Detailed Member Calculations

Units: N&mm

Regulation: ASCE 41-17

Calculation No. 1

column C1, Floor 1

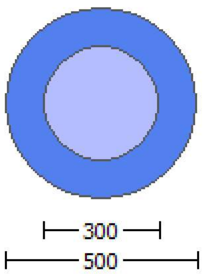
Limit State: Operational Level (data interpolation between analysis steps 1 and 2)

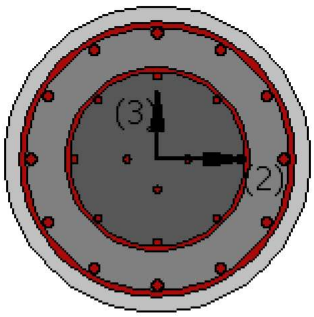
Analysis: Uniform +X

Check: Shear capacity 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, κ = 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 Secondary Member: Concrete Strength, fc = fc_lower_bound = 25.00
New material of Secondary Member: Steel Strength, fs = fs_lower_bound = 500.00

Concrete Elasticity, Ec = 26999.444
Steel Elasticity, Es = 200000.00
Existing Column
Existing material of Secondary Member: Concrete Strength, fc = fc_lower_bound = 16.00
Existing material of Secondary Member: Steel Strength, fs = fs_lower_bound = 400.00
Concrete Elasticity, Ec = 21019.039
Steel Elasticity, Es = 200000.00
##
Note: Especially for the calculation of θy 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, fc = fcm = 33.00
New material: Steel Strength, fs = fsm = 555.56
Existing Column
Existing material: Concrete Strength, fc = fcm = 20.00
Existing material: Steel Strength, fs = fsm = 444.44
##
External Diameter, D = 500.00
Internal Diameter, D = 300.00
Cover Thickness, c = 25.00
Element Length, L = 3000.00
Secondary Member
Ribbed Bars
Ductile Steel
Without Detailing for Earthquake Resistance (including stirrups not closed at 135°)
Longitudinal Bars With Ends Lapped Starting at the End Sections
Inadequate Lap Length with lo/lou,min =lb/ld = 0.30
No FRP Wrapping

Stepwise Properties

EDGE -A-
Bending Moment, Ma = -2.6175E+007
Shear Force, Va = -8722.76
EDGE -B-
Bending Moment, Mb = 0.00906183
Shear Force, Vb = 8722.76
BOTH EDGES
Axial Force, F = -7422.368
Longitudinal Reinforcement Area Distribution (in 2 divisions)
-Tension: Aslt = 1272.345
-Compression: Aslc = 1781.283
Longitudinal Reinforcement Area Distribution (in 3 divisions)
-Tension: Asl,ten = 1017.876
-Compression: Asl,com = 1017.876
-Middle: Asl,mid = 1017.876
Mean Diameter of Tension Reinforcement, DbL,ten = 18.00

Existing component: From table 7-7, ASCE 41_17: Final Shear Capacity VR = κ*Vn = 339071.742
Vn ((10.3), ASCE 41-17) = knl*VCol0 = 339071.742
VCol = 339071.742
knl = 1.00
displacement_ductility_demand = 0.03516802

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

λ = 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 = 4.00
Mu = 2.6175E+007

Vu = 8722.76
d = 0.8*D = 400.00
Nu = 7422.368
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 = π/2*A_stirrup = 123370.055
fy = 500.00
s = 100.00
Vs1 is multiplied by αCol1 = 1.00
s/d = 0.25
Vs2 = 0.00 is calculated for core, with:
Av = π/2*A_stirrup = 78956.835
fy = 400.00
s = 250.00
Vs2 is multiplied by αCol2 = 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 = π*d*d/4 = 125663.706

---

displacement_ductility_demand is calculated as θ/θy

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

From analysis, chord rotation θ = 0.00029873
θy = (My*Ls/3)/EIeff = 0.00849448 ((4.29),Biskinis Phd))
My = 2.0498E+008
Ls = M/V (with Ls >0.1*L and Ls < 2*L) = 3000.758
From table 10.5, ASCE 41_17: EIeff = 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 = 7422.368
Ec*Ig = Ec_jacket*Ig_jacket + Ec_core*Ig_core = 8.0455E+013

Calculation of Yielding Moment My

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

---

My = Min(My_ten,My_com) = 2.0498E+008
Φy = 5.8526392E-006
My_ten (8c) = 2.0498E+008
Φ_ten (7c) = 64.04195
error of function (7c) = 8.3474283E-005
My_com (8d) = 7.5621E+008
Φ_com (7d) = 64.56829
error of function (7d) = -0.0072183
with ((10.1), ASCE 41-17) ey = Min(ey, 1.25*ey*(lb/ld)^2/3) = 0.0027778
eco = 0.002
apl = 0.35 ((9a) in Biskinis and Fardis for no FRP Wrap)
d1 = 44.00
R = 250.00
v = 0.00114551
N = 7422.368
Ac = 196349.541
((10.1), ASCE 41-17) ω = Min(ω, 1.25*ω*(lb/ld)^2/3) = 0.26182028
with fc = 33.00

Calculation of ratio lb/ld

Inadequate Lap Length with lb/ld = 0.30

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

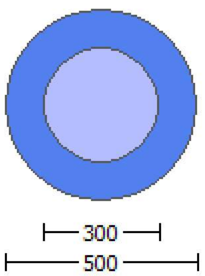
Limit State: Operational Level (data interpolation between analysis steps 1 and 2)

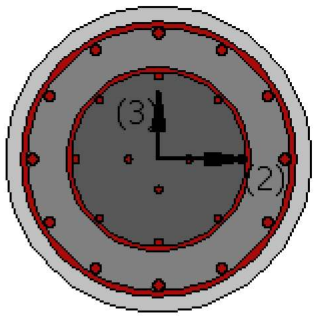
Analysis: Uniform +X

Check: Chord rotation capacity (θu)

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, κ = 1.00
Mean strength values are used for both shear and moment calculations.
Consequently:
Jacket
New material of Secondary Member: Concrete Strength, fc = fcm = 33.00
New material of Secondary Member: Steel Strength, fs = fsm = 555.56
Concrete Elasticity, Ec = 26999.444
Steel Elasticity, Es = 200000.00
Existing Column
Existing material of Secondary Member: Concrete Strength, fc = fcm = 20.00
Existing material of Secondary Member: Steel Strength, fs = fsm = 444.44
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 = 694.45
Existing Column
Existing material: Steel Strength, fs = 1.25*fsm = 555.55
###
External Diameter, D = 500.00
Internal Diameter, D = 300.00
Cover Thickness, c = 25.00
Mean Confinement Factor overall section = 1.00
Element Length, L = 3000.00
Secondary Member
Ribbed Bars
Ductile Steel
Without Detailing for Earthquake Resistance (including stirrups not closed at 135°)
Longitudinal Bars With Ends Lapped Starting at the End Sections
Inadequate Lap Length with lo/lou,min = 0.30
No FRP Wrapping

Stepwise Properties

At local axis: 3
EDGE -A-
Shear Force, Va = -7.1742060E-031
EDGE -B-
Shear Force, Vb = 7.1742060E-031
BOTH EDGES
Axial Force, F = -7425.858
Longitudinal Reinforcement Area Distribution (in 2 divisions)
-Tension: Aslt = 0.00
-Compression: Aslc = 3053.628
Longitudinal Reinforcement Area Distribution (in 3 divisions)
-Tension: Asl,ten = 1017.876
-Compression: Asl,com = 1017.876
-Middle: Asl,mid = 1017.876

Calculation of Shear Capacity ratio , Ve/Vr = 0.32864977
Member Controlled by Flexure (Ve/Vr < 1)
Calculation of Shear Demand from fig. R18.6.5, ACI 318-14 Ve = (Mpr1 + Mpr2)/ln = 159269.81
with
Mpr1 = Max(Mu1+ , Mu1-) = 2.3890E+008
Mu1+ = 2.3890E+008, is the ultimate moment strength at the edge 1 of the member in the actual moment direction which is defined for the static loading combination
Mu1- = 2.3890E+008, is the ultimate moment strength at the edge 1 of the member in the opposite moment direction which is defined for the static loading combination
Mpr2 = Max(Mu2+ , Mu2-) = 2.3890E+008
Mu2+ = 2.3890E+008, is the ultimate moment strength at the edge 2 of the member in the actual moment direction which is defined for the the static loading combination
Mu2- = 2.3890E+008, is the ultimate moment strength at the edge 2 of the member in the opposite moment direction which is defined for the the static loading combination

----
Calculation of Mu1+

Calculation of ultimate Moment Strength ((3.67), Biskinis Phd), Mu
Mu = 2.3890E+008

φ = 0.90757121
φ' = 0.80580716
error of function (3.68), Biskinis Phd = 59442.345

From 5A.2, TBDY: fcc = fc*λc = 33.00
conf. factor λc = 1.00
fc = 33.00
From 10.3.5, ASCE 41-17, Final value of fy: fy*Min(1,1.25*(lb/ld)^2/3) = 389.0139
lb/ld = 0.30
d1 = 44.00
R = 250.00
v = 0.0011456
N = 7425.858
Ac = 196349.541
ω = ω*Min(1,1.25*(lb/ld)^2/3) = 0.14666533

Calculation of ratio lb/ld

Inadequate Lap Length with lb/ld = 0.30

Calculation of Mu1-

Calculation of ultimate Moment Strength ((3.67), Biskinis Phd), Mu
Mu = 2.3890E+008

φ = 0.90757121
φ' = 0.80580716
error of function (3.68), Biskinis Phd = 59442.345
From 5A.2, TBDY: fcc = fc*λc = 33.00
conf. factor λc = 1.00
fc = 33.00
From 10.3.5, ASCE 41-17, Final value of fy: fy*Min(1,1.25*(lb/ld)^2/3) = 389.0139
lb/ld = 0.30
d1 = 44.00
R = 250.00
v = 0.0011456
N = 7425.858
Ac = 196349.541
ω = ω*Min(1,1.25*(lb/ld)^2/3) = 0.14666533

Calculation of ratio lb/ld

Inadequate Lap Length with lb/ld = 0.30

Calculation of Mu2+

Calculation of ultimate Moment Strength ((3.67), Biskinis Phd), Mu
Mu = 2.3890E+008

φ = 0.90757121
φ' = 0.80580716
error of function (3.68), Biskinis Phd = 59442.345
From 5A.2, TBDY: fcc = fc*λc = 33.00
conf. factor λc = 1.00
fc = 33.00
From 10.3.5, ASCE 41-17, Final value of fy: fy*Min(1,1.25*(lb/ld)^2/3) = 389.0139
lb/ld = 0.30
d1 = 44.00
R = 250.00
v = 0.0011456
N = 7425.858

Ac = 196349.541
ω = ω*Min(1,1.25*(lb/ld)^2/3) = 0.14666533

Calculation of ratio lb/ld

Inadequate Lap Length with lb/ld = 0.30

Calculation of Mu2-

Calculation of ultimate Moment Strength ((3.67), Biskinis Phd), Mu
Mu = 2.3890E+008

φ = 0.90757121
φ' = 0.80580716
error of function (3.68), Biskinis Phd = 59442.345
From 5A.2, TBDY: fcc = fc*λc = 33.00
conf. factor λc = 1.00
fc = 33.00
From 10.3.5, ASCE 41-17, Final value of fy: fy*Min(1,1.25*(lb/ld)^2/3) = 389.0139
lb/ld = 0.30
d1 = 44.00
R = 250.00
v = 0.0011456
N = 7425.858
Ac = 196349.541
ω = ω*Min(1,1.25*(lb/ld)^2/3) = 0.14666533

Calculation of ratio lb/ld

Inadequate Lap Length with lb/ld = 0.30

Calculation of Shear Strength Vr = Min(Vr1,Vr2) = 484618.662

Calculation of Shear Strength at edge 1, Vr1 = 484618.662
Vr1 = VCol ((10.3), ASCE 41-17) = knl*VCol0
VCol0 = 484618.662
knl = 1 (zero step-static loading)

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

λ = 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 = 1.3305601E-011
Vu = 7.1742060E-031
d = 0.8*D = 400.00
Nu = 7425.858
Ag = 196349.541
From (11.5.4.8), ACI 318-14: Vs = Vs1 + Vs2 = 274157.871
Vs1 = 274157.871 is calculated for jacket, with:
Av = π/2*A_stirrup = 123370.055
fy = 555.56
s = 100.00
Vs1 is multiplied by αCol1 = 1.00
s/d = 0.25
Vs2 = 0.00 is calculated for core, with:

Av = π/2*A_stirrup = 78956.835
fy = 444.44
s = 250.00
Vs2 is multiplied by αCol2 = 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 = π*d*d/4 = 125663.706

Calculation of Shear Strength at edge 2, Vr2 = 484618.662
Vr2 = VCol ((10.3), ASCE 41-17) = knl*VCol0
VCol0 = 484618.662
knl = 1 (zero step-static loading)

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

λ = 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 = 1.3305601E-011
Vu = 7.1742060E-031
d = 0.8*D = 400.00
Nu = 7425.858
Ag = 196349.541
From (11.5.4.8), ACI 318-14: Vs = Vs1 + Vs2 = 274157.871
Vs1 = 274157.871 is calculated for jacket, with:
Av = π/2*A_stirrup = 123370.055
fy = 555.56
s = 100.00
Vs1 is multiplied by αCol1 = 1.00
s/d = 0.25
Vs2 = 0.00 is calculated for core, with:
Av = π/2*A_stirrup = 78956.835
fy = 444.44
s = 250.00
Vs2 is multiplied by αCol2 = 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 = π*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, κ = 1.00
Mean strength values are used for both shear and moment calculations.
Consequently:
Jacket
New material of Secondary Member: Concrete Strength, fc = fcm = 33.00
New material of Secondary Member: Steel Strength, fs = fsm = 555.56
Concrete Elasticity, Ec = 26999.444
Steel Elasticity, Es = 200000.00
Existing Column

Existing material of Secondary Member: Concrete Strength, fc = fcm = 20.00
Existing material of Secondary Member: Steel Strength, fs = fsm = 444.44
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 = 694.45
Existing Column
Existing material: Steel Strength, fs = 1.25*fsm = 555.55
##
External Diameter, D = 500.00
Internal Diameter, D = 300.00
Cover Thickness, c = 25.00
Mean Confinement Factor overall section = 1.00
Element Length, L = 3000.00
Secondary Member
Ribbed Bars
Ductile Steel
Without Detailing for Earthquake Resistance (including stirrups not closed at 135°)
Longitudinal Bars With Ends Lapped Starting at the End Sections
Inadequate Lap Length with lo/lou,min = 0.30
No FRP Wrapping

Stepwise Properties

At local axis: 2
EDGE -A-
Shear Force, Va = -3.8672673E-031
EDGE -B-
Shear Force, Vb = 3.8672673E-031
BOTH EDGES
Axial Force, F = -7425.858
Longitudinal Reinforcement Area Distribution (in 2 divisions)
-Tension: Aslt = 0.00
-Compression: Aslc = 3053.628
Longitudinal Reinforcement Area Distribution (in 3 divisions)
-Tension: Asl,ten = 1017.876
-Compression: Asl,com = 1017.876
-Middle: Asl,mid = 1017.876

Calculation of Shear Capacity ratio , Ve/Vr = 0.32864977
Member Controlled by Flexure (Ve/Vr < 1)
Calculation of Shear Demand from fig. R18.6.5, ACI 318-14 Ve = (Mpr1 + Mpr2)/ln = 159269.81
with
Mpr1 = Max(Mu1+ , Mu1-) = 2.3890E+008
Mu1+ = 2.3890E+008, is the ultimate moment strength at the edge 1 of the member in the actual moment direction which is defined for the static loading combination
Mu1- = 2.3890E+008, is the ultimate moment strength at the edge 1 of the member in the opposite moment direction which is defined for the static loading combination
Mpr2 = Max(Mu2+ , Mu2-) = 2.3890E+008
Mu2+ = 2.3890E+008, is the ultimate moment strength at the edge 2 of the member in the actual moment direction which is defined for the the static loading combination
Mu2- = 2.3890E+008, is the ultimate moment strength at the edge 2 of the member in the opposite moment direction which is defined for the the static loading combination

----
Calculation of Mu1+

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

Mu = 2.3890E+008

φ = 0.90757121
φ' = 0.80580716
error of function (3.68), Biskinis Phd = 59442.345
From 5A.2, TBDY: fcc = fc*λc = 33.00
conf. factor λc = 1.00
fc = 33.00
From 10.3.5, ASCE 41-17, Final value of fy: fy*Min(1,1.25*(lb/ld)^2/3) = 389.0139
lb/ld = 0.30
d1 = 44.00
R = 250.00
v = 0.0011456
N = 7425.858
Ac = 196349.541
ω = ω*Min(1,1.25*(lb/ld)^2/3) = 0.14666533

Calculation of ratio lb/ld

Inadequate Lap Length with lb/ld = 0.30

Calculation of Mu1-

Calculation of ultimate Moment Strength ((3.67), Biskinis Phd), Mu
Mu = 2.3890E+008

φ = 0.90757121
φ' = 0.80580716
error of function (3.68), Biskinis Phd = 59442.345
From 5A.2, TBDY: fcc = fc*λc = 33.00
conf. factor λc = 1.00
fc = 33.00
From 10.3.5, ASCE 41-17, Final value of fy: fy*Min(1,1.25*(lb/ld)^2/3) = 389.0139
lb/ld = 0.30
d1 = 44.00
R = 250.00
v = 0.0011456
N = 7425.858
Ac = 196349.541
ω = ω*Min(1,1.25*(lb/ld)^2/3) = 0.14666533

Calculation of ratio lb/ld

Inadequate Lap Length with lb/ld = 0.30

Calculation of Mu2+

Calculation of ultimate Moment Strength ((3.67), Biskinis Phd), Mu
Mu = 2.3890E+008

φ = 0.90757121
φ' = 0.80580716
error of function (3.68), Biskinis Phd = 59442.345
From 5A.2, TBDY: fcc = fc*λc = 33.00
conf. factor λc = 1.00
fc = 33.00
From 10.3.5, ASCE 41-17, Final value of fy: fy*Min(1,1.25*(lb/ld)^2/3) = 389.0139

lb/ld = 0.30
d1 = 44.00
R = 250.00
v = 0.0011456
N = 7425.858
Ac = 196349.541
ω = ω*Min(1,1.25*(lb/ld)^2/3) = 0.14666533

Calculation of ratio lb/ld

Inadequate Lap Length with lb/ld = 0.30

Calculation of Mu2-

Calculation of ultimate Moment Strength ((3.67), Biskinis Phd), Mu
Mu = 2.3890E+008

φ = 0.90757121
φ' = 0.80580716
error of function (3.68), Biskinis Phd = 59442.345
From 5A.2, TBDY: fcc = fc*λc = 33.00
conf. factor λc = 1.00
fc = 33.00
From 10.3.5, ASCE 41-17, Final value of fy: fy*Min(1,1.25*(lb/ld)^2/3) = 389.0139
lb/ld = 0.30
d1 = 44.00
R = 250.00
v = 0.0011456
N = 7425.858
Ac = 196349.541
ω = ω*Min(1,1.25*(lb/ld)^2/3) = 0.14666533

Calculation of ratio lb/ld

Inadequate Lap Length with lb/ld = 0.30

Calculation of Shear Strength Vr = Min(Vr1,Vr2) = 484618.662

Calculation of Shear Strength at edge 1, Vr1 = 484618.662
Vr1 = VCol ((10.3), ASCE 41-17) = knl*VCol0
VCol0 = 484618.662
knl = 1 (zero step-static loading)

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

λ = 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 = 2.2610821E-011
Vu = 3.8672673E-031
d = 0.8*D = 400.00
Nu = 7425.858
Ag = 196349.541
From (11.5.4.8), ACI 318-14: Vs = Vs1 + Vs2 = 274157.871
Vs1 = 274157.871 is calculated for jacket, with:
Av = π/2*A_stirrup = 123370.055

fy = 555.56
s = 100.00
Vs1 is multiplied by αCol1 = 1.00
s/d = 0.25
Vs2 = 0.00 is calculated for core, with:
Av = π/2*A_stirrup = 78956.835
fy = 444.44
s = 250.00
Vs2 is multiplied by αCol2 = 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 = π*d*d/4 = 125663.706

Calculation of Shear Strength at edge 2, Vr2 = 484618.662
Vr2 = VCol ((10.3), ASCE 41-17) = knl*VCol0
VCol0 = 484618.662
knl = 1 (zero step-static loading)

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

λ = 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 = 2.2610821E-011
Vu = 3.8672673E-031
d = 0.8*D = 400.00
Nu = 7425.858
Ag = 196349.541
From (11.5.4.8), ACI 318-14: Vs = Vs1 + Vs2 = 274157.871
Vs1 = 274157.871 is calculated for jacket, with:
Av = π/2*A_stirrup = 123370.055
fy = 555.56
s = 100.00
Vs1 is multiplied by αCol1 = 1.00
s/d = 0.25
Vs2 = 0.00 is calculated for core, with:
Av = π/2*A_stirrup = 78956.835
fy = 444.44
s = 250.00
Vs2 is multiplied by αCol2 = 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 = π*d*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, κ = 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 Secondary Member: Concrete Strength, fc = fcm = 33.00
New material of Secondary Member: Steel Strength, fs = fsm = 555.56
Concrete Elasticity, Ec = 26999.444
Steel Elasticity, Es = 200000.00
Existing Column
Existing material of Secondary Member: Concrete Strength, fc = fcm = 20.00
Existing material of Secondary Member: Steel Strength, fs = fsm = 444.44
Concrete Elasticity, Ec = 21019.039
Steel Elasticity, Es = 200000.00
External Diameter, D = 500.00
Internal Diameter, D = 300.00
Cover Thickness, c = 25.00
Element Length, L = 3000.00
Secondary Member
Ribbed Bars
Ductile Steel
Without Detailing for Earthquake Resistance (including stirrups not closed at 135°)
Longitudinal Bars With Ends Lapped Starting at the End Sections
Inadequate Lap Length with lb/ld = 0.30
No FRP Wrapping

Stepwise Properties

Bending Moment, M = 2.8960610E-010
Shear Force, V2 = -8722.76
Shear Force, V3 = -1.4777839E-013
Axial Force, F = -7422.368
Longitudinal Reinforcement Area Distribution (in 2 divisions)
-Tension: Aslt = 1272.345
-Compression: Aslc = 1781.283
Longitudinal Reinforcement Area Distribution (in 3 divisions)
-Tension: Asl,ten = 1017.876
-Compression: Asl,com = 1017.876
-Middle: Asl,mid = 1017.876
Mean Diameter of Tension Reinforcement, DbL = 18.00

Existing component: From table 7-7, ASCE 41_17: Final chord rotation Capacity θu,R = κ*θu = 0.00424617
θu = θy + θp = 0.00424617

- Calculation of θy -

θy = (My*Ls/3)/EIeff = 0.00424617 ((4.29),Biskinis Phd))
My = 2.0498E+008
Ls = M/V (with Ls >0.1*L and Ls < 2*L) = 1500.00
From table 10.5, ASCE 41_17: EIeff = 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 = 7422.368
Ec*Ig = Ec_jacket*Ig_jacket + Ec_core*Ig_core = 8.0455E+013

Calculation of Yielding Moment My

Calculation of Φy and My according to (7) - (8) in Biskinis and Fardis
----
My = Min(My_ten,My_com) = 2.0498E+008
Φy = 5.8526392E-006
My_ten (8c) = 2.0498E+008
Φ_ten (7c) = 64.04195

error of function (7c) = 8.3474283E-005
My_com (8d) = 7.5621E+008
Φ_com (7d) = 64.56829
error of function (7d) = -0.0072183
with ((10.1), ASCE 41-17) ey = Min(ey, 1.25*ey*(lb/ld)^2/3) = 0.0027778
eco = 0.002
apl = 0.35 ((9a) in Biskinis and Fardis for no FRP Wrap)
d1 = 44.00
R = 250.00
v = 0.00114551
N = 7422.368
Ac = 196349.541
((10.1), ASCE 41-17) ω = Min(ω, 1.25*ω*(lb/ld)^2/3) = 0.26182028
with fc = 33.00

Calculation of ratio lb/ld

Inadequate Lap Length with lb/ld = 0.30

- Calculation of θp -

From table 10-9: θp = 0.00
with:
- Columns controlled by inadequate development or splicing along the clear height because lb/ld < 1
shear control ratio VyE/VCol0E = 0.32864977
d = d_external = 0.00
s = s_external = 0.00
ρt = ρs1 + ρs2 + 2*tf/bw*(ffe/fs) = 0.00323428
jacket: ρs1 = Av1*(π*Dc1/2)/(s1*Ag) = 0.0027646
Av1 = 78.53982, is the area of stirrup
Dc1 = Dext - 2*cover - External Hoop Diameter = 440.00, is the total Length of all stirrups parallel to loading (shear) direction
s1 = 100.00
core: ρs2 = Av2*(π*Dc2/2)/(s2*Ag) = 0.00046968
Av2 = 50.26548, is the area of stirrup
Dc2 = Dint - Internal Hoop Diameter = 292.00, is the total Length of all stirrups parallel to loading (shear) direction
s2 = 250.00
The term 2*tf/bw*(ffe/fs) is implemented to account for FRP contribution
where ρf = 2*tf/bw is FRP ratio (EC8 - 3, A.4.4.3(6)) and ffe/fs normalises ρf to steel strength
All these variables have already been given in Shear control ratio calculation.
For the normalisation fs of jacket is used.
NUD = 7422.368
Ag = 196349.541
f'cE = (fc_jacket*Area_jacket+fc_core*Area_core)/section_area = 28.32
fylE = (fy_ext_Long_Reinf*Area_ext_Long_Reinf+fy_int_Long_Reinf*Area_int_Long_Reinf)/Area_Tot_Long_Rein = 2.1219958E-314
fytE = (fy_ext_Trans_Reinf*ρs1+fy_int_Trans_Reinf*ρs2)/(ρs1+ρs2) = 539.4232
pl = 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: (a)

Calculation No. 3

column C1, Floor 1

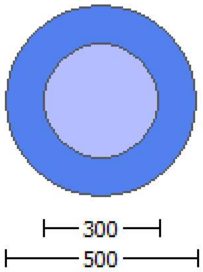
Limit State: Operational Level (data interpolation between analysis steps 1 and 2)

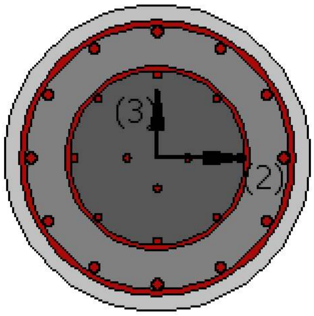
Analysis: Uniform +X

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, κ = 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 Secondary Member: Concrete Strength, fc = fc_lower_bound = 25.00
New material of Secondary Member: Steel Strength, fs = fs_lower_bound = 500.00
Concrete Elasticity, Ec = 26999.444
Steel Elasticity, Es = 200000.00
Existing Column
Existing material of Secondary Member: Concrete Strength, fc = fc_lower_bound = 16.00
Existing material of Secondary Member: Steel Strength, fs = fs_lower_bound = 400.00
Concrete Elasticity, Ec = 21019.039
Steel Elasticity, Es = 200000.00
##
Note: Especially for the calculation of θy 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, fc = fcm = 33.00
New material: Steel Strength, fs = fsm = 555.56
Existing Column
Existing material: Concrete Strength, fc = fcm = 20.00
Existing material: Steel Strength, fs = fsm = 444.44
##

External Diameter, D = 500.00
Internal Diameter, D = 300.00
Cover Thickness, c = 25.00
Element Length, L = 3000.00
Secondary Member
Ribbed Bars
Ductile Steel
Without Detailing for Earthquake Resistance (including stirrups not closed at 135°)
Longitudinal Bars With Ends Lapped Starting at the End Sections
Inadequate Lap Length with lo/lou,min =lb/ld = 0.30
No FRP Wrapping

Stepwise Properties

EDGE -A-
Bending Moment, Ma = 2.8960610E-010
Shear Force, Va = -1.4777839E-013
EDGE -B-
Bending Moment, Mb = 1.5368128E-010
Shear Force, Vb = 1.4777839E-013
BOTH EDGES
Axial Force, F = -7422.368
Longitudinal Reinforcement Area Distribution (in 2 divisions)
-Tension: Aslt = 1272.345
-Compression: Aslc = 1781.283
Longitudinal Reinforcement Area Distribution (in 3 divisions)
-Tension: Asl,ten = 1017.876
-Compression: Asl,com = 1017.876
-Middle: Asl,mid = 1017.876
Mean Diameter of Tension Reinforcement, DbL,ten = 18.00

Existing component: From table 7-7, ASCE 41_17: Final Shear Capacity VR = κ*Vn = 431403.373
Vn ((10.3), ASCE 41-17) = knl*VCol0 = 431403.373
VCol = 431403.373
knl = 1.00
displacement_ductility_demand = 0.00

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

λ = 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 = 2.8960610E-010
Vu = 1.4777839E-013
d = 0.8*D = 400.00
Nu = 7422.368
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 = π/2*A_stirrup = 123370.055
fy = 500.00
s = 100.00
Vs1 is multiplied by αCol1 = 1.00
s/d = 0.25
Vs2 = 0.00 is calculated for core, with:
Av = π/2*A_stirrup = 78956.835
fy = 400.00
s = 250.00
Vs2 is multiplied by αCol2 = 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 = π*d*d/4 = 125663.706

----
displacement_ductility_demand is calculated as θ/θy

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

From analysis, chord rotation θ = 2.1158200E-021
θy = (My*Ls/3)/EIeff = 0.00424617 ((4.29),Biskinis Phd))
My = 2.0498E+008
Ls = M/V (with Ls >0.1*L and Ls < 2*L) = 1500.00
From table 10.5, ASCE 41_17: EIeff = 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 = 7422.368
Ec*Ig = Ec_jacket*Ig_jacket + Ec_core*Ig_core = 8.0455E+013

Calculation of Yielding Moment My

Calculation of Φy and My according to (7) - (8) in Biskinis and Fardis
----
My = Min(My_ten,My_com) = 2.0498E+008
Φy = 5.8526392E-006
My_ten (8c) = 2.0498E+008
Φ_ten (7c) = 64.04195
error of function (7c) = 8.3474283E-005
My_com (8d) = 7.5621E+008
Φ_com (7d) = 64.56829
error of function (7d) = -0.0072183
with ((10.1), ASCE 41-17) ey = Min(ey, 1.25*ey*(lb/ld)^2/3) = 0.0027778
eco = 0.002
apl = 0.35 ((9a) in Biskinis and Fardis for no FRP Wrap)
d1 = 44.00
R = 250.00
v = 0.00114551
N = 7422.368
Ac = 196349.541
((10.1), ASCE 41-17) ω = Min(ω, 1.25*ω*(lb/ld)^2/3) = 0.26182028
with fc = 33.00

Calculation of ratio lb/ld

Inadequate Lap Length with lb/ld = 0.30

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

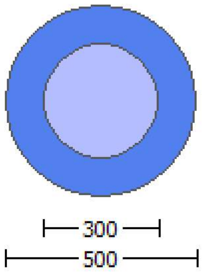
Limit State: Operational Level (data interpolation between analysis steps 1 and 2)

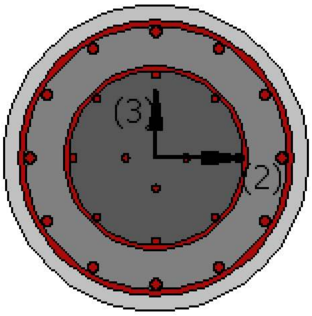
Analysis: Uniform +X

Check: Chord rotation capacity (θu)

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, κ = 1.00
Mean strength values are used for both shear and moment calculations.
Consequently:
Jacket
New material of Secondary Member: Concrete Strength, fc = fcm = 33.00
New material of Secondary Member: Steel Strength, fs = fsm = 555.56
Concrete Elasticity, Ec = 26999.444
Steel Elasticity, Es = 200000.00
Existing Column
Existing material of Secondary Member: Concrete Strength, fc = fcm = 20.00
Existing material of Secondary Member: Steel Strength, fs = fsm = 444.44
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 = 694.45
Existing Column
Existing material: Steel Strength, fs = 1.25*fsm = 555.55
##
External Diameter, D = 500.00
Internal Diameter, D = 300.00
Cover Thickness, c = 25.00
Mean Confinement Factor overall section = 1.00
Element Length, L = 3000.00
Secondary Member
Ribbed Bars
Ductile Steel
Without Detailing for Earthquake Resistance (including stirrups not closed at 135°)
Longitudinal Bars With Ends Lapped Starting at the End Sections
Inadequate Lap Length with lo/lou,min = 0.30

No FRP Wrapping

Stepwise Properties

At local axis: 3
EDGE -A-
Shear Force, Va = -7.1742060E-031
EDGE -B-
Shear Force, Vb = 7.1742060E-031
BOTH EDGES
Axial Force, F = -7425.858
Longitudinal Reinforcement Area Distribution (in 2 divisions)
-Tension: Aslt = 0.00
-Compression: Aslc = 3053.628
Longitudinal Reinforcement Area Distribution (in 3 divisions)
-Tension: Asl,ten = 1017.876
-Compression: Asl,com = 1017.876
-Middle: Asl,mid = 1017.876

Calculation of Shear Capacity ratio , Ve/Vr = 0.32864977
Member Controlled by Flexure (Ve/Vr < 1)
Calculation of Shear Demand from fig. R18.6.5, ACI 318-14 Ve = (Mpr1 + Mpr2)/ln = 159269.81
with
Mpr1 = Max(Mu1+ , Mu1-) = 2.3890E+008
Mu1+ = 2.3890E+008, is the ultimate moment strength at the edge 1 of the member in the actual moment direction which is defined for the static loading combination
Mu1- = 2.3890E+008, is the ultimate moment strength at the edge 1 of the member in the opposite moment direction which is defined for the static loading combination
Mpr2 = Max(Mu2+ , Mu2-) = 2.3890E+008
Mu2+ = 2.3890E+008, is the ultimate moment strength at the edge 2 of the member in the actual moment direction which is defined for the the static loading combination
Mu2- = 2.3890E+008, is the ultimate moment strength at the edge 2 of the member in the opposite moment direction which is defined for the the static loading combination

----
Calculation of Mu1+

Calculation of ultimate Moment Strength ((3.67), Biskinis Phd), Mu
Mu = 2.3890E+008

φ = 0.90757121
φ' = 0.80580716
error of function (3.68), Biskinis Phd = 59442.345
From 5A.2, TBDY: fcc = fc*λc = 33.00
conf. factor λc = 1.00
fc = 33.00
From 10.3.5, ASCE 41-17, Final value of fy: fy*Min(1,1.25*(lb/ld)^2/3) = 389.0139
lb/ld = 0.30
d1 = 44.00
R = 250.00
v = 0.0011456
N = 7425.858
Ac = 196349.541
ω = ω*Min(1,1.25*(lb/ld)^2/3) = 0.14666533

Calculation of ratio lb/ld

Inadequate Lap Length with lb/ld = 0.30

Calculation of Mu1-

Calculation of ultimate Moment Strength ((3.67), Biskinis Phd), Mu
Mu = 2.3890E+008

φ = 0.90757121
φ' = 0.80580716
error of function (3.68), Biskinis Phd = 59442.345
From 5A.2, TBDY: fcc = fc*λc = 33.00
conf. factor λc = 1.00
fc = 33.00
From 10.3.5, ASCE 41-17, Final value of fy: fy*Min(1,1.25*(lb/ld)^2/3) = 389.0139
lb/ld = 0.30
d1 = 44.00
R = 250.00
v = 0.0011456
N = 7425.858
Ac = 196349.541
ω = ω*Min(1,1.25*(lb/ld)^2/3) = 0.14666533

Calculation of ratio lb/ld

Inadequate Lap Length with lb/ld = 0.30

Calculation of Mu2+

Calculation of ultimate Moment Strength ((3.67), Biskinis Phd), Mu
Mu = 2.3890E+008

φ = 0.90757121
φ' = 0.80580716
error of function (3.68), Biskinis Phd = 59442.345
From 5A.2, TBDY: fcc = fc*λc = 33.00
conf. factor λc = 1.00
fc = 33.00
From 10.3.5, ASCE 41-17, Final value of fy: fy*Min(1,1.25*(lb/ld)^2/3) = 389.0139
lb/ld = 0.30
d1 = 44.00
R = 250.00
v = 0.0011456
N = 7425.858
Ac = 196349.541
ω = ω*Min(1,1.25*(lb/ld)^2/3) = 0.14666533

Calculation of ratio lb/ld

Inadequate Lap Length with lb/ld = 0.30

Calculation of Mu2-

Calculation of ultimate Moment Strength ((3.67), Biskinis Phd), Mu
Mu = 2.3890E+008

φ = 0.90757121

φ' = 0.80580716
error of function (3.68), Biskinis Phd = 59442.345
From 5A.2, TBDY: fcc = fc*λc = 33.00
conf. factor λc = 1.00
fc = 33.00
From 10.3.5, ASCE 41-17, Final value of fy: fy*Min(1,1.25*(lb/ld)^2/3) = 389.0139
lb/ld = 0.30
d1 = 44.00
R = 250.00
v = 0.0011456
N = 7425.858
Ac = 196349.541
ω = ω*Min(1,1.25*(lb/ld)^2/3) = 0.14666533

Calculation of ratio lb/ld

Inadequate Lap Length with lb/ld = 0.30

Calculation of Shear Strength Vr = Min(Vr1,Vr2) = 484618.662

Calculation of Shear Strength at edge 1, Vr1 = 484618.662
Vr1 = VCol ((10.3), ASCE 41-17) = knl*VCol0
VCol0 = 484618.662
knl = 1 (zero step-static loading)

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

λ = 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 = 1.3305601E-011
Vu = 7.1742060E-031
d = 0.8*D = 400.00
Nu = 7425.858
Ag = 196349.541
From (11.5.4.8), ACI 318-14: Vs = Vs1 + Vs2 = 274157.871
Vs1 = 274157.871 is calculated for jacket, with:
Av = π/2*A_stirrup = 123370.055
fy = 555.56
s = 100.00
Vs1 is multiplied by αCol1 = 1.00
s/d = 0.25
Vs2 = 0.00 is calculated for core, with:
Av = π/2*A_stirrup = 78956.835
fy = 444.44
s = 250.00
Vs2 is multiplied by αCol2 = 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 = π*d*d/4 = 125663.706

Calculation of Shear Strength at edge 2, Vr2 = 484618.662
Vr2 = VCol ((10.3), ASCE 41-17) = knl*VCol0
VCol0 = 484618.662
knl = 1 (zero step-static loading)

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

λ = 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 = 1.3305601E-011
Vu = 7.1742060E-031
d = 0.8*D = 400.00
Nu = 7425.858
Ag = 196349.541
From (11.5.4.8), ACI 318-14: Vs = Vs1 + Vs2 = 274157.871
Vs1 = 274157.871 is calculated for jacket, with:
Av = π/2*A_stirrup = 123370.055
fy = 555.56
s = 100.00
Vs1 is multiplied by αCol1 = 1.00
s/d = 0.25
Vs2 = 0.00 is calculated for core, with:
Av = π/2*A_stirrup = 78956.835
fy = 444.44
s = 250.00
Vs2 is multiplied by αCol2 = 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 = π*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, κ = 1.00
Mean strength values are used for both shear and moment calculations.
Consequently:
Jacket
New material of Secondary Member: Concrete Strength, fc = fcm = 33.00
New material of Secondary Member: Steel Strength, fs = fsm = 555.56
Concrete Elasticity, Ec = 26999.444
Steel Elasticity, Es = 200000.00
Existing Column
Existing material of Secondary Member: Concrete Strength, fc = fcm = 20.00
Existing material of Secondary Member: Steel Strength, fs = fsm = 444.44
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 = 694.45
Existing Column
Existing material: Steel Strength, fs = 1.25*fsm = 555.55
##
External Diameter, D = 500.00
Internal Diameter, D = 300.00
Cover Thickness, c = 25.00
Mean Confinement Factor overall section = 1.00
Element Length, L = 3000.00
Secondary Member

Ribbed Bars
Ductile Steel
Without Detailing for Earthquake Resistance (including stirrups not closed at 135°)
Longitudinal Bars With Ends Lapped Starting at the End Sections
Inadequate Lap Length with lo/lou,min = 0.30
No FRP Wrapping

Stepwise Properties

At local axis: 2
EDGE -A-
Shear Force, Va = -3.8672673E-031
EDGE -B-
Shear Force, Vb = 3.8672673E-031
BOTH EDGES
Axial Force, F = -7425.858
Longitudinal Reinforcement Area Distribution (in 2 divisions)
-Tension: Aslt = 0.00
-Compression: Aslc = 3053.628
Longitudinal Reinforcement Area Distribution (in 3 divisions)
-Tension: Asl,ten = 1017.876
-Compression: Asl,com = 1017.876
-Middle: Asl,mid = 1017.876

Calculation of Shear Capacity ratio , Ve/Vr = 0.32864977
Member Controlled by Flexure (Ve/Vr < 1)
Calculation of Shear Demand from fig. R18.6.5, ACI 318-14 Ve = (Mpr1 + Mpr2)/ln = 159269.81
with
Mpr1 = Max(Mu1+ , Mu1-) = 2.3890E+008
Mu1+ = 2.3890E+008, is the ultimate moment strength at the edge 1 of the member in the actual moment direction which is defined for the static loading combination
Mu1- = 2.3890E+008, is the ultimate moment strength at the edge 1 of the member in the opposite moment direction which is defined for the static loading combination
Mpr2 = Max(Mu2+ , Mu2-) = 2.3890E+008
Mu2+ = 2.3890E+008, is the ultimate moment strength at the edge 2 of the member in the actual moment direction which is defined for the the static loading combination
Mu2- = 2.3890E+008, is the ultimate moment strength at the edge 2 of the member in the opposite moment direction which is defined for the the static loading combination

----
Calculation of Mu1+

Calculation of ultimate Moment Strength ((3.67), Biskinis Phd), Mu
Mu = 2.3890E+008

φ = 0.90757121
φ' = 0.80580716
error of function (3.68), Biskinis Phd = 59442.345
From 5A.2, TBDY: fcc = fc*λc = 33.00
conf. factor λc = 1.00
fc = 33.00
From 10.3.5, ASCE 41-17, Final value of fy: fy*Min(1,1.25*(lb/ld)^2/3) = 389.0139
lb/ld = 0.30
d1 = 44.00
R = 250.00
v = 0.0011456
N = 7425.858
Ac = 196349.541
ω = ω*Min(1,1.25*(lb/ld)^2/3) = 0.14666533

Calculation of ratio lb/ld

Inadequate Lap Length with lb/ld = 0.30

Calculation of Mu1-

Calculation of ultimate Moment Strength ((3.67), Biskinis Phd), Mu
Mu = 2.3890E+008

φ = 0.90757121
φ' = 0.80580716
error of function (3.68), Biskinis Phd = 59442.345
From 5A.2, TBDY: fcc = fc*λc = 33.00
conf. factor λc = 1.00
fc = 33.00
From 10.3.5, ASCE 41-17, Final value of fy: fy*Min(1,1.25*(lb/ld)^2/3) = 389.0139
lb/ld = 0.30
d1 = 44.00
R = 250.00
v = 0.0011456
N = 7425.858
Ac = 196349.541
ω = ω*Min(1,1.25*(lb/ld)^2/3) = 0.14666533

Calculation of ratio lb/ld

Inadequate Lap Length with lb/ld = 0.30

Calculation of Mu2+

Calculation of ultimate Moment Strength ((3.67), Biskinis Phd), Mu
Mu = 2.3890E+008

φ = 0.90757121
φ' = 0.80580716
error of function (3.68), Biskinis Phd = 59442.345
From 5A.2, TBDY: fcc = fc*λc = 33.00
conf. factor λc = 1.00
fc = 33.00
From 10.3.5, ASCE 41-17, Final value of fy: fy*Min(1,1.25*(lb/ld)^2/3) = 389.0139
lb/ld = 0.30
d1 = 44.00
R = 250.00
v = 0.0011456
N = 7425.858
Ac = 196349.541
ω = ω*Min(1,1.25*(lb/ld)^2/3) = 0.14666533

Calculation of ratio lb/ld

Inadequate Lap Length with lb/ld = 0.30

Calculation of Mu2-

Calculation of ultimate Moment Strength ((3.67), Biskinis Phd), Mu
Mu = 2.3890E+008

φ = 0.90757121
φ' = 0.80580716
error of function (3.68), Biskinis Phd = 59442.345
From 5A.2, TBDY: fcc = fc*λc = 33.00
conf. factor λc = 1.00
fc = 33.00
From 10.3.5, ASCE 41-17, Final value of fy: fy*Min(1,1.25*(lb/ld)^2/3) = 389.0139
lb/ld = 0.30
d1 = 44.00
R = 250.00
v = 0.0011456
N = 7425.858
Ac = 196349.541
ω = ω*Min(1,1.25*(lb/ld)^2/3) = 0.14666533

Calculation of ratio lb/ld

Inadequate Lap Length with lb/ld = 0.30

Calculation of Shear Strength Vr = Min(Vr1,Vr2) = 484618.662

Calculation of Shear Strength at edge 1, Vr1 = 484618.662
Vr1 = VCol ((10.3), ASCE 41-17) = knl*VCol0
VCol0 = 484618.662
knl = 1 (zero step-static loading)

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

λ = 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 = 2.2610821E-011
Vu = 3.8672673E-031
d = 0.8*D = 400.00
Nu = 7425.858
Ag = 196349.541
From (11.5.4.8), ACI 318-14: Vs = Vs1 + Vs2 = 274157.871
Vs1 = 274157.871 is calculated for jacket, with:
Av = π/2*A_stirrup = 123370.055
fy = 555.56
s = 100.00
Vs1 is multiplied by αCol1 = 1.00
s/d = 0.25
Vs2 = 0.00 is calculated for core, with:
Av = π/2*A_stirrup = 78956.835
fy = 444.44
s = 250.00
Vs2 is multiplied by αCol2 = 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 = π*d*d/4 = 125663.706

Calculation of Shear Strength at edge 2, Vr2 = 484618.662
Vr2 = VCol ((10.3), ASCE 41-17) = knl*VCol0
VCol0 = 484618.662

knl = 1 (zero step-static loading)

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

λ = 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 = 2.2610821E-011
Vu = 3.8672673E-031
d = 0.8*D = 400.00
Nu = 7425.858
Ag = 196349.541
From (11.5.4.8), ACI 318-14: Vs = Vs1 + Vs2 = 274157.871
Vs1 = 274157.871 is calculated for jacket, with:
Av = π/2*A_stirrup = 123370.055
fy = 555.56
s = 100.00
Vs1 is multiplied by αCol1 = 1.00
s/d = 0.25
Vs2 = 0.00 is calculated for core, with:
Av = π/2*A_stirrup = 78956.835
fy = 444.44
s = 250.00
Vs2 is multiplied by αCol2 = 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 = π*d*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, κ = 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 Secondary Member: Concrete Strength, fc = fcm = 33.00
New material of Secondary Member: Steel Strength, fs = fsm = 555.56
Concrete Elasticity, Ec = 26999.444
Steel Elasticity, Es = 200000.00
Existing Column
Existing material of Secondary Member: Concrete Strength, fc = fcm = 20.00
Existing material of Secondary Member: Steel Strength, fs = fsm = 444.44
Concrete Elasticity, Ec = 21019.039
Steel Elasticity, Es = 200000.00
External Diameter, D = 500.00
Internal Diameter, D = 300.00
Cover Thickness, c = 25.00
Element Length, L = 3000.00
Secondary Member
Ribbed Bars
Ductile Steel
Without Detailing for Earthquake Resistance (including stirrups not closed at 135°)

Longitudinal Bars With Ends Lapped Starting at the End Sections
Inadequate Lap Length with lb/ld = 0.30
No FRP Wrapping

Stepwise Properties

Bending Moment, M = -2.6175E+007
Shear Force, V2 = -8722.76
Shear Force, V3 = -1.4777839E-013
Axial Force, F = -7422.368
Longitudinal Reinforcement Area Distribution (in 2 divisions)
-Tension: Aslt = 1272.345
-Compression: Aslc = 1781.283
Longitudinal Reinforcement Area Distribution (in 3 divisions)
-Tension: Asl,ten = 1017.876
-Compression: Asl,com = 1017.876
-Middle: Asl,mid = 1017.876
Mean Diameter of Tension Reinforcement, DbL = 18.00

Existing component: From table 7-7, ASCE 41_17: Final chord rotation Capacity θu,R = κ*θu = 0.00849448
θu = θy + θp = 0.00849448

- Calculation of θy -

θy = (My*Ls/3)/Eleff = 0.00849448 ((4.29),Biskinis Phd))
My = 2.0498E+008
Ls = M/V (with Ls >0.1*L and Ls < 2*L) = 3000.758
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 = 7422.368
Ec*Ig = Ec_jacket*Ig_jacket + Ec_core*Ig_core = 8.0455E+013

Calculation of Yielding Moment My

Calculation of Φy and My according to (7) - (8) in Biskinis and Fardis
----
My = Min(My_ten,My_com) = 2.0498E+008
Φy = 5.8526392E-006
My_ten (8c) = 2.0498E+008
Φ_ten (7c) = 64.04195
error of function (7c) = 8.3474283E-005
My_com (8d) = 7.5621E+008
Φ_com (7d) = 64.56829
error of function (7d) = -0.0072183
with ((10.1), ASCE 41-17) ey = Min(ey, 1.25*ey*(lb/ld)^2/3) = 0.0027778
eco = 0.002
apl = 0.35 ((9a) in Biskinis and Fardis for no FRP Wrap)
d1 = 44.00
R = 250.00
v = 0.00114551
N = 7422.368
Ac = 196349.541
((10.1), ASCE 41-17) ω = Min(ω, 1.25*ω*(lb/ld)^2/3) = 0.26182028
with fc = 33.00

Calculation of ratio lb/ld

Inadequate Lap Length with lb/ld = 0.30

- Calculation of θp -

From table 10-9: θp = 0.00
with:
- Columns controlled by inadequate development or splicing along the clear height because lb/ld < 1
shear control ratio VyE/VCol0E = 0.32864977
d = d_external = 0.00
s = s_external = 0.00
ρt = ρs1 + ρs2 + 2*tf/bw*(ffe/fs) = 0.00323428
jacket: ρs1 = Av1*(π*Dc1/2)/(s1*Ag) = 0.0027646
Av1 = 78.53982, is the area of stirrup
Dc1 = Dext - 2*cover - External Hoop Diameter = 440.00, is the total Length of all stirrups parallel to loading (shear) direction
s1 = 100.00
core: ρs2 = Av2*(π*Dc2/2)/(s2*Ag) = 0.00046968
Av2 = 50.26548, is the area of stirrup
Dc2 = Dint - Internal Hoop Diameter = 292.00, is the total Length of all stirrups parallel to loading (shear) direction
s2 = 250.00
The term 2*tf/bw*(ffe/fs) is implemented to account for FRP contribution
where ρf = 2*tf/bw is FRP ratio (EC8 - 3, A.4.4.3(6)) and ffe/fs normalises ρf to steel strength
All these variables have already been given in Shear control ratio calculation.
For the normalisation fs of jacket is used.
NUD = 7422.368
Ag = 196349.541
f'cE = (fc_jacket*Area_jacket+fc_core*Area_core)/section_area = 28.32
fylE = (fy_ext_Long_Reinf*Area_ext_Long_Reinf+fy_int_Long_Reinf*Area_int_Long_Reinf)/Area_Tot_Long_Rein = 2.1219958E-314
fytE = (fy_ext_Trans_Reinf*ρs1+fy_int_Trans_Reinf*ρs2)/(ρs1+ρs2) = 539.4232
pl = 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: 3
Integration Section: (a)

Calculation No. 5

column C1, Floor 1

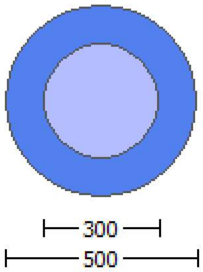
Limit State: Operational Level (data interpolation between analysis steps 1 and 2)

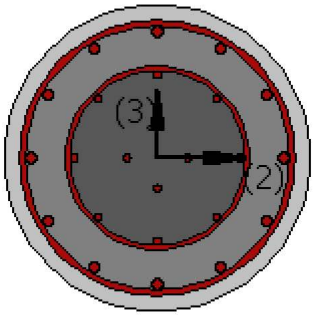
Analysis: Uniform +X

Check: Shear capacity VRd

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, κ = 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 Secondary Member: Concrete Strength, fc = fc_lower_bound = 25.00
New material of Secondary Member: Steel Strength, fs = fs_lower_bound = 500.00
Concrete Elasticity, Ec = 26999.444
Steel Elasticity, Es = 200000.00
Existing Column
Existing material of Secondary Member: Concrete Strength, fc = fc_lower_bound = 16.00
Existing material of Secondary Member: Steel Strength, fs = fs_lower_bound = 400.00
Concrete Elasticity, Ec = 21019.039
Steel Elasticity, Es = 200000.00
###
Note: Especially for the calculation of θy 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, fc = fcm = 33.00
New material: Steel Strength, fs = fsm = 555.56
Existing Column
Existing material: Concrete Strength, fc = fcm = 20.00
Existing material: Steel Strength, fs = fsm = 444.44
###
External Diameter, D = 500.00
Internal Diameter, D = 300.00
Cover Thickness, c = 25.00
Element Length, L = 3000.00
Secondary Member
Ribbed Bars
Ductile Steel
Without Detailing for Earthquake Resistance (including stirrups not closed at 135°)
Longitudinal Bars With Ends Lapped Starting at the End Sections
Inadequate Lap Length with lo/lou,min =lb/ld = 0.30
No FRP Wrapping

Stepwise Properties

EDGE -A-
Bending Moment, Ma = -2.6175E+007
Shear Force, Va = -8722.76
EDGE -B-
Bending Moment, Mb = 0.00906183

Shear Force, Vb = 8722.76
BOTH EDGES
Axial Force, F = -7422.368
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
Mean Diameter of Tension Reinforcement, DbL,ten = 18.00

Existing component: From table 7-7, ASCE 41_17: Final Shear Capacity VR = κ*Vn = 431403.373
Vn ((10.3), ASCE 41-17) = knl*VCol0 = 431403.373
VCol = 431403.373
knl = 1.00
displacement_ductility_demand = 0.19152641

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

λ = 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 = 0.00906183
Vu = 8722.76
d = 0.8*D = 400.00
Nu = 7422.368
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 = π/2*A_stirrup = 123370.055
fy = 500.00
s = 100.00
Vs1 is multiplied by αCol1 = 1.00
s/d = 0.25
Vs2 = 0.00 is calculated for core, with:
Av = π/2*A_stirrup = 78956.835
fy = 400.00
s = 250.00
Vs2 is multiplied by αCol2 = 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 = π*d*d/4 = 125663.706

----
displacement_ductility_demand is calculated as θ/θy

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

From analysis, chord rotation θ = 0.00016265
θy = (My*Ls/3)/EIeff = 0.00084923 ((4.29),Biskinis Phd))
My = 2.0498E+008
Ls = M/V (with Ls >0.1*L and Ls < 2*L) = 300.00
From table 10.5, ASCE 41_17: EIeff = 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 = 7422.368
Ec*Ig = Ec_jacket*Ig_jacket + Ec_core*Ig_core = 8.0455E+013

Calculation of Yielding Moment My

Calculation of Φy and My according to (7) - (8) in Biskinis and Fardis
----
My = Min(My_ten,My_com) = 2.0498E+008
Φy = 5.8526392E-006
My_ten (8c) = 2.0498E+008
Φ_ten (7c) = 64.04195
error of function (7c) = 8.3474283E-005
My_com (8d) = 7.5621E+008
Φ_com (7d) = 64.56829
error of function (7d) = -0.0072183
with ((10.1), ASCE 41-17) ey = Min(ey, 1.25*ey*(lb/ld)^2/3) = 0.0027778
eco = 0.002
apl = 0.35 ((9a) in Biskinis and Fardis for no FRP Wrap)
d1 = 44.00
R = 250.00
v = 0.00114551
N = 7422.368
Ac = 196349.541
((10.1), ASCE 41-17) ω = Min(ω, 1.25*ω*(lb/ld)^2/3) = 0.26182028
with fc = 33.00

Calculation of ratio lb/ld

Inadequate Lap Length with lb/ld = 0.30

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

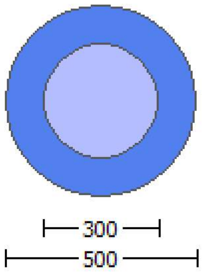
Limit State: Operational Level (data interpolation between analysis steps 1 and 2)

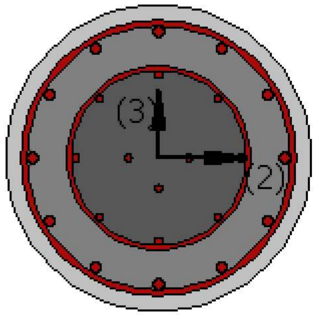
Analysis: Uniform +X

Check: Chord rotation capacity (θu)

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, κ = 1.00
Mean strength values are used for both shear and moment calculations.
Consequently:
Jacket
New material of Secondary Member: Concrete Strength, fc = fcm = 33.00
New material of Secondary Member: Steel Strength, fs = fsm = 555.56
Concrete Elasticity, Ec = 26999.444
Steel Elasticity, Es = 200000.00
Existing Column
Existing material of Secondary Member: Concrete Strength, fc = fcm = 20.00
Existing material of Secondary Member: Steel Strength, fs = fsm = 444.44
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 = 694.45
Existing Column
Existing material: Steel Strength, fs = 1.25*fsm = 555.55
##
External Diameter, D = 500.00
Internal Diameter, D = 300.00
Cover Thickness, c = 25.00
Mean Confinement Factor overall section = 1.00
Element Length, L = 3000.00
Secondary Member
Ribbed Bars
Ductile Steel
Without Detailing for Earthquake Resistance (including stirrups not closed at 135°)
Longitudinal Bars With Ends Lapped Starting at the End Sections
Inadequate Lap Length with lo/lou,min = 0.30
No FRP Wrapping

Stepwise Properties

At local axis: 3
EDGE -A-
Shear Force, Va = -7.1742060E-031
EDGE -B-
Shear Force, Vb = 7.1742060E-031
BOTH EDGES
Axial Force, F = -7425.858
Longitudinal Reinforcement Area Distribution (in 2 divisions)

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

Calculation of Shear Capacity ratio , Ve/Vr = 0.32864977
Member Controlled by Flexure (Ve/Vr < 1)
Calculation of Shear Demand from fig. R18.6.5, ACI 318-14 Ve = (Mpr1 + Mpr2)/ln = 159269.81
with
Mpr1 = Max(Mu1+ , Mu1-) = 2.3890E+008
Mu1+ = 2.3890E+008, is the ultimate moment strength at the edge 1 of the member in the actual moment direction which is defined for the static loading combination
Mu1- = 2.3890E+008, is the ultimate moment strength at the edge 1 of the member in the opposite moment direction which is defined for the static loading combination
Mpr2 = Max(Mu2+ , Mu2-) = 2.3890E+008
Mu2+ = 2.3890E+008, is the ultimate moment strength at the edge 2 of the member in the actual moment direction which is defined for the the static loading combination
Mu2- = 2.3890E+008, is the ultimate moment strength at the edge 2 of the member in the opposite moment direction which is defined for the the static loading combination

----
Calculation of Mu1+

Calculation of ultimate Moment Strength ((3.67), Biskinis Phd), Mu
Mu = 2.3890E+008

φ = 0.90757121
φ' = 0.80580716
error of function (3.68), Biskinis Phd = 59442.345
From 5A.2, TBDY: fcc = fc*λc = 33.00
conf. factor λc = 1.00
fc = 33.00
From 10.3.5, ASCE 41-17, Final value of fy: fy*Min(1,1.25*(lb/ld)^2/3) = 389.0139
lb/ld = 0.30
d1 = 44.00
R = 250.00
v = 0.0011456
N = 7425.858
Ac = 196349.541
ω = ω*Min(1,1.25*(lb/ld)^2/3) = 0.14666533

Calculation of ratio lb/ld

Inadequate Lap Length with lb/ld = 0.30

Calculation of Mu1-

Calculation of ultimate Moment Strength ((3.67), Biskinis Phd), Mu
Mu = 2.3890E+008

φ = 0.90757121
φ' = 0.80580716
error of function (3.68), Biskinis Phd = 59442.345
From 5A.2, TBDY: fcc = fc*λc = 33.00
conf. factor λc = 1.00

fc = 33.00
From 10.3.5, ASCE 41-17, Final value of fy: fy*Min(1,1.25*(lb/ld)^2/3) = 389.0139
lb/ld = 0.30
d1 = 44.00
R = 250.00
v = 0.0011456
N = 7425.858
Ac = 196349.541
ω = ω*Min(1,1.25*(lb/ld)^2/3) = 0.14666533

Calculation of ratio lb/ld

Inadequate Lap Length with lb/ld = 0.30

Calculation of Mu2+

Calculation of ultimate Moment Strength ((3.67), Biskinis Phd), Mu
Mu = 2.3890E+008

φ = 0.90757121
φ' = 0.80580716
error of function (3.68), Biskinis Phd = 59442.345
From 5A.2, TBDY: fcc = fc*λc = 33.00
conf. factor λc = 1.00
fc = 33.00
From 10.3.5, ASCE 41-17, Final value of fy: fy*Min(1,1.25*(lb/ld)^2/3) = 389.0139
lb/ld = 0.30
d1 = 44.00
R = 250.00
v = 0.0011456
N = 7425.858
Ac = 196349.541
ω = ω*Min(1,1.25*(lb/ld)^2/3) = 0.14666533

Calculation of ratio lb/ld

Inadequate Lap Length with lb/ld = 0.30

Calculation of Mu2-

Calculation of ultimate Moment Strength ((3.67), Biskinis Phd), Mu
Mu = 2.3890E+008

φ = 0.90757121
φ' = 0.80580716
error of function (3.68), Biskinis Phd = 59442.345
From 5A.2, TBDY: fcc = fc*λc = 33.00
conf. factor λc = 1.00
fc = 33.00
From 10.3.5, ASCE 41-17, Final value of fy: fy*Min(1,1.25*(lb/ld)^2/3) = 389.0139
lb/ld = 0.30
d1 = 44.00
R = 250.00
v = 0.0011456
N = 7425.858
Ac = 196349.541
ω = ω*Min(1,1.25*(lb/ld)^2/3) = 0.14666533

Calculation of ratio lb/ld

Inadequate Lap Length with lb/ld = 0.30

Calculation of Shear Strength Vr = Min(Vr1,Vr2) = 484618.662

Calculation of Shear Strength at edge 1, Vr1 = 484618.662
Vr1 = VCol ((10.3), ASCE 41-17) = knl*VCol0
VCol0 = 484618.662
knl = 1 (zero step-static loading)

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

λ = 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 = 1.3305601E-011
Vu = 7.1742060E-031
d = 0.8*D = 400.00
Nu = 7425.858
Ag = 196349.541
From (11.5.4.8), ACI 318-14: Vs = Vs1 + Vs2 = 274157.871
Vs1 = 274157.871 is calculated for jacket, with:
Av = π/2*A_stirrup = 123370.055
fy = 555.56
s = 100.00
Vs1 is multiplied by αCol1 = 1.00
s/d = 0.25
Vs2 = 0.00 is calculated for core, with:
Av = π/2*A_stirrup = 78956.835
fy = 444.44
s = 250.00
Vs2 is multiplied by αCol2 = 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 = π*d*d/4 = 125663.706

Calculation of Shear Strength at edge 2, Vr2 = 484618.662
Vr2 = VCol ((10.3), ASCE 41-17) = knl*VCol0
VCol0 = 484618.662
knl = 1 (zero step-static loading)

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

λ = 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 = 1.3305601E-011
Vu = 7.1742060E-031
d = 0.8*D = 400.00
Nu = 7425.858
Ag = 196349.541
From (11.5.4.8), ACI 318-14: Vs = Vs1 + Vs2 = 274157.871
Vs1 = 274157.871 is calculated for jacket, with:
Av = π/2*A_stirrup = 123370.055
fy = 555.56

s = 100.00
Vs1 is multiplied by αCol1 = 1.00
s/d = 0.25
Vs2 = 0.00 is calculated for core, with:
Av = π/2*A_stirrup = 78956.835
fy = 444.44
s = 250.00
Vs2 is multiplied by αCol2 = 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 = π*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, κ = 1.00
Mean strength values are used for both shear and moment calculations.
Consequently:
Jacket
New material of Secondary Member: Concrete Strength, fc = fcm = 33.00
New material of Secondary Member: Steel Strength, fs = fsm = 555.56
Concrete Elasticity, Ec = 26999.444
Steel Elasticity, Es = 200000.00
Existing Column
Existing material of Secondary Member: Concrete Strength, fc = fcm = 20.00
Existing material of Secondary Member: Steel Strength, fs = fsm = 444.44
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 = 694.45
Existing Column
Existing material: Steel Strength, fs = 1.25*fsm = 555.55
###
External Diameter, D = 500.00
Internal Diameter, D = 300.00
Cover Thickness, c = 25.00
Mean Confinement Factor overall section = 1.00
Element Length, L = 3000.00
Secondary Member
Ribbed Bars
Ductile Steel
Without Detailing for Earthquake Resistance (including stirrups not closed at 135°)
Longitudinal Bars With Ends Lapped Starting at the End Sections
Inadequate Lap Length with lo/lou,min = 0.30
No FRP Wrapping

Stepwise Properties

At local axis: 2
EDGE -A-
Shear Force, Va = -3.8672673E-031

EDGE -B-
Shear Force, Vb = 3.8672673E-031
BOTH EDGES
Axial Force, F = -7425.858
Longitudinal Reinforcement Area Distribution (in 2 divisions)
-Tension: Aslt = 0.00
-Compression: Aslc = 3053.628
Longitudinal Reinforcement Area Distribution (in 3 divisions)
-Tension: Asl,ten = 1017.876
-Compression: Asl,com = 1017.876
-Middle: Asl,mid = 1017.876

Calculation of Shear Capacity ratio , Ve/Vr = 0.32864977
Member Controlled by Flexure (Ve/Vr < 1)
Calculation of Shear Demand from fig. R18.6.5, ACI 318-14 Ve = (Mpr1 + Mpr2)/ln = 159269.81
with
Mpr1 = Max(Mu1+ , Mu1-) = 2.3890E+008
Mu1+ = 2.3890E+008, is the ultimate moment strength at the edge 1 of the member in the actual moment direction which is defined for the static loading combination
Mu1- = 2.3890E+008, is the ultimate moment strength at the edge 1 of the member in the opposite moment direction which is defined for the static loading combination
Mpr2 = Max(Mu2+ , Mu2-) = 2.3890E+008
Mu2+ = 2.3890E+008, is the ultimate moment strength at the edge 2 of the member in the actual moment direction which is defined for the the static loading combination
Mu2- = 2.3890E+008, is the ultimate moment strength at the edge 2 of the member in the opposite moment direction which is defined for the the static loading combination

----
Calculation of Mu1+

Calculation of ultimate Moment Strength ((3.67), Biskinis Phd), Mu
Mu = 2.3890E+008

φ = 0.90757121
φ' = 0.80580716
error of function (3.68), Biskinis Phd = 59442.345
From 5A.2, TBDY: fcc = fc*λc = 33.00
conf. factor λc = 1.00
fc = 33.00
From 10.3.5, ASCE 41-17, Final value of fy: fy*Min(1,1.25*(lb/ld)^2/3) = 389.0139
lb/ld = 0.30
d1 = 44.00
R = 250.00
v = 0.0011456
N = 7425.858
Ac = 196349.541
ω = ω*Min(1,1.25*(lb/ld)^2/3) = 0.14666533

Calculation of ratio lb/ld

Inadequate Lap Length with lb/ld = 0.30

Calculation of Mu1-

Calculation of ultimate Moment Strength ((3.67), Biskinis Phd), Mu
Mu = 2.3890E+008

φ = 0.90757121
φ' = 0.80580716
error of function (3.68), Biskinis Phd = 59442.345
From 5A.2, TBDY: fcc = fc*λc = 33.00
conf. factor λc = 1.00
fc = 33.00
From 10.3.5, ASCE 41-17, Final value of fy: fy*Min(1,1.25*(lb/ld)^2/3) = 389.0139
lb/ld = 0.30
d1 = 44.00
R = 250.00
v = 0.0011456
N = 7425.858
Ac = 196349.541
ω = ω*Min(1,1.25*(lb/ld)^2/3) = 0.14666533

Calculation of ratio lb/ld

Inadequate Lap Length with lb/ld = 0.30

Calculation of Mu2+

Calculation of ultimate Moment Strength ((3.67), Biskinis Phd), Mu
Mu = 2.3890E+008

φ = 0.90757121
φ' = 0.80580716
error of function (3.68), Biskinis Phd = 59442.345
From 5A.2, TBDY: fcc = fc*λc = 33.00
conf. factor λc = 1.00
fc = 33.00
From 10.3.5, ASCE 41-17, Final value of fy: fy*Min(1,1.25*(lb/ld)^2/3) = 389.0139
lb/ld = 0.30
d1 = 44.00
R = 250.00
v = 0.0011456
N = 7425.858
Ac = 196349.541
ω = ω*Min(1,1.25*(lb/ld)^2/3) = 0.14666533

Calculation of ratio lb/ld

Inadequate Lap Length with lb/ld = 0.30

Calculation of Mu2-

Calculation of ultimate Moment Strength ((3.67), Biskinis Phd), Mu
Mu = 2.3890E+008

φ = 0.90757121
φ' = 0.80580716
error of function (3.68), Biskinis Phd = 59442.345
From 5A.2, TBDY: fcc = fc*λc = 33.00
conf. factor λc = 1.00
fc = 33.00
From 10.3.5, ASCE 41-17, Final value of fy: fy*Min(1,1.25*(lb/ld)^2/3) = 389.0139
lb/ld = 0.30
d1 = 44.00

R = 250.00
v = 0.0011456
N = 7425.858
Ac = 196349.541
ω = ω*Min(1,1.25*(lb/ld)^2/3) = 0.14666533

Calculation of ratio lb/ld

Inadequate Lap Length with lb/ld = 0.30

Calculation of Shear Strength Vr = Min(Vr1,Vr2) = 484618.662

Calculation of Shear Strength at edge 1, Vr1 = 484618.662
Vr1 = VCol ((10.3), ASCE 41-17) = knl*VCol0
VCol0 = 484618.662
knl = 1 (zero step-static loading)

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

λ = 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 = 2.2610821E-011
Vu = 3.8672673E-031
d = 0.8*D = 400.00
Nu = 7425.858
Ag = 196349.541
From (11.5.4.8), ACI 318-14: Vs = Vs1 + Vs2 = 274157.871
Vs1 = 274157.871 is calculated for jacket, with:
Av = π/2*A_stirrup = 123370.055
fy = 555.56
s = 100.00
Vs1 is multiplied by αCol1 = 1.00
s/d = 0.25
Vs2 = 0.00 is calculated for core, with:
Av = π/2*A_stirrup = 78956.835
fy = 444.44
s = 250.00
Vs2 is multiplied by αCol2 = 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 = π*d*d/4 = 125663.706

Calculation of Shear Strength at edge 2, Vr2 = 484618.662
Vr2 = VCol ((10.3), ASCE 41-17) = knl*VCol0
VCol0 = 484618.662
knl = 1 (zero step-static loading)

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

λ = 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 = 2.2610821E-011
Vu = 3.8672673E-031
d = 0.8*D = 400.00
Nu = 7425.858

Ag = 196349.541
From (11.5.4.8), ACI 318-14: Vs = Vs1 + Vs2 = 274157.871
Vs1 = 274157.871 is calculated for jacket, with:
Av = π/2*A_stirrup = 123370.055
fy = 555.56
s = 100.00
Vs1 is multiplied by αCol1 = 1.00
s/d = 0.25
Vs2 = 0.00 is calculated for core, with:
Av = π/2*A_stirrup = 78956.835
fy = 444.44
s = 250.00
Vs2 is multiplied by αCol2 = 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 = π*d*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: (b)
Section Type: rcjcs

Constant Properties

Knowledge Factor, κ = 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 Secondary Member: Concrete Strength, fc = fcm = 33.00
New material of Secondary Member: Steel Strength, fs = fsm = 555.56
Concrete Elasticity, Ec = 26999.444
Steel Elasticity, Es = 200000.00
Existing Column
Existing material of Secondary Member: Concrete Strength, fc = fcm = 20.00
Existing material of Secondary Member: Steel Strength, fs = fsm = 444.44
Concrete Elasticity, Ec = 21019.039
Steel Elasticity, Es = 200000.00
External Diameter, D = 500.00
Internal Diameter, D = 300.00
Cover Thickness, c = 25.00
Element Length, L = 3000.00
Secondary Member
Ribbed Bars
Ductile Steel
Without Detailing for Earthquake Resistance (including stirrups not closed at 135°)
Longitudinal Bars With Ends Lapped Starting at the End Sections
Inadequate Lap Length with lb/ld = 0.30
No FRP Wrapping

Stepwise Properties

Bending Moment, M = 1.5368128E-010
Shear Force, V2 = 8722.76
Shear Force, V3 = 1.4777839E-013
Axial Force, F = -7422.368
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
Mean Diameter of Tension Reinforcement, DbL = 18.00

Existing component: From table 7-7, ASCE 41_17: Final chord rotation Capacity θu,R = κ*θu = 0.00424617
θu = θy + θp = 0.00424617

- Calculation of θy -

θy = (My*Ls/3)/EIeff = 0.00424617 ((4.29),Biskinis Phd))
My = 2.0498E+008
Ls = M/V (with Ls >0.1*L and Ls < 2*L) = 1500.00
From table 10.5, ASCE 41_17: EIeff = 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 = 7422.368
Ec*Ig = Ec_jacket*Ig_jacket + Ec_core*Ig_core = 8.0455E+013

Calculation of Yielding Moment My

Calculation of Φy and My according to (7) - (8) in Biskinis and Fardis
----
My = Min(My_ten,My_com) = 2.0498E+008
Φy = 5.8526392E-006
My_ten (8c) = 2.0498E+008
Φ_ten (7c) = 64.04195
error of function (7c) = 8.3474283E-005
My_com (8d) = 7.5621E+008
Φ_com (7d) = 64.56829
error of function (7d) = -0.0072183
with ((10.1), ASCE 41-17) ey = Min(ey, 1.25*ey*(lb/ld)^2/3) = 0.0027778
eco = 0.002
apl = 0.35 ((9a) in Biskinis and Fardis for no FRP Wrap)
d1 = 44.00
R = 250.00
v = 0.00114551
N = 7422.368
Ac = 196349.541
((10.1), ASCE 41-17) ω = Min(ω, 1.25*ω*(lb/ld)^2/3) = 0.26182028
with fc = 33.00

Calculation of ratio lb/ld

Inadequate Lap Length with lb/ld = 0.30

- Calculation of θp -

From table 10-9: θp = 0.00
with:
- Columns controlled by inadequate development or splicing along the clear height because lb/ld < 1
shear control ratio VyE/VColOE = 0.32864977
d = d_external = 0.00
s = s_external = 0.00
ρt = ρs1 + ρs2 + 2*tf/bw*(ffe/fs) = 0.00323428

jacket: ρs1 = Av1*(π*Dc1/2)/(s1*Ag) = 0.0027646
Av1 = 78.53982, is the area of stirrup
Dc1 = Dext - 2*cover - External Hoop Diameter = 440.00, is the total Length of all stirrups parallel to loading (shear) direction
s1 = 100.00
core: ρs2 = Av2*(π*Dc2/2)/(s2*Ag) = 0.00046968
Av2 = 50.26548, is the area of stirrup
Dc2 = Dint - Internal Hoop Diameter = 292.00, is the total Length of all stirrups parallel to loading (shear) direction
s2 = 250.00
The term 2*tf/bw*(ffe/fs) is implemented to account for FRP contribution
where ρf = 2*tf/bw is FRP ratio (EC8 - 3, A.4.4.3(6)) and ffe/fs normalises ρf to steel strength
All these variables have already been given in Shear control ratio calculation.
For the normalisation fs of jacket is used.
NUD = 7422.368
Ag = 196349.541
f'cE = (fc_jacket*Area_jacket+fc_core*Area_core)/section_area = 28.32
fylE = (fy_ext_Long_Reinf*Area_ext_Long_Reinf+fy_int_Long_Reinf*Area_int_Long_Reinf)/Area_Tot_Long_Rein = 2.1219958E-314
fytE = (fy_ext_Trans_Reinf*ρs1+fy_int_Trans_Reinf*ρs2)/(ρs1+ρs2) = 539.4232
pl = 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

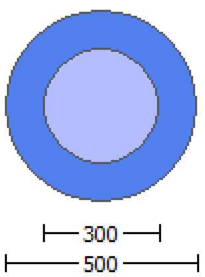
Limit State: Operational Level (data interpolation between analysis steps 1 and 2)

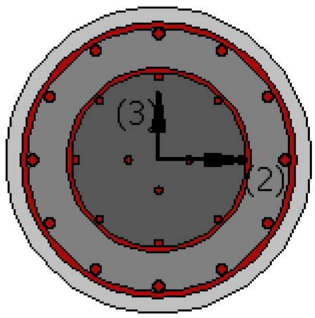
Analysis: Uniform +X

Check: Shear capacity VRd

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, κ = 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 Secondary Member: Concrete Strength, fc = fc_lower_bound = 25.00
New material of Secondary Member: Steel Strength, fs = fs_lower_bound = 500.00
Concrete Elasticity, Ec = 26999.444
Steel Elasticity, Es = 200000.00
Existing Column
Existing material of Secondary Member: Concrete Strength, fc = fc_lower_bound = 16.00
Existing material of Secondary Member: Steel Strength, fs = fs_lower_bound = 400.00
Concrete Elasticity, Ec = 21019.039
Steel Elasticity, Es = 200000.00
###
Note: Especially for the calculation of θy 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, fc = fcm = 33.00
New material: Steel Strength, fs = fsm = 555.56
Existing Column
Existing material: Concrete Strength, fc = fcm = 20.00
Existing material: Steel Strength, fs = fsm = 444.44
###
External Diameter, D = 500.00
Internal Diameter, D = 300.00
Cover Thickness, c = 25.00
Element Length, L = 3000.00
Secondary Member
Ribbed Bars
Ductile Steel
Without Detailing for Earthquake Resistance (including stirrups not closed at 135°)
Longitudinal Bars With Ends Lapped Starting at the End Sections
Inadequate Lap Length with lo/lou,min =lb/ld = 0.30
No FRP Wrapping

Stepwise Properties

EDGE -A-
Bending Moment, Ma = 2.8960610E-010
Shear Force, Va = -1.4777839E-013
EDGE -B-
Bending Moment, Mb = 1.5368128E-010
Shear Force, Vb = 1.4777839E-013
BOTH EDGES
Axial Force, F = -7422.368
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
Mean Diameter of Tension Reinforcement, DbL,ten = 18.00

Existing component: From table 7-7, ASCE 41_17: Final Shear Capacity VR = κ*Vn = 431403.373

Vn ((10.3), ASCE 41-17) = knl*VCol0 = 431403.373
VCol = 431403.373
knl = 1.00
displacement_ductility_demand = 0.00

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

λ = 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 = 1.5368128E-010
Vu = 1.4777839E-013
d = 0.8*D = 400.00
Nu = 7422.368
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 = π/2*A_stirrup = 123370.055
fy = 500.00
s = 100.00
Vs1 is multiplied by αCol1 = 1.00
s/d = 0.25
Vs2 = 0.00 is calculated for core, with:
Av = π/2*A_stirrup = 78956.835
fy = 400.00
s = 250.00
Vs2 is multiplied by αCol2 = 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 = π*d*d/4 = 125663.706

----
displacement_ductility_demand is calculated as θ/θy

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

From analysis, chord rotation θ = 1.4965376E-022
θy = (My*Ls/3)/EIeff = 0.00424617 ((4.29),Biskinis Phd))
My = 2.0498E+008
Ls = M/V (with Ls >0.1*L and Ls < 2*L) = 1500.00
From table 10.5, ASCE 41_17: EIeff = 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 = 7422.368
Ec*Ig = Ec_jacket*Ig_jacket + Ec_core*Ig_core = 8.0455E+013

Calculation of Yielding Moment My

Calculation of Φy and My according to (7) - (8) in Biskinis and Fardis
----
My = Min(My_ten,My_com) = 2.0498E+008
Φy = 5.8526392E-006
My_ten (8c) = 2.0498E+008
Φ_ten (7c) = 64.04195
error of function (7c) = 8.3474283E-005
My_com (8d) = 7.5621E+008
Φ_com (7d) = 64.56829
error of function (7d) = -0.0072183
with ((10.1), ASCE 41-17) ey = Min(ey, 1.25*ey*(lb/ld)^2/3) = 0.0027778

eco = 0.002
apl = 0.35 ((9a) in Biskinis and Fardis for no FRP Wrap)
d1 = 44.00
R = 250.00
v = 0.00114551
N = 7422.368
Ac = 196349.541
((10.1), ASCE 41-17) ω = Min(ω, 1.25*ω*(lb/ld)^2/3) = 0.26182028
with fc = 33.00

Calculation of ratio lb/ld

Inadequate Lap Length with lb/ld = 0.30

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

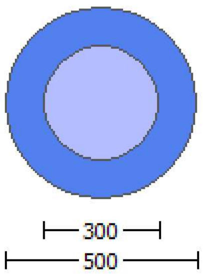
Limit State: Operational Level (data interpolation between analysis steps 1 and 2)

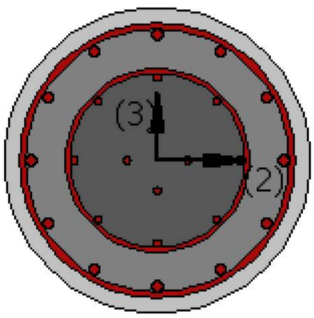
Analysis: Uniform +X

Check: Chord rotation capacity (θu)

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, κ = 1.00

Mean strength values are used for both shear and moment calculations.
Consequently:
Jacket
New material of Secondary Member: Concrete Strength, fc = fcm = 33.00
New material of Secondary Member: Steel Strength, fs = fsm = 555.56
Concrete Elasticity, Ec = 26999.444
Steel Elasticity, Es = 200000.00
Existing Column
Existing material of Secondary Member: Concrete Strength, fc = fcm = 20.00
Existing material of Secondary Member: Steel Strength, fs = fsm = 444.44
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 = 694.45
Existing Column
Existing material: Steel Strength, fs = 1.25*fsm = 555.55
##
External Diameter, D = 500.00
Internal Diameter, D = 300.00
Cover Thickness, c = 25.00
Mean Confinement Factor overall section = 1.00
Element Length, L = 3000.00
Secondary Member
Ribbed Bars
Ductile Steel
Without Detailing for Earthquake Resistance (including stirrups not closed at 135°)
Longitudinal Bars With Ends Lapped Starting at the End Sections
Inadequate Lap Length with lo/lou,min = 0.30
No FRP Wrapping

Stepwise Properties

At local axis: 3
EDGE -A-
Shear Force, Va = -7.1742060E-031
EDGE -B-
Shear Force, Vb = 7.1742060E-031
BOTH EDGES
Axial Force, F = -7425.858
Longitudinal Reinforcement Area Distribution (in 2 divisions)
-Tension: Aslt = 0.00
-Compression: Aslc = 3053.628
Longitudinal Reinforcement Area Distribution (in 3 divisions)
-Tension: Asl,ten = 1017.876
-Compression: Asl,com = 1017.876
-Middle: Asl,mid = 1017.876

Calculation of Shear Capacity ratio , Ve/Vr = 0.32864977
Member Controlled by Flexure (Ve/Vr < 1)
Calculation of Shear Demand from fig. R18.6.5, ACI 318-14 Ve = (Mpr1 + Mpr2)/ln = 159269.81
with
Mpr1 = Max(Mu1+ , Mu1-) = 2.3890E+008
Mu1+ = 2.3890E+008, is the ultimate moment strength at the edge 1 of the member in the actual moment direction which is defined for the static loading combination
Mu1- = 2.3890E+008, is the ultimate moment strength at the edge 1 of the member in the opposite moment direction which is defined for the static loading combination
Mpr2 = Max(Mu2+ , Mu2-) = 2.3890E+008
Mu2+ = 2.3890E+008, is the ultimate moment strength at the edge 2 of the member in the actual moment direction which is defined for the the static loading combination
Mu2- = 2.3890E+008, is the ultimate moment strength at the edge 2 of the member in the opposite moment

direction which is defined for the the static loading combination

----
Calculation of Mu1+

Calculation of ultimate Moment Strength ((3.67), Biskinis Phd), Mu
Mu = 2.3890E+008

φ = 0.90757121
φ' = 0.80580716
error of function (3.68), Biskinis Phd = 59442.345
From 5A.2, TBDY: fcc = fc*λc = 33.00
conf. factor λc = 1.00
fc = 33.00
From 10.3.5, ASCE 41-17, Final value of fy: fy*Min(1,1.25*(lb/ld)^2/3) = 389.0139
lb/ld = 0.30
d1 = 44.00
R = 250.00
v = 0.0011456
N = 7425.858
Ac = 196349.541
ω = ω*Min(1,1.25*(lb/ld)^2/3) = 0.14666533

Calculation of ratio lb/ld

Inadequate Lap Length with lb/ld = 0.30

Calculation of Mu1-

Calculation of ultimate Moment Strength ((3.67), Biskinis Phd), Mu
Mu = 2.3890E+008

φ = 0.90757121
φ' = 0.80580716
error of function (3.68), Biskinis Phd = 59442.345
From 5A.2, TBDY: fcc = fc*λc = 33.00
conf. factor λc = 1.00
fc = 33.00
From 10.3.5, ASCE 41-17, Final value of fy: fy*Min(1,1.25*(lb/ld)^2/3) = 389.0139
lb/ld = 0.30
d1 = 44.00
R = 250.00
v = 0.0011456
N = 7425.858
Ac = 196349.541
ω = ω*Min(1,1.25*(lb/ld)^2/3) = 0.14666533

Calculation of ratio lb/ld

Inadequate Lap Length with lb/ld = 0.30

Calculation of Mu2+

Calculation of ultimate Moment Strength ((3.67), Biskinis Phd), Mu
Mu = 2.3890E+008

φ = 0.90757121
φ' = 0.80580716
error of function (3.68), Biskinis Phd = 59442.345
From 5A.2, TBDY: fcc = fc*λc = 33.00
conf. factor λc = 1.00
fc = 33.00
From 10.3.5, ASCE 41-17, Final value of fy: fy*Min(1,1.25*(lb/ld)^2/3) = 389.0139
lb/ld = 0.30
d1 = 44.00
R = 250.00
v = 0.0011456
N = 7425.858
Ac = 196349.541
ω = ω*Min(1,1.25*(lb/ld)^2/3) = 0.14666533

Calculation of ratio lb/ld

Inadequate Lap Length with lb/ld = 0.30

Calculation of Mu2-

Calculation of ultimate Moment Strength ((3.67), Biskinis Phd), Mu
Mu = 2.3890E+008

φ = 0.90757121
φ' = 0.80580716
error of function (3.68), Biskinis Phd = 59442.345
From 5A.2, TBDY: fcc = fc*λc = 33.00
conf. factor λc = 1.00
fc = 33.00
From 10.3.5, ASCE 41-17, Final value of fy: fy*Min(1,1.25*(lb/ld)^2/3) = 389.0139
lb/ld = 0.30
d1 = 44.00
R = 250.00
v = 0.0011456
N = 7425.858
Ac = 196349.541
ω = ω*Min(1,1.25*(lb/ld)^2/3) = 0.14666533

Calculation of ratio lb/ld

Inadequate Lap Length with lb/ld = 0.30

Calculation of Shear Strength Vr = Min(Vr1,Vr2) = 484618.662

Calculation of Shear Strength at edge 1, Vr1 = 484618.662
Vr1 = VCol ((10.3), ASCE 41-17) = knl*VCol0
VCol0 = 484618.662
knl = 1 (zero step-static loading)

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

λ = 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 = 1.3305601E-011
Vu = 7.1742060E-031
d = 0.8*D = 400.00
Nu = 7425.858
Ag = 196349.541
From (11.5.4.8), ACI 318-14: Vs = Vs1 + Vs2 = 274157.871
Vs1 = 274157.871 is calculated for jacket, with:
Av = π/2*A_stirrup = 123370.055
fy = 555.56
s = 100.00
Vs1 is multiplied by αCol1 = 1.00
s/d = 0.25
Vs2 = 0.00 is calculated for core, with:
Av = π/2*A_stirrup = 78956.835
fy = 444.44
s = 250.00
Vs2 is multiplied by αCol2 = 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 = π*d*d/4 = 125663.706

Calculation of Shear Strength at edge 2, Vr2 = 484618.662
Vr2 = VCol ((10.3), ASCE 41-17) = knl*VCol0
VCol0 = 484618.662
knl = 1 (zero step-static loading)

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

λ = 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 = 1.3305601E-011
Vu = 7.1742060E-031
d = 0.8*D = 400.00
Nu = 7425.858
Ag = 196349.541
From (11.5.4.8), ACI 318-14: Vs = Vs1 + Vs2 = 274157.871
Vs1 = 274157.871 is calculated for jacket, with:
Av = π/2*A_stirrup = 123370.055
fy = 555.56
s = 100.00
Vs1 is multiplied by αCol1 = 1.00
s/d = 0.25
Vs2 = 0.00 is calculated for core, with:
Av = π/2*A_stirrup = 78956.835
fy = 444.44
s = 250.00
Vs2 is multiplied by αCol2 = 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 = π*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, κ = 1.00
Mean strength values are used for both shear and moment calculations.
Consequently:
Jacket
New material of Secondary Member: Concrete Strength, fc = fcm = 33.00
New material of Secondary Member: Steel Strength, fs = fsm = 555.56
Concrete Elasticity, Ec = 26999.444
Steel Elasticity, Es = 200000.00
Existing Column
Existing material of Secondary Member: Concrete Strength, fc = fcm = 20.00
Existing material of Secondary Member: Steel Strength, fs = fsm = 444.44
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 = 694.45
Existing Column
Existing material: Steel Strength, fs = 1.25*fsm = 555.55
###
External Diameter, D = 500.00
Internal Diameter, D = 300.00
Cover Thickness, c = 25.00
Mean Confinement Factor overall section = 1.00
Element Length, L = 3000.00
Secondary Member
Ribbed Bars
Ductile Steel
Without Detailing for Earthquake Resistance (including stirrups not closed at 135°)
Longitudinal Bars With Ends Lapped Starting at the End Sections
Inadequate Lap Length with lo/lou,min = 0.30
No FRP Wrapping

Stepwise Properties

At local axis: 2
EDGE -A-
Shear Force, Va = -3.8672673E-031
EDGE -B-
Shear Force, Vb = 3.8672673E-031
BOTH EDGES
Axial Force, F = -7425.858
Longitudinal Reinforcement Area Distribution (in 2 divisions)
-Tension: Aslt = 0.00
-Compression: Aslc = 3053.628
Longitudinal Reinforcement Area Distribution (in 3 divisions)
-Tension: Asl,ten = 1017.876
-Compression: Asl,com = 1017.876
-Middle: Asl,mid = 1017.876

Calculation of Shear Capacity ratio , Ve/Vr = 0.32864977
Member Controlled by Flexure (Ve/Vr < 1)
Calculation of Shear Demand from fig. R18.6.5, ACI 318-14 Ve = (Mpr1 + Mpr2)/ln = 159269.81
with
Mpr1 = Max(Mu1+ , Mu1-) = 2.3890E+008
Mu1+ = 2.3890E+008, is the ultimate moment strength at the edge 1 of the member in the actual moment direction which is defined for the static loading combination
Mu1- = 2.3890E+008, is the ultimate moment strength at the edge 1 of the member in the opposite moment

direction which is defined for the static loading combination
Mpr2 = Max(Mu2+ , Mu2-) = 2.3890E+008
Mu2+ = 2.3890E+008, is the ultimate moment strength at the edge 2 of the member in the actual moment direction which is defined for the the static loading combination
Mu2- = 2.3890E+008, is the ultimate moment strength at the edge 2 of the member in the opposite moment direction which is defined for the the static loading combination

----
Calculation of Mu1+

Calculation of ultimate Moment Strength ((3.67), Biskinis Phd), Mu
Mu = 2.3890E+008

φ = 0.90757121
φ' = 0.80580716
error of function (3.68), Biskinis Phd = 59442.345
From 5A.2, TBDY: fcc = fc*λc = 33.00
conf. factor λc = 1.00
fc = 33.00
From 10.3.5, ASCE 41-17, Final value of fy: fy*Min(1,1.25*(lb/ld)^2/3) = 389.0139
lb/ld = 0.30
d1 = 44.00
R = 250.00
v = 0.0011456
N = 7425.858
Ac = 196349.541
ω = ω*Min(1,1.25*(lb/ld)^2/3) = 0.14666533

Calculation of ratio lb/ld

Inadequate Lap Length with lb/ld = 0.30

Calculation of Mu1-

Calculation of ultimate Moment Strength ((3.67), Biskinis Phd), Mu
Mu = 2.3890E+008

φ = 0.90757121
φ' = 0.80580716
error of function (3.68), Biskinis Phd = 59442.345
From 5A.2, TBDY: fcc = fc*λc = 33.00
conf. factor λc = 1.00
fc = 33.00
From 10.3.5, ASCE 41-17, Final value of fy: fy*Min(1,1.25*(lb/ld)^2/3) = 389.0139
lb/ld = 0.30
d1 = 44.00
R = 250.00
v = 0.0011456
N = 7425.858
Ac = 196349.541
ω = ω*Min(1,1.25*(lb/ld)^2/3) = 0.14666533

Calculation of ratio lb/ld

Inadequate Lap Length with lb/ld = 0.30

Calculation of Mu2+

Calculation of ultimate Moment Strength ((3.67), Biskinis Phd), Mu
Mu = 2.3890E+008

φ = 0.90757121
φ' = 0.80580716
error of function (3.68), Biskinis Phd = 59442.345
From 5A.2, TBDY: fcc = fc*λc = 33.00
conf. factor λc = 1.00
fc = 33.00
From 10.3.5, ASCE 41-17, Final value of fy: fy*Min(1,1.25*(lb/ld)^2/3) = 389.0139
lb/ld = 0.30
d1 = 44.00
R = 250.00
v = 0.0011456
N = 7425.858
Ac = 196349.541
ω = ω*Min(1,1.25*(lb/ld)^2/3) = 0.14666533

Calculation of ratio lb/ld

Inadequate Lap Length with lb/ld = 0.30

Calculation of Mu2-

Calculation of ultimate Moment Strength ((3.67), Biskinis Phd), Mu
Mu = 2.3890E+008

φ = 0.90757121
φ' = 0.80580716
error of function (3.68), Biskinis Phd = 59442.345
From 5A.2, TBDY: fcc = fc*λc = 33.00
conf. factor λc = 1.00
fc = 33.00
From 10.3.5, ASCE 41-17, Final value of fy: fy*Min(1,1.25*(lb/ld)^2/3) = 389.0139
lb/ld = 0.30
d1 = 44.00
R = 250.00
v = 0.0011456
N = 7425.858
Ac = 196349.541
ω = ω*Min(1,1.25*(lb/ld)^2/3) = 0.14666533

Calculation of ratio lb/ld

Inadequate Lap Length with lb/ld = 0.30

Calculation of Shear Strength Vr = Min(Vr1,Vr2) = 484618.662

Calculation of Shear Strength at edge 1, Vr1 = 484618.662
Vr1 = VCol ((10.3), ASCE 41-17) = knl*VCol0
VCol0 = 484618.662
knl = 1 (zero step-static loading)

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

λ = 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 = 2.2610821E-011
Vu = 3.8672673E-031
d = 0.8*D = 400.00
Nu = 7425.858
Ag = 196349.541
From (11.5.4.8), ACI 318-14: Vs = Vs1 + Vs2 = 274157.871
Vs1 = 274157.871 is calculated for jacket, with:
Av = π/2*A_stirrup = 123370.055
fy = 555.56
s = 100.00
Vs1 is multiplied by αCol1 = 1.00
s/d = 0.25
Vs2 = 0.00 is calculated for core, with:
Av = π/2*A_stirrup = 78956.835
fy = 444.44
s = 250.00
Vs2 is multiplied by αCol2 = 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 = π*d*d/4 = 125663.706

Calculation of Shear Strength at edge 2, Vr2 = 484618.662
Vr2 = VCol ((10.3), ASCE 41-17) = knl*VCol0
VCol0 = 484618.662
knl = 1 (zero step-static loading)

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

λ = 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 = 2.2610821E-011
Vu = 3.8672673E-031
d = 0.8*D = 400.00
Nu = 7425.858
Ag = 196349.541
From (11.5.4.8), ACI 318-14: Vs = Vs1 + Vs2 = 274157.871
Vs1 = 274157.871 is calculated for jacket, with:
Av = π/2*A_stirrup = 123370.055
fy = 555.56
s = 100.00
Vs1 is multiplied by αCol1 = 1.00
s/d = 0.25
Vs2 = 0.00 is calculated for core, with:
Av = π/2*A_stirrup = 78956.835
fy = 444.44
s = 250.00
Vs2 is multiplied by αCol2 = 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 = π*d*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, κ = 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 Secondary Member: Concrete Strength, fc = fcm = 33.00
New material of Secondary Member: Steel Strength, fs = fsm = 555.56
Concrete Elasticity, Ec = 26999.444
Steel Elasticity, Es = 200000.00
Existing Column
Existing material of Secondary Member: Concrete Strength, fc = fcm = 20.00
Existing material of Secondary Member: Steel Strength, fs = fsm = 444.44
Concrete Elasticity, Ec = 21019.039
Steel Elasticity, Es = 200000.00
External Diameter, D = 500.00
Internal Diameter, D = 300.00
Cover Thickness, c = 25.00
Element Length, L = 3000.00
Secondary Member
Ribbed Bars
Ductile Steel
Without Detailing for Earthquake Resistance (including stirrups not closed at 135°)
Longitudinal Bars With Ends Lapped Starting at the End Sections
Inadequate Lap Length with lb/ld = 0.30
No FRP Wrapping

Stepwise Properties

Bending Moment, M = 0.00906183
Shear Force, V2 = 8722.76
Shear Force, V3 = 1.4777839E-013
Axial Force, F = -7422.368
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
Mean Diameter of Tension Reinforcement, DbL = 18.00

Existing component: From table 7-7, ASCE 41_17: Final chord rotation Capacity θu,R = κ*θu = 0.00084923
θu = θy + θp = 0.00084923

- Calculation of θy -

θy = (My*Ls/3)/EIeff = 0.00084923 ((4.29),Biskinis Phd))
My = 2.0498E+008
Ls = M/V (with Ls >0.1*L and Ls < 2*L) = 300.00
From table 10.5, ASCE 41_17: EIeff = 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 = 7422.368
Ec*Ig = Ec_jacket*Ig_jacket + Ec_core*Ig_core = 8.0455E+013

Calculation of Yielding Moment My

Calculation of Φy and My according to (7) - (8) in Biskinis and Fardis
----
My = Min(My_ten,My_com) = 2.0498E+008
Φy = 5.8526392E-006
My_ten (8c) = 2.0498E+008
Φ_ten (7c) = 64.04195
error of function (7c) = 8.3474283E-005
My_com (8d) = 7.5621E+008
Φ_com (7d) = 64.56829
error of function (7d) = -0.0072183
with ((10.1), ASCE 41-17) ey = Min(ey, 1.25*ey*(lb/ld)^2/3) = 0.0027778
eco = 0.002
apl = 0.35 ((9a) in Biskinis and Fardis for no FRP Wrap)
d1 = 44.00
R = 250.00
v = 0.00114551
N = 7422.368
Ac = 196349.541
((10.1), ASCE 41-17) ω = Min(ω, 1.25*ω*(lb/ld)^2/3) = 0.26182028
with fc = 33.00

Calculation of ratio lb/ld

Inadequate Lap Length with lb/ld = 0.30

- Calculation of θp -

From table 10-9: θp = 0.00
with:
- Columns controlled by inadequate development or splicing along the clear height because lb/ld < 1
shear control ratio VyE/VCol0E = 0.32864977
d = d_external = 0.00
s = s_external = 0.00
ρt = ρs1 + ρs2 + 2*tf/bw*(ffe/fs) = 0.00323428
jacket: ρs1 = Av1*(π*Dc1/2)/(s1*Ag) = 0.0027646
Av1 = 78.53982, is the area of stirrup
Dc1 = Dext - 2*cover - External Hoop Diameter = 440.00, is the total Length of all stirrups parallel to loading (shear) direction
s1 = 100.00
core: ρs2 = Av2*(π*Dc2/2)/(s2*Ag) = 0.00046968
Av2 = 50.26548, is the area of stirrup
Dc2 = Dint - Internal Hoop Diameter = 292.00, is the total Length of all stirrups parallel to loading (shear) direction
s2 = 250.00
The term 2*tf/bw*(ffe/fs) is implemented to account for FRP contribution
where ρf = 2*tf/bw is FRP ratio (EC8 - 3, A.4.4.3(6)) and ffe/fs normalises ρf to steel strength
All these variables have already been given in Shear control ratio calculation.
For the normalisation fs of jacket is used.
NUD = 7422.368
Ag = 196349.541
f'cE = (fc_jacket*Area_jacket+fc_core*Area_core)/section_area = 28.32
fylE = (fy_ext_Long_Reinf*Area_ext_Long_Reinf+fy_int_Long_Reinf*Area_int_Long_Reinf)/Area_Tot_Long_Rein = 2.1219958E-314
fytE = (fy_ext_Trans_Reinf*ρs1+fy_int_Trans_Reinf*ρs2)/(ρs1+ρs2) = 539.4232
pl = 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: 3
Integration Section: (b)

Calculation No. 9

column C1, Floor 1

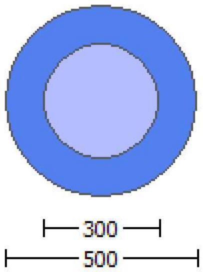
Limit State: Life Safety (data interpolation between analysis steps 1 and 2)

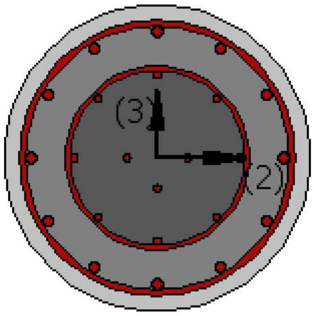
Analysis: Uniform +X

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, κ = 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 Secondary Member: Concrete Strength, fc = fc_lower_bound = 25.00
New material of Secondary Member: Steel Strength, fs = fs_lower_bound = 500.00
Concrete Elasticity, Ec = 26999.444
Steel Elasticity, Es = 200000.00
Existing Column
Existing material of Secondary Member: Concrete Strength, fc = fc_lower_bound = 16.00
Existing material of Secondary Member: Steel Strength, fs = fs_lower_bound = 400.00
Concrete Elasticity, Ec = 21019.039
Steel Elasticity, Es = 200000.00
##
Note: Especially for the calculation of θy 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, fc = fcm = 33.00
New material: Steel Strength, fs = fsm = 555.56
Existing Column
Existing material: Concrete Strength, fc = fcm = 20.00
Existing material: Steel Strength, fs = fsm = 444.44
###
External Diameter, D = 500.00
Internal Diameter, D = 300.00
Cover Thickness, c = 25.00
Element Length, L = 3000.00
Secondary Member
Ribbed Bars
Ductile Steel
Without Detailing for Earthquake Resistance (including stirrups not closed at 135°)
Longitudinal Bars With Ends Lapped Starting at the End Sections
Inadequate Lap Length with lo/lou,min =lb/ld = 0.30
No FRP Wrapping

Stepwise Properties

EDGE -A-
Bending Moment, Ma = -2.1652E+007
Shear Force, Va = -7215.465
EDGE -B-
Bending Moment, Mb = 0.00749595
Shear Force, Vb = 7215.465
BOTH EDGES
Axial Force, F = -7422.971
Longitudinal Reinforcement Area Distribution (in 2 divisions)
-Tension: Aslt = 1272.345
-Compression: Aslc = 1781.283
Longitudinal Reinforcement Area Distribution (in 3 divisions)
-Tension: Asl,ten = 1017.876
-Compression: Asl,com = 1017.876
-Middle: Asl,mid = 1017.876
Mean Diameter of Tension Reinforcement, DbL,ten = 18.00

Existing component: From table 7-7, ASCE 41_17: Final Shear Capacity VR = κ*Vn = 339071.802
Vn ((10.3), ASCE 41-17) = knl*VCol0 = 339071.802
VCol = 339071.802
knl = 1.00
displacement_ductility_demand = 0.02909096

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

λ = 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 = 4.00
Mu = 2.1652E+007
Vu = 7215.465
d = 0.8*D = 400.00
Nu = 7422.971
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 = π/2*A_stirrup = 123370.055
fy = 500.00
s = 100.00

Vs1 is multiplied by αCol1 = 1.00
s/d = 0.25
Vs2 = 0.00 is calculated for core, with:
Av = π/2*A_stirrup = 78956.835
fy = 400.00
s = 250.00
Vs2 is multiplied by αCol2 = 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 = π*d*d/4 = 125663.706

----
displacement_ductility_demand is calculated as θ/θy

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

From analysis, chord rotation θ = 0.00024711
θy = (My*Ls/3)/Eleff = 0.00849448 ((4.29),Biskinis Phd))
My = 2.0498E+008
Ls = M/V (with Ls >0.1*L and Ls < 2*L) = 3000.758
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 = 7422.971
Ec*Ig = Ec_jacket*Ig_jacket + Ec_core*Ig_core = 8.0455E+013

Calculation of Yielding Moment My

Calculation of Φy and My according to (7) - (8) in Biskinis and Fardis
----
My = Min(My_ten,My_com) = 2.0498E+008
Φy = 5.8526403E-006
My_ten (8c) = 2.0498E+008
Φ_ten (7c) = 64.04196
error of function (7c) = 8.3473441E-005
My_com (8d) = 7.5621E+008
Φ_com (7d) = 64.56829
error of function (7d) = -0.00721829
with ((10.1), ASCE 41-17) ey = Min(ey, 1.25*ey*(lb/ld)^2/3) = 0.0027778
eco = 0.002
apl = 0.35 ((9a) in Biskinis and Fardis for no FRP Wrap)
d1 = 44.00
R = 250.00
v = 0.0011456
N = 7422.971
Ac = 196349.541
((10.1), ASCE 41-17) ω = Min(ω, 1.25*ω*(lb/ld)^2/3) = 0.26182028
with fc = 33.00

Calculation of ratio lb/ld

Inadequate Lap Length with lb/ld = 0.30

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

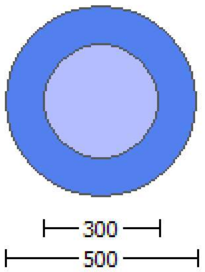
Limit State: Life Safety (data interpolation between analysis steps 1 and 2)

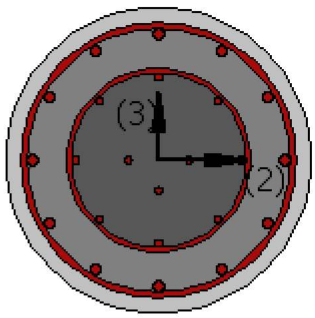
Analysis: Uniform +X

Check: Chord rotation capacity (θu)

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, κ = 1.00
Mean strength values are used for both shear and moment calculations.
Consequently:
Jacket
New material of Secondary Member: Concrete Strength, fc = fcm = 33.00
New material of Secondary Member: Steel Strength, fs = fsm = 555.56
Concrete Elasticity, Ec = 26999.444
Steel Elasticity, Es = 200000.00
Existing Column
Existing material of Secondary Member: Concrete Strength, fc = fcm = 20.00
Existing material of Secondary Member: Steel Strength, fs = fsm = 444.44
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 = 694.45
Existing Column
Existing material: Steel Strength, fs = 1.25*fsm = 555.55
##
External Diameter, D = 500.00
Internal Diameter, D = 300.00

Cover Thickness, c = 25.00
Mean Confinement Factor overall section = 1.00
Element Length, L = 3000.00
Secondary Member
Ribbed Bars
Ductile Steel
Without Detailing for Earthquake Resistance (including stirrups not closed at 135°)
Longitudinal Bars With Ends Lapped Starting at the End Sections
Inadequate Lap Length with lo/lou,min = 0.30
No FRP Wrapping

Stepwise Properties

At local axis: 3
EDGE -A-
Shear Force, Va = -7.1742060E-031
EDGE -B-
Shear Force, Vb = 7.1742060E-031
BOTH EDGES
Axial Force, F = -7425.858
Longitudinal Reinforcement Area Distribution (in 2 divisions)
-Tension: Aslt = 0.00
-Compression: Aslc = 3053.628
Longitudinal Reinforcement Area Distribution (in 3 divisions)
-Tension: Asl,ten = 1017.876
-Compression: Asl,com = 1017.876
-Middle: Asl,mid = 1017.876

Calculation of Shear Capacity ratio , Ve/Vr = 0.32864977
Member Controlled by Flexure (Ve/Vr < 1)
Calculation of Shear Demand from fig. R18.6.5, ACI 318-14 Ve = (Mpr1 + Mpr2)/ln = 159269.81
with
Mpr1 = Max(Mu1+ , Mu1-) = 2.3890E+008
Mu1+ = 2.3890E+008, is the ultimate moment strength at the edge 1 of the member in the actual moment direction which is defined for the static loading combination
Mu1- = 2.3890E+008, is the ultimate moment strength at the edge 1 of the member in the opposite moment direction which is defined for the static loading combination
Mpr2 = Max(Mu2+ , Mu2-) = 2.3890E+008
Mu2+ = 2.3890E+008, is the ultimate moment strength at the edge 2 of the member in the actual moment direction which is defined for the the static loading combination
Mu2- = 2.3890E+008, is the ultimate moment strength at the edge 2 of the member in the opposite moment direction which is defined for the the static loading combination

----
Calculation of Mu1+

Calculation of ultimate Moment Strength ((3.67), Biskinis Phd), Mu
Mu = 2.3890E+008

φ = 0.90757121
φ' = 0.80580716
error of function (3.68), Biskinis Phd = 59442.345
From 5A.2, TBDY: fcc = fc*λc = 33.00
conf. factor λc = 1.00
fc = 33.00
From 10.3.5, ASCE 41-17, Final value of fy: fy*Min(1,1.25*(lb/ld)^2/3) = 389.0139
lb/ld = 0.30
d1 = 44.00
R = 250.00
v = 0.0011456
N = 7425.858

Ac = 196349.541
ω = ω*Min(1,1.25*(lb/ld)^2/3) = 0.14666533

Calculation of ratio lb/ld

Inadequate Lap Length with lb/ld = 0.30

Calculation of Mu1-

Calculation of ultimate Moment Strength ((3.67), Biskinis Phd), Mu
Mu = 2.3890E+008

φ = 0.90757121
φ' = 0.80580716
error of function (3.68), Biskinis Phd = 59442.345
From 5A.2, TBDY: fcc = fc*λc = 33.00
conf. factor λc = 1.00
fc = 33.00
From 10.3.5, ASCE 41-17, Final value of fy: fy*Min(1,1.25*(lb/ld)^2/3) = 389.0139
lb/ld = 0.30
d1 = 44.00
R = 250.00
v = 0.0011456
N = 7425.858
Ac = 196349.541
ω = ω*Min(1,1.25*(lb/ld)^2/3) = 0.14666533

Calculation of ratio lb/ld

Inadequate Lap Length with lb/ld = 0.30

Calculation of Mu2+

Calculation of ultimate Moment Strength ((3.67), Biskinis Phd), Mu
Mu = 2.3890E+008

φ = 0.90757121
φ' = 0.80580716
error of function (3.68), Biskinis Phd = 59442.345
From 5A.2, TBDY: fcc = fc*λc = 33.00
conf. factor λc = 1.00
fc = 33.00
From 10.3.5, ASCE 41-17, Final value of fy: fy*Min(1,1.25*(lb/ld)^2/3) = 389.0139
lb/ld = 0.30
d1 = 44.00
R = 250.00
v = 0.0011456
N = 7425.858
Ac = 196349.541
ω = ω*Min(1,1.25*(lb/ld)^2/3) = 0.14666533

Calculation of ratio lb/ld

Inadequate Lap Length with lb/ld = 0.30

Calculation of Mu2-

Calculation of ultimate Moment Strength ((3.67), Biskinis Phd), Mu
Mu = 2.3890E+008

φ = 0.90757121
φ' = 0.80580716
error of function (3.68), Biskinis Phd = 59442.345
From 5A.2, TBDY: fcc = fc*λc = 33.00
conf. factor λc = 1.00
fc = 33.00
From 10.3.5, ASCE 41-17, Final value of fy: fy*Min(1,1.25*(lb/ld)^2/3) = 389.0139
lb/ld = 0.30
d1 = 44.00
R = 250.00
v = 0.0011456
N = 7425.858
Ac = 196349.541
ω = ω*Min(1,1.25*(lb/ld)^2/3) = 0.14666533

Calculation of ratio lb/ld

Inadequate Lap Length with lb/ld = 0.30

Calculation of Shear Strength Vr = Min(Vr1,Vr2) = 484618.662

Calculation of Shear Strength at edge 1, Vr1 = 484618.662
Vr1 = VCol ((10.3), ASCE 41-17) = knl*VCol0
VCol0 = 484618.662
knl = 1 (zero step-static loading)

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

λ = 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 = 1.3305601E-011
Vu = 7.1742060E-031
d = 0.8*D = 400.00
Nu = 7425.858
Ag = 196349.541
From (11.5.4.8), ACI 318-14: Vs = Vs1 + Vs2 = 274157.871
Vs1 = 274157.871 is calculated for jacket, with:
Av = π/2*A_stirrup = 123370.055
fy = 555.56
s = 100.00
Vs1 is multiplied by αCol1 = 1.00
s/d = 0.25
Vs2 = 0.00 is calculated for core, with:
Av = π/2*A_stirrup = 78956.835
fy = 444.44
s = 250.00
Vs2 is multiplied by αCol2 = 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 = π*d*d/4 = 125663.706

Calculation of Shear Strength at edge 2, Vr2 = 484618.662
Vr2 = VCol ((10.3), ASCE 41-17) = knl*VCol0
VCol0 = 484618.662
knl = 1 (zero step-static loading)

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

λ = 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 = 1.3305601E-011
Vu = 7.1742060E-031
d = 0.8*D = 400.00
Nu = 7425.858
Ag = 196349.541
From (11.5.4.8), ACI 318-14: Vs = Vs1 + Vs2 = 274157.871
Vs1 = 274157.871 is calculated for jacket, with:
Av = π/2*A_stirrup = 123370.055
fy = 555.56
s = 100.00
Vs1 is multiplied by αCol1 = 1.00
s/d = 0.25
Vs2 = 0.00 is calculated for core, with:
Av = π/2*A_stirrup = 78956.835
fy = 444.44
s = 250.00
Vs2 is multiplied by αCol2 = 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 = π*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, κ = 1.00
Mean strength values are used for both shear and moment calculations.
Consequently:
Jacket
New material of Secondary Member: Concrete Strength, fc = fcm = 33.00
New material of Secondary Member: Steel Strength, fs = fsm = 555.56
Concrete Elasticity, Ec = 26999.444
Steel Elasticity, Es = 200000.00
Existing Column
Existing material of Secondary Member: Concrete Strength, fc = fcm = 20.00
Existing material of Secondary Member: Steel Strength, fs = fsm = 444.44
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 = 694.45

Existing Column
Existing material: Steel Strength, fs = 1.25*fsm = 555.55
##
External Diameter, D = 500.00
Internal Diameter, D = 300.00
Cover Thickness, c = 25.00
Mean Confinement Factor overall section = 1.00
Element Length, L = 3000.00
Secondary Member
Ribbed Bars
Ductile Steel
Without Detailing for Earthquake Resistance (including stirrups not closed at 135°)
Longitudinal Bars With Ends Lapped Starting at the End Sections
Inadequate Lap Length with lo/lou,min = 0.30
No FRP Wrapping

Stepwise Properties

At local axis: 2
EDGE -A-
Shear Force, Va = -3.8672673E-031
EDGE -B-
Shear Force, Vb = 3.8672673E-031
BOTH EDGES
Axial Force, F = -7425.858
Longitudinal Reinforcement Area Distribution (in 2 divisions)
-Tension: Aslt = 0.00
-Compression: Aslc = 3053.628
Longitudinal Reinforcement Area Distribution (in 3 divisions)
-Tension: Asl,ten = 1017.876
-Compression: Asl,com = 1017.876
-Middle: Asl,mid = 1017.876

Calculation of Shear Capacity ratio , Ve/Vr = 0.32864977
Member Controlled by Flexure (Ve/Vr < 1)
Calculation of Shear Demand from fig. R18.6.5, ACI 318-14 Ve = (Mpr1 + Mpr2)/ln = 159269.81
with
Mpr1 = Max(Mu1+ , Mu1-) = 2.3890E+008
Mu1+ = 2.3890E+008, is the ultimate moment strength at the edge 1 of the member in the actual moment direction which is defined for the static loading combination
Mu1- = 2.3890E+008, is the ultimate moment strength at the edge 1 of the member in the opposite moment direction which is defined for the static loading combination
Mpr2 = Max(Mu2+ , Mu2-) = 2.3890E+008
Mu2+ = 2.3890E+008, is the ultimate moment strength at the edge 2 of the member in the actual moment direction which is defined for the the static loading combination
Mu2- = 2.3890E+008, is the ultimate moment strength at the edge 2 of the member in the opposite moment direction which is defined for the the static loading combination

----
Calculation of Mu1+

Calculation of ultimate Moment Strength ((3.67), Biskinis Phd), Mu
Mu = 2.3890E+008

φ = 0.90757121
φ' = 0.80580716
error of function (3.68), Biskinis Phd = 59442.345
From 5A.2, TBDY: fcc = fc*λc = 33.00
conf. factor λc = 1.00
fc = 33.00
From 10.3.5, ASCE 41-17, Final value of fy: fy*Min(1,1.25*(lb/ld)^2/3) = 389.0139

lb/ld = 0.30
d1 = 44.00
R = 250.00
v = 0.0011456
N = 7425.858
Ac = 196349.541
ω = ω*Min(1,1.25*(lb/ld)^2/3) = 0.14666533

Calculation of ratio lb/ld

Inadequate Lap Length with lb/ld = 0.30

Calculation of Mu1-

Calculation of ultimate Moment Strength ((3.67), Biskinis Phd), Mu
Mu = 2.3890E+008

φ = 0.90757121
φ' = 0.80580716
error of function (3.68), Biskinis Phd = 59442.345
From 5A.2, TBDY: fcc = fc*λc = 33.00
conf. factor λc = 1.00
fc = 33.00
From 10.3.5, ASCE 41-17, Final value of fy: fy*Min(1,1.25*(lb/ld)^2/3) = 389.0139
lb/ld = 0.30
d1 = 44.00
R = 250.00
v = 0.0011456
N = 7425.858
Ac = 196349.541
ω = ω*Min(1,1.25*(lb/ld)^2/3) = 0.14666533

Calculation of ratio lb/ld

Inadequate Lap Length with lb/ld = 0.30

Calculation of Mu2+

Calculation of ultimate Moment Strength ((3.67), Biskinis Phd), Mu
Mu = 2.3890E+008

φ = 0.90757121
φ' = 0.80580716
error of function (3.68), Biskinis Phd = 59442.345
From 5A.2, TBDY: fcc = fc*λc = 33.00
conf. factor λc = 1.00
fc = 33.00
From 10.3.5, ASCE 41-17, Final value of fy: fy*Min(1,1.25*(lb/ld)^2/3) = 389.0139
lb/ld = 0.30
d1 = 44.00
R = 250.00
v = 0.0011456
N = 7425.858
Ac = 196349.541
ω = ω*Min(1,1.25*(lb/ld)^2/3) = 0.14666533

Calculation of ratio lb/ld

Inadequate Lap Length with lb/ld = 0.30

Calculation of Mu2-

Calculation of ultimate Moment Strength ((3.67), Biskinis Phd), Mu
Mu = 2.3890E+008

φ = 0.90757121
φ' = 0.80580716
error of function (3.68), Biskinis Phd = 59442.345
From 5A.2, TBDY: fcc = fc*λc = 33.00
conf. factor λc = 1.00
fc = 33.00
From 10.3.5, ASCE 41-17, Final value of fy: fy*Min(1,1.25*(lb/ld)^2/3) = 389.0139
lb/ld = 0.30
d1 = 44.00
R = 250.00
v = 0.0011456
N = 7425.858
Ac = 196349.541
ω = ω*Min(1,1.25*(lb/ld)^2/3) = 0.14666533

Calculation of ratio lb/ld

Inadequate Lap Length with lb/ld = 0.30

Calculation of Shear Strength Vr = Min(Vr1,Vr2) = 484618.662

Calculation of Shear Strength at edge 1, Vr1 = 484618.662
Vr1 = VCol ((10.3), ASCE 41-17) = knl*VCol0
VCol0 = 484618.662
knl = 1 (zero step-static loading)

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

λ = 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 = 2.2610821E-011
Vu = 3.8672673E-031
d = 0.8*D = 400.00
Nu = 7425.858
Ag = 196349.541
From (11.5.4.8), ACI 318-14: Vs = Vs1 + Vs2 = 274157.871
Vs1 = 274157.871 is calculated for jacket, with:
Av = π/2*A_stirrup = 123370.055
fy = 555.56
s = 100.00
Vs1 is multiplied by αCol1 = 1.00
s/d = 0.25
Vs2 = 0.00 is calculated for core, with:
Av = π/2*A_stirrup = 78956.835
fy = 444.44
s = 250.00
Vs2 is multiplied by αCol2 = 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 = π*d*d/4 = 125663.706

Calculation of Shear Strength at edge 2, Vr2 = 484618.662
Vr2 = VCol ((10.3), ASCE 41-17) = knl*VCol0
VCol0 = 484618.662
knl = 1 (zero step-static loading)

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

λ = 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 = 2.2610821E-011
Vu = 3.8672673E-031
d = 0.8*D = 400.00
Nu = 7425.858
Ag = 196349.541
From (11.5.4.8), ACI 318-14: Vs = Vs1 + Vs2 = 274157.871
Vs1 = 274157.871 is calculated for jacket, with:
Av = π/2*A_stirrup = 123370.055
fy = 555.56
s = 100.00
Vs1 is multiplied by αCol1 = 1.00
s/d = 0.25
Vs2 = 0.00 is calculated for core, with:
Av = π/2*A_stirrup = 78956.835
fy = 444.44
s = 250.00
Vs2 is multiplied by αCol2 = 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 = π*d*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, κ = 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 Secondary Member: Concrete Strength, fc = fcm = 33.00
New material of Secondary Member: Steel Strength, fs = fsm = 555.56
Concrete Elasticity, Ec = 26999.444
Steel Elasticity, Es = 200000.00
Existing Column
Existing material of Secondary Member: Concrete Strength, fc = fcm = 20.00
Existing material of Secondary Member: Steel Strength, fs = fsm = 444.44
Concrete Elasticity, Ec = 21019.039

Steel Elasticity, Es = 200000.00
External Diameter, D = 500.00
Internal Diameter, D = 300.00
Cover Thickness, c = 25.00
Element Length, L = 3000.00
Secondary Member
Ribbed Bars
Ductile Steel
Without Detailing for Earthquake Resistance (including stirrups not closed at 135°)
Longitudinal Bars With Ends Lapped Starting at the End Sections
Inadequate Lap Length with lb/ld = 0.30
No FRP Wrapping

Stepwise Properties

Bending Moment, M = 2.4186131E-010
Shear Force, V2 = -7215.465
Shear Force, V3 = -1.2224225E-013
Axial Force, F = -7422.971
Longitudinal Reinforcement Area Distribution (in 2 divisions)
-Tension: Aslt = 1272.345
-Compression: Aslc = 1781.283
Longitudinal Reinforcement Area Distribution (in 3 divisions)
-Tension: Asl,ten = 1017.876
-Compression: Asl,com = 1017.876
-Middle: Asl,mid = 1017.876
Mean Diameter of Tension Reinforcement, DbL = 18.00

Existing component: From table 7-7, ASCE 41_17: Final chord rotation Capacity θu,R = κ*θu = 0.02540186
θu = θy + θp = 0.02540186

- Calculation of θy -

θy = (My*Ls/3)/EIeff = 0.00424617 ((4.29),Biskinis Phd))
My = 2.0498E+008
Ls = M/V (with Ls >0.1*L and Ls < 2*L) = 1500.00
From table 10.5, ASCE 41_17: EIeff = 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 = 7422.971
Ec*Ig = Ec_jacket*Ig_jacket + Ec_core*Ig_core = 8.0455E+013

Calculation of Yielding Moment My

Calculation of Φy and My according to (7) - (8) in Biskinis and Fardis
----
My = Min(My_ten,My_com) = 2.0498E+008
Φy = 5.8526403E-006
My_ten (8c) = 2.0498E+008
Φ_ten (7c) = 64.04196
error of function (7c) = 8.3473441E-005
My_com (8d) = 7.5621E+008
Φ_com (7d) = 64.56829
error of function (7d) = -0.00721829
with ((10.1), ASCE 41-17) ey = Min(ey, 1.25*ey*(lb/ld)^2/3) = 0.0027778
eco = 0.002
apl = 0.35 ((9a) in Biskinis and Fardis for no FRP Wrap)
d1 = 44.00
R = 250.00

v = 0.0011456
N = 7422.971
Ac = 196349.541
((10.1), ASCE 41-17) ω = Min(ω, 1.25*ω*(lb/ld)^2/3) = 0.26182028
with fc = 33.00

Calculation of ratio lb/ld

Inadequate Lap Length with lb/ld = 0.30

- Calculation of θp -

From table 10-9: θp = 0.02115569
with:
- Columns controlled by inadequate development or splicing along the clear height because lb/ld < 1
shear control ratio VyE/VCol0E = 0.32864977
d = d_external = 0.00
s = s_external = 0.00
ρt = ρs1 + ρs2 + 2*tf/bw*(ffe/fs) = 0.00323428
jacket: ρs1 = Av1*(π*Dc1/2)/(s1*Ag) = 0.0027646
Av1 = 78.53982, is the area of stirrup
Dc1 = Dext - 2*cover - External Hoop Diameter = 440.00, is the total Length of all stirrups parallel to loading (shear) direction
s1 = 100.00
core: ρs2 = Av2*(π*Dc2/2)/(s2*Ag) = 0.00046968
Av2 = 50.26548, is the area of stirrup
Dc2 = Dint - Internal Hoop Diameter = 292.00, is the total Length of all stirrups parallel to loading (shear) direction
s2 = 250.00
The term 2*tf/bw*(ffe/fs) is implemented to account for FRP contribution
where ρf = 2*tf/bw is FRP ratio (EC8 - 3, A.4.4.3(6)) and ffe/fs normalises ρf to steel strength
All these variables have already been given in Shear control ratio calculation.
For the normalisation fs of jacket is used.
NUD = 7422.971
Ag = 196349.541
f'cE = (fc_jacket*Area_jacket+fc_core*Area_core)/section_area = 28.32
fylE = (fy_ext_Long_Reinf*Area_ext_Long_Reinf+fy_int_Long_Reinf*Area_int_Long_Reinf)/Area_Tot_Long_Rein = 2.1219958E-314
fytE = (fy_ext_Trans_Reinf*ρs1+fy_int_Trans_Reinf*ρs2)/(ρs1+ρs2) = 539.4232
pl = 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: (a)

Calculation No. 11

column C1, Floor 1

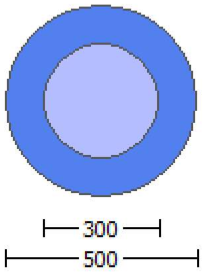
Limit State: Life Safety (data interpolation between analysis steps 1 and 2)

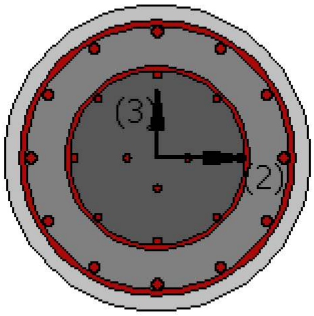
Analysis: Uniform +X

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, κ = 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 Secondary Member: Concrete Strength, fc = fc_lower_bound = 25.00
New material of Secondary Member: Steel Strength, fs = fs_lower_bound = 500.00
Concrete Elasticity, Ec = 26999.444
Steel Elasticity, Es = 200000.00
Existing Column
Existing material of Secondary Member: Concrete Strength, fc = fc_lower_bound = 16.00
Existing material of Secondary Member: Steel Strength, fs = fs_lower_bound = 400.00
Concrete Elasticity, Ec = 21019.039
Steel Elasticity, Es = 200000.00
###
Note: Especially for the calculation of θy 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, fc = fcm = 33.00
New material: Steel Strength, fs = fsm = 555.56
Existing Column
Existing material: Concrete Strength, fc = fcm = 20.00
Existing material: Steel Strength, fs = fsm = 444.44
###
External Diameter, D = 500.00
Internal Diameter, D = 300.00
Cover Thickness, c = 25.00
Element Length, L = 3000.00
Secondary Member
Ribbed Bars
Ductile Steel

Without Detailing for Earthquake Resistance (including stirrups not closed at 135°)
Longitudinal Bars With Ends Lapped Starting at the End Sections
Inadequate Lap Length with lo/lou,min =lb/ld = 0.30
No FRP Wrapping

Stepwise Properties

EDGE -A-
Bending Moment, Ma = 2.4186131E-010
Shear Force, Va = -1.2224225E-013
EDGE -B-
Bending Moment, Mb = 1.2482591E-010
Shear Force, Vb = 1.2224225E-013
BOTH EDGES
Axial Force, F = -7422.971
Longitudinal Reinforcement Area Distribution (in 2 divisions)
-Tension: Aslt = 1272.345
-Compression: Aslc = 1781.283
Longitudinal Reinforcement Area Distribution (in 3 divisions)
-Tension: Asl,ten = 1017.876
-Compression: Asl,com = 1017.876
-Middle: Asl,mid = 1017.876
Mean Diameter of Tension Reinforcement, DbL,ten = 18.00

Existing component: From table 7-7, ASCE 41_17: Final Shear Capacity VR = κ*Vn = 431403.493
Vn ((10.3), ASCE 41-17) = knl*VCol0 = 431403.493
VCol = 431403.493
knl = 1.00
displacement_ductility_demand = 0.00

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

λ = 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 = 2.4186131E-010
Vu = 1.2224225E-013
d = 0.8*D = 400.00
Nu = 7422.971
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 = π/2*A_stirrup = 123370.055
fy = 500.00
s = 100.00
Vs1 is multiplied by αCol1 = 1.00
s/d = 0.25
Vs2 = 0.00 is calculated for core, with:
Av = π/2*A_stirrup = 78956.835
fy = 400.00
s = 250.00
Vs2 is multiplied by αCol2 = 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 = π*d*d/4 = 125663.706

----
displacement_ductility_demand is calculated as θ/θy

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

From analysis, chord rotation θ = 1.7502059E-021
θy = (My*Ls/3)/EIeff = 0.00424617 ((4.29),Biskinis Phd))
My = 2.0498E+008
Ls = M/V (with Ls >0.1*L and Ls < 2*L) = 1500.00
From table 10.5, ASCE 41_17: EIeff = 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 = 7422.971
Ec*Ig = Ec_jacket*Ig_jacket + Ec_core*Ig_core = 8.0455E+013

Calculation of Yielding Moment My

Calculation of Φy and My according to (7) - (8) in Biskinis and Fardis
----
My = Min(My_ten,My_com) = 2.0498E+008
Φy = 5.8526403E-006
My_ten (8c) = 2.0498E+008
Φ_ten (7c) = 64.04196
error of function (7c) = 8.3473441E-005
My_com (8d) = 7.5621E+008
Φ_com (7d) = 64.56829
error of function (7d) = -0.00721829
with ((10.1), ASCE 41-17) ey = Min(ey, 1.25*ey*(lb/ld)^2/3) = 0.0027778
eco = 0.002
apl = 0.35 ((9a) in Biskinis and Fardis for no FRP Wrap)
d1 = 44.00
R = 250.00
v = 0.0011456
N = 7422.971
Ac = 196349.541
((10.1), ASCE 41-17) ω = Min(ω, 1.25*ω*(lb/ld)^2/3) = 0.26182028
with fc = 33.00

Calculation of ratio lb/ld

Inadequate Lap Length with lb/ld = 0.30

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

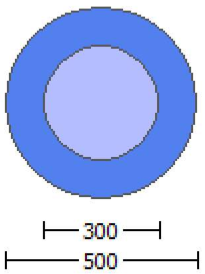
Limit State: Life Safety (data interpolation between analysis steps 1 and 2)

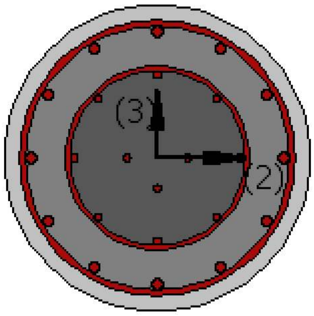
Analysis: Uniform +X

Check: Chord rotation capacity (θu)

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, κ = 1.00
Mean strength values are used for both shear and moment calculations.
Consequently:
Jacket
New material of Secondary Member: Concrete Strength, fc = fcm = 33.00
New material of Secondary Member: Steel Strength, fs = fsm = 555.56
Concrete Elasticity, Ec = 26999.444
Steel Elasticity, Es = 200000.00
Existing Column
Existing material of Secondary Member: Concrete Strength, fc = fcm = 20.00
Existing material of Secondary Member: Steel Strength, fs = fsm = 444.44
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 = 694.45
Existing Column
Existing material: Steel Strength, fs = 1.25*fsm = 555.55
###
External Diameter, D = 500.00
Internal Diameter, D = 300.00
Cover Thickness, c = 25.00
Mean Confinement Factor overall section = 1.00
Element Length, L = 3000.00
Secondary Member
Ribbed Bars
Ductile Steel
Without Detailing for Earthquake Resistance (including stirrups not closed at 135°)
Longitudinal Bars With Ends Lapped Starting at the End Sections
Inadequate Lap Length with lo/lou,min = 0.30

No FRP Wrapping

Stepwise Properties

At local axis: 3
EDGE -A-
Shear Force, Va = -7.1742060E-031
EDGE -B-
Shear Force, Vb = 7.1742060E-031
BOTH EDGES
Axial Force, F = -7425.858
Longitudinal Reinforcement Area Distribution (in 2 divisions)
-Tension: Aslt = 0.00
-Compression: Aslc = 3053.628
Longitudinal Reinforcement Area Distribution (in 3 divisions)
-Tension: Asl,ten = 1017.876
-Compression: Asl,com = 1017.876
-Middle: Asl,mid = 1017.876

Calculation of Shear Capacity ratio , Ve/Vr = 0.32864977
Member Controlled by Flexure (Ve/Vr < 1)
Calculation of Shear Demand from fig. R18.6.5, ACI 318-14 Ve = (Mpr1 + Mpr2)/ln = 159269.81
with
Mpr1 = Max(Mu1+ , Mu1-) = 2.3890E+008
Mu1+ = 2.3890E+008, is the ultimate moment strength at the edge 1 of the member in the actual moment direction which is defined for the static loading combination
Mu1- = 2.3890E+008, is the ultimate moment strength at the edge 1 of the member in the opposite moment direction which is defined for the static loading combination
Mpr2 = Max(Mu2+ , Mu2-) = 2.3890E+008
Mu2+ = 2.3890E+008, is the ultimate moment strength at the edge 2 of the member in the actual moment direction which is defined for the the static loading combination
Mu2- = 2.3890E+008, is the ultimate moment strength at the edge 2 of the member in the opposite moment direction which is defined for the the static loading combination

----
Calculation of Mu1+

Calculation of ultimate Moment Strength ((3.67), Biskinis Phd), Mu
Mu = 2.3890E+008

φ = 0.90757121
φ' = 0.80580716
error of function (3.68), Biskinis Phd = 59442.345
From 5A.2, TBDY: fcc = fc*λc = 33.00
conf. factor λc = 1.00
fc = 33.00
From 10.3.5, ASCE 41-17, Final value of fy: fy*Min(1,1.25*(lb/ld)^2/3) = 389.0139
lb/ld = 0.30
d1 = 44.00
R = 250.00
v = 0.0011456
N = 7425.858
Ac = 196349.541
ω = ω*Min(1,1.25*(lb/ld)^2/3) = 0.14666533

Calculation of ratio lb/ld

Inadequate Lap Length with lb/ld = 0.30

Calculation of Mu1-

Calculation of ultimate Moment Strength ((3.67), Biskinis Phd), Mu
Mu = 2.3890E+008

φ = 0.90757121
φ' = 0.80580716
error of function (3.68), Biskinis Phd = 59442.345
From 5A.2, TBDY: fcc = fc*λc = 33.00
conf. factor λc = 1.00
fc = 33.00
From 10.3.5, ASCE 41-17, Final value of fy: fy*Min(1,1.25*(lb/ld)^2/3) = 389.0139
lb/ld = 0.30
d1 = 44.00
R = 250.00
v = 0.0011456
N = 7425.858
Ac = 196349.541
ω = ω*Min(1,1.25*(lb/ld)^2/3) = 0.14666533

Calculation of ratio lb/ld

Inadequate Lap Length with lb/ld = 0.30

Calculation of Mu2+

Calculation of ultimate Moment Strength ((3.67), Biskinis Phd), Mu
Mu = 2.3890E+008

φ = 0.90757121
φ' = 0.80580716
error of function (3.68), Biskinis Phd = 59442.345
From 5A.2, TBDY: fcc = fc*λc = 33.00
conf. factor λc = 1.00
fc = 33.00
From 10.3.5, ASCE 41-17, Final value of fy: fy*Min(1,1.25*(lb/ld)^2/3) = 389.0139
lb/ld = 0.30
d1 = 44.00
R = 250.00
v = 0.0011456
N = 7425.858
Ac = 196349.541
ω = ω*Min(1,1.25*(lb/ld)^2/3) = 0.14666533

Calculation of ratio lb/ld

Inadequate Lap Length with lb/ld = 0.30

Calculation of Mu2-

Calculation of ultimate Moment Strength ((3.67), Biskinis Phd), Mu
Mu = 2.3890E+008

φ = 0.90757121

φ' = 0.80580716
error of function (3.68), Biskinis Phd = 59442.345
From 5A.2, TBDY: fcc = fc*λc = 33.00
conf. factor λc = 1.00
fc = 33.00
From 10.3.5, ASCE 41-17, Final value of fy: fy*Min(1,1.25*(lb/ld)^2/3) = 389.0139
lb/ld = 0.30
d1 = 44.00
R = 250.00
v = 0.0011456
N = 7425.858
Ac = 196349.541
ω = ω*Min(1,1.25*(lb/ld)^2/3) = 0.14666533

Calculation of ratio lb/ld

Inadequate Lap Length with lb/ld = 0.30

Calculation of Shear Strength Vr = Min(Vr1,Vr2) = 484618.662

Calculation of Shear Strength at edge 1, Vr1 = 484618.662
Vr1 = VCol ((10.3), ASCE 41-17) = knl*VCol0
VCol0 = 484618.662
knl = 1 (zero step-static loading)

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

λ = 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 = 1.3305601E-011
Vu = 7.1742060E-031
d = 0.8*D = 400.00
Nu = 7425.858
Ag = 196349.541
From (11.5.4.8), ACI 318-14: Vs = Vs1 + Vs2 = 274157.871
Vs1 = 274157.871 is calculated for jacket, with:
Av = π/2*A_stirrup = 123370.055
fy = 555.56
s = 100.00
Vs1 is multiplied by αCol1 = 1.00
s/d = 0.25
Vs2 = 0.00 is calculated for core, with:
Av = π/2*A_stirrup = 78956.835
fy = 444.44
s = 250.00
Vs2 is multiplied by αCol2 = 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 = π*d*d/4 = 125663.706

Calculation of Shear Strength at edge 2, Vr2 = 484618.662
Vr2 = VCol ((10.3), ASCE 41-17) = knl*VCol0
VCol0 = 484618.662
knl = 1 (zero step-static loading)

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

λ = 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 = 1.3305601E-011
Vu = 7.1742060E-031
d = 0.8*D = 400.00
Nu = 7425.858
Ag = 196349.541
From (11.5.4.8), ACI 318-14: Vs = Vs1 + Vs2 = 274157.871
Vs1 = 274157.871 is calculated for jacket, with:
Av = π/2*A_stirrup = 123370.055
fy = 555.56
s = 100.00
Vs1 is multiplied by αCol1 = 1.00
s/d = 0.25
Vs2 = 0.00 is calculated for core, with:
Av = π/2*A_stirrup = 78956.835
fy = 444.44
s = 250.00
Vs2 is multiplied by αCol2 = 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 = π*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, κ = 1.00
Mean strength values are used for both shear and moment calculations.
Consequently:
Jacket
New material of Secondary Member: Concrete Strength, fc = fcm = 33.00
New material of Secondary Member: Steel Strength, fs = fsm = 555.56
Concrete Elasticity, Ec = 26999.444
Steel Elasticity, Es = 200000.00
Existing Column
Existing material of Secondary Member: Concrete Strength, fc = fcm = 20.00
Existing material of Secondary Member: Steel Strength, fs = fsm = 444.44
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 = 694.45
Existing Column
Existing material: Steel Strength, fs = 1.25*fsm = 555.55
###
External Diameter, D = 500.00
Internal Diameter, D = 300.00
Cover Thickness, c = 25.00
Mean Confinement Factor overall section = 1.00
Element Length, L = 3000.00
Secondary Member

Ribbed Bars
Ductile Steel
Without Detailing for Earthquake Resistance (including stirrups not closed at 135°)
Longitudinal Bars With Ends Lapped Starting at the End Sections
Inadequate Lap Length with lo/lou,min = 0.30
No FRP Wrapping

Stepwise Properties

At local axis: 2
EDGE -A-
Shear Force, Va = -3.8672673E-031
EDGE -B-
Shear Force, Vb = 3.8672673E-031
BOTH EDGES
Axial Force, F = -7425.858
Longitudinal Reinforcement Area Distribution (in 2 divisions)
-Tension: Aslt = 0.00
-Compression: Aslc = 3053.628
Longitudinal Reinforcement Area Distribution (in 3 divisions)
-Tension: Asl,ten = 1017.876
-Compression: Asl,com = 1017.876
-Middle: Asl,mid = 1017.876

Calculation of Shear Capacity ratio , Ve/Vr = 0.32864977
Member Controlled by Flexure (Ve/Vr < 1)
Calculation of Shear Demand from fig. R18.6.5, ACI 318-14 Ve = (Mpr1 + Mpr2)/ln = 159269.81
with
Mpr1 = Max(Mu1+ , Mu1-) = 2.3890E+008
Mu1+ = 2.3890E+008, is the ultimate moment strength at the edge 1 of the member in the actual moment direction which is defined for the static loading combination
Mu1- = 2.3890E+008, is the ultimate moment strength at the edge 1 of the member in the opposite moment direction which is defined for the static loading combination
Mpr2 = Max(Mu2+ , Mu2-) = 2.3890E+008
Mu2+ = 2.3890E+008, is the ultimate moment strength at the edge 2 of the member in the actual moment direction which is defined for the the static loading combination
Mu2- = 2.3890E+008, is the ultimate moment strength at the edge 2 of the member in the opposite moment direction which is defined for the the static loading combination

----
Calculation of Mu1+

Calculation of ultimate Moment Strength ((3.67), Biskinis Phd), Mu
Mu = 2.3890E+008

φ = 0.90757121
φ' = 0.80580716
error of function (3.68), Biskinis Phd = 59442.345
From 5A.2, TBDY: fcc = fc*λc = 33.00
conf. factor λc = 1.00
fc = 33.00
From 10.3.5, ASCE 41-17, Final value of fy: fy*Min(1,1.25*(lb/ld)^2/3) = 389.0139
lb/ld = 0.30
d1 = 44.00
R = 250.00
v = 0.0011456
N = 7425.858
Ac = 196349.541
ω = ω*Min(1,1.25*(lb/ld)^2/3) = 0.14666533

Calculation of ratio lb/ld

Inadequate Lap Length with lb/ld = 0.30

Calculation of Mu1-

Calculation of ultimate Moment Strength ((3.67), Biskinis Phd), Mu
Mu = 2.3890E+008

φ = 0.90757121
φ' = 0.80580716
error of function (3.68), Biskinis Phd = 59442.345
From 5A.2, TBDY: fcc = fc*λc = 33.00
conf. factor λc = 1.00
fc = 33.00
From 10.3.5, ASCE 41-17, Final value of fy: fy*Min(1,1.25*(lb/ld)^2/3) = 389.0139
lb/ld = 0.30
d1 = 44.00
R = 250.00
v = 0.0011456
N = 7425.858
Ac = 196349.541
ω = ω*Min(1,1.25*(lb/ld)^2/3) = 0.14666533

Calculation of ratio lb/ld

Inadequate Lap Length with lb/ld = 0.30

Calculation of Mu2+

Calculation of ultimate Moment Strength ((3.67), Biskinis Phd), Mu
Mu = 2.3890E+008

φ = 0.90757121
φ' = 0.80580716
error of function (3.68), Biskinis Phd = 59442.345
From 5A.2, TBDY: fcc = fc*λc = 33.00
conf. factor λc = 1.00
fc = 33.00
From 10.3.5, ASCE 41-17, Final value of fy: fy*Min(1,1.25*(lb/ld)^2/3) = 389.0139
lb/ld = 0.30
d1 = 44.00
R = 250.00
v = 0.0011456
N = 7425.858
Ac = 196349.541
ω = ω*Min(1,1.25*(lb/ld)^2/3) = 0.14666533

Calculation of ratio lb/ld

Inadequate Lap Length with lb/ld = 0.30

Calculation of Mu2-

Calculation of ultimate Moment Strength ((3.67), Biskinis Phd), Mu
Mu = 2.3890E+008

φ = 0.90757121
φ' = 0.80580716
error of function (3.68), Biskinis Phd = 59442.345
From 5A.2, TBDY: fcc = fc*λc = 33.00
conf. factor λc = 1.00
fc = 33.00
From 10.3.5, ASCE 41-17, Final value of fy: fy*Min(1,1.25*(lb/ld)^2/3) = 389.0139
lb/ld = 0.30
d1 = 44.00
R = 250.00
v = 0.0011456
N = 7425.858
Ac = 196349.541
ω = ω*Min(1,1.25*(lb/ld)^2/3) = 0.14666533

Calculation of ratio lb/ld

Inadequate Lap Length with lb/ld = 0.30

Calculation of Shear Strength Vr = Min(Vr1,Vr2) = 484618.662

Calculation of Shear Strength at edge 1, Vr1 = 484618.662
Vr1 = VCol ((10.3), ASCE 41-17) = knl*VCol0
VCol0 = 484618.662
knl = 1 (zero step-static loading)

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

λ = 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 = 2.2610821E-011
Vu = 3.8672673E-031
d = 0.8*D = 400.00
Nu = 7425.858
Ag = 196349.541
From (11.5.4.8), ACI 318-14: Vs = Vs1 + Vs2 = 274157.871
Vs1 = 274157.871 is calculated for jacket, with:
Av = π/2*A_stirrup = 123370.055
fy = 555.56
s = 100.00
Vs1 is multiplied by αCol1 = 1.00
s/d = 0.25
Vs2 = 0.00 is calculated for core, with:
Av = π/2*A_stirrup = 78956.835
fy = 444.44
s = 250.00
Vs2 is multiplied by αCol2 = 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 = π*d*d/4 = 125663.706

Calculation of Shear Strength at edge 2, Vr2 = 484618.662
Vr2 = VCol ((10.3), ASCE 41-17) = knl*VCol0
VCol0 = 484618.662

knl = 1 (zero step-static loading)

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

λ = 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 = 2.2610821E-011
Vu = 3.8672673E-031
d = 0.8*D = 400.00
Nu = 7425.858
Ag = 196349.541
From (11.5.4.8), ACI 318-14: Vs = Vs1 + Vs2 = 274157.871
Vs1 = 274157.871 is calculated for jacket, with:
Av = π/2*A_stirrup = 123370.055
fy = 555.56
s = 100.00
Vs1 is multiplied by αCol1 = 1.00
s/d = 0.25
Vs2 = 0.00 is calculated for core, with:
Av = π/2*A_stirrup = 78956.835
fy = 444.44
s = 250.00
Vs2 is multiplied by αCol2 = 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 = π*d*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, κ = 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 Secondary Member: Concrete Strength, fc = fcm = 33.00
New material of Secondary Member: Steel Strength, fs = fsm = 555.56
Concrete Elasticity, Ec = 26999.444
Steel Elasticity, Es = 200000.00
Existing Column
Existing material of Secondary Member: Concrete Strength, fc = fcm = 20.00
Existing material of Secondary Member: Steel Strength, fs = fsm = 444.44
Concrete Elasticity, Ec = 21019.039
Steel Elasticity, Es = 200000.00
External Diameter, D = 500.00
Internal Diameter, D = 300.00
Cover Thickness, c = 25.00
Element Length, L = 3000.00
Secondary Member
Ribbed Bars
Ductile Steel
Without Detailing for Earthquake Resistance (including stirrups not closed at 135°)

Longitudinal Bars With Ends Lapped Starting at the End Sections
Inadequate Lap Length with lb/ld = 0.30
No FRP Wrapping

Stepwise Properties

Bending Moment, M = -2.1652E+007
Shear Force, V2 = -7215.465
Shear Force, V3 = -1.2224225E-013
Axial Force, F = -7422.971
Longitudinal Reinforcement Area Distribution (in 2 divisions)
-Tension: Aslt = 1272.345
-Compression: Aslc = 1781.283
Longitudinal Reinforcement Area Distribution (in 3 divisions)
-Tension: Asl,ten = 1017.876
-Compression: Asl,com = 1017.876
-Middle: Asl,mid = 1017.876
Mean Diameter of Tension Reinforcement, DbL = 18.00

Existing component: From table 7-7, ASCE 41_17: Final chord rotation Capacity θu,R = κ*θu = 0.02965018
θu = θy + θp = 0.02965018

- Calculation of θy -

θy = (My*Ls/3)/Eleff = 0.00849448 ((4.29),Biskinis Phd))
My = 2.0498E+008
Ls = M/V (with Ls >0.1*L and Ls < 2*L) = 3000.758
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 = 7422.971
Ec*Ig = Ec_jacket*Ig_jacket + Ec_core*Ig_core = 8.0455E+013

Calculation of Yielding Moment My

Calculation of Φy and My according to (7) - (8) in Biskinis and Fardis
----
My = Min(My_ten,My_com) = 2.0498E+008
Φy = 5.8526403E-006
My_ten (8c) = 2.0498E+008
Φ_ten (7c) = 64.04196
error of function (7c) = 8.3473441E-005
My_com (8d) = 7.5621E+008
Φ_com (7d) = 64.56829
error of function (7d) = -0.00721829
with ((10.1), ASCE 41-17) ey = Min(ey, 1.25*ey*(lb/ld)^2/3) = 0.0027778
eco = 0.002
apl = 0.35 ((9a) in Biskinis and Fardis for no FRP Wrap)
d1 = 44.00
R = 250.00
v = 0.0011456
N = 7422.971
Ac = 196349.541
((10.1), ASCE 41-17) ω = Min(ω, 1.25*ω*(lb/ld)^2/3) = 0.26182028
with fc = 33.00

Calculation of ratio lb/ld

Inadequate Lap Length with lb/ld = 0.30

 - Calculation of θp -

From table 10-9: θp = 0.02115569
with:
 - Columns controlled by inadequate development or splicing along the clear height because lb/ld < 1
   shear control ratio VyE/VCol0E = 0.32864977
   d = d_external = 0.00
   s = s_external = 0.00
   ρt = ρs1 + ρs2 + 2*tf/bw*(ffe/fs) = 0.00323428
   jacket: ρs1 = Av1*(π*Dc1/2)/(s1*Ag) = 0.0027646
          Av1 = 78.53982, is the area of stirrup
          Dc1 = Dext - 2*cover - External Hoop Diameter = 440.00, is the total Length of all stirrups parallel to loading (shear) direction
          s1 = 100.00
   core: ρs2 = Av2*(π*Dc2/2)/(s2*Ag) = 0.00046968
          Av2 = 50.26548, is the area of stirrup
          Dc2 = Dint - Internal Hoop Diameter = 292.00, is the total Length of all stirrups parallel to loading (shear) direction
          s2 = 250.00
   The term 2*tf/bw*(ffe/fs) is implemented to account for FRP contribution
   where ρf = 2*tf/bw is FRP ratio (EC8 - 3, A.4.4.3(6)) and ffe/fs normalises ρf to steel strength
   All these variables have already been given in Shear control ratio calculation.
   For the normalisation fs of jacket is used.
     NUD = 7422.971
     Ag = 196349.541
     f'cE = (fc_jacket*Area_jacket+fc_core*Area_core)/section_area = 28.32
     fylE = (fy_ext_Long_Reinf*Area_ext_Long_Reinf+fy_int_Long_Reinf*Area_int_Long_Reinf)/Area_Tot_Long_Rein = 2.1219958E-314
     fytE = (fy_ext_Trans_Reinf*ρs1+fy_int_Trans_Reinf*ρs2)/(ρs1+ρs2) = 539.4232
pl = 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: 3
Integration Section: (a)

Calculation No. 13

column C1, Floor 1

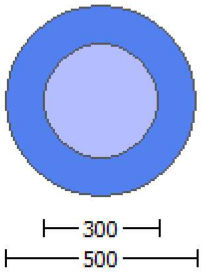
Limit State: Life Safety (data interpolation between analysis steps 1 and 2)

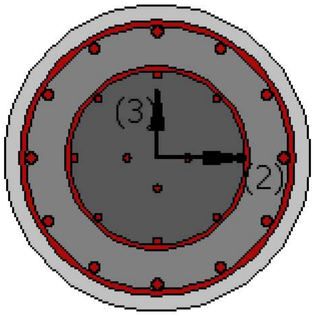
Analysis: Uniform +X

Check: Shear capacity VRd

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, κ = 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 Secondary Member: Concrete Strength, fc = fc_lower_bound = 25.00
New material of Secondary Member: Steel Strength, fs = fs_lower_bound = 500.00
Concrete Elasticity, Ec = 26999.444
Steel Elasticity, Es = 200000.00
Existing Column
Existing material of Secondary Member: Concrete Strength, fc = fc_lower_bound = 16.00
Existing material of Secondary Member: Steel Strength, fs = fs_lower_bound = 400.00
Concrete Elasticity, Ec = 21019.039
Steel Elasticity, Es = 200000.00
###
Note: Especially for the calculation of θy 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, fc = fcm = 33.00
New material: Steel Strength, fs = fsm = 555.56
Existing Column
Existing material: Concrete Strength, fc = fcm = 20.00
Existing material: Steel Strength, fs = fsm = 444.44
###
External Diameter, D = 500.00
Internal Diameter, D = 300.00
Cover Thickness, c = 25.00
Element Length, L = 3000.00
Secondary Member
Ribbed Bars
Ductile Steel
Without Detailing for Earthquake Resistance (including stirrups not closed at 135°)
Longitudinal Bars With Ends Lapped Starting at the End Sections
Inadequate Lap Length with lo/lou,min =lb/ld = 0.30
No FRP Wrapping

Stepwise Properties

EDGE -A-
Bending Moment, Ma = -2.1652E+007
Shear Force, Va = -7215.465
EDGE -B-
Bending Moment, Mb = 0.00749595

Shear Force, Vb = 7215.465
BOTH EDGES
Axial Force, F = -7422.971
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
Mean Diameter of Tension Reinforcement, DbL,ten = 18.00

Existing component: From table 7-7, ASCE 41_17: Final Shear Capacity VR = κ*Vn = 431403.493
Vn ((10.3), ASCE 41-17) = knl*VCol0 = 431403.493
VCol = 431403.493
knl = 1.00
displacement_ductility_demand = 0.15843052

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

λ = 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 = 0.00749595
Vu = 7215.465
d = 0.8*D = 400.00
Nu = 7422.971
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 = π/2*A_stirrup = 123370.055
fy = 500.00
s = 100.00
Vs1 is multiplied by αCol1 = 1.00
s/d = 0.25
Vs2 = 0.00 is calculated for core, with:
Av = π/2*A_stirrup = 78956.835
fy = 400.00
s = 250.00
Vs2 is multiplied by αCol2 = 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 = π*d*d/4 = 125663.706

----
displacement_ductility_demand is calculated as θ/θy

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

From analysis, chord rotation θ = 0.00013454
θy = (My*Ls/3)/EIeff = 0.00084923 ((4.29),Biskinis Phd))
My = 2.0498E+008
Ls = M/V (with Ls >0.1*L and Ls < 2*L) = 300.00
From table 10.5, ASCE 41_17: EIeff = 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 = 7422.971
Ec*Ig = Ec_jacket*Ig_jacket + Ec_core*Ig_core = 8.0455E+013

Calculation of Yielding Moment My

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

----

My = Min(My_ten,My_com) = 2.0498E+008
Φy = 5.8526403E-006
My_ten (8c) = 2.0498E+008
Φ_ten (7c) = 64.04196
error of function (7c) = 8.3473441E-005
My_com (8d) = 7.5621E+008
Φ_com (7d) = 64.56829
error of function (7d) = -0.00721829
with ((10.1), ASCE 41-17) ey = Min(ey, 1.25*ey*(lb/ld)^2/3) = 0.0027778
eco = 0.002
apl = 0.35 ((9a) in Biskinis and Fardis for no FRP Wrap)
d1 = 44.00
R = 250.00
v = 0.0011456
N = 7422.971
Ac = 196349.541
((10.1), ASCE 41-17) ω = Min(ω, 1.25*ω*(lb/ld)^2/3) = 0.26182028
with fc = 33.00

Calculation of ratio lb/ld

Inadequate Lap Length with lb/ld = 0.30

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

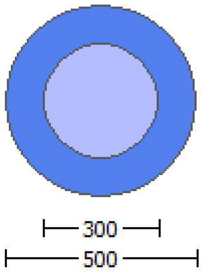
Limit State: Life Safety (data interpolation between analysis steps 1 and 2)

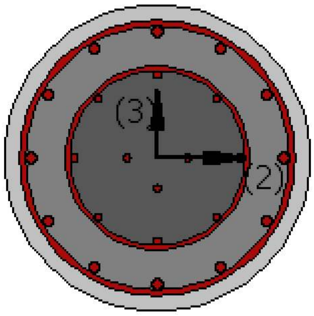
Analysis: Uniform +X

Check: Chord rotation capacity (θu)

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, κ = 1.00
Mean strength values are used for both shear and moment calculations.
Consequently:
Jacket
New material of Secondary Member: Concrete Strength, fc = fcm = 33.00
New material of Secondary Member: Steel Strength, fs = fsm = 555.56
Concrete Elasticity, Ec = 26999.444
Steel Elasticity, Es = 200000.00
Existing Column
Existing material of Secondary Member: Concrete Strength, fc = fcm = 20.00
Existing material of Secondary Member: Steel Strength, fs = fsm = 444.44
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 = 694.45
Existing Column
Existing material: Steel Strength, fs = 1.25*fsm = 555.55
##
External Diameter, D = 500.00
Internal Diameter, D = 300.00
Cover Thickness, c = 25.00
Mean Confinement Factor overall section = 1.00
Element Length, L = 3000.00
Secondary Member
Ribbed Bars
Ductile Steel
Without Detailing for Earthquake Resistance (including stirrups not closed at 135°)
Longitudinal Bars With Ends Lapped Starting at the End Sections
Inadequate Lap Length with lo/lou,min = 0.30
No FRP Wrapping

Stepwise Properties

At local axis: 3
EDGE -A-
Shear Force, Va = -7.1742060E-031
EDGE -B-
Shear Force, Vb = 7.1742060E-031
BOTH EDGES
Axial Force, F = -7425.858
Longitudinal Reinforcement Area Distribution (in 2 divisions)

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

Calculation of Shear Capacity ratio , Ve/Vr = 0.32864977
Member Controlled by Flexure (Ve/Vr < 1)
Calculation of Shear Demand from fig. R18.6.5, ACI 318-14 Ve = (Mpr1 + Mpr2)/ln = 159269.81
with
Mpr1 = Max(Mu1+ , Mu1-) = 2.3890E+008
Mu1+ = 2.3890E+008, is the ultimate moment strength at the edge 1 of the member in the actual moment direction which is defined for the static loading combination
Mu1- = 2.3890E+008, is the ultimate moment strength at the edge 1 of the member in the opposite moment direction which is defined for the static loading combination
Mpr2 = Max(Mu2+ , Mu2-) = 2.3890E+008
Mu2+ = 2.3890E+008, is the ultimate moment strength at the edge 2 of the member in the actual moment direction which is defined for the the static loading combination
Mu2- = 2.3890E+008, is the ultimate moment strength at the edge 2 of the member in the opposite moment direction which is defined for the the static loading combination

----
Calculation of Mu1+

Calculation of ultimate Moment Strength ((3.67), Biskinis Phd), Mu
Mu = 2.3890E+008

φ = 0.90757121
φ' = 0.80580716
error of function (3.68), Biskinis Phd = 59442.345
From 5A.2, TBDY: fcc = fc*λc = 33.00
conf. factor λc = 1.00
fc = 33.00
From 10.3.5, ASCE 41-17, Final value of fy: fy*Min(1,1.25*(lb/ld)^2/3) = 389.0139
lb/ld = 0.30
d1 = 44.00
R = 250.00
v = 0.0011456
N = 7425.858
Ac = 196349.541
ω = ω*Min(1,1.25*(lb/ld)^2/3) = 0.14666533

Calculation of ratio lb/ld

Inadequate Lap Length with lb/ld = 0.30

Calculation of Mu1-

Calculation of ultimate Moment Strength ((3.67), Biskinis Phd), Mu
Mu = 2.3890E+008

φ = 0.90757121
φ' = 0.80580716
error of function (3.68), Biskinis Phd = 59442.345
From 5A.2, TBDY: fcc = fc*λc = 33.00
conf. factor λc = 1.00

fc = 33.00
From 10.3.5, ASCE 41-17, Final value of fy: fy*Min(1,1.25*(lb/ld)^2/3) = 389.0139
lb/ld = 0.30
d1 = 44.00
R = 250.00
v = 0.0011456
N = 7425.858
Ac = 196349.541
ω = ω*Min(1,1.25*(lb/ld)^2/3) = 0.14666533

Calculation of ratio lb/ld

Inadequate Lap Length with lb/ld = 0.30

Calculation of Mu2+

Calculation of ultimate Moment Strength ((3.67), Biskinis Phd), Mu
Mu = 2.3890E+008

φ = 0.90757121
φ' = 0.80580716
error of function (3.68), Biskinis Phd = 59442.345
From 5A.2, TBDY: fcc = fc*λc = 33.00
conf. factor λc = 1.00
fc = 33.00
From 10.3.5, ASCE 41-17, Final value of fy: fy*Min(1,1.25*(lb/ld)^2/3) = 389.0139
lb/ld = 0.30
d1 = 44.00
R = 250.00
v = 0.0011456
N = 7425.858
Ac = 196349.541
ω = ω*Min(1,1.25*(lb/ld)^2/3) = 0.14666533

Calculation of ratio lb/ld

Inadequate Lap Length with lb/ld = 0.30

Calculation of Mu2-

Calculation of ultimate Moment Strength ((3.67), Biskinis Phd), Mu
Mu = 2.3890E+008

φ = 0.90757121
φ' = 0.80580716
error of function (3.68), Biskinis Phd = 59442.345
From 5A.2, TBDY: fcc = fc*λc = 33.00
conf. factor λc = 1.00
fc = 33.00
From 10.3.5, ASCE 41-17, Final value of fy: fy*Min(1,1.25*(lb/ld)^2/3) = 389.0139
lb/ld = 0.30
d1 = 44.00
R = 250.00
v = 0.0011456
N = 7425.858
Ac = 196349.541
ω = ω*Min(1,1.25*(lb/ld)^2/3) = 0.14666533

Calculation of ratio lb/ld

Inadequate Lap Length with lb/ld = 0.30

Calculation of Shear Strength Vr = Min(Vr1,Vr2) = 484618.662

Calculation of Shear Strength at edge 1, Vr1 = 484618.662
Vr1 = VCol ((10.3), ASCE 41-17) = knl*VCol0
VCol0 = 484618.662
knl = 1 (zero step-static loading)

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

λ = 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 = 1.3305601E-011
Vu = 7.1742060E-031
d = 0.8*D = 400.00
Nu = 7425.858
Ag = 196349.541
From (11.5.4.8), ACI 318-14: Vs = Vs1 + Vs2 = 274157.871
Vs1 = 274157.871 is calculated for jacket, with:
Av = π/2*A_stirrup = 123370.055
fy = 555.56
s = 100.00
Vs1 is multiplied by αCol1 = 1.00
s/d = 0.25
Vs2 = 0.00 is calculated for core, with:
Av = π/2*A_stirrup = 78956.835
fy = 444.44
s = 250.00
Vs2 is multiplied by αCol2 = 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 = π*d*d/4 = 125663.706

Calculation of Shear Strength at edge 2, Vr2 = 484618.662
Vr2 = VCol ((10.3), ASCE 41-17) = knl*VCol0
VCol0 = 484618.662
knl = 1 (zero step-static loading)

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

λ = 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 = 1.3305601E-011
Vu = 7.1742060E-031
d = 0.8*D = 400.00
Nu = 7425.858
Ag = 196349.541
From (11.5.4.8), ACI 318-14: Vs = Vs1 + Vs2 = 274157.871
Vs1 = 274157.871 is calculated for jacket, with:
Av = π/2*A_stirrup = 123370.055
fy = 555.56

s = 100.00
Vs1 is multiplied by αCol1 = 1.00
s/d = 0.25
Vs2 = 0.00 is calculated for core, with:
Av = π/2*A_stirrup = 78956.835
fy = 444.44
s = 250.00
Vs2 is multiplied by αCol2 = 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 = π*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, κ = 1.00
Mean strength values are used for both shear and moment calculations.
Consequently:
Jacket
New material of Secondary Member: Concrete Strength, fc = fcm = 33.00
New material of Secondary Member: Steel Strength, fs = fsm = 555.56
Concrete Elasticity, Ec = 26999.444
Steel Elasticity, Es = 200000.00
Existing Column
Existing material of Secondary Member: Concrete Strength, fc = fcm = 20.00
Existing material of Secondary Member: Steel Strength, fs = fsm = 444.44
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 = 694.45
Existing Column
Existing material: Steel Strength, fs = 1.25*fsm = 555.55
##
External Diameter, D = 500.00
Internal Diameter, D = 300.00
Cover Thickness, c = 25.00
Mean Confinement Factor overall section = 1.00
Element Length, L = 3000.00
Secondary Member
Ribbed Bars
Ductile Steel
Without Detailing for Earthquake Resistance (including stirrups not closed at 135°)
Longitudinal Bars With Ends Lapped Starting at the End Sections
Inadequate Lap Length with lo/lou,min = 0.30
No FRP Wrapping

Stepwise Properties

At local axis: 2
EDGE -A-
Shear Force, Va = -3.8672673E-031

EDGE -B-
Shear Force, Vb = 3.8672673E-031
BOTH EDGES
Axial Force, F = -7425.858
Longitudinal Reinforcement Area Distribution (in 2 divisions)
-Tension: Aslt = 0.00
-Compression: Aslc = 3053.628
Longitudinal Reinforcement Area Distribution (in 3 divisions)
-Tension: Asl,ten = 1017.876
-Compression: Asl,com = 1017.876
-Middle: Asl,mid = 1017.876

Calculation of Shear Capacity ratio , Ve/Vr = 0.32864977
Member Controlled by Flexure (Ve/Vr < 1)
Calculation of Shear Demand from fig. R18.6.5, ACI 318-14 Ve = (Mpr1 + Mpr2)/ln = 159269.81
with
Mpr1 = Max(Mu1+ , Mu1-) = 2.3890E+008
Mu1+ = 2.3890E+008, is the ultimate moment strength at the edge 1 of the member in the actual moment direction which is defined for the static loading combination
Mu1- = 2.3890E+008, is the ultimate moment strength at the edge 1 of the member in the opposite moment direction which is defined for the static loading combination
Mpr2 = Max(Mu2+ , Mu2-) = 2.3890E+008
Mu2+ = 2.3890E+008, is the ultimate moment strength at the edge 2 of the member in the actual moment direction which is defined for the the static loading combination
Mu2- = 2.3890E+008, is the ultimate moment strength at the edge 2 of the member in the opposite moment direction which is defined for the the static loading combination

----
Calculation of Mu1+

Calculation of ultimate Moment Strength ((3.67), Biskinis Phd), Mu
Mu = 2.3890E+008

φ = 0.90757121
φ' = 0.80580716
error of function (3.68), Biskinis Phd = 59442.345
From 5A.2, TBDY: fcc = fc*λc = 33.00
conf. factor λc = 1.00
fc = 33.00
From 10.3.5, ASCE 41-17, Final value of fy: fy*Min(1,1.25*(lb/ld)^2/3) = 389.0139
lb/ld = 0.30
d1 = 44.00
R = 250.00
v = 0.0011456
N = 7425.858
Ac = 196349.541
ω = ω*Min(1,1.25*(lb/ld)^2/3) = 0.14666533

Calculation of ratio lb/ld

Inadequate Lap Length with lb/ld = 0.30

Calculation of Mu1-

Calculation of ultimate Moment Strength ((3.67), Biskinis Phd), Mu
Mu = 2.3890E+008

φ = 0.90757121
φ' = 0.80580716
error of function (3.68), Biskinis Phd = 59442.345
From 5A.2, TBDY: fcc = fc*λc = 33.00
conf. factor λc = 1.00
fc = 33.00
From 10.3.5, ASCE 41-17, Final value of fy: fy*Min(1,1.25*(lb/ld)^2/3) = 389.0139
lb/ld = 0.30
d1 = 44.00
R = 250.00
v = 0.0011456
N = 7425.858
Ac = 196349.541
ω = ω*Min(1,1.25*(lb/ld)^2/3) = 0.14666533

Calculation of ratio lb/ld

Inadequate Lap Length with lb/ld = 0.30

Calculation of Mu2+

Calculation of ultimate Moment Strength ((3.67), Biskinis Phd), Mu
Mu = 2.3890E+008

φ = 0.90757121
φ' = 0.80580716
error of function (3.68), Biskinis Phd = 59442.345
From 5A.2, TBDY: fcc = fc*λc = 33.00
conf. factor λc = 1.00
fc = 33.00
From 10.3.5, ASCE 41-17, Final value of fy: fy*Min(1,1.25*(lb/ld)^2/3) = 389.0139
lb/ld = 0.30
d1 = 44.00
R = 250.00
v = 0.0011456
N = 7425.858
Ac = 196349.541
ω = ω*Min(1,1.25*(lb/ld)^2/3) = 0.14666533

Calculation of ratio lb/ld

Inadequate Lap Length with lb/ld = 0.30

Calculation of Mu2-

Calculation of ultimate Moment Strength ((3.67), Biskinis Phd), Mu
Mu = 2.3890E+008

φ = 0.90757121
φ' = 0.80580716
error of function (3.68), Biskinis Phd = 59442.345
From 5A.2, TBDY: fcc = fc*λc = 33.00
conf. factor λc = 1.00
fc = 33.00
From 10.3.5, ASCE 41-17, Final value of fy: fy*Min(1,1.25*(lb/ld)^2/3) = 389.0139
lb/ld = 0.30
d1 = 44.00

R = 250.00
v = 0.0011456
N = 7425.858
Ac = 196349.541
ω = ω*Min(1,1.25*(lb/ld)^2/3) = 0.14666533

Calculation of ratio lb/ld

Inadequate Lap Length with lb/ld = 0.30

Calculation of Shear Strength Vr = Min(Vr1,Vr2) = 484618.662

Calculation of Shear Strength at edge 1, Vr1 = 484618.662
Vr1 = VCol ((10.3), ASCE 41-17) = knl*VCol0
VCol0 = 484618.662
knl = 1 (zero step-static loading)

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

λ = 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 = 2.2610821E-011
Vu = 3.8672673E-031
d = 0.8*D = 400.00
Nu = 7425.858
Ag = 196349.541
From (11.5.4.8), ACI 318-14: Vs = Vs1 + Vs2 = 274157.871
Vs1 = 274157.871 is calculated for jacket, with:
Av = π/2*A_stirrup = 123370.055
fy = 555.56
s = 100.00
Vs1 is multiplied by αCol1 = 1.00
s/d = 0.25
Vs2 = 0.00 is calculated for core, with:
Av = π/2*A_stirrup = 78956.835
fy = 444.44
s = 250.00
Vs2 is multiplied by αCol2 = 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 = π*d*d/4 = 125663.706

Calculation of Shear Strength at edge 2, Vr2 = 484618.662
Vr2 = VCol ((10.3), ASCE 41-17) = knl*VCol0
VCol0 = 484618.662
knl = 1 (zero step-static loading)

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

λ = 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 = 2.2610821E-011
Vu = 3.8672673E-031
d = 0.8*D = 400.00
Nu = 7425.858

Ag = 196349.541
From (11.5.4.8), ACI 318-14: Vs = Vs1 + Vs2 = 274157.871
Vs1 = 274157.871 is calculated for jacket, with:
Av = π/2*A_stirrup = 123370.055
fy = 555.56
s = 100.00
Vs1 is multiplied by αCol1 = 1.00
s/d = 0.25
Vs2 = 0.00 is calculated for core, with:
Av = π/2*A_stirrup = 78956.835
fy = 444.44
s = 250.00
Vs2 is multiplied by αCol2 = 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 = π*d*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: (b)
Section Type: rcjcs

Constant Properties

Knowledge Factor, κ = 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 Secondary Member: Concrete Strength, fc = fcm = 33.00
New material of Secondary Member: Steel Strength, fs = fsm = 555.56
Concrete Elasticity, Ec = 26999.444
Steel Elasticity, Es = 200000.00
Existing Column
Existing material of Secondary Member: Concrete Strength, fc = fcm = 20.00
Existing material of Secondary Member: Steel Strength, fs = fsm = 444.44
Concrete Elasticity, Ec = 21019.039
Steel Elasticity, Es = 200000.00
External Diameter, D = 500.00
Internal Diameter, D = 300.00
Cover Thickness, c = 25.00
Element Length, L = 3000.00
Secondary Member
Ribbed Bars
Ductile Steel
Without Detailing for Earthquake Resistance (including stirrups not closed at 135°)
Longitudinal Bars With Ends Lapped Starting at the End Sections
Inadequate Lap Length with lb/ld = 0.30
No FRP Wrapping

Stepwise Properties

Bending Moment, M = 1.2482591E-010
Shear Force, V2 = 7215.465
Shear Force, V3 = 1.2224225E-013
Axial Force, F = -7422.971
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
Mean Diameter of Tension Reinforcement, DbL = 18.00

Existing component: From table 7-7, ASCE 41_17: Final chord rotation Capacity θu,R = κ*θu = 0.02540186
θu = θy + θp = 0.02540186

- Calculation of θy -

θy = (My*Ls/3)/EIeff = 0.00424617 ((4.29),Biskinis Phd))
My = 2.0498E+008
Ls = M/V (with Ls >0.1*L and Ls < 2*L) = 1500.00
From table 10.5, ASCE 41_17: EIeff = 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 = 7422.971
Ec*Ig = Ec_jacket*Ig_jacket + Ec_core*Ig_core = 8.0455E+013

Calculation of Yielding Moment My

Calculation of Φy and My according to (7) - (8) in Biskinis and Fardis
----
My = Min(My_ten,My_com) = 2.0498E+008
Φy = 5.8526403E-006
My_ten (8c) = 2.0498E+008
Φ_ten (7c) = 64.04196
error of function (7c) = 8.3473441E-005
My_com (8d) = 7.5621E+008
Φ_com (7d) = 64.56829
error of function (7d) = -0.00721829
with ((10.1), ASCE 41-17) ey = Min(ey, 1.25*ey*(lb/ld)^2/3) = 0.0027778
eco = 0.002
apl = 0.35 ((9a) in Biskinis and Fardis for no FRP Wrap)
d1 = 44.00
R = 250.00
v = 0.0011456
N = 7422.971
Ac = 196349.541
((10.1), ASCE 41-17) ω = Min(ω, 1.25*ω*(lb/ld)^2/3) = 0.26182028
with fc = 33.00

Calculation of ratio lb/ld

Inadequate Lap Length with lb/ld = 0.30

- Calculation of θp -

From table 10-9: θp = 0.02115569
with:
- Columns controlled by inadequate development or splicing along the clear height because lb/ld < 1
shear control ratio VyE/VCol0E = 0.32864977
d = d_external = 0.00
s = s_external = 0.00
ρt = ρs1 + ρs2 + 2*tf/bw*(ffe/fs) = 0.00323428

jacket: ρs1 = Av1*(π*Dc1/2)/(s1*Ag) = 0.0027646
Av1 = 78.53982, is the area of stirrup
Dc1 = Dext - 2*cover - External Hoop Diameter = 440.00, is the total Length of all stirrups parallel to loading (shear) direction
s1 = 100.00
core: ρs2 = Av2*(π*Dc2/2)/(s2*Ag) = 0.00046968
Av2 = 50.26548, is the area of stirrup
Dc2 = Dint - Internal Hoop Diameter = 292.00, is the total Length of all stirrups parallel to loading (shear) direction
s2 = 250.00
The term 2*tf/bw*(ffe/fs) is implemented to account for FRP contribution
where ρf = 2*tf/bw is FRP ratio (EC8 - 3, A.4.4.3(6)) and ffe/fs normalises ρf to steel strength
All these variables have already been given in Shear control ratio calculation.
For the normalisation fs of jacket is used.
NUD = 7422.971
Ag = 196349.541
f'cE = (fc_jacket*Area_jacket+fc_core*Area_core)/section_area = 28.32
fylE = (fy_ext_Long_Reinf*Area_ext_Long_Reinf+fy_int_Long_Reinf*Area_int_Long_Reinf)/Area_Tot_Long_Rein = 2.1219958E-314
fytE = (fy_ext_Trans_Reinf*ρs1+fy_int_Trans_Reinf*ρs2)/(ρs1+ρs2) = 539.4232
pl = 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. 15

column C1, Floor 1

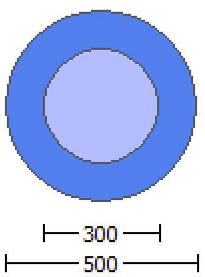
Limit State: Life Safety (data interpolation between analysis steps 1 and 2)

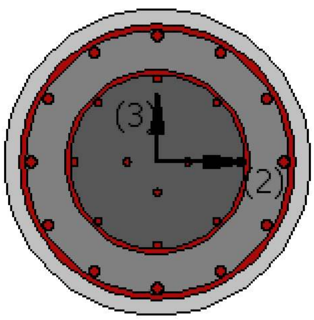
Analysis: Uniform +X

Check: Shear capacity VRd

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, κ = 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 Secondary Member: Concrete Strength, fc = fc_lower_bound = 25.00
New material of Secondary Member: Steel Strength, fs = fs_lower_bound = 500.00
Concrete Elasticity, Ec = 26999.444
Steel Elasticity, Es = 200000.00
Existing Column
Existing material of Secondary Member: Concrete Strength, fc = fc_lower_bound = 16.00
Existing material of Secondary Member: Steel Strength, fs = fs_lower_bound = 400.00
Concrete Elasticity, Ec = 21019.039
Steel Elasticity, Es = 200000.00
##
Note: Especially for the calculation of θy 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, fc = fcm = 33.00
New material: Steel Strength, fs = fsm = 555.56
Existing Column
Existing material: Concrete Strength, fc = fcm = 20.00
Existing material: Steel Strength, fs = fsm = 444.44
##
External Diameter, D = 500.00
Internal Diameter, D = 300.00
Cover Thickness, c = 25.00
Element Length, L = 3000.00
Secondary Member
Ribbed Bars
Ductile Steel
Without Detailing for Earthquake Resistance (including stirrups not closed at 135°)
Longitudinal Bars With Ends Lapped Starting at the End Sections
Inadequate Lap Length with lo/lou,min =lb/ld = 0.30
No FRP Wrapping

Stepwise Properties

EDGE -A-
Bending Moment, Ma = 2.4186131E-010
Shear Force, Va = -1.2224225E-013
EDGE -B-
Bending Moment, Mb = 1.2482591E-010
Shear Force, Vb = 1.2224225E-013
BOTH EDGES
Axial Force, F = -7422.971
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
Mean Diameter of Tension Reinforcement, DbL,ten = 18.00

Existing component: From table 7-7, ASCE 41_17: Final Shear Capacity VR = κ*Vn = 431403.493

Vn ((10.3), ASCE 41-17) = knl*VCol0 = 431403.493
VCol = 431403.493
knl = 1.00
displacement_ductility_demand = 0.00

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

λ = 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 = 1.2482591E-010
Vu = 1.2224225E-013
d = 0.8*D = 400.00
Nu = 7422.971
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 = π/2*A_stirrup = 123370.055
fy = 500.00
s = 100.00
Vs1 is multiplied by αCol1 = 1.00
s/d = 0.25
Vs2 = 0.00 is calculated for core, with:
Av = π/2*A_stirrup = 78956.835
fy = 400.00
s = 250.00
Vs2 is multiplied by αCol2 = 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 = π*d*d/4 = 125663.706

----
displacement_ductility_demand is calculated as θ/θy

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

From analysis, chord rotation θ = 1.2379356E-022
θy = (My*Ls/3)/EIeff = 0.00424617 ((4.29),Biskinis Phd))
My = 2.0498E+008
Ls = M/V (with Ls >0.1*L and Ls < 2*L) = 1500.00
From table 10.5, ASCE 41_17: EIeff = 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 = 7422.971
Ec*Ig = Ec_jacket*Ig_jacket + Ec_core*Ig_core = 8.0455E+013

Calculation of Yielding Moment My

Calculation of Φy and My according to (7) - (8) in Biskinis and Fardis
----
My = Min(My_ten,My_com) = 2.0498E+008
Φy = 5.8526403E-006
My_ten (8c) = 2.0498E+008
Φ_ten (7c) = 64.04196
error of function (7c) = 8.3473441E-005
My_com (8d) = 7.5621E+008
Φ_com (7d) = 64.56829
error of function (7d) = -0.00721829
with ((10.1), ASCE 41-17) ey = Min(ey, 1.25*ey*(lb/ld)^2/3) = 0.0027778

eco = 0.002
apl = 0.35 ((9a) in Biskinis and Fardis for no FRP Wrap)
d1 = 44.00
R = 250.00
v = 0.0011456
N = 7422.971
Ac = 196349.541
((10.1), ASCE 41-17) ω = Min(ω, 1.25*ω*(lb/ld)^2/3) = 0.26182028
with fc = 33.00

Calculation of ratio lb/ld

Inadequate Lap Length with lb/ld = 0.30

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

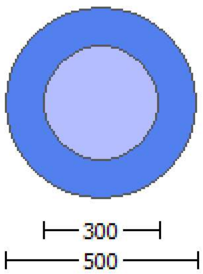
Limit State: Life Safety (data interpolation between analysis steps 1 and 2)

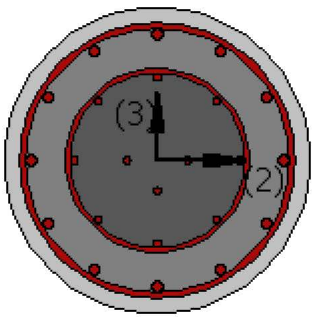
Analysis: Uniform +X

Check: Chord rotation capacity (θu)

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, κ = 1.00

Mean strength values are used for both shear and moment calculations.
Consequently:
Jacket
New material of Secondary Member: Concrete Strength, fc = fcm = 33.00
New material of Secondary Member: Steel Strength, fs = fsm = 555.56
Concrete Elasticity, Ec = 26999.444
Steel Elasticity, Es = 200000.00
Existing Column
Existing material of Secondary Member: Concrete Strength, fc = fcm = 20.00
Existing material of Secondary Member: Steel Strength, fs = fsm = 444.44
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 = 694.45
Existing Column
Existing material: Steel Strength, fs = 1.25*fsm = 555.55
##
External Diameter, D = 500.00
Internal Diameter, D = 300.00
Cover Thickness, c = 25.00
Mean Confinement Factor overall section = 1.00
Element Length, L = 3000.00
Secondary Member
Ribbed Bars
Ductile Steel
Without Detailing for Earthquake Resistance (including stirrups not closed at 135°)
Longitudinal Bars With Ends Lapped Starting at the End Sections
Inadequate Lap Length with lo/lou,min = 0.30
No FRP Wrapping

Stepwise Properties

At local axis: 3
EDGE -A-
Shear Force, Va = -7.1742060E-031
EDGE -B-
Shear Force, Vb = 7.1742060E-031
BOTH EDGES
Axial Force, F = -7425.858
Longitudinal Reinforcement Area Distribution (in 2 divisions)
-Tension: Aslt = 0.00
-Compression: Aslc = 3053.628
Longitudinal Reinforcement Area Distribution (in 3 divisions)
-Tension: Asl,ten = 1017.876
-Compression: Asl,com = 1017.876
-Middle: Asl,mid = 1017.876

Calculation of Shear Capacity ratio , Ve/Vr = 0.32864977
Member Controlled by Flexure (Ve/Vr < 1)
Calculation of Shear Demand from fig. R18.6.5, ACI 318-14 Ve = (Mpr1 + Mpr2)/ln = 159269.81
with
Mpr1 = Max(Mu1+ , Mu1-) = 2.3890E+008
Mu1+ = 2.3890E+008, is the ultimate moment strength at the edge 1 of the member in the actual moment direction which is defined for the static loading combination
Mu1- = 2.3890E+008, is the ultimate moment strength at the edge 1 of the member in the opposite moment direction which is defined for the static loading combination
Mpr2 = Max(Mu2+ , Mu2-) = 2.3890E+008
Mu2+ = 2.3890E+008, is the ultimate moment strength at the edge 2 of the member in the actual moment direction which is defined for the the static loading combination
Mu2- = 2.3890E+008, is the ultimate moment strength at the edge 2 of the member in the opposite moment

direction which is defined for the the static loading combination

----
Calculation of Mu1+

Calculation of ultimate Moment Strength ((3.67), Biskinis Phd), Mu
Mu = 2.3890E+008

φ = 0.90757121
φ' = 0.80580716
error of function (3.68), Biskinis Phd = 59442.345
From 5A.2, TBDY: fcc = fc*λc = 33.00
conf. factor λc = 1.00
fc = 33.00
From 10.3.5, ASCE 41-17, Final value of fy: fy*Min(1,1.25*(lb/ld)^2/3) = 389.0139
lb/ld = 0.30
d1 = 44.00
R = 250.00
v = 0.0011456
N = 7425.858
Ac = 196349.541
ω = ω*Min(1,1.25*(lb/ld)^2/3) = 0.14666533

Calculation of ratio lb/ld

Inadequate Lap Length with lb/ld = 0.30

Calculation of Mu1-

Calculation of ultimate Moment Strength ((3.67), Biskinis Phd), Mu
Mu = 2.3890E+008

φ = 0.90757121
φ' = 0.80580716
error of function (3.68), Biskinis Phd = 59442.345
From 5A.2, TBDY: fcc = fc*λc = 33.00
conf. factor λc = 1.00
fc = 33.00
From 10.3.5, ASCE 41-17, Final value of fy: fy*Min(1,1.25*(lb/ld)^2/3) = 389.0139
lb/ld = 0.30
d1 = 44.00
R = 250.00
v = 0.0011456
N = 7425.858
Ac = 196349.541
ω = ω*Min(1,1.25*(lb/ld)^2/3) = 0.14666533

Calculation of ratio lb/ld

Inadequate Lap Length with lb/ld = 0.30

Calculation of Mu2+

Calculation of ultimate Moment Strength ((3.67), Biskinis Phd), Mu
Mu = 2.3890E+008

φ = 0.90757121
φ' = 0.80580716
error of function (3.68), Biskinis Phd = 59442.345
From 5A.2, TBDY: fcc = fc*λc = 33.00
conf. factor λc = 1.00
fc = 33.00
From 10.3.5, ASCE 41-17, Final value of fy: fy*Min(1,1.25*(lb/ld)^2/3) = 389.0139
lb/ld = 0.30
d1 = 44.00
R = 250.00
v = 0.0011456
N = 7425.858
Ac = 196349.541
ω = ω*Min(1,1.25*(lb/ld)^2/3) = 0.14666533

Calculation of ratio lb/ld

Inadequate Lap Length with lb/ld = 0.30

Calculation of Mu2-

Calculation of ultimate Moment Strength ((3.67), Biskinis Phd), Mu
Mu = 2.3890E+008

φ = 0.90757121
φ' = 0.80580716
error of function (3.68), Biskinis Phd = 59442.345
From 5A.2, TBDY: fcc = fc*λc = 33.00
conf. factor λc = 1.00
fc = 33.00
From 10.3.5, ASCE 41-17, Final value of fy: fy*Min(1,1.25*(lb/ld)^2/3) = 389.0139
lb/ld = 0.30
d1 = 44.00
R = 250.00
v = 0.0011456
N = 7425.858
Ac = 196349.541
ω = ω*Min(1,1.25*(lb/ld)^2/3) = 0.14666533

Calculation of ratio lb/ld

Inadequate Lap Length with lb/ld = 0.30

Calculation of Shear Strength Vr = Min(Vr1,Vr2) = 484618.662

Calculation of Shear Strength at edge 1, Vr1 = 484618.662
Vr1 = VCol ((10.3), ASCE 41-17) = knl*VCol0
VCol0 = 484618.662
knl = 1 (zero step-static loading)

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

λ = 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 = 1.3305601E-011
Vu = 7.1742060E-031
d = 0.8*D = 400.00
Nu = 7425.858
Ag = 196349.541
From (11.5.4.8), ACI 318-14: Vs = Vs1 + Vs2 = 274157.871
Vs1 = 274157.871 is calculated for jacket, with:
Av = π/2*A_stirrup = 123370.055
fy = 555.56
s = 100.00
Vs1 is multiplied by αCol1 = 1.00
s/d = 0.25
Vs2 = 0.00 is calculated for core, with:
Av = π/2*A_stirrup = 78956.835
fy = 444.44
s = 250.00
Vs2 is multiplied by αCol2 = 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 = π*d*d/4 = 125663.706

Calculation of Shear Strength at edge 2, Vr2 = 484618.662
Vr2 = VCol ((10.3), ASCE 41-17) = knl*VCol0
VCol0 = 484618.662
knl = 1 (zero step-static loading)

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

λ = 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 = 1.3305601E-011
Vu = 7.1742060E-031
d = 0.8*D = 400.00
Nu = 7425.858
Ag = 196349.541
From (11.5.4.8), ACI 318-14: Vs = Vs1 + Vs2 = 274157.871
Vs1 = 274157.871 is calculated for jacket, with:
Av = π/2*A_stirrup = 123370.055
fy = 555.56
s = 100.00
Vs1 is multiplied by αCol1 = 1.00
s/d = 0.25
Vs2 = 0.00 is calculated for core, with:
Av = π/2*A_stirrup = 78956.835
fy = 444.44
s = 250.00
Vs2 is multiplied by αCol2 = 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 = π*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, κ = 1.00
Mean strength values are used for both shear and moment calculations.
Consequently:
Jacket
New material of Secondary Member: Concrete Strength, fc = fcm = 33.00
New material of Secondary Member: Steel Strength, fs = fsm = 555.56
Concrete Elasticity, Ec = 26999.444
Steel Elasticity, Es = 200000.00
Existing Column
Existing material of Secondary Member: Concrete Strength, fc = fcm = 20.00
Existing material of Secondary Member: Steel Strength, fs = fsm = 444.44
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 = 694.45
Existing Column
Existing material: Steel Strength, fs = 1.25*fsm = 555.55
##
External Diameter, D = 500.00
Internal Diameter, D = 300.00
Cover Thickness, c = 25.00
Mean Confinement Factor overall section = 1.00
Element Length, L = 3000.00
Secondary Member
Ribbed Bars
Ductile Steel
Without Detailing for Earthquake Resistance (including stirrups not closed at 135°)
Longitudinal Bars With Ends Lapped Starting at the End Sections
Inadequate Lap Length with lo/lou,min = 0.30
No FRP Wrapping

Stepwise Properties

At local axis: 2
EDGE -A-
Shear Force, Va = -3.8672673E-031
EDGE -B-
Shear Force, Vb = 3.8672673E-031
BOTH EDGES
Axial Force, F = -7425.858
Longitudinal Reinforcement Area Distribution (in 2 divisions)
-Tension: Aslt = 0.00
-Compression: Aslc = 3053.628
Longitudinal Reinforcement Area Distribution (in 3 divisions)
-Tension: Asl,ten = 1017.876
-Compression: Asl,com = 1017.876
-Middle: Asl,mid = 1017.876

Calculation of Shear Capacity ratio , Ve/Vr = 0.32864977
Member Controlled by Flexure (Ve/Vr < 1)
Calculation of Shear Demand from fig. R18.6.5, ACI 318-14 Ve = (Mpr1 + Mpr2)/ln = 159269.81
with
Mpr1 = Max(Mu1+ , Mu1-) = 2.3890E+008
Mu1+ = 2.3890E+008, is the ultimate moment strength at the edge 1 of the member in the actual moment direction which is defined for the static loading combination
Mu1- = 2.3890E+008, is the ultimate moment strength at the edge 1 of the member in the opposite moment

direction which is defined for the static loading combination
Mpr2 = Max(Mu2+ , Mu2-) = 2.3890E+008
Mu2+ = 2.3890E+008, is the ultimate moment strength at the edge 2 of the member in the actual moment direction which is defined for the the static loading combination
Mu2- = 2.3890E+008, is the ultimate moment strength at the edge 2 of the member in the opposite moment direction which is defined for the the static loading combination

----
Calculation of Mu1+

Calculation of ultimate Moment Strength ((3.67), Biskinis Phd), Mu
Mu = 2.3890E+008

φ = 0.90757121
φ' = 0.80580716
error of function (3.68), Biskinis Phd = 59442.345
From 5A.2, TBDY: fcc = fc*λc = 33.00
conf. factor λc = 1.00
fc = 33.00
From 10.3.5, ASCE 41-17, Final value of fy: fy*Min(1,1.25*(lb/ld)^2/3) = 389.0139
lb/ld = 0.30
d1 = 44.00
R = 250.00
v = 0.0011456
N = 7425.858
Ac = 196349.541
ω = ω*Min(1,1.25*(lb/ld)^2/3) = 0.14666533

Calculation of ratio lb/ld

Inadequate Lap Length with lb/ld = 0.30

Calculation of Mu1-

Calculation of ultimate Moment Strength ((3.67), Biskinis Phd), Mu
Mu = 2.3890E+008

φ = 0.90757121
φ' = 0.80580716
error of function (3.68), Biskinis Phd = 59442.345
From 5A.2, TBDY: fcc = fc*λc = 33.00
conf. factor λc = 1.00
fc = 33.00
From 10.3.5, ASCE 41-17, Final value of fy: fy*Min(1,1.25*(lb/ld)^2/3) = 389.0139
lb/ld = 0.30
d1 = 44.00
R = 250.00
v = 0.0011456
N = 7425.858
Ac = 196349.541
ω = ω*Min(1,1.25*(lb/ld)^2/3) = 0.14666533

Calculation of ratio lb/ld

Inadequate Lap Length with lb/ld = 0.30

Calculation of Mu2+

Calculation of ultimate Moment Strength ((3.67), Biskinis Phd), Mu
Mu = 2.3890E+008

φ = 0.90757121
φ' = 0.80580716
error of function (3.68), Biskinis Phd = 59442.345
From 5A.2, TBDY: fcc = fc*λc = 33.00
conf. factor λc = 1.00
fc = 33.00
From 10.3.5, ASCE 41-17, Final value of fy: fy*Min(1,1.25*(lb/ld)^2/3) = 389.0139
lb/ld = 0.30
d1 = 44.00
R = 250.00
v = 0.0011456
N = 7425.858
Ac = 196349.541
ω = ω*Min(1,1.25*(lb/ld)^2/3) = 0.14666533

Calculation of ratio lb/ld

Inadequate Lap Length with lb/ld = 0.30

Calculation of Mu2-

Calculation of ultimate Moment Strength ((3.67), Biskinis Phd), Mu
Mu = 2.3890E+008

φ = 0.90757121
φ' = 0.80580716
error of function (3.68), Biskinis Phd = 59442.345
From 5A.2, TBDY: fcc = fc*λc = 33.00
conf. factor λc = 1.00
fc = 33.00
From 10.3.5, ASCE 41-17, Final value of fy: fy*Min(1,1.25*(lb/ld)^2/3) = 389.0139
lb/ld = 0.30
d1 = 44.00
R = 250.00
v = 0.0011456
N = 7425.858
Ac = 196349.541
ω = ω*Min(1,1.25*(lb/ld)^2/3) = 0.14666533

Calculation of ratio lb/ld

Inadequate Lap Length with lb/ld = 0.30

Calculation of Shear Strength Vr = Min(Vr1,Vr2) = 484618.662

Calculation of Shear Strength at edge 1, Vr1 = 484618.662
Vr1 = VCol ((10.3), ASCE 41-17) = knl*VCol0
VCol0 = 484618.662
knl = 1 (zero step-static loading)

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

λ = 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 = 2.2610821E-011
Vu = 3.8672673E-031
d = 0.8*D = 400.00
Nu = 7425.858
Ag = 196349.541
From (11.5.4.8), ACI 318-14: Vs = Vs1 + Vs2 = 274157.871
Vs1 = 274157.871 is calculated for jacket, with:
Av = π/2*A_stirrup = 123370.055
fy = 555.56
s = 100.00
Vs1 is multiplied by αCol1 = 1.00
s/d = 0.25
Vs2 = 0.00 is calculated for core, with:
Av = π/2*A_stirrup = 78956.835
fy = 444.44
s = 250.00
Vs2 is multiplied by αCol2 = 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 = π*d*d/4 = 125663.706

Calculation of Shear Strength at edge 2, Vr2 = 484618.662
Vr2 = VCol ((10.3), ASCE 41-17) = knl*VCol0
VCol0 = 484618.662
knl = 1 (zero step-static loading)

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

λ = 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 = 2.2610821E-011
Vu = 3.8672673E-031
d = 0.8*D = 400.00
Nu = 7425.858
Ag = 196349.541
From (11.5.4.8), ACI 318-14: Vs = Vs1 + Vs2 = 274157.871
Vs1 = 274157.871 is calculated for jacket, with:
Av = π/2*A_stirrup = 123370.055
fy = 555.56
s = 100.00
Vs1 is multiplied by αCol1 = 1.00
s/d = 0.25
Vs2 = 0.00 is calculated for core, with:
Av = π/2*A_stirrup = 78956.835
fy = 444.44
s = 250.00
Vs2 is multiplied by αCol2 = 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 = π*d*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, κ = 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 Secondary Member: Concrete Strength, fc = fcm = 33.00
New material of Secondary Member: Steel Strength, fs = fsm = 555.56
Concrete Elasticity, Ec = 26999.444
Steel Elasticity, Es = 200000.00
Existing Column
Existing material of Secondary Member: Concrete Strength, fc = fcm = 20.00
Existing material of Secondary Member: Steel Strength, fs = fsm = 444.44
Concrete Elasticity, Ec = 21019.039
Steel Elasticity, Es = 200000.00
External Diameter, D = 500.00
Internal Diameter, D = 300.00
Cover Thickness, c = 25.00
Element Length, L = 3000.00
Secondary Member
Ribbed Bars
Ductile Steel
Without Detailing for Earthquake Resistance (including stirrups not closed at 135°)
Longitudinal Bars With Ends Lapped Starting at the End Sections
Inadequate Lap Length with lb/ld = 0.30
No FRP Wrapping

Stepwise Properties

Bending Moment, M = 0.00749595
Shear Force, V2 = 7215.465
Shear Force, V3 = 1.2224225E-013
Axial Force, F = -7422.971
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
Mean Diameter of Tension Reinforcement, DbL = 18.00

Existing component: From table 7-7, ASCE 41_17: Final chord rotation Capacity θu,R = κ*θu = 0.02200493
θu = θy + θp = 0.02200493

- Calculation of θy -

θy = (My*Ls/3)/EIeff = 0.00084923 ((4.29),Biskinis Phd))
My = 2.0498E+008
Ls = M/V (with Ls >0.1*L and Ls < 2*L) = 300.00
From table 10.5, ASCE 41_17: EIeff = 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 = 7422.971
Ec*Ig = Ec_jacket*Ig_jacket + Ec_core*Ig_core = 8.0455E+013

Calculation of Yielding Moment My

Calculation of Φy and My according to (7) - (8) in Biskinis and Fardis
----
My = Min(My_ten,My_com) = 2.0498E+008
Φy = 5.8526403E-006
My_ten (8c) = 2.0498E+008
Φ_ten (7c) = 64.04196
error of function (7c) = 8.3473441E-005
My_com (8d) = 7.5621E+008
Φ_com (7d) = 64.56829
error of function (7d) = -0.00721829
with ((10.1), ASCE 41-17) ey = Min(ey, 1.25*ey*(lb/ld)^2/3) = 0.0027778
eco = 0.002
apl = 0.35 ((9a) in Biskinis and Fardis for no FRP Wrap)
d1 = 44.00
R = 250.00
v = 0.0011456
N = 7422.971
Ac = 196349.541
((10.1), ASCE 41-17) ω = Min(ω, 1.25*ω*(lb/ld)^2/3) = 0.26182028
with fc = 33.00

Calculation of ratio lb/ld

Inadequate Lap Length with lb/ld = 0.30

- Calculation of θp -

From table 10-9: θp = 0.02115569
with:
- Columns controlled by inadequate development or splicing along the clear height because lb/ld < 1
shear control ratio VyE/VCol0E = 0.32864977
d = d_external = 0.00
s = s_external = 0.00
ρt = ρs1 + ρs2 + 2*tf/bw*(ffe/fs) = 0.00323428
jacket: ρs1 = Av1*(π*Dc1/2)/(s1*Ag) = 0.0027646
Av1 = 78.53982, is the area of stirrup
Dc1 = Dext - 2*cover - External Hoop Diameter = 440.00, is the total Length of all stirrups parallel to loading (shear) direction
s1 = 100.00
core: ρs2 = Av2*(π*Dc2/2)/(s2*Ag) = 0.00046968
Av2 = 50.26548, is the area of stirrup
Dc2 = Dint - Internal Hoop Diameter = 292.00, is the total Length of all stirrups parallel to loading (shear) direction
s2 = 250.00
The term 2*tf/bw*(ffe/fs) is implemented to account for FRP contribution
where ρf = 2*tf/bw is FRP ratio (EC8 - 3, A.4.4.3(6)) and ffe/fs normalises ρf to steel strength
All these variables have already been given in Shear control ratio calculation.
For the normalisation fs of jacket is used.
NUD = 7422.971
Ag = 196349.541
f'cE = (fc_jacket*Area_jacket+fc_core*Area_core)/section_area = 28.32
fylE = (fy_ext_Long_Reinf*Area_ext_Long_Reinf+fy_int_Long_Reinf*Area_int_Long_Reinf)/Area_Tot_Long_Rein = 2.1219958E-314
fytE = (fy_ext_Trans_Reinf*ρs1+fy_int_Trans_Reinf*ρs2)/(ρs1+ρs2) = 539.4232
pl = 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: 3
Integration Section: (b)
