

Detailed Member Calculations

Units: N&mm

Regulation: ASCE 41-17

Calculation No. 1

column C1, Floor 1

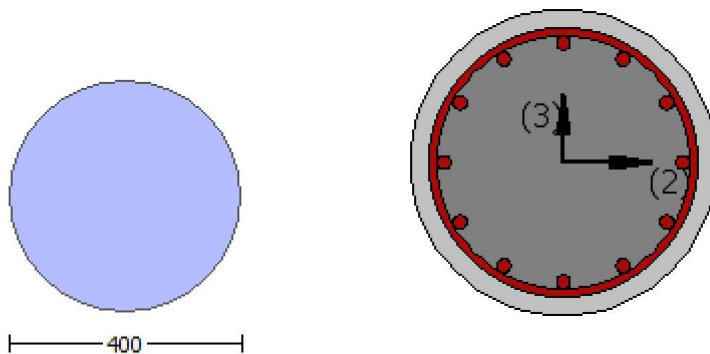
Limit State: Immediate Occupancy (data interpolation between analysis steps 1 and 2)

Analysis: Uniform +X

Check: Shear capacity VR_d

Edge: Start

Local Axis: (2)



Start Of Calculation of Shear Capacity for element: column CC1 of floor 1

At local axis: 2

Integration Section: (a)

Section Type: rccs

Constant Properties

Knowledge Factor, $\gamma = 1.00$

Member Shear Force is generally considered as Force-Controlled Action according to Table C7-1, ASCE41-17.

Lower-bound strengths are used for Force-Controlled Actions according to 7.5.1.3, ASCE 41-17

Consequently:

New material of Primary Member: Concrete Strength, $f_c = f_{c_lower_bound} = 20.00$

New material of Primary Member: Steel Strength, $f_s = f_{s_lower_bound} = 400.00$

Concrete Elasticity, $E_c = 24870.062$

Steel Elasticity, $E_s = 200000.00$

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Note: Especially for the calculation of γ for displacement ductility demand, the expected (mean value) strengths are used (7.5.1.3, ASCE41-17) because bending is considered as Deformation-Controlled Action (Table C7-1, ASCE41-17).

New material: Concrete Strength, $f_c = f_{cm} = 28.00$

New material: Steel Strength, $f_s = f_{sm} = 420.00$

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Diameter, $D = 400.00$

Cover Thickness, $c = 25.00$

Element Length, $L = 3000.00$

Primary Member

Smooth Bars

Ductile Steel

With Detailing for Earthquake Resistance (including stirrups closed at 135°)

Longitudinal Bars With Ends Lapped Starting at the End Sections

Adequate Lap Length ($l_o/l_{ou,min} = l_b/l_d \geq 1$)

No FRP Wrapping

Stepwise Properties

EDGE -A-

Bending Moment, $M_a = -1.3765E+007$

Shear Force, $V_a = -4586.336$

EDGE -B-

Bending Moment, $M_b = 0.04476539$

Shear Force, $V_b = 4586.336$

BOTH EDGES

Axial Force, $F = -4769.398$

Longitudinal Reinforcement Area Distribution (in 2 divisions)

-Tension: $A_{sl,t} = 1272.345$

-Compression: $A_{sl,c} = 1781.283$

Longitudinal Reinforcement Area Distribution (in 3 divisions)

-Tension: $A_{sl,ten} = 1017.876$

-Compression: $A_{sl,com} = 1017.876$

-Middle: $A_{sl,mid} = 1017.876$

Mean Diameter of Tension Reinforcement, $Db_{L,ten} = 18.00$

New component: From table 7-7, ASCE 41_17: Final Shear Capacity $V_R = 1.0 \cdot V_n = 214387.932$

V_n ((10.3), ASCE 41-17) = $k_n \cdot V_{CoIO} = 214387.932$

$V_{CoI} = 214387.932$

$k_n = 1.00$

displacement_ductility_demand = 0.01738074

NOTE: In expression (10-3) ' $V_s = A_v \cdot f_y \cdot d/s$ ' is replaced by ' $V_s + f \cdot V_f$ ' where V_f is the contribution of FRPs ((11.3), ACI 440).

= 1 (normal-weight concrete)

$f_c' = 20.00$, but $f_c^{0.5} \leq 8.3$ MPa ((22.5.3.1, ACI 318-14)

$M/Vd = 4.00$

$M_u = 1.3765E+007$

$V_u = 4586.336$

$d = 0.8 \cdot D = 320.00$

$N_u = 4769.398$

$A_g = 125663.706$

From ((11.5.4.8), ACI 318-14: $V_s = 157913.67$

$A_v = /2 \cdot A_{stirrup} = 123370.055$

$f_y = 400.00$

$s = 100.00$

V_s is multiplied by $CoI = 0.00$

$s/d = 0.3125$

V_f ((11-3)-(11.4), ACI 440) = 0.00

From ((11-11), ACI 440: $V_s + V_f \leq 238930.50$

$$b_w \cdot d = \frac{I_d}{I_g} = 80424.772$$

displacement_ductility_demand is calculated as $\frac{\Delta}{y}$

- Calculation of $\frac{\Delta}{y}$ for END A -
for rotation axis 3 and integ. section (a)

From analysis, chord rotation $\theta = 0.00039905$
 $y = (M_y \cdot L_s / 3) / E_{eff} = 0.02295905$ ((4.29), Biskinis Phd)
 $M_y = 2.1517E+008$
 $L_s = M/V$ (with $L_s > 0.1 \cdot L$ and $L_s < 2 \cdot L$) = 3001.241
 From table 10.5, ASCE 41_17: $E_{eff} = \text{factor} \cdot E_c \cdot I_g = 9.3758E+012$
 factor = 0.30
 $A_g = 125663.706$
 $f_c' = 28.00$
 $N = 4769.398$
 $E_c \cdot I_g = 3.1253E+013$

Calculation of Yielding Moment M_y

Calculation of $\frac{\Delta}{y}$ and M_y according to (7) - (8) in Biskinis and Fardis

$M_y = \min(M_{y_ten}, M_{y_com}) = 2.1517E+008$
 $y \text{ ((10a) or (10b))} = 1.1483164E-005$
 $M_{y_ten} \text{ (8a)} = 2.1517E+008$
 $\frac{\Delta}{y} \text{ (7a)} = 72.79824$
 error of function (7a) = 0.00155286
 $M_{y_com} \text{ (8b)} = 4.0201E+008$
 $\frac{\Delta}{y} \text{ (7b)} = 69.01016$
 error of function (7b) = -0.00040881
 with $e_y = 0.0021$
 $e_{co} = 0.002$
 $a_{pl} = 0.35$ ((9a) in Biskinis and Fardis for no FRP Wrap)
 $d_1 = 44.00$
 $R = 200.00$
 $v = 0.00135549$
 $N = 4769.398$
 $A_c = 125663.706$
 $\rho = 0.3645$
 with $f_c = 28.00$

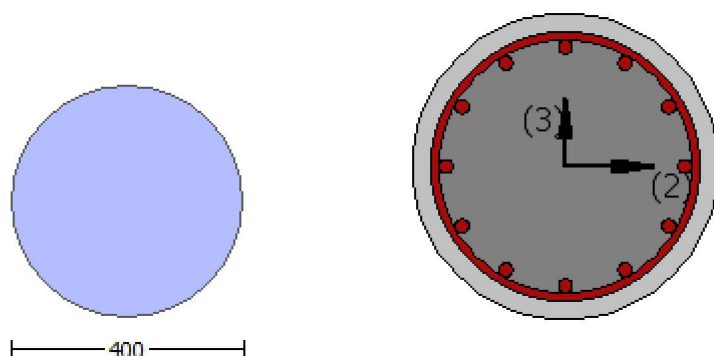
Calculation of ratio I_b/I_d

Adequate Lap Length: $I_b/I_d \geq 1$

End Of Calculation of Shear Capacity for element: column CC1 of floor 1
 At local axis: 2
 Integration Section: (a)

Calculation No. 2

column C1, Floor 1
 Limit State: Immediate Occupancy (data interpolation between analysis steps 1 and 2)
 Analysis: Uniform +X
 Check: Chord rotation capacity (ϕ)
 Edge: Start
 Local Axis: (2)



Start Of Calculation of Shear Capacity ratio for element: column CC1 of floor 1
 At Shear local axis: 3
 (Bending local axis: 2)
 Section Type: rccs

Constant Properties

Knowledge Factor, $\gamma = 1.00$
 Mean strength values are used for both shear and moment calculations.
 Consequently:
 New material of Primary Member: Concrete Strength, $f_c = f_{cm} = 28.00$
 New material of Primary Member: Steel Strength, $f_s = f_{sm} = 420.00$
 Concrete Elasticity, $E_c = 24870.062$
 Steel Elasticity, $E_s = 200000.00$
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 Note: Especially for the calculation of moment strengths,
 the above steel re-bar strengths are multiplied by 1.25 according to R18.6.5, ACI 318-14
 New material: Steel Strength, $f_s = 1.25 \cdot f_{sm} = 525.00$
 #####
 Diameter, $D = 400.00$
 Cover Thickness, $c = 25.00$
 Mean Confinement Factor overall section = 1.32561
 Element Length, $L = 3000.00$
 Primary Member
 Smooth Bars
 Ductile Steel
 With Detailing for Earthquake Resistance (including stirrups closed at 135°)
 Longitudinal Bars With Ends Lapped Starting at the End Sections
 Adequate Lap Length ($l_o/l_{ou,min} \geq 1$)
 No FRP Wrapping

Stepwise Properties

At local axis: 3
 EDGE -A-
 Shear Force, $V_a = -3.7372036E-031$
 EDGE -B-
 Shear Force, $V_b = 3.7372036E-031$

BOTH EDGES

Axial Force, $F = -4771.233$

Longitudinal Reinforcement Area Distribution (in 2 divisions)

-Tension: $As_t = 0.00$

-Compression: $As_c = 3053.628$

Longitudinal Reinforcement Area Distribution (in 3 divisions)

-Tension: $As_{t,ten} = 1017.876$

-Compression: $As_{c,com} = 1017.876$

-Middle: $As_{mid} = 1017.876$

Calculation of Shear Capacity ratio, $V_e/V_r = 0.51034251$

Member Controlled by Flexure ($V_e/V_r < 1$)

Calculation of Shear Demand from fig. R18.6.5, ACI 318-14 $V_e = (M_{pr1} + M_{pr2})/l_n = 152734.698$ with

$M_{pr1} = \text{Max}(\mu_{u1+}, \mu_{u1-}) = 2.2910\text{E}+008$

$\mu_{u1+} = 2.2910\text{E}+008$, is the ultimate moment strength at the edge 1 of the member in the actual moment direction which is defined for the static loading combination

$\mu_{u1-} = 2.2910\text{E}+008$, is the ultimate moment strength at the edge 1 of the member in the opposite moment direction which is defined for the static loading combination

$M_{pr2} = \text{Max}(\mu_{u2+}, \mu_{u2-}) = 2.2910\text{E}+008$

$\mu_{u2+} = 2.2910\text{E}+008$, is the ultimate moment strength at the edge 2 of the member in the actual moment direction which is defined for the static loading combination

$\mu_{u2-} = 2.2910\text{E}+008$, is the ultimate moment strength at the edge 2 of the member in the opposite moment direction which is defined for the static loading combination

Calculation of μ_{u1+}

Calculation of ultimate Moment Strength ((3.67), Biskinis Phd), μ_u

$\mu_u = 2.2910\text{E}+008$

$\phi = 1.0472$

$\phi' = 0.92729522$

error of function (3.68), Biskinis Phd = 58526.963

From 5A.2, TBDY: $f_{cc} = f_c \cdot c = 37.11712$

conf. factor $c = 1.32561$

$f_c = 28.00$

From 10.3.5, ASCE41-17, Final value of f_y : $f_y \cdot \text{Min}(1, 1.25 \cdot (l_b/d)^{2/3}) = 525.00$

$l_b/d = 1.00$

$d_1 = 44.00$

$R = 200.00$

$v = 0.00135568$

$N = 4771.233$

$A_c = 125663.706$

$\phi' \cdot \text{Min}(1, 1.25 \cdot (l_b/d)^{2/3}) = 0.3645$

Calculation of ratio l_b/d

Adequate Lap Length: $l_b/d \geq 1$

Calculation of μ_{u1-}

Calculation of ultimate Moment Strength ((3.67), Biskinis Phd), μ_u

$\mu_u = 2.2910\text{E}+008$

$\phi = 1.0472$

$\phi' = 0.92729522$

error of function (3.68), Biskinis Phd = 58526.963
 From 5A.2, TBDY: $f_{cc} = f_c \cdot c = 37.11712$
 conf. factor $c = 1.32561$
 $f_c = 28.00$
 From 10.3.5, ASCE41-17, Final value of f_y : $f_y \cdot \text{Min}(1, 1.25 \cdot (l_b/d)^{2/3}) = 525.00$
 $l_b/d = 1.00$
 $d_1 = 44.00$
 $R = 200.00$
 $v = 0.00135568$
 $N = 4771.233$
 $A_c = 125663.706$
 $= \cdot \text{Min}(1, 1.25 \cdot (l_b/d)^{2/3}) = 0.3645$

Calculation of ratio l_b/d

Adequate Lap Length: $l_b/d \geq 1$

Calculation of μ_{2+}

Calculation of ultimate Moment Strength ((3.67), Biskinis Phd), μ
 $\mu = 2.2910E+008$

$= 1.0472$
 $' = 0.92729522$
 error of function (3.68), Biskinis Phd = 58526.963
 From 5A.2, TBDY: $f_{cc} = f_c \cdot c = 37.11712$
 conf. factor $c = 1.32561$
 $f_c = 28.00$
 From 10.3.5, ASCE41-17, Final value of f_y : $f_y \cdot \text{Min}(1, 1.25 \cdot (l_b/d)^{2/3}) = 525.00$
 $l_b/d = 1.00$
 $d_1 = 44.00$
 $R = 200.00$
 $v = 0.00135568$
 $N = 4771.233$
 $A_c = 125663.706$
 $= \cdot \text{Min}(1, 1.25 \cdot (l_b/d)^{2/3}) = 0.3645$

Calculation of ratio l_b/d

Adequate Lap Length: $l_b/d \geq 1$

Calculation of μ_{2-}

Calculation of ultimate Moment Strength ((3.67), Biskinis Phd), μ
 $\mu = 2.2910E+008$

$= 1.0472$
 $' = 0.92729522$
 error of function (3.68), Biskinis Phd = 58526.963
 From 5A.2, TBDY: $f_{cc} = f_c \cdot c = 37.11712$
 conf. factor $c = 1.32561$
 $f_c = 28.00$
 From 10.3.5, ASCE41-17, Final value of f_y : $f_y \cdot \text{Min}(1, 1.25 \cdot (l_b/d)^{2/3}) = 525.00$
 $l_b/d = 1.00$
 $d_1 = 44.00$
 $R = 200.00$
 $v = 0.00135568$

$$N = 4771.233$$

$$A_c = 125663.706$$

$$= \text{Min}(1, 1.25 \cdot (l_b/d)^{2/3}) = 0.3645$$

Calculation of ratio l_b/d

Adequate Lap Length: $l_b/d \geq 1$

Calculation of Shear Strength $V_r = \text{Min}(V_{r1}, V_{r2}) = 299278.805$

Calculation of Shear Strength at edge 1, $V_{r1} = 299278.805$

$V_{r1} = V_{c1}$ ((10.3), ASCE 41-17) = $k_n \cdot V_{c10}$

$V_{c10} = 299278.805$

$k_n = 1$ (zero step-static loading)

NOTE: In expression (10-3) ' $V_s = A_v \cdot f_y \cdot d/s$ ' is replaced by ' $V_s + f \cdot V_f$ ' where V_f is the contribution of FRPs ((11.3), ACI 440).

= 1 (normal-weight concrete)

$f'_c = 28.00$, but $f_c^{0.5} \leq 8.3$ MPa ((22.5.3.1, ACI 318-14)

$M/Vd = 2.00$

$\mu = 1.0979367E-011$

$\nu = 3.7372036E-031$

$d = 0.8 \cdot D = 320.00$

$N_u = 4771.233$

$A_g = 125663.706$

From ((11.5.4.8), ACI 318-14: $V_s = 165809.354$

$A_v = \sqrt{2} \cdot A_{\text{stirrup}} = 123370.055$

$f_y = 420.00$

$s = 100.00$

V_s is multiplied by $\text{Col} = 0.00$

$s/d = 0.3125$

V_f ((11-3)-(11.4), ACI 440) = 0.00

From ((11-11), ACI 440: $V_s + V_f \leq 282706.38$

$b_w \cdot d = \sqrt{d} \cdot d/4 = 80424.772$

Calculation of Shear Strength at edge 2, $V_{r2} = 299278.805$

$V_{r2} = V_{c2}$ ((10.3), ASCE 41-17) = $k_n \cdot V_{c10}$

$V_{c10} = 299278.805$

$k_n = 1$ (zero step-static loading)

NOTE: In expression (10-3) ' $V_s = A_v \cdot f_y \cdot d/s$ ' is replaced by ' $V_s + f \cdot V_f$ ' where V_f is the contribution of FRPs ((11.3), ACI 440).

= 1 (normal-weight concrete)

$f'_c = 28.00$, but $f_c^{0.5} \leq 8.3$ MPa ((22.5.3.1, ACI 318-14)

$M/Vd = 2.00$

$\mu = 1.0979367E-011$

$\nu = 3.7372036E-031$

$d = 0.8 \cdot D = 320.00$

$N_u = 4771.233$

$A_g = 125663.706$

From ((11.5.4.8), ACI 318-14: $V_s = 165809.354$

$A_v = \sqrt{2} \cdot A_{\text{stirrup}} = 123370.055$

$f_y = 420.00$

$s = 100.00$

V_s is multiplied by $\text{Col} = 0.00$

$s/d = 0.3125$

V_f ((11-3)-(11.4), ACI 440) = 0.00

From ((11-11), ACI 440: $V_s + V_f \leq 282706.38$

$b_w \cdot d = \sqrt{d} \cdot d/4 = 80424.772$

End Of Calculation of Shear Capacity ratio for element: column CC1 of floor 1
At local axis: 3

Start Of Calculation of Shear Capacity ratio for element: column CC1 of floor 1
At Shear local axis: 2
(Bending local axis: 3)
Section Type: rccs

Constant Properties

Knowledge Factor, $\gamma = 1.00$
Mean strength values are used for both shear and moment calculations.
Consequently:
New material of Primary Member: Concrete Strength, $f_c = f_{cm} = 28.00$
New material of Primary Member: Steel Strength, $f_s = f_{sm} = 420.00$
Concrete Elasticity, $E_c = 24870.062$
Steel Elasticity, $E_s = 200000.00$

Note: Especially for the calculation of moment strengths,
the above steel re-bar strengths are multiplied by 1.25 according to R18.6.5, ACI 318-14
New material: Steel Strength, $f_s = 1.25 \cdot f_{sm} = 525.00$

Diameter, $D = 400.00$
Cover Thickness, $c = 25.00$
Mean Confinement Factor overall section = 1.32561
Element Length, $L = 3000.00$
Primary Member
Smooth Bars
Ductile Steel
With Detailing for Earthquake Resistance (including stirrups closed at 135°)
Longitudinal Bars With Ends Lapped Starting at the End Sections
Adequate Lap Length ($l_o/l_{ou,min} \geq 1$)
No FRP Wrapping

Stepwise Properties

At local axis: 2
EDGE -A-
Shear Force, $V_a = -9.8626872E-031$
EDGE -B-
Shear Force, $V_b = 9.8626872E-031$
BOTH EDGES
Axial Force, $F = -4771.233$
Longitudinal Reinforcement Area Distribution (in 2 divisions)
-Tension: $A_{st} = 0.00$
-Compression: $A_{sc} = 3053.628$
Longitudinal Reinforcement Area Distribution (in 3 divisions)
-Tension: $A_{st,ten} = 1017.876$
-Compression: $A_{st,com} = 1017.876$
-Middle: $A_{st,mid} = 1017.876$

Calculation of Shear Capacity ratio, $V_e/V_r = 0.51034251$
Member Controlled by Flexure ($V_e/V_r < 1$)
Calculation of Shear Demand from fig. R18.6.5, ACI 318-14 $V_e = (M_{pr1} + M_{pr2})/l_n = 152734.698$
with
 $M_{pr1} = \max(M_{u1+}, M_{u1-}) = 2.2910E+008$
 $M_{u1+} = 2.2910E+008$, is the ultimate moment strength at the edge 1 of the member in the actual moment direction
which is defined for the static loading combination
 $M_{u1-} = 2.2910E+008$, is the ultimate moment strength at the edge 1 of the member in the opposite moment

direction which is defined for the static loading combination

$$M_{pr2} = \text{Max}(M_{u2+}, M_{u2-}) = 2.2910\text{E}+008$$

$M_{u2+} = 2.2910\text{E}+008$, is the ultimate moment strength at the edge 2 of the member in the actual moment direction which is defined for the the static loading combination

$M_{u2-} = 2.2910\text{E}+008$, is the ultimate moment strength at the edge 2 of the member in the opposite moment direction which is defined for the the static loading combination

Calculation of M_{u1+}

Calculation of ultimate Moment Strength ((3.67), Biskinis Phd), M_u
 $M_u = 2.2910\text{E}+008$

$$= 1.0472$$

$$\lambda = 0.92729522$$

error of function (3.68), Biskinis Phd = 58526.963

From 5A.2, TB DY: $f_{cc} = f_c \cdot c = 37.11712$

conf. factor $c = 1.32561$

$f_c = 28.00$

From 10.3.5, ASCE41-17, Final value of f_y : $f_y \cdot \text{Min}(1, 1.25 \cdot (l_b/d)^{2/3}) = 525.00$

$$l_b/d = 1.00$$

$$d_1 = 44.00$$

$$R = 200.00$$

$$v = 0.00135568$$

$$N = 4771.233$$

$$A_c = 125663.706$$

$$= \lambda \cdot \text{Min}(1, 1.25 \cdot (l_b/d)^{2/3}) = 0.3645$$

Calculation of ratio l_b/d

Adequate Lap Length: $l_b/d \geq 1$

Calculation of M_{u1-}

Calculation of ultimate Moment Strength ((3.67), Biskinis Phd), M_u
 $M_u = 2.2910\text{E}+008$

$$= 1.0472$$

$$\lambda = 0.92729522$$

error of function (3.68), Biskinis Phd = 58526.963

From 5A.2, TB DY: $f_{cc} = f_c \cdot c = 37.11712$

conf. factor $c = 1.32561$

$f_c = 28.00$

From 10.3.5, ASCE41-17, Final value of f_y : $f_y \cdot \text{Min}(1, 1.25 \cdot (l_b/d)^{2/3}) = 525.00$

$$l_b/d = 1.00$$

$$d_1 = 44.00$$

$$R = 200.00$$

$$v = 0.00135568$$

$$N = 4771.233$$

$$A_c = 125663.706$$

$$= \lambda \cdot \text{Min}(1, 1.25 \cdot (l_b/d)^{2/3}) = 0.3645$$

Calculation of ratio l_b/d

Adequate Lap Length: $l_b/d \geq 1$

Calculation of M_{u2+}

Calculation of ultimate Moment Strength ((3.67), Biskinis Phd), Mu
Mu = 2.2910E+008

= 1.0472
' = 0.92729522
error of function (3.68), Biskinis Phd = 58526.963
From 5A.2, TBDY: fcc = fc* c = 37.11712
conf. factor c = 1.32561
fc = 28.00
From 10.3.5, ASCE41-17, Final value of fy: fy*Min(1,1.25*(lb/d)^ 2/3) = 525.00
lb/d = 1.00
d1 = 44.00
R = 200.00
v = 0.00135568
N = 4771.233
Ac = 125663.706
= *Min(1,1.25*(lb/d)^ 2/3) = 0.3645

Calculation of ratio lb/d

Adequate Lap Length: lb/d >= 1

Calculation of Mu2-

Calculation of ultimate Moment Strength ((3.67), Biskinis Phd), Mu
Mu = 2.2910E+008

= 1.0472
' = 0.92729522
error of function (3.68), Biskinis Phd = 58526.963
From 5A.2, TBDY: fcc = fc* c = 37.11712
conf. factor c = 1.32561
fc = 28.00
From 10.3.5, ASCE41-17, Final value of fy: fy*Min(1,1.25*(lb/d)^ 2/3) = 525.00
lb/d = 1.00
d1 = 44.00
R = 200.00
v = 0.00135568
N = 4771.233
Ac = 125663.706
= *Min(1,1.25*(lb/d)^ 2/3) = 0.3645

Calculation of ratio lb/d

Adequate Lap Length: lb/d >= 1

Calculation of Shear Strength Vr = Min(Vr1,Vr2) = 299278.805

Calculation of Shear Strength at edge 1, Vr1 = 299278.805

Vr1 = VCol ((10.3), ASCE 41-17) = knl*VColO

VColO = 299278.805

knl = 1 (zero step-static loading)

NOTE: In expression (10-3) 'Vs = Av*fy*d/s' is replaced by 'Vs+ f*Vf'
where Vf is the contribution of FRPs (11.3), ACI 440).

= 1 (normal-weight concrete)
 $f_c' = 28.00$, but $f_c'^{0.5} \leq 8.3$ MPa (22.5.3.1, ACI 318-14)
 $M/Vd = 2.00$
 $\mu_u = 9.9556147E-012$
 $V_u = 9.8626872E-031$
 $d = 0.8 \cdot D = 320.00$
 $N_u = 4771.233$
 $A_g = 125663.706$
 From (11.5.4.8), ACI 318-14: $V_s = 165809.354$
 $A_v = \frac{1}{2} A_{\text{stirrup}} = 123370.055$
 $f_y = 420.00$
 $s = 100.00$
 V_s is multiplied by $\text{Col} = 0.00$
 $s/d = 0.3125$
 $V_f ((11-3)-(11.4), \text{ACI } 440) = 0.00$
 From (11-11), ACI 440: $V_s + V_f \leq 282706.38$
 $b_w \cdot d = \frac{1}{4} \cdot d \cdot d = 80424.772$

Calculation of Shear Strength at edge 2, $V_{r2} = 299278.805$
 $V_{r2} = V_{\text{Col}} ((10.3), \text{ASCE } 41-17) = k_{nl} \cdot V_{\text{Col}0}$
 $V_{\text{Col}0} = 299278.805$
 $k_{nl} = 1$ (zero step-static loading)

NOTE: In expression (10-3) ' $V_s = A_v \cdot f_y \cdot d / s$ ' is replaced by ' $V_s + f \cdot V_f$ '
 where V_f is the contribution of FRPs (11.3), ACI 440).

= 1 (normal-weight concrete)
 $f_c' = 28.00$, but $f_c'^{0.5} \leq 8.3$ MPa (22.5.3.1, ACI 318-14)
 $M/Vd = 2.00$
 $\mu_u = 9.9556147E-012$
 $V_u = 9.8626872E-031$
 $d = 0.8 \cdot D = 320.00$
 $N_u = 4771.233$
 $A_g = 125663.706$
 From (11.5.4.8), ACI 318-14: $V_s = 165809.354$
 $A_v = \frac{1}{2} A_{\text{stirrup}} = 123370.055$
 $f_y = 420.00$
 $s = 100.00$
 V_s is multiplied by $\text{Col} = 0.00$
 $s/d = 0.3125$
 $V_f ((11-3)-(11.4), \text{ACI } 440) = 0.00$
 From (11-11), ACI 440: $V_s + V_f \leq 282706.38$
 $b_w \cdot d = \frac{1}{4} \cdot d \cdot d = 80424.772$

End Of Calculation of Shear Capacity ratio for element: column CC1 of floor 1
 At local axis: 2

Start Of Calculation of Chord Rotation Capacity for element: column CC1 of floor 1
 At local axis: 2
 Integration Section: (a)
 Section Type: rccs

Constant Properties

Knowledge Factor, $\gamma = 1.00$
 Chord Rotation is generally considered as Deformation-Controlled Action according to Table C7-1, ASCE41-17.
 Mean strengths are used for Deformation-Controlled Actions according to 7.5.1.3, ASCE 41-17
 Consequently:
 New material of Primary Member: Concrete Strength, $f_c = f_{cm} = 28.00$
 New material of Primary Member: Steel Strength, $f_s = f_{sm} = 420.00$
 Concrete Elasticity, $E_c = 24870.062$

Steel Elasticity, $E_s = 200000.00$
 Diameter, $D = 400.00$
 Cover Thickness, $c = 25.00$
 Element Length, $L = 3000.00$
 Primary Member
 Smooth Bars
 Ductile Steel
 With Detailing for Earthquake Resistance (including stirrups closed at 135°)
 Longitudinal Bars With Ends Lapped Starting at the End Sections
 Adequate Lap Length ($l_b/d \geq 1$)
 No FRP Wrapping

Stepwise Properties

Bending Moment, $M = -1.6986061E-011$
 Shear Force, $V_2 = -4586.336$
 Shear Force, $V_3 = 8.2104443E-015$
 Axial Force, $F = -4769.398$
 Longitudinal Reinforcement Area Distribution (in 2 divisions)
 -Tension: $A_{st} = 1272.345$
 -Compression: $A_{sc} = 1781.283$
 Longitudinal Reinforcement Area Distribution (in 3 divisions)
 -Tension: $A_{st,ten} = 1017.876$
 -Compression: $A_{sc,com} = 1017.876$
 -Middle: $A_{st,mid} = 1017.876$
 Mean Diameter of Tension Reinforcement, $D_bL = 18.00$

New component: From table 7-7, ASCE 41_17: Final chord rotation Capacity $u_R = 1.0^*$ $u = 0.01647478$
 $u = y + p = 0.01647478$

- Calculation of y -

$y = (M_y * L_s / 3) / E_{eff} = 0.01147478$ ((4.29), Biskinis Phd))
 $M_y = 2.1517E+008$
 $L_s = M/V$ (with $L_s > 0.1 * L$ and $L_s < 2 * L$) = 1500.00
 From table 10.5, ASCE 41_17: $E_{eff} = factor * E_c * I_g = 9.3758E+012$
 $factor = 0.30$
 $A_g = 125663.706$
 $f_c' = 28.00$
 $N = 4769.398$
 $E_c * I_g = 3.1253E+013$

Calculation of Yielding Moment M_y

Calculation of y and M_y according to (7) - (8) in Biskinis and Fardis

$M_y = \min(M_{y_ten}, M_{y_com}) = 2.1517E+008$
 y ((10a) or (10b)) = $1.1483164E-005$
 M_{y_ten} (8a) = $2.1517E+008$
 $_{ten}$ (7a) = 72.79824
 error of function (7a) = 0.00155286
 M_{y_com} (8b) = $4.0201E+008$
 $_{com}$ (7b) = 69.01016
 error of function (7b) = -0.00040881
 with $e_y = 0.0021$
 $e_{co} = 0.002$
 $apl = 0.35$ ((9a) in Biskinis and Fardis for no FRP Wrap)
 $d_1 = 44.00$
 $R = 200.00$
 $v = 0.00135549$

N = 4769.398
Ac = 125663.706
= 0.3645
with fc = 28.00

Calculation of ratio lb/ld

Adequate Lap Length: lb/ld >= 1

- Calculation of p -

From table 10-9: p = 0.005

with:

- Columns controlled by inadequate development or splicing along the clear height because lb/ld < 1
shear control ratio VyE/VColOE = 0.51034251

d = 0.00

s = 0.00

t = 2*Av/(dc*s) + 4*tf/D*(ffe/fs) = 0.00

Av = 78.53982, is the area of the circular stirrup

dc = D - 2*cover - Hoop Diameter = 340.00

The term 2*tf/bw*(ffe/fs) is implemented to account for FRP contribution

where f = 2*tf/bw is FRP ratio (EC8 - 3, A.4.4.3(6)) and ffe/fs normalises f to steel strength

All these variables have already been given in Shear control ratio calculation.

NUD = 4769.398

Ag = 125663.706

fcE = 28.00

fytE = fyle = 420.00

pl = Area_Tot_Long_Rein/(Ag) = 0.0243

fcE = 28.00

End Of Calculation of Chord Rotation Capacity for element: column CC1 of floor 1

At local axis: 2

Integration Section: (a)

Calculation No. 3

column C1, Floor 1

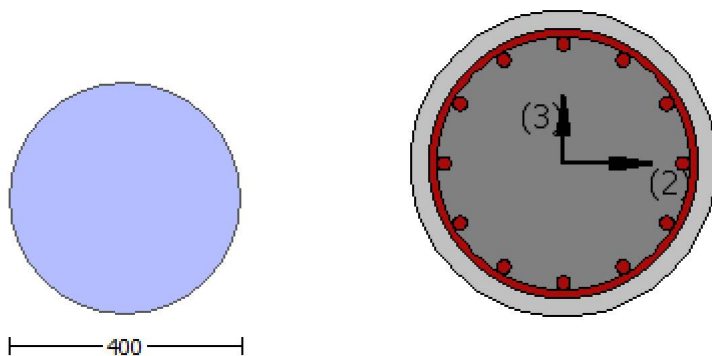
Limit State: Immediate Occupancy (data interpolation between analysis steps 1 and 2)

Analysis: Uniform +X

Check: Shear capacity VRd

Edge: Start

Local Axis: (3)



Start Of Calculation of Shear Capacity for element: column CC1 of floor 1

At local axis: 3

Integration Section: (a)

Section Type: rccs

Constant Properties

Knowledge Factor, $\gamma = 1.00$

Member Shear Force is generally considered as Force-Controlled Action according to Table C7-1, ASCE41-17.

Lower-bound strengths are used for Force-Controlled Actions according to 7.5.1.3, ASCE 41-17

Consequently:

New material of Primary Member: Concrete Strength, $f_c = f_{c_lower_bound} = 20.00$

New material of Primary Member: Steel Strength, $f_s = f_{s_lower_bound} = 400.00$

Concrete Elasticity, $E_c = 24870.062$

Steel Elasticity, $E_s = 200000.00$

#####

Note: Especially for the calculation of γ for displacement ductility demand, the expected (mean value) strengths are used (7.5.1.3, ASCE41-17) because bending is considered as Deformation-Controlled Action (Table C7-1, ASCE41-17).

New material: Concrete Strength, $f_c = f_{cm} = 28.00$

New material: Steel Strength, $f_s = f_{sm} = 420.00$

#####

Diameter, $D = 400.00$

Cover Thickness, $c = 25.00$

Element Length, $L = 3000.00$

Primary Member

Smooth Bars

Ductile Steel

With Detailing for Earthquake Resistance (including stirrups closed at 135°)

Longitudinal Bars With Ends Lapped Starting at the End Sections

Adequate Lap Length ($l_o/l_{ou,min} = l_b/l_d \geq 1$)

No FRP Wrapping

Stepwise Properties

EDGE -A-

Bending Moment, $M_a = -1.6986061E-011$

Shear Force, $V_a = 8.2104443E-015$

EDGE -B-

Bending Moment, $M_b = -7.7176173E-012$

Shear Force, $V_b = -8.2104443E-015$

BOTH EDGES

Axial Force, $F = -4769.398$

Longitudinal Reinforcement Area Distribution (in 2 divisions)

-Tension: $As_t = 1272.345$

-Compression: $As_c = 1781.283$

Longitudinal Reinforcement Area Distribution (in 3 divisions)

-Tension: $As_{t,ten} = 1017.876$

-Compression: $As_{l,com} = 1017.876$

-Middle: $As_{l,mid} = 1017.876$

Mean Diameter of Tension Reinforcement, $Db_{L,ten} = 18.00$

New component: From table 7-7, ASCE 41_17: Final Shear Capacity $V_R = 1.0 \cdot V_n = 270862.194$
 V_n ((10.3), ASCE 41-17) = $k_n \cdot V_{Col0} = 270862.194$
 $V_{Col} = 270862.194$
 $k_n = 1.00$
 $displacement_ductility_demand = 0.00$

NOTE: In expression (10-3) ' $V_s = A_v \cdot f_y \cdot d / s$ ' is replaced by ' $V_s + f \cdot V_f$ '
 where V_f is the contribution of FRPs ((11.3), ACI 440).

$= 1$ (normal-weight concrete)
 $f'_c = 20.00$, but $f'_c^{0.5} \leq 8.3$ MPa ((22.5.3.1, ACI 318-14))
 $M/Vd = 2.00$
 $\mu_u = 1.6986061E-011$
 $V_u = 8.2104443E-015$
 $d = 0.8 \cdot D = 320.00$
 $N_u = 4769.398$
 $A_g = 125663.706$
 From ((11.5.4.8), ACI 318-14: $V_s = 157913.67$
 $A_v = \sqrt{2} \cdot A_{stirrup} = 123370.055$
 $f_y = 400.00$
 $s = 100.00$
 V_s is multiplied by $Col = 0.00$
 $s/d = 0.3125$
 V_f ((11-3)-(11.4), ACI 440) = 0.00
 From ((11-11), ACI 440: $V_s + V_f \leq 238930.50$
 $b_w \cdot d = \sqrt{2} \cdot d^2 / 4 = 80424.772$

$displacement_ductility_demand$ is calculated as ϕ / y

- Calculation of ϕ / y for END A -
 for rotation axis 2 and integ. section (a)

From analysis, chord rotation $\phi = 5.0574879E-021$
 $y = (M_y \cdot L_s / 3) / E_{eff} = 0.01147478$ ((4.29), Biskinis Phd))
 $M_y = 2.1517E+008$
 $L_s = M/V$ (with $L_s > 0.1 \cdot L$ and $L_s < 2 \cdot L$) = 1500.00
 From table 10.5, ASCE 41_17: $E_{eff} = factor \cdot E_c \cdot I_g = 9.3758E+012$
 $factor = 0.30$
 $A_g = 125663.706$
 $f'_c = 28.00$
 $N = 4769.398$
 $E_c \cdot I_g = 3.1253E+013$

Calculation of Yielding Moment M_y

Calculation of ϕ and M_y according to (7) - (8) in Biskinis and Fardis

$M_y = \min(M_{y_ten}, M_{y_com}) = 2.1517E+008$
 ϕ ((10a) or (10b)) = $1.1483164E-005$
 M_{y_ten} (8a) = $2.1517E+008$
 ϕ_{ten} (7a) = 72.79824
 error of function (7a) = 0.00155286
 M_{y_com} (8b) = $4.0201E+008$
 ϕ_{com} (7b) = 69.01016
 error of function (7b) = -0.00040881
 with $e_y = 0.0021$
 $e_{co} = 0.002$
 $\alpha_{pl} = 0.35$ ((9a) in Biskinis and Fardis for no FRP Wrap)
 $d1 = 44.00$

R = 200.00
v = 0.00135549
N = 4769.398
Ac = 125663.706
= 0.3645
with $f_c = 28.00$

Calculation of ratio I_b/I_d

Adequate Lap Length: $I_b/I_d \geq 1$

End Of Calculation of Shear Capacity for element: column CC1 of floor 1

At local axis: 3

Integration Section: (a)

Calculation No. 4

column C1, Floor 1

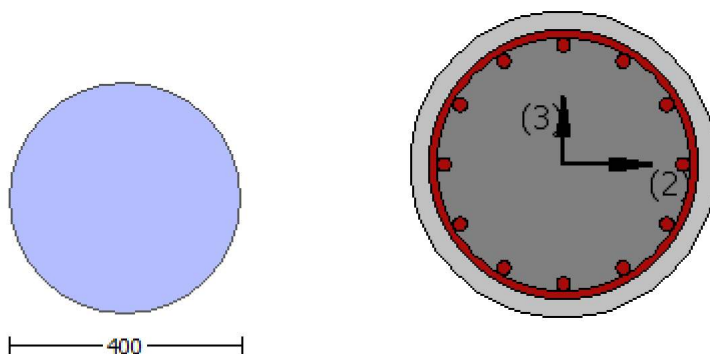
Limit State: Immediate Occupancy (data interpolation between analysis steps 1 and 2)

Analysis: Uniform +X

Check: Chord rotation capacity (ϕ)

Edge: Start

Local Axis: (3)



Start Of Calculation of Shear Capacity ratio for element: column CC1 of floor 1

At Shear local axis: 3

(Bending local axis: 2)

Section Type: rccs

Constant Properties

Knowledge Factor, $\gamma = 1.00$

Mean strength values are used for both shear and moment calculations.

Consequently:

New material of Primary Member: Concrete Strength, $f_c = f_{cm} = 28.00$

New material of Primary Member: Steel Strength, $f_s = f_{sm} = 420.00$

Concrete Elasticity, $E_c = 24870.062$

Steel Elasticity, $E_s = 200000.00$

#####

Note: Especially for the calculation of moment strengths,
the above steel re-bar strengths are multiplied by 1.25 according to R18.6.5, ACI 318-14
New material: Steel Strength, $f_s = 1.25 \cdot f_{sm} = 525.00$

#####

Diameter, $D = 400.00$

Cover Thickness, $c = 25.00$

Mean Confinement Factor overall section = 1.32561

Element Length, $L = 3000.00$

Primary Member

Smooth Bars

Ductile Steel

With Detailing for Earthquake Resistance (including stirrups closed at 135°)

Longitudinal Bars With Ends Lapped Starting at the End Sections

Adequate Lap Length ($l_o/l_{ou}, \min \geq 1$)

No FRP Wrapping

Stepwise Properties

At local axis: 3

EDGE -A-

Shear Force, $V_a = -3.7372036E-031$

EDGE -B-

Shear Force, $V_b = 3.7372036E-031$

BOTH EDGES

Axial Force, $F = -4771.233$

Longitudinal Reinforcement Area Distribution (in 2 divisions)

-Tension: $A_{slt} = 0.00$

-Compression: $A_{slc} = 3053.628$

Longitudinal Reinforcement Area Distribution (in 3 divisions)

-Tension: $A_{sl,ten} = 1017.876$

-Compression: $A_{sl,com} = 1017.876$

-Middle: $A_{sl,mid} = 1017.876$

Calculation of Shear Capacity ratio, $V_e/V_r = 0.51034251$

Member Controlled by Flexure ($V_e/V_r < 1$)

Calculation of Shear Demand from fig. R18.6.5, ACI 318-14 $V_e = (M_{pr1} + M_{pr2})/l_n = 152734.698$
with

$M_{pr1} = \max(\mu_{u1+}, \mu_{u1-}) = 2.2910E+008$

$\mu_{u1+} = 2.2910E+008$, is the ultimate moment strength at the edge 1 of the member in the actual moment direction
which is defined for the static loading combination

$\mu_{u1-} = 2.2910E+008$, is the ultimate moment strength at the edge 1 of the member in the opposite moment
direction which is defined for the static loading combination

$M_{pr2} = \max(\mu_{u2+}, \mu_{u2-}) = 2.2910E+008$

$\mu_{u2+} = 2.2910E+008$, is the ultimate moment strength at the edge 2 of the member in the actual moment direction
which is defined for the static loading combination

$\mu_{u2-} = 2.2910E+008$, is the ultimate moment strength at the edge 2 of the member in the opposite moment
direction which is defined for the static loading combination

Calculation of μ_{u1+}

Calculation of ultimate Moment Strength ((3.67), Biskinis Phd), μ_u

$\mu_u = 2.2910E+008$

$\lambda = 1.0472$

$\lambda' = 0.92729522$

error of function (3.68), Biskinis Phd = 58526.963

From 5A.2, TBDY: $f_{cc} = f_c^* \cdot c = 37.11712$

conf. factor $c = 1.32561$

$f_c = 28.00$
 From 10.3.5, ASCE41-17, Final value of f_y : $f_y \cdot \text{Min}(1, 1.25 \cdot (l_b/d)^{2/3}) = 525.00$
 $l_b/d = 1.00$
 $d_1 = 44.00$
 $R = 200.00$
 $v = 0.00135568$
 $N = 4771.233$
 $A_c = 125663.706$
 $= \cdot \text{Min}(1, 1.25 \cdot (l_b/d)^{2/3}) = 0.3645$

Calculation of ratio l_b/d

Adequate Lap Length: $l_b/d \geq 1$

Calculation of μ_1 -

Calculation of ultimate Moment Strength ((3.67), Biskinis Phd), μ
 $\mu = 2.2910\text{E}+008$

$= 1.0472$
 $' = 0.92729522$
 error of function (3.68), Biskinis Phd = 58526.963
 From 5A.2, TBDY: $f_{cc} = f_c \cdot c = 37.11712$
 conf. factor $c = 1.32561$
 $f_c = 28.00$
 From 10.3.5, ASCE41-17, Final value of f_y : $f_y \cdot \text{Min}(1, 1.25 \cdot (l_b/d)^{2/3}) = 525.00$
 $l_b/d = 1.00$
 $d_1 = 44.00$
 $R = 200.00$
 $v = 0.00135568$
 $N = 4771.233$
 $A_c = 125663.706$
 $= \cdot \text{Min}(1, 1.25 \cdot (l_b/d)^{2/3}) = 0.3645$

Calculation of ratio l_b/d

Adequate Lap Length: $l_b/d \geq 1$

Calculation of μ_2 +

Calculation of ultimate Moment Strength ((3.67), Biskinis Phd), μ
 $\mu = 2.2910\text{E}+008$

$= 1.0472$
 $' = 0.92729522$
 error of function (3.68), Biskinis Phd = 58526.963
 From 5A.2, TBDY: $f_{cc} = f_c \cdot c = 37.11712$
 conf. factor $c = 1.32561$
 $f_c = 28.00$
 From 10.3.5, ASCE41-17, Final value of f_y : $f_y \cdot \text{Min}(1, 1.25 \cdot (l_b/d)^{2/3}) = 525.00$
 $l_b/d = 1.00$
 $d_1 = 44.00$
 $R = 200.00$
 $v = 0.00135568$
 $N = 4771.233$
 $A_c = 125663.706$
 $= \cdot \text{Min}(1, 1.25 \cdot (l_b/d)^{2/3}) = 0.3645$

Calculation of ratio l_b/d

Adequate Lap Length: $l_b/d \geq 1$

Calculation of μ_2

Calculation of ultimate Moment Strength ((3.67), Biskinis Phd), μ
 $\mu = 2.2910E+008$

$$= 1.0472$$

$$' = 0.92729522$$

error of function (3.68), Biskinis Phd = 58526.963

From 5A.2, TBDY: $f_{cc} = f_c' \cdot c = 37.11712$

conf. factor $c = 1.32561$

$$f_c = 28.00$$

From 10.3.5, ASCE41-17, Final value of f_y : $f_y \cdot \text{Min}(1, 1.25 \cdot (l_b/d)^{2/3}) = 525.00$

$$l_b/d = 1.00$$

$$d_1 = 44.00$$

$$R = 200.00$$

$$v = 0.00135568$$

$$N = 4771.233$$

$$A_c = 125663.706$$

$$= \cdot \text{Min}(1, 1.25 \cdot (l_b/d)^{2/3}) = 0.3645$$

Calculation of ratio l_b/d

Adequate Lap Length: $l_b/d \geq 1$

Calculation of Shear Strength $V_r = \text{Min}(V_{r1}, V_{r2}) = 299278.805$

Calculation of Shear Strength at edge 1, $V_{r1} = 299278.805$

$V_{r1} = V_{Col} \text{ ((10.3), ASCE 41-17)} = k_{nl} \cdot V_{Col0}$

$$V_{Col0} = 299278.805$$

$k_{nl} = 1$ (zero step-static loading)

NOTE: In expression (10-3) ' $V_s = A_v \cdot f_y \cdot d/s$ ' is replaced by ' $V_s + f \cdot V_f$ ' where V_f is the contribution of FRPs (11.3), ACI 440).

$= 1$ (normal-weight concrete)

$$f_c' = 28.00, \text{ but } f_c'^{0.5} \leq 8.3 \text{ MPa (22.5.3.1, ACI 318-14)}$$

$$M/Vd = 2.00$$

$$\mu = 1.0979367E-011$$

$$V_u = 3.7372036E-031$$

$$d = 0.8 \cdot D = 320.00$$

$$N_u = 4771.233$$

$$A_g = 125663.706$$

From (11.5.4.8), ACI 318-14: $V_s = 165809.354$

$$A_v = \cdot /2 \cdot A_{\text{stirrup}} = 123370.055$$

$$f_y = 420.00$$

$$s = 100.00$$

V_s is multiplied by $Col = 0.00$

$$s/d = 0.3125$$

$$V_f \text{ ((11-3)-(11.4), ACI 440)} = 0.00$$

From (11-11), ACI 440: $V_s + V_f \leq 282706.38$

$$b_w \cdot d = \cdot d \cdot d/4 = 80424.772$$

Calculation of Shear Strength at edge 2, $V_{r2} = 299278.805$

$V_{r2} = V_{Col} ((10.3), ASCE 41-17) = k_{nl} * V_{Col0}$

$V_{Col0} = 299278.805$

$k_{nl} = 1$ (zero step-static loading)

NOTE: In expression (10-3) ' $V_s = A_v * f_y * d / s$ ' is replaced by ' $V_s + f * V_f$ '
where V_f is the contribution of FRPs (11.3), ACI 440).

$= 1$ (normal-weight concrete)

$f'_c = 28.00$, but $f_c^{0.5} \leq 8.3$ MPa (22.5.3.1, ACI 318-14)

$M/Vd = 2.00$

$\mu_u = 1.0979367E-011$

$\nu_u = 3.7372036E-031$

$d = 0.8 * D = 320.00$

$N_u = 4771.233$

$A_g = 125663.706$

From (11.5.4.8), ACI 318-14: $V_s = 165809.354$

$A_v = \sqrt{2} * A_{stirrup} = 123370.055$

$f_y = 420.00$

$s = 100.00$

V_s is multiplied by $Col = 0.00$

$s/d = 0.3125$

$V_f ((11-3)-(11.4), ACI 440) = 0.00$

From (11-11), ACI 440: $V_s + V_f \leq 282706.38$

$b_w * d = \sqrt{2} * d * d / 4 = 80424.772$

End Of Calculation of Shear Capacity ratio for element: column CC1 of floor 1

At local axis: 3

Start Of Calculation of Shear Capacity ratio for element: column CC1 of floor 1

At Shear local axis: 2

(Bending local axis: 3)

Section Type: rccs

Constant Properties

Knowledge Factor, $\phi = 1.00$

Mean strength values are used for both shear and moment calculations.

Consequently:

New material of Primary Member: Concrete Strength, $f_c = f_{cm} = 28.00$

New material of Primary Member: Steel Strength, $f_s = f_{sm} = 420.00$

Concrete Elasticity, $E_c = 24870.062$

Steel Elasticity, $E_s = 200000.00$

#####

Note: Especially for the calculation of moment strengths,

the above steel re-bar strengths are multiplied by 1.25 according to R18.6.5, ACI 318-14

New material: Steel Strength, $f_s = 1.25 * f_{sm} = 525.00$

#####

Diameter, $D = 400.00$

Cover Thickness, $c = 25.00$

Mean Confinement Factor overall section = 1.32561

Element Length, $L = 3000.00$

Primary Member

Smooth Bars

Ductile Steel

With Detailing for Earthquake Resistance (including stirrups closed at 135°)

Longitudinal Bars With Ends Lapped Starting at the End Sections

Adequate Lap Length ($l_o/l_{ou}, \min \geq 1$)

No FRP Wrapping

Stepwise Properties

At local axis: 2
EDGE -A-
Shear Force, $V_a = -9.8626872E-031$
EDGE -B-
Shear Force, $V_b = 9.8626872E-031$
BOTH EDGES
Axial Force, $F = -4771.233$
Longitudinal Reinforcement Area Distribution (in 2 divisions)
-Tension: $A_{st} = 0.00$
-Compression: $A_{sc} = 3053.628$
Longitudinal Reinforcement Area Distribution (in 3 divisions)
-Tension: $A_{st,ten} = 1017.876$
-Compression: $A_{st,com} = 1017.876$
-Middle: $A_{st,mid} = 1017.876$

Calculation of Shear Capacity ratio, $V_e/V_r = 0.51034251$
Member Controlled by Flexure ($V_e/V_r < 1$)
Calculation of Shear Demand from fig. R18.6.5, ACI 318-14 $V_e = (M_{pr1} + M_{pr2})/l_n = 152734.698$
with
 $M_{pr1} = \text{Max}(\mu_{u1+}, \mu_{u1-}) = 2.2910E+008$
 $\mu_{u1+} = 2.2910E+008$, is the ultimate moment strength at the edge 1 of the member in the actual moment direction which is defined for the static loading combination
 $\mu_{u1-} = 2.2910E+008$, is the ultimate moment strength at the edge 1 of the member in the opposite moment direction which is defined for the static loading combination
 $M_{pr2} = \text{Max}(\mu_{u2+}, \mu_{u2-}) = 2.2910E+008$
 $\mu_{u2+} = 2.2910E+008$, is the ultimate moment strength at the edge 2 of the member in the actual moment direction which is defined for the the static loading combination
 $\mu_{u2-} = 2.2910E+008$, is the ultimate moment strength at the edge 2 of the member in the opposite moment direction which is defined for the the static loading combination

Calculation of μ_{u1+}

Calculation of ultimate Moment Strength ((3.67), Biskinis Phd), μ_u
 $\mu_u = 2.2910E+008$

$\lambda = 1.0472$
 $\lambda' = 0.92729522$
error of function (3.68), Biskinis Phd = 58526.963
From 5A.2, TB DY: $f_{cc} = f_c^* \quad c = 37.11712$
conf. factor $c = 1.32561$
 $f_c = 28.00$
From 10.3.5, ASCE41-17, Final value of f_y : $f_y \cdot \text{Min}(1, 1.25 \cdot (l_b/d)^{2/3}) = 525.00$
 $l_b/d = 1.00$
 $d_1 = 44.00$
 $R = 200.00$
 $v = 0.00135568$
 $N = 4771.233$
 $A_c = 125663.706$
 $= \lambda' \cdot \text{Min}(1, 1.25 \cdot (l_b/d)^{2/3}) = 0.3645$

Calculation of ratio l_b/d

Adequate Lap Length: $l_b/d \geq 1$

Calculation of μ_{u1-}

Calculation of ultimate Moment Strength ((3.67), Biskinis Phd), Mu
Mu = 2.2910E+008

= 1.0472
' = 0.92729522
error of function (3.68), Biskinis Phd = 58526.963
From 5A.2, TBDY: fcc = fc* c = 37.11712
conf. factor c = 1.32561
fc = 28.00
From 10.3.5, ASCE41-17, Final value of fy: fy*Min(1,1.25*(lb/d)^ 2/3) = 525.00
lb/d = 1.00
d1 = 44.00
R = 200.00
v = 0.00135568
N = 4771.233
Ac = 125663.706
= *Min(1,1.25*(lb/d)^ 2/3) = 0.3645

Calculation of ratio lb/d

Adequate Lap Length: lb/d >= 1

Calculation of Mu2+

Calculation of ultimate Moment Strength ((3.67), Biskinis Phd), Mu
Mu = 2.2910E+008

= 1.0472
' = 0.92729522
error of function (3.68), Biskinis Phd = 58526.963
From 5A.2, TBDY: fcc = fc* c = 37.11712
conf. factor c = 1.32561
fc = 28.00
From 10.3.5, ASCE41-17, Final value of fy: fy*Min(1,1.25*(lb/d)^ 2/3) = 525.00
lb/d = 1.00
d1 = 44.00
R = 200.00
v = 0.00135568
N = 4771.233
Ac = 125663.706
= *Min(1,1.25*(lb/d)^ 2/3) = 0.3645

Calculation of ratio lb/d

Adequate Lap Length: lb/d >= 1

Calculation of Mu2-

Calculation of ultimate Moment Strength ((3.67), Biskinis Phd), Mu
Mu = 2.2910E+008

= 1.0472
' = 0.92729522
error of function (3.68), Biskinis Phd = 58526.963
From 5A.2, TBDY: fcc = fc* c = 37.11712
conf. factor c = 1.32561
fc = 28.00

From 10.3.5, ASCE41-17, Final value of f_y : $f_y \cdot \text{Min}(1, 1.25 \cdot (l_b/d)^{2/3}) = 525.00$

$$l_b/d = 1.00$$

$$d_1 = 44.00$$

$$R = 200.00$$

$$v = 0.00135568$$

$$N = 4771.233$$

$$A_c = 125663.706$$

$$= \cdot \text{Min}(1, 1.25 \cdot (l_b/d)^{2/3}) = 0.3645$$

Calculation of ratio l_b/d

Adequate Lap Length: $l_b/d \geq 1$

Calculation of Shear Strength $V_r = \text{Min}(V_{r1}, V_{r2}) = 299278.805$

Calculation of Shear Strength at edge 1, $V_{r1} = 299278.805$

$$V_{r1} = V_{\text{Col}} ((10.3), \text{ASCE 41-17}) = k_{nl} \cdot V_{\text{ColO}}$$

$$V_{\text{ColO}} = 299278.805$$

$$k_{nl} = 1 \text{ (zero step-static loading)}$$

NOTE: In expression (10-3) ' $V_s = A_v \cdot f_y \cdot d/s$ ' is replaced by ' $V_s + f \cdot V_f$ ' where V_f is the contribution of FRPs (11.3), ACI 440).

$$= 1 \text{ (normal-weight concrete)}$$

$$f'_c = 28.00, \text{ but } f'_c^{0.5} \leq 8.3 \text{ MPa (22.5.3.1, ACI 318-14)}$$

$$M/Vd = 2.00$$

$$\mu_u = 9.9556147\text{E-}012$$

$$V_u = 9.8626872\text{E-}031$$

$$d = 0.8 \cdot D = 320.00$$

$$N_u = 4771.233$$

$$A_g = 125663.706$$

$$\text{From (11.5.4.8), ACI 318-14: } V_s = 165809.354$$

$$A_v = \cdot /2 \cdot A_{\text{stirrup}} = 123370.055$$

$$f_y = 420.00$$

$$s = 100.00$$

$$V_s \text{ is multiplied by } \text{Col} = 0.00$$

$$s/d = 0.3125$$

$$V_f ((11-3)-(11.4), \text{ACI 440}) = 0.00$$

$$\text{From (11-11), ACI 440: } V_s + V_f \leq 282706.38$$

$$b_w \cdot d = \cdot d \cdot d/4 = 80424.772$$

Calculation of Shear Strength at edge 2, $V_{r2} = 299278.805$

$$V_{r2} = V_{\text{Col}} ((10.3), \text{ASCE 41-17}) = k_{nl} \cdot V_{\text{ColO}}$$

$$V_{\text{ColO}} = 299278.805$$

$$k_{nl} = 1 \text{ (zero step-static loading)}$$

NOTE: In expression (10-3) ' $V_s = A_v \cdot f_y \cdot d/s$ ' is replaced by ' $V_s + f \cdot V_f$ ' where V_f is the contribution of FRPs (11.3), ACI 440).

$$= 1 \text{ (normal-weight concrete)}$$

$$f'_c = 28.00, \text{ but } f'_c^{0.5} \leq 8.3 \text{ MPa (22.5.3.1, ACI 318-14)}$$

$$M/Vd = 2.00$$

$$\mu_u = 9.9556147\text{E-}012$$

$$V_u = 9.8626872\text{E-}031$$

$$d = 0.8 \cdot D = 320.00$$

$$N_u = 4771.233$$

$$A_g = 125663.706$$

$$\text{From (11.5.4.8), ACI 318-14: } V_s = 165809.354$$

$$A_v = \cdot /2 \cdot A_{\text{stirrup}} = 123370.055$$

$$f_y = 420.00$$

$$s = 100.00$$

$$V_s \text{ is multiplied by } \text{Col} = 0.00$$

$s/d = 0.3125$
 $V_f ((11-3)-(11.4), ACI 440) = 0.00$
 From (11-11), ACI 440: $V_s + V_f \leq 282706.38$
 $b_w \cdot d = \frac{1}{4} \cdot d \cdot d = 80424.772$

 End Of Calculation of Shear Capacity ratio for element: column CC1 of floor 1
 At local axis: 2

Start Of Calculation of Chord Rotation Capacity for element: column CC1 of floor 1
 At local axis: 3
 Integration Section: (a)
 Section Type: rccs

Constant Properties

 Knowledge Factor, $\phi = 1.00$
 Chord Rotation is generally considered as Deformation-Controlled Action according to Table C7-1, ASCE41-17.
 Mean strengths are used for Deformation-Controlled Actions according to 7.5.1.3, ASCE 41-17
 Consequently:
 New material of Primary Member: Concrete Strength, $f_c = f_{cm} = 28.00$
 New material of Primary Member: Steel Strength, $f_s = f_{sm} = 420.00$
 Concrete Elasticity, $E_c = 24870.062$
 Steel Elasticity, $E_s = 200000.00$
 Diameter, $D = 400.00$
 Cover Thickness, $c = 25.00$
 Element Length, $L = 3000.00$
 Primary Member
 Smooth Bars
 Ductile Steel
 With Detailing for Earthquake Resistance (including stirrups closed at 135°)
 Longitudinal Bars With Ends Lapped Starting at the End Sections
 Adequate Lap Length ($l_b/d \geq 1$)
 No FRP Wrapping

Stepwise Properties

 Bending Moment, $M = -1.3765E+007$
 Shear Force, $V_2 = -4586.336$
 Shear Force, $V_3 = 8.2104443E-015$
 Axial Force, $F = -4769.398$
 Longitudinal Reinforcement Area Distribution (in 2 divisions)
 -Tension: $A_{st} = 1272.345$
 -Compression: $A_{sc} = 1781.283$
 Longitudinal Reinforcement Area Distribution (in 3 divisions)
 -Tension: $A_{st,ten} = 1017.876$
 -Compression: $A_{st,com} = 1017.876$
 -Middle: $A_{st,mid} = 1017.876$
 Mean Diameter of Tension Reinforcement, $D_{bL} = 18.00$

New component: From table 7-7, ASCE 41_17: Final chord rotation Capacity $u_R = 1.0^*$ $u = 0.02795905$
 $u = y + p = 0.02795905$

 - Calculation of y -

$y = (M_y \cdot L_s / 3) / E_{eff} = 0.02295905 ((4.29), Biskinis Phd)$
 $M_y = 2.1517E+008$
 $L_s = M/V$ (with $L_s > 0.1 \cdot L$ and $L_s < 2 \cdot L$) = 3001.241
 From table 10.5, ASCE 41_17: $E_{eff} = factor \cdot E_c \cdot I_g = 9.3758E+012$

factor = 0.30
Ag = 125663.706
fc' = 28.00
N = 4769.398
Ec*Ig = 3.1253E+013

Calculation of Yielding Moment My

Calculation of μ_y and My according to (7) - (8) in Biskinis and Fardis

My = Min(My_ten, My_com) = 2.1517E+008
 μ_y ((10a) or (10b)) = 1.1483164E-005
My_ten (8a) = 2.1517E+008
 μ_{ten} (7a) = 72.79824
error of function (7a) = 0.00155286
My_com (8b) = 4.0201E+008
 μ_{com} (7b) = 69.01016
error of function (7b) = -0.00040881
with $\epsilon_y = 0.0021$
 $\epsilon_{co} = 0.002$
 $\alpha_{pl} = 0.35$ ((9a) in Biskinis and Fardis for no FRP Wrap)
d1 = 44.00
R = 200.00
 $\nu = 0.00135549$
N = 4769.398
Ac = 125663.706
= 0.3645
with fc = 28.00

Calculation of ratio lb/d

Adequate Lap Length: lb/d >= 1

- Calculation of ρ_p -

From table 10-9: $\rho_p = 0.005$

with:

- Columns controlled by inadequate development or splicing along the clear height because lb/d < 1
shear control ratio $V_y E / V_{co} I_{OE} = 0.51034251$
d = 0.00
s = 0.00
 $t = 2 * A_v / (d_c * s) + 4 * t_f / D * (f_{fe} / f_s) = 0.00$
Av = 78.53982, is the area of the circular stirrup
dc = D - 2*cover - Hoop Diameter = 340.00
The term $2 * t_f / b_w * (f_{fe} / f_s)$ is implemented to account for FRP contribution
where $f = 2 * t_f / b_w$ is FRP ratio (EC8 - 3, A.4.4.3(6)) and f_{fe} / f_s normalises f to steel strength
All these variables have already been given in Shear control ratio calculation.
NUD = 4769.398
Ag = 125663.706
fcE = 28.00
fytE = fytE = 420.00
 $\rho_l = \text{Area_Tot_Long_Rein} / (Ag) = 0.0243$
fcE = 28.00

End Of Calculation of Chord Rotation Capacity for element: column CC1 of floor 1

At local axis: 3

Integration Section: (a)

Calculation No. 5

column C1, Floor 1

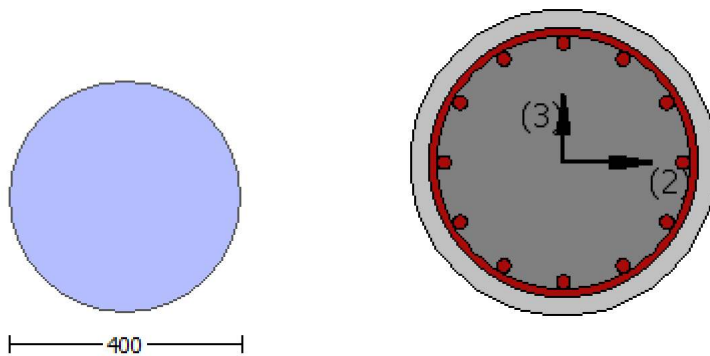
Limit State: Immediate Occupancy (data interpolation between analysis steps 1 and 2)

Analysis: Uniform +X

Check: Shear capacity VR_d

Edge: End

Local Axis: (2)



Start Of Calculation of Shear Capacity for element: column CC1 of floor 1

At local axis: 2

Integration Section: (b)

Section Type: rccs

Constant Properties

Knowledge Factor, $\gamma = 1.00$

Member Shear Force is generally considered as Force-Controlled Action according to Table C7-1, ASCE41-17.

Lower-bound strengths are used for Force-Controlled Actions according to 7.5.1.3, ASCE 41-17

Consequently:

New material of Primary Member: Concrete Strength, $f_c = f_{c_lower_bound} = 20.00$

New material of Primary Member: Steel Strength, $f_s = f_{s_lower_bound} = 400.00$

Concrete Elasticity, $E_c = 24870.062$

Steel Elasticity, $E_s = 200000.00$

#####

Note: Especially for the calculation of γ for displacement ductility demand, the expected (mean value) strengths are used (7.5.1.3, ASCE41-17) because bending is considered as Deformation-Controlled Action (Table C7-1, ASCE41-17).

New material: Concrete Strength, $f_c = f_{cm} = 28.00$

New material: Steel Strength, $f_s = f_{sm} = 420.00$

#####

Diameter, $D = 400.00$

Cover Thickness, $c = 25.00$

Element Length, $L = 3000.00$

Primary Member

Smooth Bars

Ductile Steel

With Detailing for Earthquake Resistance (including stirrups closed at 135°)

Longitudinal Bars With Ends Lapped Starting at the End Sections

Adequate Lap Length ($l_o/l_{ou,min} = l_b/l_d \geq 1$)

No FRP Wrapping

Stepwise Properties

EDGE -A-

Bending Moment, $M_a = -1.3765E+007$

Shear Force, $V_a = -4586.336$

EDGE -B-

Bending Moment, $M_b = 0.04476539$

Shear Force, $V_b = 4586.336$

BOTH EDGES

Axial Force, $F = -4769.398$

Longitudinal Reinforcement Area Distribution (in 2 divisions)

-Tension: $A_{st} = 0.00$

-Compression: $A_{sc} = 3053.628$

Longitudinal Reinforcement Area Distribution (in 3 divisions)

-Tension: $A_{st,ten} = 1017.876$

-Compression: $A_{sc,com} = 1017.876$

-Middle: $A_{st,mid} = 1017.876$

Mean Diameter of Tension Reinforcement, $Db_{L,ten} = 18.00$

New component: From table 7-7, ASCE 41_17: Final Shear Capacity $V_R = 1.0 \cdot V_n = 270862.194$

V_n ((10.3), ASCE 41-17) = $k_n \cdot V_{Col} = 270862.194$

$V_{Col} = 270862.194$

$k_n = 1.00$

$displacement_ductility_demand = 0.09594703$

NOTE: In expression (10-3) ' $V_s = A_v \cdot f_y \cdot d / s$ ' is replaced by ' $V_s + f \cdot V_f$ ' where V_f is the contribution of FRPs (11.3), ACI 440).

= 1 (normal-weight concrete)

$f'_c = 20.00$, but $f_c^{0.5} \leq 8.3$ MPa (22.5.3.1, ACI 318-14)

$M/Vd = 2.00$

$M_u = 0.04476539$

$V_u = 4586.336$

$d = 0.8 \cdot D = 320.00$

$N_u = 4769.398$

$A_g = 125663.706$

From (11.5.4.8), ACI 318-14: $V_s = 157913.67$

$A_v = \sqrt{2} \cdot A_{stirrup} = 123370.055$

$f_y = 400.00$

$s = 100.00$

V_s is multiplied by $Col = 0.00$

$s/d = 0.3125$

V_f ((11-3)-(11.4), ACI 440) = 0.00

From (11-11), ACI 440: $V_s + V_f \leq 238930.50$

$b_w \cdot d = \sqrt{2} \cdot d \cdot d / 4 = 80424.772$

$displacement_ductility_demand$ is calculated as ϕ / y

- Calculation of ϕ / y for END B -

for rotation axis 3 and integ. section (b)

From analysis, chord rotation $\phi = 0.00022019$

$y = (M_y \cdot L_s / 3) / E_{eff} = 0.00229496$ ((4.29), Biskinis Phd))

$M_y = 2.1517E+008$

$L_s = M/V$ (with $L_s > 0.1 \cdot L$ and $L_s < 2 \cdot L$) = 300.00

From table 10.5, ASCE 41_17: $E_{eff} = factor \cdot E_c \cdot I_g = 9.3758E+012$

$factor = 0.30$

$A_g = 125663.706$

$f'_c = 28.00$

$N = 4769.398$

$$E_c \cdot I_g = 3.1253E+013$$

Calculation of Yielding Moment M_y

Calculation of ρ_y and M_y according to (7) - (8) in Biskinis and Fardis

$$M_y = \min(M_{y_ten}, M_{y_com}) = 2.1517E+008$$

$$\rho_y ((10a) \text{ or } (10b)) = 1.1483164E-005$$

$$M_{y_ten} (8a) = 2.1517E+008$$

$$\rho_{y_ten} (7a) = 72.79824$$

$$\text{error of function (7a)} = 0.00155286$$

$$M_{y_com} (8b) = 4.0201E+008$$

$$\rho_{y_com} (7b) = 69.01016$$

$$\text{error of function (7b)} = -0.00040881$$

$$\text{with } e_y = 0.0021$$

$$e_{co} = 0.002$$

$$\alpha_{pl} = 0.35 ((9a) \text{ in Biskinis and Fardis for no FRP Wrap})$$

$$d_1 = 44.00$$

$$R = 200.00$$

$$v = 0.00135549$$

$$N = 4769.398$$

$$A_c = 125663.706$$

$$= 0.3645$$

$$\text{with } f_c = 28.00$$

Calculation of ratio I_b/I_d

Adequate Lap Length: $I_b/I_d \geq 1$

End Of Calculation of Shear Capacity for element: column CC1 of floor 1

At local axis: 2

Integration Section: (b)

Calculation No. 6

column C1, Floor 1

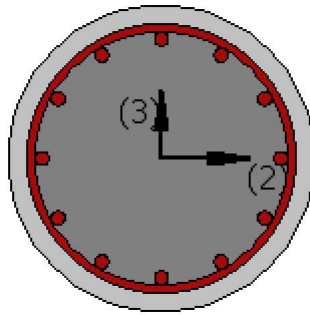
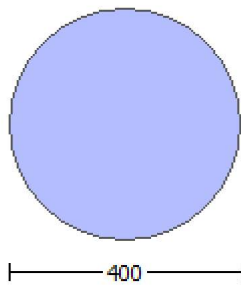
Limit State: Immediate Occupancy (data interpolation between analysis steps 1 and 2)

Analysis: Uniform +X

Check: Chord rotation capacity (ϕ_u)

Edge: End

Local Axis: (2)



Start Of Calculation of Shear Capacity ratio for element: column CC1 of floor 1

At Shear local axis: 3

(Bending local axis: 2)

Section Type: rccs

Constant Properties

Knowledge Factor, $\gamma = 1.00$

Mean strength values are used for both shear and moment calculations.

Consequently:

New material of Primary Member: Concrete Strength, $f_c = f_{cm} = 28.00$

New material of Primary Member: Steel Strength, $f_s = f_{sm} = 420.00$

Concrete Elasticity, $E_c = 24870.062$

Steel Elasticity, $E_s = 200000.00$

#####

Note: Especially for the calculation of moment strengths,

the above steel re-bar strengths are multiplied by 1.25 according to R18.6.5, ACI 318-14

New material: Steel Strength, $f_s = 1.25 \cdot f_{sm} = 525.00$

#####

Diameter, $D = 400.00$

Cover Thickness, $c = 25.00$

Mean Confinement Factor overall section = 1.32561

Element Length, $L = 3000.00$

Primary Member

Smooth Bars

Ductile Steel

With Detailing for Earthquake Resistance (including stirrups closed at 135°)

Longitudinal Bars With Ends Lapped Starting at the End Sections

Adequate Lap Length ($l_o/l_{ou, \min} \geq 1$)

No FRP Wrapping

Stepwise Properties

At local axis: 3

EDGE -A-

Shear Force, $V_a = -3.7372036E-031$

EDGE -B-

Shear Force, $V_b = 3.7372036E-031$

BOTH EDGES

Axial Force, $F = -4771.233$

Longitudinal Reinforcement Area Distribution (in 2 divisions)

-Tension: $As_t = 0.00$

-Compression: $As_c = 3053.628$

Longitudinal Reinforcement Area Distribution (in 3 divisions)

-Tension: $As_{t, \text{ten}} = 1017.876$

-Compression: $As_{c, \text{com}} = 1017.876$

-Middle: $As_{c, \text{mid}} = 1017.876$

Calculation of Shear Capacity ratio , $V_e/V_r = 0.51034251$

Member Controlled by Flexure ($V_e/V_r < 1$)

Calculation of Shear Demand from fig. R18.6.5, ACI 318-14 $V_e = (M_{pr1} + M_{pr2})/l_n = 152734.698$

with

$M_{pr1} = \text{Max}(M_{u1+}, M_{u1-}) = 2.2910\text{E}+008$

$M_{u1+} = 2.2910\text{E}+008$, is the ultimate moment strength at the edge 1 of the member in the actual moment direction which is defined for the static loading combination

$M_{u1-} = 2.2910\text{E}+008$, is the ultimate moment strength at the edge 1 of the member in the opposite moment direction which is defined for the static loading combination

$M_{pr2} = \text{Max}(M_{u2+}, M_{u2-}) = 2.2910\text{E}+008$

$M_{u2+} = 2.2910\text{E}+008$, is the ultimate moment strength at the edge 2 of the member in the actual moment direction which is defined for the static loading combination

$M_{u2-} = 2.2910\text{E}+008$, is the ultimate moment strength at the edge 2 of the member in the opposite moment direction which is defined for the static loading combination

Calculation of M_{u1+}

Calculation of ultimate Moment Strength ((3.67), Biskinis Phd), M_u

$M_u = 2.2910\text{E}+008$

$\phi = 1.0472$

$\phi' = 0.92729522$

error of function (3.68), Biskinis Phd = 58526.963

From 5A.2, TB DY: $f_{cc} = f_c \cdot c = 37.11712$

conf. factor $c = 1.32561$

$f_c = 28.00$

From 10.3.5, ASCE41-17, Final value of f_y : $f_y \cdot \text{Min}(1, 1.25 \cdot (l_b/d)^{2/3}) = 525.00$

$l_b/d = 1.00$

$d_1 = 44.00$

$R = 200.00$

$v = 0.00135568$

$N = 4771.233$

$A_c = 125663.706$

$= \phi' \cdot \text{Min}(1, 1.25 \cdot (l_b/d)^{2/3}) = 0.3645$

Calculation of ratio l_b/d

Adequate Lap Length: $l_b/d \geq 1$

Calculation of M_{u1-}

Calculation of ultimate Moment Strength ((3.67), Biskinis Phd), M_u

$M_u = 2.2910\text{E}+008$

$\phi = 1.0472$

$\phi' = 0.92729522$

error of function (3.68), Biskinis Phd = 58526.963

From 5A.2, TB DY: $f_{cc} = f_c \cdot c = 37.11712$

conf. factor $c = 1.32561$

$f_c = 28.00$

From 10.3.5, ASCE41-17, Final value of f_y : $f_y \cdot \text{Min}(1, 1.25 \cdot (l_b/d)^{2/3}) = 525.00$

$l_b/d = 1.00$

$d_1 = 44.00$

$R = 200.00$

$v = 0.00135568$

$N = 4771.233$

$A_c = 125663.706$

$= \phi' \cdot \text{Min}(1, 1.25 \cdot (l_b/d)^{2/3}) = 0.3645$

Calculation of ratio l_b/d

Adequate Lap Length: $l_b/d \geq 1$

Calculation of μ_{2+}

Calculation of ultimate Moment Strength ((3.67), Biskinis Phd), μ
 $\mu = 2.2910E+008$

$$= 1.0472$$

$$' = 0.92729522$$

error of function (3.68), Biskinis Phd = 58526.963

From 5A.2, TBDY: $f_{cc} = f_c \cdot c = 37.11712$

conf. factor $c = 1.32561$

$f_c = 28.00$

From 10.3.5, ASCE41-17, Final value of f_y : $f_y \cdot \text{Min}(1, 1.25 \cdot (l_b/d)^{2/3}) = 525.00$

$$l_b/d = 1.00$$

$$d_1 = 44.00$$

$$R = 200.00$$

$$v = 0.00135568$$

$$N = 4771.233$$

$$A_c = 125663.706$$

$$= \cdot \text{Min}(1, 1.25 \cdot (l_b/d)^{2/3}) = 0.3645$$

Calculation of ratio l_b/d

Adequate Lap Length: $l_b/d \geq 1$

Calculation of μ_{2-}

Calculation of ultimate Moment Strength ((3.67), Biskinis Phd), μ
 $\mu = 2.2910E+008$

$$= 1.0472$$

$$' = 0.92729522$$

error of function (3.68), Biskinis Phd = 58526.963

From 5A.2, TBDY: $f_{cc} = f_c \cdot c = 37.11712$

conf. factor $c = 1.32561$

$f_c = 28.00$

From 10.3.5, ASCE41-17, Final value of f_y : $f_y \cdot \text{Min}(1, 1.25 \cdot (l_b/d)^{2/3}) = 525.00$

$$l_b/d = 1.00$$

$$d_1 = 44.00$$

$$R = 200.00$$

$$v = 0.00135568$$

$$N = 4771.233$$

$$A_c = 125663.706$$

$$= \cdot \text{Min}(1, 1.25 \cdot (l_b/d)^{2/3}) = 0.3645$$

Calculation of ratio l_b/d

Adequate Lap Length: $l_b/d \geq 1$

Calculation of Shear Strength $V_r = \text{Min}(V_{r1}, V_{r2}) = 299278.805$

Calculation of Shear Strength at edge 1, $V_{r1} = 299278.805$

$V_{r1} = V_{Col} \text{ ((10.3), ASCE 41-17)} = k_{nl} * V_{Col0}$

$V_{Col0} = 299278.805$

$k_{nl} = 1$ (zero step-static loading)

NOTE: In expression (10-3) ' $V_s = A_v * f_y * d / s$ ' is replaced by ' $V_s + f * V_f$ ' where V_f is the contribution of FRPs (11.3), ACI 440).

$= 1$ (normal-weight concrete)

$f'_c = 28.00$, but $f'_c^{0.5} \leq 8.3$ MPa (22.5.3.1, ACI 318-14)

$M/Vd = 2.00$

$\mu_u = 1.0979367E-011$

$\nu_u = 3.7372036E-031$

$d = 0.8 * D = 320.00$

$N_u = 4771.233$

$A_g = 125663.706$

From (11.5.4.8), ACI 318-14: $V_s = 165809.354$

$A_v = \sqrt{2} * A_{stirrup} = 123370.055$

$f_y = 420.00$

$s = 100.00$

V_s is multiplied by $Col = 0.00$

$s/d = 0.3125$

$V_f \text{ ((11-3)-(11.4), ACI 440)} = 0.00$

From (11-11), ACI 440: $V_s + V_f \leq 282706.38$

$b_w * d = \sqrt{3} * d * d / 4 = 80424.772$

Calculation of Shear Strength at edge 2, $V_{r2} = 299278.805$

$V_{r2} = V_{Col} \text{ ((10.3), ASCE 41-17)} = k_{nl} * V_{Col0}$

$V_{Col0} = 299278.805$

$k_{nl} = 1$ (zero step-static loading)

NOTE: In expression (10-3) ' $V_s = A_v * f_y * d / s$ ' is replaced by ' $V_s + f * V_f$ ' where V_f is the contribution of FRPs (11.3), ACI 440).

$= 1$ (normal-weight concrete)

$f'_c = 28.00$, but $f'_c^{0.5} \leq 8.3$ MPa (22.5.3.1, ACI 318-14)

$M/Vd = 2.00$

$\mu_u = 1.0979367E-011$

$\nu_u = 3.7372036E-031$

$d = 0.8 * D = 320.00$

$N_u = 4771.233$

$A_g = 125663.706$

From (11.5.4.8), ACI 318-14: $V_s = 165809.354$

$A_v = \sqrt{2} * A_{stirrup} = 123370.055$

$f_y = 420.00$

$s = 100.00$

V_s is multiplied by $Col = 0.00$

$s/d = 0.3125$

$V_f \text{ ((11-3)-(11.4), ACI 440)} = 0.00$

From (11-11), ACI 440: $V_s + V_f \leq 282706.38$

$b_w * d = \sqrt{3} * d * d / 4 = 80424.772$

End Of Calculation of Shear Capacity ratio for element: column CC1 of floor 1

At local axis: 3

Start Of Calculation of Shear Capacity ratio for element: column CC1 of floor 1

At Shear local axis: 2

(Bending local axis: 3)

Section Type: rccs

Constant Properties

Knowledge Factor, $\phi = 1.00$
 Mean strength values are used for both shear and moment calculations.
 Consequently:
 New material of Primary Member: Concrete Strength, $f_c = f_{cm} = 28.00$
 New material of Primary Member: Steel Strength, $f_s = f_{sm} = 420.00$
 Concrete Elasticity, $E_c = 24870.062$
 Steel Elasticity, $E_s = 200000.00$
 #####
 Note: Especially for the calculation of moment strengths,
 the above steel re-bar strengths are multiplied by 1.25 according to R18.6.5, ACI 318-14
 New material: Steel Strength, $f_s = 1.25 \cdot f_{sm} = 525.00$
 #####
 Diameter, $D = 400.00$
 Cover Thickness, $c = 25.00$
 Mean Confinement Factor overall section = 1.32561
 Element Length, $L = 3000.00$
 Primary Member
 Smooth Bars
 Ductile Steel
 With Detailing for Earthquake Resistance (including stirrups closed at 135°)
 Longitudinal Bars With Ends Lapped Starting at the End Sections
 Adequate Lap Length ($l_o/l_{o,min} \geq 1$)
 No FRP Wrapping

Stepwise Properties

At local axis: 2
 EDGE -A-
 Shear Force, $V_a = -9.8626872E-031$
 EDGE -B-
 Shear Force, $V_b = 9.8626872E-031$
 BOTH EDGES
 Axial Force, $F = -4771.233$
 Longitudinal Reinforcement Area Distribution (in 2 divisions)
 -Tension: $A_{sl,t} = 0.00$
 -Compression: $A_{sl,c} = 3053.628$
 Longitudinal Reinforcement Area Distribution (in 3 divisions)
 -Tension: $A_{sl,ten} = 1017.876$
 -Compression: $A_{sl,com} = 1017.876$
 -Middle: $A_{sl,mid} = 1017.876$

Calculation of Shear Capacity ratio, $V_e/V_r = 0.51034251$
 Member Controlled by Flexure ($V_e/V_r < 1$)
 Calculation of Shear Demand from fig. R18.6.5, ACI 318-14 $V_e = (M_{pr1} + M_{pr2})/l_n = 152734.698$
 with
 $M_{pr1} = \max(\mu_{u1+}, \mu_{u1-}) = 2.2910E+008$
 $\mu_{u1+} = 2.2910E+008$, is the ultimate moment strength at the edge 1 of the member in the actual moment direction
 which is defined for the static loading combination
 $\mu_{u1-} = 2.2910E+008$, is the ultimate moment strength at the edge 1 of the member in the opposite moment
 direction which is defined for the static loading combination
 $M_{pr2} = \max(\mu_{u2+}, \mu_{u2-}) = 2.2910E+008$
 $\mu_{u2+} = 2.2910E+008$, is the ultimate moment strength at the edge 2 of the member in the actual moment direction
 which is defined for the the static loading combination
 $\mu_{u2-} = 2.2910E+008$, is the ultimate moment strength at the edge 2 of the member in the opposite moment
 direction which is defined for the the static loading combination

Calculation of μ_{u1+}

Calculation of ultimate Moment Strength ((3.67), Biskinis Phd), μ_u

Mu = 2.2910E+008

= 1.0472

' = 0.92729522

error of function (3.68), Biskinis Phd = 58526.963

From 5A.2, TBDY: $f_{cc} = f_c \cdot c = 37.11712$

conf. factor $c = 1.32561$

$f_c = 28.00$

From 10.3.5, ASCE41-17, Final value of f_y : $f_y \cdot \text{Min}(1, 1.25 \cdot (l_b/l_d)^{2/3}) = 525.00$

$l_b/l_d = 1.00$

$d_1 = 44.00$

$R = 200.00$

$v = 0.00135568$

$N = 4771.233$

$A_c = 125663.706$

= $\cdot \text{Min}(1, 1.25 \cdot (l_b/l_d)^{2/3}) = 0.3645$

Calculation of ratio l_b/l_d

Adequate Lap Length: $l_b/l_d \geq 1$

Calculation of Mu1-

Calculation of ultimate Moment Strength ((3.67), Biskinis Phd), Mu

Mu = 2.2910E+008

= 1.0472

' = 0.92729522

error of function (3.68), Biskinis Phd = 58526.963

From 5A.2, TBDY: $f_{cc} = f_c \cdot c = 37.11712$

conf. factor $c = 1.32561$

$f_c = 28.00$

From 10.3.5, ASCE41-17, Final value of f_y : $f_y \cdot \text{Min}(1, 1.25 \cdot (l_b/l_d)^{2/3}) = 525.00$

$l_b/l_d = 1.00$

$d_1 = 44.00$

$R = 200.00$

$v = 0.00135568$

$N = 4771.233$

$A_c = 125663.706$

= $\cdot \text{Min}(1, 1.25 \cdot (l_b/l_d)^{2/3}) = 0.3645$

Calculation of ratio l_b/l_d

Adequate Lap Length: $l_b/l_d \geq 1$

Calculation of Mu2+

Calculation of ultimate Moment Strength ((3.67), Biskinis Phd), Mu

Mu = 2.2910E+008

= 1.0472

' = 0.92729522

error of function (3.68), Biskinis Phd = 58526.963

From 5A.2, TBDY: $f_{cc} = f_c \cdot c = 37.11712$

conf. factor $c = 1.32561$

$f_c = 28.00$

From 10.3.5, ASCE41-17, Final value of f_y : $f_y \cdot \text{Min}(1, 1.25 \cdot (l_b/l_d)^{2/3}) = 525.00$

$l_b/d = 1.00$
 $d_1 = 44.00$
 $R = 200.00$
 $v = 0.00135568$
 $N = 4771.233$
 $A_c = 125663.706$
 $= \text{Min}(1, 1.25 \cdot (l_b/d)^{2/3}) = 0.3645$

Calculation of ratio l_b/d

Adequate Lap Length: $l_b/d \geq 1$

Calculation of μ_2

Calculation of ultimate Moment Strength ((3.67), Biskinis Phd), μ
 $\mu = 2.2910 \times 10^8$

$= 1.0472$
 $' = 0.92729522$
 error of function (3.68), Biskinis Phd = 58526.963
 From 5A.2, TBDY: $f_{cc} = f_c \cdot c = 37.11712$
 conf. factor $c = 1.32561$
 $f_c = 28.00$
 From 10.3.5, ASCE41-17, Final value of f_y : $f_y \cdot \text{Min}(1, 1.25 \cdot (l_b/d)^{2/3}) = 525.00$
 $l_b/d = 1.00$
 $d_1 = 44.00$
 $R = 200.00$
 $v = 0.00135568$
 $N = 4771.233$
 $A_c = 125663.706$
 $= \text{Min}(1, 1.25 \cdot (l_b/d)^{2/3}) = 0.3645$

Calculation of ratio l_b/d

Adequate Lap Length: $l_b/d \geq 1$

Calculation of Shear Strength $V_r = \text{Min}(V_{r1}, V_{r2}) = 299278.805$

Calculation of Shear Strength at edge 1, $V_{r1} = 299278.805$

$V_{r1} = V_{co1}$ ((10.3), ASCE 41-17) = $k_{nl} \cdot V_{co1}$
 $V_{co1} = 299278.805$
 $k_{nl} = 1$ (zero step-static loading)

NOTE: In expression (10-3) ' $V_s = A_v \cdot f_y \cdot d/s$ ' is replaced by ' $V_s + f \cdot V_f$ ' where V_f is the contribution of FRPs (11.3), ACI 440).

$= 1$ (normal-weight concrete)
 $f_c' = 28.00$, but $f_c'^{0.5} \leq 8.3$ MPa (22.5.3.1, ACI 318-14)
 $M/Vd = 2.00$
 $\mu = 9.9556147 \times 10^{-12}$
 $V_u = 9.8626872 \times 10^{-31}$
 $d = 0.8 \cdot D = 320.00$
 $N_u = 4771.233$
 $A_g = 125663.706$
 From (11.5.4.8), ACI 318-14: $V_s = 165809.354$
 $A_v = \frac{1}{2} \cdot A_{stirrup} = 123370.055$
 $f_y = 420.00$
 $s = 100.00$

Vs is multiplied by Col = 0.00
s/d = 0.3125
Vf ((11-3)-(11.4), ACI 440) = 0.00
From (11-11), ACI 440: Vs + Vf <= 282706.38
bw*d = *d*d/4 = 80424.772

Calculation of Shear Strength at edge 2, Vr2 = 299278.805
Vr2 = VCol ((10.3), ASCE 41-17) = knl*VCol0
VCol0 = 299278.805
knl = 1 (zero step-static loading)

NOTE: In expression (10-3) 'Vs = Av*fy*d/s' is replaced by 'Vs+ f*VF'
where Vf is the contribution of FRPs (11.3), ACI 440).

= 1 (normal-weight concrete)
fc' = 28.00, but $fc'^{0.5} \leq 8.3$ MPa (22.5.3.1, ACI 318-14)
M/Vd = 2.00
Mu = 9.9556147E-012
Vu = 9.8626872E-031
d = 0.8*D = 320.00
Nu = 4771.233
Ag = 125663.706
From (11.5.4.8), ACI 318-14: Vs = 165809.354
Av = /2*A_stirrup = 123370.055
fy = 420.00
s = 100.00
Vs is multiplied by Col = 0.00
s/d = 0.3125
Vf ((11-3)-(11.4), ACI 440) = 0.00
From (11-11), ACI 440: Vs + Vf <= 282706.38
bw*d = *d*d/4 = 80424.772

End Of Calculation of Shear Capacity ratio for element: column CC1 of floor 1
At local axis: 2

Start Of Calculation of Chord Rotation Capacity for element: column CC1 of floor 1
At local axis: 2
Integration Section: (b)
Section Type: rccs

Constant Properties

Knowledge Factor, = 1.00
Chord Rotation is generally considered as Deformation-Controlled Action according to Table C7-1, ASCE41-17.
Mean strengths are used for Deformation-Controlled Actions according to 7.5.1.3, ASCE 41-17
Consequently:
New material of Primary Member: Concrete Strength, fc = fcm = 28.00
New material of Primary Member: Steel Strength, fs = fsm = 420.00
Concrete Elasticity, Ec = 24870.062
Steel Elasticity, Es = 200000.00
Diameter, D = 400.00
Cover Thickness, c = 25.00
Element Length, L = 3000.00
Primary Member
Smooth Bars
Ductile Steel
With Detailing for Earthquake Resistance (including stirrups closed at 135°)
Longitudinal Bars With Ends Lapped Starting at the End Sections
Adequate Lap Length ($l_b/d \geq 1$)
No FRP Wrapping

Stepwise Properties

Bending Moment, $M = -7.7176173\text{E-}012$

Shear Force, $V2 = 4586.336$

Shear Force, $V3 = -8.2104443\text{E-}015$

Axial Force, $F = -4769.398$

Longitudinal Reinforcement Area Distribution (in 2 divisions)

-Tension: $As_t = 0.00$

-Compression: $As_c = 3053.628$

Longitudinal Reinforcement Area Distribution (in 3 divisions)

-Tension: $As_{t,ten} = 1017.876$

-Compression: $As_{c,com} = 1017.876$

-Middle: $As_{mid} = 1017.876$

Mean Diameter of Tension Reinforcement, $Db_L = 18.00$

New component: From table 7-7, ASCE 41_17: Final chord rotation Capacity $u_R = 1.0^*$ $u = 0.01647478$

$u = y + p = 0.01647478$

- Calculation of y -

$y = (My * L_s / 3) / E_{eff} = 0.01147478$ ((4.29), Biskinis Phd))

$My = 2.1517\text{E+}008$

$L_s = M/V$ (with $L_s > 0.1 * L$ and $L_s < 2 * L$) = 1500.00

From table 10.5, ASCE 41_17: $E_{eff} = \text{factor} * E_c * I_g = 9.3758\text{E+}012$

factor = 0.30

$A_g = 125663.706$

$f_c' = 28.00$

$N = 4769.398$

$E_c * I_g = 3.1253\text{E+}013$

Calculation of Yielding Moment My

Calculation of y and My according to (7) - (8) in Biskinis and Fardis

$My = \text{Min}(My_{ten}, My_{com}) = 2.1517\text{E+}008$

y ((10a) or (10b)) = 1.1483164E-005

My_{ten} (8a) = 2.1517E+008

y_{ten} (7a) = 72.79824

error of function (7a) = 0.00155286

My_{com} (8b) = 4.0201E+008

y_{com} (7b) = 69.01016

error of function (7b) = -0.00040881

with $e_y = 0.0021$

$e_{co} = 0.002$

$a_{pl} = 0.35$ ((9a) in Biskinis and Fardis for no FRP Wrap)

$d_1 = 44.00$

$R = 200.00$

$v = 0.00135549$

$N = 4769.398$

$A_c = 125663.706$

= 0.3645

with $f_c = 28.00$

Calculation of ratio l_b / l_d

Adequate Lap Length: $l_b / l_d \geq 1$

- Calculation of p -

From table 10-9: $p = 0.005$

with:

- Columns controlled by inadequate development or splicing along the clear height because $l_b/l_d < 1$

shear control ratio $V_y E / V_{col} E = 0.51034251$

$d = 0.00$

$s = 0.00$

$t = 2 \cdot A_v / (d_c \cdot s) + 4 \cdot t_f / D \cdot (f_{fe} / f_s) = 0.00$

$A_v = 78.53982$, is the area of the circular stirrup

$d_c = D - 2 \cdot \text{cover}$ - Hoop Diameter = 340.00

The term $2 \cdot t_f / b_w \cdot (f_{fe} / f_s)$ is implemented to account for FRP contribution

where $f = 2 \cdot t_f / b_w$ is FRP ratio (EC8 - 3, A.4.4.3(6)) and f_{fe} / f_s normalises f to steel strength

All these variables have already been given in Shear control ratio calculation.

$NUD = 4769.398$

$A_g = 125663.706$

$f_{cE} = 28.00$

$f_{yE} = f_{yL} = 420.00$

$p_l = \text{Area_Tot_Long_Rein} / (A_g) = 0.0243$

$f_{cE} = 28.00$

End Of Calculation of Chord Rotation Capacity for element: column CC1 of floor 1

At local axis: 2

Integration Section: (b)

Calculation No. 7

column C1, Floor 1

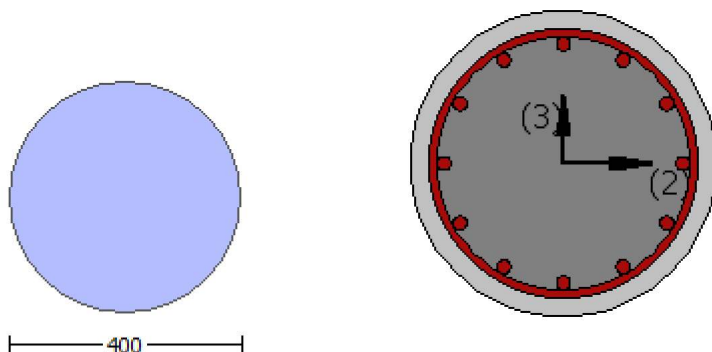
Limit State: Immediate Occupancy (data interpolation between analysis steps 1 and 2)

Analysis: Uniform +X

Check: Shear capacity VR_d

Edge: End

Local Axis: (3)



Start Of Calculation of Shear Capacity for element: column CC1 of floor 1

At local axis: 3

Integration Section: (b)

Section Type: rccs

Constant Properties

Knowledge Factor, $\gamma = 1.00$

Member Shear Force is generally considered as Force-Controlled Action according to Table C7-1, ASCE41-17.

Lower-bound strengths are used for Force-Controlled Actions according to 7.5.1.3, ASCE 41-17

Consequently:

New material of Primary Member: Concrete Strength, $f_c = f_{c_lower_bound} = 20.00$

New material of Primary Member: Steel Strength, $f_s = f_{s_lower_bound} = 400.00$

Concrete Elasticity, $E_c = 24870.062$

Steel Elasticity, $E_s = 200000.00$

#####

Note: Especially for the calculation of γ for displacement ductility demand, the expected (mean value) strengths are used (7.5.1.3, ASCE41-17) because bending is considered as Deformation-Controlled Action (Table C7-1, ASCE41-17).

New material: Concrete Strength, $f_c = f_{cm} = 28.00$

New material: Steel Strength, $f_s = f_{sm} = 420.00$

#####

Diameter, $D = 400.00$

Cover Thickness, $c = 25.00$

Element Length, $L = 3000.00$

Primary Member

Smooth Bars

Ductile Steel

With Detailing for Earthquake Resistance (including stirrups closed at 135°)

Longitudinal Bars With Ends Lapped Starting at the End Sections

Adequate Lap Length ($l_o/l_{ou,min} = l_b/l_d \geq 1$)

No FRP Wrapping

Stepwise Properties

EDGE -A-

Bending Moment, $M_a = -1.6986061E-011$

Shear Force, $V_a = 8.2104443E-015$

EDGE -B-

Bending Moment, $M_b = -7.7176173E-012$

Shear Force, $V_b = -8.2104443E-015$

BOTH EDGES

Axial Force, $F = -4769.398$

Longitudinal Reinforcement Area Distribution (in 2 divisions)

-Tension: $As_t = 0.00$

-Compression: $As_c = 3053.628$

Longitudinal Reinforcement Area Distribution (in 3 divisions)

-Tension: $As_{t,ten} = 1017.876$

-Compression: $As_{l,com} = 1017.876$

-Middle: $As_{l,mid} = 1017.876$

Mean Diameter of Tension Reinforcement, $Db_{L,ten} = 18.00$

New component: From table 7-7, ASCE 41_17: Final Shear Capacity $V_R = 1.0 \cdot V_n = 270862.194$

V_n ((10.3), ASCE 41-17) = $kn_l \cdot V_{CoIO} = 270862.194$

$V_{CoI} = 270862.194$

$kn_l = 1.00$

displacement_ductility_demand = 0.00

NOTE: In expression (10-3) ' $V_s = A_v \cdot f_y \cdot d / s$ ' is replaced by ' $V_{s+} + f^* V_f$ ' where V_f is the contribution of FRPs (11.3), ACI 440).

= 1 (normal-weight concrete)

$f'_c = 20.00$, but $f'_c^{0.5} \leq 8.3$ MPa (22.5.3.1, ACI 318-14)

$M/Vd = 2.00$

$M_u = 7.7176173E-012$

$V_u = 8.2104443E-015$

$d = 0.8 \cdot D = 320.00$

$N_u = 4769.398$

$A_g = 125663.706$
 From (11.5.4.8), ACI 318-14: $V_s = 157913.67$
 $A_v = \sqrt{2} A_{stirrup} = 123370.055$
 $f_y = 400.00$
 $s = 100.00$
 V_s is multiplied by $Col = 0.00$
 $s/d = 0.3125$
 $V_f ((11-3)-(11.4), ACI 440) = 0.00$
 From (11-11), ACI 440: $V_s + V_f \leq 238930.50$
 $b_w d = \sqrt{2} d^2 / 4 = 80424.772$

displacement ductility demand is calculated as δ / y

- Calculation of δ / y for END B -
 for rotation axis 2 and integ. section (b)

From analysis, chord rotation $\theta = 3.9748299E-022$
 $y = (M_y * L_s / 3) / E_{eff} = 0.01147478 ((4.29), Biskinis Phd)$
 $M_y = 2.1517E+008$
 $L_s = M/V$ (with $L_s > 0.1 * L$ and $L_s < 2 * L$) = 1500.00
 From table 10.5, ASCE 41_17: $E_{eff} = factor * E_c * I_g = 9.3758E+012$
 $factor = 0.30$
 $A_g = 125663.706$
 $f_c' = 28.00$
 $N = 4769.398$
 $E_c * I_g = 3.1253E+013$

Calculation of Yielding Moment M_y

Calculation of δ and M_y according to (7) - (8) in Biskinis and Fardis

$M_y = \min(M_{y_ten}, M_{y_com}) = 2.1517E+008$
 $y ((10a) \text{ or } (10b)) = 1.1483164E-005$
 $M_{y_ten} (8a) = 2.1517E+008$
 $\delta_{ten} (7a) = 72.79824$
 $error \text{ of function } (7a) = 0.00155286$
 $M_{y_com} (8b) = 4.0201E+008$
 $\delta_{com} (7b) = 69.01016$
 $error \text{ of function } (7b) = -0.00040881$
 with $e_y = 0.0021$
 $e_{co} = 0.002$
 $apl = 0.35 ((9a) \text{ in Biskinis and Fardis for no FRP Wrap})$
 $d1 = 44.00$
 $R = 200.00$
 $v = 0.00135549$
 $N = 4769.398$
 $A_c = 125663.706$
 $= 0.3645$
 with $f_c = 28.00$

Calculation of ratio I_b / I_d

Adequate Lap Length: $I_b / I_d \geq 1$

End Of Calculation of Shear Capacity for element: column CC1 of floor 1

At local axis: 3

Integration Section: (b)

Calculation No. 8

column C1, Floor 1

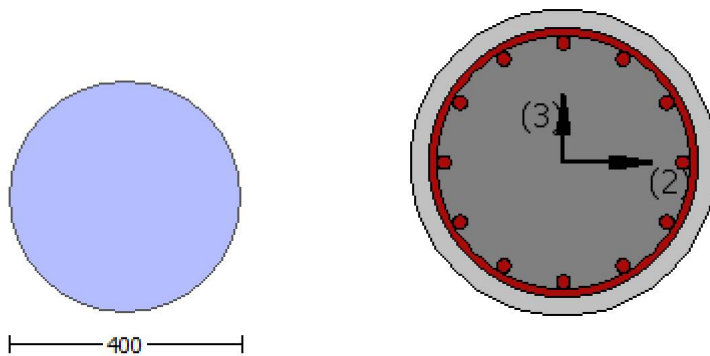
Limit State: Immediate Occupancy (data interpolation between analysis steps 1 and 2)

Analysis: Uniform +X

Check: Chord rotation capacity (ϕ)

Edge: End

Local Axis: (3)



Start Of Calculation of Shear Capacity ratio for element: column CC1 of floor 1

At Shear local axis: 3

(Bending local axis: 2)

Section Type: rccs

Constant Properties

Knowledge Factor, $\gamma = 1.00$

Mean strength values are used for both shear and moment calculations.

Consequently:

New material of Primary Member: Concrete Strength, $f_c = f_{cm} = 28.00$

New material of Primary Member: Steel Strength, $f_s = f_{sm} = 420.00$

Concrete Elasticity, $E_c = 24870.062$

Steel Elasticity, $E_s = 200000.00$

#####

Note: Especially for the calculation of moment strengths,

the above steel re-bar strengths are multiplied by 1.25 according to R18.6.5, ACI 318-14

New material: Steel Strength, $f_s = 1.25 \cdot f_{sm} = 525.00$

#####

Diameter, $D = 400.00$

Cover Thickness, $c = 25.00$

Mean Confinement Factor overall section = 1.32561

Element Length, $L = 3000.00$

Primary Member

Smooth Bars

Ductile Steel

With Detailing for Earthquake Resistance (including stirrups closed at 135°)

Longitudinal Bars With Ends Lapped Starting at the End Sections

Adequate Lap Length ($l_o/l_{ou,min} \geq 1$)

No FRP Wrapping

Stepwise Properties

At local axis: 3

EDGE -A-

Shear Force, $V_a = -3.7372036E-031$

EDGE -B-

Shear Force, $V_b = 3.7372036E-031$

BOTH EDGES

Axial Force, $F = -4771.233$

Longitudinal Reinforcement Area Distribution (in 2 divisions)

-Tension: $As_t = 0.00$

-Compression: $As_c = 3053.628$

Longitudinal Reinforcement Area Distribution (in 3 divisions)

-Tension: $As_{t,ten} = 1017.876$

-Compression: $As_{l,com} = 1017.876$

-Middle: $As_{l,mid} = 1017.876$

Calculation of Shear Capacity ratio, $V_e/V_r = 0.51034251$

Member Controlled by Flexure ($V_e/V_r < 1$)

Calculation of Shear Demand from fig. R18.6.5, ACI 318-14 $V_e = (M_{pr1} + M_{pr2})/l_n = 152734.698$
with

$M_{pr1} = \text{Max}(\mu_{1+}, \mu_{1-}) = 2.2910E+008$

$\mu_{1+} = 2.2910E+008$, is the ultimate moment strength at the edge 1 of the member in the actual moment direction which is defined for the static loading combination

$\mu_{1-} = 2.2910E+008$, is the ultimate moment strength at the edge 1 of the member in the opposite moment direction which is defined for the static loading combination

$M_{pr2} = \text{Max}(\mu_{2+}, \mu_{2-}) = 2.2910E+008$

$\mu_{2+} = 2.2910E+008$, is the ultimate moment strength at the edge 2 of the member in the actual moment direction which is defined for the static loading combination

$\mu_{2-} = 2.2910E+008$, is the ultimate moment strength at the edge 2 of the member in the opposite moment direction which is defined for the static loading combination

Calculation of μ_{1+}

Calculation of ultimate Moment Strength ((3.67), Biskinis Phd), μ_u

$\mu_u = 2.2910E+008$

$\lambda = 1.0472$

$\lambda' = 0.92729522$

error of function (3.68), Biskinis Phd = 58526.963

From 5A.2, TDY: $f_{cc} = f_c^* \quad c = 37.11712$

conf. factor $c = 1.32561$

$f_c = 28.00$

From 10.3.5, ASCE41-17, Final value of f_y : $f_y \cdot \text{Min}(1, 1.25 \cdot (l_b/d)^{2/3}) = 525.00$

$l_b/d = 1.00$

$d_1 = 44.00$

$R = 200.00$

$v = 0.00135568$

$N = 4771.233$

$A_c = 125663.706$

$= \lambda \cdot \text{Min}(1, 1.25 \cdot (l_b/d)^{2/3}) = 0.3645$

Calculation of ratio l_b/d

Adequate Lap Length: $l_b/d \geq 1$

Calculation of μ_{1-}

Calculation of ultimate Moment Strength ((3.67), Biskinis Phd), Mu
Mu = 2.2910E+008

= 1.0472
' = 0.92729522
error of function (3.68), Biskinis Phd = 58526.963
From 5A.2, TDY: fcc = fc* c = 37.11712
conf. factor c = 1.32561
fc = 28.00
From 10.3.5, ASCE41-17, Final value of fy: fy*Min(1,1.25*(lb/d)^ 2/3) = 525.00
lb/d = 1.00
d1 = 44.00
R = 200.00
v = 0.00135568
N = 4771.233
Ac = 125663.706
= *Min(1,1.25*(lb/d)^ 2/3) = 0.3645

Calculation of ratio lb/d

Adequate Lap Length: lb/d >= 1

Calculation of Mu2+

Calculation of ultimate Moment Strength ((3.67), Biskinis Phd), Mu
Mu = 2.2910E+008

= 1.0472
' = 0.92729522
error of function (3.68), Biskinis Phd = 58526.963
From 5A.2, TDY: fcc = fc* c = 37.11712
conf. factor c = 1.32561
fc = 28.00
From 10.3.5, ASCE41-17, Final value of fy: fy*Min(1,1.25*(lb/d)^ 2/3) = 525.00
lb/d = 1.00
d1 = 44.00
R = 200.00
v = 0.00135568
N = 4771.233
Ac = 125663.706
= *Min(1,1.25*(lb/d)^ 2/3) = 0.3645

Calculation of ratio lb/d

Adequate Lap Length: lb/d >= 1

Calculation of Mu2-

Calculation of ultimate Moment Strength ((3.67), Biskinis Phd), Mu
Mu = 2.2910E+008

= 1.0472
' = 0.92729522
error of function (3.68), Biskinis Phd = 58526.963
From 5A.2, TDY: fcc = fc* c = 37.11712

conf. factor $c = 1.32561$
 $f_c = 28.00$
 From 10.3.5, ASCE41-17, Final value of f_y : $f_y \cdot \text{Min}(1, 1.25 \cdot (l_b/d)^{2/3}) = 525.00$
 $l_b/d = 1.00$
 $d_1 = 44.00$
 $R = 200.00$
 $v = 0.00135568$
 $N = 4771.233$
 $A_c = 125663.706$
 $= \cdot \text{Min}(1, 1.25 \cdot (l_b/d)^{2/3}) = 0.3645$

Calculation of ratio l_b/d

Adequate Lap Length: $l_b/d \geq 1$

Calculation of Shear Strength $V_r = \text{Min}(V_{r1}, V_{r2}) = 299278.805$

Calculation of Shear Strength at edge 1, $V_{r1} = 299278.805$

$V_{r1} = V_{Col}$ ((10.3), ASCE 41-17) = $k_n \cdot V_{ColO}$

$V_{ColO} = 299278.805$

$k_n = 1$ (zero step-static loading)

NOTE: In expression (10-3) ' $V_s = A_v \cdot f_y \cdot d/s$ ' is replaced by ' $V_s + f \cdot V_f$ ' where V_f is the contribution of FRPs ((11.3), ACI 440).

$= 1$ (normal-weight concrete)

$f'_c = 28.00$, but $f_c^{0.5} \leq 8.3$ MPa ((22.5.3.1, ACI 318-14)

$M/Vd = 2.00$

$\mu_u = 1.0979367E-011$

$\nu_u = 3.7372036E-031$

$d = 0.8 \cdot D = 320.00$

$N_u = 4771.233$

$A_g = 125663.706$

From ((11.5.4.8), ACI 318-14: $V_s = 165809.354$

$A_v = \cdot /2 \cdot A_{stirrup} = 123370.055$

$f_y = 420.00$

$s = 100.00$

V_s is multiplied by $Col = 0.00$

$s/d = 0.3125$

V_f ((11-3)-(11.4), ACI 440) = 0.00

From ((11-11), ACI 440: $V_s + V_f \leq 282706.38$

$b_w \cdot d = \cdot d \cdot d/4 = 80424.772$

Calculation of Shear Strength at edge 2, $V_{r2} = 299278.805$

$V_{r2} = V_{Col}$ ((10.3), ASCE 41-17) = $k_n \cdot V_{ColO}$

$V_{ColO} = 299278.805$

$k_n = 1$ (zero step-static loading)

NOTE: In expression (10-3) ' $V_s = A_v \cdot f_y \cdot d/s$ ' is replaced by ' $V_s + f \cdot V_f$ ' where V_f is the contribution of FRPs ((11.3), ACI 440).

$= 1$ (normal-weight concrete)

$f'_c = 28.00$, but $f_c^{0.5} \leq 8.3$ MPa ((22.5.3.1, ACI 318-14)

$M/Vd = 2.00$

$\mu_u = 1.0979367E-011$

$\nu_u = 3.7372036E-031$

$d = 0.8 \cdot D = 320.00$

$N_u = 4771.233$

$A_g = 125663.706$

From ((11.5.4.8), ACI 318-14: $V_s = 165809.354$

$A_v = \cdot /2 \cdot A_{stirrup} = 123370.055$

$f_y = 420.00$

s = 100.00
Vs is multiplied by Col = 0.00
s/d = 0.3125
Vf ((11-3)-(11.4), ACI 440) = 0.00
From (11-11), ACI 440: Vs + Vf <= 282706.38
bw*d = *d*d/4 = 80424.772

End Of Calculation of Shear Capacity ratio for element: column CC1 of floor 1
At local axis: 3

Start Of Calculation of Shear Capacity ratio for element: column CC1 of floor 1
At Shear local axis: 2
(Bending local axis: 3)
Section Type: rccs

Constant Properties

Knowledge Factor, = 1.00
Mean strength values are used for both shear and moment calculations.
Consequently:
New material of Primary Member: Concrete Strength, fc = fcm = 28.00
New material of Primary Member: Steel Strength, fs = fsm = 420.00
Concrete Elasticity, Ec = 24870.062
Steel Elasticity, Es = 200000.00

Note: Especially for the calculation of moment strengths,
the above steel re-bar strengths are multiplied by 1.25 according to R18.6.5, ACI 318-14
New material: Steel Strength, fs = 1.25*fsm = 525.00

Diameter, D = 400.00
Cover Thickness, c = 25.00
Mean Confinement Factor overall section = 1.32561
Element Length, L = 3000.00
Primary Member
Smooth Bars
Ductile Steel
With Detailing for Earthquake Resistance (including stirrups closed at 135°)
Longitudinal Bars With Ends Lapped Starting at the End Sections
Adequate Lap Length (lo/lo_u, min >= 1)
No FRP Wrapping

Stepwise Properties

At local axis: 2
EDGE -A-
Shear Force, Va = -9.8626872E-031
EDGE -B-
Shear Force, Vb = 9.8626872E-031
BOTH EDGES
Axial Force, F = -4771.233
Longitudinal Reinforcement Area Distribution (in 2 divisions)
-Tension: Aslt = 0.00
-Compression: Aslc = 3053.628
Longitudinal Reinforcement Area Distribution (in 3 divisions)
-Tension: Asl,ten = 1017.876
-Compression: Asl,com = 1017.876
-Middle: Asl,mid = 1017.876

Calculation of Shear Capacity ratio , Ve/Vr = 0.51034251

Member Controlled by Flexure ($V_e/V_r < 1$)

Calculation of Shear Demand from fig. R18.6.5, ACI 318-14 $V_e = (M_{pr1} + M_{pr2})/l_n = 152734.698$
with

$M_{pr1} = \text{Max}(\mu_{u1+}, \mu_{u1-}) = 2.2910\text{E}+008$

$\mu_{u1+} = 2.2910\text{E}+008$, is the ultimate moment strength at the edge 1 of the member in the actual moment direction which is defined for the static loading combination

$\mu_{u1-} = 2.2910\text{E}+008$, is the ultimate moment strength at the edge 1 of the member in the opposite moment direction which is defined for the static loading combination

$M_{pr2} = \text{Max}(\mu_{u2+}, \mu_{u2-}) = 2.2910\text{E}+008$

$\mu_{u2+} = 2.2910\text{E}+008$, is the ultimate moment strength at the edge 2 of the member in the actual moment direction which is defined for the static loading combination

$\mu_{u2-} = 2.2910\text{E}+008$, is the ultimate moment strength at the edge 2 of the member in the opposite moment direction which is defined for the static loading combination

Calculation of μ_{u1+}

Calculation of ultimate Moment Strength ((3.67), Biskinis Phd), μ_u
 $\mu_u = 2.2910\text{E}+008$

$\phi = 1.0472$

$\phi' = 0.92729522$

error of function (3.68), Biskinis Phd = 58526.963

From 5A.2, TB DY: $f_{cc} = f_c^* \quad c = 37.11712$

conf. factor $c = 1.32561$

$f_c = 28.00$

From 10.3.5, ASCE41-17, Final value of f_y : $f_y \cdot \text{Min}(1, 1.25 \cdot (l_b/d)^{2/3}) = 525.00$

$l_b/d = 1.00$

$d_1 = 44.00$

$R = 200.00$

$v = 0.00135568$

$N = 4771.233$

$A_c = 125663.706$

$= \phi' \cdot \text{Min}(1, 1.25 \cdot (l_b/d)^{2/3}) = 0.3645$

Calculation of ratio l_b/d

Adequate Lap Length: $l_b/d \geq 1$

Calculation of μ_{u1-}

Calculation of ultimate Moment Strength ((3.67), Biskinis Phd), μ_u
 $\mu_u = 2.2910\text{E}+008$

$\phi = 1.0472$

$\phi' = 0.92729522$

error of function (3.68), Biskinis Phd = 58526.963

From 5A.2, TB DY: $f_{cc} = f_c^* \quad c = 37.11712$

conf. factor $c = 1.32561$

$f_c = 28.00$

From 10.3.5, ASCE41-17, Final value of f_y : $f_y \cdot \text{Min}(1, 1.25 \cdot (l_b/d)^{2/3}) = 525.00$

$l_b/d = 1.00$

$d_1 = 44.00$

$R = 200.00$

$v = 0.00135568$

$N = 4771.233$

$A_c = 125663.706$

$= \phi' \cdot \text{Min}(1, 1.25 \cdot (l_b/d)^{2/3}) = 0.3645$

Calculation of ratio l_b/d

Adequate Lap Length: $l_b/d \geq 1$

Calculation of μ_{2+}

Calculation of ultimate Moment Strength ((3.67), Biskinis Phd), μ
 $\mu = 2.2910E+008$

$$= 1.0472$$

$$' = 0.92729522$$

error of function (3.68), Biskinis Phd = 58526.963

From 5A.2, TDY: $f_{cc} = f_c \cdot c = 37.11712$

conf. factor $c = 1.32561$

$f_c = 28.00$

From 10.3.5, ASCE41-17, Final value of f_y : $f_y \cdot \text{Min}(1, 1.25 \cdot (l_b/d)^{2/3}) = 525.00$

$$l_b/d = 1.00$$

$$d_1 = 44.00$$

$$R = 200.00$$

$$v = 0.00135568$$

$$N = 4771.233$$

$$A_c = 125663.706$$

$$= \cdot \text{Min}(1, 1.25 \cdot (l_b/d)^{2/3}) = 0.3645$$

Calculation of ratio l_b/d

Adequate Lap Length: $l_b/d \geq 1$

Calculation of μ_{2-}

Calculation of ultimate Moment Strength ((3.67), Biskinis Phd), μ
 $\mu = 2.2910E+008$

$$= 1.0472$$

$$' = 0.92729522$$

error of function (3.68), Biskinis Phd = 58526.963

From 5A.2, TDY: $f_{cc} = f_c \cdot c = 37.11712$

conf. factor $c = 1.32561$

$f_c = 28.00$

From 10.3.5, ASCE41-17, Final value of f_y : $f_y \cdot \text{Min}(1, 1.25 \cdot (l_b/d)^{2/3}) = 525.00$

$$l_b/d = 1.00$$

$$d_1 = 44.00$$

$$R = 200.00$$

$$v = 0.00135568$$

$$N = 4771.233$$

$$A_c = 125663.706$$

$$= \cdot \text{Min}(1, 1.25 \cdot (l_b/d)^{2/3}) = 0.3645$$

Calculation of ratio l_b/d

Adequate Lap Length: $l_b/d \geq 1$

Calculation of Shear Strength $V_r = \text{Min}(V_{r1}, V_{r2}) = 299278.805$

Calculation of Shear Strength at edge 1, $V_{r1} = 299278.805$

$V_{r1} = V_{Col} \text{ ((10.3), ASCE 41-17)} = knl * V_{ColO}$

$V_{ColO} = 299278.805$

$knl = 1$ (zero step-static loading)

NOTE: In expression (10-3) ' $V_s = A_v * f_y * d / s$ ' is replaced by ' $V_s + f * V_f$ ' where V_f is the contribution of FRPs (11.3), ACI 440).

$= 1$ (normal-weight concrete)

$f'_c = 28.00$, but $f'_c^{0.5} \leq 8.3$ MPa (22.5.3.1, ACI 318-14)

$M/Vd = 2.00$

$\mu_u = 9.9556147E-012$

$V_u = 9.8626872E-031$

$d = 0.8 * D = 320.00$

$N_u = 4771.233$

$A_g = 125663.706$

From (11.5.4.8), ACI 318-14: $V_s = 165809.354$

$A_v = \sqrt{2} * A_{stirrup} = 123370.055$

$f_y = 420.00$

$s = 100.00$

V_s is multiplied by $Col = 0.00$

$s/d = 0.3125$

$V_f \text{ ((11-3)-(11.4), ACI 440)} = 0.00$

From (11-11), ACI 440: $V_s + V_f \leq 282706.38$

$bw * d = \sqrt{2} * d^2 / 4 = 80424.772$

Calculation of Shear Strength at edge 2, $V_{r2} = 299278.805$

$V_{r2} = V_{Col} \text{ ((10.3), ASCE 41-17)} = knl * V_{ColO}$

$V_{ColO} = 299278.805$

$knl = 1$ (zero step-static loading)

NOTE: In expression (10-3) ' $V_s = A_v * f_y * d / s$ ' is replaced by ' $V_s + f * V_f$ ' where V_f is the contribution of FRPs (11.3), ACI 440).

$= 1$ (normal-weight concrete)

$f'_c = 28.00$, but $f'_c^{0.5} \leq 8.3$ MPa (22.5.3.1, ACI 318-14)

$M/Vd = 2.00$

$\mu_u = 9.9556147E-012$

$V_u = 9.8626872E-031$

$d = 0.8 * D = 320.00$

$N_u = 4771.233$

$A_g = 125663.706$

From (11.5.4.8), ACI 318-14: $V_s = 165809.354$

$A_v = \sqrt{2} * A_{stirrup} = 123370.055$

$f_y = 420.00$

$s = 100.00$

V_s is multiplied by $Col = 0.00$

$s/d = 0.3125$

$V_f \text{ ((11-3)-(11.4), ACI 440)} = 0.00$

From (11-11), ACI 440: $V_s + V_f \leq 282706.38$

$bw * d = \sqrt{2} * d^2 / 4 = 80424.772$

End Of Calculation of Shear Capacity ratio for element: column CC1 of floor 1

At local axis: 2

Start Of Calculation of Chord Rotation Capacity for element: column CC1 of floor 1

At local axis: 3

Integration Section: (b)

Section Type: rccs

Constant Properties

Knowledge Factor, $\phi = 1.00$
 Chord Rotation is generally considered as Deformation-Controlled Action according to Table C7-1, ASCE41-17.
 Mean strengths are used for Deformation-Controlled Actions according to 7.5.1.3, ASCE 41-17
 Consequently:
 New material of Primary Member: Concrete Strength, $f_c = f_{cm} = 28.00$
 New material of Primary Member: Steel Strength, $f_s = f_{sm} = 420.00$
 Concrete Elasticity, $E_c = 24870.062$
 Steel Elasticity, $E_s = 200000.00$
 Diameter, $D = 400.00$
 Cover Thickness, $c = 25.00$
 Element Length, $L = 3000.00$
 Primary Member
 Smooth Bars
 Ductile Steel
 With Detailing for Earthquake Resistance (including stirrups closed at 135°)
 Longitudinal Bars With Ends Lapped Starting at the End Sections
 Adequate Lap Length ($l_b/l_d \geq 1$)
 No FRP Wrapping

Stepwise Properties

Bending Moment, $M = 0.04476539$
 Shear Force, $V_2 = 4586.336$
 Shear Force, $V_3 = -8.2104443E-015$
 Axial Force, $F = -4769.398$
 Longitudinal Reinforcement Area Distribution (in 2 divisions)
 -Tension: $A_{st} = 0.00$
 -Compression: $A_{sc} = 3053.628$
 Longitudinal Reinforcement Area Distribution (in 3 divisions)
 -Tension: $A_{st,ten} = 1017.876$
 -Compression: $A_{st,com} = 1017.876$
 -Middle: $A_{st,mid} = 1017.876$
 Mean Diameter of Tension Reinforcement, $D_bL = 18.00$

New component: From table 7-7, ASCE 41_17: Final chord rotation Capacity $u_R = 1.0^*$ $u = 0.00729496$
 $u = y + p = 0.00729496$

- Calculation of y -

$y = (M_y * L_s / 3) / E_{eff} = 0.00229496$ ((4.29), Biskinis Phd))
 $M_y = 2.1517E+008$
 $L_s = M/V$ (with $L_s > 0.1 * L$ and $L_s < 2 * L$) = 300.00
 From table 10.5, ASCE 41_17: $E_{eff} = factor * E_c * I_g = 9.3758E+012$
 $factor = 0.30$
 $A_g = 125663.706$
 $f_c' = 28.00$
 $N = 4769.398$
 $E_c * I_g = 3.1253E+013$

Calculation of Yielding Moment M_y

Calculation of y and M_y according to (7) - (8) in Biskinis and Fardis

$M_y = \min(M_{y_ten}, M_{y_com}) = 2.1517E+008$
 y ((10a) or (10b)) = $1.1483164E-005$
 M_{y_ten} (8a) = $2.1517E+008$
 y_{ten} (7a) = 72.79824
 error of function (7a) = 0.00155286
 M_{y_com} (8b) = $4.0201E+008$
 y_{com} (7b) = 69.01016

error of function (7b) = -0.00040881

with $\epsilon_y = 0.0021$

$\epsilon_{co} = 0.002$

$\alpha_{pl} = 0.35$ ((9a) in Biskinis and Fardis for no FRP Wrap)

$d_1 = 44.00$

$R = 200.00$

$v = 0.00135549$

$N = 4769.398$

$A_c = 125663.706$

$= 0.3645$

with $f_c = 28.00$

Calculation of ratio l_b/d

Adequate Lap Length: $l_b/d \geq 1$

- Calculation of p -

From table 10-9: $p = 0.005$

with:

- Columns controlled by inadequate development or splicing along the clear height because $l_b/d < 1$

shear control ratio $V_y E / V_{co} I_{OE} = 0.51034251$

$d = 0.00$

$s = 0.00$

$t = 2 \cdot A_v / (d_c \cdot s) + 4 \cdot t_f / D \cdot (f_{fe} / f_s) = 0.00$

$A_v = 78.53982$, is the area of the circular stirrup

$d_c = D - 2 \cdot \text{cover} - \text{Hoop Diameter} = 340.00$

The term $2 \cdot t_f / b_w \cdot (f_{fe} / f_s)$ is implemented to account for FRP contribution

where $f = 2 \cdot t_f / b_w$ is FRP ratio (EC8 - 3, A.4.4.3(6)) and f_{fe} / f_s normalises f to steel strength

All these variables have already been given in Shear control ratio calculation.

$N_{UD} = 4769.398$

$A_g = 125663.706$

$f_{cE} = 28.00$

$f_{yE} = f_{yI} = 420.00$

$p_l = \text{Area_Tot_Long_Rein} / (A_g) = 0.0243$

$f_{cE} = 28.00$

End Of Calculation of Chord Rotation Capacity for element: column CC1 of floor 1

At local axis: 3

Integration Section: (b)

Calculation No. 9

column C1, Floor 1

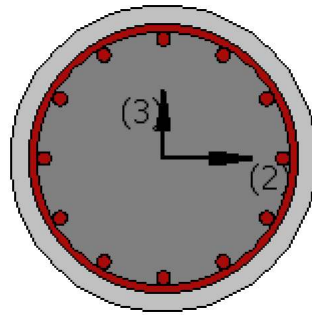
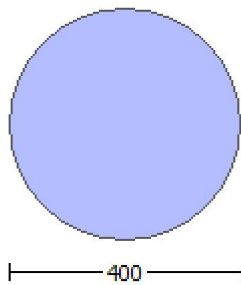
Limit State: Collapse Prevention (data interpolation between analysis steps 1 and 2)

Analysis: Uniform +X

Check: Shear capacity V_{Rd}

Edge: Start

Local Axis: (2)



Start Of Calculation of Shear Capacity for element: column CC1 of floor 1

At local axis: 2

Integration Section: (a)

Section Type: rccs

Constant Properties

Knowledge Factor, $\gamma = 1.00$

Member Shear Force is generally considered as Force-Controlled Action according to Table C7-1, ASCE41-17.

Lower-bound strengths are used for Force-Controlled Actions according to 7.5.1.3, ASCE 41-17

Consequently:

New material of Primary Member: Concrete Strength, $f_c = f_{c_lower_bound} = 20.00$

New material of Primary Member: Steel Strength, $f_s = f_{s_lower_bound} = 400.00$

Concrete Elasticity, $E_c = 24870.062$

Steel Elasticity, $E_s = 200000.00$

#####

Note: Especially for the calculation of γ for displacement ductility demand, the expected (mean value) strengths are used (7.5.1.3, ASCE41-17) because bending is considered as Deformation-Controlled Action (Table C7-1, ASCE41-17).

New material: Concrete Strength, $f_c = f_{cm} = 28.00$

New material: Steel Strength, $f_s = f_{sm} = 420.00$

#####

Diameter, $D = 400.00$

Cover Thickness, $c = 25.00$

Element Length, $L = 3000.00$

Primary Member

Smooth Bars

Ductile Steel

With Detailing for Earthquake Resistance (including stirrups closed at 135°)

Longitudinal Bars With Ends Lapped Starting at the End Sections

Adequate Lap Length ($l_o/l_{ou,min} = l_b/l_d \geq 1$)

No FRP Wrapping

Stepwise Properties

EDGE -A-

Bending Moment, $M_a = -8.6921E+006$

Shear Force, $V_a = -2896.171$

EDGE -B-

Bending Moment, $M_b = 0.02826837$

Shear Force, $V_b = 2896.171$

BOTH EDGES

Axial Force, $F = -4770.074$

Longitudinal Reinforcement Area Distribution (in 2 divisions)

-Tension: $A_{sl,t} = 1272.345$

-Compression: $A_{sl,c} = 1781.283$

Longitudinal Reinforcement Area Distribution (in 3 divisions)

-Tension: $A_{sl,ten} = 1017.876$

-Compression: $A_{sl,com} = 1017.876$

-Middle: $A_{sl,mid} = 1017.876$

Mean Diameter of Tension Reinforcement, $Db_{L,ten} = 18.00$

New component: From table 7-7, ASCE 41_17: Final Shear Capacity $V_R = 1.0 \cdot V_n = 214387.999$
 V_n ((10.3), ASCE 41-17) = $k_n \cdot V_{Col0} = 214387.999$
 $V_{Col} = 214387.999$
 $k_n = 1.00$
 $displacement_ductility_demand = 0.01097555$

NOTE: In expression (10-3) ' $V_s = A_v \cdot f_y \cdot d / s$ ' is replaced by ' $V_s + f \cdot V_f$ '
 where V_f is the contribution of FRPs (11.3), ACI 440).

$= 1$ (normal-weight concrete)
 $f_c' = 20.00$, but $f_c'^{0.5} \leq 8.3$ MPa (22.5.3.1, ACI 318-14)
 $M/Vd = 4.00$
 $\mu_u = 8.6921E+006$
 $V_u = 2896.171$
 $d = 0.8 \cdot D = 320.00$
 $N_u = 4770.074$
 $A_g = 125663.706$
 From (11.5.4.8), ACI 318-14: $V_s = 157913.67$
 $A_v = \sqrt{2} \cdot A_{stirrup} = 123370.055$
 $f_y = 400.00$
 $s = 100.00$
 V_s is multiplied by $Col = 0.00$
 $s/d = 0.3125$
 V_f ((11-3)-(11.4), ACI 440) = 0.00
 From (11-11), ACI 440: $V_s + V_f \leq 238930.50$
 $b_w \cdot d = \mu_u \cdot d / 4 = 80424.772$

$displacement_ductility_demand$ is calculated as ϕ / y

- Calculation of ϕ / y for END A -
 for rotation axis 3 and integ. section (a)

From analysis, chord rotation $\phi = 0.00025199$
 $y = (M_y \cdot L_s / 3) / E_{eff} = 0.02295907$ ((4.29), Biskinis Phd))
 $M_y = 2.1517E+008$
 $L_s = M/V$ (with $L_s > 0.1 \cdot L$ and $L_s < 2 \cdot L$) = 3001.241
 From table 10.5, ASCE 41_17: $E_{eff} = factor \cdot E_c \cdot I_g = 9.3758E+012$
 $factor = 0.30$
 $A_g = 125663.706$
 $f_c' = 28.00$
 $N = 4770.074$
 $E_c \cdot I_g = 3.1253E+013$

Calculation of Yielding Moment M_y

Calculation of ϕ and M_y according to (7) - (8) in Biskinis and Fardis

$M_y = \min(M_{y_ten}, M_{y_com}) = 2.1517E+008$
 ϕ ((10a) or (10b)) = $1.1483167E-005$
 M_{y_ten} (8a) = $2.1517E+008$
 ϕ_{ten} (7a) = 72.79826
 error of function (7a) = 0.00155288
 M_{y_com} (8b) = $4.0201E+008$
 ϕ_{com} (7b) = 69.01017
 error of function (7b) = -0.00040881
 with $e_y = 0.0021$
 $e_{co} = 0.002$
 $apl = 0.35$ ((9a) in Biskinis and Fardis for no FRP Wrap)
 $d1 = 44.00$

$R = 200.00$
 $v = 0.00135568$
 $N = 4770.074$
 $A_c = 125663.706$
 $= 0.3645$
 with $f_c = 28.00$

Calculation of ratio I_b/I_d

Adequate Lap Length: $I_b/I_d \geq 1$

End Of Calculation of Shear Capacity for element: column CC1 of floor 1

At local axis: 2

Integration Section: (a)

Calculation No. 10

column C1, Floor 1

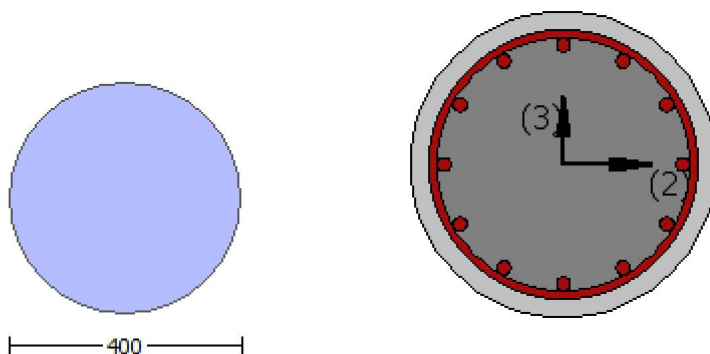
Limit State: Collapse Prevention (data interpolation between analysis steps 1 and 2)

Analysis: Uniform +X

Check: Chord rotation capacity (ϕ)

Edge: Start

Local Axis: (2)



Start Of Calculation of Shear Capacity ratio for element: column CC1 of floor 1

At Shear local axis: 3

(Bending local axis: 2)

Section Type: rccs

Constant Properties

Knowledge Factor, $\gamma = 1.00$

Mean strength values are used for both shear and moment calculations.

Consequently:

New material of Primary Member: Concrete Strength, $f_c = f_{cm} = 28.00$

New material of Primary Member: Steel Strength, $f_s = f_{sm} = 420.00$

Concrete Elasticity, $E_c = 24870.062$

```

Steel Elasticity, Es = 200000.00
#####
Note: Especially for the calculation of moment strengths,
the above steel re-bar strengths are multiplied by 1.25 according to R18.6.5, ACI 318-14
New material: Steel Strength, fs = 1.25*fsm = 525.00
#####
Diameter, D = 400.00
Cover Thickness, c = 25.00
Mean Confinement Factor overall section = 1.32561
Element Length, L = 3000.00
Primary Member
Smooth Bars
Ductile Steel
With Detailing for Earthquake Resistance (including stirrups closed at 135°)
Longitudinal Bars With Ends Lapped Starting at the End Sections
Adequate Lap Length (lo/lou,min>=1)
No FRP Wrapping
-----

Stepwise Properties
-----
At local axis: 3
EDGE -A-
Shear Force, Va = -3.7372036E-031
EDGE -B-
Shear Force, Vb = 3.7372036E-031
BOTH EDGES
Axial Force, F = -4771.233
Longitudinal Reinforcement Area Distribution (in 2 divisions)
-Tension: Aslt = 0.00
-Compression: Aslc = 3053.628
Longitudinal Reinforcement Area Distribution (in 3 divisions)
-Tension: Asl,ten = 1017.876
-Compression: Asl,com = 1017.876
-Middle: Asl,mid = 1017.876
-----
-----

Calculation of Shear Capacity ratio , Ve/Vr = 0.51034251
Member Controlled by Flexure (Ve/Vr < 1)
Calculation of Shear Demand from fig. R18.6.5, ACI 318-14 Ve = (Mpr1 + Mpr2)/ln = 152734.698
with
Mpr1 = Max(Mu1+ , Mu1-) = 2.2910E+008
Mu1+ = 2.2910E+008, is the ultimate moment strength at the edge 1 of the member in the actual moment direction
which is defined for the static loading combination
Mu1- = 2.2910E+008, is the ultimate moment strength at the edge 1 of the member in the opposite moment
direction which is defined for the static loading combination
Mpr2 = Max(Mu2+ , Mu2-) = 2.2910E+008
Mu2+ = 2.2910E+008, is the ultimate moment strength at the edge 2 of the member in the actual moment direction
which is defined for the the static loading combination
Mu2- = 2.2910E+008, is the ultimate moment strength at the edge 2 of the member in the opposite moment
direction which is defined for the the static loading combination
-----

Calculation of Mu1+
-----

Calculation of ultimate Moment Strength ((3.67), Biskinis Phd), Mu
Mu = 2.2910E+008
-----
= 1.0472
' = 0.92729522
error of function (3.68), Biskinis Phd = 58526.963
From 5A.2, TBDY: fcc = fc* c = 37.11712
conf. factor c = 1.32561

```

$f_c = 28.00$
 From 10.3.5, ASCE41-17, Final value of f_y : $f_y \cdot \text{Min}(1, 1.25 \cdot (l_b/d)^{2/3}) = 525.00$
 $l_b/d = 1.00$
 $d_1 = 44.00$
 $R = 200.00$
 $v = 0.00135568$
 $N = 4771.233$
 $A_c = 125663.706$
 $= \cdot \text{Min}(1, 1.25 \cdot (l_b/d)^{2/3}) = 0.3645$

Calculation of ratio l_b/d

Adequate Lap Length: $l_b/d \geq 1$

Calculation of μ_1 -

Calculation of ultimate Moment Strength ((3.67), Biskinis Phd), μ
 $\mu = 2.2910\text{E}+008$

$= 1.0472$
 $' = 0.92729522$
 error of function (3.68), Biskinis Phd = 58526.963
 From 5A.2, TBDY: $f_{cc} = f_c \cdot c = 37.11712$
 conf. factor $c = 1.32561$
 $f_c = 28.00$
 From 10.3.5, ASCE41-17, Final value of f_y : $f_y \cdot \text{Min}(1, 1.25 \cdot (l_b/d)^{2/3}) = 525.00$
 $l_b/d = 1.00$
 $d_1 = 44.00$
 $R = 200.00$
 $v = 0.00135568$
 $N = 4771.233$
 $A_c = 125663.706$
 $= \cdot \text{Min}(1, 1.25 \cdot (l_b/d)^{2/3}) = 0.3645$

Calculation of ratio l_b/d

Adequate Lap Length: $l_b/d \geq 1$

Calculation of μ_2 +

Calculation of ultimate Moment Strength ((3.67), Biskinis Phd), μ
 $\mu = 2.2910\text{E}+008$

$= 1.0472$
 $' = 0.92729522$
 error of function (3.68), Biskinis Phd = 58526.963
 From 5A.2, TBDY: $f_{cc} = f_c \cdot c = 37.11712$
 conf. factor $c = 1.32561$
 $f_c = 28.00$
 From 10.3.5, ASCE41-17, Final value of f_y : $f_y \cdot \text{Min}(1, 1.25 \cdot (l_b/d)^{2/3}) = 525.00$
 $l_b/d = 1.00$
 $d_1 = 44.00$
 $R = 200.00$
 $v = 0.00135568$
 $N = 4771.233$
 $A_c = 125663.706$
 $= \cdot \text{Min}(1, 1.25 \cdot (l_b/d)^{2/3}) = 0.3645$

Calculation of ratio l_b/d

Adequate Lap Length: $l_b/d \geq 1$

Calculation of μ_2

Calculation of ultimate Moment Strength ((3.67), Biskinis Phd), μ
 $\mu = 2.2910E+008$

$$= 1.0472$$

$$' = 0.92729522$$

error of function (3.68), Biskinis Phd = 58526.963

From 5A.2, TBDY: $f_{cc} = f_c' \cdot c = 37.11712$

conf. factor $c = 1.32561$

$$f_c = 28.00$$

From 10.3.5, ASCE41-17, Final value of f_y : $f_y \cdot \text{Min}(1, 1.25 \cdot (l_b/d)^{2/3}) = 525.00$

$$l_b/d = 1.00$$

$$d_1 = 44.00$$

$$R = 200.00$$

$$v = 0.00135568$$

$$N = 4771.233$$

$$A_c = 125663.706$$

$$= \cdot \text{Min}(1, 1.25 \cdot (l_b/d)^{2/3}) = 0.3645$$

Calculation of ratio l_b/d

Adequate Lap Length: $l_b/d \geq 1$

Calculation of Shear Strength $V_r = \text{Min}(V_{r1}, V_{r2}) = 299278.805$

Calculation of Shear Strength at edge 1, $V_{r1} = 299278.805$

$V_{r1} = V_{Col} \text{ ((10.3), ASCE 41-17)} = k_{nl} \cdot V_{Col0}$

$$V_{Col0} = 299278.805$$

$k_{nl} = 1$ (zero step-static loading)

NOTE: In expression (10-3) ' $V_s = A_v \cdot f_y \cdot d/s$ ' is replaced by ' $V_s + f \cdot V_f$ ' where V_f is the contribution of FRPs (11.3), ACI 440).

$= 1$ (normal-weight concrete)

$f_c' = 28.00$, but $f_c'^{0.5} \leq 8.3 \text{ MPa}$ (22.5.3.1, ACI 318-14)

$$M/Vd = 2.00$$

$$\mu = 1.0979367E-011$$

$$V_u = 3.7372036E-031$$

$$d = 0.8 \cdot D = 320.00$$

$$N_u = 4771.233$$

$$A_g = 125663.706$$

From (11.5.4.8), ACI 318-14: $V_s = 165809.354$

$$A_v = \cdot /2 \cdot A_{\text{stirrup}} = 123370.055$$

$$f_y = 420.00$$

$$s = 100.00$$

V_s is multiplied by $Col = 0.00$

$$s/d = 0.3125$$

$V_f \text{ ((11-3)-(11.4), ACI 440)} = 0.00$

From (11-11), ACI 440: $V_s + V_f \leq 282706.38$

$$b_w \cdot d = \cdot d \cdot d/4 = 80424.772$$

Calculation of Shear Strength at edge 2, $V_{r2} = 299278.805$

$V_{r2} = V_{Col} ((10.3), ASCE 41-17) = knl * V_{Col0}$

$V_{Col0} = 299278.805$

$knl = 1$ (zero step-static loading)

NOTE: In expression (10-3) ' $V_s = A_v * f_y * d / s$ ' is replaced by ' $V_s + f * V_f$ '
where V_f is the contribution of FRPs (11.3), ACI 440).

$= 1$ (normal-weight concrete)

$f'_c = 28.00$, but $f_c^{0.5} \leq 8.3$ MPa (22.5.3.1, ACI 318-14)

$M/Vd = 2.00$

$\mu_u = 1.0979367E-011$

$\nu_u = 3.7372036E-031$

$d = 0.8 * D = 320.00$

$N_u = 4771.233$

$A_g = 125663.706$

From (11.5.4.8), ACI 318-14: $V_s = 165809.354$

$A_v = /2 * A_{stirrup} = 123370.055$

$f_y = 420.00$

$s = 100.00$

V_s is multiplied by $Col = 0.00$

$s/d = 0.3125$

$V_f ((11-3)-(11.4), ACI 440) = 0.00$

From (11-11), ACI 440: $V_s + V_f \leq 282706.38$

$bw * d = *d * d / 4 = 80424.772$

End Of Calculation of Shear Capacity ratio for element: column CC1 of floor 1

At local axis: 3

Start Of Calculation of Shear Capacity ratio for element: column CC1 of floor 1

At Shear local axis: 2

(Bending local axis: 3)

Section Type: rccs

Constant Properties

Knowledge Factor, $= 1.00$

Mean strength values are used for both shear and moment calculations.

Consequently:

New material of Primary Member: Concrete Strength, $f_c = f_{cm} = 28.00$

New material of Primary Member: Steel Strength, $f_s = f_{sm} = 420.00$

Concrete Elasticity, $E_c = 24870.062$

Steel Elasticity, $E_s = 200000.00$

#####

Note: Especially for the calculation of moment strengths,

the above steel re-bar strengths are multiplied by 1.25 according to R18.6.5, ACI 318-14

New material: Steel Strength, $f_s = 1.25 * f_{sm} = 525.00$

#####

Diameter, $D = 400.00$

Cover Thickness, $c = 25.00$

Mean Confinement Factor overall section = 1.32561

Element Length, $L = 3000.00$

Primary Member

Smooth Bars

Ductile Steel

With Detailing for Earthquake Resistance (including stirrups closed at 135°)

Longitudinal Bars With Ends Lapped Starting at the End Sections

Adequate Lap Length ($l_o/l_{ou}, \min \geq 1$)

No FRP Wrapping

Stepwise Properties

At local axis: 2
EDGE -A-
Shear Force, $V_a = -9.8626872E-031$
EDGE -B-
Shear Force, $V_b = 9.8626872E-031$
BOTH EDGES
Axial Force, $F = -4771.233$
Longitudinal Reinforcement Area Distribution (in 2 divisions)
-Tension: $A_{st} = 0.00$
-Compression: $A_{sc} = 3053.628$
Longitudinal Reinforcement Area Distribution (in 3 divisions)
-Tension: $A_{st,ten} = 1017.876$
-Compression: $A_{st,com} = 1017.876$
-Middle: $A_{st,mid} = 1017.876$

Calculation of Shear Capacity ratio, $V_e/V_r = 0.51034251$
Member Controlled by Flexure ($V_e/V_r < 1$)
Calculation of Shear Demand from fig. R18.6.5, ACI 318-14 $V_e = (M_{pr1} + M_{pr2})/l_n = 152734.698$
with
 $M_{pr1} = \text{Max}(\mu_{u1+}, \mu_{u1-}) = 2.2910E+008$
 $\mu_{u1+} = 2.2910E+008$, is the ultimate moment strength at the edge 1 of the member in the actual moment direction which is defined for the static loading combination
 $\mu_{u1-} = 2.2910E+008$, is the ultimate moment strength at the edge 1 of the member in the opposite moment direction which is defined for the static loading combination
 $M_{pr2} = \text{Max}(\mu_{u2+}, \mu_{u2-}) = 2.2910E+008$
 $\mu_{u2+} = 2.2910E+008$, is the ultimate moment strength at the edge 2 of the member in the actual moment direction which is defined for the static loading combination
 $\mu_{u2-} = 2.2910E+008$, is the ultimate moment strength at the edge 2 of the member in the opposite moment direction which is defined for the static loading combination

Calculation of μ_{u1+}

Calculation of ultimate Moment Strength ((3.67), Biskinis Phd), μ_u
 $\mu_u = 2.2910E+008$

$\lambda = 1.0472$
 $\lambda' = 0.92729522$
error of function (3.68), Biskinis Phd = 58526.963
From 5A.2, TB DY: $f_{cc} = f_c^* \quad c = 37.11712$
conf. factor $c = 1.32561$
 $f_c = 28.00$
From 10.3.5, ASCE41-17, Final value of f_y : $f_y \cdot \text{Min}(1, 1.25 \cdot (l_b/d)^{2/3}) = 525.00$
 $l_b/d = 1.00$
 $d_1 = 44.00$
 $R = 200.00$
 $v = 0.00135568$
 $N = 4771.233$
 $A_c = 125663.706$
 $= \lambda' \cdot \text{Min}(1, 1.25 \cdot (l_b/d)^{2/3}) = 0.3645$

Calculation of ratio l_b/d

Adequate Lap Length: $l_b/d \geq 1$

Calculation of μ_{u1-}

Calculation of ultimate Moment Strength ((3.67), Biskinis Phd), Mu
Mu = 2.2910E+008

= 1.0472
' = 0.92729522
error of function (3.68), Biskinis Phd = 58526.963
From 5A.2, TBDY: fcc = fc* c = 37.11712
conf. factor c = 1.32561
fc = 28.00
From 10.3.5, ASCE41-17, Final value of fy: fy*Min(1,1.25*(lb/d)^ 2/3) = 525.00
lb/d = 1.00
d1 = 44.00
R = 200.00
v = 0.00135568
N = 4771.233
Ac = 125663.706
= *Min(1,1.25*(lb/d)^ 2/3) = 0.3645

Calculation of ratio lb/d

Adequate Lap Length: lb/d >= 1

Calculation of Mu2+

Calculation of ultimate Moment Strength ((3.67), Biskinis Phd), Mu
Mu = 2.2910E+008

= 1.0472
' = 0.92729522
error of function (3.68), Biskinis Phd = 58526.963
From 5A.2, TBDY: fcc = fc* c = 37.11712
conf. factor c = 1.32561
fc = 28.00
From 10.3.5, ASCE41-17, Final value of fy: fy*Min(1,1.25*(lb/d)^ 2/3) = 525.00
lb/d = 1.00
d1 = 44.00
R = 200.00
v = 0.00135568
N = 4771.233
Ac = 125663.706
= *Min(1,1.25*(lb/d)^ 2/3) = 0.3645

Calculation of ratio lb/d

Adequate Lap Length: lb/d >= 1

Calculation of Mu2-

Calculation of ultimate Moment Strength ((3.67), Biskinis Phd), Mu
Mu = 2.2910E+008

= 1.0472
' = 0.92729522
error of function (3.68), Biskinis Phd = 58526.963
From 5A.2, TBDY: fcc = fc* c = 37.11712
conf. factor c = 1.32561
fc = 28.00

From 10.3.5, ASCE41-17, Final value of f_y : $f_y \cdot \text{Min}(1, 1.25 \cdot (l_b/d)^{2/3}) = 525.00$

$$l_b/d = 1.00$$

$$d_1 = 44.00$$

$$R = 200.00$$

$$v = 0.00135568$$

$$N = 4771.233$$

$$A_c = 125663.706$$

$$= \cdot \text{Min}(1, 1.25 \cdot (l_b/d)^{2/3}) = 0.3645$$

Calculation of ratio l_b/d

Adequate Lap Length: $l_b/d \geq 1$

Calculation of Shear Strength $V_r = \text{Min}(V_{r1}, V_{r2}) = 299278.805$

Calculation of Shear Strength at edge 1, $V_{r1} = 299278.805$

$$V_{r1} = V_{\text{Col}} ((10.3), \text{ASCE } 41-17) = k_{nl} \cdot V_{\text{Col}0}$$

$$V_{\text{Col}0} = 299278.805$$

$$k_{nl} = 1 \text{ (zero step-static loading)}$$

NOTE: In expression (10-3) ' $V_s = A_v \cdot f_y \cdot d/s$ ' is replaced by ' $V_s + f \cdot V_f$ ' where V_f is the contribution of FRPs (11.3), ACI 440).

$$= 1 \text{ (normal-weight concrete)}$$

$$f'_c = 28.00, \text{ but } f'_c^{0.5} \leq 8.3 \text{ MPa (22.5.3.1, ACI 318-14)}$$

$$M/Vd = 2.00$$

$$\mu_u = 9.9556147\text{E-}012$$

$$V_u = 9.8626872\text{E-}031$$

$$d = 0.8 \cdot D = 320.00$$

$$N_u = 4771.233$$

$$A_g = 125663.706$$

$$\text{From (11.5.4.8), ACI 318-14: } V_s = 165809.354$$

$$A_v = \cdot /2 \cdot A_{\text{stirrup}} = 123370.055$$

$$f_y = 420.00$$

$$s = 100.00$$

$$V_s \text{ is multiplied by } \text{Col} = 0.00$$

$$s/d = 0.3125$$

$$V_f ((11-3)-(11.4), \text{ACI } 440) = 0.00$$

$$\text{From (11-11), ACI } 440: V_s + V_f \leq 282706.38$$

$$b_w \cdot d = \cdot d \cdot d/4 = 80424.772$$

Calculation of Shear Strength at edge 2, $V_{r2} = 299278.805$

$$V_{r2} = V_{\text{Col}} ((10.3), \text{ASCE } 41-17) = k_{nl} \cdot V_{\text{Col}0}$$

$$V_{\text{Col}0} = 299278.805$$

$$k_{nl} = 1 \text{ (zero step-static loading)}$$

NOTE: In expression (10-3) ' $V_s = A_v \cdot f_y \cdot d/s$ ' is replaced by ' $V_s + f \cdot V_f$ ' where V_f is the contribution of FRPs (11.3), ACI 440).

$$= 1 \text{ (normal-weight concrete)}$$

$$f'_c = 28.00, \text{ but } f'_c^{0.5} \leq 8.3 \text{ MPa (22.5.3.1, ACI 318-14)}$$

$$M/Vd = 2.00$$

$$\mu_u = 9.9556147\text{E-}012$$

$$V_u = 9.8626872\text{E-}031$$

$$d = 0.8 \cdot D = 320.00$$

$$N_u = 4771.233$$

$$A_g = 125663.706$$

$$\text{From (11.5.4.8), ACI 318-14: } V_s = 165809.354$$

$$A_v = \cdot /2 \cdot A_{\text{stirrup}} = 123370.055$$

$$f_y = 420.00$$

$$s = 100.00$$

$$V_s \text{ is multiplied by } \text{Col} = 0.00$$

s/d = 0.3125
Vf ((11-3)-(11.4), ACI 440) = 0.00
From (11-11), ACI 440: Vs + Vf <= 282706.38
bw*d = *d*d/4 = 80424.772

End Of Calculation of Shear Capacity ratio for element: column CC1 of floor 1
At local axis: 2

Start Of Calculation of Chord Rotation Capacity for element: column CC1 of floor 1
At local axis: 2
Integration Section: (a)
Section Type: rccs

Constant Properties

Knowledge Factor, = 1.00
Chord Rotation is generally considered as Deformation-Controlled Action according to Table C7-1, ASCE41-17.
Mean strengths are used for Deformation-Controlled Actions according to 7.5.1.3, ASCE 41-17
Consequently:
New material of Primary Member: Concrete Strength, fc = fcm = 28.00
New material of Primary Member: Steel Strength, fs = fsm = 420.00
Concrete Elasticity, Ec = 24870.062
Steel Elasticity, Es = 200000.00
Diameter, D = 400.00
Cover Thickness, c = 25.00
Element Length, L = 3000.00
Primary Member
Smooth Bars
Ductile Steel
With Detailing for Earthquake Resistance (including stirrups closed at 135°)
Longitudinal Bars With Ends Lapped Starting at the End Sections
Adequate Lap Length (lb/d >= 1)
No FRP Wrapping

Stepwise Properties

Bending Moment, M = -6.6801918E-012
Shear Force, V2 = -2896.171
Shear Force, V3 = 5.1847167E-015
Axial Force, F = -4770.074
Longitudinal Reinforcement Area Distribution (in 2 divisions)
-Tension: Aslt = 1272.345
-Compression: Aslc = 1781.283
Longitudinal Reinforcement Area Distribution (in 3 divisions)
-Tension: Asl,ten = 1017.876
-Compression: Asl,com = 1017.876
-Middle: Asl,mid = 1017.876
Mean Diameter of Tension Reinforcement, DbL = 18.00

New component: From table 7-7, ASCE 41_17: Final chord rotation Capacity u,R = 1.0* u = 0.07134961
u = y + p = 0.07134961

- Calculation of y -

y = (My*Ls/3)/Eleff = 0.01147479 ((4.29),Biskinis Phd))
My = 2.1517E+008
Ls = M/V (with Ls > 0.1*L and Ls < 2*L) = 1500.00
From table 10.5, ASCE 41_17: Eleff = factor*Ec*lg = 9.3758E+012

factor = 0.30
Ag = 125663.706
fc' = 28.00
N = 4770.074
Ec*Ig = 3.1253E+013

Calculation of Yielding Moment My

Calculation of η and My according to (7) - (8) in Biskinis and Fardis

My = Min(My_ten, My_com) = 2.1517E+008
 η ((10a) or (10b)) = 1.1483167E-005
My_ten (8a) = 2.1517E+008
 η _ten (7a) = 72.79826
error of function (7a) = 0.00155288
My_com (8b) = 4.0201E+008
 η _com (7b) = 69.01017
error of function (7b) = -0.00040881
with ϵ_y = 0.0021
 ϵ_{co} = 0.002
 α_{pl} = 0.35 ((9a) in Biskinis and Fardis for no FRP Wrap)
d1 = 44.00
R = 200.00
 ν = 0.00135568
N = 4770.074
Ac = 125663.706
= 0.3645
with fc = 28.00

Calculation of ratio lb/d

Adequate Lap Length: lb/d >= 1

- Calculation of ρ -

From table 10-9: ρ = 0.05987483

with:

- Columns controlled by inadequate development or splicing along the clear height because lb/d < 1
shear control ratio $V_y E / V_{co} I_{OE}$ = 0.51034251
d = 0.00
s = 0.00
 $t = 2 \cdot A_v / (d_c \cdot s) + 4 \cdot t_f / D \cdot (f_{fe} / f_s) = 0.00$
Av = 78.53982, is the area of the circular stirrup
dc = D - 2*cover - Hoop Diameter = 340.00
The term $2 \cdot t_f / b_w \cdot (f_{fe} / f_s)$ is implemented to account for FRP contribution
where $f = 2 \cdot t_f / b_w$ is FRP ratio (EC8 - 3, A.4.4.3(6)) and f_{fe} / f_s normalises f to steel strength
All these variables have already been given in Shear control ratio calculation.
NUD = 4770.074
Ag = 125663.706
fcE = 28.00
fytE = fytE = 420.00
 $\rho_l = \text{Area_Tot_Long_Rein} / (Ag) = 0.0243$
fcE = 28.00

End Of Calculation of Chord Rotation Capacity for element: column CC1 of floor 1

At local axis: 2

Integration Section: (a)

Calculation No. 11

column C1, Floor 1

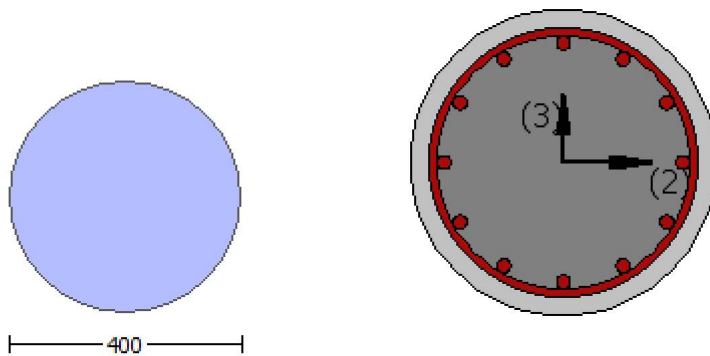
Limit State: Collapse Prevention (data interpolation between analysis steps 1 and 2)

Analysis: Uniform +X

Check: Shear capacity V_{Rd}

Edge: Start

Local Axis: (3)



Start Of Calculation of Shear Capacity for element: column CC1 of floor 1

At local axis: 3

Integration Section: (a)

Section Type: rccs

Constant Properties

Knowledge Factor, $\gamma = 1.00$

Member Shear Force is generally considered as Force-Controlled Action according to Table C7-1, ASCE41-17.

Lower-bound strengths are used for Force-Controlled Actions according to 7.5.1.3, ASCE 41-17

Consequently:

New material of Primary Member: Concrete Strength, $f_c = f_{c_lower_bound} = 20.00$

New material of Primary Member: Steel Strength, $f_s = f_{s_lower_bound} = 400.00$

Concrete Elasticity, $E_c = 24870.062$

Steel Elasticity, $E_s = 200000.00$

#####

Note: Especially for the calculation of γ for displacement ductility demand, the expected (mean value) strengths are used (7.5.1.3, ASCE41-17) because bending is considered as Deformation-Controlled Action (Table C7-1, ASCE41-17).

New material: Concrete Strength, $f_c = f_{cm} = 28.00$

New material: Steel Strength, $f_s = f_{sm} = 420.00$

#####

Diameter, $D = 400.00$

Cover Thickness, $c = 25.00$

Element Length, $L = 3000.00$

Primary Member

Smooth Bars

Ductile Steel

With Detailing for Earthquake Resistance (including stirrups closed at 135°)

Longitudinal Bars With Ends Lapped Starting at the End Sections

Adequate Lap Length ($l_o/l_{ou,min} = l_b/l_d \geq 1$)

No FRP Wrapping

Stepwise Properties

EDGE -A-

Bending Moment, $M_a = -6.6801918E-012$

Shear Force, $V_a = 5.1847167E-015$

EDGE -B-

Bending Moment, $M_b = -8.9196430E-012$

Shear Force, $V_b = -5.1847167E-015$

BOTH EDGES

Axial Force, $F = -4770.074$

Longitudinal Reinforcement Area Distribution (in 2 divisions)

-Tension: $As_t = 1272.345$

-Compression: $As_c = 1781.283$

Longitudinal Reinforcement Area Distribution (in 3 divisions)

-Tension: $As_{t,ten} = 1017.876$

-Compression: $As_{c,com} = 1017.876$

-Middle: $As_{mid} = 1017.876$

Mean Diameter of Tension Reinforcement, $Db_{L,ten} = 18.00$

New component: From table 7-7, ASCE 41_17: Final Shear Capacity $V_R = 1.0 \cdot V_n = 270862.328$

V_n ((10.3), ASCE 41-17) = $k_n \cdot V_{Col} = 270862.328$

$V_{Col} = 270862.328$

$k_n = 1.00$

$displacement_ductility_demand = 0.00$

NOTE: In expression (10-3) ' $V_s = A_v \cdot f_y \cdot d / s$ ' is replaced by ' $V_s + f \cdot V_f$ ' where V_f is the contribution of FRPs (11.3), ACI 440).

= 1 (normal-weight concrete)

$f'_c = 20.00$, but $f_c^{0.5} \leq 8.3$ MPa (22.5.3.1, ACI 318-14)

$M/Vd = 2.00$

$M_u = 6.6801918E-012$

$V_u = 5.1847167E-015$

$d = 0.8 \cdot D = 320.00$

$N_u = 4770.074$

$A_g = 125663.706$

From (11.5.4.8), ACI 318-14: $V_s = 157913.67$

$A_v = \sqrt{2} \cdot A_{stirrup} = 123370.055$

$f_y = 400.00$

$s = 100.00$

V_s is multiplied by $Col = 0.00$

$s/d = 0.3125$

V_f ((11-3)-(11.4), ACI 440) = 0.00

From (11-11), ACI 440: $V_s + V_f \leq 238930.50$

$bw \cdot d = \frac{1}{4} \cdot d \cdot d = 80424.772$

$displacement_ductility_demand$ is calculated as $\frac{V_u}{V_R} \cdot \frac{1}{y}$

- Calculation of $\frac{V_u}{V_R} \cdot \frac{1}{y}$ for END A -
for rotation axis 2 and integ. section (a)

From analysis, chord rotation $\theta = 3.1936934E-021$

$y = (M_y \cdot L_s / 3) / Eleff = 0.01147479$ ((4.29), Biskinis Phd))

$M_y = 2.1517E+008$

$L_s = M/V$ (with $L_s > 0.1 \cdot L$ and $L_s < 2 \cdot L$) = 1500.00

From table 10.5, ASCE 41_17: $Eleff = factor \cdot E_c \cdot I_g = 9.3758E+012$

$factor = 0.30$

$A_g = 125663.706$

$f'_c = 28.00$

$N = 4770.074$

$$E_c \cdot I_g = 3.1253E+013$$

Calculation of Yielding Moment M_y

Calculation of ϕ_y and M_y according to (7) - (8) in Biskinis and Fardis

$$M_y = \min(M_{y_ten}, M_{y_com}) = 2.1517E+008$$

$$\phi_y ((10a) \text{ or } (10b)) = 1.1483167E-005$$

$$M_{y_ten} (8a) = 2.1517E+008$$

$$\phi_{y_ten} (7a) = 72.79826$$

$$\text{error of function (7a)} = 0.00155288$$

$$M_{y_com} (8b) = 4.0201E+008$$

$$\phi_{y_com} (7b) = 69.01017$$

$$\text{error of function (7b)} = -0.00040881$$

$$\text{with } e_y = 0.0021$$

$$e_{co} = 0.002$$

$$\alpha_{pl} = 0.35 ((9a) \text{ in Biskinis and Fardis for no FRP Wrap})$$

$$d_1 = 44.00$$

$$R = 200.00$$

$$v = 0.00135568$$

$$N = 4770.074$$

$$A_c = 125663.706$$

$$= 0.3645$$

$$\text{with } f_c = 28.00$$

Calculation of ratio I_b/I_d

Adequate Lap Length: $I_b/I_d \geq 1$

End Of Calculation of Shear Capacity for element: column CC1 of floor 1

At local axis: 3

Integration Section: (a)

Calculation No. 12

column C1, Floor 1

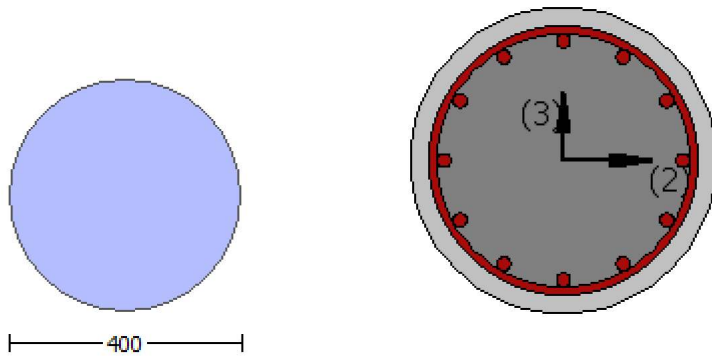
Limit State: Collapse Prevention (data interpolation between analysis steps 1 and 2)

Analysis: Uniform +X

Check: Chord rotation capacity (ϕ_u)

Edge: Start

Local Axis: (3)



Start Of Calculation of Shear Capacity ratio for element: column CC1 of floor 1

At Shear local axis: 3

(Bending local axis: 2)

Section Type: rccs

Constant Properties

Knowledge Factor, $\gamma = 1.00$

Mean strength values are used for both shear and moment calculations.

Consequently:

New material of Primary Member: Concrete Strength, $f_c = f_{cm} = 28.00$

New material of Primary Member: Steel Strength, $f_s = f_{sm} = 420.00$

Concrete Elasticity, $E_c = 24870.062$

Steel Elasticity, $E_s = 200000.00$

#####

Note: Especially for the calculation of moment strengths,

the above steel re-bar strengths are multiplied by 1.25 according to R18.6.5, ACI 318-14

New material: Steel Strength, $f_s = 1.25 \cdot f_{sm} = 525.00$

#####

Diameter, $D = 400.00$

Cover Thickness, $c = 25.00$

Mean Confinement Factor overall section = 1.32561

Element Length, $L = 3000.00$

Primary Member

Smooth Bars

Ductile Steel

With Detailing for Earthquake Resistance (including stirrups closed at 135°)

Longitudinal Bars With Ends Lapped Starting at the End Sections

Adequate Lap Length ($l_o/l_{ou, \min} \geq 1$)

No FRP Wrapping

Stepwise Properties

At local axis: 3

EDGE -A-

Shear Force, $V_a = -3.7372036E-031$

EDGE -B-

Shear Force, $V_b = 3.7372036E-031$

BOTH EDGES

Axial Force, $F = -4771.233$

Longitudinal Reinforcement Area Distribution (in 2 divisions)

-Tension: $As_t = 0.00$

-Compression: $As_c = 3053.628$

Longitudinal Reinforcement Area Distribution (in 3 divisions)

-Tension: $As_{t, \text{ten}} = 1017.876$

-Compression: $As_{c, \text{com}} = 1017.876$

-Middle: $As_{\text{mid}} = 1017.876$

Calculation of Shear Capacity ratio , $V_e/V_r = 0.51034251$

Member Controlled by Flexure ($V_e/V_r < 1$)

Calculation of Shear Demand from fig. R18.6.5, ACI 318-14 $V_e = (M_{pr1} + M_{pr2})/l_n = 152734.698$

with

$M_{pr1} = \text{Max}(M_{u1+}, M_{u1-}) = 2.2910\text{E}+008$

$M_{u1+} = 2.2910\text{E}+008$, is the ultimate moment strength at the edge 1 of the member in the actual moment direction which is defined for the static loading combination

$M_{u1-} = 2.2910\text{E}+008$, is the ultimate moment strength at the edge 1 of the member in the opposite moment direction which is defined for the static loading combination

$M_{pr2} = \text{Max}(M_{u2+}, M_{u2-}) = 2.2910\text{E}+008$

$M_{u2+} = 2.2910\text{E}+008$, is the ultimate moment strength at the edge 2 of the member in the actual moment direction which is defined for the static loading combination

$M_{u2-} = 2.2910\text{E}+008$, is the ultimate moment strength at the edge 2 of the member in the opposite moment direction which is defined for the static loading combination

Calculation of M_{u1+}

Calculation of ultimate Moment Strength ((3.67), Biskinis Phd), M_u

$M_u = 2.2910\text{E}+008$

$\phi = 1.0472$

$\phi' = 0.92729522$

error of function (3.68), Biskinis Phd = 58526.963

From 5A.2, TBDY: $f_{cc} = f_c \cdot c = 37.11712$

conf. factor $c = 1.32561$

$f_c = 28.00$

From 10.3.5, ASCE41-17, Final value of f_y : $f_y \cdot \text{Min}(1, 1.25 \cdot (l_b/d)^{2/3}) = 525.00$

$l_b/d = 1.00$

$d_1 = 44.00$

$R = 200.00$

$v = 0.00135568$

$N = 4771.233$

$A_c = 125663.706$

$= \phi' \cdot \text{Min}(1, 1.25 \cdot (l_b/d)^{2/3}) = 0.3645$

Calculation of ratio l_b/d

Adequate Lap Length: $l_b/d \geq 1$

Calculation of M_{u1-}

Calculation of ultimate Moment Strength ((3.67), Biskinis Phd), M_u

$M_u = 2.2910\text{E}+008$

$\phi = 1.0472$

$\phi' = 0.92729522$

error of function (3.68), Biskinis Phd = 58526.963

From 5A.2, TBDY: $f_{cc} = f_c \cdot c = 37.11712$

conf. factor $c = 1.32561$

$f_c = 28.00$

From 10.3.5, ASCE41-17, Final value of f_y : $f_y \cdot \text{Min}(1, 1.25 \cdot (l_b/d)^{2/3}) = 525.00$

$l_b/d = 1.00$

$d_1 = 44.00$

$R = 200.00$

$v = 0.00135568$

$N = 4771.233$

$A_c = 125663.706$

$= \phi' \cdot \text{Min}(1, 1.25 \cdot (l_b/d)^{2/3}) = 0.3645$

Calculation of ratio l_b/d

Adequate Lap Length: $l_b/d \geq 1$

Calculation of μ_{2+}

Calculation of ultimate Moment Strength ((3.67), Biskinis Phd), μ
 $\mu = 2.2910E+008$

$$= 1.0472$$

$$' = 0.92729522$$

error of function (3.68), Biskinis Phd = 58526.963

From 5A.2, TBDY: $f_{cc} = f_c \cdot c = 37.11712$

conf. factor $c = 1.32561$

$f_c = 28.00$

From 10.3.5, ASCE41-17, Final value of f_y : $f_y \cdot \text{Min}(1, 1.25 \cdot (l_b/d)^{2/3}) = 525.00$

$$l_b/d = 1.00$$

$$d_1 = 44.00$$

$$R = 200.00$$

$$v = 0.00135568$$

$$N = 4771.233$$

$$A_c = 125663.706$$

$$= \cdot \text{Min}(1, 1.25 \cdot (l_b/d)^{2/3}) = 0.3645$$

Calculation of ratio l_b/d

Adequate Lap Length: $l_b/d \geq 1$

Calculation of μ_{2-}

Calculation of ultimate Moment Strength ((3.67), Biskinis Phd), μ
 $\mu = 2.2910E+008$

$$= 1.0472$$

$$' = 0.92729522$$

error of function (3.68), Biskinis Phd = 58526.963

From 5A.2, TBDY: $f_{cc} = f_c \cdot c = 37.11712$

conf. factor $c = 1.32561$

$f_c = 28.00$

From 10.3.5, ASCE41-17, Final value of f_y : $f_y \cdot \text{Min}(1, 1.25 \cdot (l_b/d)^{2/3}) = 525.00$

$$l_b/d = 1.00$$

$$d_1 = 44.00$$

$$R = 200.00$$

$$v = 0.00135568$$

$$N = 4771.233$$

$$A_c = 125663.706$$

$$= \cdot \text{Min}(1, 1.25 \cdot (l_b/d)^{2/3}) = 0.3645$$

Calculation of ratio l_b/d

Adequate Lap Length: $l_b/d \geq 1$

Calculation of Shear Strength $V_r = \text{Min}(V_{r1}, V_{r2}) = 299278.805$

Calculation of Shear Strength at edge 1, $V_{r1} = 299278.805$

$V_{r1} = V_{Col} \text{ ((10.3), ASCE 41-17)} = k_{nl} * V_{Col0}$

$V_{Col0} = 299278.805$

$k_{nl} = 1$ (zero step-static loading)

NOTE: In expression (10-3) ' $V_s = A_v * f_y * d / s$ ' is replaced by ' $V_s + f * V_f$ ' where V_f is the contribution of FRPs (11.3), ACI 440).

$= 1$ (normal-weight concrete)

$f'_c = 28.00$, but $f'_c^{0.5} \leq 8.3$ MPa (22.5.3.1, ACI 318-14)

$M/Vd = 2.00$

$\mu_u = 1.0979367E-011$

$\nu_u = 3.7372036E-031$

$d = 0.8 * D = 320.00$

$N_u = 4771.233$

$A_g = 125663.706$

From (11.5.4.8), ACI 318-14: $V_s = 165809.354$

$A_v = \sqrt{2} * A_{stirrup} = 123370.055$

$f_y = 420.00$

$s = 100.00$

V_s is multiplied by $Col = 0.00$

$s/d = 0.3125$

$V_f \text{ ((11-3)-(11.4), ACI 440)} = 0.00$

From (11-11), ACI 440: $V_s + V_f \leq 282706.38$

$b_w * d = \sqrt{3} * d * d / 4 = 80424.772$

Calculation of Shear Strength at edge 2, $V_{r2} = 299278.805$

$V_{r2} = V_{Col} \text{ ((10.3), ASCE 41-17)} = k_{nl} * V_{Col0}$

$V_{Col0} = 299278.805$

$k_{nl} = 1$ (zero step-static loading)

NOTE: In expression (10-3) ' $V_s = A_v * f_y * d / s$ ' is replaced by ' $V_s + f * V_f$ ' where V_f is the contribution of FRPs (11.3), ACI 440).

$= 1$ (normal-weight concrete)

$f'_c = 28.00$, but $f'_c^{0.5} \leq 8.3$ MPa (22.5.3.1, ACI 318-14)

$M/Vd = 2.00$

$\mu_u = 1.0979367E-011$

$\nu_u = 3.7372036E-031$

$d = 0.8 * D = 320.00$

$N_u = 4771.233$

$A_g = 125663.706$

From (11.5.4.8), ACI 318-14: $V_s = 165809.354$

$A_v = \sqrt{2} * A_{stirrup} = 123370.055$

$f_y = 420.00$

$s = 100.00$

V_s is multiplied by $Col = 0.00$

$s/d = 0.3125$

$V_f \text{ ((11-3)-(11.4), ACI 440)} = 0.00$

From (11-11), ACI 440: $V_s + V_f \leq 282706.38$

$b_w * d = \sqrt{3} * d * d / 4 = 80424.772$

End Of Calculation of Shear Capacity ratio for element: column CC1 of floor 1

At local axis: 3

Start Of Calculation of Shear Capacity ratio for element: column CC1 of floor 1

At Shear local axis: 2

(Bending local axis: 3)

Section Type: rccs

Constant Properties

Knowledge Factor, $\phi = 1.00$
Mean strength values are used for both shear and moment calculations.
Consequently:
New material of Primary Member: Concrete Strength, $f_c = f_{cm} = 28.00$
New material of Primary Member: Steel Strength, $f_s = f_{sm} = 420.00$
Concrete Elasticity, $E_c = 24870.062$
Steel Elasticity, $E_s = 200000.00$

Note: Especially for the calculation of moment strengths,
the above steel re-bar strengths are multiplied by 1.25 according to R18.6.5, ACI 318-14
New material: Steel Strength, $f_s = 1.25 \cdot f_{sm} = 525.00$

Diameter, $D = 400.00$
Cover Thickness, $c = 25.00$
Mean Confinement Factor overall section = 1.32561
Element Length, $L = 3000.00$
Primary Member
Smooth Bars
Ductile Steel
With Detailing for Earthquake Resistance (including stirrups closed at 135°)
Longitudinal Bars With Ends Lapped Starting at the End Sections
Adequate Lap Length ($l_o/l_{o,min} \geq 1$)
No FRP Wrapping

Stepwise Properties

At local axis: 2
EDGE -A-
Shear Force, $V_a = -9.8626872E-031$
EDGE -B-
Shear Force, $V_b = 9.8626872E-031$
BOTH EDGES
Axial Force, $F = -4771.233$
Longitudinal Reinforcement Area Distribution (in 2 divisions)
-Tension: $As_t = 0.00$
-Compression: $As_c = 3053.628$
Longitudinal Reinforcement Area Distribution (in 3 divisions)
-Tension: $As_{t,ten} = 1017.876$
-Compression: $As_{c,com} = 1017.876$
-Middle: $As_{mid} = 1017.876$

Calculation of Shear Capacity ratio, $V_e/V_r = 0.51034251$
Member Controlled by Flexure ($V_e/V_r < 1$)
Calculation of Shear Demand from fig. R18.6.5, ACI 318-14 $V_e = (M_{pr1} + M_{pr2})/l_n = 152734.698$
with
 $M_{pr1} = \max(\mu_{u1+}, \mu_{u1-}) = 2.2910E+008$
 $\mu_{u1+} = 2.2910E+008$, is the ultimate moment strength at the edge 1 of the member in the actual moment direction
which is defined for the static loading combination
 $\mu_{u1-} = 2.2910E+008$, is the ultimate moment strength at the edge 1 of the member in the opposite moment
direction which is defined for the static loading combination
 $M_{pr2} = \max(\mu_{u2+}, \mu_{u2-}) = 2.2910E+008$
 $\mu_{u2+} = 2.2910E+008$, is the ultimate moment strength at the edge 2 of the member in the actual moment direction
which is defined for the the static loading combination
 $\mu_{u2-} = 2.2910E+008$, is the ultimate moment strength at the edge 2 of the member in the opposite moment
direction which is defined for the the static loading combination

Calculation of μ_{u1+}

Calculation of ultimate Moment Strength ((3.67), Biskinis Phd), μ_u

Mu = 2.2910E+008

= 1.0472

' = 0.92729522

error of function (3.68), Biskinis Phd = 58526.963

From 5A.2, TBDY: $f_{cc} = f_c \cdot c = 37.11712$

conf. factor $c = 1.32561$

$f_c = 28.00$

From 10.3.5, ASCE41-17, Final value of f_y : $f_y \cdot \text{Min}(1, 1.25 \cdot (l_b/d)^{2/3}) = 525.00$

$l_b/d = 1.00$

$d_1 = 44.00$

$R = 200.00$

$v = 0.00135568$

$N = 4771.233$

$A_c = 125663.706$

= $\cdot \text{Min}(1, 1.25 \cdot (l_b/d)^{2/3}) = 0.3645$

Calculation of ratio l_b/d

Adequate Lap Length: $l_b/d \geq 1$

Calculation of Mu1-

Calculation of ultimate Moment Strength ((3.67), Biskinis Phd), Mu

Mu = 2.2910E+008

= 1.0472

' = 0.92729522

error of function (3.68), Biskinis Phd = 58526.963

From 5A.2, TBDY: $f_{cc} = f_c \cdot c = 37.11712$

conf. factor $c = 1.32561$

$f_c = 28.00$

From 10.3.5, ASCE41-17, Final value of f_y : $f_y \cdot \text{Min}(1, 1.25 \cdot (l_b/d)^{2/3}) = 525.00$

$l_b/d = 1.00$

$d_1 = 44.00$

$R = 200.00$

$v = 0.00135568$

$N = 4771.233$

$A_c = 125663.706$

= $\cdot \text{Min}(1, 1.25 \cdot (l_b/d)^{2/3}) = 0.3645$

Calculation of ratio l_b/d

Adequate Lap Length: $l_b/d \geq 1$

Calculation of Mu2+

Calculation of ultimate Moment Strength ((3.67), Biskinis Phd), Mu

Mu = 2.2910E+008

= 1.0472

' = 0.92729522

error of function (3.68), Biskinis Phd = 58526.963

From 5A.2, TBDY: $f_{cc} = f_c \cdot c = 37.11712$

conf. factor $c = 1.32561$

$f_c = 28.00$

From 10.3.5, ASCE41-17, Final value of f_y : $f_y \cdot \text{Min}(1, 1.25 \cdot (l_b/d)^{2/3}) = 525.00$

$l_b/d = 1.00$
 $d_1 = 44.00$
 $R = 200.00$
 $v = 0.00135568$
 $N = 4771.233$
 $A_c = 125663.706$
 $= \text{Min}(1, 1.25 \cdot (l_b/d)^{2/3}) = 0.3645$

Calculation of ratio l_b/d

Adequate Lap Length: $l_b/d \geq 1$

Calculation of μ_2 -

Calculation of ultimate Moment Strength ((3.67), Biskinis Phd), μ
 $\mu = 2.2910 \times 10^8$

$= 1.0472$

$' = 0.92729522$

error of function (3.68), Biskinis Phd = 58526.963

From 5A.2, TBDY: $f_{cc} = f_c' \cdot c = 37.11712$

conf. factor $c = 1.32561$

$f_c = 28.00$

From 10.3.5, ASCE41-17, Final value of f_y : $f_y \cdot \text{Min}(1, 1.25 \cdot (l_b/d)^{2/3}) = 525.00$

$l_b/d = 1.00$

$d_1 = 44.00$

$R = 200.00$

$v = 0.00135568$

$N = 4771.233$

$A_c = 125663.706$

$= \text{Min}(1, 1.25 \cdot (l_b/d)^{2/3}) = 0.3645$

Calculation of ratio l_b/d

Adequate Lap Length: $l_b/d \geq 1$

Calculation of Shear Strength $V_r = \text{Min}(V_{r1}, V_{r2}) = 299278.805$

Calculation of Shear Strength at edge 1, $V_{r1} = 299278.805$

$V_{r1} = V_{co1}$ ((10.3), ASCE 41-17) = $k_n \cdot V_{co10}$

$V_{co10} = 299278.805$

$k_n = 1$ (zero step-static loading)

NOTE: In expression (10-3) ' $V_s = A_v \cdot f_y \cdot d/s$ ' is replaced by ' $V_s + f \cdot V_f$ ' where V_f is the contribution of FRPs (11.3), ACI 440).

$= 1$ (normal-weight concrete)

$f_c' = 28.00$, but $f_c'^{0.5} \leq 8.3$ MPa (22.5.3.1, ACI 318-14)

$M/Vd = 2.00$

$\mu = 9.9556147 \times 10^{-12}$

$V_u = 9.8626872 \times 10^{-31}$

$d = 0.8 \cdot D = 320.00$

$N_u = 4771.233$

$A_g = 125663.706$

From (11.5.4.8), ACI 318-14: $V_s = 165809.354$

$A_v = \frac{1}{2} \cdot A_{\text{stirrup}} = 123370.055$

$f_y = 420.00$

$s = 100.00$

Vs is multiplied by Col = 0.00
s/d = 0.3125
Vf ((11-3)-(11.4), ACI 440) = 0.00
From (11-11), ACI 440: Vs + Vf <= 282706.38
bw*d = *d*d/4 = 80424.772

Calculation of Shear Strength at edge 2, Vr2 = 299278.805
Vr2 = VCol ((10.3), ASCE 41-17) = knl*VCol0
VCol0 = 299278.805
knl = 1 (zero step-static loading)

NOTE: In expression (10-3) 'Vs = Av*fy*d/s' is replaced by 'Vs+ f*VF'
where Vf is the contribution of FRPs (11.3), ACI 440).

= 1 (normal-weight concrete)
fc' = 28.00, but $fc'^{0.5} \leq 8.3$ MPa (22.5.3.1, ACI 318-14)
M/Vd = 2.00
Mu = 9.9556147E-012
Vu = 9.8626872E-031
d = 0.8*D = 320.00
Nu = 4771.233
Ag = 125663.706
From (11.5.4.8), ACI 318-14: Vs = 165809.354
Av = /2*A_stirrup = 123370.055
fy = 420.00
s = 100.00
Vs is multiplied by Col = 0.00
s/d = 0.3125
Vf ((11-3)-(11.4), ACI 440) = 0.00
From (11-11), ACI 440: Vs + Vf <= 282706.38
bw*d = *d*d/4 = 80424.772

End Of Calculation of Shear Capacity ratio for element: column CC1 of floor 1
At local axis: 2

Start Of Calculation of Chord Rotation Capacity for element: column CC1 of floor 1
At local axis: 3
Integration Section: (a)
Section Type: rccs

Constant Properties

Knowledge Factor, = 1.00
Chord Rotation is generally considered as Deformation-Controlled Action according to Table C7-1, ASCE41-17.
Mean strengths are used for Deformation-Controlled Actions according to 7.5.1.3, ASCE 41-17
Consequently:
New material of Primary Member: Concrete Strength, fc = fcm = 28.00
New material of Primary Member: Steel Strength, fs = fsm = 420.00
Concrete Elasticity, Ec = 24870.062
Steel Elasticity, Es = 200000.00
Diameter, D = 400.00
Cover Thickness, c = 25.00
Element Length, L = 3000.00
Primary Member
Smooth Bars
Ductile Steel
With Detailing for Earthquake Resistance (including stirrups closed at 135°)
Longitudinal Bars With Ends Lapped Starting at the End Sections
Adequate Lap Length ($l_b/d \geq 1$)
No FRP Wrapping

Stepwise Properties

Bending Moment, $M = -8.6921\text{E}+006$

Shear Force, $V2 = -2896.171$

Shear Force, $V3 = 5.1847167\text{E}-015$

Axial Force, $F = -4770.074$

Longitudinal Reinforcement Area Distribution (in 2 divisions)

-Tension: $As_t = 1272.345$

-Compression: $As_c = 1781.283$

Longitudinal Reinforcement Area Distribution (in 3 divisions)

-Tension: $As_{t,ten} = 1017.876$

-Compression: $As_{c,com} = 1017.876$

-Middle: $As_{mid} = 1017.876$

Mean Diameter of Tension Reinforcement, $Db_L = 18.00$

New component: From table 7-7, ASCE 41_17: Final chord rotation Capacity $u_R = 1.0^*$ $u = 0.08283389$

$u = y + p = 0.08283389$

- Calculation of y -

$y = (My * L_s / 3) / E_{eff} = 0.02295907$ ((4.29), Biskinis Phd))

$My = 2.1517\text{E}+008$

$L_s = M/V$ (with $L_s > 0.1 * L$ and $L_s < 2 * L$) = 3001.241

From table 10.5, ASCE 41_17: $E_{eff} = \text{factor} * E_c * I_g = 9.3758\text{E}+012$

factor = 0.30

$A_g = 125663.706$

$f_c' = 28.00$

$N = 4770.074$

$E_c * I_g = 3.1253\text{E}+013$

Calculation of Yielding Moment My

Calculation of y and My according to (7) - (8) in Biskinis and Fardis

$My = \text{Min}(My_{ten}, My_{com}) = 2.1517\text{E}+008$

y ((10a) or (10b)) = 1.1483167E-005

My_{ten} (8a) = 2.1517E+008

y_{ten} (7a) = 72.79826

error of function (7a) = 0.00155288

My_{com} (8b) = 4.0201E+008

y_{com} (7b) = 69.01017

error of function (7b) = -0.00040881

with $e_y = 0.0021$

$e_{co} = 0.002$

$a_{pl} = 0.35$ ((9a) in Biskinis and Fardis for no FRP Wrap)

$d_1 = 44.00$

$R = 200.00$

$v = 0.00135568$

$N = 4770.074$

$A_c = 125663.706$

= 0.3645

with $f_c = 28.00$

Calculation of ratio l_b / l_d

Adequate Lap Length: $l_b / l_d \geq 1$

- Calculation of p -

From table 10-9: $p = 0.05987483$

with:

- Columns controlled by inadequate development or splicing along the clear height because $l_b/l_d < 1$

shear control ratio $V_{yE}/V_{CoIE} = 0.51034251$

$d = 0.00$

$s = 0.00$

$t = 2 \cdot A_v / (d_c \cdot s) + 4 \cdot t_f / D \cdot (f_{fe} / f_s) = 0.00$

$A_v = 78.53982$, is the area of the circular stirrup

$d_c = D - 2 \cdot \text{cover}$ - Hoop Diameter = 340.00

The term $2 \cdot t_f / b_w \cdot (f_{fe} / f_s)$ is implemented to account for FRP contribution

where $f = 2 \cdot t_f / b_w$ is FRP ratio (EC8 - 3, A.4.4.3(6)) and f_{fe} / f_s normalises f to steel strength

All these variables have already been given in Shear control ratio calculation.

$NUD = 4770.074$

$A_g = 125663.706$

$f_{cE} = 28.00$

$f_{yE} = f_{yIE} = 420.00$

$p_l = \text{Area_Tot_Long_Rein} / (A_g) = 0.0243$

$f_{cE} = 28.00$

End Of Calculation of Chord Rotation Capacity for element: column CC1 of floor 1

At local axis: 3

Integration Section: (a)

Calculation No. 13

column C1, Floor 1

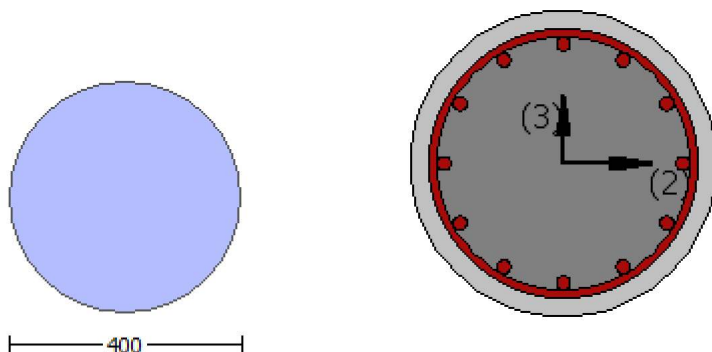
Limit State: Collapse Prevention (data interpolation between analysis steps 1 and 2)

Analysis: Uniform +X

Check: Shear capacity VR_d

Edge: End

Local Axis: (2)



Start Of Calculation of Shear Capacity for element: column CC1 of floor 1

At local axis: 2

Integration Section: (b)

Section Type: rccs

Constant Properties

Knowledge Factor, $\phi = 1.00$

Member Shear Force is generally considered as Force-Controlled Action according to Table C7-1, ASCE41-17.

Lower-bound strengths are used for Force-Controlled Actions according to 7.5.1.3, ASCE 41-17

Consequently:

New material of Primary Member: Concrete Strength, $f_c = f_{c_lower_bound} = 20.00$

New material of Primary Member: Steel Strength, $f_s = f_{s_lower_bound} = 400.00$

Concrete Elasticity, $E_c = 24870.062$

Steel Elasticity, $E_s = 200000.00$

#####

Note: Especially for the calculation of ϕ for displacement ductility demand, the expected (mean value) strengths are used (7.5.1.3, ASCE41-17) because bending is considered as Deformation-Controlled Action (Table C7-1, ASCE41-17).

New material: Concrete Strength, $f_c = f_{cm} = 28.00$

New material: Steel Strength, $f_s = f_{sm} = 420.00$

#####

Diameter, $D = 400.00$

Cover Thickness, $c = 25.00$

Element Length, $L = 3000.00$

Primary Member

Smooth Bars

Ductile Steel

With Detailing for Earthquake Resistance (including stirrups closed at 135°)

Longitudinal Bars With Ends Lapped Starting at the End Sections

Adequate Lap Length ($l_o/l_{ou,min} = l_b/l_d \geq 1$)

No FRP Wrapping

Stepwise Properties

EDGE -A-

Bending Moment, $M_a = -8.6921E+006$

Shear Force, $V_a = -2896.171$

EDGE -B-

Bending Moment, $M_b = 0.02826837$

Shear Force, $V_b = 2896.171$

BOTH EDGES

Axial Force, $F = -4770.074$

Longitudinal Reinforcement Area Distribution (in 2 divisions)

-Tension: $A_{sl,t} = 0.00$

-Compression: $A_{sl,c} = 3053.628$

Longitudinal Reinforcement Area Distribution (in 3 divisions)

-Tension: $A_{sl,ten} = 1017.876$

-Compression: $A_{sl,com} = 1017.876$

-Middle: $A_{sl,mid} = 1017.876$

Mean Diameter of Tension Reinforcement, $D_{bL,ten} = 18.00$

New component: From table 7-7, ASCE 41_17: Final Shear Capacity $V_R = 1.0 \cdot V_n = 270862.328$

V_n ((10.3), ASCE 41-17) = $k_n \cdot V_{CoI} = 270862.328$

$V_{CoI} = 270862.328$

$k_n = 1.00$

$displacement_ductility_demand = 0.06058843$

NOTE: In expression (10-3) ' $V_s = A_v \cdot f_y \cdot d/s$ ' is replaced by ' $V_{s+} = f^* V_f$ ' where V_f is the contribution of FRPs (11.3), ACI 440).

$\phi = 1$ (normal-weight concrete)

$f'_c = 20.00$, but $f'_c^{0.5} \leq 8.3$ MPa (22.5.3.1, ACI 318-14)

$M/V_d = 2.00$

$M_u = 0.02826837$

$V_u = 2896.171$

$d = 0.8 \cdot D = 320.00$

$N_u = 4770.074$

$A_g = 125663.706$
 From (11.5.4.8), ACI 318-14: $V_s = 157913.67$
 $A_v = \sqrt{2} A_{stirrup} = 123370.055$
 $f_y = 400.00$
 $s = 100.00$
 V_s is multiplied by $Col = 0.00$
 $s/d = 0.3125$
 $V_f ((11-3)-(11.4), ACI 440) = 0.00$
 From (11-11), ACI 440: $V_s + V_f \leq 238930.50$
 $b_w d = \sqrt{2} d^2 / 4 = 80424.772$

displacement ductility demand is calculated as δ / y

- Calculation of δ / y for END B -
 for rotation axis 3 and integ. section (b)

From analysis, chord rotation $\theta = 0.00013905$
 $y = (M_y * L_s / 3) / E I_{eff} = 0.00229496 ((4.29), Biskinis Phd)$
 $M_y = 2.1517E+008$
 $L_s = M/V$ (with $L_s > 0.1 * L$ and $L_s < 2 * L$) = 300.00
 From table 10.5, ASCE 41_17: $E I_{eff} = factor * E_c * I_g = 9.3758E+012$
 $factor = 0.30$
 $A_g = 125663.706$
 $f_c' = 28.00$
 $N = 4770.074$
 $E_c * I_g = 3.1253E+013$

Calculation of Yielding Moment M_y

Calculation of δ and M_y according to (7) - (8) in Biskinis and Fardis

$M_y = \min(M_{y_ten}, M_{y_com}) = 2.1517E+008$
 $y ((10a) \text{ or } (10b)) = 1.1483167E-005$
 $M_{y_ten} (8a) = 2.1517E+008$
 $\delta_{ten} (7a) = 72.79826$
 $error \text{ of function } (7a) = 0.00155288$
 $M_{y_com} (8b) = 4.0201E+008$
 $\delta_{com} (7b) = 69.01017$
 $error \text{ of function } (7b) = -0.00040881$
 with $e_y = 0.0021$
 $e_{co} = 0.002$
 $a_{pl} = 0.35 ((9a) \text{ in Biskinis and Fardis for no FRP Wrap})$
 $d_1 = 44.00$
 $R = 200.00$
 $v = 0.00135568$
 $N = 4770.074$
 $A_c = 125663.706$
 $= 0.3645$
 with $f_c = 28.00$

Calculation of ratio I_b / I_d

Adequate Lap Length: $I_b / I_d \geq 1$

End Of Calculation of Shear Capacity for element: column CC1 of floor 1

At local axis: 2

Integration Section: (b)

Calculation No. 14

column C1, Floor 1

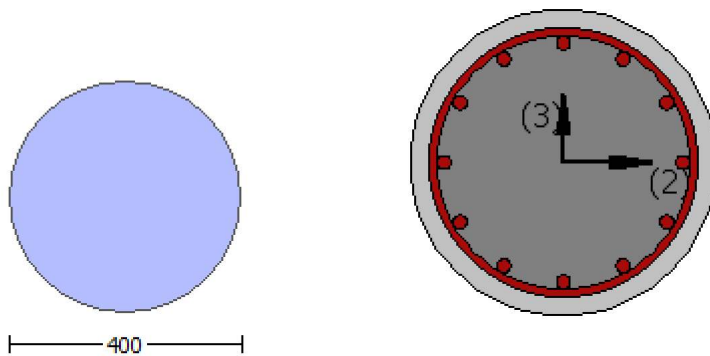
Limit State: Collapse Prevention (data interpolation between analysis steps 1 and 2)

Analysis: Uniform +X

Check: Chord rotation capacity (μ)

Edge: End

Local Axis: (2)



Start Of Calculation of Shear Capacity ratio for element: column CC1 of floor 1

At Shear local axis: 3

(Bending local axis: 2)

Section Type: rccs

Constant Properties

Knowledge Factor, $\gamma = 1.00$

Mean strength values are used for both shear and moment calculations.

Consequently:

New material of Primary Member: Concrete Strength, $f_c = f_{cm} = 28.00$

New material of Primary Member: Steel Strength, $f_s = f_{sm} = 420.00$

Concrete Elasticity, $E_c = 24870.062$

Steel Elasticity, $E_s = 200000.00$

#####

Note: Especially for the calculation of moment strengths,

the above steel re-bar strengths are multiplied by 1.25 according to R18.6.5, ACI 318-14

New material: Steel Strength, $f_s = 1.25 \cdot f_{sm} = 525.00$

#####

Diameter, $D = 400.00$

Cover Thickness, $c = 25.00$

Mean Confinement Factor overall section = 1.32561

Element Length, $L = 3000.00$

Primary Member

Smooth Bars

Ductile Steel

With Detailing for Earthquake Resistance (including stirrups closed at 135°)

Longitudinal Bars With Ends Lapped Starting at the End Sections

Adequate Lap Length ($l_o/l_{ou, \min} \geq 1$)

No FRP Wrapping

Stepwise Properties

At local axis: 3

EDGE -A-

Shear Force, $V_a = -3.7372036\text{E-}031$

EDGE -B-

Shear Force, $V_b = 3.7372036\text{E-}031$

BOTH EDGES

Axial Force, $F = -4771.233$

Longitudinal Reinforcement Area Distribution (in 2 divisions)

-Tension: $As_t = 0.00$

-Compression: $As_c = 3053.628$

Longitudinal Reinforcement Area Distribution (in 3 divisions)

-Tension: $As_{l,ten} = 1017.876$

-Compression: $As_{l,com} = 1017.876$

-Middle: $As_{l,mid} = 1017.876$

Calculation of Shear Capacity ratio, $V_e/V_r = 0.51034251$

Member Controlled by Flexure ($V_e/V_r < 1$)

Calculation of Shear Demand from fig. R18.6.5, ACI 318-14 $V_e = (M_{pr1} + M_{pr2})/l_n = 152734.698$
with

$M_{pr1} = \text{Max}(Mu_{1+}, Mu_{1-}) = 2.2910\text{E}+008$

$Mu_{1+} = 2.2910\text{E}+008$, is the ultimate moment strength at the edge 1 of the member in the actual moment direction which is defined for the static loading combination

$Mu_{1-} = 2.2910\text{E}+008$, is the ultimate moment strength at the edge 1 of the member in the opposite moment direction which is defined for the static loading combination

$M_{pr2} = \text{Max}(Mu_{2+}, Mu_{2-}) = 2.2910\text{E}+008$

$Mu_{2+} = 2.2910\text{E}+008$, is the ultimate moment strength at the edge 2 of the member in the actual moment direction which is defined for the static loading combination

$Mu_{2-} = 2.2910\text{E}+008$, is the ultimate moment strength at the edge 2 of the member in the opposite moment direction which is defined for the static loading combination

Calculation of Mu_{1+}

Calculation of ultimate Moment Strength ((3.67), Biskinis Phd), M_u

$M_u = 2.2910\text{E}+008$

$\phi = 1.0472$

$\phi' = 0.92729522$

error of function (3.68), Biskinis Phd = 58526.963

From 5A.2, TDY: $f_{cc} = f_c^* \quad c = 37.11712$

conf. factor $c = 1.32561$

$f_c = 28.00$

From 10.3.5, ASCE41-17, Final value of f_y : $f_y \cdot \text{Min}(1, 1.25 \cdot (l_b/d)^{2/3}) = 525.00$

$l_b/d = 1.00$

$d_1 = 44.00$

$R = 200.00$

$v = 0.00135568$

$N = 4771.233$

$A_c = 125663.706$

$= \phi' \cdot \text{Min}(1, 1.25 \cdot (l_b/d)^{2/3}) = 0.3645$

Calculation of ratio l_b/d

Adequate Lap Length: $l_b/d \geq 1$

Calculation of Mu_{1-}

Calculation of ultimate Moment Strength ((3.67), Biskinis Phd), Mu
Mu = 2.2910E+008

= 1.0472
' = 0.92729522
error of function (3.68), Biskinis Phd = 58526.963
From 5A.2, TDY: fcc = fc* c = 37.11712
conf. factor c = 1.32561
fc = 28.00
From 10.3.5, ASCE41-17, Final value of fy: fy*Min(1,1.25*(lb/d)^ 2/3) = 525.00
lb/d = 1.00
d1 = 44.00
R = 200.00
v = 0.00135568
N = 4771.233
Ac = 125663.706
= *Min(1,1.25*(lb/d)^ 2/3) = 0.3645

Calculation of ratio lb/d

Adequate Lap Length: lb/d >= 1

Calculation of Mu2+

Calculation of ultimate Moment Strength ((3.67), Biskinis Phd), Mu
Mu = 2.2910E+008

= 1.0472
' = 0.92729522
error of function (3.68), Biskinis Phd = 58526.963
From 5A.2, TDY: fcc = fc* c = 37.11712
conf. factor c = 1.32561
fc = 28.00
From 10.3.5, ASCE41-17, Final value of fy: fy*Min(1,1.25*(lb/d)^ 2/3) = 525.00
lb/d = 1.00
d1 = 44.00
R = 200.00
v = 0.00135568
N = 4771.233
Ac = 125663.706
= *Min(1,1.25*(lb/d)^ 2/3) = 0.3645

Calculation of ratio lb/d

Adequate Lap Length: lb/d >= 1

Calculation of Mu2-

Calculation of ultimate Moment Strength ((3.67), Biskinis Phd), Mu
Mu = 2.2910E+008

= 1.0472
' = 0.92729522
error of function (3.68), Biskinis Phd = 58526.963
From 5A.2, TDY: fcc = fc* c = 37.11712

conf. factor $c = 1.32561$
 $f_c = 28.00$
 From 10.3.5, ASCE41-17, Final value of f_y : $f_y * \text{Min}(1, 1.25 * (l_b / d)^{2/3}) = 525.00$
 $l_b / d = 1.00$
 $d_1 = 44.00$
 $R = 200.00$
 $v = 0.00135568$
 $N = 4771.233$
 $A_c = 125663.706$
 $= * \text{Min}(1, 1.25 * (l_b / d)^{2/3}) = 0.3645$

Calculation of ratio l_b / d

Adequate Lap Length: $l_b / d \geq 1$

Calculation of Shear Strength $V_r = \text{Min}(V_{r1}, V_{r2}) = 299278.805$

Calculation of Shear Strength at edge 1, $V_{r1} = 299278.805$

$V_{r1} = V_{Col}$ ((10.3), ASCE 41-17) = $k_n l * V_{ColO}$

$V_{ColO} = 299278.805$

$k_n l = 1$ (zero step-static loading)

NOTE: In expression (10-3) ' $V_s = A_v * f_y * d / s$ ' is replaced by ' $V_s + f * V_f$ ' where V_f is the contribution of FRPs ((11.3), ACI 440).

$= 1$ (normal-weight concrete)

$f'_c = 28.00$, but $f_c^{0.5} \leq 8.3$ MPa ((22.5.3.1, ACI 318-14)

$M / V d = 2.00$

$\mu_u = 1.0979367E-011$

$\nu_u = 3.7372036E-031$

$d = 0.8 * D = 320.00$

$N_u = 4771.233$

$A_g = 125663.706$

From ((11.5.4.8), ACI 318-14: $V_s = 165809.354$

$A_v = / 2 * A_{stirrup} = 123370.055$

$f_y = 420.00$

$s = 100.00$

V_s is multiplied by $Col = 0.00$

$s / d = 0.3125$

V_f ((11-3)-(11.4), ACI 440) = 0.00

From ((11-11), ACI 440: $V_s + V_f \leq 282706.38$

$b_w * d = * d * d / 4 = 80424.772$

Calculation of Shear Strength at edge 2, $V_{r2} = 299278.805$

$V_{r2} = V_{Col}$ ((10.3), ASCE 41-17) = $k_n l * V_{ColO}$

$V_{ColO} = 299278.805$

$k_n l = 1$ (zero step-static loading)

NOTE: In expression (10-3) ' $V_s = A_v * f_y * d / s$ ' is replaced by ' $V_s + f * V_f$ ' where V_f is the contribution of FRPs ((11.3), ACI 440).

$= 1$ (normal-weight concrete)

$f'_c = 28.00$, but $f_c^{0.5} \leq 8.3$ MPa ((22.5.3.1, ACI 318-14)

$M / V d = 2.00$

$\mu_u = 1.0979367E-011$

$\nu_u = 3.7372036E-031$

$d = 0.8 * D = 320.00$

$N_u = 4771.233$

$A_g = 125663.706$

From ((11.5.4.8), ACI 318-14: $V_s = 165809.354$

$A_v = / 2 * A_{stirrup} = 123370.055$

$f_y = 420.00$

$s = 100.00$
 V_s is multiplied by $Col = 0.00$
 $s/d = 0.3125$
 $V_f ((11-3)-(11.4), ACI 440) = 0.00$
 From (11-11), ACI 440: $V_s + V_f \leq 282706.38$
 $bw*d = *d*d/4 = 80424.772$

 End Of Calculation of Shear Capacity ratio for element: column CC1 of floor 1
 At local axis: 3

Start Of Calculation of Shear Capacity ratio for element: column CC1 of floor 1
 At Shear local axis: 2
 (Bending local axis: 3)
 Section Type: rccs

Constant Properties

 Knowledge Factor, $\gamma = 1.00$
 Mean strength values are used for both shear and moment calculations.
 Consequently:
 New material of Primary Member: Concrete Strength, $f_c = f_{cm} = 28.00$
 New material of Primary Member: Steel Strength, $f_s = f_{sm} = 420.00$
 Concrete Elasticity, $E_c = 24870.062$
 Steel Elasticity, $E_s = 200000.00$
 #####
 Note: Especially for the calculation of moment strengths,
 the above steel re-bar strengths are multiplied by 1.25 according to R18.6.5, ACI 318-14
 New material: Steel Strength, $f_s = 1.25*f_{sm} = 525.00$
 #####
 Diameter, $D = 400.00$
 Cover Thickness, $c = 25.00$
 Mean Confinement Factor overall section = 1.32561
 Element Length, $L = 3000.00$
 Primary Member
 Smooth Bars
 Ductile Steel
 With Detailing for Earthquake Resistance (including stirrups closed at 135°)
 Longitudinal Bars With Ends Lapped Starting at the End Sections
 Adequate Lap Length ($l_o/l_{ou,min} \geq 1$)
 No FRP Wrapping

Stepwise Properties

 At local axis: 2
 EDGE -A-
 Shear Force, $V_a = -9.8626872E-031$
 EDGE -B-
 Shear Force, $V_b = 9.8626872E-031$
 BOTH EDGES
 Axial Force, $F = -4771.233$
 Longitudinal Reinforcement Area Distribution (in 2 divisions)
 -Tension: $As_t = 0.00$
 -Compression: $As_c = 3053.628$
 Longitudinal Reinforcement Area Distribution (in 3 divisions)
 -Tension: $As_{t,ten} = 1017.876$
 -Compression: $As_{l,com} = 1017.876$
 -Middle: $As_{l,mid} = 1017.876$

Calculation of Shear Capacity ratio , $V_e/V_r = 0.51034251$

Member Controlled by Flexure ($V_e/V_r < 1$)

Calculation of Shear Demand from fig. R18.6.5, ACI 318-14 $V_e = (M_{pr1} + M_{pr2})/l_n = 152734.698$
with

$M_{pr1} = \text{Max}(\mu_{u1+}, \mu_{u1-}) = 2.2910\text{E}+008$

$\mu_{u1+} = 2.2910\text{E}+008$, is the ultimate moment strength at the edge 1 of the member in the actual moment direction which is defined for the static loading combination

$\mu_{u1-} = 2.2910\text{E}+008$, is the ultimate moment strength at the edge 1 of the member in the opposite moment direction which is defined for the static loading combination

$M_{pr2} = \text{Max}(\mu_{u2+}, \mu_{u2-}) = 2.2910\text{E}+008$

$\mu_{u2+} = 2.2910\text{E}+008$, is the ultimate moment strength at the edge 2 of the member in the actual moment direction which is defined for the static loading combination

$\mu_{u2-} = 2.2910\text{E}+008$, is the ultimate moment strength at the edge 2 of the member in the opposite moment direction which is defined for the static loading combination

Calculation of μ_{u1+}

Calculation of ultimate Moment Strength ((3.67), Biskinis Phd), μ_u
 $\mu_u = 2.2910\text{E}+008$

$\phi = 1.0472$

$\phi' = 0.92729522$

error of function (3.68), Biskinis Phd = 58526.963

From 5A.2, TB DY: $f_{cc} = f_c^* \quad c = 37.11712$

conf. factor $c = 1.32561$

$f_c = 28.00$

From 10.3.5, ASCE41-17, Final value of f_y : $f_y \cdot \text{Min}(1, 1.25 \cdot (l_b/d)^{2/3}) = 525.00$

$l_b/d = 1.00$

$d_1 = 44.00$

$R = 200.00$

$v = 0.00135568$

$N = 4771.233$

$A_c = 125663.706$

$= \phi' \cdot \text{Min}(1, 1.25 \cdot (l_b/d)^{2/3}) = 0.3645$

Calculation of ratio l_b/d

Adequate Lap Length: $l_b/d \geq 1$

Calculation of μ_{u1-}

Calculation of ultimate Moment Strength ((3.67), Biskinis Phd), μ_u
 $\mu_u = 2.2910\text{E}+008$

$\phi = 1.0472$

$\phi' = 0.92729522$

error of function (3.68), Biskinis Phd = 58526.963

From 5A.2, TB DY: $f_{cc} = f_c^* \quad c = 37.11712$

conf. factor $c = 1.32561$

$f_c = 28.00$

From 10.3.5, ASCE41-17, Final value of f_y : $f_y \cdot \text{Min}(1, 1.25 \cdot (l_b/d)^{2/3}) = 525.00$

$l_b/d = 1.00$

$d_1 = 44.00$

$R = 200.00$

$v = 0.00135568$

$N = 4771.233$

$A_c = 125663.706$

$= \phi' \cdot \text{Min}(1, 1.25 \cdot (l_b/d)^{2/3}) = 0.3645$

Calculation of ratio l_b/d

Adequate Lap Length: $l_b/d \geq 1$

Calculation of μ_{2+}

Calculation of ultimate Moment Strength ((3.67), Biskinis Phd), μ
 $\mu = 2.2910E+008$

$$= 1.0472$$

$$' = 0.92729522$$

error of function (3.68), Biskinis Phd = 58526.963

From 5A.2, TDY: $f_{cc} = f_c \cdot c = 37.11712$

conf. factor $c = 1.32561$

$f_c = 28.00$

From 10.3.5, ASCE41-17, Final value of f_y : $f_y \cdot \text{Min}(1, 1.25 \cdot (l_b/d)^{2/3}) = 525.00$

$$l_b/d = 1.00$$

$$d_1 = 44.00$$

$$R = 200.00$$

$$v = 0.00135568$$

$$N = 4771.233$$

$$A_c = 125663.706$$

$$= \cdot \text{Min}(1, 1.25 \cdot (l_b/d)^{2/3}) = 0.3645$$

Calculation of ratio l_b/d

Adequate Lap Length: $l_b/d \geq 1$

Calculation of μ_{2-}

Calculation of ultimate Moment Strength ((3.67), Biskinis Phd), μ
 $\mu = 2.2910E+008$

$$= 1.0472$$

$$' = 0.92729522$$

error of function (3.68), Biskinis Phd = 58526.963

From 5A.2, TDY: $f_{cc} = f_c \cdot c = 37.11712$

conf. factor $c = 1.32561$

$f_c = 28.00$

From 10.3.5, ASCE41-17, Final value of f_y : $f_y \cdot \text{Min}(1, 1.25 \cdot (l_b/d)^{2/3}) = 525.00$

$$l_b/d = 1.00$$

$$d_1 = 44.00$$

$$R = 200.00$$

$$v = 0.00135568$$

$$N = 4771.233$$

$$A_c = 125663.706$$

$$= \cdot \text{Min}(1, 1.25 \cdot (l_b/d)^{2/3}) = 0.3645$$

Calculation of ratio l_b/d

Adequate Lap Length: $l_b/d \geq 1$

Calculation of Shear Strength $V_r = \text{Min}(V_{r1}, V_{r2}) = 299278.805$

Calculation of Shear Strength at edge 1, $V_{r1} = 299278.805$

$V_{r1} = V_{Col} ((10.3), ASCE 41-17) = knl * V_{ColO}$

$V_{ColO} = 299278.805$

$knl = 1$ (zero step-static loading)

NOTE: In expression (10-3) ' $V_s = A_v * f_y * d / s$ ' is replaced by ' $V_s + f * V_f$ ' where V_f is the contribution of FRPs (11.3), ACI 440).

$= 1$ (normal-weight concrete)

$f'_c = 28.00$, but $f_c^{0.5} \leq 8.3$ MPa (22.5.3.1, ACI 318-14)

$M/Vd = 2.00$

$\mu_u = 9.9556147E-012$

$\nu_u = 9.8626872E-031$

$d = 0.8 * D = 320.00$

$N_u = 4771.233$

$A_g = 125663.706$

From (11.5.4.8), ACI 318-14: $V_s = 165809.354$

$A_v = /2 * A_{stirrup} = 123370.055$

$f_y = 420.00$

$s = 100.00$

V_s is multiplied by $Col = 0.00$

$s/d = 0.3125$

$V_f ((11-3)-(11.4), ACI 440) = 0.00$

From (11-11), ACI 440: $V_s + V_f \leq 282706.38$

$bw * d = *d * d / 4 = 80424.772$

Calculation of Shear Strength at edge 2, $V_{r2} = 299278.805$

$V_{r2} = V_{Col} ((10.3), ASCE 41-17) = knl * V_{ColO}$

$V_{ColO} = 299278.805$

$knl = 1$ (zero step-static loading)

NOTE: In expression (10-3) ' $V_s = A_v * f_y * d / s$ ' is replaced by ' $V_s + f * V_f$ ' where V_f is the contribution of FRPs (11.3), ACI 440).

$= 1$ (normal-weight concrete)

$f'_c = 28.00$, but $f_c^{0.5} \leq 8.3$ MPa (22.5.3.1, ACI 318-14)

$M/Vd = 2.00$

$\mu_u = 9.9556147E-012$

$\nu_u = 9.8626872E-031$

$d = 0.8 * D = 320.00$

$N_u = 4771.233$

$A_g = 125663.706$

From (11.5.4.8), ACI 318-14: $V_s = 165809.354$

$A_v = /2 * A_{stirrup} = 123370.055$

$f_y = 420.00$

$s = 100.00$

V_s is multiplied by $Col = 0.00$

$s/d = 0.3125$

$V_f ((11-3)-(11.4), ACI 440) = 0.00$

From (11-11), ACI 440: $V_s + V_f \leq 282706.38$

$bw * d = *d * d / 4 = 80424.772$

End Of Calculation of Shear Capacity ratio for element: column CC1 of floor 1

At local axis: 2

Start Of Calculation of Chord Rotation Capacity for element: column CC1 of floor 1

At local axis: 2

Integration Section: (b)

Section Type: rccs

Constant Properties

Knowledge Factor, $\phi = 1.00$
 Chord Rotation is generally considered as Deformation-Controlled Action according to Table C7-1, ASCE41-17.
 Mean strengths are used for Deformation-Controlled Actions according to 7.5.1.3, ASCE 41-17
 Consequently:
 New material of Primary Member: Concrete Strength, $f_c = f_{cm} = 28.00$
 New material of Primary Member: Steel Strength, $f_s = f_{sm} = 420.00$
 Concrete Elasticity, $E_c = 24870.062$
 Steel Elasticity, $E_s = 200000.00$
 Diameter, $D = 400.00$
 Cover Thickness, $c = 25.00$
 Element Length, $L = 3000.00$
 Primary Member
 Smooth Bars
 Ductile Steel
 With Detailing for Earthquake Resistance (including stirrups closed at 135°)
 Longitudinal Bars With Ends Lapped Starting at the End Sections
 Adequate Lap Length ($l_b/l_d \geq 1$)
 No FRP Wrapping

Stepwise Properties

Bending Moment, $M = -8.9196430E-012$
 Shear Force, $V_2 = 2896.171$
 Shear Force, $V_3 = -5.1847167E-015$
 Axial Force, $F = -4770.074$
 Longitudinal Reinforcement Area Distribution (in 2 divisions)
 -Tension: $A_{st} = 0.00$
 -Compression: $A_{sc} = 3053.628$
 Longitudinal Reinforcement Area Distribution (in 3 divisions)
 -Tension: $A_{st,ten} = 1017.876$
 -Compression: $A_{st,com} = 1017.876$
 -Middle: $A_{st,mid} = 1017.876$
 Mean Diameter of Tension Reinforcement, $D_bL = 18.00$

New component: From table 7-7, ASCE 41_17: Final chord rotation Capacity $u_R = 1.0^*$ $u = 0.07134961$
 $u = y + p = 0.07134961$

- Calculation of y -

$y = (M_y * L_s / 3) / E_{eff} = 0.01147479$ ((4.29), Biskinis Phd))
 $M_y = 2.1517E+008$
 $L_s = M/V$ (with $L_s > 0.1 * L$ and $L_s < 2 * L$) = 1500.00
 From table 10.5, ASCE 41_17: $E_{eff} = factor * E_c * I_g = 9.3758E+012$
 $factor = 0.30$
 $A_g = 125663.706$
 $f_c' = 28.00$
 $N = 4770.074$
 $E_c * I_g = 3.1253E+013$

Calculation of Yielding Moment M_y

Calculation of y and M_y according to (7) - (8) in Biskinis and Fardis

$M_y = \min(M_{y_ten}, M_{y_com}) = 2.1517E+008$
 y ((10a) or (10b)) = $1.1483167E-005$
 M_{y_ten} (8a) = $2.1517E+008$
 y_{ten} (7a) = 72.79826
 error of function (7a) = 0.00155288
 M_{y_com} (8b) = $4.0201E+008$
 y_{com} (7b) = 69.01017

error of function (7b) = -0.00040881

with $\epsilon_y = 0.0021$

$\epsilon_{co} = 0.002$

$\alpha_{pl} = 0.35$ ((9a) in Biskinis and Fardis for no FRP Wrap)

$d_1 = 44.00$

$R = 200.00$

$v = 0.00135568$

$N = 4770.074$

$A_c = 125663.706$

$= 0.3645$

with $f_c = 28.00$

Calculation of ratio l_b/d

Adequate Lap Length: $l_b/d \geq 1$

- Calculation of ρ -

From table 10-9: $\rho = 0.05987483$

with:

- Columns controlled by inadequate development or splicing along the clear height because $l_b/d < 1$

shear control ratio $V_y E / V_{co} I_{OE} = 0.51034251$

$d = 0.00$

$s = 0.00$

$t = 2 \cdot A_v / (d_c \cdot s) + 4 \cdot t_f / D \cdot (f_{fe} / f_s) = 0.00$

$A_v = 78.53982$, is the area of the circular stirrup

$d_c = D - 2 \cdot \text{cover} - \text{Hoop Diameter} = 340.00$

The term $2 \cdot t_f / b_w \cdot (f_{fe} / f_s)$ is implemented to account for FRP contribution

where $f = 2 \cdot t_f / b_w$ is FRP ratio (EC8 - 3, A.4.4.3(6)) and f_{fe} / f_s normalises f to steel strength

All these variables have already been given in Shear control ratio calculation.

$N_{UD} = 4770.074$

$A_g = 125663.706$

$f_{cE} = 28.00$

$f_{yE} = f_{yI} = 420.00$

$\rho_l = \text{Area_Tot_Long_Rein} / (A_g) = 0.0243$

$f_{cE} = 28.00$

End Of Calculation of Chord Rotation Capacity for element: column CC1 of floor 1

At local axis: 2

Integration Section: (b)

Calculation No. 15

column C1, Floor 1

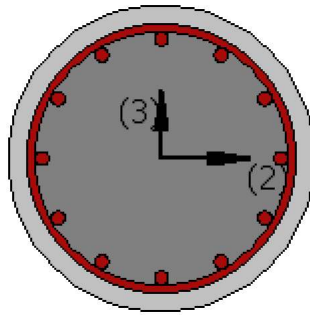
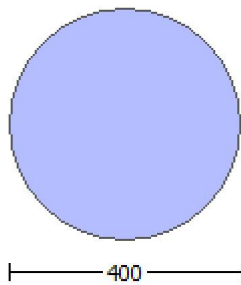
Limit State: Collapse Prevention (data interpolation between analysis steps 1 and 2)

Analysis: Uniform +X

Check: Shear capacity V_{Rd}

Edge: End

Local Axis: (3)



Start Of Calculation of Shear Capacity for element: column CC1 of floor 1

At local axis: 3

Integration Section: (b)

Section Type: rccs

Constant Properties

Knowledge Factor, $\gamma = 1.00$

Member Shear Force is generally considered as Force-Controlled Action according to Table C7-1, ASCE41-17.

Lower-bound strengths are used for Force-Controlled Actions according to 7.5.1.3, ASCE 41-17

Consequently:

New material of Primary Member: Concrete Strength, $f_c = f_{c_lower_bound} = 20.00$

New material of Primary Member: Steel Strength, $f_s = f_{s_lower_bound} = 400.00$

Concrete Elasticity, $E_c = 24870.062$

Steel Elasticity, $E_s = 200000.00$

#####

Note: Especially for the calculation of γ for displacement ductility demand, the expected (mean value) strengths are used (7.5.1.3, ASCE41-17) because bending is considered as Deformation-Controlled Action (Table C7-1, ASCE41-17).

New material: Concrete Strength, $f_c = f_{cm} = 28.00$

New material: Steel Strength, $f_s = f_{sm} = 420.00$

#####

Diameter, $D = 400.00$

Cover Thickness, $c = 25.00$

Element Length, $L = 3000.00$

Primary Member

Smooth Bars

Ductile Steel

With Detailing for Earthquake Resistance (including stirrups closed at 135°)

Longitudinal Bars With Ends Lapped Starting at the End Sections

Adequate Lap Length ($l_o/l_{ou,min} = l_b/l_d \geq 1$)

No FRP Wrapping

Stepwise Properties

EDGE -A-

Bending Moment, $M_a = -6.6801918E-012$

Shear Force, $V_a = 5.1847167E-015$

EDGE -B-

Bending Moment, $M_b = -8.9196430E-012$

Shear Force, $V_b = -5.1847167E-015$

BOTH EDGES

Axial Force, $F = -4770.074$

Longitudinal Reinforcement Area Distribution (in 2 divisions)

-Tension: $As_t = 0.00$

-Compression: $As_c = 3053.628$

Longitudinal Reinforcement Area Distribution (in 3 divisions)

-Tension: $As_{t,ten} = 1017.876$

-Compression: $As_{l,com} = 1017.876$

-Middle: $As_{l,mid} = 1017.876$

Mean Diameter of Tension Reinforcement, $Db_{L,ten} = 18.00$

New component: From table 7-7, ASCE 41_17: Final Shear Capacity $V_R = 1.0 \cdot V_n = 270862.328$
 V_n ((10.3), ASCE 41-17) = $k_n \cdot V_{Col0} = 270862.328$
 $V_{Col} = 270862.328$
 $k_n = 1.00$
 $displacement_ductility_demand = 0.00$

NOTE: In expression (10-3) ' $V_s = A_v \cdot f_y \cdot d / s$ ' is replaced by ' $V_s + f \cdot V_f$ '
 where V_f is the contribution of FRPs ((11.3), ACI 440).

$= 1$ (normal-weight concrete)
 $f'_c = 20.00$, but $f'_c^{0.5} \leq 8.3$ MPa ((22.5.3.1, ACI 318-14))
 $M/Vd = 2.00$
 $\mu_u = 8.9196430E-012$
 $\nu_u = 5.1847167E-015$
 $d = 0.8 \cdot D = 320.00$
 $N_u = 4770.074$
 $A_g = 125663.706$
 From ((11.5.4.8), ACI 318-14: $V_s = 157913.67$
 $A_v = \sqrt{2} \cdot A_{stirrup} = 123370.055$
 $f_y = 400.00$
 $s = 100.00$
 V_s is multiplied by $Col = 0.00$
 $s/d = 0.3125$
 V_f ((11-3)-(11.4), ACI 440) = 0.00
 From ((11-11), ACI 440: $V_s + V_f \leq 238930.50$
 $b_w \cdot d = \sqrt{2} \cdot d^2 / 4 = 80424.772$

$displacement_ductility_demand$ is calculated as ϕ / y

- Calculation of ϕ / y for END B -
 for rotation axis 2 and integ. section (b)

From analysis, chord rotation $\phi = 2.5100185E-022$
 $y = (M_y \cdot L_s / 3) / E_{eff} = 0.01147479$ ((4.29), Biskinis Phd))
 $M_y = 2.1517E+008$
 $L_s = M/V$ (with $L_s > 0.1 \cdot L$ and $L_s < 2 \cdot L$) = 1500.00
 From table 10.5, ASCE 41_17: $E_{eff} = factor \cdot E_c \cdot I_g = 9.3758E+012$
 $factor = 0.30$
 $A_g = 125663.706$
 $f'_c = 28.00$
 $N = 4770.074$
 $E_c \cdot I_g = 3.1253E+013$

Calculation of Yielding Moment M_y

Calculation of ϕ and M_y according to (7) - (8) in Biskinis and Fardis

$M_y = \min(M_{y_ten}, M_{y_com}) = 2.1517E+008$
 ϕ ((10a) or (10b)) = $1.1483167E-005$
 M_{y_ten} (8a) = $2.1517E+008$
 ϕ_{ten} (7a) = 72.79826
 error of function (7a) = 0.00155288
 M_{y_com} (8b) = $4.0201E+008$
 ϕ_{com} (7b) = 69.01017
 error of function (7b) = -0.00040881
 with $e_y = 0.0021$
 $e_{co} = 0.002$
 $\alpha_{pl} = 0.35$ ((9a) in Biskinis and Fardis for no FRP Wrap)
 $d_1 = 44.00$

R = 200.00
v = 0.00135568
N = 4770.074
Ac = 125663.706
= 0.3645
with fc = 28.00

Calculation of ratio lb/d

Adequate Lap Length: lb/d >= 1

End Of Calculation of Shear Capacity for element: column CC1 of floor 1
At local axis: 3
Integration Section: (b)