Detailed Member Calculations

Units: N&mm

Regulation: ASCE 41-17

Calculation No. 1

column C1, Floor 1

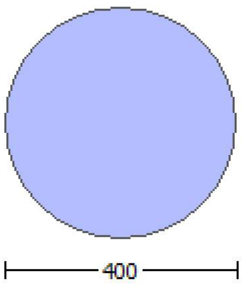
Limit State: Operational Level (data interpolation between analysis steps 2 and 3)

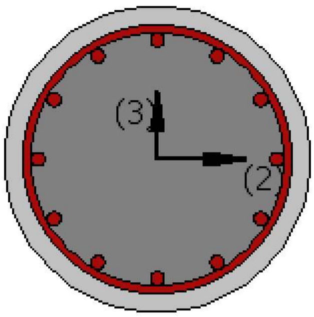
Analysis: Uniform +X

Check: Shear capacity VRd

Edge: Start

Local Axis: (2)





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

Constant Properties

Knowledge Factor, κ = 0.80
Member Shear Force is generally considered as Force-Controlled Action according to Table C7-1, ASCE41-17.
Lower-bound strengths are used for Force-Controlled Actions according to 7.5.1.3, ASCE 41-17
Consequently:
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, ASCE41-17) because bending is considered as
Deformation-Controlled Action (Table C7-1, ASCE41-17).
Existing material: Concrete Strength, fc = fcm = 20.00
Existing material: Steel Strength, fs = fsm = 444.44
###
Diameter, D = 400.00
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 = -1.3475E+007
Shear Force, Va = -4488.117
EDGE -B-
Bending Moment, Mb = 2717.168
Shear Force, Vb = 4488.117
BOTH EDGES
Axial Force, F = -4783.291
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 = 166781.653
Vn ((10.3), ASCE 41-17) = knl*VCol0 = 208477.066
VCol = 208477.066
knl = 1.00
displacement_ductility_demand = 0.03279029

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)
fc' = 16.00, but fc^0.5 <= 8.3 MPa (22.5.3.1, ACI 318-14)
M/Vd = 4.00
Mu = 1.3475E+007
Vu = 4488.117
d = 0.8*D = 320.00
Nu = 4783.291
Ag = 125663.706
From (11.5.4.8), ACI 318-14: Vs = 157913.67
Av = π/2*A_stirrup = 123370.055
fy = 400.00
s = 100.00
Vs is multiplied by αCol = 0.00
s/d = 0.3125
Vf ((11-3)-(11.4), ACI 440) = 0.00
From (11-11), ACI 440: Vs + Vf <= 213705.936

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

----

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 = 0.00053866
θy = (My*Ls/3)/Eleff = 0.01642731 ((4.29),Biskinis Phd))
My = 1.3007E+008
Ls = M/V (with Ls >0.1*L and Ls < 2*L) = 3002.333
From table 10.5, ASCE 41_17: Eleff = factor*Ec*Ig = 7.9240E+012
factor = 0.30
Ag = 125663.706
fc' = 20.00
N = 4783.291
Ec*Ig = 2.6413E+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) = 1.3007E+008
Φy = 7.1483870E-006
My_ten (8c) = 1.3007E+008
Φ_ten (7c) = 75.93176
error of function (7c) = 0.00012645
My_com (8d) = 3.4649E+008
Φ_com (7d) = 70.96949
error of function (7d) = -0.0005182
with ((10.1), ASCE 41-17) ey = Min(ey, 1.25*ey*(lb/ld)^2/3) = 0.0022222
eco = 0.002
apl = 0.35 ((9a) in Biskinis and Fardis for no FRP Wrap)
d1 = 44.00
R = 200.00
v = 0.00190321
N = 4783.291
Ac = 125663.706
((10.1), ASCE 41-17) ω = Min(ω, 1.25*ω*(lb/ld)^2/3) = 0.5399946
with fc = 20.00

Calculation of ratio lb/ld

Inadequate Lap Length with lb/ld = 0.30

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

Calculation No. 2

column C1, Floor 1

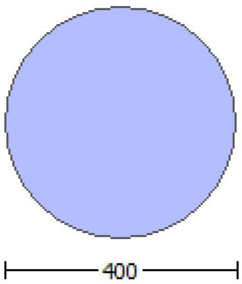
Limit State: Operational Level (data interpolation between analysis steps 2 and 3)

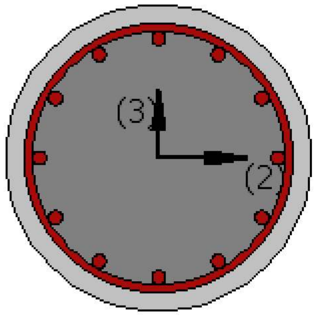
Analysis: Uniform +X

Check: Chord rotation capacity (θu)

Edge: Start

Local Axis: (2)





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

Constant Properties

Knowledge Factor, κ = 0.80
Mean strength values are used for both shear and moment calculations.
Consequently:
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
Existing material: Steel Strength, fs = 1.25*fsm = 555.55
##
Diameter, D = 400.00
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 = -4.2497275E-031
EDGE -B-
Shear Force, Vb = 4.2497275E-031

BOTH EDGES
Axial Force, F = -4771.233
Longitudinal Reinforcement Area Distribution (in 2 divisions)
-Tension: Aslt = 0.00
-Compression: Aslc = 3053.628
Longitudinal Reinforcement Area Distribution (in 3 divisions)
-Tension: Asl,ten = 1017.876
-Compression: Asl,com = 1017.876
-Middle: Asl,mid = 1017.876

Calculation of Shear Capacity ratio , Ve/Vr = 0.30828827
Member Controlled by Flexure (Ve/Vr < 1)
Calculation of Shear Demand from fig. R18.6.5, ACI 318-14 Ve = (Mpr1 + Mpr2)/ln = 88912.422
with
Mpr1 = Max(Mu1+ , Mu1-) = 1.3337E+008
Mu1+ = 1.3337E+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- = 1.3337E+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-) = 1.3337E+008
Mu2+ = 1.3337E+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- = 1.3337E+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 = 1.3337E+008

φ = 1.06465
φ' = 0.94240061
error of function (3.68), Biskinis Phd = 25149.978
From 5A.2, TBDY: fcc = fc*λc = 20.00
conf. factor λc = 1.00
fc = 20.00
From 10.3.5, ASCE41-17, Final value of fy: fy*Min(1,1.25*(lb/ld)^2/3) = 311.2056
lb/ld = 0.30
d1 = 44.00
R = 200.00
v = 0.00189953
N = 4771.233
Ac = 125663.706
ω = ω*Min(1,1.25*(lb/ld)^2/3) = 0.3024918

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 = 1.3337E+008

φ = 1.06465
φ' = 0.94240061

error of function (3.68), Biskinis Phd = 25149.978
From 5A.2, TBDY: fcc = fc*λc = 20.00
conf. factor λc = 1.00
fc = 20.00
From 10.3.5, ASCE41-17, Final value of fy: fy*Min(1,1.25*(lb/ld)^2/3) = 311.2056
lb/ld = 0.30
d1 = 44.00
R = 200.00
v = 0.00189953
N = 4771.233
Ac = 125663.706
ω = ω*Min(1,1.25*(lb/ld)^2/3) = 0.3024918

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 = 1.3337E+008

φ = 1.06465
φ' = 0.94240061
error of function (3.68), Biskinis Phd = 25149.978
From 5A.2, TBDY: fcc = fc*λc = 20.00
conf. factor λc = 1.00
fc = 20.00
From 10.3.5, ASCE41-17, Final value of fy: fy*Min(1,1.25*(lb/ld)^2/3) = 311.2056
lb/ld = 0.30
d1 = 44.00
R = 200.00
v = 0.00189953
N = 4771.233
Ac = 125663.706
ω = ω*Min(1,1.25*(lb/ld)^2/3) = 0.3024918

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 = 1.3337E+008

φ = 1.06465
φ' = 0.94240061
error of function (3.68), Biskinis Phd = 25149.978
From 5A.2, TBDY: fcc = fc*λc = 20.00
conf. factor λc = 1.00
fc = 20.00
From 10.3.5, ASCE41-17, Final value of fy: fy*Min(1,1.25*(lb/ld)^2/3) = 311.2056
lb/ld = 0.30
d1 = 44.00
R = 200.00
v = 0.00189953

N = 4771.233
Ac = 125663.706
ω = ω*Min(1,1.25*(lb/ld)^2/3) = 0.3024918

Calculation of ratio lb/ld

Inadequate Lap Length with lb/ld = 0.30

Calculation of Shear Strength Vr = Min(Vr1,Vr2) = 288406.767

Calculation of Shear Strength at edge 1, Vr1 = 288406.767
Vr1 = VCol ((10.3), ASCE 41-17) = knl*VCol0
VCol0 = 288406.767
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)
fc' = 20.00, but fc^0.5 <= 8.3 MPa (22.5.3.1, ACI 318-14)
M/Vd = 2.00
Mu = 2.9652000E-012
Vu = 4.2497275E-031
d = 0.8*D = 320.00
Nu = 4771.233
Ag = 125663.706
From (11.5.4.8), ACI 318-14: Vs = 175457.879
Av = π/2*A_stirrup = 123370.055
fy = 444.44
s = 100.00
Vs is multiplied by αCol = 0.00
s/d = 0.3125
Vf ((11-3)-(11.4), ACI 440) = 0.00
From (11-11), ACI 440: Vs + Vf <= 238930.50
bw*d = π*d*d/4 = 80424.772

Calculation of Shear Strength at edge 2, Vr2 = 288406.767
Vr2 = VCol ((10.3), ASCE 41-17) = knl*VCol0
VCol0 = 288406.767
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)
fc' = 20.00, but fc^0.5 <= 8.3 MPa (22.5.3.1, ACI 318-14)
M/Vd = 2.00
Mu = 2.9652000E-012
Vu = 4.2497275E-031
d = 0.8*D = 320.00
Nu = 4771.233
Ag = 125663.706
From (11.5.4.8), ACI 318-14: Vs = 175457.879
Av = π/2*A_stirrup = 123370.055
fy = 444.44
s = 100.00
Vs is multiplied by αCol = 0.00
s/d = 0.3125
Vf ((11-3)-(11.4), ACI 440) = 0.00
From (11-11), ACI 440: Vs + Vf <= 238930.50
bw*d = π*d*d/4 = 80424.772

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

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

Constant Properties

Knowledge Factor, κ = 0.80
Mean strength values are used for both shear and moment calculations.
Consequently:
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
Existing material: Steel Strength, fs = 1.25*fsm = 555.55
###
Diameter, D = 400.00
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 = 3.9443045E-031
EDGE -B-
Shear Force, Vb = -3.9443045E-031
BOTH EDGES
Axial Force, F = -4771.233
Longitudinal Reinforcement Area Distribution (in 2 divisions)
-Tension: Aslt = 0.00
-Compression: Aslc = 3053.628
Longitudinal Reinforcement Area Distribution (in 3 divisions)
-Tension: Asl,ten = 1017.876
-Compression: Asl,com = 1017.876
-Middle: Asl,mid = 1017.876

Calculation of Shear Capacity ratio , Ve/Vr = 0.30828827
Member Controlled by Flexure (Ve/Vr < 1)
Calculation of Shear Demand from fig. R18.6.5, ACI 318-14 Ve = (Mpr1 + Mpr2)/ln = 88912.422
with
Mpr1 = Max(Mu1+ , Mu1-) = 1.3337E+008
Mu1+ = 1.3337E+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- = 1.3337E+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-) = 1.3337E+008
Mu2+ = 1.3337E+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- = 1.3337E+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 = 1.3337E+008

φ = 1.06465
φ' = 0.94240061
error of function (3.68), Biskinis Phd = 25149.978
From 5A.2, TBDY: fcc = fc*λc = 20.00
conf. factor λc = 1.00
fc = 20.00
From 10.3.5, ASCE41-17, Final value of fy: fy*Min(1,1.25*(lb/ld)^2/3) = 311.2056
lb/ld = 0.30
d1 = 44.00
R = 200.00
v = 0.00189953
N = 4771.233
Ac = 125663.706
ω = ω*Min(1,1.25*(lb/ld)^2/3) = 0.3024918

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 = 1.3337E+008

φ = 1.06465
φ' = 0.94240061
error of function (3.68), Biskinis Phd = 25149.978
From 5A.2, TBDY: fcc = fc*λc = 20.00
conf. factor λc = 1.00
fc = 20.00
From 10.3.5, ASCE41-17, Final value of fy: fy*Min(1,1.25*(lb/ld)^2/3) = 311.2056
lb/ld = 0.30
d1 = 44.00
R = 200.00
v = 0.00189953
N = 4771.233
Ac = 125663.706
ω = ω*Min(1,1.25*(lb/ld)^2/3) = 0.3024918

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 = 1.3337E+008

φ = 1.06465
φ' = 0.94240061
error of function (3.68), Biskinis Phd = 25149.978
From 5A.2, TBDY: fcc = fc*λc = 20.00
conf. factor λc = 1.00
fc = 20.00
From 10.3.5, ASCE41-17, Final value of fy: fy*Min(1,1.25*(lb/ld)^2/3) = 311.2056
lb/ld = 0.30
d1 = 44.00
R = 200.00
v = 0.00189953
N = 4771.233
Ac = 125663.706
ω = ω*Min(1,1.25*(lb/ld)^2/3) = 0.3024918

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 = 1.3337E+008

φ = 1.06465
φ' = 0.94240061
error of function (3.68), Biskinis Phd = 25149.978
From 5A.2, TBDY: fcc = fc*λc = 20.00
conf. factor λc = 1.00
fc = 20.00
From 10.3.5, ASCE41-17, Final value of fy: fy*Min(1,1.25*(lb/ld)^2/3) = 311.2056
lb/ld = 0.30
d1 = 44.00
R = 200.00
v = 0.00189953
N = 4771.233
Ac = 125663.706
ω = ω*Min(1,1.25*(lb/ld)^2/3) = 0.3024918

Calculation of ratio lb/ld

Inadequate Lap Length with lb/ld = 0.30

Calculation of Shear Strength Vr = Min(Vr1,Vr2) = 288406.767

Calculation of Shear Strength at edge 1, Vr1 = 288406.767
Vr1 = VCol ((10.3), ASCE 41-17) = knl*VCol0
VCol0 = 288406.767
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)
fc' = 20.00, but fc^0.5 <= 8.3 MPa (22.5.3.1, ACI 318-14)
M/Vd = 2.00
Mu = 8.1661822E-012
Vu = 3.9443045E-031
d = 0.8*D = 320.00
Nu = 4771.233
Ag = 125663.706
From (11.5.4.8), ACI 318-14: Vs = 175457.879
Av = π/2*A_stirrup = 123370.055
fy = 444.44
s = 100.00
Vs is multiplied by αCol = 0.00
s/d = 0.3125
Vf ((11-3)-(11.4), ACI 440) = 0.00
From (11-11), ACI 440: Vs + Vf <= 238930.50
bw*d = π*d*d/4 = 80424.772

Calculation of Shear Strength at edge 2, Vr2 = 288406.767
Vr2 = VCol ((10.3), ASCE 41-17) = knl*VCol0
VCol0 = 288406.767
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)
fc' = 20.00, but fc^0.5 <= 8.3 MPa (22.5.3.1, ACI 318-14)
M/Vd = 2.00
Mu = 8.1661822E-012
Vu = 3.9443045E-031
d = 0.8*D = 320.00
Nu = 4771.233
Ag = 125663.706
From (11.5.4.8), ACI 318-14: Vs = 175457.879
Av = π/2*A_stirrup = 123370.055
fy = 444.44
s = 100.00
Vs is multiplied by αCol = 0.00
s/d = 0.3125
Vf ((11-3)-(11.4), ACI 440) = 0.00
From (11-11), ACI 440: Vs + Vf <= 238930.50
bw*d = π*d*d/4 = 80424.772

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

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

Constant Properties

Knowledge Factor, κ = 0.80
Chord Rotation is generally considered as Deformation-Controlled Action according to Table C7-1, ASCE41-17.
Mean strengths are used for Deformation-Controlled Actions according to 7.5.1.3, ASCE 41-17
Consequently:
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
Diameter, D = 400.00
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 = 4.2097093E-010
Shear Force, V2 = -4488.117
Shear Force, V3 = -6.4424309E-014
Axial Force, F = -4783.291
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 = 0.00656582
θu = θy + θp = 0.00820727

- Calculation of θy -

θy = (My*Ls/3)/EIeff = 0.00820727 ((4.29),Biskinis Phd))
My = 1.3007E+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 = 7.9240E+012
factor = 0.30
Ag = 125663.706
fc' = 20.00
N = 4783.291
Ec*Ig = 2.6413E+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) = 1.3007E+008
Φy = 7.1483870E-006
My_ten (8c) = 1.3007E+008
Φ_ten (7c) = 75.93176
error of function (7c) = 0.00012645
My_com (8d) = 3.4649E+008
Φ_com (7d) = 70.96949
error of function (7d) = -0.0005182
with ((10.1), ASCE 41-17) ey = Min(ey, 1.25*ey*(lb/ld)^2/3) = 0.0022222
eco = 0.002
apl = 0.35 ((9a) in Biskinis and Fardis for no FRP Wrap)
d1 = 44.00
R = 200.00
v = 0.00190321

N = 4783.291
Ac = 125663.706
((10.1), ASCE 41-17) ω = Min(ω, 1.25*ω*(lb/ld)^2/3) = 0.5399946
with fc = 20.00

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 not controlled by inadequate development or splicing along the clear height because lb/ld >= 1
shear control ratio VyE/VCol0E = 0.30828827
d = 0.00
s = 0.00
ρt = 2*Av/(dc*s) + 4*tf/D*(ffe/fs) = 0.00
Av = 78.53982, is the area of the circular stirrup
dc = D - 2*cover - Hoop Diameter = 340.00
The term 2*tf/bw*(ffe/fs) is implemented to account for FRP contribution
where 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.
NUD = 4783.291
Ag = 125663.706
f'cE = 20.00
fytE = fylE = 444.44
pl = Area_Tot_Long_Rein/(Ag) = 0.0243
f'cE = 20.00

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

Calculation No. 3

column C1, Floor 1

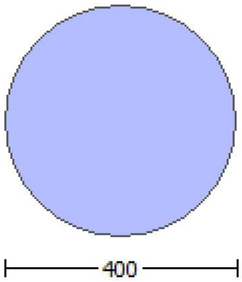
Limit State: Operational Level (data interpolation between analysis steps 2 and 3)

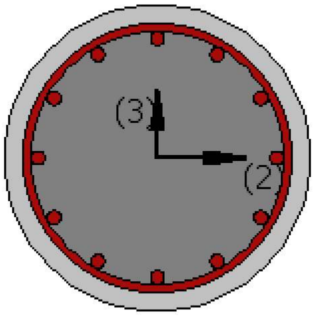
Analysis: Uniform +X

Check: Shear capacity VRd

Edge: Start

Local Axis: (3)





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

Constant Properties

Knowledge Factor, κ = 0.80
Member Shear Force is generally considered as Force-Controlled Action according to Table C7-1, ASCE41-17.
Lower-bound strengths are used for Force-Controlled Actions according to 7.5.1.3, ASCE 41-17
Consequently:
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, ASCE41-17) because bending is considered as
Deformation-Controlled Action (Table C7-1, ASCE41-17).
Existing material: Concrete Strength, fc = fcm = 20.00
Existing material: Steel Strength, fs = fsm = 444.44
##
Diameter, D = 400.00
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 = 4.2097093E-010
Shear Force, Va = -6.4424309E-014
EDGE -B-
Bending Moment, Mb = -2.2752425E-010
Shear Force, Vb = 6.4424309E-014
BOTH EDGES
Axial Force, F = -4783.291
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 = 207232.369
Vn ((10.3), ASCE 41-17) = knl*VCol0 = 259040.461
VCol = 259040.461
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)
fc' = 16.00, but fc^0.5 <= 8.3 MPa (22.5.3.1, ACI 318-14)
M/Vd = 2.00
Mu = 4.2097093E-010
Vu = 6.4424309E-014
d = 0.8*D = 320.00
Nu = 4783.291
Ag = 125663.706
From (11.5.4.8), ACI 318-14: Vs = 157913.67
Av = π/2*A_stirrup = 123370.055
fy = 400.00
s = 100.00
Vs is multiplied by αCol = 0.00
s/d = 0.3125
Vf ((11-3)-(11.4), ACI 440) = 0.00
From (11-11), ACI 440: Vs + Vf <= 213705.936
bw*d = π*d*d/4 = 80424.772

----
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 = 1.2360212E-020
θy = (My*Ls/3)/Eleff = 0.00820727 ((4.29),Biskinis Phd))
My = 1.3007E+008
Ls = M/V (with Ls >0.1*L and Ls < 2*L) = 1500.00
From table 10.5, ASCE 41_17: Eleff = factor*Ec*Ig = 7.9240E+012
factor = 0.30
Ag = 125663.706
fc' = 20.00
N = 4783.291
Ec*Ig = 2.6413E+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) = 1.3007E+008
Φy = 7.1483870E-006
My_ten (8c) = 1.3007E+008
Φ_ten (7c) = 75.93176
error of function (7c) = 0.00012645
My_com (8d) = 3.4649E+008
Φ_com (7d) = 70.96949
error of function (7d) = -0.0005182
with ((10.1), ASCE 41-17) ey = Min(ey, 1.25*ey*(lb/ld)^2/3) = 0.0022222
eco = 0.002
apl = 0.35 ((9a) in Biskinis and Fardis for no FRP Wrap)
d1 = 44.00

R = 200.00
v = 0.00190321
N = 4783.291
Ac = 125663.706
((10.1), ASCE 41-17) ω = Min(ω, 1.25*ω*(lb/ld)^2/3) = 0.5399946
with fc = 20.00

Calculation of ratio lb/ld

Inadequate Lap Length with lb/ld = 0.30

End Of Calculation of Shear Capacity for element: column CC1 of floor 1
At local axis: 3
Integration Section: (a)

Calculation No. 4

column C1, Floor 1

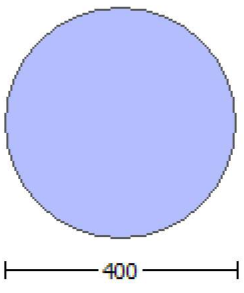
Limit State: Operational Level (data interpolation between analysis steps 2 and 3)

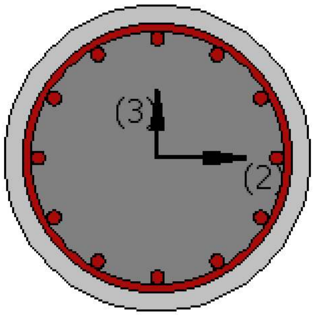
Analysis: Uniform +X

Check: Chord rotation capacity (θu)

Edge: Start

Local Axis: (3)





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

Constant Properties

Knowledge Factor, κ = 0.80
Mean strength values are used for both shear and moment calculations.
Consequently:
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
Existing material: Steel Strength, fs = 1.25*fsm = 555.55
##
Diameter, D = 400.00
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 = -4.2497275E-031
EDGE -B-
Shear Force, Vb = 4.2497275E-031
BOTH EDGES
Axial Force, F = -4771.233
Longitudinal Reinforcement Area Distribution (in 2 divisions)
-Tension: Aslt = 0.00
-Compression: Aslc = 3053.628
Longitudinal Reinforcement Area Distribution (in 3 divisions)
-Tension: Asl,ten = 1017.876
-Compression: Asl,com = 1017.876
-Middle: Asl,mid = 1017.876

Calculation of Shear Capacity ratio , Ve/Vr = 0.30828827
Member Controlled by Flexure (Ve/Vr < 1)
Calculation of Shear Demand from fig. R18.6.5, ACI 318-14 Ve = (Mpr1 + Mpr2)/ln = 88912.422
with
Mpr1 = Max(Mu1+ , Mu1-) = 1.3337E+008
Mu1+ = 1.3337E+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- = 1.3337E+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-) = 1.3337E+008
Mu2+ = 1.3337E+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- = 1.3337E+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 = 1.3337E+008

φ = 1.06465
φ' = 0.94240061
error of function (3.68), Biskinis Phd = 25149.978
From 5A.2, TBDY: fcc = fc*λc = 20.00

conf. factor λc = 1.00
fc = 20.00
From 10.3.5, ASCE41-17, Final value of fy: fy*Min(1,1.25*(lb/ld)^2/3) = 311.2056
lb/ld = 0.30
d1 = 44.00
R = 200.00
v = 0.00189953
N = 4771.233
Ac = 125663.706
ω = ω*Min(1,1.25*(lb/ld)^2/3) = 0.3024918

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 = 1.3337E+008

φ = 1.06465
φ' = 0.94240061
error of function (3.68), Biskinis Phd = 25149.978
From 5A.2, TBDY: fcc = fc*λc = 20.00
conf. factor λc = 1.00
fc = 20.00
From 10.3.5, ASCE41-17, Final value of fy: fy*Min(1,1.25*(lb/ld)^2/3) = 311.2056
lb/ld = 0.30
d1 = 44.00
R = 200.00
v = 0.00189953
N = 4771.233
Ac = 125663.706
ω = ω*Min(1,1.25*(lb/ld)^2/3) = 0.3024918

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 = 1.3337E+008

φ = 1.06465
φ' = 0.94240061
error of function (3.68), Biskinis Phd = 25149.978
From 5A.2, TBDY: fcc = fc*λc = 20.00
conf. factor λc = 1.00
fc = 20.00
From 10.3.5, ASCE41-17, Final value of fy: fy*Min(1,1.25*(lb/ld)^2/3) = 311.2056
lb/ld = 0.30
d1 = 44.00
R = 200.00
v = 0.00189953
N = 4771.233
Ac = 125663.706

ω = ω*Min(1,1.25*(lb/ld)^2/3) = 0.3024918

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 = 1.3337E+008

φ = 1.06465
φ' = 0.94240061
error of function (3.68), Biskinis Phd = 25149.978
From 5A.2, TBDY: fcc = fc*λc = 20.00
conf. factor λc = 1.00
fc = 20.00
From 10.3.5, ASCE41-17, Final value of fy: fy*Min(1,1.25*(lb/ld)^2/3) = 311.2056
lb/ld = 0.30
d1 = 44.00
R = 200.00
v = 0.00189953
N = 4771.233
Ac = 125663.706
ω = ω*Min(1,1.25*(lb/ld)^2/3) = 0.3024918

Calculation of ratio lb/ld

Inadequate Lap Length with lb/ld = 0.30

Calculation of Shear Strength Vr = Min(Vr1,Vr2) = 288406.767

Calculation of Shear Strength at edge 1, Vr1 = 288406.767
Vr1 = VCol ((10.3), ASCE 41-17) = knl*VCol0
VCol0 = 288406.767
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)
fc' = 20.00, but fc^0.5 <= 8.3 MPa (22.5.3.1, ACI 318-14)
M/Vd = 2.00
Mu = 2.9652000E-012
Vu = 4.2497275E-031
d = 0.8*D = 320.00
Nu = 4771.233
Ag = 125663.706
From (11.5.4.8), ACI 318-14: Vs = 175457.879
Av = π/2*A_stirrup = 123370.055
fy = 444.44
s = 100.00
Vs is multiplied by αCol = 0.00
s/d = 0.3125
Vf ((11-3)-(11.4), ACI 440) = 0.00
From (11-11), ACI 440: Vs + Vf <= 238930.50
bw*d = π*d*d/4 = 80424.772

Calculation of Shear Strength at edge 2, Vr2 = 288406.767
Vr2 = VCol ((10.3), ASCE 41-17) = knl*VCol0
VCol0 = 288406.767
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)
fc' = 20.00, but fc^0.5 <= 8.3 MPa (22.5.3.1, ACI 318-14)
M/Vd = 2.00
Mu = 2.9652000E-012
Vu = 4.2497275E-031
d = 0.8*D = 320.00
Nu = 4771.233
Ag = 125663.706
From (11.5.4.8), ACI 318-14: Vs = 175457.879
Av = π/2*A_stirrup = 123370.055
fy = 444.44
s = 100.00
Vs is multiplied by αCol = 0.00
s/d = 0.3125
Vf ((11-3)-(11.4), ACI 440) = 0.00
From (11-11), ACI 440: Vs + Vf <= 238930.50
bw*d = π*d*d/4 = 80424.772

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

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

Constant Properties

Knowledge Factor, κ = 0.80
Mean strength values are used for both shear and moment calculations.
Consequently:
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
Existing material: Steel Strength, fs = 1.25*fsm = 555.55
###
Diameter, D = 400.00
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 = 3.9443045E-031
EDGE -B-
Shear Force, Vb = -3.9443045E-031
BOTH EDGES
Axial Force, F = -4771.233
Longitudinal Reinforcement Area Distribution (in 2 divisions)
-Tension: Aslt = 0.00
-Compression: Aslc = 3053.628
Longitudinal Reinforcement Area Distribution (in 3 divisions)
-Tension: Asl,ten = 1017.876
-Compression: Asl,com = 1017.876
-Middle: Asl,mid = 1017.876

Calculation of Shear Capacity ratio , Ve/Vr = 0.30828827
Member Controlled by Flexure (Ve/Vr < 1)
Calculation of Shear Demand from fig. R18.6.5, ACI 318-14 Ve = (Mpr1 + Mpr2)/ln = 88912.422
with
Mpr1 = Max(Mu1+ , Mu1-) = 1.3337E+008
Mu1+ = 1.3337E+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- = 1.3337E+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-) = 1.3337E+008
Mu2+ = 1.3337E+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- = 1.3337E+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 = 1.3337E+008

φ = 1.06465
φ' = 0.94240061
error of function (3.68), Biskinis Phd = 25149.978
From 5A.2, TBDY: fcc = fc*λc = 20.00
conf. factor λc = 1.00
fc = 20.00
From 10.3.5, ASCE41-17, Final value of fy: fy*Min(1,1.25*(lb/ld)^2/3) = 311.2056
lb/ld = 0.30
d1 = 44.00
R = 200.00
v = 0.00189953
N = 4771.233
Ac = 125663.706
ω = ω*Min(1,1.25*(lb/ld)^2/3) = 0.3024918

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 = 1.3337E+008

φ = 1.06465
φ' = 0.94240061
error of function (3.68), Biskinis Phd = 25149.978
From 5A.2, TBDY: fcc = fc*λc = 20.00
conf. factor λc = 1.00
fc = 20.00
From 10.3.5, ASCE41-17, Final value of fy: fy*Min(1,1.25*(lb/ld)^2/3) = 311.2056
lb/ld = 0.30
d1 = 44.00
R = 200.00
v = 0.00189953
N = 4771.233
Ac = 125663.706
ω = ω*Min(1,1.25*(lb/ld)^2/3) = 0.3024918

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 = 1.3337E+008

φ = 1.06465
φ' = 0.94240061
error of function (3.68), Biskinis Phd = 25149.978
From 5A.2, TBDY: fcc = fc*λc = 20.00
conf. factor λc = 1.00
fc = 20.00
From 10.3.5, ASCE41-17, Final value of fy: fy*Min(1,1.25*(lb/ld)^2/3) = 311.2056
lb/ld = 0.30
d1 = 44.00
R = 200.00
v = 0.00189953
N = 4771.233
Ac = 125663.706
ω = ω*Min(1,1.25*(lb/ld)^2/3) = 0.3024918

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 = 1.3337E+008

φ = 1.06465
φ' = 0.94240061
error of function (3.68), Biskinis Phd = 25149.978
From 5A.2, TBDY: fcc = fc*λc = 20.00
conf. factor λc = 1.00

fc = 20.00
From 10.3.5, ASCE41-17, Final value of fy: fy*Min(1,1.25*(lb/ld)^2/3) = 311.2056
lb/ld = 0.30
d1 = 44.00
R = 200.00
v = 0.00189953
N = 4771.233
Ac = 125663.706
ω = ω*Min(1,1.25*(lb/ld)^2/3) = 0.3024918

Calculation of ratio lb/ld

Inadequate Lap Length with lb/ld = 0.30

Calculation of Shear Strength Vr = Min(Vr1,Vr2) = 288406.767

Calculation of Shear Strength at edge 1, Vr1 = 288406.767
Vr1 = VCol ((10.3), ASCE 41-17) = knl*VCol0
VCol0 = 288406.767
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)
fc' = 20.00, but fc^0.5 <= 8.3 MPa (22.5.3.1, ACI 318-14)
M/Vd = 2.00
Mu = 8.1661822E-012
Vu = 3.9443045E-031
d = 0.8*D = 320.00
Nu = 4771.233
Ag = 125663.706
From (11.5.4.8), ACI 318-14: Vs = 175457.879
Av = π/2*A_stirrup = 123370.055
fy = 444.44
s = 100.00
Vs is multiplied by αCol = 0.00
s/d = 0.3125
Vf ((11-3)-(11.4), ACI 440) = 0.00
From (11-11), ACI 440: Vs + Vf <= 238930.50
bw*d = π*d*d/4 = 80424.772

Calculation of Shear Strength at edge 2, Vr2 = 288406.767
Vr2 = VCol ((10.3), ASCE 41-17) = knl*VCol0
VCol0 = 288406.767
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)
fc' = 20.00, but fc^0.5 <= 8.3 MPa (22.5.3.1, ACI 318-14)
M/Vd = 2.00
Mu = 8.1661822E-012
Vu = 3.9443045E-031
d = 0.8*D = 320.00
Nu = 4771.233
Ag = 125663.706
From (11.5.4.8), ACI 318-14: Vs = 175457.879
Av = π/2*A_stirrup = 123370.055
fy = 444.44
s = 100.00

Vs is multiplied by αCol = 0.00
s/d = 0.3125
Vf ((11-3)-(11.4), ACI 440) = 0.00
From (11-11), ACI 440: Vs + Vf <= 238930.50
bw*d = π*d*d/4 = 80424.772

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

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

Constant Properties

Knowledge Factor, κ = 0.80
Chord Rotation is generally considered as Deformation-Controlled Action according to Table C7-1, ASCE41-17.
Mean strengths are used for Deformation-Controlled Actions according to 7.5.1.3, ASCE 41-17
Consequently:
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
Diameter, D = 400.00
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 = -1.3475E+007
Shear Force, V2 = -4488.117
Shear Force, V3 = -6.4424309E-014
Axial Force, F = -4783.291
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 = 0.01314185
θu = θy + θp = 0.01642731

- Calculation of θy -

θy = (My*Ls/3)/EIeff = 0.01642731 ((4.29),Biskinis Phd))
My = 1.3007E+008
Ls = M/V (with Ls >0.1*L and Ls < 2*L) = 3002.333

From table 10.5, ASCE 41_17: Eleff = factor*Ec*Ig = 7.9240E+012
factor = 0.30
Ag = 125663.706
fc' = 20.00
N = 4783.291
Ec*Ig = 2.6413E+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) = 1.3007E+008
Φy = 7.1483870E-006
My_ten (8c) = 1.3007E+008
Φ_ten (7c) = 75.93176
error of function (7c) = 0.00012645
My_com (8d) = 3.4649E+008
Φ_com (7d) = 70.96949
error of function (7d) = -0.0005182
with ((10.1), ASCE 41-17) ey = Min(ey, 1.25*ey*(lb/ld)^2/3) = 0.0022222
eco = 0.002
apl = 0.35 ((9a) in Biskinis and Fardis for no FRP Wrap)
d1 = 44.00
R = 200.00
v = 0.00190321
N = 4783.291
Ac = 125663.706
((10.1), ASCE 41-17) ω = Min(ω, 1.25*ω*(lb/ld)^2/3) = 0.5399946
with fc = 20.00

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 not controlled by inadequate development or splicing along the clear height because lb/ld >= 1
shear control ratio VyE/VCol0E = 0.30828827
d = 0.00
s = 0.00
ρt = 2*Av/(dc*s) + 4*tf/D*(ffe/fs) = 0.00
Av = 78.53982, is the area of the circular stirrup
dc = D - 2*cover - Hoop Diameter = 340.00
The term 2*tf/bw*(ffe/fs) is implemented to account for FRP contribution
where 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.
NUD = 4783.291
Ag = 125663.706
f'cE = 20.00
fytE = fylE = 444.44
pl = Area_Tot_Long_Rein/(Ag) = 0.0243
f'cE = 20.00

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

Calculation No. 5

column C1, Floor 1

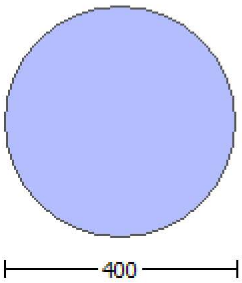
Limit State: Operational Level (data interpolation between analysis steps 2 and 3)

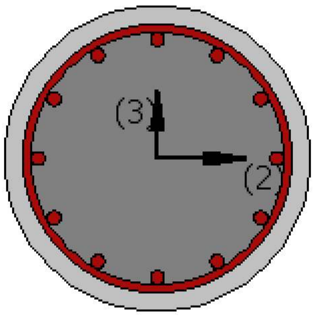
Analysis: Uniform +X

Check: Shear capacity VRd

Edge: End

Local Axis: (2)





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

Constant Properties

Knowledge Factor, κ = 0.80
Member Shear Force is generally considered as Force-Controlled Action according to Table C7-1, ASCE41-17.
Lower-bound strengths are used for Force-Controlled Actions according to 7.5.1.3, ASCE 41-17
Consequently:
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, ASCE41-17) because bending is considered as
Deformation-Controlled Action (Table C7-1, ASCE41-17).
Existing material: Concrete Strength, fc = fcm = 20.00
Existing material: Steel Strength, fs = fsm = 444.44
###
Diameter, D = 400.00
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 = -1.3475E+007
Shear Force, Va = -4488.117
EDGE -B-
Bending Moment, Mb = 2717.168
Shear Force, Vb = 4488.117
BOTH EDGES
Axial Force, F = -4783.291
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 = 207232.369
Vn ((10.3), ASCE 41-17) = knl*VCol0 = 259040.461
VCol = 259040.461
knl = 1.00
displacement_ductility_demand = 0.15543679

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)
fc' = 16.00, but fc^0.5 <= 8.3 MPa (22.5.3.1, ACI 318-14)
M/Vd = 2.00
Mu = 2717.168
Vu = 4488.117
d = 0.8*D = 320.00
Nu = 4783.291
Ag = 125663.706
From (11.5.4.8), ACI 318-14: Vs = 157913.67
Av = π/2*A_stirrup = 123370.055
fy = 400.00
s = 100.00
Vs is multiplied by αCol = 0.00
s/d = 0.3125
Vf ((11-3)-(11.4), ACI 440) = 0.00
From (11-11), ACI 440: Vs + Vf <= 213705.936
bw*d = π*d*d/4 = 80424.772

----
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 = 0.00025514
θy = (My*Ls/3)/Eleff = 0.00164145 ((4.29),Biskinis Phd))
My = 1.3007E+008
Ls = M/V (with Ls >0.1*L and Ls < 2*L) = 300.00
From table 10.5, ASCE 41_17: Eleff = factor*Ec*Ig = 7.9240E+012
factor = 0.30
Ag = 125663.706
fc' = 20.00

N = 4783.291
Ec*Ig = 2.6413E+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) = 1.3007E+008
Φy = 7.1483870E-006
My_ten (8c) = 1.3007E+008
Φ_ten (7c) = 75.93176
error of function (7c) = 0.00012645
My_com (8d) = 3.4649E+008
Φ_com (7d) = 70.96949
error of function (7d) = -0.0005182
with ((10.1), ASCE 41-17) ey = Min(ey, 1.25*ey*(lb/ld)^2/3) = 0.0022222
eco = 0.002
apl = 0.35 ((9a) in Biskinis and Fardis for no FRP Wrap)
d1 = 44.00
R = 200.00
v = 0.00190321
N = 4783.291
Ac = 125663.706
((10.1), ASCE 41-17) ω = Min(ω, 1.25*ω*(lb/ld)^2/3) = 0.5399946
with fc = 20.00

Calculation of ratio lb/ld

Inadequate Lap Length with lb/ld = 0.30

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

Calculation No. 6

column C1, Floor 1

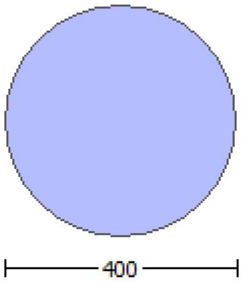
Limit State: Operational Level (data interpolation between analysis steps 2 and 3)

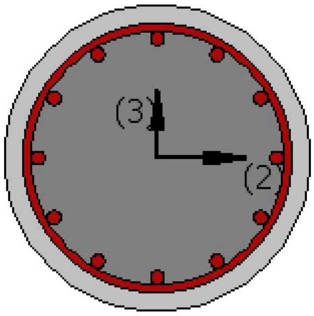
Analysis: Uniform +X

Check: Chord rotation capacity (θu)

Edge: End

Local Axis: (2)





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

Constant Properties

Knowledge Factor, κ = 0.80
Mean strength values are used for both shear and moment calculations.
Consequently:
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
Existing material: Steel Strength, fs = 1.25*fsm = 555.55
##
Diameter, D = 400.00
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 = -4.2497275E-031
EDGE -B-
Shear Force, Vb = 4.2497275E-031
BOTH EDGES
Axial Force, F = -4771.233
Longitudinal Reinforcement Area Distribution (in 2 divisions)
-Tension: Aslt = 0.00
-Compression: Aslc = 3053.628
Longitudinal Reinforcement Area Distribution (in 3 divisions)
-Tension: Asl,ten = 1017.876
-Compression: Asl,com = 1017.876
-Middle: Asl,mid = 1017.876

Calculation of Shear Capacity ratio , Ve/Vr = 0.30828827
Member Controlled by Flexure (Ve/Vr < 1)
Calculation of Shear Demand from fig. R18.6.5, ACI 318-14 Ve = (Mpr1 + Mpr2)/ln = 88912.422
with
Mpr1 = Max(Mu1+ , Mu1-) = 1.3337E+008
Mu1+ = 1.3337E+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- = 1.3337E+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-) = 1.3337E+008
Mu2+ = 1.3337E+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- = 1.3337E+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 = 1.3337E+008

φ = 1.06465
φ' = 0.94240061
error of function (3.68), Biskinis Phd = 25149.978
From 5A.2, TBDY: fcc = fc*λc = 20.00
conf. factor λc = 1.00
fc = 20.00
From 10.3.5, ASCE41-17, Final value of fy: fy*Min(1,1.25*(lb/ld)^2/3) = 311.2056
lb/ld = 0.30
d1 = 44.00
R = 200.00
v = 0.00189953
N = 4771.233
Ac = 125663.706
ω = ω*Min(1,1.25*(lb/ld)^2/3) = 0.3024918

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 = 1.3337E+008

φ = 1.06465
φ' = 0.94240061
error of function (3.68), Biskinis Phd = 25149.978
From 5A.2, TBDY: fcc = fc*λc = 20.00
conf. factor λc = 1.00
fc = 20.00
From 10.3.5, ASCE41-17, Final value of fy: fy*Min(1,1.25*(lb/ld)^2/3) = 311.2056
lb/ld = 0.30
d1 = 44.00
R = 200.00
v = 0.00189953
N = 4771.233
Ac = 125663.706
ω = ω*Min(1,1.25*(lb/ld)^2/3) = 0.3024918

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 = 1.3337E+008

φ = 1.06465
φ' = 0.94240061
error of function (3.68), Biskinis Phd = 25149.978
From 5A.2, TBDY: fcc = fc*λc = 20.00
conf. factor λc = 1.00
fc = 20.00
From 10.3.5, ASCE41-17, Final value of fy: fy*Min(1,1.25*(lb/ld)^2/3) = 311.2056
lb/ld = 0.30
d1 = 44.00
R = 200.00
v = 0.00189953
N = 4771.233
Ac = 125663.706
ω = ω*Min(1,1.25*(lb/ld)^2/3) = 0.3024918

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 = 1.3337E+008

φ = 1.06465
φ' = 0.94240061
error of function (3.68), Biskinis Phd = 25149.978
From 5A.2, TBDY: fcc = fc*λc = 20.00
conf. factor λc = 1.00
fc = 20.00
From 10.3.5, ASCE41-17, Final value of fy: fy*Min(1,1.25*(lb/ld)^2/3) = 311.2056
lb/ld = 0.30
d1 = 44.00
R = 200.00
v = 0.00189953
N = 4771.233
Ac = 125663.706
ω = ω*Min(1,1.25*(lb/ld)^2/3) = 0.3024918

Calculation of ratio lb/ld

Inadequate Lap Length with lb/ld = 0.30

Calculation of Shear Strength Vr = Min(Vr1,Vr2) = 288406.767

Calculation of Shear Strength at edge 1, Vr1 = 288406.767
Vr1 = VCol ((10.3), ASCE 41-17) = knl*VCol0
VCol0 = 288406.767
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)
fc' = 20.00, but fc^0.5 <= 8.3 MPa (22.5.3.1, ACI 318-14)
M/Vd = 2.00
Mu = 2.9652000E-012
Vu = 4.2497275E-031
d = 0.8*D = 320.00
Nu = 4771.233
Ag = 125663.706
From (11.5.4.8), ACI 318-14: Vs = 175457.879
Av = π/2*A_stirrup = 123370.055
fy = 444.44
s = 100.00
Vs is multiplied by αCol = 0.00
s/d = 0.3125
Vf ((11-3)-(11.4), ACI 440) = 0.00
From (11-11), ACI 440: Vs + Vf <= 238930.50
bw*d = π*d*d/4 = 80424.772

Calculation of Shear Strength at edge 2, Vr2 = 288406.767
Vr2 = VCol ((10.3), ASCE 41-17) = knl*VCol0
VCol0 = 288406.767
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)
fc' = 20.00, but fc^0.5 <= 8.3 MPa (22.5.3.1, ACI 318-14)
M/Vd = 2.00
Mu = 2.9652000E-012
Vu = 4.2497275E-031
d = 0.8*D = 320.00
Nu = 4771.233
Ag = 125663.706
From (11.5.4.8), ACI 318-14: Vs = 175457.879
Av = π/2*A_stirrup = 123370.055
fy = 444.44
s = 100.00
Vs is multiplied by αCol = 0.00
s/d = 0.3125
Vf ((11-3)-(11.4), ACI 440) = 0.00
From (11-11), ACI 440: Vs + Vf <= 238930.50
bw*d = π*d*d/4 = 80424.772

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

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

Constant Properties

Knowledge Factor, κ = 0.80
Mean strength values are used for both shear and moment calculations.
Consequently:
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
Existing material: Steel Strength, fs = 1.25*fsm = 555.55
###
Diameter, D = 400.00
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 = 3.9443045E-031
EDGE -B-
Shear Force, Vb = -3.9443045E-031
BOTH EDGES
Axial Force, F = -4771.233
Longitudinal Reinforcement Area Distribution (in 2 divisions)
-Tension: Aslt = 0.00
-Compression: Aslc = 3053.628
Longitudinal Reinforcement Area Distribution (in 3 divisions)
-Tension: Asl,ten = 1017.876
-Compression: Asl,com = 1017.876
-Middle: Asl,mid = 1017.876

Calculation of Shear Capacity ratio , Ve/Vr = 0.30828827
Member Controlled by Flexure (Ve/Vr < 1)
Calculation of Shear Demand from fig. R18.6.5, ACI 318-14 Ve = (Mpr1 + Mpr2)/ln = 88912.422
with
Mpr1 = Max(Mu1+ , Mu1-) = 1.3337E+008
Mu1+ = 1.3337E+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- = 1.3337E+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-) = 1.3337E+008
Mu2+ = 1.3337E+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- = 1.3337E+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 = 1.3337E+008

φ = 1.06465
φ' = 0.94240061
error of function (3.68), Biskinis Phd = 25149.978
From 5A.2, TBDY: fcc = fc*λc = 20.00
conf. factor λc = 1.00
fc = 20.00
From 10.3.5, ASCE41-17, Final value of fy: fy*Min(1,1.25*(lb/ld)^2/3) = 311.2056
lb/ld = 0.30
d1 = 44.00
R = 200.00
v = 0.00189953
N = 4771.233
Ac = 125663.706
ω = ω*Min(1,1.25*(lb/ld)^2/3) = 0.3024918

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 = 1.3337E+008

φ = 1.06465
φ' = 0.94240061
error of function (3.68), Biskinis Phd = 25149.978
From 5A.2, TBDY: fcc = fc*λc = 20.00
conf. factor λc = 1.00
fc = 20.00
From 10.3.5, ASCE41-17, Final value of fy: fy*Min(1,1.25*(lb/ld)^2/3) = 311.2056
lb/ld = 0.30
d1 = 44.00
R = 200.00
v = 0.00189953
N = 4771.233
Ac = 125663.706
ω = ω*Min(1,1.25*(lb/ld)^2/3) = 0.3024918

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 = 1.3337E+008

φ = 1.06465
φ' = 0.94240061
error of function (3.68), Biskinis Phd = 25149.978
From 5A.2, TBDY: fcc = fc*λc = 20.00
conf. factor λc = 1.00
fc = 20.00
From 10.3.5, ASCE41-17, Final value of fy: fy*Min(1,1.25*(lb/ld)^2/3) = 311.2056

lb/ld = 0.30
d1 = 44.00
R = 200.00
v = 0.00189953
N = 4771.233
Ac = 125663.706
ω = ω*Min(1,1.25*(lb/ld)^2/3) = 0.3024918

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 = 1.3337E+008

φ = 1.06465
φ' = 0.94240061
error of function (3.68), Biskinis Phd = 25149.978
From 5A.2, TBDY: fcc = fc*λc = 20.00
conf. factor λc = 1.00
fc = 20.00
From 10.3.5, ASCE41-17, Final value of fy: fy*Min(1,1.25*(lb/ld)^2/3) = 311.2056
lb/ld = 0.30
d1 = 44.00
R = 200.00
v = 0.00189953
N = 4771.233
Ac = 125663.706
ω = ω*Min(1,1.25*(lb/ld)^2/3) = 0.3024918

Calculation of ratio lb/ld

Inadequate Lap Length with lb/ld = 0.30

Calculation of Shear Strength Vr = Min(Vr1,Vr2) = 288406.767

Calculation of Shear Strength at edge 1, Vr1 = 288406.767
Vr1 = VCol ((10.3), ASCE 41-17) = knl*VCol0
VCol0 = 288406.767
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)
fc' = 20.00, but fc^0.5 <= 8.3 MPa (22.5.3.1, ACI 318-14)
M/Vd = 2.00
Mu = 8.1661822E-012
Vu = 3.9443045E-031
d = 0.8*D = 320.00
Nu = 4771.233
Ag = 125663.706
From (11.5.4.8), ACI 318-14: Vs = 175457.879
Av = π/2*A_stirrup = 123370.055
fy = 444.44
s = 100.00

Vs is multiplied by αCol = 0.00
s/d = 0.3125
Vf ((11-3)-(11.4), ACI 440) = 0.00
From (11-11), ACI 440: Vs + Vf <= 238930.50
bw*d = π*d*d/4 = 80424.772

Calculation of Shear Strength at edge 2, Vr2 = 288406.767
Vr2 = VCol ((10.3), ASCE 41-17) = knl*VCol0
VCol0 = 288406.767
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)
fc' = 20.00, but fc^0.5 <= 8.3 MPa (22.5.3.1, ACI 318-14)
M/Vd = 2.00
Mu = 8.1661822E-012
Vu = 3.9443045E-031
d = 0.8*D = 320.00
Nu = 4771.233
Ag = 125663.706
From (11.5.4.8), ACI 318-14: Vs = 175457.879
Av = π/2*A_stirrup = 123370.055
fy = 444.44
s = 100.00
Vs is multiplied by αCol = 0.00
s/d = 0.3125
Vf ((11-3)-(11.4), ACI 440) = 0.00
From (11-11), ACI 440: Vs + Vf <= 238930.50
bw*d = π*d*d/4 = 80424.772

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

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

Constant Properties

Knowledge Factor, κ = 0.80
Chord Rotation is generally considered as Deformation-Controlled Action according to Table C7-1, ASCE41-17.
Mean strengths are used for Deformation-Controlled Actions according to 7.5.1.3, ASCE 41-17
Consequently:
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
Diameter, D = 400.00
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 = -2.2752425E-010
Shear Force, V2 = 4488.117
Shear Force, V3 = 6.4424309E-014
Axial Force, F = -4783.291
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 = 0.00656582
θu = θy + θp = 0.00820727

- Calculation of θy -

θy = (My*Ls/3)/EIeff = 0.00820727 ((4.29),Biskinis Phd))
My = 1.3007E+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 = 7.9240E+012
factor = 0.30
Ag = 125663.706
fc' = 20.00
N = 4783.291
Ec*Ig = 2.6413E+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) = 1.3007E+008
Φy = 7.1483870E-006
My_ten (8c) = 1.3007E+008
Φ_ten (7c) = 75.93176
error of function (7c) = 0.00012645
My_com (8d) = 3.4649E+008
Φ_com (7d) = 70.96949
error of function (7d) = -0.0005182
with ((10.1), ASCE 41-17) ey = Min(ey, 1.25*ey*(lb/ld)^2/3) = 0.0022222
eco = 0.002
apl = 0.35 ((9a) in Biskinis and Fardis for no FRP Wrap)
d1 = 44.00
R = 200.00
v = 0.00190321
N = 4783.291
Ac = 125663.706
((10.1), ASCE 41-17) ω = Min(ω, 1.25*ω*(lb/ld)^2/3) = 0.5399946
with fc = 20.00

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 not controlled by inadequate development or splicing along the clear height because lb/ld >= 1
  shear control ratio VyE/VCol0E = 0.30828827
  d = 0.00
  s = 0.00
  ρt = 2*Av/(dc*s) + 4*tf/D*(ffe/fs) = 0.00
  Av = 78.53982, is the area of the circular stirrup
  dc = D - 2*cover - Hoop Diameter = 340.00
  The term 2*tf/bw*(ffe/fs) is implemented to account for FRP contribution
  where 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.
    NUD = 4783.291
    Ag = 125663.706
    f'cE = 20.00
    fytE = fylE = 444.44
ρl = Area_Tot_Long_Rein/(Ag) = 0.0243
    f'cE = 20.00

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

Calculation No. 7

column C1, Floor 1

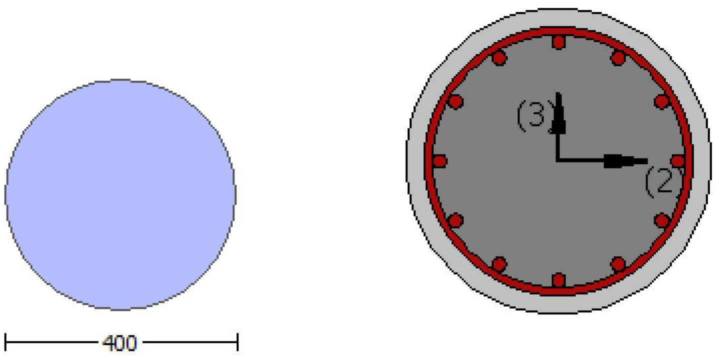
Limit State: Operational Level (data interpolation between analysis steps 2 and 3)

Analysis: Uniform +X

Check: Shear capacity VRd

Edge: End

Local Axis: (3)



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

Constant Properties

Knowledge Factor, κ = 0.80
Member Shear Force is generally considered as Force-Controlled Action according to Table C7-1, ASCE41-17.
Lower-bound strengths are used for Force-Controlled Actions according to 7.5.1.3, ASCE 41-17
Consequently:
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, ASCE41-17) because bending is considered as
Deformation-Controlled Action (Table C7-1, ASCE41-17).
Existing material: Concrete Strength, fc = fcm = 20.00
Existing material: Steel Strength, fs = fsm = 444.44
##
Diameter, D = 400.00
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 = 4.2097093E-010
Shear Force, Va = -6.4424309E-014
EDGE -B-
Bending Moment, Mb = -2.2752425E-010
Shear Force, Vb = 6.4424309E-014
BOTH EDGES
Axial Force, F = -4783.291
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 = 207232.369
Vn ((10.3), ASCE 41-17) = knl*VCol0 = 259040.461
VCol = 259040.461
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)
fc' = 16.00, but fc^0.5 <= 8.3 MPa (22.5.3.1, ACI 318-14)
M/Vd = 2.00
Mu = 2.2752425E-010
Vu = 6.4424309E-014
d = 0.8*D = 320.00

Nu = 4783.291
Ag = 125663.706
From (11.5.4.8), ACI 318-14: Vs = 157913.67
Av = π/2*A_stirrup = 123370.055
fy = 400.00
s = 100.00
Vs is multiplied by αCol = 0.00
s/d = 0.3125
Vf ((11-3)-(11.4), ACI 440) = 0.00
From (11-11), ACI 440: Vs + Vf <= 213705.936
bw*d = π*d*d/4 = 80424.772

----
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 = 1.5212680E-020
θy = (My*Ls/3)/Eleff = 0.00820727 ((4.29),Biskinis Phd))
My = 1.3007E+008
Ls = M/V (with Ls >0.1*L and Ls < 2*L) = 1500.00
From table 10.5, ASCE 41_17: Eleff = factor*Ec*Ig = 7.9240E+012
factor = 0.30
Ag = 125663.706
fc' = 20.00
N = 4783.291
Ec*Ig = 2.6413E+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) = 1.3007E+008
Φy = 7.1483870E-006
My_ten (8c) = 1.3007E+008
Φ_ten (7c) = 75.93176
error of function (7c) = 0.00012645
My_com (8d) = 3.4649E+008
Φ_com (7d) = 70.96949
error of function (7d) = -0.0005182
with ((10.1), ASCE 41-17) ey = Min(ey, 1.25*ey*(lb/ld)^2/3) = 0.0022222
eco = 0.002
apl = 0.35 ((9a) in Biskinis and Fardis for no FRP Wrap)
d1 = 44.00
R = 200.00
v = 0.00190321
N = 4783.291
Ac = 125663.706
((10.1), ASCE 41-17) ω = Min(ω, 1.25*ω*(lb/ld)^2/3) = 0.5399946
with fc = 20.00

Calculation of ratio lb/ld

Inadequate Lap Length with lb/ld = 0.30

End Of Calculation of Shear Capacity for element: column CC1 of floor 1
At local axis: 3
Integration Section: (b)

Calculation No. 8

column C1, Floor 1

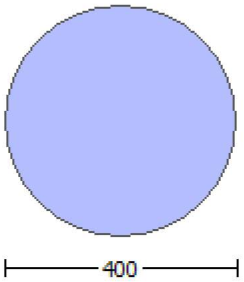
Limit State: Operational Level (data interpolation between analysis steps 2 and 3)

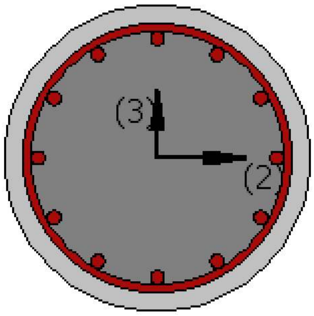
Analysis: Uniform +X

Check: Chord rotation capacity (θu)

Edge: End

Local Axis: (3)





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

Constant Properties

Knowledge Factor, κ = 0.80
Mean strength values are used for both shear and moment calculations.
Consequently:
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
Existing material: Steel Strength, fs = 1.25*fsm = 555.55
###
Diameter, D = 400.00
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 = -4.2497275E-031
EDGE -B-
Shear Force, Vb = 4.2497275E-031
BOTH EDGES
Axial Force, F = -4771.233
Longitudinal Reinforcement Area Distribution (in 2 divisions)
-Tension: Aslt = 0.00
-Compression: Aslc = 3053.628
Longitudinal Reinforcement Area Distribution (in 3 divisions)
-Tension: Asl,ten = 1017.876
-Compression: Asl,com = 1017.876
-Middle: Asl,mid = 1017.876

Calculation of Shear Capacity ratio , Ve/Vr = 0.30828827
Member Controlled by Flexure (Ve/Vr < 1)
Calculation of Shear Demand from fig. R18.6.5, ACI 318-14 Ve = (Mpr1 + Mpr2)/ln = 88912.422
with
Mpr1 = Max(Mu1+ , Mu1-) = 1.3337E+008
Mu1+ = 1.3337E+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- = 1.3337E+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-) = 1.3337E+008
Mu2+ = 1.3337E+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- = 1.3337E+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 = 1.3337E+008

φ = 1.06465
φ' = 0.94240061
error of function (3.68), Biskinis Phd = 25149.978
From 5A.2, TBDY: fcc = fc*λc = 20.00
conf. factor λc = 1.00
fc = 20.00
From 10.3.5, ASCE41-17, Final value of fy: fy*Min(1,1.25*(lb/ld)^2/3) = 311.2056
lb/ld = 0.30
d1 = 44.00
R = 200.00
v = 0.00189953
N = 4771.233
Ac = 125663.706
ω = ω*Min(1,1.25*(lb/ld)^2/3) = 0.3024918

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 = 1.3337E+008

φ = 1.06465
φ' = 0.94240061
error of function (3.68), Biskinis Phd = 25149.978
From 5A.2, TBDY: fcc = fc*λc = 20.00
conf. factor λc = 1.00
fc = 20.00
From 10.3.5, ASCE41-17, Final value of fy: fy*Min(1,1.25*(lb/ld)^2/3) = 311.2056
lb/ld = 0.30
d1 = 44.00
R = 200.00
v = 0.00189953
N = 4771.233
Ac = 125663.706
ω = ω*Min(1,1.25*(lb/ld)^2/3) = 0.3024918

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 = 1.3337E+008

φ = 1.06465
φ' = 0.94240061
error of function (3.68), Biskinis Phd = 25149.978
From 5A.2, TBDY: fcc = fc*λc = 20.00
conf. factor λc = 1.00
fc = 20.00
From 10.3.5, ASCE41-17, Final value of fy: fy*Min(1,1.25*(lb/ld)^2/3) = 311.2056
lb/ld = 0.30
d1 = 44.00
R = 200.00
v = 0.00189953
N = 4771.233
Ac = 125663.706
ω = ω*Min(1,1.25*(lb/ld)^2/3) = 0.3024918

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 = 1.3337E+008

φ = 1.06465
φ' = 0.94240061

error of function (3.68), Biskinis Phd = 25149.978
From 5A.2, TBDY: fcc = fc*λc = 20.00
conf. factor λc = 1.00
fc = 20.00
From 10.3.5, ASCE41-17, Final value of fy: fy*Min(1,1.25*(lb/ld)^2/3) = 311.2056
lb/ld = 0.30
d1 = 44.00
R = 200.00
v = 0.00189953
N = 4771.233
Ac = 125663.706
ω = ω*Min(1,1.25*(lb/ld)^2/3) = 0.3024918

Calculation of ratio lb/ld

Inadequate Lap Length with lb/ld = 0.30

Calculation of Shear Strength Vr = Min(Vr1,Vr2) = 288406.767

Calculation of Shear Strength at edge 1, Vr1 = 288406.767
Vr1 = VCol ((10.3), ASCE 41-17) = knl*VCol0
VCol0 = 288406.767
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)
fc' = 20.00, but fc^0.5 <= 8.3 MPa (22.5.3.1, ACI 318-14)
M/Vd = 2.00
Mu = 2.9652000E-012
Vu = 4.2497275E-031
d = 0.8*D = 320.00
Nu = 4771.233
Ag = 125663.706
From (11.5.4.8), ACI 318-14: Vs = 175457.879
Av = π/2*A_stirrup = 123370.055
fy = 444.44
s = 100.00
Vs is multiplied by αCol = 0.00
s/d = 0.3125
Vf ((11-3)-(11.4), ACI 440) = 0.00
From (11-11), ACI 440: Vs + Vf <= 238930.50
bw*d = π*d*d/4 = 80424.772

Calculation of Shear Strength at edge 2, Vr2 = 288406.767
Vr2 = VCol ((10.3), ASCE 41-17) = knl*VCol0
VCol0 = 288406.767
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)
fc' = 20.00, but fc^0.5 <= 8.3 MPa (22.5.3.1, ACI 318-14)
M/Vd = 2.00
Mu = 2.9652000E-012
Vu = 4.2497275E-031
d = 0.8*D = 320.00
Nu = 4771.233
Ag = 125663.706
From (11.5.4.8), ACI 318-14: Vs = 175457.879

Av = π/2*A_stirrup = 123370.055
fy = 444.44
s = 100.00
Vs is multiplied by αCol = 0.00
s/d = 0.3125
Vf ((11-3)-(11.4), ACI 440) = 0.00
From (11-11), ACI 440: Vs + Vf <= 238930.50
bw*d = π*d*d/4 = 80424.772

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

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

Constant Properties

Knowledge Factor, κ = 0.80
Mean strength values are used for both shear and moment calculations.
Consequently:
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
Existing material: Steel Strength, fs = 1.25*fsm = 555.55
##
Diameter, D = 400.00
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 = 3.9443045E-031
EDGE -B-
Shear Force, Vb = -3.9443045E-031
BOTH EDGES
Axial Force, F = -4771.233
Longitudinal Reinforcement Area Distribution (in 2 divisions)
-Tension: Aslt = 0.00
-Compression: Aslc = 3053.628
Longitudinal Reinforcement Area Distribution (in 3 divisions)
-Tension: Asl,ten = 1017.876
-Compression: Asl,com = 1017.876
-Middle: Asl,mid = 1017.876

Calculation of Shear Capacity ratio , Ve/Vr = 0.30828827
Member Controlled by Flexure (Ve/Vr < 1)
Calculation of Shear Demand from fig. R18.6.5, ACI 318-14 Ve = (Mpr1 + Mpr2)/ln = 88912.422
with
Mpr1 = Max(Mu1+ , Mu1-) = 1.3337E+008
Mu1+ = 1.3337E+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- = 1.3337E+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-) = 1.3337E+008
Mu2+ = 1.3337E+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- = 1.3337E+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 = 1.3337E+008

φ = 1.06465
φ' = 0.94240061
error of function (3.68), Biskinis Phd = 25149.978
From 5A.2, TBDY: fcc = fc*λc = 20.00
conf. factor λc = 1.00
fc = 20.00
From 10.3.5, ASCE41-17, Final value of fy: fy*Min(1,1.25*(lb/ld)^2/3) = 311.2056
lb/ld = 0.30
d1 = 44.00
R = 200.00
v = 0.00189953
N = 4771.233
Ac = 125663.706
ω = ω*Min(1,1.25*(lb/ld)^2/3) = 0.3024918

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 = 1.3337E+008

φ = 1.06465
φ' = 0.94240061
error of function (3.68), Biskinis Phd = 25149.978
From 5A.2, TBDY: fcc = fc*λc = 20.00
conf. factor λc = 1.00
fc = 20.00
From 10.3.5, ASCE41-17, Final value of fy: fy*Min(1,1.25*(lb/ld)^2/3) = 311.2056
lb/ld = 0.30
d1 = 44.00
R = 200.00
v = 0.00189953
N = 4771.233
Ac = 125663.706
ω = ω*Min(1,1.25*(lb/ld)^2/3) = 0.3024918

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 = 1.3337E+008

φ = 1.06465
φ' = 0.94240061
error of function (3.68), Biskinis Phd = 25149.978
From 5A.2, TBDY: fcc = fc*λc = 20.00
conf. factor λc = 1.00
fc = 20.00
From 10.3.5, ASCE41-17, Final value of fy: fy*Min(1,1.25*(lb/ld)^2/3) = 311.2056
lb/ld = 0.30
d1 = 44.00
R = 200.00
v = 0.00189953
N = 4771.233
Ac = 125663.706
ω = ω*Min(1,1.25*(lb/ld)^2/3) = 0.3024918

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 = 1.3337E+008

φ = 1.06465
φ' = 0.94240061
error of function (3.68), Biskinis Phd = 25149.978
From 5A.2, TBDY: fcc = fc*λc = 20.00
conf. factor λc = 1.00
fc = 20.00
From 10.3.5, ASCE41-17, Final value of fy: fy*Min(1,1.25*(lb/ld)^2/3) = 311.2056
lb/ld = 0.30
d1 = 44.00
R = 200.00
v = 0.00189953
N = 4771.233
Ac = 125663.706
ω = ω*Min(1,1.25*(lb/ld)^2/3) = 0.3024918

Calculation of ratio lb/ld

Inadequate Lap Length with lb/ld = 0.30

Calculation of Shear Strength Vr = Min(Vr1,Vr2) = 288406.767

Calculation of Shear Strength at edge 1, Vr1 = 288406.767
Vr1 = VCol ((10.3), ASCE 41-17) = knl*VCol0
VCol0 = 288406.767
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)
fc' = 20.00, but fc^0.5 <= 8.3 MPa (22.5.3.1, ACI 318-14)
M/Vd = 2.00
Mu = 8.1661822E-012
Vu = 3.9443045E-031
d = 0.8*D = 320.00
Nu = 4771.233
Ag = 125663.706
From (11.5.4.8), ACI 318-14: Vs = 175457.879
Av = π/2*A_stirrup = 123370.055
fy = 444.44
s = 100.00
Vs is multiplied by αCol = 0.00
s/d = 0.3125
Vf ((11-3)-(11.4), ACI 440) = 0.00
From (11-11), ACI 440: Vs + Vf <= 238930.50
bw*d = π*d*d/4 = 80424.772

Calculation of Shear Strength at edge 2, Vr2 = 288406.767
Vr2 = VCol ((10.3), ASCE 41-17) = knl*VCol0
VCol0 = 288406.767
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)
fc' = 20.00, but fc^0.5 <= 8.3 MPa (22.5.3.1, ACI 318-14)
M/Vd = 2.00
Mu = 8.1661822E-012
Vu = 3.9443045E-031
d = 0.8*D = 320.00
Nu = 4771.233
Ag = 125663.706
From (11.5.4.8), ACI 318-14: Vs = 175457.879
Av = π/2*A_stirrup = 123370.055
fy = 444.44
s = 100.00
Vs is multiplied by αCol = 0.00
s/d = 0.3125
Vf ((11-3)-(11.4), ACI 440) = 0.00
From (11-11), ACI 440: Vs + Vf <= 238930.50
bw*d = π*d*d/4 = 80424.772

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

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

Constant Properties

Knowledge Factor, κ = 0.80
Chord Rotation is generally considered as Deformation-Controlled Action according to Table C7-1, ASCE41-17.
Mean strengths are used for Deformation-Controlled Actions according to 7.5.1.3, ASCE 41-17
Consequently:
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
Diameter, D = 400.00
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 = 2717.168
Shear Force, V2 = 4488.117
Shear Force, V3 = 6.4424309E-014
Axial Force, F = -4783.291
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 = 0.00131316
θu = θy + θp = 0.00164145

- Calculation of θy -

θy = (My*Ls/3)/EIeff = 0.00164145 ((4.29),Biskinis Phd))
My = 1.3007E+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 = 7.9240E+012
factor = 0.30
Ag = 125663.706
fc' = 20.00
N = 4783.291
Ec*Ig = 2.6413E+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) = 1.3007E+008
Φy = 7.1483870E-006
My_ten (8c) = 1.3007E+008
Φ_ten (7c) = 75.93176
error of function (7c) = 0.00012645

My_com (8d) = 3.4649E+008
Φ_com (7d) = 70.96949
error of function (7d) = -0.0005182
with ((10.1), ASCE 41-17) ey = Min(ey, 1.25*ey*(lb/ld)^2/3) = 0.0022222
eco = 0.002
apl = 0.35 ((9a) in Biskinis and Fardis for no FRP Wrap)
d1 = 44.00
R = 200.00
v = 0.00190321
N = 4783.291
Ac = 125663.706
((10.1), ASCE 41-17) ω = Min(ω, 1.25*ω*(lb/ld)^2/3) = 0.5399946
with fc = 20.00

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 not controlled by inadequate development or splicing along the clear height because lb/ld >= 1
shear control ratio VyE/VCol0E = 0.30828827
d = 0.00
s = 0.00
ρt = 2*Av/(dc*s) + 4*tf/D*(ffe/fs) = 0.00
Av = 78.53982, is the area of the circular stirrup
dc = D - 2*cover - Hoop Diameter = 340.00
The term 2*tf/bw*(ffe/fs) is implemented to account for FRP contribution
where 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.
NUD = 4783.291
Ag = 125663.706
f'cE = 20.00
fytE = fylE = 444.44
pl = Area_Tot_Long_Rein/(Ag) = 0.0243
f'cE = 20.00

End Of Calculation of Chord Rotation Capacity for element: column CC1 of floor 1
At local axis: 3
Integration Section: (b)

Calculation No. 9

column C1, Floor 1

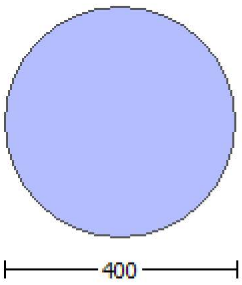
Limit State: Life Safety (data interpolation between analysis steps 2 and 3)

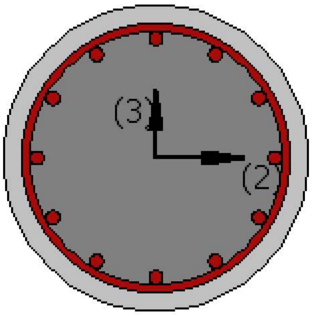
Analysis: Uniform +X

Check: Shear capacity VRd

Edge: Start

Local Axis: (2)





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

Constant Properties

Knowledge Factor, κ = 0.80
Member Shear Force is generally considered as Force-Controlled Action according to Table C7-1, ASCE41-17.
Lower-bound strengths are used for Force-Controlled Actions according to 7.5.1.3, ASCE 41-17
Consequently:
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, ASCE41-17) because bending is considered as
Deformation-Controlled Action (Table C7-1, ASCE41-17).
Existing material: Concrete Strength, fc = fcm = 20.00
Existing material: Steel Strength, fs = fsm = 444.44
##
Diameter, D = 400.00
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 = -1.2370E+007
Shear Force, Va = -4121.027

EDGE -B-
Bending Moment, Mb = 876.0544
Shear Force, Vb = 4121.027
BOTH EDGES
Axial Force, F = -4774.051
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 = 166780.92
Vn ((10.3), ASCE 41-17) = knl*VCol0 = 208476.15
VCol = 208476.15
knl = 1.00
displacement_ductility_demand = 0.02707656

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)
fc' = 16.00, but fc^0.5 <= 8.3 MPa (22.5.3.1, ACI 318-14)
M/Vd = 4.00
Mu = 1.2370E+007
Vu = 4121.027
d = 0.8*D = 320.00
Nu = 4774.051
Ag = 125663.706
From (11.5.4.8), ACI 318-14: Vs = 157913.67
Av = π/2*A_stirrup = 123370.055
fy = 400.00
s = 100.00
Vs is multiplied by αCol = 0.00
s/d = 0.3125
Vf ((11-3)-(11.4), ACI 440) = 0.00
From (11-11), ACI 440: Vs + Vf <= 213705.936
bw*d = π*d*d/4 = 80424.772

----
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 = 0.0004447
θy = (My*Ls/3)/EIeff = 0.01642397 ((4.29),Biskinis Phd))
My = 1.3007E+008
Ls = M/V (with Ls >0.1*L and Ls < 2*L) = 3001.757
From table 10.5, ASCE 41_17: EIeff = factor*Ec*Ig = 7.9240E+012
factor = 0.30
Ag = 125663.706
fc' = 20.00
N = 4774.051
Ec*Ig = 2.6413E+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) = 1.3007E+008
Φy = 7.1483407E-006
My_ten (8c) = 1.3007E+008
Φ_ten (7c) = 75.93136
error of function (7c) = 0.00012641
My_com (8d) = 3.4649E+008
Φ_com (7d) = 70.9694
error of function (7d) = -0.0005181
with ((10.1), ASCE 41-17) ey = Min(ey, 1.25*ey*(lb/ld)^2/3) = 0.0022222
eco = 0.002
apl = 0.35 ((9a) in Biskinis and Fardis for no FRP Wrap)
d1 = 44.00
R = 200.00
v = 0.00189953
N = 4774.051
Ac = 125663.706
((10.1), ASCE 41-17) ω = Min(ω, 1.25*ω*(lb/ld)^2/3) = 0.5399946
with fc = 20.00

Calculation of ratio lb/ld

Inadequate Lap Length with lb/ld = 0.30

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

Calculation No. 10

column C1, Floor 1

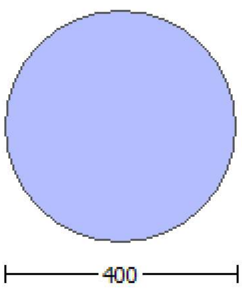
Limit State: Life Safety (data interpolation between analysis steps 2 and 3)

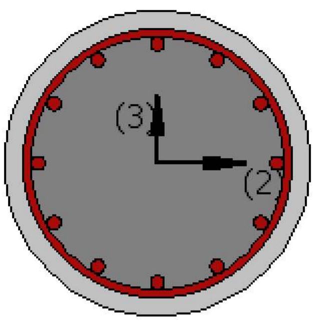
Analysis: Uniform +X

Check: Chord rotation capacity (θu)

Edge: Start

Local Axis: (2)





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

Constant Properties

Knowledge Factor, κ = 0.80
Mean strength values are used for both shear and moment calculations.
Consequently:
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
Existing material: Steel Strength, fs = 1.25*fsm = 555.55
###
Diameter, D = 400.00
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 = -4.2497275E-031
EDGE -B-
Shear Force, Vb = 4.2497275E-031
BOTH EDGES
Axial Force, F = -4771.233
Longitudinal Reinforcement Area Distribution (in 2 divisions)
-Tension: Aslt = 0.00
-Compression: Aslc = 3053.628
Longitudinal Reinforcement Area Distribution (in 3 divisions)
-Tension: Asl,ten = 1017.876
-Compression: Asl,com = 1017.876
-Middle: Asl,mid = 1017.876

Calculation of Shear Capacity ratio , Ve/Vr = 0.30828827
Member Controlled by Flexure (Ve/Vr < 1)
Calculation of Shear Demand from fig. R18.6.5, ACI 318-14 Ve = (Mpr1 + Mpr2)/ln = 88912.422
with
Mpr1 = Max(Mu1+ , Mu1-) = 1.3337E+008
Mu1+ = 1.3337E+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- = 1.3337E+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-) = 1.3337E+008
Mu2+ = 1.3337E+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- = 1.3337E+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 = 1.3337E+008

φ = 1.06465
φ' = 0.94240061
error of function (3.68), Biskinis Phd = 25149.978
From 5A.2, TBDY: fcc = fc*λc = 20.00
conf. factor λc = 1.00
fc = 20.00
From 10.3.5, ASCE41-17, Final value of fy: fy*Min(1,1.25*(lb/ld)^2/3) = 311.2056
lb/ld = 0.30
d1 = 44.00
R = 200.00
v = 0.00189953
N = 4771.233
Ac = 125663.706
ω = ω*Min(1,1.25*(lb/ld)^2/3) = 0.3024918

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 = 1.3337E+008

φ = 1.06465
φ' = 0.94240061
error of function (3.68), Biskinis Phd = 25149.978
From 5A.2, TBDY: fcc = fc*λc = 20.00
conf. factor λc = 1.00
fc = 20.00
From 10.3.5, ASCE41-17, Final value of fy: fy*Min(1,1.25*(lb/ld)^2/3) = 311.2056
lb/ld = 0.30
d1 = 44.00
R = 200.00
v = 0.00189953
N = 4771.233
Ac = 125663.706
ω = ω*Min(1,1.25*(lb/ld)^2/3) = 0.3024918

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 = 1.3337E+008

φ = 1.06465
φ' = 0.94240061
error of function (3.68), Biskinis Phd = 25149.978
From 5A.2, TBDY: fcc = fc*λc = 20.00
conf. factor λc = 1.00
fc = 20.00
From 10.3.5, ASCE41-17, Final value of fy: fy*Min(1,1.25*(lb/ld)^2/3) = 311.2056
lb/ld = 0.30
d1 = 44.00
R = 200.00
v = 0.00189953
N = 4771.233
Ac = 125663.706
ω = ω*Min(1,1.25*(lb/ld)^2/3) = 0.3024918

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 = 1.3337E+008

φ = 1.06465
φ' = 0.94240061
error of function (3.68), Biskinis Phd = 25149.978
From 5A.2, TBDY: fcc = fc*λc = 20.00
conf. factor λc = 1.00
fc = 20.00
From 10.3.5, ASCE41-17, Final value of fy: fy*Min(1,1.25*(lb/ld)^2/3) = 311.2056
lb/ld = 0.30
d1 = 44.00
R = 200.00
v = 0.00189953
N = 4771.233
Ac = 125663.706
ω = ω*Min(1,1.25*(lb/ld)^2/3) = 0.3024918

Calculation of ratio lb/ld

Inadequate Lap Length with lb/ld = 0.30

Calculation of Shear Strength Vr = Min(Vr1,Vr2) = 288406.767

Calculation of Shear Strength at edge 1, Vr1 = 288406.767
Vr1 = VCol ((10.3), ASCE 41-17) = knl*VCol0
VCol0 = 288406.767
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)
fc' = 20.00, but fc^0.5 <= 8.3 MPa (22.5.3.1, ACI 318-14)
M/Vd = 2.00
Mu = 2.9652000E-012

Vu = 4.2497275E-031
d = 0.8*D = 320.00
Nu = 4771.233
Ag = 125663.706
From (11.5.4.8), ACI 318-14: Vs = 175457.879
Av = π/2*A_stirrup = 123370.055
fy = 444.44
s = 100.00
Vs is multiplied by αCol = 0.00
s/d = 0.3125
Vf ((11-3)-(11.4), ACI 440) = 0.00
From (11-11), ACI 440: Vs + Vf <= 238930.50
bw*d = π*d*d/4 = 80424.772

Calculation of Shear Strength at edge 2, Vr2 = 288406.767
Vr2 = VCol ((10.3), ASCE 41-17) = knl*VCol0
VCol0 = 288406.767
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)
fc' = 20.00, but fc^0.5 <= 8.3 MPa (22.5.3.1, ACI 318-14)
M/Vd = 2.00
Mu = 2.9652000E-012
Vu = 4.2497275E-031
d = 0.8*D = 320.00
Nu = 4771.233
Ag = 125663.706
From (11.5.4.8), ACI 318-14: Vs = 175457.879
Av = π/2*A_stirrup = 123370.055
fy = 444.44
s = 100.00
Vs is multiplied by αCol = 0.00
s/d = 0.3125
Vf ((11-3)-(11.4), ACI 440) = 0.00
From (11-11), ACI 440: Vs + Vf <= 238930.50
bw*d = π*d*d/4 = 80424.772

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

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

Constant Properties

Knowledge Factor, κ = 0.80
Mean strength values are used for both shear and moment calculations.
Consequently:
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
Existing material: Steel Strength, fs = 1.25*fsm = 555.55
##

Diameter, D = 400.00
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 = 3.9443045E-031
EDGE -B-
Shear Force, Vb = -3.9443045E-031
BOTH EDGES
Axial Force, F = -4771.233
Longitudinal Reinforcement Area Distribution (in 2 divisions)
-Tension: Aslt = 0.00
-Compression: Aslc = 3053.628
Longitudinal Reinforcement Area Distribution (in 3 divisions)
-Tension: Asl,ten = 1017.876
-Compression: Asl,com = 1017.876
-Middle: Asl,mid = 1017.876

Calculation of Shear Capacity ratio , Ve/Vr = 0.30828827
Member Controlled by Flexure (Ve/Vr < 1)
Calculation of Shear Demand from fig. R18.6.5, ACI 318-14 Ve = (Mpr1 + Mpr2)/ln = 88912.422
with
Mpr1 = Max(Mu1+ , Mu1-) = 1.3337E+008
Mu1+ = 1.3337E+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- = 1.3337E+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-) = 1.3337E+008
Mu2+ = 1.3337E+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- = 1.3337E+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 = 1.3337E+008

φ = 1.06465
φ' = 0.94240061
error of function (3.68), Biskinis Phd = 25149.978
From 5A.2, TBDY: fcc = fc*λc = 20.00
conf. factor λc = 1.00
fc = 20.00
From 10.3.5, ASCE41-17, Final value of fy: fy*Min(1,1.25*(lb/ld)^2/3) = 311.2056
lb/ld = 0.30
d1 = 44.00
R = 200.00
v = 0.00189953

N = 4771.233
Ac = 125663.706
ω = ω*Min(1,1.25*(lb/ld)^2/3) = 0.3024918

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 = 1.3337E+008

φ = 1.06465
φ' = 0.94240061
error of function (3.68), Biskinis Phd = 25149.978
From 5A.2, TBDY: fcc = fc*λc = 20.00
conf. factor λc = 1.00
fc = 20.00
From 10.3.5, ASCE41-17, Final value of fy: fy*Min(1,1.25*(lb/ld)^2/3) = 311.2056
lb/ld = 0.30
d1 = 44.00
R = 200.00
v = 0.00189953
N = 4771.233
Ac = 125663.706
ω = ω*Min(1,1.25*(lb/ld)^2/3) = 0.3024918

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 = 1.3337E+008

φ = 1.06465
φ' = 0.94240061
error of function (3.68), Biskinis Phd = 25149.978
From 5A.2, TBDY: fcc = fc*λc = 20.00
conf. factor λc = 1.00
fc = 20.00
From 10.3.5, ASCE41-17, Final value of fy: fy*Min(1,1.25*(lb/ld)^2/3) = 311.2056
lb/ld = 0.30
d1 = 44.00
R = 200.00
v = 0.00189953
N = 4771.233
Ac = 125663.706
ω = ω*Min(1,1.25*(lb/ld)^2/3) = 0.3024918

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 = 1.3337E+008

φ = 1.06465
φ' = 0.94240061
error of function (3.68), Biskinis Phd = 25149.978
From 5A.2, TBDY: fcc = fc*λc = 20.00
conf. factor λc = 1.00
fc = 20.00
From 10.3.5, ASCE41-17, Final value of fy: fy*Min(1,1.25*(lb/ld)^2/3) = 311.2056
lb/ld = 0.30
d1 = 44.00
R = 200.00
v = 0.00189953
N = 4771.233
Ac = 125663.706
ω = ω*Min(1,1.25*(lb/ld)^2/3) = 0.3024918

Calculation of ratio lb/ld

Inadequate Lap Length with lb/ld = 0.30

Calculation of Shear Strength Vr = Min(Vr1,Vr2) = 288406.767

Calculation of Shear Strength at edge 1, Vr1 = 288406.767
Vr1 = VCol ((10.3), ASCE 41-17) = knl*VCol0
VCol0 = 288406.767
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)
fc' = 20.00, but fc^0.5 <= 8.3 MPa (22.5.3.1, ACI 318-14)
M/Vd = 2.00
Mu = 8.1661822E-012
Vu = 3.9443045E-031
d = 0.8*D = 320.00
Nu = 4771.233
Ag = 125663.706
From (11.5.4.8), ACI 318-14: Vs = 175457.879
Av = π/2*A_stirrup = 123370.055
fy = 444.44
s = 100.00
Vs is multiplied by αCol = 0.00
s/d = 0.3125
Vf ((11-3)-(11.4), ACI 440) = 0.00
From (11-11), ACI 440: Vs + Vf <= 238930.50
bw*d = π*d*d/4 = 80424.772

Calculation of Shear Strength at edge 2, Vr2 = 288406.767
Vr2 = VCol ((10.3), ASCE 41-17) = knl*VCol0
VCol0 = 288406.767
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)
fc' = 20.00, but fc^0.5 <= 8.3 MPa (22.5.3.1, ACI 318-14)
M/Vd = 2.00
Mu = 8.1661822E-012
Vu = 3.9443045E-031
d = 0.8*D = 320.00
Nu = 4771.233
Ag = 125663.706
From (11.5.4.8), ACI 318-14: Vs = 175457.879
Av = π/2*A_stirrup = 123370.055
fy = 444.44
s = 100.00
Vs is multiplied by αCol = 0.00
s/d = 0.3125
Vf ((11-3)-(11.4), ACI 440) = 0.00
From (11-11), ACI 440: Vs + Vf <= 238930.50
bw*d = π*d*d/4 = 80424.772

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

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

Constant Properties

Knowledge Factor, κ = 0.80
Chord Rotation is generally considered as Deformation-Controlled Action according to Table C7-1, ASCE41-17.
Mean strengths are used for Deformation-Controlled Actions according to 7.5.1.3, ASCE 41-17
Consequently:
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
Diameter, D = 400.00
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 = 6.6860422E-010
Shear Force, V2 = -4121.027
Shear Force, V3 = -2.0556142E-013
Axial Force, F = -4774.051
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 = 0.03014169
θu = θy + θp = 0.03767711

- Calculation of θy -

θy = (My*Ls/3)/EIeff = 0.00820718 ((4.29),Biskinis Phd))
My = 1.3007E+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 = 7.9240E+012
factor = 0.30
Ag = 125663.706
fc' = 20.00
N = 4774.051
Ec*Ig = 2.6413E+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) = 1.3007E+008
Φy = 7.1483407E-006
My_ten (8c) = 1.3007E+008
Φ_ten (7c) = 75.93136
error of function (7c) = 0.00012641
My_com (8d) = 3.4649E+008
Φ_com (7d) = 70.9694
error of function (7d) = -0.0005181
with ((10.1), ASCE 41-17) ey = Min(ey, 1.25*ey*(lb/ld)^2/3) = 0.0022222
eco = 0.002
apl = 0.35 ((9a) in Biskinis and Fardis for no FRP Wrap)
d1 = 44.00
R = 200.00
v = 0.00189953
N = 4774.051
Ac = 125663.706
((10.1), ASCE 41-17) ω = Min(ω, 1.25*ω*(lb/ld)^2/3) = 0.5399946
with fc = 20.00

Calculation of ratio lb/ld

Inadequate Lap Length with lb/ld = 0.30

- Calculation of θp -

From table 10-9: θp = 0.02946994
with:
- Columns not controlled by inadequate development or splicing along the clear height because lb/ld >= 1
shear control ratio VyE/VColOE = 0.30828827
d = 0.00
s = 0.00
ρt = 2*Av/(dc*s) + 4*tf/D*(ffe/fs) = 0.00
Av = 78.53982, is the area of the circular stirrup
dc = D - 2*cover - Hoop Diameter = 340.00
The term 2*tf/bw*(ffe/fs) is implemented to account for FRP contribution
where 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.

NUD = 4774.051
Ag = 125663.706
f'cE = 20.00
fytE = fylE = 444.44
pl = Area_Tot_Long_Rein/(Ag) = 0.0243
f'cE = 20.00

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

Calculation No. 11

column C1, Floor 1

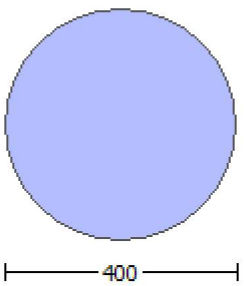
Limit State: Life Safety (data interpolation between analysis steps 2 and 3)

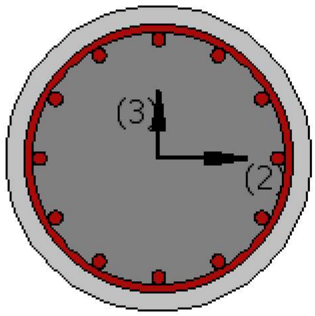
Analysis: Uniform +X

Check: Shear capacity VRd

Edge: Start

Local Axis: (3)





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

Constant Properties

Knowledge Factor, κ = 0.80
Member Shear Force is generally considered as Force-Controlled Action according to Table C7-1, ASCE41-17.
Lower-bound strengths are used for Force-Controlled Actions according to 7.5.1.3, ASCE 41-17
Consequently:
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, ASCE41-17) because bending is considered as Deformation-Controlled Action (Table C7-1, ASCE41-17).
Existing material: Concrete Strength, fc = fcm = 20.00
Existing material: Steel Strength, fs = fsm = 444.44
##
Diameter, D = 400.00
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 = 6.6860422E-010
Shear Force, Va = -2.0556142E-013
EDGE -B-
Bending Moment, Mb = -5.1597350E-011
Shear Force, Vb = 2.0556142E-013
BOTH EDGES
Axial Force, F = -4774.051
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 = 207230.904
Vn ((10.3), ASCE 41-17) = knl*VCol0 = 259038.631
VCol = 259038.631
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)
fc' = 16.00, but fc^0.5 <= 8.3 MPa (22.5.3.1, ACI 318-14)
M/Vd = 2.00
Mu = 6.6860422E-010
Vu = 2.0556142E-013
d = 0.8*D = 320.00
Nu = 4774.051
Ag = 125663.706
From (11.5.4.8), ACI 318-14: Vs = 157913.67
Av = π/2*A_stirrup = 123370.055
fy = 400.00
s = 100.00
Vs is multiplied by αCol = 0.00
s/d = 0.3125
Vf ((11-3)-(11.4), ACI 440) = 0.00
From (11-11), ACI 440: Vs + Vf <= 213705.936
bw*d = π*d*d/4 = 80424.772

----
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 = 2.2679671E-020
θy = (My*Ls/3)/Eleff = 0.00820718 ((4.29),Biskinis Phd))
My = 1.3007E+008
Ls = M/V (with Ls >0.1*L and Ls < 2*L) = 1500.00
From table 10.5, ASCE 41_17: Eleff = factor*Ec*Ig = 7.9240E+012
factor = 0.30
Ag = 125663.706
fc' = 20.00
N = 4774.051
Ec*Ig = 2.6413E+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) = 1.3007E+008
Φy = 7.1483407E-006
My_ten (8c) = 1.3007E+008
Φ_ten (7c) = 75.93136
error of function (7c) = 0.00012641
My_com (8d) = 3.4649E+008
Φ_com (7d) = 70.9694
error of function (7d) = -0.0005181
with ((10.1), ASCE 41-17) ey = Min(ey, 1.25*ey*(lb/ld)^2/3) = 0.0022222
eco = 0.002
apl = 0.35 ((9a) in Biskinis and Fardis for no FRP Wrap)
d1 = 44.00
R = 200.00
v = 0.00189953
N = 4774.051
Ac = 125663.706
((10.1), ASCE 41-17) ω = Min(ω, 1.25*ω*(lb/ld)^2/3) = 0.5399946
with fc = 20.00

Calculation of ratio lb/ld

Inadequate Lap Length with lb/ld = 0.30

End Of Calculation of Shear Capacity for element: column CC1 of floor 1
At local axis: 3
Integration Section: (a)

Calculation No. 12

column C1, Floor 1

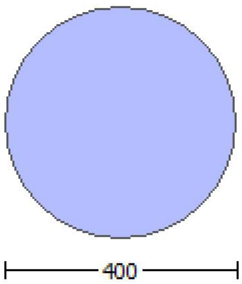
Limit State: Life Safety (data interpolation between analysis steps 2 and 3)

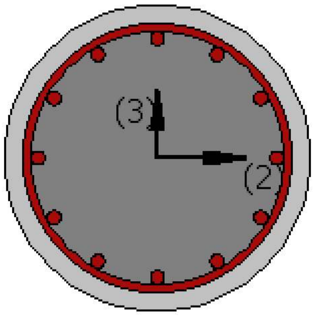
Analysis: Uniform +X

Check: Chord rotation capacity (θu)

Edge: Start

Local Axis: (3)





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

Constant Properties

Knowledge Factor, κ = 0.80
Mean strength values are used for both shear and moment calculations.
Consequently:
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
Existing material: Steel Strength, fs = 1.25*fsm = 555.55
##
Diameter, D = 400.00
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 = -4.2497275E-031
EDGE -B-
Shear Force, Vb = 4.2497275E-031

BOTH EDGES
Axial Force, F = -4771.233
Longitudinal Reinforcement Area Distribution (in 2 divisions)
-Tension: Aslt = 0.00
-Compression: Aslc = 3053.628
Longitudinal Reinforcement Area Distribution (in 3 divisions)
-Tension: Asl,ten = 1017.876
-Compression: Asl,com = 1017.876
-Middle: Asl,mid = 1017.876

Calculation of Shear Capacity ratio , Ve/Vr = 0.30828827
Member Controlled by Flexure (Ve/Vr < 1)
Calculation of Shear Demand from fig. R18.6.5, ACI 318-14 Ve = (Mpr1 + Mpr2)/ln = 88912.422
with
Mpr1 = Max(Mu1+ , Mu1-) = 1.3337E+008
Mu1+ = 1.3337E+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- = 1.3337E+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-) = 1.3337E+008
Mu2+ = 1.3337E+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- = 1.3337E+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 = 1.3337E+008

φ = 1.06465
φ' = 0.94240061
error of function (3.68), Biskinis Phd = 25149.978
From 5A.2, TBDY: fcc = fc*λc = 20.00
conf. factor λc = 1.00
fc = 20.00
From 10.3.5, ASCE41-17, Final value of fy: fy*Min(1,1.25*(lb/ld)^2/3) = 311.2056
lb/ld = 0.30
d1 = 44.00
R = 200.00
v = 0.00189953
N = 4771.233
Ac = 125663.706
ω = ω*Min(1,1.25*(lb/ld)^2/3) = 0.3024918

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 = 1.3337E+008

φ = 1.06465
φ' = 0.94240061

error of function (3.68), Biskinis Phd = 25149.978
From 5A.2, TBDY: fcc = fc*λc = 20.00
conf. factor λc = 1.00
fc = 20.00
From 10.3.5, ASCE41-17, Final value of fy: fy*Min(1,1.25*(lb/ld)^2/3) = 311.2056
lb/ld = 0.30
d1 = 44.00
R = 200.00
v = 0.00189953
N = 4771.233
Ac = 125663.706
ω = ω*Min(1,1.25*(lb/ld)^2/3) = 0.3024918

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 = 1.3337E+008

φ = 1.06465
φ' = 0.94240061
error of function (3.68), Biskinis Phd = 25149.978
From 5A.2, TBDY: fcc = fc*λc = 20.00
conf. factor λc = 1.00
fc = 20.00
From 10.3.5, ASCE41-17, Final value of fy: fy*Min(1,1.25*(lb/ld)^2/3) = 311.2056
lb/ld = 0.30
d1 = 44.00
R = 200.00
v = 0.00189953
N = 4771.233
Ac = 125663.706
ω = ω*Min(1,1.25*(lb/ld)^2/3) = 0.3024918

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 = 1.3337E+008

φ = 1.06465
φ' = 0.94240061
error of function (3.68), Biskinis Phd = 25149.978
From 5A.2, TBDY: fcc = fc*λc = 20.00
conf. factor λc = 1.00
fc = 20.00
From 10.3.5, ASCE41-17, Final value of fy: fy*Min(1,1.25*(lb/ld)^2/3) = 311.2056
lb/ld = 0.30
d1 = 44.00
R = 200.00
v = 0.00189953

N = 4771.233
Ac = 125663.706
ω = ω*Min(1,1.25*(lb/ld)^2/3) = 0.3024918

Calculation of ratio lb/ld

Inadequate Lap Length with lb/ld = 0.30

Calculation of Shear Strength Vr = Min(Vr1,Vr2) = 288406.767

Calculation of Shear Strength at edge 1, Vr1 = 288406.767
Vr1 = VCol ((10.3), ASCE 41-17) = knl*VCol0
VCol0 = 288406.767
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)
fc' = 20.00, but fc^0.5 <= 8.3 MPa (22.5.3.1, ACI 318-14)
M/Vd = 2.00
Mu = 2.9652000E-012
Vu = 4.2497275E-031
d = 0.8*D = 320.00
Nu = 4771.233
Ag = 125663.706
From (11.5.4.8), ACI 318-14: Vs = 175457.879
Av = π/2*A_stirrup = 123370.055
fy = 444.44
s = 100.00
Vs is multiplied by αCol = 0.00
s/d = 0.3125
Vf ((11-3)-(11.4), ACI 440) = 0.00
From (11-11), ACI 440: Vs + Vf <= 238930.50
bw*d = π*d*d/4 = 80424.772

Calculation of Shear Strength at edge 2, Vr2 = 288406.767
Vr2 = VCol ((10.3), ASCE 41-17) = knl*VCol0
VCol0 = 288406.767
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)
fc' = 20.00, but fc^0.5 <= 8.3 MPa (22.5.3.1, ACI 318-14)
M/Vd = 2.00
Mu = 2.9652000E-012
Vu = 4.2497275E-031
d = 0.8*D = 320.00
Nu = 4771.233
Ag = 125663.706
From (11.5.4.8), ACI 318-14: Vs = 175457.879
Av = π/2*A_stirrup = 123370.055
fy = 444.44
s = 100.00
Vs is multiplied by αCol = 0.00
s/d = 0.3125
Vf ((11-3)-(11.4), ACI 440) = 0.00
From (11-11), ACI 440: Vs + Vf <= 238930.50
bw*d = π*d*d/4 = 80424.772

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

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

Constant Properties

Knowledge Factor, κ = 0.80
Mean strength values are used for both shear and moment calculations.
Consequently:
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
Existing material: Steel Strength, fs = 1.25*fsm = 555.55
##
Diameter, D = 400.00
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 = 3.9443045E-031
EDGE -B-
Shear Force, Vb = -3.9443045E-031
BOTH EDGES
Axial Force, F = -4771.233
Longitudinal Reinforcement Area Distribution (in 2 divisions)
-Tension: Aslt = 0.00
-Compression: Aslc = 3053.628
Longitudinal Reinforcement Area Distribution (in 3 divisions)
-Tension: Asl,ten = 1017.876
-Compression: Asl,com = 1017.876
-Middle: Asl,mid = 1017.876

Calculation of Shear Capacity ratio , Ve/Vr = 0.30828827
Member Controlled by Flexure (Ve/Vr < 1)
Calculation of Shear Demand from fig. R18.6.5, ACI 318-14 Ve = (Mpr1 + Mpr2)/ln = 88912.422
with
Mpr1 = Max(Mu1+ , Mu1-) = 1.3337E+008
Mu1+ = 1.3337E+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- = 1.3337E+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-) = 1.3337E+008
Mu2+ = 1.3337E+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- = 1.3337E+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 = 1.3337E+008

φ = 1.06465
φ' = 0.94240061
error of function (3.68), Biskinis Phd = 25149.978
From 5A.2, TBDY: fcc = fc*λc = 20.00
conf. factor λc = 1.00
fc = 20.00
From 10.3.5, ASCE41-17, Final value of fy: fy*Min(1,1.25*(lb/ld)^2/3) = 311.2056
lb/ld = 0.30
d1 = 44.00
R = 200.00
v = 0.00189953
N = 4771.233
Ac = 125663.706
ω = ω*Min(1,1.25*(lb/ld)^2/3) = 0.3024918

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 = 1.3337E+008

φ = 1.06465
φ' = 0.94240061
error of function (3.68), Biskinis Phd = 25149.978
From 5A.2, TBDY: fcc = fc*λc = 20.00
conf. factor λc = 1.00
fc = 20.00
From 10.3.5, ASCE41-17, Final value of fy: fy*Min(1,1.25*(lb/ld)^2/3) = 311.2056
lb/ld = 0.30
d1 = 44.00
R = 200.00
v = 0.00189953
N = 4771.233
Ac = 125663.706
ω = ω*Min(1,1.25*(lb/ld)^2/3) = 0.3024918

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 = 1.3337E+008

φ = 1.06465
φ' = 0.94240061
error of function (3.68), Biskinis Phd = 25149.978
From 5A.2, TBDY: fcc = fc*λc = 20.00
conf. factor λc = 1.00
fc = 20.00
From 10.3.5, ASCE41-17, Final value of fy: fy*Min(1,1.25*(lb/ld)^2/3) = 311.2056
lb/ld = 0.30
d1 = 44.00
R = 200.00
v = 0.00189953
N = 4771.233
Ac = 125663.706
ω = ω*Min(1,1.25*(lb/ld)^2/3) = 0.3024918

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 = 1.3337E+008

φ = 1.06465
φ' = 0.94240061
error of function (3.68), Biskinis Phd = 25149.978
From 5A.2, TBDY: fcc = fc*λc = 20.00
conf. factor λc = 1.00
fc = 20.00
From 10.3.5, ASCE41-17, Final value of fy: fy*Min(1,1.25*(lb/ld)^2/3) = 311.2056
lb/ld = 0.30
d1 = 44.00
R = 200.00
v = 0.00189953
N = 4771.233
Ac = 125663.706
ω = ω*Min(1,1.25*(lb/ld)^2/3) = 0.3024918

Calculation of ratio lb/ld

Inadequate Lap Length with lb/ld = 0.30

Calculation of Shear Strength Vr = Min(Vr1,Vr2) = 288406.767

Calculation of Shear Strength at edge 1, Vr1 = 288406.767
Vr1 = VCol ((10.3), ASCE 41-17) = knl*VCol0
VCol0 = 288406.767
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)
fc' = 20.00, but fc^0.5 <= 8.3 MPa (22.5.3.1, ACI 318-14)
M/Vd = 2.00
Mu = 8.1661822E-012
Vu = 3.9443045E-031
d = 0.8*D = 320.00
Nu = 4771.233
Ag = 125663.706
From (11.5.4.8), ACI 318-14: Vs = 175457.879
Av = π/2*A_stirrup = 123370.055
fy = 444.44
s = 100.00
Vs is multiplied by αCol = 0.00
s/d = 0.3125
Vf ((11-3)-(11.4), ACI 440) = 0.00
From (11-11), ACI 440: Vs + Vf <= 238930.50
bw*d = π*d*d/4 = 80424.772

Calculation of Shear Strength at edge 2, Vr2 = 288406.767
Vr2 = VCol ((10.3), ASCE 41-17) = knl*VCol0
VCol0 = 288406.767
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)
fc' = 20.00, but fc^0.5 <= 8.3 MPa (22.5.3.1, ACI 318-14)
M/Vd = 2.00
Mu = 8.1661822E-012
Vu = 3.9443045E-031
d = 0.8*D = 320.00
Nu = 4771.233
Ag = 125663.706
From (11.5.4.8), ACI 318-14: Vs = 175457.879
Av = π/2*A_stirrup = 123370.055
fy = 444.44
s = 100.00
Vs is multiplied by αCol = 0.00
s/d = 0.3125
Vf ((11-3)-(11.4), ACI 440) = 0.00
From (11-11), ACI 440: Vs + Vf <= 238930.50
bw*d = π*d*d/4 = 80424.772

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

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

Constant Properties

Knowledge Factor, κ = 0.80
Chord Rotation is generally considered as Deformation-Controlled Action according to Table C7-1, ASCE41-17.
Mean strengths are used for Deformation-Controlled Actions according to 7.5.1.3, ASCE 41-17
Consequently:
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
Diameter, D = 400.00
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 = -1.2370E+007
Shear Force, V2 = -4121.027
Shear Force, V3 = -2.0556142E-013
Axial Force, F = -4774.051
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 = 0.03671513
θu = θy + θp = 0.04589391

- Calculation of θy -

θy = (My*Ls/3)/Eleff = 0.01642397 ((4.29),Biskinis Phd))
My = 1.3007E+008
Ls = M/V (with Ls >0.1*L and Ls < 2*L) = 3001.757
From table 10.5, ASCE 41_17: Eleff = factor*Ec*Ig = 7.9240E+012
factor = 0.30
Ag = 125663.706
fc' = 20.00
N = 4774.051
Ec*Ig = 2.6413E+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) = 1.3007E+008
Φy = 7.1483407E-006
My_ten (8c) = 1.3007E+008
Φ_ten (7c) = 75.93136
error of function (7c) = 0.00012641
My_com (8d) = 3.4649E+008
Φ_com (7d) = 70.9694
error of function (7d) = -0.0005181
with ((10.1), ASCE 41-17) ey = Min(ey, 1.25*ey*(lb/ld)^2/3) = 0.0022222
eco = 0.002
apl = 0.35 ((9a) in Biskinis and Fardis for no FRP Wrap)
d1 = 44.00
R = 200.00
v = 0.00189953

N = 4774.051
Ac = 125663.706
((10.1), ASCE 41-17) ω = Min(ω, 1.25*ω*(lb/ld)^2/3) = 0.5399946
with fc = 20.00

Calculation of ratio lb/ld

Inadequate Lap Length with lb/ld = 0.30

- Calculation of θp -

From table 10-9: θp = 0.02946994
with:
- Columns not controlled by inadequate development or splicing along the clear height because lb/ld >= 1
shear control ratio VyE/VCol0E = 0.30828827
d = 0.00
s = 0.00
ρt = 2*Av/(dc*s) + 4*tf/D*(ffe/fs) = 0.00
Av = 78.53982, is the area of the circular stirrup
dc = D - 2*cover - Hoop Diameter = 340.00
The term 2*tf/bw*(ffe/fs) is implemented to account for FRP contribution
where 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.
NUD = 4774.051
Ag = 125663.706
f'cE = 20.00
fytE = fylE = 444.44
pl = Area_Tot_Long_Rein/(Ag) = 0.0243
f'cE = 20.00

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

Calculation No. 13

column C1, Floor 1

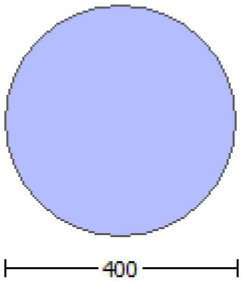
Limit State: Life Safety (data interpolation between analysis steps 2 and 3)

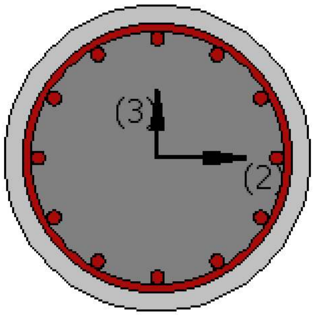
Analysis: Uniform +X

Check: Shear capacity VRd

Edge: End

Local Axis: (2)





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

Constant Properties

Knowledge Factor, κ = 0.80
Member Shear Force is generally considered as Force-Controlled Action according to Table C7-1, ASCE41-17.
Lower-bound strengths are used for Force-Controlled Actions according to 7.5.1.3, ASCE 41-17
Consequently:
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, ASCE41-17) because bending is considered as
Deformation-Controlled Action (Table C7-1, ASCE41-17).
Existing material: Concrete Strength, fc = fcm = 20.00
Existing material: Steel Strength, fs = fsm = 444.44
###
Diameter, D = 400.00
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 = -1.2370E+007
Shear Force, Va = -4121.027
EDGE -B-
Bending Moment, Mb = 876.0544
Shear Force, Vb = 4121.027
BOTH EDGES
Axial Force, F = -4774.051
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 = 207230.904
Vn ((10.3), ASCE 41-17) = knl*VCol0 = 259038.631
VCol = 259038.631
knl = 1.00
displacement_ductility_demand = 0.14266428

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)
fc' = 16.00, but fc^0.5 <= 8.3 MPa (22.5.3.1, ACI 318-14)
M/Vd = 2.00
Mu = 876.0544
Vu = 4121.027
d = 0.8*D = 320.00
Nu = 4774.051
Ag = 125663.706
From (11.5.4.8), ACI 318-14: Vs = 157913.67
Av = π/2*A_stirrup = 123370.055
fy = 400.00
s = 100.00
Vs is multiplied by αCol = 0.00
s/d = 0.3125
Vf ((11-3)-(11.4), ACI 440) = 0.00
From (11-11), ACI 440: Vs + Vf <= 213705.936
bw*d = π*d*d/4 = 80424.772

----
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 = 0.00023417
θy = (My*Ls/3)/Eleff = 0.00164144 ((4.29),Biskinis Phd))
My = 1.3007E+008
Ls = M/V (with Ls >0.1*L and Ls < 2*L) = 300.00
From table 10.5, ASCE 41_17: Eleff = factor*Ec*Ig = 7.9240E+012
factor = 0.30
Ag = 125663.706
fc' = 20.00
N = 4774.051
Ec*Ig = 2.6413E+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) = 1.3007E+008
Φy = 7.1483407E-006
My_ten (8c) = 1.3007E+008
Φ_ten (7c) = 75.93136
error of function (7c) = 0.00012641
My_com (8d) = 3.4649E+008
Φ_com (7d) = 70.9694
error of function (7d) = -0.0005181
with ((10.1), ASCE 41-17) ey = Min(ey, 1.25*ey*(lb/ld)^2/3) = 0.0022222
eco = 0.002
apl = 0.35 ((9a) in Biskinis and Fardis for no FRP Wrap)
d1 = 44.00

R = 200.00
v = 0.00189953
N = 4774.051
Ac = 125663.706
((10.1), ASCE 41-17) ω = Min(ω, 1.25*ω*(lb/ld)^2/3) = 0.5399946
with fc = 20.00

Calculation of ratio lb/ld

Inadequate Lap Length with lb/ld = 0.30

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

Calculation No. 14

column C1, Floor 1

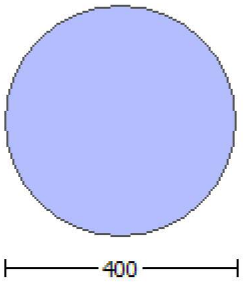
Limit State: Life Safety (data interpolation between analysis steps 2 and 3)

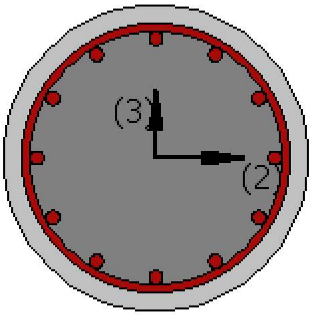
Analysis: Uniform +X

Check: Chord rotation capacity (θu)

Edge: End

Local Axis: (2)





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

Constant Properties

Knowledge Factor, κ = 0.80
Mean strength values are used for both shear and moment calculations.
Consequently:
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
Existing material: Steel Strength, fs = 1.25*fsm = 555.55
##
Diameter, D = 400.00
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 = -4.2497275E-031
EDGE -B-
Shear Force, Vb = 4.2497275E-031
BOTH EDGES
Axial Force, F = -4771.233
Longitudinal Reinforcement Area Distribution (in 2 divisions)
-Tension: Aslt = 0.00
-Compression: Aslc = 3053.628
Longitudinal Reinforcement Area Distribution (in 3 divisions)
-Tension: Asl,ten = 1017.876
-Compression: Asl,com = 1017.876
-Middle: Asl,mid = 1017.876

Calculation of Shear Capacity ratio , Ve/Vr = 0.30828827
Member Controlled by Flexure (Ve/Vr < 1)
Calculation of Shear Demand from fig. R18.6.5, ACI 318-14 Ve = (Mpr1 + Mpr2)/ln = 88912.422
with
Mpr1 = Max(Mu1+ , Mu1-) = 1.3337E+008
Mu1+ = 1.3337E+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- = 1.3337E+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-) = 1.3337E+008
Mu2+ = 1.3337E+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- = 1.3337E+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 = 1.3337E+008

φ = 1.06465
φ' = 0.94240061
error of function (3.68), Biskinis Phd = 25149.978
From 5A.2, TBDY: fcc = fc*λc = 20.00

conf. factor λc = 1.00
fc = 20.00
From 10.3.5, ASCE41-17, Final value of fy: fy*Min(1,1.25*(lb/ld)^2/3) = 311.2056
lb/ld = 0.30
d1 = 44.00
R = 200.00
v = 0.00189953
N = 4771.233
Ac = 125663.706
ω = ω*Min(1,1.25*(lb/ld)^2/3) = 0.3024918

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 = 1.3337E+008

φ = 1.06465
φ' = 0.94240061
error of function (3.68), Biskinis Phd = 25149.978
From 5A.2, TBDY: fcc = fc*λc = 20.00
conf. factor λc = 1.00
fc = 20.00
From 10.3.5, ASCE41-17, Final value of fy: fy*Min(1,1.25*(lb/ld)^2/3) = 311.2056
lb/ld = 0.30
d1 = 44.00
R = 200.00
v = 0.00189953
N = 4771.233
Ac = 125663.706
ω = ω*Min(1,1.25*(lb/ld)^2/3) = 0.3024918

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 = 1.3337E+008

φ = 1.06465
φ' = 0.94240061
error of function (3.68), Biskinis Phd = 25149.978
From 5A.2, TBDY: fcc = fc*λc = 20.00
conf. factor λc = 1.00
fc = 20.00
From 10.3.5, ASCE41-17, Final value of fy: fy*Min(1,1.25*(lb/ld)^2/3) = 311.2056
lb/ld = 0.30
d1 = 44.00
R = 200.00
v = 0.00189953
N = 4771.233
Ac = 125663.706

ω = ω*Min(1,1.25*(lb/ld)^2/3) = 0.3024918

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 = 1.3337E+008

φ = 1.06465
φ' = 0.94240061
error of function (3.68), Biskinis Phd = 25149.978
From 5A.2, TBDY: fcc = fc*λc = 20.00
conf. factor λc = 1.00
fc = 20.00
From 10.3.5, ASCE41-17, Final value of fy: fy*Min(1,1.25*(lb/ld)^2/3) = 311.2056
lb/ld = 0.30
d1 = 44.00
R = 200.00
v = 0.00189953
N = 4771.233
Ac = 125663.706
ω = ω*Min(1,1.25*(lb/ld)^2/3) = 0.3024918

Calculation of ratio lb/ld

Inadequate Lap Length with lb/ld = 0.30

Calculation of Shear Strength Vr = Min(Vr1,Vr2) = 288406.767

Calculation of Shear Strength at edge 1, Vr1 = 288406.767
Vr1 = VCol ((10.3), ASCE 41-17) = knl*VCol0
VCol0 = 288406.767
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)
fc' = 20.00, but fc^0.5 <= 8.3 MPa (22.5.3.1, ACI 318-14)
M/Vd = 2.00
Mu = 2.9652000E-012
Vu = 4.2497275E-031
d = 0.8*D = 320.00
Nu = 4771.233
Ag = 125663.706
From (11.5.4.8), ACI 318-14: Vs = 175457.879
Av = π/2*A_stirrup = 123370.055
fy = 444.44
s = 100.00
Vs is multiplied by αCol = 0.00
s/d = 0.3125
Vf ((11-3)-(11.4), ACI 440) = 0.00
From (11-11), ACI 440: Vs + Vf <= 238930.50
bw*d = π*d*d/4 = 80424.772

Calculation of Shear Strength at edge 2, Vr2 = 288406.767
Vr2 = VCol ((10.3), ASCE 41-17) = knl*VCol0
VCol0 = 288406.767
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)
fc' = 20.00, but fc^0.5 <= 8.3 MPa (22.5.3.1, ACI 318-14)
M/Vd = 2.00
Mu = 2.9652000E-012
Vu = 4.2497275E-031
d = 0.8*D = 320.00
Nu = 4771.233
Ag = 125663.706
From (11.5.4.8), ACI 318-14: Vs = 175457.879
Av = π/2*A_stirrup = 123370.055
fy = 444.44
s = 100.00
Vs is multiplied by αCol = 0.00
s/d = 0.3125
Vf ((11-3)-(11.4), ACI 440) = 0.00
From (11-11), ACI 440: Vs + Vf <= 238930.50
bw*d = π*d*d/4 = 80424.772

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

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

Constant Properties

Knowledge Factor, κ = 0.80
Mean strength values are used for both shear and moment calculations.
Consequently:
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
Existing material: Steel Strength, fs = 1.25*fsm = 555.55
###
Diameter, D = 400.00
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 = 3.9443045E-031
EDGE -B-
Shear Force, Vb = -3.9443045E-031
BOTH EDGES
Axial Force, F = -4771.233
Longitudinal Reinforcement Area Distribution (in 2 divisions)
-Tension: Aslt = 0.00
-Compression: Aslc = 3053.628
Longitudinal Reinforcement Area Distribution (in 3 divisions)
-Tension: Asl,ten = 1017.876
-Compression: Asl,com = 1017.876
-Middle: Asl,mid = 1017.876

Calculation of Shear Capacity ratio , Ve/Vr = 0.30828827
Member Controlled by Flexure (Ve/Vr < 1)
Calculation of Shear Demand from fig. R18.6.5, ACI 318-14 Ve = (Mpr1 + Mpr2)/ln = 88912.422
with
Mpr1 = Max(Mu1+ , Mu1-) = 1.3337E+008
Mu1+ = 1.3337E+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- = 1.3337E+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-) = 1.3337E+008
Mu2+ = 1.3337E+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- = 1.3337E+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 = 1.3337E+008

φ = 1.06465
φ' = 0.94240061
error of function (3.68), Biskinis Phd = 25149.978
From 5A.2, TBDY: fcc = fc*λc = 20.00
conf. factor λc = 1.00
fc = 20.00
From 10.3.5, ASCE41-17, Final value of fy: fy*Min(1,1.25*(lb/ld)^2/3) = 311.2056
lb/ld = 0.30
d1 = 44.00
R = 200.00
v = 0.00189953
N = 4771.233
Ac = 125663.706
ω = ω*Min(1,1.25*(lb/ld)^2/3) = 0.3024918

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 = 1.3337E+008

φ = 1.06465
φ' = 0.94240061
error of function (3.68), Biskinis Phd = 25149.978
From 5A.2, TBDY: fcc = fc*λc = 20.00
conf. factor λc = 1.00
fc = 20.00
From 10.3.5, ASCE41-17, Final value of fy: fy*Min(1,1.25*(lb/ld)^2/3) = 311.2056
lb/ld = 0.30
d1 = 44.00
R = 200.00
v = 0.00189953
N = 4771.233
Ac = 125663.706
ω = ω*Min(1,1.25*(lb/ld)^2/3) = 0.3024918

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 = 1.3337E+008

φ = 1.06465
φ' = 0.94240061
error of function (3.68), Biskinis Phd = 25149.978
From 5A.2, TBDY: fcc = fc*λc = 20.00
conf. factor λc = 1.00
fc = 20.00
From 10.3.5, ASCE41-17, Final value of fy: fy*Min(1,1.25*(lb/ld)^2/3) = 311.2056
lb/ld = 0.30
d1 = 44.00
R = 200.00
v = 0.00189953
N = 4771.233
Ac = 125663.706
ω = ω*Min(1,1.25*(lb/ld)^2/3) = 0.3024918

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 = 1.3337E+008

φ = 1.06465
φ' = 0.94240061
error of function (3.68), Biskinis Phd = 25149.978
From 5A.2, TBDY: fcc = fc*λc = 20.00
conf. factor λc = 1.00

fc = 20.00
From 10.3.5, ASCE41-17, Final value of fy: fy*Min(1,1.25*(lb/ld)^2/3) = 311.2056
lb/ld = 0.30
d1 = 44.00
R = 200.00
v = 0.00189953
N = 4771.233
Ac = 125663.706
ω = ω*Min(1,1.25*(lb/ld)^2/3) = 0.3024918

Calculation of ratio lb/ld

Inadequate Lap Length with lb/ld = 0.30

Calculation of Shear Strength Vr = Min(Vr1,Vr2) = 288406.767

Calculation of Shear Strength at edge 1, Vr1 = 288406.767
Vr1 = VCol ((10.3), ASCE 41-17) = knl*VCol0
VCol0 = 288406.767
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)
fc' = 20.00, but fc^0.5 <= 8.3 MPa (22.5.3.1, ACI 318-14)
M/Vd = 2.00
Mu = 8.1661822E-012
Vu = 3.9443045E-031
d = 0.8*D = 320.00
Nu = 4771.233
Ag = 125663.706
From (11.5.4.8), ACI 318-14: Vs = 175457.879
Av = π/2*A_stirrup = 123370.055
fy = 444.44
s = 100.00
Vs is multiplied by αCol = 0.00
s/d = 0.3125
Vf ((11-3)-(11.4), ACI 440) = 0.00
From (11-11), ACI 440: Vs + Vf <= 238930.50
bw*d = π*d*d/4 = 80424.772

Calculation of Shear Strength at edge 2, Vr2 = 288406.767
Vr2 = VCol ((10.3), ASCE 41-17) = knl*VCol0
VCol0 = 288406.767
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)
fc' = 20.00, but fc^0.5 <= 8.3 MPa (22.5.3.1, ACI 318-14)
M/Vd = 2.00
Mu = 8.1661822E-012
Vu = 3.9443045E-031
d = 0.8*D = 320.00
Nu = 4771.233
Ag = 125663.706
From (11.5.4.8), ACI 318-14: Vs = 175457.879
Av = π/2*A_stirrup = 123370.055
fy = 444.44
s = 100.00

Vs is multiplied by αCol = 0.00
s/d = 0.3125
Vf ((11-3)-(11.4), ACI 440) = 0.00
From (11-11), ACI 440: Vs + Vf <= 238930.50
bw*d = π*d*d/4 = 80424.772

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

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

Constant Properties

Knowledge Factor, κ = 0.80
Chord Rotation is generally considered as Deformation-Controlled Action according to Table C7-1, ASCE41-17.
Mean strengths are used for Deformation-Controlled Actions according to 7.5.1.3, ASCE 41-17
Consequently:
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
Diameter, D = 400.00
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 = -5.1597350E-011
Shear Force, V2 = 4121.027
Shear Force, V3 = 2.0556142E-013
Axial Force, F = -4774.051
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 = 0.03014169
θu = θy + θp = 0.03767711

- Calculation of θy -

θy = (My*Ls/3)/EIeff = 0.00820718 ((4.29),Biskinis Phd))
My = 1.3007E+008
Ls = M/V (with Ls >0.1*L and Ls < 2*L) = 1500.00

From table 10.5, ASCE 41_17: Eleff = factor*Ec*Ig = 7.9240E+012
factor = 0.30
Ag = 125663.706
fc' = 20.00
N = 4774.051
Ec*Ig = 2.6413E+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) = 1.3007E+008
Φy = 7.1483407E-006
My_ten (8c) = 1.3007E+008
Φ_ten (7c) = 75.93136
error of function (7c) = 0.00012641
My_com (8d) = 3.4649E+008
Φ_com (7d) = 70.9694
error of function (7d) = -0.0005181
with ((10.1), ASCE 41-17) ey = Min(ey, 1.25*ey*(lb/ld)^2/3) = 0.0022222
eco = 0.002
apl = 0.35 ((9a) in Biskinis and Fardis for no FRP Wrap)
d1 = 44.00
R = 200.00
v = 0.00189953
N = 4774.051
Ac = 125663.706
((10.1), ASCE 41-17) ω = Min(ω, 1.25*ω*(lb/ld)^2/3) = 0.5399946
with fc = 20.00

Calculation of ratio lb/ld

Inadequate Lap Length with lb/ld = 0.30

- Calculation of θp -

From table 10-9: θp = 0.02946994
with:
- Columns not controlled by inadequate development or splicing along the clear height because lb/ld >= 1
shear control ratio VyE/VCol0E = 0.30828827
d = 0.00
s = 0.00
ρt = 2*Av/(dc*s) + 4*tf/D*(ffe/fs) = 0.00
Av = 78.53982, is the area of the circular stirrup
dc = D - 2*cover - Hoop Diameter = 340.00
The term 2*tf/bw*(ffe/fs) is implemented to account for FRP contribution
where 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.
NUD = 4774.051
Ag = 125663.706
f'cE = 20.00
fytE = fylE = 444.44
pl = Area_Tot_Long_Rein/(Ag) = 0.0243
f'cE = 20.00

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

Calculation No. 15

column C1, Floor 1

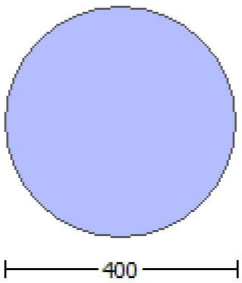
Limit State: Life Safety (data interpolation between analysis steps 2 and 3)

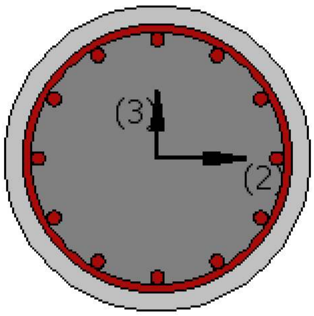
Analysis: Uniform +X

Check: Shear capacity VRd

Edge: End

Local Axis: (3)





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

Constant Properties

Knowledge Factor, κ = 0.80
Member Shear Force is generally considered as Force-Controlled Action according to Table C7-1, ASCE41-17.
Lower-bound strengths are used for Force-Controlled Actions according to 7.5.1.3, ASCE 41-17
Consequently:
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, ASCE41-17) because bending is considered as
Deformation-Controlled Action (Table C7-1, ASCE41-17).
Existing material: Concrete Strength, fc = fcm = 20.00
Existing material: Steel Strength, fs = fsm = 444.44
##
Diameter, D = 400.00
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 = 6.6860422E-010
Shear Force, Va = -2.0556142E-013
EDGE -B-
Bending Moment, Mb = -5.1597350E-011
Shear Force, Vb = 2.0556142E-013
BOTH EDGES
Axial Force, F = -4774.051
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 = 207230.904
Vn ((10.3), ASCE 41-17) = knl*VCol0 = 259038.631
VCol = 259038.631
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)
fc' = 16.00, but fc^0.5 <= 8.3 MPa (22.5.3.1, ACI 318-14)
M/Vd = 2.00
Mu = 5.1597350E-011
Vu = 2.0556142E-013
d = 0.8*D = 320.00
Nu = 4774.051
Ag = 125663.706
From (11.5.4.8), ACI 318-14: Vs = 157913.67
Av = π/2*A_stirrup = 123370.055
fy = 400.00
s = 100.00
Vs is multiplied by αCol = 0.00
s/d = 0.3125
Vf ((11-3)-(11.4), ACI 440) = 0.00
From (11-11), ACI 440: Vs + Vf <= 213705.936
bw*d = π*d*d/4 = 80424.772

----
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 = 1.3951720E-020
θy = (My*Ls/3)/Eleff = 0.00820718 ((4.29),Biskinis Phd))
My = 1.3007E+008
Ls = M/V (with Ls >0.1*L and Ls < 2*L) = 1500.00
From table 10.5, ASCE 41_17: Eleff = factor*Ec*Ig = 7.9240E+012
factor = 0.30
Ag = 125663.706
fc' = 20.00

N = 4774.051
Ec*Ig = 2.6413E+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) = 1.3007E+008
Φy = 7.1483407E-006
My_ten (8c) = 1.3007E+008
Φ_ten (7c) = 75.93136
error of function (7c) = 0.00012641
My_com (8d) = 3.4649E+008
Φ_com (7d) = 70.9694
error of function (7d) = -0.0005181
with ((10.1), ASCE 41-17) ey = Min(ey, 1.25*ey*(lb/ld)^2/3) = 0.0022222
eco = 0.002
apl = 0.35 ((9a) in Biskinis and Fardis for no FRP Wrap)
d1 = 44.00
R = 200.00
v = 0.00189953
N = 4774.051
Ac = 125663.706
((10.1), ASCE 41-17) ω = Min(ω, 1.25*ω*(lb/ld)^2/3) = 0.5399946
with fc = 20.00

Calculation of ratio lb/ld

Inadequate Lap Length with lb/ld = 0.30

End Of Calculation of Shear Capacity for element: column CC1 of floor 1
At local axis: 3
Integration Section: (b)

Calculation No. 16

column C1, Floor 1

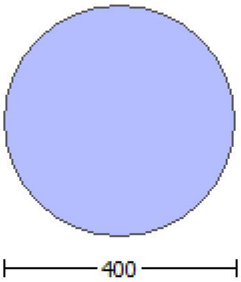
Limit State: Life Safety (data interpolation between analysis steps 2 and 3)

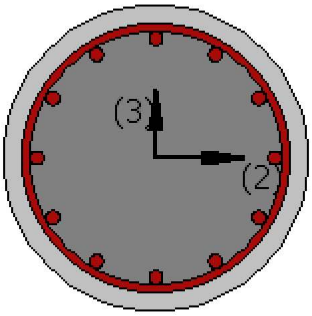
Analysis: Uniform +X

Check: Chord rotation capacity (θu)

Edge: End

Local Axis: (3)





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

Constant Properties

Knowledge Factor, κ = 0.80
Mean strength values are used for both shear and moment calculations.
Consequently:
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
Existing material: Steel Strength, fs = 1.25*fsm = 555.55
###
Diameter, D = 400.00
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 = -4.2497275E-031
EDGE -B-
Shear Force, Vb = 4.2497275E-031
BOTH EDGES
Axial Force, F = -4771.233
Longitudinal Reinforcement Area Distribution (in 2 divisions)
-Tension: Aslt = 0.00
-Compression: Aslc = 3053.628
Longitudinal Reinforcement Area Distribution (in 3 divisions)
-Tension: Asl,ten = 1017.876
-Compression: Asl,com = 1017.876
-Middle: Asl,mid = 1017.876

Calculation of Shear Capacity ratio , Ve/Vr = 0.30828827
Member Controlled by Flexure (Ve/Vr < 1)
Calculation of Shear Demand from fig. R18.6.5, ACI 318-14 Ve = (Mpr1 + Mpr2)/ln = 88912.422
with
Mpr1 = Max(Mu1+ , Mu1-) = 1.3337E+008
Mu1+ = 1.3337E+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- = 1.3337E+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-) = 1.3337E+008
Mu2+ = 1.3337E+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- = 1.3337E+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 = 1.3337E+008

φ = 1.06465
φ' = 0.94240061
error of function (3.68), Biskinis Phd = 25149.978
From 5A.2, TBDY: fcc = fc*λc = 20.00
conf. factor λc = 1.00
fc = 20.00
From 10.3.5, ASCE41-17, Final value of fy: fy*Min(1,1.25*(lb/ld)^2/3) = 311.2056
lb/ld = 0.30
d1 = 44.00
R = 200.00
v = 0.00189953
N = 4771.233
Ac = 125663.706
ω = ω*Min(1,1.25*(lb/ld)^2/3) = 0.3024918

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 = 1.3337E+008

φ = 1.06465
φ' = 0.94240061
error of function (3.68), Biskinis Phd = 25149.978
From 5A.2, TBDY: fcc = fc*λc = 20.00
conf. factor λc = 1.00
fc = 20.00
From 10.3.5, ASCE41-17, Final value of fy: fy*Min(1,1.25*(lb/ld)^2/3) = 311.2056
lb/ld = 0.30
d1 = 44.00
R = 200.00
v = 0.00189953
N = 4771.233
Ac = 125663.706
ω = ω*Min(1,1.25*(lb/ld)^2/3) = 0.3024918

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 = 1.3337E+008

φ = 1.06465
φ' = 0.94240061
error of function (3.68), Biskinis Phd = 25149.978
From 5A.2, TBDY: fcc = fc*λc = 20.00
conf. factor λc = 1.00
fc = 20.00
From 10.3.5, ASCE41-17, Final value of fy: fy*Min(1,1.25*(lb/ld)^2/3) = 311.2056
lb/ld = 0.30
d1 = 44.00
R = 200.00
v = 0.00189953
N = 4771.233
Ac = 125663.706
ω = ω*Min(1,1.25*(lb/ld)^2/3) = 0.3024918

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 = 1.3337E+008

φ = 1.06465
φ' = 0.94240061
error of function (3.68), Biskinis Phd = 25149.978
From 5A.2, TBDY: fcc = fc*λc = 20.00
conf. factor λc = 1.00
fc = 20.00
From 10.3.5, ASCE41-17, Final value of fy: fy*Min(1,1.25*(lb/ld)^2/3) = 311.2056
lb/ld = 0.30
d1 = 44.00
R = 200.00
v = 0.00189953
N = 4771.233
Ac = 125663.706
ω = ω*Min(1,1.25*(lb/ld)^2/3) = 0.3024918

Calculation of ratio lb/ld

Inadequate Lap Length with lb/ld = 0.30

Calculation of Shear Strength Vr = Min(Vr1,Vr2) = 288406.767

Calculation of Shear Strength at edge 1, Vr1 = 288406.767
Vr1 = VCol ((10.3), ASCE 41-17) = knl*VCol0
VCol0 = 288406.767
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)
fc' = 20.00, but fc^0.5 <= 8.3 MPa (22.5.3.1, ACI 318-14)
M/Vd = 2.00
Mu = 2.9652000E-012
Vu = 4.2497275E-031
d = 0.8*D = 320.00
Nu = 4771.233
Ag = 125663.706
From (11.5.4.8), ACI 318-14: Vs = 175457.879
Av = π/2*A_stirrup = 123370.055
fy = 444.44
s = 100.00
Vs is multiplied by αCol = 0.00
s/d = 0.3125
Vf ((11-3)-(11.4), ACI 440) = 0.00
From (11-11), ACI 440: Vs + Vf <= 238930.50
bw*d = π*d*d/4 = 80424.772

Calculation of Shear Strength at edge 2, Vr2 = 288406.767
Vr2 = VCol ((10.3), ASCE 41-17) = knl*VCol0
VCol0 = 288406.767
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)
fc' = 20.00, but fc^0.5 <= 8.3 MPa (22.5.3.1, ACI 318-14)
M/Vd = 2.00
Mu = 2.9652000E-012
Vu = 4.2497275E-031
d = 0.8*D = 320.00
Nu = 4771.233
Ag = 125663.706
From (11.5.4.8), ACI 318-14: Vs = 175457.879
Av = π/2*A_stirrup = 123370.055
fy = 444.44
s = 100.00
Vs is multiplied by αCol = 0.00
s/d = 0.3125
Vf ((11-3)-(11.4), ACI 440) = 0.00
From (11-11), ACI 440: Vs + Vf <= 238930.50
bw*d = π*d*d/4 = 80424.772

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

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

Constant Properties

Knowledge Factor, κ = 0.80
Mean strength values are used for both shear and moment calculations.
Consequently:
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
Existing material: Steel Strength, fs = 1.25*fsm = 555.55
###
Diameter, D = 400.00
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 = 3.9443045E-031
EDGE -B-
Shear Force, Vb = -3.9443045E-031
BOTH EDGES
Axial Force, F = -4771.233
Longitudinal Reinforcement Area Distribution (in 2 divisions)
-Tension: Aslt = 0.00
-Compression: Aslc = 3053.628
Longitudinal Reinforcement Area Distribution (in 3 divisions)
-Tension: Asl,ten = 1017.876
-Compression: Asl,com = 1017.876
-Middle: Asl,mid = 1017.876

Calculation of Shear Capacity ratio , Ve/Vr = 0.30828827
Member Controlled by Flexure (Ve/Vr < 1)
Calculation of Shear Demand from fig. R18.6.5, ACI 318-14 Ve = (Mpr1 + Mpr2)/ln = 88912.422
with
Mpr1 = Max(Mu1+ , Mu1-) = 1.3337E+008
Mu1+ = 1.3337E+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- = 1.3337E+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-) = 1.3337E+008
Mu2+ = 1.3337E+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- = 1.3337E+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 = 1.3337E+008

φ = 1.06465
φ' = 0.94240061
error of function (3.68), Biskinis Phd = 25149.978
From 5A.2, TBDY: fcc = fc*λc = 20.00
conf. factor λc = 1.00
fc = 20.00
From 10.3.5, ASCE41-17, Final value of fy: fy*Min(1,1.25*(lb/ld)^2/3) = 311.2056
lb/ld = 0.30
d1 = 44.00
R = 200.00
v = 0.00189953
N = 4771.233
Ac = 125663.706
ω = ω*Min(1,1.25*(lb/ld)^2/3) = 0.3024918

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 = 1.3337E+008

φ = 1.06465
φ' = 0.94240061
error of function (3.68), Biskinis Phd = 25149.978
From 5A.2, TBDY: fcc = fc*λc = 20.00
conf. factor λc = 1.00
fc = 20.00
From 10.3.5, ASCE41-17, Final value of fy: fy*Min(1,1.25*(lb/ld)^2/3) = 311.2056
lb/ld = 0.30
d1 = 44.00
R = 200.00
v = 0.00189953
N = 4771.233
Ac = 125663.706
ω = ω*Min(1,1.25*(lb/ld)^2/3) = 0.3024918

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 = 1.3337E+008

φ = 1.06465
φ' = 0.94240061
error of function (3.68), Biskinis Phd = 25149.978
From 5A.2, TBDY: fcc = fc*λc = 20.00
conf. factor λc = 1.00
fc = 20.00
From 10.3.5, ASCE41-17, Final value of fy: fy*Min(1,1.25*(lb/ld)^2/3) = 311.2056

lb/ld = 0.30
d1 = 44.00
R = 200.00
v = 0.00189953
N = 4771.233
Ac = 125663.706
ω = ω*Min(1,1.25*(lb/ld)^2/3) = 0.3024918

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 = 1.3337E+008

φ = 1.06465
φ' = 0.94240061
error of function (3.68), Biskinis Phd = 25149.978
From 5A.2, TBDY: fcc = fc*λc = 20.00
conf. factor λc = 1.00
fc = 20.00
From 10.3.5, ASCE41-17, Final value of fy: fy*Min(1,1.25*(lb/ld)^2/3) = 311.2056
lb/ld = 0.30
d1 = 44.00
R = 200.00
v = 0.00189953
N = 4771.233
Ac = 125663.706
ω = ω*Min(1,1.25*(lb/ld)^2/3) = 0.3024918

Calculation of ratio lb/ld

Inadequate Lap Length with lb/ld = 0.30

Calculation of Shear Strength Vr = Min(Vr1,Vr2) = 288406.767

Calculation of Shear Strength at edge 1, Vr1 = 288406.767
Vr1 = VCol ((10.3), ASCE 41-17) = knl*VCol0
VCol0 = 288406.767
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)
fc' = 20.00, but fc^0.5 <= 8.3 MPa (22.5.3.1, ACI 318-14)
M/Vd = 2.00
Mu = 8.1661822E-012
Vu = 3.9443045E-031
d = 0.8*D = 320.00
Nu = 4771.233
Ag = 125663.706
From (11.5.4.8), ACI 318-14: Vs = 175457.879
Av = π/2*A_stirrup = 123370.055
fy = 444.44
s = 100.00

Vs is multiplied by αCol = 0.00
s/d = 0.3125
Vf ((11-3)-(11.4), ACI 440) = 0.00
From (11-11), ACI 440: Vs + Vf <= 238930.50
bw*d = π*d*d/4 = 80424.772

Calculation of Shear Strength at edge 2, Vr2 = 288406.767
Vr2 = VCol ((10.3), ASCE 41-17) = knl*VCol0
VCol0 = 288406.767
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)
fc' = 20.00, but fc^0.5 <= 8.3 MPa (22.5.3.1, ACI 318-14)
M/Vd = 2.00
Mu = 8.1661822E-012
Vu = 3.9443045E-031
d = 0.8*D = 320.00
Nu = 4771.233
Ag = 125663.706
From (11.5.4.8), ACI 318-14: Vs = 175457.879
Av = π/2*A_stirrup = 123370.055
fy = 444.44
s = 100.00
Vs is multiplied by αCol = 0.00
s/d = 0.3125
Vf ((11-3)-(11.4), ACI 440) = 0.00
From (11-11), ACI 440: Vs + Vf <= 238930.50
bw*d = π*d*d/4 = 80424.772

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

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

Constant Properties

Knowledge Factor, κ = 0.80
Chord Rotation is generally considered as Deformation-Controlled Action according to Table C7-1, ASCE41-17.
Mean strengths are used for Deformation-Controlled Actions according to 7.5.1.3, ASCE 41-17
Consequently:
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
Diameter, D = 400.00
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 = 876.0544
Shear Force, V2 = 4121.027
Shear Force, V3 = 2.0556142E-013
Axial Force, F = -4774.051
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 = 0.0248891
θu = θy + θp = 0.03111137

- Calculation of θy -

θy = (My*Ls/3)/EIeff = 0.00164144 ((4.29),Biskinis Phd))
My = 1.3007E+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 = 7.9240E+012
factor = 0.30
Ag = 125663.706
fc' = 20.00
N = 4774.051
Ec*Ig = 2.6413E+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) = 1.3007E+008
Φy = 7.1483407E-006
My_ten (8c) = 1.3007E+008
Φ_ten (7c) = 75.93136
error of function (7c) = 0.00012641
My_com (8d) = 3.4649E+008
Φ_com (7d) = 70.9694
error of function (7d) = -0.0005181
with ((10.1), ASCE 41-17) ey = Min(ey, 1.25*ey*(lb/ld)^2/3) = 0.0022222
eco = 0.002
apl = 0.35 ((9a) in Biskinis and Fardis for no FRP Wrap)
d1 = 44.00
R = 200.00
v = 0.00189953
N = 4774.051
Ac = 125663.706
((10.1), ASCE 41-17) ω = Min(ω, 1.25*ω*(lb/ld)^2/3) = 0.5399946
with fc = 20.00

Calculation of ratio lb/ld

Inadequate Lap Length with lb/ld = 0.30

- Calculation of θp -

From table 10-9: θp = 0.02946994
with:
- Columns not controlled by inadequate development or splicing along the clear height because lb/ld >= 1
shear control ratio VyE/VCol0E = 0.30828827
d = 0.00
s = 0.00
ρt = 2*Av/(dc*s) + 4*tf/D*(ffe/fs) = 0.00
Av = 78.53982, is the area of the circular stirrup
dc = D - 2*cover - Hoop Diameter = 340.00
The term 2*tf/bw*(ffe/fs) is implemented to account for FRP contribution
where 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.
NUD = 4774.051
Ag = 125663.706
f'cE = 20.00
fytE = fylE = 444.44
pl = Area_Tot_Long_Rein/(Ag) = 0.0243
f'cE = 20.00

End Of Calculation of Chord Rotation Capacity for element: column CC1 of floor 1
At local axis: 3
Integration Section: (b)
