

Detailed Member Calculations

Units: N&mm

Regulation: ASCE 41-17

Calculation No. 1

column C1, Floor 1

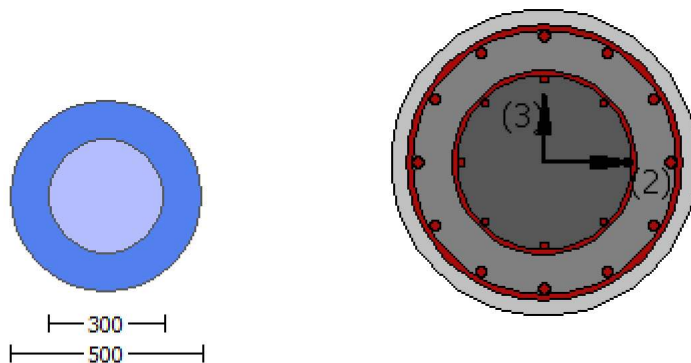
Limit State: Operational Level (data interpolation between analysis steps 1 and 2)

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, $\gamma = 1.00$

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 Primary Member: Concrete Strength, $f_c = f_{c_lower_bound} = 25.00$

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

Concrete Elasticity, $E_c = 26999.444$
Steel Elasticity, $E_s = 200000.00$
Existing Column
Existing material of Primary Member: Concrete Strength, $f_c = f_{c_lower_bound} = 16.00$
Existing material of Primary Member: Steel Strength, $f_s = f_{s_lower_bound} = 400.00$
Concrete Elasticity, $E_c = 21019.039$
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, $f_c = f_{cm} = 33.00$
New material: Steel Strength, $f_s = f_{sm} = 555.5556$
Existing Column
Existing material: Concrete Strength, $f_c = f_{cm} = 20.00$
Existing material: Steel Strength, $f_s = f_{sm} = 444.4444$

External Diameter, $D = 500.00$
Internal Diameter, $D = 300.00$
Cover Thickness, $c = 25.00$
Element Length, $L = 3000.00$
Primary Member
Ribbed Bars
Ductile Steel
With Detailing for Earthquake Resistance (including stirrups closed at 135°)
Longitudinal Bars With Ends Lapped Starting at the End Sections
Adequate Lap Length ($l_o/l_{ou,min} = l_b/l_d \geq 1$)
No FRP Wrapping

Stepwise Properties

EDGE -A-
Bending Moment, $M_a = -1.4007E+007$
Shear Force, $V_a = -4667.691$
EDGE -B-
Bending Moment, $M_b = 0.03183135$
Shear Force, $V_b = 4667.691$
BOTH EDGES
Axial Force, $F = -7387.347$
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 $V_R = V_n = 338743.63$
 V_n ((10.3), ASCE 41-17) = $k_n \cdot V_{CoI0} = 338743.63$
 $V_{CoI} = 338743.63$
 $k_n = 1.00$
displacement_ductility_demand = 0.01065065

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

$\gamma = 1$ (normal-weight concrete)
Mean concrete strength: $f_c' = (f_c'_{jacket} \cdot Area_{jacket} + f_c'_{core} \cdot Area_{core}) / Area_{section} = 21.76$, but $f_c'^{0.5} \leq 8.3$
MPa (22.5.3.1, ACI 318-14)
 $M/Vd = 4.00$
 $\mu_u = 1.4007E+007$

$V_u = 4667.691$
 $d = 0.8 \cdot D = 400.00$
 $N_u = 7387.347$
 $A_g = 196349.541$
 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:
 $A_v = \frac{1}{2} \cdot A_{stirrup} = 78956.835$
 $f_y = 400.00$
 $s = 250.00$
 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: $V_s + V_f \leq 389409.072$
 $b_w \cdot d = \frac{1}{4} \cdot d \cdot d = 125663.706$

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

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

From analysis, chord rotation $\theta = 0.00016004$
 $y = (M_y \cdot L_s / 3) / E_{eff} = 0.01502587$ ((4.29), Biskinis Phd))
 $M_y = 3.6258E+008$
 $L_s = M/V$ (with $L_s > 0.1 \cdot L$ and $L_s < 2 \cdot L$) = 3000.755
 From table 10.5, ASCE 41_17: $E_{eff} = factor \cdot E_c \cdot I_g = 2.4137E+013$
 $factor = 0.30$
 $A_g = 196349.541$
 Mean concrete strength: $f'_c = (f'_c_{jacket} \cdot Area_{jacket} + f'_c_{core} \cdot Area_{core}) / Area_{section} = 28.32$
 $N = 7387.347$
 $E_c \cdot I_g = E_{c,jacket} \cdot I_{g,jacket} + E_{c,core} \cdot I_{g,core} = 8.0455E+013$

Calculation of Yielding Moment M_y

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

$M_y = \min(M_{y,ten}, M_{y,com}) = 3.6258E+008$
 $y ((10a) \text{ or } (10b)) = 1.0622220E-005$
 $M_{y,ten} (8a) = 3.6258E+008$
 $\Delta_{ten} (7a) = 65.43627$
 error of function (7a) = 0.00293095
 $M_{y,com} (8b) = 7.5621E+008$
 $\Delta_{com} (7b) = 64.56804$
 error of function (7b) = -0.00721905
 with $e_y = 0.00277778$
 $e_{co} = 0.002$
 $apl = 0.35$ ((9a) in Biskinis and Fardis for no FRP Wrap)
 $d1 = 44.00$
 $R = 250.00$
 $v = 0.0011401$
 $N = 7387.347$
 $A_c = 196349.541$
 $= 0.26181818$
 with $f_c = 33.00$

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

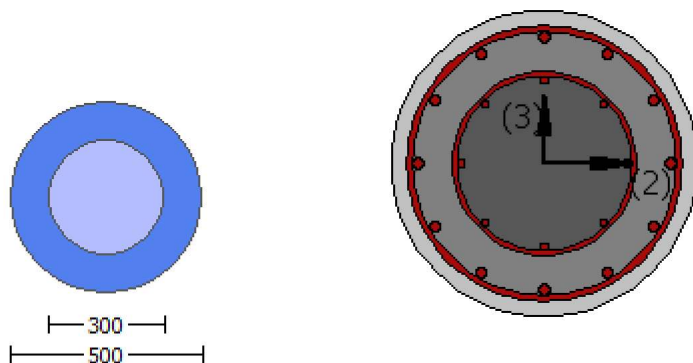
Limit State: Operational Level (data interpolation between analysis steps 1 and 2)

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, $\gamma = 1.00$

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

Consequently:

Jacket

New material of Primary Member: Concrete Strength, $f_c = f_{cm} = 33.00$

New material of Primary Member: Steel Strength, $f_s = f_{sm} = 555.5556$

Concrete Elasticity, $E_c = 26999.444$

Steel Elasticity, $E_s = 200000.00$

Existing Column

Existing material of Primary Member: Concrete Strength, $f_c = f_{cm} = 20.00$

Existing material of Primary Member: Steel Strength, $f_s = f_{sm} = 444.4444$

Concrete Elasticity, $E_c = 21019.039$

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, $f_s = 1.25 \cdot f_{sm} = 694.4444$

Existing Column

Existing material: Steel Strength, $f_s = 1.25 \cdot f_{sm} = 555.5556$

#####

External Diameter, $D = 500.00$

Internal Diameter, $D = 300.00$

Cover Thickness, $c = 25.00$

Mean Confinement Factor overall section = 1.30349

Element Length, $L = 3000.00$

Primary Member

Ribbed Bars

Ductile Steel

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

Longitudinal Bars With Ends Lapped Starting at the End Sections

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

No FRP Wrapping

Stepwise Properties

At local axis: 3

EDGE -A-

Shear Force, $V_a = -1.4969033E-031$

EDGE -B-

Shear Force, $V_b = 1.4969033E-031$

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, $V_e/V_r = 0.5636717$

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

Calculation of Shear Demand from fig. R18.6.5, ACI 318-14 $V_e = (M_{pr1} + M_{pr2})/l_n = 272742.977$

with

$M_{pr1} = \max(\mu_{1+}, \mu_{1-}) = 4.0911E+008$

$\mu_{1+} = 4.0911E+008$, 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

$\mu_{1-} = 4.0911E+008$, 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

$M_{pr2} = \max(\mu_{2+}, \mu_{2-}) = 4.0911E+008$

$\mu_{2+} = 4.0911E+008$, 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

$\mu_{2-} = 4.0911E+008$, 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

$\mu_u = 4.0911E+008$

$= 0.97738438$

$' = 0.86668818$

error of function (3.68), Biskinis Phd = 94699.84

From 5A.2, TBDY: $f_{cc} = f_c^* \quad c = 43.01524$

conf. factor $c = 1.30349$
 $f_c = 33.00$
 From 10.3.5, ASCE 41-17, Final value of f_y : $f_y \cdot \text{Min}(1, 1.25 \cdot (l_b/d)^{2/3}) = 694.4444$
 $l_b/d = 1.00$
 $d_1 = 44.00$
 $R = 250.00$
 $v = 0.00114003$
 $N = 7389.214$
 $Ac = 196349.541$
 $= \cdot \text{Min}(1, 1.25 \cdot (l_b/d)^{2/3}) = 0.26181818$

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
 $\mu = 4.0911\text{E}+008$

$= 0.97738438$
 $' = 0.86668818$
 error of function (3.68), Biskinis Phd = 94699.84
 From 5A.2, TBDY: $f_{cc} = f_c \cdot c = 43.01524$
 conf. factor $c = 1.30349$
 $f_c = 33.00$
 From 10.3.5, ASCE 41-17, Final value of f_y : $f_y \cdot \text{Min}(1, 1.25 \cdot (l_b/d)^{2/3}) = 694.4444$
 $l_b/d = 1.00$
 $d_1 = 44.00$
 $R = 250.00$
 $v = 0.00114003$
 $N = 7389.214$
 $Ac = 196349.541$
 $= \cdot \text{Min}(1, 1.25 \cdot (l_b/d)^{2/3}) = 0.26181818$

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
 $\mu = 4.0911\text{E}+008$

$= 0.97738438$
 $' = 0.86668818$
 error of function (3.68), Biskinis Phd = 94699.84
 From 5A.2, TBDY: $f_{cc} = f_c \cdot c = 43.01524$
 conf. factor $c = 1.30349$
 $f_c = 33.00$
 From 10.3.5, ASCE 41-17, Final value of f_y : $f_y \cdot \text{Min}(1, 1.25 \cdot (l_b/d)^{2/3}) = 694.4444$
 $l_b/d = 1.00$
 $d_1 = 44.00$
 $R = 250.00$
 $v = 0.00114003$
 $N = 7389.214$
 $Ac = 196349.541$

$$= \text{Min}(1, 1.25 \cdot (l_b/d)^{2/3}) = 0.26181818$$

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
 $\mu = 4.0911 \text{E}+008$

$$= 0.97738438$$

$$\lambda = 0.86668818$$

error of function (3.68), Biskinis Phd = 94699.84

From 5A.2, TBDY: $f_{cc} = f_c \cdot \lambda = 43.01524$

conf. factor $\lambda = 1.30349$

$f_c = 33.00$

From 10.3.5, ASCE 41-17, Final value of f_y : $f_y \cdot \text{Min}(1, 1.25 \cdot (l_b/d)^{2/3}) = 694.4444$

$$l_b/d = 1.00$$

$$d_1 = 44.00$$

$$R = 250.00$$

$$v = 0.00114003$$

$$N = 7389.214$$

$$A_c = 196349.541$$

$$= \text{Min}(1, 1.25 \cdot (l_b/d)^{2/3}) = 0.26181818$$

Calculation of ratio l_b/d

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

Calculation of Shear Strength $V_r = \text{Min}(V_{r1}, V_{r2}) = 483868.491$

Calculation of Shear Strength at edge 1, $V_{r1} = 483868.491$

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

$$V_{c10} = 483868.491$$

$k_n = 1$ (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: $f_c' = (f_c'_{\text{jacket}} \cdot \text{Area}_{\text{jacket}} + f_c'_{\text{core}} \cdot \text{Area}_{\text{core}}) / \text{Area}_{\text{section}} = 28.32$, but $f_c'^{0.5} \leq 8.3$ MPa ((22.5.3.1, ACI 318-14)

$$M/Vd = 2.00$$

$$\mu = 1.4802042 \text{E}-011$$

$$V_u = 1.4969033 \text{E}-031$$

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

$$N_u = 7389.214$$

$$A_g = 196349.541$$

From ((11.5.4.8), ACI 318-14: $V_s = V_{s1} + V_{s2} = 274155.678$

$V_{s1} = 274155.678$ is calculated for jacket, with:

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

$$f_y = 555.5556$$

$$s = 100.00$$

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

$$s/d = 0.25$$

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

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

$$f_y = 444.4444$$

$$s = 250.00$$

Vs2 is multiplied by Col2 = 0.00

$$s/d = 1.04167$$

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

$$\text{From (11-11), ACI } 440: V_s + V_f \leq 444245.712$$

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

Calculation of Shear Strength at edge 2, $V_{r2} = 483868.491$

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

$$V_{\text{Col}0} = 483868.491$$

$k_n l = 1$ (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)

$$\text{Mean concrete strength: } f'_c = (f'_{c_jacket} \cdot \text{Area}_{jacket} + f'_{c_core} \cdot \text{Area}_{core}) / \text{Area}_{section} = 28.32, \text{ but } f_c^{0.5} \leq 8.3 \text{ MPa (22.5.3.1, ACI 318-14)}$$

$$M/Vd = 2.00$$

$$M_u = 1.4802042\text{E-}011$$

$$V_u = 1.4969033\text{E-}031$$

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

$$N_u = 7389.214$$

$$A_g = 196349.541$$

$$\text{From (11.5.4.8), ACI 318-14: } V_s = V_{s1} + V_{s2} = 274155.678$$

$V_{s1} = 274155.678$ is calculated for jacket, with:

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

$$f_y = 555.5556$$

$$s = 100.00$$

V_{s1} is multiplied by Col1 = 1.00

$$s/d = 0.25$$

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

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

$$f_y = 444.4444$$

$$s = 250.00$$

Vs2 is multiplied by Col2 = 0.00

$$s/d = 1.04167$$

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

$$\text{From (11-11), ACI } 440: V_s + V_f \leq 444245.712$$

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

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, = 1.00

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

Consequently:

Jacket

New material of Primary Member: Concrete Strength, $f_c = f_{cm} = 33.00$

New material of Primary Member: Steel Strength, $f_s = f_{sm} = 555.5556$

Concrete Elasticity, $E_c = 26999.444$

Steel Elasticity, $E_s = 200000.00$

Existing Column

Existing material of Primary Member: Concrete Strength, $f_c = f_{cm} = 20.00$

Existing material of Primary Member: Steel Strength, $f_s = f_{sm} = 444.4444$
Concrete Elasticity, $E_c = 21019.039$
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, $f_s = 1.25 \cdot f_{sm} = 694.4444$
Existing Column
Existing material: Steel Strength, $f_s = 1.25 \cdot f_{sm} = 555.5556$

External Diameter, $D = 500.00$
Internal Diameter, $D = 300.00$
Cover Thickness, $c = 25.00$
Mean Confinement Factor overall section = 1.30349
Element Length, $L = 3000.00$
Primary Member
Ribbed Bars
Ductile Steel
With Detailing for Earthquake Resistance (including stirrups closed at 135°)
Longitudinal Bars With Ends Lapped Starting at the End Sections
Adequate Lap Length ($l_o/l_{ou,min} > 1$)
No FRP Wrapping

Stepwise Properties

At local axis: 2
EDGE -A-
Shear Force, $V_a = -2.6832056E-030$
EDGE -B-
Shear Force, $V_b = 2.6832056E-030$
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 , $V_e/V_r = 0.5636717$
Member Controlled by Flexure ($V_e/V_r < 1$)
Calculation of Shear Demand from fig. R18.6.5, ACI 318-14 $V_e = (M_{pr1} + M_{pr2})/l_n = 272742.977$
with
 $M_{pr1} = \text{Max}(Mu_{1+} , Mu_{1-}) = 4.0911E+008$
 $Mu_{1+} = 4.0911E+008$, 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
 $Mu_{1-} = 4.0911E+008$, 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
 $M_{pr2} = \text{Max}(Mu_{2+} , Mu_{2-}) = 4.0911E+008$
 $Mu_{2+} = 4.0911E+008$, 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
 $Mu_{2-} = 4.0911E+008$, 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
 $Mu = 4.0911E+008$

```

= 0.97738438
' = 0.86668818
error of function (3.68), Biskinis Phd = 94699.84
From 5A.2, TBDY: fcc = fc* c = 43.01524
conf. factor c = 1.30349
fc = 33.00
From 10.3.5, ASCE 41-17, Final value of fy: fy*Min(1,1.25*(lb/d)^ 2/3) = 694.4444
lb/d = 1.00
d1 = 44.00
R = 250.00
v = 0.00114003
N = 7389.214
Ac = 196349.541
= *Min(1,1.25*(lb/d)^ 2/3) = 0.26181818

```

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 = 4.0911E+008

```

= 0.97738438
' = 0.86668818
error of function (3.68), Biskinis Phd = 94699.84
From 5A.2, TBDY: fcc = fc* c = 43.01524
conf. factor c = 1.30349
fc = 33.00
From 10.3.5, ASCE 41-17, Final value of fy: fy*Min(1,1.25*(lb/d)^ 2/3) = 694.4444
lb/d = 1.00
d1 = 44.00
R = 250.00
v = 0.00114003
N = 7389.214
Ac = 196349.541
= *Min(1,1.25*(lb/d)^ 2/3) = 0.26181818

```

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 = 4.0911E+008

```

= 0.97738438
' = 0.86668818
error of function (3.68), Biskinis Phd = 94699.84
From 5A.2, TBDY: fcc = fc* c = 43.01524
conf. factor c = 1.30349
fc = 33.00
From 10.3.5, ASCE 41-17, Final value of fy: fy*Min(1,1.25*(lb/d)^ 2/3) = 694.4444
lb/d = 1.00

```

$d1 = 44.00$
 $R = 250.00$
 $v = 0.00114003$
 $N = 7389.214$
 $Ac = 196349.541$
 $= *Min(1, 1.25*(lb/ld)^{2/3}) = 0.26181818$

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

$Mu = 4.0911E+008$

$= 0.97738438$

$' = 0.86668818$

error of function (3.68), Biskinis Phd = 94699.84

From 5A.2, TDY: $f_{cc} = f_c \cdot c = 43.01524$

conf. factor $c = 1.30349$

$f_c = 33.00$

From 10.3.5, ASCE 41-17, Final value of f_y : $f_y * Min(1, 1.25*(lb/ld)^{2/3}) = 694.4444$

$lb/ld = 1.00$

$d1 = 44.00$

$R = 250.00$

$v = 0.00114003$

$N = 7389.214$

$Ac = 196349.541$

$= *Min(1, 1.25*(lb/ld)^{2/3}) = 0.26181818$

Calculation of ratio lb/ld

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

Calculation of Shear Strength $V_r = Min(V_{r1}, V_{r2}) = 483868.491$

Calculation of Shear Strength at edge 1, $V_{r1} = 483868.491$

$V_{r1} = V_{CoI} ((10.3), ASCE 41-17) = knl * V_{CoI0}$

$V_{CoI0} = 483868.491$

$knl = 1$ (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: $f'_c = (f'_c_jacket * Area_jacket + f'_c_core * Area_core) / Area_section = 28.32$, but $f'_c^{0.5} \leq 8.3$ MPa (22.5.3.1, ACI 318-14)

$M/Vd = 2.00$

$Mu = 2.6359963E-011$

$Vu = 2.6832056E-030$

$d = 0.8 * D = 400.00$

$Nu = 7389.214$

$Ag = 196349.541$

From (11.5.4.8), ACI 318-14: $V_s = V_{s1} + V_{s2} = 274155.678$

$V_{s1} = 274155.678$ is calculated for jacket, with:

$A_v = /2 * A_stirrup = 123370.055$

$f_y = 555.5556$

$s = 100.00$
 V_{s1} is multiplied by $Col1 = 1.00$
 $s/d = 0.25$
 $V_{s2} = 0.00$ is calculated for core, with:
 $A_v = \sqrt{2} \cdot A_{stirrup} = 78956.835$
 $f_y = 444.4444$
 $s = 250.00$
 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: $V_s + V_f \leq 444245.712$
 $b_w \cdot d = \sqrt{d} \cdot d / 4 = 125663.706$

Calculation of Shear Strength at edge 2, $V_{r2} = 483868.491$
 $V_{r2} = V_{Col} ((10.3), ASCE 41-17) = knl \cdot V_{Col0}$
 $V_{Col0} = 483868.491$
 $knl = 1$ (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: $f'_c = (f'_c \cdot A_{jacket} + f'_c \cdot A_{core}) / A_{section} = 28.32$, but $f'_c^{0.5} \leq 8.3$
 MPa (22.5.3.1, ACI 318-14)
 $M/Vd = 2.00$
 $\mu_u = 2.6359963E-011$
 $V_u = 2.6832056E-030$
 $d = 0.8 \cdot D = 400.00$
 $N_u = 7389.214$
 $A_g = 196349.541$
 From (11.5.4.8), ACI 318-14: $V_s = V_{s1} + V_{s2} = 274155.678$
 $V_{s1} = 274155.678$ is calculated for jacket, with:
 $A_v = \sqrt{2} \cdot A_{stirrup} = 123370.055$
 $f_y = 555.5556$
 $s = 100.00$
 V_{s1} is multiplied by $Col1 = 1.00$
 $s/d = 0.25$
 $V_{s2} = 0.00$ is calculated for core, with:
 $A_v = \sqrt{2} \cdot A_{stirrup} = 78956.835$
 $f_y = 444.4444$
 $s = 250.00$
 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: $V_s + V_f \leq 444245.712$
 $b_w \cdot d = \sqrt{d} \cdot d / 4 = 125663.706$

End Of Calculation of Shear Capacity ratio for element: column JCC1 of floor 1
 At local axis: 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, $= 1.00$
 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 Primary Member: Concrete Strength, $f_c = f_{cm} = 33.00$
 New material of Primary Member: Steel Strength, $f_s = f_{sm} = 555.5556$
 Concrete Elasticity, $E_c = 26999.444$
 Steel Elasticity, $E_s = 200000.00$
 Existing Column
 Existing material of Primary Member: Concrete Strength, $f_c = f_{cm} = 20.00$
 Existing material of Primary Member: Steel Strength, $f_s = f_{sm} = 444.4444$
 Concrete Elasticity, $E_c = 21019.039$
 Steel Elasticity, $E_s = 200000.00$
 External Diameter, $D = 500.00$
 Internal Diameter, $D = 300.00$
 Cover Thickness, $c = 25.00$
 Element Length, $L = 3000.00$
 Primary Member
 Ribbed Bars
 Ductile Steel
 With Detailing for Earthquake Resistance (including stirrups closed at 135°)
 Longitudinal Bars With Ends Lapped Starting at the End Sections
 Adequate Lap Length ($l_b/l_d \geq 1$)
 No FRP Wrapping

Stepwise Properties

Bending Moment, $M = 2.3398468E-011$
 Shear Force, $V_2 = -4667.691$
 Shear Force, $V_3 = -8.7343036E-015$
 Axial Force, $F = -7387.347$
 Longitudinal Reinforcement Area Distribution (in 2 divisions)
 -Tension: $A_{st} = 0.00$
 -Compression: $A_{sc} = 3053.628$
 Longitudinal Reinforcement Area Distribution (in 3 divisions)
 -Tension: $A_{st,ten} = 1017.876$
 -Compression: $A_{st,com} = 1017.876$
 -Middle: $A_{st,mid} = 1017.876$
 Mean Diameter of Tension Reinforcement, $D_bL = 18.00$

Existing component: From table 7-7, ASCE 41_17: Final chord rotation Capacity $u_{R} = u = 0.01251105$
 $u = y + p = 0.01251105$

- Calculation of y -

$y = (M_y * L_s / 3) / E_{eff} = 0.00751105$ ((4.29), Biskinis Phd))
 $M_y = 3.6258E+008$
 $L_s = M/V$ (with $L_s > 0.1 * L$ and $L_s < 2 * L$) = 1500.00
 From table 10.5, ASCE 41_17: $E_{eff} = factor * E_c * I_g = 2.4137E+013$
 $factor = 0.30$
 $A_g = 196349.541$
 Mean concrete strength: $f_c' = (f_c'_{jacket} * Area_{jacket} + f_c'_{core} * Area_{core}) / Area_{section} = 28.32$
 $N = 7387.347$
 $E_c * I_g = E_c_{jacket} * I_{g,jacket} + E_c_{core} * I_{g,core} = 8.0455E+013$

Calculation of Yielding Moment M_y

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

$M_y = \min(M_{y,ten}, M_{y,com}) = 3.6258E+008$
 y ((10a) or (10b)) = $1.0622220E-005$
 $M_{y,ten}$ (8a) = $3.6258E+008$
 y_{ten} (7a) = 65.43627
 error of function (7a) = 0.00293095

$M_{y_com} (8b) = 7.5621E+008$
 $_{com} (7b) = 64.56804$
error of function (7b) = -0.00721905
with $e_y = 0.00277778$
 $e_{co} = 0.002$
 $a_{pl} = 0.35$ ((9a) in Biskinis and Fardis for no FRP Wrap)
 $d_1 = 44.00$
 $R = 250.00$
 $v = 0.0011401$
 $N = 7387.347$
 $A_c = 196349.541$
 $= 0.26181818$
with $f_c = 33.00$

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 $V_y E / V_{col} E = 0.5636717$

$d = d_{external} = 0.00$

$s = s_{external} = 0.00$

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

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.

$N_{UD} = 7387.347$

$A_g = 196349.541$

$f_{cE} = (f_{c_jacket} \cdot \text{Area_jacket} + f_{c_core} \cdot \text{Area_core}) / \text{section_area} = 28.32$

$f_{yE} = (f_{y_ext_Long_Reinf} \cdot \text{Area_ext_Long_Reinf} + f_{y_int_Long_Reinf} \cdot \text{Area_int_Long_Reinf}) / \text{Area_Tot_Long_Rein} = 21219958E-314$

$f_{yE} = (f_{y_ext_Trans_Reinf} \cdot \text{Area_ext_Trans_Reinf} + f_{y_int_Trans_Reinf} \cdot \text{Area_int_Trans_Reinf}) / \text{Area_Tot_Trans_Rein} = 539.4201$

$p_l = \text{Area_Tot_Long_Rein} / (A_g) = 0.015552$

$f_{cE} = 28.32$

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

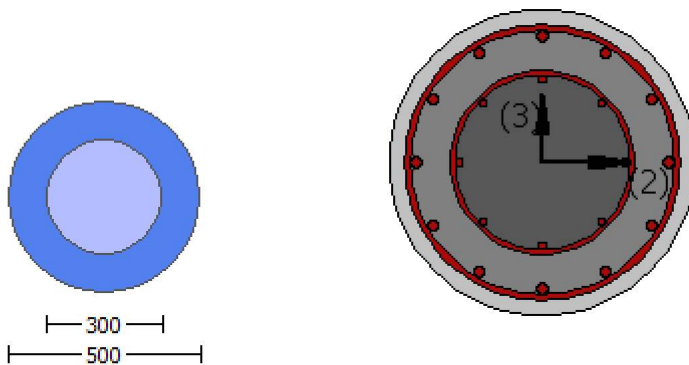
Limit State: Operational Level (data interpolation between analysis steps 1 and 2)

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, $\gamma = 1.00$

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 Primary Member: Concrete Strength, $f_c = f_{c_lower_bound} = 25.00$

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

Concrete Elasticity, $E_c = 26999.444$

Steel Elasticity, $E_s = 200000.00$

Existing Column

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

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

Concrete Elasticity, $E_c = 21019.039$

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, $f_c = f_{cm} = 33.00$

New material: Steel Strength, $f_s = f_{sm} = 555.5556$

Existing Column

Existing material: Concrete Strength, $f_c = f_{cm} = 20.00$

Existing material: Steel Strength, $f_s = f_{sm} = 444.4444$

#####

External Diameter, D = 500.00
 Internal Diameter, D = 300.00
 Cover Thickness, c = 25.00
 Element Length, L = 3000.00
 Primary Member
 Ribbed Bars
 Ductile Steel
 With Detailing for Earthquake Resistance (including stirrups closed at 135°)
 Longitudinal Bars With Ends Lapped Starting at the End Sections
 Adequate Lap Length ($l_o/l_{o,u,min} = l_b/l_d \geq 1$)
 No FRP Wrapping

Stepwise Properties

EDGE -A-
 Bending Moment, $M_a = 2.3398468E-011$
 Shear Force, $V_a = -8.7343036E-015$
 EDGE -B-
 Bending Moment, $M_b = 2.7436834E-012$
 Shear Force, $V_b = 8.7343036E-015$
 BOTH EDGES
 Axial Force, $F = -7387.347$
 Longitudinal Reinforcement Area Distribution (in 2 divisions)
 -Tension: $A_{sl,t} = 0.00$
 -Compression: $A_{sl,c} = 3053.628$
 Longitudinal Reinforcement Area Distribution (in 3 divisions)
 -Tension: $A_{sl,ten} = 1017.876$
 -Compression: $A_{sl,com} = 1017.876$
 -Middle: $A_{sl,mid} = 1017.876$
 Mean Diameter of Tension Reinforcement, $Db_{L,ten} = 18.00$

Existing component: From table 7-7, ASCE 41-17: Final Shear Capacity $V_R = V_n = 430747.15$
 V_n ((10.3), ASCE 41-17) = $k_n \cdot V_{CoI} = 430747.15$
 $V_{CoI} = 430747.15$
 $k_n = 1.00$
 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: $f'_c = (f'_c_{jacket} \cdot Area_{jacket} + f'_c_{core} \cdot Area_{core}) / Area_{section} = 21.76$, but $f'_c^{0.5} \leq 8.3$
 MPa ((22.5.3.1, ACI 318-14)
 $M/V_d = 2.00$
 $\mu_u = 2.3398468E-011$
 $V_u = 8.7343036E-015$
 $d = 0.8 \cdot D = 400.00$
 $N_u = 7387.347$
 $A_g = 196349.541$
 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:
 $A_v = \pi/2 \cdot A_{stirrup} = 123370.055$
 $f_y = 500.00$
 $s = 100.00$
 V_{s1} is multiplied by $Col1 = 1.00$
 $s/d = 0.25$
 $V_{s2} = 0.00$ is calculated for core, with:
 $A_v = \pi/2 \cdot A_{stirrup} = 78956.835$
 $f_y = 400.00$
 $s = 250.00$
 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: $V_s + V_f \leq 389409.072$
 $b_w \cdot d = \frac{V_s \cdot d}{4} = 125663.706$

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 $\theta = 2.7397291E-021$
 $y = \frac{M_y \cdot L_s / 3}{E_{eff}} = 0.00751105$ ((4.29), Biskinis Phd))
 $M_y = 3.6258E+008$
 $L_s = M/V$ (with $L_s > 0.1 \cdot L$ and $L_s < 2 \cdot L$) = 1500.00
From table 10.5, ASCE 41_17: $E_{eff} = \text{factor} \cdot E_c \cdot I_g = 2.4137E+013$
factor = 0.30
 $A_g = 196349.541$
Mean concrete strength: $f'_c = \frac{(f'_c)_{\text{jacket}} \cdot \text{Area}_{\text{jacket}} + (f'_c)_{\text{core}} \cdot \text{Area}_{\text{core}}}{\text{Area}_{\text{section}}} = 28.32$
 $N = 7387.347$
 $E_c \cdot I_g = E_{c,\text{jacket}} \cdot I_{g,\text{jacket}} + E_{c,\text{core}} \cdot I_{g,\text{core}} = 8.0455E+013$

Calculation of Yielding Moment M_y

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

$M_y = \min(M_{y,\text{ten}}, M_{y,\text{com}}) = 3.6258E+008$
 y ((10a) or (10b)) = 1.0622220E-005
 $M_{y,\text{ten}}$ (8a) = 3.6258E+008
 $\frac{\Delta}{y}$ (7a) = 65.43627
error of function (7a) = 0.00293095
 $M_{y,\text{com}}$ (8b) = 7.5621E+008
 $\frac{\Delta}{y}$ (7b) = 64.56804
error of function (7b) = -0.00721905
with $e_y = 0.00277778$
 $e_{co} = 0.002$
 $a_{pl} = 0.35$ ((9a) in Biskinis and Fardis for no FRP Wrap)
 $d_1 = 44.00$
 $R = 250.00$
 $v = 0.0011401$
 $N = 7387.347$
 $A_c = 196349.541$
 $\frac{A_c}{A_g} = 0.26181818$
with $f_c = 33.00$

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

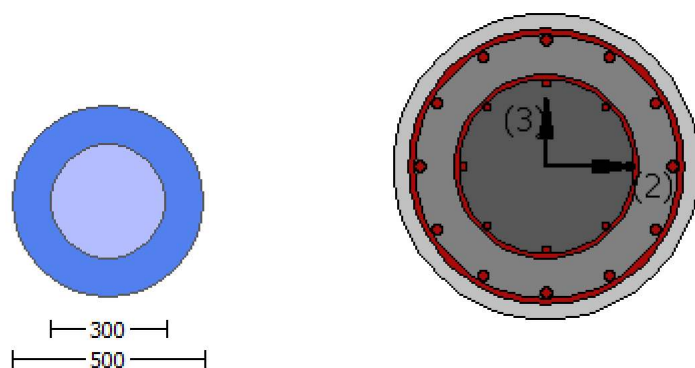
Limit State: Operational Level (data interpolation between analysis steps 1 and 2)

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, $\gamma = 1.00$

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

Consequently:

Jacket

New material of Primary Member: Concrete Strength, $f_c = f_{cm} = 33.00$

New material of Primary Member: Steel Strength, $f_s = f_{sm} = 555.5556$

Concrete Elasticity, $E_c = 26999.444$

Steel Elasticity, $E_s = 200000.00$

Existing Column

Existing material of Primary Member: Concrete Strength, $f_c = f_{cm} = 20.00$

Existing material of Primary Member: Steel Strength, $f_s = f_{sm} = 444.4444$

Concrete Elasticity, $E_c = 21019.039$

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, $f_s = 1.25 \cdot f_{sm} = 694.4444$

Existing Column

Existing material: Steel Strength, $f_s = 1.25 \cdot f_{sm} = 555.5556$

#####

External Diameter, $D = 500.00$

Internal Diameter, $D = 300.00$

Cover Thickness, $c = 25.00$

Mean Confinement Factor overall section = 1.30349

Element Length, $L = 3000.00$

Primary Member

Ribbed Bars

Ductile Steel

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

Longitudinal Bars With Ends Lapped Starting at the End Sections

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

No FRP Wrapping

Stepwise Properties

At local axis: 3

EDGE -A-

Shear Force, $V_a = -1.4969033\text{E}-031$

EDGE -B-

Shear Force, $V_b = 1.4969033\text{E}-031$

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, $V_e/V_r = 0.5636717$

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

Calculation of Shear Demand from fig. R18.6.5, ACI 318-14 $V_e = (M_{pr1} + M_{pr2})/l_n = 272742.977$

with

$M_{pr1} = \text{Max}(Mu_{1+}, Mu_{1-}) = 4.0911\text{E}+008$

$Mu_{1+} = 4.0911\text{E}+008$, is the ultimate moment strength at the edge 1 of the member in the actual moment direction which is defined for the static loading combination

$Mu_{1-} = 4.0911\text{E}+008$, is the ultimate moment strength at the edge 1 of the member in the opposite moment direction which is defined for the static loading combination

$M_{pr2} = \text{Max}(Mu_{2+}, Mu_{2-}) = 4.0911\text{E}+008$

$Mu_{2+} = 4.0911\text{E}+008$, is the ultimate moment strength at the edge 2 of the member in the actual moment direction which is defined for the static loading combination

$Mu_{2-} = 4.0911\text{E}+008$, is the ultimate moment strength at the edge 2 of the member in the opposite moment direction which is defined for the static loading combination

Calculation of Mu_{1+}

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

$M_u = 4.0911\text{E}+008$

$\phi = 0.97738438$

$\phi' = 0.86668818$

error of function (3.68), Biskinis Phd = 94699.84

From 5A.2, TBDY: $f_{cc} = f_c \cdot c = 43.01524$

conf. factor $c = 1.30349$

$f_c = 33.00$

From 10.3.5, ASCE 41-17, Final value of f_y : $f_y \cdot \text{Min}(1, 1.25 \cdot (l_b/l_d)^{2/3}) = 694.4444$

$l_b/l_d = 1.00$

$d_1 = 44.00$

$R = 250.00$

$v = 0.00114003$

$N = 7389.214$

$A_c = 196349.541$

$= \phi' \cdot \text{Min}(1, 1.25 \cdot (l_b/l_d)^{2/3}) = 0.26181818$

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 = 4.0911E+008

= 0.97738438
' = 0.86668818
error of function (3.68), Biskinis Phd = 94699.84
From 5A.2, TBDY: $f_{cc} = f_c \cdot c = 43.01524$
conf. factor $c = 1.30349$
 $f_c = 33.00$
From 10.3.5, ASCE 41-17, Final value of f_y : $f_y \cdot \text{Min}(1, 1.25 \cdot (l_b/d)^{2/3}) = 694.4444$
 $l_b/d = 1.00$
 $d_1 = 44.00$
 $R = 250.00$
 $v = 0.00114003$
 $N = 7389.214$
 $A_c = 196349.541$
= $\cdot \text{Min}(1, 1.25 \cdot (l_b/d)^{2/3}) = 0.26181818$

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 = 4.0911E+008

= 0.97738438
' = 0.86668818
error of function (3.68), Biskinis Phd = 94699.84
From 5A.2, TBDY: $f_{cc} = f_c \cdot c = 43.01524$
conf. factor $c = 1.30349$
 $f_c = 33.00$
From 10.3.5, ASCE 41-17, Final value of f_y : $f_y \cdot \text{Min}(1, 1.25 \cdot (l_b/d)^{2/3}) = 694.4444$
 $l_b/d = 1.00$
 $d_1 = 44.00$
 $R = 250.00$
 $v = 0.00114003$
 $N = 7389.214$
 $A_c = 196349.541$
= $\cdot \text{Min}(1, 1.25 \cdot (l_b/d)^{2/3}) = 0.26181818$

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 = 4.0911E+008

= 0.97738438

$\rho = 0.86668818$
 error of function (3.68), Biskinis Phd = 94699.84
 From 5A.2, TBDY: $f_{cc} = f_c' \cdot c = 43.01524$
 conf. factor $c = 1.30349$
 $f_c = 33.00$
 From 10.3.5, ASCE 41-17, Final value of f_y : $f_y \cdot \text{Min}(1, 1.25 \cdot (l_b/l_d)^{2/3}) = 694.4444$
 $l_b/l_d = 1.00$
 $d_1 = 44.00$
 $R = 250.00$
 $v = 0.00114003$
 $N = 7389.214$
 $A_c = 196349.541$
 $= \cdot \text{Min}(1, 1.25 \cdot (l_b/l_d)^{2/3}) = 0.26181818$

Calculation of ratio l_b/l_d

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

Calculation of Shear Strength $V_r = \text{Min}(V_{r1}, V_{r2}) = 483868.491$

Calculation of Shear Strength at edge 1, $V_{r1} = 483868.491$

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

$V_{ColO} = 483868.491$

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

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

$= 1$ (normal-weight concrete)

Mean concrete strength: $f_c' = (f_c'_{\text{jacket}} \cdot \text{Area}_{\text{jacket}} + f_c'_{\text{core}} \cdot \text{Area}_{\text{core}}) / \text{Area}_{\text{section}} = 28.32$, but $f_c'^{0.5} \leq 8.3$ MPa (22.5.3.1, ACI 318-14)

$M/Vd = 2.00$

$\mu_u = 1.4802042E-011$

$\nu_u = 1.4969033E-031$

$d = 0.8 \cdot D = 400.00$

$N_u = 7389.214$

$A_g = 196349.541$

From (11.5.4.8), ACI 318-14: $V_s = V_{s1} + V_{s2} = 274155.678$

$V_{s1} = 274155.678$ is calculated for jacket, with:

$A_v = \rho_s \cdot A_{\text{stirrup}} = 123370.055$

$f_y = 555.5556$

$s = 100.00$

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

$s/d = 0.25$

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

$A_v = \rho_s \cdot A_{\text{stirrup}} = 78956.835$

$f_y = 444.4444$

$s = 250.00$

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: $V_s + V_f \leq 444245.712$

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

Calculation of Shear Strength at edge 2, $V_{r2} = 483868.491$

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

$V_{ColO} = 483868.491$

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

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

= 1 (normal-weight concrete)
Mean concrete strength: $f_c' = (f_c'_{\text{jacket}} \cdot \text{Area}_{\text{jacket}} + f_c'_{\text{core}} \cdot \text{Area}_{\text{core}}) / \text{Area}_{\text{section}} = 28.32$, but $f_c'^{0.5} \leq 8.3$ MPa (22.5.3.1, ACI 318-14)
 $M/Vd = 2.00$
 $\mu_u = 1.4802042\text{E-}011$
 $\mu_v = 1.4969033\text{E-}031$
 $d = 0.8 \cdot D = 400.00$
 $N_u = 7389.214$
 $A_g = 196349.541$
From (11.5.4.8), ACI 318-14: $V_s = V_{s1} + V_{s2} = 274155.678$
 $V_{s1} = 274155.678$ is calculated for jacket, with:
 $A_v = \frac{1}{2} \cdot A_{\text{stirrup}} = 123370.055$
 $f_y = 555.5556$
 $s = 100.00$
 V_{s1} is multiplied by $\text{Col1} = 1.00$
 $s/d = 0.25$
 $V_{s2} = 0.00$ is calculated for core, with:
 $A_v = \frac{1}{2} \cdot A_{\text{stirrup}} = 78956.835$
 $f_y = 444.4444$
 $s = 250.00$
 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: $V_s + V_f \leq 444245.712$
 $b_w \cdot d = \frac{1}{4} \cdot d \cdot d = 125663.706$

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, $\phi = 1.00$
Mean strength values are used for both shear and moment calculations.
Consequently:
Jacket
New material of Primary Member: Concrete Strength, $f_c = f_{cm} = 33.00$
New material of Primary Member: Steel Strength, $f_s = f_{sm} = 555.5556$
Concrete Elasticity, $E_c = 26999.444$
Steel Elasticity, $E_s = 200000.00$
Existing Column
Existing material of Primary Member: Concrete Strength, $f_c = f_{cm} = 20.00$
Existing material of Primary Member: Steel Strength, $f_s = f_{sm} = 444.4444$
Concrete Elasticity, $E_c = 21019.039$
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, $f_s = 1.25 \cdot f_{sm} = 694.4444$
Existing Column
Existing material: Steel Strength, $f_s = 1.25 \cdot f_{sm} = 555.5556$

External Diameter, $D = 500.00$
Internal Diameter, $D = 300.00$
Cover Thickness, $c = 25.00$
Mean Confinement Factor overall section = 1.30349
Element Length, $L = 3000.00$
Primary Member

Ribbed Bars
 Ductile Steel
 With Detailing for Earthquake Resistance (including stirrups closed at 135°)
 Longitudinal Bars With Ends Lapped Starting at the End Sections
 Adequate Lap Length ($l_o/l_{ou,min} \geq 1$)
 No FRP Wrapping

Stepwise Properties

At local axis: 2
 EDGE -A-
 Shear Force, $V_a = -2.6832056E-030$
 EDGE -B-
 Shear Force, $V_b = 2.6832056E-030$
 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, $V_e/V_r = 0.5636717$
 Member Controlled by Flexure ($V_e/V_r < 1$)
 Calculation of Shear Demand from fig. R18.6.5, ACI 318-14 $V_e = (M_{pr1} + M_{pr2})/l_n = 272742.977$
 with
 $M_{pr1} = \max(\mu_{u1+}, \mu_{u1-}) = 4.0911E+008$
 $\mu_{u1+} = 4.0911E+008$, 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
 $\mu_{u1-} = 4.0911E+008$, 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
 $M_{pr2} = \max(\mu_{u2+}, \mu_{u2-}) = 4.0911E+008$
 $\mu_{u2+} = 4.0911E+008$, 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
 $\mu_{u2-} = 4.0911E+008$, 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
 $\mu_u = 4.0911E+008$

$\phi = 0.97738438$
 $\phi' = 0.86668818$
 error of function (3.68), Biskinis Phd = 94699.84
 From 5A.2, TBDY: $f_{cc} = f_c' \cdot c = 43.01524$
 conf. factor $c = 1.30349$
 $f_c = 33.00$
 From 10.3.5, ASCE 41-17, Final value of f_y : $f_y \cdot \min(1, 1.25 \cdot (l_b/l_d)^{2/3}) = 694.4444$
 $l_b/l_d = 1.00$
 $d_1 = 44.00$
 $R = 250.00$
 $v = 0.00114003$
 $N = 7389.214$
 $A_c = 196349.541$
 $\phi' \cdot \min(1, 1.25 \cdot (l_b/l_d)^{2/3}) = 0.26181818$

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
 $\mu = 4.0911E+008$

$$= 0.97738438$$

$$' = 0.86668818$$

error of function (3.68), Biskinis Phd = 94699.84

From 5A.2, TDY: $f_{cc} = f_c \cdot c = 43.01524$

conf. factor $c = 1.30349$

$f_c = 33.00$

From 10.3.5, ASCE 41-17, Final value of f_y : $f_y \cdot \text{Min}(1, 1.25 \cdot (l_b/d)^{2/3}) = 694.4444$

$$l_b/d = 1.00$$

$$d_1 = 44.00$$

$$R = 250.00$$

$$v = 0.00114003$$

$$N = 7389.214$$

$$A_c = 196349.541$$

$$= \cdot \text{Min}(1, 1.25 \cdot (l_b/d)^{2/3}) = 0.26181818$$

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
 $\mu = 4.0911E+008$

$$= 0.97738438$$

$$' = 0.86668818$$

error of function (3.68), Biskinis Phd = 94699.84

From 5A.2, TDY: $f_{cc} = f_c \cdot c = 43.01524$

conf. factor $c = 1.30349$

$f_c = 33.00$

From 10.3.5, ASCE 41-17, Final value of f_y : $f_y \cdot \text{Min}(1, 1.25 \cdot (l_b/d)^{2/3}) = 694.4444$

$$l_b/d = 1.00$$

$$d_1 = 44.00$$

$$R = 250.00$$

$$v = 0.00114003$$

$$N = 7389.214$$

$$A_c = 196349.541$$

$$= \cdot \text{Min}(1, 1.25 \cdot (l_b/d)^{2/3}) = 0.26181818$$

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 = 4.0911E+008

= 0.97738438
' = 0.86668818
error of function (3.68), Biskinis Phd = 94699.84
From 5A.2, TBDY: $f_{cc} = f_c' \cdot c = 43.01524$
conf. factor $c = 1.30349$
 $f_c = 33.00$
From 10.3.5, ASCE 41-17, Final value of f_y : $f_y \cdot \text{Min}(1, 1.25 \cdot (l_b/d)^{2/3}) = 694.4444$
 $l_b/d = 1.00$
 $d_1 = 44.00$
 $R = 250.00$
 $v = 0.00114003$
 $N = 7389.214$
 $A_c = 196349.541$
= $\cdot \text{Min}(1, 1.25 \cdot (l_b/d)^{2/3}) = 0.26181818$

Calculation of ratio l_b/d

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

Calculation of Shear Strength $V_r = \text{Min}(V_{r1}, V_{r2}) = 483868.491$

Calculation of Shear Strength at edge 1, $V_{r1} = 483868.491$
 $V_{r1} = V_{Col} \text{ ((10.3), ASCE 41-17)} = k_{nl} \cdot V_{ColO}$
 $V_{ColO} = 483868.491$
 $k_{nl} = 1$ (zero step-static loading)

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

= 1 (normal-weight concrete)
Mean concrete strength: $f_c' = (f_c'_{\text{jacket}} \cdot \text{Area}_{\text{jacket}} + f_c'_{\text{core}} \cdot \text{Area}_{\text{core}}) / \text{Area}_{\text{section}} = 28.32$, but $f_c'^{0.5} \leq 8.3$
MPa (22.5.3.1, ACI 318-14)
 $M/Vd = 2.00$
 $M_u = 2.6359963E-011$
 $V_u = 2.6832056E-030$
 $d = 0.8 \cdot D = 400.00$
 $N_u = 7389.214$
 $A_g = 196349.541$
From (11.5.4.8), ACI 318-14: $V_s = V_{s1} + V_{s2} = 274155.678$
 $V_{s1} = 274155.678$ is calculated for jacket, with:
 $A_v = \sqrt{2} \cdot A_{\text{stirrup}} = 123370.055$
 $f_y = 555.5556$
 $s = 100.00$
 V_{s1} is multiplied by $\text{Col1} = 1.00$
 $s/d = 0.25$
 $V_{s2} = 0.00$ is calculated for core, with:
 $A_v = \sqrt{2} \cdot A_{\text{stirrup}} = 78956.835$
 $f_y = 444.4444$
 $s = 250.00$
 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: $V_s + V_f \leq 444245.712$
 $b_w \cdot d = \cdot d \cdot d/4 = 125663.706$

Calculation of Shear Strength at edge 2, $V_{r2} = 483868.491$
 $V_{r2} = V_{Col} \text{ ((10.3), ASCE 41-17)} = k_{nl} \cdot V_{ColO}$
 $V_{ColO} = 483868.491$

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: $f_c' = (f_c'_{\text{jacket}} \cdot \text{Area}_{\text{jacket}} + f_c'_{\text{core}} \cdot \text{Area}_{\text{core}}) / \text{Area}_{\text{section}} = 28.32$, but $f_c'^{0.5} \leq 8.3$
MPa (22.5.3.1, ACI 318-14)

$M/Vd = 2.00$

$\mu_u = 2.6359963E-011$

$\nu_u = 2.6832056E-030$

$d = 0.8 \cdot D = 400.00$

$N_u = 7389.214$

$A_g = 196349.541$

From (11.5.4.8), ACI 318-14: $V_s = V_{s1} + V_{s2} = 274155.678$

$V_{s1} = 274155.678$ is calculated for jacket, with:

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

$f_y = 555.5556$

$s = 100.00$

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

$s/d = 0.25$

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

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

$f_y = 444.4444$

$s = 250.00$

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: $V_s + V_f \leq 444245.712$

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

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: 3

Integration Section: (a)

Section Type: rcjcs

Constant Properties

Knowledge Factor, = 1.00

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 Primary Member: Concrete Strength, $f_c = f_{cm} = 33.00$

New material of Primary Member: Steel Strength, $f_s = f_{sm} = 555.5556$

Concrete Elasticity, $E_c = 26999.444$

Steel Elasticity, $E_s = 200000.00$

Existing Column

Existing material of Primary Member: Concrete Strength, $f_c = f_{cm} = 20.00$

Existing material of Primary Member: Steel Strength, $f_s = f_{sm} = 444.4444$

Concrete Elasticity, $E_c = 21019.039$

Steel Elasticity, $E_s = 200000.00$

External Diameter, $D = 500.00$

Internal Diameter, $D = 300.00$

Cover Thickness, $c = 25.00$

Element Length, $L = 3000.00$

Primary Member

Ribbed Bars

Ductile Steel

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

Longitudinal Bars With Ends Lapped Starting at the End Sections
Adequate Lap Length ($l_b/l_d \geq 1$)
No FRP Wrapping

Stepwise Properties

Bending Moment, $M = -1.4007\text{E}+007$
Shear Force, $V2 = -4667.691$
Shear Force, $V3 = -8.7343036\text{E}-015$
Axial Force, $F = -7387.347$
Longitudinal Reinforcement Area Distribution (in 2 divisions)
-Tension: $A_{st} = 0.00$
-Compression: $A_{sc} = 3053.628$
Longitudinal Reinforcement Area Distribution (in 3 divisions)
-Tension: $A_{st,ten} = 1017.876$
-Compression: $A_{sc,com} = 1017.876$
-Middle: $A_{st,mid} = 1017.876$
Mean Diameter of Tension Reinforcement, $D_bL = 18.00$

Existing component: From table 7-7, ASCE 41_17: Final chord rotation Capacity $\phi_{u,R} = \phi_u = 0.02002587$
 $\phi_u = \phi_y + \phi_p = 0.02002587$

- Calculation of ϕ_y -

$\phi_y = (M_y \cdot L_s / 3) / E_{eff} = 0.01502587$ ((4.29), Biskinis Phd))
 $M_y = 3.6258\text{E}+008$
 $L_s = M/V$ (with $L_s > 0.1 \cdot L$ and $L_s < 2 \cdot L$) = 3000.755
From table 10.5, ASCE 41_17: $E_{eff} = \text{factor} \cdot E_c \cdot I_g = 2.4137\text{E}+013$
factor = 0.30
 $A_g = 196349.541$
Mean concrete strength: $f'_c = (f'_{c,jacket} \cdot \text{Area}_{jacket} + f'_{c,core} \cdot \text{Area}_{core}) / \text{Area}_{section} = 28.32$
 $N = 7387.347$
 $E_c \cdot I_g = E_{c,jacket} \cdot I_{g,jacket} + E_{c,core} \cdot I_{g,core} = 8.0455\text{E}+013$

Calculation of Yielding Moment M_y

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

$M_y = \min(M_{y,ten}, M_{y,com}) = 3.6258\text{E}+008$
 ϕ_y ((10a) or (10b)) = $1.0622220\text{E}-005$
 $M_{y,ten}$ (8a) = $3.6258\text{E}+008$
 $\phi_{y,ten}$ (7a) = 65.43627
error of function (7a) = 0.00293095
 $M_{y,com}$ (8b) = $7.5621\text{E}+008$
 $\phi_{y,com}$ (7b) = 64.56804
error of function (7b) = -0.00721905
with $e_y = 0.00277778$
 $e_{co} = 0.002$
 $a_{pl} = 0.35$ ((9a) in Biskinis and Fardis for no FRP Wrap)
 $d1 = 44.00$
 $R = 250.00$
 $v = 0.0011401$
 $N = 7387.347$
 $A_c = 196349.541$
= 0.26181818
with $f_c = 33.00$

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 $V_{yE}/V_{Col0E} = 0.5636717$

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

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

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

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

where $f = 2 \cdot t_f/bw$ 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.

$NUD = 7387.347$

$A_g = 196349.541$

$f_{cE} = (f_{c,jacket} \cdot \text{Area}_{jacket} + f_{c,core} \cdot \text{Area}_{core})/\text{section_area} = 28.32$

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

$f_{yIE} = (f_{y,ext_Trans_Reinf} \cdot \text{Area}_{ext_Trans_Reinf} + f_{y,int_Trans_Reinf} \cdot \text{Area}_{int_Trans_Reinf})/\text{Area}_{Tot_Trans_Rein} = 539.4201$

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

$f_{cE} = 28.32$

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

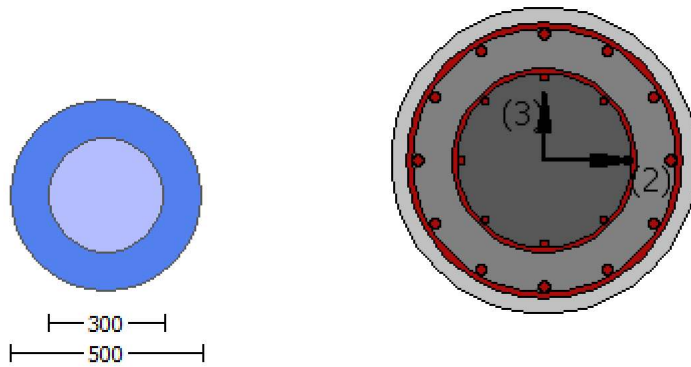
Limit State: Operational Level (data interpolation between analysis steps 1 and 2)

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, $\gamma = 1.00$

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 Primary Member: Concrete Strength, $f_c = f_{c_lower_bound} = 25.00$

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

Concrete Elasticity, $E_c = 26999.444$

Steel Elasticity, $E_s = 200000.00$

Existing Column

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

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

Concrete Elasticity, $E_c = 21019.039$

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, $f_c = f_{cm} = 33.00$

New material: Steel Strength, $f_s = f_{sm} = 555.5556$

Existing Column

Existing material: Concrete Strength, $f_c = f_{cm} = 20.00$

Existing material: Steel Strength, $f_s = f_{sm} = 444.4444$

#####

External Diameter, $D = 500.00$

Internal Diameter, $D = 300.00$

Cover Thickness, $c = 25.00$

Element Length, $L = 3000.00$

Primary Member

Ribbed Bars

Ductile Steel

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

Longitudinal Bars With Ends Lapped Starting at the End Sections

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

No FRP Wrapping

Stepwise Properties

EDGE -A-

Bending Moment, $M_a = -1.4007E+007$

Shear Force, $V_a = -4667.691$

EDGE -B-

Bending Moment, $M_b = 0.03183135$

Shear Force, $V_b = 4667.691$
 BOTH EDGES
 Axial Force, $F = -7387.347$
 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 $V_R = V_n = 430747.15$
 $V_n ((10.3), ASCE 41-17) = knl * V_{Col0} = 430747.15$
 $V_{Col} = 430747.15$
 $knl = 1.00$
 $displacement_ductility_demand = 0.05793927$

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).

$= 1$ (normal-weight concrete)
 Mean concrete strength: $fc' = (fc'_{jacket} * Area_{jacket} + fc'_{core} * Area_{core}) / Area_{section} = 21.76$, but $fc'^{0.5} \leq 8.3$
 MPa (22.5.3.1, ACI 318-14)
 $M/Vd = 2.00$
 $\mu_u = 0.03183135$
 $V_u = 4667.691$
 $d = 0.8 * D = 400.00$
 $N_u = 7387.347$
 $A_g = 196349.541$
 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:
 $A_v = A_{stirrup} / 2 = 78956.835$
 $f_y = 400.00$
 $s = 250.00$
 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: $V_s + V_f \leq 389409.072$
 $b_w * d = 125663.706$

$displacement_ductility_demand$ is calculated as ϕ / y

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

From analysis, chord rotation $\phi = 8.7036909E-005$
 $y = (M_y * L_s / 3) / E_{eff} = 0.00150221 ((4.29), Biskinis Phd)$
 $M_y = 3.6258E+008$
 $L_s = M/V$ (with $L_s > 0.1 * L$ and $L_s < 2 * L$) = 300.00
 From table 10.5, ASCE 41_17: $E_{eff} = factor * E_c * I_g = 2.4137E+013$
 $factor = 0.30$
 $A_g = 196349.541$
 Mean concrete strength: $fc' = (fc'_{jacket} * Area_{jacket} + fc'_{core} * Area_{core}) / Area_{section} = 28.32$
 $N = 7387.347$
 $E_c * I_g = E_{c,jacket} * I_{g,jacket} + E_{c,core} * I_{g,core} = 8.0455E+013$

Calculation of Yielding Moment M_y

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

$M_y = \min(M_{y_ten}, M_{y_com}) = 3.6258E+008$

$\rho_y ((10a) \text{ or } (10b)) = 1.0622220E-005$

$M_{y_ten} (8a) = 3.6258E+008$

$\rho_{y_ten} (7a) = 65.43627$

error of function (7a) = 0.00293095

$M_{y_com} (8b) = 7.5621E+008$

$\rho_{y_com} (7b) = 64.56804$

error of function (7b) = -0.00721905

with $e_y = 0.00277778$

$e_{co} = 0.002$

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

$d_1 = 44.00$

$R = 250.00$

$v = 0.0011401$

$N = 7387.347$

$A_c = 196349.541$

$= 0.26181818$

with $f_c = 33.00$

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

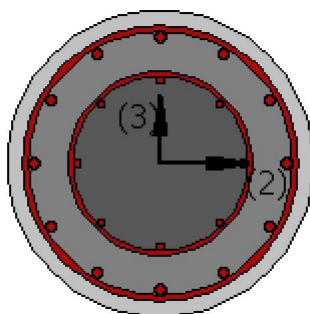
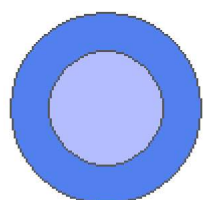
Limit State: Operational Level (data interpolation between analysis steps 1 and 2)

Analysis: Uniform +X

Check: Chord rotation capacity (ϕ_u)

Edge: End

Local Axis: (2)



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, $\gamma = 1.00$

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

Consequently:

Jacket

New material of Primary Member: Concrete Strength, $f_c = f_{cm} = 33.00$

New material of Primary Member: Steel Strength, $f_s = f_{sm} = 555.5556$

Concrete Elasticity, $E_c = 26999.444$

Steel Elasticity, $E_s = 200000.00$

Existing Column

Existing material of Primary Member: Concrete Strength, $f_c = f_{cm} = 20.00$

Existing material of Primary Member: Steel Strength, $f_s = f_{sm} = 444.4444$

Concrete Elasticity, $E_c = 21019.039$

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, $f_s = 1.25 \cdot f_{sm} = 694.4444$

Existing Column

Existing material: Steel Strength, $f_s = 1.25 \cdot f_{sm} = 555.5556$

#####

External Diameter, $D = 500.00$

Internal Diameter, $D = 300.00$

Cover Thickness, $c = 25.00$

Mean Confinement Factor overall section = 1.30349

Element Length, $L = 3000.00$

Primary Member

Ribbed Bars

Ductile Steel

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

Longitudinal Bars With Ends Lapped Starting at the End Sections

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

No FRP Wrapping

Stepwise Properties

At local axis: 3

EDGE -A-

Shear Force, $V_a = -1.4969033E-031$

EDGE -B-

Shear Force, $V_b = 1.4969033E-031$

BOTH EDGES

Axial Force, $F = -7389.214$

Longitudinal Reinforcement Area Distribution (in 2 divisions)

-Tension: Aslt = 0.00
 -Compression: Aslc = 3053.628
 Longitudinal Reinforcement Area Distribution (in 3 divisions)
 -Tension: Asl,ten = 1017.876
 -Compression: Asl,com = 1017.876
 -Middle: Asl,mid = 1017.876

Calculation of Shear Capacity ratio , $V_e/V_r = 0.5636717$
 Member Controlled by Flexure ($V_e/V_r < 1$)
 Calculation of Shear Demand from fig. R18.6.5, ACI 318-14 $V_e = (M_{pr1} + M_{pr2})/l_n = 272742.977$
 with
 $M_{pr1} = \text{Max}(Mu_{1+} , Mu_{1-}) = 4.0911E+008$
 $Mu_{1+} = 4.0911E+008$, 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
 $Mu_{1-} = 4.0911E+008$, 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
 $M_{pr2} = \text{Max}(Mu_{2+} , Mu_{2-}) = 4.0911E+008$
 $Mu_{2+} = 4.0911E+008$, 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
 $Mu_{2-} = 4.0911E+008$, 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
 $Mu = 4.0911E+008$

$\phi = 0.97738438$
 $\lambda = 0.86668818$
 error of function (3.68), Biskinis Phd = 94699.84
 From 5A.2, TBDY: $f_{cc} = f_c^* c = 43.01524$
 conf. factor $c = 1.30349$
 $f_c = 33.00$
 From 10.3.5, ASCE 41-17, Final value of f_y : $f_y * \text{Min}(1, 1.25 * (l_b/d)^{2/3}) = 694.4444$
 $l_b/d = 1.00$
 $d_1 = 44.00$
 $R = 250.00$
 $v = 0.00114003$
 $N = 7389.214$
 $A_c = 196349.541$
 $\phi * \text{Min}(1, 1.25 * (l_b/d)^{2/3}) = 0.26181818$

Calculation of ratio l_b/d

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

Calculation of Mu_{1-}

Calculation of ultimate Moment Strength ((3.67), Biskinis Phd), Mu
 $Mu = 4.0911E+008$

$\phi = 0.97738438$
 $\lambda = 0.86668818$
 error of function (3.68), Biskinis Phd = 94699.84
 From 5A.2, TBDY: $f_{cc} = f_c^* c = 43.01524$
 conf. factor $c = 1.30349$

$f_c = 33.00$
 From 10.3.5, ASCE 41-17, Final value of f_y : $f_y \cdot \text{Min}(1, 1.25 \cdot (l_b/d)^{2/3}) = 694.4444$
 $l_b/d = 1.00$
 $d_1 = 44.00$
 $R = 250.00$
 $v = 0.00114003$
 $N = 7389.214$
 $Ac = 196349.541$
 $= \cdot \text{Min}(1, 1.25 \cdot (l_b/d)^{2/3}) = 0.26181818$

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
 $\mu = 4.0911\text{E}+008$

$= 0.97738438$
 $' = 0.86668818$
 error of function (3.68), Biskinis Phd = 94699.84
 From 5A.2, TBDY: $f_{cc} = f_c \cdot c = 43.01524$
 conf. factor $c = 1.30349$
 $f_c = 33.00$
 From 10.3.5, ASCE 41-17, Final value of f_y : $f_y \cdot \text{Min}(1, 1.25 \cdot (l_b/d)^{2/3}) = 694.4444$
 $l_b/d = 1.00$
 $d_1 = 44.00$
 $R = 250.00$
 $v = 0.00114003$
 $N = 7389.214$
 $Ac = 196349.541$
 $= \cdot \text{Min}(1, 1.25 \cdot (l_b/d)^{2/3}) = 0.26181818$

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
 $\mu = 4.0911\text{E}+008$

$= 0.97738438$
 $' = 0.86668818$
 error of function (3.68), Biskinis Phd = 94699.84
 From 5A.2, TBDY: $f_{cc} = f_c \cdot c = 43.01524$
 conf. factor $c = 1.30349$
 $f_c = 33.00$
 From 10.3.5, ASCE 41-17, Final value of f_y : $f_y \cdot \text{Min}(1, 1.25 \cdot (l_b/d)^{2/3}) = 694.4444$
 $l_b/d = 1.00$
 $d_1 = 44.00$
 $R = 250.00$
 $v = 0.00114003$
 $N = 7389.214$
 $Ac = 196349.541$
 $= \cdot \text{Min}(1, 1.25 \cdot (l_b/d)^{2/3}) = 0.26181818$

Calculation of ratio l_b/d

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

Calculation of Shear Strength $V_r = \text{Min}(V_{r1}, V_{r2}) = 483868.491$

Calculation of Shear Strength at edge 1, $V_{r1} = 483868.491$

$V_{r1} = V_{col} \text{ ((10.3), ASCE 41-17)} = k_n l V_{col0}$

$V_{col0} = 483868.491$

$k_n l = 1$ (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: $f'_c = (f'_{c_jacket} \text{Area}_{jacket} + f'_{c_core} \text{Area}_{core}) / \text{Area}_{section} = 28.32$, but $f'_c^{0.5} \leq 8.3$ MPa (22.5.3.1, ACI 318-14)

$M/Vd = 2.00$

$\mu_u = 1.4802042E-011$

$\nu_u = 1.4969033E-031$

$d = 0.8 \cdot D = 400.00$

$N_u = 7389.214$

$A_g = 196349.541$

From (11.5.4.8), ACI 318-14: $V_s = V_{s1} + V_{s2} = 274155.678$

$V_{s1} = 274155.678$ is calculated for jacket, with:

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

$f_y = 555.5556$

$s = 100.00$

V_{s1} is multiplied by $\phi_{col1} = 1.00$

$s/d = 0.25$

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

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

$f_y = 444.4444$

$s = 250.00$

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: $V_s + V_f \leq 444245.712$

$b_w d = A_{stirrup} d / 4 = 125663.706$

Calculation of Shear Strength at edge 2, $V_{r2} = 483868.491$

$V_{r2} = V_{col} \text{ ((10.3), ASCE 41-17)} = k_n l V_{col0}$

$V_{col0} = 483868.491$

$k_n l = 1$ (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: $f'_c = (f'_{c_jacket} \text{Area}_{jacket} + f'_{c_core} \text{Area}_{core}) / \text{Area}_{section} = 28.32$, but $f'_c^{0.5} \leq 8.3$ MPa (22.5.3.1, ACI 318-14)

$M/Vd = 2.00$

$\mu_u = 1.4802042E-011$

$\nu_u = 1.4969033E-031$

$d = 0.8 \cdot D = 400.00$

$N_u = 7389.214$

$A_g = 196349.541$

From (11.5.4.8), ACI 318-14: $V_s = V_{s1} + V_{s2} = 274155.678$

$V_{s1} = 274155.678$ is calculated for jacket, with:

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

$f_y = 555.5556$

$s = 100.00$
 $Vs1$ is multiplied by $Col1 = 1.00$
 $s/d = 0.25$
 $Vs2 = 0.00$ is calculated for core, with:
 $Av = \frac{1}{2} A_{stirrup} = 78956.835$
 $fy = 444.4444$
 $s = 250.00$
 $Vs2$ is multiplied by $Col2 = 0.00$
 $s/d = 1.04167$
 $Vf ((11-3)-(11.4), ACI 440) = 0.00$
 From $(11-11)$, ACI 440: $Vs + Vf \leq 444245.712$
 $bw*d = \frac{1}{4} d*d = 125663.706$

End Of Calculation of Shear Capacity ratio for element: column JCC1 of floor 1
 At local axis: 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, $\phi = 1.00$
 Mean strength values are used for both shear and moment calculations.
 Consequently:
 Jacket
 New material of Primary Member: Concrete Strength, $fc = f_{cm} = 33.00$
 New material of Primary Member: Steel Strength, $fs = f_{sm} = 555.5556$
 Concrete Elasticity, $Ec = 26999.444$
 Steel Elasticity, $Es = 200000.00$
 Existing Column
 Existing material of Primary Member: Concrete Strength, $fc = f_{cm} = 20.00$
 Existing material of Primary Member: Steel Strength, $fs = f_{sm} = 444.4444$
 Concrete Elasticity, $Ec = 21019.039$
 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, $fs = 1.25*f_{sm} = 694.4444$
 Existing Column
 Existing material: Steel Strength, $fs = 1.25*f_{sm} = 555.5556$
 #####
 External Diameter, $D = 500.00$
 Internal Diameter, $D = 300.00$
 Cover Thickness, $c = 25.00$
 Mean Confinement Factor overall section = 1.30349
 Element Length, $L = 3000.00$
 Primary Member
 Ribbed Bars
 Ductile Steel
 With Detailing for Earthquake Resistance (including stirrups closed at 135°)
 Longitudinal Bars With Ends Lapped Starting at the End Sections
 Adequate Lap Length ($l_o/l_{ou}, \min \geq 1$)
 No FRP Wrapping

Stepwise Properties

At local axis: 2
 EDGE -A-
 Shear Force, $Va = -2.6832056E-030$

EDGE -B-

Shear Force, $V_b = 2.6832056E-030$

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, $V_e/V_r = 0.5636717$

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

Calculation of Shear Demand from fig. R18.6.5, ACI 318-14 $V_e = (M_{pr1} + M_{pr2})/l_n = 272742.977$
with

$M_{pr1} = \text{Max}(Mu_{1+}, Mu_{1-}) = 4.0911E+008$

$Mu_{1+} = 4.0911E+008$, 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

$Mu_{1-} = 4.0911E+008$, 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

$M_{pr2} = \text{Max}(Mu_{2+}, Mu_{2-}) = 4.0911E+008$

$Mu_{2+} = 4.0911E+008$, 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

$Mu_{2-} = 4.0911E+008$, 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), Mu

$Mu = 4.0911E+008$

$= 0.97738438$

$' = 0.86668818$

error of function (3.68), Biskinis Phd = 94699.84

From 5A.2, TBDY: $f_{cc} = f_c' \cdot c = 43.01524$

conf. factor $c = 1.30349$

$f_c = 33.00$

From 10.3.5, ASCE 41-17, Final value of f_y : $f_y \cdot \text{Min}(1, 1.25 \cdot (l_b/l_d)^{2/3}) = 694.4444$

$l_b/l_d = 1.00$

$d_1 = 44.00$

$R = 250.00$

$v = 0.00114003$

$N = 7389.214$

$A_c = 196349.541$

$= \cdot \text{Min}(1, 1.25 \cdot (l_b/l_d)^{2/3}) = 0.26181818$

Calculation of ratio l_b/l_d

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

Calculation of Mu_{1-}

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

$Mu = 4.0911E+008$

```

= 0.97738438
' = 0.86668818
error of function (3.68), Biskinis Phd = 94699.84
From 5A.2, TBDY: fcc = fc* c = 43.01524
conf. factor c = 1.30349
fc = 33.00
From 10.3.5, ASCE 41-17, Final value of fy: fy*Min(1,1.25*(lb/d)^ 2/3) = 694.4444
lb/d = 1.00
d1 = 44.00
R = 250.00
v = 0.00114003
N = 7389.214
Ac = 196349.541
= *Min(1,1.25*(lb/d)^ 2/3) = 0.26181818

```

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 = 4.0911E+008

```

= 0.97738438
' = 0.86668818
error of function (3.68), Biskinis Phd = 94699.84
From 5A.2, TBDY: fcc = fc* c = 43.01524
conf. factor c = 1.30349
fc = 33.00
From 10.3.5, ASCE 41-17, Final value of fy: fy*Min(1,1.25*(lb/d)^ 2/3) = 694.4444
lb/d = 1.00
d1 = 44.00
R = 250.00
v = 0.00114003
N = 7389.214
Ac = 196349.541
= *Min(1,1.25*(lb/d)^ 2/3) = 0.26181818

```

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 = 4.0911E+008

```

= 0.97738438
' = 0.86668818
error of function (3.68), Biskinis Phd = 94699.84
From 5A.2, TBDY: fcc = fc* c = 43.01524
conf. factor c = 1.30349
fc = 33.00
From 10.3.5, ASCE 41-17, Final value of fy: fy*Min(1,1.25*(lb/d)^ 2/3) = 694.4444
lb/d = 1.00
d1 = 44.00

```

$R = 250.00$
 $v = 0.00114003$
 $N = 7389.214$
 $Ac = 196349.541$
 $= \text{Min}(1, 1.25 \cdot (lb/d)^{2/3}) = 0.26181818$

Calculation of ratio lb/d

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

Calculation of Shear Strength $V_r = \text{Min}(V_{r1}, V_{r2}) = 483868.491$

Calculation of Shear Strength at edge 1, $V_{r1} = 483868.491$

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

$V_{ColO} = 483868.491$

$knl = 1$ (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: $f'_c = (f'_c \cdot \text{jacket} \cdot \text{Area}_{\text{jacket}} + f'_c \cdot \text{core} \cdot \text{Area}_{\text{core}}) / \text{Area}_{\text{section}} = 28.32$, but $f'_c^{0.5} \leq 8.3$ MPa (22.5.3.1, ACI 318-14)

$M/Vd = 2.00$

$\mu_u = 2.6359963E-011$

$\nu_u = 2.6832056E-030$

$d = 0.8 \cdot D = 400.00$

$N_u = 7389.214$

$A_g = 196349.541$

From (11.5.4.8), ACI 318-14: $V_s = V_{s1} + V_{s2} = 274155.678$

$V_{s1} = 274155.678$ is calculated for jacket, with:

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

$f_y = 555.5556$

$s = 100.00$

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

$s/d = 0.25$

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

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

$f_y = 444.4444$

$s = 250.00$

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: $V_s + V_f \leq 444245.712$

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

Calculation of Shear Strength at edge 2, $V_{r2} = 483868.491$

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

$V_{ColO} = 483868.491$

$knl = 1$ (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: $f'_c = (f'_c \cdot \text{jacket} \cdot \text{Area}_{\text{jacket}} + f'_c \cdot \text{core} \cdot \text{Area}_{\text{core}}) / \text{Area}_{\text{section}} = 28.32$, but $f'_c^{0.5} \leq 8.3$ MPa (22.5.3.1, ACI 318-14)

$M/Vd = 2.00$

$\mu_u = 2.6359963E-011$

$\nu_u = 2.6832056E-030$

$d = 0.8 \cdot D = 400.00$

$N_u = 7389.214$

$A_g = 196349.541$
 From (11.5.4.8), ACI 318-14: $V_s = V_{s1} + V_{s2} = 274155.678$
 $V_{s1} = 274155.678$ is calculated for jacket, with:
 $A_v = \frac{1}{2} A_{stirrup} = 123370.055$
 $f_y = 555.5556$
 $s = 100.00$
 V_{s1} is multiplied by $Col1 = 1.00$
 $s/d = 0.25$
 $V_{s2} = 0.00$ is calculated for core, with:
 $A_v = \frac{1}{2} A_{stirrup} = 78956.835$
 $f_y = 444.4444$
 $s = 250.00$
 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: $V_s + V_f \leq 444245.712$
 $b_w * d = \frac{1}{4} * d * d = 125663.706$

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, $\phi = 1.00$
 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 Primary Member: Concrete Strength, $f_c = f_{cm} = 33.00$
 New material of Primary Member: Steel Strength, $f_s = f_{sm} = 555.5556$
 Concrete Elasticity, $E_c = 26999.444$
 Steel Elasticity, $E_s = 200000.00$
 Existing Column
 Existing material of Primary Member: Concrete Strength, $f_c = f_{cm} = 20.00$
 Existing material of Primary Member: Steel Strength, $f_s = f_{sm} = 444.4444$
 Concrete Elasticity, $E_c = 21019.039$
 Steel Elasticity, $E_s = 200000.00$
 External Diameter, $D = 500.00$
 Internal Diameter, $D = 300.00$
 Cover Thickness, $c = 25.00$
 Element Length, $L = 3000.00$
 Primary Member
 Ribbed Bars
 Ductile Steel
 With Detailing for Earthquake Resistance (including stirrups closed at 135°)
 Longitudinal Bars With Ends Lapped Starting at the End Sections
 Adequate Lap Length ($l_b/d \geq 1$)
 No FRP Wrapping

Stepwise Properties

Bending Moment, $M = 2.7436834E-012$
 Shear Force, $V2 = 4667.691$
 Shear Force, $V3 = 8.7343036E-015$
 Axial Force, $F = -7387.347$
 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: $As_{ten} = 1017.876$
 -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 $u_R = \gamma + p = 0.01251105$
 $u = \gamma + p = 0.01251105$

- Calculation of γ -

$\gamma = (M_y \cdot L_s / 3) / E_{eff} = 0.00751105$ ((4.29), Biskinis Phd))
 $M_y = 3.6258E+008$
 $L_s = M/V$ (with $L_s > 0.1 \cdot L$ and $L_s < 2 \cdot L$) = 1500.00
 From table 10.5, ASCE 41_17: $E_{eff} = factor \cdot E_c \cdot I_g = 2.4137E+013$
 $factor = 0.30$
 $A_g = 196349.541$
 Mean concrete strength: $fc' = (fc'_{jacket} \cdot Area_{jacket} + fc'_{core} \cdot Area_{core}) / Area_{section} = 28.32$
 $N = 7387.347$
 $E_c \cdot I_g = E_{c_jacket} \cdot I_{g_jacket} + E_{c_core} \cdot I_{g_core} = 8.0455E+013$

Calculation of Yielding Moment M_y

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

$M_y = \min(M_{y_ten}, M_{y_com}) = 3.6258E+008$
 γ ((10a) or (10b)) = 1.0622220E-005
 M_{y_ten} (8a) = 3.6258E+008
 γ_{ten} (7a) = 65.43627
 error of function (7a) = 0.00293095
 M_{y_com} (8b) = 7.5621E+008
 γ_{com} (7b) = 64.56804
 error of function (7b) = -0.00721905
 with $e_y = 0.00277778$
 $e_{co} = 0.002$
 $apl = 0.35$ ((9a) in Biskinis and Fardis for no FRP Wrap)
 $d_1 = 44.00$
 $R = 250.00$
 $v = 0.0011401$
 $N = 7387.347$
 $A_c = 196349.541$
 $= 0.26181818$
 with $fc = 33.00$

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:

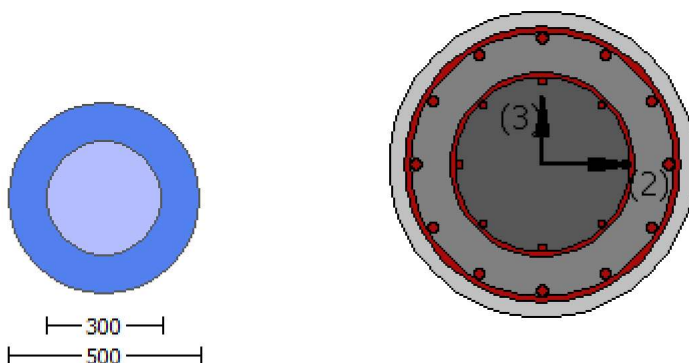
- Columns not controlled by inadequate development or splicing along the clear height because $l_b/d \geq 1$
 shear control ratio $V_y E / V_{col} E = 0.5636717$
 $d = d_{external} = 0.00$
 $s = s_{external} = 0.00$
 $t = s_1 + s_2 + 2 \cdot t_f / b_w \cdot (f_{fe} / f_s) = 0.00323428$
 jacket: $s_1 = A_{v1} \cdot (D_c / 2) / (s_1 \cdot A_g) = 0.0027646$

$Av1 = 78.53982$, 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.
 $NUD = 7387.347$
 $Ag = 196349.541$
 $f_{cE} = (f_{c,jacket} \cdot Area_{jacket} + f_{c,core} \cdot Area_{core}) / section_area = 28.32$
 $f_{yE} = (f_{y,ext_Long_Reinf} \cdot Area_{ext_Long_Reinf} + f_{y,int_Long_Reinf} \cdot Area_{int_Long_Reinf}) / Area_{Tot_Long_Rein} = 2.1219958E-314$
 $f_{yE} = (f_{y,ext_Trans_Reinf} \cdot Area_{ext_Trans_Reinf} + f_{y,int_Trans_Reinf} \cdot Area_{int_Trans_Reinf}) / Area_{Tot_Trans_Rein} = 539.4201$
 $p_l = Area_{Tot_Long_Rein} / (Ag) = 0.015552$
 $f_{cE} = 28.32$

 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, $\gamma = 1.00$
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 Primary Member: Concrete Strength, $f_c = f_{c_lower_bound} = 25.00$
New material of Primary Member: Steel Strength, $f_s = f_{s_lower_bound} = 500.00$
Concrete Elasticity, $E_c = 26999.444$
Steel Elasticity, $E_s = 200000.00$
Existing Column
Existing material of Primary Member: Concrete Strength, $f_c = f_{c_lower_bound} = 16.00$
Existing material of Primary Member: Steel Strength, $f_s = f_{s_lower_bound} = 400.00$
Concrete Elasticity, $E_c = 21019.039$
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, $f_c = f_{cm} = 33.00$
New material: Steel Strength, $f_s = f_{sm} = 555.5556$
Existing Column
Existing material: Concrete Strength, $f_c = f_{cm} = 20.00$
Existing material: Steel Strength, $f_s = f_{sm} = 444.4444$

External Diameter, $D = 500.00$
Internal Diameter, $D = 300.00$
Cover Thickness, $c = 25.00$
Element Length, $L = 3000.00$
Primary Member
Ribbed Bars
Ductile Steel
With Detailing for Earthquake Resistance (including stirrups closed at 135°)
Longitudinal Bars With Ends Lapped Starting at the End Sections
Adequate Lap Length ($l_o/l_{ou,min} = l_b/l_d \geq 1$)
No FRP Wrapping

Stepwise Properties

EDGE -A-
Bending Moment, $M_a = 2.3398468E-011$
Shear Force, $V_a = -8.7343036E-015$
EDGE -B-
Bending Moment, $M_b = 2.7436834E-012$
Shear Force, $V_b = 8.7343036E-015$
BOTH EDGES
Axial Force, $F = -7387.347$
Longitudinal Reinforcement Area Distribution (in 2 divisions)
-Tension: $A_{st} = 0.00$
-Compression: $A_{sc} = 3053.628$
Longitudinal Reinforcement Area Distribution (in 3 divisions)
-Tension: $A_{st,ten} = 1017.876$
-Compression: $A_{st,com} = 1017.876$
-Middle: $A_{st,mid} = 1017.876$
Mean Diameter of Tension Reinforcement, $Db_{L,ten} = 18.00$

Existing component: From table 7-7, ASCE 41_17: Final Shear Capacity $VR = *V_n = 430747.15$

Vn ((10.3), ASCE 41-17) = knl*VColO = 430747.15

VCol = 430747.15

knl = 1.00

displacement_ductility_demand = 0.00

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 = 21.76, but fc'^0.5 <= 8.3 MPa (22.5.3.1, ACI 318-14)

M/Vd = 2.00

Mu = 2.7436834E-012

Vu = 8.7343036E-015

d = 0.8*D = 400.00

Nu = 7387.347

Ag = 196349.541

From (11.5.4.8), ACI 318-14: Vs = Vs1 + Vs2 = 246740.11

Vs1 = 246740.11 is calculated for jacket, with:

Av = /2*A_stirrup = 123370.055

fy = 500.00

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 = 400.00

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 <= 389409.072

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 = 7.1729375E-023

y = (My*Ls/3)/Eleff = 0.00751105 ((4.29),Biskinis Phd))

My = 3.6258E+008

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

From table 10.5, ASCE 41_17: Eleff = factor*Ec*Ig = 2.4137E+013

factor = 0.30

Ag = 196349.541

Mean concrete strength: fc' = (fc'_jacket*Area_jacket + fc'_core*Area_core)/Area_section = 28.32

N = 7387.347

Ec*Ig = Ec_jacket*Ig_jacket + Ec_core*Ig_core = 8.0455E+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) = 3.6258E+008

y ((10a) or (10b)) = 1.0622220E-005

My_ten (8a) = 3.6258E+008

_ten (7a) = 65.43627

error of function (7a) = 0.00293095

My_com (8b) = 7.5621E+008

_com (7b) = 64.56804

error of function (7b) = -0.00721905

with ey = 0.00277778

$\rho_{co} = 0.002$
 $\rho_{pl} = 0.35$ ((9a) in Biskinis and Fardis for no FRP Wrap)
 $d_1 = 44.00$
 $R = 250.00$
 $v = 0.0011401$
 $N = 7387.347$
 $A_c = 196349.541$
 $= 0.26181818$
with $f_c = 33.00$

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

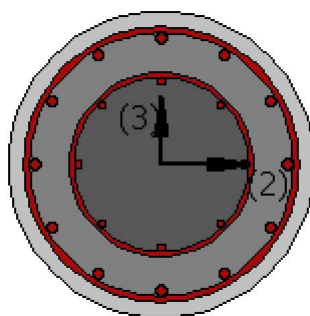
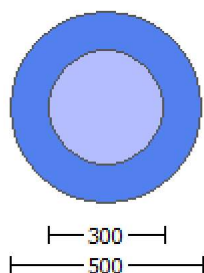
Limit State: Operational Level (data interpolation between analysis steps 1 and 2)

Analysis: Uniform +X

Check: Chord rotation capacity (ρ)

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, $\gamma = 1.00$

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

Consequently:

Jacket

New material of Primary Member: Concrete Strength, $f_c = f_{cm} = 33.00$

New material of Primary Member: Steel Strength, $f_s = f_{sm} = 555.5556$

Concrete Elasticity, $E_c = 26999.444$

Steel Elasticity, $E_s = 200000.00$

Existing Column

Existing material of Primary Member: Concrete Strength, $f_c = f_{cm} = 20.00$

Existing material of Primary Member: Steel Strength, $f_s = f_{sm} = 444.4444$

Concrete Elasticity, $E_c = 21019.039$

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, $f_s = 1.25 \cdot f_{sm} = 694.4444$

Existing Column

Existing material: Steel Strength, $f_s = 1.25 \cdot f_{sm} = 555.5556$

#####

External Diameter, $D = 500.00$

Internal Diameter, $D = 300.00$

Cover Thickness, $c = 25.00$

Mean Confinement Factor overall section = 1.30349

Element Length, $L = 3000.00$

Primary Member

Ribbed Bars

Ductile Steel

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

Longitudinal Bars With Ends Lapped Starting at the End Sections

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

No FRP Wrapping

Stepwise Properties

At local axis: 3

EDGE -A-

Shear Force, $V_a = -1.4969033E-031$

EDGE -B-

Shear Force, $V_b = 1.4969033E-031$

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: $As_{l,mid} = 1017.876$

Calculation of Shear Capacity ratio, $V_e/V_r = 0.5636717$

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

Calculation of Shear Demand from fig. R18.6.5, ACI 318-14 $V_e = (M_{pr1} + M_{pr2})/l_n = 272742.977$

with

$M_{pr1} = \max(\mu_{u1+}, \mu_{u1-}) = 4.0911E+008$

$\mu_{u1+} = 4.0911E+008$, 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

$\mu_{u1-} = 4.0911E+008$, 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

$M_{pr2} = \max(\mu_{u2+}, \mu_{u2-}) = 4.0911E+008$

$\mu_{u2+} = 4.0911E+008$, 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

$\mu_{u2-} = 4.0911E+008$, 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 = 4.0911E+008

= 0.97738438
' = 0.86668818
error of function (3.68), Biskinis Phd = 94699.84
From 5A.2, TBDY: $f_{cc} = f_c \cdot c = 43.01524$
conf. factor $c = 1.30349$
 $f_c = 33.00$
From 10.3.5, ASCE 41-17, Final value of f_y : $f_y \cdot \text{Min}(1, 1.25 \cdot (l_b/d)^{2/3}) = 694.4444$
 $l_b/d = 1.00$
 $d1 = 44.00$
 $R = 250.00$
 $v = 0.00114003$
 $N = 7389.214$
 $Ac = 196349.541$
= $\cdot \text{Min}(1, 1.25 \cdot (l_b/d)^{2/3}) = 0.26181818$

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 = 4.0911E+008

= 0.97738438
' = 0.86668818
error of function (3.68), Biskinis Phd = 94699.84
From 5A.2, TBDY: $f_{cc} = f_c \cdot c = 43.01524$
conf. factor $c = 1.30349$
 $f_c = 33.00$
From 10.3.5, ASCE 41-17, Final value of f_y : $f_y \cdot \text{Min}(1, 1.25 \cdot (l_b/d)^{2/3}) = 694.4444$
 $l_b/d = 1.00$
 $d1 = 44.00$
 $R = 250.00$
 $v = 0.00114003$
 $N = 7389.214$
 $Ac = 196349.541$
= $\cdot \text{Min}(1, 1.25 \cdot (l_b/d)^{2/3}) = 0.26181818$

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 = 4.0911E+008

= 0.97738438
 ' = 0.86668818
 error of function (3.68), Biskinis Phd = 94699.84
 From 5A.2, TBDY: $f_{cc} = f_c^* c = 43.01524$
 conf. factor $c = 1.30349$
 $f_c = 33.00$
 From 10.3.5, ASCE 41-17, Final value of f_y : $f_y \cdot \text{Min}(1, 1.25 \cdot (l_b/l_d)^{2/3}) = 694.4444$
 $l_b/l_d = 1.00$
 $d_1 = 44.00$
 $R = 250.00$
 $v = 0.00114003$
 $N = 7389.214$
 $A_c = 196349.541$
 $= \cdot \text{Min}(1, 1.25 \cdot (l_b/l_d)^{2/3}) = 0.26181818$

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
 $\mu = 4.0911\text{E}+008$

= 0.97738438
 ' = 0.86668818
 error of function (3.68), Biskinis Phd = 94699.84
 From 5A.2, TBDY: $f_{cc} = f_c^* c = 43.01524$
 conf. factor $c = 1.30349$
 $f_c = 33.00$
 From 10.3.5, ASCE 41-17, Final value of f_y : $f_y \cdot \text{Min}(1, 1.25 \cdot (l_b/l_d)^{2/3}) = 694.4444$
 $l_b/l_d = 1.00$
 $d_1 = 44.00$
 $R = 250.00$
 $v = 0.00114003$
 $N = 7389.214$
 $A_c = 196349.541$
 $= \cdot \text{Min}(1, 1.25 \cdot (l_b/l_d)^{2/3}) = 0.26181818$

Calculation of ratio l_b/l_d

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

Calculation of Shear Strength $V_r = \text{Min}(V_{r1}, V_{r2}) = 483868.491$

Calculation of Shear Strength at edge 1, $V_{r1} = 483868.491$

$V_{r1} = V_{CoI} ((10.3), \text{ASCE 41-17}) = k_{nl} \cdot V_{CoI0}$

$V_{CoI0} = 483868.491$

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

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

= 1 (normal-weight concrete)
 Mean concrete strength: $f_c' = (f_c'_{\text{jacket}} \cdot \text{Area}_{\text{jacket}} + f_c'_{\text{core}} \cdot \text{Area}_{\text{core}}) / \text{Area}_{\text{section}} = 28.32$, but $f_c'^{0.5} \leq 8.3$
 MPa (22.5.3.1, ACI 318-14)
 $M/Vd = 2.00$
 $\mu = 1.4802042\text{E}-011$

$V_u = 1.4969033E-031$
 $d = 0.8 \cdot D = 400.00$
 $N_u = 7389.214$
 $A_g = 196349.541$
 From (11.5.4.8), ACI 318-14: $V_s = V_{s1} + V_{s2} = 274155.678$
 $V_{s1} = 274155.678$ is calculated for jacket, with:
 $A_v = /2 \cdot A_{stirrup} = 123370.055$
 $f_y = 555.5556$
 $s = 100.00$
 V_{s1} is multiplied by $Col1 = 1.00$
 $s/d = 0.25$
 $V_{s2} = 0.00$ is calculated for core, with:
 $A_v = /2 \cdot A_{stirrup} = 78956.835$
 $f_y = 444.4444$
 $s = 250.00$
 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: $V_s + V_f \leq 444245.712$
 $b_w \cdot d = *d \cdot d/4 = 125663.706$

Calculation of Shear Strength at edge 2, $V_{r2} = 483868.491$
 $V_{r2} = V_{Col} ((10.3), ASCE 41-17) = knl \cdot V_{Col0}$
 $V_{Col0} = 483868.491$
 $knl = 1$ (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: $f_c' = (f_c'_{jacket} \cdot Area_{jacket} + f_c'_{core} \cdot Area_{core}) / Area_{section} = 28.32$, but $f_c'^{0.5} \leq 8.3$
 MPa (22.5.3.1, ACI 318-14)
 $M/Vd = 2.00$
 $M_u = 1.4802042E-011$
 $V_u = 1.4969033E-031$
 $d = 0.8 \cdot D = 400.00$
 $N_u = 7389.214$
 $A_g = 196349.541$
 From (11.5.4.8), ACI 318-14: $V_s = V_{s1} + V_{s2} = 274155.678$
 $V_{s1} = 274155.678$ is calculated for jacket, with:
 $A_v = /2 \cdot A_{stirrup} = 123370.055$
 $f_y = 555.5556$
 $s = 100.00$
 V_{s1} is multiplied by $Col1 = 1.00$
 $s/d = 0.25$
 $V_{s2} = 0.00$ is calculated for core, with:
 $A_v = /2 \cdot A_{stirrup} = 78956.835$
 $f_y = 444.4444$
 $s = 250.00$
 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: $V_s + V_f \leq 444245.712$
 $b_w \cdot d = *d \cdot d/4 = 125663.706$

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, $\phi = 1.00$

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

Consequently:

Jacket

New material of Primary Member: Concrete Strength, $f_c = f_{cm} = 33.00$

New material of Primary Member: Steel Strength, $f_s = f_{sm} = 555.5556$

Concrete Elasticity, $E_c = 26999.444$

Steel Elasticity, $E_s = 200000.00$

Existing Column

Existing material of Primary Member: Concrete Strength, $f_c = f_{cm} = 20.00$

Existing material of Primary Member: Steel Strength, $f_s = f_{sm} = 444.4444$

Concrete Elasticity, $E_c = 21019.039$

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, $f_s = 1.25 \cdot f_{sm} = 694.4444$

Existing Column

Existing material: Steel Strength, $f_s = 1.25 \cdot f_{sm} = 555.5556$

#####

External Diameter, $D = 500.00$

Internal Diameter, $D = 300.00$

Cover Thickness, $c = 25.00$

Mean Confinement Factor overall section = 1.30349

Element Length, $L = 3000.00$

Primary Member

Ribbed Bars

Ductile Steel

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

Longitudinal Bars With Ends Lapped Starting at the End Sections

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

No FRP Wrapping

Stepwise Properties

At local axis: 2

EDGE -A-

Shear Force, $V_a = -2.6832056E-030$

EDGE -B-

Shear Force, $V_b = 2.6832056E-030$

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: $A_{st, \text{ten}} = 1017.876$

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

-Middle: $A_{sc, \text{mid}} = 1017.876$

Calculation of Shear Capacity ratio, $V_e/V_r = 0.5636717$

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

Calculation of Shear Demand from fig. R18.6.5, ACI 318-14 $V_e = (M_{pr1} + M_{pr2})/l_n = 272742.977$

with

$M_{pr1} = \text{Max}(M_{u1+}, M_{u1-}) = 4.0911E+008$

$M_{u1+} = 4.0911E+008$, 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

$M_{u1-} = 4.0911E+008$, 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

$$M_{pr2} = \text{Max}(M_{u2+}, M_{u2-}) = 4.0911\text{E}+008$$

$M_{u2+} = 4.0911\text{E}+008$, 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

$M_{u2-} = 4.0911\text{E}+008$, 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
 $M_u = 4.0911\text{E}+008$

$$= 0.97738438$$

$$\lambda = 0.86668818$$

error of function (3.68), Biskinis Phd = 94699.84

From 5A.2, TB DY: $f_{cc} = f_c \cdot c = 43.01524$

conf. factor $c = 1.30349$

$$f_c = 33.00$$

From 10.3.5, ASCE 41-17, Final value of f_y : $f_y \cdot \text{Min}(1, 1.25 \cdot (l_b/d)^{2/3}) = 694.4444$

$$l_b/d = 1.00$$

$$d_1 = 44.00$$

$$R = 250.00$$

$$v = 0.00114003$$

$$N = 7389.214$$

$$A_c = 196349.541$$

$$= \lambda \cdot \text{Min}(1, 1.25 \cdot (l_b/d)^{2/3}) = 0.26181818$$

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
 $M_u = 4.0911\text{E}+008$

$$= 0.97738438$$

$$\lambda = 0.86668818$$

error of function (3.68), Biskinis Phd = 94699.84

From 5A.2, TB DY: $f_{cc} = f_c \cdot c = 43.01524$

conf. factor $c = 1.30349$

$$f_c = 33.00$$

From 10.3.5, ASCE 41-17, Final value of f_y : $f_y \cdot \text{Min}(1, 1.25 \cdot (l_b/d)^{2/3}) = 694.4444$

$$l_b/d = 1.00$$

$$d_1 = 44.00$$

$$R = 250.00$$

$$v = 0.00114003$$

$$N = 7389.214$$

$$A_c = 196349.541$$

$$= \lambda \cdot \text{Min}(1, 1.25 \cdot (l_b/d)^{2/3}) = 0.26181818$$

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 = 4.0911E+008

= 0.97738438
' = 0.86668818
error of function (3.68), Biskinis Phd = 94699.84
From 5A.2, TBDY: fcc = fc* c = 43.01524
conf. factor c = 1.30349
fc = 33.00
From 10.3.5, ASCE 41-17, Final value of fy: fy*Min(1,1.25*(lb/d)^ 2/3) = 694.4444
lb/d = 1.00
d1 = 44.00
R = 250.00
v = 0.00114003
N = 7389.214
Ac = 196349.541
= *Min(1,1.25*(lb/d)^ 2/3) = 0.26181818

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 = 4.0911E+008

= 0.97738438
' = 0.86668818
error of function (3.68), Biskinis Phd = 94699.84
From 5A.2, TBDY: fcc = fc* c = 43.01524
conf. factor c = 1.30349
fc = 33.00
From 10.3.5, ASCE 41-17, Final value of fy: fy*Min(1,1.25*(lb/d)^ 2/3) = 694.4444
lb/d = 1.00
d1 = 44.00
R = 250.00
v = 0.00114003
N = 7389.214
Ac = 196349.541
= *Min(1,1.25*(lb/d)^ 2/3) = 0.26181818

Calculation of ratio lb/d

Adequate Lap Length: lb/d >= 1

Calculation of Shear Strength Vr = Min(Vr1,Vr2) = 483868.491

Calculation of Shear Strength at edge 1, Vr1 = 483868.491
Vr1 = VCol ((10.3), ASCE 41-17) = knl*VColO
VColO = 483868.491
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: $fc' = (fc'_{jacket} \cdot Area_{jacket} + fc'_{core} \cdot Area_{core}) / Area_{section} = 28.32$, but $fc'^{0.5} \leq 8.3$ MPa (22.5.3.1, ACI 318-14)
 $M/Vd = 2.00$
 $\mu_u = 2.6359963E-011$
 $V_u = 2.6832056E-030$
 $d = 0.8 \cdot D = 400.00$
 $N_u = 7389.214$
 $A_g = 196349.541$
From (11.5.4.8), ACI 318-14: $V_s = V_{s1} + V_{s2} = 274155.678$
 $V_{s1} = 274155.678$ is calculated for jacket, with:
 $A_v = \frac{1}{2} \cdot A_{stirrup} = 123370.055$
 $f_y = 555.5556$
 $s = 100.00$
 V_{s1} is multiplied by $Col1 = 1.00$
 $s/d = 0.25$
 $V_{s2} = 0.00$ is calculated for core, with:
 $A_v = \frac{1}{2} \cdot A_{stirrup} = 78956.835$
 $f_y = 444.4444$
 $s = 250.00$
 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: $V_s + V_f \leq 444245.712$
 $b_w \cdot d = \frac{1}{4} \cdot d \cdot d = 125663.706$

Calculation of Shear Strength at edge 2, $V_{r2} = 483868.491$
 $V_{r2} = V_{Col} ((10.3), ASCE 41-17) = knl \cdot V_{Col0}$
 $V_{Col0} = 483868.491$
 $knl = 1$ (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: $fc' = (fc'_{jacket} \cdot Area_{jacket} + fc'_{core} \cdot Area_{core}) / Area_{section} = 28.32$, but $fc'^{0.5} \leq 8.3$ MPa (22.5.3.1, ACI 318-14)
 $M/Vd = 2.00$
 $\mu_u = 2.6359963E-011$
 $V_u = 2.6832056E-030$
 $d = 0.8 \cdot D = 400.00$
 $N_u = 7389.214$
 $A_g = 196349.541$
From (11.5.4.8), ACI 318-14: $V_s = V_{s1} + V_{s2} = 274155.678$
 $V_{s1} = 274155.678$ is calculated for jacket, with:
 $A_v = \frac{1}{2} \cdot A_{stirrup} = 123370.055$
 $f_y = 555.5556$
 $s = 100.00$
 V_{s1} is multiplied by $Col1 = 1.00$
 $s/d = 0.25$
 $V_{s2} = 0.00$ is calculated for core, with:
 $A_v = \frac{1}{2} \cdot A_{stirrup} = 78956.835$
 $f_y = 444.4444$
 $s = 250.00$
 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: $V_s + V_f \leq 444245.712$
 $b_w \cdot d = \frac{1}{4} \cdot d \cdot d = 125663.706$

End Of Calculation of Shear Capacity ratio for element: column JCC1 of floor 1
At local axis: 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, $\gamma = 1.00$

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 Primary Member: Concrete Strength, $f_c = f_{cm} = 33.00$

New material of Primary Member: Steel Strength, $f_s = f_{sm} = 555.5556$

Concrete Elasticity, $E_c = 26999.444$

Steel Elasticity, $E_s = 200000.00$

Existing Column

Existing material of Primary Member: Concrete Strength, $f_c = f_{cm} = 20.00$

Existing material of Primary Member: Steel Strength, $f_s = f_{sm} = 444.4444$

Concrete Elasticity, $E_c = 21019.039$

Steel Elasticity, $E_s = 200000.00$

External Diameter, $D = 500.00$

Internal Diameter, $D = 300.00$

Cover Thickness, $c = 25.00$

Element Length, $L = 3000.00$

Primary Member

Ribbed Bars

Ductile Steel

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

Longitudinal Bars With Ends Lapped Starting at the End Sections

Adequate Lap Length ($l_b/d \geq 1$)

No FRP Wrapping

Stepwise Properties

Bending Moment, $M = 0.03183135$

Shear Force, $V_2 = 4667.691$

Shear Force, $V_3 = 8.7343036E-015$

Axial Force, $F = -7387.347$

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 $u_R = \gamma \cdot u = 0.00650221$

$u = \gamma \cdot u + p = 0.00650221$

- Calculation of γ -

$\gamma = (M_y \cdot L_s / 3) / E_{eff} = 0.00150221$ ((4.29), Biskinis Phd)

$M_y = 3.6258E+008$

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

From table 10.5, ASCE 41_17: $E_{eff} = factor \cdot E_c \cdot I_g = 2.4137E+013$

factor = 0.30

$A_g = 196349.541$

Mean concrete strength: $f_c' = (f_c'_{jacket} \cdot Area_{jacket} + f_c'_{core} \cdot Area_{core}) / Area_{section} = 28.32$

$N = 7387.347$

$$E_c I_g = E_c I_{g_jacket} + E_c I_{g_core} = 8.0455E+013$$

Calculation of Yielding Moment M_y

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

$$\begin{aligned} M_y &= \min(M_{y_ten}, M_{y_com}) = 3.6258E+008 \\ \gamma &((10a) \text{ or } (10b)) = 1.0622220E-005 \\ M_{y_ten} (8a) &= 3.6258E+008 \\ \gamma_{ten} (7a) &= 65.43627 \\ \text{error of function (7a)} &= 0.00293095 \\ M_{y_com} (8b) &= 7.5621E+008 \\ \gamma_{com} (7b) &= 64.56804 \\ \text{error of function (7b)} &= -0.00721905 \\ \text{with } e_y &= 0.00277778 \\ e_{co} &= 0.002 \\ a_{pl} &= 0.35 ((9a) \text{ in Biskinis and Fardis for no FRP Wrap}) \\ d_1 &= 44.00 \\ R &= 250.00 \\ v &= 0.0011401 \\ N &= 7387.347 \\ A_c &= 196349.541 \\ &= 0.26181818 \\ \text{with } f_c &= 33.00 \end{aligned}$$

Calculation of ratio I_b/I_d

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

- Calculation of ρ -

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

with:

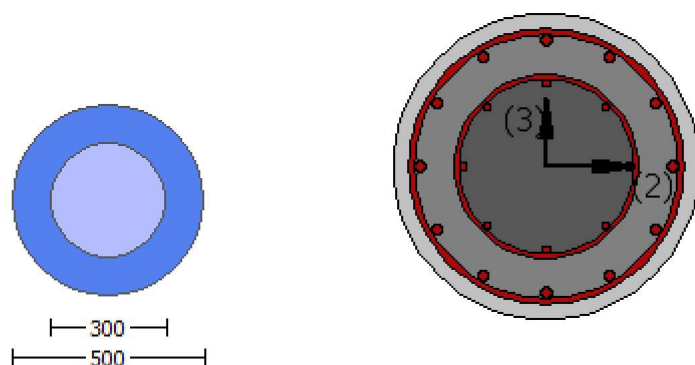
$$\begin{aligned} &\text{- Columns not controlled by inadequate development or splicing along the clear height because } I_b/I_d \geq 1 \\ &\text{shear control ratio } V_y E / V_{col} E = 0.5636717 \\ &d = d_{external} = 0.00 \\ &s = s_{external} = 0.00 \\ &t = s_1 + s_2 + 2 * t_f / b_w * (f_{fe} / f_s) = 0.00323428 \\ &\text{jacket: } s_1 = A_{v1} * (D_{c1} / 2) / (s_1 * A_g) = 0.0027646 \\ &\quad A_{v1} = 78.53982, \text{ is the area of stirrup} \\ &\quad D_{c1} = D_{ext} - 2 * \text{cover} - \text{External Hoop Diameter} = 440.00, \text{ is the total Length of all stirrups parallel to loading} \\ &\text{(shear) direction} \\ &\quad s_1 = 100.00 \\ &\text{core: } s_2 = A_{v2} * (D_{c2} / 2) / (s_2 * A_g) = 0.00046968 \\ &\quad A_{v2} = 50.26548, \text{ is the area of stirrup} \\ &\quad D_{c2} = D_{int} - \text{Internal Hoop Diameter} = 292.00, \text{ is the total Length of all stirrups parallel to loading (shear)} \\ &\text{direction} \\ &\quad s_2 = 250.00 \\ &\text{The term } 2 * t_f / b_w * (f_{fe} / f_s) \text{ is implemented to account for FRP contribution} \\ &\text{where } f = 2 * t_f / b_w \text{ is FRP ratio (EC8 - 3, A.4.4.3(6)) and } f_{fe} / f_s \text{ normalises } f \text{ to steel strength} \\ &\text{All these variables have already been given in Shear control ratio calculation.} \\ &\text{For the normalisation } f_s \text{ of jacket is used.} \\ &\quad NUD = 7387.347 \\ &\quad A_g = 196349.541 \\ &\quad f_{cE} = (f_c I_{jacket} * A_{jacket} + f_c I_{core} * A_{core}) / \text{section_area} = 28.32 \\ &\quad f_{yLE} = (f_{y_ext_Long_Reinf} * A_{ext_Long_Reinf} + f_{y_int_Long_Reinf} * A_{int_Long_Reinf}) / A_{Tot_Long_Rein} = \\ &2.1219958E-314 \\ &\quad f_{yTE} = (f_{y_ext_Trans_Reinf} * A_{ext_Trans_Reinf} + f_{y_int_Trans_Reinf} * A_{int_Trans_Reinf}) / A_{Tot_Trans_Rein} = \\ &539.4201 \\ &\quad \rho_l = A_{Tot_Long_Rein} / (A_g) = 0.015552 \\ &\quad f_{cE} = 28.32 \end{aligned}$$

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, $\gamma = 1.00$
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 Primary Member: Concrete Strength, $f_c = f_{c_lower_bound} = 25.00$
New material of Primary Member: Steel Strength, $f_s = f_{s_lower_bound} = 500.00$
Concrete Elasticity, $E_c = 26999.444$
Steel Elasticity, $E_s = 200000.00$
Existing Column
Existing material of Primary Member: Concrete Strength, $f_c = f_{c_lower_bound} = 16.00$
Existing material of Primary Member: Steel Strength, $f_s = f_{s_lower_bound} = 400.00$
Concrete Elasticity, $E_c = 21019.039$
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, $f_c = f_{cm} = 33.00$

New material: Steel Strength, $f_s = f_{sm} = 555.5556$

Existing Column

Existing material: Concrete Strength, $f_c = f_{cm} = 20.00$

Existing material: Steel Strength, $f_s = f_{sm} = 444.4444$

#####

External Diameter, $D = 500.00$

Internal Diameter, $D = 300.00$

Cover Thickness, $c = 25.00$

Element Length, $L = 3000.00$

Primary Member

Ribbed Bars

Ductile Steel

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

Longitudinal Bars With Ends Lapped Starting at the End Sections

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

No FRP Wrapping

Stepwise Properties

EDGE -A-

Bending Moment, $M_a = -1.7490E+007$

Shear Force, $V_a = -5828.436$

EDGE -B-

Bending Moment, $M_b = 0.03974705$

Shear Force, $V_b = 5828.436$

BOTH EDGES

Axial Force, $F = -7386.882$

Longitudinal Reinforcement Area Distribution (in 2 divisions)

-Tension: $A_{st} = 0.00$

-Compression: $A_{sc} = 3053.628$

Longitudinal Reinforcement Area Distribution (in 3 divisions)

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

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

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

Mean Diameter of Tension Reinforcement, $D_{bL,ten} = 18.00$

Existing component: From table 7-7, ASCE 41_17: Final Shear Capacity $V_R = V_n = 338743.584$

V_n ((10.3), ASCE 41-17) = $k_n \cdot V_{CoI} = 338743.584$

$V_{CoI} = 338743.584$

$k_n = 1.00$

displacement_ductility_demand = 0.01329921

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: $f_c' = (f_c'_{jacket} \cdot Area_{jacket} + f_c'_{core} \cdot Area_{core}) / Area_{section} = 21.76$, but $f_c'^{0.5} \leq 8.3$ MPa ((22.5.3.1, ACI 318-14)

$M / V_d = 4.00$

$M_u = 1.7490E+007$

$V_u = 5828.436$

$d = 0.8 \cdot D = 400.00$

$N_u = 7386.882$

$A_g = 196349.541$

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 $Col1 = 1.00$

$s/d = 0.25$
 $Vs2 = 0.00$ is calculated for core, with:
 $Av = \sqrt{2} \cdot A_{stirrup} = 78956.835$
 $fy = 400.00$
 $s = 250.00$
 $Vs2$ is multiplied by $Col2 = 0.00$
 $s/d = 1.04167$
 $Vf ((11-3)-(11.4), ACI 440) = 0.00$
 From (11-11), ACI 440: $Vs + Vf \leq 389409.072$
 $bw \cdot d = \sqrt{d} \cdot d/4 = 125663.706$

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 $\theta = 0.00019983$
 $y = (My \cdot Ls/3)/Eleff = 0.01502587 ((4.29), Biskinis Phd)$
 $My = 3.6258E+008$
 $Ls = M/V$ (with $Ls > 0.1 \cdot L$ and $Ls < 2 \cdot L$) = 3000.755
 From table 10.5, ASCE 41_17: $Eleff = factor \cdot Ec \cdot Ig = 2.4137E+013$
 $factor = 0.30$
 $Ag = 196349.541$
 Mean concrete strength: $fc' = (fc'_{jacket} \cdot Area_{jacket} + fc'_{core} \cdot Area_{core})/Area_{section} = 28.32$
 $N = 7386.882$
 $Ec \cdot Ig = Ec_{jacket} \cdot Ig_{jacket} + Ec_{core} \cdot Ig_{core} = 8.0455E+013$

Calculation of Yielding Moment My

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

$My = \min(My_{ten}, My_{com}) = 3.6258E+008$
 $y ((10a) \text{ or } (10b)) = 1.0622219E-005$
 $My_{ten} (8a) = 3.6258E+008$
 $\delta_{ten} (7a) = 65.43626$
 error of function (7a) = 0.00293096
 $My_{com} (8b) = 7.5621E+008$
 $\delta_{com} (7b) = 64.56804$
 error of function (7b) = -0.00721906
 with $ey = 0.00277778$
 $eco = 0.002$
 $apl = 0.35 ((9a) \text{ in Biskinis and Fardis for no FRP Wrap})$
 $d1 = 44.00$
 $R = 250.00$
 $v = 0.00114003$
 $N = 7386.882$
 $Ac = 196349.541$
 $= 0.26181818$
 with $fc = 33.00$

Calculation of ratio lb/d

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

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

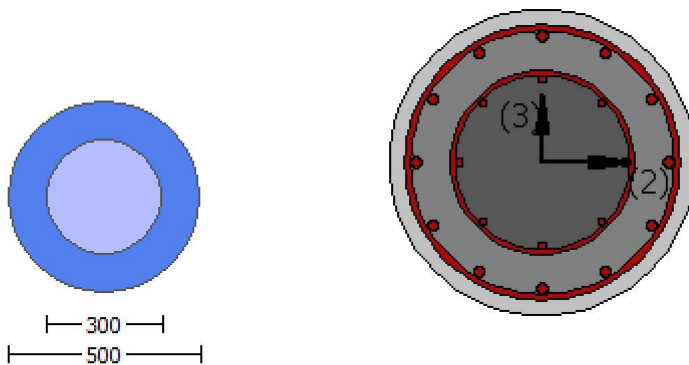
Limit State: Life Safety (data interpolation between analysis steps 1 and 2)

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, $\gamma = 1.00$

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

Consequently:

Jacket

New material of Primary Member: Concrete Strength, $f_c = f_{cm} = 33.00$

New material of Primary Member: Steel Strength, $f_s = f_{sm} = 555.5556$

Concrete Elasticity, $E_c = 26999.444$

Steel Elasticity, $E_s = 200000.00$

Existing Column

Existing material of Primary Member: Concrete Strength, $f_c = f_{cm} = 20.00$

Existing material of Primary Member: Steel Strength, $f_s = f_{sm} = 444.4444$

Concrete Elasticity, $E_c = 21019.039$

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, $f_s = 1.25 \cdot f_{sm} = 694.4444$

Existing Column

Existing material: Steel Strength, $f_s = 1.25 \cdot f_{sm} = 555.5556$

#####

External Diameter, $D = 500.00$

Internal Diameter, $D = 300.00$

Cover Thickness, $c = 25.00$

Mean Confinement Factor overall section = 1.30349

Element Length, L = 3000.00
Primary Member
Ribbed Bars
Ductile Steel
With Detailing for Earthquake Resistance (including stirrups closed at 135°)
Longitudinal Bars With Ends Lapped Starting at the End Sections
Adequate Lap Length ($l_o/l_{ou}, \min \geq 1$)
No FRP Wrapping

Stepwise Properties

At local axis: 3
EDGE -A-
Shear Force, $V_a = -1.4969033E-031$
EDGE -B-
Shear Force, $V_b = 1.4969033E-031$
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, $V_e/V_r = 0.5636717$
Member Controlled by Flexure ($V_e/V_r < 1$)
Calculation of Shear Demand from fig. R18.6.5, ACI 318-14 $V_e = (M_{pr1} + M_{pr2})/l_n = 272742.977$
with
 $M_{pr1} = \max(\mu_{u1+}, \mu_{u1-}) = 4.0911E+008$
 $\mu_{u1+} = 4.0911E+008$, 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
 $\mu_{u1-} = 4.0911E+008$, 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
 $M_{pr2} = \max(\mu_{u2+}, \mu_{u2-}) = 4.0911E+008$
 $\mu_{u2+} = 4.0911E+008$, 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
 $\mu_{u2-} = 4.0911E+008$, 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
 $\mu_u = 4.0911E+008$

$\lambda = 0.97738438$
 $\lambda' = 0.86668818$
error of function (3.68), Biskinis Phd = 94699.84
From 5A.2, TBDY: $f_{cc} = f_c^* \cdot c = 43.01524$
conf. factor $c = 1.30349$
 $f_c = 33.00$
From 10.3.5, ASCE 41-17, Final value of f_y : $f_y \cdot \min(1, 1.25 \cdot (l_b/l_d)^{2/3}) = 694.4444$
 $l_b/l_d = 1.00$
 $d_1 = 44.00$
 $R = 250.00$
 $v = 0.00114003$
 $N = 7389.214$
 $A_c = 196349.541$
 $\lambda = \lambda' \cdot \min(1, 1.25 \cdot (l_b/l_d)^{2/3}) = 0.26181818$

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
 $\mu_u = 4.0911E+008$

$$= 0.97738438$$

$$' = 0.86668818$$

error of function (3.68), Biskinis Phd = 94699.84

From 5A.2, TBDY: $f_{cc} = f_c \cdot c = 43.01524$

conf. factor $c = 1.30349$

$f_c = 33.00$

From 10.3.5, ASCE 41-17, Final value of f_y : $f_y \cdot \text{Min}(1, 1.25 \cdot (l_b/d)^{2/3}) = 694.4444$

$$l_b/d = 1.00$$

$$d_1 = 44.00$$

$$R = 250.00$$

$$v = 0.00114003$$

$$N = 7389.214$$

$$A_c = 196349.541$$

$$= \cdot \text{Min}(1, 1.25 \cdot (l_b/d)^{2/3}) = 0.26181818$$

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
 $\mu_u = 4.0911E+008$

$$= 0.97738438$$

$$' = 0.86668818$$

error of function (3.68), Biskinis Phd = 94699.84

From 5A.2, TBDY: $f_{cc} = f_c \cdot c = 43.01524$

conf. factor $c = 1.30349$

$f_c = 33.00$

From 10.3.5, ASCE 41-17, Final value of f_y : $f_y \cdot \text{Min}(1, 1.25 \cdot (l_b/d)^{2/3}) = 694.4444$

$$l_b/d = 1.00$$

$$d_1 = 44.00$$

$$R = 250.00$$

$$v = 0.00114003$$

$$N = 7389.214$$

$$A_c = 196349.541$$

$$= \cdot \text{Min}(1, 1.25 \cdot (l_b/d)^{2/3}) = 0.26181818$$

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), Mu
Mu = 4.0911E+008

= 0.97738438
' = 0.86668818
error of function (3.68), Biskinis Phd = 94699.84
From 5A.2, TBDY: fcc = fc* c = 43.01524
conf. factor c = 1.30349
fc = 33.00
From 10.3.5, ASCE 41-17, Final value of fy: fy*Min(1,1.25*(lb/d)^ 2/3) = 694.4444
lb/d = 1.00
d1 = 44.00
R = 250.00
v = 0.00114003
N = 7389.214
Ac = 196349.541
= *Min(1,1.25*(lb/d)^ 2/3) = 0.26181818

Calculation of ratio lb/d

Adequate Lap Length: lb/d >= 1

Calculation of Shear Strength Vr = Min(Vr1,Vr2) = 483868.491

Calculation of Shear Strength at edge 1, Vr1 = 483868.491

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

VColO = 483868.491

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 = 28.32, but fc'^0.5 <= 8.3
MPa (22.5.3.1, ACI 318-14)
M/Vd = 2.00
Mu = 1.4802042E-011
Vu = 1.4969033E-031
d = 0.8*D = 400.00
Nu = 7389.214
Ag = 196349.541
From (11.5.4.8), ACI 318-14: Vs = Vs1 + Vs2 = 274155.678
Vs1 = 274155.678 is calculated for jacket, with:
Av = /2*A_stirrup = 123370.055
fy = 555.5556
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 = 444.4444
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 <= 444245.712
bw*d = *d*d/4 = 125663.706

Calculation of Shear Strength at edge 2, Vr2 = 483868.491

Vr2 = VCol ((10.3), ASCE 41-17) = knl*VColO

VColO = 483868.491

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 = 28.32, but $fc'^{0.5} \leq 8.3$ MPa (22.5.3.1, ACI 318-14)

M/Vd = 2.00

Mu = 1.4802042E-011

Vu = 1.4969033E-031

d = 0.8*D = 400.00

Nu = 7389.214

Ag = 196349.541

From (11.5.4.8), ACI 318-14: Vs = Vs1 + Vs2 = 274155.678

Vs1 = 274155.678 is calculated for jacket, with:

Av = /2*A_stirrup = 123370.055

fy = 555.5556

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 = 444.4444

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 <= 444245.712

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, = 1.00

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

Consequently:

Jacket

New material of Primary Member: Concrete Strength, fc = fcm = 33.00

New material of Primary Member: Steel Strength, fs = fsm = 555.5556

Concrete Elasticity, Ec = 26999.444

Steel Elasticity, Es = 200000.00

Existing Column

Existing material of Primary Member: Concrete Strength, fc = fcm = 20.00

Existing material of Primary Member: Steel Strength, fs = fsm = 444.4444

Concrete Elasticity, Ec = 21019.039

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 = 694.4444

Existing Column

Existing material: Steel Strength, fs = 1.25*fsm = 555.5556

#####

External Diameter, D = 500.00
Internal Diameter, D = 300.00
Cover Thickness, c = 25.00
Mean Confinement Factor overall section = 1.30349
Element Length, L = 3000.00
Primary Member
Ribbed Bars
Ductile Steel
With Detailing for Earthquake Resistance (including stirrups closed at 135°)
Longitudinal Bars With Ends Lapped Starting at the End Sections
Adequate Lap Length ($l_o/l_{ou}, \min \geq 1$)
No FRP Wrapping

Stepwise Properties

At local axis: 2
EDGE -A-
Shear Force, $V_a = -2.6832056E-030$
EDGE -B-
Shear Force, $V_b = 2.6832056E-030$
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, $V_e/V_r = 0.5636717$
Member Controlled by Flexure ($V_e/V_r < 1$)
Calculation of Shear Demand from fig. R18.6.5, ACI 318-14 $V_e = (M_{pr1} + M_{pr2})/l_n = 272742.977$
with
 $M_{pr1} = \max(\mu_{u1+}, \mu_{u1-}) = 4.0911E+008$
 $\mu_{u1+} = 4.0911E+008$, 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
 $\mu_{u1-} = 4.0911E+008$, 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
 $M_{pr2} = \max(\mu_{u2+}, \mu_{u2-}) = 4.0911E+008$
 $\mu_{u2+} = 4.0911E+008$, 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
 $\mu_{u2-} = 4.0911E+008$, 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
 $\mu_u = 4.0911E+008$

$\lambda = 0.97738438$
 $\lambda' = 0.86668818$
error of function (3.68), Biskinis Phd = 94699.84
From 5A.2, TBDY: $f_{cc} = f_c \cdot c = 43.01524$
conf. factor $c = 1.30349$
 $f_c = 33.00$
From 10.3.5, ASCE 41-17, Final value of f_y : $f_y \cdot \min(1, 1.25 \cdot (l_b/l_d)^{2/3}) = 694.4444$
 $l_b/l_d = 1.00$
 $d_1 = 44.00$

R = 250.00
v = 0.00114003
N = 7389.214
Ac = 196349.541
= *Min(1,1.25*(lb/d)^ 2/3) = 0.26181818

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 = 4.0911E+008

= 0.97738438
' = 0.86668818
error of function (3.68), Biskinis Phd = 94699.84
From 5A.2, TBDY: fcc = fc* c = 43.01524
conf. factor c = 1.30349
fc = 33.00
From 10.3.5, ASCE 41-17, Final value of fy: fy*Min(1,1.25*(lb/d)^ 2/3) = 694.4444
lb/d = 1.00
d1 = 44.00
R = 250.00
v = 0.00114003
N = 7389.214
Ac = 196349.541
= *Min(1,1.25*(lb/d)^ 2/3) = 0.26181818

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 = 4.0911E+008

= 0.97738438
' = 0.86668818
error of function (3.68), Biskinis Phd = 94699.84
From 5A.2, TBDY: fcc = fc* c = 43.01524
conf. factor c = 1.30349
fc = 33.00
From 10.3.5, ASCE 41-17, Final value of fy: fy*Min(1,1.25*(lb/d)^ 2/3) = 694.4444
lb/d = 1.00
d1 = 44.00
R = 250.00
v = 0.00114003
N = 7389.214
Ac = 196349.541
= *Min(1,1.25*(lb/d)^ 2/3) = 0.26181818

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
 $\mu = 4.0911 \times 10^8$

$$= 0.97738438$$

$$\mu = 0.86668818$$

error of function (3.68), Biskinis Phd = 94699.84

From 5A.2, TBDY: $f_{cc} = f_c \cdot c = 43.01524$

conf. factor $c = 1.30349$

$$f_c = 33.00$$

From 10.3.5, ASCE 41-17, Final value of f_y : $f_y \cdot \text{Min}(1, 1.25 \cdot (l_b/d)^{2/3}) = 694.4444$

$$l_b/d = 1.00$$

$$d_1 = 44.00$$

$$R = 250.00$$

$$v = 0.00114003$$

$$N = 7389.214$$

$$A_c = 196349.541$$

$$= \cdot \text{Min}(1, 1.25 \cdot (l_b/d)^{2/3}) = 0.26181818$$

Calculation of ratio l_b/d

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

Calculation of Shear Strength $V_r = \text{Min}(V_{r1}, V_{r2}) = 483868.491$

Calculation of Shear Strength at edge 1, $V_{r1} = 483868.491$

$V_{r1} = V_{c0} \text{ ((10.3), ASCE 41-17)} = k_{nl} \cdot V_{c0}$

$$V_{c0} = 483868.491$$

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

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

= 1 (normal-weight concrete)

Mean concrete strength: $f'_c = (f'_{c_jacket} \cdot \text{Area}_{jacket} + f'_{c_core} \cdot \text{Area}_{core}) / \text{Area}_{section} = 28.32$, but $f'_c^{0.5} \leq 8.3$
MPa ((22.5.3.1, ACI 318-14)

$$M/Vd = 2.00$$

$$\mu = 2.6359963 \times 10^{11}$$

$$V_u = 2.6832056 \times 10^{30}$$

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

$$N_u = 7389.214$$

$$A_g = 196349.541$$

From ((11.5.4.8), ACI 318-14: $V_s = V_{s1} + V_{s2} = 274155.678$

$V_{s1} = 274155.678$ is calculated for jacket, with:

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

$$f_y = 555.5556$$

$$s = 100.00$$

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

$$s/d = 0.25$$

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

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

$$f_y = 444.4444$$

$$s = 250.00$$

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: $V_s + V_f \leq 444245.712$
 $b_w \cdot d = \frac{V_s \cdot d}{4} = 125663.706$

Calculation of Shear Strength at edge 2, $V_{r2} = 483868.491$
 $V_{r2} = V_{Col} \text{ ((10.3), ASCE 41-17)} = k_n l \cdot V_{Col0}$
 $V_{Col0} = 483868.491$
 $k_n l = 1$ (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).

$f_c = 1$ (normal-weight concrete)
Mean concrete strength: $f_c' = (f_c'_{jacket} \cdot Area_{jacket} + f_c'_{core} \cdot Area_{core}) / Area_{section} = 28.32$, but $f_c'^{0.5} \leq 8.3$ MPa (22.5.3.1, ACI 318-14)
 $M / Vd = 2.00$
 $\mu_u = 2.6359963E-011$
 $V_u = 2.6832056E-030$
 $d = 0.8 \cdot D = 400.00$
 $N_u = 7389.214$
 $A_g = 196349.541$
From (11.5.4.8), ACI 318-14: $V_s = V_{s1} + V_{s2} = 274155.678$
 $V_{s1} = 274155.678$ is calculated for jacket, with:
 $A_v = \frac{V_{s1}}{2 \cdot A_{stirrup}} = 123370.055$
 $f_y = 555.5556$
 $s = 100.00$
 V_{s1} is multiplied by $Col1 = 1.00$
 $s/d = 0.25$
 $V_{s2} = 0.00$ is calculated for core, with:
 $A_v = \frac{V_{s2}}{2 \cdot A_{stirrup}} = 78956.835$
 $f_y = 444.4444$
 $s = 250.00$
 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: $V_s + V_f \leq 444245.712$
 $b_w \cdot d = \frac{V_s \cdot d}{4} = 125663.706$

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, $\phi = 1.00$
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 Primary Member: Concrete Strength, $f_c = f_{cm} = 33.00$
New material of Primary Member: Steel Strength, $f_s = f_{sm} = 555.5556$
Concrete Elasticity, $E_c = 26999.444$
Steel Elasticity, $E_s = 200000.00$
Existing Column
Existing material of Primary Member: Concrete Strength, $f_c = f_{cm} = 20.00$
Existing material of Primary Member: Steel Strength, $f_s = f_{sm} = 444.4444$
Concrete Elasticity, $E_c = 21019.039$
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
 Primary Member
 Ribbed Bars
 Ductile Steel
 With Detailing for Earthquake Resistance (including stirrups closed at 135°)
 Longitudinal Bars With Ends Lapped Starting at the End Sections
 Adequate Lap Length ($l_b/l_d > 1$)
 No FRP Wrapping

Stepwise Properties

Bending Moment, M = 2.5536198E-011
 Shear Force, V2 = -5828.436
 Shear Force, V3 = -1.0906320E-014
 Axial Force, F = -7386.882
 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_{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 $u_R = \gamma + \rho = 0.04874756$

- Calculation of γ -

$\gamma = (M_y \cdot L_s / 3) / E_{eff} = 0.00751104$ ((4.29), Biskinis Phd))
 $M_y = 3.6258E+008$
 $L_s = M/V$ (with $L_s > 0.1 \cdot L$ and $L_s < 2 \cdot L$) = 1500.00
 From table 10.5, ASCE 41_17: $E_{eff} = \text{factor} \cdot E_c \cdot I_g = 2.4137E+013$
 factor = 0.30
 $A_g = 196349.541$
 Mean concrete strength: $f_c' = (f_c'_{\text{jacket}} \cdot \text{Area}_{\text{jacket}} + f_c'_{\text{core}} \cdot \text{Area}_{\text{core}}) / \text{Area}_{\text{section}} = 28.32$
 $N = 7386.882$
 $E_c \cdot I_g = E_c \cdot I_{g,\text{jacket}} + E_c \cdot I_{g,\text{core}} = 8.0455E+013$

Calculation of Yielding Moment M_y

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

$M_y = \text{Min}(M_{y,\text{ten}}, M_{y,\text{com}}) = 3.6258E+008$
 γ ((10a) or (10b)) = 1.0622219E-005
 $M_{y,\text{ten}}$ (8a) = 3.6258E+008
 γ_{ten} (7a) = 65.43626
 error of function (7a) = 0.00293096
 $M_{y,\text{com}}$ (8b) = 7.5621E+008
 γ_{com} (7b) = 64.56804
 error of function (7b) = -0.00721906
 with $e_y = 0.00277778$
 $e_{co} = 0.002$
 $a_{pl} = 0.35$ ((9a) in Biskinis and Fardis for no FRP Wrap)
 $d_1 = 44.00$
 $R = 250.00$
 $v = 0.00114003$
 $N = 7386.882$

$A_c = 196349.541$
 $= 0.26181818$
with $f_c = 33.00$

Calculation of ratio l_b/l_d

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

- Calculation of p -

From table 10-9: $p = 0.04123652$

with:

- Columns not controlled by inadequate development or splicing along the clear height because $l_b/l_d \geq 1$
shear control ratio $V_y E / V_{CoI} E = 0.5636717$

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

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

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

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.

$N_{UD} = 7386.882$

$A_g = 196349.541$

$f_{cE} = (f_{c_jacket} \cdot \text{Area}_{\text{jacket}} + f_{c_core} \cdot \text{Area}_{\text{core}}) / \text{section_area} = 28.32$

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

$f_{yE} = (f_{y_ext_Trans_Reinf} \cdot \text{Area}_{\text{ext_Trans_Reinf}} + f_{y_int_Trans_Reinf} \cdot \text{Area}_{\text{int_Trans_Reinf}}) / \text{Area}_{\text{Tot_Trans_Rein}} = 539.4201$

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

$f_{cE} = 28.32$

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

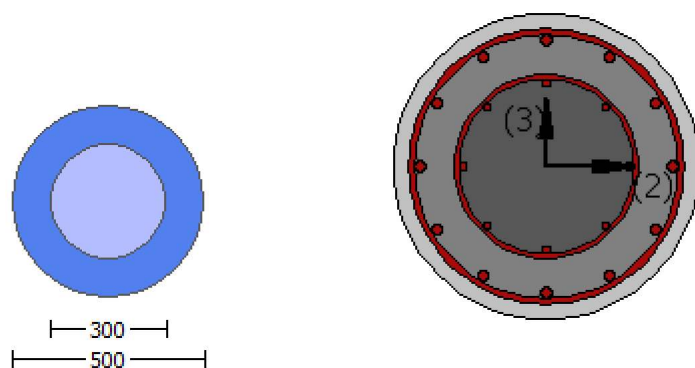
Limit State: Life Safety (data interpolation between analysis steps 1 and 2)

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, $\gamma = 1.00$

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 Primary Member: Concrete Strength, $f_c = f_{c_lower_bound} = 25.00$

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

Concrete Elasticity, $E_c = 26999.444$

Steel Elasticity, $E_s = 200000.00$

Existing Column

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

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

Concrete Elasticity, $E_c = 21019.039$

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, $f_c = f_{cm} = 33.00$

New material: Steel Strength, $f_s = f_{sm} = 555.5556$

Existing Column

Existing material: Concrete Strength, $f_c = f_{cm} = 20.00$

Existing material: Steel Strength, $f_s = f_{sm} = 444.4444$

#####

External Diameter, $D = 500.00$

Internal Diameter, $D = 300.00$

Cover Thickness, $c = 25.00$

Element Length, $L = 3000.00$

Primary Member

Ribbed Bars

Ductile Steel

With Detailing for Earthquake Resistance (including stirrups closed at 135°)
Longitudinal Bars With Ends Lapped Starting at the End Sections
Adequate Lap Length ($l_o/l_{o,min} = l_b/l_d \geq 1$)
No FRP Wrapping

Stepwise Properties

EDGE -A-
Bending Moment, $M_a = 2.5536198E-011$
Shear Force, $V_a = -1.0906320E-014$
EDGE -B-
Bending Moment, $M_b = 7.1068928E-012$
Shear Force, $V_b = 1.0906320E-014$
BOTH EDGES
Axial Force, $F = -7386.882$
Longitudinal Reinforcement Area Distribution (in 2 divisions)
-Tension: $A_{sl,t} = 0.00$
-Compression: $A_{sl,c} = 3053.628$
Longitudinal Reinforcement Area Distribution (in 3 divisions)
-Tension: $A_{sl,ten} = 1017.876$
-Compression: $A_{sl,com} = 1017.876$
-Middle: $A_{sl,mid} = 1017.876$
Mean Diameter of Tension Reinforcement, $D_{bL,ten} = 18.00$

Existing component: From table 7-7, ASCE 41_17: Final Shear Capacity $V_R = V_n = 430747.058$
 $V_n ((10.3), ASCE 41-17) = k_n \cdot V_{CoI} = 430747.058$
 $V_{CoI} = 430747.058$
 $k_n = 1.00$
 $displacement_ductility_demand = 0.00$

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: $f'_c = (f'_c_jacket \cdot Area_jacket + f'_c_core \cdot Area_core) / Area_section = 21.76$, but $f'_c^{0.5} \leq 8.3$
MPa (22.5.3.1, ACI 318-14)
 $M/Vd = 2.00$
 $M_u = 2.5536198E-011$
 $V_u = 1.0906320E-014$
 $d = 0.8 \cdot D = 400.00$
 $N_u = 7386.882$
 $A_g = 196349.541$
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:
 $A_v = \sqrt{2} \cdot A_{stirrup} = 78956.835$
 $f_y = 400.00$
 $s = 250.00$
 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: $V_s + V_f \leq 389409.072$
 $b_w \cdot d = \sqrt{2} \cdot d \cdot d / 4 = 125663.706$

$displacement_ductility_demand$ is calculated as δ / y

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

From analysis, chord rotation $\theta = 3.4210353E-021$

$y = (M_y * L_s / 3) / E_{eff} = 0.00751104$ ((4.29), Biskinis Phd))

$M_y = 3.6258E+008$

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

From table 10.5, ASCE 41_17: $E_{eff} = factor * E_c * I_g = 2.4137E+013$

factor = 0.30

$A_g = 196349.541$

Mean concrete strength: $f'_c = (f'_{c_jacket} * Area_jacket + f'_{c_core} * Area_core) / Area_section = 28.32$

$N = 7386.882$

$E_c * I_g = E_{c_jacket} * I_{g_jacket} + E_{c_core} * I_{g_core} = 8.0455E+013$

Calculation of Yielding Moment M_y

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

$M_y = \min(M_{y_ten}, M_{y_com}) = 3.6258E+008$

y ((10a) or (10b)) = $1.0622219E-005$

M_{y_ten} (8a) = $3.6258E+008$

ϕ_{y_ten} (7a) = 65.43626

error of function (7a) = 0.00293096

M_{y_com} (8b) = $7.5621E+008$

ϕ_{y_com} (7b) = 64.56804

error of function (7b) = -0.00721906

with $e_y = 0.00277778$

$e_{co} = 0.002$

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

$d_1 = 44.00$

$R = 250.00$

$v = 0.00114003$

$N = 7386.882$

$A_c = 196349.541$

$= 0.26181818$

with $f_c = 33.00$

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

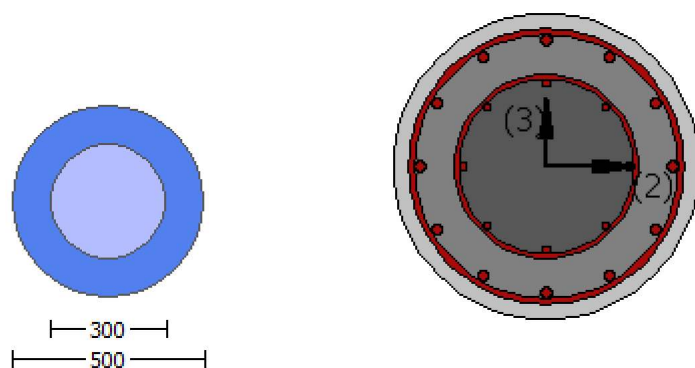
Limit State: Life Safety (data interpolation between analysis steps 1 and 2)

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, $\gamma = 1.00$

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

Consequently:

Jacket

New material of Primary Member: Concrete Strength, $f_c = f_{cm} = 33.00$

New material of Primary Member: Steel Strength, $f_s = f_{sm} = 555.5556$

Concrete Elasticity, $E_c = 26999.444$

Steel Elasticity, $E_s = 200000.00$

Existing Column

Existing material of Primary Member: Concrete Strength, $f_c = f_{cm} = 20.00$

Existing material of Primary Member: Steel Strength, $f_s = f_{sm} = 444.4444$

Concrete Elasticity, $E_c = 21019.039$

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, $f_s = 1.25 \cdot f_{sm} = 694.4444$

Existing Column

Existing material: Steel Strength, $f_s = 1.25 \cdot f_{sm} = 555.5556$

#####

External Diameter, $D = 500.00$

Internal Diameter, $D = 300.00$

Cover Thickness, $c = 25.00$

Mean Confinement Factor overall section = 1.30349

Element Length, $L = 3000.00$

Primary Member

Ribbed Bars

Ductile Steel

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

Longitudinal Bars With Ends Lapped Starting at the End Sections

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

No FRP Wrapping

Stepwise Properties

At local axis: 3

EDGE -A-

Shear Force, $V_a = -1.4969033\text{E}-031$

EDGE -B-

Shear Force, $V_b = 1.4969033\text{E}-031$

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, $V_e/V_r = 0.5636717$

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

Calculation of Shear Demand from fig. R18.6.5, ACI 318-14 $V_e = (M_{pr1} + M_{pr2})/l_n = 272742.977$

with

$M_{pr1} = \text{Max}(Mu_{1+}, Mu_{1-}) = 4.0911\text{E}+008$

$Mu_{1+} = 4.0911\text{E}+008$, is the ultimate moment strength at the edge 1 of the member in the actual moment direction which is defined for the static loading combination

$Mu_{1-} = 4.0911\text{E}+008$, is the ultimate moment strength at the edge 1 of the member in the opposite moment direction which is defined for the static loading combination

$M_{pr2} = \text{Max}(Mu_{2+}, Mu_{2-}) = 4.0911\text{E}+008$

$Mu_{2+} = 4.0911\text{E}+008$, is the ultimate moment strength at the edge 2 of the member in the actual moment direction which is defined for the static loading combination

$Mu_{2-} = 4.0911\text{E}+008$, is the ultimate moment strength at the edge 2 of the member in the opposite moment direction which is defined for the static loading combination

Calculation of Mu_{1+}

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

$M_u = 4.0911\text{E}+008$

$\phi = 0.97738438$

$\phi' = 0.86668818$

error of function (3.68), Biskinis Phd = 94699.84

From 5A.2, TBDY: $f_{cc} = f_c' \cdot c = 43.01524$

conf. factor $c = 1.30349$

$f_c = 33.00$

From 10.3.5, ASCE 41-17, Final value of f_y : $f_y \cdot \text{Min}(1, 1.25 \cdot (l_b/d)^{2/3}) = 694.4444$

$l_b/d = 1.00$

$d_1 = 44.00$

$R = 250.00$

$v = 0.00114003$

$N = 7389.214$

$A_c = 196349.541$

$= \phi' \cdot \text{Min}(1, 1.25 \cdot (l_b/d)^{2/3}) = 0.26181818$

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 = 4.0911E+008

= 0.97738438
' = 0.86668818
error of function (3.68), Biskinis Phd = 94699.84
From 5A.2, TBDY: $f_{cc} = f_c^* c = 43.01524$
conf. factor $c = 1.30349$
 $f_c = 33.00$
From 10.3.5, ASCE 41-17, Final value of f_y : $f_y * \text{Min}(1, 1.25 * (l_b / d)^{2/3}) = 694.4444$
 $l_b / d = 1.00$
 $d_1 = 44.00$
 $R = 250.00$
 $v = 0.00114003$
 $N = 7389.214$
 $A_c = 196349.541$
= $*\text{Min}(1, 1.25 * (l_b / d)^{2/3}) = 0.26181818$

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 = 4.0911E+008

= 0.97738438
' = 0.86668818
error of function (3.68), Biskinis Phd = 94699.84
From 5A.2, TBDY: $f_{cc} = f_c^* c = 43.01524$
conf. factor $c = 1.30349$
 $f_c = 33.00$
From 10.3.5, ASCE 41-17, Final value of f_y : $f_y * \text{Min}(1, 1.25 * (l_b / d)^{2/3}) = 694.4444$
 $l_b / d = 1.00$
 $d_1 = 44.00$
 $R = 250.00$
 $v = 0.00114003$
 $N = 7389.214$
 $A_c = 196349.541$
= $*\text{Min}(1, 1.25 * (l_b / d)^{2/3}) = 0.26181818$

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 = 4.0911E+008

= 0.97738438

$\rho = 0.86668818$
 error of function (3.68), Biskinis Phd = 94699.84
 From 5A.2, TBDY: $f_{cc} = f_c' \cdot c = 43.01524$
 conf. factor $c = 1.30349$
 $f_c = 33.00$
 From 10.3.5, ASCE 41-17, Final value of f_y : $f_y \cdot \text{Min}(1, 1.25 \cdot (l_b/l_d)^{2/3}) = 694.4444$
 $l_b/l_d = 1.00$
 $d_1 = 44.00$
 $R = 250.00$
 $v = 0.00114003$
 $N = 7389.214$
 $A_c = 196349.541$
 $= \cdot \text{Min}(1, 1.25 \cdot (l_b/l_d)^{2/3}) = 0.26181818$

Calculation of ratio l_b/l_d

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

Calculation of Shear Strength $V_r = \text{Min}(V_{r1}, V_{r2}) = 483868.491$

Calculation of Shear Strength at edge 1, $V_{r1} = 483868.491$

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

$V_{ColO} = 483868.491$

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

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

$= 1$ (normal-weight concrete)

Mean concrete strength: $f_c' = (f_c'_{\text{jacket}} \cdot \text{Area}_{\text{jacket}} + f_c'_{\text{core}} \cdot \text{Area}_{\text{core}}) / \text{Area}_{\text{section}} = 28.32$, but $f_c'^{0.5} \leq 8.3$ MPa (22.5.3.1, ACI 318-14)

$M/Vd = 2.00$

$\mu_u = 1.4802042E-011$

$\nu_u = 1.4969033E-031$

$d = 0.8 \cdot D = 400.00$

$N_u = 7389.214$

$A_g = 196349.541$

From (11.5.4.8), ACI 318-14: $V_s = V_{s1} + V_{s2} = 274155.678$

$V_{s1} = 274155.678$ is calculated for jacket, with:

$A_v = \rho_s \cdot A_{\text{stirrup}} = 123370.055$

$f_y = 555.5556$

$s = 100.00$

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

$s/d = 0.25$

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

$A_v = \rho_s \cdot A_{\text{stirrup}} = 78956.835$

$f_y = 444.4444$

$s = 250.00$

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: $V_s + V_f \leq 444245.712$

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

Calculation of Shear Strength at edge 2, $V_{r2} = 483868.491$

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

$V_{ColO} = 483868.491$

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

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

= 1 (normal-weight concrete)
Mean concrete strength: $f'_c = (f'_{c_jacket} \cdot Area_jacket + f'_{c_core} \cdot Area_core) / Area_section = 28.32$, but $f'_c^{0.5} \leq 8.3$ MPa (22.5.3.1, ACI 318-14)
 $M/Vd = 2.00$
 $\mu_u = 1.4802042E-011$
 $\mu_v = 1.4969033E-031$
 $d = 0.8 \cdot D = 400.00$
 $N_u = 7389.214$
 $A_g = 196349.541$
From (11.5.4.8), ACI 318-14: $V_s = V_{s1} + V_{s2} = 274155.678$
 $V_{s1} = 274155.678$ is calculated for jacket, with:
 $A_v = \frac{1}{2} \cdot A_{stirrup} = 123370.055$
 $f_y = 555.5556$
 $s = 100.00$
 V_{s1} is multiplied by $Col1 = 1.00$
 $s/d = 0.25$
 $V_{s2} = 0.00$ is calculated for core, with:
 $A_v = \frac{1}{2} \cdot A_{stirrup} = 78956.835$
 $f_y = 444.4444$
 $s = 250.00$
 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: $V_s + V_f \leq 444245.712$
 $b_w \cdot d = \frac{1}{4} \cdot d \cdot d = 125663.706$

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, $\phi = 1.00$
Mean strength values are used for both shear and moment calculations.
Consequently:
Jacket
New material of Primary Member: Concrete Strength, $f_c = f_{cm} = 33.00$
New material of Primary Member: Steel Strength, $f_s = f_{sm} = 555.5556$
Concrete Elasticity, $E_c = 26999.444$
Steel Elasticity, $E_s = 200000.00$
Existing Column
Existing material of Primary Member: Concrete Strength, $f_c = f_{cm} = 20.00$
Existing material of Primary Member: Steel Strength, $f_s = f_{sm} = 444.4444$
Concrete Elasticity, $E_c = 21019.039$
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, $f_s = 1.25 \cdot f_{sm} = 694.4444$
Existing Column
Existing material: Steel Strength, $f_s = 1.25 \cdot f_{sm} = 555.5556$

External Diameter, $D = 500.00$
Internal Diameter, $D = 300.00$
Cover Thickness, $c = 25.00$
Mean Confinement Factor overall section = 1.30349
Element Length, $L = 3000.00$
Primary Member

Ribbed Bars
Ductile Steel
With Detailing for Earthquake Resistance (including stirrups closed at 135°)
Longitudinal Bars With Ends Lapped Starting at the End Sections
Adequate Lap Length ($l_o/l_{ou,min} \geq 1$)
No FRP Wrapping

Stepwise Properties

At local axis: 2
EDGE -A-
Shear Force, $V_a = -2.6832056E-030$
EDGE -B-
Shear Force, $V_b = 2.6832056E-030$
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, $V_e/V_r = 0.5636717$
Member Controlled by Flexure ($V_e/V_r < 1$)
Calculation of Shear Demand from fig. R18.6.5, ACI 318-14 $V_e = (M_{pr1} + M_{pr2})/l_n = 272742.977$
with
 $M_{pr1} = \max(\mu_{u1+}, \mu_{u1-}) = 4.0911E+008$
 $\mu_{u1+} = 4.0911E+008$, 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
 $\mu_{u1-} = 4.0911E+008$, 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
 $M_{pr2} = \max(\mu_{u2+}, \mu_{u2-}) = 4.0911E+008$
 $\mu_{u2+} = 4.0911E+008$, 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
 $\mu_{u2-} = 4.0911E+008$, 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
 $\mu_u = 4.0911E+008$

$\phi = 0.97738438$
 $\phi' = 0.86668818$
error of function (3.68), Biskinis Phd = 94699.84
From 5A.2, TBDY: $f_{cc} = f_c' \cdot c = 43.01524$
conf. factor $c = 1.30349$
 $f_c = 33.00$
From 10.3.5, ASCE 41-17, Final value of f_y : $f_y \cdot \min(1, 1.25 \cdot (l_b/l_d)^{2/3}) = 694.4444$
 $l_b/l_d = 1.00$
 $d_1 = 44.00$
 $R = 250.00$
 $v = 0.00114003$
 $N = 7389.214$
 $A_c = 196349.541$
 $= \phi' \cdot \min(1, 1.25 \cdot (l_b/l_d)^{2/3}) = 0.26181818$

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
 $\mu = 4.0911E+008$

$$= 0.97738438$$

$$' = 0.86668818$$

error of function (3.68), Biskinis Phd = 94699.84

From 5A.2, TDY: $f_{cc} = f_c \cdot c = 43.01524$

conf. factor $c = 1.30349$

$f_c = 33.00$

From 10.3.5, ASCE 41-17, Final value of f_y : $f_y \cdot \text{Min}(1, 1.25 \cdot (l_b/d)^{2/3}) = 694.4444$

$$l_b/d = 1.00$$

$$d_1 = 44.00$$

$$R = 250.00$$

$$v = 0.00114003$$

$$N = 7389.214$$

$$A_c = 196349.541$$

$$= \cdot \text{Min}(1, 1.25 \cdot (l_b/d)^{2/3}) = 0.26181818$$

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
 $\mu = 4.0911E+008$

$$= 0.97738438$$

$$' = 0.86668818$$

error of function (3.68), Biskinis Phd = 94699.84

From 5A.2, TDY: $f_{cc} = f_c \cdot c = 43.01524$

conf. factor $c = 1.30349$

$f_c = 33.00$

From 10.3.5, ASCE 41-17, Final value of f_y : $f_y \cdot \text{Min}(1, 1.25 \cdot (l_b/d)^{2/3}) = 694.4444$

$$l_b/d = 1.00$$

$$d_1 = 44.00$$

$$R = 250.00$$

$$v = 0.00114003$$

$$N = 7389.214$$

$$A_c = 196349.541$$

$$= \cdot \text{Min}(1, 1.25 \cdot (l_b/d)^{2/3}) = 0.26181818$$

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), M_u
 $M_u = 4.0911E+008$

$= 0.97738438$
 $' = 0.86668818$
error of function (3.68), Biskinis Phd = 94699.84
From 5A.2, TBDY: $f_{cc} = f_c' \cdot c = 43.01524$
conf. factor $c = 1.30349$
 $f_c = 33.00$
From 10.3.5, ASCE 41-17, Final value of f_y : $f_y \cdot \text{Min}(1, 1.25 \cdot (l_b/d)^{2/3}) = 694.4444$
 $l_b/d = 1.00$
 $d_1 = 44.00$
 $R = 250.00$
 $v = 0.00114003$
 $N = 7389.214$
 $A_c = 196349.541$
 $= \cdot \text{Min}(1, 1.25 \cdot (l_b/d)^{2/3}) = 0.26181818$

Calculation of ratio l_b/d

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

Calculation of Shear Strength $V_r = \text{Min}(V_{r1}, V_{r2}) = 483868.491$

Calculation of Shear Strength at edge 1, $V_{r1} = 483868.491$
 $V_{r1} = V_{Col} \text{ ((10.3), ASCE 41-17)} = k_{nl} \cdot V_{ColO}$
 $V_{ColO} = 483868.491$
 $k_{nl} = 1$ (zero step-static loading)

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

$= 1$ (normal-weight concrete)
Mean concrete strength: $f_c' = (f_c'_{\text{jacket}} \cdot \text{Area}_{\text{jacket}} + f_c'_{\text{core}} \cdot \text{Area}_{\text{core}}) / \text{Area}_{\text{section}} = 28.32$, but $f_c'^{0.5} \leq 8.3$
MPa (22.5.3.1, ACI 318-14)
 $M/Vd = 2.00$
 $M_u = 2.6359963E-011$
 $V_u = 2.6832056E-030$
 $d = 0.8 \cdot D = 400.00$
 $N_u = 7389.214$
 $A_g = 196349.541$
From (11.5.4.8), ACI 318-14: $V_s = V_{s1} + V_{s2} = 274155.678$
 $V_{s1} = 274155.678$ is calculated for jacket, with:
 $A_v = \cdot /2 \cdot A_{\text{stirrup}} = 123370.055$
 $f_y = 555.5556$
 $s = 100.00$
 V_{s1} is multiplied by $Col1 = 1.00$
 $s/d = 0.25$
 $V_{s2} = 0.00$ is calculated for core, with:
 $A_v = \cdot /2 \cdot A_{\text{stirrup}} = 78956.835$
 $f_y = 444.4444$
 $s = 250.00$
 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: $V_s + V_f \leq 444245.712$
 $b_w \cdot d = \cdot d \cdot d/4 = 125663.706$

Calculation of Shear Strength at edge 2, $V_{r2} = 483868.491$
 $V_{r2} = V_{Col} \text{ ((10.3), ASCE 41-17)} = k_{nl} \cdot V_{ColO}$
 $V_{ColO} = 483868.491$

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: $f_c' = (f_c'_{\text{jacket}} \cdot \text{Area}_{\text{jacket}} + f_c'_{\text{core}} \cdot \text{Area}_{\text{core}}) / \text{Area}_{\text{section}} = 28.32$, but $f_c'^{0.5} \leq 8.3$
MPa (22.5.3.1, ACI 318-14)

$M/Vd = 2.00$

$\mu_u = 2.6359963E-011$

$\nu_u = 2.6832056E-030$

$d = 0.8 \cdot D = 400.00$

$N_u = 7389.214$

$A_g = 196349.541$

From (11.5.4.8), ACI 318-14: $V_s = V_{s1} + V_{s2} = 274155.678$

$V_{s1} = 274155.678$ is calculated for jacket, with:

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

$f_y = 555.5556$

$s = 100.00$

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

$s/d = 0.25$

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

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

$f_y = 444.4444$

$s = 250.00$

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: $V_s + V_f \leq 444245.712$

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

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: 3

Integration Section: (a)

Section Type: rcjcs

Constant Properties

Knowledge Factor, = 1.00

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 Primary Member: Concrete Strength, $f_c = f_{cm} = 33.00$

New material of Primary Member: Steel Strength, $f_s = f_{sm} = 555.5556$

Concrete Elasticity, $E_c = 26999.444$

Steel Elasticity, $E_s = 200000.00$

Existing Column

Existing material of Primary Member: Concrete Strength, $f_c = f_{cm} = 20.00$

Existing material of Primary Member: Steel Strength, $f_s = f_{sm} = 444.4444$

Concrete Elasticity, $E_c = 21019.039$

Steel Elasticity, $E_s = 200000.00$

External Diameter, $D = 500.00$

Internal Diameter, $D = 300.00$

Cover Thickness, $c = 25.00$

Element Length, $L = 3000.00$

Primary Member

Ribbed Bars

Ductile Steel

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

Longitudinal Bars With Ends Lapped Starting at the End Sections
Adequate Lap Length ($l_b/l_d \geq 1$)
No FRP Wrapping

Stepwise Properties

Bending Moment, $M = -1.7490\text{E}+007$
Shear Force, $V2 = -5828.436$
Shear Force, $V3 = -1.0906320\text{E}-014$
Axial Force, $F = -7386.882$
Longitudinal Reinforcement Area Distribution (in 2 divisions)
-Tension: $A_{sl,t} = 0.00$
-Compression: $A_{sl,c} = 3053.628$
Longitudinal Reinforcement Area Distribution (in 3 divisions)
-Tension: $A_{sl,ten} = 1017.876$
-Compression: $A_{sl,com} = 1017.876$
-Middle: $A_{sl,mid} = 1017.876$
Mean Diameter of Tension Reinforcement, $DbL = 18.00$

Existing component: From table 7-7, ASCE 41_17: Final chord rotation Capacity $u_R = \phi \cdot u = 0.05626239$
 $u = y + p = 0.05626239$

- Calculation of y -

$y = (M_y \cdot L_s / 3) / E_{eff} = 0.01502587$ ((4.29), Biskinis Phd))
 $M_y = 3.6258\text{E}+008$
 $L_s = M/V$ (with $L_s > 0.1 \cdot L$ and $L_s < 2 \cdot L$) = 3000.755
From table 10.5, ASCE 41_17: $E_{eff} = \text{factor} \cdot E_c \cdot I_g = 2.4137\text{E}+013$
factor = 0.30
 $A_g = 196349.541$
Mean concrete strength: $f'_c = (f'_c \cdot \text{jacket} \cdot \text{Area}_{\text{jacket}} + f'_c \cdot \text{core} \cdot \text{Area}_{\text{core}}) / \text{Area}_{\text{section}} = 28.32$
 $N = 7386.882$
 $E_c \cdot I_g = E_{c \cdot \text{jacket}} \cdot I_{g \cdot \text{jacket}} + E_{c \cdot \text{core}} \cdot I_{g \cdot \text{core}} = 8.0455\text{E}+013$

Calculation of Yielding Moment M_y

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

$M_y = \min(M_{y,ten}, M_{y,com}) = 3.6258\text{E}+008$
 y ((10a) or (10b)) = $1.0622219\text{E}-005$
 $M_{y,ten}$ (8a) = $3.6258\text{E}+008$
 y_{ten} (7a) = 65.43626
error of function (7a) = 0.00293096
 $M_{y,com}$ (8b) = $7.5621\text{E}+008$
 y_{com} (7b) = 64.56804
error of function (7b) = -0.00721906
with $e_y = 0.00277778$
 $e_{co} = 0.002$
 $a_{pl} = 0.35$ ((9a) in Biskinis and Fardis for no FRP Wrap)
 $d1 = 44.00$
 $R = 250.00$
 $v = 0.00114003$
 $N = 7386.882$
 $A_c = 196349.541$
= 0.26181818
with $f_c = 33.00$

Calculation of ratio l_b/l_d

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

- Calculation of p -

From table 10-9: $p = 0.04123652$

with:

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

shear control ratio $V_y E / V_{Col0E} = 0.5636717$

$d = d_{external} = 0.00$

$s = s_{external} = 0.00$

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

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.

$NUD = 7386.882$

$A_g = 196349.541$

$f_{cE} = (f_{c,jacket} \cdot \text{Area}_{jacket} + f_{c,core} \cdot \text{Area}_{core}) / \text{section_area} = 28.32$

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

$f_{yIE} = (f_{y,ext_Trans_Reinf} \cdot \text{Area}_{ext_Trans_Reinf} + f_{y,int_Trans_Reinf} \cdot \text{Area}_{int_Trans_Reinf}) / \text{Area}_{Tot_Trans_Rein} = 539.4201$

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

$f_{cE} = 28.32$

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

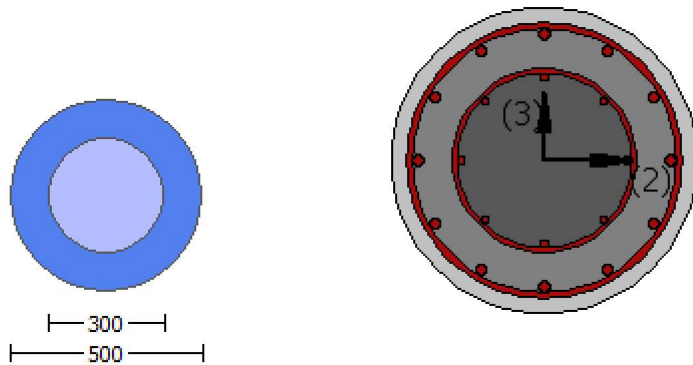
Limit State: Life Safety (data interpolation between analysis steps 1 and 2)

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, $\gamma = 1.00$

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 Primary Member: Concrete Strength, $f_c = f_{c_lower_bound} = 25.00$

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

Concrete Elasticity, $E_c = 26999.444$

Steel Elasticity, $E_s = 200000.00$

Existing Column

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

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

Concrete Elasticity, $E_c = 21019.039$

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, $f_c = f_{cm} = 33.00$

New material: Steel Strength, $f_s = f_{sm} = 555.5556$

Existing Column

Existing material: Concrete Strength, $f_c = f_{cm} = 20.00$

Existing material: Steel Strength, $f_s = f_{sm} = 444.4444$

#####

External Diameter, $D = 500.00$

Internal Diameter, $D = 300.00$

Cover Thickness, $c = 25.00$

Element Length, $L = 3000.00$

Primary Member

Ribbed Bars

Ductile Steel

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

Longitudinal Bars With Ends Lapped Starting at the End Sections

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

No FRP Wrapping

Stepwise Properties

EDGE -A-

Bending Moment, $M_a = -1.7490E+007$

Shear Force, $V_a = -5828.436$

EDGE -B-

Bending Moment, $M_b = 0.03974705$

Shear Force, $V_b = 5828.436$
 BOTH EDGES
 Axial Force, $F = -7386.882$
 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 $V_R = V_n = 430747.058$
 $V_n ((10.3), ASCE 41-17) = knl * V_{Col0} = 430747.058$
 $V_{Col} = 430747.058$
 $knl = 1.00$
 $displacement_ductility_demand = 0.07234743$

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).

$= 1$ (normal-weight concrete)
 Mean concrete strength: $f'_c = (f'_{c,jacket} * Area_{jacket} + f'_{c,core} * Area_{core}) / Area_{section} = 21.76$, but $f'_c^{0.5} \leq 8.3$
 MPa (22.5.3.1, ACI 318-14)
 $M/Vd = 2.00$
 $\mu_u = 0.03974705$
 $V_u = 5828.436$
 $d = 0.8 * D = 400.00$
 $N_u = 7386.882$
 $A_g = 196349.541$
 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:
 $A_v = A_{stirrup} / 2 = 78956.835$
 $f_y = 400.00$
 $s = 250.00$
 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: $V_s + V_f \leq 389409.072$
 $b_w * d = 125663.706$

$displacement_ductility_demand$ is calculated as δ_u / y

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

From analysis, chord rotation $\theta = 0.00010868$
 $y = (M_y * L_s / 3) / E_{eff} = 0.00150221$ ((4.29), Biskinis Phd))
 $M_y = 3.6258E+008$
 $L_s = M/V$ (with $L_s > 0.1 * L$ and $L_s < 2 * L$) = 300.00
 From table 10.5, ASCE 41_17: $E_{eff} = factor * E_c * I_g = 2.4137E+013$
 $factor = 0.30$
 $A_g = 196349.541$
 Mean concrete strength: $f'_c = (f'_{c,jacket} * Area_{jacket} + f'_{c,core} * Area_{core}) / Area_{section} = 28.32$
 $N = 7386.882$
 $E_c * I_g = E_{c,jacket} * I_{g,jacket} + E_{c,core} * I_{g,core} = 8.0455E+013$

Calculation of Yielding Moment M_y

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

$M_y = \min(M_{y_ten}, M_{y_com}) = 3.6258E+008$

$\rho_y ((10a) \text{ or } (10b)) = 1.0622219E-005$

$M_{y_ten} (8a) = 3.6258E+008$

$\rho_{y_ten} (7a) = 65.43626$

error of function (7a) = 0.00293096

$M_{y_com} (8b) = 7.5621E+008$

$\rho_{y_com} (7b) = 64.56804$

error of function (7b) = -0.00721906

with $e_y = 0.00277778$

$e_{co} = 0.002$

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

$d_1 = 44.00$

$R = 250.00$

$v = 0.00114003$

$N = 7386.882$

$A_c = 196349.541$

$= 0.26181818$

with $f_c = 33.00$

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

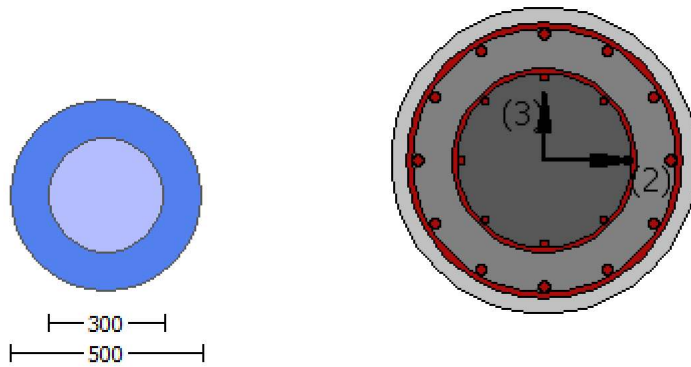
Limit State: Life Safety (data interpolation between analysis steps 1 and 2)

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, $\gamma = 1.00$

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

Consequently:

Jacket

New material of Primary Member: Concrete Strength, $f_c = f_{cm} = 33.00$

New material of Primary Member: Steel Strength, $f_s = f_{sm} = 555.5556$

Concrete Elasticity, $E_c = 26999.444$

Steel Elasticity, $E_s = 200000.00$

Existing Column

Existing material of Primary Member: Concrete Strength, $f_c = f_{cm} = 20.00$

Existing material of Primary Member: Steel Strength, $f_s = f_{sm} = 444.4444$

Concrete Elasticity, $E_c = 21019.039$

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, $f_s = 1.25 \cdot f_{sm} = 694.4444$

Existing Column

Existing material: Steel Strength, $f_s = 1.25 \cdot f_{sm} = 555.5556$

#####

External Diameter, $D = 500.00$

Internal Diameter, $D = 300.00$

Cover Thickness, $c = 25.00$

Mean Confinement Factor overall section = 1.30349

Element Length, $L = 3000.00$

Primary Member

Ribbed Bars

Ductile Steel

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

Longitudinal Bars With Ends Lapped Starting at the End Sections

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

No FRP Wrapping

Stepwise Properties

At local axis: 3

EDGE -A-

Shear Force, $V_a = -1.4969033E-031$

EDGE -B-

Shear Force, $V_b = 1.4969033E-031$

BOTH EDGES

Axial Force, $F = -7389.214$

Longitudinal Reinforcement Area Distribution (in 2 divisions)

-Tension: Aslt = 0.00
 -Compression: Aslc = 3053.628
 Longitudinal Reinforcement Area Distribution (in 3 divisions)
 -Tension: Asl,ten = 1017.876
 -Compression: Asl,com = 1017.876
 -Middle: Asl,mid = 1017.876

Calculation of Shear Capacity ratio , $V_e/V_r = 0.5636717$
 Member Controlled by Flexure ($V_e/V_r < 1$)
 Calculation of Shear Demand from fig. R18.6.5, ACI 318-14 $V_e = (M_{pr1} + M_{pr2})/l_n = 272742.977$
 with
 $M_{pr1} = \text{Max}(Mu_{1+}, Mu_{1-}) = 4.0911E+008$
 $Mu_{1+} = 4.0911E+008$, 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
 $Mu_{1-} = 4.0911E+008$, 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
 $M_{pr2} = \text{Max}(Mu_{2+}, Mu_{2-}) = 4.0911E+008$
 $Mu_{2+} = 4.0911E+008$, 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
 $Mu_{2-} = 4.0911E+008$, 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
 $Mu = 4.0911E+008$

$\phi = 0.97738438$
 $\lambda = 0.86668818$
 error of function (3.68), Biskinis Phd = 94699.84
 From 5A.2, TBDY: $f_{cc} = f_c^* c = 43.01524$
 conf. factor $c = 1.30349$
 $f_c = 33.00$
 From 10.3.5, ASCE 41-17, Final value of f_y : $f_y * \text{Min}(1, 1.25 * (l_b/d)^{2/3}) = 694.4444$
 $l_b/d = 1.00$
 $d_1 = 44.00$
 $R = 250.00$
 $v = 0.00114003$
 $N = 7389.214$
 $A_c = 196349.541$
 $\phi * \text{Min}(1, 1.25 * (l_b/d)^{2/3}) = 0.26181818$

Calculation of ratio l_b/d

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

Calculation of Mu_{1-}

Calculation of ultimate Moment Strength ((3.67), Biskinis Phd), Mu
 $Mu = 4.0911E+008$

$\phi = 0.97738438$
 $\lambda = 0.86668818$
 error of function (3.68), Biskinis Phd = 94699.84
 From 5A.2, TBDY: $f_{cc} = f_c^* c = 43.01524$
 conf. factor $c = 1.30349$

$f_c = 33.00$
 From 10.3.5, ASCE 41-17, Final value of f_y : $f_y \cdot \text{Min}(1, 1.25 \cdot (l_b/d)^{2/3}) = 694.4444$
 $l_b/d = 1.00$
 $d_1 = 44.00$
 $R = 250.00$
 $v = 0.00114003$
 $N = 7389.214$
 $A_c = 196349.541$
 $= \cdot \text{Min}(1, 1.25 \cdot (l_b/d)^{2/3}) = 0.26181818$

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
 $\mu = 4.0911\text{E}+008$

$= 0.97738438$
 $' = 0.86668818$
 error of function (3.68), Biskinis Phd = 94699.84
 From 5A.2, TBDY: $f_{cc} = f_c \cdot c = 43.01524$
 conf. factor $c = 1.30349$
 $f_c = 33.00$
 From 10.3.5, ASCE 41-17, Final value of f_y : $f_y \cdot \text{Min}(1, 1.25 \cdot (l_b/d)^{2/3}) = 694.4444$
 $l_b/d = 1.00$
 $d_1 = 44.00$
 $R = 250.00$
 $v = 0.00114003$
 $N = 7389.214$
 $A_c = 196349.541$
 $= \cdot \text{Min}(1, 1.25 \cdot (l_b/d)^{2/3}) = 0.26181818$

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
 $\mu = 4.0911\text{E}+008$

$= 0.97738438$
 $' = 0.86668818$
 error of function (3.68), Biskinis Phd = 94699.84
 From 5A.2, TBDY: $f_{cc} = f_c \cdot c = 43.01524$
 conf. factor $c = 1.30349$
 $f_c = 33.00$
 From 10.3.5, ASCE 41-17, Final value of f_y : $f_y \cdot \text{Min}(1, 1.25 \cdot (l_b/d)^{2/3}) = 694.4444$
 $l_b/d = 1.00$
 $d_1 = 44.00$
 $R = 250.00$
 $v = 0.00114003$
 $N = 7389.214$
 $A_c = 196349.541$
 $= \cdot \text{Min}(1, 1.25 \cdot (l_b/d)^{2/3}) = 0.26181818$

Calculation of ratio l_b/d

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

Calculation of Shear Strength $V_r = \text{Min}(V_{r1}, V_{r2}) = 483868.491$

Calculation of Shear Strength at edge 1, $V_{r1} = 483868.491$

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

$V_{ColO} = 483868.491$

$k_n l = 1$ (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: $f'_c = (f'_{c_jacket} \text{Area}_{jacket} + f'_{c_core} \text{Area}_{core}) / \text{Area}_{section} = 28.32$, but $f'_c^{0.5} \leq 8.3$ MPa (22.5.3.1, ACI 318-14)

$M/Vd = 2.00$

$\mu_u = 1.4802042E-011$

$\nu_u = 1.4969033E-031$

$d = 0.8 \cdot D = 400.00$

$N_u = 7389.214$

$A_g = 196349.541$

From (11.5.4.8), ACI 318-14: $V_s = V_{s1} + V_{s2} = 274155.678$

$V_{s1} = 274155.678$ is calculated for jacket, with:

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

$f_y = 555.5556$

$s = 100.00$

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

$s/d = 0.25$

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

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

$f_y = 444.4444$

$s = 250.00$

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: $V_s + V_f \leq 444245.712$

$b_w d = A_{stirrup} d / 4 = 125663.706$

Calculation of Shear Strength at edge 2, $V_{r2} = 483868.491$

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

$V_{ColO} = 483868.491$

$k_n l = 1$ (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: $f'_c = (f'_{c_jacket} \text{Area}_{jacket} + f'_{c_core} \text{Area}_{core}) / \text{Area}_{section} = 28.32$, but $f'_c^{0.5} \leq 8.3$ MPa (22.5.3.1, ACI 318-14)

$M/Vd = 2.00$

$\mu_u = 1.4802042E-011$

$\nu_u = 1.4969033E-031$

$d = 0.8 \cdot D = 400.00$

$N_u = 7389.214$

$A_g = 196349.541$

From (11.5.4.8), ACI 318-14: $V_s = V_{s1} + V_{s2} = 274155.678$

$V_{s1} = 274155.678$ is calculated for jacket, with:

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

$f_y = 555.5556$

$s = 100.00$
 V_{s1} is multiplied by $Col1 = 1.00$
 $s/d = 0.25$
 $V_{s2} = 0.00$ is calculated for core, with:
 $A_v = \sqrt{2} \cdot A_{stirrup} = 78956.835$
 $f_y = 444.4444$
 $s = 250.00$
 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: $V_s + V_f \leq 444245.712$
 $b_w \cdot d = \sqrt{d} \cdot d/4 = 125663.706$

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, $\phi = 1.00$
 Mean strength values are used for both shear and moment calculations.
 Consequently:
 Jacket
 New material of Primary Member: Concrete Strength, $f_c = f_{cm} = 33.00$
 New material of Primary Member: Steel Strength, $f_s = f_{sm} = 555.5556$
 Concrete Elasticity, $E_c = 26999.444$
 Steel Elasticity, $E_s = 200000.00$
 Existing Column
 Existing material of Primary Member: Concrete Strength, $f_c = f_{cm} = 20.00$
 Existing material of Primary Member: Steel Strength, $f_s = f_{sm} = 444.4444$
 Concrete Elasticity, $E_c = 21019.039$
 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, $f_s = 1.25 \cdot f_{sm} = 694.4444$
 Existing Column
 Existing material: Steel Strength, $f_s = 1.25 \cdot f_{sm} = 555.5556$
 #####
 External Diameter, $D = 500.00$
 Internal Diameter, $D = 300.00$
 Cover Thickness, $c = 25.00$
 Mean Confinement Factor overall section = 1.30349
 Element Length, $L = 3000.00$
 Primary Member
 Ribbed Bars
 Ductile Steel
 With Detailing for Earthquake Resistance (including stirrups closed at 135°)
 Longitudinal Bars With Ends Lapped Starting at the End Sections
 Adequate Lap Length ($l_o/l_{ou}, \min \geq 1$)
 No FRP Wrapping

Stepwise Properties

At local axis: 2
 EDGE -A-
 Shear Force, $V_a = -2.6832056E-030$

EDGE -B-

Shear Force, $V_b = 2.6832056E-030$

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, $V_e/V_r = 0.5636717$

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

Calculation of Shear Demand from fig. R18.6.5, ACI 318-14 $V_e = (M_{pr1} + M_{pr2})/l_n = 272742.977$
with

$M_{pr1} = \text{Max}(Mu_{1+}, Mu_{1-}) = 4.0911E+008$

$Mu_{1+} = 4.0911E+008$, 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

$Mu_{1-} = 4.0911E+008$, 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

$M_{pr2} = \text{Max}(Mu_{2+}, Mu_{2-}) = 4.0911E+008$

$Mu_{2+} = 4.0911E+008$, 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

$Mu_{2-} = 4.0911E+008$, 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), Mu

$Mu = 4.0911E+008$

$\phi = 0.97738438$

$\phi' = 0.86668818$

error of function (3.68), Biskinis Phd = 94699.84

From 5A.2, TBDY: $f_{cc} = f_c' \cdot c = 43.01524$

conf. factor $c = 1.30349$

$f_c = 33.00$

From 10.3.5, ASCE 41-17, Final value of f_y : $f_y \cdot \text{Min}(1, 1.25 \cdot (l_b/l_d)^{2/3}) = 694.4444$

$l_b/l_d = 1.00$

$d_1 = 44.00$

$R = 250.00$

$v = 0.00114003$

$N = 7389.214$

$A_c = 196349.541$

$\phi' \cdot \text{Min}(1, 1.25 \cdot (l_b/l_d)^{2/3}) = 0.26181818$

Calculation of ratio l_b/l_d

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

Calculation of Mu_{1-}

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

$Mu = 4.0911E+008$

```

= 0.97738438
' = 0.86668818
error of function (3.68), Biskinis Phd = 94699.84
From 5A.2, TBDY: fcc = fc* c = 43.01524
conf. factor c = 1.30349
fc = 33.00
From 10.3.5, ASCE 41-17, Final value of fy: fy*Min(1,1.25*(lb/d)^ 2/3) = 694.4444
lb/d = 1.00
d1 = 44.00
R = 250.00
v = 0.00114003
N = 7389.214
Ac = 196349.541
= *Min(1,1.25*(lb/d)^ 2/3) = 0.26181818

```

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 = 4.0911E+008

```

= 0.97738438
' = 0.86668818
error of function (3.68), Biskinis Phd = 94699.84
From 5A.2, TBDY: fcc = fc* c = 43.01524
conf. factor c = 1.30349
fc = 33.00
From 10.3.5, ASCE 41-17, Final value of fy: fy*Min(1,1.25*(lb/d)^ 2/3) = 694.4444
lb/d = 1.00
d1 = 44.00
R = 250.00
v = 0.00114003
N = 7389.214
Ac = 196349.541
= *Min(1,1.25*(lb/d)^ 2/3) = 0.26181818

```

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 = 4.0911E+008

```

= 0.97738438
' = 0.86668818
error of function (3.68), Biskinis Phd = 94699.84
From 5A.2, TBDY: fcc = fc* c = 43.01524
conf. factor c = 1.30349
fc = 33.00
From 10.3.5, ASCE 41-17, Final value of fy: fy*Min(1,1.25*(lb/d)^ 2/3) = 694.4444
lb/d = 1.00
d1 = 44.00

```

$$\begin{aligned}
 R &= 250.00 \\
 v &= 0.00114003 \\
 N &= 7389.214 \\
 A_c &= 196349.541 \\
 &= *Min(1, 1.25*(lb/d)^{2/3}) = 0.26181818
 \end{aligned}$$

Calculation of ratio lb/d

Adequate Lap Length: lb/d \geq 1

Calculation of Shear Strength $V_r = \text{Min}(V_{r1}, V_{r2}) = 483868.491$

Calculation of Shear Strength at edge 1, $V_{r1} = 483868.491$

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

$V_{ColO} = 483868.491$

$knl = 1$ (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: $f'_c = (f'_c_{\text{jacket}} * \text{Area}_{\text{jacket}} + f'_c_{\text{core}} * \text{Area}_{\text{core}}) / \text{Area}_{\text{section}} = 28.32$, but $f'_c^{0.5} \leq 8.3$ MPa (22.5.3.1, ACI 318-14)

$M/Vd = 2.00$

$\mu_u = 2.6359963E-011$

$\nu_u = 2.6832056E-030$

$d = 0.8 * D = 400.00$

$N_u = 7389.214$

$A_g = 196349.541$

From (11.5.4.8), ACI 318-14: $V_s = V_{s1} + V_{s2} = 274155.678$

$V_{s1} = 274155.678$ is calculated for jacket, with:

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

$f_y = 555.5556$

$s = 100.00$

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

$s/d = 0.25$

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

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

$f_y = 444.4444$

$s = 250.00$

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: $V_s + V_f \leq 444245.712$

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

Calculation of Shear Strength at edge 2, $V_{r2} = 483868.491$

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

$V_{ColO} = 483868.491$

$knl = 1$ (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: $f'_c = (f'_c_{\text{jacket}} * \text{Area}_{\text{jacket}} + f'_c_{\text{core}} * \text{Area}_{\text{core}}) / \text{Area}_{\text{section}} = 28.32$, but $f'_c^{0.5} \leq 8.3$ MPa (22.5.3.1, ACI 318-14)

$M/Vd = 2.00$

$\mu_u = 2.6359963E-011$

$\nu_u = 2.6832056E-030$

$d = 0.8 * D = 400.00$

$N_u = 7389.214$

$A_g = 196349.541$
 From (11.5.4.8), ACI 318-14: $V_s = V_{s1} + V_{s2} = 274155.678$
 $V_{s1} = 274155.678$ is calculated for jacket, with:
 $A_v = \frac{1}{2} A_{stirrup} = 123370.055$
 $f_y = 555.5556$
 $s = 100.00$
 V_{s1} is multiplied by $Col1 = 1.00$
 $s/d = 0.25$
 $V_{s2} = 0.00$ is calculated for core, with:
 $A_v = \frac{1}{2} A_{stirrup} = 78956.835$
 $f_y = 444.4444$
 $s = 250.00$
 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: $V_s + V_f \leq 444245.712$
 $b_w * d = \frac{1}{4} * d * d = 125663.706$

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, $\phi = 1.00$
 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 Primary Member: Concrete Strength, $f_c = f_{cm} = 33.00$
 New material of Primary Member: Steel Strength, $f_s = f_{sm} = 555.5556$
 Concrete Elasticity, $E_c = 26999.444$
 Steel Elasticity, $E_s = 200000.00$
 Existing Column
 Existing material of Primary Member: Concrete Strength, $f_c = f_{cm} = 20.00$
 Existing material of Primary Member: Steel Strength, $f_s = f_{sm} = 444.4444$
 Concrete Elasticity, $E_c = 21019.039$
 Steel Elasticity, $E_s = 200000.00$
 External Diameter, $D = 500.00$
 Internal Diameter, $D = 300.00$
 Cover Thickness, $c = 25.00$
 Element Length, $L = 3000.00$
 Primary Member
 Ribbed Bars
 Ductile Steel
 With Detailing for Earthquake Resistance (including stirrups closed at 135°)
 Longitudinal Bars With Ends Lapped Starting at the End Sections
 Adequate Lap Length ($l_b/d \geq 1$)
 No FRP Wrapping

Stepwise Properties

Bending Moment, $M = 7.1068928E-012$
 Shear Force, $V_2 = 5828.436$
 Shear Force, $V_3 = 1.0906320E-014$
 Axial Force, $F = -7386.882$
 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 $u_R = * u = 0.04874756$
 $u = y + p = 0.04874756$

- Calculation of y -

$y = (My * L_s / 3) / E_{eff} = 0.00751104$ ((4.29), Biskinis Phd))
 $My = 3.6258E+008$
 $L_s = M/V$ (with $L_s > 0.1 * L$ and $L_s < 2 * L$) = 1500.00
 From table 10.5, ASCE 41_17: $E_{eff} = factor * E_c * I_g = 2.4137E+013$
 $factor = 0.30$
 $Ag = 196349.541$
 Mean concrete strength: $fc' = (fc'_{jacket} * Area_{jacket} + fc'_{core} * Area_{core}) / Area_{section} = 28.32$
 $N = 7386.882$
 $E_c * I_g = E_{c_jacket} * I_{g_jacket} + E_{c_core} * I_{g_core} = 8.0455E+013$

Calculation of Yielding Moment My

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

$My = \min(My_{ten}, My_{com}) = 3.6258E+008$
 y ((10a) or (10b)) = 1.0622219E-005
 My_{ten} (8a) = 3.6258E+008
 $_{ten}$ (7a) = 65.43626
 error of function (7a) = 0.00293096
 My_{com} (8b) = 7.5621E+008
 $_{com}$ (7b) = 64.56804
 error of function (7b) = -0.00721906
 with $ey = 0.00277778$
 $eco = 0.002$
 $apl = 0.35$ ((9a) in Biskinis and Fardis for no FRP Wrap)
 $d1 = 44.00$
 $R = 250.00$
 $v = 0.00114003$
 $N = 7386.882$
 $Ac = 196349.541$
 $= 0.26181818$
 with $fc = 33.00$

Calculation of ratio lb/d

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

- Calculation of p -

From table 10-9: $p = 0.04123652$

with:

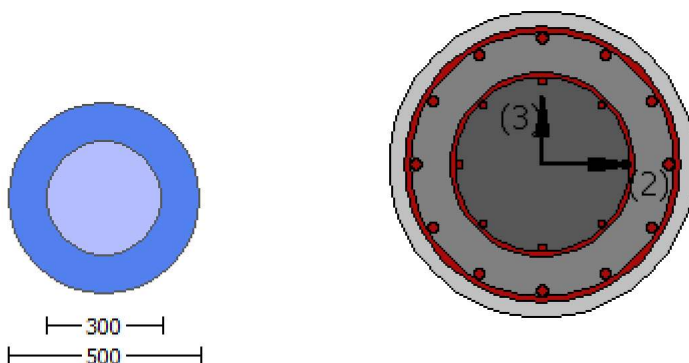
- Columns not controlled by inadequate development or splicing along the clear height because $lb/d \geq 1$
 shear control ratio $VyE/VCoIE = 0.5636717$
 $d = d_{external} = 0.00$
 $s = s_{external} = 0.00$
 $t = s1 + s2 + 2 * tf / bw * (ffe/fs) = 0.00323428$
 jacket: $s1 = Av1 * (* Dc / 2) / (s1 * Ag) = 0.0027646$

$A_{v1} = 78.53982$, 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.
 $NUD = 7386.882$
 $A_g = 196349.541$
 $f_{cE} = (f_{c_jacket} \cdot \text{Area_jacket} + f_{c_core} \cdot \text{Area_core}) / \text{section_area} = 28.32$
 $f_{yE} = (f_{y_ext_Long_Reinf} \cdot \text{Area_ext_Long_Reinf} + f_{y_int_Long_Reinf} \cdot \text{Area_int_Long_Reinf}) / \text{Area_Tot_Long_Rein} = 2.1219958E-314$
 $f_{yE} = (f_{y_ext_Trans_Reinf} \cdot \text{Area_ext_Trans_Reinf} + f_{y_int_Trans_Reinf} \cdot \text{Area_int_Trans_Reinf}) / \text{Area_Tot_Trans_Rein} = 539.4201$
 $p_l = \text{Area_Tot_Long_Rein} / (A_g) = 0.015552$
 $f_{cE} = 28.32$

 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: rcjcs

Constant Properties

Knowledge Factor, $\gamma = 1.00$

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 Primary Member: Concrete Strength, $f_c = f_{c_lower_bound} = 25.00$

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

Concrete Elasticity, $E_c = 26999.444$

Steel Elasticity, $E_s = 200000.00$

Existing Column

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

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

Concrete Elasticity, $E_c = 21019.039$

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, $f_c = f_{cm} = 33.00$

New material: Steel Strength, $f_s = f_{sm} = 555.5556$

Existing Column

Existing material: Concrete Strength, $f_c = f_{cm} = 20.00$

Existing material: Steel Strength, $f_s = f_{sm} = 444.4444$

#####

External Diameter, $D = 500.00$

Internal Diameter, $D = 300.00$

Cover Thickness, $c = 25.00$

Element Length, $L = 3000.00$

Primary Member

Ribbed Bars

Ductile Steel

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

Longitudinal Bars With Ends Lapped Starting at the End Sections

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

No FRP Wrapping

Stepwise Properties

EDGE -A-

Bending Moment, $M_a = 2.5536198E-011$

Shear Force, $V_a = -1.0906320E-014$

EDGE -B-

Bending Moment, $M_b = 7.1068928E-012$

Shear Force, $V_b = 1.0906320E-014$

BOTH EDGES

Axial Force, $F = -7386.882$

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 $VR = *V_n = 430747.058$

V_n ((10.3), ASCE 41-17) = $k_n \cdot V_{Col0} = 430747.058$

$V_{Col} = 430747.058$

$k_n = 1.00$

$\text{displacement_ductility_demand} = 0.00$

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: $f_c' = (f_c'_{\text{jacket}} \cdot \text{Area}_{\text{jacket}} + f_c'_{\text{core}} \cdot \text{Area}_{\text{core}}) / \text{Area}_{\text{section}} = 21.76$, but $f_c'^{0.5} \leq 8.3$ MPa (22.5.3.1, ACI 318-14)

$M/Vd = 2.00$

$\mu_u = 7.1068928E-012$

$\nu_u = 1.0906320E-014$

$d = 0.8 \cdot D = 400.00$

$N_u = 7386.882$

$A_g = 196349.541$

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:

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

$f_y = 500.00$

$s = 100.00$

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

$s/d = 0.25$

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

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

$f_y = 400.00$

$s = 250.00$

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: $V_s + V_f \leq 389409.072$

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

$\text{displacement_ductility_demand}$ is calculated as $\frac{1}{y}$

- Calculation of $\frac{1}{y}$ for END B -
for rotation axis 2 and integ. section (b)

From analysis, chord rotation $\theta = 8.9566785E-023$

$y = (M_y \cdot L_s / 3) / E_{\text{eff}} = 0.00751104$ ((4.29), Biskinis Phd))

$M_y = 3.6258E+008$

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

From table 10.5, ASCE 41_17: $E_{\text{eff}} = \text{factor} \cdot E_c \cdot I_g = 2.4137E+013$

$\text{factor} = 0.30$

$A_g = 196349.541$

Mean concrete strength: $f_c' = (f_c'_{\text{jacket}} \cdot \text{Area}_{\text{jacket}} + f_c'_{\text{core}} \cdot \text{Area}_{\text{core}}) / \text{Area}_{\text{section}} = 28.32$

$N = 7386.882$

$E_c \cdot I_g = E_{c_{\text{jacket}}} \cdot I_{g_{\text{jacket}}} + E_{c_{\text{core}}} \cdot I_{g_{\text{core}}} = 8.0455E+013$

Calculation of Yielding Moment M_y

Calculation of $\frac{1}{y}$ and M_y according to (7) - (8) in Biskinis and Fardis

$M_y = \text{Min}(M_{y_{\text{ten}}}, M_{y_{\text{com}}}) = 3.6258E+008$

y ((10a) or (10b)) = 1.0622219E-005

$M_{y_{\text{ten}}} (8a) = 3.6258E+008$

$\frac{1}{y_{\text{ten}}} (7a) = 65.43626$

error of function (7a) = 0.00293096

$M_{y_{\text{com}}} (8b) = 7.5621E+008$

$\frac{1}{y_{\text{com}}} (7b) = 64.56804$

error of function (7b) = -0.00721906

with $e_y = 0.00277778$

$e_{co} = 0.002$
 $apl = 0.35$ ((9a) in Biskinis and Fardis for no FRP Wrap)
 $d1 = 44.00$
 $R = 250.00$
 $v = 0.00114003$
 $N = 7386.882$
 $Ac = 196349.541$
 $= 0.26181818$
 with $f_c = 33.00$

Calculation of ratio lb/d

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

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

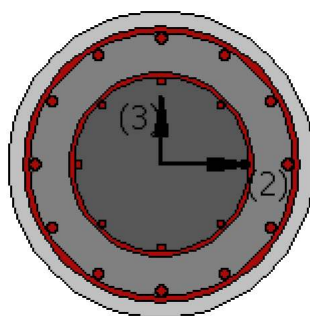
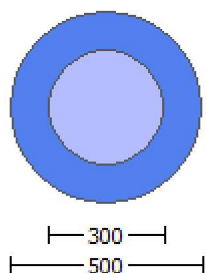
Limit State: Life Safety (data interpolation between analysis steps 1 and 2)

Analysis: Uniform +X

Check: Chord rotation capacity (μ)

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, $= 1.00$

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

Consequently:

Jacket

New material of Primary Member: Concrete Strength, $f_c = f_{cm} = 33.00$

New material of Primary Member: Steel Strength, $f_s = f_{sm} = 555.5556$

Concrete Elasticity, $E_c = 26999.444$

Steel Elasticity, $E_s = 200000.00$

Existing Column

Existing material of Primary Member: Concrete Strength, $f_c = f_{cm} = 20.00$

Existing material of Primary Member: Steel Strength, $f_s = f_{sm} = 444.4444$

Concrete Elasticity, $E_c = 21019.039$

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, $f_s = 1.25 \cdot f_{sm} = 694.4444$

Existing Column

Existing material: Steel Strength, $f_s = 1.25 \cdot f_{sm} = 555.5556$

#####

External Diameter, $D = 500.00$

Internal Diameter, $D = 300.00$

Cover Thickness, $c = 25.00$

Mean Confinement Factor overall section = 1.30349

Element Length, $L = 3000.00$

Primary Member

Ribbed Bars

Ductile Steel

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

Longitudinal Bars With Ends Lapped Starting at the End Sections

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

No FRP Wrapping

Stepwise Properties

At local axis: 3

EDGE -A-

Shear Force, $V_a = -1.4969033E-031$

EDGE -B-

Shear Force, $V_b = 1.4969033E-031$

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: $As_{l,mid} = 1017.876$

Calculation of Shear Capacity ratio, $V_e/V_r = 0.5636717$

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

Calculation of Shear Demand from fig. R18.6.5, ACI 318-14 $V_e = (M_{pr1} + M_{pr2})/l_n = 272742.977$

with

$M_{pr1} = \max(\mu_{u1+}, \mu_{u1-}) = 4.0911E+008$

$\mu_{u1+} = 4.0911E+008$, 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

$\mu_{u1-} = 4.0911E+008$, 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

$M_{pr2} = \max(\mu_{u2+}, \mu_{u2-}) = 4.0911E+008$

$\mu_{u2+} = 4.0911E+008$, 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

$\mu_{u2-} = 4.0911E+008$, 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 = 4.0911E+008

= 0.97738438
' = 0.86668818
error of function (3.68), Biskinis Phd = 94699.84
From 5A.2, TBDY: $f_{cc} = f_c \cdot c = 43.01524$
conf. factor $c = 1.30349$
 $f_c = 33.00$
From 10.3.5, ASCE 41-17, Final value of f_y : $f_y \cdot \text{Min}(1, 1.25 \cdot (l_b/d)^{2/3}) = 694.4444$
 $l_b/d = 1.00$
 $d_1 = 44.00$
 $R = 250.00$
 $v = 0.00114003$
 $N = 7389.214$
 $A_c = 196349.541$
= $\cdot \text{Min}(1, 1.25 \cdot (l_b/d)^{2/3}) = 0.26181818$

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 = 4.0911E+008

= 0.97738438
' = 0.86668818
error of function (3.68), Biskinis Phd = 94699.84
From 5A.2, TBDY: $f_{cc} = f_c \cdot c = 43.01524$
conf. factor $c = 1.30349$
 $f_c = 33.00$
From 10.3.5, ASCE 41-17, Final value of f_y : $f_y \cdot \text{Min}(1, 1.25 \cdot (l_b/d)^{2/3}) = 694.4444$
 $l_b/d = 1.00$
 $d_1 = 44.00$
 $R = 250.00$
 $v = 0.00114003$
 $N = 7389.214$
 $A_c = 196349.541$
= $\cdot \text{Min}(1, 1.25 \cdot (l_b/d)^{2/3}) = 0.26181818$

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 = 4.0911E+008

```

= 0.97738438
' = 0.86668818
error of function (3.68), Biskinis Phd = 94699.84
From 5A.2, TBDY: fcc = fc* c = 43.01524
conf. factor c = 1.30349
fc = 33.00
From 10.3.5, ASCE 41-17, Final value of fy: fy*Min(1,1.25*(lb/d)^ 2/3) = 694.4444
lb/d = 1.00
d1 = 44.00
R = 250.00
v = 0.00114003
N = 7389.214
Ac = 196349.541
= *Min(1,1.25*(lb/d)^ 2/3) = 0.26181818

```

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 = 4.0911E+008

```

= 0.97738438
' = 0.86668818
error of function (3.68), Biskinis Phd = 94699.84
From 5A.2, TBDY: fcc = fc* c = 43.01524
conf. factor c = 1.30349
fc = 33.00
From 10.3.5, ASCE 41-17, Final value of fy: fy*Min(1,1.25*(lb/d)^ 2/3) = 694.4444
lb/d = 1.00
d1 = 44.00
R = 250.00
v = 0.00114003
N = 7389.214
Ac = 196349.541
= *Min(1,1.25*(lb/d)^ 2/3) = 0.26181818

```

Calculation of ratio lb/d

Adequate Lap Length: lb/d >= 1

Calculation of Shear Strength Vr = Min(Vr1,Vr2) = 483868.491

Calculation of Shear Strength at edge 1, Vr1 = 483868.491

Vr1 = VCol ((10.3), ASCE 41-17) = knl*VCol0

VCol0 = 483868.491

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 = 28.32, but fc'^0.5 <= 8.3
MPa (22.5.3.1, ACI 318-14)
M/Vd = 2.00
Mu = 1.4802042E-011

```

$V_u = 1.4969033E-031$
 $d = 0.8 \cdot D = 400.00$
 $N_u = 7389.214$
 $A_g = 196349.541$
 From (11.5.4.8), ACI 318-14: $V_s = V_{s1} + V_{s2} = 274155.678$
 $V_{s1} = 274155.678$ is calculated for jacket, with:
 $A_v = \frac{1}{2} \cdot A_{stirrup} = 123370.055$
 $f_y = 555.5556$
 $s = 100.00$
 V_{s1} is multiplied by $Col1 = 1.00$
 $s/d = 0.25$
 $V_{s2} = 0.00$ is calculated for core, with:
 $A_v = \frac{1}{2} \cdot A_{stirrup} = 78956.835$
 $f_y = 444.4444$
 $s = 250.00$
 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: $V_s + V_f \leq 444245.712$
 $b_w \cdot d = \frac{1}{4} \cdot d \cdot d = 125663.706$

Calculation of Shear Strength at edge 2, $V_{r2} = 483868.491$
 $V_{r2} = V_{Col} ((10.3), ASCE 41-17) = knl \cdot V_{Col0}$
 $V_{Col0} = 483868.491$
 $knl = 1$ (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: $f_c' = (f_c'_{jacket} \cdot Area_{jacket} + f_c'_{core} \cdot Area_{core}) / Area_{section} = 28.32$, but $f_c'^{0.5} \leq 8.3$
 MPa (22.5.3.1, ACI 318-14)
 $M/Vd = 2.00$
 $M_u = 1.4802042E-011$
 $V_u = 1.4969033E-031$
 $d = 0.8 \cdot D = 400.00$
 $N_u = 7389.214$
 $A_g = 196349.541$
 From (11.5.4.8), ACI 318-14: $V_s = V_{s1} + V_{s2} = 274155.678$
 $V_{s1} = 274155.678$ is calculated for jacket, with:
 $A_v = \frac{1}{2} \cdot A_{stirrup} = 123370.055$
 $f_y = 555.5556$
 $s = 100.00$
 V_{s1} is multiplied by $Col1 = 1.00$
 $s/d = 0.25$
 $V_{s2} = 0.00$ is calculated for core, with:
 $A_v = \frac{1}{2} \cdot A_{stirrup} = 78956.835$
 $f_y = 444.4444$
 $s = 250.00$
 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: $V_s + V_f \leq 444245.712$
 $b_w \cdot d = \frac{1}{4} \cdot d \cdot d = 125663.706$

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, $\phi = 1.00$

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

Consequently:

Jacket

New material of Primary Member: Concrete Strength, $f_c = f_{cm} = 33.00$

New material of Primary Member: Steel Strength, $f_s = f_{sm} = 555.5556$

Concrete Elasticity, $E_c = 26999.444$

Steel Elasticity, $E_s = 200000.00$

Existing Column

Existing material of Primary Member: Concrete Strength, $f_c = f_{cm} = 20.00$

Existing material of Primary Member: Steel Strength, $f_s = f_{sm} = 444.4444$

Concrete Elasticity, $E_c = 21019.039$

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, $f_s = 1.25 \cdot f_{sm} = 694.4444$

Existing Column

Existing material: Steel Strength, $f_s = 1.25 \cdot f_{sm} = 555.5556$

#####

External Diameter, $D = 500.00$

Internal Diameter, $D = 300.00$

Cover Thickness, $c = 25.00$

Mean Confinement Factor overall section = 1.30349

Element Length, $L = 3000.00$

Primary Member

Ribbed Bars

Ductile Steel

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

Longitudinal Bars With Ends Lapped Starting at the End Sections

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

No FRP Wrapping

Stepwise Properties

At local axis: 2

EDGE -A-

Shear Force, $V_a = -2.6832056E-030$

EDGE -B-

Shear Force, $V_b = 2.6832056E-030$

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: $A_{st, \text{ten}} = 1017.876$

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

-Middle: $A_{sc, \text{mid}} = 1017.876$

Calculation of Shear Capacity ratio, $V_e/V_r = 0.5636717$

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

Calculation of Shear Demand from fig. R18.6.5, ACI 318-14 $V_e = (M_{pr1} + M_{pr2})/l_n = 272742.977$

with

$M_{pr1} = \text{Max}(M_{u1+}, M_{u1-}) = 4.0911E+008$

$M_{u1+} = 4.0911E+008$, 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

$M_{u1-} = 4.0911E+008$, 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

$$M_{pr2} = \text{Max}(M_{u2+}, M_{u2-}) = 4.0911\text{E}+008$$

$M_{u2+} = 4.0911\text{E}+008$, 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

$M_{u2-} = 4.0911\text{E}+008$, 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
 $M_u = 4.0911\text{E}+008$

$$= 0.97738438$$

$$\lambda = 0.86668818$$

error of function (3.68), Biskinis Phd = 94699.84

From 5A.2, TDY: $f_{cc} = f_c \cdot c = 43.01524$

conf. factor $c = 1.30349$

$$f_c = 33.00$$

From 10.3.5, ASCE 41-17, Final value of f_y : $f_y \cdot \text{Min}(1, 1.25 \cdot (l_b/d)^{2/3}) = 694.4444$

$$l_b/d = 1.00$$

$$d_1 = 44.00$$

$$R = 250.00$$

$$v = 0.00114003$$

$$N = 7389.214$$

$$A_c = 196349.541$$

$$= \lambda \cdot \text{Min}(1, 1.25 \cdot (l_b/d)^{2/3}) = 0.26181818$$

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
 $M_u = 4.0911\text{E}+008$

$$= 0.97738438$$

$$\lambda = 0.86668818$$

error of function (3.68), Biskinis Phd = 94699.84

From 5A.2, TDY: $f_{cc} = f_c \cdot c = 43.01524$

conf. factor $c = 1.30349$

$$f_c = 33.00$$

From 10.3.5, ASCE 41-17, Final value of f_y : $f_y \cdot \text{Min}(1, 1.25 \cdot (l_b/d)^{2/3}) = 694.4444$

$$l_b/d = 1.00$$

$$d_1 = 44.00$$

$$R = 250.00$$

$$v = 0.00114003$$

$$N = 7389.214$$

$$A_c = 196349.541$$

$$= \lambda \cdot \text{Min}(1, 1.25 \cdot (l_b/d)^{2/3}) = 0.26181818$$

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 = 4.0911E+008

= 0.97738438
' = 0.86668818
error of function (3.68), Biskinis Phd = 94699.84
From 5A.2, TBDY: fcc = fc* c = 43.01524
conf. factor c = 1.30349
fc = 33.00
From 10.3.5, ASCE 41-17, Final value of fy: fy*Min(1,1.25*(lb/d)^ 2/3) = 694.4444
lb/d = 1.00
d1 = 44.00
R = 250.00
v = 0.00114003
N = 7389.214
Ac = 196349.541
= *Min(1,1.25*(lb/d)^ 2/3) = 0.26181818

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 = 4.0911E+008

= 0.97738438
' = 0.86668818
error of function (3.68), Biskinis Phd = 94699.84
From 5A.2, TBDY: fcc = fc* c = 43.01524
conf. factor c = 1.30349
fc = 33.00
From 10.3.5, ASCE 41-17, Final value of fy: fy*Min(1,1.25*(lb/d)^ 2/3) = 694.4444
lb/d = 1.00
d1 = 44.00
R = 250.00
v = 0.00114003
N = 7389.214
Ac = 196349.541
= *Min(1,1.25*(lb/d)^ 2/3) = 0.26181818

Calculation of ratio lb/d

Adequate Lap Length: lb/d >= 1

Calculation of Shear Strength Vr = Min(Vr1,Vr2) = 483868.491

Calculation of Shear Strength at edge 1, Vr1 = 483868.491
Vr1 = VCol ((10.3), ASCE 41-17) = knl*VColO
VColO = 483868.491
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: $fc' = (fc'_{jacket} \cdot Area_{jacket} + fc'_{core} \cdot Area_{core}) / Area_{section} = 28.32$, but $fc'^{0.5} \leq 8.3$ MPa (22.5.3.1, ACI 318-14)
 $M/Vd = 2.00$
 $\mu_u = 2.6359963E-011$
 $V_u = 2.6832056E-030$
 $d = 0.8 \cdot D = 400.00$
 $N_u = 7389.214$
 $A_g = 196349.541$
From (11.5.4.8), ACI 318-14: $V_s = V_{s1} + V_{s2} = 274155.678$
 $V_{s1} = 274155.678$ is calculated for jacket, with:
 $A_v = \sqrt{2} \cdot A_{stirrup} = 123370.055$
 $f_y = 555.5556$
 $s = 100.00$
 V_{s1} is multiplied by $Col1 = 1.00$
 $s/d = 0.25$
 $V_{s2} = 0.00$ is calculated for core, with:
 $A_v = \sqrt{2} \cdot A_{stirrup} = 78956.835$
 $f_y = 444.4444$
 $s = 250.00$
 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: $V_s + V_f \leq 444245.712$
 $b_w \cdot d = \sqrt{3} \cdot d \cdot d / 4 = 125663.706$

Calculation of Shear Strength at edge 2, $V_{r2} = 483868.491$
 $V_{r2} = V_{Col} ((10.3), ASCE 41-17) = knl \cdot V_{Col0}$
 $V_{Col0} = 483868.491$
 $knl = 1$ (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: $fc' = (fc'_{jacket} \cdot Area_{jacket} + fc'_{core} \cdot Area_{core}) / Area_{section} = 28.32$, but $fc'^{0.5} \leq 8.3$ MPa (22.5.3.1, ACI 318-14)
 $M/Vd = 2.00$
 $\mu_u = 2.6359963E-011$
 $V_u = 2.6832056E-030$
 $d = 0.8 \cdot D = 400.00$
 $N_u = 7389.214$
 $A_g = 196349.541$
From (11.5.4.8), ACI 318-14: $V_s = V_{s1} + V_{s2} = 274155.678$
 $V_{s1} = 274155.678$ is calculated for jacket, with:
 $A_v = \sqrt{2} \cdot A_{stirrup} = 123370.055$
 $f_y = 555.5556$
 $s = 100.00$
 V_{s1} is multiplied by $Col1 = 1.00$
 $s/d = 0.25$
 $V_{s2} = 0.00$ is calculated for core, with:
 $A_v = \sqrt{2} \cdot A_{stirrup} = 78956.835$
 $f_y = 444.4444$
 $s = 250.00$
 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: $V_s + V_f \leq 444245.712$
 $b_w \cdot d = \sqrt{3} \cdot d \cdot d / 4 = 125663.706$

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: 3

Integration Section: (b)

Section Type: rcjcs

Constant Properties

Knowledge Factor, $\gamma = 1.00$

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 Primary Member: Concrete Strength, $f_c = f_{cm} = 33.00$

New material of Primary Member: Steel Strength, $f_s = f_{sm} = 555.5556$

Concrete Elasticity, $E_c = 26999.444$

Steel Elasticity, $E_s = 200000.00$

Existing Column

Existing material of Primary Member: Concrete Strength, $f_c = f_{cm} = 20.00$

Existing material of Primary Member: Steel Strength, $f_s = f_{sm} = 444.4444$

Concrete Elasticity, $E_c = 21019.039$

Steel Elasticity, $E_s = 200000.00$

External Diameter, $D = 500.00$

Internal Diameter, $D = 300.00$

Cover Thickness, $c = 25.00$

Element Length, $L = 3000.00$

Primary Member

Ribbed Bars

Ductile Steel

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

Longitudinal Bars With Ends Lapped Starting at the End Sections

Adequate Lap Length ($l_b/d \geq 1$)

No FRP Wrapping

Stepwise Properties

Bending Moment, $M = 0.03974705$

Shear Force, $V_2 = 5828.436$

Shear Force, $V_3 = 1.0906320E-014$

Axial Force, $F = -7386.882$

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 $u_R = \gamma \cdot u = 0.04273873$

$u = \gamma \cdot u + p = 0.04273873$

- Calculation of γ -

$\gamma = (M_y \cdot L_s / 3) / E_{eff} = 0.00150221$ ((4.29), Biskinis Phd)

$M_y = 3.6258E+008$

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

From table 10.5, ASCE 41_17: $E_{eff} = factor \cdot E_c \cdot I_g = 2.4137E+013$

factor = 0.30

$A_g = 196349.541$

Mean concrete strength: $f_c' = (f_c'_{jacket} \cdot Area_{jacket} + f_c'_{core} \cdot Area_{core}) / Area_{section} = 28.32$

$N = 7386.882$

$$E_c I_g = E_c I_{g_jacket} + E_c I_{g_core} = 8.0455E+013$$

Calculation of Yielding Moment M_y

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

$$M_y = \min(M_{y_ten}, M_{y_com}) = 3.6258E+008$$

$$\rho_y ((10a) \text{ or } (10b)) = 1.0622219E-005$$

$$M_{y_ten} (8a) = 3.6258E+008$$

$$\rho_{y_ten} (7a) = 65.43626$$

$$\text{error of function } (7a) = 0.00293096$$

$$M_{y_com} (8b) = 7.5621E+008$$

$$\rho_{y_com} (7b) = 64.56804$$

$$\text{error of function } (7b) = -0.00721906$$

$$\text{with } e_y = 0.00277778$$

$$e_{co} = 0.002$$

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

$$d_1 = 44.00$$

$$R = 250.00$$

$$v = 0.00114003$$

$$N = 7386.882$$

$$A_c = 196349.541$$

$$= 0.26181818$$

$$\text{with } f_c = 33.00$$

Calculation of ratio l_b/d

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

- Calculation of ρ_p -

From table 10-9: $\rho_p = 0.04123652$

with:

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

$$\text{shear control ratio } V_y E / V_{col} E = 0.5636717$$

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

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

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

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

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

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

$$s_1 = 100.00$$

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

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

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

$$s_2 = 250.00$$

The term $2 * t_f / b_w * (f_{fe} / f_s)$ is implemented to account for FRP contribution where $f = 2 * t_f / b_w$ is FRP ratio (EC8 - 3, A.4.4.3(6)) and f_{fe} / f_s normalises f to steel strength. All these variables have already been given in Shear control ratio calculation. For the normalisation f_s of jacket is used.

$$N_{UD} = 7386.882$$

$$A_g = 196349.541$$

$$f_{cE} = (f_{c_jacket} * \text{Area}_{\text{jacket}} + f_{c_core} * \text{Area}_{\text{core}}) / \text{section_area} = 28.32$$

$$f_{yLE} = (f_{y_ext_Long_Reinf} * \text{Area}_{\text{ext_Long_Reinf}} + f_{y_int_Long_Reinf} * \text{Area}_{\text{int_Long_Reinf}}) / \text{Area}_{\text{Tot_Long_Rein}} = 21219958E-314$$

$$f_{yTE} = (f_{y_ext_Trans_Reinf} * \text{Area}_{\text{ext_Trans_Reinf}} + f_{y_int_Trans_Reinf} * \text{Area}_{\text{int_Trans_Reinf}}) / \text{Area}_{\text{Tot_Trans_Rein}} = 539.4201$$

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

$$f_{cE} = 28.32$$

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

At local axis: 3
Integration Section: (b)
