Detailed Member Calculations

Units: N&mm

Regulation: ASCE 41-17

Calculation No. 1

column C1, Floor 1

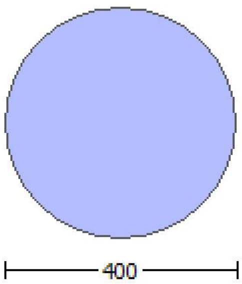
Limit State: Operational Level (data interpolation between analysis steps 1 and 2)

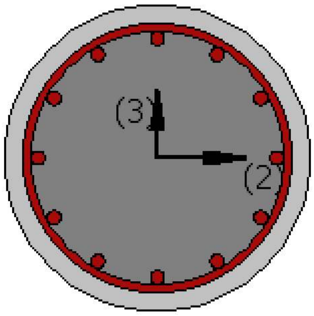
Analysis: Uniform +X

Check: Shear capacity VRd

Edge: Start

Local Axis: (2)





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

Constant Properties

Knowledge Factor, κ = 1.00
Member Shear Force is generally considered as Force-Controlled Action according to Table C7-1, ASCE41-17.
Lower-bound strengths are used for Force-Controlled Actions according to 7.5.1.3, ASCE 41-17
Consequently:
Existing material of Primary Member: Concrete Strength, fc = fc_lower_bound = 16.00
Existing material of Primary Member: Steel Strength, fs = fs_lower_bound = 400.00
Concrete Elasticity, Ec = 21019.039

Steel Elasticity, Es = 200000.00
##
Note: Especially for the calculation of θy for displacement ductility demand,
the expected (mean value) strengths are used (7.5.1.3, ASCE41-17) because bending is considered as Deformation-Controlled Action (Table C7-1, ASCE41-17).
Existing material: Concrete Strength, fc = fcm = 20.00
Existing material: Steel Strength, fs = fsm = 444.4444
##
Diameter, D = 400.00
Cover Thickness, c = 25.00
Element Length, L = 3000.00
Primary Member
Ribbed Bars
Ductile Steel
With Detailing for Earthquake Resistance (including stirrups closed at 135°)
Longitudinal Bars With Ends Lapped Starting at the End Sections
Adequate Lap Length (lo/lou,min = lb/ld >=1)
No FRP Wrapping

Stepwise Properties

EDGE -A-
Bending Moment, Ma = -8.3435E+006
Shear Force, Va = -2779.824
EDGE -B-
Bending Moment, Mb = 0.04084082
Shear Force, Vb = 2779.824
BOTH EDGES
Axial Force, F = -4770.12
Longitudinal Reinforcement Area Distribution (in 2 divisions)
-Tension: Aslt = 1272.345
-Compression: Aslc = 1781.283
Longitudinal Reinforcement Area Distribution (in 3 divisions)
-Tension: Asl,ten = 1017.876
-Compression: Asl,com = 1017.876
-Middle: Asl,mid = 1017.876
Mean Diameter of Tension Reinforcement, DbL,ten = 18.00

Existing component: From table 7-7, ASCE 41_17: Final Shear Capacity VR = κ*Vn = 208475.761
Vn ((10.3), ASCE 41-17) = knl*VCol0 = 208475.761
VCol = 208475.761
knl = 1.00
displacement_ductility_demand = 0.0095812

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

λ = 1 (normal-weight concrete)
fc' = 16.00, but fc^0.5 <= 8.3 MPa (22.5.3.1, ACI 318-14)
M/Vd = 4.00
Mu = 8.3435E+006
Vu = 2779.824
d = 0.8*D = 320.00
Nu = 4770.12
Ag = 125663.706
From (11.5.4.8), ACI 318-14: Vs = 157913.67
Av = π/2*A_stirrup = 123370.055
fy = 400.00
s = 100.00
Vs is multiplied by αCol = 0.00
s/d = 0.3125
Vf ((11-3)-(11.4), ACI 440) = 0.00
From (11-11), ACI 440: Vs + Vf <= 213705.936

bw*d = π*d*d/4 = 80424.772

----
displacement_ductility_demand is calculated as θ/θy

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

From analysis, chord rotation θ = 0.00028196
θy = (My*Ls/3)/EIeff = 0.02942871 ((4.29),Biskinis Phd))
My = 2.3308E+008
Ls = M/V (with Ls >0.1*L and Ls < 2*L) = 3001.447
From table 10.5, ASCE 41_17: EIeff = factor*Ec*Ig = 7.9240E+012
factor = 0.30
Ag = 125663.706
fc' = 20.00
N = 4770.12
Ec*Ig = 2.6413E+013

Calculation of Yielding Moment My

Calculation of Φy and My according to (7) - (8) in Biskinis and Fardis
----
My = Min(My_ten,My_com) = 2.3308E+008
Φy ((10a) or (10b)) = 1.3315994E-005
My_ten (8a) = 2.3308E+008
Φ_ten (7a) = 78.48339
error of function (7a) = 0.00010055
My_com (8b) = 3.4649E+008
Φ_com (7b) = 70.96936
error of function (7b) = -0.00051806
with ey = 0.00222222
eco = 0.002
apl = 0.35 ((9a) in Biskinis and Fardis for no FRP Wrap)
d1 = 44.00
R = 200.00
v = 0.00189797
N = 4770.12
Ac = 125663.706
ω = 0.54
with fc = 20.00

Calculation of ratio lb/ld

Adequate Lap Length: lb/ld >= 1

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

Calculation No. 2

column C1, Floor 1

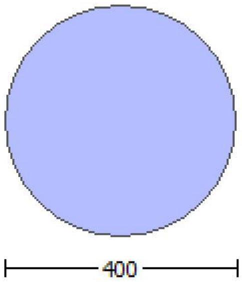
Limit State: Operational Level (data interpolation between analysis steps 1 and 2)

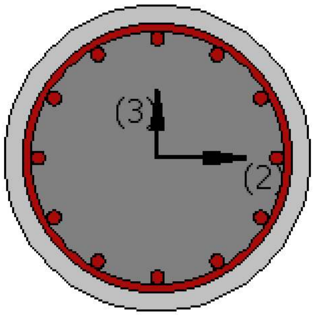
Analysis: Uniform +X

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 = 1.00
Mean strength values are used for both shear and moment calculations.
Consequently:
Existing material of Primary Member: Concrete Strength, fc = fcm = 20.00
Existing material of Primary Member: Steel Strength, fs = fsm = 444.4444
Concrete Elasticity, Ec = 21019.039
Steel Elasticity, Es = 200000.00
##
Note: Especially for the calculation of moment strengths,
the above steel re-bar strengths are multiplied by 1.25 according to R18.6.5, ACI 318-14
Existing material: Steel Strength, fs = 1.25*fsm = 555.5556
##
Diameter, D = 400.00
Cover Thickness, c = 25.00
Mean Confinement Factor overall section = 1.45937
Element Length, L = 3000.00
Primary Member
Ribbed Bars
Ductile Steel
With Detailing for Earthquake Resistance (including stirrups closed at 135°)
Longitudinal Bars With Ends Lapped Starting at the End Sections
Adequate Lap Length (lo/lou,min>=1)
No FRP Wrapping

Stepwise Properties

At local axis: 3
EDGE -A-
Shear Force, Va = -6.3906440E-031
EDGE -B-
Shear Force, Vb = 6.3906440E-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 = 0.53837083
Member Controlled by Flexure (Ve/Vr < 1)
Calculation of Shear Demand from fig. R18.6.5, ACI 318-14 Ve = (Mpr1 + Mpr2)/ln = 155270.734
with
Mpr1 = Max(Mu1+ , Mu1-) = 2.3291E+008
Mu1+ = 2.3291E+008, is the ultimate moment strength at the edge 1 of the member in the actual moment direction which is defined for the static loading combination
Mu1- = 2.3291E+008, is the ultimate moment strength at the edge 1 of the member in the opposite moment direction which is defined for the static loading combination
Mpr2 = Max(Mu2+ , Mu2-) = 2.3291E+008
Mu2+ = 2.3291E+008, is the ultimate moment strength at the edge 2 of the member in the actual moment direction which is defined for the the static loading combination
Mu2- = 2.3291E+008, is the ultimate moment strength at the edge 2 of the member in the opposite moment direction which is defined for the the static loading combination

----
Calculation of Mu1+

Calculation of ultimate Moment Strength ((3.67), Biskinis Phd), Mu
Mu = 2.3291E+008

φ = 1.11701
φ' = 0.98759739
error of function (3.68), Biskinis Phd = 59859.019
From 5A.2, TBDY: fcc = fc*λc = 29.18743
conf. factor λc = 1.45937
fc = 20.00
From 10.3.5, ASCE41-17, Final value of fy: fy*Min(1,1.25*(lb/ld)^2/3) = 555.5556
lb/ld = 1.00
d1 = 44.00
R = 200.00
v = 0.00189786
N = 4771.233
Ac = 125663.706
ω = ω*Min(1,1.25*(lb/ld)^2/3) = 0.54

Calculation of ratio lb/ld

Adequate Lap Length: lb/ld >= 1

Calculation of Mu1-

Calculation of ultimate Moment Strength ((3.67), Biskinis Phd), Mu
Mu = 2.3291E+008

φ = 1.11701
φ' = 0.98759739

error of function (3.68), Biskinis Phd = 59859.019
From 5A.2, TBDY: fcc = fc*λc = 29.18743
conf. factor λc = 1.45937
fc = 20.00
From 10.3.5, ASCE41-17, Final value of fy: fy*Min(1,1.25*(lb/ld)^2/3) = 555.5556
lb/ld = 1.00
d1 = 44.00
R = 200.00
v = 0.00189786
N = 4771.233
Ac = 125663.706
ω = ω*Min(1,1.25*(lb/ld)^2/3) = 0.54

Calculation of ratio lb/ld

Adequate Lap Length: lb/ld >= 1

Calculation of Mu2+

Calculation of ultimate Moment Strength ((3.67), Biskinis Phd), Mu
Mu = 2.3291E+008

φ = 1.11701
φ' = 0.98759739
error of function (3.68), Biskinis Phd = 59859.019
From 5A.2, TBDY: fcc = fc*λc = 29.18743
conf. factor λc = 1.45937
fc = 20.00
From 10.3.5, ASCE41-17, Final value of fy: fy*Min(1,1.25*(lb/ld)^2/3) = 555.5556
lb/ld = 1.00
d1 = 44.00
R = 200.00
v = 0.00189786
N = 4771.233
Ac = 125663.706
ω = ω*Min(1,1.25*(lb/ld)^2/3) = 0.54

Calculation of ratio lb/ld

Adequate Lap Length: lb/ld >= 1

Calculation of Mu2-

Calculation of ultimate Moment Strength ((3.67), Biskinis Phd), Mu
Mu = 2.3291E+008

φ = 1.11701
φ' = 0.98759739
error of function (3.68), Biskinis Phd = 59859.019
From 5A.2, TBDY: fcc = fc*λc = 29.18743
conf. factor λc = 1.45937
fc = 20.00
From 10.3.5, ASCE41-17, Final value of fy: fy*Min(1,1.25*(lb/ld)^2/3) = 555.5556
lb/ld = 1.00
d1 = 44.00
R = 200.00
v = 0.00189786

N = 4771.233
Ac = 125663.706
ω = ω*Min(1,1.25*(lb/ld)^2/3) = 0.54

Calculation of ratio lb/ld

Adequate Lap Length: lb/ld >= 1

Calculation of Shear Strength Vr = Min(Vr1,Vr2) = 288408.521

Calculation of Shear Strength at edge 1, Vr1 = 288408.521
Vr1 = VCol ((10.3), ASCE 41-17) = knl*VCol0
VCol0 = 288408.521
knl = 1 (zero step-static loading)

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

λ = 1 (normal-weight concrete)
fc' = 20.00, but fc^0.5 <= 8.3 MPa (22.5.3.1, ACI 318-14)
M/Vd = 2.00
Mu = 1.4857213E-011
Vu = 6.3906440E-031
d = 0.8*D = 320.00
Nu = 4771.233
Ag = 125663.706
From (11.5.4.8), ACI 318-14: Vs = 175459.634
Av = π/2*A_stirrup = 123370.055
fy = 444.4444
s = 100.00
Vs is multiplied by αCol = 0.00
s/d = 0.3125
Vf ((11-3)-(11.4), ACI 440) = 0.00
From (11-11), ACI 440: Vs + Vf <= 238930.50
bw*d = π*d*d/4 = 80424.772

Calculation of Shear Strength at edge 2, Vr2 = 288408.521
Vr2 = VCol ((10.3), ASCE 41-17) = knl*VCol0
VCol0 = 288408.521
knl = 1 (zero step-static loading)

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

λ = 1 (normal-weight concrete)
fc' = 20.00, but fc^0.5 <= 8.3 MPa (22.5.3.1, ACI 318-14)
M/Vd = 2.00
Mu = 1.4857213E-011
Vu = 6.3906440E-031
d = 0.8*D = 320.00
Nu = 4771.233
Ag = 125663.706
From (11.5.4.8), ACI 318-14: Vs = 175459.634
Av = π/2*A_stirrup = 123370.055
fy = 444.4444
s = 100.00
Vs is multiplied by αCol = 0.00
s/d = 0.3125
Vf ((11-3)-(11.4), ACI 440) = 0.00
From (11-11), ACI 440: Vs + Vf <= 238930.50
bw*d = π*d*d/4 = 80424.772

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

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

Constant Properties

Knowledge Factor, κ = 1.00
Mean strength values are used for both shear and moment calculations.
Consequently:
Existing material of Primary Member: Concrete Strength, fc = fcm = 20.00
Existing material of Primary Member: Steel Strength, fs = fsm = 444.4444
Concrete Elasticity, Ec = 21019.039
Steel Elasticity, Es = 200000.00
###
Note: Especially for the calculation of moment strengths,
the above steel re-bar strengths are multiplied by 1.25 according to R18.6.5, ACI 318-14
Existing material: Steel Strength, fs = 1.25*fsm = 555.5556
###
Diameter, D = 400.00
Cover Thickness, c = 25.00
Mean Confinement Factor overall section = 1.45937
Element Length, L = 3000.00
Primary Member
Ribbed Bars
Ductile Steel
With Detailing for Earthquake Resistance (including stirrups closed at 135°)
Longitudinal Bars With Ends Lapped Starting at the End Sections
Adequate Lap Length (lo/lou,min>=1)
No FRP Wrapping

Stepwise Properties

At local axis: 2
EDGE -A-
Shear Force, Va = 3.9130116E-047
EDGE -B-
Shear Force, Vb = -3.9130116E-047
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 = 0.53837083
Member Controlled by Flexure (Ve/Vr < 1)
Calculation of Shear Demand from fig. R18.6.5, ACI 318-14 Ve = (Mpr1 + Mpr2)/ln = 155270.734
with
Mpr1 = Max(Mu1+ , Mu1-) = 2.3291E+008
Mu1+ = 2.3291E+008, is the ultimate moment strength at the edge 1 of the member in the actual moment direction which is defined for the static loading combination
Mu1- = 2.3291E+008, is the ultimate moment strength at the edge 1 of the member in the opposite moment

direction which is defined for the static loading combination
Mpr2 = Max(Mu2+ , Mu2-) = 2.3291E+008
Mu2+ = 2.3291E+008, is the ultimate moment strength at the edge 2 of the member in the actual moment direction which is defined for the the static loading combination
Mu2- = 2.3291E+008, is the ultimate moment strength at the edge 2 of the member in the opposite moment direction which is defined for the the static loading combination

----
Calculation of Mu1+

Calculation of ultimate Moment Strength ((3.67), Biskinis Phd), Mu
Mu = 2.3291E+008

φ = 1.11701
φ' = 0.98759739
error of function (3.68), Biskinis Phd = 59859.019
From 5A.2, TBDY: fcc = fc*λc = 29.18743
conf. factor λc = 1.45937
fc = 20.00
From 10.3.5, ASCE41-17, Final value of fy: fy*Min(1,1.25*(lb/ld)^2/3) = 555.5556
lb/ld = 1.00
d1 = 44.00
R = 200.00
v = 0.00189786
N = 4771.233
Ac = 125663.706
ω = ω*Min(1,1.25*(lb/ld)^2/3) = 0.54

Calculation of ratio lb/ld

Adequate Lap Length: lb/ld >= 1

Calculation of Mu1-

Calculation of ultimate Moment Strength ((3.67), Biskinis Phd), Mu
Mu = 2.3291E+008

φ = 1.11701
φ' = 0.98759739
error of function (3.68), Biskinis Phd = 59859.019
From 5A.2, TBDY: fcc = fc*λc = 29.18743
conf. factor λc = 1.45937
fc = 20.00
From 10.3.5, ASCE41-17, Final value of fy: fy*Min(1,1.25*(lb/ld)^2/3) = 555.5556
lb/ld = 1.00
d1 = 44.00
R = 200.00
v = 0.00189786
N = 4771.233
Ac = 125663.706
ω = ω*Min(1,1.25*(lb/ld)^2/3) = 0.54

Calculation of ratio lb/ld

Adequate Lap Length: lb/ld >= 1

Calculation of Mu2+

Calculation of ultimate Moment Strength ((3.67), Biskinis Phd), Mu
Mu = 2.3291E+008

φ = 1.11701
φ' = 0.98759739
error of function (3.68), Biskinis Phd = 59859.019
From 5A.2, TBDY: fcc = fc*λc = 29.18743
 conf. factor λc = 1.45937
 fc = 20.00
From 10.3.5, ASCE41-17, Final value of fy: fy*Min(1,1.25*(lb/ld)^2/3) = 555.5556
  lb/ld = 1.00
  d1 = 44.00
  R = 200.00
  v = 0.00189786
   N = 4771.233
   Ac = 125663.706
  ω = ω*Min(1,1.25*(lb/ld)^2/3) = 0.54

Calculation of ratio lb/ld

Adequate Lap Length: lb/ld >= 1

Calculation of Mu2-

Calculation of ultimate Moment Strength ((3.67), Biskinis Phd), Mu
Mu = 2.3291E+008

φ = 1.11701
φ' = 0.98759739
error of function (3.68), Biskinis Phd = 59859.019
From 5A.2, TBDY: fcc = fc*λc = 29.18743
 conf. factor λc = 1.45937
 fc = 20.00
From 10.3.5, ASCE41-17, Final value of fy: fy*Min(1,1.25*(lb/ld)^2/3) = 555.5556
  lb/ld = 1.00
  d1 = 44.00
  R = 200.00
  v = 0.00189786
   N = 4771.233
   Ac = 125663.706
  ω = ω*Min(1,1.25*(lb/ld)^2/3) = 0.54

Calculation of ratio lb/ld

Adequate Lap Length: lb/ld >= 1

Calculation of Shear Strength Vr = Min(Vr1,Vr2) = 288408.521

Calculation of Shear Strength at edge 1, Vr1 = 288408.521
Vr1 = VCol ((10.3), ASCE 41-17) = knl*VCol0
 VCol0 = 288408.521
 knl = 1 (zero step-static loading)

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

λ = 1 (normal-weight concrete)
fc' = 20.00, but fc^0.5 <= 8.3 MPa (22.5.3.1, ACI 318-14)
M/Vd = 2.00
Mu = 7.3393178E-012
Vu = 3.9130116E-047
d = 0.8*D = 320.00
Nu = 4771.233
Ag = 125663.706
From (11.5.4.8), ACI 318-14: Vs = 175459.634
Av = π/2*A_stirrup = 123370.055
fy = 444.4444
s = 100.00
Vs is multiplied by αCol = 0.00
s/d = 0.3125
Vf ((11-3)-(11.4), ACI 440) = 0.00
From (11-11), ACI 440: Vs + Vf <= 238930.50
bw*d = π*d*d/4 = 80424.772

Calculation of Shear Strength at edge 2, Vr2 = 288408.521
Vr2 = VCol ((10.3), ASCE 41-17) = knl*VCol0
VCol0 = 288408.521
knl = 1 (zero step-static loading)

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

λ = 1 (normal-weight concrete)
fc' = 20.00, but fc^0.5 <= 8.3 MPa (22.5.3.1, ACI 318-14)
M/Vd = 2.00
Mu = 7.3393178E-012
Vu = 3.9130116E-047
d = 0.8*D = 320.00
Nu = 4771.233
Ag = 125663.706
From (11.5.4.8), ACI 318-14: Vs = 175459.634
Av = π/2*A_stirrup = 123370.055
fy = 444.4444
s = 100.00
Vs is multiplied by αCol = 0.00
s/d = 0.3125
Vf ((11-3)-(11.4), ACI 440) = 0.00
From (11-11), ACI 440: Vs + Vf <= 238930.50
bw*d = π*d*d/4 = 80424.772

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

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

Constant Properties

Knowledge Factor, κ = 1.00
Chord Rotation is generally considered as Deformation-Controlled Action according to Table C7-1, ASCE41-17.
Mean strengths are used for Deformation-Controlled Actions according to 7.5.1.3, ASCE 41-17
Consequently:
Existing material of Primary Member: Concrete Strength, fc = fcm = 20.00
Existing material of Primary Member: Steel Strength, fs = fsm = 444.4444
Concrete Elasticity, Ec = 21019.039

Steel Elasticity, Es = 200000.00
Diameter, D = 400.00
Cover Thickness, c = 25.00
Element Length, L = 3000.00
Primary Member
Ribbed Bars
Ductile Steel
With Detailing for Earthquake Resistance (including stirrups closed at 135°)
Longitudinal Bars With Ends Lapped Starting at the End Sections
Adequate Lap Length (lb/ld>=1)
No FRP Wrapping

Stepwise Properties

Bending Moment, M = 3.8560899E-010
Shear Force, V2 = -2779.824
Shear Force, V3 = -1.0292144E-013
Axial Force, F = -4770.12
Longitudinal Reinforcement Area Distribution (in 2 divisions)
-Tension: Aslt = 1272.345
-Compression: Aslc = 1781.283
Longitudinal Reinforcement Area Distribution (in 3 divisions)
-Tension: Asl,ten = 1017.876
-Compression: Asl,com = 1017.876
-Middle: Asl,mid = 1017.876
Mean Diameter of Tension Reinforcement, DbL = 18.00

Existing component: From table 7-7, ASCE 41_17: Final chord rotation Capacity θu,R = κ*θu = 0.01970726
θu = θy + θp = 0.01970726

- Calculation of θy -

θy = (My*Ls/3)/EIeff = 0.01470726 ((4.29),Biskinis Phd))
My = 2.3308E+008
Ls = M/V (with Ls >0.1*L and Ls < 2*L) = 1500.00
From table 10.5, ASCE 41_17: EIeff = factor*Ec*Ig = 7.9240E+012
factor = 0.30
Ag = 125663.706
fc' = 20.00
N = 4770.12
Ec*Ig = 2.6413E+013

Calculation of Yielding Moment My

Calculation of Φy and My according to (7) - (8) in Biskinis and Fardis
----
My = Min(My_ten,My_com) = 2.3308E+008
Φy ((10a) or (10b)) = 1.3315994E-005
My_ten (8a) = 2.3308E+008
Φ_ten (7a) = 78.48339
error of function (7a) = 0.00010055
My_com (8b) = 3.4649E+008
Φ_com (7b) = 70.96936
error of function (7b) = -0.00051806
with ey = 0.00222222
eco = 0.002
apl = 0.35 ((9a) in Biskinis and Fardis for no FRP Wrap)
d1 = 44.00
R = 200.00
v = 0.00189797

N = 4770.12
Ac = 125663.706
ω = 0.54
with fc = 20.00

Calculation of ratio lb/ld

Adequate Lap Length: lb/ld >= 1

- Calculation of θp -

From table 10-9: θp = 0.005
with:
- Columns controlled by inadequate development or splicing along the clear height because lb/ld < 1
shear control ratio VyE/VCol0E = 0.53837083
d = 707.00
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 = 4770.12
Ag = 125663.706
f'cE = 20.00
fytE = fylE = 444.4444
pl = Area_Tot_Long_Rein/(Ag) = 0.0243
f'cE = 20.00

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

Calculation No. 3

column C1, Floor 1

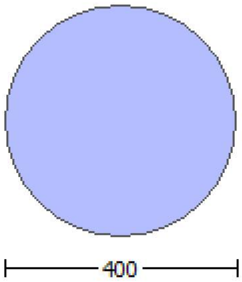
Limit State: Operational Level (data interpolation between analysis steps 1 and 2)

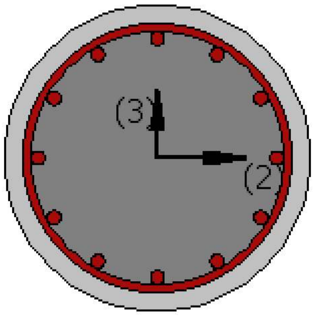
Analysis: Uniform +X

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 = 1.00
Member Shear Force is generally considered as Force-Controlled Action according to Table C7-1, ASCE41-17.
Lower-bound strengths are used for Force-Controlled Actions according to 7.5.1.3, ASCE 41-17
Consequently:
Existing material of Primary Member: Concrete Strength, fc = fc_lower_bound = 16.00
Existing material of Primary Member: Steel Strength, fs = fs_lower_bound = 400.00
Concrete Elasticity, Ec = 21019.039
Steel Elasticity, Es = 200000.00
###
Note: Especially for the calculation of θy for displacement ductility demand,
the expected (mean value) strengths are used (7.5.1.3, ASCE41-17) because bending is considered as
Deformation-Controlled Action (Table C7-1, ASCE41-17).
Existing material: Concrete Strength, fc = fcm = 20.00
Existing material: Steel Strength, fs = fsm = 444.4444
###
Diameter, D = 400.00
Cover Thickness, c = 25.00
Element Length, L = 3000.00
Primary Member
Ribbed Bars
Ductile Steel
With Detailing for Earthquake Resistance (including stirrups closed at 135°)
Longitudinal Bars With Ends Lapped Starting at the End Sections
Adequate Lap Length (lo/lou,min = lb/ld >=1)
No FRP Wrapping

Stepwise Properties

EDGE -A-
Bending Moment, Ma = 3.8560899E-010
Shear Force, Va = -1.0292144E-013
EDGE -B-
Bending Moment, Mb = -7.6585865E-011
Shear Force, Vb = 1.0292144E-013
BOTH EDGES
Axial Force, F = -4770.12
Longitudinal Reinforcement Area Distribution (in 2 divisions)
-Tension: Aslt = 1272.345
-Compression: Aslc = 1781.283
Longitudinal Reinforcement Area Distribution (in 3 divisions)
-Tension: Asl,ten = 1017.876
-Compression: Asl,com = 1017.876
-Middle: Asl,mid = 1017.876
Mean Diameter of Tension Reinforcement, DbL,ten = 18.00

Existing component: From table 7-7, ASCE 41_17: Final Shear Capacity VR = κ*Vn = 259037.852
Vn ((10.3), ASCE 41-17) = knl*VCol0 = 259037.852
VCol = 259037.852
knl = 1.00
displacement_ductility_demand = 0.00

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

λ = 1 (normal-weight concrete)
fc' = 16.00, but fc^0.5 <= 8.3 MPa (22.5.3.1, ACI 318-14)
M/Vd = 2.00
Mu = 3.8560899E-010
Vu = 1.0292144E-013
d = 0.8*D = 320.00
Nu = 4770.12
Ag = 125663.706
From (11.5.4.8), ACI 318-14: Vs = 157913.67
Av = π/2*A_stirrup = 123370.055
fy = 400.00
s = 100.00
Vs is multiplied by αCol = 0.00
s/d = 0.3125
Vf ((11-3)-(11.4), ACI 440) = 0.00
From (11-11), ACI 440: Vs + Vf <= 213705.936
bw*d = π*d*d/4 = 80424.772

----
displacement_ductility_demand is calculated as θ/θy

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

From analysis, chord rotation θ = 1.8120356E-020
θy = (My*Ls/3)/Eleff = 0.01470726 ((4.29),Biskinis Phd))
My = 2.3308E+008
Ls = M/V (with Ls >0.1*L and Ls < 2*L) = 1500.00
From table 10.5, ASCE 41_17: Eleff = factor*Ec*Ig = 7.9240E+012
factor = 0.30
Ag = 125663.706
fc' = 20.00
N = 4770.12
Ec*Ig = 2.6413E+013

Calculation of Yielding Moment My

Calculation of Φy and My according to (7) - (8) in Biskinis and Fardis
----
My = Min(My_ten,My_com) = 2.3308E+008
Φy ((10a) or (10b)) = 1.3315994E-005
My_ten (8a) = 2.3308E+008
Φ_ten (7a) = 78.48339
error of function (7a) = 0.00010055
My_com (8b) = 3.4649E+008
Φ_com (7b) = 70.96936
error of function (7b) = -0.00051806
with ey = 0.00222222
eco = 0.002
apl = 0.35 ((9a) in Biskinis and Fardis for no FRP Wrap)
d1 = 44.00

R = 200.00
v = 0.00189797
N = 4770.12
Ac = 125663.706
ω = 0.54
with fc = 20.00

Calculation of ratio lb/ld

Adequate Lap Length: lb/ld >= 1

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

Calculation No. 4

column C1, Floor 1

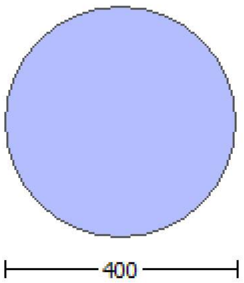
Limit State: Operational Level (data interpolation between analysis steps 1 and 2)

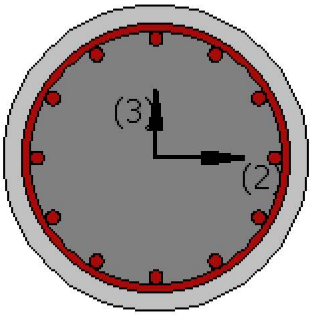
Analysis: Uniform +X

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 = 1.00
Mean strength values are used for both shear and moment calculations.
Consequently:
Existing material of Primary Member: Concrete Strength, fc = fcm = 20.00
Existing material of Primary Member: Steel Strength, fs = fsm = 444.4444
Concrete Elasticity, Ec = 21019.039

Steel Elasticity, Es = 200000.00
##
Note: Especially for the calculation of moment strengths,
the above steel re-bar strengths are multiplied by 1.25 according to R18.6.5, ACI 318-14
Existing material: Steel Strength, fs = 1.25*fsm = 555.5556
##
Diameter, D = 400.00
Cover Thickness, c = 25.00
Mean Confinement Factor overall section = 1.45937
Element Length, L = 3000.00
Primary Member
Ribbed Bars
Ductile Steel
With Detailing for Earthquake Resistance (including stirrups closed at 135°)
Longitudinal Bars With Ends Lapped Starting at the End Sections
Adequate Lap Length (lo/lou,min>=1)
No FRP Wrapping

Stepwise Properties

At local axis: 3
EDGE -A-
Shear Force, Va = -6.3906440E-031
EDGE -B-
Shear Force, Vb = 6.3906440E-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 = 0.53837083
Member Controlled by Flexure (Ve/Vr < 1)
Calculation of Shear Demand from fig. R18.6.5, ACI 318-14 Ve = (Mpr1 + Mpr2)/ln = 155270.734
with
Mpr1 = Max(Mu1+ , Mu1-) = 2.3291E+008
Mu1+ = 2.3291E+008, is the ultimate moment strength at the edge 1 of the member in the actual moment direction which is defined for the static loading combination
Mu1- = 2.3291E+008, is the ultimate moment strength at the edge 1 of the member in the opposite moment direction which is defined for the static loading combination
Mpr2 = Max(Mu2+ , Mu2-) = 2.3291E+008
Mu2+ = 2.3291E+008, is the ultimate moment strength at the edge 2 of the member in the actual moment direction which is defined for the the static loading combination
Mu2- = 2.3291E+008, is the ultimate moment strength at the edge 2 of the member in the opposite moment direction which is defined for the the static loading combination

----
Calculation of Mu1+

Calculation of ultimate Moment Strength ((3.67), Biskinis Phd), Mu
Mu = 2.3291E+008

φ = 1.11701
φ' = 0.98759739
error of function (3.68), Biskinis Phd = 59859.019
From 5A.2, TBDY: fcc = fc*λc = 29.18743
conf. factor λc = 1.45937

fc = 20.00
From 10.3.5, ASCE41-17, Final value of fy: fy*Min(1,1.25*(lb/ld)^2/3) = 555.5556
lb/ld = 1.00
d1 = 44.00
R = 200.00
v = 0.00189786
N = 4771.233
Ac = 125663.706
ω = ω*Min(1,1.25*(lb/ld)^2/3) = 0.54

Calculation of ratio lb/ld

Adequate Lap Length: lb/ld >= 1

Calculation of Mu1-

Calculation of ultimate Moment Strength ((3.67), Biskinis Phd), Mu
Mu = 2.3291E+008

φ = 1.11701
φ' = 0.98759739
error of function (3.68), Biskinis Phd = 59859.019
From 5A.2, TBDY: fcc = fc*λc = 29.18743
conf. factor λc = 1.45937
fc = 20.00
From 10.3.5, ASCE41-17, Final value of fy: fy*Min(1,1.25*(lb/ld)^2/3) = 555.5556
lb/ld = 1.00
d1 = 44.00
R = 200.00
v = 0.00189786
N = 4771.233
Ac = 125663.706
ω = ω*Min(1,1.25*(lb/ld)^2/3) = 0.54

Calculation of ratio lb/ld

Adequate Lap Length: lb/ld >= 1

Calculation of Mu2+

Calculation of ultimate Moment Strength ((3.67), Biskinis Phd), Mu
Mu = 2.3291E+008

φ = 1.11701
φ' = 0.98759739
error of function (3.68), Biskinis Phd = 59859.019
From 5A.2, TBDY: fcc = fc*λc = 29.18743
conf. factor λc = 1.45937
fc = 20.00
From 10.3.5, ASCE41-17, Final value of fy: fy*Min(1,1.25*(lb/ld)^2/3) = 555.5556
lb/ld = 1.00
d1 = 44.00
R = 200.00
v = 0.00189786
N = 4771.233
Ac = 125663.706
ω = ω*Min(1,1.25*(lb/ld)^2/3) = 0.54

Calculation of ratio lb/ld

Adequate Lap Length: lb/ld >= 1

Calculation of Mu2-

Calculation of ultimate Moment Strength ((3.67), Biskinis Phd), Mu
Mu = 2.3291E+008

φ = 1.11701
φ' = 0.98759739
error of function (3.68), Biskinis Phd = 59859.019
From 5A.2, TBDY: fcc = fc*λc = 29.18743
  conf. factor λc = 1.45937
  fc = 20.00
From 10.3.5, ASCE41-17, Final value of fy: fy*Min(1,1.25*(lb/ld)^2/3) = 555.5556
    lb/ld = 1.00
    d1 = 44.00
    R = 200.00
    v = 0.00189786
      N = 4771.233
      Ac = 125663.706
    ω = ω*Min(1,1.25*(lb/ld)^2/3) = 0.54

Calculation of ratio lb/ld

Adequate Lap Length: lb/ld >= 1

Calculation of Shear Strength Vr = Min(Vr1,Vr2) = 288408.521

Calculation of Shear Strength at edge 1, Vr1 = 288408.521
Vr1 = VCol ((10.3), ASCE 41-17) = knl*VCol0
  VCol0 = 288408.521
  knl = 1 (zero step-static loading)

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

  λ = 1 (normal-weight concrete)
  fc' = 20.00, but fc^0.5 <= 8.3 MPa (22.5.3.1, ACI 318-14)
  M/Vd = 2.00
    Mu = 1.4857213E-011
    Vu = 6.3906440E-031
  d = 0.8*D = 320.00
  Nu = 4771.233
  Ag = 125663.706
  From (11.5.4.8), ACI 318-14: Vs = 175459.634
    Av = π/2*A_stirrup = 123370.055
    fy = 444.4444
    s = 100.00
  Vs is multiplied by αCol = 0.00
    s/d = 0.3125
  Vf ((11-3)-(11.4), ACI 440) = 0.00
  From (11-11), ACI 440: Vs + Vf <= 238930.50
    bw*d = π*d*d/4 = 80424.772

Calculation of Shear Strength at edge 2, Vr2 = 288408.521
Vr2 = VCol ((10.3), ASCE 41-17) = knl*VCol0
VCol0 = 288408.521
knl = 1 (zero step-static loading)

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

λ = 1 (normal-weight concrete)
fc' = 20.00, but fc^0.5 <= 8.3 MPa (22.5.3.1, ACI 318-14)
M/Vd = 2.00
Mu = 1.4857213E-011
Vu = 6.3906440E-031
d = 0.8*D = 320.00
Nu = 4771.233
Ag = 125663.706
From (11.5.4.8), ACI 318-14: Vs = 175459.634
Av = π/2*A_stirrup = 123370.055
fy = 444.4444
s = 100.00
Vs is multiplied by αCol = 0.00
s/d = 0.3125
Vf ((11-3)-(11.4), ACI 440) = 0.00
From (11-11), ACI 440: Vs + Vf <= 238930.50
bw*d = π*d*d/4 = 80424.772

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

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

Constant Properties

Knowledge Factor, κ = 1.00
Mean strength values are used for both shear and moment calculations.
Consequently:
Existing material of Primary Member: Concrete Strength, fc = fcm = 20.00
Existing material of Primary Member: Steel Strength, fs = fsm = 444.4444
Concrete Elasticity, Ec = 21019.039
Steel Elasticity, Es = 200000.00
##
Note: Especially for the calculation of moment strengths,
the above steel re-bar strengths are multiplied by 1.25 according to R18.6.5, ACI 318-14
Existing material: Steel Strength, fs = 1.25*fsm = 555.5556
##
Diameter, D = 400.00
Cover Thickness, c = 25.00
Mean Confinement Factor overall section = 1.45937
Element Length, L = 3000.00
Primary Member
Ribbed Bars
Ductile Steel
With Detailing for Earthquake Resistance (including stirrups closed at 135°)
Longitudinal Bars With Ends Lapped Starting at the End Sections
Adequate Lap Length (lo/lou,min>=1)
No FRP Wrapping

Stepwise Properties

At local axis: 2
EDGE -A-
Shear Force, Va = 3.9130116E-047
EDGE -B-
Shear Force, Vb = -3.9130116E-047
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 = 0.53837083
Member Controlled by Flexure (Ve/Vr < 1)
Calculation of Shear Demand from fig. R18.6.5, ACI 318-14 Ve = (Mpr1 + Mpr2)/ln = 155270.734
with
Mpr1 = Max(Mu1+ , Mu1-) = 2.3291E+008
Mu1+ = 2.3291E+008, is the ultimate moment strength at the edge 1 of the member in the actual moment direction which is defined for the static loading combination
Mu1- = 2.3291E+008, is the ultimate moment strength at the edge 1 of the member in the opposite moment direction which is defined for the static loading combination
Mpr2 = Max(Mu2+ , Mu2-) = 2.3291E+008
Mu2+ = 2.3291E+008, is the ultimate moment strength at the edge 2 of the member in the actual moment direction which is defined for the the static loading combination
Mu2- = 2.3291E+008, is the ultimate moment strength at the edge 2 of the member in the opposite moment direction which is defined for the the static loading combination

----
Calculation of Mu1+

Calculation of ultimate Moment Strength ((3.67), Biskinis Phd), Mu
Mu = 2.3291E+008

φ = 1.11701
φ' = 0.98759739
error of function (3.68), Biskinis Phd = 59859.019
From 5A.2, TBDY: fcc = fc*λc = 29.18743
conf. factor λc = 1.45937
fc = 20.00
From 10.3.5, ASCE41-17, Final value of fy: fy*Min(1,1.25*(lb/ld)^2/3) = 555.5556
lb/ld = 1.00
d1 = 44.00
R = 200.00
v = 0.00189786
N = 4771.233
Ac = 125663.706
ω = ω*Min(1,1.25*(lb/ld)^2/3) = 0.54

Calculation of ratio lb/ld

Adequate Lap Length: lb/ld >= 1

Calculation of Mu1-

Calculation of ultimate Moment Strength ((3.67), Biskinis Phd), Mu
Mu = 2.3291E+008

φ = 1.11701
φ' = 0.98759739
error of function (3.68), Biskinis Phd = 59859.019
From 5A.2, TBDY: fcc = fc*λc = 29.18743
conf. factor λc = 1.45937
fc = 20.00
From 10.3.5, ASCE41-17, Final value of fy: fy*Min(1,1.25*(lb/ld)^2/3) = 555.5556
lb/ld = 1.00
d1 = 44.00
R = 200.00
v = 0.00189786
N = 4771.233
Ac = 125663.706
ω = ω*Min(1,1.25*(lb/ld)^2/3) = 0.54

Calculation of ratio lb/ld

Adequate Lap Length: lb/ld >= 1

Calculation of Mu2+

Calculation of ultimate Moment Strength ((3.67), Biskinis Phd), Mu
Mu = 2.3291E+008

φ = 1.11701
φ' = 0.98759739
error of function (3.68), Biskinis Phd = 59859.019
From 5A.2, TBDY: fcc = fc*λc = 29.18743
conf. factor λc = 1.45937
fc = 20.00
From 10.3.5, ASCE41-17, Final value of fy: fy*Min(1,1.25*(lb/ld)^2/3) = 555.5556
lb/ld = 1.00
d1 = 44.00
R = 200.00
v = 0.00189786
N = 4771.233
Ac = 125663.706
ω = ω*Min(1,1.25*(lb/ld)^2/3) = 0.54

Calculation of ratio lb/ld

Adequate Lap Length: lb/ld >= 1

Calculation of Mu2-

Calculation of ultimate Moment Strength ((3.67), Biskinis Phd), Mu
Mu = 2.3291E+008

φ = 1.11701
φ' = 0.98759739
error of function (3.68), Biskinis Phd = 59859.019
From 5A.2, TBDY: fcc = fc*λc = 29.18743
conf. factor λc = 1.45937
fc = 20.00

From 10.3.5, ASCE41-17, Final value of fy: fy*Min(1,1.25*(lb/ld)^2/3) = 555.5556
lb/ld = 1.00
d1 = 44.00
R = 200.00
v = 0.00189786
N = 4771.233
Ac = 125663.706
ω = ω*Min(1,1.25*(lb/ld)^2/3) = 0.54

Calculation of ratio lb/ld

Adequate Lap Length: lb/ld >= 1

Calculation of Shear Strength Vr = Min(Vr1,Vr2) = 288408.521

Calculation of Shear Strength at edge 1, Vr1 = 288408.521
Vr1 = VCol ((10.3), ASCE 41-17) = knl*VCol0
VCol0 = 288408.521
knl = 1 (zero step-static loading)

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

λ = 1 (normal-weight concrete)
fc' = 20.00, but fc^0.5 <= 8.3 MPa (22.5.3.1, ACI 318-14)
M/Vd = 2.00
Mu = 7.3393178E-012
Vu = 3.9130116E-047
d = 0.8*D = 320.00
Nu = 4771.233
Ag = 125663.706
From (11.5.4.8), ACI 318-14: Vs = 175459.634
Av = π/2*A_stirrup = 123370.055
fy = 444.4444
s = 100.00
Vs is multiplied by αCol = 0.00
s/d = 0.3125
Vf ((11-3)-(11.4), ACI 440) = 0.00
From (11-11), ACI 440: Vs + Vf <= 238930.50
bw*d = π*d*d/4 = 80424.772

Calculation of Shear Strength at edge 2, Vr2 = 288408.521
Vr2 = VCol ((10.3), ASCE 41-17) = knl*VCol0
VCol0 = 288408.521
knl = 1 (zero step-static loading)

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

λ = 1 (normal-weight concrete)
fc' = 20.00, but fc^0.5 <= 8.3 MPa (22.5.3.1, ACI 318-14)
M/Vd = 2.00
Mu = 7.3393178E-012
Vu = 3.9130116E-047
d = 0.8*D = 320.00
Nu = 4771.233
Ag = 125663.706
From (11.5.4.8), ACI 318-14: Vs = 175459.634
Av = π/2*A_stirrup = 123370.055
fy = 444.4444
s = 100.00
Vs is multiplied by αCol = 0.00

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

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

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

Constant Properties

Knowledge Factor, κ = 1.00
Chord Rotation is generally considered as Deformation-Controlled Action according to Table C7-1, ASCE41-17.
Mean strengths are used for Deformation-Controlled Actions according to 7.5.1.3, ASCE 41-17
Consequently:
Existing material of Primary Member: Concrete Strength, fc = fcm = 20.00
Existing material of Primary Member: Steel Strength, fs = fsm = 444.4444
Concrete Elasticity, Ec = 21019.039
Steel Elasticity, Es = 200000.00
Diameter, D = 400.00
Cover Thickness, c = 25.00
Element Length, L = 3000.00
Primary Member
Ribbed Bars
Ductile Steel
With Detailing for Earthquake Resistance (including stirrups closed at 135°)
Longitudinal Bars With Ends Lapped Starting at the End Sections
Adequate Lap Length (lb/ld>=1)
No FRP Wrapping

Stepwise Properties

Bending Moment, M = -8.3435E+006
Shear Force, V2 = -2779.824
Shear Force, V3 = -1.0292144E-013
Axial Force, F = -4770.12
Longitudinal Reinforcement Area Distribution (in 2 divisions)
-Tension: Aslt = 1272.345
-Compression: Aslc = 1781.283
Longitudinal Reinforcement Area Distribution (in 3 divisions)
-Tension: Asl,ten = 1017.876
-Compression: Asl,com = 1017.876
-Middle: Asl,mid = 1017.876
Mean Diameter of Tension Reinforcement, DbL = 18.00

Existing component: From table 7-7, ASCE 41_17: Final chord rotation Capacity θu,R = κ*θu = 0.03442871
θu = θy + θp = 0.03442871

- Calculation of θy -

θy = (My*Ls/3)/EIeff = 0.02942871 ((4.29),Biskinis Phd))
My = 2.3308E+008
Ls = M/V (with Ls >0.1*L and Ls < 2*L) = 3001.447
From table 10.5, ASCE 41_17: EIeff = factor*Ec*Ig = 7.9240E+012

factor = 0.30
Ag = 125663.706
fc' = 20.00
N = 4770.12
Ec*Ig = 2.6413E+013

Calculation of Yielding Moment My

Calculation of Φy and My according to (7) - (8) in Biskinis and Fardis
----
My = Min(My_ten,My_com) = 2.3308E+008
Φy ((10a) or (10b)) = 1.3315994E-005
My_ten (8a) = 2.3308E+008
Φ_ten (7a) = 78.48339
error of function (7a) = 0.00010055
My_com (8b) = 3.4649E+008
Φ_com (7b) = 70.96936
error of function (7b) = -0.00051806
with ey = 0.00222222
eco = 0.002
apl = 0.35 ((9a) in Biskinis and Fardis for no FRP Wrap)
d1 = 44.00
R = 200.00
v = 0.00189797
N = 4770.12
Ac = 125663.706
ω = 0.54
with fc = 20.00

Calculation of ratio lb/ld

Adequate Lap Length: lb/ld >= 1

- Calculation of θp -

From table 10-9: θp = 0.005
with:
- Columns controlled by inadequate development or splicing along the clear height because lb/ld < 1
shear control ratio VyE/VCol0E = 0.53837083
d = 707.00
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 = 4770.12
Ag = 125663.706
f'cE = 20.00
fytE = fylE = 444.4444
pl = Area_Tot_Long_Rein/(Ag) = 0.0243
f'cE = 20.00

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

Calculation No. 5

column C1, Floor 1

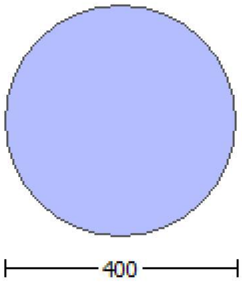
Limit State: Operational Level (data interpolation between analysis steps 1 and 2)

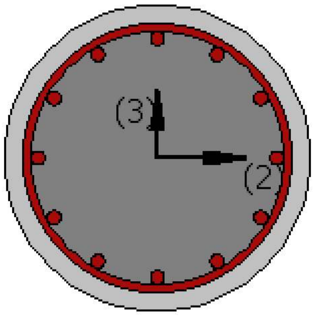
Analysis: Uniform +X

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 = 1.00
Member Shear Force is generally considered as Force-Controlled Action according to Table C7-1, ASCE41-17.
Lower-bound strengths are used for Force-Controlled Actions according to 7.5.1.3, ASCE 41-17
Consequently:
Existing material of Primary Member: Concrete Strength, fc = fc_lower_bound = 16.00
Existing material of Primary Member: Steel Strength, fs = fs_lower_bound = 400.00
Concrete Elasticity, Ec = 21019.039
Steel Elasticity, Es = 200000.00
###
Note: Especially for the calculation of θy for displacement ductility demand,
the expected (mean value) strengths are used (7.5.1.3, ASCE41-17) because bending is considered as
Deformation-Controlled Action (Table C7-1, ASCE41-17).
Existing material: Concrete Strength, fc = fcm = 20.00
Existing material: Steel Strength, fs = fsm = 444.4444
###
Diameter, D = 400.00
Cover Thickness, c = 25.00
Element Length, L = 3000.00
Primary Member
Ribbed Bars
Ductile Steel
With Detailing for Earthquake Resistance (including stirrups closed at 135°)
Longitudinal Bars With Ends Lapped Starting at the End Sections
Adequate Lap Length (lo/lou,min = lb/ld >=1)
No FRP Wrapping

Stepwise Properties

EDGE -A-
Bending Moment, Ma = -8.3435E+006
Shear Force, Va = -2779.824
EDGE -B-
Bending Moment, Mb = 0.04084082
Shear Force, Vb = 2779.824
BOTH EDGES
Axial Force, F = -4770.12
Longitudinal Reinforcement Area Distribution (in 2 divisions)
-Tension: Aslt = 0.00
-Compression: Aslc = 3053.628
Longitudinal Reinforcement Area Distribution (in 3 divisions)
-Tension: Asl,ten = 1017.876
-Compression: Asl,com = 1017.876
-Middle: Asl,mid = 1017.876
Mean Diameter of Tension Reinforcement, DbL,ten = 18.00

Existing component: From table 7-7, ASCE 41_17: Final Shear Capacity VR = κ*Vn = 259037.852
Vn ((10.3), ASCE 41-17) = knl*VCol0 = 259037.852
VCol = 259037.852
knl = 1.00
displacement_ductility_demand = 0.05368949

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

λ = 1 (normal-weight concrete)
fc' = 16.00, but fc^0.5 <= 8.3 MPa (22.5.3.1, ACI 318-14)
M/Vd = 2.00
Mu = 0.04084082
Vu = 2779.824
d = 0.8*D = 320.00
Nu = 4770.12
Ag = 125663.706
From (11.5.4.8), ACI 318-14: Vs = 157913.67
Av = π/2*A_stirrup = 123370.055
fy = 400.00
s = 100.00
Vs is multiplied by αCol = 0.00
s/d = 0.3125
Vf ((11-3)-(11.4), ACI 440) = 0.00
From (11-11), ACI 440: Vs + Vf <= 213705.936
bw*d = π*d*d/4 = 80424.772

----

displacement_ductility_demand is calculated as θ/θy

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

From analysis, chord rotation θ = 0.00015793
θy = (My*Ls/3)/Eleff = 0.00294145 ((4.29),Biskinis Phd))
My = 2.3308E+008
Ls = M/V (with Ls >0.1*L and Ls < 2*L) = 300.00
From table 10.5, ASCE 41_17: Eleff = factor*Ec*Ig = 7.9240E+012
factor = 0.30
Ag = 125663.706
fc' = 20.00
N = 4770.12

Ec*Ig = 2.6413E+013

Calculation of Yielding Moment My

Calculation of Φy and My according to (7) - (8) in Biskinis and Fardis
----
My = Min(My_ten,My_com) = 2.3308E+008
Φy ((10a) or (10b)) = 1.3315994E-005
My_ten (8a) = 2.3308E+008
Φ_ten (7a) = 78.48339
error of function (7a) = 0.00010055
My_com (8b) = 3.4649E+008
Φ_com (7b) = 70.96936
error of function (7b) = -0.00051806
with ey = 0.00222222
eco = 0.002
apl = 0.35 ((9a) in Biskinis and Fardis for no FRP Wrap)
d1 = 44.00
R = 200.00
v = 0.00189797
N = 4770.12
Ac = 125663.706
ω = 0.54
with fc = 20.00

Calculation of ratio lb/ld

Adequate Lap Length: lb/ld >= 1

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

Calculation No. 6

column C1, Floor 1

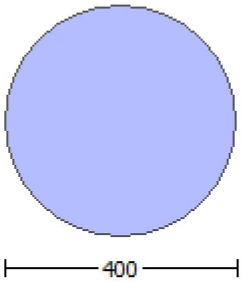
Limit State: Operational Level (data interpolation between analysis steps 1 and 2)

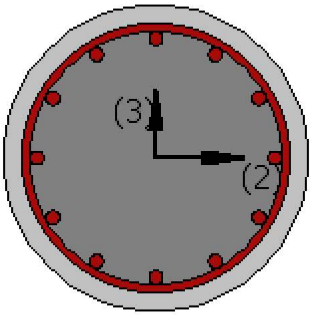
Analysis: Uniform +X

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 = 1.00
Mean strength values are used for both shear and moment calculations.
Consequently:
Existing material of Primary Member: Concrete Strength, fc = fcm = 20.00
Existing material of Primary Member: Steel Strength, fs = fsm = 444.4444
Concrete Elasticity, Ec = 21019.039
Steel Elasticity, Es = 200000.00
###
Note: Especially for the calculation of moment strengths,
the above steel re-bar strengths are multiplied by 1.25 according to R18.6.5, ACI 318-14
Existing material: Steel Strength, fs = 1.25*fsm = 555.5556
###
Diameter, D = 400.00
Cover Thickness, c = 25.00
Mean Confinement Factor overall section = 1.45937
Element Length, L = 3000.00
Primary Member
Ribbed Bars
Ductile Steel
With Detailing for Earthquake Resistance (including stirrups closed at 135°)
Longitudinal Bars With Ends Lapped Starting at the End Sections
Adequate Lap Length (lo/lou,min>=1)
No FRP Wrapping

Stepwise Properties

At local axis: 3
EDGE -A-
Shear Force, Va = -6.3906440E-031
EDGE -B-
Shear Force, Vb = 6.3906440E-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 = 0.53837083
Member Controlled by Flexure (Ve/Vr < 1)
Calculation of Shear Demand from fig. R18.6.5, ACI 318-14 Ve = (Mpr1 + Mpr2)/ln = 155270.734
with
Mpr1 = Max(Mu1+ , Mu1-) = 2.3291E+008
Mu1+ = 2.3291E+008, is the ultimate moment strength at the edge 1 of the member in the actual moment direction which is defined for the static loading combination
Mu1- = 2.3291E+008, is the ultimate moment strength at the edge 1 of the member in the opposite moment direction which is defined for the static loading combination
Mpr2 = Max(Mu2+ , Mu2-) = 2.3291E+008
Mu2+ = 2.3291E+008, is the ultimate moment strength at the edge 2 of the member in the actual moment direction which is defined for the the static loading combination
Mu2- = 2.3291E+008, is the ultimate moment strength at the edge 2 of the member in the opposite moment direction which is defined for the the static loading combination

----
Calculation of Mu1+

Calculation of ultimate Moment Strength ((3.67), Biskinis Phd), Mu
Mu = 2.3291E+008

φ = 1.11701
φ' = 0.98759739
error of function (3.68), Biskinis Phd = 59859.019
From 5A.2, TBDY: fcc = fc*λc = 29.18743
conf. factor λc = 1.45937
fc = 20.00
From 10.3.5, ASCE41-17, Final value of fy: fy*Min(1,1.25*(lb/ld)^2/3) = 555.5556
lb/ld = 1.00
d1 = 44.00
R = 200.00
v = 0.00189786
N = 4771.233
Ac = 125663.706
ω = ω*Min(1,1.25*(lb/ld)^2/3) = 0.54

Calculation of ratio lb/ld

Adequate Lap Length: lb/ld >= 1

Calculation of Mu1-

Calculation of ultimate Moment Strength ((3.67), Biskinis Phd), Mu
Mu = 2.3291E+008

φ = 1.11701
φ' = 0.98759739
error of function (3.68), Biskinis Phd = 59859.019
From 5A.2, TBDY: fcc = fc*λc = 29.18743
conf. factor λc = 1.45937
fc = 20.00
From 10.3.5, ASCE41-17, Final value of fy: fy*Min(1,1.25*(lb/ld)^2/3) = 555.5556
lb/ld = 1.00
d1 = 44.00
R = 200.00
v = 0.00189786
N = 4771.233
Ac = 125663.706
ω = ω*Min(1,1.25*(lb/ld)^2/3) = 0.54

Calculation of ratio lb/ld

Adequate Lap Length: lb/ld >= 1

Calculation of Mu2+

Calculation of ultimate Moment Strength ((3.67), Biskinis Phd), Mu
Mu = 2.3291E+008

φ = 1.11701
φ' = 0.98759739
error of function (3.68), Biskinis Phd = 59859.019
From 5A.2, TBDY: fcc = fc*λc = 29.18743
conf. factor λc = 1.45937
fc = 20.00
From 10.3.5, ASCE41-17, Final value of fy: fy*Min(1,1.25*(lb/ld)^2/3) = 555.5556
lb/ld = 1.00
d1 = 44.00
R = 200.00
v = 0.00189786
N = 4771.233
Ac = 125663.706
ω = ω*Min(1,1.25*(lb/ld)^2/3) = 0.54

Calculation of ratio lb/ld

Adequate Lap Length: lb/ld >= 1

Calculation of Mu2-

Calculation of ultimate Moment Strength ((3.67), Biskinis Phd), Mu
Mu = 2.3291E+008

φ = 1.11701
φ' = 0.98759739
error of function (3.68), Biskinis Phd = 59859.019
From 5A.2, TBDY: fcc = fc*λc = 29.18743
conf. factor λc = 1.45937
fc = 20.00
From 10.3.5, ASCE41-17, Final value of fy: fy*Min(1,1.25*(lb/ld)^2/3) = 555.5556
lb/ld = 1.00
d1 = 44.00
R = 200.00
v = 0.00189786
N = 4771.233
Ac = 125663.706
ω = ω*Min(1,1.25*(lb/ld)^2/3) = 0.54

Calculation of ratio lb/ld

Adequate Lap Length: lb/ld >= 1

Calculation of Shear Strength Vr = Min(Vr1,Vr2) = 288408.521

Calculation of Shear Strength at edge 1, Vr1 = 288408.521
Vr1 = VCol ((10.3), ASCE 41-17) = knl*VCol0
VCol0 = 288408.521
knl = 1 (zero step-static loading)

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

λ = 1 (normal-weight concrete)
fc' = 20.00, but fc^0.5 <= 8.3 MPa (22.5.3.1, ACI 318-14)
M/Vd = 2.00
Mu = 1.4857213E-011
Vu = 6.3906440E-031
d = 0.8*D = 320.00
Nu = 4771.233
Ag = 125663.706
From (11.5.4.8), ACI 318-14: Vs = 175459.634
Av = π/2*A_stirrup = 123370.055
fy = 444.4444
s = 100.00
Vs is multiplied by αCol = 0.00
s/d = 0.3125
Vf ((11-3)-(11.4), ACI 440) = 0.00
From (11-11), ACI 440: Vs + Vf <= 238930.50
bw*d = π*d*d/4 = 80424.772

Calculation of Shear Strength at edge 2, Vr2 = 288408.521
Vr2 = VCol ((10.3), ASCE 41-17) = knl*VCol0
VCol0 = 288408.521
knl = 1 (zero step-static loading)

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

λ = 1 (normal-weight concrete)
fc' = 20.00, but fc^0.5 <= 8.3 MPa (22.5.3.1, ACI 318-14)
M/Vd = 2.00
Mu = 1.4857213E-011
Vu = 6.3906440E-031
d = 0.8*D = 320.00
Nu = 4771.233
Ag = 125663.706
From (11.5.4.8), ACI 318-14: Vs = 175459.634
Av = π/2*A_stirrup = 123370.055
fy = 444.4444
s = 100.00
Vs is multiplied by αCol = 0.00
s/d = 0.3125
Vf ((11-3)-(11.4), ACI 440) = 0.00
From (11-11), ACI 440: Vs + Vf <= 238930.50
bw*d = π*d*d/4 = 80424.772

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

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

Constant Properties

Knowledge Factor, κ = 1.00
Mean strength values are used for both shear and moment calculations.
Consequently:
Existing material of Primary Member: Concrete Strength, fc = fcm = 20.00
Existing material of Primary Member: Steel Strength, fs = fsm = 444.4444
Concrete Elasticity, Ec = 21019.039
Steel Elasticity, Es = 200000.00
###
Note: Especially for the calculation of moment strengths,
the above steel re-bar strengths are multiplied by 1.25 according to R18.6.5, ACI 318-14
Existing material: Steel Strength, fs = 1.25*fsm = 555.5556
###
Diameter, D = 400.00
Cover Thickness, c = 25.00
Mean Confinement Factor overall section = 1.45937
Element Length, L = 3000.00
Primary Member
Ribbed Bars
Ductile Steel
With Detailing for Earthquake Resistance (including stirrups closed at 135°)
Longitudinal Bars With Ends Lapped Starting at the End Sections
Adequate Lap Length (lo/lou,min>=1)
No FRP Wrapping

Stepwise Properties

At local axis: 2
EDGE -A-
Shear Force, Va = 3.9130116E-047
EDGE -B-
Shear Force, Vb = -3.9130116E-047
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 = 0.53837083
Member Controlled by Flexure (Ve/Vr < 1)
Calculation of Shear Demand from fig. R18.6.5, ACI 318-14 Ve = (Mpr1 + Mpr2)/ln = 155270.734
with
Mpr1 = Max(Mu1+ , Mu1-) = 2.3291E+008
Mu1+ = 2.3291E+008, is the ultimate moment strength at the edge 1 of the member in the actual moment direction which is defined for the static loading combination
Mu1- = 2.3291E+008, is the ultimate moment strength at the edge 1 of the member in the opposite moment direction which is defined for the static loading combination
Mpr2 = Max(Mu2+ , Mu2-) = 2.3291E+008
Mu2+ = 2.3291E+008, is the ultimate moment strength at the edge 2 of the member in the actual moment direction which is defined for the the static loading combination
Mu2- = 2.3291E+008, is the ultimate moment strength at the edge 2 of the member in the opposite moment direction which is defined for the the static loading combination

----
Calculation of Mu1+

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

Mu = 2.3291E+008

φ = 1.11701
φ' = 0.98759739
error of function (3.68), Biskinis Phd = 59859.019
From 5A.2, TBDY: fcc = fc*λc = 29.18743
conf. factor λc = 1.45937
fc = 20.00
From 10.3.5, ASCE41-17, Final value of fy: fy*Min(1,1.25*(lb/ld)^2/3) = 555.5556
lb/ld = 1.00
d1 = 44.00
R = 200.00
v = 0.00189786
N = 4771.233
Ac = 125663.706
ω = ω*Min(1,1.25*(lb/ld)^2/3) = 0.54

Calculation of ratio lb/ld

Adequate Lap Length: lb/ld >= 1

Calculation of Mu1-

Calculation of ultimate Moment Strength ((3.67), Biskinis Phd), Mu
Mu = 2.3291E+008

φ = 1.11701
φ' = 0.98759739
error of function (3.68), Biskinis Phd = 59859.019
From 5A.2, TBDY: fcc = fc*λc = 29.18743
conf. factor λc = 1.45937
fc = 20.00
From 10.3.5, ASCE41-17, Final value of fy: fy*Min(1,1.25*(lb/ld)^2/3) = 555.5556
lb/ld = 1.00
d1 = 44.00
R = 200.00
v = 0.00189786
N = 4771.233
Ac = 125663.706
ω = ω*Min(1,1.25*(lb/ld)^2/3) = 0.54

Calculation of ratio lb/ld

Adequate Lap Length: lb/ld >= 1

Calculation of Mu2+

Calculation of ultimate Moment Strength ((3.67), Biskinis Phd), Mu
Mu = 2.3291E+008

φ = 1.11701
φ' = 0.98759739
error of function (3.68), Biskinis Phd = 59859.019
From 5A.2, TBDY: fcc = fc*λc = 29.18743
conf. factor λc = 1.45937
fc = 20.00
From 10.3.5, ASCE41-17, Final value of fy: fy*Min(1,1.25*(lb/ld)^2/3) = 555.5556

lb/ld = 1.00
d1 = 44.00
R = 200.00
v = 0.00189786
N = 4771.233
Ac = 125663.706
ω = ω*Min(1,1.25*(lb/ld)^2/3) = 0.54

Calculation of ratio lb/ld

Adequate Lap Length: lb/ld >= 1

Calculation of Mu2-

Calculation of ultimate Moment Strength ((3.67), Biskinis Phd), Mu
Mu = 2.3291E+008

φ = 1.11701
φ' = 0.98759739
error of function (3.68), Biskinis Phd = 59859.019
From 5A.2, TBDY: fcc = fc*λc = 29.18743
conf. factor λc = 1.45937
fc = 20.00
From 10.3.5, ASCE41-17, Final value of fy: fy*Min(1,1.25*(lb/ld)^2/3) = 555.5556
lb/ld = 1.00
d1 = 44.00
R = 200.00
v = 0.00189786
N = 4771.233
Ac = 125663.706
ω = ω*Min(1,1.25*(lb/ld)^2/3) = 0.54

Calculation of ratio lb/ld

Adequate Lap Length: lb/ld >= 1

Calculation of Shear Strength Vr = Min(Vr1,Vr2) = 288408.521

Calculation of Shear Strength at edge 1, Vr1 = 288408.521
Vr1 = VCol ((10.3), ASCE 41-17) = knl*VCol0
VCol0 = 288408.521
knl = 1 (zero step-static loading)

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

λ = 1 (normal-weight concrete)
fc' = 20.00, but fc^0.5 <= 8.3 MPa (22.5.3.1, ACI 318-14)
M/Vd = 2.00
Mu = 7.3393178E-012
Vu = 3.9130116E-047
d = 0.8*D = 320.00
Nu = 4771.233
Ag = 125663.706
From (11.5.4.8), ACI 318-14: Vs = 175459.634
Av = π/2*A_stirrup = 123370.055
fy = 444.4444
s = 100.00

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

Calculation of Shear Strength at edge 2, Vr2 = 288408.521
Vr2 = VCol ((10.3), ASCE 41-17) = knl*VCol0
VCol0 = 288408.521
knl = 1 (zero step-static loading)

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

λ = 1 (normal-weight concrete)
fc' = 20.00, but fc^0.5 <= 8.3 MPa (22.5.3.1, ACI 318-14)
M/Vd = 2.00
Mu = 7.3393178E-012
Vu = 3.9130116E-047
d = 0.8*D = 320.00
Nu = 4771.233
Ag = 125663.706
From (11.5.4.8), ACI 318-14: Vs = 175459.634
Av = π/2*A_stirrup = 123370.055
fy = 444.4444
s = 100.00
Vs is multiplied by αCol = 0.00
s/d = 0.3125
Vf ((11-3)-(11.4), ACI 440) = 0.00
From (11-11), ACI 440: Vs + Vf <= 238930.50
bw*d = π*d*d/4 = 80424.772

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

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

Constant Properties

Knowledge Factor, κ = 1.00
Chord Rotation is generally considered as Deformation-Controlled Action according to Table C7-1, ASCE41-17.
Mean strengths are used for Deformation-Controlled Actions according to 7.5.1.3, ASCE 41-17
Consequently:
Existing material of Primary Member: Concrete Strength, fc = fcm = 20.00
Existing material of Primary Member: Steel Strength, fs = fsm = 444.4444
Concrete Elasticity, Ec = 21019.039
Steel Elasticity, Es = 200000.00
Diameter, D = 400.00
Cover Thickness, c = 25.00
Element Length, L = 3000.00
Primary Member
Ribbed Bars
Ductile Steel
With Detailing for Earthquake Resistance (including stirrups closed at 135°)
Longitudinal Bars With Ends Lapped Starting at the End Sections
Adequate Lap Length (lb/ld>=1)
No FRP Wrapping

Stepwise Properties

Bending Moment, M = -7.6585865E-011
Shear Force, V2 = 2779.824
Shear Force, V3 = 1.0292144E-013
Axial Force, F = -4770.12
Longitudinal Reinforcement Area Distribution (in 2 divisions)
-Tension: Aslt = 0.00
-Compression: Aslc = 3053.628
Longitudinal Reinforcement Area Distribution (in 3 divisions)
-Tension: Asl,ten = 1017.876
-Compression: Asl,com = 1017.876
-Middle: Asl,mid = 1017.876
Mean Diameter of Tension Reinforcement, DbL = 18.00

Existing component: From table 7-7, ASCE 41_17: Final chord rotation Capacity θu,R = κ*θu = 0.01970726
θu = θy + θp = 0.01970726

- Calculation of θy -

θy = (My*Ls/3)/EIeff = 0.01470726 ((4.29),Biskinis Phd))
My = 2.3308E+008
Ls = M/V (with Ls >0.1*L and Ls < 2*L) = 1500.00
From table 10.5, ASCE 41_17: EIeff = factor*Ec*Ig = 7.9240E+012
factor = 0.30
Ag = 125663.706
fc' = 20.00
N = 4770.12
Ec*Ig = 2.6413E+013

Calculation of Yielding Moment My

Calculation of Φy and My according to (7) - (8) in Biskinis and Fardis
----
My = Min(My_ten,My_com) = 2.3308E+008
Φy ((10a) or (10b)) = 1.3315994E-005
My_ten (8a) = 2.3308E+008
Φ_ten (7a) = 78.48339
error of function (7a) = 0.00010055
My_com (8b) = 3.4649E+008
Φ_com (7b) = 70.96936
error of function (7b) = -0.00051806
with ey = 0.00222222
eco = 0.002
apl = 0.35 ((9a) in Biskinis and Fardis for no FRP Wrap)
d1 = 44.00
R = 200.00
v = 0.00189797
N = 4770.12
Ac = 125663.706
ω = 0.54
with fc = 20.00

Calculation of ratio lb/ld

Adequate Lap Length: lb/ld >= 1

- Calculation of θp -

From table 10-9: θp = 0.005
with:
- Columns controlled by inadequate development or splicing along the clear height because lb/ld < 1
shear control ratio VyE/VCol0E = 0.53837083
d = 707.00
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 = 4770.12
Ag = 125663.706
f'cE = 20.00
fytE = fylE = 444.4444
pl = Area_Tot_Long_Rein/(Ag) = 0.0243
f'cE = 20.00

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

Calculation No. 7

column C1, Floor 1

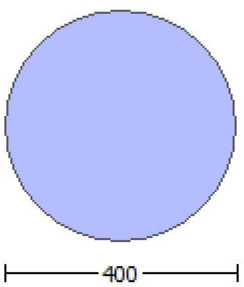
Limit State: Operational Level (data interpolation between analysis steps 1 and 2)

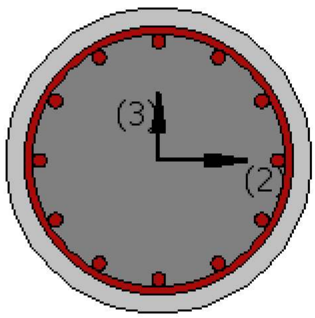
Analysis: Uniform +X

Check: Shear capacity VRd

Edge: End

Local Axis: (3)





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

Constant Properties

Knowledge Factor, κ = 1.00
Member Shear Force is generally considered as Force-Controlled Action according to Table C7-1, ASCE41-17.
Lower-bound strengths are used for Force-Controlled Actions according to 7.5.1.3, ASCE 41-17
Consequently:
Existing material of Primary Member: Concrete Strength, fc = fc_lower_bound = 16.00
Existing material of Primary Member: Steel Strength, fs = fs_lower_bound = 400.00
Concrete Elasticity, Ec = 21019.039
Steel Elasticity, Es = 200000.00
##
Note: Especially for the calculation of θy for displacement ductility demand,
the expected (mean value) strengths are used (7.5.1.3, ASCE41-17) because bending is considered as
Deformation-Controlled Action (Table C7-1, ASCE41-17).
Existing material: Concrete Strength, fc = fcm = 20.00
Existing material: Steel Strength, fs = fsm = 444.4444
##
Diameter, D = 400.00
Cover Thickness, c = 25.00
Element Length, L = 3000.00
Primary Member
Ribbed Bars
Ductile Steel
With Detailing for Earthquake Resistance (including stirrups closed at 135°)
Longitudinal Bars With Ends Lapped Starting at the End Sections
Adequate Lap Length (lo/lou,min = lb/ld >=1)
No FRP Wrapping

Stepwise Properties

EDGE -A-
Bending Moment, Ma = 3.8560899E-010
Shear Force, Va = -1.0292144E-013
EDGE -B-
Bending Moment, Mb = -7.6585865E-011
Shear Force, Vb = 1.0292144E-013
BOTH EDGES
Axial Force, F = -4770.12
Longitudinal Reinforcement Area Distribution (in 2 divisions)
-Tension: Aslt = 0.00
-Compression: Aslc = 3053.628
Longitudinal Reinforcement Area Distribution (in 3 divisions)
-Tension: Asl,ten = 1017.876
-Compression: Asl,com = 1017.876
-Middle: Asl,mid = 1017.876
Mean Diameter of Tension Reinforcement, DbL,ten = 18.00

Existing component: From table 7-7, ASCE 41_17: Final Shear Capacity VR = κ*Vn = 259037.852
Vn ((10.3), ASCE 41-17) = knl*VCol0 = 259037.852
VCol = 259037.852
knl = 1.00
displacement_ductility_demand = 0.00

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

λ = 1 (normal-weight concrete)
fc' = 16.00, but fc^0.5 <= 8.3 MPa (22.5.3.1, ACI 318-14)
M/Vd = 2.00
Mu = 7.6585865E-011
Vu = 1.0292144E-013
d = 0.8*D = 320.00
Nu = 4770.12

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

----
displacement_ductility_demand is calculated as θ/θy

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

From analysis, chord rotation θ = 9.2409501E-021
θy = (My*Ls/3)/Eleff = 0.01470726 ((4.29),Biskinis Phd))
My = 2.3308E+008
Ls = M/V (with Ls >0.1*L and Ls < 2*L) = 1500.00
From table 10.5, ASCE 41_17: Eleff = factor*Ec*Ig = 7.9240E+012
factor = 0.30
Ag = 125663.706
fc' = 20.00
N = 4770.12
Ec*Ig = 2.6413E+013

Calculation of Yielding Moment My

Calculation of Φy and My according to (7) - (8) in Biskinis and Fardis
----
My = Min(My_ten,My_com) = 2.3308E+008
Φy ((10a) or (10b)) = 1.3315994E-005
My_ten (8a) = 2.3308E+008
Φ_ten (7a) = 78.48339
error of function (7a) = 0.00010055
My_com (8b) = 3.4649E+008
Φ_com (7b) = 70.96936
error of function (7b) = -0.00051806
with ey = 0.00222222
eco = 0.002
apl = 0.35 ((9a) in Biskinis and Fardis for no FRP Wrap)
d1 = 44.00
R = 200.00
v = 0.00189797
N = 4770.12
Ac = 125663.706
ω = 0.54
with fc = 20.00

Calculation of ratio lb/ld

Adequate Lap Length: lb/ld >= 1

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

Calculation No. 8

column C1, Floor 1

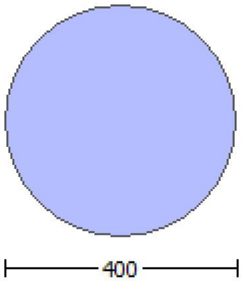
Limit State: Operational Level (data interpolation between analysis steps 1 and 2)

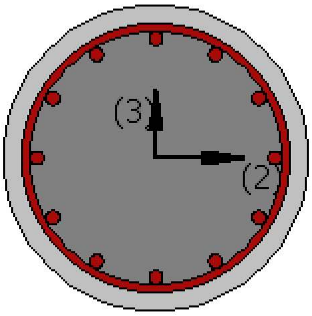
Analysis: Uniform +X

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 = 1.00
Mean strength values are used for both shear and moment calculations.
Consequently:
Existing material of Primary Member: Concrete Strength, fc = fcm = 20.00
Existing material of Primary Member: Steel Strength, fs = fsm = 444.4444
Concrete Elasticity, Ec = 21019.039
Steel Elasticity, Es = 200000.00
##
Note: Especially for the calculation of moment strengths,
the above steel re-bar strengths are multiplied by 1.25 according to R18.6.5, ACI 318-14
Existing material: Steel Strength, fs = 1.25*fsm = 555.5556
##
Diameter, D = 400.00
Cover Thickness, c = 25.00
Mean Confinement Factor overall section = 1.45937
Element Length, L = 3000.00
Primary Member
Ribbed Bars
Ductile Steel
With Detailing for Earthquake Resistance (including stirrups closed at 135°)
Longitudinal Bars With Ends Lapped Starting at the End Sections
Adequate Lap Length (lo/lou,min>=1)
No FRP Wrapping

Stepwise Properties

At local axis: 3
EDGE -A-
Shear Force, Va = -6.3906440E-031
EDGE -B-
Shear Force, Vb = 6.3906440E-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 = 0.53837083
Member Controlled by Flexure (Ve/Vr < 1)
Calculation of Shear Demand from fig. R18.6.5, ACI 318-14 Ve = (Mpr1 + Mpr2)/ln = 155270.734
with
Mpr1 = Max(Mu1+ , Mu1-) = 2.3291E+008
Mu1+ = 2.3291E+008, is the ultimate moment strength at the edge 1 of the member in the actual moment direction which is defined for the static loading combination
Mu1- = 2.3291E+008, is the ultimate moment strength at the edge 1 of the member in the opposite moment direction which is defined for the static loading combination
Mpr2 = Max(Mu2+ , Mu2-) = 2.3291E+008
Mu2+ = 2.3291E+008, is the ultimate moment strength at the edge 2 of the member in the actual moment direction which is defined for the the static loading combination
Mu2- = 2.3291E+008, is the ultimate moment strength at the edge 2 of the member in the opposite moment direction which is defined for the the static loading combination

----
Calculation of Mu1+

Calculation of ultimate Moment Strength ((3.67), Biskinis Phd), Mu
Mu = 2.3291E+008

φ = 1.11701
φ' = 0.98759739
error of function (3.68), Biskinis Phd = 59859.019
From 5A.2, TBDY: fcc = fc*λc = 29.18743
conf. factor λc = 1.45937
fc = 20.00
From 10.3.5, ASCE41-17, Final value of fy: fy*Min(1,1.25*(lb/ld)^2/3) = 555.5556
lb/ld = 1.00
d1 = 44.00
R = 200.00
v = 0.00189786
N = 4771.233
Ac = 125663.706
ω = ω*Min(1,1.25*(lb/ld)^2/3) = 0.54

Calculation of ratio lb/ld

Adequate Lap Length: lb/ld >= 1

Calculation of Mu1-

Calculation of ultimate Moment Strength ((3.67), Biskinis Phd), Mu
Mu = 2.3291E+008

φ = 1.11701
φ' = 0.98759739
error of function (3.68), Biskinis Phd = 59859.019
From 5A.2, TBDY: fcc = fc*λc = 29.18743
conf. factor λc = 1.45937
fc = 20.00
From 10.3.5, ASCE41-17, Final value of fy: fy*Min(1,1.25*(lb/ld)^2/3) = 555.5556
lb/ld = 1.00
d1 = 44.00
R = 200.00
v = 0.00189786
N = 4771.233
Ac = 125663.706
ω = ω*Min(1,1.25*(lb/ld)^2/3) = 0.54

Calculation of ratio lb/ld

Adequate Lap Length: lb/ld >= 1

Calculation of Mu2+

Calculation of ultimate Moment Strength ((3.67), Biskinis Phd), Mu
Mu = 2.3291E+008

φ = 1.11701
φ' = 0.98759739
error of function (3.68), Biskinis Phd = 59859.019
From 5A.2, TBDY: fcc = fc*λc = 29.18743
conf. factor λc = 1.45937
fc = 20.00
From 10.3.5, ASCE41-17, Final value of fy: fy*Min(1,1.25*(lb/ld)^2/3) = 555.5556
lb/ld = 1.00
d1 = 44.00
R = 200.00
v = 0.00189786
N = 4771.233
Ac = 125663.706
ω = ω*Min(1,1.25*(lb/ld)^2/3) = 0.54

Calculation of ratio lb/ld

Adequate Lap Length: lb/ld >= 1

Calculation of Mu2-

Calculation of ultimate Moment Strength ((3.67), Biskinis Phd), Mu
Mu = 2.3291E+008

φ = 1.11701
φ' = 0.98759739
error of function (3.68), Biskinis Phd = 59859.019
From 5A.2, TBDY: fcc = fc*λc = 29.18743

conf. factor λc = 1.45937
fc = 20.00
From 10.3.5, ASCE41-17, Final value of fy: fy*Min(1,1.25*(lb/ld)^2/3) = 555.5556
lb/ld = 1.00
d1 = 44.00
R = 200.00
v = 0.00189786
N = 4771.233
Ac = 125663.706
ω = ω*Min(1,1.25*(lb/ld)^2/3) = 0.54

Calculation of ratio lb/ld

Adequate Lap Length: lb/ld >= 1

Calculation of Shear Strength Vr = Min(Vr1,Vr2) = 288408.521

Calculation of Shear Strength at edge 1, Vr1 = 288408.521
Vr1 = VCol ((10.3), ASCE 41-17) = knl*VCol0
VCol0 = 288408.521
knl = 1 (zero step-static loading)

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

λ = 1 (normal-weight concrete)
fc' = 20.00, but fc^0.5 <= 8.3 MPa (22.5.3.1, ACI 318-14)
M/Vd = 2.00
Mu = 1.4857213E-011
Vu = 6.3906440E-031
d = 0.8*D = 320.00
Nu = 4771.233
Ag = 125663.706
From (11.5.4.8), ACI 318-14: Vs = 175459.634
Av = π/2*A_stirrup = 123370.055
fy = 444.4444
s = 100.00
Vs is multiplied by αCol = 0.00
s/d = 0.3125
Vf ((11-3)-(11.4), ACI 440) = 0.00
From (11-11), ACI 440: Vs + Vf <= 238930.50
bw*d = π*d*d/4 = 80424.772

Calculation of Shear Strength at edge 2, Vr2 = 288408.521
Vr2 = VCol ((10.3), ASCE 41-17) = knl*VCol0
VCol0 = 288408.521
knl = 1 (zero step-static loading)

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

λ = 1 (normal-weight concrete)
fc' = 20.00, but fc^0.5 <= 8.3 MPa (22.5.3.1, ACI 318-14)
M/Vd = 2.00
Mu = 1.4857213E-011
Vu = 6.3906440E-031
d = 0.8*D = 320.00
Nu = 4771.233
Ag = 125663.706
From (11.5.4.8), ACI 318-14: Vs = 175459.634
Av = π/2*A_stirrup = 123370.055
fy = 444.4444

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

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

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

Constant Properties

Knowledge Factor, κ = 1.00
Mean strength values are used for both shear and moment calculations.
Consequently:
Existing material of Primary Member: Concrete Strength, fc = fcm = 20.00
Existing material of Primary Member: Steel Strength, fs = fsm = 444.4444
Concrete Elasticity, Ec = 21019.039
Steel Elasticity, Es = 200000.00
###
Note: Especially for the calculation of moment strengths,
the above steel re-bar strengths are multiplied by 1.25 according to R18.6.5, ACI 318-14
Existing material: Steel Strength, fs = 1.25*fsm = 555.5556
###
Diameter, D = 400.00
Cover Thickness, c = 25.00
Mean Confinement Factor overall section = 1.45937
Element Length, L = 3000.00
Primary Member
Ribbed Bars
Ductile Steel
With Detailing for Earthquake Resistance (including stirrups closed at 135°)
Longitudinal Bars With Ends Lapped Starting at the End Sections
Adequate Lap Length (lo/lou,min>=1)
No FRP Wrapping

Stepwise Properties

At local axis: 2
EDGE -A-
Shear Force, Va = 3.9130116E-047
EDGE -B-
Shear Force, Vb = -3.9130116E-047
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 = 0.53837083

Member Controlled by Flexure (Ve/Vr < 1)
Calculation of Shear Demand from fig. R18.6.5, ACI 318-14 Ve = (Mpr1 + Mpr2)/ln = 155270.734
with
Mpr1 = Max(Mu1+ , Mu1-) = 2.3291E+008
Mu1+ = 2.3291E+008, is the ultimate moment strength at the edge 1 of the member in the actual moment direction which is defined for the static loading combination
Mu1- = 2.3291E+008, is the ultimate moment strength at the edge 1 of the member in the opposite moment direction which is defined for the static loading combination
Mpr2 = Max(Mu2+ , Mu2-) = 2.3291E+008
Mu2+ = 2.3291E+008, is the ultimate moment strength at the edge 2 of the member in the actual moment direction which is defined for the the static loading combination
Mu2- = 2.3291E+008, is the ultimate moment strength at the edge 2 of the member in the opposite moment direction which is defined for the the static loading combination

----
Calculation of Mu1+

Calculation of ultimate Moment Strength ((3.67), Biskinis Phd), Mu
Mu = 2.3291E+008

φ = 1.11701
φ' = 0.98759739
error of function (3.68), Biskinis Phd = 59859.019
From 5A.2, TBDY: fcc = fc*λc = 29.18743
conf. factor λc = 1.45937
fc = 20.00
From 10.3.5, ASCE41-17, Final value of fy: fy*Min(1,1.25*(lb/ld)^2/3) = 555.5556
lb/ld = 1.00
d1 = 44.00
R = 200.00
v = 0.00189786
N = 4771.233
Ac = 125663.706
ω = ω*Min(1,1.25*(lb/ld)^2/3) = 0.54

Calculation of ratio lb/ld

Adequate Lap Length: lb/ld >= 1

Calculation of Mu1-

Calculation of ultimate Moment Strength ((3.67), Biskinis Phd), Mu
Mu = 2.3291E+008

φ = 1.11701
φ' = 0.98759739
error of function (3.68), Biskinis Phd = 59859.019
From 5A.2, TBDY: fcc = fc*λc = 29.18743
conf. factor λc = 1.45937
fc = 20.00
From 10.3.5, ASCE41-17, Final value of fy: fy*Min(1,1.25*(lb/ld)^2/3) = 555.5556
lb/ld = 1.00
d1 = 44.00
R = 200.00
v = 0.00189786
N = 4771.233
Ac = 125663.706
ω = ω*Min(1,1.25*(lb/ld)^2/3) = 0.54

Calculation of ratio lb/ld

Adequate Lap Length: lb/ld >= 1

Calculation of Mu2+

Calculation of ultimate Moment Strength ((3.67), Biskinis Phd), Mu
Mu = 2.3291E+008

φ = 1.11701
φ' = 0.98759739
error of function (3.68), Biskinis Phd = 59859.019
From 5A.2, TBDY: fcc = fc*λc = 29.18743
conf. factor λc = 1.45937
fc = 20.00
From 10.3.5, ASCE41-17, Final value of fy: fy*Min(1,1.25*(lb/ld)^2/3) = 555.5556
lb/ld = 1.00
d1 = 44.00
R = 200.00
v = 0.00189786
N = 4771.233
Ac = 125663.706
ω = ω*Min(1,1.25*(lb/ld)^2/3) = 0.54

Calculation of ratio lb/ld

Adequate Lap Length: lb/ld >= 1

Calculation of Mu2-

Calculation of ultimate Moment Strength ((3.67), Biskinis Phd), Mu
Mu = 2.3291E+008

φ = 1.11701
φ' = 0.98759739
error of function (3.68), Biskinis Phd = 59859.019
From 5A.2, TBDY: fcc = fc*λc = 29.18743
conf. factor λc = 1.45937
fc = 20.00
From 10.3.5, ASCE41-17, Final value of fy: fy*Min(1,1.25*(lb/ld)^2/3) = 555.5556
lb/ld = 1.00
d1 = 44.00
R = 200.00
v = 0.00189786
N = 4771.233
Ac = 125663.706
ω = ω*Min(1,1.25*(lb/ld)^2/3) = 0.54

Calculation of ratio lb/ld

Adequate Lap Length: lb/ld >= 1

Calculation of Shear Strength Vr = Min(Vr1,Vr2) = 288408.521

Calculation of Shear Strength at edge 1, Vr1 = 288408.521
Vr1 = VCol ((10.3), ASCE 41-17) = knl*VCol0
VCol0 = 288408.521
knl = 1 (zero step-static loading)

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

λ = 1 (normal-weight concrete)
fc' = 20.00, but fc^0.5 <= 8.3 MPa (22.5.3.1, ACI 318-14)
M/Vd = 2.00
Mu = 7.3393178E-012
Vu = 3.9130116E-047
d = 0.8*D = 320.00
Nu = 4771.233
Ag = 125663.706
From (11.5.4.8), ACI 318-14: Vs = 175459.634
Av = π/2*A_stirrup = 123370.055
fy = 444.4444
s = 100.00
Vs is multiplied by αCol = 0.00
s/d = 0.3125
Vf ((11-3)-(11.4), ACI 440) = 0.00
From (11-11), ACI 440: Vs + Vf <= 238930.50
bw*d = π*d*d/4 = 80424.772

Calculation of Shear Strength at edge 2, Vr2 = 288408.521
Vr2 = VCol ((10.3), ASCE 41-17) = knl*VCol0
VCol0 = 288408.521
knl = 1 (zero step-static loading)

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

λ = 1 (normal-weight concrete)
fc' = 20.00, but fc^0.5 <= 8.3 MPa (22.5.3.1, ACI 318-14)
M/Vd = 2.00
Mu = 7.3393178E-012
Vu = 3.9130116E-047
d = 0.8*D = 320.00
Nu = 4771.233
Ag = 125663.706
From (11.5.4.8), ACI 318-14: Vs = 175459.634
Av = π/2*A_stirrup = 123370.055
fy = 444.4444
s = 100.00
Vs is multiplied by αCol = 0.00
s/d = 0.3125
Vf ((11-3)-(11.4), ACI 440) = 0.00
From (11-11), ACI 440: Vs + Vf <= 238930.50
bw*d = π*d*d/4 = 80424.772

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

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

Constant Properties

Knowledge Factor, κ = 1.00
Chord Rotation is generally considered as Deformation-Controlled Action according to Table C7-1, ASCE41-17.
Mean strengths are used for Deformation-Controlled Actions according to 7.5.1.3, ASCE 41-17
Consequently:
Existing material of Primary Member: Concrete Strength, fc = fcm = 20.00
Existing material of Primary Member: Steel Strength, fs = fsm = 444.4444
Concrete Elasticity, Ec = 21019.039
Steel Elasticity, Es = 200000.00
Diameter, D = 400.00
Cover Thickness, c = 25.00
Element Length, L = 3000.00
Primary Member
Ribbed Bars
Ductile Steel
With Detailing for Earthquake Resistance (including stirrups closed at 135°)
Longitudinal Bars With Ends Lapped Starting at the End Sections
Adequate Lap Length (lb/ld>=1)
No FRP Wrapping

Stepwise Properties

Bending Moment, M = 0.04084082
Shear Force, V2 = 2779.824
Shear Force, V3 = 1.0292144E-013
Axial Force, F = -4770.12
Longitudinal Reinforcement Area Distribution (in 2 divisions)
-Tension: Aslt = 0.00
-Compression: Aslc = 3053.628
Longitudinal Reinforcement Area Distribution (in 3 divisions)
-Tension: Asl,ten = 1017.876
-Compression: Asl,com = 1017.876
-Middle: Asl,mid = 1017.876
Mean Diameter of Tension Reinforcement, DbL = 18.00

Existing component: From table 7-7, ASCE 41_17: Final chord rotation Capacity θu,R = κ*θu = 0.00794145
θu = θy + θp = 0.00794145

- Calculation of θy -

θy = (My*Ls/3)/EIeff = 0.00294145 ((4.29),Biskinis Phd))
My = 2.3308E+008
Ls = M/V (with Ls >0.1*L and Ls < 2*L) = 300.00
From table 10.5, ASCE 41_17: EIeff = factor*Ec*Ig = 7.9240E+012
factor = 0.30
Ag = 125663.706
fc' = 20.00
N = 4770.12
Ec*Ig = 2.6413E+013

Calculation of Yielding Moment My

Calculation of Φy and My according to (7) - (8) in Biskinis and Fardis
----
My = Min(My_ten,My_com) = 2.3308E+008
Φy ((10a) or (10b)) = 1.3315994E-005
My_ten (8a) = 2.3308E+008
Φ_ten (7a) = 78.48339
error of function (7a) = 0.00010055
My_com (8b) = 3.4649E+008
Φ_com (7b) = 70.96936

error of function (7b) = -0.00051806
with ey = 0.00222222
eco = 0.002
apl = 0.35 ((9a) in Biskinis and Fardis for no FRP Wrap)
d1 = 44.00
R = 200.00
v = 0.00189797
N = 4770.12
Ac = 125663.706
ω = 0.54
with fc = 20.00

Calculation of ratio lb/ld

Adequate Lap Length: lb/ld >= 1

- Calculation of θp -

From table 10-9: θp = 0.005
with:
- Columns controlled by inadequate development or splicing along the clear height because lb/ld < 1
shear control ratio VyE/VCol0E = 0.53837083
d = 707.00
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 = 4770.12
Ag = 125663.706
f'cE = 20.00
fytE = fylE = 444.4444
pl = Area_Tot_Long_Rein/(Ag) = 0.0243
f'cE = 20.00

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

Calculation No. 9

column C1, Floor 1

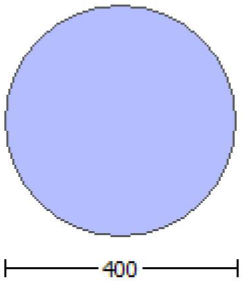
Limit State: Life Safety (data interpolation between analysis steps 1 and 2)

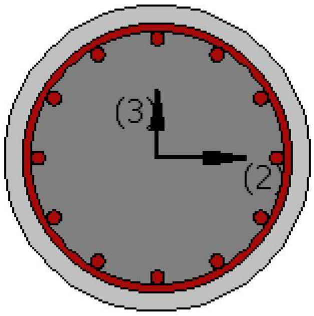
Analysis: Uniform +X

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 = 1.00
Member Shear Force is generally considered as Force-Controlled Action according to Table C7-1, ASCE41-17.
Lower-bound strengths are used for Force-Controlled Actions according to 7.5.1.3, ASCE 41-17
Consequently:
Existing material of Primary Member: Concrete Strength, fc = fc_lower_bound = 16.00
Existing material of Primary Member: Steel Strength, fs = fs_lower_bound = 400.00
Concrete Elasticity, Ec = 21019.039
Steel Elasticity, Es = 200000.00
##
Note: Especially for the calculation of θy for displacement ductility demand,
the expected (mean value) strengths are used (7.5.1.3, ASCE41-17) because bending is considered as
Deformation-Controlled Action (Table C7-1, ASCE41-17).
Existing material: Concrete Strength, fc = fcm = 20.00
Existing material: Steel Strength, fs = fsm = 444.4444
##
Diameter, D = 400.00
Cover Thickness, c = 25.00
Element Length, L = 3000.00
Primary Member
Ribbed Bars
Ductile Steel
With Detailing for Earthquake Resistance (including stirrups closed at 135°)
Longitudinal Bars With Ends Lapped Starting at the End Sections
Adequate Lap Length (lo/lou,min = lb/ld >=1)
No FRP Wrapping

Stepwise Properties

EDGE -A-
Bending Moment, Ma = -1.0433E+007
Shear Force, Va = -3476.079
EDGE -B-
Bending Moment, Mb = 0.0510701
Shear Force, Vb = 3476.079
BOTH EDGES
Axial Force, F = -4769.841
Longitudinal Reinforcement Area Distribution (in 2 divisions)
-Tension: Aslt = 1272.345
-Compression: Aslc = 1781.283
Longitudinal Reinforcement Area Distribution (in 3 divisions)
-Tension: Asl,ten = 1017.876
-Compression: Asl,com = 1017.876
-Middle: Asl,mid = 1017.876
Mean Diameter of Tension Reinforcement, DbL,ten = 18.00

Existing component: From table 7-7, ASCE 41_17: Final Shear Capacity VR = κ*Vn = 208475.733
Vn ((10.3), ASCE 41-17) = knl*VCol0 = 208475.733
VCol = 208475.733
knl = 1.00
displacement_ductility_demand = 0.01198098

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

λ = 1 (normal-weight concrete)
fc' = 16.00, but fc^0.5 <= 8.3 MPa (22.5.3.1, ACI 318-14)
M/Vd = 4.00
Mu = 1.0433E+007
Vu = 3476.079
d = 0.8*D = 320.00
Nu = 4769.841
Ag = 125663.706
From (11.5.4.8), ACI 318-14: Vs = 157913.67
Av = π/2*A_stirrup = 123370.055
fy = 400.00
s = 100.00
Vs is multiplied by αCol = 0.00
s/d = 0.3125
Vf ((11-3)-(11.4), ACI 440) = 0.00
From (11-11), ACI 440: Vs + Vf <= 213705.936
bw*d = π*d*d/4 = 80424.772

----
displacement_ductility_demand is calculated as θ/θy

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

From analysis, chord rotation θ = 0.00035258
θy = (My*Ls/3)/Eleff = 0.0294287 ((4.29),Biskinis Phd))
My = 2.3308E+008
Ls = M/V (with Ls >0.1*L and Ls < 2*L) = 3001.447
From table 10.5, ASCE 41_17: Eleff = factor*Ec*Ig = 7.9240E+012
factor = 0.30
Ag = 125663.706
fc' = 20.00
N = 4769.841
Ec*Ig = 2.6413E+013

Calculation of Yielding Moment My

Calculation of Φy and My according to (7) - (8) in Biskinis and Fardis
----
My = Min(My_ten,My_com) = 2.3308E+008
Φy ((10a) or (10b)) = 1.3315992E-005
My_ten (8a) = 2.3308E+008
Φ_ten (7a) = 78.48338
error of function (7a) = 0.00010055
My_com (8b) = 3.4649E+008
Φ_com (7b) = 70.96935
error of function (7b) = -0.00051806
with ey = 0.00222222
eco = 0.002
apl = 0.35 ((9a) in Biskinis and Fardis for no FRP Wrap)
d1 = 44.00

R = 200.00
v = 0.00189786
N = 4769.841
Ac = 125663.706
ω = 0.54
with fc = 20.00

Calculation of ratio lb/ld

Adequate Lap Length: lb/ld >= 1

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

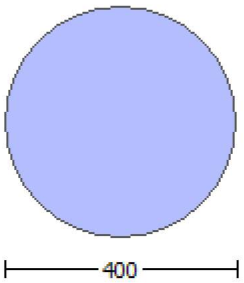
Limit State: Life Safety (data interpolation between analysis steps 1 and 2)

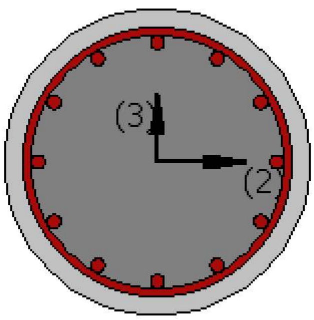
Analysis: Uniform +X

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 = 1.00
Mean strength values are used for both shear and moment calculations.
Consequently:
Existing material of Primary Member: Concrete Strength, fc = fcm = 20.00
Existing material of Primary Member: Steel Strength, fs = fsm = 444.4444
Concrete Elasticity, Ec = 21019.039

Steel Elasticity, Es = 200000.00
##
Note: Especially for the calculation of moment strengths,
the above steel re-bar strengths are multiplied by 1.25 according to R18.6.5, ACI 318-14
Existing material: Steel Strength, fs = 1.25*fsm = 555.5556
##
Diameter, D = 400.00
Cover Thickness, c = 25.00
Mean Confinement Factor overall section = 1.45937
Element Length, L = 3000.00
Primary Member
Ribbed Bars
Ductile Steel
With Detailing for Earthquake Resistance (including stirrups closed at 135°)
Longitudinal Bars With Ends Lapped Starting at the End Sections
Adequate Lap Length (lo/lou,min>=1)
No FRP Wrapping

Stepwise Properties

At local axis: 3
EDGE -A-
Shear Force, Va = -6.3906440E-031
EDGE -B-
Shear Force, Vb = 6.3906440E-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 = 0.53837083
Member Controlled by Flexure (Ve/Vr < 1)
Calculation of Shear Demand from fig. R18.6.5, ACI 318-14 Ve = (Mpr1 + Mpr2)/ln = 155270.734
with
Mpr1 = Max(Mu1+ , Mu1-) = 2.3291E+008
Mu1+ = 2.3291E+008, is the ultimate moment strength at the edge 1 of the member in the actual moment direction which is defined for the static loading combination
Mu1- = 2.3291E+008, is the ultimate moment strength at the edge 1 of the member in the opposite moment direction which is defined for the static loading combination
Mpr2 = Max(Mu2+ , Mu2-) = 2.3291E+008
Mu2+ = 2.3291E+008, is the ultimate moment strength at the edge 2 of the member in the actual moment direction which is defined for the the static loading combination
Mu2- = 2.3291E+008, is the ultimate moment strength at the edge 2 of the member in the opposite moment direction which is defined for the the static loading combination

----
Calculation of Mu1+

Calculation of ultimate Moment Strength ((3.67), Biskinis Phd), Mu
Mu = 2.3291E+008

φ = 1.11701
φ' = 0.98759739
error of function (3.68), Biskinis Phd = 59859.019
From 5A.2, TBDY: fcc = fc*λc = 29.18743
conf. factor λc = 1.45937

fc = 20.00
From 10.3.5, ASCE41-17, Final value of fy: fy*Min(1,1.25*(lb/ld)^2/3) = 555.5556
lb/ld = 1.00
d1 = 44.00
R = 200.00
v = 0.00189786
N = 4771.233
Ac = 125663.706
ω = ω*Min(1,1.25*(lb/ld)^2/3) = 0.54

Calculation of ratio lb/ld

Adequate Lap Length: lb/ld >= 1

Calculation of Mu1-

Calculation of ultimate Moment Strength ((3.67), Biskinis Phd), Mu
Mu = 2.3291E+008

φ = 1.11701
φ' = 0.98759739
error of function (3.68), Biskinis Phd = 59859.019
From 5A.2, TBDY: fcc = fc*λc = 29.18743
conf. factor λc = 1.45937
fc = 20.00
From 10.3.5, ASCE41-17, Final value of fy: fy*Min(1,1.25*(lb/ld)^2/3) = 555.5556
lb/ld = 1.00
d1 = 44.00
R = 200.00
v = 0.00189786
N = 4771.233
Ac = 125663.706
ω = ω*Min(1,1.25*(lb/ld)^2/3) = 0.54

Calculation of ratio lb/ld

Adequate Lap Length: lb/ld >= 1

Calculation of Mu2+

Calculation of ultimate Moment Strength ((3.67), Biskinis Phd), Mu
Mu = 2.3291E+008

φ = 1.11701
φ' = 0.98759739
error of function (3.68), Biskinis Phd = 59859.019
From 5A.2, TBDY: fcc = fc*λc = 29.18743
conf. factor λc = 1.45937
fc = 20.00
From 10.3.5, ASCE41-17, Final value of fy: fy*Min(1,1.25*(lb/ld)^2/3) = 555.5556
lb/ld = 1.00
d1 = 44.00
R = 200.00
v = 0.00189786
N = 4771.233
Ac = 125663.706
ω = ω*Min(1,1.25*(lb/ld)^2/3) = 0.54

Calculation of ratio lb/ld

Adequate Lap Length: lb/ld >= 1

Calculation of Mu2-

Calculation of ultimate Moment Strength ((3.67), Biskinis Phd), Mu
Mu = 2.3291E+008

φ = 1.11701
φ' = 0.98759739
error of function (3.68), Biskinis Phd = 59859.019
From 5A.2, TBDY: fcc = fc*λc = 29.18743
conf. factor λc = 1.45937
fc = 20.00
From 10.3.5, ASCE41-17, Final value of fy: fy*Min(1,1.25*(lb/ld)^2/3) = 555.5556
lb/ld = 1.00
d1 = 44.00
R = 200.00
v = 0.00189786
N = 4771.233
Ac = 125663.706
ω = ω*Min(1,1.25*(lb/ld)^2/3) = 0.54

Calculation of ratio lb/ld

Adequate Lap Length: lb/ld >= 1

Calculation of Shear Strength Vr = Min(Vr1,Vr2) = 288408.521

Calculation of Shear Strength at edge 1, Vr1 = 288408.521
Vr1 = VCol ((10.3), ASCE 41-17) = knl*VCol0
VCol0 = 288408.521
knl = 1 (zero step-static loading)

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

λ = 1 (normal-weight concrete)
fc' = 20.00, but fc^0.5 <= 8.3 MPa (22.5.3.1, ACI 318-14)
M/Vd = 2.00
Mu = 1.4857213E-011
Vu = 6.3906440E-031
d = 0.8*D = 320.00
Nu = 4771.233
Ag = 125663.706
From (11.5.4.8), ACI 318-14: Vs = 175459.634
Av = π/2*A_stirrup = 123370.055
fy = 444.4444
s = 100.00
Vs is multiplied by αCol = 0.00
s/d = 0.3125
Vf ((11-3)-(11.4), ACI 440) = 0.00
From (11-11), ACI 440: Vs + Vf <= 238930.50
bw*d = π*d*d/4 = 80424.772

Calculation of Shear Strength at edge 2, Vr2 = 288408.521
Vr2 = VCol ((10.3), ASCE 41-17) = knl*VCol0
VCol0 = 288408.521
knl = 1 (zero step-static loading)

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

λ = 1 (normal-weight concrete)
fc' = 20.00, but fc^0.5 <= 8.3 MPa (22.5.3.1, ACI 318-14)
M/Vd = 2.00
Mu = 1.4857213E-011
Vu = 6.3906440E-031
d = 0.8*D = 320.00
Nu = 4771.233
Ag = 125663.706
From (11.5.4.8), ACI 318-14: Vs = 175459.634
Av = π/2*A_stirrup = 123370.055
fy = 444.4444
s = 100.00
Vs is multiplied by αCol = 0.00
s/d = 0.3125
Vf ((11-3)-(11.4), ACI 440) = 0.00
From (11-11), ACI 440: Vs + Vf <= 238930.50
bw*d = π*d*d/4 = 80424.772

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

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

Constant Properties

Knowledge Factor, κ = 1.00
Mean strength values are used for both shear and moment calculations.
Consequently:
Existing material of Primary Member: Concrete Strength, fc = fcm = 20.00
Existing material of Primary Member: Steel Strength, fs = fsm = 444.4444
Concrete Elasticity, Ec = 21019.039
Steel Elasticity, Es = 200000.00
##
Note: Especially for the calculation of moment strengths,
the above steel re-bar strengths are multiplied by 1.25 according to R18.6.5, ACI 318-14
Existing material: Steel Strength, fs = 1.25*fsm = 555.5556
##
Diameter, D = 400.00
Cover Thickness, c = 25.00
Mean Confinement Factor overall section = 1.45937
Element Length, L = 3000.00
Primary Member
Ribbed Bars
Ductile Steel
With Detailing for Earthquake Resistance (including stirrups closed at 135°)
Longitudinal Bars With Ends Lapped Starting at the End Sections
Adequate Lap Length (lo/lou,min>=1)
No FRP Wrapping

Stepwise Properties

At local axis: 2
EDGE -A-
Shear Force, Va = 3.9130116E-047
EDGE -B-
Shear Force, Vb = -3.9130116E-047
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 = 0.53837083
Member Controlled by Flexure (Ve/Vr < 1)
Calculation of Shear Demand from fig. R18.6.5, ACI 318-14 Ve = (Mpr1 + Mpr2)/ln = 155270.734
with
Mpr1 = Max(Mu1+ , Mu1-) = 2.3291E+008
Mu1+ = 2.3291E+008, is the ultimate moment strength at the edge 1 of the member in the actual moment direction which is defined for the static loading combination
Mu1- = 2.3291E+008, is the ultimate moment strength at the edge 1 of the member in the opposite moment direction which is defined for the static loading combination
Mpr2 = Max(Mu2+ , Mu2-) = 2.3291E+008
Mu2+ = 2.3291E+008, is the ultimate moment strength at the edge 2 of the member in the actual moment direction which is defined for the the static loading combination
Mu2- = 2.3291E+008, is the ultimate moment strength at the edge 2 of the member in the opposite moment direction which is defined for the the static loading combination

----
Calculation of Mu1+

Calculation of ultimate Moment Strength ((3.67), Biskinis Phd), Mu
Mu = 2.3291E+008

φ = 1.11701
φ' = 0.98759739
error of function (3.68), Biskinis Phd = 59859.019
From 5A.2, TBDY: fcc = fc*λc = 29.18743
conf. factor λc = 1.45937
fc = 20.00
From 10.3.5, ASCE41-17, Final value of fy: fy*Min(1,1.25*(lb/ld)^2/3) = 555.5556
lb/ld = 1.00
d1 = 44.00
R = 200.00
v = 0.00189786
N = 4771.233
Ac = 125663.706
ω = ω*Min(1,1.25*(lb/ld)^2/3) = 0.54

Calculation of ratio lb/ld

Adequate Lap Length: lb/ld >= 1

Calculation of Mu1-

Calculation of ultimate Moment Strength ((3.67), Biskinis Phd), Mu
Mu = 2.3291E+008

φ = 1.11701
φ' = 0.98759739
error of function (3.68), Biskinis Phd = 59859.019
From 5A.2, TBDY: fcc = fc*λc = 29.18743
conf. factor λc = 1.45937
fc = 20.00
From 10.3.5, ASCE41-17, Final value of fy: fy*Min(1,1.25*(lb/ld)^2/3) = 555.5556
lb/ld = 1.00
d1 = 44.00
R = 200.00
v = 0.00189786
N = 4771.233
Ac = 125663.706
ω = ω*Min(1,1.25*(lb/ld)^2/3) = 0.54

Calculation of ratio lb/ld

Adequate Lap Length: lb/ld >= 1

Calculation of Mu2+

Calculation of ultimate Moment Strength ((3.67), Biskinis Phd), Mu
Mu = 2.3291E+008

φ = 1.11701
φ' = 0.98759739
error of function (3.68), Biskinis Phd = 59859.019
From 5A.2, TBDY: fcc = fc*λc = 29.18743
conf. factor λc = 1.45937
fc = 20.00
From 10.3.5, ASCE41-17, Final value of fy: fy*Min(1,1.25*(lb/ld)^2/3) = 555.5556
lb/ld = 1.00
d1 = 44.00
R = 200.00
v = 0.00189786
N = 4771.233
Ac = 125663.706
ω = ω*Min(1,1.25*(lb/ld)^2/3) = 0.54

Calculation of ratio lb/ld

Adequate Lap Length: lb/ld >= 1

Calculation of Mu2-

Calculation of ultimate Moment Strength ((3.67), Biskinis Phd), Mu
Mu = 2.3291E+008

φ = 1.11701
φ' = 0.98759739
error of function (3.68), Biskinis Phd = 59859.019
From 5A.2, TBDY: fcc = fc*λc = 29.18743
conf. factor λc = 1.45937
fc = 20.00

From 10.3.5, ASCE41-17, Final value of fy: fy*Min(1,1.25*(lb/ld)^2/3) = 555.5556
lb/ld = 1.00
d1 = 44.00
R = 200.00
v = 0.00189786
N = 4771.233
Ac = 125663.706
ω = ω*Min(1,1.25*(lb/ld)^2/3) = 0.54

Calculation of ratio lb/ld

Adequate Lap Length: lb/ld >= 1

Calculation of Shear Strength Vr = Min(Vr1,Vr2) = 288408.521

Calculation of Shear Strength at edge 1, Vr1 = 288408.521
Vr1 = VCol ((10.3), ASCE 41-17) = knl*VCol0
VCol0 = 288408.521
knl = 1 (zero step-static loading)

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

λ = 1 (normal-weight concrete)
fc' = 20.00, but fc^0.5 <= 8.3 MPa (22.5.3.1, ACI 318-14)
M/Vd = 2.00
Mu = 7.3393178E-012
Vu = 3.9130116E-047
d = 0.8*D = 320.00
Nu = 4771.233
Ag = 125663.706
From (11.5.4.8), ACI 318-14: Vs = 175459.634
Av = π/2*A_stirrup = 123370.055
fy = 444.4444
s = 100.00
Vs is multiplied by αCol = 0.00
s/d = 0.3125
Vf ((11-3)-(11.4), ACI 440) = 0.00
From (11-11), ACI 440: Vs + Vf <= 238930.50
bw*d = π*d*d/4 = 80424.772

Calculation of Shear Strength at edge 2, Vr2 = 288408.521
Vr2 = VCol ((10.3), ASCE 41-17) = knl*VCol0
VCol0 = 288408.521
knl = 1 (zero step-static loading)

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

λ = 1 (normal-weight concrete)
fc' = 20.00, but fc^0.5 <= 8.3 MPa (22.5.3.1, ACI 318-14)
M/Vd = 2.00
Mu = 7.3393178E-012
Vu = 3.9130116E-047
d = 0.8*D = 320.00
Nu = 4771.233
Ag = 125663.706
From (11.5.4.8), ACI 318-14: Vs = 175459.634
Av = π/2*A_stirrup = 123370.055
fy = 444.4444
s = 100.00
Vs is multiplied by αCol = 0.00

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

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

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

Constant Properties

Knowledge Factor, κ = 1.00
Chord Rotation is generally considered as Deformation-Controlled Action according to Table C7-1, ASCE41-17.
Mean strengths are used for Deformation-Controlled Actions according to 7.5.1.3, ASCE 41-17
Consequently:
Existing material of Primary Member: Concrete Strength, fc = fcm = 20.00
Existing material of Primary Member: Steel Strength, fs = fsm = 444.4444
Concrete Elasticity, Ec = 21019.039
Steel Elasticity, Es = 200000.00
Diameter, D = 400.00
Cover Thickness, c = 25.00
Element Length, L = 3000.00
Primary Member
Ribbed Bars
Ductile Steel
With Detailing for Earthquake Resistance (including stirrups closed at 135°)
Longitudinal Bars With Ends Lapped Starting at the End Sections
Adequate Lap Length (lb/ld>=1)
No FRP Wrapping

Stepwise Properties

Bending Moment, M = 4.7847013E-010
Shear Force, V2 = -3476.079
Shear Force, V3 = -1.2869988E-013
Axial Force, F = -4769.841
Longitudinal Reinforcement Area Distribution (in 2 divisions)
-Tension: Aslt = 1272.345
-Compression: Aslc = 1781.283
Longitudinal Reinforcement Area Distribution (in 3 divisions)
-Tension: Asl,ten = 1017.876
-Compression: Asl,com = 1017.876
-Middle: Asl,mid = 1017.876
Mean Diameter of Tension Reinforcement, DbL = 18.00

Existing component: From table 7-7, ASCE 41_17: Final chord rotation Capacity θu,R = κ*θu = 0.06197877
θu = θy + θp = 0.06197877

- Calculation of θy -

θy = (My*Ls/3)/EIeff = 0.01470726 ((4.29),Biskinis Phd))
My = 2.3308E+008
Ls = M/V (with Ls >0.1*L and Ls < 2*L) = 1500.00
From table 10.5, ASCE 41_17: EIeff = factor*Ec*Ig = 7.9240E+012

factor = 0.30
Ag = 125663.706
fc' = 20.00
N = 4769.841
Ec*Ig = 2.6413E+013

Calculation of Yielding Moment My

Calculation of Φy and My according to (7) - (8) in Biskinis and Fardis
----
My = Min(My_ten,My_com) = 2.3308E+008
Φy ((10a) or (10b)) = 1.3315992E-005
My_ten (8a) = 2.3308E+008
Φ_ten (7a) = 78.48338
error of function (7a) = 0.00010055
My_com (8b) = 3.4649E+008
Φ_com (7b) = 70.96935
error of function (7b) = -0.00051806
with ey = 0.00222222
eco = 0.002
apl = 0.35 ((9a) in Biskinis and Fardis for no FRP Wrap)
d1 = 44.00
R = 200.00
v = 0.00189786
N = 4769.841
Ac = 125663.706
ω = 0.54
with fc = 20.00

Calculation of ratio lb/ld

Adequate Lap Length: lb/ld >= 1

- Calculation of θp -

From table 10-9: θp = 0.04727152
with:
- Columns controlled by inadequate development or splicing along the clear height because lb/ld < 1
shear control ratio VyE/VCol0E = 0.53837083
d = 707.00
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 = 4769.841
Ag = 125663.706
f'cE = 20.00
fytE = fylE = 444.4444
pl = Area_Tot_Long_Rein/(Ag) = 0.0243
f'cE = 20.00

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

Calculation No. 11

column C1, Floor 1

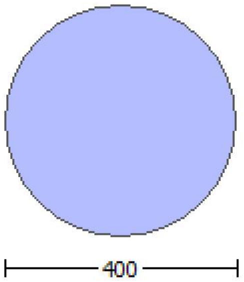
Limit State: Life Safety (data interpolation between analysis steps 1 and 2)

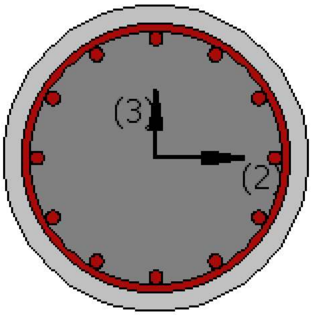
Analysis: Uniform +X

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 = 1.00
Member Shear Force is generally considered as Force-Controlled Action according to Table C7-1, ASCE41-17.
Lower-bound strengths are used for Force-Controlled Actions according to 7.5.1.3, ASCE 41-17
Consequently:
Existing material of Primary Member: Concrete Strength, fc = fc_lower_bound = 16.00
Existing material of Primary Member: Steel Strength, fs = fs_lower_bound = 400.00
Concrete Elasticity, Ec = 21019.039
Steel Elasticity, Es = 200000.00
###
Note: Especially for the calculation of θy for displacement ductility demand,
the expected (mean value) strengths are used (7.5.1.3, ASCE41-17) because bending is considered as
Deformation-Controlled Action (Table C7-1, ASCE41-17).
Existing material: Concrete Strength, fc = fcm = 20.00
Existing material: Steel Strength, fs = fsm = 444.4444
###
Diameter, D = 400.00
Cover Thickness, c = 25.00
Element Length, L = 3000.00
Primary Member
Ribbed Bars
Ductile Steel
With Detailing for Earthquake Resistance (including stirrups closed at 135°)
Longitudinal Bars With Ends Lapped Starting at the End Sections
Adequate Lap Length (lo/lou,min = lb/ld >=1)
No FRP Wrapping

Stepwise Properties

EDGE -A-
Bending Moment, Ma = 4.7847013E-010
Shear Force, Va = -1.2869988E-013
EDGE -B-
Bending Moment, Mb = -9.2046865E-011
Shear Force, Vb = 1.2869988E-013
BOTH EDGES
Axial Force, F = -4769.841
Longitudinal Reinforcement Area Distribution (in 2 divisions)
-Tension: Aslt = 1272.345
-Compression: Aslc = 1781.283
Longitudinal Reinforcement Area Distribution (in 3 divisions)
-Tension: Asl,ten = 1017.876
-Compression: Asl,com = 1017.876
-Middle: Asl,mid = 1017.876
Mean Diameter of Tension Reinforcement, DbL,ten = 18.00

Existing component: From table 7-7, ASCE 41_17: Final Shear Capacity VR = κ*Vn = 259037.796
Vn ((10.3), ASCE 41-17) = knl*VCol0 = 259037.796
VCol = 259037.796
knl = 1.00
displacement_ductility_demand = 0.00

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

λ = 1 (normal-weight concrete)
fc' = 16.00, but fc^0.5 <= 8.3 MPa (22.5.3.1, ACI 318-14)
M/Vd = 2.00
Mu = 4.7847013E-010
Vu = 1.2869988E-013
d = 0.8*D = 320.00
Nu = 4769.841
Ag = 125663.706
From (11.5.4.8), ACI 318-14: Vs = 157913.67
Av = π/2*A_stirrup = 123370.055
fy = 400.00
s = 100.00
Vs is multiplied by αCol = 0.00
s/d = 0.3125
Vf ((11-3)-(11.4), ACI 440) = 0.00
From (11-11), ACI 440: Vs + Vf <= 213705.936
bw*d = π*d*d/4 = 80424.772

----

displacement_ductility_demand is calculated as θ/θy

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

From analysis, chord rotation θ = 2.2658910E-020
θy = (My*Ls/3)/EIeff = 0.01470726 ((4.29),Biskinis Phd))
My = 2.3308E+008
Ls = M/V (with Ls >0.1*L and Ls < 2*L) = 1500.00
From table 10.5, ASCE 41_17: EIeff = factor*Ec*Ig = 7.9240E+012
factor = 0.30
Ag = 125663.706
fc' = 20.00
N = 4769.841

Ec*Ig = 2.6413E+013

Calculation of Yielding Moment My

Calculation of Φy and My according to (7) - (8) in Biskinis and Fardis
----
My = Min(My_ten,My_com) = 2.3308E+008
Φy ((10a) or (10b)) = 1.3315992E-005
My_ten (8a) = 2.3308E+008
Φ_ten (7a) = 78.48338
error of function (7a) = 0.00010055
My_com (8b) = 3.4649E+008
Φ_com (7b) = 70.96935
error of function (7b) = -0.00051806
with ey = 0.00222222
eco = 0.002
apl = 0.35 ((9a) in Biskinis and Fardis for no FRP Wrap)
d1 = 44.00
R = 200.00
v = 0.00189786
N = 4769.841
Ac = 125663.706
ω = 0.54
with fc = 20.00

Calculation of ratio lb/ld

Adequate Lap Length: lb/ld >= 1

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

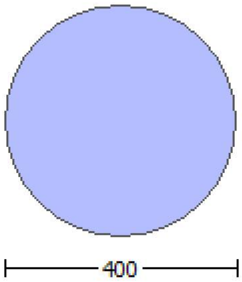
Limit State: Life Safety (data interpolation between analysis steps 1 and 2)

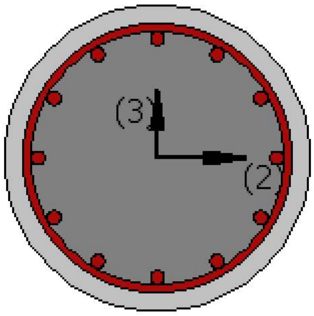
Analysis: Uniform +X

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 = 1.00
Mean strength values are used for both shear and moment calculations.
Consequently:
Existing material of Primary Member: Concrete Strength, fc = fcm = 20.00
Existing material of Primary Member: Steel Strength, fs = fsm = 444.4444
Concrete Elasticity, Ec = 21019.039
Steel Elasticity, Es = 200000.00
##
Note: Especially for the calculation of moment strengths,
the above steel re-bar strengths are multiplied by 1.25 according to R18.6.5, ACI 318-14
Existing material: Steel Strength, fs = 1.25*fsm = 555.5556
##
Diameter, D = 400.00
Cover Thickness, c = 25.00
Mean Confinement Factor overall section = 1.45937
Element Length, L = 3000.00
Primary Member
Ribbed Bars
Ductile Steel
With Detailing for Earthquake Resistance (including stirrups closed at 135°)
Longitudinal Bars With Ends Lapped Starting at the End Sections
Adequate Lap Length (lo/lou,min>=1)
No FRP Wrapping

Stepwise Properties

At local axis: 3
EDGE -A-
Shear Force, Va = -6.3906440E-031
EDGE -B-
Shear Force, Vb = 6.3906440E-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 = 0.53837083
Member Controlled by Flexure (Ve/Vr < 1)
Calculation of Shear Demand from fig. R18.6.5, ACI 318-14 Ve = (Mpr1 + Mpr2)/ln = 155270.734
with
Mpr1 = Max(Mu1+ , Mu1-) = 2.3291E+008
Mu1+ = 2.3291E+008, is the ultimate moment strength at the edge 1 of the member in the actual moment direction which is defined for the static loading combination
Mu1- = 2.3291E+008, is the ultimate moment strength at the edge 1 of the member in the opposite moment direction which is defined for the static loading combination
Mpr2 = Max(Mu2+ , Mu2-) = 2.3291E+008
Mu2+ = 2.3291E+008, is the ultimate moment strength at the edge 2 of the member in the actual moment direction which is defined for the the static loading combination
Mu2- = 2.3291E+008, is the ultimate moment strength at the edge 2 of the member in the opposite moment direction which is defined for the the static loading combination

----
Calculation of Mu1+

Calculation of ultimate Moment Strength ((3.67), Biskinis Phd), Mu
Mu = 2.3291E+008

φ = 1.11701
φ' = 0.98759739
error of function (3.68), Biskinis Phd = 59859.019
From 5A.2, TBDY: fcc = fc*λc = 29.18743
conf. factor λc = 1.45937
fc = 20.00
From 10.3.5, ASCE41-17, Final value of fy: fy*Min(1,1.25*(lb/ld)^2/3) = 555.5556
lb/ld = 1.00
d1 = 44.00
R = 200.00
v = 0.00189786
N = 4771.233
Ac = 125663.706
ω = ω*Min(1,1.25*(lb/ld)^2/3) = 0.54

Calculation of ratio lb/ld

Adequate Lap Length: lb/ld >= 1

Calculation of Mu1-

Calculation of ultimate Moment Strength ((3.67), Biskinis Phd), Mu
Mu = 2.3291E+008

φ = 1.11701
φ' = 0.98759739
error of function (3.68), Biskinis Phd = 59859.019
From 5A.2, TBDY: fcc = fc*λc = 29.18743
conf. factor λc = 1.45937
fc = 20.00
From 10.3.5, ASCE41-17, Final value of fy: fy*Min(1,1.25*(lb/ld)^2/3) = 555.5556
lb/ld = 1.00
d1 = 44.00
R = 200.00
v = 0.00189786
N = 4771.233
Ac = 125663.706
ω = ω*Min(1,1.25*(lb/ld)^2/3) = 0.54

Calculation of ratio lb/ld

Adequate Lap Length: lb/ld >= 1

Calculation of Mu2+

Calculation of ultimate Moment Strength ((3.67), Biskinis Phd), Mu
Mu = 2.3291E+008

φ = 1.11701
φ' = 0.98759739
error of function (3.68), Biskinis Phd = 59859.019
From 5A.2, TBDY: fcc = fc*λc = 29.18743
conf. factor λc = 1.45937
fc = 20.00
From 10.3.5, ASCE41-17, Final value of fy: fy*Min(1,1.25*(lb/ld)^2/3) = 555.5556
lb/ld = 1.00
d1 = 44.00
R = 200.00
v = 0.00189786
N = 4771.233
Ac = 125663.706
ω = ω*Min(1,1.25*(lb/ld)^2/3) = 0.54

Calculation of ratio lb/ld

Adequate Lap Length: lb/ld >= 1

Calculation of Mu2-

Calculation of ultimate Moment Strength ((3.67), Biskinis Phd), Mu
Mu = 2.3291E+008

φ = 1.11701
φ' = 0.98759739
error of function (3.68), Biskinis Phd = 59859.019
From 5A.2, TBDY: fcc = fc*λc = 29.18743
conf. factor λc = 1.45937
fc = 20.00
From 10.3.5, ASCE41-17, Final value of fy: fy*Min(1,1.25*(lb/ld)^2/3) = 555.5556
lb/ld = 1.00
d1 = 44.00
R = 200.00
v = 0.00189786
N = 4771.233
Ac = 125663.706
ω = ω*Min(1,1.25*(lb/ld)^2/3) = 0.54

Calculation of ratio lb/ld

Adequate Lap Length: lb/ld >= 1

Calculation of Shear Strength Vr = Min(Vr1,Vr2) = 288408.521

Calculation of Shear Strength at edge 1, Vr1 = 288408.521
Vr1 = VCol ((10.3), ASCE 41-17) = knl*VCol0
VCol0 = 288408.521
knl = 1 (zero step-static loading)

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

λ = 1 (normal-weight concrete)
fc' = 20.00, but fc^0.5 <= 8.3 MPa (22.5.3.1, ACI 318-14)
M/Vd = 2.00
Mu = 1.4857213E-011
Vu = 6.3906440E-031
d = 0.8*D = 320.00
Nu = 4771.233
Ag = 125663.706
From (11.5.4.8), ACI 318-14: Vs = 175459.634
Av = π/2*A_stirrup = 123370.055
fy = 444.4444
s = 100.00
Vs is multiplied by αCol = 0.00
s/d = 0.3125
Vf ((11-3)-(11.4), ACI 440) = 0.00
From (11-11), ACI 440: Vs + Vf <= 238930.50
bw*d = π*d*d/4 = 80424.772

Calculation of Shear Strength at edge 2, Vr2 = 288408.521
Vr2 = VCol ((10.3), ASCE 41-17) = knl*VCol0
VCol0 = 288408.521
knl = 1 (zero step-static loading)

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

λ = 1 (normal-weight concrete)
fc' = 20.00, but fc^0.5 <= 8.3 MPa (22.5.3.1, ACI 318-14)
M/Vd = 2.00
Mu = 1.4857213E-011
Vu = 6.3906440E-031
d = 0.8*D = 320.00
Nu = 4771.233
Ag = 125663.706
From (11.5.4.8), ACI 318-14: Vs = 175459.634
Av = π/2*A_stirrup = 123370.055
fy = 444.4444
s = 100.00
Vs is multiplied by αCol = 0.00
s/d = 0.3125
Vf ((11-3)-(11.4), ACI 440) = 0.00
From (11-11), ACI 440: Vs + Vf <= 238930.50
bw*d = π*d*d/4 = 80424.772

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

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

Constant Properties

Knowledge Factor, κ = 1.00
Mean strength values are used for both shear and moment calculations.
Consequently:
Existing material of Primary Member: Concrete Strength, fc = fcm = 20.00
Existing material of Primary Member: Steel Strength, fs = fsm = 444.4444
Concrete Elasticity, Ec = 21019.039
Steel Elasticity, Es = 200000.00
###
Note: Especially for the calculation of moment strengths,
the above steel re-bar strengths are multiplied by 1.25 according to R18.6.5, ACI 318-14
Existing material: Steel Strength, fs = 1.25*fsm = 555.5556
###
Diameter, D = 400.00
Cover Thickness, c = 25.00
Mean Confinement Factor overall section = 1.45937
Element Length, L = 3000.00
Primary Member
Ribbed Bars
Ductile Steel
With Detailing for Earthquake Resistance (including stirrups closed at 135°)
Longitudinal Bars With Ends Lapped Starting at the End Sections
Adequate Lap Length (lo/lou,min>=1)
No FRP Wrapping

Stepwise Properties

At local axis: 2
EDGE -A-
Shear Force, Va = 3.9130116E-047
EDGE -B-
Shear Force, Vb = -3.9130116E-047
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 = 0.53837083
Member Controlled by Flexure (Ve/Vr < 1)
Calculation of Shear Demand from fig. R18.6.5, ACI 318-14 Ve = (Mpr1 + Mpr2)/ln = 155270.734
with
Mpr1 = Max(Mu1+ , Mu1-) = 2.3291E+008
Mu1+ = 2.3291E+008, is the ultimate moment strength at the edge 1 of the member in the actual moment direction which is defined for the static loading combination
Mu1- = 2.3291E+008, is the ultimate moment strength at the edge 1 of the member in the opposite moment direction which is defined for the static loading combination
Mpr2 = Max(Mu2+ , Mu2-) = 2.3291E+008
Mu2+ = 2.3291E+008, is the ultimate moment strength at the edge 2 of the member in the actual moment direction which is defined for the the static loading combination
Mu2- = 2.3291E+008, is the ultimate moment strength at the edge 2 of the member in the opposite moment direction which is defined for the the static loading combination

----
Calculation of Mu1+

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

Mu = 2.3291E+008

φ = 1.11701
φ' = 0.98759739
error of function (3.68), Biskinis Phd = 59859.019
From 5A.2, TBDY: fcc = fc*λc = 29.18743
conf. factor λc = 1.45937
fc = 20.00
From 10.3.5, ASCE41-17, Final value of fy: fy*Min(1,1.25*(lb/ld)^2/3) = 555.5556
lb/ld = 1.00
d1 = 44.00
R = 200.00
v = 0.00189786
N = 4771.233
Ac = 125663.706
ω = ω*Min(1,1.25*(lb/ld)^2/3) = 0.54

Calculation of ratio lb/ld

Adequate Lap Length: lb/ld >= 1

Calculation of Mu1-

Calculation of ultimate Moment Strength ((3.67), Biskinis Phd), Mu
Mu = 2.3291E+008

φ = 1.11701
φ' = 0.98759739
error of function (3.68), Biskinis Phd = 59859.019
From 5A.2, TBDY: fcc = fc*λc = 29.18743
conf. factor λc = 1.45937
fc = 20.00
From 10.3.5, ASCE41-17, Final value of fy: fy*Min(1,1.25*(lb/ld)^2/3) = 555.5556
lb/ld = 1.00
d1 = 44.00
R = 200.00
v = 0.00189786
N = 4771.233
Ac = 125663.706
ω = ω*Min(1,1.25*(lb/ld)^2/3) = 0.54

Calculation of ratio lb/ld

Adequate Lap Length: lb/ld >= 1

Calculation of Mu2+

Calculation of ultimate Moment Strength ((3.67), Biskinis Phd), Mu
Mu = 2.3291E+008

φ = 1.11701
φ' = 0.98759739
error of function (3.68), Biskinis Phd = 59859.019
From 5A.2, TBDY: fcc = fc*λc = 29.18743
conf. factor λc = 1.45937
fc = 20.00
From 10.3.5, ASCE41-17, Final value of fy: fy*Min(1,1.25*(lb/ld)^2/3) = 555.5556

lb/ld = 1.00
d1 = 44.00
R = 200.00
v = 0.00189786
N = 4771.233
Ac = 125663.706
ω = ω*Min(1,1.25*(lb/ld)^2/3) = 0.54

Calculation of ratio lb/ld

Adequate Lap Length: lb/ld >= 1

Calculation of Mu2-

Calculation of ultimate Moment Strength ((3.67), Biskinis Phd), Mu
Mu = 2.3291E+008

φ = 1.11701
φ' = 0.98759739
error of function (3.68), Biskinis Phd = 59859.019
From 5A.2, TBDY: fcc = fc*λc = 29.18743
conf. factor λc = 1.45937
fc = 20.00
From 10.3.5, ASCE41-17, Final value of fy: fy*Min(1,1.25*(lb/ld)^2/3) = 555.5556
lb/ld = 1.00
d1 = 44.00
R = 200.00
v = 0.00189786
N = 4771.233
Ac = 125663.706
ω = ω*Min(1,1.25*(lb/ld)^2/3) = 0.54

Calculation of ratio lb/ld

Adequate Lap Length: lb/ld >= 1

Calculation of Shear Strength Vr = Min(Vr1,Vr2) = 288408.521

Calculation of Shear Strength at edge 1, Vr1 = 288408.521
Vr1 = VCol ((10.3), ASCE 41-17) = knl*VCol0
VCol0 = 288408.521
knl = 1 (zero step-static loading)

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

λ = 1 (normal-weight concrete)
fc' = 20.00, but fc^0.5 <= 8.3 MPa (22.5.3.1, ACI 318-14)
M/Vd = 2.00
Mu = 7.3393178E-012
Vu = 3.9130116E-047
d = 0.8*D = 320.00
Nu = 4771.233
Ag = 125663.706
From (11.5.4.8), ACI 318-14: Vs = 175459.634
Av = π/2*A_stirrup = 123370.055
fy = 444.4444
s = 100.00

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

Calculation of Shear Strength at edge 2, Vr2 = 288408.521
Vr2 = VCol ((10.3), ASCE 41-17) = knl*VCol0
VCol0 = 288408.521
knl = 1 (zero step-static loading)

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

λ = 1 (normal-weight concrete)
fc' = 20.00, but fc^0.5 <= 8.3 MPa (22.5.3.1, ACI 318-14)
M/Vd = 2.00
Mu = 7.3393178E-012
Vu = 3.9130116E-047
d = 0.8*D = 320.00
Nu = 4771.233
Ag = 125663.706
From (11.5.4.8), ACI 318-14: Vs = 175459.634
Av = π/2*A_stirrup = 123370.055
fy = 444.4444
s = 100.00
Vs is multiplied by αCol = 0.00
s/d = 0.3125
Vf ((11-3)-(11.4), ACI 440) = 0.00
From (11-11), ACI 440: Vs + Vf <= 238930.50
bw*d = π*d*d/4 = 80424.772

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

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

Constant Properties

Knowledge Factor, κ = 1.00
Chord Rotation is generally considered as Deformation-Controlled Action according to Table C7-1, ASCE41-17.
Mean strengths are used for Deformation-Controlled Actions according to 7.5.1.3, ASCE 41-17
Consequently:
Existing material of Primary Member: Concrete Strength, fc = fcm = 20.00
Existing material of Primary Member: Steel Strength, fs = fsm = 444.4444
Concrete Elasticity, Ec = 21019.039
Steel Elasticity, Es = 200000.00
Diameter, D = 400.00
Cover Thickness, c = 25.00
Element Length, L = 3000.00
Primary Member
Ribbed Bars
Ductile Steel
With Detailing for Earthquake Resistance (including stirrups closed at 135°)
Longitudinal Bars With Ends Lapped Starting at the End Sections
Adequate Lap Length (lb/ld>=1)
No FRP Wrapping

Stepwise Properties

Bending Moment, M = -1.0433E+007
Shear Force, V2 = -3476.079
Shear Force, V3 = -1.2869988E-013
Axial Force, F = -4769.841
Longitudinal Reinforcement Area Distribution (in 2 divisions)
-Tension: Aslt = 1272.345
-Compression: Aslc = 1781.283
Longitudinal Reinforcement Area Distribution (in 3 divisions)
-Tension: Asl,ten = 1017.876
-Compression: Asl,com = 1017.876
-Middle: Asl,mid = 1017.876
Mean Diameter of Tension Reinforcement, DbL = 18.00

Existing component: From table 7-7, ASCE 41_17: Final chord rotation Capacity θu,R = κ*θu = 0.07670022
θu = θy + θp = 0.07670022

- Calculation of θy -

θy = (My*Ls/3)/EIeff = 0.0294287 ((4.29),Biskinis Phd))
My = 2.3308E+008
Ls = M/V (with Ls >0.1*L and Ls < 2*L) = 3001.447
From table 10.5, ASCE 41_17: EIeff = factor*Ec*Ig = 7.9240E+012
factor = 0.30
Ag = 125663.706
fc' = 20.00
N = 4769.841
Ec*Ig = 2.6413E+013

Calculation of Yielding Moment My

Calculation of Φy and My according to (7) - (8) in Biskinis and Fardis
----
My = Min(My_ten,My_com) = 2.3308E+008
Φy ((10a) or (10b)) = 1.3315992E-005
My_ten (8a) = 2.3308E+008
Φ_ten (7a) = 78.48338
error of function (7a) = 0.00010055
My_com (8b) = 3.4649E+008
Φ_com (7b) = 70.96935
error of function (7b) = -0.00051806
with ey = 0.00222222
eco = 0.002
apl = 0.35 ((9a) in Biskinis and Fardis for no FRP Wrap)
d1 = 44.00
R = 200.00
v = 0.00189786
N = 4769.841
Ac = 125663.706
ω = 0.54
with fc = 20.00

Calculation of ratio lb/ld

Adequate Lap Length: lb/ld >= 1

- Calculation of θp -

From table 10-9: θp = 0.04727152
with:
- Columns controlled by inadequate development or splicing along the clear height because lb/ld < 1
  shear control ratio VyE/VCol0E = 0.53837083
  d = 707.00
  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 = 4769.841
    Ag = 125663.706
    f'cE = 20.00
    fytE = fylE = 444.4444
pl = Area_Tot_Long_Rein/(Ag) = 0.0243
    f'cE = 20.00

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

Calculation No. 13

column C1, Floor 1

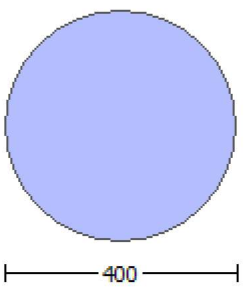
Limit State: Life Safety (data interpolation between analysis steps 1 and 2)

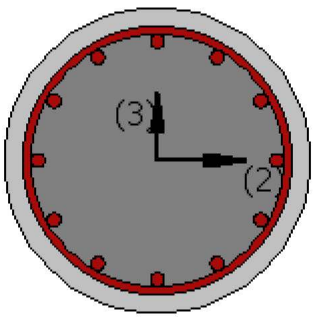
Analysis: Uniform +X

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 = 1.00
Member Shear Force is generally considered as Force-Controlled Action according to Table C7-1, ASCE41-17.
Lower-bound strengths are used for Force-Controlled Actions according to 7.5.1.3, ASCE 41-17
Consequently:
Existing material of Primary Member: Concrete Strength, fc = fc_lower_bound = 16.00
Existing material of Primary Member: Steel Strength, fs = fs_lower_bound = 400.00
Concrete Elasticity, Ec = 21019.039
Steel Elasticity, Es = 200000.00
##
Note: Especially for the calculation of θy for displacement ductility demand,
the expected (mean value) strengths are used (7.5.1.3, ASCE41-17) because bending is considered as
Deformation-Controlled Action (Table C7-1, ASCE41-17).
Existing material: Concrete Strength, fc = fcm = 20.00
Existing material: Steel Strength, fs = fsm = 444.4444
##
Diameter, D = 400.00
Cover Thickness, c = 25.00
Element Length, L = 3000.00
Primary Member
Ribbed Bars
Ductile Steel
With Detailing for Earthquake Resistance (including stirrups closed at 135°)
Longitudinal Bars With Ends Lapped Starting at the End Sections
Adequate Lap Length (lo/lou,min = lb/ld >=1)
No FRP Wrapping

Stepwise Properties

EDGE -A-
Bending Moment, Ma = -1.0433E+007
Shear Force, Va = -3476.079
EDGE -B-
Bending Moment, Mb = 0.0510701
Shear Force, Vb = 3476.079
BOTH EDGES
Axial Force, F = -4769.841
Longitudinal Reinforcement Area Distribution (in 2 divisions)
-Tension: Aslt = 0.00
-Compression: Aslc = 3053.628
Longitudinal Reinforcement Area Distribution (in 3 divisions)
-Tension: Asl,ten = 1017.876
-Compression: Asl,com = 1017.876
-Middle: Asl,mid = 1017.876
Mean Diameter of Tension Reinforcement, DbL,ten = 18.00

Existing component: From table 7-7, ASCE 41_17: Final Shear Capacity VR = κ*Vn = 259037.796
Vn ((10.3), ASCE 41-17) = knl*VCol0 = 259037.796
VCol = 259037.796
knl = 1.00
displacement_ductility_demand = 0.06713695

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

λ = 1 (normal-weight concrete)
fc' = 16.00, but fc^0.5 <= 8.3 MPa (22.5.3.1, ACI 318-14)
M/Vd = 2.00
Mu = 0.0510701
Vu = 3476.079
d = 0.8*D = 320.00
Nu = 4769.841

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

----
displacement_ductility_demand is calculated as θ/θy

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

From analysis, chord rotation θ = 0.00019748
θy = (My*Ls/3)/Eleff = 0.00294145 ((4.29),Biskinis Phd))
My = 2.3308E+008
Ls = M/V (with Ls >0.1*L and Ls < 2*L) = 300.00
From table 10.5, ASCE 41_17: Eleff = factor*Ec*Ig = 7.9240E+012
factor = 0.30
Ag = 125663.706
fc' = 20.00
N = 4769.841
Ec*Ig = 2.6413E+013

Calculation of Yielding Moment My

Calculation of Φy and My according to (7) - (8) in Biskinis and Fardis
----
My = Min(My_ten,My_com) = 2.3308E+008
Φy ((10a) or (10b)) = 1.3315992E-005
My_ten (8a) = 2.3308E+008
Φ_ten (7a) = 78.48338
error of function (7a) = 0.00010055
My_com (8b) = 3.4649E+008
Φ_com (7b) = 70.96935
error of function (7b) = -0.00051806
with ey = 0.00222222
eco = 0.002
apl = 0.35 ((9a) in Biskinis and Fardis for no FRP Wrap)
d1 = 44.00
R = 200.00
v = 0.00189786
N = 4769.841
Ac = 125663.706
ω = 0.54
with fc = 20.00

Calculation of ratio lb/ld

Adequate Lap Length: lb/ld >= 1

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

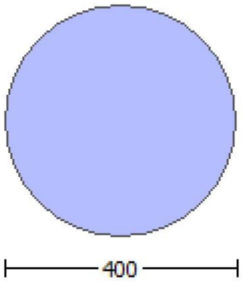
Limit State: Life Safety (data interpolation between analysis steps 1 and 2)

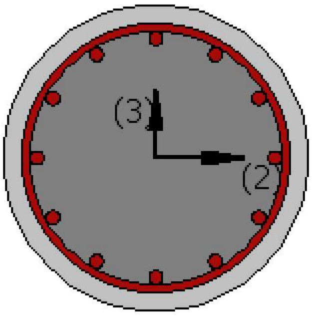
Analysis: Uniform +X

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 = 1.00
Mean strength values are used for both shear and moment calculations.
Consequently:
Existing material of Primary Member: Concrete Strength, fc = fcm = 20.00
Existing material of Primary Member: Steel Strength, fs = fsm = 444.4444
Concrete Elasticity, Ec = 21019.039
Steel Elasticity, Es = 200000.00
##
Note: Especially for the calculation of moment strengths,
the above steel re-bar strengths are multiplied by 1.25 according to R18.6.5, ACI 318-14
Existing material: Steel Strength, fs = 1.25*fsm = 555.5556
##
Diameter, D = 400.00
Cover Thickness, c = 25.00
Mean Confinement Factor overall section = 1.45937
Element Length, L = 3000.00
Primary Member
Ribbed Bars
Ductile Steel
With Detailing for Earthquake Resistance (including stirrups closed at 135°)
Longitudinal Bars With Ends Lapped Starting at the End Sections
Adequate Lap Length (lo/lou,min>=1)
No FRP Wrapping

Stepwise Properties

At local axis: 3
EDGE -A-
Shear Force, Va = -6.3906440E-031
EDGE -B-
Shear Force, Vb = 6.3906440E-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 = 0.53837083
Member Controlled by Flexure (Ve/Vr < 1)
Calculation of Shear Demand from fig. R18.6.5, ACI 318-14 Ve = (Mpr1 + Mpr2)/ln = 155270.734
with
Mpr1 = Max(Mu1+ , Mu1-) = 2.3291E+008
Mu1+ = 2.3291E+008, is the ultimate moment strength at the edge 1 of the member in the actual moment direction which is defined for the static loading combination
Mu1- = 2.3291E+008, is the ultimate moment strength at the edge 1 of the member in the opposite moment direction which is defined for the static loading combination
Mpr2 = Max(Mu2+ , Mu2-) = 2.3291E+008
Mu2+ = 2.3291E+008, is the ultimate moment strength at the edge 2 of the member in the actual moment direction which is defined for the the static loading combination
Mu2- = 2.3291E+008, is the ultimate moment strength at the edge 2 of the member in the opposite moment direction which is defined for the the static loading combination

----
Calculation of Mu1+

Calculation of ultimate Moment Strength ((3.67), Biskinis Phd), Mu
Mu = 2.3291E+008

φ = 1.11701
φ' = 0.98759739
error of function (3.68), Biskinis Phd = 59859.019
From 5A.2, TBDY: fcc = fc*λc = 29.18743
conf. factor λc = 1.45937
fc = 20.00
From 10.3.5, ASCE41-17, Final value of fy: fy*Min(1,1.25*(lb/ld)^2/3) = 555.5556
lb/ld = 1.00
d1 = 44.00
R = 200.00
v = 0.00189786
N = 4771.233
Ac = 125663.706
ω = ω*Min(1,1.25*(lb/ld)^2/3) = 0.54

Calculation of ratio lb/ld

Adequate Lap Length: lb/ld >= 1

Calculation of Mu1-

Calculation of ultimate Moment Strength ((3.67), Biskinis Phd), Mu
Mu = 2.3291E+008

φ = 1.11701
φ' = 0.98759739
error of function (3.68), Biskinis Phd = 59859.019
From 5A.2, TBDY: fcc = fc*λc = 29.18743
conf. factor λc = 1.45937
fc = 20.00
From 10.3.5, ASCE41-17, Final value of fy: fy*Min(1,1.25*(lb/ld)^2/3) = 555.5556
lb/ld = 1.00
d1 = 44.00
R = 200.00
v = 0.00189786
N = 4771.233
Ac = 125663.706
ω = ω*Min(1,1.25*(lb/ld)^2/3) = 0.54

Calculation of ratio lb/ld

Adequate Lap Length: lb/ld >= 1

Calculation of Mu2+

Calculation of ultimate Moment Strength ((3.67), Biskinis Phd), Mu
Mu = 2.3291E+008

φ = 1.11701
φ' = 0.98759739
error of function (3.68), Biskinis Phd = 59859.019
From 5A.2, TBDY: fcc = fc*λc = 29.18743
conf. factor λc = 1.45937
fc = 20.00
From 10.3.5, ASCE41-17, Final value of fy: fy*Min(1,1.25*(lb/ld)^2/3) = 555.5556
lb/ld = 1.00
d1 = 44.00
R = 200.00
v = 0.00189786
N = 4771.233
Ac = 125663.706
ω = ω*Min(1,1.25*(lb/ld)^2/3) = 0.54

Calculation of ratio lb/ld

Adequate Lap Length: lb/ld >= 1

Calculation of Mu2-

Calculation of ultimate Moment Strength ((3.67), Biskinis Phd), Mu
Mu = 2.3291E+008

φ = 1.11701
φ' = 0.98759739
error of function (3.68), Biskinis Phd = 59859.019
From 5A.2, TBDY: fcc = fc*λc = 29.18743

conf. factor λc = 1.45937
fc = 20.00
From 10.3.5, ASCE41-17, Final value of fy: fy*Min(1,1.25*(lb/ld)^2/3) = 555.5556
lb/ld = 1.00
d1 = 44.00
R = 200.00
v = 0.00189786
N = 4771.233
Ac = 125663.706
ω = ω*Min(1,1.25*(lb/ld)^2/3) = 0.54

Calculation of ratio lb/ld

Adequate Lap Length: lb/ld >= 1

Calculation of Shear Strength Vr = Min(Vr1,Vr2) = 288408.521

Calculation of Shear Strength at edge 1, Vr1 = 288408.521
Vr1 = VCol ((10.3), ASCE 41-17) = knl*VCol0
VCol0 = 288408.521
knl = 1 (zero step-static loading)

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

λ = 1 (normal-weight concrete)
fc' = 20.00, but fc^0.5 <= 8.3 MPa (22.5.3.1, ACI 318-14)
M/Vd = 2.00
Mu = 1.4857213E-011
Vu = 6.3906440E-031
d = 0.8*D = 320.00
Nu = 4771.233
Ag = 125663.706
From (11.5.4.8), ACI 318-14: Vs = 175459.634
Av = π/2*A_stirrup = 123370.055
fy = 444.4444
s = 100.00
Vs is multiplied by αCol = 0.00
s/d = 0.3125
Vf ((11-3)-(11.4), ACI 440) = 0.00
From (11-11), ACI 440: Vs + Vf <= 238930.50
bw*d = π*d*d/4 = 80424.772

Calculation of Shear Strength at edge 2, Vr2 = 288408.521
Vr2 = VCol ((10.3), ASCE 41-17) = knl*VCol0
VCol0 = 288408.521
knl = 1 (zero step-static loading)

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

λ = 1 (normal-weight concrete)
fc' = 20.00, but fc^0.5 <= 8.3 MPa (22.5.3.1, ACI 318-14)
M/Vd = 2.00
Mu = 1.4857213E-011
Vu = 6.3906440E-031
d = 0.8*D = 320.00
Nu = 4771.233
Ag = 125663.706
From (11.5.4.8), ACI 318-14: Vs = 175459.634
Av = π/2*A_stirrup = 123370.055
fy = 444.4444

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

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

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

Constant Properties

Knowledge Factor, κ = 1.00
Mean strength values are used for both shear and moment calculations.
Consequently:
Existing material of Primary Member: Concrete Strength, fc = fcm = 20.00
Existing material of Primary Member: Steel Strength, fs = fsm = 444.4444
Concrete Elasticity, Ec = 21019.039
Steel Elasticity, Es = 200000.00
###
Note: Especially for the calculation of moment strengths,
the above steel re-bar strengths are multiplied by 1.25 according to R18.6.5, ACI 318-14
Existing material: Steel Strength, fs = 1.25*fsm = 555.5556
###
Diameter, D = 400.00
Cover Thickness, c = 25.00
Mean Confinement Factor overall section = 1.45937
Element Length, L = 3000.00
Primary Member
Ribbed Bars
Ductile Steel
With Detailing for Earthquake Resistance (including stirrups closed at 135°)
Longitudinal Bars With Ends Lapped Starting at the End Sections
Adequate Lap Length (lo/lou,min>=1)
No FRP Wrapping

Stepwise Properties

At local axis: 2
EDGE -A-
Shear Force, Va = 3.9130116E-047
EDGE -B-
Shear Force, Vb = -3.9130116E-047
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 = 0.53837083

Member Controlled by Flexure (Ve/Vr < 1)
Calculation of Shear Demand from fig. R18.6.5, ACI 318-14 Ve = (Mpr1 + Mpr2)/ln = 155270.734
with
Mpr1 = Max(Mu1+ , Mu1-) = 2.3291E+008
Mu1+ = 2.3291E+008, is the ultimate moment strength at the edge 1 of the member in the actual moment direction which is defined for the static loading combination
Mu1- = 2.3291E+008, is the ultimate moment strength at the edge 1 of the member in the opposite moment direction which is defined for the static loading combination
Mpr2 = Max(Mu2+ , Mu2-) = 2.3291E+008
Mu2+ = 2.3291E+008, is the ultimate moment strength at the edge 2 of the member in the actual moment direction which is defined for the the static loading combination
Mu2- = 2.3291E+008, is the ultimate moment strength at the edge 2 of the member in the opposite moment direction which is defined for the the static loading combination

----
Calculation of Mu1+

Calculation of ultimate Moment Strength ((3.67), Biskinis Phd), Mu
Mu = 2.3291E+008

φ = 1.11701
φ' = 0.98759739
error of function (3.68), Biskinis Phd = 59859.019
From 5A.2, TBDY: fcc = fc*λc = 29.18743
conf. factor λc = 1.45937
fc = 20.00
From 10.3.5, ASCE41-17, Final value of fy: fy*Min(1,1.25*(lb/ld)^2/3) = 555.5556
lb/ld = 1.00
d1 = 44.00
R = 200.00
v = 0.00189786
N = 4771.233
Ac = 125663.706
ω = ω*Min(1,1.25*(lb/ld)^2/3) = 0.54

Calculation of ratio lb/ld

Adequate Lap Length: lb/ld >= 1

Calculation of Mu1-

Calculation of ultimate Moment Strength ((3.67), Biskinis Phd), Mu
Mu = 2.3291E+008

φ = 1.11701
φ' = 0.98759739
error of function (3.68), Biskinis Phd = 59859.019
From 5A.2, TBDY: fcc = fc*λc = 29.18743
conf. factor λc = 1.45937
fc = 20.00
From 10.3.5, ASCE41-17, Final value of fy: fy*Min(1,1.25*(lb/ld)^2/3) = 555.5556
lb/ld = 1.00
d1 = 44.00
R = 200.00
v = 0.00189786
N = 4771.233
Ac = 125663.706
ω = ω*Min(1,1.25*(lb/ld)^2/3) = 0.54

Calculation of ratio lb/ld

Adequate Lap Length: lb/ld >= 1

Calculation of Mu2+

Calculation of ultimate Moment Strength ((3.67), Biskinis Phd), Mu
Mu = 2.3291E+008

φ = 1.11701
φ' = 0.98759739
error of function (3.68), Biskinis Phd = 59859.019
From 5A.2, TBDY: fcc = fc*λc = 29.18743
conf. factor λc = 1.45937
fc = 20.00
From 10.3.5, ASCE41-17, Final value of fy: fy*Min(1,1.25*(lb/ld)^2/3) = 555.5556
lb/ld = 1.00
d1 = 44.00
R = 200.00
v = 0.00189786
N = 4771.233
Ac = 125663.706
ω = ω*Min(1,1.25*(lb/ld)^2/3) = 0.54

Calculation of ratio lb/ld

Adequate Lap Length: lb/ld >= 1

Calculation of Mu2-

Calculation of ultimate Moment Strength ((3.67), Biskinis Phd), Mu
Mu = 2.3291E+008

φ = 1.11701
φ' = 0.98759739
error of function (3.68), Biskinis Phd = 59859.019
From 5A.2, TBDY: fcc = fc*λc = 29.18743
conf. factor λc = 1.45937
fc = 20.00
From 10.3.5, ASCE41-17, Final value of fy: fy*Min(1,1.25*(lb/ld)^2/3) = 555.5556
lb/ld = 1.00
d1 = 44.00
R = 200.00
v = 0.00189786
N = 4771.233
Ac = 125663.706
ω = ω*Min(1,1.25*(lb/ld)^2/3) = 0.54

Calculation of ratio lb/ld

Adequate Lap Length: lb/ld >= 1

Calculation of Shear Strength Vr = Min(Vr1,Vr2) = 288408.521

Calculation of Shear Strength at edge 1, Vr1 = 288408.521
Vr1 = VCol ((10.3), ASCE 41-17) = knl*VCol0
VCol0 = 288408.521
knl = 1 (zero step-static loading)

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

λ = 1 (normal-weight concrete)
fc' = 20.00, but fc^0.5 <= 8.3 MPa (22.5.3.1, ACI 318-14)
M/Vd = 2.00
Mu = 7.3393178E-012
Vu = 3.9130116E-047
d = 0.8*D = 320.00
Nu = 4771.233
Ag = 125663.706
From (11.5.4.8), ACI 318-14: Vs = 175459.634
Av = π/2*A_stirrup = 123370.055
fy = 444.4444
s = 100.00
Vs is multiplied by αCol = 0.00
s/d = 0.3125
Vf ((11-3)-(11.4), ACI 440) = 0.00
From (11-11), ACI 440: Vs + Vf <= 238930.50
bw*d = π*d*d/4 = 80424.772

Calculation of Shear Strength at edge 2, Vr2 = 288408.521
Vr2 = VCol ((10.3), ASCE 41-17) = knl*VCol0
VCol0 = 288408.521
knl = 1 (zero step-static loading)

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

λ = 1 (normal-weight concrete)
fc' = 20.00, but fc^0.5 <= 8.3 MPa (22.5.3.1, ACI 318-14)
M/Vd = 2.00
Mu = 7.3393178E-012
Vu = 3.9130116E-047
d = 0.8*D = 320.00
Nu = 4771.233
Ag = 125663.706
From (11.5.4.8), ACI 318-14: Vs = 175459.634
Av = π/2*A_stirrup = 123370.055
fy = 444.4444
s = 100.00
Vs is multiplied by αCol = 0.00
s/d = 0.3125
Vf ((11-3)-(11.4), ACI 440) = 0.00
From (11-11), ACI 440: Vs + Vf <= 238930.50
bw*d = π*d*d/4 = 80424.772

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

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

Constant Properties

Knowledge Factor, κ = 1.00
Chord Rotation is generally considered as Deformation-Controlled Action according to Table C7-1, ASCE41-17.
Mean strengths are used for Deformation-Controlled Actions according to 7.5.1.3, ASCE 41-17
Consequently:
Existing material of Primary Member: Concrete Strength, fc = fcm = 20.00
Existing material of Primary Member: Steel Strength, fs = fsm = 444.4444
Concrete Elasticity, Ec = 21019.039
Steel Elasticity, Es = 200000.00
Diameter, D = 400.00
Cover Thickness, c = 25.00
Element Length, L = 3000.00
Primary Member
Ribbed Bars
Ductile Steel
With Detailing for Earthquake Resistance (including stirrups closed at 135°)
Longitudinal Bars With Ends Lapped Starting at the End Sections
Adequate Lap Length (lb/ld>=1)
No FRP Wrapping

Stepwise Properties

Bending Moment, M = -9.2046865E-011
Shear Force, V2 = 3476.079
Shear Force, V3 = 1.2869988E-013
Axial Force, F = -4769.841
Longitudinal Reinforcement Area Distribution (in 2 divisions)
-Tension: Aslt = 0.00
-Compression: Aslc = 3053.628
Longitudinal Reinforcement Area Distribution (in 3 divisions)
-Tension: Asl,ten = 1017.876
-Compression: Asl,com = 1017.876
-Middle: Asl,mid = 1017.876
Mean Diameter of Tension Reinforcement, DbL = 18.00

Existing component: From table 7-7, ASCE 41_17: Final chord rotation Capacity θu,R = κ*θu = 0.06197877
θu = θy + θp = 0.06197877

- Calculation of θy -

θy = (My*Ls/3)/EIeff = 0.01470726 ((4.29),Biskinis Phd))
My = 2.3308E+008
Ls = M/V (with Ls >0.1*L and Ls < 2*L) = 1500.00
From table 10.5, ASCE 41_17: EIeff = factor*Ec*Ig = 7.9240E+012
factor = 0.30
Ag = 125663.706
fc' = 20.00
N = 4769.841
Ec*Ig = 2.6413E+013

Calculation of Yielding Moment My

Calculation of Φy and My according to (7) - (8) in Biskinis and Fardis
----
My = Min(My_ten,My_com) = 2.3308E+008
Φy ((10a) or (10b)) = 1.3315992E-005
My_ten (8a) = 2.3308E+008
Φ_ten (7a) = 78.48338
error of function (7a) = 0.00010055
My_com (8b) = 3.4649E+008
Φ_com (7b) = 70.96935

error of function (7b) = -0.00051806
with ey = 0.00222222
eco = 0.002
apl = 0.35 ((9a) in Biskinis and Fardis for no FRP Wrap)
d1 = 44.00
R = 200.00
v = 0.00189786
N = 4769.841
Ac = 125663.706
ω = 0.54
with fc = 20.00

Calculation of ratio lb/ld

Adequate Lap Length: lb/ld >= 1

- Calculation of θp -

From table 10-9: θp = 0.04727152
with:
- Columns controlled by inadequate development or splicing along the clear height because lb/ld < 1
shear control ratio VyE/VCol0E = 0.53837083
d = 707.00
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 = 4769.841
Ag = 125663.706
f'cE = 20.00
fytE = fylE = 444.4444
pl = Area_Tot_Long_Rein/(Ag) = 0.0243
f'cE = 20.00

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

Calculation No. 15

column C1, Floor 1

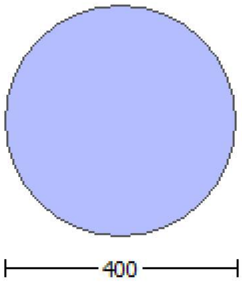
Limit State: Life Safety (data interpolation between analysis steps 1 and 2)

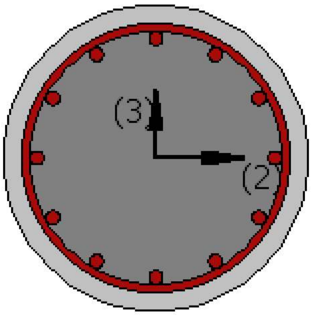
Analysis: Uniform +X

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 = 1.00
Member Shear Force is generally considered as Force-Controlled Action according to Table C7-1, ASCE41-17.
Lower-bound strengths are used for Force-Controlled Actions according to 7.5.1.3, ASCE 41-17
Consequently:
Existing material of Primary Member: Concrete Strength, fc = fc_lower_bound = 16.00
Existing material of Primary Member: Steel Strength, fs = fs_lower_bound = 400.00
Concrete Elasticity, Ec = 21019.039
Steel Elasticity, Es = 200000.00
##
Note: Especially for the calculation of θy for displacement ductility demand,
the expected (mean value) strengths are used (7.5.1.3, ASCE41-17) because bending is considered as
Deformation-Controlled Action (Table C7-1, ASCE41-17).
Existing material: Concrete Strength, fc = fcm = 20.00
Existing material: Steel Strength, fs = fsm = 444.4444
##
Diameter, D = 400.00
Cover Thickness, c = 25.00
Element Length, L = 3000.00
Primary Member
Ribbed Bars
Ductile Steel
With Detailing for Earthquake Resistance (including stirrups closed at 135°)
Longitudinal Bars With Ends Lapped Starting at the End Sections
Adequate Lap Length (lo/lou,min = lb/ld >=1)
No FRP Wrapping

Stepwise Properties

EDGE -A-
Bending Moment, Ma = 4.7847013E-010
Shear Force, Va = -1.2869988E-013
EDGE -B-
Bending Moment, Mb = -9.2046865E-011
Shear Force, Vb = 1.2869988E-013
BOTH EDGES
Axial Force, F = -4769.841
Longitudinal Reinforcement Area Distribution (in 2 divisions)
-Tension: Aslt = 0.00
-Compression: Aslc = 3053.628
Longitudinal Reinforcement Area Distribution (in 3 divisions)
-Tension: Asl,ten = 1017.876
-Compression: Asl,com = 1017.876
-Middle: Asl,mid = 1017.876
Mean Diameter of Tension Reinforcement, DbL,ten = 18.00

Existing component: From table 7-7, ASCE 41_17: Final Shear Capacity VR = κ*Vn = 259037.796
Vn ((10.3), ASCE 41-17) = knl*VCol0 = 259037.796
VCol = 259037.796
knl = 1.00
displacement_ductility_demand = 0.00

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

λ = 1 (normal-weight concrete)
fc' = 16.00, but fc^0.5 <= 8.3 MPa (22.5.3.1, ACI 318-14)
M/Vd = 2.00
Mu = 9.2046865E-011
Vu = 1.2869988E-013
d = 0.8*D = 320.00
Nu = 4769.841
Ag = 125663.706
From (11.5.4.8), ACI 318-14: Vs = 157913.67
Av = π/2*A_stirrup = 123370.055
fy = 400.00
s = 100.00
Vs is multiplied by αCol = 0.00
s/d = 0.3125
Vf ((11-3)-(11.4), ACI 440) = 0.00
From (11-11), ACI 440: Vs + Vf <= 213705.936
bw*d = π*d*d/4 = 80424.772

----
displacement_ductility_demand is calculated as θ/θy

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

From analysis, chord rotation θ = 1.1555505E-020
θy = (My*Ls/3)/Eleff = 0.01470726 ((4.29),Biskinis Phd))
My = 2.3308E+008
Ls = M/V (with Ls >0.1*L and Ls < 2*L) = 1500.00
From table 10.5, ASCE 41_17: Eleff = factor*Ec*Ig = 7.9240E+012
factor = 0.30
Ag = 125663.706
fc' = 20.00
N = 4769.841
Ec*Ig = 2.6413E+013

Calculation of Yielding Moment My

Calculation of Φy and My according to (7) - (8) in Biskinis and Fardis
----
My = Min(My_ten,My_com) = 2.3308E+008
Φy ((10a) or (10b)) = 1.3315992E-005
My_ten (8a) = 2.3308E+008
Φ_ten (7a) = 78.48338
error of function (7a) = 0.00010055
My_com (8b) = 3.4649E+008
Φ_com (7b) = 70.96935
error of function (7b) = -0.00051806
with ey = 0.00222222
eco = 0.002
apl = 0.35 ((9a) in Biskinis and Fardis for no FRP Wrap)
d1 = 44.00

R = 200.00
v = 0.00189786
N = 4769.841
Ac = 125663.706
ω = 0.54
with fc = 20.00

Calculation of ratio lb/ld

Adequate Lap Length: lb/ld >= 1

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

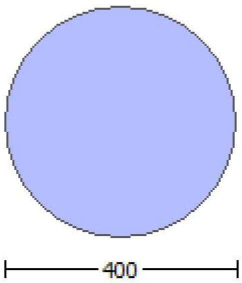
Limit State: Life Safety (data interpolation between analysis steps 1 and 2)

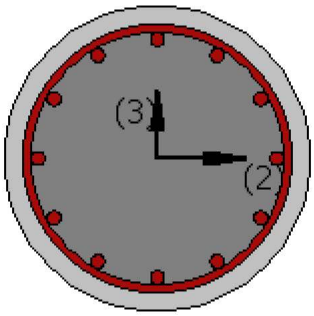
Analysis: Uniform +X

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 = 1.00
Mean strength values are used for both shear and moment calculations.
Consequently:
Existing material of Primary Member: Concrete Strength, fc = fcm = 20.00
Existing material of Primary Member: Steel Strength, fs = fsm = 444.4444
Concrete Elasticity, Ec = 21019.039

Steel Elasticity, Es = 200000.00
###
Note: Especially for the calculation of moment strengths,
the above steel re-bar strengths are multiplied by 1.25 according to R18.6.5, ACI 318-14
Existing material: Steel Strength, fs = 1.25*fsm = 555.5556
###
Diameter, D = 400.00
Cover Thickness, c = 25.00
Mean Confinement Factor overall section = 1.45937
Element Length, L = 3000.00
Primary Member
Ribbed Bars
Ductile Steel
With Detailing for Earthquake Resistance (including stirrups closed at 135°)
Longitudinal Bars With Ends Lapped Starting at the End Sections
Adequate Lap Length (lo/lou,min>=1)
No FRP Wrapping

Stepwise Properties

At local axis: 3
EDGE -A-
Shear Force, Va = -6.3906440E-031
EDGE -B-
Shear Force, Vb = 6.3906440E-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 = 0.53837083
Member Controlled by Flexure (Ve/Vr < 1)
Calculation of Shear Demand from fig. R18.6.5, ACI 318-14 Ve = (Mpr1 + Mpr2)/ln = 155270.734
with
Mpr1 = Max(Mu1+ , Mu1-) = 2.3291E+008
Mu1+ = 2.3291E+008, is the ultimate moment strength at the edge 1 of the member in the actual moment direction which is defined for the static loading combination
Mu1- = 2.3291E+008, is the ultimate moment strength at the edge 1 of the member in the opposite moment direction which is defined for the static loading combination
Mpr2 = Max(Mu2+ , Mu2-) = 2.3291E+008
Mu2+ = 2.3291E+008, is the ultimate moment strength at the edge 2 of the member in the actual moment direction which is defined for the the static loading combination
Mu2- = 2.3291E+008, is the ultimate moment strength at the edge 2 of the member in the opposite moment direction which is defined for the the static loading combination

----
Calculation of Mu1+

Calculation of ultimate Moment Strength ((3.67), Biskinis Phd), Mu
Mu = 2.3291E+008

φ = 1.11701
φ' = 0.98759739
error of function (3.68), Biskinis Phd = 59859.019
From 5A.2, TBDY: fcc = fc*λc = 29.18743
conf. factor λc = 1.45937

fc = 20.00
From 10.3.5, ASCE41-17, Final value of fy: fy*Min(1,1.25*(lb/ld)^2/3) = 555.5556
lb/ld = 1.00
d1 = 44.00
R = 200.00
v = 0.00189786
N = 4771.233
Ac = 125663.706
ω = ω*Min(1,1.25*(lb/ld)^2/3) = 0.54

Calculation of ratio lb/ld

Adequate Lap Length: lb/ld >= 1

Calculation of Mu1-

Calculation of ultimate Moment Strength ((3.67), Biskinis Phd), Mu
Mu = 2.3291E+008

φ = 1.11701
φ' = 0.98759739
error of function (3.68), Biskinis Phd = 59859.019
From 5A.2, TBDY: fcc = fc*λc = 29.18743
conf. factor λc = 1.45937
fc = 20.00
From 10.3.5, ASCE41-17, Final value of fy: fy*Min(1,1.25*(lb/ld)^2/3) = 555.5556
lb/ld = 1.00
d1 = 44.00
R = 200.00
v = 0.00189786
N = 4771.233
Ac = 125663.706
ω = ω*Min(1,1.25*(lb/ld)^2/3) = 0.54

Calculation of ratio lb/ld

Adequate Lap Length: lb/ld >= 1

Calculation of Mu2+

Calculation of ultimate Moment Strength ((3.67), Biskinis Phd), Mu
Mu = 2.3291E+008

φ = 1.11701
φ' = 0.98759739
error of function (3.68), Biskinis Phd = 59859.019
From 5A.2, TBDY: fcc = fc*λc = 29.18743
conf. factor λc = 1.45937
fc = 20.00
From 10.3.5, ASCE41-17, Final value of fy: fy*Min(1,1.25*(lb/ld)^2/3) = 555.5556
lb/ld = 1.00
d1 = 44.00
R = 200.00
v = 0.00189786
N = 4771.233
Ac = 125663.706
ω = ω*Min(1,1.25*(lb/ld)^2/3) = 0.54

Calculation of ratio lb/ld

Adequate Lap Length: lb/ld >= 1

Calculation of Mu2-

Calculation of ultimate Moment Strength ((3.67), Biskinis Phd), Mu
Mu = 2.3291E+008

φ = 1.11701
φ' = 0.98759739
error of function (3.68), Biskinis Phd = 59859.019
From 5A.2, TBDY: fcc = fc*λc = 29.18743
conf. factor λc = 1.45937
fc = 20.00
From 10.3.5, ASCE41-17, Final value of fy: fy*Min(1,1.25*(lb/ld)^2/3) = 555.5556
lb/ld = 1.00
d1 = 44.00
R = 200.00
v = 0.00189786
N = 4771.233
Ac = 125663.706
ω = ω*Min(1,1.25*(lb/ld)^2/3) = 0.54

Calculation of ratio lb/ld

Adequate Lap Length: lb/ld >= 1

Calculation of Shear Strength Vr = Min(Vr1,Vr2) = 288408.521

Calculation of Shear Strength at edge 1, Vr1 = 288408.521
Vr1 = VCol ((10.3), ASCE 41-17) = knl*VCol0
VCol0 = 288408.521
knl = 1 (zero step-static loading)

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

λ = 1 (normal-weight concrete)
fc' = 20.00, but fc^0.5 <= 8.3 MPa (22.5.3.1, ACI 318-14)
M/Vd = 2.00
Mu = 1.4857213E-011
Vu = 6.3906440E-031
d = 0.8*D = 320.00
Nu = 4771.233
Ag = 125663.706
From (11.5.4.8), ACI 318-14: Vs = 175459.634
Av = π/2*A_stirrup = 123370.055
fy = 444.4444
s = 100.00
Vs is multiplied by αCol = 0.00
s/d = 0.3125
Vf ((11-3)-(11.4), ACI 440) = 0.00
From (11-11), ACI 440: Vs + Vf <= 238930.50
bw*d = π*d*d/4 = 80424.772

Calculation of Shear Strength at edge 2, Vr2 = 288408.521
Vr2 = VCol ((10.3), ASCE 41-17) = knl*VCol0
VCol0 = 288408.521
knl = 1 (zero step-static loading)

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

λ = 1 (normal-weight concrete)
fc' = 20.00, but fc^0.5 <= 8.3 MPa (22.5.3.1, ACI 318-14)
M/Vd = 2.00
Mu = 1.4857213E-011
Vu = 6.3906440E-031
d = 0.8*D = 320.00
Nu = 4771.233
Ag = 125663.706
From (11.5.4.8), ACI 318-14: Vs = 175459.634
Av = π/2*A_stirrup = 123370.055
fy = 444.4444
s = 100.00
Vs is multiplied by αCol = 0.00
s/d = 0.3125
Vf ((11-3)-(11.4), ACI 440) = 0.00
From (11-11), ACI 440: Vs + Vf <= 238930.50
bw*d = π*d*d/4 = 80424.772

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

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

Constant Properties

Knowledge Factor, κ = 1.00
Mean strength values are used for both shear and moment calculations.
Consequently:
Existing material of Primary Member: Concrete Strength, fc = fcm = 20.00
Existing material of Primary Member: Steel Strength, fs = fsm = 444.4444
Concrete Elasticity, Ec = 21019.039
Steel Elasticity, Es = 200000.00
###
Note: Especially for the calculation of moment strengths,
the above steel re-bar strengths are multiplied by 1.25 according to R18.6.5, ACI 318-14
Existing material: Steel Strength, fs = 1.25*fsm = 555.5556
###
Diameter, D = 400.00
Cover Thickness, c = 25.00
Mean Confinement Factor overall section = 1.45937
Element Length, L = 3000.00
Primary Member
Ribbed Bars
Ductile Steel
With Detailing for Earthquake Resistance (including stirrups closed at 135°)
Longitudinal Bars With Ends Lapped Starting at the End Sections
Adequate Lap Length (lo/lou,min>=1)
No FRP Wrapping

Stepwise Properties

At local axis: 2
EDGE -A-
Shear Force, Va = 3.9130116E-047
EDGE -B-
Shear Force, Vb = -3.9130116E-047
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 = 0.53837083
Member Controlled by Flexure (Ve/Vr < 1)
Calculation of Shear Demand from fig. R18.6.5, ACI 318-14 Ve = (Mpr1 + Mpr2)/ln = 155270.734
with
Mpr1 = Max(Mu1+ , Mu1-) = 2.3291E+008
Mu1+ = 2.3291E+008, is the ultimate moment strength at the edge 1 of the member in the actual moment direction which is defined for the static loading combination
Mu1- = 2.3291E+008, is the ultimate moment strength at the edge 1 of the member in the opposite moment direction which is defined for the static loading combination
Mpr2 = Max(Mu2+ , Mu2-) = 2.3291E+008
Mu2+ = 2.3291E+008, is the ultimate moment strength at the edge 2 of the member in the actual moment direction which is defined for the the static loading combination
Mu2- = 2.3291E+008, is the ultimate moment strength at the edge 2 of the member in the opposite moment direction which is defined for the the static loading combination

----
Calculation of Mu1+

Calculation of ultimate Moment Strength ((3.67), Biskinis Phd), Mu
Mu = 2.3291E+008

φ = 1.11701
φ' = 0.98759739
error of function (3.68), Biskinis Phd = 59859.019
From 5A.2, TBDY: fcc = fc*λc = 29.18743
conf. factor λc = 1.45937
fc = 20.00
From 10.3.5, ASCE41-17, Final value of fy: fy*Min(1,1.25*(lb/ld)^2/3) = 555.5556
lb/ld = 1.00
d1 = 44.00
R = 200.00
v = 0.00189786
N = 4771.233
Ac = 125663.706
ω = ω*Min(1,1.25*(lb/ld)^2/3) = 0.54

Calculation of ratio lb/ld

Adequate Lap Length: lb/ld >= 1

Calculation of Mu1-

Calculation of ultimate Moment Strength ((3.67), Biskinis Phd), Mu
Mu = 2.3291E+008

φ = 1.11701
φ' = 0.98759739
error of function (3.68), Biskinis Phd = 59859.019
From 5A.2, TBDY: fcc = fc*λc = 29.18743
conf. factor λc = 1.45937
fc = 20.00
From 10.3.5, ASCE41-17, Final value of fy: fy*Min(1,1.25*(lb/ld)^2/3) = 555.5556
lb/ld = 1.00
d1 = 44.00
R = 200.00
v = 0.00189786
N = 4771.233
Ac = 125663.706
ω = ω*Min(1,1.25*(lb/ld)^2/3) = 0.54

Calculation of ratio lb/ld

Adequate Lap Length: lb/ld >= 1

Calculation of Mu2+

Calculation of ultimate Moment Strength ((3.67), Biskinis Phd), Mu
Mu = 2.3291E+008

φ = 1.11701
φ' = 0.98759739
error of function (3.68), Biskinis Phd = 59859.019
From 5A.2, TBDY: fcc = fc*λc = 29.18743
conf. factor λc = 1.45937
fc = 20.00
From 10.3.5, ASCE41-17, Final value of fy: fy*Min(1,1.25*(lb/ld)^2/3) = 555.5556
lb/ld = 1.00
d1 = 44.00
R = 200.00
v = 0.00189786
N = 4771.233
Ac = 125663.706
ω = ω*Min(1,1.25*(lb/ld)^2/3) = 0.54

Calculation of ratio lb/ld

Adequate Lap Length: lb/ld >= 1

Calculation of Mu2-

Calculation of ultimate Moment Strength ((3.67), Biskinis Phd), Mu
Mu = 2.3291E+008

φ = 1.11701
φ' = 0.98759739
error of function (3.68), Biskinis Phd = 59859.019
From 5A.2, TBDY: fcc = fc*λc = 29.18743
conf. factor λc = 1.45937
fc = 20.00

From 10.3.5, ASCE41-17, Final value of fy: fy*Min(1,1.25*(lb/ld)^2/3) = 555.5556
lb/ld = 1.00
d1 = 44.00
R = 200.00
v = 0.00189786
N = 4771.233
Ac = 125663.706
ω = ω*Min(1,1.25*(lb/ld)^2/3) = 0.54

Calculation of ratio lb/ld

Adequate Lap Length: lb/ld >= 1

Calculation of Shear Strength Vr = Min(Vr1,Vr2) = 288408.521

Calculation of Shear Strength at edge 1, Vr1 = 288408.521
Vr1 = VCol ((10.3), ASCE 41-17) = knl*VCol0
VCol0 = 288408.521
knl = 1 (zero step-static loading)

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

λ = 1 (normal-weight concrete)
fc' = 20.00, but fc^0.5 <= 8.3 MPa (22.5.3.1, ACI 318-14)
M/Vd = 2.00
Mu = 7.3393178E-012
Vu = 3.9130116E-047
d = 0.8*D = 320.00
Nu = 4771.233
Ag = 125663.706
From (11.5.4.8), ACI 318-14: Vs = 175459.634
Av = π/2*A_stirrup = 123370.055
fy = 444.4444
s = 100.00
Vs is multiplied by αCol = 0.00
s/d = 0.3125
Vf ((11-3)-(11.4), ACI 440) = 0.00
From (11-11), ACI 440: Vs + Vf <= 238930.50
bw*d = π*d*d/4 = 80424.772

Calculation of Shear Strength at edge 2, Vr2 = 288408.521
Vr2 = VCol ((10.3), ASCE 41-17) = knl*VCol0
VCol0 = 288408.521
knl = 1 (zero step-static loading)

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

λ = 1 (normal-weight concrete)
fc' = 20.00, but fc^0.5 <= 8.3 MPa (22.5.3.1, ACI 318-14)
M/Vd = 2.00
Mu = 7.3393178E-012
Vu = 3.9130116E-047
d = 0.8*D = 320.00
Nu = 4771.233
Ag = 125663.706
From (11.5.4.8), ACI 318-14: Vs = 175459.634
Av = π/2*A_stirrup = 123370.055
fy = 444.4444
s = 100.00
Vs is multiplied by αCol = 0.00

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

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

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

Constant Properties

Knowledge Factor, κ = 1.00
Chord Rotation is generally considered as Deformation-Controlled Action according to Table C7-1, ASCE41-17.
Mean strengths are used for Deformation-Controlled Actions according to 7.5.1.3, ASCE 41-17
Consequently:
Existing material of Primary Member: Concrete Strength, fc = fcm = 20.00
Existing material of Primary Member: Steel Strength, fs = fsm = 444.4444
Concrete Elasticity, Ec = 21019.039
Steel Elasticity, Es = 200000.00
Diameter, D = 400.00
Cover Thickness, c = 25.00
Element Length, L = 3000.00
Primary Member
Ribbed Bars
Ductile Steel
With Detailing for Earthquake Resistance (including stirrups closed at 135°)
Longitudinal Bars With Ends Lapped Starting at the End Sections
Adequate Lap Length (lb/ld>=1)
No FRP Wrapping

Stepwise Properties

Bending Moment, M = 0.0510701
Shear Force, V2 = 3476.079
Shear Force, V3 = 1.2869988E-013
Axial Force, F = -4769.841
Longitudinal Reinforcement Area Distribution (in 2 divisions)
-Tension: Aslt = 0.00
-Compression: Aslc = 3053.628
Longitudinal Reinforcement Area Distribution (in 3 divisions)
-Tension: Asl,ten = 1017.876
-Compression: Asl,com = 1017.876
-Middle: Asl,mid = 1017.876
Mean Diameter of Tension Reinforcement, DbL = 18.00

Existing component: From table 7-7, ASCE 41_17: Final chord rotation Capacity θu,R = κ*θu = 0.05021297
θu = θy + θp = 0.05021297

- Calculation of θy -

θy = (My*Ls/3)/EIeff = 0.00294145 ((4.29),Biskinis Phd))
My = 2.3308E+008
Ls = M/V (with Ls >0.1*L and Ls < 2*L) = 300.00
From table 10.5, ASCE 41_17: EIeff = factor*Ec*Ig = 7.9240E+012

factor = 0.30
Ag = 125663.706
fc' = 20.00
N = 4769.841
Ec*Ig = 2.6413E+013

Calculation of Yielding Moment My

Calculation of Φy and My according to (7) - (8) in Biskinis and Fardis
----
My = Min(My_ten,My_com) = 2.3308E+008
Φy ((10a) or (10b)) = 1.3315992E-005
My_ten (8a) = 2.3308E+008
Φ_ten (7a) = 78.48338
error of function (7a) = 0.00010055
My_com (8b) = 3.4649E+008
Φ_com (7b) = 70.96935
error of function (7b) = -0.00051806
with ey = 0.00222222
eco = 0.002
apl = 0.35 ((9a) in Biskinis and Fardis for no FRP Wrap)
d1 = 44.00
R = 200.00
v = 0.00189786
N = 4769.841
Ac = 125663.706
ω = 0.54
with fc = 20.00

Calculation of ratio lb/ld

Adequate Lap Length: lb/ld >= 1

- Calculation of θp -

From table 10-9: θp = 0.04727152
with:
- Columns controlled by inadequate development or splicing along the clear height because lb/ld < 1
shear control ratio VyE/VCol0E = 0.53837083
d = 707.00
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 = 4769.841
Ag = 125663.706
f'cE = 20.00
fytE = fylE = 444.4444
pl = Area_Tot_Long_Rein/(Ag) = 0.0243
f'cE = 20.00

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