Detailed Member Calculations

Units: N&mm

Regulation: ASCE 41-17

Calculation No. 1

column C1, Floor 1

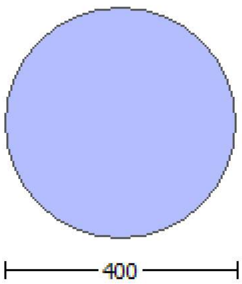
Limit State: Immediate Occupancy (data interpolation between analysis steps 1 and 2)

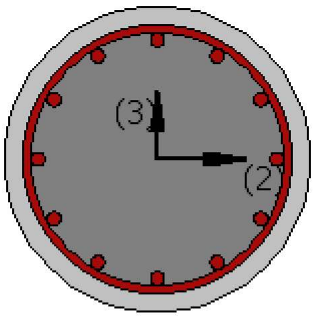
Analysis: Uniform +X

Check: Shear capacity 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 = 0.75
Member Shear Force is generally considered as Force-Controlled Action according to Table C7-1, ASCE41-17.
Lower-bound strengths are used for Force-Controlled Actions according to 7.5.1.3, ASCE 41-17
Consequently:
New material of Primary Member: Concrete Strength, fc = fc_lower_bound = 25.00
New material of Primary Member: Steel Strength, fs = fs_lower_bound = 500.00
Concrete Elasticity, Ec = 26999.444

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).
New material: Concrete Strength, fc = fcm = 33.00
New material: Steel Strength, fs = fsm = 555.56
##
Diameter, D = 400.00
Cover Thickness, c = 25.00
Element Length, L = 3000.00
Primary 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 = -8.9264E+006
Shear Force, Va = -2974.321
EDGE -B-
Bending Moment, Mb = 0.00304337
Shear Force, Vb = 2974.321
BOTH EDGES
Axial Force, F = -4770.044
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

New component: From table 7-7, ASCE 41_17: Final Shear Capacity VR = 1.0*Vn = 260476.45
Vn ((10.3), ASCE 41-17) = knl*VCol0 = 260476.45
VCol = 260476.45
knl = 1.00
displacement_ductility_demand = 0.01534626

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' = 25.00, but fc^0.5 <= 8.3 MPa (22.5.3.1, ACI 318-14)
M/Vd = 4.00
Mu = 8.9264E+006
Vu = 2974.321
d = 0.8*D = 320.00
Nu = 4770.044
Ag = 125663.706
From (11.5.4.8), ACI 318-14: Vs = 197392.088
Av = π/2*A_stirrup = 123370.055
fy = 500.00
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 <= 267132.42

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 = 0.0002399
θy = (My*Ls/3)/EIeff = 0.01563237 ((4.29),Biskinis Phd))
My = 1.5905E+008
Ls = M/V (with Ls >0.1*L and Ls < 2*L) = 3001.15
From table 10.5, ASCE 41_17: EIeff = factor*Ec*Ig = 1.0179E+013
factor = 0.30
Ag = 125663.706
fc' = 33.00
N = 4770.044
Ec*Ig = 3.3929E+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) = 1.5905E+008
Φy = 8.1162666E-006
My_ten (8c) = 1.5905E+008
Φ_ten (7c) = 69.54133
error of function (7c) = 0.00365843
My_com (8d) = 4.3759E+008
Φ_com (7d) = 67.95301
error of function (7d) = -0.00152245
with ((10.1), ASCE 41-17) ey = Min(ey, 1.25*ey*(lb/ld)^2/3) = 0.0027778
eco = 0.002
apl = 0.35 ((9a) in Biskinis and Fardis for no FRP Wrap)
d1 = 44.00
R = 200.00
v = 0.00115027
N = 4770.044
Ac = 125663.706
((10.1), ASCE 41-17) ω = Min(ω, 1.25*ω*(lb/ld)^2/3) = 0.40909418
with fc = 33.00

Calculation of ratio lb/ld

Inadequate Lap Length with lb/ld = 0.30

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

Calculation No. 2

column C1, Floor 1

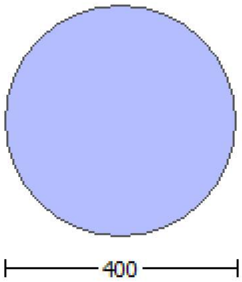
Limit State: Immediate Occupancy (data interpolation between analysis steps 1 and 2)

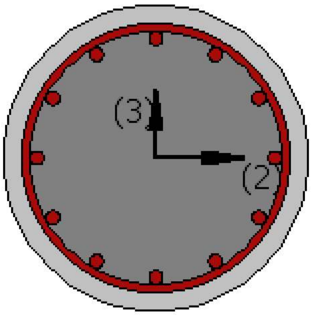
Analysis: Uniform +X

Check: Chord rotation capacity (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 = 0.75
Mean strength values are used for both shear and moment calculations.
Consequently:
New material of Primary Member: Concrete Strength, fc = fcm = 33.00
New material of Primary Member: Steel Strength, fs = fsm = 555.56
Concrete Elasticity, Ec = 26999.444
Steel Elasticity, Es = 200000.00
##
Note: Especially for the calculation of moment strengths,
the above steel re-bar strengths are multiplied by 1.25 according to R18.6.5, ACI 318-14
New material: Steel Strength, fs = 1.25*fsm = 694.45
##
Diameter, D = 400.00
Cover Thickness, c = 25.00
Mean Confinement Factor overall section = 1.00
Element Length, L = 3000.00
Primary 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 = 1.0793611E-031
EDGE -B-
Shear Force, Vb = -1.0793611E-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 = 0.31452971
Member Controlled by Flexure (Ve/Vr < 1)
Calculation of Shear Demand from fig. R18.6.5, ACI 318-14 Ve = (Mpr1 + Mpr2)/ln = 114533.572
with
Mpr1 = Max(Mu1+ , Mu1-) = 1.7180E+008
Mu1+ = 1.7180E+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- = 1.7180E+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-) = 1.7180E+008
Mu2+ = 1.7180E+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- = 1.7180E+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 = 1.7180E+008

φ = 0.99483767
φ' = 0.88186679
error of function (3.68), Biskinis Phd = 24343.261
From 5A.2, TBDY: fcc = fc*λc = 33.00
conf. factor λc = 1.00
fc = 33.00
From 10.3.5, ASCE41-17, Final value of fy: fy*Min(1,1.25*(lb/ld)^2/3) = 389.0139
lb/ld = 0.30
d1 = 44.00
R = 200.00
v = 0.0011501
N = 4771.233
Ac = 125663.706
ω = ω*Min(1,1.25*(lb/ld)^2/3) = 0.22916458

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 = 1.7180E+008

φ = 0.99483767
φ' = 0.88186679

error of function (3.68), Biskinis Phd = 24343.261
From 5A.2, TBDY: fcc = fc*λc = 33.00
conf. factor λc = 1.00
fc = 33.00
From 10.3.5, ASCE41-17, Final value of fy: fy*Min(1,1.25*(lb/ld)^2/3) = 389.0139
lb/ld = 0.30
d1 = 44.00
R = 200.00
v = 0.0011501
N = 4771.233
Ac = 125663.706
ω = ω*Min(1,1.25*(lb/ld)^2/3) = 0.22916458

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 = 1.7180E+008

φ = 0.99483767
φ' = 0.88186679
error of function (3.68), Biskinis Phd = 24343.261
From 5A.2, TBDY: fcc = fc*λc = 33.00
conf. factor λc = 1.00
fc = 33.00
From 10.3.5, ASCE41-17, Final value of fy: fy*Min(1,1.25*(lb/ld)^2/3) = 389.0139
lb/ld = 0.30
d1 = 44.00
R = 200.00
v = 0.0011501
N = 4771.233
Ac = 125663.706
ω = ω*Min(1,1.25*(lb/ld)^2/3) = 0.22916458

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 = 1.7180E+008

φ = 0.99483767
φ' = 0.88186679
error of function (3.68), Biskinis Phd = 24343.261
From 5A.2, TBDY: fcc = fc*λc = 33.00
conf. factor λc = 1.00
fc = 33.00
From 10.3.5, ASCE41-17, Final value of fy: fy*Min(1,1.25*(lb/ld)^2/3) = 389.0139
lb/ld = 0.30
d1 = 44.00
R = 200.00
v = 0.0011501

N = 4771.233
Ac = 125663.706
ω = ω*Min(1,1.25*(lb/ld)^2/3) = 0.22916458

Calculation of ratio lb/ld

Inadequate Lap Length with lb/ld = 0.30

Calculation of Shear Strength Vr = Min(Vr1,Vr2) = 364142.298

Calculation of Shear Strength at edge 1, Vr1 = 364142.298
Vr1 = VCol ((10.3), ASCE 41-17) = knl*VCol0
VCol0 = 364142.298
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' = 33.00, but fc^0.5 <= 8.3 MPa (22.5.3.1, ACI 318-14)
M/Vd = 2.00
Mu = 1.7170334E-011
Vu = 1.0793611E-031
d = 0.8*D = 320.00
Nu = 4771.233
Ag = 125663.706
From (11.5.4.8), ACI 318-14: Vs = 219326.297
Av = π/2*A_stirrup = 123370.055
fy = 555.56
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 <= 306911.784
bw*d = π*d*d/4 = 80424.772

Calculation of Shear Strength at edge 2, Vr2 = 364142.298
Vr2 = VCol ((10.3), ASCE 41-17) = knl*VCol0
VCol0 = 364142.298
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' = 33.00, but fc^0.5 <= 8.3 MPa (22.5.3.1, ACI 318-14)
M/Vd = 2.00
Mu = 1.7170334E-011
Vu = 1.0793611E-031
d = 0.8*D = 320.00
Nu = 4771.233
Ag = 125663.706
From (11.5.4.8), ACI 318-14: Vs = 219326.297
Av = π/2*A_stirrup = 123370.055
fy = 555.56
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 <= 306911.784
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 = 0.75
Mean strength values are used for both shear and moment calculations.
Consequently:
New material of Primary Member: Concrete Strength, fc = fcm = 33.00
New material of Primary Member: Steel Strength, fs = fsm = 555.56
Concrete Elasticity, Ec = 26999.444
Steel Elasticity, Es = 200000.00
##
Note: Especially for the calculation of moment strengths,
the above steel re-bar strengths are multiplied by 1.25 according to R18.6.5, ACI 318-14
New material: Steel Strength, fs = 1.25*fsm = 694.45
##
Diameter, D = 400.00
Cover Thickness, c = 25.00
Mean Confinement Factor overall section = 1.00
Element Length, L = 3000.00
Primary 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 = -6.6089624E-048
EDGE -B-
Shear Force, Vb = 6.6089624E-048
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 = 0.31452971
Member Controlled by Flexure (Ve/Vr < 1)
Calculation of Shear Demand from fig. R18.6.5, ACI 318-14 Ve = (Mpr1 + Mpr2)/ln = 114533.572
with
Mpr1 = Max(Mu1+ , Mu1-) = 1.7180E+008
Mu1+ = 1.7180E+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- = 1.7180E+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-) = 1.7180E+008
Mu2+ = 1.7180E+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- = 1.7180E+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 = 1.7180E+008

φ = 0.99483767
φ' = 0.88186679
error of function (3.68), Biskinis Phd = 24343.261
From 5A.2, TBDY: fcc = fc*λc = 33.00
conf. factor λc = 1.00
fc = 33.00
From 10.3.5, ASCE41-17, Final value of fy: fy*Min(1,1.25*(lb/ld)^2/3) = 389.0139
lb/ld = 0.30
d1 = 44.00
R = 200.00
v = 0.0011501
N = 4771.233
Ac = 125663.706
ω = ω*Min(1,1.25*(lb/ld)^2/3) = 0.22916458

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 = 1.7180E+008

φ = 0.99483767
φ' = 0.88186679
error of function (3.68), Biskinis Phd = 24343.261
From 5A.2, TBDY: fcc = fc*λc = 33.00
conf. factor λc = 1.00
fc = 33.00
From 10.3.5, ASCE41-17, Final value of fy: fy*Min(1,1.25*(lb/ld)^2/3) = 389.0139
lb/ld = 0.30
d1 = 44.00
R = 200.00
v = 0.0011501
N = 4771.233
Ac = 125663.706
ω = ω*Min(1,1.25*(lb/ld)^2/3) = 0.22916458

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 = 1.7180E+008

φ = 0.99483767
φ' = 0.88186679
error of function (3.68), Biskinis Phd = 24343.261
From 5A.2, TBDY: fcc = fc*λc = 33.00
conf. factor λc = 1.00
fc = 33.00
From 10.3.5, ASCE41-17, Final value of fy: fy*Min(1,1.25*(lb/ld)^2/3) = 389.0139
lb/ld = 0.30
d1 = 44.00
R = 200.00
v = 0.0011501
N = 4771.233
Ac = 125663.706
ω = ω*Min(1,1.25*(lb/ld)^2/3) = 0.22916458

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 = 1.7180E+008

φ = 0.99483767
φ' = 0.88186679
error of function (3.68), Biskinis Phd = 24343.261
From 5A.2, TBDY: fcc = fc*λc = 33.00
conf. factor λc = 1.00
fc = 33.00
From 10.3.5, ASCE41-17, Final value of fy: fy*Min(1,1.25*(lb/ld)^2/3) = 389.0139
lb/ld = 0.30
d1 = 44.00
R = 200.00
v = 0.0011501
N = 4771.233
Ac = 125663.706
ω = ω*Min(1,1.25*(lb/ld)^2/3) = 0.22916458

Calculation of ratio lb/ld

Inadequate Lap Length with lb/ld = 0.30

Calculation of Shear Strength Vr = Min(Vr1,Vr2) = 364142.298

Calculation of Shear Strength at edge 1, Vr1 = 364142.298
Vr1 = VCol ((10.3), ASCE 41-17) = knl*VCol0
VCol0 = 364142.298
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' = 33.00, but fc^0.5 <= 8.3 MPa (22.5.3.1, ACI 318-14)
M/Vd = 2.00
Mu = 5.2883590E-012
Vu = 6.6089624E-048
d = 0.8*D = 320.00
Nu = 4771.233
Ag = 125663.706
From (11.5.4.8), ACI 318-14: Vs = 219326.297
Av = π/2*A_stirrup = 123370.055
fy = 555.56
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 <= 306911.784
bw*d = π*d*d/4 = 80424.772

Calculation of Shear Strength at edge 2, Vr2 = 364142.298
Vr2 = VCol ((10.3), ASCE 41-17) = knl*VCol0
VCol0 = 364142.298
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' = 33.00, but fc^0.5 <= 8.3 MPa (22.5.3.1, ACI 318-14)
M/Vd = 2.00
Mu = 5.2883590E-012
Vu = 6.6089624E-048
d = 0.8*D = 320.00
Nu = 4771.233
Ag = 125663.706
From (11.5.4.8), ACI 318-14: Vs = 219326.297
Av = π/2*A_stirrup = 123370.055
fy = 555.56
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 <= 306911.784
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 = 0.75
Chord Rotation is generally considered as Deformation-Controlled Action according to Table C7-1, ASCE41-17.
Mean strengths are used for Deformation-Controlled Actions according to 7.5.1.3, ASCE 41-17
Consequently:
New material of Primary Member: Concrete Strength, fc = fcm = 33.00
New material of Primary Member: Steel Strength, fs = fsm = 555.56
Concrete Elasticity, Ec = 26999.444

Steel Elasticity, Es = 200000.00
Diameter, D = 400.00
Cover Thickness, c = 25.00
Element Length, L = 3000.00
Primary 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 = 4.6357277E-010
Shear Force, V2 = -2974.321
Shear Force, V3 = -1.3556528E-013
Axial Force, F = -4770.044
Longitudinal Reinforcement Area Distribution (in 2 divisions)
-Tension: Aslt = 1272.345
-Compression: Aslc = 1781.283
Longitudinal Reinforcement Area Distribution (in 3 divisions)
-Tension: Asl,ten = 1017.876
-Compression: Asl,com = 1017.876
-Middle: Asl,mid = 1017.876
Mean Diameter of Tension Reinforcement, DbL = 18.00

New component: From table 7-7, ASCE 41_17: Final chord rotation Capacity θu,R = 1.0*θu = 0.00781319
θu = θy + θp = 0.00781319

- Calculation of θy -

θy = (My*Ls/3)/EIeff = 0.00781319 ((4.29),Biskinis Phd))
My = 1.5905E+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 = 1.0179E+013
factor = 0.30
Ag = 125663.706
fc' = 33.00
N = 4770.044
Ec*Ig = 3.3929E+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) = 1.5905E+008
Φy = 8.1162666E-006
My_ten (8c) = 1.5905E+008
Φ_ten (7c) = 69.54133
error of function (7c) = 0.00365843
My_com (8d) = 4.3759E+008
Φ_com (7d) = 67.95301
error of function (7d) = -0.00152245
with ((10.1), ASCE 41-17) ey = Min(ey, 1.25*ey*(lb/ld)^2/3) = 0.0027778
eco = 0.002
apl = 0.35 ((9a) in Biskinis and Fardis for no FRP Wrap)
d1 = 44.00
R = 200.00
v = 0.00115027

N = 4770.044
Ac = 125663.706
((10.1), ASCE 41-17) ω = Min(ω, 1.25*ω*(lb/ld)^2/3) = 0.40909418
with fc = 33.00

Calculation of ratio lb/ld

Inadequate Lap Length with lb/ld = 0.30

- Calculation of θp -

From table 10-9: θp = 0.00
with:
- Columns not controlled by inadequate development or splicing along the clear height because lb/ld >= 1
shear control ratio VyE/VCol0E = 0.31452971
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 = 4770.044
Ag = 125663.706
f'cE = 33.00
fytE = fylE = 555.56
pl = Area_Tot_Long_Rein/(Ag) = 0.0243
f'cE = 33.00

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

Calculation No. 3

column C1, Floor 1

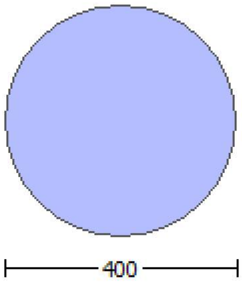
Limit State: Immediate Occupancy (data interpolation between analysis steps 1 and 2)

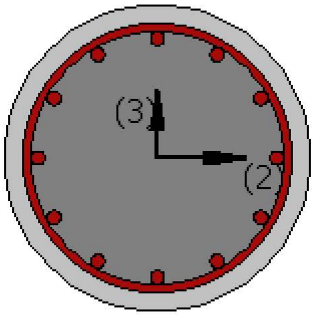
Analysis: Uniform +X

Check: Shear capacity VRd

Edge: Start

Local Axis: (3)





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

Constant Properties

Knowledge Factor, κ = 0.75
Member Shear Force is generally considered as Force-Controlled Action according to Table C7-1, ASCE41-17.
Lower-bound strengths are used for Force-Controlled Actions according to 7.5.1.3, ASCE 41-17
Consequently:
New material of Primary Member: Concrete Strength, fc = fc_lower_bound = 25.00
New material of Primary Member: Steel Strength, fs = fs_lower_bound = 500.00
Concrete Elasticity, Ec = 26999.444
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).
New material: Concrete Strength, fc = fcm = 33.00
New material: Steel Strength, fs = fsm = 555.56
###
Diameter, D = 400.00
Cover Thickness, c = 25.00
Element Length, L = 3000.00
Primary 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 = 4.6357277E-010
Shear Force, Va = -1.3556528E-013
EDGE -B-
Bending Moment, Mb = -5.6659975E-011
Shear Force, Vb = 1.3556528E-013
BOTH EDGES
Axial Force, F = -4770.044
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

New component: From table 7-7, ASCE 41_17: Final Shear Capacity VR = 1.0*Vn = 323560.812
Vn ((10.3), ASCE 41-17) = knl*VCol0 = 323560.812
VCol = 323560.812
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' = 25.00, but fc^0.5 <= 8.3 MPa (22.5.3.1, ACI 318-14)
M/Vd = 2.00
Mu = 4.6357277E-010
Vu = 1.3556528E-013
d = 0.8*D = 320.00
Nu = 4770.044
Ag = 125663.706
From (11.5.4.8), ACI 318-14: Vs = 197392.088
Av = π/2*A_stirrup = 123370.055
fy = 500.00
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 <= 267132.42
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 = 1.5198218E-020
θy = (My*Ls/3)/Eleff = 0.00781319 ((4.29),Biskinis Phd))
My = 1.5905E+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 = 1.0179E+013
factor = 0.30
Ag = 125663.706
fc' = 33.00
N = 4770.044
Ec*Ig = 3.3929E+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) = 1.5905E+008
Φy = 8.1162666E-006
My_ten (8c) = 1.5905E+008
Φ_ten (7c) = 69.54133
error of function (7c) = 0.00365843
My_com (8d) = 4.3759E+008
Φ_com (7d) = 67.95301
error of function (7d) = -0.00152245
with ((10.1), ASCE 41-17) ey = Min(ey, 1.25*ey*(lb/ld)^2/3) = 0.0027778
eco = 0.002
apl = 0.35 ((9a) in Biskinis and Fardis for no FRP Wrap)
d1 = 44.00

R = 200.00
v = 0.00115027
N = 4770.044
Ac = 125663.706
((10.1), ASCE 41-17) ω = Min(ω, 1.25*ω*(lb/ld)^2/3) = 0.40909418
with fc = 33.00

Calculation of ratio lb/ld

Inadequate Lap Length with lb/ld = 0.30

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

Calculation No. 4

column C1, Floor 1

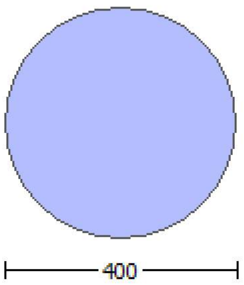
Limit State: Immediate Occupancy (data interpolation between analysis steps 1 and 2)

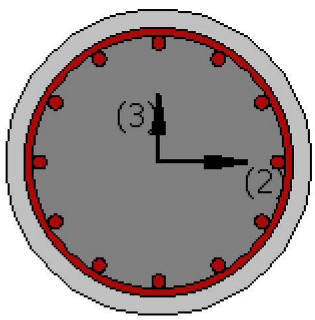
Analysis: Uniform +X

Check: Chord rotation capacity (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 = 0.75
Mean strength values are used for both shear and moment calculations.
Consequently:
New material of Primary Member: Concrete Strength, fc = fcm = 33.00
New material of Primary Member: Steel Strength, fs = fsm = 555.56

Concrete Elasticity, Ec = 26999.444
Steel Elasticity, Es = 200000.00
##
Note: Especially for the calculation of moment strengths,
the above steel re-bar strengths are multiplied by 1.25 according to R18.6.5, ACI 318-14
New material: Steel Strength, fs = 1.25*fsm = 694.45
##
Diameter, D = 400.00
Cover Thickness, c = 25.00
Mean Confinement Factor overall section = 1.00
Element Length, L = 3000.00
Primary 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 = 1.0793611E-031
EDGE -B-
Shear Force, Vb = -1.0793611E-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 = 0.31452971
Member Controlled by Flexure (Ve/Vr < 1)
Calculation of Shear Demand from fig. R18.6.5, ACI 318-14 Ve = (Mpr1 + Mpr2)/ln = 114533.572
with
Mpr1 = Max(Mu1+ , Mu1-) = 1.7180E+008
Mu1+ = 1.7180E+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- = 1.7180E+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-) = 1.7180E+008
Mu2+ = 1.7180E+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- = 1.7180E+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 = 1.7180E+008

φ = 0.99483767
φ' = 0.88186679
error of function (3.68), Biskinis Phd = 24343.261
From 5A.2, TBDY: fcc = fc*λc = 33.00

conf. factor λc = 1.00
fc = 33.00
From 10.3.5, ASCE41-17, Final value of fy: fy*Min(1,1.25*(lb/ld)^2/3) = 389.0139
lb/ld = 0.30
d1 = 44.00
R = 200.00
v = 0.0011501
N = 4771.233
Ac = 125663.706
ω = ω*Min(1,1.25*(lb/ld)^2/3) = 0.22916458

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 = 1.7180E+008

φ = 0.99483767
φ' = 0.88186679
error of function (3.68), Biskinis Phd = 24343.261
From 5A.2, TBDY: fcc = fc*λc = 33.00
conf. factor λc = 1.00
fc = 33.00
From 10.3.5, ASCE41-17, Final value of fy: fy*Min(1,1.25*(lb/ld)^2/3) = 389.0139
lb/ld = 0.30
d1 = 44.00
R = 200.00
v = 0.0011501
N = 4771.233
Ac = 125663.706
ω = ω*Min(1,1.25*(lb/ld)^2/3) = 0.22916458

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 = 1.7180E+008

φ = 0.99483767
φ' = 0.88186679
error of function (3.68), Biskinis Phd = 24343.261
From 5A.2, TBDY: fcc = fc*λc = 33.00
conf. factor λc = 1.00
fc = 33.00
From 10.3.5, ASCE41-17, Final value of fy: fy*Min(1,1.25*(lb/ld)^2/3) = 389.0139
lb/ld = 0.30
d1 = 44.00
R = 200.00
v = 0.0011501
N = 4771.233
Ac = 125663.706

ω = ω*Min(1,1.25*(lb/ld)^2/3) = 0.22916458

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 = 1.7180E+008

φ = 0.99483767
φ' = 0.88186679
error of function (3.68), Biskinis Phd = 24343.261
From 5A.2, TBDY: fcc = fc*λc = 33.00
conf. factor λc = 1.00
fc = 33.00
From 10.3.5, ASCE41-17, Final value of fy: fy*Min(1,1.25*(lb/ld)^2/3) = 389.0139
lb/ld = 0.30
d1 = 44.00
R = 200.00
v = 0.0011501
N = 4771.233
Ac = 125663.706
ω = ω*Min(1,1.25*(lb/ld)^2/3) = 0.22916458

Calculation of ratio lb/ld

Inadequate Lap Length with lb/ld = 0.30

Calculation of Shear Strength Vr = Min(Vr1,Vr2) = 364142.298

Calculation of Shear Strength at edge 1, Vr1 = 364142.298
Vr1 = VCol ((10.3), ASCE 41-17) = knl*VCol0
VCol0 = 364142.298
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' = 33.00, but fc^0.5 <= 8.3 MPa (22.5.3.1, ACI 318-14)
M/Vd = 2.00
Mu = 1.7170334E-011
Vu = 1.0793611E-031
d = 0.8*D = 320.00
Nu = 4771.233
Ag = 125663.706
From (11.5.4.8), ACI 318-14: Vs = 219326.297
Av = π/2*A_stirrup = 123370.055
fy = 555.56
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 <= 306911.784
bw*d = π*d*d/4 = 80424.772

Calculation of Shear Strength at edge 2, Vr2 = 364142.298
Vr2 = VCol ((10.3), ASCE 41-17) = knl*VCol0
VCol0 = 364142.298
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' = 33.00, but fc^0.5 <= 8.3 MPa (22.5.3.1, ACI 318-14)
M/Vd = 2.00
Mu = 1.7170334E-011
Vu = 1.0793611E-031
d = 0.8*D = 320.00
Nu = 4771.233
Ag = 125663.706
From (11.5.4.8), ACI 318-14: Vs = 219326.297
Av = π/2*A_stirrup = 123370.055
fy = 555.56
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 <= 306911.784
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 = 0.75
Mean strength values are used for both shear and moment calculations.
Consequently:
New material of Primary Member: Concrete Strength, fc = fcm = 33.00
New material of Primary Member: Steel Strength, fs = fsm = 555.56
Concrete Elasticity, Ec = 26999.444
Steel Elasticity, Es = 200000.00
##
Note: Especially for the calculation of moment strengths,
the above steel re-bar strengths are multiplied by 1.25 according to R18.6.5, ACI 318-14
New material: Steel Strength, fs = 1.25*fsm = 694.45
##
Diameter, D = 400.00
Cover Thickness, c = 25.00
Mean Confinement Factor overall section = 1.00
Element Length, L = 3000.00
Primary 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 = -6.6089624E-048
EDGE -B-
Shear Force, Vb = 6.6089624E-048
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 = 0.31452971
Member Controlled by Flexure (Ve/Vr < 1)
Calculation of Shear Demand from fig. R18.6.5, ACI 318-14 Ve = (Mpr1 + Mpr2)/ln = 114533.572
with
Mpr1 = Max(Mu1+ , Mu1-) = 1.7180E+008
Mu1+ = 1.7180E+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- = 1.7180E+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-) = 1.7180E+008
Mu2+ = 1.7180E+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- = 1.7180E+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 = 1.7180E+008

φ = 0.99483767
φ' = 0.88186679
error of function (3.68), Biskinis Phd = 24343.261
From 5A.2, TBDY: fcc = fc*λc = 33.00
conf. factor λc = 1.00
fc = 33.00
From 10.3.5, ASCE41-17, Final value of fy: fy*Min(1,1.25*(lb/ld)^2/3) = 389.0139
lb/ld = 0.30
d1 = 44.00
R = 200.00
v = 0.0011501
N = 4771.233
Ac = 125663.706
ω = ω*Min(1,1.25*(lb/ld)^2/3) = 0.22916458

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 = 1.7180E+008

φ = 0.99483767
φ' = 0.88186679
error of function (3.68), Biskinis Phd = 24343.261
From 5A.2, TBDY: fcc = fc*λc = 33.00
conf. factor λc = 1.00
fc = 33.00
From 10.3.5, ASCE41-17, Final value of fy: fy*Min(1,1.25*(lb/ld)^2/3) = 389.0139
lb/ld = 0.30
d1 = 44.00
R = 200.00
v = 0.0011501
N = 4771.233
Ac = 125663.706
ω = ω*Min(1,1.25*(lb/ld)^2/3) = 0.22916458

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 = 1.7180E+008

φ = 0.99483767
φ' = 0.88186679
error of function (3.68), Biskinis Phd = 24343.261
From 5A.2, TBDY: fcc = fc*λc = 33.00
conf. factor λc = 1.00
fc = 33.00
From 10.3.5, ASCE41-17, Final value of fy: fy*Min(1,1.25*(lb/ld)^2/3) = 389.0139
lb/ld = 0.30
d1 = 44.00
R = 200.00
v = 0.0011501
N = 4771.233
Ac = 125663.706
ω = ω*Min(1,1.25*(lb/ld)^2/3) = 0.22916458

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 = 1.7180E+008

φ = 0.99483767
φ' = 0.88186679
error of function (3.68), Biskinis Phd = 24343.261
From 5A.2, TBDY: fcc = fc*λc = 33.00
conf. factor λc = 1.00

fc = 33.00
From 10.3.5, ASCE41-17, Final value of fy: fy*Min(1,1.25*(lb/ld)^2/3) = 389.0139
lb/ld = 0.30
d1 = 44.00
R = 200.00
v = 0.0011501
N = 4771.233
Ac = 125663.706
ω = ω*Min(1,1.25*(lb/ld)^2/3) = 0.22916458

Calculation of ratio lb/ld

Inadequate Lap Length with lb/ld = 0.30

Calculation of Shear Strength Vr = Min(Vr1,Vr2) = 364142.298

Calculation of Shear Strength at edge 1, Vr1 = 364142.298
Vr1 = VCol ((10.3), ASCE 41-17) = knl*VCol0
VCol0 = 364142.298
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' = 33.00, but fc^0.5 <= 8.3 MPa (22.5.3.1, ACI 318-14)
M/Vd = 2.00
Mu = 5.2883590E-012
Vu = 6.6089624E-048
d = 0.8*D = 320.00
Nu = 4771.233
Ag = 125663.706
From (11.5.4.8), ACI 318-14: Vs = 219326.297
Av = π/2*A_stirrup = 123370.055
fy = 555.56
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 <= 306911.784
bw*d = π*d*d/4 = 80424.772

Calculation of Shear Strength at edge 2, Vr2 = 364142.298
Vr2 = VCol ((10.3), ASCE 41-17) = knl*VCol0
VCol0 = 364142.298
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' = 33.00, but fc^0.5 <= 8.3 MPa (22.5.3.1, ACI 318-14)
M/Vd = 2.00
Mu = 5.2883590E-012
Vu = 6.6089624E-048
d = 0.8*D = 320.00
Nu = 4771.233
Ag = 125663.706
From (11.5.4.8), ACI 318-14: Vs = 219326.297
Av = π/2*A_stirrup = 123370.055
fy = 555.56
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 <= 306911.784
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 = 0.75
Chord Rotation is generally considered as Deformation-Controlled Action according to Table C7-1, ASCE41-17.
Mean strengths are used for Deformation-Controlled Actions according to 7.5.1.3, ASCE 41-17
Consequently:
New material of Primary Member: Concrete Strength, fc = fcm = 33.00
New material of Primary Member: Steel Strength, fs = fsm = 555.56
Concrete Elasticity, Ec = 26999.444
Steel Elasticity, Es = 200000.00
Diameter, D = 400.00
Cover Thickness, c = 25.00
Element Length, L = 3000.00
Primary 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 = -8.9264E+006
Shear Force, V2 = -2974.321
Shear Force, V3 = -1.3556528E-013
Axial Force, F = -4770.044
Longitudinal Reinforcement Area Distribution (in 2 divisions)
-Tension: Aslt = 1272.345
-Compression: Aslc = 1781.283
Longitudinal Reinforcement Area Distribution (in 3 divisions)
-Tension: Asl,ten = 1017.876
-Compression: Asl,com = 1017.876
-Middle: Asl,mid = 1017.876
Mean Diameter of Tension Reinforcement, DbL = 18.00

New component: From table 7-7, ASCE 41_17: Final chord rotation Capacity θu,R = 1.0*θu = 0.01563237
θu = θy + θp = 0.01563237

- Calculation of θy -

θy = (My*Ls/3)/EIeff = 0.01563237 ((4.29),Biskinis Phd))
My = 1.5905E+008
Ls = M/V (with Ls >0.1*L and Ls < 2*L) = 3001.15

From table 10.5, ASCE 41_17: Eleff = factor*Ec*Ig = 1.0179E+013
factor = 0.30
Ag = 125663.706
fc' = 33.00
N = 4770.044
Ec*Ig = 3.3929E+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) = 1.5905E+008
Φy = 8.1162666E-006
My_ten (8c) = 1.5905E+008
Φ_ten (7c) = 69.54133
error of function (7c) = 0.00365843
My_com (8d) = 4.3759E+008
Φ_com (7d) = 67.95301
error of function (7d) = -0.00152245
with ((10.1), ASCE 41-17) ey = Min(ey, 1.25*ey*(lb/ld)^2/3) = 0.0027778
eco = 0.002
apl = 0.35 ((9a) in Biskinis and Fardis for no FRP Wrap)
d1 = 44.00
R = 200.00
v = 0.00115027
N = 4770.044
Ac = 125663.706
((10.1), ASCE 41-17) ω = Min(ω, 1.25*ω*(lb/ld)^2/3) = 0.40909418
with fc = 33.00

Calculation of ratio lb/ld

Inadequate Lap Length with lb/ld = 0.30

- Calculation of θp -

From table 10-9: θp = 0.00
with:
- Columns not controlled by inadequate development or splicing along the clear height because lb/ld >= 1
shear control ratio VyE/VCol0E = 0.31452971
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 = 4770.044
Ag = 125663.706
f'cE = 33.00
fytE = fylE = 555.56
pl = Area_Tot_Long_Rein/(Ag) = 0.0243
f'cE = 33.00

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

Calculation No. 5

column C1, Floor 1

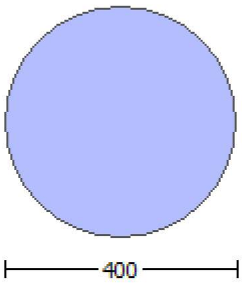
Limit State: Immediate Occupancy (data interpolation between analysis steps 1 and 2)

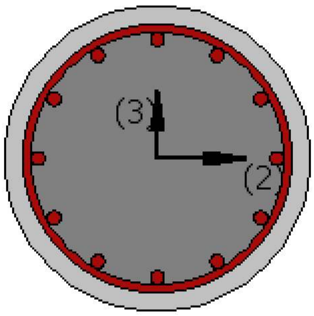
Analysis: Uniform +X

Check: Shear capacity 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 = 0.75
Member Shear Force is generally considered as Force-Controlled Action according to Table C7-1, ASCE41-17.
Lower-bound strengths are used for Force-Controlled Actions according to 7.5.1.3, ASCE 41-17
Consequently:
New material of Primary Member: Concrete Strength, fc = fc_lower_bound = 25.00
New material of Primary Member: Steel Strength, fs = fs_lower_bound = 500.00
Concrete Elasticity, Ec = 26999.444
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).
New material: Concrete Strength, fc = fcm = 33.00
New material: Steel Strength, fs = fsm = 555.56
###
Diameter, D = 400.00
Cover Thickness, c = 25.00
Element Length, L = 3000.00
Primary 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 = -8.9264E+006
Shear Force, Va = -2974.321
EDGE -B-
Bending Moment, Mb = 0.00304337
Shear Force, Vb = 2974.321
BOTH EDGES
Axial Force, F = -4770.044
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

New component: From table 7-7, ASCE 41_17: Final Shear Capacity VR = 1.0*Vn = 323560.812
Vn ((10.3), ASCE 41-17) = knl*VCol0 = 323560.812
VCol = 323560.812
knl = 1.00
displacement_ductility_demand = 0.08417412

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' = 25.00, but fc^0.5 <= 8.3 MPa (22.5.3.1, ACI 318-14)
M/Vd = 2.00
Mu = 0.00304337
Vu = 2974.321
d = 0.8*D = 320.00
Nu = 4770.044
Ag = 125663.706
From (11.5.4.8), ACI 318-14: Vs = 197392.088
Av = π/2*A_stirrup = 123370.055
fy = 500.00
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 <= 267132.42
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 = 0.00013153
θy = (My*Ls/3)/Eleff = 0.00156264 ((4.29),Biskinis Phd))
My = 1.5905E+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 = 1.0179E+013
factor = 0.30
Ag = 125663.706
fc' = 33.00

N = 4770.044
Ec*Ig = 3.3929E+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) = 1.5905E+008
Φy = 8.1162666E-006
My_ten (8c) = 1.5905E+008
Φ_ten (7c) = 69.54133
error of function (7c) = 0.00365843
My_com (8d) = 4.3759E+008
Φ_com (7d) = 67.95301
error of function (7d) = -0.00152245
with ((10.1), ASCE 41-17) ey = Min(ey, 1.25*ey*(lb/ld)^2/3) = 0.0027778
eco = 0.002
apl = 0.35 ((9a) in Biskinis and Fardis for no FRP Wrap)
d1 = 44.00
R = 200.00
v = 0.00115027
N = 4770.044
Ac = 125663.706
((10.1), ASCE 41-17) ω = Min(ω, 1.25*ω*(lb/ld)^2/3) = 0.40909418
with fc = 33.00

Calculation of ratio lb/ld

Inadequate Lap Length with lb/ld = 0.30

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

Calculation No. 6

column C1, Floor 1

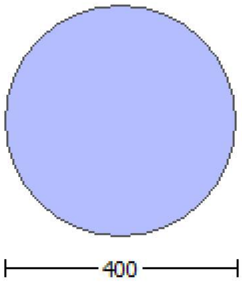
Limit State: Immediate Occupancy (data interpolation between analysis steps 1 and 2)

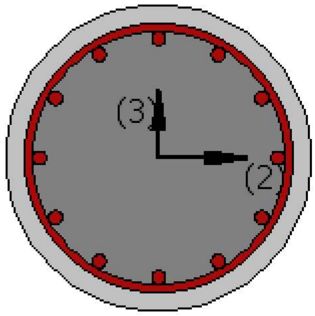
Analysis: Uniform +X

Check: Chord rotation capacity (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 = 0.75
Mean strength values are used for both shear and moment calculations.
Consequently:
New material of Primary Member: Concrete Strength, fc = fcm = 33.00
New material of Primary Member: Steel Strength, fs = fsm = 555.56
Concrete Elasticity, Ec = 26999.444
Steel Elasticity, Es = 200000.00
###
Note: Especially for the calculation of moment strengths,
the above steel re-bar strengths are multiplied by 1.25 according to R18.6.5, ACI 318-14
New material: Steel Strength, fs = 1.25*fsm = 694.45
###
Diameter, D = 400.00
Cover Thickness, c = 25.00
Mean Confinement Factor overall section = 1.00
Element Length, L = 3000.00
Primary 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 = 1.0793611E-031
EDGE -B-
Shear Force, Vb = -1.0793611E-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 = 0.31452971
Member Controlled by Flexure (Ve/Vr < 1)
Calculation of Shear Demand from fig. R18.6.5, ACI 318-14 Ve = (Mpr1 + Mpr2)/ln = 114533.572
with
Mpr1 = Max(Mu1+ , Mu1-) = 1.7180E+008
Mu1+ = 1.7180E+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- = 1.7180E+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-) = 1.7180E+008
Mu2+ = 1.7180E+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- = 1.7180E+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 = 1.7180E+008

φ = 0.99483767
φ' = 0.88186679
error of function (3.68), Biskinis Phd = 24343.261
From 5A.2, TBDY: fcc = fc*λc = 33.00
conf. factor λc = 1.00
fc = 33.00
From 10.3.5, ASCE41-17, Final value of fy: fy*Min(1,1.25*(lb/ld)^2/3) = 389.0139
lb/ld = 0.30
d1 = 44.00
R = 200.00
v = 0.0011501
N = 4771.233
Ac = 125663.706
ω = ω*Min(1,1.25*(lb/ld)^2/3) = 0.22916458

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 = 1.7180E+008

φ = 0.99483767
φ' = 0.88186679
error of function (3.68), Biskinis Phd = 24343.261
From 5A.2, TBDY: fcc = fc*λc = 33.00
conf. factor λc = 1.00
fc = 33.00
From 10.3.5, ASCE41-17, Final value of fy: fy*Min(1,1.25*(lb/ld)^2/3) = 389.0139
lb/ld = 0.30
d1 = 44.00
R = 200.00
v = 0.0011501
N = 4771.233
Ac = 125663.706
ω = ω*Min(1,1.25*(lb/ld)^2/3) = 0.22916458

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 = 1.7180E+008

φ = 0.99483767
φ' = 0.88186679
error of function (3.68), Biskinis Phd = 24343.261
From 5A.2, TBDY: fcc = fc*λc = 33.00
conf. factor λc = 1.00
fc = 33.00
From 10.3.5, ASCE41-17, Final value of fy: fy*Min(1,1.25*(lb/ld)^2/3) = 389.0139
lb/ld = 0.30
d1 = 44.00
R = 200.00
v = 0.0011501
N = 4771.233
Ac = 125663.706
ω = ω*Min(1,1.25*(lb/ld)^2/3) = 0.22916458

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 = 1.7180E+008

φ = 0.99483767
φ' = 0.88186679
error of function (3.68), Biskinis Phd = 24343.261
From 5A.2, TBDY: fcc = fc*λc = 33.00
conf. factor λc = 1.00
fc = 33.00
From 10.3.5, ASCE41-17, Final value of fy: fy*Min(1,1.25*(lb/ld)^2/3) = 389.0139
lb/ld = 0.30
d1 = 44.00
R = 200.00
v = 0.0011501
N = 4771.233
Ac = 125663.706
ω = ω*Min(1,1.25*(lb/ld)^2/3) = 0.22916458

Calculation of ratio lb/ld

Inadequate Lap Length with lb/ld = 0.30

Calculation of Shear Strength Vr = Min(Vr1,Vr2) = 364142.298

Calculation of Shear Strength at edge 1, Vr1 = 364142.298
Vr1 = VCol ((10.3), ASCE 41-17) = knl*VCol0
VCol0 = 364142.298
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' = 33.00, but fc^0.5 <= 8.3 MPa (22.5.3.1, ACI 318-14)
M/Vd = 2.00
Mu = 1.7170334E-011
Vu = 1.0793611E-031
d = 0.8*D = 320.00
Nu = 4771.233
Ag = 125663.706
From (11.5.4.8), ACI 318-14: Vs = 219326.297
Av = π/2*A_stirrup = 123370.055
fy = 555.56
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 <= 306911.784
bw*d = π*d*d/4 = 80424.772

Calculation of Shear Strength at edge 2, Vr2 = 364142.298
Vr2 = VCol ((10.3), ASCE 41-17) = knl*VCol0
VCol0 = 364142.298
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' = 33.00, but fc^0.5 <= 8.3 MPa (22.5.3.1, ACI 318-14)
M/Vd = 2.00
Mu = 1.7170334E-011
Vu = 1.0793611E-031
d = 0.8*D = 320.00
Nu = 4771.233
Ag = 125663.706
From (11.5.4.8), ACI 318-14: Vs = 219326.297
Av = π/2*A_stirrup = 123370.055
fy = 555.56
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 <= 306911.784
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 = 0.75
Mean strength values are used for both shear and moment calculations.
Consequently:
New material of Primary Member: Concrete Strength, fc = fcm = 33.00
New material of Primary Member: Steel Strength, fs = fsm = 555.56
Concrete Elasticity, Ec = 26999.444
Steel Elasticity, Es = 200000.00
##
Note: Especially for the calculation of moment strengths,
the above steel re-bar strengths are multiplied by 1.25 according to R18.6.5, ACI 318-14
New material: Steel Strength, fs = 1.25*fsm = 694.45
##
Diameter, D = 400.00
Cover Thickness, c = 25.00
Mean Confinement Factor overall section = 1.00
Element Length, L = 3000.00
Primary 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 = -6.6089624E-048
EDGE -B-
Shear Force, Vb = 6.6089624E-048
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 = 0.31452971
Member Controlled by Flexure (Ve/Vr < 1)
Calculation of Shear Demand from fig. R18.6.5, ACI 318-14 Ve = (Mpr1 + Mpr2)/ln = 114533.572
with
Mpr1 = Max(Mu1+ , Mu1-) = 1.7180E+008
Mu1+ = 1.7180E+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- = 1.7180E+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-) = 1.7180E+008
Mu2+ = 1.7180E+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- = 1.7180E+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 = 1.7180E+008

φ = 0.99483767
φ' = 0.88186679
error of function (3.68), Biskinis Phd = 24343.261
From 5A.2, TBDY: fcc = fc*λc = 33.00
conf. factor λc = 1.00
fc = 33.00
From 10.3.5, ASCE41-17, Final value of fy: fy*Min(1,1.25*(lb/ld)^2/3) = 389.0139
lb/ld = 0.30
d1 = 44.00
R = 200.00
v = 0.0011501
N = 4771.233
Ac = 125663.706
ω = ω*Min(1,1.25*(lb/ld)^2/3) = 0.22916458

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 = 1.7180E+008

φ = 0.99483767
φ' = 0.88186679
error of function (3.68), Biskinis Phd = 24343.261
From 5A.2, TBDY: fcc = fc*λc = 33.00
conf. factor λc = 1.00
fc = 33.00
From 10.3.5, ASCE41-17, Final value of fy: fy*Min(1,1.25*(lb/ld)^2/3) = 389.0139
lb/ld = 0.30
d1 = 44.00
R = 200.00
v = 0.0011501
N = 4771.233
Ac = 125663.706
ω = ω*Min(1,1.25*(lb/ld)^2/3) = 0.22916458

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 = 1.7180E+008

φ = 0.99483767
φ' = 0.88186679
error of function (3.68), Biskinis Phd = 24343.261
From 5A.2, TBDY: fcc = fc*λc = 33.00
conf. factor λc = 1.00
fc = 33.00
From 10.3.5, ASCE41-17, Final value of fy: fy*Min(1,1.25*(lb/ld)^2/3) = 389.0139

lb/ld = 0.30
d1 = 44.00
R = 200.00
v = 0.0011501
N = 4771.233
Ac = 125663.706
ω = ω*Min(1,1.25*(lb/ld)^2/3) = 0.22916458

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 = 1.7180E+008

φ = 0.99483767
φ' = 0.88186679
error of function (3.68), Biskinis Phd = 24343.261
From 5A.2, TBDY: fcc = fc*λc = 33.00
conf. factor λc = 1.00
fc = 33.00
From 10.3.5, ASCE41-17, Final value of fy: fy*Min(1,1.25*(lb/ld)^2/3) = 389.0139
lb/ld = 0.30
d1 = 44.00
R = 200.00
v = 0.0011501
N = 4771.233
Ac = 125663.706
ω = ω*Min(1,1.25*(lb/ld)^2/3) = 0.22916458

Calculation of ratio lb/ld

Inadequate Lap Length with lb/ld = 0.30

Calculation of Shear Strength Vr = Min(Vr1,Vr2) = 364142.298

Calculation of Shear Strength at edge 1, Vr1 = 364142.298
Vr1 = VCol ((10.3), ASCE 41-17) = knl*VCol0
VCol0 = 364142.298
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' = 33.00, but fc^0.5 <= 8.3 MPa (22.5.3.1, ACI 318-14)
M/Vd = 2.00
Mu = 5.2883590E-012
Vu = 6.6089624E-048
d = 0.8*D = 320.00
Nu = 4771.233
Ag = 125663.706
From (11.5.4.8), ACI 318-14: Vs = 219326.297
Av = π/2*A_stirrup = 123370.055
fy = 555.56
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 <= 306911.784
bw*d = π*d*d/4 = 80424.772

Calculation of Shear Strength at edge 2, Vr2 = 364142.298
Vr2 = VCol ((10.3), ASCE 41-17) = knl*VCol0
VCol0 = 364142.298
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' = 33.00, but fc^0.5 <= 8.3 MPa (22.5.3.1, ACI 318-14)
M/Vd = 2.00
Mu = 5.2883590E-012
Vu = 6.6089624E-048
d = 0.8*D = 320.00
Nu = 4771.233
Ag = 125663.706
From (11.5.4.8), ACI 318-14: Vs = 219326.297
Av = π/2*A_stirrup = 123370.055
fy = 555.56
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 <= 306911.784
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 = 0.75
Chord Rotation is generally considered as Deformation-Controlled Action according to Table C7-1, ASCE41-17.
Mean strengths are used for Deformation-Controlled Actions according to 7.5.1.3, ASCE 41-17
Consequently:
New material of Primary Member: Concrete Strength, fc = fcm = 33.00
New material of Primary Member: Steel Strength, fs = fsm = 555.56
Concrete Elasticity, Ec = 26999.444
Steel Elasticity, Es = 200000.00
Diameter, D = 400.00
Cover Thickness, c = 25.00
Element Length, L = 3000.00
Primary 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 = -5.6659975E-011
Shear Force, V2 = 2974.321
Shear Force, V3 = 1.3556528E-013
Axial Force, F = -4770.044
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

New component: From table 7-7, ASCE 41_17: Final chord rotation Capacity θu,R = 1.0*θu = 0.00781319
θu = θy + θp = 0.00781319

- Calculation of θy -

θy = (My*Ls/3)/EIeff = 0.00781319 ((4.29),Biskinis Phd))
My = 1.5905E+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 = 1.0179E+013
factor = 0.30
Ag = 125663.706
fc' = 33.00
N = 4770.044
Ec*Ig = 3.3929E+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) = 1.5905E+008
Φy = 8.1162666E-006
My_ten (8c) = 1.5905E+008
Φ_ten (7c) = 69.54133
error of function (7c) = 0.00365843
My_com (8d) = 4.3759E+008
Φ_com (7d) = 67.95301
error of function (7d) = -0.00152245
with ((10.1), ASCE 41-17) ey = Min(ey, 1.25*ey*(lb/ld)^2/3) = 0.0027778
eco = 0.002
apl = 0.35 ((9a) in Biskinis and Fardis for no FRP Wrap)
d1 = 44.00
R = 200.00
v = 0.00115027
N = 4770.044
Ac = 125663.706
((10.1), ASCE 41-17) ω = Min(ω, 1.25*ω*(lb/ld)^2/3) = 0.40909418
with fc = 33.00

Calculation of ratio lb/ld

Inadequate Lap Length with lb/ld = 0.30

- Calculation of θp -

From table 10-9: θp = 0.00
with:
- Columns not controlled by inadequate development or splicing along the clear height because lb/ld >= 1
  shear control ratio VyE/VCol0E = 0.31452971
  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 = 4770.044
    Ag = 125663.706
    f'cE = 33.00
    fytE = fylE = 555.56
ρl = Area_Tot_Long_Rein/(Ag) = 0.0243
    f'cE = 33.00

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

Calculation No. 7

column C1, Floor 1

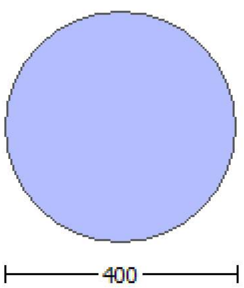
Limit State: Immediate Occupancy (data interpolation between analysis steps 1 and 2)

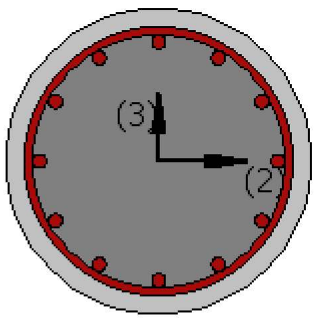
Analysis: Uniform +X

Check: Shear capacity 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 = 0.75
Member Shear Force is generally considered as Force-Controlled Action according to Table C7-1, ASCE41-17.
Lower-bound strengths are used for Force-Controlled Actions according to 7.5.1.3, ASCE 41-17
Consequently:
New material of Primary Member: Concrete Strength, fc = fc_lower_bound = 25.00
New material of Primary Member: Steel Strength, fs = fs_lower_bound = 500.00
Concrete Elasticity, Ec = 26999.444
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).
New material: Concrete Strength, fc = fcm = 33.00
New material: Steel Strength, fs = fsm = 555.56
##
Diameter, D = 400.00
Cover Thickness, c = 25.00
Element Length, L = 3000.00
Primary 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 = 4.6357277E-010
Shear Force, Va = -1.3556528E-013
EDGE -B-
Bending Moment, Mb = -5.6659975E-011
Shear Force, Vb = 1.3556528E-013
BOTH EDGES
Axial Force, F = -4770.044
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

New component: From table 7-7, ASCE 41_17: Final Shear Capacity VR = 1.0*Vn = 323560.812
Vn ((10.3), ASCE 41-17) = knl*VCol0 = 323560.812
VCol = 323560.812
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' = 25.00, but fc^0.5 <= 8.3 MPa (22.5.3.1, ACI 318-14)
M/Vd = 2.00
Mu = 5.6659975E-011
Vu = 1.3556528E-013
d = 0.8*D = 320.00

Nu = 4770.044
Ag = 125663.706
From (11.5.4.8), ACI 318-14: Vs = 197392.088
Av = π/2*A_stirrup = 123370.055
fy = 500.00
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 <= 267132.42
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 = 7.6394471E-021
θy = (My*Ls/3)/EIeff = 0.00781319 ((4.29),Biskinis Phd))
My = 1.5905E+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 = 1.0179E+013
factor = 0.30
Ag = 125663.706
fc' = 33.00
N = 4770.044
Ec*Ig = 3.3929E+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) = 1.5905E+008
Φy = 8.1162666E-006
My_ten (8c) = 1.5905E+008
Φ_ten (7c) = 69.54133
error of function (7c) = 0.00365843
My_com (8d) = 4.3759E+008
Φ_com (7d) = 67.95301
error of function (7d) = -0.00152245
with ((10.1), ASCE 41-17) ey = Min(ey, 1.25*ey*(lb/ld)^2/3) = 0.0027778
eco = 0.002
apl = 0.35 ((9a) in Biskinis and Fardis for no FRP Wrap)
d1 = 44.00
R = 200.00
v = 0.00115027
N = 4770.044
Ac = 125663.706
((10.1), ASCE 41-17) ω = Min(ω, 1.25*ω*(lb/ld)^2/3) = 0.40909418
with fc = 33.00

Calculation of ratio lb/ld

Inadequate Lap Length with lb/ld = 0.30

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

Calculation No. 8

column C1, Floor 1

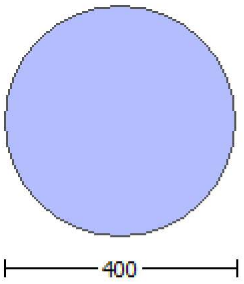
Limit State: Immediate Occupancy (data interpolation between analysis steps 1 and 2)

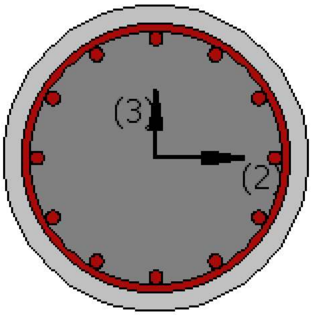
Analysis: Uniform +X

Check: Chord rotation capacity (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 = 0.75
Mean strength values are used for both shear and moment calculations.
Consequently:
New material of Primary Member: Concrete Strength, fc = fcm = 33.00
New material of Primary Member: Steel Strength, fs = fsm = 555.56
Concrete Elasticity, Ec = 26999.444
Steel Elasticity, Es = 200000.00
##
Note: Especially for the calculation of moment strengths,
the above steel re-bar strengths are multiplied by 1.25 according to R18.6.5, ACI 318-14
New material: Steel Strength, fs = 1.25*fsm = 694.45
##
Diameter, D = 400.00
Cover Thickness, c = 25.00
Mean Confinement Factor overall section = 1.00
Element Length, L = 3000.00
Primary 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 = 1.0793611E-031
EDGE -B-
Shear Force, Vb = -1.0793611E-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 = 0.31452971
Member Controlled by Flexure (Ve/Vr < 1)
Calculation of Shear Demand from fig. R18.6.5, ACI 318-14 Ve = (Mpr1 + Mpr2)/ln = 114533.572
with
Mpr1 = Max(Mu1+ , Mu1-) = 1.7180E+008
Mu1+ = 1.7180E+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- = 1.7180E+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-) = 1.7180E+008
Mu2+ = 1.7180E+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- = 1.7180E+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 = 1.7180E+008

φ = 0.99483767
φ' = 0.88186679
error of function (3.68), Biskinis Phd = 24343.261
From 5A.2, TBDY: fcc = fc*λc = 33.00
conf. factor λc = 1.00
fc = 33.00
From 10.3.5, ASCE41-17, Final value of fy: fy*Min(1,1.25*(lb/ld)^2/3) = 389.0139
lb/ld = 0.30
d1 = 44.00
R = 200.00
v = 0.0011501
N = 4771.233
Ac = 125663.706
ω = ω*Min(1,1.25*(lb/ld)^2/3) = 0.22916458

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 = 1.7180E+008

φ = 0.99483767
φ' = 0.88186679
error of function (3.68), Biskinis Phd = 24343.261
From 5A.2, TBDY: fcc = fc*λc = 33.00
conf. factor λc = 1.00
fc = 33.00
From 10.3.5, ASCE41-17, Final value of fy: fy*Min(1,1.25*(lb/ld)^2/3) = 389.0139
lb/ld = 0.30
d1 = 44.00
R = 200.00
v = 0.0011501
N = 4771.233
Ac = 125663.706
ω = ω*Min(1,1.25*(lb/ld)^2/3) = 0.22916458

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 = 1.7180E+008

φ = 0.99483767
φ' = 0.88186679
error of function (3.68), Biskinis Phd = 24343.261
From 5A.2, TBDY: fcc = fc*λc = 33.00
conf. factor λc = 1.00
fc = 33.00
From 10.3.5, ASCE41-17, Final value of fy: fy*Min(1,1.25*(lb/ld)^2/3) = 389.0139
lb/ld = 0.30
d1 = 44.00
R = 200.00
v = 0.0011501
N = 4771.233
Ac = 125663.706
ω = ω*Min(1,1.25*(lb/ld)^2/3) = 0.22916458

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 = 1.7180E+008

φ = 0.99483767
φ' = 0.88186679

error of function (3.68), Biskinis Phd = 24343.261
From 5A.2, TBDY: fcc = fc*λc = 33.00
conf. factor λc = 1.00
fc = 33.00
From 10.3.5, ASCE41-17, Final value of fy: fy*Min(1,1.25*(lb/ld)^2/3) = 389.0139
lb/ld = 0.30
d1 = 44.00
R = 200.00
v = 0.0011501
N = 4771.233
Ac = 125663.706
ω = ω*Min(1,1.25*(lb/ld)^2/3) = 0.22916458

Calculation of ratio lb/ld

Inadequate Lap Length with lb/ld = 0.30

Calculation of Shear Strength Vr = Min(Vr1,Vr2) = 364142.298

Calculation of Shear Strength at edge 1, Vr1 = 364142.298
Vr1 = VCol ((10.3), ASCE 41-17) = knl*VCol0
VCol0 = 364142.298
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' = 33.00, but fc^0.5 <= 8.3 MPa (22.5.3.1, ACI 318-14)
M/Vd = 2.00
Mu = 1.7170334E-011
Vu = 1.0793611E-031
d = 0.8*D = 320.00
Nu = 4771.233
Ag = 125663.706
From (11.5.4.8), ACI 318-14: Vs = 219326.297
Av = π/2*A_stirrup = 123370.055
fy = 555.56
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 <= 306911.784
bw*d = π*d*d/4 = 80424.772

Calculation of Shear Strength at edge 2, Vr2 = 364142.298
Vr2 = VCol ((10.3), ASCE 41-17) = knl*VCol0
VCol0 = 364142.298
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' = 33.00, but fc^0.5 <= 8.3 MPa (22.5.3.1, ACI 318-14)
M/Vd = 2.00
Mu = 1.7170334E-011
Vu = 1.0793611E-031
d = 0.8*D = 320.00
Nu = 4771.233
Ag = 125663.706
From (11.5.4.8), ACI 318-14: Vs = 219326.297

Av = π/2*A_stirrup = 123370.055
fy = 555.56
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 <= 306911.784
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 = 0.75
Mean strength values are used for both shear and moment calculations.
Consequently:
New material of Primary Member: Concrete Strength, fc = fcm = 33.00
New material of Primary Member: Steel Strength, fs = fsm = 555.56
Concrete Elasticity, Ec = 26999.444
Steel Elasticity, Es = 200000.00
###
Note: Especially for the calculation of moment strengths,
the above steel re-bar strengths are multiplied by 1.25 according to R18.6.5, ACI 318-14
New material: Steel Strength, fs = 1.25*fsm = 694.45
###
Diameter, D = 400.00
Cover Thickness, c = 25.00
Mean Confinement Factor overall section = 1.00
Element Length, L = 3000.00
Primary 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 = -6.6089624E-048
EDGE -B-
Shear Force, Vb = 6.6089624E-048
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 = 0.31452971
Member Controlled by Flexure (Ve/Vr < 1)
Calculation of Shear Demand from fig. R18.6.5, ACI 318-14 Ve = (Mpr1 + Mpr2)/ln = 114533.572
with
Mpr1 = Max(Mu1+ , Mu1-) = 1.7180E+008
Mu1+ = 1.7180E+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- = 1.7180E+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-) = 1.7180E+008
Mu2+ = 1.7180E+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- = 1.7180E+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 = 1.7180E+008

φ = 0.99483767
φ' = 0.88186679
error of function (3.68), Biskinis Phd = 24343.261
From 5A.2, TBDY: fcc = fc*λc = 33.00
conf. factor λc = 1.00
fc = 33.00
From 10.3.5, ASCE41-17, Final value of fy: fy*Min(1,1.25*(lb/ld)^2/3) = 389.0139
lb/ld = 0.30
d1 = 44.00
R = 200.00
v = 0.0011501
N = 4771.233
Ac = 125663.706
ω = ω*Min(1,1.25*(lb/ld)^2/3) = 0.22916458

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 = 1.7180E+008

φ = 0.99483767
φ' = 0.88186679
error of function (3.68), Biskinis Phd = 24343.261
From 5A.2, TBDY: fcc = fc*λc = 33.00
conf. factor λc = 1.00
fc = 33.00
From 10.3.5, ASCE41-17, Final value of fy: fy*Min(1,1.25*(lb/ld)^2/3) = 389.0139
lb/ld = 0.30
d1 = 44.00
R = 200.00
v = 0.0011501
N = 4771.233
Ac = 125663.706
ω = ω*Min(1,1.25*(lb/ld)^2/3) = 0.22916458

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 = 1.7180E+008

φ = 0.99483767
φ' = 0.88186679
error of function (3.68), Biskinis Phd = 24343.261
From 5A.2, TBDY: fcc = fc*λc = 33.00
conf. factor λc = 1.00
fc = 33.00
From 10.3.5, ASCE41-17, Final value of fy: fy*Min(1,1.25*(lb/ld)^2/3) = 389.0139
lb/ld = 0.30
d1 = 44.00
R = 200.00
v = 0.0011501
N = 4771.233
Ac = 125663.706
ω = ω*Min(1,1.25*(lb/ld)^2/3) = 0.22916458

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 = 1.7180E+008

φ = 0.99483767
φ' = 0.88186679
error of function (3.68), Biskinis Phd = 24343.261
From 5A.2, TBDY: fcc = fc*λc = 33.00
conf. factor λc = 1.00
fc = 33.00
From 10.3.5, ASCE41-17, Final value of fy: fy*Min(1,1.25*(lb/ld)^2/3) = 389.0139
lb/ld = 0.30
d1 = 44.00
R = 200.00
v = 0.0011501
N = 4771.233
Ac = 125663.706
ω = ω*Min(1,1.25*(lb/ld)^2/3) = 0.22916458

Calculation of ratio lb/ld

Inadequate Lap Length with lb/ld = 0.30

Calculation of Shear Strength Vr = Min(Vr1,Vr2) = 364142.298

Calculation of Shear Strength at edge 1, Vr1 = 364142.298
Vr1 = VCol ((10.3), ASCE 41-17) = knl*VCol0
VCol0 = 364142.298
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' = 33.00, but fc^0.5 <= 8.3 MPa (22.5.3.1, ACI 318-14)
M/Vd = 2.00
Mu = 5.2883590E-012
Vu = 6.6089624E-048
d = 0.8*D = 320.00
Nu = 4771.233
Ag = 125663.706
From (11.5.4.8), ACI 318-14: Vs = 219326.297
Av = π/2*A_stirrup = 123370.055
fy = 555.56
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 <= 306911.784
bw*d = π*d*d/4 = 80424.772

Calculation of Shear Strength at edge 2, Vr2 = 364142.298
Vr2 = VCol ((10.3), ASCE 41-17) = knl*VCol0
VCol0 = 364142.298
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' = 33.00, but fc^0.5 <= 8.3 MPa (22.5.3.1, ACI 318-14)
M/Vd = 2.00
Mu = 5.2883590E-012
Vu = 6.6089624E-048
d = 0.8*D = 320.00
Nu = 4771.233
Ag = 125663.706
From (11.5.4.8), ACI 318-14: Vs = 219326.297
Av = π/2*A_stirrup = 123370.055
fy = 555.56
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 <= 306911.784
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 = 0.75
Chord Rotation is generally considered as Deformation-Controlled Action according to Table C7-1, ASCE41-17.
Mean strengths are used for Deformation-Controlled Actions according to 7.5.1.3, ASCE 41-17
Consequently:
New material of Primary Member: Concrete Strength, fc = fcm = 33.00
New material of Primary Member: Steel Strength, fs = fsm = 555.56
Concrete Elasticity, Ec = 26999.444
Steel Elasticity, Es = 200000.00
Diameter, D = 400.00
Cover Thickness, c = 25.00
Element Length, L = 3000.00
Primary Member
Ribbed Bars
Ductile Steel
Without Detailing for Earthquake Resistance (including stirrups not closed at 135°)
Longitudinal Bars With Ends Lapped Starting at the End Sections
Inadequate Lap Length with lb/ld = 0.30
No FRP Wrapping

Stepwise Properties

Bending Moment, M = 0.00304337
Shear Force, V2 = 2974.321
Shear Force, V3 = 1.3556528E-013
Axial Force, F = -4770.044
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

New component: From table 7-7, ASCE 41_17: Final chord rotation Capacity θu,R = 1.0*θu = 0.00156264
θu = θy + θp = 0.00156264

- Calculation of θy -

θy = (My*Ls/3)/EIeff = 0.00156264 ((4.29),Biskinis Phd))
My = 1.5905E+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 = 1.0179E+013
factor = 0.30
Ag = 125663.706
fc' = 33.00
N = 4770.044
Ec*Ig = 3.3929E+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) = 1.5905E+008
Φy = 8.1162666E-006
My_ten (8c) = 1.5905E+008
Φ_ten (7c) = 69.54133
error of function (7c) = 0.00365843

My_com (8d) = 4.3759E+008
Φ_com (7d) = 67.95301
error of function (7d) = -0.00152245
with ((10.1), ASCE 41-17) ey = Min(ey, 1.25*ey*(lb/ld)^2/3) = 0.0027778
eco = 0.002
apl = 0.35 ((9a) in Biskinis and Fardis for no FRP Wrap)
d1 = 44.00
R = 200.00
v = 0.00115027
N = 4770.044
Ac = 125663.706
((10.1), ASCE 41-17) ω = Min(ω, 1.25*ω*(lb/ld)^2/3) = 0.40909418
with fc = 33.00

Calculation of ratio lb/ld

Inadequate Lap Length with lb/ld = 0.30

- Calculation of θp -

From table 10-9: θp = 0.00
with:
- Columns not controlled by inadequate development or splicing along the clear height because lb/ld >= 1
shear control ratio VyE/VCol0E = 0.31452971
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 = 4770.044
Ag = 125663.706
f'cE = 33.00
fytE = fylE = 555.56
pl = Area_Tot_Long_Rein/(Ag) = 0.0243
f'cE = 33.00

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

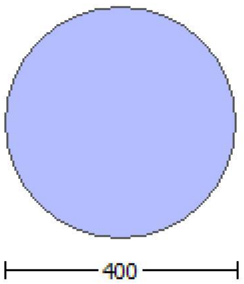
Limit State: Life Safety (data interpolation between analysis steps 1 and 2)

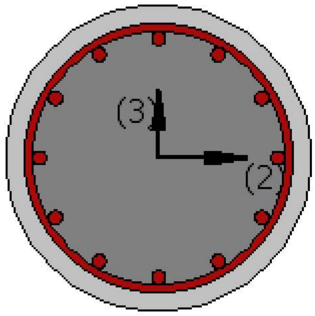
Analysis: Uniform +X

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 = 0.75
Member Shear Force is generally considered as Force-Controlled Action according to Table C7-1, ASCE41-17.
Lower-bound strengths are used for Force-Controlled Actions according to 7.5.1.3, ASCE 41-17
Consequently:
New material of Primary Member: Concrete Strength, fc = fc_lower_bound = 25.00
New material of Primary Member: Steel Strength, fs = fs_lower_bound = 500.00
Concrete Elasticity, Ec = 26999.444
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).
New material: Concrete Strength, fc = fcm = 33.00
New material: Steel Strength, fs = fsm = 555.56
###
Diameter, D = 400.00
Cover Thickness, c = 25.00
Element Length, L = 3000.00
Primary 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 = -1.4130E+007
Shear Force, Va = -4708.281

EDGE -B-
Bending Moment, Mb = 0.00481758
Shear Force, Vb = 4708.281
BOTH EDGES
Axial Force, F = -4769.35
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

New component: From table 7-7, ASCE 41_17: Final Shear Capacity VR = 1.0*Vn = 260476.381
Vn ((10.3), ASCE 41-17) = knl*VCol0 = 260476.381
VCol = 260476.381
knl = 1.00
displacement_ductility_demand = 0.0242928

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' = 25.00, but fc^0.5 <= 8.3 MPa (22.5.3.1, ACI 318-14)
M/Vd = 4.00
Mu = 1.4130E+007
Vu = 4708.281
d = 0.8*D = 320.00
Nu = 4769.35
Ag = 125663.706
From (11.5.4.8), ACI 318-14: Vs = 197392.088
Av = π/2*A_stirrup = 123370.055
fy = 500.00
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 <= 267132.42
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 = 0.00037975
θy = (My*Ls/3)/Eleff = 0.01563236 ((4.29),Biskinis Phd))
My = 1.5905E+008
Ls = M/V (with Ls >0.1*L and Ls < 2*L) = 3001.15
From table 10.5, ASCE 41_17: EIeff = factor*Ec*Ig = 1.0179E+013
factor = 0.30
Ag = 125663.706
fc' = 33.00
N = 4769.35
Ec*Ig = 3.3929E+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) = 1.5905E+008
Φy = 8.1162642E-006
My_ten (8c) = 1.5905E+008
Φ_ten (7c) = 69.54131
error of function (7c) = 0.0036586
My_com (8d) = 4.3759E+008
Φ_com (7d) = 67.953
error of function (7d) = -0.00152246
with ((10.1), ASCE 41-17) ey = Min(ey, 1.25*ey*(lb/ld)^2/3) = 0.0027778
eco = 0.002
apl = 0.35 ((9a) in Biskinis and Fardis for no FRP Wrap)
d1 = 44.00
R = 200.00
v = 0.0011501
N = 4769.35
Ac = 125663.706
((10.1), ASCE 41-17) ω = Min(ω, 1.25*ω*(lb/ld)^2/3) = 0.40909418
with fc = 33.00

Calculation of ratio lb/ld

Inadequate Lap Length with lb/ld = 0.30

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

Calculation No. 10

column C1, Floor 1

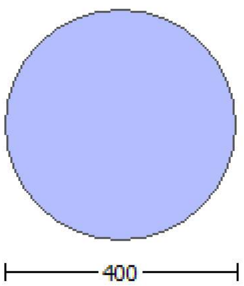
Limit State: Life Safety (data interpolation between analysis steps 1 and 2)

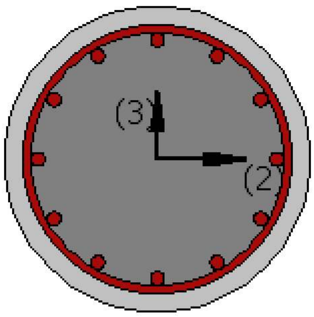
Analysis: Uniform +X

Check: Chord rotation capacity (θu)

Edge: Start

Local Axis: (2)





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

Constant Properties

Knowledge Factor, κ = 0.75
Mean strength values are used for both shear and moment calculations.
Consequently:
New material of Primary Member: Concrete Strength, fc = fcm = 33.00
New material of Primary Member: Steel Strength, fs = fsm = 555.56
Concrete Elasticity, Ec = 26999.444
Steel Elasticity, Es = 200000.00
##
Note: Especially for the calculation of moment strengths,
the above steel re-bar strengths are multiplied by 1.25 according to R18.6.5, ACI 318-14
New material: Steel Strength, fs = 1.25*fsm = 694.45
##
Diameter, D = 400.00
Cover Thickness, c = 25.00
Mean Confinement Factor overall section = 1.00
Element Length, L = 3000.00
Primary 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 = 1.0793611E-031
EDGE -B-
Shear Force, Vb = -1.0793611E-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 = 0.31452971
Member Controlled by Flexure (Ve/Vr < 1)
Calculation of Shear Demand from fig. R18.6.5, ACI 318-14 Ve = (Mpr1 + Mpr2)/ln = 114533.572
with
Mpr1 = Max(Mu1+ , Mu1-) = 1.7180E+008
Mu1+ = 1.7180E+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- = 1.7180E+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-) = 1.7180E+008
Mu2+ = 1.7180E+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- = 1.7180E+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 = 1.7180E+008

φ = 0.99483767
φ' = 0.88186679
error of function (3.68), Biskinis Phd = 24343.261
From 5A.2, TBDY: fcc = fc*λc = 33.00
conf. factor λc = 1.00
fc = 33.00
From 10.3.5, ASCE41-17, Final value of fy: fy*Min(1,1.25*(lb/ld)^2/3) = 389.0139
lb/ld = 0.30
d1 = 44.00
R = 200.00
v = 0.0011501
N = 4771.233
Ac = 125663.706
ω = ω*Min(1,1.25*(lb/ld)^2/3) = 0.22916458

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 = 1.7180E+008

φ = 0.99483767
φ' = 0.88186679
error of function (3.68), Biskinis Phd = 24343.261
From 5A.2, TBDY: fcc = fc*λc = 33.00
conf. factor λc = 1.00
fc = 33.00
From 10.3.5, ASCE41-17, Final value of fy: fy*Min(1,1.25*(lb/ld)^2/3) = 389.0139
lb/ld = 0.30
d1 = 44.00
R = 200.00
v = 0.0011501
N = 4771.233
Ac = 125663.706
ω = ω*Min(1,1.25*(lb/ld)^2/3) = 0.22916458

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 = 1.7180E+008

φ = 0.99483767
φ' = 0.88186679
error of function (3.68), Biskinis Phd = 24343.261
From 5A.2, TBDY: fcc = fc*λc = 33.00
conf. factor λc = 1.00
fc = 33.00
From 10.3.5, ASCE41-17, Final value of fy: fy*Min(1,1.25*(lb/ld)^2/3) = 389.0139
lb/ld = 0.30
d1 = 44.00
R = 200.00
v = 0.0011501
N = 4771.233
Ac = 125663.706
ω = ω*Min(1,1.25*(lb/ld)^2/3) = 0.22916458

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 = 1.7180E+008

φ = 0.99483767
φ' = 0.88186679
error of function (3.68), Biskinis Phd = 24343.261
From 5A.2, TBDY: fcc = fc*λc = 33.00
conf. factor λc = 1.00
fc = 33.00
From 10.3.5, ASCE41-17, Final value of fy: fy*Min(1,1.25*(lb/ld)^2/3) = 389.0139
lb/ld = 0.30
d1 = 44.00
R = 200.00
v = 0.0011501
N = 4771.233
Ac = 125663.706
ω = ω*Min(1,1.25*(lb/ld)^2/3) = 0.22916458

Calculation of ratio lb/ld

Inadequate Lap Length with lb/ld = 0.30

Calculation of Shear Strength Vr = Min(Vr1,Vr2) = 364142.298

Calculation of Shear Strength at edge 1, Vr1 = 364142.298
Vr1 = VCol ((10.3), ASCE 41-17) = knl*VCol0
VCol0 = 364142.298
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' = 33.00, but fc^0.5 <= 8.3 MPa (22.5.3.1, ACI 318-14)
M/Vd = 2.00
Mu = 1.7170334E-011

Vu = 1.0793611E-031
d = 0.8*D = 320.00
Nu = 4771.233
Ag = 125663.706
From (11.5.4.8), ACI 318-14: Vs = 219326.297
Av = π/2*A_stirrup = 123370.055
fy = 555.56
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 <= 306911.784
bw*d = π*d*d/4 = 80424.772

Calculation of Shear Strength at edge 2, Vr2 = 364142.298
Vr2 = VCol ((10.3), ASCE 41-17) = knl*VCol0
VCol0 = 364142.298
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' = 33.00, but fc^0.5 <= 8.3 MPa (22.5.3.1, ACI 318-14)
M/Vd = 2.00
Mu = 1.7170334E-011
Vu = 1.0793611E-031
d = 0.8*D = 320.00
Nu = 4771.233
Ag = 125663.706
From (11.5.4.8), ACI 318-14: Vs = 219326.297
Av = π/2*A_stirrup = 123370.055
fy = 555.56
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 <= 306911.784
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 = 0.75
Mean strength values are used for both shear and moment calculations.
Consequently:
New material of Primary Member: Concrete Strength, fc = fcm = 33.00
New material of Primary Member: Steel Strength, fs = fsm = 555.56
Concrete Elasticity, Ec = 26999.444
Steel Elasticity, Es = 200000.00
##
Note: Especially for the calculation of moment strengths,
the above steel re-bar strengths are multiplied by 1.25 according to R18.6.5, ACI 318-14
New material: Steel Strength, fs = 1.25*fsm = 694.45
##

Diameter, D = 400.00
Cover Thickness, c = 25.00
Mean Confinement Factor overall section = 1.00
Element Length, L = 3000.00
Primary 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 = -6.6089624E-048
EDGE -B-
Shear Force, Vb = 6.6089624E-048
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 = 0.31452971
Member Controlled by Flexure (Ve/Vr < 1)
Calculation of Shear Demand from fig. R18.6.5, ACI 318-14 Ve = (Mpr1 + Mpr2)/ln = 114533.572
with
Mpr1 = Max(Mu1+ , Mu1-) = 1.7180E+008
Mu1+ = 1.7180E+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- = 1.7180E+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-) = 1.7180E+008
Mu2+ = 1.7180E+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- = 1.7180E+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 = 1.7180E+008

φ = 0.99483767
φ' = 0.88186679
error of function (3.68), Biskinis Phd = 24343.261
From 5A.2, TBDY: fcc = fc*λc = 33.00
conf. factor λc = 1.00
fc = 33.00
From 10.3.5, ASCE41-17, Final value of fy: fy*Min(1,1.25*(lb/ld)^2/3) = 389.0139
lb/ld = 0.30
d1 = 44.00
R = 200.00
v = 0.0011501

N = 4771.233
Ac = 125663.706
ω = ω*Min(1,1.25*(lb/ld)^2/3) = 0.22916458

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 = 1.7180E+008

φ = 0.99483767
φ' = 0.88186679
error of function (3.68), Biskinis Phd = 24343.261
From 5A.2, TBDY: fcc = fc*λc = 33.00
conf. factor λc = 1.00
fc = 33.00
From 10.3.5, ASCE41-17, Final value of fy: fy*Min(1,1.25*(lb/ld)^2/3) = 389.0139
lb/ld = 0.30
d1 = 44.00
R = 200.00
v = 0.0011501
N = 4771.233
Ac = 125663.706
ω = ω*Min(1,1.25*(lb/ld)^2/3) = 0.22916458

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 = 1.7180E+008

φ = 0.99483767
φ' = 0.88186679
error of function (3.68), Biskinis Phd = 24343.261
From 5A.2, TBDY: fcc = fc*λc = 33.00
conf. factor λc = 1.00
fc = 33.00
From 10.3.5, ASCE41-17, Final value of fy: fy*Min(1,1.25*(lb/ld)^2/3) = 389.0139
lb/ld = 0.30
d1 = 44.00
R = 200.00
v = 0.0011501
N = 4771.233
Ac = 125663.706
ω = ω*Min(1,1.25*(lb/ld)^2/3) = 0.22916458

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 = 1.7180E+008

φ = 0.99483767
φ' = 0.88186679
error of function (3.68), Biskinis Phd = 24343.261
From 5A.2, TBDY: fcc = fc*λc = 33.00
conf. factor λc = 1.00
fc = 33.00
From 10.3.5, ASCE41-17, Final value of fy: fy*Min(1,1.25*(lb/ld)^2/3) = 389.0139
lb/ld = 0.30
d1 = 44.00
R = 200.00
v = 0.0011501
N = 4771.233
Ac = 125663.706
ω = ω*Min(1,1.25*(lb/ld)^2/3) = 0.22916458

Calculation of ratio lb/ld

Inadequate Lap Length with lb/ld = 0.30

Calculation of Shear Strength Vr = Min(Vr1,Vr2) = 364142.298

Calculation of Shear Strength at edge 1, Vr1 = 364142.298
Vr1 = VCol ((10.3), ASCE 41-17) = knl*VCol0
VCol0 = 364142.298
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' = 33.00, but fc^0.5 <= 8.3 MPa (22.5.3.1, ACI 318-14)
M/Vd = 2.00
Mu = 5.2883590E-012
Vu = 6.6089624E-048
d = 0.8*D = 320.00
Nu = 4771.233
Ag = 125663.706
From (11.5.4.8), ACI 318-14: Vs = 219326.297
Av = π/2*A_stirrup = 123370.055
fy = 555.56
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 <= 306911.784
bw*d = π*d*d/4 = 80424.772

Calculation of Shear Strength at edge 2, Vr2 = 364142.298
Vr2 = VCol ((10.3), ASCE 41-17) = knl*VCol0
VCol0 = 364142.298
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' = 33.00, but fc^0.5 <= 8.3 MPa (22.5.3.1, ACI 318-14)
M/Vd = 2.00
Mu = 5.2883590E-012
Vu = 6.6089624E-048
d = 0.8*D = 320.00
Nu = 4771.233
Ag = 125663.706
From (11.5.4.8), ACI 318-14: Vs = 219326.297
Av = π/2*A_stirrup = 123370.055
fy = 555.56
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 <= 306911.784
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 = 0.75
Chord Rotation is generally considered as Deformation-Controlled Action according to Table C7-1, ASCE41-17.
Mean strengths are used for Deformation-Controlled Actions according to 7.5.1.3, ASCE 41-17
Consequently:
New material of Primary Member: Concrete Strength, fc = fcm = 33.00
New material of Primary Member: Steel Strength, fs = fsm = 555.56
Concrete Elasticity, Ec = 26999.444
Steel Elasticity, Es = 200000.00
Diameter, D = 400.00
Cover Thickness, c = 25.00
Element Length, L = 3000.00
Primary 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 = 7.2381509E-010
Shear Force, V2 = -4708.281
Shear Force, V3 = -2.1459671E-013
Axial Force, F = -4769.35
Longitudinal Reinforcement Area Distribution (in 2 divisions)
-Tension: Aslt = 1272.345
-Compression: Aslc = 1781.283
Longitudinal Reinforcement Area Distribution (in 3 divisions)
-Tension: Asl,ten = 1017.876
-Compression: Asl,com = 1017.876
-Middle: Asl,mid = 1017.876

Mean Diameter of Tension Reinforcement, DbL = 18.00

New component: From table 7-7, ASCE 41_17: Final chord rotation Capacity θu,R = 1.0*θu = 0.03728312
θu = θy + θp = 0.03728312

- Calculation of θy -

θy = (My*Ls/3)/EIeff = 0.00781318 ((4.29),Biskinis Phd))
My = 1.5905E+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 = 1.0179E+013
factor = 0.30
Ag = 125663.706
fc' = 33.00
N = 4769.35
Ec*Ig = 3.3929E+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) = 1.5905E+008
Φy = 8.1162642E-006
My_ten (8c) = 1.5905E+008
Φ_ten (7c) = 69.54131
error of function (7c) = 0.0036586
My_com (8d) = 4.3759E+008
Φ_com (7d) = 67.953
error of function (7d) = -0.00152246
with ((10.1), ASCE 41-17) ey = Min(ey, 1.25*ey*(lb/ld)^2/3) = 0.0027778
eco = 0.002
apl = 0.35 ((9a) in Biskinis and Fardis for no FRP Wrap)
d1 = 44.00
R = 200.00
v = 0.0011501
N = 4769.35
Ac = 125663.706
((10.1), ASCE 41-17) ω = Min(ω, 1.25*ω*(lb/ld)^2/3) = 0.40909418
with fc = 33.00

Calculation of ratio lb/ld

Inadequate Lap Length with lb/ld = 0.30

- Calculation of θp -

From table 10-9: θp = 0.02946994
with:
- Columns not controlled by inadequate development or splicing along the clear height because lb/ld >= 1
shear control ratio VyE/VCol0E = 0.31452971
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 = 4769.35
Ag = 125663.706
f'cE = 33.00
fytE = fylE = 555.56
pl = Area_Tot_Long_Rein/(Ag) = 0.0243
f'cE = 33.00

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

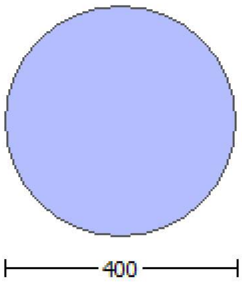
Limit State: Life Safety (data interpolation between analysis steps 1 and 2)

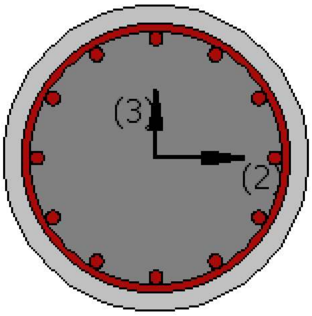
Analysis: Uniform +X

Check: Shear capacity VRd

Edge: Start

Local Axis: (3)





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

Constant Properties

Knowledge Factor, κ = 0.75
Member Shear Force is generally considered as Force-Controlled Action according to Table C7-1, ASCE41-17.
Lower-bound strengths are used for Force-Controlled Actions according to 7.5.1.3, ASCE 41-17
Consequently:
New material of Primary Member: Concrete Strength, fc = fc_lower_bound = 25.00
New material of Primary Member: Steel Strength, fs = fs_lower_bound = 500.00
Concrete Elasticity, Ec = 26999.444
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).
New material: Concrete Strength, fc = fcm = 33.00
New material: Steel Strength, fs = fsm = 555.56
##
Diameter, D = 400.00
Cover Thickness, c = 25.00
Element Length, L = 3000.00
Primary 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 = 7.2381509E-010
Shear Force, Va = -2.1459671E-013
EDGE -B-
Bending Moment, Mb = -7.9681523E-011
Shear Force, Vb = 2.1459671E-013
BOTH EDGES
Axial Force, F = -4769.35
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

New component: From table 7-7, ASCE 41_17: Final Shear Capacity VR = 1.0*Vn = 323560.674
Vn ((10.3), ASCE 41-17) = knl*VCol0 = 323560.674
VCol = 323560.674
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' = 25.00, but fc^0.5 <= 8.3 MPa (22.5.3.1, ACI 318-14)
M/Vd = 2.00
Mu = 7.2381509E-010
Vu = 2.1459671E-013
d = 0.8*D = 320.00
Nu = 4769.35
Ag = 125663.706
From (11.5.4.8), ACI 318-14: Vs = 197392.088
Av = π/2*A_stirrup = 123370.055
fy = 500.00
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 <= 267132.42
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 = 2.4058428E-020
θy = (My*Ls/3)/Eleff = 0.00781318 ((4.29),Biskinis Phd))
My = 1.5905E+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 = 1.0179E+013
factor = 0.30
Ag = 125663.706
fc' = 33.00
N = 4769.35
Ec*Ig = 3.3929E+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) = 1.5905E+008
Φy = 8.1162642E-006
My_ten (8c) = 1.5905E+008
Φ_ten (7c) = 69.54131
error of function (7c) = 0.0036586
My_com (8d) = 4.3759E+008
Φ_com (7d) = 67.953
error of function (7d) = -0.00152246
with ((10.1), ASCE 41-17) ey = Min(ey, 1.25*ey*(lb/ld)^2/3) = 0.0027778
eco = 0.002
apl = 0.35 ((9a) in Biskinis and Fardis for no FRP Wrap)
d1 = 44.00
R = 200.00
v = 0.0011501
N = 4769.35
Ac = 125663.706
((10.1), ASCE 41-17) ω = Min(ω, 1.25*ω*(lb/ld)^2/3) = 0.40909418
with fc = 33.00

Calculation of ratio lb/ld

Inadequate Lap Length with lb/ld = 0.30

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

Calculation No. 12

column C1, Floor 1

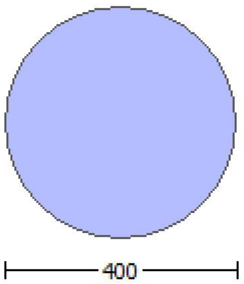
Limit State: Life Safety (data interpolation between analysis steps 1 and 2)

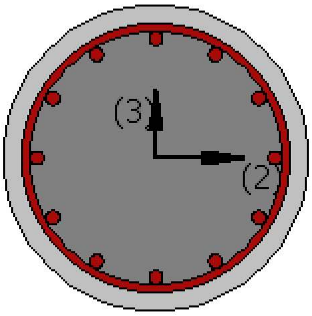
Analysis: Uniform +X

Check: Chord rotation capacity (θu)

Edge: Start

Local Axis: (3)





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

Constant Properties

Knowledge Factor, κ = 0.75
Mean strength values are used for both shear and moment calculations.
Consequently:
New material of Primary Member: Concrete Strength, fc = fcm = 33.00
New material of Primary Member: Steel Strength, fs = fsm = 555.56
Concrete Elasticity, Ec = 26999.444
Steel Elasticity, Es = 200000.00
###
Note: Especially for the calculation of moment strengths,
the above steel re-bar strengths are multiplied by 1.25 according to R18.6.5, ACI 318-14
New material: Steel Strength, fs = 1.25*fsm = 694.45
###
Diameter, D = 400.00
Cover Thickness, c = 25.00
Mean Confinement Factor overall section = 1.00
Element Length, L = 3000.00
Primary 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 = 1.0793611E-031
EDGE -B-
Shear Force, Vb = -1.0793611E-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 = 0.31452971
Member Controlled by Flexure (Ve/Vr < 1)
Calculation of Shear Demand from fig. R18.6.5, ACI 318-14 Ve = (Mpr1 + Mpr2)/ln = 114533.572
with
Mpr1 = Max(Mu1+ , Mu1-) = 1.7180E+008
Mu1+ = 1.7180E+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- = 1.7180E+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-) = 1.7180E+008
Mu2+ = 1.7180E+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- = 1.7180E+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 = 1.7180E+008

φ = 0.99483767
φ' = 0.88186679
error of function (3.68), Biskinis Phd = 24343.261
From 5A.2, TBDY: fcc = fc*λc = 33.00
conf. factor λc = 1.00
fc = 33.00
From 10.3.5, ASCE41-17, Final value of fy: fy*Min(1,1.25*(lb/ld)^2/3) = 389.0139
lb/ld = 0.30
d1 = 44.00
R = 200.00
v = 0.0011501
N = 4771.233
Ac = 125663.706
ω = ω*Min(1,1.25*(lb/ld)^2/3) = 0.22916458

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 = 1.7180E+008

φ = 0.99483767
φ' = 0.88186679

error of function (3.68), Biskinis Phd = 24343.261
From 5A.2, TBDY: fcc = fc*λc = 33.00
conf. factor λc = 1.00
fc = 33.00
From 10.3.5, ASCE41-17, Final value of fy: fy*Min(1,1.25*(lb/ld)^2/3) = 389.0139
lb/ld = 0.30
d1 = 44.00
R = 200.00
v = 0.0011501
N = 4771.233
Ac = 125663.706
ω = ω*Min(1,1.25*(lb/ld)^2/3) = 0.22916458

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 = 1.7180E+008

φ = 0.99483767
φ' = 0.88186679
error of function (3.68), Biskinis Phd = 24343.261
From 5A.2, TBDY: fcc = fc*λc = 33.00
conf. factor λc = 1.00
fc = 33.00
From 10.3.5, ASCE41-17, Final value of fy: fy*Min(1,1.25*(lb/ld)^2/3) = 389.0139
lb/ld = 0.30
d1 = 44.00
R = 200.00
v = 0.0011501
N = 4771.233
Ac = 125663.706
ω = ω*Min(1,1.25*(lb/ld)^2/3) = 0.22916458

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 = 1.7180E+008

φ = 0.99483767
φ' = 0.88186679
error of function (3.68), Biskinis Phd = 24343.261
From 5A.2, TBDY: fcc = fc*λc = 33.00
conf. factor λc = 1.00
fc = 33.00
From 10.3.5, ASCE41-17, Final value of fy: fy*Min(1,1.25*(lb/ld)^2/3) = 389.0139
lb/ld = 0.30
d1 = 44.00
R = 200.00
v = 0.0011501

N = 4771.233
Ac = 125663.706
ω = ω*Min(1,1.25*(lb/ld)^2/3) = 0.22916458

Calculation of ratio lb/ld

Inadequate Lap Length with lb/ld = 0.30

Calculation of Shear Strength Vr = Min(Vr1,Vr2) = 364142.298

Calculation of Shear Strength at edge 1, Vr1 = 364142.298
Vr1 = VCol ((10.3), ASCE 41-17) = knl*VCol0
VCol0 = 364142.298
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' = 33.00, but fc^0.5 <= 8.3 MPa (22.5.3.1, ACI 318-14)
M/Vd = 2.00
Mu = 1.7170334E-011
Vu = 1.0793611E-031
d = 0.8*D = 320.00
Nu = 4771.233
Ag = 125663.706
From (11.5.4.8), ACI 318-14: Vs = 219326.297
Av = π/2*A_stirrup = 123370.055
fy = 555.56
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 <= 306911.784
bw*d = π*d*d/4 = 80424.772

Calculation of Shear Strength at edge 2, Vr2 = 364142.298
Vr2 = VCol ((10.3), ASCE 41-17) = knl*VCol0
VCol0 = 364142.298
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' = 33.00, but fc^0.5 <= 8.3 MPa (22.5.3.1, ACI 318-14)
M/Vd = 2.00
Mu = 1.7170334E-011
Vu = 1.0793611E-031
d = 0.8*D = 320.00
Nu = 4771.233
Ag = 125663.706
From (11.5.4.8), ACI 318-14: Vs = 219326.297
Av = π/2*A_stirrup = 123370.055
fy = 555.56
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 <= 306911.784
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 = 0.75
Mean strength values are used for both shear and moment calculations.
Consequently:
New material of Primary Member: Concrete Strength, fc = fcm = 33.00
New material of Primary Member: Steel Strength, fs = fsm = 555.56
Concrete Elasticity, Ec = 26999.444
Steel Elasticity, Es = 200000.00
###
Note: Especially for the calculation of moment strengths,
the above steel re-bar strengths are multiplied by 1.25 according to R18.6.5, ACI 318-14
New material: Steel Strength, fs = 1.25*fsm = 694.45
###
Diameter, D = 400.00
Cover Thickness, c = 25.00
Mean Confinement Factor overall section = 1.00
Element Length, L = 3000.00
Primary 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 = -6.6089624E-048
EDGE -B-
Shear Force, Vb = 6.6089624E-048
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 = 0.31452971
Member Controlled by Flexure (Ve/Vr < 1)
Calculation of Shear Demand from fig. R18.6.5, ACI 318-14 Ve = (Mpr1 + Mpr2)/ln = 114533.572
with
Mpr1 = Max(Mu1+ , Mu1-) = 1.7180E+008
Mu1+ = 1.7180E+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- = 1.7180E+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-) = 1.7180E+008
Mu2+ = 1.7180E+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- = 1.7180E+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 = 1.7180E+008

φ = 0.99483767
φ' = 0.88186679
error of function (3.68), Biskinis Phd = 24343.261
From 5A.2, TBDY: fcc = fc*λc = 33.00
conf. factor λc = 1.00
fc = 33.00
From 10.3.5, ASCE41-17, Final value of fy: fy*Min(1,1.25*(lb/ld)^2/3) = 389.0139
lb/ld = 0.30
d1 = 44.00
R = 200.00
v = 0.0011501
N = 4771.233
Ac = 125663.706
ω = ω*Min(1,1.25*(lb/ld)^2/3) = 0.22916458

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 = 1.7180E+008

φ = 0.99483767
φ' = 0.88186679
error of function (3.68), Biskinis Phd = 24343.261
From 5A.2, TBDY: fcc = fc*λc = 33.00
conf. factor λc = 1.00
fc = 33.00
From 10.3.5, ASCE41-17, Final value of fy: fy*Min(1,1.25*(lb/ld)^2/3) = 389.0139
lb/ld = 0.30
d1 = 44.00
R = 200.00
v = 0.0011501
N = 4771.233
Ac = 125663.706
ω = ω*Min(1,1.25*(lb/ld)^2/3) = 0.22916458

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 = 1.7180E+008

φ = 0.99483767
φ' = 0.88186679
error of function (3.68), Biskinis Phd = 24343.261
From 5A.2, TBDY: fcc = fc*λc = 33.00
conf. factor λc = 1.00
fc = 33.00
From 10.3.5, ASCE41-17, Final value of fy: fy*Min(1,1.25*(lb/ld)^2/3) = 389.0139
lb/ld = 0.30
d1 = 44.00
R = 200.00
v = 0.0011501
N = 4771.233
Ac = 125663.706
ω = ω*Min(1,1.25*(lb/ld)^2/3) = 0.22916458

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 = 1.7180E+008

φ = 0.99483767
φ' = 0.88186679
error of function (3.68), Biskinis Phd = 24343.261
From 5A.2, TBDY: fcc = fc*λc = 33.00
conf. factor λc = 1.00
fc = 33.00
From 10.3.5, ASCE41-17, Final value of fy: fy*Min(1,1.25*(lb/ld)^2/3) = 389.0139
lb/ld = 0.30
d1 = 44.00
R = 200.00
v = 0.0011501
N = 4771.233
Ac = 125663.706
ω = ω*Min(1,1.25*(lb/ld)^2/3) = 0.22916458

Calculation of ratio lb/ld

Inadequate Lap Length with lb/ld = 0.30

Calculation of Shear Strength Vr = Min(Vr1,Vr2) = 364142.298

Calculation of Shear Strength at edge 1, Vr1 = 364142.298
Vr1 = VCol ((10.3), ASCE 41-17) = knl*VCol0
VCol0 = 364142.298
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' = 33.00, but fc^0.5 <= 8.3 MPa (22.5.3.1, ACI 318-14)
M/Vd = 2.00
Mu = 5.2883590E-012
Vu = 6.6089624E-048
d = 0.8*D = 320.00
Nu = 4771.233
Ag = 125663.706
From (11.5.4.8), ACI 318-14: Vs = 219326.297
Av = π/2*A_stirrup = 123370.055
fy = 555.56
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 <= 306911.784
bw*d = π*d*d/4 = 80424.772

Calculation of Shear Strength at edge 2, Vr2 = 364142.298
Vr2 = VCol ((10.3), ASCE 41-17) = knl*VCol0
VCol0 = 364142.298
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' = 33.00, but fc^0.5 <= 8.3 MPa (22.5.3.1, ACI 318-14)
M/Vd = 2.00
Mu = 5.2883590E-012
Vu = 6.6089624E-048
d = 0.8*D = 320.00
Nu = 4771.233
Ag = 125663.706
From (11.5.4.8), ACI 318-14: Vs = 219326.297
Av = π/2*A_stirrup = 123370.055
fy = 555.56
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 <= 306911.784
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 = 0.75
Chord Rotation is generally considered as Deformation-Controlled Action according to Table C7-1, ASCE41-17.
Mean strengths are used for Deformation-Controlled Actions according to 7.5.1.3, ASCE 41-17
Consequently:
New material of Primary Member: Concrete Strength, fc = fcm = 33.00
New material of Primary Member: Steel Strength, fs = fsm = 555.56
Concrete Elasticity, Ec = 26999.444

Steel Elasticity, Es = 200000.00
Diameter, D = 400.00
Cover Thickness, c = 25.00
Element Length, L = 3000.00
Primary 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 = -1.4130E+007
Shear Force, V2 = -4708.281
Shear Force, V3 = -2.1459671E-013
Axial Force, F = -4769.35
Longitudinal Reinforcement Area Distribution (in 2 divisions)
-Tension: Aslt = 1272.345
-Compression: Aslc = 1781.283
Longitudinal Reinforcement Area Distribution (in 3 divisions)
-Tension: Asl,ten = 1017.876
-Compression: Asl,com = 1017.876
-Middle: Asl,mid = 1017.876
Mean Diameter of Tension Reinforcement, DbL = 18.00

New component: From table 7-7, ASCE 41_17: Final chord rotation Capacity θu,R = 1.0*θu = 0.04510229
θu = θy + θp = 0.04510229

- Calculation of θy -

θy = (My*Ls/3)/EIeff = 0.01563236 ((4.29),Biskinis Phd))
My = 1.5905E+008
Ls = M/V (with Ls >0.1*L and Ls < 2*L) = 3001.15
From table 10.5, ASCE 41_17: EIeff = factor*Ec*Ig = 1.0179E+013
factor = 0.30
Ag = 125663.706
fc' = 33.00
N = 4769.35
Ec*Ig = 3.3929E+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) = 1.5905E+008
Φy = 8.1162642E-006
My_ten (8c) = 1.5905E+008
Φ_ten (7c) = 69.54131
error of function (7c) = 0.0036586
My_com (8d) = 4.3759E+008
Φ_com (7d) = 67.953
error of function (7d) = -0.00152246
with ((10.1), ASCE 41-17) ey = Min(ey, 1.25*ey*(lb/ld)^2/3) = 0.0027778
eco = 0.002
apl = 0.35 ((9a) in Biskinis and Fardis for no FRP Wrap)
d1 = 44.00
R = 200.00
v = 0.0011501

N = 4769.35
Ac = 125663.706
((10.1), ASCE 41-17) ω = Min(ω, 1.25*ω*(lb/ld)^2/3) = 0.40909418
with fc = 33.00

Calculation of ratio lb/ld

Inadequate Lap Length with lb/ld = 0.30

- Calculation of θp -

From table 10-9: θp = 0.02946994
with:
- Columns not controlled by inadequate development or splicing along the clear height because lb/ld >= 1
shear control ratio VyE/VCol0E = 0.31452971
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 = 4769.35
Ag = 125663.706
f'cE = 33.00
fytE = fylE = 555.56
pl = Area_Tot_Long_Rein/(Ag) = 0.0243
f'cE = 33.00

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

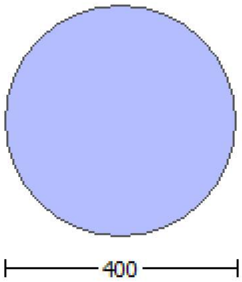
Limit State: Life Safety (data interpolation between analysis steps 1 and 2)

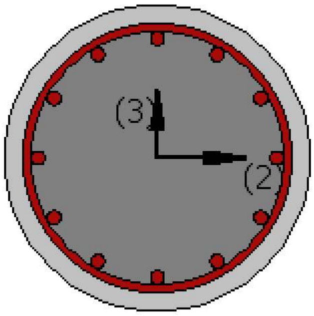
Analysis: Uniform +X

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 = 0.75
Member Shear Force is generally considered as Force-Controlled Action according to Table C7-1, ASCE41-17.
Lower-bound strengths are used for Force-Controlled Actions according to 7.5.1.3, ASCE 41-17
Consequently:
New material of Primary Member: Concrete Strength, fc = fc_lower_bound = 25.00
New material of Primary Member: Steel Strength, fs = fs_lower_bound = 500.00
Concrete Elasticity, Ec = 26999.444
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).
New material: Concrete Strength, fc = fcm = 33.00
New material: Steel Strength, fs = fsm = 555.56
##
Diameter, D = 400.00
Cover Thickness, c = 25.00
Element Length, L = 3000.00
Primary 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 = -1.4130E+007
Shear Force, Va = -4708.281
EDGE -B-
Bending Moment, Mb = 0.00481758
Shear Force, Vb = 4708.281
BOTH EDGES
Axial Force, F = -4769.35
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

New component: From table 7-7, ASCE 41_17: Final Shear Capacity VR = 1.0*Vn = 323560.674
Vn ((10.3), ASCE 41-17) = knl*VCol0 = 323560.674
VCol = 323560.674
knl = 1.00
displacement_ductility_demand = 0.13324578

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' = 25.00, but fc^0.5 <= 8.3 MPa (22.5.3.1, ACI 318-14)
M/Vd = 2.00
Mu = 0.00481758
Vu = 4708.281
d = 0.8*D = 320.00
Nu = 4769.35
Ag = 125663.706
From (11.5.4.8), ACI 318-14: Vs = 197392.088
Av = π/2*A_stirrup = 123370.055
fy = 500.00
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 <= 267132.42
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 = 0.00020821
θy = (My*Ls/3)/Eleff = 0.00156264 ((4.29),Biskinis Phd))
My = 1.5905E+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 = 1.0179E+013
factor = 0.30
Ag = 125663.706
fc' = 33.00
N = 4769.35
Ec*Ig = 3.3929E+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) = 1.5905E+008
Φy = 8.1162642E-006
My_ten (8c) = 1.5905E+008
Φ_ten (7c) = 69.54131
error of function (7c) = 0.0036586
My_com (8d) = 4.3759E+008
Φ_com (7d) = 67.953
error of function (7d) = -0.00152246
with ((10.1), ASCE 41-17) ey = Min(ey, 1.25*ey*(lb/ld)^2/3) = 0.0027778
eco = 0.002
apl = 0.35 ((9a) in Biskinis and Fardis for no FRP Wrap)
d1 = 44.00

R = 200.00
v = 0.0011501
N = 4769.35
Ac = 125663.706
((10.1), ASCE 41-17) ω = Min(ω, 1.25*ω*(lb/ld)^2/3) = 0.40909418
with fc = 33.00

Calculation of ratio lb/ld

Inadequate Lap Length with lb/ld = 0.30

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

Calculation No. 14

column C1, Floor 1

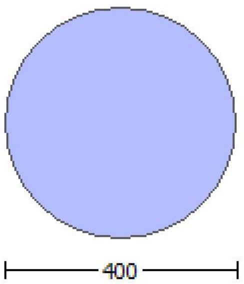
Limit State: Life Safety (data interpolation between analysis steps 1 and 2)

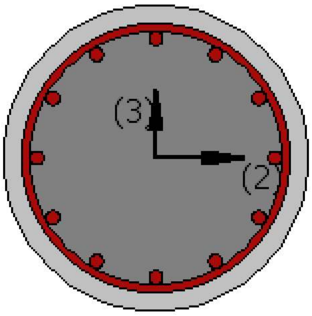
Analysis: Uniform +X

Check: Chord rotation capacity (θu)

Edge: End

Local Axis: (2)





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

Constant Properties

Knowledge Factor, κ = 0.75
Mean strength values are used for both shear and moment calculations.
Consequently:
New material of Primary Member: Concrete Strength, fc = fcm = 33.00
New material of Primary Member: Steel Strength, fs = fsm = 555.56

Concrete Elasticity, Ec = 26999.444
Steel Elasticity, Es = 200000.00
##
Note: Especially for the calculation of moment strengths,
the above steel re-bar strengths are multiplied by 1.25 according to R18.6.5, ACI 318-14
New material: Steel Strength, fs = 1.25*fsm = 694.45
##
Diameter, D = 400.00
Cover Thickness, c = 25.00
Mean Confinement Factor overall section = 1.00
Element Length, L = 3000.00
Primary 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 = 1.0793611E-031
EDGE -B-
Shear Force, Vb = -1.0793611E-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 = 0.31452971
Member Controlled by Flexure (Ve/Vr < 1)
Calculation of Shear Demand from fig. R18.6.5, ACI 318-14 Ve = (Mpr1 + Mpr2)/ln = 114533.572
with
Mpr1 = Max(Mu1+ , Mu1-) = 1.7180E+008
Mu1+ = 1.7180E+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- = 1.7180E+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-) = 1.7180E+008
Mu2+ = 1.7180E+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- = 1.7180E+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 = 1.7180E+008

φ = 0.99483767
φ' = 0.88186679
error of function (3.68), Biskinis Phd = 24343.261
From 5A.2, TBDY: fcc = fc*λc = 33.00

conf. factor λc = 1.00
fc = 33.00
From 10.3.5, ASCE41-17, Final value of fy: fy*Min(1,1.25*(lb/ld)^2/3) = 389.0139
lb/ld = 0.30
d1 = 44.00
R = 200.00
v = 0.0011501
N = 4771.233
Ac = 125663.706
ω = ω*Min(1,1.25*(lb/ld)^2/3) = 0.22916458

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 = 1.7180E+008

φ = 0.99483767
φ' = 0.88186679
error of function (3.68), Biskinis Phd = 24343.261
From 5A.2, TBDY: fcc = fc*λc = 33.00
conf. factor λc = 1.00
fc = 33.00
From 10.3.5, ASCE41-17, Final value of fy: fy*Min(1,1.25*(lb/ld)^2/3) = 389.0139
lb/ld = 0.30
d1 = 44.00
R = 200.00
v = 0.0011501
N = 4771.233
Ac = 125663.706
ω = ω*Min(1,1.25*(lb/ld)^2/3) = 0.22916458

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 = 1.7180E+008

φ = 0.99483767
φ' = 0.88186679
error of function (3.68), Biskinis Phd = 24343.261
From 5A.2, TBDY: fcc = fc*λc = 33.00
conf. factor λc = 1.00
fc = 33.00
From 10.3.5, ASCE41-17, Final value of fy: fy*Min(1,1.25*(lb/ld)^2/3) = 389.0139
lb/ld = 0.30
d1 = 44.00
R = 200.00
v = 0.0011501
N = 4771.233
Ac = 125663.706

ω = ω*Min(1,1.25*(lb/ld)^2/3) = 0.22916458

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 = 1.7180E+008

φ = 0.99483767
φ' = 0.88186679
error of function (3.68), Biskinis Phd = 24343.261
From 5A.2, TBDY: fcc = fc*λc = 33.00
conf. factor λc = 1.00
fc = 33.00
From 10.3.5, ASCE41-17, Final value of fy: fy*Min(1,1.25*(lb/ld)^2/3) = 389.0139
lb/ld = 0.30
d1 = 44.00
R = 200.00
v = 0.0011501
N = 4771.233
Ac = 125663.706
ω = ω*Min(1,1.25*(lb/ld)^2/3) = 0.22916458

Calculation of ratio lb/ld

Inadequate Lap Length with lb/ld = 0.30

Calculation of Shear Strength Vr = Min(Vr1,Vr2) = 364142.298

Calculation of Shear Strength at edge 1, Vr1 = 364142.298
Vr1 = VCol ((10.3), ASCE 41-17) = knl*VCol0
VCol0 = 364142.298
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' = 33.00, but fc^0.5 <= 8.3 MPa (22.5.3.1, ACI 318-14)
M/Vd = 2.00
Mu = 1.7170334E-011
Vu = 1.0793611E-031
d = 0.8*D = 320.00
Nu = 4771.233
Ag = 125663.706
From (11.5.4.8), ACI 318-14: Vs = 219326.297
Av = π/2*A_stirrup = 123370.055
fy = 555.56
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 <= 306911.784
bw*d = π*d*d/4 = 80424.772

Calculation of Shear Strength at edge 2, Vr2 = 364142.298
Vr2 = VCol ((10.3), ASCE 41-17) = knl*VCol0
VCol0 = 364142.298
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' = 33.00, but fc^0.5 <= 8.3 MPa (22.5.3.1, ACI 318-14)
M/Vd = 2.00
Mu = 1.7170334E-011
Vu = 1.0793611E-031
d = 0.8*D = 320.00
Nu = 4771.233
Ag = 125663.706
From (11.5.4.8), ACI 318-14: Vs = 219326.297
Av = π/2*A_stirrup = 123370.055
fy = 555.56
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 <= 306911.784
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 = 0.75
Mean strength values are used for both shear and moment calculations.
Consequently:
New material of Primary Member: Concrete Strength, fc = fcm = 33.00
New material of Primary Member: Steel Strength, fs = fsm = 555.56
Concrete Elasticity, Ec = 26999.444
Steel Elasticity, Es = 200000.00
###
Note: Especially for the calculation of moment strengths,
the above steel re-bar strengths are multiplied by 1.25 according to R18.6.5, ACI 318-14
New material: Steel Strength, fs = 1.25*fsm = 694.45
###
Diameter, D = 400.00
Cover Thickness, c = 25.00
Mean Confinement Factor overall section = 1.00
Element Length, L = 3000.00
Primary 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 = -6.6089624E-048
EDGE -B-
Shear Force, Vb = 6.6089624E-048
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 = 0.31452971
Member Controlled by Flexure (Ve/Vr < 1)
Calculation of Shear Demand from fig. R18.6.5, ACI 318-14 Ve = (Mpr1 + Mpr2)/ln = 114533.572
with
Mpr1 = Max(Mu1+ , Mu1-) = 1.7180E+008
Mu1+ = 1.7180E+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- = 1.7180E+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-) = 1.7180E+008
Mu2+ = 1.7180E+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- = 1.7180E+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 = 1.7180E+008

φ = 0.99483767
φ' = 0.88186679
error of function (3.68), Biskinis Phd = 24343.261
From 5A.2, TBDY: fcc = fc*λc = 33.00
conf. factor λc = 1.00
fc = 33.00
From 10.3.5, ASCE41-17, Final value of fy: fy*Min(1,1.25*(lb/ld)^2/3) = 389.0139
lb/ld = 0.30
d1 = 44.00
R = 200.00
v = 0.0011501
N = 4771.233
Ac = 125663.706
ω = ω*Min(1,1.25*(lb/ld)^2/3) = 0.22916458

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 = 1.7180E+008

φ = 0.99483767
φ' = 0.88186679
error of function (3.68), Biskinis Phd = 24343.261
From 5A.2, TBDY: fcc = fc*λc = 33.00
conf. factor λc = 1.00
fc = 33.00
From 10.3.5, ASCE41-17, Final value of fy: fy*Min(1,1.25*(lb/ld)^2/3) = 389.0139
lb/ld = 0.30
d1 = 44.00
R = 200.00
v = 0.0011501
N = 4771.233
Ac = 125663.706
ω = ω*Min(1,1.25*(lb/ld)^2/3) = 0.22916458

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 = 1.7180E+008

φ = 0.99483767
φ' = 0.88186679
error of function (3.68), Biskinis Phd = 24343.261
From 5A.2, TBDY: fcc = fc*λc = 33.00
conf. factor λc = 1.00
fc = 33.00
From 10.3.5, ASCE41-17, Final value of fy: fy*Min(1,1.25*(lb/ld)^2/3) = 389.0139
lb/ld = 0.30
d1 = 44.00
R = 200.00
v = 0.0011501
N = 4771.233
Ac = 125663.706
ω = ω*Min(1,1.25*(lb/ld)^2/3) = 0.22916458

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 = 1.7180E+008

φ = 0.99483767
φ' = 0.88186679
error of function (3.68), Biskinis Phd = 24343.261
From 5A.2, TBDY: fcc = fc*λc = 33.00
conf. factor λc = 1.00

fc = 33.00
From 10.3.5, ASCE41-17, Final value of fy: fy*Min(1,1.25*(lb/ld)^2/3) = 389.0139
lb/ld = 0.30
d1 = 44.00
R = 200.00
v = 0.0011501
N = 4771.233
Ac = 125663.706
ω = ω*Min(1,1.25*(lb/ld)^2/3) = 0.22916458

Calculation of ratio lb/ld

Inadequate Lap Length with lb/ld = 0.30

Calculation of Shear Strength Vr = Min(Vr1,Vr2) = 364142.298

Calculation of Shear Strength at edge 1, Vr1 = 364142.298
Vr1 = VCol ((10.3), ASCE 41-17) = knl*VCol0
VCol0 = 364142.298
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' = 33.00, but fc^0.5 <= 8.3 MPa (22.5.3.1, ACI 318-14)
M/Vd = 2.00
Mu = 5.2883590E-012
Vu = 6.6089624E-048
d = 0.8*D = 320.00
Nu = 4771.233
Ag = 125663.706
From (11.5.4.8), ACI 318-14: Vs = 219326.297
Av = π/2*A_stirrup = 123370.055
fy = 555.56
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 <= 306911.784
bw*d = π*d*d/4 = 80424.772

Calculation of Shear Strength at edge 2, Vr2 = 364142.298
Vr2 = VCol ((10.3), ASCE 41-17) = knl*VCol0
VCol0 = 364142.298
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' = 33.00, but fc^0.5 <= 8.3 MPa (22.5.3.1, ACI 318-14)
M/Vd = 2.00
Mu = 5.2883590E-012
Vu = 6.6089624E-048
d = 0.8*D = 320.00
Nu = 4771.233
Ag = 125663.706
From (11.5.4.8), ACI 318-14: Vs = 219326.297
Av = π/2*A_stirrup = 123370.055
fy = 555.56
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 <= 306911.784
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 = 0.75
Chord Rotation is generally considered as Deformation-Controlled Action according to Table C7-1, ASCE41-17.
Mean strengths are used for Deformation-Controlled Actions according to 7.5.1.3, ASCE 41-17
Consequently:
New material of Primary Member: Concrete Strength, fc = fcm = 33.00
New material of Primary Member: Steel Strength, fs = fsm = 555.56
Concrete Elasticity, Ec = 26999.444
Steel Elasticity, Es = 200000.00
Diameter, D = 400.00
Cover Thickness, c = 25.00
Element Length, L = 3000.00
Primary 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 = -7.9681523E-011
Shear Force, V2 = 4708.281
Shear Force, V3 = 2.1459671E-013
Axial Force, F = -4769.35
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

New component: From table 7-7, ASCE 41_17: Final chord rotation Capacity θu,R = 1.0*θu = 0.03728312
θu = θy + θp = 0.03728312

- Calculation of θy -

θy = (My*Ls/3)/EIeff = 0.00781318 ((4.29),Biskinis Phd))
My = 1.5905E+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 = 1.0179E+013
factor = 0.30
Ag = 125663.706
fc' = 33.00
N = 4769.35
Ec*Ig = 3.3929E+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) = 1.5905E+008
Φy = 8.1162642E-006
My_ten (8c) = 1.5905E+008
Φ_ten (7c) = 69.54131
error of function (7c) = 0.0036586
My_com (8d) = 4.3759E+008
Φ_com (7d) = 67.953
error of function (7d) = -0.00152246
with ((10.1), ASCE 41-17) ey = Min(ey, 1.25*ey*(lb/ld)^2/3) = 0.0027778
eco = 0.002
apl = 0.35 ((9a) in Biskinis and Fardis for no FRP Wrap)
d1 = 44.00
R = 200.00
v = 0.0011501
N = 4769.35
Ac = 125663.706
((10.1), ASCE 41-17) ω = Min(ω, 1.25*ω*(lb/ld)^2/3) = 0.40909418
with fc = 33.00

Calculation of ratio lb/ld

Inadequate Lap Length with lb/ld = 0.30

- Calculation of θp -

From table 10-9: θp = 0.02946994
with:
- Columns not controlled by inadequate development or splicing along the clear height because lb/ld >= 1
shear control ratio VyE/VCol0E = 0.31452971
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 = 4769.35
Ag = 125663.706
f'cE = 33.00
fytE = fylE = 555.56
pl = Area_Tot_Long_Rein/(Ag) = 0.0243
f'cE = 33.00

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

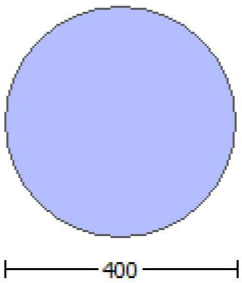
Limit State: Life Safety (data interpolation between analysis steps 1 and 2)

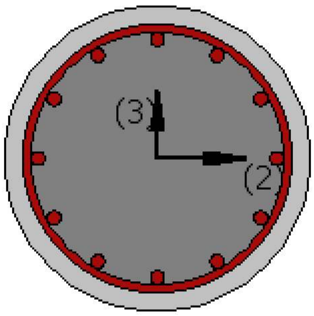
Analysis: Uniform +X

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 = 0.75
Member Shear Force is generally considered as Force-Controlled Action according to Table C7-1, ASCE41-17.
Lower-bound strengths are used for Force-Controlled Actions according to 7.5.1.3, ASCE 41-17
Consequently:
New material of Primary Member: Concrete Strength, fc = fc_lower_bound = 25.00
New material of Primary Member: Steel Strength, fs = fs_lower_bound = 500.00
Concrete Elasticity, Ec = 26999.444
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).
New material: Concrete Strength, fc = fcm = 33.00
New material: Steel Strength, fs = fsm = 555.56
###
Diameter, D = 400.00
Cover Thickness, c = 25.00
Element Length, L = 3000.00
Primary 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 = 7.2381509E-010
Shear Force, Va = -2.1459671E-013
EDGE -B-
Bending Moment, Mb = -7.9681523E-011
Shear Force, Vb = 2.1459671E-013
BOTH EDGES
Axial Force, F = -4769.35
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

New component: From table 7-7, ASCE 41_17: Final Shear Capacity VR = 1.0*Vn = 323560.674
Vn ((10.3), ASCE 41-17) = knl*VCol0 = 323560.674
VCol = 323560.674
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' = 25.00, but fc^0.5 <= 8.3 MPa (22.5.3.1, ACI 318-14)
M/Vd = 2.00
Mu = 7.9681523E-011
Vu = 2.1459671E-013
d = 0.8*D = 320.00
Nu = 4769.35
Ag = 125663.706
From (11.5.4.8), ACI 318-14: Vs = 197392.088
Av = π/2*A_stirrup = 123370.055
fy = 500.00
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 <= 267132.42
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 = 1.2093068E-020
θy = (My*Ls/3)/Eleff = 0.00781318 ((4.29),Biskinis Phd))
My = 1.5905E+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 = 1.0179E+013
factor = 0.30
Ag = 125663.706
fc' = 33.00

N = 4769.35
Ec*Ig = 3.3929E+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) = 1.5905E+008
Φy = 8.1162642E-006
My_ten (8c) = 1.5905E+008
Φ_ten (7c) = 69.54131
error of function (7c) = 0.0036586
My_com (8d) = 4.3759E+008
Φ_com (7d) = 67.953
error of function (7d) = -0.00152246
with ((10.1), ASCE 41-17) ey = Min(ey, 1.25*ey*(lb/ld)^2/3) = 0.0027778
eco = 0.002
apl = 0.35 ((9a) in Biskinis and Fardis for no FRP Wrap)
d1 = 44.00
R = 200.00
v = 0.0011501
N = 4769.35
Ac = 125663.706
((10.1), ASCE 41-17) ω = Min(ω, 1.25*ω*(lb/ld)^2/3) = 0.40909418
with fc = 33.00

Calculation of ratio lb/ld

Inadequate Lap Length with lb/ld = 0.30

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

Calculation No. 16

column C1, Floor 1

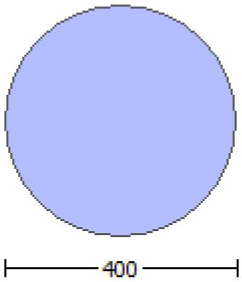
Limit State: Life Safety (data interpolation between analysis steps 1 and 2)

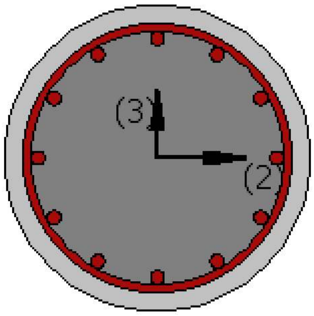
Analysis: Uniform +X

Check: Chord rotation capacity (θu)

Edge: End

Local Axis: (3)





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

Constant Properties

Knowledge Factor, κ = 0.75
Mean strength values are used for both shear and moment calculations.
Consequently:
New material of Primary Member: Concrete Strength, fc = fcm = 33.00
New material of Primary Member: Steel Strength, fs = fsm = 555.56
Concrete Elasticity, Ec = 26999.444
Steel Elasticity, Es = 200000.00
##
Note: Especially for the calculation of moment strengths,
the above steel re-bar strengths are multiplied by 1.25 according to R18.6.5, ACI 318-14
New material: Steel Strength, fs = 1.25*fsm = 694.45
##
Diameter, D = 400.00
Cover Thickness, c = 25.00
Mean Confinement Factor overall section = 1.00
Element Length, L = 3000.00
Primary 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 = 1.0793611E-031
EDGE -B-
Shear Force, Vb = -1.0793611E-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 = 0.31452971
Member Controlled by Flexure (Ve/Vr < 1)
Calculation of Shear Demand from fig. R18.6.5, ACI 318-14 Ve = (Mpr1 + Mpr2)/ln = 114533.572
with
Mpr1 = Max(Mu1+ , Mu1-) = 1.7180E+008
Mu1+ = 1.7180E+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- = 1.7180E+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-) = 1.7180E+008
Mu2+ = 1.7180E+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- = 1.7180E+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 = 1.7180E+008

φ = 0.99483767
φ' = 0.88186679
error of function (3.68), Biskinis Phd = 24343.261
From 5A.2, TBDY: fcc = fc*λc = 33.00
conf. factor λc = 1.00
fc = 33.00
From 10.3.5, ASCE41-17, Final value of fy: fy*Min(1,1.25*(lb/ld)^2/3) = 389.0139
lb/ld = 0.30
d1 = 44.00
R = 200.00
v = 0.0011501
N = 4771.233
Ac = 125663.706
ω = ω*Min(1,1.25*(lb/ld)^2/3) = 0.22916458

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 = 1.7180E+008

φ = 0.99483767
φ' = 0.88186679
error of function (3.68), Biskinis Phd = 24343.261
From 5A.2, TBDY: fcc = fc*λc = 33.00
conf. factor λc = 1.00
fc = 33.00
From 10.3.5, ASCE41-17, Final value of fy: fy*Min(1,1.25*(lb/ld)^2/3) = 389.0139
lb/ld = 0.30
d1 = 44.00
R = 200.00
v = 0.0011501
N = 4771.233
Ac = 125663.706
ω = ω*Min(1,1.25*(lb/ld)^2/3) = 0.22916458

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 = 1.7180E+008

φ = 0.99483767
φ' = 0.88186679
error of function (3.68), Biskinis Phd = 24343.261
From 5A.2, TBDY: fcc = fc*λc = 33.00
conf. factor λc = 1.00
fc = 33.00
From 10.3.5, ASCE41-17, Final value of fy: fy*Min(1,1.25*(lb/ld)^2/3) = 389.0139
lb/ld = 0.30
d1 = 44.00
R = 200.00
v = 0.0011501
N = 4771.233
Ac = 125663.706
ω = ω*Min(1,1.25*(lb/ld)^2/3) = 0.22916458

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 = 1.7180E+008

φ = 0.99483767
φ' = 0.88186679
error of function (3.68), Biskinis Phd = 24343.261
From 5A.2, TBDY: fcc = fc*λc = 33.00
conf. factor λc = 1.00
fc = 33.00
From 10.3.5, ASCE41-17, Final value of fy: fy*Min(1,1.25*(lb/ld)^2/3) = 389.0139
lb/ld = 0.30
d1 = 44.00
R = 200.00
v = 0.0011501
N = 4771.233
Ac = 125663.706
ω = ω*Min(1,1.25*(lb/ld)^2/3) = 0.22916458

Calculation of ratio lb/ld

Inadequate Lap Length with lb/ld = 0.30

Calculation of Shear Strength Vr = Min(Vr1,Vr2) = 364142.298

Calculation of Shear Strength at edge 1, Vr1 = 364142.298
Vr1 = VCol ((10.3), ASCE 41-17) = knl*VCol0
VCol0 = 364142.298
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' = 33.00, but fc^0.5 <= 8.3 MPa (22.5.3.1, ACI 318-14)
M/Vd = 2.00
Mu = 1.7170334E-011
Vu = 1.0793611E-031
d = 0.8*D = 320.00
Nu = 4771.233
Ag = 125663.706
From (11.5.4.8), ACI 318-14: Vs = 219326.297
Av = π/2*A_stirrup = 123370.055
fy = 555.56
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 <= 306911.784
bw*d = π*d*d/4 = 80424.772

Calculation of Shear Strength at edge 2, Vr2 = 364142.298
Vr2 = VCol ((10.3), ASCE 41-17) = knl*VCol0
VCol0 = 364142.298
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' = 33.00, but fc^0.5 <= 8.3 MPa (22.5.3.1, ACI 318-14)
M/Vd = 2.00
Mu = 1.7170334E-011
Vu = 1.0793611E-031
d = 0.8*D = 320.00
Nu = 4771.233
Ag = 125663.706
From (11.5.4.8), ACI 318-14: Vs = 219326.297
Av = π/2*A_stirrup = 123370.055
fy = 555.56
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 <= 306911.784
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 = 0.75
Mean strength values are used for both shear and moment calculations.
Consequently:
New material of Primary Member: Concrete Strength, fc = fcm = 33.00
New material of Primary Member: Steel Strength, fs = fsm = 555.56
Concrete Elasticity, Ec = 26999.444
Steel Elasticity, Es = 200000.00
##
Note: Especially for the calculation of moment strengths,
the above steel re-bar strengths are multiplied by 1.25 according to R18.6.5, ACI 318-14
New material: Steel Strength, fs = 1.25*fsm = 694.45
##
Diameter, D = 400.00
Cover Thickness, c = 25.00
Mean Confinement Factor overall section = 1.00
Element Length, L = 3000.00
Primary 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 = -6.6089624E-048
EDGE -B-
Shear Force, Vb = 6.6089624E-048
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 = 0.31452971
Member Controlled by Flexure (Ve/Vr < 1)
Calculation of Shear Demand from fig. R18.6.5, ACI 318-14 Ve = (Mpr1 + Mpr2)/ln = 114533.572
with
Mpr1 = Max(Mu1+ , Mu1-) = 1.7180E+008
Mu1+ = 1.7180E+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- = 1.7180E+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-) = 1.7180E+008
Mu2+ = 1.7180E+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- = 1.7180E+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 = 1.7180E+008

φ = 0.99483767
φ' = 0.88186679
error of function (3.68), Biskinis Phd = 24343.261
From 5A.2, TBDY: fcc = fc*λc = 33.00
conf. factor λc = 1.00
fc = 33.00
From 10.3.5, ASCE41-17, Final value of fy: fy*Min(1,1.25*(lb/ld)^2/3) = 389.0139
lb/ld = 0.30
d1 = 44.00
R = 200.00
v = 0.0011501
N = 4771.233
Ac = 125663.706
ω = ω*Min(1,1.25*(lb/ld)^2/3) = 0.22916458

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 = 1.7180E+008

φ = 0.99483767
φ' = 0.88186679
error of function (3.68), Biskinis Phd = 24343.261
From 5A.2, TBDY: fcc = fc*λc = 33.00
conf. factor λc = 1.00
fc = 33.00
From 10.3.5, ASCE41-17, Final value of fy: fy*Min(1,1.25*(lb/ld)^2/3) = 389.0139
lb/ld = 0.30
d1 = 44.00
R = 200.00
v = 0.0011501
N = 4771.233
Ac = 125663.706
ω = ω*Min(1,1.25*(lb/ld)^2/3) = 0.22916458

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 = 1.7180E+008

φ = 0.99483767
φ' = 0.88186679
error of function (3.68), Biskinis Phd = 24343.261
From 5A.2, TBDY: fcc = fc*λc = 33.00
conf. factor λc = 1.00
fc = 33.00
From 10.3.5, ASCE41-17, Final value of fy: fy*Min(1,1.25*(lb/ld)^2/3) = 389.0139

lb/ld = 0.30
d1 = 44.00
R = 200.00
v = 0.0011501
N = 4771.233
Ac = 125663.706
ω = ω*Min(1,1.25*(lb/ld)^2/3) = 0.22916458

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 = 1.7180E+008

φ = 0.99483767
φ' = 0.88186679
error of function (3.68), Biskinis Phd = 24343.261
From 5A.2, TBDY: fcc = fc*λc = 33.00
conf. factor λc = 1.00
fc = 33.00
From 10.3.5, ASCE41-17, Final value of fy: fy*Min(1,1.25*(lb/ld)^2/3) = 389.0139
lb/ld = 0.30
d1 = 44.00
R = 200.00
v = 0.0011501
N = 4771.233
Ac = 125663.706
ω = ω*Min(1,1.25*(lb/ld)^2/3) = 0.22916458

Calculation of ratio lb/ld

Inadequate Lap Length with lb/ld = 0.30

Calculation of Shear Strength Vr = Min(Vr1,Vr2) = 364142.298

Calculation of Shear Strength at edge 1, Vr1 = 364142.298
Vr1 = VCol ((10.3), ASCE 41-17) = knl*VCol0
VCol0 = 364142.298
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' = 33.00, but fc^0.5 <= 8.3 MPa (22.5.3.1, ACI 318-14)
M/Vd = 2.00
Mu = 5.2883590E-012
Vu = 6.6089624E-048
d = 0.8*D = 320.00
Nu = 4771.233
Ag = 125663.706
From (11.5.4.8), ACI 318-14: Vs = 219326.297
Av = π/2*A_stirrup = 123370.055
fy = 555.56
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 <= 306911.784
bw*d = π*d*d/4 = 80424.772

Calculation of Shear Strength at edge 2, Vr2 = 364142.298
Vr2 = VCol ((10.3), ASCE 41-17) = knl*VCol0
VCol0 = 364142.298
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' = 33.00, but fc^0.5 <= 8.3 MPa (22.5.3.1, ACI 318-14)
M/Vd = 2.00
Mu = 5.2883590E-012
Vu = 6.6089624E-048
d = 0.8*D = 320.00
Nu = 4771.233
Ag = 125663.706
From (11.5.4.8), ACI 318-14: Vs = 219326.297
Av = π/2*A_stirrup = 123370.055
fy = 555.56
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 <= 306911.784
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 = 0.75
Chord Rotation is generally considered as Deformation-Controlled Action according to Table C7-1, ASCE41-17.
Mean strengths are used for Deformation-Controlled Actions according to 7.5.1.3, ASCE 41-17
Consequently:
New material of Primary Member: Concrete Strength, fc = fcm = 33.00
New material of Primary Member: Steel Strength, fs = fsm = 555.56
Concrete Elasticity, Ec = 26999.444
Steel Elasticity, Es = 200000.00
Diameter, D = 400.00
Cover Thickness, c = 25.00
Element Length, L = 3000.00
Primary Member
Ribbed Bars
Ductile Steel
Without Detailing for Earthquake Resistance (including stirrups not closed at 135°)
Longitudinal Bars With Ends Lapped Starting at the End Sections
Inadequate Lap Length with lb/ld = 0.30
No FRP Wrapping

Stepwise Properties

Bending Moment, M = 0.00481758
Shear Force, V2 = 4708.281
Shear Force, V3 = 2.1459671E-013
Axial Force, F = -4769.35
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

New component: From table 7-7, ASCE 41_17: Final chord rotation Capacity θu,R = 1.0*θu = 0.03103257
θu = θy + θp = 0.03103257

- Calculation of θy -

θy = (My*Ls/3)/EIeff = 0.00156264 ((4.29),Biskinis Phd))
My = 1.5905E+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 = 1.0179E+013
factor = 0.30
Ag = 125663.706
fc' = 33.00
N = 4769.35
Ec*Ig = 3.3929E+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) = 1.5905E+008
Φy = 8.1162642E-006
My_ten (8c) = 1.5905E+008
Φ_ten (7c) = 69.54131
error of function (7c) = 0.0036586
My_com (8d) = 4.3759E+008
Φ_com (7d) = 67.953
error of function (7d) = -0.00152246
with ((10.1), ASCE 41-17) ey = Min(ey, 1.25*ey*(lb/ld)^2/3) = 0.0027778
eco = 0.002
apl = 0.35 ((9a) in Biskinis and Fardis for no FRP Wrap)
d1 = 44.00
R = 200.00
v = 0.0011501
N = 4769.35
Ac = 125663.706
((10.1), ASCE 41-17) ω = Min(ω, 1.25*ω*(lb/ld)^2/3) = 0.40909418
with fc = 33.00

Calculation of ratio lb/ld

Inadequate Lap Length with lb/ld = 0.30

- Calculation of θp -

From table 10-9: θp = 0.02946994
with:
- Columns not controlled by inadequate development or splicing along the clear height because lb/ld >= 1
  shear control ratio VyE/VCol0E = 0.31452971
  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 = 4769.35
    Ag = 125663.706
    f'cE = 33.00
    fytE = fylE = 555.56
pl = Area_Tot_Long_Rein/(Ag) = 0.0243
    f'cE = 33.00

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