

Detailed Member Calculations

Units: N&mm

Regulation: ASCE 41-17

Calculation No. 1

column C1, Floor 1

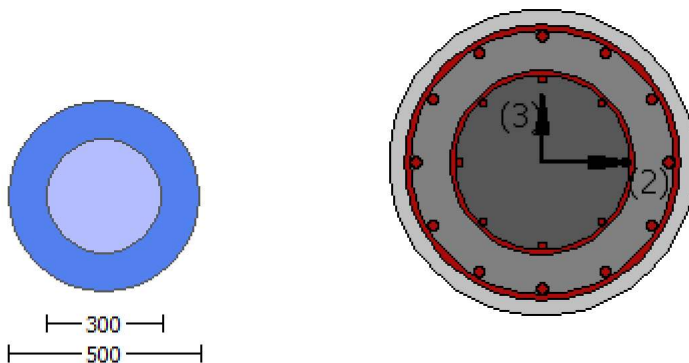
Limit State: Operational Level (data interpolation between analysis steps 1 and 2)

Analysis: Uniform +X

Check: Shear capacity VR_d

Edge: Start

Local Axis: (2)



Start Of Calculation of Shear Capacity for element: column JCC1 of floor 1

At local axis: 2

Integration Section: (a)

Section Type: rcjcs

Constant Properties

Knowledge Factor, $\gamma = 0.80$

Member Shear Force is generally considered as Force-Controlled Action according to Table C7-1, ASCE 41-17.

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

Consequently:

Jacket

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

New material of Secondary Member: Steel Strength, $f_s = f_{s_lower_bound} = 500.00$

Concrete Elasticity, $E_c = 25742.96$
Steel Elasticity, $E_s = 200000.00$
Existing Column
Existing material of Secondary Member: Concrete Strength, $f_c = f_{c_lower_bound} = 16.00$
Existing material of Secondary Member: Steel Strength, $f_s = f_{s_lower_bound} = 420.00$
Concrete Elasticity, $E_c = 23025.204$
Steel Elasticity, $E_s = 200000.00$

Note: Especially for the calculation of μ_y for displacement ductility demand,
the expected (mean value) strengths are used (7.5.1.3, ASCE 41-17) because bending is considered as
Deformation-Controlled Action (Table C7-1, ASCE 41-17).
Jacket
New material: Concrete Strength, $f_c = f_{cm} = 30.00$
New material: Steel Strength, $f_s = f_{sm} = 625.00$
Existing Column
Existing material: Concrete Strength, $f_c = f_{cm} = 24.00$
Existing material: Steel Strength, $f_s = f_{sm} = 525.00$

External Diameter, $D = 500.00$
Internal Diameter, $D = 300.00$
Cover Thickness, $c = 25.00$
Element Length, $L = 3000.00$
Secondary Member
Smooth Bars
Ductile Steel
Without Detailing for Earthquake Resistance (including stirrups not 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, $M_a = -1.7325E+007$
Shear Force, $V_a = -5773.639$
EDGE -B-
Bending Moment, $M_b = -0.02600098$
Shear Force, $V_b = 5773.639$
BOTH EDGES
Axial Force, $F = -7386.905$
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: $As_{l,ten} = 1017.876$
-Compression: $As_{l,com} = 1017.876$
-Middle: $As_{l,mid} = 1017.876$
Mean Diameter of Tension Reinforcement, $Db_{L,ten} = 18.00$

Existing component: From table 7-7, ASCE 41_17: Final Shear Capacity $V_R = V_n = 265412.603$
 V_n ((10.3), ASCE 41-17) = $k_n l V_{CoIO} = 331765.754$
 $V_{CoI} = 331765.754$
 $k_n l = 1.00$
displacement_ductility_demand = 0.01172462

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).

$\mu = 1$ (normal-weight concrete)
Mean concrete strength: $f'_c = (f'_c_jacket * Area_jacket + f'_c_core * Area_core) / Area_section = 18.56$, but $f'_c^{0.5} \leq 8.3$
MPa (22.5.3.1, ACI 318-14)
 $M/Vd = 4.00$
 $\mu_u = 1.7325E+007$

$V_u = 5773.639$
 $d = 0.8 \cdot D = 400.00$
 $N_u = 7386.905$
 $A_g = 196349.541$
 From (11.5.4.8), ACI 318-14: $V_s = V_{s1} + V_{s2} = 246740.11$
 $V_{s1} = 246740.11$ is calculated for jacket, with:
 $A_v = \frac{1}{2} \cdot A_{stirrup} = 123370.055$
 $f_y = 500.00$
 $s = 100.00$
 V_{s1} is multiplied by $Col1 = 1.00$
 $s/d = 0.25$
 $V_{s2} = 0.00$ is calculated for core, with:
 $A_v = \frac{1}{2} \cdot A_{stirrup} = 78956.835$
 $f_y = 420.00$
 $s = 250.00$
 V_{s2} is multiplied by $Col2 = 0.00$
 $s/d = 1.04167$
 $V_f ((11-3)-(11.4), ACI 440) = 0.00$
 From (11-11), ACI 440: $V_s + V_f \leq 359638.026$
 $b_w \cdot d = \frac{1}{4} \cdot d \cdot d = 125663.706$

displacement ductility demand is calculated as δ / y

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

From analysis, chord rotation $\theta = 0.00020385$
 $y = (M_y \cdot L_s / 3) / E_{eff} = 0.01738647$ ((4.29), Biskinis Phd))
 $M_y = 4.0621E+008$
 $L_s = M/V$ (with $L_s > 0.1 \cdot L$ and $L_s < 2 \cdot L$) = 3000.778
 From table 10.5, ASCE 41_17: $E_{eff} = factor \cdot E_c \cdot I_g = 2.3369E+013$
 $factor = 0.30$
 $A_g = 196349.541$
 Mean concrete strength: $f_c' = (f_c'_{jacket} \cdot Area_{jacket} + f_c'_{core} \cdot Area_{core}) / Area_{section} = 27.84$
 $N = 7386.905$
 $E_c \cdot I_g = E_{c,jacket} \cdot I_{g,jacket} + E_{c,core} \cdot I_{g,core} = 7.7898E+013$

Calculation of Yielding Moment M_y

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

$M_y = \min(M_{y,ten}, M_{y,com}) = 4.0621E+008$
 $y ((10a) \text{ or } (10b)) = 1.2225563E-005$
 $M_{y,ten} (8a) = 4.0621E+008$
 $\delta_{ten} (7a) = 67.28293$
 error of function (7a) = 0.00093796
 $M_{y,com} (8b) = 7.1091E+008$
 $\delta_{com} (7b) = 65.27568$
 error of function (7b) = -0.00596424
 with $e_y = 0.003125$
 $e_{co} = 0.002$
 $apl = 0.35$ ((9a) in Biskinis and Fardis for no FRP Wrap)
 $d1 = 44.00$
 $R = 250.00$
 $v = 0.00125404$
 $N = 7386.905$
 $A_c = 196349.541$
 $= 0.324$
 with $f_c = 30.00$

Calculation of ratio I_b/I_d

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

End Of Calculation of Shear Capacity for element: column JCC1 of floor 1

At local axis: 2

Integration Section: (a)

Calculation No. 2

column C1, Floor 1

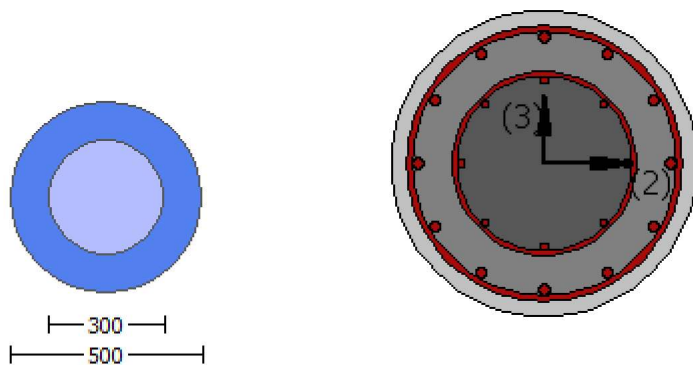
Limit State: Operational Level (data interpolation between analysis steps 1 and 2)

Analysis: Uniform +X

Check: Chord rotation capacity (ϕ)

Edge: Start

Local Axis: (2)



Start Of Calculation of Shear Capacity ratio for element: column JCC1 of floor 1

At Shear local axis: 3

(Bending local axis: 2)

Section Type: rcjcs

Constant Properties

Knowledge Factor, $\gamma = 0.80$

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

Consequently:

Jacket

New material of Secondary Member: Concrete Strength, $f_c = f_{cm} = 30.00$

New material of Secondary Member: Steel Strength, $f_s = f_{sm} = 625.00$

Concrete Elasticity, $E_c = 25742.96$

Steel Elasticity, $E_s = 200000.00$

Existing Column

Existing material of Secondary Member: Concrete Strength, $f_c = f_{cm} = 24.00$

Existing material of Secondary Member: Steel Strength, $f_s = f_{sm} = 525.00$

Concrete Elasticity, $E_c = 23025.204$

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

Jacket

New material: Steel Strength, $f_s = 1.25 \cdot f_{sm} = 781.25$

Existing Column

Existing material: Steel Strength, $f_s = 1.25 \cdot f_{sm} = 656.25$

#####

External Diameter, $D = 500.00$

Internal Diameter, $D = 300.00$

Cover Thickness, $c = 25.00$

Mean Confinement Factor overall section = 1.00

Element Length, $L = 3000.00$

Secondary Member

Smooth Bars

Ductile Steel

Without Detailing for Earthquake Resistance (including stirrups not closed at 135°)

Longitudinal Bars With Ends Lapped Starting at the End Sections

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

No FRP Wrapping

Stepwise Properties

At local axis: 3

EDGE -A-

Shear Force, $V_a = 3.5877337E-031$

EDGE -B-

Shear Force, $V_b = -3.5877337E-031$

BOTH EDGES

Axial Force, $F = -7389.214$

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, $V_e/V_r = 0.5552125$

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

Calculation of Shear Demand from fig. R18.6.5, ACI 318-14 $V_e = (M_{pr1} + M_{pr2})/l_n = 286692.647$

with

$M_{pr1} = \text{Max}(\mu_{1+}, \mu_{1-}) = 4.3004E+008$

$\mu_{1+} = 4.3004E+008$, 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

$\mu_{1-} = 4.3004E+008$, 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

$M_{pr2} = \text{Max}(\mu_{2+}, \mu_{2-}) = 4.3004E+008$

$\mu_{2+} = 4.3004E+008$, 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

$\mu_{2-} = 4.3004E+008$, 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

$\mu_u = 4.3004E+008$

$\lambda = 1.0821$

$\lambda' = 0.95748645$

error of function (3.68), Biskinis Phd = 71900.796

From 5A.2, TBDY: $f_{cc} = f_c \cdot c = 30.00$

conf. factor $c = 1.00$
 $f_c = 30.00$
 From 10.3.5, ASCE 41-17, Final value of f_y : $f_y \cdot \text{Min}(1, 1.25 \cdot (l_b/d)^{2/3}) = 781.25$
 $l_b/d = 1.00$
 $d_1 = 44.00$
 $R = 250.00$
 $v = 0.00125383$
 $N = 7389.214$
 $A_c = 196349.541$
 $= \cdot \text{Min}(1, 1.25 \cdot (l_b/d)^{2/3}) = 0.324$

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
 $\mu = 4.3004\text{E}+008$

$= 1.0821$
 $' = 0.95748645$
 error of function (3.68), Biskinis Phd = 71900.796
 From 5A.2, TBDY: $f_{cc} = f_c \cdot c = 30.00$
 conf. factor $c = 1.00$
 $f_c = 30.00$
 From 10.3.5, ASCE 41-17, Final value of f_y : $f_y \cdot \text{Min}(1, 1.25 \cdot (l_b/d)^{2/3}) = 781.25$
 $l_b/d = 1.00$
 $d_1 = 44.00$
 $R = 250.00$
 $v = 0.00125383$
 $N = 7389.214$
 $A_c = 196349.541$
 $= \cdot \text{Min}(1, 1.25 \cdot (l_b/d)^{2/3}) = 0.324$

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
 $\mu = 4.3004\text{E}+008$

$= 1.0821$
 $' = 0.95748645$
 error of function (3.68), Biskinis Phd = 71900.796
 From 5A.2, TBDY: $f_{cc} = f_c \cdot c = 30.00$
 conf. factor $c = 1.00$
 $f_c = 30.00$
 From 10.3.5, ASCE 41-17, Final value of f_y : $f_y \cdot \text{Min}(1, 1.25 \cdot (l_b/d)^{2/3}) = 781.25$
 $l_b/d = 1.00$
 $d_1 = 44.00$
 $R = 250.00$
 $v = 0.00125383$
 $N = 7389.214$
 $A_c = 196349.541$

$$= \text{Min}(1, 1.25 \cdot (l_b/d)^{2/3}) = 0.324$$

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
 $\mu = 4.3004 \times 10^8$

$$= 1.0821$$

$$\lambda = 0.95748645$$

error of function (3.68), Biskinis Phd = 71900.796

From 5A.2, TBDY: $f_{cc} = f_c \cdot c = 30.00$

conf. factor $c = 1.00$

$f_c = 30.00$

From 10.3.5, ASCE 41-17, Final value of f_y : $f_y \cdot \text{Min}(1, 1.25 \cdot (l_b/d)^{2/3}) = 781.25$

$$l_b/d = 1.00$$

$$d_1 = 44.00$$

$$R = 250.00$$

$$v = 0.00125383$$

$$N = 7389.214$$

$$A_c = 196349.541$$

$$= \text{Min}(1, 1.25 \cdot (l_b/d)^{2/3}) = 0.324$$

Calculation of ratio l_b/d

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

Calculation of Shear Strength $V_r = \text{Min}(V_{r1}, V_{r2}) = 516365.617$

Calculation of Shear Strength at edge 1, $V_{r1} = 516365.617$

$V_{r1} = V_{co1}$ ((10.3), ASCE 41-17) = $k_n \cdot V_{co1}$

$$V_{co1} = 516365.617$$

$k_n = 1$ (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)

Mean concrete strength: $f_c' = (f_c'_{\text{jacket}} \cdot \text{Area}_{\text{jacket}} + f_c'_{\text{core}} \cdot \text{Area}_{\text{core}}) / \text{Area}_{\text{section}} = 27.84$, but $f_c'^{0.5} \leq 8.3$
 MPa ((22.5.3.1, ACI 318-14)

$$M/Vd = 2.00$$

$$\mu_u = 1.6653780 \times 10^{-11}$$

$$V_u = 3.5877337 \times 10^{-31}$$

$$d = 0.8 \cdot D = 400.00$$

$$N_u = 7389.214$$

$$A_g = 196349.541$$

From ((11.5.4.8), ACI 318-14: $V_s = V_{s1} + V_{s2} = 308425.138$

$V_{s1} = 308425.138$ is calculated for jacket, with:

$$A_v = \frac{1}{2} \cdot A_{\text{stirrup}} = 123370.055$$

$$f_y = 625.00$$

$$s = 100.00$$

V_{s1} is multiplied by $\text{Col1} = 1.00$

$$s/d = 0.25$$

$V_{s2} = 0.00$ is calculated for core, with:

$$A_v = \frac{1}{2} \cdot A_{\text{stirrup}} = 78956.835$$

$f_y = 525.00$

$s = 250.00$

Vs2 is multiplied by Col2 = 0.00

$s/d = 1.04167$

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

From (11-11), ACI 440: $V_s + V_f \leq 440464.828$

$bw*d = *d*d/4 = 125663.706$

Calculation of Shear Strength at edge 2, $V_{r2} = 516365.617$

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

$V_{Col0} = 516365.617$

$knl = 1$ (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)

Mean concrete strength: $f'_c = (f'_c_{jacket} * Area_{jacket} + f'_c_{core} * Area_{core}) / Area_{section} = 27.84$, but $f'_c^{0.5} \leq 8.3$ MPa (22.5.3.1, ACI 318-14)

$M/Vd = 2.00$

$\mu_u = 1.6653780E-011$

$\mu_v = 3.5877337E-031$

$d = 0.8 * D = 400.00$

$N_u = 7389.214$

$A_g = 196349.541$

From (11.5.4.8), ACI 318-14: $V_s = V_{s1} + V_{s2} = 308425.138$

$V_{s1} = 308425.138$ is calculated for jacket, with:

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

$f_y = 625.00$

$s = 100.00$

V_{s1} is multiplied by Col1 = 1.00

$s/d = 0.25$

$V_{s2} = 0.00$ is calculated for core, with:

$A_v = /2 * A_{stirrup} = 78956.835$

$f_y = 525.00$

$s = 250.00$

Vs2 is multiplied by Col2 = 0.00

$s/d = 1.04167$

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

From (11-11), ACI 440: $V_s + V_f \leq 440464.828$

$bw*d = *d*d/4 = 125663.706$

End Of Calculation of Shear Capacity ratio for element: column JCC1 of floor 1

At local axis: 3

Start Of Calculation of Shear Capacity ratio for element: column JCC1 of floor 1

At Shear local axis: 2

(Bending local axis: 3)

Section Type: rcjcs

Constant Properties

Knowledge Factor, = 0.80

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

Consequently:

Jacket

New material of Secondary Member: Concrete Strength, $f_c = f_{cm} = 30.00$

New material of Secondary Member: Steel Strength, $f_s = f_{sm} = 625.00$

Concrete Elasticity, $E_c = 25742.96$

Steel Elasticity, $E_s = 200000.00$

Existing Column

Existing material of Secondary Member: Concrete Strength, $f_c = f_{cm} = 24.00$

Existing material of Secondary Member: Steel Strength, $f_s = f_{sm} = 525.00$
Concrete Elasticity, $E_c = 23025.204$
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
Jacket
New material: Steel Strength, $f_s = 1.25 \cdot f_{sm} = 781.25$
Existing Column
Existing material: Steel Strength, $f_s = 1.25 \cdot f_{sm} = 656.25$

External Diameter, $D = 500.00$
Internal Diameter, $D = 300.00$
Cover Thickness, $c = 25.00$
Mean Confinement Factor overall section = 1.00
Element Length, $L = 3000.00$
Secondary Member
Smooth Bars
Ductile Steel
Without Detailing for Earthquake Resistance (including stirrups not closed at 135°)
Longitudinal Bars With Ends Lapped Starting at the End Sections
Adequate Lap Length ($l_o/l_{ou,min} > 1$)
No FRP Wrapping

Stepwise Properties

At local axis: 2
EDGE -A-
Shear Force, $V_a = -7.8886091E-031$
EDGE -B-
Shear Force, $V_b = 7.8886091E-031$
BOTH EDGES
Axial Force, $F = -7389.214$
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 , $V_e/V_r = 0.5552125$
Member Controlled by Flexure ($V_e/V_r < 1$)
Calculation of Shear Demand from fig. R18.6.5, ACI 318-14 $V_e = (M_{pr1} + M_{pr2})/l_n = 286692.647$
with
 $M_{pr1} = \text{Max}(Mu_{1+} , Mu_{1-}) = 4.3004E+008$
 $Mu_{1+} = 4.3004E+008$, 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
 $Mu_{1-} = 4.3004E+008$, 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
 $M_{pr2} = \text{Max}(Mu_{2+} , Mu_{2-}) = 4.3004E+008$
 $Mu_{2+} = 4.3004E+008$, 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
 $Mu_{2-} = 4.3004E+008$, 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 Mu_{1+}

Calculation of ultimate Moment Strength ((3.67), Biskinis Phd), Mu
 $Mu = 4.3004E+008$

= 1.0821
 ' = 0.95748645
 error of function (3.68), Biskinis Phd = 71900.796
 From 5A.2, TBDY: $f_{cc} = f_c \cdot c = 30.00$
 conf. factor $c = 1.00$
 $f_c = 30.00$
 From 10.3.5, ASCE 41-17, Final value of f_y : $f_y \cdot \text{Min}(1, 1.25 \cdot (l_b/l_d)^{2/3}) = 781.25$
 $l_b/l_d = 1.00$
 $d_1 = 44.00$
 $R = 250.00$
 $v = 0.00125383$
 $N = 7389.214$
 $Ac = 196349.541$
 $= \cdot \text{Min}(1, 1.25 \cdot (l_b/l_d)^{2/3}) = 0.324$

Calculation of ratio l_b/l_d

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

Calculation of μ_1 -

Calculation of ultimate Moment Strength ((3.67), Biskinis Phd), μ
 $\mu = 4.3004E+008$

= 1.0821
 ' = 0.95748645
 error of function (3.68), Biskinis Phd = 71900.796
 From 5A.2, TBDY: $f_{cc} = f_c \cdot c = 30.00$
 conf. factor $c = 1.00$
 $f_c = 30.00$
 From 10.3.5, ASCE 41-17, Final value of f_y : $f_y \cdot \text{Min}(1, 1.25 \cdot (l_b/l_d)^{2/3}) = 781.25$
 $l_b/l_d = 1.00$
 $d_1 = 44.00$
 $R = 250.00$
 $v = 0.00125383$
 $N = 7389.214$
 $Ac = 196349.541$
 $= \cdot \text{Min}(1, 1.25 \cdot (l_b/l_d)^{2/3}) = 0.324$

Calculation of ratio l_b/l_d

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

Calculation of μ_2 +

Calculation of ultimate Moment Strength ((3.67), Biskinis Phd), μ
 $\mu = 4.3004E+008$

= 1.0821
 ' = 0.95748645
 error of function (3.68), Biskinis Phd = 71900.796
 From 5A.2, TBDY: $f_{cc} = f_c \cdot c = 30.00$
 conf. factor $c = 1.00$
 $f_c = 30.00$
 From 10.3.5, ASCE 41-17, Final value of f_y : $f_y \cdot \text{Min}(1, 1.25 \cdot (l_b/l_d)^{2/3}) = 781.25$
 $l_b/l_d = 1.00$

$d1 = 44.00$
 $R = 250.00$
 $v = 0.00125383$
 $N = 7389.214$
 $Ac = 196349.541$
 $= *Min(1, 1.25*(lb/ld)^{2/3}) = 0.324$

Calculation of ratio lb/ld

Adequate Lap Length: $lb/ld \geq 1$

Calculation of $Mu2$ -

Calculation of ultimate Moment Strength ((3.67), Biskinis Phd), Mu
 $Mu = 4.3004E+008$

$= 1.0821$
 $' = 0.95748645$
 error of function (3.68), Biskinis Phd = 71900.796
 From 5A.2, TDY: $f_{cc} = f_c' \cdot c = 30.00$
 conf. factor $c = 1.00$
 $f_c = 30.00$
 From 10.3.5, ASCE 41-17, Final value of f_y : $f_y * Min(1, 1.25*(lb/ld)^{2/3}) = 781.25$
 $lb/ld = 1.00$
 $d1 = 44.00$
 $R = 250.00$
 $v = 0.00125383$
 $N = 7389.214$
 $Ac = 196349.541$
 $= *Min(1, 1.25*(lb/ld)^{2/3}) = 0.324$

Calculation of ratio lb/ld

Adequate Lap Length: $lb/ld \geq 1$

Calculation of Shear Strength $Vr = Min(Vr1, Vr2) = 516365.617$

Calculation of Shear Strength at edge 1, $Vr1 = 516365.617$
 $Vr1 = V_{Col} ((10.3), ASCE 41-17) = knl * V_{Col0}$
 $V_{Col0} = 516365.617$
 $knl = 1$ (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)
 Mean concrete strength: $f_c' = (f_c'_{jacket} * Area_{jacket} + f_c'_{core} * Area_{core}) / Area_{section} = 27.84$, but $f_c'^{0.5} \leq 8.3$ MPa (22.5.3.1, ACI 318-14)
 $M/Vd = 2.00$
 $Mu = 1.7578197E-011$
 $Vu = 7.8886091E-031$
 $d = 0.8 * D = 400.00$
 $Nu = 7389.214$
 $Ag = 196349.541$
 From (11.5.4.8), ACI 318-14: $V_s = V_{s1} + V_{s2} = 308425.138$
 $V_{s1} = 308425.138$ is calculated for jacket, with:
 $A_v = /2 * A_{stirrup} = 123370.055$
 $f_y = 625.00$

$s = 100.00$
 $Vs1$ is multiplied by $Col1 = 1.00$
 $s/d = 0.25$
 $Vs2 = 0.00$ is calculated for core, with:
 $Av = \sqrt{2} \cdot A_{stirrup} = 78956.835$
 $fy = 525.00$
 $s = 250.00$
 $Vs2$ is multiplied by $Col2 = 0.00$
 $s/d = 1.04167$
 $Vf ((11-3)-(11.4), ACI 440) = 0.00$
 From (11-11), ACI 440: $Vs + Vf \leq 440464.828$
 $bw \cdot d = \sqrt{d} \cdot d/4 = 125663.706$

Calculation of Shear Strength at edge 2, $Vr2 = 516365.617$
 $Vr2 = VCol ((10.3), ASCE 41-17) = knl \cdot VCol0$
 $VCol0 = 516365.617$
 $knl = 1$ (zero step-static loading)

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

$= 1$ (normal-weight concrete)
 Mean concrete strength: $fc' = (fc'_{jacket} \cdot Area_{jacket} + fc'_{core} \cdot Area_{core}) / Area_{section} = 27.84$, but $fc'^{0.5} \leq 8.3$
 MPa (22.5.3.1, ACI 318-14)
 $M/Vd = 2.00$
 $Mu = 1.7578197E-011$
 $Vu = 7.8886091E-031$
 $d = 0.8 \cdot D = 400.00$
 $Nu = 7389.214$
 $Ag = 196349.541$
 From (11.5.4.8), ACI 318-14: $Vs = Vs1 + Vs2 = 308425.138$
 $Vs1 = 308425.138$ is calculated for jacket, with:
 $Av = \sqrt{2} \cdot A_{stirrup} = 123370.055$
 $fy = 625.00$
 $s = 100.00$
 $Vs1$ is multiplied by $Col1 = 1.00$
 $s/d = 0.25$
 $Vs2 = 0.00$ is calculated for core, with:
 $Av = \sqrt{2} \cdot A_{stirrup} = 78956.835$
 $fy = 525.00$
 $s = 250.00$
 $Vs2$ is multiplied by $Col2 = 0.00$
 $s/d = 1.04167$
 $Vf ((11-3)-(11.4), ACI 440) = 0.00$
 From (11-11), ACI 440: $Vs + Vf \leq 440464.828$
 $bw \cdot d = \sqrt{d} \cdot d/4 = 125663.706$

End Of Calculation of Shear Capacity ratio for element: column JCC1 of floor 1
 At local axis: 2

Start Of Calculation of Chord Rotation Capacity for element: column JCC1 of floor 1
 At local axis: 2
 Integration Section: (a)
 Section Type: rcjcs

Constant Properties

Knowledge Factor, $= 0.80$
 Chord Rotation is generally considered as Deformation-Controlled Action according to Table C7-1, ASCE 41-17.
 Mean strengths are used for Deformation-Controlled Actions according to 7.5.1.3, ASCE 41-17
 Consequently:
 Jacket

New material of Secondary Member: Concrete Strength, $f_c = f_{cm} = 30.00$
 New material of Secondary Member: Steel Strength, $f_s = f_{sm} = 625.00$
 Concrete Elasticity, $E_c = 25742.96$
 Steel Elasticity, $E_s = 200000.00$
 Existing Column
 Existing material of Secondary Member: Concrete Strength, $f_c = f_{cm} = 24.00$
 Existing material of Secondary Member: Steel Strength, $f_s = f_{sm} = 525.00$
 Concrete Elasticity, $E_c = 23025.204$
 Steel Elasticity, $E_s = 200000.00$
 External Diameter, $D = 500.00$
 Internal Diameter, $D = 300.00$
 Cover Thickness, $c = 25.00$
 Element Length, $L = 3000.00$
 Secondary Member
 Smooth Bars
 Ductile Steel
 Without Detailing for Earthquake Resistance (including stirrups not 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, $M = 1.2063676E-009$
 Shear Force, $V_2 = -5773.639$
 Shear Force, $V_3 = -4.2353875E-013$
 Axial Force, $F = -7386.905$
 Longitudinal Reinforcement Area Distribution (in 2 divisions)
 -Tension: $A_{slt} = 1272.345$
 -Compression: $A_{slc} = 1781.283$
 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, $DbL = 18.00$

Existing component: From table 7-7, ASCE 41_17: Final chord rotation Capacity $u_{R} = u = 0.01095279$
 $u = y + p = 0.01369098$

- Calculation of y -

$y = (M_y * L_s / 3) / E_{eff} = 0.00869098$ ((4.29), Biskinis Phd))
 $M_y = 4.0621E+008$
 $L_s = M/V$ (with $L_s > 0.1 * L$ and $L_s < 2 * L$) = 1500.00
 From table 10.5, ASCE 41_17: $E_{eff} = factor * E_c * I_g = 2.3369E+013$
 $factor = 0.30$
 $A_g = 196349.541$
 Mean concrete strength: $f'_c = (f'_{c,jacket} * Area_{jacket} + f'_{c,core} * Area_{core}) / Area_{section} = 27.84$
 $N = 7386.905$
 $E_c * I_g = E_{c,jacket} * I_{g,jacket} + E_{c,core} * I_{g,core} = 7.7898E+013$

Calculation of Yielding Moment M_y

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

$M_y = \min(M_{y,ten}, M_{y,com}) = 4.0621E+008$
 y ((10a) or (10b)) = $1.2225563E-005$
 $M_{y,ten}$ (8a) = $4.0621E+008$
 y_{ten} (7a) = 67.28293
 error of function (7a) = 0.00093796

$M_{y_com} (8b) = 7.1091E+008$
 $_{com} (7b) = 65.27568$
error of function (7b) = -0.00596424
with $e_y = 0.003125$
 $e_{co} = 0.002$
 $a_{pl} = 0.35$ ((9a) in Biskinis and Fardis for no FRP Wrap)
 $d_1 = 44.00$
 $R = 250.00$
 $v = 0.00125404$
 $N = 7386.905$
 $A_c = 196349.541$
 $= 0.324$
with $f_c = 30.00$

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 not controlled by inadequate development or splicing along the clear height because $l_b/d \geq 1$
shear control ratio $V_y E / V_{col} E = 0.5552125$
 $d = d_{external} = 0.00$
 $s = s_{external} = 0.00$
 $t = s_1 + s_2 + 2 \cdot t_f / b_w \cdot (f_{fe} / f_s) = 0.00323428$
jacket: $s_1 = A_{v1} \cdot (D_{c1} / 2) / (s_1 \cdot A_g) = 0.0027646$
 $A_{v1} = 78.53982$, is the area of stirrup
 $D_{c1} = D_{ext} - 2 \cdot cover$ - External Hoop Diameter = 440.00, is the total Length of all stirrups parallel to loading (shear) direction
 $s_1 = 100.00$
core: $s_2 = A_{v2} \cdot (D_{c2} / 2) / (s_2 \cdot A_g) = 0.00046968$
 $A_{v2} = 50.26548$, is the area of stirrup
 $D_{c2} = D_{int} - 2 \cdot cover$ - Internal Hoop Diameter = 292.00, is the total Length of all stirrups parallel to loading (shear) direction
 $s_2 = 250.00$

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.
For the normalisation f_s of jacket is used.

$N_{UD} = 7386.905$
 $A_g = 196349.541$
 $f_{cE} = (f_{c_jacket} \cdot Area_{jacket} + f_{c_core} \cdot Area_{core}) / section_area = 27.84$
 $f_{yE} = (f_{y_ext_Long_Reinf} \cdot Area_{ext_Long_Reinf} + f_{y_int_Long_Reinf} \cdot Area_{int_Long_Reinf}) / Area_{Tot_Long_Rein} = 21219958E-314$
 $f_{yE} = (f_{y_ext_Trans_Reinf} \cdot Area_{ext_Trans_Reinf} + f_{y_int_Trans_Reinf} \cdot Area_{int_Trans_Reinf}) / Area_{Tot_Trans_Rein} = 610.4781$
 $p_l = Area_{Tot_Long_Rein} / (A_g) = 0.015552$
 $f_{cE} = 27.84$

End Of Calculation of Chord Rotation Capacity for element: column JCC1 of floor 1

At local axis: 2

Integration Section: (a)

Calculation No. 3

column C1, Floor 1

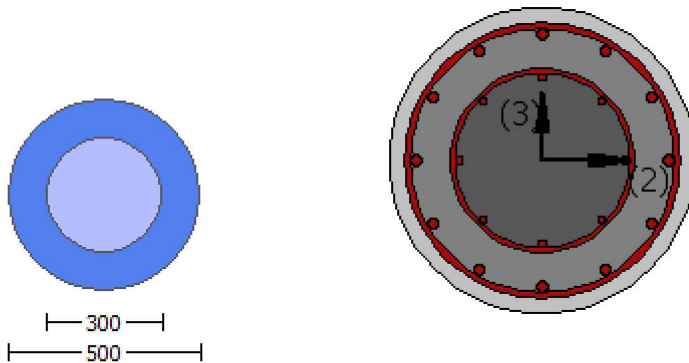
Limit State: Operational Level (data interpolation between analysis steps 1 and 2)

Analysis: Uniform +X

Check: Shear capacity VRd

Edge: Start

Local Axis: (3)



Start Of Calculation of Shear Capacity for element: column JCC1 of floor 1

At local axis: 3

Integration Section: (a)

Section Type: rcjcs

Constant Properties

Knowledge Factor, $\gamma = 0.80$

Member Shear Force is generally considered as Force-Controlled Action according to Table C7-1, ASCE 41-17.

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

Consequently:

Jacket

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

New material of Secondary Member: Steel Strength, $f_s = f_{s_lower_bound} = 500.00$

Concrete Elasticity, $E_c = 25742.96$

Steel Elasticity, $E_s = 200000.00$

Existing Column

Existing material of Secondary Member: Concrete Strength, $f_c = f_{c_lower_bound} = 16.00$

Existing material of Secondary Member: Steel Strength, $f_s = f_{s_lower_bound} = 420.00$

Concrete Elasticity, $E_c = 23025.204$

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, ASCE 41-17) because bending is considered as Deformation-Controlled Action (Table C7-1, ASCE 41-17).

Jacket

New material: Concrete Strength, $f_c = f_{cm} = 30.00$

New material: Steel Strength, $f_s = f_{sm} = 625.00$

Existing Column

Existing material: Concrete Strength, $f_c = f_{cm} = 24.00$

Existing material: Steel Strength, $f_s = f_{sm} = 525.00$

#####

External Diameter, D = 500.00
 Internal Diameter, D = 300.00
 Cover Thickness, c = 25.00
 Element Length, L = 3000.00
 Secondary Member
 Smooth Bars
 Ductile Steel
 Without Detailing for Earthquake Resistance (including stirrups not closed at 135°)
 Longitudinal Bars With Ends Lapped Starting at the End Sections
 Adequate Lap Length ($l_o/l_{o,u,min} = l_b/l_d \geq 1$)
 No FRP Wrapping

Stepwise Properties

EDGE -A-
 Bending Moment, $M_a = 1.2063676E-009$
 Shear Force, $V_a = -4.2353875E-013$
 EDGE -B-
 Bending Moment, $M_b = 6.4544616E-011$
 Shear Force, $V_b = 4.2353875E-013$
 BOTH EDGES
 Axial Force, $F = -7386.905$
 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, $Db_{L,ten} = 18.00$

Existing component: From table 7-7, ASCE 41-17: Final Shear Capacity $V_R = V_n = 333433.119$
 V_n ((10.3), ASCE 41-17) = $k_n \cdot V_{Col0} = 416791.398$
 $V_{Col} = 416791.398$
 $k_n = 1.00$
 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)
 Mean concrete strength: $f'_c = (f'_c_{jacket} \cdot Area_{jacket} + f'_c_{core} \cdot Area_{core}) / Area_{section} = 18.56$, but $f'_c^{0.5} \leq 8.3$
 MPa ((22.5.3.1, ACI 318-14))
 $M/Vd = 2.00$
 $M_u = 1.2063676E-009$
 $V_u = 4.2353875E-013$
 $d = 0.8 \cdot D = 400.00$
 $N_u = 7386.905$
 $A_g = 196349.541$
 From ((11.5.4.8), ACI 318-14): $V_s = V_{s1} + V_{s2} = 246740.11$
 $V_{s1} = 246740.11$ is calculated for jacket, with:
 $A_v = \sqrt{2} \cdot A_{stirrup} = 123370.055$
 $f_y = 500.00$
 $s = 100.00$
 V_{s1} is multiplied by $Col1 = 1.00$
 $s/d = 0.25$
 $V_{s2} = 0.00$ is calculated for core, with:
 $A_v = \sqrt{2} \cdot A_{stirrup} = 78956.835$
 $f_y = 420.00$
 $s = 250.00$
 V_{s2} is multiplied by $Col2 = 0.00$
 $s/d = 1.04167$
 V_f ((11-3)-(11.4), ACI 440) = 0.00

From (11-11), ACI 440: $V_s + V_f \leq 359638.026$
 $b_w \cdot d = \frac{V_s \cdot d}{4} = 125663.706$

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

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

From analysis, chord rotation $\theta = 1.3440380E-020$
 $y = \frac{M_y \cdot L_s / 3}{E_{eff}} = 0.00869098$ ((4.29), Biskinis Phd))
 $M_y = 4.0621E+008$
 $L_s = M/V$ (with $L_s > 0.1 \cdot L$ and $L_s < 2 \cdot L$) = 1500.00
From table 10.5, ASCE 41_17: $E_{eff} = \text{factor} \cdot E_c \cdot I_g = 2.3369E+013$
factor = 0.30
 $A_g = 196349.541$
Mean concrete strength: $f'_c = \frac{(f'_c)_{\text{jacket}} \cdot \text{Area}_{\text{jacket}} + (f'_c)_{\text{core}} \cdot \text{Area}_{\text{core}}}{\text{Area}_{\text{section}}} = 27.84$
 $N = 7386.905$
 $E_c \cdot I_g = E_{c_{\text{jacket}}} \cdot I_{g_{\text{jacket}}} + E_{c_{\text{core}}} \cdot I_{g_{\text{core}}} = 7.7898E+013$

Calculation of Yielding Moment M_y

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

$M_y = \min(M_{y_{\text{ten}}}, M_{y_{\text{com}}}) = 4.0621E+008$
 y ((10a) or (10b)) = $1.2225563E-005$
 $M_{y_{\text{ten}}} (8a) = 4.0621E+008$
 $\frac{\Delta}{y}_{\text{ten}} (7a) = 67.28293$
error of function (7a) = 0.00093796
 $M_{y_{\text{com}}} (8b) = 7.1091E+008$
 $\frac{\Delta}{y}_{\text{com}} (7b) = 65.27568$
error of function (7b) = -0.00596424
with $e_y = 0.003125$
 $e_{co} = 0.002$
 $a_{pl} = 0.35$ ((9a) in Biskinis and Fardis for no FRP Wrap)
 $d_1 = 44.00$
 $R = 250.00$
 $v = 0.00125404$
 $N = 7386.905$
 $A_c = 196349.541$
 $\rho = 0.324$
with $f_c = 30.00$

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 JCC1 of floor 1

At local axis: 3

Integration Section: (a)

Calculation No. 4

column C1, Floor 1

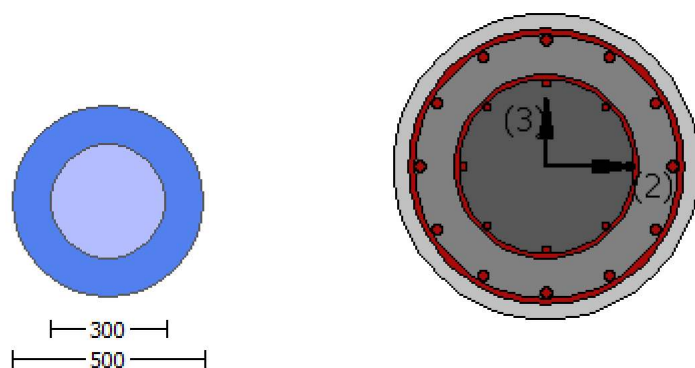
Limit State: Operational Level (data interpolation between analysis steps 1 and 2)

Analysis: Uniform +X

Check: Chord rotation capacity (ϕ)

Edge: Start

Local Axis: (3)



Start Of Calculation of Shear Capacity ratio for element: column JCC1 of floor 1

At Shear local axis: 3

(Bending local axis: 2)

Section Type: rcjcs

Constant Properties

Knowledge Factor, $\gamma = 0.80$

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

Consequently:

Jacket

New material of Secondary Member: Concrete Strength, $f_c = f_{cm} = 30.00$

New material of Secondary Member: Steel Strength, $f_s = f_{sm} = 625.00$

Concrete Elasticity, $E_c = 25742.96$

Steel Elasticity, $E_s = 200000.00$

Existing Column

Existing material of Secondary Member: Concrete Strength, $f_c = f_{cm} = 24.00$

Existing material of Secondary Member: Steel Strength, $f_s = f_{sm} = 525.00$

Concrete Elasticity, $E_c = 23025.204$

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

Jacket

New material: Steel Strength, $f_s = 1.25 \cdot f_{sm} = 781.25$

Existing Column

Existing material: Steel Strength, $f_s = 1.25 \cdot f_{sm} = 656.25$

#####

External Diameter, $D = 500.00$

Internal Diameter, $D = 300.00$

Cover Thickness, $c = 25.00$

Mean Confinement Factor overall section = 1.00

Element Length, $L = 3000.00$

Secondary Member

Smooth Bars

Ductile Steel

Without Detailing for Earthquake Resistance (including stirrups not 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, $V_a = 3.5877337E-031$

EDGE -B-

Shear Force, $V_b = -3.5877337E-031$

BOTH EDGES

Axial Force, $F = -7389.214$

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, $V_e/V_r = 0.5552125$

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

Calculation of Shear Demand from fig. R18.6.5, ACI 318-14 $V_e = (M_{pr1} + M_{pr2})/l_n = 286692.647$

with

$M_{pr1} = \text{Max}(Mu_{1+}, Mu_{1-}) = 4.3004E+008$

$Mu_{1+} = 4.3004E+008$, 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

$Mu_{1-} = 4.3004E+008$, 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

$M_{pr2} = \text{Max}(Mu_{2+}, Mu_{2-}) = 4.3004E+008$

$Mu_{2+} = 4.3004E+008$, 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

$Mu_{2-} = 4.3004E+008$, 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

$M_u = 4.3004E+008$

$\lambda = 1.0821$

$\lambda' = 0.95748645$

error of function (3.68), Biskinis Phd = 71900.796

From 5A.2, TBDY: $f_{cc} = f_c \cdot c = 30.00$

conf. factor $c = 1.00$

$f_c = 30.00$

From 10.3.5, ASCE 41-17, Final value of f_y : $f_y \cdot \text{Min}(1, 1.25 \cdot (l_b/l_d)^{2/3}) = 781.25$

$l_b/l_d = 1.00$

$d_1 = 44.00$

$R = 250.00$

$v = 0.00125383$

$N = 7389.214$

$A_c = 196349.541$

$= \lambda \cdot \text{Min}(1, 1.25 \cdot (l_b/l_d)^{2/3}) = 0.324$

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 = 4.3004E+008

= 1.0821
' = 0.95748645
error of function (3.68), Biskinis Phd = 71900.796
From 5A.2, TBDY: $f_{cc} = f_c \cdot c = 30.00$
conf. factor $c = 1.00$
 $f_c = 30.00$
From 10.3.5, ASCE 41-17, Final value of f_y : $f_y \cdot \text{Min}(1, 1.25 \cdot (l_b/d)^{2/3}) = 781.25$
 $l_b/d = 1.00$
 $d_1 = 44.00$
 $R = 250.00$
 $v = 0.00125383$
 $N = 7389.214$
 $A_c = 196349.541$
= $\cdot \text{Min}(1, 1.25 \cdot (l_b/d)^{2/3}) = 0.324$

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 = 4.3004E+008

= 1.0821
' = 0.95748645
error of function (3.68), Biskinis Phd = 71900.796
From 5A.2, TBDY: $f_{cc} = f_c \cdot c = 30.00$
conf. factor $c = 1.00$
 $f_c = 30.00$
From 10.3.5, ASCE 41-17, Final value of f_y : $f_y \cdot \text{Min}(1, 1.25 \cdot (l_b/d)^{2/3}) = 781.25$
 $l_b/d = 1.00$
 $d_1 = 44.00$
 $R = 250.00$
 $v = 0.00125383$
 $N = 7389.214$
 $A_c = 196349.541$
= $\cdot \text{Min}(1, 1.25 \cdot (l_b/d)^{2/3}) = 0.324$

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 = 4.3004E+008

= 1.0821

$\lambda = 0.95748645$
 error of function (3.68), Biskinis Phd = 71900.796
 From 5A.2, TBDY: $f_{cc} = f_c' \cdot \lambda = 30.00$
 conf. factor $\lambda = 1.00$
 $f_c = 30.00$
 From 10.3.5, ASCE 41-17, Final value of f_y : $f_y \cdot \text{Min}(1, 1.25 \cdot (l_b/l_d)^{2/3}) = 781.25$
 $l_b/l_d = 1.00$
 $d_1 = 44.00$
 $R = 250.00$
 $v = 0.00125383$
 $N = 7389.214$
 $A_c = 196349.541$
 $= \cdot \text{Min}(1, 1.25 \cdot (l_b/l_d)^{2/3}) = 0.324$

Calculation of ratio l_b/l_d

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

Calculation of Shear Strength $V_r = \text{Min}(V_{r1}, V_{r2}) = 516365.617$

Calculation of Shear Strength at edge 1, $V_{r1} = 516365.617$

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

$V_{ColO} = 516365.617$

$k_n l = 1$ (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)

Mean concrete strength: $f_c' = (f_c'_{\text{jacket}} \cdot \text{Area}_{\text{jacket}} + f_c'_{\text{core}} \cdot \text{Area}_{\text{core}}) / \text{Area}_{\text{section}} = 27.84$, but $f_c'^{0.5} \leq 8.3$ MPa (22.5.3.1, ACI 318-14)

$M/Vd = 2.00$

$\mu_u = 1.6653780E-011$

$\nu_u = 3.5877337E-031$

$d = 0.8 \cdot D = 400.00$

$N_u = 7389.214$

$A_g = 196349.541$

From (11.5.4.8), ACI 318-14: $V_s = V_{s1} + V_{s2} = 308425.138$

$V_{s1} = 308425.138$ is calculated for jacket, with:

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

$f_y = 625.00$

$s = 100.00$

V_{s1} is multiplied by $Col1 = 1.00$

$s/d = 0.25$

$V_{s2} = 0.00$ is calculated for core, with:

$A_v = \pi/2 \cdot A_{\text{stirrup}} = 78956.835$

$f_y = 525.00$

$s = 250.00$

V_{s2} is multiplied by $Col2 = 0.00$

$s/d = 1.04167$

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

From (11-11), ACI 440: $V_s + V_f \leq 440464.828$

$b_w \cdot d = \cdot d \cdot d/4 = 125663.706$

Calculation of Shear Strength at edge 2, $V_{r2} = 516365.617$

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

$V_{ColO} = 516365.617$

$k_n l = 1$ (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)
Mean concrete strength: $f'_c = (f'_{c_jacket} \cdot Area_jacket + f'_{c_core} \cdot Area_core) / Area_section = 27.84$, but $f'_c^{0.5} \leq 8.3$ MPa (22.5.3.1, ACI 318-14)
 $M/Vd = 2.00$
 $\mu_u = 1.6653780E-011$
 $\mu_v = 3.5877337E-031$
 $d = 0.8 \cdot D = 400.00$
 $N_u = 7389.214$
 $A_g = 196349.541$
From (11.5.4.8), ACI 318-14: $V_s = V_{s1} + V_{s2} = 308425.138$
 $V_{s1} = 308425.138$ is calculated for jacket, with:
 $A_v = \sqrt{2} \cdot A_{stirrup} = 123370.055$
 $f_y = 625.00$
 $s = 100.00$
 V_{s1} is multiplied by $Col1 = 1.00$
 $s/d = 0.25$
 $V_{s2} = 0.00$ is calculated for core, with:
 $A_v = \sqrt{2} \cdot A_{stirrup} = 78956.835$
 $f_y = 525.00$
 $s = 250.00$
 V_{s2} is multiplied by $Col2 = 0.00$
 $s/d = 1.04167$
 $V_f ((11-3)-(11.4), ACI 440) = 0.00$
From (11-11), ACI 440: $V_s + V_f \leq 440464.828$
 $b_w \cdot d = \sqrt{2} \cdot d \cdot d / 4 = 125663.706$

End Of Calculation of Shear Capacity ratio for element: column JCC1 of floor 1
At local axis: 3

Start Of Calculation of Shear Capacity ratio for element: column JCC1 of floor 1
At Shear local axis: 2
(Bending local axis: 3)
Section Type: rcjcs

Constant Properties

Knowledge Factor, $\phi = 0.80$
Mean strength values are used for both shear and moment calculations.
Consequently:
Jacket
New material of Secondary Member: Concrete Strength, $f_c = f_{cm} = 30.00$
New material of Secondary Member: Steel Strength, $f_s = f_{sm} = 625.00$
Concrete Elasticity, $E_c = 25742.96$
Steel Elasticity, $E_s = 200000.00$
Existing Column
Existing material of Secondary Member: Concrete Strength, $f_c = f_{cm} = 24.00$
Existing material of Secondary Member: Steel Strength, $f_s = f_{sm} = 525.00$
Concrete Elasticity, $E_c = 23025.204$
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
Jacket
New material: Steel Strength, $f_s = 1.25 \cdot f_{sm} = 781.25$
Existing Column
Existing material: Steel Strength, $f_s = 1.25 \cdot f_{sm} = 656.25$

External Diameter, $D = 500.00$
Internal Diameter, $D = 300.00$
Cover Thickness, $c = 25.00$
Mean Confinement Factor overall section = 1.00
Element Length, $L = 3000.00$
Secondary Member

Smooth Bars
 Ductile Steel
 Without Detailing for Earthquake Resistance (including stirrups not 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, $V_a = -7.8886091E-031$
 EDGE -B-
 Shear Force, $V_b = 7.8886091E-031$
 BOTH EDGES
 Axial Force, $F = -7389.214$
 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, $V_e/V_r = 0.5552125$
 Member Controlled by Flexure ($V_e/V_r < 1$)
 Calculation of Shear Demand from fig. R18.6.5, ACI 318-14 $V_e = (M_{pr1} + M_{pr2})/l_n = 286692.647$
 with
 $M_{pr1} = \max(\mu_{u1+}, \mu_{u1-}) = 4.3004E+008$
 $\mu_{u1+} = 4.3004E+008$, 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
 $\mu_{u1-} = 4.3004E+008$, 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
 $M_{pr2} = \max(\mu_{u2+}, \mu_{u2-}) = 4.3004E+008$
 $\mu_{u2+} = 4.3004E+008$, 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
 $\mu_{u2-} = 4.3004E+008$, 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
 $\mu_u = 4.3004E+008$

$\lambda = 1.0821$
 $\lambda' = 0.95748645$
 error of function (3.68), Biskinis Phd = 71900.796
 From 5A.2, TBDY: $f_{cc} = f_c \cdot c = 30.00$
 conf. factor $c = 1.00$
 $f_c = 30.00$
 From 10.3.5, ASCE 41-17, Final value of f_y : $f_y \cdot \min(1, 1.25 \cdot (l_b/l_d)^{2/3}) = 781.25$
 $l_b/l_d = 1.00$
 $d_1 = 44.00$
 $R = 250.00$
 $v = 0.00125383$
 $N = 7389.214$
 $A_c = 196349.541$
 $= \lambda' \cdot \min(1, 1.25 \cdot (l_b/l_d)^{2/3}) = 0.324$

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
 $\mu = 4.3004E+008$

$$= 1.0821$$

$$' = 0.95748645$$

error of function (3.68), Biskinis Phd = 71900.796

From 5A.2, TDY: $f_{cc} = f_c \cdot c = 30.00$

conf. factor $c = 1.00$

$$f_c = 30.00$$

From 10.3.5, ASCE 41-17, Final value of f_y : $f_y \cdot \text{Min}(1, 1.25 \cdot (l_b/d)^{2/3}) = 781.25$

$$l_b/d = 1.00$$

$$d_1 = 44.00$$

$$R = 250.00$$

$$v = 0.00125383$$

$$N = 7389.214$$

$$A_c = 196349.541$$

$$= \cdot \text{Min}(1, 1.25 \cdot (l_b/d)^{2/3}) = 0.324$$

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
 $\mu = 4.3004E+008$

$$= 1.0821$$

$$' = 0.95748645$$

error of function (3.68), Biskinis Phd = 71900.796

From 5A.2, TDY: $f_{cc} = f_c \cdot c = 30.00$

conf. factor $c = 1.00$

$$f_c = 30.00$$

From 10.3.5, ASCE 41-17, Final value of f_y : $f_y \cdot \text{Min}(1, 1.25 \cdot (l_b/d)^{2/3}) = 781.25$

$$l_b/d = 1.00$$

$$d_1 = 44.00$$

$$R = 250.00$$

$$v = 0.00125383$$

$$N = 7389.214$$

$$A_c = 196349.541$$

$$= \cdot \text{Min}(1, 1.25 \cdot (l_b/d)^{2/3}) = 0.324$$

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), Mu
Mu = 4.3004E+008

= 1.0821
' = 0.95748645
error of function (3.68), Biskinis Phd = 71900.796
From 5A.2, TBDY: fcc = fc* c = 30.00
conf. factor c = 1.00
fc = 30.00
From 10.3.5, ASCE 41-17, Final value of fy: fy*Min(1,1.25*(lb/d)^ 2/3) = 781.25
lb/d = 1.00
d1 = 44.00
R = 250.00
v = 0.00125383
N = 7389.214
Ac = 196349.541
= *Min(1,1.25*(lb/d)^ 2/3) = 0.324

Calculation of ratio lb/d

Adequate Lap Length: lb/d >= 1

Calculation of Shear Strength Vr = Min(Vr1,Vr2) = 516365.617

Calculation of Shear Strength at edge 1, Vr1 = 516365.617
Vr1 = VCol ((10.3), ASCE 41-17) = knl*VColO
VColO = 516365.617
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)
Mean concrete strength: fc' = (fc'_jacket*Area_jacket + fc'_core*Area_core)/Area_section = 27.84, but fc'^0.5 <= 8.3
MPa (22.5.3.1, ACI 318-14)
M/Vd = 2.00
Mu = 1.7578197E-011
Vu = 7.8886091E-031
d = 0.8*D = 400.00
Nu = 7389.214
Ag = 196349.541
From (11.5.4.8), ACI 318-14: Vs = Vs1 + Vs2 = 308425.138
Vs1 = 308425.138 is calculated for jacket, with:
Av = /2*A_stirrup = 123370.055
fy = 625.00
s = 100.00
Vs1 is multiplied by Col1 = 1.00
s/d = 0.25
Vs2 = 0.00 is calculated for core, with:
Av = /2*A_stirrup = 78956.835
fy = 525.00
s = 250.00
Vs2 is multiplied by Col2 = 0.00
s/d = 1.04167
Vf ((11-3)-(11.4), ACI 440) = 0.00
From (11-11), ACI 440: Vs + Vf <= 440464.828
bw*d = *d*d/4 = 125663.706

Calculation of Shear Strength at edge 2, Vr2 = 516365.617
Vr2 = VCol ((10.3), ASCE 41-17) = knl*VColO
VColO = 516365.617

knl = 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)

Mean concrete strength: $f_c' = (f_c'_{\text{jacket}} \cdot \text{Area}_{\text{jacket}} + f_c'_{\text{core}} \cdot \text{Area}_{\text{core}}) / \text{Area}_{\text{section}} = 27.84$, but $f_c'^{0.5} \leq 8.3$
MPa (22.5.3.1, ACI 318-14)

$M/Vd = 2.00$

$\mu_u = 1.7578197\text{E-}011$

$\nu_u = 7.8886091\text{E-}031$

$d = 0.8 \cdot D = 400.00$

$N_u = 7389.214$

$A_g = 196349.541$

From (11.5.4.8), ACI 318-14: $V_s = V_{s1} + V_{s2} = 308425.138$

$V_{s1} = 308425.138$ is calculated for jacket, with:

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

$f_y = 625.00$

$s = 100.00$

V_{s1} is multiplied by $\text{Col1} = 1.00$

$s/d = 0.25$

$V_{s2} = 0.00$ is calculated for core, with:

$A_v = \pi/2 \cdot A_{\text{stirrup}} = 78956.835$

$f_y = 525.00$

$s = 250.00$

V_{s2} is multiplied by $\text{Col2} = 0.00$

$s/d = 1.04167$

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

From (11-11), ACI 440: $V_s + V_f \leq 440464.828$

$b_w \cdot d = \pi \cdot d^2 / 4 = 125663.706$

End Of Calculation of Shear Capacity ratio for element: column JCC1 of floor 1
At local axis: 2

Start Of Calculation of Chord Rotation Capacity for element: column JCC1 of floor 1

At local axis: 3

Integration Section: (a)

Section Type: rcjcs

Constant Properties

Knowledge Factor, $\phi = 0.80$

Chord Rotation is generally considered as Deformation-Controlled Action according to Table C7-1, ASCE 41-17.

Mean strengths are used for Deformation-Controlled Actions according to 7.5.1.3, ASCE 41-17

Consequently:

Jacket

New material of Secondary Member: Concrete Strength, $f_c = f_{cm} = 30.00$

New material of Secondary Member: Steel Strength, $f_s = f_{sm} = 625.00$

Concrete Elasticity, $E_c = 25742.96$

Steel Elasticity, $E_s = 200000.00$

Existing Column

Existing material of Secondary Member: Concrete Strength, $f_c = f_{cm} = 24.00$

Existing material of Secondary Member: Steel Strength, $f_s = f_{sm} = 525.00$

Concrete Elasticity, $E_c = 23025.204$

Steel Elasticity, $E_s = 200000.00$

External Diameter, $D = 500.00$

Internal Diameter, $D = 300.00$

Cover Thickness, $c = 25.00$

Element Length, $L = 3000.00$

Secondary Member

Smooth Bars

Ductile Steel

Without Detailing for Earthquake Resistance (including stirrups not 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, $M = -1.7325E+007$
Shear Force, $V2 = -5773.639$
Shear Force, $V3 = -4.2353875E-013$
Axial Force, $F = -7386.905$
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, $Db_L = 18.00$

Existing component: From table 7-7, ASCE 41_17: Final chord rotation Capacity $u_R = \phi \cdot u = 0.01790918$
 $u = y + p = 0.02238647$

- Calculation of y -

$y = (M \cdot L_s / 3) / E_{eff} = 0.01738647$ ((4.29), Biskinis Phd))
 $M_y = 4.0621E+008$
 $L_s = M/V$ (with $L_s > 0.1 \cdot L$ and $L_s < 2 \cdot L$) = 3000.778
From table 10.5, ASCE 41_17: $E_{eff} = factor \cdot E_c \cdot I_g = 2.3369E+013$
 $factor = 0.30$
 $A_g = 196349.541$
Mean concrete strength: $fc' = (fc'_{jacket} \cdot Area_{jacket} + fc'_{core} \cdot Area_{core}) / Area_{section} = 27.84$
 $N = 7386.905$
 $E_c \cdot I_g = E_{c,jacket} \cdot I_{g,jacket} + E_{c,core} \cdot I_{g,core} = 7.7898E+013$

Calculation of Yielding Moment M_y

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

$M_y = \min(M_{y,ten}, M_{y,com}) = 4.0621E+008$
 y ((10a) or (10b)) = $1.2225563E-005$
 $M_{y,ten}$ (8a) = $4.0621E+008$
 y_{ten} (7a) = 67.28293
error of function (7a) = 0.00093796
 $M_{y,com}$ (8b) = $7.1091E+008$
 y_{com} (7b) = 65.27568
error of function (7b) = -0.00596424
with $e_y = 0.003125$
 $e_{co} = 0.002$
 $a_{pl} = 0.35$ ((9a) in Biskinis and Fardis for no FRP Wrap)
 $d_1 = 44.00$
 $R = 250.00$
 $v = 0.00125404$
 $N = 7386.905$
 $A_c = 196349.541$
 $= 0.324$
with $fc = 30.00$

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 not controlled by inadequate development or splicing along the clear height because $l_b/l_d \geq 1$

shear control ratio $V_y E / V_{Col} I_{OE} = 0.5552125$

$d = d_{external} = 0.00$

$s = s_{external} = 0.00$

$t = s_1 + s_2 + 2 \cdot t_f / b_w \cdot (f_{fe} / f_s) = 0.00323428$

jacket: $s_1 = A_{v1} \cdot (D_{c1} / 2) / (s_1 \cdot A_g) = 0.0027646$

$A_{v1} = 78.53982$, is the area of stirrup

$D_{c1} = D_{ext} - 2 \cdot \text{cover} - \text{External Hoop Diameter} = 440.00$, is the total Length of all stirrups parallel to loading (shear) direction

$s_1 = 100.00$

core: $s_2 = A_{v2} \cdot (D_{c2} / 2) / (s_2 \cdot A_g) = 0.00046968$

$A_{v2} = 50.26548$, is the area of stirrup

$D_{c2} = D_{int} - \text{Internal Hoop Diameter} = 292.00$, is the total Length of all stirrups parallel to loading (shear) direction

$s_2 = 250.00$

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.

For the normalisation f_s of jacket is used.

$NUD = 7386.905$

$A_g = 196349.541$

$f_{cE} = (f_{c,jacket} \cdot \text{Area}_{jacket} + f_{c,core} \cdot \text{Area}_{core}) / \text{section_area} = 27.84$

$f_{yIE} = (f_{y,ext_Long_Reinf} \cdot \text{Area}_{ext_Long_Reinf} + f_{y,int_Long_Reinf} \cdot \text{Area}_{int_Long_Reinf}) / \text{Area}_{Tot_Long_Rein} = 21219958E-314$

$f_{yIE} = (f_{y,ext_Trans_Reinf} \cdot \text{Area}_{ext_Trans_Reinf} + f_{y,int_Trans_Reinf} \cdot \text{Area}_{int_Trans_Reinf}) / \text{Area}_{Tot_Trans_Rein} = 610.4781$

$p_l = \text{Area}_{Tot_Long_Rein} / (A_g) = 0.015552$

$f_{cE} = 27.84$

End Of Calculation of Chord Rotation Capacity for element: column JCC1 of floor 1

At local axis: 3

Integration Section: (a)

Calculation No. 5

column C1, Floor 1

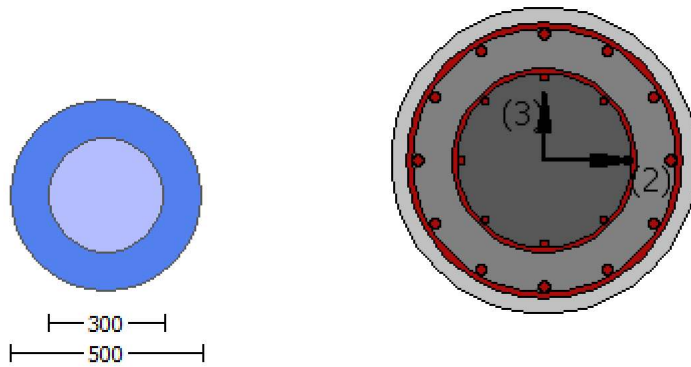
Limit State: Operational Level (data interpolation between analysis steps 1 and 2)

Analysis: Uniform +X

Check: Shear capacity V_{Rd}

Edge: End

Local Axis: (2)



Start Of Calculation of Shear Capacity for element: column JCC1 of floor 1

At local axis: 2

Integration Section: (b)

Section Type: rcjcs

Constant Properties

Knowledge Factor, $\gamma = 0.80$

Member Shear Force is generally considered as Force-Controlled Action according to Table C7-1, ASCE 41-17.

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

Consequently:

Jacket

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

New material of Secondary Member: Steel Strength, $f_s = f_{s_lower_bound} = 500.00$

Concrete Elasticity, $E_c = 25742.96$

Steel Elasticity, $E_s = 200000.00$

Existing Column

Existing material of Secondary Member: Concrete Strength, $f_c = f_{c_lower_bound} = 16.00$

Existing material of Secondary Member: Steel Strength, $f_s = f_{s_lower_bound} = 420.00$

Concrete Elasticity, $E_c = 23025.204$

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, ASCE 41-17) because bending is considered as Deformation-Controlled Action (Table C7-1, ASCE 41-17).

Jacket

New material: Concrete Strength, $f_c = f_{cm} = 30.00$

New material: Steel Strength, $f_s = f_{sm} = 625.00$

Existing Column

Existing material: Concrete Strength, $f_c = f_{cm} = 24.00$

Existing material: Steel Strength, $f_s = f_{sm} = 525.00$

#####

External Diameter, $D = 500.00$

Internal Diameter, $D = 300.00$

Cover Thickness, $c = 25.00$

Element Length, $L = 3000.00$

Secondary Member

Smooth Bars

Ductile Steel

Without Detailing for Earthquake Resistance (including stirrups not 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, $M_a = -1.7325E+007$

Shear Force, $V_a = -5773.639$

EDGE -B-

Bending Moment, $M_b = -0.02600098$

Shear Force, $V_b = 5773.639$
 BOTH EDGES
 Axial Force, $F = -7386.905$
 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, $Db_{L,ten} = 18.00$

Existing component: From table 7-7, ASCE 41_17: Final Shear Capacity $V_R = \phi V_n = 333433.119$
 $V_n ((10.3), ASCE 41-17) = knl * V_{Col0} = 416791.398$
 $V_{Col} = 416791.398$
 $knl = 1.00$
 $displacement_ductility_demand = 0.06397132$

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

$= 1$ (normal-weight concrete)
 Mean concrete strength: $fc' = (fc'_{jacket} * Area_{jacket} + fc'_{core} * Area_{core}) / Area_{section} = 18.56$, but $fc'^{0.5} \leq 8.3$
 MPa (22.5.3.1, ACI 318-14)
 $M/Vd = 2.00$
 $\mu_u = 0.02600098$
 $V_u = 5773.639$
 $d = 0.8 * D = 400.00$
 $N_u = 7386.905$
 $A_g = 196349.541$
 From (11.5.4.8), ACI 318-14: $V_s = V_{s1} + V_{s2} = 246740.11$
 $V_{s1} = 246740.11$ is calculated for jacket, with:
 $A_v = \phi / 2 * A_{stirrup} = 123370.055$
 $f_y = 500.00$
 $s = 100.00$
 V_{s1} is multiplied by $Col1 = 1.00$
 $s/d = 0.25$
 $V_{s2} = 0.00$ is calculated for core, with:
 $A_v = \phi / 2 * A_{stirrup} = 78956.835$
 $f_y = 420.00$
 $s = 250.00$
 V_{s2} is multiplied by $Col2 = 0.00$
 $s/d = 1.04167$
 $V_f ((11-3)-(11.4), ACI 440) = 0.00$
 From (11-11), ACI 440: $V_s + V_f \leq 359638.026$
 $b_w * d = \phi * d * d / 4 = 125663.706$

$displacement_ductility_demand$ is calculated as ϕ / y

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

From analysis, chord rotation $\theta = 0.00011119$
 $y = (M_y * L_s / 3) / E_{eff} = 0.0017382 ((4.29), Biskinis Phd)$
 $M_y = 4.0621E+008$
 $L_s = M/V$ (with $L_s > 0.1 * L$ and $L_s < 2 * L$) = 300.00
 From table 10.5, ASCE 41_17: $E_{eff} = factor * E_c * I_g = 2.3369E+013$
 $factor = 0.30$
 $A_g = 196349.541$
 Mean concrete strength: $fc' = (fc'_{jacket} * Area_{jacket} + fc'_{core} * Area_{core}) / Area_{section} = 27.84$
 $N = 7386.905$
 $E_c * I_g = E_{c,jacket} * I_{g,jacket} + E_{c,core} * I_{g,core} = 7.7898E+013$

Calculation of Yielding Moment M_y

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

$M_y = \min(M_{y_ten}, M_{y_com}) = 4.0621E+008$

$\rho_y ((10a) \text{ or } (10b)) = 1.2225563E-005$

$M_{y_ten} (8a) = 4.0621E+008$

$\rho_{y_ten} (7a) = 67.28293$

error of function (7a) = 0.00093796

$M_{y_com} (8b) = 7.1091E+008$

$\rho_{y_com} (7b) = 65.27568$

error of function (7b) = -0.00596424

with $e_y = 0.003125$

$e_{co} = 0.002$

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

$d_1 = 44.00$

$R = 250.00$

$v = 0.00125404$

$N = 7386.905$

$A_c = 196349.541$

$= 0.324$

with $f_c = 30.00$

Calculation of ratio l_b/d

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

End Of Calculation of Shear Capacity for element: column JCC1 of floor 1

At local axis: 2

Integration Section: (b)

Calculation No. 6

column C1, Floor 1

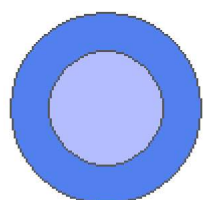
Limit State: Operational Level (data interpolation between analysis steps 1 and 2)

Analysis: Uniform +X

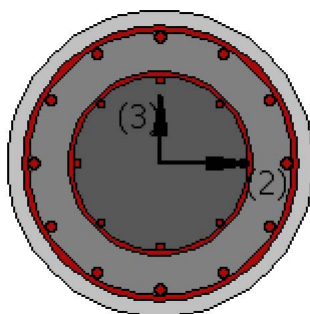
Check: Chord rotation capacity (ϕ_u)

Edge: End

Local Axis: (2)



300
500



Start Of Calculation of Shear Capacity ratio for element: column JCC1 of floor 1

At Shear local axis: 3

(Bending local axis: 2)

Section Type: rcjcs

Constant Properties

Knowledge Factor, $\gamma = 0.80$

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

Consequently:

Jacket

New material of Secondary Member: Concrete Strength, $f_c = f_{cm} = 30.00$

New material of Secondary Member: Steel Strength, $f_s = f_{sm} = 625.00$

Concrete Elasticity, $E_c = 25742.96$

Steel Elasticity, $E_s = 200000.00$

Existing Column

Existing material of Secondary Member: Concrete Strength, $f_c = f_{cm} = 24.00$

Existing material of Secondary Member: Steel Strength, $f_s = f_{sm} = 525.00$

Concrete Elasticity, $E_c = 23025.204$

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

Jacket

New material: Steel Strength, $f_s = 1.25 \cdot f_{sm} = 781.25$

Existing Column

Existing material: Steel Strength, $f_s = 1.25 \cdot f_{sm} = 656.25$

#####

External Diameter, $D = 500.00$

Internal Diameter, $D = 300.00$

Cover Thickness, $c = 25.00$

Mean Confinement Factor overall section = 1.00

Element Length, $L = 3000.00$

Secondary Member

Smooth Bars

Ductile Steel

Without Detailing for Earthquake Resistance (including stirrups not 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, $V_a = 3.5877337E-031$

EDGE -B-

Shear Force, $V_b = -3.5877337E-031$

BOTH EDGES

Axial Force, $F = -7389.214$

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: $A_{sl,t,ten} = 1017.876$
 -Compression: $A_{sl,c,com} = 1017.876$
 -Middle: $A_{sl,mid} = 1017.876$

Calculation of Shear Capacity ratio, $V_e/V_r = 0.5552125$
 Member Controlled by Flexure ($V_e/V_r < 1$)
 Calculation of Shear Demand from fig. R18.6.5, ACI 318-14 $V_e = (M_{pr1} + M_{pr2})/l_n = 286692.647$
 with
 $M_{pr1} = \max(\mu_{u1+}, \mu_{u1-}) = 4.3004E+008$
 $\mu_{u1+} = 4.3004E+008$, 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
 $\mu_{u1-} = 4.3004E+008$, 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
 $M_{pr2} = \max(\mu_{u2+}, \mu_{u2-}) = 4.3004E+008$
 $\mu_{u2+} = 4.3004E+008$, 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
 $\mu_{u2-} = 4.3004E+008$, 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
 $\mu_u = 4.3004E+008$

$\phi = 1.0821$
 $\phi' = 0.95748645$
 error of function (3.68), Biskinis Phd = 71900.796
 From 5A.2, TB DY: $f_{cc} = f_c' \cdot c = 30.00$
 conf. factor $c = 1.00$
 $f_c = 30.00$
 From 10.3.5, ASCE 41-17, Final value of f_y : $f_y \cdot \min(1, 1.25 \cdot (l_b/d)^{2/3}) = 781.25$
 $l_b/d = 1.00$
 $d_1 = 44.00$
 $R = 250.00$
 $v = 0.00125383$
 $N = 7389.214$
 $A_c = 196349.541$
 $\phi' \cdot \min(1, 1.25 \cdot (l_b/d)^{2/3}) = 0.324$

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
 $\mu_u = 4.3004E+008$

$\phi = 1.0821$
 $\phi' = 0.95748645$
 error of function (3.68), Biskinis Phd = 71900.796
 From 5A.2, TB DY: $f_{cc} = f_c' \cdot c = 30.00$
 conf. factor $c = 1.00$

$f_c = 30.00$
 From 10.3.5, ASCE 41-17, Final value of f_y : $f_y \cdot \text{Min}(1, 1.25 \cdot (l_b/d)^{2/3}) = 781.25$
 $l_b/d = 1.00$
 $d_1 = 44.00$
 $R = 250.00$
 $v = 0.00125383$
 $N = 7389.214$
 $A_c = 196349.541$
 $= \cdot \text{Min}(1, 1.25 \cdot (l_b/d)^{2/3}) = 0.324$

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
 $\mu = 4.3004E+008$

$= 1.0821$
 $' = 0.95748645$
 error of function (3.68), Biskinis Phd = 71900.796
 From 5A.2, TBDY: $f_{cc} = f_c \cdot c = 30.00$
 conf. factor $c = 1.00$
 $f_c = 30.00$
 From 10.3.5, ASCE 41-17, Final value of f_y : $f_y \cdot \text{Min}(1, 1.25 \cdot (l_b/d)^{2/3}) = 781.25$
 $l_b/d = 1.00$
 $d_1 = 44.00$
 $R = 250.00$
 $v = 0.00125383$
 $N = 7389.214$
 $A_c = 196349.541$
 $= \cdot \text{Min}(1, 1.25 \cdot (l_b/d)^{2/3}) = 0.324$

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
 $\mu = 4.3004E+008$

$= 1.0821$
 $' = 0.95748645$
 error of function (3.68), Biskinis Phd = 71900.796
 From 5A.2, TBDY: $f_{cc} = f_c \cdot c = 30.00$
 conf. factor $c = 1.00$
 $f_c = 30.00$
 From 10.3.5, ASCE 41-17, Final value of f_y : $f_y \cdot \text{Min}(1, 1.25 \cdot (l_b/d)^{2/3}) = 781.25$
 $l_b/d = 1.00$
 $d_1 = 44.00$
 $R = 250.00$
 $v = 0.00125383$
 $N = 7389.214$
 $A_c = 196349.541$
 $= \cdot \text{Min}(1, 1.25 \cdot (l_b/d)^{2/3}) = 0.324$

Calculation of ratio l_b/d

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

Calculation of Shear Strength $V_r = \text{Min}(V_{r1}, V_{r2}) = 516365.617$

Calculation of Shear Strength at edge 1, $V_{r1} = 516365.617$

$V_{r1} = V_{Col} \text{ ((10.3), ASCE 41-17)} = k_n l V_{Col0}$

$V_{Col0} = 516365.617$

$k_n l = 1$ (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)

Mean concrete strength: $f'_c = (f'_{c_jacket} \text{Area}_{jacket} + f'_{c_core} \text{Area}_{core}) / \text{Area}_{section} = 27.84$, but $f'_c^{0.5} \leq 8.3$ MPa (22.5.3.1, ACI 318-14)

$M/Vd = 2.00$

$\mu_u = 1.6653780E-011$

$\nu_u = 3.5877337E-031$

$d = 0.8 \cdot D = 400.00$

$N_u = 7389.214$

$A_g = 196349.541$

From (11.5.4.8), ACI 318-14: $V_s = V_{s1} + V_{s2} = 308425.138$

$V_{s1} = 308425.138$ is calculated for jacket, with:

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

$f_y = 625.00$

$s = 100.00$

V_{s1} is multiplied by $\text{Col1} = 1.00$

$s/d = 0.25$

$V_{s2} = 0.00$ is calculated for core, with:

$A_v = \sqrt{2} \cdot A_{stirrup} = 78956.835$

$f_y = 525.00$

$s = 250.00$

V_{s2} is multiplied by $\text{Col2} = 0.00$

$s/d = 1.04167$

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

From (11-11), ACI 440: $V_s + V_f \leq 440464.828$

$b_w d = \frac{1}{4} d^2 = 125663.706$

Calculation of Shear Strength at edge 2, $V_{r2} = 516365.617$

$V_{r2} = V_{Col} \text{ ((10.3), ASCE 41-17)} = k_n l V_{Col0}$

$V_{Col0} = 516365.617$

$k_n l = 1$ (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)

Mean concrete strength: $f'_c = (f'_{c_jacket} \text{Area}_{jacket} + f'_{c_core} \text{Area}_{core}) / \text{Area}_{section} = 27.84$, but $f'_c^{0.5} \leq 8.3$ MPa (22.5.3.1, ACI 318-14)

$M/Vd = 2.00$

$\mu_u = 1.6653780E-011$

$\nu_u = 3.5877337E-031$

$d = 0.8 \cdot D = 400.00$

$N_u = 7389.214$

$A_g = 196349.541$

From (11.5.4.8), ACI 318-14: $V_s = V_{s1} + V_{s2} = 308425.138$

$V_{s1} = 308425.138$ is calculated for jacket, with:

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

$f_y = 625.00$

$s = 100.00$
 V_{s1} is multiplied by $Col1 = 1.00$
 $s/d = 0.25$
 $V_{s2} = 0.00$ is calculated for core, with:
 $A_v = \sqrt{2} \cdot A_{stirrup} = 78956.835$
 $f_y = 525.00$
 $s = 250.00$
 V_{s2} is multiplied by $Col2 = 0.00$
 $s/d = 1.04167$
 $V_f ((11-3)-(11.4), ACI 440) = 0.00$
 From (11-11), ACI 440: $V_s + V_f \leq 440464.828$
 $b_w \cdot d = \sqrt{d} \cdot d / 4 = 125663.706$

End Of Calculation of Shear Capacity ratio for element: column JCC1 of floor 1
 At local axis: 3

Start Of Calculation of Shear Capacity ratio for element: column JCC1 of floor 1
 At Shear local axis: 2
 (Bending local axis: 3)
 Section Type: rcjcs

Constant Properties

Knowledge Factor, $\phi = 0.80$
 Mean strength values are used for both shear and moment calculations.
 Consequently:
 Jacket
 New material of Secondary Member: Concrete Strength, $f_c = f_{cm} = 30.00$
 New material of Secondary Member: Steel Strength, $f_s = f_{sm} = 625.00$
 Concrete Elasticity, $E_c = 25742.96$
 Steel Elasticity, $E_s = 200000.00$
 Existing Column
 Existing material of Secondary Member: Concrete Strength, $f_c = f_{cm} = 24.00$
 Existing material of Secondary Member: Steel Strength, $f_s = f_{sm} = 525.00$
 Concrete Elasticity, $E_c = 23025.204$
 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
 Jacket
 New material: Steel Strength, $f_s = 1.25 \cdot f_{sm} = 781.25$
 Existing Column
 Existing material: Steel Strength, $f_s = 1.25 \cdot f_{sm} = 656.25$
 #####
 External Diameter, $D = 500.00$
 Internal Diameter, $D = 300.00$
 Cover Thickness, $c = 25.00$
 Mean Confinement Factor overall section = 1.00
 Element Length, $L = 3000.00$
 Secondary Member
 Smooth Bars
 Ductile Steel
 Without Detailing for Earthquake Resistance (including stirrups not 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, $V_a = -7.8886091E-031$

EDGE -B-

Shear Force, $V_b = 7.8886091\text{E-}031$

BOTH EDGES

Axial Force, $F = -7389.214$

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: $A_{st,mid} = 1017.876$

Calculation of Shear Capacity ratio, $V_e/V_r = 0.5552125$

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

Calculation of Shear Demand from fig. R18.6.5, ACI 318-14 $V_e = (M_{pr1} + M_{pr2})/l_n = 286692.647$ with

$M_{pr1} = \text{Max}(M_{u1+}, M_{u1-}) = 4.3004\text{E}+008$

$M_{u1+} = 4.3004\text{E}+008$, 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

$M_{u1-} = 4.3004\text{E}+008$, 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

$M_{pr2} = \text{Max}(M_{u2+}, M_{u2-}) = 4.3004\text{E}+008$

$M_{u2+} = 4.3004\text{E}+008$, 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

$M_{u2-} = 4.3004\text{E}+008$, 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

$M_u = 4.3004\text{E}+008$

$\phi = 1.0821$

$\phi' = 0.95748645$

error of function (3.68), Biskinis Phd = 71900.796

From 5A.2, TBDY: $f_{cc} = f_c' \cdot c = 30.00$

conf. factor $c = 1.00$

$f_c = 30.00$

From 10.3.5, ASCE 41-17, Final value of f_y : $f_y \cdot \text{Min}(1, 1.25 \cdot (l_b/l_d)^{2/3}) = 781.25$

$l_b/l_d = 1.00$

$d_1 = 44.00$

$R = 250.00$

$v = 0.00125383$

$N = 7389.214$

$A_c = 196349.541$

$= \phi' \cdot \text{Min}(1, 1.25 \cdot (l_b/l_d)^{2/3}) = 0.324$

Calculation of ratio l_b/l_d

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

Calculation of M_{u1-}

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

$M_u = 4.3004\text{E}+008$

```

= 1.0821
' = 0.95748645
error of function (3.68), Biskinis Phd = 71900.796
From 5A.2, TBDY: fcc = fc* c = 30.00
  conf. factor c = 1.00
  fc = 30.00
From 10.3.5, ASCE 41-17, Final value of fy: fy*Min(1,1.25*(lb/d)^ 2/3) = 781.25
  lb/d = 1.00
  d1 = 44.00
  R = 250.00
  v = 0.00125383
  N = 7389.214
  Ac = 196349.541
  = *Min(1,1.25*(lb/d)^ 2/3) = 0.324

```

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 = 4.3004E+008

```

= 1.0821
' = 0.95748645
error of function (3.68), Biskinis Phd = 71900.796
From 5A.2, TBDY: fcc = fc* c = 30.00
  conf. factor c = 1.00
  fc = 30.00
From 10.3.5, ASCE 41-17, Final value of fy: fy*Min(1,1.25*(lb/d)^ 2/3) = 781.25
  lb/d = 1.00
  d1 = 44.00
  R = 250.00
  v = 0.00125383
  N = 7389.214
  Ac = 196349.541
  = *Min(1,1.25*(lb/d)^ 2/3) = 0.324

```

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 = 4.3004E+008

```

= 1.0821
' = 0.95748645
error of function (3.68), Biskinis Phd = 71900.796
From 5A.2, TBDY: fcc = fc* c = 30.00
  conf. factor c = 1.00
  fc = 30.00
From 10.3.5, ASCE 41-17, Final value of fy: fy*Min(1,1.25*(lb/d)^ 2/3) = 781.25
  lb/d = 1.00
  d1 = 44.00

```

$R = 250.00$
 $v = 0.00125383$
 $N = 7389.214$
 $Ac = 196349.541$
 $= \text{Min}(1, 1.25 \cdot (lb/d)^{2/3}) = 0.324$

Calculation of ratio lb/d

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

Calculation of Shear Strength $V_r = \text{Min}(V_{r1}, V_{r2}) = 516365.617$

Calculation of Shear Strength at edge 1, $V_{r1} = 516365.617$

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

$V_{ColO} = 516365.617$

$knl = 1$ (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)

Mean concrete strength: $f'_c = (f'_c \cdot \text{jacket} \cdot \text{Area}_{\text{jacket}} + f'_c \cdot \text{core} \cdot \text{Area}_{\text{core}}) / \text{Area}_{\text{section}} = 27.84$, but $f'_c^{0.5} \leq 8.3$ MPa (22.5.3.1, ACI 318-14)

$M/Vd = 2.00$

$\mu_u = 1.7578197E-011$

$\nu_u = 7.8886091E-031$

$d = 0.8 \cdot D = 400.00$

$N_u = 7389.214$

$A_g = 196349.541$

From (11.5.4.8), ACI 318-14: $V_s = V_{s1} + V_{s2} = 308425.138$

$V_{s1} = 308425.138$ is calculated for jacket, with:

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

$f_y = 625.00$

$s = 100.00$

V_{s1} is multiplied by $\text{Col1} = 1.00$

$s/d = 0.25$

$V_{s2} = 0.00$ is calculated for core, with:

$A_v = \pi/2 \cdot A_{\text{stirrup}} = 78956.835$

$f_y = 525.00$

$s = 250.00$

V_{s2} is multiplied by $\text{Col2} = 0.00$

$s/d = 1.04167$

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

From (11-11), ACI 440: $V_s + V_f \leq 440464.828$

$b_w \cdot d = \pi \cdot d^2 / 4 = 125663.706$

Calculation of Shear Strength at edge 2, $V_{r2} = 516365.617$

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

$V_{ColO} = 516365.617$

$knl = 1$ (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)

Mean concrete strength: $f'_c = (f'_c \cdot \text{jacket} \cdot \text{Area}_{\text{jacket}} + f'_c \cdot \text{core} \cdot \text{Area}_{\text{core}}) / \text{Area}_{\text{section}} = 27.84$, but $f'_c^{0.5} \leq 8.3$ MPa (22.5.3.1, ACI 318-14)

$M/Vd = 2.00$

$\mu_u = 1.7578197E-011$

$\nu_u = 7.8886091E-031$

$d = 0.8 \cdot D = 400.00$

$N_u = 7389.214$

$A_g = 196349.541$
 From (11.5.4.8), ACI 318-14: $V_s = V_{s1} + V_{s2} = 308425.138$
 $V_{s1} = 308425.138$ is calculated for jacket, with:
 $A_v = \frac{1}{2} A_{stirrup} = 123370.055$
 $f_y = 625.00$
 $s = 100.00$
 V_{s1} is multiplied by $Col1 = 1.00$
 $s/d = 0.25$
 $V_{s2} = 0.00$ is calculated for core, with:
 $A_v = \frac{1}{2} A_{stirrup} = 78956.835$
 $f_y = 525.00$
 $s = 250.00$
 V_{s2} is multiplied by $Col2 = 0.00$
 $s/d = 1.04167$
 $V_f ((11-3)-(11.4), ACI 440) = 0.00$
 From (11-11), ACI 440: $V_s + V_f \leq 440464.828$
 $b_w * d = \frac{1}{4} * d * d = 125663.706$

End Of Calculation of Shear Capacity ratio for element: column JCC1 of floor 1
 At local axis: 2

Start Of Calculation of Chord Rotation Capacity for element: column JCC1 of floor 1
 At local axis: 2
 Integration Section: (b)
 Section Type: rcjcs

Constant Properties

Knowledge Factor, $\phi = 0.80$
 Chord Rotation is generally considered as Deformation-Controlled Action according to Table C7-1, ASCE 41-17.
 Mean strengths are used for Deformation-Controlled Actions according to 7.5.1.3, ASCE 41-17
 Consequently:
 Jacket
 New material of Secondary Member: Concrete Strength, $f_c = f_{cm} = 30.00$
 New material of Secondary Member: Steel Strength, $f_s = f_{sm} = 625.00$
 Concrete Elasticity, $E_c = 25742.96$
 Steel Elasticity, $E_s = 200000.00$
 Existing Column
 Existing material of Secondary Member: Concrete Strength, $f_c = f_{cm} = 24.00$
 Existing material of Secondary Member: Steel Strength, $f_s = f_{sm} = 525.00$
 Concrete Elasticity, $E_c = 23025.204$
 Steel Elasticity, $E_s = 200000.00$
 External Diameter, $D = 500.00$
 Internal Diameter, $D = 300.00$
 Cover Thickness, $c = 25.00$
 Element Length, $L = 3000.00$
 Secondary Member
 Smooth Bars
 Ductile Steel
 Without Detailing for Earthquake Resistance (including stirrups not 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, $M = 6.4544616E-011$
 Shear Force, $V_2 = 5773.639$
 Shear Force, $V_3 = 4.2353875E-013$
 Axial Force, $F = -7386.905$
 Longitudinal Reinforcement Area Distribution (in 2 divisions)
 -Tension: $A_{st} = 0.00$

-Compression: $As_c = 3053.628$
 Longitudinal Reinforcement Area Distribution (in 3 divisions)
 -Tension: $As_{ten} = 1017.876$
 -Compression: $As_{com} = 1017.876$
 -Middle: $As_{mid} = 1017.876$
 Mean Diameter of Tension Reinforcement, $Db_L = 18.00$

Existing component: From table 7-7, ASCE 41_17: Final chord rotation Capacity $u_R = * u = 0.01095279$
 $u = y + p = 0.01369098$

- Calculation of y -

$y = (M_y * L_s / 3) / E_{eff} = 0.00869098$ ((4.29), Biskinis Phd))
 $M_y = 4.0621E+008$
 $L_s = M/V$ (with $L_s > 0.1 * L$ and $L_s < 2 * L$) = 1500.00
 From table 10.5, ASCE 41_17: $E_{eff} = factor * E_c * I_g = 2.3369E+013$
 $factor = 0.30$
 $A_g = 196349.541$
 Mean concrete strength: $fc' = (fc'_{jacket} * Area_{jacket} + fc'_{core} * Area_{core}) / Area_{section} = 27.84$
 $N = 7386.905$
 $E_c * I_g = E_{c_jacket} * I_{g_jacket} + E_{c_core} * I_{g_core} = 7.7898E+013$

Calculation of Yielding Moment M_y

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

$M_y = \min(M_{y_ten}, M_{y_com}) = 4.0621E+008$
 y ((10a) or (10b)) = 1.2225563E-005
 M_{y_ten} (8a) = 4.0621E+008
 $_{ten}$ (7a) = 67.28293
 error of function (7a) = 0.00093796
 M_{y_com} (8b) = 7.1091E+008
 $_{com}$ (7b) = 65.27568
 error of function (7b) = -0.00596424
 with $e_y = 0.003125$
 $eco = 0.002$
 $apl = 0.35$ ((9a) in Biskinis and Fardis for no FRP Wrap)
 $d1 = 44.00$
 $R = 250.00$
 $v = 0.00125404$
 $N = 7386.905$
 $Ac = 196349.541$
 $= 0.324$
 with $fc = 30.00$

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:

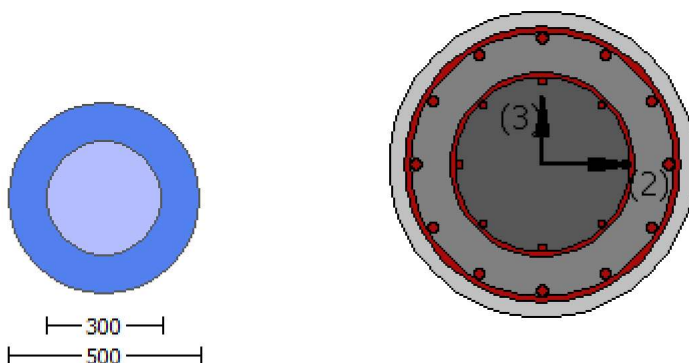
- Columns not controlled by inadequate development or splicing along the clear height because $l_b/d \geq 1$
 shear control ratio $V_y E / V_{col} E = 0.5552125$
 $d = d_{external} = 0.00$
 $s = s_{external} = 0.00$
 $t = s1 + s2 + 2 * t_f / b_w * (f_{fe} / f_s) = 0.00323428$
 jacket: $s1 = A_{v1} * (* D_c / 2) / (s1 * A_g) = 0.0027646$

$Av1 = 78.53982$, is the area of stirrup
 $Dc1 = D_{ext} - 2 \cdot cover - \text{External Hoop Diameter} = 440.00$, is the total Length of all stirrups parallel to loading (shear) direction
 $s1 = 100.00$
 core: $s2 = Av2 \cdot (Dc2/2) / (s2 \cdot Ag) = 0.00046968$
 $Av2 = 50.26548$, is the area of stirrup
 $Dc2 = D_{int} - \text{Internal Hoop Diameter} = 292.00$, is the total Length of all stirrups parallel to loading (shear) direction
 $s2 = 250.00$
 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. For the normalisation f_s of jacket is used.
 $NUD = 7386.905$
 $Ag = 196349.541$
 $f_{cE} = (f_{c_jacket} \cdot Area_jacket + f_{c_core} \cdot Area_core) / section_area = 27.84$
 $f_{yE} = (f_{y_ext_Long_Reinf} \cdot Area_ext_Long_Reinf + f_{y_int_Long_Reinf} \cdot Area_int_Long_Reinf) / Area_Tot_Long_Rein = 2.1219958E-314$
 $f_{yE} = (f_{y_ext_Trans_Reinf} \cdot Area_ext_Trans_Reinf + f_{y_int_Trans_Reinf} \cdot Area_int_Trans_Reinf) / Area_Tot_Trans_Rein = 610.4781$
 $p_l = Area_Tot_Long_Rein / (Ag) = 0.015552$
 $f_{cE} = 27.84$

 End Of Calculation of Chord Rotation Capacity for element: column JCC1 of floor 1
 At local axis: 2
 Integration Section: (b)

Calculation No. 7

column C1, Floor 1
 Limit State: Operational Level (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 JCC1 of floor 1

At local axis: 3
Integration Section: (b)
Section Type: rcjcs

Constant Properties

Knowledge Factor, $\gamma = 0.80$
Member Shear Force is generally considered as Force-Controlled Action according to Table C7-1, ASCE 41-17.
Lower-bound strengths are used for Force-Controlled Actions according to 7.5.1.3, ASCE 41-17
Consequently:
Jacket
New material of Secondary Member: Concrete Strength, $f_c = f_{c_lower_bound} = 20.00$
New material of Secondary Member: Steel Strength, $f_s = f_{s_lower_bound} = 500.00$
Concrete Elasticity, $E_c = 25742.96$
Steel Elasticity, $E_s = 200000.00$
Existing Column
Existing material of Secondary Member: Concrete Strength, $f_c = f_{c_lower_bound} = 16.00$
Existing material of Secondary Member: Steel Strength, $f_s = f_{s_lower_bound} = 420.00$
Concrete Elasticity, $E_c = 23025.204$
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, ASCE 41-17) because bending is considered as
Deformation-Controlled Action (Table C7-1, ASCE 41-17).
Jacket
New material: Concrete Strength, $f_c = f_{cm} = 30.00$
New material: Steel Strength, $f_s = f_{sm} = 625.00$
Existing Column
Existing material: Concrete Strength, $f_c = f_{cm} = 24.00$
Existing material: Steel Strength, $f_s = f_{sm} = 525.00$

External Diameter, $D = 500.00$
Internal Diameter, $D = 300.00$
Cover Thickness, $c = 25.00$
Element Length, $L = 3000.00$
Secondary Member
Smooth Bars
Ductile Steel
Without Detailing for Earthquake Resistance (including stirrups not 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, $M_a = 1.2063676E-009$
Shear Force, $V_a = -4.2353875E-013$
EDGE -B-
Bending Moment, $M_b = 6.4544616E-011$
Shear Force, $V_b = 4.2353875E-013$
BOTH EDGES
Axial Force, $F = -7386.905$
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: $As_{l,mid} = 1017.876$
Mean Diameter of Tension Reinforcement, $Db_{L,ten} = 18.00$

Existing component: From table 7-7, ASCE 41_17: Final Shear Capacity $VR = *V_n = 333433.119$

$V_n ((10.3), ASCE 41-17) = knl * V_{ColO} = 416791.398$

$V_{Col} = 416791.398$

$knl = 1.00$

$displacement_ductility_demand = 0.00$

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)

Mean concrete strength: $f'_c = (f'_{c_jacket} * Area_jacket + f'_{c_core} * Area_core) / Area_section = 18.56$, but $f'_c^{0.5} \leq 8.3$ MPa (22.5.3.1, ACI 318-14)

$M/Vd = 2.00$

$\mu_u = 6.4544616E-011$

$\nu_u = 4.2353875E-013$

$d = 0.8 * D = 400.00$

$N_u = 7386.905$

$A_g = 196349.541$

From (11.5.4.8), ACI 318-14: $V_s = V_{s1} + V_{s2} = 246740.11$

$V_{s1} = 246740.11$ is calculated for jacket, with:

$A_v = A_s / 2 = 123370.055$

$f_y = 500.00$

$s = 100.00$

V_{s1} is multiplied by $Col1 = 1.00$

$s/d = 0.25$

$V_{s2} = 0.00$ is calculated for core, with:

$A_v = A_s / 2 = 78956.835$

$f_y = 420.00$

$s = 250.00$

V_{s2} is multiplied by $Col2 = 0.00$

$s/d = 1.04167$

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

From (11-11), ACI 440: $V_s + V_f \leq 359638.026$

$b_w * d = A_s * d / 4 = 125663.706$

$displacement_ductility_demand$ is calculated as δ_u / y

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

From analysis, chord rotation $\theta = 6.7088938E-021$

$y = (M_y * L_s / 3) / E_{eff} = 0.00869098$ ((4.29), Biskinis Phd))

$M_y = 4.0621E+008$

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

From table 10.5, ASCE 41_17: $E_{eff} = factor * E_c * I_g = 2.3369E+013$

$factor = 0.30$

$A_g = 196349.541$

Mean concrete strength: $f'_c = (f'_{c_jacket} * Area_jacket + f'_{c_core} * Area_core) / Area_section = 27.84$

$N = 7386.905$

$E_c * I_g = E_{c_jacket} * I_{g_jacket} + E_{c_core} * I_{g_core} = 7.7898E+013$

Calculation of Yielding Moment M_y

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

$M_y = \min(M_{y_ten}, M_{y_com}) = 4.0621E+008$

$y ((10a) \text{ or } (10b)) = 1.2225563E-005$

$M_{y_ten} (8a) = 4.0621E+008$

$\delta_{y_ten} (7a) = 67.28293$

error of function (7a) = 0.00093796

$M_{y_com} (8b) = 7.1091E+008$

$\delta_{y_com} (7b) = 65.27568$

error of function (7b) = -0.00596424

with $e_y = 0.003125$

$\epsilon_{co} = 0.002$
 $\alpha_{pl} = 0.35$ ((9a) in Biskinis and Fardis for no FRP Wrap)
 $d_1 = 44.00$
 $R = 250.00$
 $v = 0.00125404$
 $N = 7386.905$
 $A_c = 196349.541$
 $= 0.324$
 with $f_c = 30.00$

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 JCC1 of floor 1

At local axis: 3

Integration Section: (b)

Calculation No. 8

column C1, Floor 1

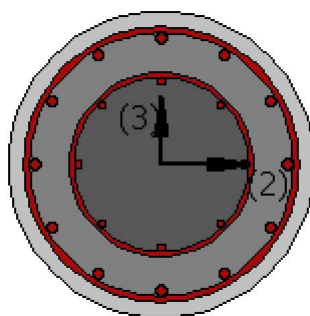
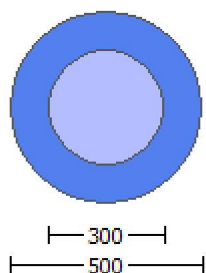
Limit State: Operational Level (data interpolation between analysis steps 1 and 2)

Analysis: Uniform +X

Check: Chord rotation capacity (ϕ)

Edge: End

Local Axis: (3)



Start Of Calculation of Shear Capacity ratio for element: column JCC1 of floor 1

At Shear local axis: 3

(Bending local axis: 2)

Section Type: rcjcs

Constant Properties

Knowledge Factor, $\phi = 0.80$

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

```

Consequently:
Jacket
New material of Secondary Member: Concrete Strength,  $f_c = f_{cm} = 30.00$ 
New material of Secondary Member: Steel Strength,  $f_s = f_{sm} = 625.00$ 
Concrete Elasticity,  $E_c = 25742.96$ 
Steel Elasticity,  $E_s = 200000.00$ 
Existing Column
Existing material of Secondary Member: Concrete Strength,  $f_c = f_{cm} = 24.00$ 
Existing material of Secondary Member: Steel Strength,  $f_s = f_{sm} = 525.00$ 
Concrete Elasticity,  $E_c = 23025.204$ 
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
Jacket
New material: Steel Strength,  $f_s = 1.25 \cdot f_{sm} = 781.25$ 
Existing Column
Existing material: Steel Strength,  $f_s = 1.25 \cdot f_{sm} = 656.25$ 
#####
External Diameter,  $D = 500.00$ 
Internal Diameter,  $D = 300.00$ 
Cover Thickness,  $c = 25.00$ 
Mean Confinement Factor overall section = 1.00
Element Length,  $L = 3000.00$ 
Secondary Member
Smooth Bars
Ductile Steel
Without Detailing for Earthquake Resistance (including stirrups not closed at 135°)
Longitudinal Bars With Ends Lapped Starting at the End Sections
Adequate Lap Length ( $l_o/l_{ou}, \min >= 1$ )
No FRP Wrapping
-----

Stepwise Properties
-----
At local axis: 3
EDGE -A-
Shear Force,  $V_a = 3.5877337E-031$ 
EDGE -B-
Shear Force,  $V_b = -3.5877337E-031$ 
BOTH EDGES
Axial Force,  $F = -7389.214$ 
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 ,  $V_e/V_r = 0.5552125$ 
Member Controlled by Flexure ( $V_e/V_r < 1$ )
Calculation of Shear Demand from fig. R18.6.5, ACI 318-14  $V_e = (M_{pr1} + M_{pr2})/l_n = 286692.647$ 
with
 $M_{pr1} = \max(\mu_{u1+}, \mu_{u1-}) = 4.3004E+008$ 
 $\mu_{u1+} = 4.3004E+008$ , 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
 $\mu_{u1-} = 4.3004E+008$ , 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
 $M_{pr2} = \max(\mu_{u2+}, \mu_{u2-}) = 4.3004E+008$ 
 $\mu_{u2+} = 4.3004E+008$ , 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
 $\mu_{u2-} = 4.3004E+008$ , 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 Mu1+

Calculation of ultimate Moment Strength ((3.67), Biskinis Phd), Mu
Mu = 4.3004E+008

= 1.0821
' = 0.95748645
error of function (3.68), Biskinis Phd = 71900.796
From 5A.2, TBDY: fcc = fc* c = 30.00
conf. factor c = 1.00
fc = 30.00
From 10.3.5, ASCE 41-17, Final value of fy: fy*Min(1,1.25*(lb/d)^ 2/3) = 781.25
lb/d = 1.00
d1 = 44.00
R = 250.00
v = 0.00125383
N = 7389.214
Ac = 196349.541
= *Min(1,1.25*(lb/d)^ 2/3) = 0.324

Calculation of ratio lb/d

Adequate Lap Length: lb/d >= 1

Calculation of Mu1-

Calculation of ultimate Moment Strength ((3.67), Biskinis Phd), Mu
Mu = 4.3004E+008

= 1.0821
' = 0.95748645
error of function (3.68), Biskinis Phd = 71900.796
From 5A.2, TBDY: fcc = fc* c = 30.00
conf. factor c = 1.00
fc = 30.00
From 10.3.5, ASCE 41-17, Final value of fy: fy*Min(1,1.25*(lb/d)^ 2/3) = 781.25
lb/d = 1.00
d1 = 44.00
R = 250.00
v = 0.00125383
N = 7389.214
Ac = 196349.541
= *Min(1,1.25*(lb/d)^ 2/3) = 0.324

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 = 4.3004E+008

$\lambda = 1.0821$
 $\lambda' = 0.95748645$
error of function (3.68), Biskinis Phd = 71900.796
From 5A.2, TBDY: $f_{cc} = f_c' \cdot c = 30.00$
conf. factor $c = 1.00$
 $f_c = 30.00$
From 10.3.5, ASCE 41-17, Final value of f_y : $f_y \cdot \text{Min}(1, 1.25 \cdot (l_b/l_d)^{2/3}) = 781.25$
 $l_b/l_d = 1.00$
 $d_1 = 44.00$
 $R = 250.00$
 $v = 0.00125383$
 $N = 7389.214$
 $A_c = 196349.541$
 $= \lambda' \cdot \text{Min}(1, 1.25 \cdot (l_b/l_d)^{2/3}) = 0.324$

Calculation of ratio l_b/l_d

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

Calculation of μ_2 -

Calculation of ultimate Moment Strength ((3.67), Biskinis Phd), μ
 $\mu = 4.3004E+008$

$\lambda = 1.0821$
 $\lambda' = 0.95748645$
error of function (3.68), Biskinis Phd = 71900.796
From 5A.2, TBDY: $f_{cc} = f_c' \cdot c = 30.00$
conf. factor $c = 1.00$
 $f_c = 30.00$
From 10.3.5, ASCE 41-17, Final value of f_y : $f_y \cdot \text{Min}(1, 1.25 \cdot (l_b/l_d)^{2/3}) = 781.25$
 $l_b/l_d = 1.00$
 $d_1 = 44.00$
 $R = 250.00$
 $v = 0.00125383$
 $N = 7389.214$
 $A_c = 196349.541$
 $= \lambda' \cdot \text{Min}(1, 1.25 \cdot (l_b/l_d)^{2/3}) = 0.324$

Calculation of ratio l_b/l_d

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

Calculation of Shear Strength $V_r = \text{Min}(V_{r1}, V_{r2}) = 516365.617$

Calculation of Shear Strength at edge 1, $V_{r1} = 516365.617$

$V_{r1} = V_{CoI} \text{ ((10.3), ASCE 41-17)} = k_n \cdot V_{CoI0}$
 $V_{CoI0} = 516365.617$
 $k_n = 1$ (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).

$\lambda = 1$ (normal-weight concrete)
Mean concrete strength: $f_c' = (f_c'_{\text{jacket}} \cdot \text{Area}_{\text{jacket}} + f_c'_{\text{core}} \cdot \text{Area}_{\text{core}}) / \text{Area}_{\text{section}} = 27.84$, but $f_c'^{0.5} \leq 8.3$ MPa (22.5.3.1, ACI 318-14)
 $M/Vd = 2.00$
 $\mu = 1.6653780E-011$

$V_u = 3.5877337E-031$
 $d = 0.8 \cdot D = 400.00$
 $N_u = 7389.214$
 $A_g = 196349.541$
 From (11.5.4.8), ACI 318-14: $V_s = V_{s1} + V_{s2} = 308425.138$
 $V_{s1} = 308425.138$ is calculated for jacket, with:
 $A_v = \frac{1}{2} \cdot A_{stirrup} = 123370.055$
 $f_y = 625.00$
 $s = 100.00$
 V_{s1} is multiplied by $Col1 = 1.00$
 $s/d = 0.25$
 $V_{s2} = 0.00$ is calculated for core, with:
 $A_v = \frac{1}{2} \cdot A_{stirrup} = 78956.835$
 $f_y = 525.00$
 $s = 250.00$
 V_{s2} is multiplied by $Col2 = 0.00$
 $s/d = 1.04167$
 $V_f ((11-3)-(11.4), ACI 440) = 0.00$
 From (11-11), ACI 440: $V_s + V_f \leq 440464.828$
 $b_w \cdot d = \frac{1}{4} \cdot d \cdot d = 125663.706$

Calculation of Shear Strength at edge 2, $V_{r2} = 516365.617$
 $V_{r2} = V_{Col} ((10.3), ASCE 41-17) = knl \cdot V_{Col0}$
 $V_{Col0} = 516365.617$
 $knl = 1$ (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)
 Mean concrete strength: $f_c' = (f_c'_{jacket} \cdot Area_{jacket} + f_c'_{core} \cdot Area_{core}) / Area_{section} = 27.84$, but $f_c'^{0.5} \leq 8.3$
 MPa (22.5.3.1, ACI 318-14)
 $M/Vd = 2.00$
 $M_u = 1.6653780E-011$
 $V_u = 3.5877337E-031$
 $d = 0.8 \cdot D = 400.00$
 $N_u = 7389.214$
 $A_g = 196349.541$
 From (11.5.4.8), ACI 318-14: $V_s = V_{s1} + V_{s2} = 308425.138$
 $V_{s1} = 308425.138$ is calculated for jacket, with:
 $A_v = \frac{1}{2} \cdot A_{stirrup} = 123370.055$
 $f_y = 625.00$
 $s = 100.00$
 V_{s1} is multiplied by $Col1 = 1.00$
 $s/d = 0.25$
 $V_{s2} = 0.00$ is calculated for core, with:
 $A_v = \frac{1}{2} \cdot A_{stirrup} = 78956.835$
 $f_y = 525.00$
 $s = 250.00$
 V_{s2} is multiplied by $Col2 = 0.00$
 $s/d = 1.04167$
 $V_f ((11-3)-(11.4), ACI 440) = 0.00$
 From (11-11), ACI 440: $V_s + V_f \leq 440464.828$
 $b_w \cdot d = \frac{1}{4} \cdot d \cdot d = 125663.706$

End Of Calculation of Shear Capacity ratio for element: column JCC1 of floor 1
 At local axis: 3

Start Of Calculation of Shear Capacity ratio for element: column JCC1 of floor 1
 At Shear local axis: 2
 (Bending local axis: 3)
 Section Type: rcjcs

Constant Properties

Knowledge Factor, $\phi = 0.80$

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

Consequently:

Jacket

New material of Secondary Member: Concrete Strength, $f_c = f_{cm} = 30.00$

New material of Secondary Member: Steel Strength, $f_s = f_{sm} = 625.00$

Concrete Elasticity, $E_c = 25742.96$

Steel Elasticity, $E_s = 200000.00$

Existing Column

Existing material of Secondary Member: Concrete Strength, $f_c = f_{cm} = 24.00$

Existing material of Secondary Member: Steel Strength, $f_s = f_{sm} = 525.00$

Concrete Elasticity, $E_c = 23025.204$

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

Jacket

New material: Steel Strength, $f_s = 1.25 \cdot f_{sm} = 781.25$

Existing Column

Existing material: Steel Strength, $f_s = 1.25 \cdot f_{sm} = 656.25$

#####

External Diameter, $D = 500.00$

Internal Diameter, $D = 300.00$

Cover Thickness, $c = 25.00$

Mean Confinement Factor overall section = 1.00

Element Length, $L = 3000.00$

Secondary Member

Smooth Bars

Ductile Steel

Without Detailing for Earthquake Resistance (including stirrups not 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, $V_a = -7.8886091E-031$

EDGE -B-

Shear Force, $V_b = 7.8886091E-031$

BOTH EDGES

Axial Force, $F = -7389.214$

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: $A_{st,mid} = 1017.876$

Calculation of Shear Capacity ratio, $V_e/V_r = 0.5552125$

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

Calculation of Shear Demand from fig. R18.6.5, ACI 318-14 $V_e = (M_{pr1} + M_{pr2})/l_n = 286692.647$

with

$M_{pr1} = \max(M_{u1+}, M_{u1-}) = 4.3004E+008$

$M_{u1+} = 4.3004E+008$, 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

$M_{u1-} = 4.3004E+008$, 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

$$M_{pr2} = \text{Max}(M_{u2+}, M_{u2-}) = 4.3004\text{E}+008$$

$M_{u2+} = 4.3004\text{E}+008$, 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

$M_{u2-} = 4.3004\text{E}+008$, 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
 $M_u = 4.3004\text{E}+008$

$$= 1.0821$$

$$\lambda = 0.95748645$$

error of function (3.68), Biskinis Phd = 71900.796

From 5A.2, TB DY: $f_{cc} = f_c \cdot c = 30.00$

conf. factor $c = 1.00$

$$f_c = 30.00$$

From 10.3.5, ASCE 41-17, Final value of f_y : $f_y \cdot \text{Min}(1, 1.25 \cdot (l_b/d)^{2/3}) = 781.25$

$$l_b/d = 1.00$$

$$d_1 = 44.00$$

$$R = 250.00$$

$$v = 0.00125383$$

$$N = 7389.214$$

$$A_c = 196349.541$$

$$= \lambda \cdot \text{Min}(1, 1.25 \cdot (l_b/d)^{2/3}) = 0.324$$

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
 $M_u = 4.3004\text{E}+008$

$$= 1.0821$$

$$\lambda = 0.95748645$$

error of function (3.68), Biskinis Phd = 71900.796

From 5A.2, TB DY: $f_{cc} = f_c \cdot c = 30.00$

conf. factor $c = 1.00$

$$f_c = 30.00$$

From 10.3.5, ASCE 41-17, Final value of f_y : $f_y \cdot \text{Min}(1, 1.25 \cdot (l_b/d)^{2/3}) = 781.25$

$$l_b/d = 1.00$$

$$d_1 = 44.00$$

$$R = 250.00$$

$$v = 0.00125383$$

$$N = 7389.214$$

$$A_c = 196349.541$$

$$= \lambda \cdot \text{Min}(1, 1.25 \cdot (l_b/d)^{2/3}) = 0.324$$

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 = 4.3004E+008

= 1.0821
' = 0.95748645
error of function (3.68), Biskinis Phd = 71900.796
From 5A.2, TBDY: fcc = fc* c = 30.00
conf. factor c = 1.00
fc = 30.00
From 10.3.5, ASCE 41-17, Final value of fy: fy*Min(1,1.25*(lb/d)^ 2/3) = 781.25
lb/d = 1.00
d1 = 44.00
R = 250.00
v = 0.00125383
N = 7389.214
Ac = 196349.541
= *Min(1,1.25*(lb/d)^ 2/3) = 0.324

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 = 4.3004E+008

= 1.0821
' = 0.95748645
error of function (3.68), Biskinis Phd = 71900.796
From 5A.2, TBDY: fcc = fc* c = 30.00
conf. factor c = 1.00
fc = 30.00
From 10.3.5, ASCE 41-17, Final value of fy: fy*Min(1,1.25*(lb/d)^ 2/3) = 781.25
lb/d = 1.00
d1 = 44.00
R = 250.00
v = 0.00125383
N = 7389.214
Ac = 196349.541
= *Min(1,1.25*(lb/d)^ 2/3) = 0.324

Calculation of ratio lb/d

Adequate Lap Length: lb/d >= 1

Calculation of Shear Strength Vr = Min(Vr1,Vr2) = 516365.617

Calculation of Shear Strength at edge 1, Vr1 = 516365.617
Vr1 = VCol ((10.3), ASCE 41-17) = knl*VColO
VColO = 516365.617
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)
Mean concrete strength: $fc' = (fc'_{jacket} \cdot Area_{jacket} + fc'_{core} \cdot Area_{core}) / Area_{section} = 27.84$, but $fc'^{0.5} \leq 8.3$ MPa (22.5.3.1, ACI 318-14)
 $M/Vd = 2.00$
 $\mu_u = 1.7578197E-011$
 $V_u = 7.8886091E-031$
 $d = 0.8 \cdot D = 400.00$
 $N_u = 7389.214$
 $A_g = 196349.541$
From (11.5.4.8), ACI 318-14: $V_s = V_{s1} + V_{s2} = 308425.138$
 $V_{s1} = 308425.138$ is calculated for jacket, with:
 $A_v = \sqrt{2} \cdot A_{stirrup} = 123370.055$
 $f_y = 625.00$
 $s = 100.00$
 V_{s1} is multiplied by $Col1 = 1.00$
 $s/d = 0.25$
 $V_{s2} = 0.00$ is calculated for core, with:
 $A_v = \sqrt{2} \cdot A_{stirrup} = 78956.835$
 $f_y = 525.00$
 $s = 250.00$
 V_{s2} is multiplied by $Col2 = 0.00$
 $s/d = 1.04167$
 $V_f ((11-3)-(11.4), ACI 440) = 0.00$
From (11-11), ACI 440: $V_s + V_f \leq 440464.828$
 $b_w \cdot d = \sqrt{3} \cdot d^2 / 4 = 125663.706$

Calculation of Shear Strength at edge 2, $V_{r2} = 516365.617$
 $V_{r2} = V_{Col} ((10.3), ASCE 41-17) = knl \cdot V_{Col0}$
 $V_{Col0} = 516365.617$
 $knl = 1$ (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)
Mean concrete strength: $fc' = (fc'_{jacket} \cdot Area_{jacket} + fc'_{core} \cdot Area_{core}) / Area_{section} = 27.84$, but $fc'^{0.5} \leq 8.3$ MPa (22.5.3.1, ACI 318-14)
 $M/Vd = 2.00$
 $\mu_u = 1.7578197E-011$
 $V_u = 7.8886091E-031$
 $d = 0.8 \cdot D = 400.00$
 $N_u = 7389.214$
 $A_g = 196349.541$
From (11.5.4.8), ACI 318-14: $V_s = V_{s1} + V_{s2} = 308425.138$
 $V_{s1} = 308425.138$ is calculated for jacket, with:
 $A_v = \sqrt{2} \cdot A_{stirrup} = 123370.055$
 $f_y = 625.00$
 $s = 100.00$
 V_{s1} is multiplied by $Col1 = 1.00$
 $s/d = 0.25$
 $V_{s2} = 0.00$ is calculated for core, with:
 $A_v = \sqrt{2} \cdot A_{stirrup} = 78956.835$
 $f_y = 525.00$
 $s = 250.00$
 V_{s2} is multiplied by $Col2 = 0.00$
 $s/d = 1.04167$
 $V_f ((11-3)-(11.4), ACI 440) = 0.00$
From (11-11), ACI 440: $V_s + V_f \leq 440464.828$
 $b_w \cdot d = \sqrt{3} \cdot d^2 / 4 = 125663.706$

End Of Calculation of Shear Capacity ratio for element: column JCC1 of floor 1
At local axis: 2

Start Of Calculation of Chord Rotation Capacity for element: column JCC1 of floor 1

At local axis: 3

Integration Section: (b)

Section Type: rcjcs

Constant Properties

Knowledge Factor, $\gamma = 0.80$

Chord Rotation is generally considered as Deformation-Controlled Action according to Table C7-1, ASCE 41-17.

Mean strengths are used for Deformation-Controlled Actions according to 7.5.1.3, ASCE 41-17

Consequently:

Jacket

New material of Secondary Member: Concrete Strength, $f_c = f_{cm} = 30.00$

New material of Secondary Member: Steel Strength, $f_s = f_{sm} = 625.00$

Concrete Elasticity, $E_c = 25742.96$

Steel Elasticity, $E_s = 200000.00$

Existing Column

Existing material of Secondary Member: Concrete Strength, $f_c = f_{cm} = 24.00$

Existing material of Secondary Member: Steel Strength, $f_s = f_{sm} = 525.00$

Concrete Elasticity, $E_c = 23025.204$

Steel Elasticity, $E_s = 200000.00$

External Diameter, $D = 500.00$

Internal Diameter, $D = 300.00$

Cover Thickness, $c = 25.00$

Element Length, $L = 3000.00$

Secondary Member

Smooth Bars

Ductile Steel

Without Detailing for Earthquake Resistance (including stirrups not 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, $M = -0.02600098$

Shear Force, $V_2 = 5773.639$

Shear Force, $V_3 = 4.2353875E-013$

Axial Force, $F = -7386.905$

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, $Db_L = 18.00$

Existing component: From table 7-7, ASCE 41_17: Final chord rotation Capacity $u_R = \gamma \cdot u = 0.00539056$

$u = \gamma \cdot y + p = 0.0067382$

- Calculation of γ -

$\gamma = (M_y \cdot L_s / 3) / E_{eff} = 0.0017382 ((4.29), \text{Biskinis Phd})$

$M_y = 4.0621E+008$

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

From table 10.5, ASCE 41_17: $E_{eff} = \text{factor} \cdot E_c \cdot I_g = 2.3369E+013$

factor = 0.30

$A_g = 196349.541$

Mean concrete strength: $f_c' = (f_c'_{\text{jacket}} \cdot \text{Area}_{\text{jacket}} + f_c'_{\text{core}} \cdot \text{Area}_{\text{core}}) / \text{Area}_{\text{section}} = 27.84$

$N = 7386.905$

$$E_c I_g = E_c I_{g_jacket} + E_c I_{g_core} = 7.7898E+013$$

Calculation of Yielding Moment M_y

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

$$\begin{aligned} M_y &= \min(M_{y_ten}, M_{y_com}) = 4.0621E+008 \\ \gamma &((10a) \text{ or } (10b)) = 1.2225563E-005 \\ M_{y_ten} (8a) &= 4.0621E+008 \\ \gamma_{ten} (7a) &= 67.28293 \\ \text{error of function (7a)} &= 0.00093796 \\ M_{y_com} (8b) &= 7.1091E+008 \\ \gamma_{com} (7b) &= 65.27568 \\ \text{error of function (7b)} &= -0.00596424 \\ \text{with } e_y &= 0.003125 \\ e_{co} &= 0.002 \\ a_{pl} &= 0.35 ((9a) \text{ in Biskinis and Fardis for no FRP Wrap}) \\ d_1 &= 44.00 \\ R &= 250.00 \\ v &= 0.00125404 \\ N &= 7386.905 \\ A_c &= 196349.541 \\ &= 0.324 \\ \text{with } f_c &= 30.00 \end{aligned}$$

Calculation of ratio l_b/d

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

- Calculation of ρ -

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

with:

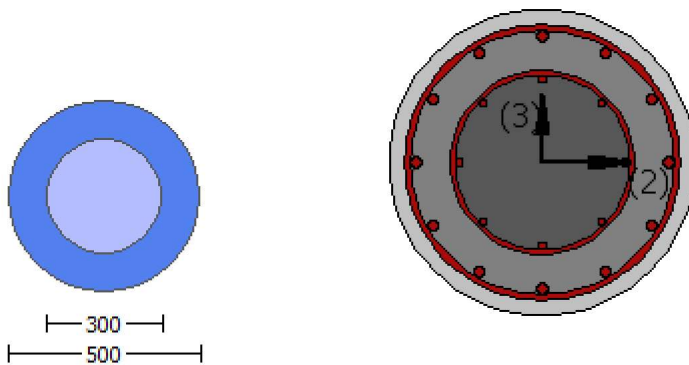
$$\begin{aligned} &\text{- Columns not controlled by inadequate development or splicing along the clear height because } l_b/d \geq 1 \\ &\text{shear control ratio } V_y E / V_{col} E = 0.5552125 \\ &d = d_{external} = 0.00 \\ &s = s_{external} = 0.00 \\ &t = s_1 + s_2 + 2 * t_f / b_w * (f_{fe} / f_s) = 0.00323428 \\ &\text{jacket: } s_1 = A_{v1} * (D_{c1} / 2) / (s_1 * A_g) = 0.0027646 \\ &\quad A_{v1} = 78.53982, \text{ is the area of stirrup} \\ &\quad D_{c1} = D_{ext} - 2 * \text{cover} - \text{External Hoop Diameter} = 440.00, \text{ is the total Length of all stirrups parallel to loading} \\ &\text{(shear) direction} \\ &\quad s_1 = 100.00 \\ &\text{core: } s_2 = A_{v2} * (D_{c2} / 2) / (s_2 * A_g) = 0.00046968 \\ &\quad A_{v2} = 50.26548, \text{ is the area of stirrup} \\ &\quad D_{c2} = D_{int} - \text{Internal Hoop Diameter} = 292.00, \text{ is the total Length of all stirrups parallel to loading (shear)} \\ &\text{direction} \\ &\quad s_2 = 250.00 \\ &\text{The term } 2 * t_f / b_w * (f_{fe} / f_s) \text{ is implemented to account for FRP contribution} \\ &\text{where } f = 2 * t_f / b_w \text{ is FRP ratio (EC8 - 3, A.4.4.3(6)) and } f_{fe} / f_s \text{ normalises } f \text{ to steel strength} \\ &\text{All these variables have already been given in Shear control ratio calculation.} \\ &\text{For the normalisation } f_s \text{ of jacket is used.} \\ &\quad NUD = 7386.905 \\ &\quad A_g = 196349.541 \\ &\quad f_{cE} = (f_c I_{jacket} * Area_{jacket} + f_c I_{core} * Area_{core}) / section_area = 27.84 \\ &\quad f_{yLE} = (f_{y_ext_Long_Reinf} * Area_{ext_Long_Reinf} + f_{y_int_Long_Reinf} * Area_{int_Long_Reinf}) / Area_{Tot_Long_Rein} = \\ &2.1219958E-314 \\ &\quad f_{yTE} = (f_{y_ext_Trans_Reinf} * Area_{ext_Trans_Reinf} + f_{y_int_Trans_Reinf} * Area_{int_Trans_Reinf}) / Area_{Tot_Trans_Rein} = \\ &610.4781 \\ &\quad \rho_l = Area_{Tot_Long_Rein} / (A_g) = 0.015552 \\ &\quad f_{cE} = 27.84 \end{aligned}$$

End Of Calculation of Chord Rotation Capacity for element: column JCC1 of floor 1

At local axis: 3
Integration Section: (b)

Calculation No. 9

column C1, Floor 1
Limit State: Life Safety (data interpolation between analysis steps 1 and 2)
Analysis: Uniform +X
Check: Shear capacity VRd
Edge: Start
Local Axis: (2)



Start Of Calculation of Shear Capacity for element: column JCC1 of floor 1
At local axis: 2
Integration Section: (a)
Section Type: rcjcs

Constant Properties

Knowledge Factor, $\gamma = 0.80$
Member Shear Force is generally considered as Force-Controlled Action according to Table C7-1, ASCE 41-17.
Lower-bound strengths are used for Force-Controlled Actions according to 7.5.1.3, ASCE 41-17
Consequently:
Jacket
New material of Secondary Member: Concrete Strength, $f_c = f_{c_lower_bound} = 20.00$
New material of Secondary Member: Steel Strength, $f_s = f_{s_lower_bound} = 500.00$
Concrete Elasticity, $E_c = 25742.96$
Steel Elasticity, $E_s = 200000.00$
Existing Column
Existing material of Secondary Member: Concrete Strength, $f_c = f_{c_lower_bound} = 16.00$
Existing material of Secondary Member: Steel Strength, $f_s = f_{s_lower_bound} = 420.00$
Concrete Elasticity, $E_c = 23025.204$
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, ASCE 41-17) because bending is considered as

Deformation-Controlled Action (Table C7-1, ASCE 41-17).

Jacket

New material: Concrete Strength, $f_c = f_{cm} = 30.00$

New material: Steel Strength, $f_s = f_{sm} = 625.00$

Existing Column

Existing material: Concrete Strength, $f_c = f_{cm} = 24.00$

Existing material: Steel Strength, $f_s = f_{sm} = 525.00$

#####

External Diameter, $D = 500.00$

Internal Diameter, $D = 300.00$

Cover Thickness, $c = 25.00$

Element Length, $L = 3000.00$

Secondary Member

Smooth Bars

Ductile Steel

Without Detailing for Earthquake Resistance (including stirrups not 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, $M_a = -2.6502E+007$

Shear Force, $V_a = -8831.613$

EDGE -B-

Bending Moment, $M_b = -0.03977224$

Shear Force, $V_b = 8831.613$

BOTH EDGES

Axial Force, $F = -7385.681$

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, $D_{bL,ten} = 18.00$

Existing component: From table 7-7, ASCE 41_17: Final Shear Capacity $V_R = V_n = 265412.506$

V_n ((10.3), ASCE 41-17) = $k_n \cdot V_{CoI} = 331765.633$

$V_{CoI} = 331765.633$

$k_n = 1.00$

displacement_ductility_demand = 0.01793451

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)

Mean concrete strength: $f_c' = (f_c'_{jacket} \cdot Area_{jacket} + f_c'_{core} \cdot Area_{core}) / Area_{section} = 18.56$, but $f_c'^{0.5} \leq 8.3$ MPa (22.5.3.1, ACI 318-14)

$M/V_d = 4.00$

$M_u = 2.6502E+007$

$V_u = 8831.613$

$d = 0.8 \cdot D = 400.00$

$N_u = 7385.681$

$A_g = 196349.541$

From (11.5.4.8), ACI 318-14: $V_s = V_{s1} + V_{s2} = 246740.11$

$V_{s1} = 246740.11$ is calculated for jacket, with:

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

$f_y = 500.00$

$s = 100.00$

V_{s1} is multiplied by $Col1 = 1.00$

$s/d = 0.25$
 $V_{s2} = 0.00$ is calculated for core, with:
 $A_v = \sqrt{2} \cdot A_{stirrup} = 78956.835$
 $f_y = 420.00$
 $s = 250.00$
 V_{s2} is multiplied by $Col2 = 0.00$
 $s/d = 1.04167$
 $V_f ((11-3)-(11.4), ACI 440) = 0.00$
 From (11-11), ACI 440: $V_s + V_f \leq 359638.026$
 $b_w \cdot d = \sqrt{d} \cdot d / 4 = 125663.706$

displacement ductility demand is calculated as δ / y

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

From analysis, chord rotation $\theta = 0.00031182$
 $y = (M_y \cdot L_s / 3) / E_{eff} = 0.01738646$ ((4.29), Biskinis Phd))
 $M_y = 4.0620E+008$
 $L_s = M/V$ (with $L_s > 0.1 \cdot L$ and $L_s < 2 \cdot L$) = 3000.778
 From table 10.5, ASCE 41_17: $E_{eff} = factor \cdot E_c \cdot I_g = 2.3369E+013$
 $factor = 0.30$
 $A_g = 196349.541$
 Mean concrete strength: $f_c' = (f_c'_{jacket} \cdot Area_{jacket} + f_c'_{core} \cdot Area_{core}) / Area_{section} = 27.84$
 $N = 7385.681$
 $E_c \cdot I_g = E_c \cdot I_{g,jacket} + E_c \cdot I_{g,core} = 7.7898E+013$

Calculation of Yielding Moment M_y

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

$M_y = \min(M_{y,ten}, M_{y,com}) = 4.0620E+008$
 $y ((10a) \text{ or } (10b)) = 1.2225560E-005$
 $M_{y,ten} (8a) = 4.0620E+008$
 $\delta_{ten} (7a) = 67.28291$
 error of function (7a) = 0.00093798
 $M_{y,com} (8b) = 7.1091E+008$
 $\delta_{com} (7b) = 65.27567$
 error of function (7b) = -0.00596426
 with $e_y = 0.003125$
 $e_{co} = 0.002$
 $apl = 0.35$ ((9a) in Biskinis and Fardis for no FRP Wrap)
 $d1 = 44.00$
 $R = 250.00$
 $v = 0.00125383$
 $N = 7385.681$
 $A_c = 196349.541$
 $\delta = 0.324$
 with $f_c = 30.00$

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 JCC1 of floor 1

At local axis: 2

Integration Section: (a)

Calculation No. 10

column C1, Floor 1

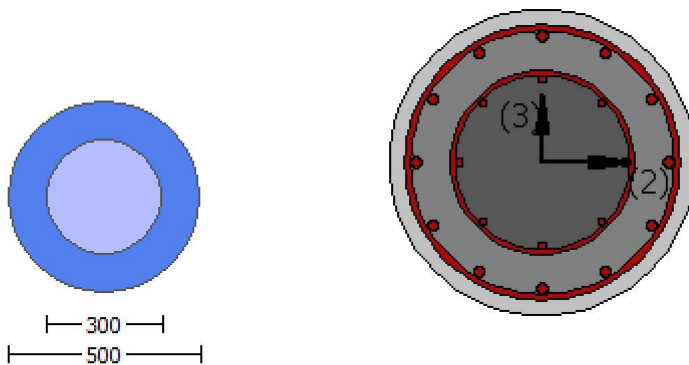
Limit State: Life Safety (data interpolation between analysis steps 1 and 2)

Analysis: Uniform +X

Check: Chord rotation capacity (ϕ)

Edge: Start

Local Axis: (2)



Start Of Calculation of Shear Capacity ratio for element: column JCC1 of floor 1

At Shear local axis: 3

(Bending local axis: 2)

Section Type: rcjcs

Constant Properties

Knowledge Factor, $\phi = 0.80$

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

Consequently:

Jacket

New material of Secondary Member: Concrete Strength, $f_c = f_{cm} = 30.00$

New material of Secondary Member: Steel Strength, $f_s = f_{sm} = 625.00$

Concrete Elasticity, $E_c = 25742.96$

Steel Elasticity, $E_s = 200000.00$

Existing Column

Existing material of Secondary Member: Concrete Strength, $f_c = f_{cm} = 24.00$

Existing material of Secondary Member: Steel Strength, $f_s = f_{sm} = 525.00$

Concrete Elasticity, $E_c = 23025.204$

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

Jacket

New material: Steel Strength, $f_s = 1.25 \cdot f_{sm} = 781.25$

Existing Column

Existing material: Steel Strength, $f_s = 1.25 \cdot f_{sm} = 656.25$

#####

External Diameter, $D = 500.00$

Internal Diameter, $D = 300.00$

Cover Thickness, $c = 25.00$

Mean Confinement Factor overall section = 1.00

Element Length, L = 3000.00
Secondary Member
Smooth Bars
Ductile Steel
Without Detailing for Earthquake Resistance (including stirrups not closed at 135°)
Longitudinal Bars With Ends Lapped Starting at the End Sections
Adequate Lap Length ($l_o/l_{ou}, \min > 1$)
No FRP Wrapping

Stepwise Properties

At local axis: 3
EDGE -A-
Shear Force, $V_a = 3.5877337E-031$
EDGE -B-
Shear Force, $V_b = -3.5877337E-031$
BOTH EDGES
Axial Force, $F = -7389.214$
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, $V_e/V_r = 0.5552125$
Member Controlled by Flexure ($V_e/V_r < 1$)
Calculation of Shear Demand from fig. R18.6.5, ACI 318-14 $V_e = (M_{pr1} + M_{pr2})/l_n = 286692.647$
with
 $M_{pr1} = \max(\mu_{u1+}, \mu_{u1-}) = 4.3004E+008$
 $\mu_{u1+} = 4.3004E+008$, 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
 $\mu_{u1-} = 4.3004E+008$, 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
 $M_{pr2} = \max(\mu_{u2+}, \mu_{u2-}) = 4.3004E+008$
 $\mu_{u2+} = 4.3004E+008$, 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
 $\mu_{u2-} = 4.3004E+008$, 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
 $\mu_u = 4.3004E+008$

$\beta_1 = 1.0821$
 $\beta_1 = 0.95748645$
error of function (3.68), Biskinis Phd = 71900.796
From 5A.2, TDY: $f_{cc} = f_c \cdot c = 30.00$
conf. factor $c = 1.00$
 $f_c = 30.00$
From 10.3.5, ASCE 41-17, Final value of f_y : $f_y \cdot \min(1, 1.25 \cdot (l_b/l_d)^{2/3}) = 781.25$
 $l_b/l_d = 1.00$
 $d_1 = 44.00$
 $R = 250.00$
 $v = 0.00125383$
 $N = 7389.214$
 $A_c = 196349.541$
 $= \cdot \min(1, 1.25 \cdot (l_b/l_d)^{2/3}) = 0.324$

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
 $\mu_u = 4.3004E+008$

$$= 1.0821$$

$$' = 0.95748645$$

error of function (3.68), Biskinis Phd = 71900.796

From 5A.2, TBDY: $f_{cc} = f_c \cdot c = 30.00$

conf. factor $c = 1.00$

$$f_c = 30.00$$

From 10.3.5, ASCE 41-17, Final value of f_y : $f_y \cdot \text{Min}(1, 1.25 \cdot (l_b/d)^{2/3}) = 781.25$

$$l_b/d = 1.00$$

$$d_1 = 44.00$$

$$R = 250.00$$

$$v = 0.00125383$$

$$N = 7389.214$$

$$A_c = 196349.541$$

$$= \cdot \text{Min}(1, 1.25 \cdot (l_b/d)^{2/3}) = 0.324$$

Calculation of ratio l_b/d

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

Calculation of μ_{u2+}

Calculation of ultimate Moment Strength ((3.67), Biskinis Phd), μ_u
 $\mu_u = 4.3004E+008$

$$= 1.0821$$

$$' = 0.95748645$$

error of function (3.68), Biskinis Phd = 71900.796

From 5A.2, TBDY: $f_{cc} = f_c \cdot c = 30.00$

conf. factor $c = 1.00$

$$f_c = 30.00$$

From 10.3.5, ASCE 41-17, Final value of f_y : $f_y \cdot \text{Min}(1, 1.25 \cdot (l_b/d)^{2/3}) = 781.25$

$$l_b/d = 1.00$$

$$d_1 = 44.00$$

$$R = 250.00$$

$$v = 0.00125383$$

$$N = 7389.214$$

$$A_c = 196349.541$$

$$= \cdot \text{Min}(1, 1.25 \cdot (l_b/d)^{2/3}) = 0.324$$

Calculation of ratio l_b/d

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

Calculation of μ_{u2-}

Calculation of ultimate Moment Strength ((3.67), Biskinis Phd), μ
 $\mu = 4.3004E+008$

$\phi = 1.0821$
 $\phi' = 0.95748645$
error of function (3.68), Biskinis Phd = 71900.796
From 5A.2, TBDY: $f_{cc} = f_c \cdot c = 30.00$
conf. factor $c = 1.00$
 $f_c = 30.00$
From 10.3.5, ASCE 41-17, Final value of f_y : $f_y \cdot \text{Min}(1, 1.25 \cdot (l_b/l_d)^{2/3}) = 781.25$
 $l_b/l_d = 1.00$
 $d_1 = 44.00$
 $R = 250.00$
 $v = 0.00125383$
 $N = 7389.214$
 $A_c = 196349.541$
 $= \cdot \text{Min}(1, 1.25 \cdot (l_b/l_d)^{2/3}) = 0.324$

Calculation of ratio l_b/l_d

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

Calculation of Shear Strength $V_r = \text{Min}(V_{r1}, V_{r2}) = 516365.617$

Calculation of Shear Strength at edge 1, $V_{r1} = 516365.617$

$V_{r1} = V_{co1}$ ((10.3), ASCE 41-17) = $k_{nl} \cdot V_{co1}$

$V_{co1} = 516365.617$

$k_{nl} = 1$ (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).

$\phi = 1$ (normal-weight concrete)
Mean concrete strength: $f_c' = (f_c'_{\text{jacket}} \cdot \text{Area}_{\text{jacket}} + f_c'_{\text{core}} \cdot \text{Area}_{\text{core}}) / \text{Area}_{\text{section}} = 27.84$, but $f_c'^{0.5} \leq 8.3$
MPa (22.5.3.1, ACI 318-14)
 $M/Vd = 2.00$
 $\mu = 1.6653780E-011$
 $V_u = 3.5877337E-031$
 $d = 0.8 \cdot D = 400.00$
 $N_u = 7389.214$
 $A_g = 196349.541$
From (11.5.4.8), ACI 318-14: $V_s = V_{s1} + V_{s2} = 308425.138$
 $V_{s1} = 308425.138$ is calculated for jacket, with:
 $A_v = \phi/2 \cdot A_{\text{stirrup}} = 123370.055$
 $f_y = 625.00$
 $s = 100.00$
 V_{s1} is multiplied by $\phi_{col1} = 1.00$
 $s/d = 0.25$
 $V_{s2} = 0.00$ is calculated for core, with:
 $A_v = \phi/2 \cdot A_{\text{stirrup}} = 78956.835$
 $f_y = 525.00$
 $s = 250.00$
 V_{s2} is multiplied by $\phi_{col2} = 0.00$
 $s/d = 1.04167$
 V_f ((11-3)-(11.4), ACI 440) = 0.00
From (11-11), ACI 440: $V_{s+} + V_f \leq 440464.828$
 $b_w \cdot d = \phi \cdot d \cdot d/4 = 125663.706$

Calculation of Shear Strength at edge 2, $V_{r2} = 516365.617$

Vr2 = VCol ((10.3), ASCE 41-17) = knl*VCol0
VCol0 = 516365.617
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)
Mean concrete strength: fc' = (fc'_jacket*Area_jacket + fc'_core*Area_core)/Area_section = 27.84, but $fc'^{0.5} \leq 8.3$
MPa (22.5.3.1, ACI 318-14)
M/Vd = 2.00
Mu = 1.6653780E-011
Vu = 3.5877337E-031
d = 0.8*D = 400.00
Nu = 7389.214
Ag = 196349.541
From (11.5.4.8), ACI 318-14: Vs = Vs1 + Vs2 = 308425.138
Vs1 = 308425.138 is calculated for jacket, with:
Av = /2*A_stirrup = 123370.055
fy = 625.00
s = 100.00
Vs1 is multiplied by Col1 = 1.00
s/d = 0.25
Vs2 = 0.00 is calculated for core, with:
Av = /2*A_stirrup = 78956.835
fy = 525.00
s = 250.00
Vs2 is multiplied by Col2 = 0.00
s/d = 1.04167
Vf ((11-3)-(11.4), ACI 440) = 0.00
From (11-11), ACI 440: Vs + Vf <= 440464.828
bw*d = *d*d/4 = 125663.706

End Of Calculation of Shear Capacity ratio for element: column JCC1 of floor 1
At local axis: 3

Start Of Calculation of Shear Capacity ratio for element: column JCC1 of floor 1
At Shear local axis: 2
(Bending local axis: 3)
Section Type: rcjs

Constant Properties

Knowledge Factor, = 0.80
Mean strength values are used for both shear and moment calculations.
Consequently:
Jacket
New material of Secondary Member: Concrete Strength, fc = fcm = 30.00
New material of Secondary Member: Steel Strength, fs = fsm = 625.00
Concrete Elasticity, Ec = 25742.96
Steel Elasticity, Es = 200000.00
Existing Column
Existing material of Secondary Member: Concrete Strength, fc = fcm = 24.00
Existing material of Secondary Member: Steel Strength, fs = fsm = 525.00
Concrete Elasticity, Ec = 23025.204
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
Jacket
New material: Steel Strength, fs = 1.25*fsm = 781.25
Existing Column
Existing material: Steel Strength, fs = 1.25*fsm = 656.25

#####

External Diameter, D = 500.00
Internal Diameter, D = 300.00
Cover Thickness, c = 25.00
Mean Confinement Factor overall section = 1.00
Element Length, L = 3000.00
Secondary Member
Smooth Bars
Ductile Steel
Without Detailing for Earthquake Resistance (including stirrups not 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, $V_a = -7.8886091E-031$
EDGE -B-
Shear Force, $V_b = 7.8886091E-031$
BOTH EDGES
Axial Force, $F = -7389.214$
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, $V_e/V_r = 0.5552125$
Member Controlled by Flexure ($V_e/V_r < 1$)
Calculation of Shear Demand from fig. R18.6.5, ACI 318-14 $V_e = (M_{pr1} + M_{pr2})/l_n = 286692.647$
with
 $M_{pr1} = \max(\mu_{u1+}, \mu_{u1-}) = 4.3004E+008$
 $\mu_{u1+} = 4.3004E+008$, 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
 $\mu_{u1-} = 4.3004E+008$, 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
 $M_{pr2} = \max(\mu_{u2+}, \mu_{u2-}) = 4.3004E+008$
 $\mu_{u2+} = 4.3004E+008$, 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
 $\mu_{u2-} = 4.3004E+008$, 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
 $\mu_u = 4.3004E+008$

$\phi = 1.0821$
 $\phi' = 0.95748645$
error of function (3.68), Biskinis Phd = 71900.796
From 5A.2, TBDY: $f_{cc} = f_c \cdot c = 30.00$
conf. factor $c = 1.00$
 $f_c = 30.00$
From 10.3.5, ASCE 41-17, Final value of f_y : $f_y \cdot \min(1, 1.25 \cdot (l_b/l_d)^{2/3}) = 781.25$
 $l_b/l_d = 1.00$
 $d_1 = 44.00$

R = 250.00
v = 0.00125383
N = 7389.214
Ac = 196349.541
= *Min(1,1.25*(lb/d)^ 2/3) = 0.324

Calculation of ratio lb/d

Adequate Lap Length: lb/d >= 1

Calculation of Mu1-

Calculation of ultimate Moment Strength ((3.67), Biskinis Phd), Mu
Mu = 4.3004E+008

= 1.0821
' = 0.95748645
error of function (3.68), Biskinis Phd = 71900.796
From 5A.2, TBDY: fcc = fc* c = 30.00
conf. factor c = 1.00
fc = 30.00
From 10.3.5, ASCE 41-17, Final value of fy: fy*Min(1,1.25*(lb/d)^ 2/3) = 781.25
lb/d = 1.00
d1 = 44.00
R = 250.00
v = 0.00125383
N = 7389.214
Ac = 196349.541
= *Min(1,1.25*(lb/d)^ 2/3) = 0.324

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 = 4.3004E+008

= 1.0821
' = 0.95748645
error of function (3.68), Biskinis Phd = 71900.796
From 5A.2, TBDY: fcc = fc* c = 30.00
conf. factor c = 1.00
fc = 30.00
From 10.3.5, ASCE 41-17, Final value of fy: fy*Min(1,1.25*(lb/d)^ 2/3) = 781.25
lb/d = 1.00
d1 = 44.00
R = 250.00
v = 0.00125383
N = 7389.214
Ac = 196349.541
= *Min(1,1.25*(lb/d)^ 2/3) = 0.324

Calculation of ratio lb/d

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

Calculation of μ_2 -

Calculation of ultimate Moment Strength ((3.67), Biskinis Phd), μ
 $\mu = 4.3004 \times 10^8$

$$= 1.0821$$

$$\gamma = 0.95748645$$

error of function (3.68), Biskinis Phd = 71900.796

From 5A.2, TDY: $f_{cc} = f_c \cdot c = 30.00$

conf. factor $c = 1.00$

$$f_c = 30.00$$

From 10.3.5, ASCE 41-17, Final value of f_y : $f_y \cdot \min(1, 1.25 \cdot (l_b/d)^{2/3}) = 781.25$

$$l_b/d = 1.00$$

$$d_1 = 44.00$$

$$R = 250.00$$

$$v = 0.00125383$$

$$N = 7389.214$$

$$A_c = 196349.541$$

$$= \cdot \min(1, 1.25 \cdot (l_b/d)^{2/3}) = 0.324$$

Calculation of ratio l_b/d

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

Calculation of Shear Strength $V_r = \min(V_{r1}, V_{r2}) = 516365.617$

Calculation of Shear Strength at edge 1, $V_{r1} = 516365.617$

$V_{r1} = V_{co1}$ ((10.3), ASCE 41-17) = $k_n \cdot V_{co1}$

$$V_{co1} = 516365.617$$

$k_n = 1$ (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)

Mean concrete strength: $f'_c = (f'_{c_jacket} \cdot Area_{jacket} + f'_{c_core} \cdot Area_{core}) / Area_{section} = 27.84$, but $f'_c^{0.5} \leq 8.3$
MPa ((22.5.3.1), ACI 318-14)

$$M/Vd = 2.00$$

$$\mu = 1.7578197 \times 10^{-11}$$

$$V_u = 7.8886091 \times 10^{-31}$$

$$d = 0.8 \cdot D = 400.00$$

$$N_u = 7389.214$$

$$A_g = 196349.541$$

From ((11.5.4.8), ACI 318-14: $V_s = V_{s1} + V_{s2} = 308425.138$

$V_{s1} = 308425.138$ is calculated for jacket, with:

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

$$f_y = 625.00$$

$$s = 100.00$$

V_{s1} is multiplied by $\phi_1 = 1.00$

$$s/d = 0.25$$

$V_{s2} = 0.00$ is calculated for core, with:

$$A_v = \pi/2 \cdot A_{stirrup} = 78956.835$$

$$f_y = 525.00$$

$$s = 250.00$$

V_{s2} is multiplied by $\phi_2 = 0.00$

$$s/d = 1.04167$$

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

From (11-11), ACI 440: $V_s + V_f \leq 440464.828$
 $b_w d = \frac{V_s}{\phi} = 125663.706$

Calculation of Shear Strength at edge 2, $V_{r2} = 516365.617$
 $V_{r2} = V_{Col} ((10.3), ASCE 41-17) = k_{nl} V_{Col0}$
 $V_{Col0} = 516365.617$
 $k_{nl} = 1$ (zero step-static loading)

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

$\phi = 1$ (normal-weight concrete)
Mean concrete strength: $f_c' = (f_c'_{jacket} \text{Area}_{jacket} + f_c'_{core} \text{Area}_{core}) / \text{Area}_{section} = 27.84$, but $f_c'^{0.5} \leq 8.3$ MPa (22.5.3.1, ACI 318-14)
 $M/Vd = 2.00$
 $M_u = 1.7578197E-011$
 $V_u = 7.8886091E-031$
 $d = 0.8D = 400.00$
 $N_u = 7389.214$
 $A_g = 196349.541$
From (11.5.4.8), ACI 318-14: $V_s = V_{s1} + V_{s2} = 308425.138$
 $V_{s1} = 308425.138$ is calculated for jacket, with:
 $A_v = \frac{V_{s1}}{f_y} = 123370.055$
 $f_y = 625.00$
 $s = 100.00$
 V_{s1} is multiplied by $\phi_{Col1} = 1.00$
 $s/d = 0.25$
 $V_{s2} = 0.00$ is calculated for core, with:
 $A_v = \frac{V_{s2}}{f_y} = 78956.835$
 $f_y = 525.00$
 $s = 250.00$
 V_{s2} is multiplied by $\phi_{Col2} = 0.00$
 $s/d = 1.04167$
 $V_f ((11-3)-(11.4), ACI 440) = 0.00$
From (11-11), ACI 440: $V_s + V_f \leq 440464.828$
 $b_w d = \frac{V_s}{\phi} = 125663.706$

End Of Calculation of Shear Capacity ratio for element: column JCC1 of floor 1
At local axis: 2

Start Of Calculation of Chord Rotation Capacity for element: column JCC1 of floor 1

At local axis: 2

Integration Section: (a)

Section Type: rcjcs

Constant Properties

Knowledge Factor, $\phi = 0.80$

Chord Rotation is generally considered as Deformation-Controlled Action according to Table C7-1, ASCE 41-17.

Mean strengths are used for Deformation-Controlled Actions according to 7.5.1.3, ASCE 41-17

Consequently:

Jacket

New material of Secondary Member: Concrete Strength, $f_c = f_{cm} = 30.00$

New material of Secondary Member: Steel Strength, $f_s = f_{sm} = 625.00$

Concrete Elasticity, $E_c = 25742.96$

Steel Elasticity, $E_s = 200000.00$

Existing Column

Existing material of Secondary Member: Concrete Strength, $f_c = f_{cm} = 24.00$

Existing material of Secondary Member: Steel Strength, $f_s = f_{sm} = 525.00$

Concrete Elasticity, $E_c = 23025.204$

Steel Elasticity, $E_s = 200000.00$

External Diameter, $D = 500.00$

Internal Diameter, D = 300.00
 Cover Thickness, c = 25.00
 Element Length, L = 3000.00
 Secondary Member
 Smooth Bars
 Ductile Steel
 Without Detailing for Earthquake Resistance (including stirrups not closed at 135°)
 Longitudinal Bars With Ends Lapped Starting at the End Sections
 Adequate Lap Length ($l_b/l_d > 1$)
 No FRP Wrapping

Stepwise Properties

Bending Moment, M = 1.8364926E-009
 Shear Force, V2 = -8831.613
 Shear Force, V3 = -6.4786357E-013
 Axial Force, F = -7385.681
 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_{t,com} = 1017.876
 -Middle: As_{t,mid} = 1017.876
 Mean Diameter of Tension Reinforcement, DbL = 18.00

Existing component: From table 7-7, ASCE 41_17: Final chord rotation Capacity $u_R = \frac{1}{2} u = 0.04140005$
 $u = y + p = 0.05175007$

- Calculation of y -

$y = (M_y * L_s / 3) / E_{eff} = 0.00869098$ ((4.29), Biskinis Phd))
 $M_y = 4.0620E+008$
 $L_s = M/V$ (with $L_s > 0.1 * L$ and $L_s < 2 * L$) = 1500.00
 From table 10.5, ASCE 41_17: $E_{eff} = factor * E_c * I_g = 2.3369E+013$
 $factor = 0.30$
 $A_g = 196349.541$
 Mean concrete strength: $fc' = (fc'_{jacket} * Area_{jacket} + fc'_{core} * Area_{core}) / Area_{section} = 27.84$
 $N = 7385.681$
 $E_c * I_g = E_c * I_{g,jacket} + E_c * I_{g,core} = 7.7898E+013$

Calculation of Yielding Moment M_y

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

$M_y = \min(M_{y,ten}, M_{y,com}) = 4.0620E+008$
 y ((10a) or (10b)) = 1.2225560E-005
 $M_{y,ten}$ (8a) = 4.0620E+008
 y_{ten} (7a) = 67.28291
 error of function (7a) = 0.00093798
 $M_{y,com}$ (8b) = 7.1091E+008
 y_{com} (7b) = 65.27567
 error of function (7b) = -0.00596426
 with $e_y = 0.003125$
 $e_{co} = 0.002$
 $apl = 0.35$ ((9a) in Biskinis and Fardis for no FRP Wrap)
 $d1 = 44.00$
 $R = 250.00$
 $v = 0.00125383$
 $N = 7385.681$

$A_c = 196349.541$
 $= 0.324$
with $f_c = 30.00$

Calculation of ratio l_b/l_d

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

- Calculation of p -

From table 10-9: $p = 0.04305909$

with:

- Columns not controlled by inadequate development or splicing along the clear height because $l_b/l_d \geq 1$
shear control ratio $V_y E / V_{CoI} E = 0.5552125$

$d = d_{\text{external}} = 0.00$

$s = s_{\text{external}} = 0.00$

$t = s_1 + s_2 + 2 \cdot t_f / b_w \cdot (f_{fe} / f_s) = 0.00323428$

jacket: $s_1 = A_{v1} \cdot (D_{c1} / 2) / (s_1 \cdot A_g) = 0.0027646$

$A_{v1} = 78.53982$, is the area of stirrup

$D_{c1} = D_{\text{ext}} - 2 \cdot \text{cover} - \text{External Hoop Diameter} = 440.00$, is the total Length of all stirrups parallel to loading (shear) direction

$s_1 = 100.00$

core: $s_2 = A_{v2} \cdot (D_{c2} / 2) / (s_2 \cdot A_g) = 0.00046968$

$A_{v2} = 50.26548$, is the area of stirrup

$D_{c2} = D_{\text{int}} - \text{Internal Hoop Diameter} = 292.00$, is the total Length of all stirrups parallel to loading (shear) direction

$s_2 = 250.00$

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.

For the normalisation f_s of jacket is used.

$N_{UD} = 7385.681$

$A_g = 196349.541$

$f_{cE} = (f_{c_jacket} \cdot \text{Area}_{\text{jacket}} + f_{c_core} \cdot \text{Area}_{\text{core}}) / \text{section_area} = 27.84$

$f_{yE} = (f_{y_ext_Long_Reinf} \cdot \text{Area}_{\text{ext_Long_Reinf}} + f_{y_int_Long_Reinf} \cdot \text{Area}_{\text{int_Long_Reinf}}) / \text{Area}_{\text{Tot_Long_Rein}} = 2.1219958E-314$

$f_{yE} = (f_{y_ext_Trans_Reinf} \cdot \text{Area}_{\text{ext_Trans_Reinf}} + f_{y_int_Trans_Reinf} \cdot \text{Area}_{\text{int_Trans_Reinf}}) / \text{Area}_{\text{Tot_Trans_Rein}} = 610.4781$

$p_l = \text{Area}_{\text{Tot_Long_Rein}} / (A_g) = 0.015552$

$f_{cE} = 27.84$

End Of Calculation of Chord Rotation Capacity for element: column JCC1 of floor 1

At local axis: 2

Integration Section: (a)

Calculation No. 11

column C1, Floor 1

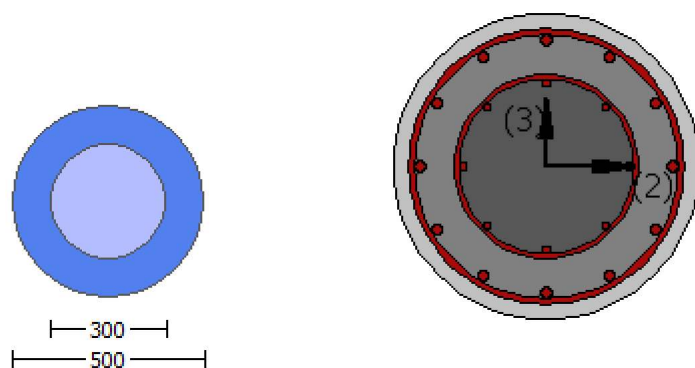
Limit State: Life Safety (data interpolation between analysis steps 1 and 2)

Analysis: Uniform +X

Check: Shear capacity VRd

Edge: Start

Local Axis: (3)



Start Of Calculation of Shear Capacity for element: column JCC1 of floor 1

At local axis: 3

Integration Section: (a)

Section Type: rcjcs

Constant Properties

Knowledge Factor, $\gamma = 0.80$

Member Shear Force is generally considered as Force-Controlled Action according to Table C7-1, ASCE 41-17.

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

Consequently:

Jacket

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

New material of Secondary Member: Steel Strength, $f_s = f_{s_lower_bound} = 500.00$

Concrete Elasticity, $E_c = 25742.96$

Steel Elasticity, $E_s = 200000.00$

Existing Column

Existing material of Secondary Member: Concrete Strength, $f_c = f_{c_lower_bound} = 16.00$

Existing material of Secondary Member: Steel Strength, $f_s = f_{s_lower_bound} = 420.00$

Concrete Elasticity, $E_c = 23025.204$

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, ASCE 41-17) because bending is considered as Deformation-Controlled Action (Table C7-1, ASCE 41-17).

Jacket

New material: Concrete Strength, $f_c = f_{cm} = 30.00$

New material: Steel Strength, $f_s = f_{sm} = 625.00$

Existing Column

Existing material: Concrete Strength, $f_c = f_{cm} = 24.00$

Existing material: Steel Strength, $f_s = f_{sm} = 525.00$

#####

External Diameter, $D = 500.00$

Internal Diameter, $D = 300.00$

Cover Thickness, $c = 25.00$

Element Length, $L = 3000.00$

Secondary Member

Smooth Bars

Ductile Steel

Without Detailing for Earthquake Resistance (including stirrups not closed at 135°)
Longitudinal Bars With Ends Lapped Starting at the End Sections
Adequate Lap Length ($l_o/l_{o,min} = l_b/l_d \geq 1$)
No FRP Wrapping

Stepwise Properties

EDGE -A-
Bending Moment, $M_a = 1.8364926E-009$
Shear Force, $V_a = -6.4786357E-013$
EDGE -B-
Bending Moment, $M_b = 1.0755087E-010$
Shear Force, $V_b = 6.4786357E-013$
BOTH EDGES
Axial Force, $F = -7385.681$
Longitudinal Reinforcement Area Distribution (in 2 divisions)
-Tension: $A_{s,t} = 1272.345$
-Compression: $A_{s,c} = 1781.283$
Longitudinal Reinforcement Area Distribution (in 3 divisions)
-Tension: $A_{s,ten} = 1017.876$
-Compression: $A_{s,com} = 1017.876$
-Middle: $A_{s,mid} = 1017.876$
Mean Diameter of Tension Reinforcement, $D_{bL,ten} = 18.00$

Existing component: From table 7-7, ASCE 41_17: Final Shear Capacity $V_R = V_n = 333432.925$
 $V_n ((10.3), ASCE 41-17) = k_n \cdot V_{CoI0} = 416791.156$
 $V_{CoI} = 416791.156$
 $k_n = 1.00$
 $displacement_ductility_demand = 0.00$

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)
Mean concrete strength: $f'_c = (f'_c_jacket \cdot Area_jacket + f'_c_core \cdot Area_core) / Area_section = 18.56$, but $f'_c^{0.5} \leq 8.3$
MPa (22.5.3.1, ACI 318-14)
 $M/Vd = 2.00$
 $M_u = 1.8364926E-009$
 $V_u = 6.4786357E-013$
 $d = 0.8 \cdot D = 400.00$
 $N_u = 7385.681$
 $A_g = 196349.541$
From (11.5.4.8), ACI 318-14: $V_s = V_{s1} + V_{s2} = 246740.11$
 $V_{s1} = 246740.11$ is calculated for jacket, with:
 $A_v = \sqrt{2} \cdot A_{stirrup} = 123370.055$
 $f_y = 500.00$
 $s = 100.00$
 V_{s1} is multiplied by $Col1 = 1.00$
 $s/d = 0.25$
 $V_{s2} = 0.00$ is calculated for core, with:
 $A_v = \sqrt{2} \cdot A_{stirrup} = 78956.835$
 $f_y = 420.00$
 $s = 250.00$
 V_{s2} is multiplied by $Col2 = 0.00$
 $s/d = 1.04167$
 $V_f ((11-3)-(11.4), ACI 440) = 0.00$
From (11-11), ACI 440: $V_s + V_f \leq 359638.026$
 $b_w \cdot d = \sqrt{2} \cdot d^2 / 4 = 125663.706$

$displacement_ductility_demand$ is calculated as $\frac{V_u}{V_R}$

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

From analysis, chord rotation $\theta = 2.0558999E-020$

$\phi_y = (M_y * L_s / 3) / E_{eff} = 0.00869098$ ((4.29), Biskinis Phd))

$M_y = 4.0620E+008$

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

From table 10.5, ASCE 41_17: $E_{eff} = \text{factor} * E_c * I_g = 2.3369E+013$

factor = 0.30

$A_g = 196349.541$

Mean concrete strength: $f'_c = (f'_{c_jacket} * \text{Area}_{jacket} + f'_{c_core} * \text{Area}_{core}) / \text{Area}_{section} = 27.84$

$N = 7385.681$

$E_c * I_g = E_{c_jacket} * I_{g_jacket} + E_{c_core} * I_{g_core} = 7.7898E+013$

Calculation of Yielding Moment M_y

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

$M_y = \text{Min}(M_{y_ten}, M_{y_com}) = 4.0620E+008$

ϕ_y ((10a) or (10b)) = 1.2225560E-005

M_{y_ten} (8a) = 4.0620E+008

ϕ_{y_ten} (7a) = 67.28291

error of function (7a) = 0.00093798

M_{y_com} (8b) = 7.1091E+008

ϕ_{y_com} (7b) = 65.27567

error of function (7b) = -0.00596426

with $e_y = 0.003125$

$e_{co} = 0.002$

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

$d_1 = 44.00$

$R = 250.00$

$v = 0.00125383$

$N = 7385.681$

$A_c = 196349.541$

$= 0.324$

with $f_c = 30.00$

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 JCC1 of floor 1

At local axis: 3

Integration Section: (a)

Calculation No. 12

column C1, Floor 1

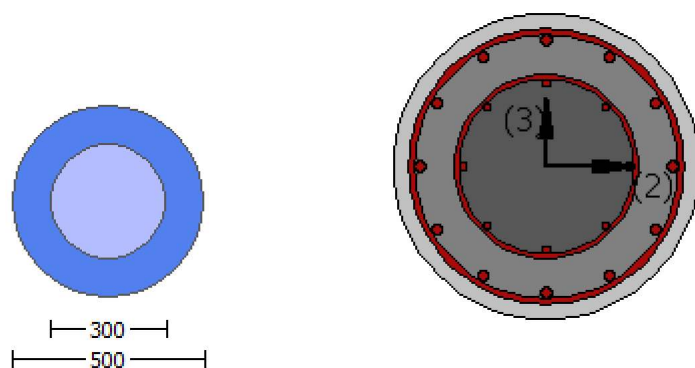
Limit State: Life Safety (data interpolation between analysis steps 1 and 2)

Analysis: Uniform +X

Check: Chord rotation capacity (ϕ)

Edge: Start

Local Axis: (3)



Start Of Calculation of Shear Capacity ratio for element: column JCC1 of floor 1

At Shear local axis: 3

(Bending local axis: 2)

Section Type: rcjcs

Constant Properties

Knowledge Factor, $\gamma = 0.80$

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

Consequently:

Jacket

New material of Secondary Member: Concrete Strength, $f_c = f_{cm} = 30.00$

New material of Secondary Member: Steel Strength, $f_s = f_{sm} = 625.00$

Concrete Elasticity, $E_c = 25742.96$

Steel Elasticity, $E_s = 200000.00$

Existing Column

Existing material of Secondary Member: Concrete Strength, $f_c = f_{cm} = 24.00$

Existing material of Secondary Member: Steel Strength, $f_s = f_{sm} = 525.00$

Concrete Elasticity, $E_c = 23025.204$

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

Jacket

New material: Steel Strength, $f_s = 1.25 \cdot f_{sm} = 781.25$

Existing Column

Existing material: Steel Strength, $f_s = 1.25 \cdot f_{sm} = 656.25$

#####

External Diameter, $D = 500.00$

Internal Diameter, $D = 300.00$

Cover Thickness, $c = 25.00$

Mean Confinement Factor overall section = 1.00

Element Length, $L = 3000.00$

Secondary Member

Smooth Bars

Ductile Steel

Without Detailing for Earthquake Resistance (including stirrups not 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, $V_a = 3.5877337E-031$

EDGE -B-

Shear Force, $V_b = -3.5877337E-031$

BOTH EDGES

Axial Force, $F = -7389.214$

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, $V_e/V_r = 0.5552125$

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

Calculation of Shear Demand from fig. R18.6.5, ACI 318-14 $V_e = (M_{pr1} + M_{pr2})/l_n = 286692.647$

with

$M_{pr1} = \text{Max}(Mu_{1+}, Mu_{1-}) = 4.3004E+008$

$Mu_{1+} = 4.3004E+008$, 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

$Mu_{1-} = 4.3004E+008$, 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

$M_{pr2} = \text{Max}(Mu_{2+}, Mu_{2-}) = 4.3004E+008$

$Mu_{2+} = 4.3004E+008$, 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

$Mu_{2-} = 4.3004E+008$, 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

$M_u = 4.3004E+008$

$\lambda = 1.0821$

$\lambda' = 0.95748645$

error of function (3.68), Biskinis Phd = 71900.796

From 5A.2, TBDY: $f_{cc} = f_c \cdot c = 30.00$

conf. factor $c = 1.00$

$f_c = 30.00$

From 10.3.5, ASCE 41-17, Final value of f_y : $f_y \cdot \text{Min}(1, 1.25 \cdot (l_b/d)^{2/3}) = 781.25$

$l_b/d = 1.00$

$d_1 = 44.00$

$R = 250.00$

$v = 0.00125383$

$N = 7389.214$

$A_c = 196349.541$

$= \lambda \cdot \text{Min}(1, 1.25 \cdot (l_b/d)^{2/3}) = 0.324$

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 = 4.3004E+008

= 1.0821
' = 0.95748645
error of function (3.68), Biskinis Phd = 71900.796
From 5A.2, TBDY: $f_{cc} = f_c \cdot c = 30.00$
conf. factor $c = 1.00$
 $f_c = 30.00$
From 10.3.5, ASCE 41-17, Final value of f_y : $f_y \cdot \text{Min}(1, 1.25 \cdot (l_b/d)^{2/3}) = 781.25$
 $l_b/d = 1.00$
 $d_1 = 44.00$
 $R = 250.00$
 $v = 0.00125383$
 $N = 7389.214$
 $A_c = 196349.541$
= $\cdot \text{Min}(1, 1.25 \cdot (l_b/d)^{2/3}) = 0.324$

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 = 4.3004E+008

= 1.0821
' = 0.95748645
error of function (3.68), Biskinis Phd = 71900.796
From 5A.2, TBDY: $f_{cc} = f_c \cdot c = 30.00$
conf. factor $c = 1.00$
 $f_c = 30.00$
From 10.3.5, ASCE 41-17, Final value of f_y : $f_y \cdot \text{Min}(1, 1.25 \cdot (l_b/d)^{2/3}) = 781.25$
 $l_b/d = 1.00$
 $d_1 = 44.00$
 $R = 250.00$
 $v = 0.00125383$
 $N = 7389.214$
 $A_c = 196349.541$
= $\cdot \text{Min}(1, 1.25 \cdot (l_b/d)^{2/3}) = 0.324$

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 = 4.3004E+008

= 1.0821

$\lambda = 0.95748645$
 error of function (3.68), Biskinis Phd = 71900.796
 From 5A.2, TBDY: $f_{cc} = f_c' \cdot \lambda = 30.00$
 conf. factor $\lambda = 1.00$
 $f_c = 30.00$
 From 10.3.5, ASCE 41-17, Final value of f_y : $f_y \cdot \text{Min}(1, 1.25 \cdot (l_b/d)^{2/3}) = 781.25$
 $l_b/d = 1.00$
 $d_1 = 44.00$
 $R = 250.00$
 $v = 0.00125383$
 $N = 7389.214$
 $A_c = 196349.541$
 $= \cdot \text{Min}(1, 1.25 \cdot (l_b/d)^{2/3}) = 0.324$

Calculation of ratio l_b/d

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

Calculation of Shear Strength $V_r = \text{Min}(V_{r1}, V_{r2}) = 516365.617$

Calculation of Shear Strength at edge 1, $V_{r1} = 516365.617$

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

$V_{ColO} = 516365.617$

$k_n l = 1$ (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)

Mean concrete strength: $f_c' = (f_c'_{\text{jacket}} \cdot \text{Area}_{\text{jacket}} + f_c'_{\text{core}} \cdot \text{Area}_{\text{core}}) / \text{Area}_{\text{section}} = 27.84$, but $f_c'^{0.5} \leq 8.3$ MPa (22.5.3.1, ACI 318-14)

$M/Vd = 2.00$

$\mu_u = 1.6653780E-011$

$\nu_u = 3.5877337E-031$

$d = 0.8 \cdot D = 400.00$

$N_u = 7389.214$

$A_g = 196349.541$

From (11.5.4.8), ACI 318-14: $V_s = V_{s1} + V_{s2} = 308425.138$

$V_{s1} = 308425.138$ is calculated for jacket, with:

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

$f_y = 625.00$

$s = 100.00$

V_{s1} is multiplied by $\text{Col1} = 1.00$

$s/d = 0.25$

$V_{s2} = 0.00$ is calculated for core, with:

$A_v = \pi/2 \cdot A_{\text{stirrup}} = 78956.835$

$f_y = 525.00$

$s = 250.00$

V_{s2} is multiplied by $\text{Col2} = 0.00$

$s/d = 1.04167$

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

From (11-11), ACI 440: $V_s + V_f \leq 440464.828$

$b_w \cdot d = \cdot d \cdot d/4 = 125663.706$

Calculation of Shear Strength at edge 2, $V_{r2} = 516365.617$

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

$V_{ColO} = 516365.617$

$k_n l = 1$ (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)
Mean concrete strength: $f_c' = (f_c'_{\text{jacket}} \cdot \text{Area}_{\text{jacket}} + f_c'_{\text{core}} \cdot \text{Area}_{\text{core}}) / \text{Area}_{\text{section}} = 27.84$, but $f_c'^{0.5} \leq 8.3$ MPa (22.5.3.1, ACI 318-14)
 $M/Vd = 2.00$
 $\mu_u = 1.6653780\text{E-}011$
 $\mu_v = 3.5877337\text{E-}031$
 $d = 0.8 \cdot D = 400.00$
 $N_u = 7389.214$
 $A_g = 196349.541$
From (11.5.4.8), ACI 318-14: $V_s = V_{s1} + V_{s2} = 308425.138$
 $V_{s1} = 308425.138$ is calculated for jacket, with:
 $A_v = \sqrt{2} \cdot A_{\text{stirrup}} = 123370.055$
 $f_y = 625.00$
 $s = 100.00$
 V_{s1} is multiplied by $\text{Col1} = 1.00$
 $s/d = 0.25$
 $V_{s2} = 0.00$ is calculated for core, with:
 $A_v = \sqrt{2} \cdot A_{\text{stirrup}} = 78956.835$
 $f_y = 525.00$
 $s = 250.00$
 V_{s2} is multiplied by $\text{Col2} = 0.00$
 $s/d = 1.04167$
 $V_f ((11-3)-(11.4), \text{ACI } 440) = 0.00$
From (11-11), ACI 440: $V_s + V_f \leq 440464.828$
 $b_w \cdot d = \sqrt{2} \cdot d \cdot d / 4 = 125663.706$

End Of Calculation of Shear Capacity ratio for element: column JCC1 of floor 1
At local axis: 3

Start Of Calculation of Shear Capacity ratio for element: column JCC1 of floor 1
At Shear local axis: 2
(Bending local axis: 3)
Section Type: rcjcs

Constant Properties

Knowledge Factor, $\phi = 0.80$
Mean strength values are used for both shear and moment calculations.
Consequently:
Jacket
New material of Secondary Member: Concrete Strength, $f_c = f_{cm} = 30.00$
New material of Secondary Member: Steel Strength, $f_s = f_{sm} = 625.00$
Concrete Elasticity, $E_c = 25742.96$
Steel Elasticity, $E_s = 200000.00$
Existing Column
Existing material of Secondary Member: Concrete Strength, $f_c = f_{cm} = 24.00$
Existing material of Secondary Member: Steel Strength, $f_s = f_{sm} = 525.00$
Concrete Elasticity, $E_c = 23025.204$
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
Jacket
New material: Steel Strength, $f_s = 1.25 \cdot f_{sm} = 781.25$
Existing Column
Existing material: Steel Strength, $f_s = 1.25 \cdot f_{sm} = 656.25$

External Diameter, $D = 500.00$
Internal Diameter, $D = 300.00$
Cover Thickness, $c = 25.00$
Mean Confinement Factor overall section = 1.00
Element Length, $L = 3000.00$
Secondary Member

Smooth Bars
 Ductile Steel
 Without Detailing for Earthquake Resistance (including stirrups not 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, $V_a = -7.8886091E-031$
 EDGE -B-
 Shear Force, $V_b = 7.8886091E-031$
 BOTH EDGES
 Axial Force, $F = -7389.214$
 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, $V_e/V_r = 0.5552125$
 Member Controlled by Flexure ($V_e/V_r < 1$)
 Calculation of Shear Demand from fig. R18.6.5, ACI 318-14 $V_e = (M_{pr1} + M_{pr2})/l_n = 286692.647$
 with
 $M_{pr1} = \max(\mu_{u1+}, \mu_{u1-}) = 4.3004E+008$
 $\mu_{u1+} = 4.3004E+008$, 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
 $\mu_{u1-} = 4.3004E+008$, 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
 $M_{pr2} = \max(\mu_{u2+}, \mu_{u2-}) = 4.3004E+008$
 $\mu_{u2+} = 4.3004E+008$, 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
 $\mu_{u2-} = 4.3004E+008$, 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
 $\mu_u = 4.3004E+008$

$\beta_1 = 1.0821$
 $\beta_1 = 0.95748645$
 error of function (3.68), Biskinis Phd = 71900.796
 From 5A.2, TBDY: $f_{cc} = f_c \cdot \gamma_c = 30.00$
 conf. factor $\gamma_c = 1.00$
 $f_c = 30.00$
 From 10.3.5, ASCE 41-17, Final value of f_y : $f_y \cdot \min(1, 1.25 \cdot (l_b/l_d)^{2/3}) = 781.25$
 $l_b/l_d = 1.00$
 $d_1 = 44.00$
 $R = 250.00$
 $v = 0.00125383$
 $N = 7389.214$
 $A_c = 196349.541$
 $= \gamma_c \cdot \min(1, 1.25 \cdot (l_b/l_d)^{2/3}) = 0.324$

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
 $\mu = 4.3004E+008$

$$= 1.0821$$

$$' = 0.95748645$$

error of function (3.68), Biskinis Phd = 71900.796

From 5A.2, TDY: $f_{cc} = f_c \cdot c = 30.00$

conf. factor $c = 1.00$

$$f_c = 30.00$$

From 10.3.5, ASCE 41-17, Final value of f_y : $f_y \cdot \text{Min}(1, 1.25 \cdot (l_b/d)^{2/3}) = 781.25$

$$l_b/d = 1.00$$

$$d_1 = 44.00$$

$$R = 250.00$$

$$v = 0.00125383$$

$$N = 7389.214$$

$$A_c = 196349.541$$

$$= \cdot \text{Min}(1, 1.25 \cdot (l_b/d)^{2/3}) = 0.324$$

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
 $\mu = 4.3004E+008$

$$= 1.0821$$

$$' = 0.95748645$$

error of function (3.68), Biskinis Phd = 71900.796

From 5A.2, TDY: $f_{cc} = f_c \cdot c = 30.00$

conf. factor $c = 1.00$

$$f_c = 30.00$$

From 10.3.5, ASCE 41-17, Final value of f_y : $f_y \cdot \text{Min}(1, 1.25 \cdot (l_b/d)^{2/3}) = 781.25$

$$l_b/d = 1.00$$

$$d_1 = 44.00$$

$$R = 250.00$$

$$v = 0.00125383$$

$$N = 7389.214$$

$$A_c = 196349.541$$

$$= \cdot \text{Min}(1, 1.25 \cdot (l_b/d)^{2/3}) = 0.324$$

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
 $\mu = 4.3004E+008$

$\phi = 1.0821$
 $\phi' = 0.95748645$
error of function (3.68), Biskinis Phd = 71900.796
From 5A.2, TBDY: $f_{cc} = f_c' \cdot c = 30.00$
conf. factor $c = 1.00$
 $f_c = 30.00$
From 10.3.5, ASCE 41-17, Final value of f_y : $f_y \cdot \text{Min}(1, 1.25 \cdot (l_b/d)^{2/3}) = 781.25$
 $l_b/d = 1.00$
 $d_1 = 44.00$
 $R = 250.00$
 $v = 0.00125383$
 $N = 7389.214$
 $A_c = 196349.541$
 $= \phi' \cdot \text{Min}(1, 1.25 \cdot (l_b/d)^{2/3}) = 0.324$

Calculation of ratio l_b/d

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

Calculation of Shear Strength $V_r = \text{Min}(V_{r1}, V_{r2}) = 516365.617$

Calculation of Shear Strength at edge 1, $V_{r1} = 516365.617$

$V_{r1} = V_{col} \text{ ((10.3), ASCE 41-17)} = k_{nl} \cdot V_{col0}$
 $V_{col0} = 516365.617$
 $k_{nl} = 1$ (zero step-static loading)

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

$\phi = 1$ (normal-weight concrete)
Mean concrete strength: $f_c' = (f_c'_{\text{jacket}} \cdot \text{Area}_{\text{jacket}} + f_c'_{\text{core}} \cdot \text{Area}_{\text{core}}) / \text{Area}_{\text{section}} = 27.84$, but $f_c'^{0.5} \leq 8.3$
MPa (22.5.3.1, ACI 318-14)
 $M/Vd = 2.00$
 $\mu = 1.7578197E-011$
 $V_u = 7.8886091E-031$
 $d = 0.8 \cdot D = 400.00$
 $N_u = 7389.214$
 $A_g = 196349.541$
From (11.5.4.8), ACI 318-14: $V_s = V_{s1} + V_{s2} = 308425.138$
 $V_{s1} = 308425.138$ is calculated for jacket, with:
 $A_v = \phi / 2 \cdot A_{\text{stirrup}} = 123370.055$
 $f_y = 625.00$
 $s = 100.00$
 V_{s1} is multiplied by $\phi_{col1} = 1.00$
 $s/d = 0.25$
 $V_{s2} = 0.00$ is calculated for core, with:
 $A_v = \phi / 2 \cdot A_{\text{stirrup}} = 78956.835$
 $f_y = 525.00$
 $s = 250.00$
 V_{s2} is multiplied by $\phi_{col2} = 0.00$
 $s/d = 1.04167$
 $V_f \text{ ((11-3)-(11.4), ACI 440)} = 0.00$
From (11-11), ACI 440: $V_s + V_f \leq 440464.828$
 $b_w \cdot d = \phi \cdot d \cdot d / 4 = 125663.706$

Calculation of Shear Strength at edge 2, $V_{r2} = 516365.617$

$V_{r2} = V_{col} \text{ ((10.3), ASCE 41-17)} = k_{nl} \cdot V_{col0}$
 $V_{col0} = 516365.617$

knl = 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)

Mean concrete strength: $f_c' = (f_c'_{\text{jacket}} \cdot \text{Area}_{\text{jacket}} + f_c'_{\text{core}} \cdot \text{Area}_{\text{core}}) / \text{Area}_{\text{section}} = 27.84$, but $f_c'^{0.5} \leq 8.3$
MPa (22.5.3.1, ACI 318-14)

$M/Vd = 2.00$

$\mu_u = 1.7578197\text{E-}011$

$\nu_u = 7.8886091\text{E-}031$

$d = 0.8 \cdot D = 400.00$

$N_u = 7389.214$

$A_g = 196349.541$

From (11.5.4.8), ACI 318-14: $V_s = V_{s1} + V_{s2} = 308425.138$

$V_{s1} = 308425.138$ is calculated for jacket, with:

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

$f_y = 625.00$

$s = 100.00$

V_{s1} is multiplied by $\text{Col1} = 1.00$

$s/d = 0.25$

$V_{s2} = 0.00$ is calculated for core, with:

$A_v = \pi/2 \cdot A_{\text{stirrup}} = 78956.835$

$f_y = 525.00$

$s = 250.00$

V_{s2} is multiplied by $\text{Col2} = 0.00$

$s/d = 1.04167$

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

From (11-11), ACI 440: $V_s + V_f \leq 440464.828$

$b_w \cdot d = \pi \cdot d^2 / 4 = 125663.706$

End Of Calculation of Shear Capacity ratio for element: column JCC1 of floor 1
At local axis: 2

Start Of Calculation of Chord Rotation Capacity for element: column JCC1 of floor 1

At local axis: 3

Integration Section: (a)

Section Type: rcjcs

Constant Properties

Knowledge Factor, $\phi = 0.80$

Chord Rotation is generally considered as Deformation-Controlled Action according to Table C7-1, ASCE 41-17.

Mean strengths are used for Deformation-Controlled Actions according to 7.5.1.3, ASCE 41-17

Consequently:

Jacket

New material of Secondary Member: Concrete Strength, $f_c = f_{cm} = 30.00$

New material of Secondary Member: Steel Strength, $f_s = f_{sm} = 625.00$

Concrete Elasticity, $E_c = 25742.96$

Steel Elasticity, $E_s = 200000.00$

Existing Column

Existing material of Secondary Member: Concrete Strength, $f_c = f_{cm} = 24.00$

Existing material of Secondary Member: Steel Strength, $f_s = f_{sm} = 525.00$

Concrete Elasticity, $E_c = 23025.204$

Steel Elasticity, $E_s = 200000.00$

External Diameter, $D = 500.00$

Internal Diameter, $D = 300.00$

Cover Thickness, $c = 25.00$

Element Length, $L = 3000.00$

Secondary Member

Smooth Bars

Ductile Steel

Without Detailing for Earthquake Resistance (including stirrups not 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, $M = -2.6502E+007$
Shear Force, $V2 = -8831.613$
Shear Force, $V3 = -6.4786357E-013$
Axial Force, $F = -7385.681$
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, $Db_L = 18.00$

Existing component: From table 7-7, ASCE 41_17: Final chord rotation Capacity $u_R = \phi u = 0.04835644$
 $u = \gamma + p = 0.06044555$

- Calculation of γ -

$\gamma = (M \gamma_s / 3) / E_{eff} = 0.01738646$ ((4.29), Biskinis Phd))
 $M \gamma = 4.0620E+008$
 $L_s = M/V$ (with $L_s > 0.1 \cdot L$ and $L_s < 2 \cdot L$) = 3000.778
From table 10.5, ASCE 41_17: $E_{eff} = factor \cdot E_c \cdot I_g = 2.3369E+013$
 $factor = 0.30$
 $A_g = 196349.541$
Mean concrete strength: $fc' = (fc'_{jacket} \cdot Area_{jacket} + fc'_{core} \cdot Area_{core}) / Area_{section} = 27.84$
 $N = 7385.681$
 $E_c \cdot I_g = E_{c,jacket} \cdot I_{g,jacket} + E_{c,core} \cdot I_{g,core} = 7.7898E+013$

Calculation of Yielding Moment M_y

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

$M_y = \min(M_{y,ten}, M_{y,com}) = 4.0620E+008$
 γ ((10a) or (10b)) = 1.2225560E-005
 $M_{y,ten}$ (8a) = 4.0620E+008
 γ_{ten} (7a) = 67.28291
error of function (7a) = 0.00093798
 $M_{y,com}$ (8b) = 7.1091E+008
 γ_{com} (7b) = 65.27567
error of function (7b) = -0.00596426
with $e_y = 0.003125$
 $e_{co} = 0.002$
 $a_{pl} = 0.35$ ((9a) in Biskinis and Fardis for no FRP Wrap)
 $d_1 = 44.00$
 $R = 250.00$
 $v = 0.00125383$
 $N = 7385.681$
 $A_c = 196349.541$
 $= 0.324$
with $fc = 30.00$

Calculation of ratio l_b/l_d

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

- Calculation of p -

From table 10-9: $p = 0.04305909$

with:

- Columns not controlled by inadequate development or splicing along the clear height because $l_b/l_d \geq 1$

shear control ratio $V_y E / V_{Col0E} = 0.5552125$

$d = d_{external} = 0.00$

$s = s_{external} = 0.00$

$t = s_1 + s_2 + 2 \cdot t_f / b_w \cdot (f_{fe} / f_s) = 0.00323428$

jacket: $s_1 = A_{v1} \cdot (D_{c1} / 2) / (s_1 \cdot A_g) = 0.0027646$

$A_{v1} = 78.53982$, is the area of stirrup

$D_{c1} = D_{ext} - 2 \cdot \text{cover} - \text{External Hoop Diameter} = 440.00$, is the total Length of all stirrups parallel to loading (shear) direction

$s_1 = 100.00$

core: $s_2 = A_{v2} \cdot (D_{c2} / 2) / (s_2 \cdot A_g) = 0.00046968$

$A_{v2} = 50.26548$, is the area of stirrup

$D_{c2} = D_{int} - \text{Internal Hoop Diameter} = 292.00$, is the total Length of all stirrups parallel to loading (shear) direction

$s_2 = 250.00$

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.

For the normalisation f_s of jacket is used.

$NUD = 7385.681$

$A_g = 196349.541$

$f_{cE} = (f_{c,jacket} \cdot \text{Area}_{jacket} + f_{c,core} \cdot \text{Area}_{core}) / \text{section_area} = 27.84$

$f_{yIE} = (f_{y,ext_Long_Reinf} \cdot \text{Area}_{ext_Long_Reinf} + f_{y,int_Long_Reinf} \cdot \text{Area}_{int_Long_Reinf}) / \text{Area}_{Tot_Long_Rein} = 21219958E-314$

$f_{yIE} = (f_{y,ext_Trans_Reinf} \cdot \text{Area}_{ext_Trans_Reinf} + f_{y,int_Trans_Reinf} \cdot \text{Area}_{int_Trans_Reinf}) / \text{Area}_{Tot_Trans_Rein} = 610.4781$

$p_l = \text{Area}_{Tot_Long_Rein} / (A_g) = 0.015552$

$f_{cE} = 27.84$

End Of Calculation of Chord Rotation Capacity for element: column JCC1 of floor 1

At local axis: 3

Integration Section: (a)

Calculation No. 13

column C1, Floor 1

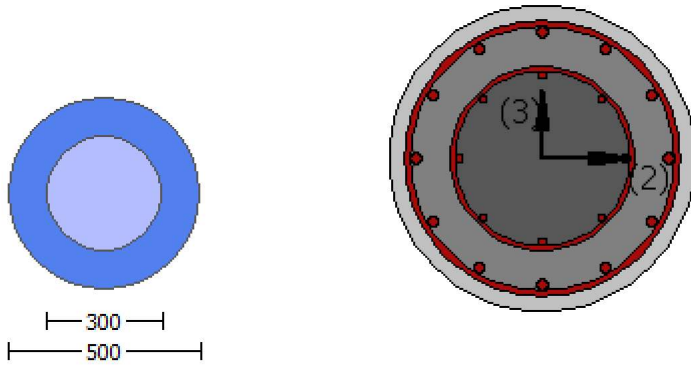
Limit State: Life Safety (data interpolation between analysis steps 1 and 2)

Analysis: Uniform +X

Check: Shear capacity V_{Rd}

Edge: End

Local Axis: (2)



Start Of Calculation of Shear Capacity for element: column JCC1 of floor 1

At local axis: 2

Integration Section: (b)

Section Type: rcjcs

Constant Properties

Knowledge Factor, $\gamma = 0.80$

Member Shear Force is generally considered as Force-Controlled Action according to Table C7-1, ASCE 41-17.

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

Consequently:

Jacket

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

New material of Secondary Member: Steel Strength, $f_s = f_{s_lower_bound} = 500.00$

Concrete Elasticity, $E_c = 25742.96$

Steel Elasticity, $E_s = 200000.00$

Existing Column

Existing material of Secondary Member: Concrete Strength, $f_c = f_{c_lower_bound} = 16.00$

Existing material of Secondary Member: Steel Strength, $f_s = f_{s_lower_bound} = 420.00$

Concrete Elasticity, $E_c = 23025.204$

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, ASCE 41-17) because bending is considered as Deformation-Controlled Action (Table C7-1, ASCE 41-17).

Jacket

New material: Concrete Strength, $f_c = f_{cm} = 30.00$

New material: Steel Strength, $f_s = f_{sm} = 625.00$

Existing Column

Existing material: Concrete Strength, $f_c = f_{cm} = 24.00$

Existing material: Steel Strength, $f_s = f_{sm} = 525.00$

#####

External Diameter, $D = 500.00$

Internal Diameter, $D = 300.00$

Cover Thickness, $c = 25.00$

Element Length, $L = 3000.00$

Secondary Member

Smooth Bars

Ductile Steel

Without Detailing for Earthquake Resistance (including stirrups not 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, $M_a = -2.6502E+007$

Shear Force, $V_a = -8831.613$

EDGE -B-

Bending Moment, $M_b = -0.03977224$

Shear Force, $V_b = 8831.613$
 BOTH EDGES
 Axial Force, $F = -7385.681$
 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, $Db_{L,ten} = 18.00$

Existing component: From table 7-7, ASCE 41_17: Final Shear Capacity $VR = \phi V_n = 333432.925$
 $V_n ((10.3), ASCE 41-17) = knl \cdot V_{Col0} = 416791.156$
 $V_{Col} = 416791.156$
 $knl = 1.00$
 $displacement_ductility_demand = 0.09785341$

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

$\phi = 1$ (normal-weight concrete)
 Mean concrete strength: $fc' = (fc'_{jacket} \cdot Area_{jacket} + fc'_{core} \cdot Area_{core}) / Area_{section} = 18.56$, but $fc'^{0.5} \leq 8.3$
 MPa (22.5.3.1, ACI 318-14)
 $M/Vd = 2.00$
 $\mu_u = 0.03977224$
 $V_u = 8831.613$
 $d = 0.8 \cdot D = 400.00$
 $N_u = 7385.681$
 $A_g = 196349.541$
 From (11.5.4.8), ACI 318-14: $V_s = V_{s1} + V_{s2} = 246740.11$
 $V_{s1} = 246740.11$ is calculated for jacket, with:
 $A_v = \phi / 2 \cdot A_{stirrup} = 123370.055$
 $f_y = 500.00$
 $s = 100.00$
 V_{s1} is multiplied by $Col1 = 1.00$
 $s/d = 0.25$
 $V_{s2} = 0.00$ is calculated for core, with:
 $A_v = \phi / 2 \cdot A_{stirrup} = 78956.835$
 $f_y = 420.00$
 $s = 250.00$
 V_{s2} is multiplied by $Col2 = 0.00$
 $s/d = 1.04167$
 $V_f ((11-3)-(11.4), ACI 440) = 0.00$
 From (11-11), ACI 440: $V_s + V_f \leq 359638.026$
 $b_w \cdot d = \phi \cdot d \cdot d / 4 = 125663.706$

$displacement_ductility_demand$ is calculated as ϕ / y

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

From analysis, chord rotation $\phi = 0.00017009$
 $y = (M_y \cdot L_s / 3) / Eleff = 0.0017382 ((4.29), Biskinis Phd)$
 $M_y = 4.0620E+008$
 $L_s = M/V$ (with $L_s > 0.1 \cdot L$ and $L_s < 2 \cdot L$) = 300.00
 From table 10.5, ASCE 41_17: $Eleff = factor \cdot E_c \cdot I_g = 2.3369E+013$
 $factor = 0.30$
 $A_g = 196349.541$
 Mean concrete strength: $fc' = (fc'_{jacket} \cdot Area_{jacket} + fc'_{core} \cdot Area_{core}) / Area_{section} = 27.84$
 $N = 7385.681$
 $E_c \cdot I_g = E_{c,jacket} \cdot I_{g,jacket} + E_{c,core} \cdot I_{g,core} = 7.7898E+013$

Calculation of Yielding Moment M_y

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

$M_y = \min(M_{y_ten}, M_{y_com}) = 4.0620E+008$

$\rho_y ((10a) \text{ or } (10b)) = 1.2225560E-005$

$M_{y_ten} (8a) = 4.0620E+008$

$\rho_{y_ten} (7a) = 67.28291$

error of function (7a) = 0.00093798

$M_{y_com} (8b) = 7.1091E+008$

$\rho_{y_com} (7b) = 65.27567$

error of function (7b) = -0.00596426

with $e_y = 0.003125$

$e_{co} = 0.002$

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

$d_1 = 44.00$

$R = 250.00$

$v = 0.00125383$

$N = 7385.681$

$A_c = 196349.541$

$= 0.324$

with $f_c = 30.00$

Calculation of ratio l_b/d

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

End Of Calculation of Shear Capacity for element: column JCC1 of floor 1

At local axis: 2

Integration Section: (b)

Calculation No. 14

column C1, Floor 1

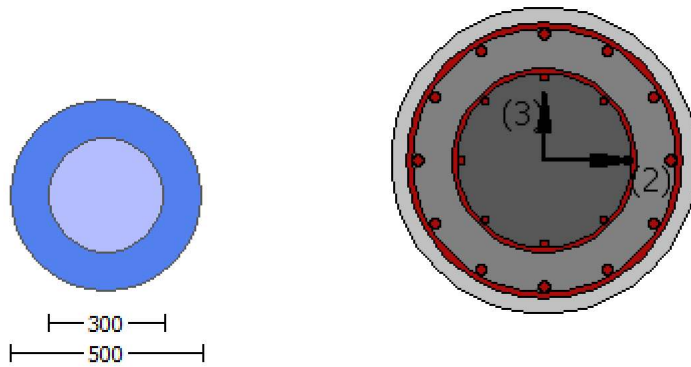
Limit State: Life Safety (data interpolation between analysis steps 1 and 2)

Analysis: Uniform +X

Check: Chord rotation capacity (ϕ_u)

Edge: End

Local Axis: (2)



Start Of Calculation of Shear Capacity ratio for element: column JCC1 of floor 1

At Shear local axis: 3

(Bending local axis: 2)

Section Type: rcjcs

Constant Properties

Knowledge Factor, $\gamma = 0.80$

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

Consequently:

Jacket

New material of Secondary Member: Concrete Strength, $f_c = f_{cm} = 30.00$

New material of Secondary Member: Steel Strength, $f_s = f_{sm} = 625.00$

Concrete Elasticity, $E_c = 25742.96$

Steel Elasticity, $E_s = 200000.00$

Existing Column

Existing material of Secondary Member: Concrete Strength, $f_c = f_{cm} = 24.00$

Existing material of Secondary Member: Steel Strength, $f_s = f_{sm} = 525.00$

Concrete Elasticity, $E_c = 23025.204$

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

Jacket

New material: Steel Strength, $f_s = 1.25 \cdot f_{sm} = 781.25$

Existing Column

Existing material: Steel Strength, $f_s = 1.25 \cdot f_{sm} = 656.25$

#####

External Diameter, $D = 500.00$

Internal Diameter, $D = 300.00$

Cover Thickness, $c = 25.00$

Mean Confinement Factor overall section = 1.00

Element Length, $L = 3000.00$

Secondary Member

Smooth Bars

Ductile Steel

Without Detailing for Earthquake Resistance (including stirrups not 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, $V_a = 3.5877337E-031$

EDGE -B-

Shear Force, $V_b = -3.5877337E-031$

BOTH EDGES

Axial Force, $F = -7389.214$

Longitudinal Reinforcement Area Distribution (in 2 divisions)

-Tension: $Asl_t = 0.00$
 -Compression: $Asl_c = 3053.628$
 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, $V_e/V_r = 0.5552125$
 Member Controlled by Flexure ($V_e/V_r < 1$)
 Calculation of Shear Demand from fig. R18.6.5, ACI 318-14 $V_e = (M_{pr1} + M_{pr2})/l_n = 286692.647$
 with

$M_{pr1} = \text{Max}(\mu_{u1+}, \mu_{u1-}) = 4.3004E+008$
 $\mu_{u1+} = 4.3004E+008$, 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
 $\mu_{u1-} = 4.3004E+008$, 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
 $M_{pr2} = \text{Max}(\mu_{u2+}, \mu_{u2-}) = 4.3004E+008$
 $\mu_{u2+} = 4.3004E+008$, 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
 $\mu_{u2-} = 4.3004E+008$, 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
 $\mu_u = 4.3004E+008$

$\phi = 1.0821$
 $\phi' = 0.95748645$
 error of function (3.68), Biskinis Phd = 71900.796
 From 5A.2, TB DY: $f_{cc} = f_c' \cdot c = 30.00$
 conf. factor $c = 1.00$
 $f_c = 30.00$
 From 10.3.5, ASCE 41-17, Final value of f_y : $f_y \cdot \text{Min}(1, 1.25 \cdot (l_b/d)^{2/3}) = 781.25$
 $l_b/d = 1.00$
 $d_1 = 44.00$
 $R = 250.00$
 $v = 0.00125383$
 $N = 7389.214$
 $A_c = 196349.541$
 $\phi' \cdot \text{Min}(1, 1.25 \cdot (l_b/d)^{2/3}) = 0.324$

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
 $\mu_u = 4.3004E+008$

$\phi = 1.0821$
 $\phi' = 0.95748645$
 error of function (3.68), Biskinis Phd = 71900.796
 From 5A.2, TB DY: $f_{cc} = f_c' \cdot c = 30.00$
 conf. factor $c = 1.00$

$f_c = 30.00$
 From 10.3.5, ASCE 41-17, Final value of f_y : $f_y \cdot \text{Min}(1, 1.25 \cdot (l_b/d)^{2/3}) = 781.25$
 $l_b/d = 1.00$
 $d_1 = 44.00$
 $R = 250.00$
 $v = 0.00125383$
 $N = 7389.214$
 $A_c = 196349.541$
 $= \cdot \text{Min}(1, 1.25 \cdot (l_b/d)^{2/3}) = 0.324$

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
 $\mu = 4.3004E+008$

$= 1.0821$
 $' = 0.95748645$
 error of function (3.68), Biskinis Phd = 71900.796
 From 5A.2, TBDY: $f_{cc} = f_c \cdot c = 30.00$
 conf. factor $c = 1.00$
 $f_c = 30.00$
 From 10.3.5, ASCE 41-17, Final value of f_y : $f_y \cdot \text{Min}(1, 1.25 \cdot (l_b/d)^{2/3}) = 781.25$
 $l_b/d = 1.00$
 $d_1 = 44.00$
 $R = 250.00$
 $v = 0.00125383$
 $N = 7389.214$
 $A_c = 196349.541$
 $= \cdot \text{Min}(1, 1.25 \cdot (l_b/d)^{2/3}) = 0.324$

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
 $\mu = 4.3004E+008$

$= 1.0821$
 $' = 0.95748645$
 error of function (3.68), Biskinis Phd = 71900.796
 From 5A.2, TBDY: $f_{cc} = f_c \cdot c = 30.00$
 conf. factor $c = 1.00$
 $f_c = 30.00$
 From 10.3.5, ASCE 41-17, Final value of f_y : $f_y \cdot \text{Min}(1, 1.25 \cdot (l_b/d)^{2/3}) = 781.25$
 $l_b/d = 1.00$
 $d_1 = 44.00$
 $R = 250.00$
 $v = 0.00125383$
 $N = 7389.214$
 $A_c = 196349.541$
 $= \cdot \text{Min}(1, 1.25 \cdot (l_b/d)^{2/3}) = 0.324$

Calculation of ratio l_b/d

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

Calculation of Shear Strength $V_r = \text{Min}(V_{r1}, V_{r2}) = 516365.617$

Calculation of Shear Strength at edge 1, $V_{r1} = 516365.617$

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

$V_{Col0} = 516365.617$

$k_{nl} = 1$ (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)

Mean concrete strength: $f'_c = (f'_{c_jacket} * \text{Area}_{jacket} + f'_{c_core} * \text{Area}_{core}) / \text{Area}_{section} = 27.84$, but $f'_c^{0.5} \leq 8.3$ MPa (22.5.3.1, ACI 318-14)

$M/Vd = 2.00$

$\mu_u = 1.6653780E-011$

$\nu_u = 3.5877337E-031$

$d = 0.8 * D = 400.00$

$N_u = 7389.214$

$A_g = 196349.541$

From (11.5.4.8), ACI 318-14: $V_s = V_{s1} + V_{s2} = 308425.138$

$V_{s1} = 308425.138$ is calculated for jacket, with:

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

$f_y = 625.00$

$s = 100.00$

V_{s1} is multiplied by $\text{Col1} = 1.00$

$s/d = 0.25$

$V_{s2} = 0.00$ is calculated for core, with:

$A_v = \sqrt{2} * A_{stirrup} = 78956.835$

$f_y = 525.00$

$s = 250.00$

V_{s2} is multiplied by $\text{Col2} = 0.00$

$s/d = 1.04167$

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

From (11-11), ACI 440: $V_s + V_f \leq 440464.828$

$b_w * d = \sqrt{2} * d^2 / 4 = 125663.706$

Calculation of Shear Strength at edge 2, $V_{r2} = 516365.617$

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

$V_{Col0} = 516365.617$

$k_{nl} = 1$ (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)

Mean concrete strength: $f'_c = (f'_{c_jacket} * \text{Area}_{jacket} + f'_{c_core} * \text{Area}_{core}) / \text{Area}_{section} = 27.84$, but $f'_c^{0.5} \leq 8.3$ MPa (22.5.3.1, ACI 318-14)

$M/Vd = 2.00$

$\mu_u = 1.6653780E-011$

$\nu_u = 3.5877337E-031$

$d = 0.8 * D = 400.00$

$N_u = 7389.214$

$A_g = 196349.541$

From (11.5.4.8), ACI 318-14: $V_s = V_{s1} + V_{s2} = 308425.138$

$V_{s1} = 308425.138$ is calculated for jacket, with:

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

$f_y = 625.00$

$s = 100.00$
 V_{s1} is multiplied by $Col1 = 1.00$
 $s/d = 0.25$
 $V_{s2} = 0.00$ is calculated for core, with:
 $A_v = \sqrt{2} \cdot A_{stirrup} = 78956.835$
 $f_y = 525.00$
 $s = 250.00$
 V_{s2} is multiplied by $Col2 = 0.00$
 $s/d = 1.04167$
 $V_f ((11-3)-(11.4), ACI 440) = 0.00$
 From (11-11), ACI 440: $V_s + V_f \leq 440464.828$
 $b_w \cdot d = \sqrt{d} \cdot d / 4 = 125663.706$

End Of Calculation of Shear Capacity ratio for element: column JCC1 of floor 1
 At local axis: 3

Start Of Calculation of Shear Capacity ratio for element: column JCC1 of floor 1
 At Shear local axis: 2
 (Bending local axis: 3)
 Section Type: rcjcs

Constant Properties

Knowledge Factor, $\phi = 0.80$
 Mean strength values are used for both shear and moment calculations.
 Consequently:
 Jacket
 New material of Secondary Member: Concrete Strength, $f_c = f_{cm} = 30.00$
 New material of Secondary Member: Steel Strength, $f_s = f_{sm} = 625.00$
 Concrete Elasticity, $E_c = 25742.96$
 Steel Elasticity, $E_s = 200000.00$
 Existing Column
 Existing material of Secondary Member: Concrete Strength, $f_c = f_{cm} = 24.00$
 Existing material of Secondary Member: Steel Strength, $f_s = f_{sm} = 525.00$
 Concrete Elasticity, $E_c = 23025.204$
 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
 Jacket
 New material: Steel Strength, $f_s = 1.25 \cdot f_{sm} = 781.25$
 Existing Column
 Existing material: Steel Strength, $f_s = 1.25 \cdot f_{sm} = 656.25$
 #####
 External Diameter, $D = 500.00$
 Internal Diameter, $D = 300.00$
 Cover Thickness, $c = 25.00$
 Mean Confinement Factor overall section = 1.00
 Element Length, $L = 3000.00$
 Secondary Member
 Smooth Bars
 Ductile Steel
 Without Detailing for Earthquake Resistance (including stirrups not 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, $V_a = -7.8886091E-031$

EDGE -B-

Shear Force, $V_b = 7.8886091\text{E-}031$

BOTH EDGES

Axial Force, $F = -7389.214$

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: $A_{st,mid} = 1017.876$

Calculation of Shear Capacity ratio, $V_e/V_r = 0.5552125$

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

Calculation of Shear Demand from fig. R18.6.5, ACI 318-14 $V_e = (M_{pr1} + M_{pr2})/l_n = 286692.647$ with

$M_{pr1} = \text{Max}(M_{u1+}, M_{u1-}) = 4.3004\text{E}+008$

$M_{u1+} = 4.3004\text{E}+008$, 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

$M_{u1-} = 4.3004\text{E}+008$, 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

$M_{pr2} = \text{Max}(M_{u2+}, M_{u2-}) = 4.3004\text{E}+008$

$M_{u2+} = 4.3004\text{E}+008$, 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

$M_{u2-} = 4.3004\text{E}+008$, 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

$M_u = 4.3004\text{E}+008$

$\phi = 1.0821$

$\phi' = 0.95748645$

error of function (3.68), Biskinis Phd = 71900.796

From 5A.2, TBDY: $f_{cc} = f_c' \cdot c = 30.00$

conf. factor $c = 1.00$

$f_c = 30.00$

From 10.3.5, ASCE 41-17, Final value of f_y : $f_y \cdot \text{Min}(1, 1.25 \cdot (l_b/l_d)^{2/3}) = 781.25$

$l_b/l_d = 1.00$

$d_1 = 44.00$

$R = 250.00$

$v = 0.00125383$

$N = 7389.214$

$A_c = 196349.541$

$\phi' \cdot \text{Min}(1, 1.25 \cdot (l_b/l_d)^{2/3}) = 0.324$

Calculation of ratio l_b/l_d

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

Calculation of M_{u1-}

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

$M_u = 4.3004\text{E}+008$

```

= 1.0821
' = 0.95748645
error of function (3.68), Biskinis Phd = 71900.796
From 5A.2, TBDY: fcc = fc* c = 30.00
  conf. factor c = 1.00
  fc = 30.00
From 10.3.5, ASCE 41-17, Final value of fy: fy*Min(1,1.25*(lb/d)^ 2/3) = 781.25
  lb/d = 1.00
  d1 = 44.00
  R = 250.00
  v = 0.00125383
  N = 7389.214
  Ac = 196349.541
  = *Min(1,1.25*(lb/d)^ 2/3) = 0.324

```

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 = 4.3004E+008

```

= 1.0821
' = 0.95748645
error of function (3.68), Biskinis Phd = 71900.796
From 5A.2, TBDY: fcc = fc* c = 30.00
  conf. factor c = 1.00
  fc = 30.00
From 10.3.5, ASCE 41-17, Final value of fy: fy*Min(1,1.25*(lb/d)^ 2/3) = 781.25
  lb/d = 1.00
  d1 = 44.00
  R = 250.00
  v = 0.00125383
  N = 7389.214
  Ac = 196349.541
  = *Min(1,1.25*(lb/d)^ 2/3) = 0.324

```

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 = 4.3004E+008

```

= 1.0821
' = 0.95748645
error of function (3.68), Biskinis Phd = 71900.796
From 5A.2, TBDY: fcc = fc* c = 30.00
  conf. factor c = 1.00
  fc = 30.00
From 10.3.5, ASCE 41-17, Final value of fy: fy*Min(1,1.25*(lb/d)^ 2/3) = 781.25
  lb/d = 1.00
  d1 = 44.00

```

$R = 250.00$
 $v = 0.00125383$
 $N = 7389.214$
 $Ac = 196349.541$
 $= \text{Min}(1, 1.25 \cdot (lb/d)^{2/3}) = 0.324$

Calculation of ratio lb/d

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

Calculation of Shear Strength $V_r = \text{Min}(V_{r1}, V_{r2}) = 516365.617$

Calculation of Shear Strength at edge 1, $V_{r1} = 516365.617$

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

$V_{ColO} = 516365.617$

$knl = 1$ (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)

Mean concrete strength: $f'_c = (f'_c \cdot \text{jacket} \cdot \text{Area}_{\text{jacket}} + f'_c \cdot \text{core} \cdot \text{Area}_{\text{core}}) / \text{Area}_{\text{section}} = 27.84$, but $f'_c^{0.5} \leq 8.3$ MPa (22.5.3.1, ACI 318-14)

$M/Vd = 2.00$

$\mu_u = 1.7578197E-011$

$\nu_u = 7.8886091E-031$

$d = 0.8 \cdot D = 400.00$

$N_u = 7389.214$

$A_g = 196349.541$

From (11.5.4.8), ACI 318-14: $V_s = V_{s1} + V_{s2} = 308425.138$

$V_{s1} = 308425.138$ is calculated for jacket, with:

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

$f_y = 625.00$

$s = 100.00$

V_{s1} is multiplied by $\text{Col1} = 1.00$

$s/d = 0.25$

$V_{s2} = 0.00$ is calculated for core, with:

$A_v = \pi/2 \cdot A_{\text{stirrup}} = 78956.835$

$f_y = 525.00$

$s = 250.00$

V_{s2} is multiplied by $\text{Col2} = 0.00$

$s/d = 1.04167$

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

From (11-11), ACI 440: $V_s + V_f \leq 440464.828$

$b_w \cdot d = \pi \cdot d^2 / 4 = 125663.706$

Calculation of Shear Strength at edge 2, $V_{r2} = 516365.617$

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

$V_{ColO} = 516365.617$

$knl = 1$ (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)

Mean concrete strength: $f'_c = (f'_c \cdot \text{jacket} \cdot \text{Area}_{\text{jacket}} + f'_c \cdot \text{core} \cdot \text{Area}_{\text{core}}) / \text{Area}_{\text{section}} = 27.84$, but $f'_c^{0.5} \leq 8.3$ MPa (22.5.3.1, ACI 318-14)

$M/Vd = 2.00$

$\mu_u = 1.7578197E-011$

$\nu_u = 7.8886091E-031$

$d = 0.8 \cdot D = 400.00$

$N_u = 7389.214$

$A_g = 196349.541$
 From (11.5.4.8), ACI 318-14: $V_s = V_{s1} + V_{s2} = 308425.138$
 $V_{s1} = 308425.138$ is calculated for jacket, with:
 $A_v = \frac{1}{2} A_{stirrup} = 123370.055$
 $f_y = 625.00$
 $s = 100.00$
 V_{s1} is multiplied by $Col1 = 1.00$
 $s/d = 0.25$
 $V_{s2} = 0.00$ is calculated for core, with:
 $A_v = \frac{1}{2} A_{stirrup} = 78956.835$
 $f_y = 525.00$
 $s = 250.00$
 V_{s2} is multiplied by $Col2 = 0.00$
 $s/d = 1.04167$
 $V_f ((11-3)-(11.4), ACI 440) = 0.00$
 From (11-11), ACI 440: $V_s + V_f \leq 440464.828$
 $b_w * d = \frac{1}{4} * d * d = 125663.706$

End Of Calculation of Shear Capacity ratio for element: column JCC1 of floor 1
 At local axis: 2

Start Of Calculation of Chord Rotation Capacity for element: column JCC1 of floor 1
 At local axis: 2
 Integration Section: (b)
 Section Type: rcjcs

Constant Properties

Knowledge Factor, $\phi = 0.80$
 Chord Rotation is generally considered as Deformation-Controlled Action according to Table C7-1, ASCE 41-17.
 Mean strengths are used for Deformation-Controlled Actions according to 7.5.1.3, ASCE 41-17
 Consequently:
 Jacket
 New material of Secondary Member: Concrete Strength, $f_c = f_{cm} = 30.00$
 New material of Secondary Member: Steel Strength, $f_s = f_{sm} = 625.00$
 Concrete Elasticity, $E_c = 25742.96$
 Steel Elasticity, $E_s = 200000.00$
 Existing Column
 Existing material of Secondary Member: Concrete Strength, $f_c = f_{cm} = 24.00$
 Existing material of Secondary Member: Steel Strength, $f_s = f_{sm} = 525.00$
 Concrete Elasticity, $E_c = 23025.204$
 Steel Elasticity, $E_s = 200000.00$
 External Diameter, $D = 500.00$
 Internal Diameter, $D = 300.00$
 Cover Thickness, $c = 25.00$
 Element Length, $L = 3000.00$
 Secondary Member
 Smooth Bars
 Ductile Steel
 Without Detailing for Earthquake Resistance (including stirrups not 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, $M = 1.0755087E-010$
 Shear Force, $V2 = 8831.613$
 Shear Force, $V3 = 6.4786357E-013$
 Axial Force, $F = -7385.681$
 Longitudinal Reinforcement Area Distribution (in 2 divisions)
 -Tension: $As_t = 0.00$

-Compression: $Asl_c = 3053.628$
 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, $DbL = 18.00$

Existing component: From table 7-7, ASCE 41_17: Final chord rotation Capacity $u_R = * u = 0.04140005$
 $u = y + p = 0.05175007$

- Calculation of y -

$y = (My * L_s / 3) / Eleff = 0.00869098$ ((4.29), Biskinis Phd))
 $My = 4.0620E+008$
 $L_s = M/V$ (with $L_s > 0.1 * L$ and $L_s < 2 * L$) = 1500.00
 From table 10.5, ASCE 41_17: $Eleff = factor * Ec * I_g = 2.3369E+013$
 $factor = 0.30$
 $Ag = 196349.541$
 Mean concrete strength: $fc' = (fc'_{jacket} * Area_{jacket} + fc'_{core} * Area_{core}) / Area_{section} = 27.84$
 $N = 7385.681$
 $Ec * I_g = Ec_{jacket} * I_{g,jacket} + Ec_{core} * I_{g,core} = 7.7898E+013$

Calculation of Yielding Moment My

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

$My = \min(My_{ten}, My_{com}) = 4.0620E+008$
 y ((10a) or (10b)) = 1.2225560E-005
 My_{ten} (8a) = 4.0620E+008
 $_{ten}$ (7a) = 67.28291
 error of function (7a) = 0.00093798
 My_{com} (8b) = 7.1091E+008
 $_{com}$ (7b) = 65.27567
 error of function (7b) = -0.00596426
 with $ey = 0.003125$
 $eco = 0.002$
 $apl = 0.35$ ((9a) in Biskinis and Fardis for no FRP Wrap)
 $d1 = 44.00$
 $R = 250.00$
 $v = 0.00125383$
 $N = 7385.681$
 $Ac = 196349.541$
 $= 0.324$
 with $fc = 30.00$

Calculation of ratio lb/d

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

- Calculation of p -

From table 10-9: $p = 0.04305909$

with:

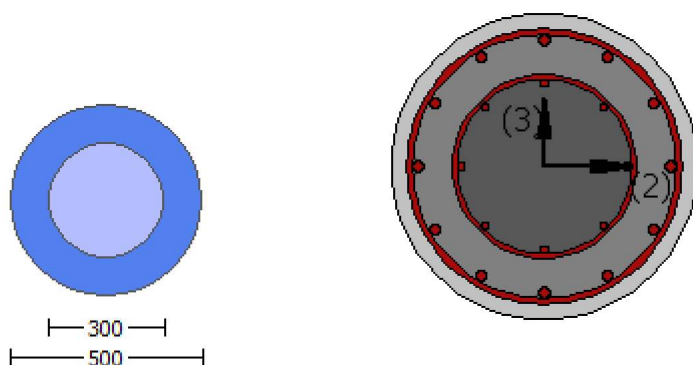
- Columns not controlled by inadequate development or splicing along the clear height because $lb/d \geq 1$
 shear control ratio $VyE/VCoIE = 0.5552125$
 $d = d_{external} = 0.00$
 $s = s_{external} = 0.00$
 $t = s1 + s2 + 2 * tf / bw * (ffe / fs) = 0.00323428$
 jacket: $s1 = Av1 * (* Dc1 / 2) / (s1 * Ag) = 0.0027646$

$A_{v1} = 78.53982$, is the area of stirrup
 $D_{c1} = D_{ext} - 2 \cdot \text{cover} - \text{External Hoop Diameter} = 440.00$, is the total Length of all stirrups parallel to loading (shear) direction
 $s_1 = 100.00$
 core: $s_2 = A_{v2} \cdot (D_{c2}/2) / (s_2 \cdot A_g) = 0.00046968$
 $A_{v2} = 50.26548$, is the area of stirrup
 $D_{c2} = D_{int} - \text{Internal Hoop Diameter} = 292.00$, is the total Length of all stirrups parallel to loading (shear) direction
 $s_2 = 250.00$
 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. For the normalisation f_s of jacket is used.
 $N_{UD} = 7385.681$
 $A_g = 196349.541$
 $f_{cE} = (f_{c_jacket} \cdot \text{Area_jacket} + f_{c_core} \cdot \text{Area_core}) / \text{section_area} = 27.84$
 $f_{yE} = (f_{y_ext_Long_Reinf} \cdot \text{Area_ext_Long_Reinf} + f_{y_int_Long_Reinf} \cdot \text{Area_int_Long_Reinf}) / \text{Area_Tot_Long_Rein} = 2.1219958E-314$
 $f_{yE} = (f_{y_ext_Trans_Reinf} \cdot \text{Area_ext_Trans_Reinf} + f_{y_int_Trans_Reinf} \cdot \text{Area_int_Trans_Reinf}) / \text{Area_Tot_Trans_Rein} = 610.4781$
 $p_l = \text{Area_Tot_Long_Rein} / (A_g) = 0.015552$
 $f_{cE} = 27.84$

 End Of Calculation of Chord Rotation Capacity for element: column JCC1 of floor 1
 At local axis: 2
 Integration Section: (b)

Calculation No. 15

column C1, Floor 1
 Limit State: Life Safety (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 JCC1 of floor 1

At local axis: 3
Integration Section: (b)
Section Type: rcjs

Constant Properties

Knowledge Factor, $\gamma = 0.80$
Member Shear Force is generally considered as Force-Controlled Action according to Table C7-1, ASCE 41-17.
Lower-bound strengths are used for Force-Controlled Actions according to 7.5.1.3, ASCE 41-17
Consequently:
Jacket
New material of Secondary Member: Concrete Strength, $f_c = f_{c_lower_bound} = 20.00$
New material of Secondary Member: Steel Strength, $f_s = f_{s_lower_bound} = 500.00$
Concrete Elasticity, $E_c = 25742.96$
Steel Elasticity, $E_s = 200000.00$
Existing Column
Existing material of Secondary Member: Concrete Strength, $f_c = f_{c_lower_bound} = 16.00$
Existing material of Secondary Member: Steel Strength, $f_s = f_{s_lower_bound} = 420.00$
Concrete Elasticity, $E_c = 23025.204$
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, ASCE 41-17) because bending is considered as
Deformation-Controlled Action (Table C7-1, ASCE 41-17).
Jacket
New material: Concrete Strength, $f_c = f_{cm} = 30.00$
New material: Steel Strength, $f_s = f_{sm} = 625.00$
Existing Column
Existing material: Concrete Strength, $f_c = f_{cm} = 24.00$
Existing material: Steel Strength, $f_s = f_{sm} = 525.00$

External Diameter, $D = 500.00$
Internal Diameter, $D = 300.00$
Cover Thickness, $c = 25.00$
Element Length, $L = 3000.00$
Secondary Member
Smooth Bars
Ductile Steel
Without Detailing for Earthquake Resistance (including stirrups not 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, $M_a = 1.8364926E-009$
Shear Force, $V_a = -6.4786357E-013$
EDGE -B-
Bending Moment, $M_b = 1.0755087E-010$
Shear Force, $V_b = 6.4786357E-013$
BOTH EDGES
Axial Force, $F = -7385.681$
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: $As_{l,mid} = 1017.876$
Mean Diameter of Tension Reinforcement, $Db_{L,ten} = 18.00$

Existing component: From table 7-7, ASCE 41_17: Final Shear Capacity $VR = *V_n = 333432.925$

$V_n ((10.3), ASCE 41-17) = knl * V_{ColO} = 416791.156$

$V_{Col} = 416791.156$

$knl = 1.00$

$displacement_ductility_demand = 0.00$

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)

Mean concrete strength: $fc' = (fc'_{jacket} * Area_{jacket} + fc'_{core} * Area_{core}) / Area_{section} = 18.56$, but $fc'^{0.5} \leq 8.3$ MPa (22.5.3.1, ACI 318-14)

$M/Vd = 2.00$

$\mu_u = 1.0755087E-010$

$\nu_u = 6.4786357E-013$

$d = 0.8 * D = 400.00$

$N_u = 7385.681$

$A_g = 196349.541$

From (11.5.4.8), ACI 318-14: $V_s = V_{s1} + V_{s2} = 246740.11$

$V_{s1} = 246740.11$ is calculated for jacket, with:

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

$f_y = 500.00$

$s = 100.00$

V_{s1} is multiplied by $Col1 = 1.00$

$s/d = 0.25$

$V_{s2} = 0.00$ is calculated for core, with:

$A_v = /2 * A_{stirrup} = 78956.835$

$f_y = 420.00$

$s = 250.00$

V_{s2} is multiplied by $Col2 = 0.00$

$s/d = 1.04167$

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

From (11-11), ACI 440: $V_s + V_f \leq 359638.026$

$bw * d = *d * d / 4 = 125663.706$

$displacement_ductility_demand$ is calculated as $/ y$

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

From analysis, chord rotation $= 1.0262220E-020$

$y = (M_y * L_s / 3) / E_{eff} = 0.00869098 ((4.29), Biskinis Phd)$

$M_y = 4.0620E+008$

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

From table 10.5, ASCE 41_17: $E_{eff} = factor * E_c * I_g = 2.3369E+013$

$factor = 0.30$

$A_g = 196349.541$

Mean concrete strength: $fc' = (fc'_{jacket} * Area_{jacket} + fc'_{core} * Area_{core}) / Area_{section} = 27.84$

$N = 7385.681$

$E_c * I_g = E_{c_jacket} * I_{g_jacket} + E_{c_core} * I_{g_core} = 7.7898E+013$

Calculation of Yielding Moment M_y

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

$M_y = \min(M_{y_ten}, M_{y_com}) = 4.0620E+008$

$y ((10a) \text{ or } (10b)) = 1.2225560E-005$

$M_{y_ten} (8a) = 4.0620E+008$

$_{ten} (7a) = 67.28291$

error of function (7a) = 0.00093798

$M_{y_com} (8b) = 7.1091E+008$

$_{com} (7b) = 65.27567$

error of function (7b) = -0.00596426

with $e_y = 0.003125$

$e_{co} = 0.002$
 $apl = 0.35$ ((9a) in Biskinis and Fardis for no FRP Wrap)
 $d1 = 44.00$
 $R = 250.00$
 $v = 0.00125383$
 $N = 7385.681$
 $Ac = 196349.541$
 $= 0.324$
 with $f_c = 30.00$

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 JCC1 of floor 1

At local axis: 3

Integration Section: (b)

Calculation No. 16

column C1, Floor 1

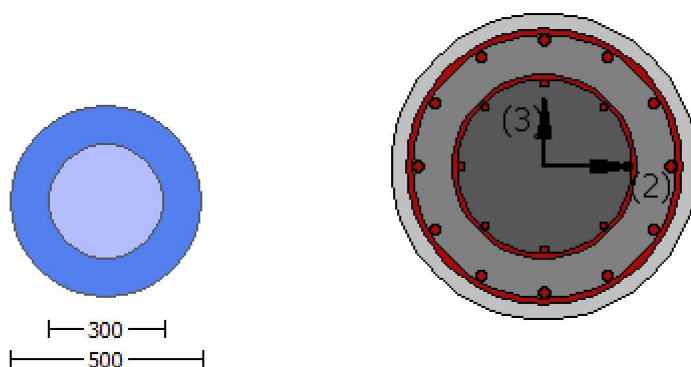
Limit State: Life Safety (data interpolation between analysis steps 1 and 2)

Analysis: Uniform +X

Check: Chord rotation capacity (ϕ)

Edge: End

Local Axis: (3)



Start Of Calculation of Shear Capacity ratio for element: column JCC1 of floor 1

At Shear local axis: 3

(Bending local axis: 2)

Section Type: rcjcs

Constant Properties

Knowledge Factor, $= 0.80$

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

```

Consequently:
Jacket
New material of Secondary Member: Concrete Strength,  $f_c = f_{cm} = 30.00$ 
New material of Secondary Member: Steel Strength,  $f_s = f_{sm} = 625.00$ 
Concrete Elasticity,  $E_c = 25742.96$ 
Steel Elasticity,  $E_s = 200000.00$ 
Existing Column
Existing material of Secondary Member: Concrete Strength,  $f_c = f_{cm} = 24.00$ 
Existing material of Secondary Member: Steel Strength,  $f_s = f_{sm} = 525.00$ 
Concrete Elasticity,  $E_c = 23025.204$ 
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
Jacket
New material: Steel Strength,  $f_s = 1.25 \cdot f_{sm} = 781.25$ 
Existing Column
Existing material: Steel Strength,  $f_s = 1.25 \cdot f_{sm} = 656.25$ 
#####
External Diameter,  $D = 500.00$ 
Internal Diameter,  $D = 300.00$ 
Cover Thickness,  $c = 25.00$ 
Mean Confinement Factor overall section = 1.00
Element Length,  $L = 3000.00$ 
Secondary Member
Smooth Bars
Ductile Steel
Without Detailing for Earthquake Resistance (including stirrups not closed at 135°)
Longitudinal Bars With Ends Lapped Starting at the End Sections
Adequate Lap Length ( $l_o/l_{ou}, \min >= 1$ )
No FRP Wrapping
-----

Stepwise Properties
-----
At local axis: 3
EDGE -A-
Shear Force,  $V_a = 3.5877337E-031$ 
EDGE -B-
Shear Force,  $V_b = -3.5877337E-031$ 
BOTH EDGES
Axial Force,  $F = -7389.214$ 
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 ,  $V_e/V_r = 0.5552125$ 
Member Controlled by Flexure ( $V_e/V_r < 1$ )
Calculation of Shear Demand from fig. R18.6.5, ACI 318-14  $V_e = (M_{pr1} + M_{pr2})/l_n = 286692.647$ 
with
 $M_{pr1} = \max(\mu_{u1+}, \mu_{u1-}) = 4.3004E+008$ 
 $\mu_{u1+} = 4.3004E+008$ , 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
 $\mu_{u1-} = 4.3004E+008$ , 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
 $M_{pr2} = \max(\mu_{u2+}, \mu_{u2-}) = 4.3004E+008$ 
 $\mu_{u2+} = 4.3004E+008$ , 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
 $\mu_{u2-} = 4.3004E+008$ , 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 = 4.3004E+008

= 1.0821
' = 0.95748645
error of function (3.68), Biskinis Phd = 71900.796
From 5A.2, TBDY: fcc = fc* c = 30.00
conf. factor c = 1.00
fc = 30.00
From 10.3.5, ASCE 41-17, Final value of fy: fy*Min(1,1.25*(lb/d)^ 2/3) = 781.25
lb/d = 1.00
d1 = 44.00
R = 250.00
v = 0.00125383
N = 7389.214
Ac = 196349.541
= *Min(1,1.25*(lb/d)^ 2/3) = 0.324

Calculation of ratio lb/d

Adequate Lap Length: lb/d >= 1

Calculation of Mu1-

Calculation of ultimate Moment Strength ((3.67), Biskinis Phd), Mu
Mu = 4.3004E+008

= 1.0821
' = 0.95748645
error of function (3.68), Biskinis Phd = 71900.796
From 5A.2, TBDY: fcc = fc* c = 30.00
conf. factor c = 1.00
fc = 30.00
From 10.3.5, ASCE 41-17, Final value of fy: fy*Min(1,1.25*(lb/d)^ 2/3) = 781.25
lb/d = 1.00
d1 = 44.00
R = 250.00
v = 0.00125383
N = 7389.214
Ac = 196349.541
= *Min(1,1.25*(lb/d)^ 2/3) = 0.324

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 = 4.3004E+008

= 1.0821
 ' = 0.95748645
 error of function (3.68), Biskinis Phd = 71900.796
 From 5A.2, TBDY: $f_{cc} = f_c' \cdot c = 30.00$
 conf. factor $c = 1.00$
 $f_c = 30.00$
 From 10.3.5, ASCE 41-17, Final value of f_y : $f_y \cdot \text{Min}(1, 1.25 \cdot (l_b/l_d)^{2/3}) = 781.25$
 $l_b/l_d = 1.00$
 $d_1 = 44.00$
 $R = 250.00$
 $v = 0.00125383$
 $N = 7389.214$
 $A_c = 196349.541$
 $= \cdot \text{Min}(1, 1.25 \cdot (l_b/l_d)^{2/3}) = 0.324$

Calculation of ratio l_b/l_d

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

Calculation of μ_2 -

Calculation of ultimate Moment Strength ((3.67), Biskinis Phd), μ
 $\mu = 4.3004E+008$

= 1.0821
 ' = 0.95748645
 error of function (3.68), Biskinis Phd = 71900.796
 From 5A.2, TBDY: $f_{cc} = f_c' \cdot c = 30.00$
 conf. factor $c = 1.00$
 $f_c = 30.00$
 From 10.3.5, ASCE 41-17, Final value of f_y : $f_y \cdot \text{Min}(1, 1.25 \cdot (l_b/l_d)^{2/3}) = 781.25$
 $l_b/l_d = 1.00$
 $d_1 = 44.00$
 $R = 250.00$
 $v = 0.00125383$
 $N = 7389.214$
 $A_c = 196349.541$
 $= \cdot \text{Min}(1, 1.25 \cdot (l_b/l_d)^{2/3}) = 0.324$

Calculation of ratio l_b/l_d

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

Calculation of Shear Strength $V_r = \text{Min}(V_{r1}, V_{r2}) = 516365.617$

Calculation of Shear Strength at edge 1, $V_{r1} = 516365.617$

$V_{r1} = V_{CoI} \text{ ((10.3), ASCE 41-17)} = k_n l \cdot V_{CoI0}$
 $V_{CoI0} = 516365.617$
 $k_n l = 1$ (zero step-static loading)

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

= 1 (normal-weight concrete)
 Mean concrete strength: $f_c' = (f_c'_{\text{jacket}} \cdot \text{Area}_{\text{jacket}} + f_c'_{\text{core}} \cdot \text{Area}_{\text{core}}) / \text{Area}_{\text{section}} = 27.84$, but $f_c'^{0.5} \leq 8.3$ MPa (22.5.3.1, ACI 318-14)
 $M/Vd = 2.00$
 $\mu = 1.6653780E-011$

$V_u = 3.5877337E-031$
 $d = 0.8 \cdot D = 400.00$
 $N_u = 7389.214$
 $A_g = 196349.541$
 From (11.5.4.8), ACI 318-14: $V_s = V_{s1} + V_{s2} = 308425.138$
 $V_{s1} = 308425.138$ is calculated for jacket, with:
 $A_v = \frac{1}{2} \cdot A_{stirrup} = 123370.055$
 $f_y = 625.00$
 $s = 100.00$
 V_{s1} is multiplied by $Col1 = 1.00$
 $s/d = 0.25$
 $V_{s2} = 0.00$ is calculated for core, with:
 $A_v = \frac{1}{2} \cdot A_{stirrup} = 78956.835$
 $f_y = 525.00$
 $s = 250.00$
 V_{s2} is multiplied by $Col2 = 0.00$
 $s/d = 1.04167$
 $V_f ((11-3)-(11.4), ACI 440) = 0.00$
 From (11-11), ACI 440: $V_s + V_f \leq 440464.828$
 $b_w \cdot d = \frac{1}{4} \cdot d \cdot d = 125663.706$

Calculation of Shear Strength at edge 2, $V_{r2} = 516365.617$
 $V_{r2} = V_{Col} ((10.3), ASCE 41-17) = knl \cdot V_{Col0}$
 $V_{Col0} = 516365.617$
 $knl = 1$ (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)
 Mean concrete strength: $f_c' = (f_c'_{jacket} \cdot Area_{jacket} + f_c'_{core} \cdot Area_{core}) / Area_{section} = 27.84$, but $f_c'^{0.5} \leq 8.3$
 MPa (22.5.3.1, ACI 318-14)
 $M/Vd = 2.00$
 $M_u = 1.6653780E-011$
 $V_u = 3.5877337E-031$
 $d = 0.8 \cdot D = 400.00$
 $N_u = 7389.214$
 $A_g = 196349.541$
 From (11.5.4.8), ACI 318-14: $V_s = V_{s1} + V_{s2} = 308425.138$
 $V_{s1} = 308425.138$ is calculated for jacket, with:
 $A_v = \frac{1}{2} \cdot A_{stirrup} = 123370.055$
 $f_y = 625.00$
 $s = 100.00$
 V_{s1} is multiplied by $Col1 = 1.00$
 $s/d = 0.25$
 $V_{s2} = 0.00$ is calculated for core, with:
 $A_v = \frac{1}{2} \cdot A_{stirrup} = 78956.835$
 $f_y = 525.00$
 $s = 250.00$
 V_{s2} is multiplied by $Col2 = 0.00$
 $s/d = 1.04167$
 $V_f ((11-3)-(11.4), ACI 440) = 0.00$
 From (11-11), ACI 440: $V_s + V_f \leq 440464.828$
 $b_w \cdot d = \frac{1}{4} \cdot d \cdot d = 125663.706$

End Of Calculation of Shear Capacity ratio for element: column JCC1 of floor 1
 At local axis: 3

Start Of Calculation of Shear Capacity ratio for element: column JCC1 of floor 1
 At Shear local axis: 2
 (Bending local axis: 3)
 Section Type: rcjcs

Constant Properties

Knowledge Factor, $\phi = 0.80$

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

Consequently:

Jacket

New material of Secondary Member: Concrete Strength, $f_c = f_{cm} = 30.00$

New material of Secondary Member: Steel Strength, $f_s = f_{sm} = 625.00$

Concrete Elasticity, $E_c = 25742.96$

Steel Elasticity, $E_s = 200000.00$

Existing Column

Existing material of Secondary Member: Concrete Strength, $f_c = f_{cm} = 24.00$

Existing material of Secondary Member: Steel Strength, $f_s = f_{sm} = 525.00$

Concrete Elasticity, $E_c = 23025.204$

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

Jacket

New material: Steel Strength, $f_s = 1.25 \cdot f_{sm} = 781.25$

Existing Column

Existing material: Steel Strength, $f_s = 1.25 \cdot f_{sm} = 656.25$

#####

External Diameter, $D = 500.00$

Internal Diameter, $D = 300.00$

Cover Thickness, $c = 25.00$

Mean Confinement Factor overall section = 1.00

Element Length, $L = 3000.00$

Secondary Member

Smooth Bars

Ductile Steel

Without Detailing for Earthquake Resistance (including stirrups not 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, $V_a = -7.8886091E-031$

EDGE -B-

Shear Force, $V_b = 7.8886091E-031$

BOTH EDGES

Axial Force, $F = -7389.214$

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: $A_{sl, \text{ten}} = 1017.876$

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

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

Calculation of Shear Capacity ratio, $V_e/V_r = 0.5552125$

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

Calculation of Shear Demand from fig. R18.6.5, ACI 318-14 $V_e = (M_{pr1} + M_{pr2})/l_n = 286692.647$

with

$M_{pr1} = \text{Max}(M_{u1+}, M_{u1-}) = 4.3004E+008$

$M_{u1+} = 4.3004E+008$, 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

$M_{u1-} = 4.3004E+008$, 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

$$M_{pr2} = \text{Max}(M_{u2+}, M_{u2-}) = 4.3004\text{E}+008$$

$M_{u2+} = 4.3004\text{E}+008$, 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

$M_{u2-} = 4.3004\text{E}+008$, 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
 $M_u = 4.3004\text{E}+008$

$$= 1.0821$$

$$\lambda = 0.95748645$$

error of function (3.68), Biskinis Phd = 71900.796

From 5A.2, TB DY: $f_{cc} = f_c \cdot c = 30.00$

conf. factor $c = 1.00$

$$f_c = 30.00$$

From 10.3.5, ASCE 41-17, Final value of f_y : $f_y \cdot \text{Min}(1, 1.25 \cdot (l_b/d)^{2/3}) = 781.25$

$$l_b/d = 1.00$$

$$d_1 = 44.00$$

$$R = 250.00$$

$$v = 0.00125383$$

$$N = 7389.214$$

$$A_c = 196349.541$$

$$= \lambda \cdot \text{Min}(1, 1.25 \cdot (l_b/d)^{2/3}) = 0.324$$

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
 $M_u = 4.3004\text{E}+008$

$$= 1.0821$$

$$\lambda = 0.95748645$$

error of function (3.68), Biskinis Phd = 71900.796

From 5A.2, TB DY: $f_{cc} = f_c \cdot c = 30.00$

conf. factor $c = 1.00$

$$f_c = 30.00$$

From 10.3.5, ASCE 41-17, Final value of f_y : $f_y \cdot \text{Min}(1, 1.25 \cdot (l_b/d)^{2/3}) = 781.25$

$$l_b/d = 1.00$$

$$d_1 = 44.00$$

$$R = 250.00$$

$$v = 0.00125383$$

$$N = 7389.214$$

$$A_c = 196349.541$$

$$= \lambda \cdot \text{Min}(1, 1.25 \cdot (l_b/d)^{2/3}) = 0.324$$

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 = 4.3004E+008

= 1.0821
' = 0.95748645
error of function (3.68), Biskinis Phd = 71900.796
From 5A.2, TBDY: fcc = fc* c = 30.00
conf. factor c = 1.00
fc = 30.00
From 10.3.5, ASCE 41-17, Final value of fy: fy*Min(1,1.25*(lb/d)^ 2/3) = 781.25
lb/d = 1.00
d1 = 44.00
R = 250.00
v = 0.00125383
N = 7389.214
Ac = 196349.541
= *Min(1,1.25*(lb/d)^ 2/3) = 0.324

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 = 4.3004E+008

= 1.0821
' = 0.95748645
error of function (3.68), Biskinis Phd = 71900.796
From 5A.2, TBDY: fcc = fc* c = 30.00
conf. factor c = 1.00
fc = 30.00
From 10.3.5, ASCE 41-17, Final value of fy: fy*Min(1,1.25*(lb/d)^ 2/3) = 781.25
lb/d = 1.00
d1 = 44.00
R = 250.00
v = 0.00125383
N = 7389.214
Ac = 196349.541
= *Min(1,1.25*(lb/d)^ 2/3) = 0.324

Calculation of ratio lb/d

Adequate Lap Length: lb/d >= 1

Calculation of Shear Strength Vr = Min(Vr1,Vr2) = 516365.617

Calculation of Shear Strength at edge 1, Vr1 = 516365.617
Vr1 = VCol ((10.3), ASCE 41-17) = knl*VColO
VColO = 516365.617
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)
Mean concrete strength: $fc' = (fc'_{jacket} \cdot Area_{jacket} + fc'_{core} \cdot Area_{core}) / Area_{section} = 27.84$, but $fc'^{0.5} \leq 8.3$ MPa (22.5.3.1, ACI 318-14)
 $M/Vd = 2.00$
 $\mu_u = 1.7578197E-011$
 $V_u = 7.8886091E-031$
 $d = 0.8 \cdot D = 400.00$
 $N_u = 7389.214$
 $A_g = 196349.541$
From (11.5.4.8), ACI 318-14: $V_s = V_{s1} + V_{s2} = 308425.138$
 $V_{s1} = 308425.138$ is calculated for jacket, with:
 $A_v = \sqrt{2} \cdot A_{stirrup} = 123370.055$
 $f_y = 625.00$
 $s = 100.00$
 V_{s1} is multiplied by $Col1 = 1.00$
 $s/d = 0.25$
 $V_{s2} = 0.00$ is calculated for core, with:
 $A_v = \sqrt{2} \cdot A_{stirrup} = 78956.835$
 $f_y = 525.00$
 $s = 250.00$
 V_{s2} is multiplied by $Col2 = 0.00$
 $s/d = 1.04167$
 $V_f ((11-3)-(11.4), ACI 440) = 0.00$
From (11-11), ACI 440: $V_s + V_f \leq 440464.828$
 $b_w \cdot d = \sqrt{3} \cdot d^2 / 4 = 125663.706$

Calculation of Shear Strength at edge 2, $V_{r2} = 516365.617$
 $V_{r2} = V_{Col} ((10.3), ASCE 41-17) = knl \cdot V_{Col0}$
 $V_{Col0} = 516365.617$
 $knl = 1$ (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)
Mean concrete strength: $fc' = (fc'_{jacket} \cdot Area_{jacket} + fc'_{core} \cdot Area_{core}) / Area_{section} = 27.84$, but $fc'^{0.5} \leq 8.3$ MPa (22.5.3.1, ACI 318-14)
 $M/Vd = 2.00$
 $\mu_u = 1.7578197E-011$
 $V_u = 7.8886091E-031$
 $d = 0.8 \cdot D = 400.00$
 $N_u = 7389.214$
 $A_g = 196349.541$
From (11.5.4.8), ACI 318-14: $V_s = V_{s1} + V_{s2} = 308425.138$
 $V_{s1} = 308425.138$ is calculated for jacket, with:
 $A_v = \sqrt{2} \cdot A_{stirrup} = 123370.055$
 $f_y = 625.00$
 $s = 100.00$
 V_{s1} is multiplied by $Col1 = 1.00$
 $s/d = 0.25$
 $V_{s2} = 0.00$ is calculated for core, with:
 $A_v = \sqrt{2} \cdot A_{stirrup} = 78956.835$
 $f_y = 525.00$
 $s = 250.00$
 V_{s2} is multiplied by $Col2 = 0.00$
 $s/d = 1.04167$
 $V_f ((11-3)-(11.4), ACI 440) = 0.00$
From (11-11), ACI 440: $V_s + V_f \leq 440464.828$
 $b_w \cdot d = \sqrt{3} \cdot d^2 / 4 = 125663.706$

End Of Calculation of Shear Capacity ratio for element: column JCC1 of floor 1
At local axis: 2

Start Of Calculation of Chord Rotation Capacity for element: column JCC1 of floor 1

At local axis: 3

Integration Section: (b)

Section Type: rcjcs

Constant Properties

Knowledge Factor, $\gamma = 0.80$

Chord Rotation is generally considered as Deformation-Controlled Action according to Table C7-1, ASCE 41-17.

Mean strengths are used for Deformation-Controlled Actions according to 7.5.1.3, ASCE 41-17

Consequently:

Jacket

New material of Secondary Member: Concrete Strength, $f_c = f_{cm} = 30.00$

New material of Secondary Member: Steel Strength, $f_s = f_{sm} = 625.00$

Concrete Elasticity, $E_c = 25742.96$

Steel Elasticity, $E_s = 200000.00$

Existing Column

Existing material of Secondary Member: Concrete Strength, $f_c = f_{cm} = 24.00$

Existing material of Secondary Member: Steel Strength, $f_s = f_{sm} = 525.00$

Concrete Elasticity, $E_c = 23025.204$

Steel Elasticity, $E_s = 200000.00$

External Diameter, $D = 500.00$

Internal Diameter, $D = 300.00$

Cover Thickness, $c = 25.00$

Element Length, $L = 3000.00$

Secondary Member

Smooth Bars

Ductile Steel

Without Detailing for Earthquake Resistance (including stirrups not 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, $M = -0.03977224$

Shear Force, $V_2 = 8831.613$

Shear Force, $V_3 = 6.4786357E-013$

Axial Force, $F = -7385.681$

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, $Db_L = 18.00$

Existing component: From table 7-7, ASCE 41_17: Final chord rotation Capacity $u_R = \gamma \cdot u = 0.03583783$

$u = \gamma + p = 0.04479728$

- Calculation of γ -

$\gamma = (M \cdot L_s / 3) / E_{eff} = 0.0017382$ ((4.29), Biskinis Phd))

$M_y = 4.0620E+008$

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

From table 10.5, ASCE 41_17: $E_{eff} = factor \cdot E_c \cdot I_g = 2.3369E+013$

factor = 0.30

$A_g = 196349.541$

Mean concrete strength: $f_c' = (f_c'_{jacket} \cdot Area_{jacket} + f_c'_{core} \cdot Area_{core}) / Area_{section} = 27.84$

$N = 7385.681$

$$E_c I_g = E_c I_{g_jacket} + E_c I_{g_core} = 7.7898E+013$$

Calculation of Yielding Moment M_y

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

$$\begin{aligned} M_y &= \min(M_{y_ten}, M_{y_com}) = 4.0620E+008 \\ \epsilon_y ((10a) \text{ or } (10b)) &= 1.2225560E-005 \\ M_{y_ten} (8a) &= 4.0620E+008 \\ \epsilon_{y_ten} (7a) &= 67.28291 \\ \text{error of function (7a)} &= 0.00093798 \\ M_{y_com} (8b) &= 7.1091E+008 \\ \epsilon_{y_com} (7b) &= 65.27567 \\ \text{error of function (7b)} &= -0.00596426 \\ \text{with } \epsilon_y &= 0.003125 \\ \epsilon_{co} &= 0.002 \\ a_{pl} &= 0.35 ((9a) \text{ in Biskinis and Fardis for no FRP Wrap}) \\ d_1 &= 44.00 \\ R &= 250.00 \\ v &= 0.00125383 \\ N &= 7385.681 \\ A_c &= 196349.541 \\ &= 0.324 \\ \text{with } f_c &= 30.00 \end{aligned}$$

Calculation of ratio I_b/I_d

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

- Calculation of ρ -

From table 10-9: $\rho = 0.04305909$

with:

$$\begin{aligned} &\text{- Columns not controlled by inadequate development or splicing along the clear height because } I_b/I_d \geq 1 \\ &\text{shear control ratio } V_y E / V_{col} E = 0.5552125 \\ &d = d_{external} = 0.00 \\ &s = s_{external} = 0.00 \\ &t = s_1 + s_2 + 2 * t_f / b_w * (f_{fe} / f_s) = 0.00323428 \\ &\text{jacket: } s_1 = A_{v1} * (D_{c1} / 2) / (s_1 * A_g) = 0.0027646 \\ &\quad A_{v1} = 78.53982, \text{ is the area of stirrup} \\ &\quad D_{c1} = D_{ext} - 2 * \text{cover} - \text{External Hoop Diameter} = 440.00, \text{ is the total Length of all stirrups parallel to loading} \\ &\text{(shear) direction} \\ &\quad s_1 = 100.00 \\ &\text{core: } s_2 = A_{v2} * (D_{c2} / 2) / (s_2 * A_g) = 0.00046968 \\ &\quad A_{v2} = 50.26548, \text{ is the area of stirrup} \\ &\quad D_{c2} = D_{int} - \text{Internal Hoop Diameter} = 292.00, \text{ is the total Length of all stirrups parallel to loading (shear)} \\ &\text{direction} \\ &\quad s_2 = 250.00 \\ &\text{The term } 2 * t_f / b_w * (f_{fe} / f_s) \text{ is implemented to account for FRP contribution} \\ &\text{where } f = 2 * t_f / b_w \text{ is FRP ratio (EC8 - 3, A.4.4.3(6)) and } f_{fe} / f_s \text{ normalises } f \text{ to steel strength} \\ &\text{All these variables have already been given in Shear control ratio calculation.} \\ &\text{For the normalisation } f_s \text{ of jacket is used.} \\ &\quad NUD = 7385.681 \\ &\quad A_g = 196349.541 \\ &\quad f_{cE} = (f_c I_{jacket} * Area_{jacket} + f_c I_{core} * Area_{core}) / section_area = 27.84 \\ &\quad f_{yLE} = (f_{y_ext_Long_Reinf} * Area_{ext_Long_Reinf} + f_{y_int_Long_Reinf} * Area_{int_Long_Reinf}) / Area_{Tot_Long_Rein} = \\ &2.1219958E-314 \\ &\quad f_{yTE} = (f_{y_ext_Trans_Reinf} * Area_{ext_Trans_Reinf} + f_{y_int_Trans_Reinf} * Area_{int_Trans_Reinf}) / Area_{Tot_Trans_Rein} = \\ &610.4781 \\ &\quad \rho I = Area_{Tot_Long_Rein} / (A_g) = 0.015552 \\ &\quad f_{cE} = 27.84 \end{aligned}$$

End Of Calculation of Chord Rotation Capacity for element: column JCC1 of floor 1

At local axis: 3
Integration Section: (b)
