

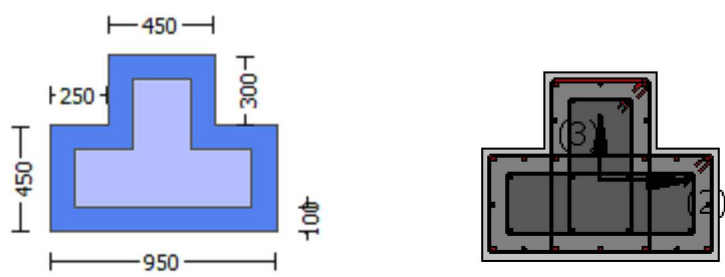
Detailed Member Calculations

Units: N&mm

Regulation: ASCE 41-17

Calculation No. 1

- column C1, Floor 1
- Limit State: Operational Level (data interpolation between analysis steps 39 and 40)
- Analysis: Uniform +X
- Check: Shear capacity VRd
- Edge: Start
- Local Axis: (2)



- Start Of Calculation of Shear Capacity for element: column JTC1 of floor 1
- At local axis: 2
- Integration Section: (a)
- Section Type: rcjtcs

Constant Properties

- Knowledge Factor, $\gamma = 0.80$
- Member Shear Force is generally considered as Force-Controlled Action according to Table C7-1, ASCE 41-17.
- Lower-bound strengths are used for Force-Controlled Actions according to 7.5.1.3, ASCE 41-17
- Consequently:
- Jacket
- New material of Primary Member: Concrete Strength, $f_c = f_{c_lower_bound} = 12.00$
- New material of Primary Member: Steel Strength, $f_s = f_{s_lower_bound} = 500.00$

Concrete Elasticity, $E_c = 19940.411$
Steel Elasticity, $E_s = 200000.00$
Existing Column
Existing material of Primary Member: Concrete Strength, $f_c = f_{c_lower_bound} = 8.00$
Existing material of Primary Member: Steel Strength, $f_s = f_{s_lower_bound} = 420.00$
Concrete Elasticity, $E_c = 16281.278$
Steel Elasticity, $E_s = 200000.00$

Note: Especially for the calculation of γ for displacement ductility demand,
the expected (mean value) strengths are used (7.5.1.3, ASCE 41-17) because bending is considered as
Deformation-Controlled Action (Table C7-1, ASCE 41-17).
Jacket
New material: Concrete Strength, $f_c = f_{cm} = 18.00$
New material: Steel Strength, $f_s = f_{sm} = 625.00$
Existing Column
Existing material: Concrete Strength, $f_c = f_{cm} = 12.00$
Existing material: Steel Strength, $f_s = f_{sm} = 525.00$

Max Height, $H_{max} = 750.00$
Min Height, $H_{min} = 450.00$
Max Width, $W_{max} = 950.00$
Min Width, $W_{min} = 450.00$
Eccentricity, $Ecc = 250.00$
Jacket Thickness, $t_j = 100.00$
Cover Thickness, $c = 25.00$
Element Length, $L = 3000.00$
Primary Member
Ribbed Bars
Ductile Steel
With Detailing for Earthquake Resistance (including stirrups closed at 135°)
Longitudinal Bars With Ends Lapped Starting at the End Sections
Adequate Lap Length ($l_o/l_{ou,min} = l_b/l_d \geq 1$)
No FRP Wrapping

Stepwise Properties

EDGE -A-
Bending Moment, $M_a = -1.2771E+009$
Shear Force, $V_a = -455948.704$
EDGE -B-
Bending Moment, $M_b = -4.0305E+008$
Shear Force, $V_b = 455948.704$
BOTH EDGES
Axial Force, $F = -7.6755E+006$
Longitudinal Reinforcement Area Distribution (in 2 divisions)
-Tension: $As_t = 2522.699$
-Compression: $As_c = 4168.893$
Longitudinal Reinforcement Area Distribution (in 3 divisions)
-Tension: $As_{t,ten} = 1539.38$
-Compression: $As_{l,com} = 1539.38$
-Middle: $As_{l,mid} = 3612.832$
Mean Diameter of Tension Reinforcement, $Db_{L,ten} = 16.57143$

Existing component: From table 7-7, ASCE 41_17: Final Shear Capacity $VR = \phi V_n = 791861.287$
 $V_n ((10.3), ASCE 41-17) = k_n l \cdot V_{CoI} = 989826.609$
 $V_{CoI} = 1.2408E+006$
 $k_n l = 0.7977232$
displacement_ductility_demand = 4.69702

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

= 1 (normal-weight concrete)

Mean concrete strength: $fc' = (fc'_{jacket} \cdot Area_{jacket} + fc'_{core} \cdot Area_{core}) / Area_{section} = 10.13333$, but $fc'^{0.5} \leq 8.3$ MPa (22.5.3.1, ACI 318-14)

$M/Vd = 3.68541$

$\mu_u = 1.2771E+009$

$V_u = 455948.704$

$d = 0.8 \cdot h = 760.00$

$N_u = 7.6755E+006$

$A_g = 427500.00$

From (11.5.4.8), ACI 318-14: $V_s = V_{sjacket} + V_{s,core} = 980981.156$

where:

$V_{sjacket} = V_{sj1} + V_{sj2} = 879645.943$

$V_{sj1} = 282743.339$ is calculated for section web jacket, with:

$d = 360.00$

$A_v = 157079.633$

$f_y = 500.00$

$s = 100.00$

V_{sj1} is multiplied by $Col,j1 = 1.00$

$s/d = 0.27777778$

$V_{sj2} = 596902.604$ is calculated for section flange jacket, with:

$d = 760.00$

$A_v = 157079.633$

$f_y = 500.00$

$s = 100.00$

V_{sj2} is multiplied by $Col,j2 = 1.00$

$s/d = 0.13157895$

$V_{s,core} = V_{s,c1} + V_{s,c2} = 101335.213$

$V_{s,c1} = 0.00$ is calculated for section web core, with:

$d = 200.00$

$A_v = 100530.965$

$f_y = 420.00$

$s = 250.00$

$V_{s,c1}$ is multiplied by $Col,c1 = 0.00$

$s/d = 1.25$

$V_{s,c2} = 101335.213$ is calculated for section flange core, with:

$d = 600.00$

$A_v = 100530.965$

$f_y = 420.00$

$s = 250.00$

$V_{s,c2}$ is multiplied by $Col,c2 = 1.00$

$s/d = 0.41666667$

$V_f ((11-3)-(11.4), ACI 440) = 0.00$

From (11-11), ACI 440: $V_s + V_f \leq 723217.666$

$bw = 450.00$

displacement_ductility_demand is calculated as ϕ / y

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

From analysis, chord rotation $\theta = 0.01366208$

$y = (M_y \cdot L_s / 3) / E_{eff} = 0.00290867$ ((4.29), Biskinis Phd))

$M_y = 1.4240E+009$

$L_s = M/V$ (with $L_s > 0.1 \cdot L$ and $L_s < 2 \cdot L$) = 2800.911

From table 10.5, ASCE 41_17: $E_{eff} = factor \cdot E_c \cdot I_g = 4.5707E+014$

factor = 0.70

$A_g = 562500.00$

Mean concrete strength: $fc' = (fc'_{jacket} \cdot Area_{jacket} + fc'_{core} \cdot Area_{core}) / Area_{section} = 15.20$

$N = 7.6755E+006$

$E_c \cdot I_g = E_{c,jacket} \cdot I_{g,jacket} + E_{c,core} \cdot I_{g,core} = 6.5295E+014$

Calculation of Yielding Moment M_y

Calculation of ϕ and M_y according to Annex 7 -

```
y = Min( y_ten, y_com)
y_ten = 7.1477677E-006
with fy = 601.9953
d = 907.00
y = 0.53571421
A = 0.04763352
B = 0.03982468
with pt = 0.0037716
pc = 0.0037716
pv = 0.00885172
N = 7.6755E+006
b = 450.00
" = 0.04740904
y_comp = 1.7850636E-006
with fc = 18.00
Ec = 19940.411
y = 1.00358
A = -0.04147374
B = 0.0085861
with Es = 200000.00
```

Calculation of ratio I_b/I_d

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

End Of Calculation of Shear Capacity for element: column JTC1 of floor 1

At local axis: 2

Integration Section: (a)

Calculation No. 2

column C1, Floor 1

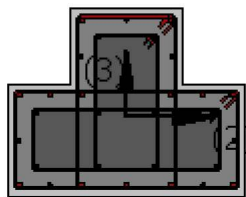
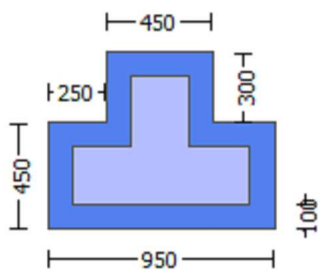
Limit State: Operational Level (data interpolation between analysis steps 39 and 40)

Analysis: Uniform +X

Check: Chord rotation capacity (ϕ)

Edge: Start

Local Axis: (2)



Start Of Calculation of Shear Capacity ratio for element: column JTC1 of floor 1

At Shear local axis: 3

(Bending local axis: 2)

Section Type: rcjtcs

Constant Properties

Knowledge Factor, $\gamma = 0.80$

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

Consequently:

Jacket

New material of Primary Member: Concrete Strength, $f_c = f_{cm} = 18.00$

New material of Primary Member: Steel Strength, $f_s = f_{sm} = 625.00$

Concrete Elasticity, $E_c = 19940.411$

Steel Elasticity, $E_s = 200000.00$

Existing Column

Existing material of Primary Member: Concrete Strength, $f_c = f_{cm} = 12.00$

Existing material of Primary Member: Steel Strength, $f_s = f_{sm} = 525.00$

Concrete Elasticity, $E_c = 16281.278$

Steel Elasticity, $E_s = 200000.00$

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Note: Especially for the calculation of moment strengths,

the above steel re-bar strengths are multiplied by 1.25 according to R18.6.5, ACI 318-14

Jacket

New material: Steel Strength, $f_s = 1.25 \cdot f_{sm} = 781.25$

Existing Column

Existing material: Steel Strength, $f_s = 1.25 \cdot f_{sm} = 656.25$

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Max Height, $H_{max} = 750.00$

Min Height, $H_{min} = 450.00$

Max Width, $W_{max} = 950.00$

Min Width, $W_{min} = 450.00$

Eccentricity, $E_{cc} = 250.00$

Jacket Thickness, $t_j = 100.00$

Cover Thickness, $c = 25.00$

Mean Confinement Factor overall section = 1.42131

Element Length, $L = 3000.00$

Primary Member

Ribbed Bars

Ductile Steel

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

Longitudinal Bars With Ends Lapped Starting at the End Sections

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

No FRP Wrapping

Stepwise Properties

At local axis: 3

EDGE -A-

Shear Force, $V_a = -227.4778$

EDGE -B-

Shear Force, $V_b = 227.4778$
 BOTH EDGES
 Axial Force, $F = -7.6839E+006$
 Longitudinal Reinforcement Area Distribution (in 2 divisions)
 -Tension: $As_t = 314.1593$
 -Compression: $As_c = 6176.371$
 Longitudinal Reinforcement Area Distribution (in 3 divisions)
 -Tension: $As_{t,ten} = 1539.38$
 -Compression: $As_{c,com} = 2475.575$
 -Middle: $As_{mid} = 2676.637$

Calculation of Shear Capacity ratio, $V_e/V_r = 1.74773$
 Member Controlled by Shear ($V_e/V_r > 1$)
 Calculation of Shear Demand from fig. R18.6.5, ACI 318-14 $V_e = (M_{pr1} + M_{pr2})/l_n = 2.0410E+006$
 with
 $M_{pr1} = \text{Max}(\mu_{u1+}, \mu_{u1-}) = 3.0615E+009$
 $\mu_{u1+} = 3.0615E+009$, is the ultimate moment strength at the edge 1 of the member in the actual moment direction which is defined for the static loading combination
 $\mu_{u1-} = 8.9683E+008$, is the ultimate moment strength at the edge 1 of the member in the opposite moment direction which is defined for the static loading combination
 $M_{pr2} = \text{Max}(\mu_{u2+}, \mu_{u2-}) = 3.0615E+009$
 $\mu_{u2+} = 3.0615E+009$, is the ultimate moment strength at the edge 2 of the member in the actual moment direction which is defined for the static loading combination
 $\mu_{u2-} = 8.9683E+008$, is the ultimate moment strength at the edge 2 of the member in the opposite moment direction which is defined for the static loading combination

Calculation of μ_{u1+}

Calculation of ultimate curvature ϕ_u according to 4.1, Biskinis/Fardis 2013:
 $\phi_u = 1.3930706E-005$
 $\mu_u = 3.0615E+009$

with full section properties:

$b = 950.00$
 $d = 707.00$
 $d' = 43.00$
 $v = 0.63557247$
 $N = 7.6839E+006$
 $f_c = 18.00$
 $\alpha = (5A.5, \text{TBDY}) = 0.002$

Final value of ϕ_u : $\phi_u^* = \text{shear_factor} * \text{Max}(\phi_u, \alpha) = 0.01486841$

The Shear_factor is considered equal to 1 (pure moment strength)

From (5.4b), TBDY: $\phi_u = 0.01486841$

we (5.4c) = 0.08077545

$\alpha_{se}((5.4d), \text{TBDY}) = (\alpha_{se1} * A_{ext} + \alpha_{se2} * A_{int}) / A_{sec} = 0.53375773$

$\alpha_{se1} = \text{Max}(((A_{conf,max1} - A_{noConf1}) / A_{conf,max1}) * (A_{conf,min1} / A_{conf,max1}), 0) = 0.53375773$

The definitions of A_{noConf} , $A_{conf,min}$ and $A_{conf,max}$ are derived from generalization of the rectangular sections confinement, which is expressed by (5.4d).

The generalization is done according to Mander, J., Priestley, M., and Park, R. (1988)

"Theoretical Stress-Strain Model for Confined Concrete."

J. Struct. Eng., 10.1061/(ASCE)0733-9445(1988)114:8(1804), 1804-1826.

$A_{conf,max1} = 464100.00$ is the confined core area at levels of member with hoops and is calculated as the area of core enclosed by the center lines of the external perimeter hoops.

$A_{conf,min1} = 395025.00$ is the confined core area at midway between the levels of hoops and is calculated by reducing all the dimensions of the area $A_{conf,max1}$ by a length equal to half the clear spacing between external hoops.

$A_{noConf1} = 173066.667$ is the unconfined external core area which is equal to $b^2/6$ as defined at (A.2).

$\alpha_{se2} (>= \alpha_{se1}) = \text{Max}(((A_{conf,max2} - A_{noConf2}) / A_{conf,max2}) * (A_{conf,min2} / A_{conf,max2}), 0) = 0.53375773$

The definitions of A_{noConf} , $A_{conf,min}$ and $A_{conf,max}$ are derived from generalization of the rectangular sections confinement, which is expressed by (5.4d).

The generalization is done according to Mander, J., Priestley, M., and Park, R. (1988)

"Theoretical Stress-Strain Model for Confined Concrete."

J. Struct. Eng., 10.1061/(ASCE)0733-9445(1988)114:8(1804), 1804-1826.

$A_{conf,max2} = 252164.00$ is the confined core area at levels of member with hoops and is calculated as the area of core enclosed by the center lines of the internal perimeter hoops.

$A_{conf,min2} = 111441.00$ is the confined core area at midway between the levels of hoops and is calculated by reducing all the dimensions of the area $A_{conf,max2}$ by a length equal to half the clear spacing between internal hoops.

$A_{noConf2} = 110709.333$ is the unconfined internal core area which is equal to $b_i^2/6$ as defined at (A.2).
 $psh_{min} * F_{ywe} = \text{Min}(psh_x * F_{ywe}, psh_y * F_{ywe}) = 2.724$

$psh_x * F_{ywe} = psh1 * F_{ywe1} + ps2 * F_{ywe2} = 2.724$

$psh1 ((5.4d), TBDY) = L_{stir1} * A_{stir1} / (A_{sec} * s1) = 0.00301593$

L_{stir1} (Length of stirrups along Y) = 2160.00

A_{stir1} (stirrups area) = 78.53982

$psh2 (5.4d) = L_{stir2} * A_{stir2} / (A_{sec} * s2) = 0.00056047$

L_{stir2} (Length of stirrups along Y) = 1568.00

A_{stir2} (stirrups area) = 50.26548

$psh_y * F_{ywe} = psh1 * F_{ywe1} + ps2 * F_{ywe2} = 3.25416$

$psh1 ((5.4d), TBDY) = L_{stir1} * A_{stir1} / (A_{sec} * s1) = 0.00357443$

L_{stir1} (Length of stirrups along X) = 2560.00

A_{stir1} (stirrups area) = 78.53982

$psh2 ((5.4d), TBDY) = L_{stir2} * A_{stir2} / (A_{sec} * s2) = 0.00070345$

L_{stir2} (Length of stirrups along X) = 1968.00

A_{stir2} (stirrups area) = 50.26548

$A_{sec} = 562500.00$

$s1 = 100.00$

$s2 = 250.00$

$f_{ywe1} = 781.25$

$f_{ywe2} = 656.25$

$f_{ce} = 18.00$

From ((5.A5), TBDY), TBDY: $cc = 0.00621307$

c = confinement factor = 1.42131

$y1 = 0.0025$

$sh1 = 0.008$

$ft1 = 907.50$

$fy1 = 756.25$

$su1 = 0.032$

using (30) in Biskinis/Fardis (2013) multiplied with shear_factor and also multiplied by the shear_factor according to 15.7.1.4, with

Shear_factor = 1.00

$lo/lo_{min} = lb/l_d = 1.00$

$su1 = 0.4 * esu1_{nominal} ((5.5), TBDY) = 0.032$

From table 5A.1, TBDY: $esu1_{nominal} = 0.08$,

For calculation of $esu1_{nominal}$ and $y1, sh1, ft1, fy1$, it is considered characteristic value $fsy1 = fs1/1.2$, from table 5.1, TBDY.

$y1, sh1, ft1, fy1$, are also multiplied by $\text{Min}(1, 1.25 * (lb/l_d)^{2/3})$, from 10.3.5, ASCE 41-17.

with $fs1 = (fs_{jacket} * A_{sl,ten,jacket} + fs_{core} * A_{sl,ten,core}) / A_{sl,ten} = 756.25$

with $Es1 = (Es_{jacket} * A_{sl,ten,jacket} + Es_{core} * A_{sl,ten,core}) / A_{sl,ten} = 200000.00$

$y2 = 0.0025$

$sh2 = 0.008$

$ft2 = 900.1904$

$fy2 = 750.1586$

$su2 = 0.032$

using (30) in Biskinis/Fardis (2013) multiplied with shear_factor and also multiplied by the shear_factor according to 15.7.1.4, with Shear_factor = 1.00

$lo/lo_{min} = lb/l_b, min = 1.00$

$su2 = 0.4 * esu2_{nominal} ((5.5), TBDY) = 0.032$

From table 5A.1, TBDY: $esu2_{nominal} = 0.08$,

For calculation of $esu2_{nominal}$ and $y2, sh2, ft2, fy2$, it is considered characteristic value $fsy2 = fs2/1.2$, from table 5.1, TBDY.

$y1, sh1, ft1, fy1$, are also multiplied by $\text{Min}(1, 1.25 * (lb/l_d)^{2/3})$, from 10.3.5, ASCE 41-17.

with $fs2 = (fs_{jacket} * A_{sl,com,jacket} + fs_{core} * A_{sl,com,core}) / A_{sl,com} = 750.1586$

with $Es2 = (Es_{jacket} * A_{sl,com,jacket} + Es_{core} * A_{sl,com,core}) / A_{sl,com} = 200000.00$

$yv = 0.0025$

```

shv = 0.008
ftv = 902.993
fyv = 752.4941
suv = 0.032
    using (30) in Biskinis/Fardis (2013) multiplied with shear_factor
    and also multiplied by the shear_factor according to 15.7.1.4, with
    Shear_factor = 1.00
    lo/lou,min = lb/ld = 1.00
    suv = 0.4*esuv_nominal ((5.5), TBDY) = 0.032
    From table 5A.1, TBDY: esuv_nominal = 0.08,
    considering characteristic value fsyv = fsv/1.2, from table 5.1, TBDY
    For calculation of esuv_nominal and yv, shv,ftv,fyv, it is considered
    characteristic value fsyv = fsv/1.2, from table 5.1, TBDY.
    y1, sh1,ft1,fy1, are also multiplied by Min(1,1.25*(lb/ld)^2/3), from 10.3.5, ASCE 41-17.
    with fsv = (fs,jacket*Asl,mid,jacket + fs,mid*Asl,mid,core)/Asl,mid = 752.4941
    with Esv = (Es,jacket*Asl,mid,jacket + Es,mid*Asl,mid,core)/Asl,mid = 200000.00
    1 = Asl,ten/(b*d)*(fs1/fc) = 0.09629324
    2 = Asl,com/(b*d)*(fs2/fc) = 0.15360794
    v = Asl,mid/(b*d)*(fsv/fc) = 0.16660079
and confined core properties:
b = 890.00
d = 677.00
d' = 13.00
fcc (5A.2, TBDY) = 25.58352
cc (5A.5, TBDY) = 0.00621307
    c = confinement factor = 1.42131
    1 = Asl,ten/(b*d)*(fs1/fc) = 0.10733965
    2 = Asl,com/(b*d)*(fs2/fc) = 0.17122928
    v = Asl,mid/(b*d)*(fsv/fc) = 0.18571261
Case/Assumption: Unconfined full section - Steel rupture
' satisfies Eq. (4.3)
---->
v < vs,y2 - LHS eq.(4.5) is not satisfied
---->
v < vs,c - RHS eq.(4.5) is not satisfied
---->
Case/Assumption Rejected.
---->
New Case/Assumption: Unconfined full section - Spalling of concrete cover
' satisfies Eq. (4.4)
---->
v < s,y1 - LHS eq.(4.7) is not satisfied
---->
v < vc,y1 - RHS eq.(4.6) is satisfied
---->
cu (4.10) = 0.58509201
MRc (4.17) = 2.3801E+009
---->
New Case/Assumption: Ultimate curvature of confined core after spalling of concrete cover
In expressions below, the following modifications have been made
- b, d, d' replaced by geometric parameters of the core: bo, do, d'o
- N, 1, 2, v normalised to bo*do, instead of b*d
- parameters of confined concrete, fcc, cc, used in lieu of fc, ecu
---->
Subcase: Rupture of tension steel
---->
v* < v*s,y2 - LHS eq.(4.5) is not satisfied
---->
v* < v*s,c - LHS eq.(4.5) is not satisfied
---->
Subcase rejected
---->
New Subcase: Failure of compression zone
---->
v* < v*c,y2 - LHS eq.(4.6) is not satisfied
---->
v* < v*c,y1 - RHS eq.(4.6) is not satisfied

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$$*cu(4.11) = 0.65878578$$

$$MRo(4.18) = 3.0615E+009$$

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$$u = cu(4.2) = 1.3930706E-005$$

$$Mu = MRo$$

Calculation of ratio lb/d

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

Calculation of $Mu1$ -

Calculation of ultimate curvature u according to 4.1, Biskinis/Fardis 2013:

$$u = 9.1773508E-006$$

$$Mu = 8.9683E+008$$

with full section properties:

$$b = 450.00$$

$$d = 707.00$$

$$d' = 43.00$$

$$v = 1.34176$$

$$N = 7.6839E+006$$

$$fc = 18.00$$

$$co(5A.5, TBDY) = 0.002$$

$$\text{Final value of } cu: cu^* = \text{shear_factor} * \text{Max}(cu, cc) = 0.01486841$$

The Shear_factor is considered equal to 1 (pure moment strength)

$$\text{From (5.4b), TBDY: } cu = 0.01486841$$

$$we(5.4c) = 0.08077545$$

$$ase((5.4d), TBDY) = (ase1 * Aext + ase2 * Aint) / Asec = 0.53375773$$

$$ase1 = \text{Max}(((Aconf,max1 - AnoConf1) / Aconf,max1) * (Aconf,min1 / Aconf,max1), 0) = 0.53375773$$

The definitions of $AnoConf$, $Aconf,min$ and $Aconf,max$ are derived from generalization of the rectangular sections confinement, which is expressed by (5.4d).

The generalization is done according to Mander, J., Priestley, M., and Park, R. (1988)

"Theoretical Stress-Strain Model for Confined Concrete."

J. Struct. Eng., 10.1061/(ASCE)0733-9445(1988)114:8(1804), 1804-1826.

$Aconf,max1 = 464100.00$ is the confined core area at levels of member with hoops and is calculated as the area of core enclosed by the center lines of the external perimeter hoops.

$Aconf,min1 = 395025.00$ is the confined core area at midway between the levels of hoops and is calculated by reducing all the dimensions of the area $Aconf,max1$ by a length equal to half the clear spacing between external hoops.

$AnoConf1 = 173066.667$ is the unconfined external core area which is equal to $bi^2/6$ as defined at (A.2).

$$ase2 (\geq ase1) = \text{Max}(((Aconf,max2 - AnoConf2) / Aconf,max2) * (Aconf,min2 / Aconf,max2), 0) = 0.53375773$$

The definitions of $AnoConf$, $Aconf,min$ and $Aconf,max$ are derived from generalization of the rectangular sections confinement, which is expressed by (5.4d).

The generalization is done according to Mander, J., Priestley, M., and Park, R. (1988)

"Theoretical Stress-Strain Model for Confined Concrete."

J. Struct. Eng., 10.1061/(ASCE)0733-9445(1988)114:8(1804), 1804-1826.

$Aconf,max2 = 252164.00$ is the confined core area at levels of member with hoops and is calculated as the area of core enclosed by the center lines of the internal perimeter hoops.

$Aconf,min2 = 111441.00$ is the confined core area at midway between the levels of hoops and is calculated by reducing all the dimensions of the area $Aconf,max2$ by a length equal to half the clear spacing between internal hoops.

$AnoConf2 = 110709.333$ is the unconfined internal core area which is equal to $bi^2/6$ as defined at (A.2).

$$psh,min * Fywe = \text{Min}(psh,x * Fywe, psh,y * Fywe) = 2.724$$

$$psh_x * Fywe = psh1 * Fywe1 + ps2 * Fywe2 = 2.724$$

$$psh1((5.4d), TBDY) = Lstir1 * Astir1 / (Asec * s1) = 0.00301593$$

$$Lstir1 (\text{Length of stirrups along } Y) = 2160.00$$

$$Astir1 (\text{stirrups area}) = 78.53982$$

$$psh2(5.4d) = Lstir2 * Astir2 / (Asec * s2) = 0.00056047$$

$$Lstir2 (\text{Length of stirrups along } Y) = 1568.00$$

Astir2 (stirrups area) = 50.26548

psh_y*Fywe = psh1*Fywe1+ps2*Fywe2 = 3.25416
psh1 ((5.4d), TBDY) = Lstir1*Astir1/(Asec*s1) = 0.00357443
Lstir1 (Length of stirrups along X) = 2560.00
Astir1 (stirrups area) = 78.53982
psh2 ((5.4d), TBDY) = Lstir2*Astir2/(Asec*s2) = 0.00070345
Lstir2 (Length of stirrups along X) = 1968.00
Astir2 (stirrups area) = 50.26548

Asec = 562500.00

s1 = 100.00

s2 = 250.00

fywe1 = 781.25

fywe2 = 656.25

fce = 18.00

From ((5.A.5), TBDY), TBDY: cc = 0.00621307

c = confinement factor = 1.42131

y1 = 0.0025

sh1 = 0.008

ft1 = 900.1904

fy1 = 750.1586

su1 = 0.032

using (30) in Biskinis/Fardis (2013) multiplied with shear_factor
and also multiplied by the shear_factor according to 15.7.1.4, with
Shear_factor = 1.00

lo/lou,min = lb/ld = 1.00

su1 = 0.4*esu1_nominal ((5.5), TBDY) = 0.032

From table 5A.1, TBDY: esu1_nominal = 0.08,

For calculation of esu1_nominal and y1, sh1,ft1,fy1, it is considered
characteristic value fsy1 = fs1/1.2, from table 5.1, TBDY.

y1, sh1,ft1,fy1, are also multiplied by Min(1,1.25*(lb/ld)^ 2/3), from 10.3.5, ASCE 41-17.

with fs1 = (fs,jacket*Asl,ten,jacket + fs,core*Asl,ten,core)/Asl,ten = 750.1586

with Es1 = (Es,jacket*Asl,ten,jacket + Es,core*Asl,ten,core)/Asl,ten = 200000.00

y2 = 0.0025

sh2 = 0.008

ft2 = 907.50

fy2 = 756.25

su2 = 0.032

using (30) in Biskinis/Fardis (2013) multiplied with shear_factor
and also multiplied by the shear_factor according to 15.7.1.4, with
Shear_factor = 1.00

lo/lou,min = lb/lb,min = 1.00

su2 = 0.4*esu2_nominal ((5.5), TBDY) = 0.032

From table 5A.1, TBDY: esu2_nominal = 0.08,

For calculation of esu2_nominal and y2, sh2,ft2,fy2, it is considered
characteristic value fsy2 = fs2/1.2, from table 5.1, TBDY.

y1, sh1,ft1,fy1, are also multiplied by Min(1,1.25*(lb/ld)^ 2/3), from 10.3.5, ASCE 41-17.

with fs2 = (fs,jacket*Asl,com,jacket + fs,core*Asl,com,core)/Asl,com = 756.25

with Es2 = (Es,jacket*Asl,com,jacket + Es,core*Asl,com,core)/Asl,com = 200000.00

yv = 0.0025

shv = 0.008

ftv = 902.993

fyv = 752.4941

suv = 0.032

using (30) in Biskinis/Fardis (2013) multiplied with shear_factor
and also multiplied by the shear_factor according to 15.7.1.4, with
Shear_factor = 1.00

lo/lou,min = lb/ld = 1.00

suv = 0.4*esuv_nominal ((5.5), TBDY) = 0.032

From table 5A.1, TBDY: esuv_nominal = 0.08,

considering characteristic value fsyv = fsv/1.2, from table 5.1, TBDY

For calculation of esuv_nominal and yv, shv,ftv,fyv, it is considered
characteristic value fsyv = fsv/1.2, from table 5.1, TBDY.

y1, sh1,ft1,fy1, are also multiplied by Min(1,1.25*(lb/ld)^ 2/3), from 10.3.5, ASCE 41-17.

with fsv = (fs,jacket*Asl,mid,jacket + fs,mid*Asl,mid,core)/Asl,mid = 752.4941

with Esv = (Es,jacket*Asl,mid,jacket + Es,mid*Asl,mid,core)/Asl,mid = 200000.00

```

1 = Asl,ten/(b*d)*(fs1/fc) = 0.32428344
2 = Asl,com/(b*d)*(fs2/fc) = 0.20328574
v = Asl,mid/(b*d)*(fsv/fc) = 0.35171278
and confined core properties:
b = 390.00
d = 677.00
d' = 13.00
fcc (5A.2, TBDY) = 25.58352
cc (5A.5, TBDY) = 0.00621307
c = confinement factor = 1.42131
1 = Asl,ten/(b*d)*(fs1/fc) = 0.39075399
2 = Asl,com/(b*d)*(fs2/fc) = 0.24495458
v = Asl,mid/(b*d)*(fsv/fc) = 0.42380571
Case/Assumption: Unconfined full section - Steel rupture
' satisfies Eq. (4.3)
---->
v < vs,y2 - LHS eq.(4.5) is not satisfied
---->
v < vs,c - RHS eq.(4.5) is not satisfied
---->
Case/Assumption Rejected.
---->
New Case/Assumption: Unconfined full section - Spalling of concrete cover
' satisfies Eq. (4.4)
---->
v < s,y1 - LHS eq.(4.7) is not satisfied
---->
v < vc,y1 - RHS eq.(4.6) is not satisfied
---->
cu (4.11) = 0.95920575
MRc (4.18) = 7.7321E+008
---->
New Case/Assumption: Ultimate curvature of confined core after spalling of concrete cover
In expressions below, the following modifications have been made
- b, d, d' replaced by geometric parameters of the core: bo, do, d'o
- N, 1, 2, v normalised to bo*do, instead of b*d
- - parameters of confined concrete, fcc, cc, used in lieu of fc, ecu
---->
Subcase: Rupture of tension steel
---->
v* < v*s,y2 - LHS eq.(4.5) is not satisfied
---->
v* < v*s,c - LHS eq.(4.5) is not satisfied
---->
Subcase rejected
---->
New Subcase: Failure of compression zone
---->
v* < v*c,y2 - LHS eq.(4.6) is not satisfied
---->
v* < v*c,y1 - RHS eq.(4.6) is not satisfied
---->
*cu (4.11) = 1.00
MRo (4.18) = 8.9683E+008
---->
u = cu (4.2) = 9.1773508E-006
Mu = MRo

```

Calculation of ratio lb/d

Adequate Lap Length: lb/d >= 1

Calculation of Mu2+

Calculation of ultimate curvature ϕ_u according to 4.1, Biskinis/Fardis 2013:

$$u = 1.3930706E-005$$

$$\mu_u = 3.0615E+009$$

with full section properties:

$$b = 950.00$$

$$d = 707.00$$

$$d' = 43.00$$

$$v = 0.63557247$$

$$N = 7.6839E+006$$

$$f_c = 18.00$$

$$\alpha_{co} (5A.5, TBDY) = 0.002$$

$$\text{Final value of } \phi_u: \phi_u^* = \text{shear_factor} * \text{Max}(\phi_u, \phi_c) = 0.01486841$$

The Shear_factor is considered equal to 1 (pure moment strength)

$$\text{From (5.4b), TBDY: } \phi_u = 0.01486841$$

$$\phi_{we} (5.4c) = 0.08077545$$

$$\phi_{ase} ((5.4d), TBDY) = (\phi_{ase1} * A_{ext} + \phi_{ase2} * A_{int}) / A_{sec} = 0.53375773$$

$$\phi_{ase1} = \text{Max}(((A_{conf,max1} - A_{noConf1}) / A_{conf,max1}) * (\phi_{conf,min1} / A_{conf,max1}), 0) = 0.53375773$$

The definitions of A_{noConf} , $A_{conf,min}$ and $A_{conf,max}$ are derived from generalization of the rectangular sections confinement, which is expressed by (5.4d).

The generalization is done according to Mander, J., Priestley, M., and Park, R. (1988)

"Theoretical Stress-Strain Model for Confined Concrete."

J. Struct. Eng., 10.1061/(ASCE)0733-9445(1988)114:8(1804), 1804-1826.

$A_{conf,max1} = 464100.00$ is the confined core area at levels of member with hoops and is calculated as the area of core enclosed by the center lines of the external perimeter hoops.

$A_{conf,min1} = 395025.00$ is the confined core area at midway between the levels of hoops and is calculated by reducing all the dimensions of the area $A_{conf,max1}$ by a length equal to half the clear spacing between external hoops.

$A_{noConf1} = 173066.667$ is the unconfined external core area which is equal to $b_i^2/6$ as defined at (A.2).

$$\phi_{ase2} (> \phi_{ase1}) = \text{Max}(((A_{conf,max2} - A_{noConf2}) / A_{conf,max2}) * (\phi_{conf,min2} / A_{conf,max2}), 0) = 0.53375773$$

The definitions of A_{noConf} , $A_{conf,min}$ and $A_{conf,max}$ are derived from generalization of the rectangular sections confinement, which is expressed by (5.4d).

The generalization is done according to Mander, J., Priestley, M., and Park, R. (1988)

"Theoretical Stress-Strain Model for Confined Concrete."

J. Struct. Eng., 10.1061/(ASCE)0733-9445(1988)114:8(1804), 1804-1826.

$A_{conf,max2} = 252164.00$ is the confined core area at levels of member with hoops and is calculated as the area of core enclosed by the center lines of the internal perimeter hoops.

$A_{conf,min2} = 111441.00$ is the confined core area at midway between the levels of hoops and is calculated by reducing all the dimensions of the area $A_{conf,max2}$ by a length equal to half the clear spacing between internal hoops.

$A_{noConf2} = 110709.333$ is the unconfined internal core area which is equal to $b_i^2/6$ as defined at (A.2).

$$\phi_{psh,min} * F_{ywe} = \text{Min}(\phi_{psh,x} * F_{ywe}, \phi_{psh,y} * F_{ywe}) = 2.724$$

$$\phi_{psh,x} * F_{ywe} = \phi_{psh1} * F_{ywe1} + \phi_{ps2} * F_{ywe2} = 2.724$$

$$\phi_{psh1} ((5.4d), TBDY) = L_{stir1} * A_{stir1} / (A_{sec} * s_1) = 0.00301593$$

$$L_{stir1} \text{ (Length of stirrups along Y)} = 2160.00$$

$$A_{stir1} \text{ (stirrups area)} = 78.53982$$

$$\phi_{psh2} (5.4d) = L_{stir2} * A_{stir2} / (A_{sec} * s_2) = 0.00056047$$

$$L_{stir2} \text{ (Length of stirrups along Y)} = 1568.00$$

$$A_{stir2} \text{ (stirrups area)} = 50.26548$$

$$\phi_{psh,y} * F_{ywe} = \phi_{psh1} * F_{ywe1} + \phi_{ps2} * F_{ywe2} = 3.25416$$

$$\phi_{psh1} ((5.4d), TBDY) = L_{stir1} * A_{stir1} / (A_{sec} * s_1) = 0.00357443$$

$$L_{stir1} \text{ (Length of stirrups along X)} = 2560.00$$

$$A_{stir1} \text{ (stirrups area)} = 78.53982$$

$$\phi_{psh2} ((5.4d), TBDY) = L_{stir2} * A_{stir2} / (A_{sec} * s_2) = 0.00070345$$

$$L_{stir2} \text{ (Length of stirrups along X)} = 1968.00$$

$$A_{stir2} \text{ (stirrups area)} = 50.26548$$

$$A_{sec} = 562500.00$$

$$s_1 = 100.00$$

$$s_2 = 250.00$$

$$f_{ywe1} = 781.25$$

$$f_{ywe2} = 656.25$$

$$f_{ce} = 18.00$$

From ((5.A.5), TBDY), TBDY: $cc = 0.00621307$
 $c = \text{confinement factor} = 1.42131$
 $y1 = 0.0025$
 $sh1 = 0.008$
 $ft1 = 907.50$
 $fy1 = 756.25$
 $su1 = 0.032$
 using (30) in Biskinis/Fardis (2013) multiplied with shear_factor
 and also multiplied by the shear_factor according to 15.7.1.4, with
 $\text{Shear_factor} = 1.00$
 $lo/lou, \min = lb/ld = 1.00$
 $su1 = 0.4 * esu1_nominal ((5.5), TBDY) = 0.032$
 From table 5A.1, TBDY: $esu1_nominal = 0.08$,
 For calculation of $esu1_nominal$ and $y1, sh1, ft1, fy1$, it is considered
 characteristic value $fsy1 = fs1/1.2$, from table 5.1, TBDY.
 $y1, sh1, ft1, fy1$, are also multiplied by $\text{Min}(1, 1.25 * (lb/ld)^{2/3})$, from 10.3.5, ASCE 41-17.
 with $fs1 = (fs_jacket * Asl, \text{ten}, \text{jacket} + fs_core * Asl, \text{ten}, \text{core}) / Asl, \text{ten} = 756.25$
 with $Es1 = (Es_jacket * Asl, \text{ten}, \text{jacket} + Es_core * Asl, \text{ten}, \text{core}) / Asl, \text{ten} = 200000.00$
 $y2 = 0.0025$
 $sh2 = 0.008$
 $ft2 = 900.1904$
 $fy2 = 750.1586$
 $su2 = 0.032$
 using (30) in Biskinis/Fardis (2013) multiplied with shear_factor
 and also multiplied by the shear_factor according to 15.7.1.4, with
 $\text{Shear_factor} = 1.00$
 $lo/lou, \min = lb/lb, \min = 1.00$
 $su2 = 0.4 * esu2_nominal ((5.5), TBDY) = 0.032$
 From table 5A.1, TBDY: $esu2_nominal = 0.08$,
 For calculation of $esu2_nominal$ and $y2, sh2, ft2, fy2$, it is considered
 characteristic value $fsy2 = fs2/1.2$, from table 5.1, TBDY.
 $y1, sh1, ft1, fy1$, are also multiplied by $\text{Min}(1, 1.25 * (lb/ld)^{2/3})$, from 10.3.5, ASCE 41-17.
 with $fs2 = (fs_jacket * Asl, \text{com}, \text{jacket} + fs_core * Asl, \text{com}, \text{core}) / Asl, \text{com} = 750.1586$
 with $Es2 = (Es_jacket * Asl, \text{com}, \text{jacket} + Es_core * Asl, \text{com}, \text{core}) / Asl, \text{com} = 200000.00$
 $yv = 0.0025$
 $shv = 0.008$
 $ftv = 902.993$
 $fyv = 752.4941$
 $suv = 0.032$
 using (30) in Biskinis/Fardis (2013) multiplied with shear_factor
 and also multiplied by the shear_factor according to 15.7.1.4, with
 $\text{Shear_factor} = 1.00$
 $lo/lou, \min = lb/ld = 1.00$
 $suv = 0.4 * esuv_nominal ((5.5), TBDY) = 0.032$
 From table 5A.1, TBDY: $esuv_nominal = 0.08$,
 considering characteristic value $fsyv = fsv/1.2$, from table 5.1, TBDY
 For calculation of $esuv_nominal$ and yv, shv, ftv, fyv , it is considered
 characteristic value $fsyv = fsv/1.2$, from table 5.1, TBDY.
 $y1, sh1, ft1, fy1$, are also multiplied by $\text{Min}(1, 1.25 * (lb/ld)^{2/3})$, from 10.3.5, ASCE 41-17.
 with $fsv = (fs_jacket * Asl, \text{mid}, \text{jacket} + fs_mid * Asl, \text{mid}, \text{core}) / Asl, \text{mid} = 752.4941$
 with $Es_v = (Es_jacket * Asl, \text{mid}, \text{jacket} + Es_mid * Asl, \text{mid}, \text{core}) / Asl, \text{mid} = 200000.00$
 $1 = Asl, \text{ten} / (b * d) * (fs1 / fc) = 0.09629324$
 $2 = Asl, \text{com} / (b * d) * (fs2 / fc) = 0.15360794$
 $v = Asl, \text{mid} / (b * d) * (fsv / fc) = 0.16660079$
 and confined core properties:
 $b = 890.00$
 $d = 677.00$
 $d' = 13.00$
 $fcc (5A.2, TBDY) = 25.58352$
 $cc (5A.5, TBDY) = 0.00621307$
 $c = \text{confinement factor} = 1.42131$
 $1 = Asl, \text{ten} / (b * d) * (fs1 / fc) = 0.10733965$
 $2 = Asl, \text{com} / (b * d) * (fs2 / fc) = 0.17122928$
 $v = Asl, \text{mid} / (b * d) * (fsv / fc) = 0.18571261$
 Case/Assumption: Unconfined full section - Steel rupture
 ' satisfies Eq. (4.3)

```

v < vs,y2 - LHS eq.(4.5) is not satisfied
---->
v < vs,c - RHS eq.(4.5) is not satisfied
---->
Case/Assumption Rejected.
---->
New Case/Assumption: Unconfined full section - Spalling of concrete cover
' satisfies Eq. (4.4)
---->
v < vs,y1 - LHS eq.(4.7) is not satisfied
---->
v < vs,c,y1 - RHS eq.(4.6) is satisfied
---->
cu (4.10) = 0.58509201
MRc (4.17) = 2.3801E+009
---->
New Case/Assumption: Ultimate curvature of confined core after spalling of concrete cover
In expressions below, the following modifications have been made
- b, d, d' replaced by geometric parameters of the core: bo, do, d'o
- N1, N2, v normalised to bo*do, instead of b*d
- parameters of confined concrete, fcc, cc, used in lieu of fc, ecu
---->
Subcase: Rupture of tension steel
---->
v* < vs*,y2 - LHS eq.(4.5) is not satisfied
---->
v* < vs*,c - LHS eq.(4.5) is not satisfied
---->
Subcase rejected
---->
New Subcase: Failure of compression zone
---->
v* < vs*,c,y2 - LHS eq.(4.6) is not satisfied
---->
v* < vs*,c,y1 - RHS eq.(4.6) is not satisfied
---->
*cu (4.11) = 0.65878578
MRo (4.18) = 3.0615E+009
---->
u = cu (4.2) = 1.3930706E-005
Mu = MRo

```

Calculation of ratio lb/d

Adequate Lap Length: lb/d >= 1

Calculation of Mu2-

Calculation of ultimate curvature u according to 4.1, Biskinis/Fardis 2013:

u = 9.1773508E-006

Mu = 8.9683E+008

with full section properties:

b = 450.00

d = 707.00

d' = 43.00

v = 1.34176

N = 7.6839E+006

fc = 18.00

co (5A.5, TBDY) = 0.002

Final value of cu: cu* = shear_factor * Max(cu, cc) = 0.01486841

The Shear_factor is considered equal to 1 (pure moment strength)

From (5.4b), TBDY: $c_u = 0.01486841$

w_e (5.4c) = 0.08077545

a_{se} ((5.4d), TBDY) = $(a_{se1} \cdot A_{ext} + a_{se2} \cdot A_{int}) / A_{sec} = 0.53375773$

$a_{se1} = \text{Max}(((A_{conf,max1} - A_{noConf1}) / A_{conf,max1}) \cdot (A_{conf,min1} / A_{conf,max1}), 0) = 0.53375773$

The definitions of A_{noConf} , $A_{conf,min}$ and $A_{conf,max}$ are derived from generalization of the rectangular sections confinement, which is expressed by (5.4d).

The generalization is done according to Mander, J., Priestley, M., and Park, R. (1988)

"Theoretical Stress-Strain Model for Confined Concrete."

J. Struct. Eng., 10.1061/(ASCE)0733-9445(1988)114:8(1804), 1804-1826.

$A_{conf,max1} = 464100.00$ is the confined core area at levels of member with hoops and

is calculated as the area of core enclosed by the center lines of the external perimeter hoops.

$A_{conf,min1} = 395025.00$ is the confined core area at midway between the levels of hoops and

is calculated by reducing all the dimensions of the area $A_{conf,max1}$ by a length

equal to half the clear spacing between external hoops.

$A_{noConf1} = 173066.667$ is the unconfined external core area which is equal to $b_i^2/6$ as defined at (A.2).

$a_{se2} (>= a_{se1}) = \text{Max}(((A_{conf,max2} - A_{noConf2}) / A_{conf,max2}) \cdot (A_{conf,min2} / A_{conf,max2}), 0) = 0.53375773$

The definitions of A_{noConf} , $A_{conf,min}$ and $A_{conf,max}$ are derived from generalization of the rectangular sections confinement, which is expressed by (5.4d).

The generalization is done according to Mander, J., Priestley, M., and Park, R. (1988)

"Theoretical Stress-Strain Model for Confined Concrete."

J. Struct. Eng., 10.1061/(ASCE)0733-9445(1988)114:8(1804), 1804-1826.

$A_{conf,max2} = 252164.00$ is the confined core area at levels of member with hoops and

is calculated as the area of core enclosed by the center lines of the internal perimeter hoops.

$A_{conf,min2} = 111441.00$ is the confined core area at midway between the levels of hoops and

is calculated by reducing all the dimensions of the area $A_{conf,max2}$ by a length

equal to half the clear spacing between internal hoops.

$A_{noConf2} = 110709.333$ is the unconfined internal core area which is equal to $b_i^2/6$ as defined at (A.2).

$p_{sh,min} \cdot F_{ywe} = \text{Min}(p_{sh,x} \cdot F_{ywe}, p_{sh,y} \cdot F_{ywe}) = 2.724$

$p_{sh,x} \cdot F_{ywe} = p_{sh1} \cdot F_{ywe1} + p_{s2} \cdot F_{ywe2} = 2.724$

p_{sh1} ((5.4d), TBDY) = $L_{stir1} \cdot A_{stir1} / (A_{sec} \cdot s_1) = 0.00301593$

L_{stir1} (Length of stirrups along Y) = 2160.00

A_{stir1} (stirrups area) = 78.53982

p_{sh2} (5.4d) = $L_{stir2} \cdot A_{stir2} / (A_{sec} \cdot s_2) = 0.00056047$

L_{stir2} (Length of stirrups along Y) = 1568.00

A_{stir2} (stirrups area) = 50.26548

$p_{sh,y} \cdot F_{ywe} = p_{sh1} \cdot F_{ywe1} + p_{s2} \cdot F_{ywe2} = 3.25416$

p_{sh1} ((5.4d), TBDY) = $L_{stir1} \cdot A_{stir1} / (A_{sec} \cdot s_1) = 0.00357443$

L_{stir1} (Length of stirrups along X) = 2560.00

A_{stir1} (stirrups area) = 78.53982

p_{sh2} ((5.4d), TBDY) = $L_{stir2} \cdot A_{stir2} / (A_{sec} \cdot s_2) = 0.00070345$

L_{stir2} (Length of stirrups along X) = 1968.00

A_{stir2} (stirrups area) = 50.26548

$A_{sec} = 562500.00$

$s_1 = 100.00$

$s_2 = 250.00$

$f_{ywe1} = 781.25$

$f_{ywe2} = 656.25$

$f_{ce} = 18.00$

From ((5.A5), TBDY), TBDY: $c_c = 0.00621307$

c = confinement factor = 1.42131

$y_1 = 0.0025$

$sh_1 = 0.008$

$ft_1 = 900.1904$

$fy_1 = 750.1586$

$su_1 = 0.032$

using (30) in Biskinis/Fardis (2013) multiplied with shear_factor

and also multiplied by the shear_factor according to 15.7.1.4, with

Shear_factor = 1.00

$l_o/l_{ou,min} = l_b/l_d = 1.00$

$su_1 = 0.4 \cdot esu_{1,nominal}$ ((5.5), TBDY) = 0.032

From table 5A.1, TBDY: $esu_{1,nominal} = 0.08$,

For calculation of $esu_{1,nominal}$ and y_1 , sh_1 , ft_1 , fy_1 , it is considered characteristic value $fsy_1 = fs_1/1.2$, from table 5.1, TBDY.

y_1 , sh_1 , ft_1 , fy_1 , are also multiplied by $\text{Min}(1, 1.25 \cdot (l_b/l_d)^{2/3})$, from 10.3.5, ASCE 41-17.

```

with fs1 = (fs,jacket*Asl,ten,jacket + fs,core*Asl,ten,core)/Asl,ten = 750.1586
with Es1 = (Es,jacket*Asl,ten,jacket + Es,core*Asl,ten,core)/Asl,ten = 200000.00
y2 = 0.0025
sh2 = 0.008
ft2 = 907.50
fy2 = 756.25
su2 = 0.032
using (30) in Biskinis/Fardis (2013) multiplied with shear_factor
and also multiplied by the shear_factor according to 15.7.1.4, with
Shear_factor = 1.00
lo/lou,min = lb/lb,min = 1.00
su2 = 0.4*esu2_nominal ((5.5), TBDY) = 0.032
From table 5A.1, TBDY: esu2_nominal = 0.08,
For calculation of esu2_nominal and y2, sh2,ft2,fy2, it is considered
characteristic value fsy2 = fs2/1.2, from table 5.1, TBDY.
y1, sh1,ft1,fy1, are also multiplied by Min(1,1.25*(lb/lb)^2/3), from 10.3.5, ASCE 41-17.
with fs2 = (fs,jacket*Asl,com,jacket + fs,core*Asl,com,core)/Asl,com = 756.25
with Es2 = (Es,jacket*Asl,com,jacket + Es,core*Asl,com,core)/Asl,com = 200000.00
yv = 0.0025
shv = 0.008
ftv = 902.993
fyv = 752.4941
suv = 0.032
using (30) in Biskinis/Fardis (2013) multiplied with shear_factor
and also multiplied by the shear_factor according to 15.7.1.4, with
Shear_factor = 1.00
lo/lou,min = lb/lb = 1.00
suv = 0.4*esuv_nominal ((5.5), TBDY) = 0.032
From table 5A.1, TBDY: esuv_nominal = 0.08,
considering characteristic value fsyv = fsv/1.2, from table 5.1, TBDY
For calculation of esuv_nominal and yv, shv,ftv,fyv, it is considered
characteristic value fsyv = fsv/1.2, from table 5.1, TBDY.
y1, sh1,ft1,fy1, are also multiplied by Min(1,1.25*(lb/lb)^2/3), from 10.3.5, ASCE 41-17.
with fsv = (fs,jacket*Asl,mid,jacket + fs,mid*Asl,mid,core)/Asl,mid = 752.4941
with Esv = (Es,jacket*Asl,mid,jacket + Es,mid*Asl,mid,core)/Asl,mid = 200000.00
1 = Asl,ten/(b*d)*(fs1/fc) = 0.32428344
2 = Asl,com/(b*d)*(fs2/fc) = 0.20328574
v = Asl,mid/(b*d)*(fsv/fc) = 0.35171278
and confined core properties:
b = 390.00
d = 677.00
d' = 13.00
fcc (5A.2, TBDY) = 25.58352
cc (5A.5, TBDY) = 0.00621307
c = confinement factor = 1.42131
1 = Asl,ten/(b*d)*(fs1/fc) = 0.39075399
2 = Asl,com/(b*d)*(fs2/fc) = 0.24495458
v = Asl,mid/(b*d)*(fsv/fc) = 0.42380571
Case/Assumption: Unconfined full section - Steel rupture
' satisfies Eq. (4.3)
---->
v < vs,y2 - LHS eq.(4.5) is not satisfied
---->
v < vs,c - RHS eq.(4.5) is not satisfied
---->
Case/Assumption Rejected.
---->
New Case/Assumption: Unconfined full section - Spalling of concrete cover
' satisfies Eq. (4.4)
---->
v < s,y1 - LHS eq.(4.7) is not satisfied
---->
v < vc,y1 - RHS eq.(4.6) is not satisfied
---->
cu (4.11) = 0.95920575
MRc (4.18) = 7.7321E+008
---->

```


New Case/Assumption: Ultimate curvature of confined core after spalling of concrete cover

In expressions below, the following modifications have been made

- b, d, d' replaced by geometric parameters of the core: bo, do, d'o
- N, 1, 2, v normalised to bo*do, instead of b*d
- parameters of confined concrete, fcc, cc, used in lieu of fc, ecu

--->

Subcase: Rupture of tension steel

--->

$v^* < v^*s_y2$ - LHS eq.(4.5) is not satisfied

--->

$v^* < v^*s_c$ - LHS eq.(4.5) is not satisfied

--->

Subcase rejected

--->

New Subcase: Failure of compression zone

--->

$v^* < v^*c_y2$ - LHS eq.(4.6) is not satisfied

--->

$v^* < v^*c_y1$ - RHS eq.(4.6) is not satisfied

--->

ϕ_{cu} (4.11) = 1.00

M_{Ro} (4.18) = 8.9683E+008

--->

$\phi_u = \phi_{cu}$ (4.2) = 9.1773508E-006

$\mu_u = M_{Ro}$

Calculation of ratio lb/d

Adequate Lap Length: lb/d >= 1

Calculation of Shear Strength $V_r = \min(V_{r1}, V_{r2}) = 1.1678E+006$

Calculation of Shear Strength at edge 1, $V_{r1} = 1.1678E+006$

$V_{r1} = V_{Col}$ ((10.3), ASCE 41-17) = knl*V_{Col0}

V_{Col0} = 1.1678E+006

knl = 1 (zero step-static loading)

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

= 1 (normal-weight concrete)

Mean concrete strength: $f'_c = (f'_{c_jacket} \cdot Area_{jacket} + f'_{c_core} \cdot Area_{core}) / Area_{section} = 15.20$, but $f'_c^{0.5} \leq 8.3$
MPa (22.5.3.1, ACI 318-14)

M/Vd = 4.00

$\mu_u = 7.6104E+008$

$V_u = 227.4778$

d = 0.8*h = 600.00

$N_u = 7.6839E+006$

Ag = 337500.00

From (11.5.4.8), ACI 318-14: $V_s = V_{s,jacket} + V_{s,core} = 1.0354E+006$

where:

$V_{s,jacket} = V_{s,j1} + V_{s,j2} = 942477.796$

$V_{s,j1} = 589048.623$ is calculated for section web jacket, with:

d = 600.00

$A_v = 157079.633$

$f_y = 625.00$

s = 100.00

$V_{s,j1}$ is multiplied by Col,j1 = 1.00

s/d = 0.16666667

$V_{s,j2} = 353429.174$ is calculated for section flange jacket, with:

d = 360.00

$A_v = 157079.633$

$f_y = 625.00$

$s = 100.00$
 V_{sj2} is multiplied by $Col,j2 = 1.00$
 $s/d = 0.27777778$
 $V_{s,core} = V_{s,c1} + V_{s,c2} = 92890.612$
 $V_{s,c1} = 92890.612$ is calculated for section web core, with:
 $d = 440.00$
 $A_v = 100530.965$
 $f_y = 525.00$
 $s = 250.00$
 $V_{s,c1}$ is multiplied by $Col,c1 = 1.00$
 $s/d = 0.56818182$
 $V_{s,c2} = 0.00$ is calculated for section flange core, with:
 $d = 200.00$
 $A_v = 100530.965$
 $f_y = 525.00$
 $s = 250.00$
 $V_{s,c2}$ is multiplied by $Col,c2 = 0.00$
 $s/d = 1.25$
 $V_f ((11-3)-(11.4), ACI 440) = 0.00$
 From (11-11), ACI 440: $V_s + V_f \leq 699281.943$
 $bw = 450.00$

Calculation of Shear Strength at edge 2, $V_{r2} = 1.1678E+006$
 $V_{r2} = V_{Col} ((10.3), ASCE 41-17) = knl * V_{Col0}$
 $V_{Col0} = 1.1678E+006$
 $knl = 1$ (zero step-static loading)

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

$= 1$ (normal-weight concrete)
 Mean concrete strength: $fc' = (fc'_{jacket} * Area_{jacket} + fc'_{core} * Area_{core}) / Area_{section} = 15.20$, but $fc'^{0.5} \leq 8.3$
 MPa (22.5.3.1, ACI 318-14)
 $M/Vd = 4.00$
 $M_u = 6.8991E+008$
 $V_u = 227.4778$
 $d = 0.8 * h = 600.00$
 $N_u = 7.6839E+006$
 $Ag = 337500.00$
 From (11.5.4.8), ACI 318-14: $V_s = V_{sjacket} + V_{s,core} = 1.0354E+006$
 where:
 $V_{sjacket} = V_{sj1} + V_{sj2} = 942477.796$
 $V_{sj1} = 589048.623$ is calculated for section web jacket, with:
 $d = 600.00$
 $A_v = 157079.633$
 $f_y = 625.00$
 $s = 100.00$
 V_{sj1} is multiplied by $Col,j1 = 1.00$
 $s/d = 0.16666667$
 $V_{sj2} = 353429.174$ is calculated for section flange jacket, with:
 $d = 360.00$
 $A_v = 157079.633$
 $f_y = 625.00$
 $s = 100.00$
 V_{sj2} is multiplied by $Col,j2 = 1.00$
 $s/d = 0.27777778$
 $V_{s,core} = V_{s,c1} + V_{s,c2} = 92890.612$
 $V_{s,c1} = 92890.612$ is calculated for section web core, with:
 $d = 440.00$
 $A_v = 100530.965$
 $f_y = 525.00$
 $s = 250.00$
 $V_{s,c1}$ is multiplied by $Col,c1 = 1.00$
 $s/d = 0.56818182$
 $V_{s,c2} = 0.00$ is calculated for section flange core, with:
 $d = 200.00$

Av = 100530.965

fy = 525.00

s = 250.00

Vs,c2 is multiplied by Col,c2 = 0.00

s/d = 1.25

Vf ((11-3)-(11.4), ACI 440) = 0.00

From (11-11), ACI 440: Vs + Vf <= 699281.943

bw = 450.00

End Of Calculation of Shear Capacity ratio for element: column JTC1 of floor 1

At local axis: 3

Start Of Calculation of Shear Capacity ratio for element: column JTC1 of floor 1

At Shear local axis: 2

(Bending local axis: 3)

Section Type: rcjtcs

Constant Properties

Knowledge Factor, $\phi = 0.80$

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

Consequently:

Jacket

New material of Primary Member: Concrete Strength, $f_c = f_{cm} = 18.00$

New material of Primary Member: Steel Strength, $f_s = f_{sm} = 625.00$

Concrete Elasticity, $E_c = 19940.411$

Steel Elasticity, $E_s = 200000.00$

Existing Column

Existing material of Primary Member: Concrete Strength, $f_c = f_{cm} = 12.00$

Existing material of Primary Member: Steel Strength, $f_s = f_{sm} = 525.00$

Concrete Elasticity, $E_c = 16281.278$

Steel Elasticity, $E_s = 200000.00$

#####

Note: Especially for the calculation of moment strengths,

the above steel re-bar strengths are multiplied by 1.25 according to R18.6.5, ACI 318-14

Jacket

New material: Steel Strength, $f_s = 1.25 \cdot f_{sm} = 781.25$

Existing Column

Existing material: Steel Strength, $f_s = 1.25 \cdot f_{sm} = 656.25$

#####

Max Height, $H_{max} = 750.00$

Min Height, $H_{min} = 450.00$

Max Width, $W_{max} = 950.00$

Min Width, $W_{min} = 450.00$

Eccentricity, $Ecc = 250.00$

Jacket Thickness, $t_j = 100.00$

Cover Thickness, $c = 25.00$

Mean Confinement Factor overall section = 1.42131

Element Length, $L = 3000.00$

Primary Member

Ribbed Bars

Ductile Steel

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

Longitudinal Bars With Ends Lapped Starting at the End Sections

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

No FRP Wrapping

Stepwise Properties

At local axis: 2

EDGE -A-

Shear Force, $V_a = -137.4894$

EDGE -B-

Shear Force, $V_b = 137.4894$

BOTH EDGES

Axial Force, $F = -7.6839E+006$

Longitudinal Reinforcement Area Distribution (in 2 divisions)

-Tension: $As_t = 314.1593$

-Compression: $As_c = 6176.371$

Longitudinal Reinforcement Area Distribution (in 3 divisions)

-Tension: $As_{t,ten} = 1539.38$

-Compression: $As_{c,com} = 1539.38$

-Middle: $As_{c,mid} = 3612.832$

Calculation of Shear Capacity ratio, $V_e/V_r = 0.86614729$

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

Calculation of Shear Demand from fig. R18.6.5, ACI 318-14 $V_e = (M_{pr1} + M_{pr2})/l_n = 1.2287E+006$ with

$M_{pr1} = \text{Max}(\mu_{u1+}, \mu_{u1-}) = 1.8431E+009$

$\mu_{u1+} = 1.8431E+009$, is the ultimate moment strength at the edge 1 of the member in the actual moment direction which is defined for the static loading combination

$\mu_{u1-} = 1.8431E+009$, is the ultimate moment strength at the edge 1 of the member in the opposite moment direction which is defined for the static loading combination

$M_{pr2} = \text{Max}(\mu_{u2+}, \mu_{u2-}) = 1.8431E+009$

$\mu_{u2+} = 1.8431E+009$, is the ultimate moment strength at the edge 2 of the member in the actual moment direction which is defined for the static loading combination

$\mu_{u2-} = 1.8431E+009$, is the ultimate moment strength at the edge 2 of the member in the opposite moment direction which is defined for the static loading combination

Calculation of μ_{u1+}

Calculation of ultimate curvature μ_u according to 4.1, Biskinis/Fardis 2013:

$\mu_u = 7.8730391E-006$

$\mu_u = 1.8431E+009$

with full section properties:

$b = 450.00$

$d = 907.00$

$d' = 43.00$

$v = 1.0459$

$N = 7.6839E+006$

$f_c = 18.00$

$\alpha = 0.002$

Final value of μ_u : $\mu_u^* = \text{shear_factor} * \text{Max}(\mu_u, \alpha) = 0.01486841$

The Shear_factor is considered equal to 1 (pure moment strength)

From (5.4b), TBDY: $\mu_u = 0.01486841$

μ_u (5.4c) = 0.08077545

μ_u (5.4d), TBDY = $(\mu_{u1} * A_{ext} + \mu_{u2} * A_{int}) / A_{sec} = 0.53375773$

$\mu_{u1} = \text{Max}((A_{conf,max1} - A_{noConf1}) / A_{conf,max1} * (A_{conf,min1} / A_{conf,max1}), 0) = 0.53375773$

The definitions of A_{noConf} , $A_{conf,min}$ and $A_{conf,max}$ are derived from generalization of the rectangular sections confinement, which is expressed by (5.4d).

The generalization is done according to Mander, J., Priestley, M., and Park, R. (1988)

"Theoretical Stress-Strain Model for Confined Concrete."

J. Struct. Eng., 10.1061/(ASCE)0733-9445(1988)114:8(1804), 1804-1826.

$A_{conf,max1} = 464100.00$ is the confined core area at levels of member with hoops and is calculated as the area of core enclosed by the center lines of the external perimeter hoops.

$A_{conf,min1} = 395025.00$ is the confined core area at midway between the levels of hoops and is calculated by reducing all the dimensions of the area $A_{conf,max1}$ by a length equal to half the clear spacing between external hoops.

$A_{noConf1} = 173066.667$ is the unconfined external core area which is equal to $b^2/6$ as defined at (A.2).

$\mu_{u2} (> \mu_{u1}) = \text{Max}((A_{conf,max2} - A_{noConf2}) / A_{conf,max2} * (A_{conf,min2} / A_{conf,max2}), 0) = 0.53375773$

The definitions of A_{noConf} , $A_{conf,min}$ and $A_{conf,max}$ are derived from generalization of the rectangular sections confinement, which is expressed by (5.4d).

The generalization is done according to Mander, J., Priestley, M., and Park, R. (1988)
 "Theoretical Stress-Strain Model for Confined Concrete."

J. Struct. Eng., 10.1061/(ASCE)0733-9445(1988)114:8(1804), 1804-1826.

$A_{conf,max2} = 252164.00$ is the confined core area at levels of member with hoops and
 is calculated as the area of core enclosed by the center lines of the internal perimeter hoops.

$A_{conf,min2} = 111441.00$ is the confined core area at midway between the levels of hoops and
 is calculated by reducing all the dimensions of the area $A_{conf,max2}$ by a length
 equal to half the clear spacing between internal hoops.

$A_{noConf2} = 110709.333$ is the unconfined internal core area which is equal to $b_i^2/6$ as defined at (A.2).
 $psh_{min} \cdot F_{ywe} = \min(psh_x \cdot F_{ywe}, psh_y \cdot F_{ywe}) = 2.724$

$psh_x \cdot F_{ywe} = psh1 \cdot F_{ywe1} + ps2 \cdot F_{ywe2} = 2.724$
 $psh1 ((5.4d), TBDY) = L_{stir1} \cdot A_{stir1} / (A_{sec} \cdot s1) = 0.00301593$
 L_{stir1} (Length of stirrups along Y) = 2160.00
 A_{stir1} (stirrups area) = 78.53982
 $psh2 (5.4d) = L_{stir2} \cdot A_{stir2} / (A_{sec} \cdot s2) = 0.00056047$
 L_{stir2} (Length of stirrups along Y) = 1568.00
 A_{stir2} (stirrups area) = 50.26548

$psh_y \cdot F_{ywe} = psh1 \cdot F_{ywe1} + ps2 \cdot F_{ywe2} = 3.25416$
 $psh1 ((5.4d), TBDY) = L_{stir1} \cdot A_{stir1} / (A_{sec} \cdot s1) = 0.00357443$
 L_{stir1} (Length of stirrups along X) = 2560.00
 A_{stir1} (stirrups area) = 78.53982
 $psh2 ((5.4d), TBDY) = L_{stir2} \cdot A_{stir2} / (A_{sec} \cdot s2) = 0.00070345$
 L_{stir2} (Length of stirrups along X) = 1968.00
 A_{stir2} (stirrups area) = 50.26548

$A_{sec} = 562500.00$

$s1 = 100.00$

$s2 = 250.00$

$f_{ywe1} = 781.25$

$f_{ywe2} = 656.25$

$f_{ce} = 18.00$

From ((5.A5), TBDY), TBDY: $cc = 0.00621307$

c = confinement factor = 1.42131

$y1 = 0.0025$

$sh1 = 0.008$

$ft1 = 907.50$

$fy1 = 756.25$

$su1 = 0.032$

using (30) in Biskinis/Fardis (2013) multiplied with shear_factor
 and also multiplied by the shear_factor according to 15.7.1.4, with
 Shear_factor = 1.00

$lo/lo_{min} = lb/ld = 1.00$

$su1 = 0.4 \cdot esu1_{nominal} ((5.5), TBDY) = 0.032$

From table 5A.1, TBDY: $esu1_{nominal} = 0.08$,

For calculation of $esu1_{nominal}$ and $y1, sh1, ft1, fy1$, it is considered
 characteristic value $fsy1 = fs1/1.2$, from table 5.1, TBDY.

$y1, sh1, ft1, fy1$, are also multiplied by $\min(1, 1.25 \cdot (lb/ld)^{2/3})$, from 10.3.5, ASCE 41-17.

with $fs1 = (fs_{jacket} \cdot A_{sl,ten,jacket} + fs_{core} \cdot A_{sl,ten,core}) / A_{sl,ten} = 756.25$

with $Es1 = (Es_{jacket} \cdot A_{sl,ten,jacket} + Es_{core} \cdot A_{sl,ten,core}) / A_{sl,ten} = 200000.00$

$y2 = 0.0025$

$sh2 = 0.008$

$ft2 = 907.50$

$fy2 = 756.25$

$su2 = 0.032$

using (30) in Biskinis/Fardis (2013) multiplied with shear_factor
 and also multiplied by the shear_factor according to 15.7.1.4, with
 Shear_factor = 1.00

$lo/lo_{min} = lb/lb_{min} = 1.00$

$su2 = 0.4 \cdot esu2_{nominal} ((5.5), TBDY) = 0.032$

From table 5A.1, TBDY: $esu2_{nominal} = 0.08$,

For calculation of $esu2_{nominal}$ and $y2, sh2, ft2, fy2$, it is considered
 characteristic value $fsy2 = fs2/1.2$, from table 5.1, TBDY.

$y1, sh1, ft1, fy1$, are also multiplied by $\min(1, 1.25 \cdot (lb/ld)^{2/3})$, from 10.3.5, ASCE 41-17.

with $fs2 = (fs_{jacket} \cdot A_{sl,com,jacket} + fs_{core} \cdot A_{sl,com,core}) / A_{sl,com} = 756.25$

with $Es2 = (Es_{jacket} \cdot A_{sl,com,jacket} + Es_{core} \cdot A_{sl,com,core}) / A_{sl,com} = 200000.00$

```

yv = 0.0025
shv = 0.008
ftv = 899.1522
fyv = 749.2935
suv = 0.032
    using (30) in Biskinis/Fardis (2013) multiplied with shear_factor
    and also multiplied by the shear_factor according to 15.7.1.4, with
    Shear_factor = 1.00
    lo/lou,min = lb/ld = 1.00
    suv = 0.4*esuv_nominal ((5.5), TBDY) = 0.032
    From table 5A.1, TBDY: esuv_nominal = 0.08,
    considering characteristic value fsyv = fsv/1.2, from table 5.1, TBDY
    For calculation of esuv_nominal and yv, shv,ftv,fyv, it is considered
    characteristic value fsyv = fsv/1.2, from table 5.1, TBDY.
    y1, sh1,ft1,fy1, are also multiplied by Min(1,1.25*(lb/ld)^2/3), from 10.3.5, ASCE 41-17.
    with fsv = (fs,jacket*Asl,mid,jacket + fs,mid*Asl,mid,core)/Asl,mid = 749.2935
    with Esv = (Es,jacket*Asl,mid,jacket + Es,mid*Asl,mid,core)/Asl,mid = 200000.00
    1 = Asl,ten/(b*d)*(fs1/fc) = 0.15845977
    2 = Asl,com/(b*d)*(fs2/fc) = 0.15845977
    v = Asl,mid/(b*d)*(fsv/fc) = 0.36847443
and confined core properties:
b = 390.00
d = 877.00
d' = 13.00
fcc (5A.2, TBDY) = 25.58352
cc (5A.5, TBDY) = 0.00621307
    c = confinement factor = 1.42131
    1 = Asl,ten/(b*d)*(fs1/fc) = 0.18909264
    2 = Asl,com/(b*d)*(fs2/fc) = 0.18909264
    v = Asl,mid/(b*d)*(fsv/fc) = 0.43970658
Case/Assumption: Unconfined full section - Steel rupture
' satisfies Eq. (4.3)
--->
v < vs,y2 - LHS eq.(4.5) is not satisfied
--->
v < vs,c - RHS eq.(4.5) is not satisfied
--->
Case/Assumption Rejected.
--->
New Case/Assumption: Unconfined full section - Spalling of concrete cover
' satisfies Eq. (4.4)
--->
v < s,y1 - LHS eq.(4.7) is not satisfied
--->
v < vc,y1 - RHS eq.(4.6) is satisfied
--->
cu (4.10) = 0.83934343
MRc (4.17) = 1.8439E+009
--->
New Case/Assumption: Ultimate curvature of confined core after spalling of concrete cover
In expressions below, the following modifications have been made
- b, d, d' replaced by geometric parameters of the core: bo, do, d'o
- N, 1, 2, v normalised to bo*do, instead of b*d
- parameters of confined concrete, fcc, cc, used in lieu of fc, ecu
--->
Subcase: Rupture of tension steel
--->
v* < v*s,y2 - LHS eq.(4.5) is not satisfied
--->
v* < v*s,c - LHS eq.(4.5) is not satisfied
--->
Subcase rejected
--->
New Subcase: Failure of compression zone
--->
v* < v*c,y2 - LHS eq.(4.6) is not satisfied
--->

```

$v^* < v^*_{c,y1}$ - RHS eq.(4.6) is not satisfied

--->

$$*cu(4.11) = 0.89983732$$

$$MRo(4.18) = 1.8431E+009$$

--->

$$u = cu(4.2) = 7.8730391E-006$$

$$Mu = MRo$$

Calculation of ratio lb/ld

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

Calculation of $Mu1$ -

Calculation of ultimate curvature u according to 4.1, Biskinis/Fardis 2013:

$$u = 7.8730391E-006$$

$$Mu = 1.8431E+009$$

with full section properties:

$$b = 450.00$$

$$d = 907.00$$

$$d' = 43.00$$

$$v = 1.0459$$

$$N = 7.6839E+006$$

$$fc = 18.00$$

$$co(5A.5, TBDY) = 0.002$$

$$\text{Final value of } cu: cu^* = \text{shear_factor} * \text{Max}(cu, cc) = 0.01486841$$

The Shear_factor is considered equal to 1 (pure moment strength)

$$\text{From (5.4b), TBDY: } cu = 0.01486841$$

$$we(5.4c) = 0.08077545$$

$$ase((5.4d), TBDY) = (ase1 * A_{ext} + ase2 * A_{int}) / A_{sec} = 0.53375773$$

$$ase1 = \text{Max}(((A_{conf,max1} - A_{noConf1}) / A_{conf,max1}) * (A_{conf,min1} / A_{conf,max1}), 0) = 0.53375773$$

The definitions of A_{noConf} , $A_{conf,min}$ and $A_{conf,max}$ are derived from generalization of the rectangular sections confinement, which is expressed by (5.4d).

The generalization is done according to Mander, J., Priestley, M., and Park, R. (1988)

"Theoretical Stress-Strain Model for Confined Concrete."

J. Struct. Eng., 10.1061/(ASCE)0733-9445(1988)114:8(1804), 1804-1826.

$A_{conf,max1} = 464100.00$ is the confined core area at levels of member with hoops and is calculated as the area of core enclosed by the center lines of the external perimeter hoops.

$A_{conf,min1} = 395025.00$ is the confined core area at midway between the levels of hoops and is calculated by reducing all the dimensions of the area $A_{conf,max1}$ by a length equal to half the clear spacing between external hoops.

$A_{noConf1} = 173066.667$ is the unconfined external core area which is equal to $b_i^2/6$ as defined at (A.2).

$$ase2 (\geq ase1) = \text{Max}(((A_{conf,max2} - A_{noConf2}) / A_{conf,max2}) * (A_{conf,min2} / A_{conf,max2}), 0) = 0.53375773$$

The definitions of A_{noConf} , $A_{conf,min}$ and $A_{conf,max}$ are derived from generalization of the rectangular sections confinement, which is expressed by (5.4d).

The generalization is done according to Mander, J., Priestley, M., and Park, R. (1988)

"Theoretical Stress-Strain Model for Confined Concrete."

J. Struct. Eng., 10.1061/(ASCE)0733-9445(1988)114:8(1804), 1804-1826.

$A_{conf,max2} = 252164.00$ is the confined core area at levels of member with hoops and is calculated as the area of core enclosed by the center lines of the internal perimeter hoops.

$A_{conf,min2} = 111441.00$ is the confined core area at midway between the levels of hoops and is calculated by reducing all the dimensions of the area $A_{conf,max2}$ by a length equal to half the clear spacing between internal hoops.

$A_{noConf2} = 110709.333$ is the unconfined internal core area which is equal to $b_i^2/6$ as defined at (A.2).

$$psh_{min} * Fy_{we} = \text{Min}(psh_x * Fy_{we}, psh_y * Fy_{we}) = 2.724$$

$$psh_x * Fy_{we} = psh1 * Fy_{we1} + psh2 * Fy_{we2} = 2.724$$

$$psh1((5.4d), TBDY) = L_{stir1} * A_{stir1} / (A_{sec} * s1) = 0.00301593$$

$$L_{stir1} (\text{Length of stirrups along } Y) = 2160.00$$

$$A_{stir1} (\text{stirrups area}) = 78.53982$$

$$psh2(5.4d) = L_{stir2} * A_{stir2} / (A_{sec} * s2) = 0.00056047$$

Lstir2 (Length of stirrups along Y) = 1568.00
Astir2 (stirrups area) = 50.26548

psh_y*Fywe = psh1*Fywe1+ps2*Fywe2 = 3.25416
psh1 ((5.4d), TBDY) = Lstir1*Astir1/(Asec*s1) = 0.00357443
Lstir1 (Length of stirrups along X) = 2560.00
Astir1 (stirrups area) = 78.53982
psh2 ((5.4d), TBDY) = Lstir2*Astir2/(Asec*s2) = 0.00070345
Lstir2 (Length of stirrups along X) = 1968.00
Astir2 (stirrups area) = 50.26548

Asec = 562500.00
s1 = 100.00
s2 = 250.00

fywe1 = 781.25
fywe2 = 656.25
fce = 18.00

From ((5.A5), TBDY), TBDY: cc = 0.00621307
c = confinement factor = 1.42131

y1 = 0.0025
sh1 = 0.008
ft1 = 907.50
fy1 = 756.25
su1 = 0.032

using (30) in Biskinis/Fardis (2013) multiplied with shear_factor
and also multiplied by the shear_factor according to 15.7.1.4, with
Shear_factor = 1.00

lo/lou,min = lb/ld = 1.00

su1 = 0.4*esu1_nominal ((5.5), TBDY) = 0.032

From table 5A.1, TBDY: esu1_nominal = 0.08,

For calculation of esu1_nominal and y1, sh1,ft1,fy1, it is considered
characteristic value fsy1 = fs1/1.2, from table 5.1, TBDY.

y1, sh1,ft1,fy1, are also multiplied by $\text{Min}(1, 1.25 \cdot (lb/ld)^{2/3})$, from 10.3.5, ASCE 41-17.

with fs1 = (fs,jacket*Asl,ten,jacket + fs,core*Asl,ten,core)/Asl,ten = 756.25

with Es1 = (Es,jacket*Asl,ten,jacket + Es,core*Asl,ten,core)/Asl,ten = 200000.00

y2 = 0.0025
sh2 = 0.008
ft2 = 907.50
fy2 = 756.25
su2 = 0.032

using (30) in Biskinis/Fardis (2013) multiplied with shear_factor
and also multiplied by the shear_factor according to 15.7.1.4, with
Shear_factor = 1.00

lo/lou,min = lb/lb,min = 1.00

su2 = 0.4*esu2_nominal ((5.5), TBDY) = 0.032

From table 5A.1, TBDY: esu2_nominal = 0.08,

For calculation of esu2_nominal and y2, sh2,ft2,fy2, it is considered
characteristic value fsy2 = fs2/1.2, from table 5.1, TBDY.

y1, sh1,ft1,fy1, are also multiplied by $\text{Min}(1, 1.25 \cdot (lb/ld)^{2/3})$, from 10.3.5, ASCE 41-17.

with fs2 = (fs,jacket*Asl,com,jacket + fs,core*Asl,com,core)/Asl,com = 756.25

with Es2 = (Es,jacket*Asl,com,jacket + Es,core*Asl,com,core)/Asl,com = 200000.00

yv = 0.0025
shv = 0.008
ftv = 899.1522
fyv = 749.2935
suv = 0.032

using (30) in Biskinis/Fardis (2013) multiplied with shear_factor
and also multiplied by the shear_factor according to 15.7.1.4, with
Shear_factor = 1.00

lo/lou,min = lb/ld = 1.00

suv = 0.4*esuv_nominal ((5.5), TBDY) = 0.032

From table 5A.1, TBDY: esuv_nominal = 0.08,

considering characteristic value fsyv = fsv/1.2, from table 5.1, TBDY
For calculation of esuv_nominal and yv, shv,ftv,fyv, it is considered
characteristic value fsyv = fsv/1.2, from table 5.1, TBDY.

y1, sh1,ft1,fy1, are also multiplied by $\text{Min}(1, 1.25 \cdot (lb/ld)^{2/3})$, from 10.3.5, ASCE 41-17.

with fsv = (fs,jacket*Asl,mid,jacket + fs,mid*Asl,mid,core)/Asl,mid = 749.2935


```

with Esv = (Esjacket*Aslmid,jacket + Esmid*Aslmid,core)/Aslmid = 200000.00
1 = Aslten/(b*d)*(fs1/fc) = 0.15845977
2 = Aslcom/(b*d)*(fs2/fc) = 0.15845977
v = Aslmid/(b*d)*(fsv/fc) = 0.36847443
and confined core properties:
b = 390.00
d = 877.00
d' = 13.00
fcc (5A.2, TBDY) = 25.58352
cc (5A.5, TBDY) = 0.00621307
c = confinement factor = 1.42131
1 = Aslten/(b*d)*(fs1/fc) = 0.18909264
2 = Aslcom/(b*d)*(fs2/fc) = 0.18909264
v = Aslmid/(b*d)*(fsv/fc) = 0.43970658
Case/Assumption: Unconfined full section - Steel rupture
' satisfies Eq. (4.3)
--->
v < vs,y2 - LHS eq.(4.5) is not satisfied
--->
v < vs,c - RHS eq.(4.5) is not satisfied
--->
Case/Assumption Rejected.
--->
New Case/Assumption: Unconfined full section - Spalling of concrete cover
' satisfies Eq. (4.4)
--->
v < sy1 - LHS eq.(4.7) is not satisfied
--->
v < vc,y1 - RHS eq.(4.6) is satisfied
--->
cu (4.10) = 0.83934343
MRc (4.17) = 1.8439E+009
--->
New Case/Assumption: Ultimate curvature of confined core after spalling of concrete cover
In expressions below, the following modifications have been made
- b, d, d' replaced by geometric parameters of the core: bo, do, d'o
- N1, 1, 2, v normalised to bo*do, instead of b*d
- - parameters of confined concrete, fcc, cc, used in lieu of fc, ecu
--->
Subcase: Rupture of tension steel
--->
v* < v*sy2 - LHS eq.(4.5) is not satisfied
--->
v* < v*sc - LHS eq.(4.5) is not satisfied
--->
Subcase rejected
--->
New Subcase: Failure of compression zone
--->
v* < v*cy2 - LHS eq.(4.6) is not satisfied
--->
v* < v*cy1 - RHS eq.(4.6) is not satisfied
--->
*cu (4.11) = 0.89983732
MRo (4.18) = 1.8431E+009
--->
u = cu (4.2) = 7.8730391E-006
Mu = MRo

```

Calculation of ratio lb/d

Adequate Lap Length: lb/d >= 1

Calculation of Mu2+

Calculation of ultimate curvature ϕ_u according to 4.1, Biskinis/Fardis 2013:

$$\phi_u = 7.8730391E-006$$

$$\mu = 1.8431E+009$$

with full section properties:

$$b = 450.00$$

$$d = 907.00$$

$$d' = 43.00$$

$$\nu = 1.0459$$

$$N = 7.6839E+006$$

$$f_c = 18.00$$

$$\alpha (5A.5, TBDY) = 0.002$$

$$\text{Final value of } \phi_u: \phi_u^* = \text{shear_factor} * \text{Max}(\phi_u, \phi_c) = 0.01486841$$

The Shear_factor is considered equal to 1 (pure moment strength)

$$\text{From (5.4b), TBDY: } \phi_u = 0.01486841$$

$$\phi_{ue} (5.4c) = 0.08077545$$

$$\alpha_{se} ((5.4d), TBDY) = (\alpha_{se1} * A_{ext} + \alpha_{se2} * A_{int}) / A_{sec} = 0.53375773$$

$$\alpha_{se1} = \text{Max}(((A_{conf,max1} - A_{noConf1}) / A_{conf,max1}) * (A_{conf,min1} / A_{conf,max1}), 0) = 0.53375773$$

The definitions of A_{noConf} , $A_{conf,min}$ and $A_{conf,max}$ are derived from generalization of the rectangular sections confinement, which is expressed by (5.4d).

The generalization is done according to Mander, J., Priestley, M., and Park, R. (1988)

"Theoretical Stress-Strain Model for Confined Concrete."

J. Struct. Eng., 10.1061/(ASCE)0733-9445(1988)114:8(1804), 1804-1826.

$A_{conf,max1} = 464100.00$ is the confined core area at levels of member with hoops and is calculated as the area of core enclosed by the center lines of the external perimeter hoops.

$A_{conf,min1} = 395025.00$ is the confined core area at midway between the levels of hoops and is calculated by reducing all the dimensions of the area $A_{conf,max1}$ by a length equal to half the clear spacing between external hoops.

$A_{noConf1} = 173066.667$ is the unconfined external core area which is equal to $b_i^2/6$ as defined at (A.2).

$$\alpha_{se2} (> \alpha_{se1}) = \text{Max}(((A_{conf,max2} - A_{noConf2}) / A_{conf,max2}) * (A_{conf,min2} / A_{conf,max2}), 0) = 0.53375773$$

The definitions of A_{noConf} , $A_{conf,min}$ and $A_{conf,max}$ are derived from generalization of the rectangular sections confinement, which is expressed by (5.4d).

The generalization is done according to Mander, J., Priestley, M., and Park, R. (1988)

"Theoretical Stress-Strain Model for Confined Concrete."

J. Struct. Eng., 10.1061/(ASCE)0733-9445(1988)114:8(1804), 1804-1826.

$A_{conf,max2} = 252164.00$ is the confined core area at levels of member with hoops and is calculated as the area of core enclosed by the center lines of the internal perimeter hoops.

$A_{conf,min2} = 111441.00$ is the confined core area at midway between the levels of hoops and is calculated by reducing all the dimensions of the area $A_{conf,max2}$ by a length equal to half the clear spacing between internal hoops.

$A_{noConf2} = 110709.333$ is the unconfined internal core area which is equal to $b_i^2/6$ as defined at (A.2).

$$\phi_{sh,min} * F_{ywe} = \text{Min}(\phi_{sh,x} * F_{ywe}, \phi_{sh,y} * F_{ywe}) = 2.724$$

$$\phi_{sh,x} * F_{ywe} = \phi_{sh1} * F_{ywe1} + \phi_{sh2} * F_{ywe2} = 2.724$$

$$\phi_{sh1} ((5.4d), TBDY) = L_{stir1} * A_{stir1} / (A_{sec} * s_1) = 0.00301593$$

$$L_{stir1} \text{ (Length of stirrups along Y)} = 2160.00$$

$$A_{stir1} \text{ (stirrups area)} = 78.53982$$

$$\phi_{sh2} (5.4d) = L_{stir2} * A_{stir2} / (A_{sec} * s_2) = 0.00056047$$

$$L_{stir2} \text{ (Length of stirrups along Y)} = 1568.00$$

$$A_{stir2} \text{ (stirrups area)} = 50.26548$$

$$\phi_{sh,y} * F_{ywe} = \phi_{sh1} * F_{ywe1} + \phi_{sh2} * F_{ywe2} = 3.25416$$

$$\phi_{sh1} ((5.4d), TBDY) = L_{stir1} * A_{stir1} / (A_{sec} * s_1) = 0.00357443$$

$$L_{stir1} \text{ (Length of stirrups along X)} = 2560.00$$

$$A_{stir1} \text{ (stirrups area)} = 78.53982$$

$$\phi_{sh2} ((5.4d), TBDY) = L_{stir2} * A_{stir2} / (A_{sec} * s_2) = 0.00070345$$

$$L_{stir2} \text{ (Length of stirrups along X)} = 1968.00$$

$$A_{stir2} \text{ (stirrups area)} = 50.26548$$

$$A_{sec} = 562500.00$$

$$s_1 = 100.00$$

$$s_2 = 250.00$$

$$f_{ywe1} = 781.25$$

$$f_{ywe2} = 656.25$$

$f_{ce} = 18.00$
 From ((5.A.5), TBDY), TBDY: $cc = 0.00621307$
 $c = \text{confinement factor} = 1.42131$
 $y_1 = 0.0025$
 $sh_1 = 0.008$
 $ft_1 = 907.50$
 $fy_1 = 756.25$
 $su_1 = 0.032$
 using (30) in Biskinis/Fardis (2013) multiplied with shear_factor
 and also multiplied by the shear_factor according to 15.7.1.4, with
 $\text{Shear_factor} = 1.00$
 $lo/lou, \min = lb/ld = 1.00$
 $su_1 = 0.4 * esu_1, \text{nominal} ((5.5), \text{TBDY}) = 0.032$
 From table 5A.1, TBDY: $esu_1, \text{nominal} = 0.08$,
 For calculation of $esu_1, \text{nominal}$ and y_1, sh_1, ft_1, fy_1 , it is considered
 characteristic value $fsy_1 = fs_1/1.2$, from table 5.1, TBDY.
 y_1, sh_1, ft_1, fy_1 , are also multiplied by $\text{Min}(1, 1.25 * (lb/ld)^{2/3})$, from 10.3.5, ASCE 41-17.
 with $fs_1 = (fs, \text{jacket} * Asl, \text{ten, jacket} + fs, \text{core} * Asl, \text{ten, core}) / Asl, \text{ten} = 756.25$
 with $Es_1 = (Es, \text{jacket} * Asl, \text{ten, jacket} + Es, \text{core} * Asl, \text{ten, core}) / Asl, \text{ten} = 200000.00$
 $y_2 = 0.0025$
 $sh_2 = 0.008$
 $ft_2 = 907.50$
 $fy_2 = 756.25$
 $su_2 = 0.032$
 using (30) in Biskinis/Fardis (2013) multiplied with shear_factor
 and also multiplied by the shear_factor according to 15.7.1.4, with
 $\text{Shear_factor} = 1.00$
 $lo/lou, \min = lb/lb, \min = 1.00$
 $su_2 = 0.4 * esu_2, \text{nominal} ((5.5), \text{TBDY}) = 0.032$
 From table 5A.1, TBDY: $esu_2, \text{nominal} = 0.08$,
 For calculation of $esu_2, \text{nominal}$ and y_2, sh_2, ft_2, fy_2 , it is considered
 characteristic value $fsy_2 = fs_2/1.2$, from table 5.1, TBDY.
 y_1, sh_1, ft_1, fy_1 , are also multiplied by $\text{Min}(1, 1.25 * (lb/ld)^{2/3})$, from 10.3.5, ASCE 41-17.
 with $fs_2 = (fs, \text{jacket} * Asl, \text{com, jacket} + fs, \text{core} * Asl, \text{com, core}) / Asl, \text{com} = 756.25$
 with $Es_2 = (Es, \text{jacket} * Asl, \text{com, jacket} + Es, \text{core} * Asl, \text{com, core}) / Asl, \text{com} = 200000.00$
 $y_v = 0.0025$
 $sh_v = 0.008$
 $ft_v = 899.1522$
 $fy_v = 749.2935$
 $suv = 0.032$
 using (30) in Biskinis/Fardis (2013) multiplied with shear_factor
 and also multiplied by the shear_factor according to 15.7.1.4, with
 $\text{Shear_factor} = 1.00$
 $lo/lou, \min = lb/ld = 1.00$
 $suv = 0.4 * esuv, \text{nominal} ((5.5), \text{TBDY}) = 0.032$
 From table 5A.1, TBDY: $esuv, \text{nominal} = 0.08$,
 considering characteristic value $fsyv = fsv/1.2$, from table 5.1, TBDY
 For calculation of $esuv, \text{nominal}$ and y_v, sh_v, ft_v, fy_v , it is considered
 characteristic value $fsyv = fsv/1.2$, from table 5.1, TBDY.
 y_1, sh_1, ft_1, fy_1 , are also multiplied by $\text{Min}(1, 1.25 * (lb/ld)^{2/3})$, from 10.3.5, ASCE 41-17.
 with $fsv = (fs, \text{jacket} * Asl, \text{mid, jacket} + fs, \text{mid} * Asl, \text{mid, core}) / Asl, \text{mid} = 749.2935$
 with $Es_v = (Es, \text{jacket} * Asl, \text{mid, jacket} + Es, \text{mid} * Asl, \text{mid, core}) / Asl, \text{mid} = 200000.00$
 $1 = Asl, \text{ten} / (b * d) * (fs_1 / f_c) = 0.15845977$
 $2 = Asl, \text{com} / (b * d) * (fs_2 / f_c) = 0.15845977$
 $v = Asl, \text{mid} / (b * d) * (fsv / f_c) = 0.36847443$
 and confined core properties:
 $b = 390.00$
 $d = 877.00$
 $d' = 13.00$
 $f_{cc} (5A.2, \text{TBDY}) = 25.58352$
 $cc (5A.5, \text{TBDY}) = 0.00621307$
 $c = \text{confinement factor} = 1.42131$
 $1 = Asl, \text{ten} / (b * d) * (fs_1 / f_c) = 0.18909264$
 $2 = Asl, \text{com} / (b * d) * (fs_2 / f_c) = 0.18909264$
 $v = Asl, \text{mid} / (b * d) * (fsv / f_c) = 0.43970658$
 Case/Assumption: Unconfined full section - Steel rupture
 ' satisfies Eq. (4.3)

```

---->
v < vs,y2 - LHS eq.(4.5) is not satisfied
---->
v < vs,c - RHS eq.(4.5) is not satisfied
---->
Case/Assumption Rejected.
---->
New Case/Assumption: Unconfined full section - Spalling of concrete cover
'satisfies Eq. (4.4)
---->
v < s,y1 - LHS eq.(4.7) is not satisfied
---->
v < vc,y1 - RHS eq.(4.6) is satisfied
---->
cu (4.10) = 0.83934343
MRc (4.17) = 1.8439E+009
---->
New Case/Assumption: Ultimate curvature of confined core after spalling of concrete cover
In expressions below, the following modifications have been made
- b, d, d' replaced by geometric parameters of the core: bo, do, d'o
- N, 1, 2, v normalised to bo*do, instead of b*d
- parameters of confined concrete, fcc, cc, used in lieu of fc, ecu
---->
Subcase: Rupture of tension steel
---->
v* < v*s,y2 - LHS eq.(4.5) is not satisfied
---->
v* < v*s,c - LHS eq.(4.5) is not satisfied
---->
Subcase rejected
---->
New Subcase: Failure of compression zone
---->
v* < v*c,y2 - LHS eq.(4.6) is not satisfied
---->
v* < v*c,y1 - RHS eq.(4.6) is not satisfied
---->
*cu (4.11) = 0.89983732
MRo (4.18) = 1.8431E+009
---->
u = cu (4.2) = 7.8730391E-006
Mu = MRo

```

Calculation of ratio lb/d

Adequate Lap Length: lb/d >= 1

Calculation of Mu2-

Calculation of ultimate curvature u according to 4.1, Biskinis/Fardis 2013:

u = 7.8730391E-006
Mu = 1.8431E+009

with full section properties:

b = 450.00
d = 907.00
d' = 43.00
v = 1.0459
N = 7.6839E+006
fc = 18.00
co (5A.5, TBDY) = 0.002
Final value of cu: cu* = shear_factor * Max(cu, cc) = 0.01486841

The Shear_factor is considered equal to 1 (pure moment strength)

From (5.4b), TBDY: $c_u = 0.01486841$

w_e (5.4c) = 0.08077545

a_{se} ((5.4d), TBDY) = $(a_{se1} * A_{ext} + a_{se2} * A_{int}) / A_{sec} = 0.53375773$

$a_{se1} = \text{Max}(((A_{conf,max1} - A_{noConf1}) / A_{conf,max1}) * (A_{conf,min1} / A_{conf,max1}), 0) = 0.53375773$

The definitions of A_{noConf} , $A_{conf,min}$ and $A_{conf,max}$ are derived from generalization of the rectangular sections confinement, which is expressed by (5.4d).

The generalization is done according to Mander, J., Priestley, M., and Park, R. (1988)

"Theoretical Stress-Strain Model for Confined Concrete."

J. Struct. Eng., 10.1061/(ASCE)0733-9445(1988)114:8(1804), 1804-1826.

$A_{conf,max1} = 464100.00$ is the confined core area at levels of member with hoops and is calculated as the area of core enclosed by the center lines of the external perimeter hoops.

$A_{conf,min1} = 395025.00$ is the confined core area at midway between the levels of hoops and is calculated by reducing all the dimensions of the area $A_{conf,max1}$ by a length equal to half the clear spacing between external hoops.

$A_{noConf1} = 173066.667$ is the unconfined external core area which is equal to $b_i^2/6$ as defined at (A.2).

$a_{se2} (>= a_{se1}) = \text{Max}(((A_{conf,max2} - A_{noConf2}) / A_{conf,max2}) * (A_{conf,min2} / A_{conf,max2}), 0) = 0.53375773$

The definitions of A_{noConf} , $A_{conf,min}$ and $A_{conf,max}$ are derived from generalization of the rectangular sections confinement, which is expressed by (5.4d).

The generalization is done according to Mander, J., Priestley, M., and Park, R. (1988)

"Theoretical Stress-Strain Model for Confined Concrete."

J. Struct. Eng., 10.1061/(ASCE)0733-9445(1988)114:8(1804), 1804-1826.

$A_{conf,max2} = 252164.00$ is the confined core area at levels of member with hoops and is calculated as the area of core enclosed by the center lines of the internal perimeter hoops.

$A_{conf,min2} = 111441.00$ is the confined core area at midway between the levels of hoops and is calculated by reducing all the dimensions of the area $A_{conf,max2}$ by a length equal to half the clear spacing between internal hoops.

$A_{noConf2} = 110709.333$ is the unconfined internal core area which is equal to $b_i^2/6$ as defined at (A.2).

$p_{sh,min} * F_{ywe} = \text{Min}(p_{sh,x} * F_{ywe}, p_{sh,y} * F_{ywe}) = 2.724$

$p_{sh,x} * F_{ywe} = p_{sh1} * F_{ywe1} + p_{s2} * F_{ywe2} = 2.724$

p_{sh1} ((5.4d), TBDY) = $L_{stir1} * A_{stir1} / (A_{sec} * s_1) = 0.00301593$

L_{stir1} (Length of stirrups along Y) = 2160.00

A_{stir1} (stirrups area) = 78.53982

p_{sh2} (5.4d) = $L_{stir2} * A_{stir2} / (A_{sec} * s_2) = 0.00056047$

L_{stir2} (Length of stirrups along Y) = 1568.00

A_{stir2} (stirrups area) = 50.26548

$p_{sh,y} * F_{ywe} = p_{sh1} * F_{ywe1} + p_{s2} * F_{ywe2} = 3.25416$

p_{sh1} ((5.4d), TBDY) = $L_{stir1} * A_{stir1} / (A_{sec} * s_1) = 0.00357443$

L_{stir1} (Length of stirrups along X) = 2560.00

A_{stir1} (stirrups area) = 78.53982

p_{sh2} ((5.4d), TBDY) = $L_{stir2} * A_{stir2} / (A_{sec} * s_2) = 0.00070345$

L_{stir2} (Length of stirrups along X) = 1968.00

A_{stir2} (stirrups area) = 50.26548

$A_{sec} = 562500.00$

$s_1 = 100.00$

$s_2 = 250.00$

$f_{ywe1} = 781.25$

$f_{ywe2} = 656.25$

$f_{ce} = 18.00$

From ((5.A5), TBDY), TBDY: $c_c = 0.00621307$

c = confinement factor = 1.42131

$y_1 = 0.0025$

$sh_1 = 0.008$

$ft_1 = 907.50$

$fy_1 = 756.25$

$su_1 = 0.032$

using (30) in Biskinis/Fardis (2013) multiplied with shear_factor

and also multiplied by the shear_factor according to 15.7.1.4, with

Shear_factor = 1.00

$l_o/l_{ou,min} = l_b/l_d = 1.00$

$su_1 = 0.4 * esu_{1,nominal}$ ((5.5), TBDY) = 0.032

From table 5A.1, TBDY: $esu_{1,nominal} = 0.08$,

For calculation of $esu_{1,nominal}$ and y_1 , sh_1 , ft_1 , fy_1 , it is considered characteristic value $fsy_1 = fs_1/1.2$, from table 5.1, TBDY.

$y1, sh1, ft1, fy1$, are also multiplied by $\text{Min}(1, 1.25 \cdot (lb/ld)^{2/3})$, from 10.3.5, ASCE 41-17.
 with $fs1 = (fs_{jacket} \cdot A_{sl,ten,jacket} + fs_{core} \cdot A_{sl,ten,core}) / A_{sl,ten} = 756.25$
 with $Es1 = (Es_{jacket} \cdot A_{sl,ten,jacket} + Es_{core} \cdot A_{sl,ten,core}) / A_{sl,ten} = 200000.00$
 $y2 = 0.0025$
 $sh2 = 0.008$
 $ft2 = 907.50$
 $fy2 = 756.25$
 $su2 = 0.032$
 using (30) in Biskinis/Fardis (2013) multiplied with shear_factor
 and also multiplied by the shear_factor according to 15.7.1.4, with
 Shear_factor = 1.00
 $lo/lou, min = lb/lb, min = 1.00$
 $su2 = 0.4 \cdot esu2_nominal \cdot ((5.5), TBDY) = 0.032$
 From table 5A.1, TBDY: $esu2_nominal = 0.08$,
 For calculation of $esu2_nominal$ and $y2, sh2, ft2, fy2$, it is considered
 characteristic value $fsy2 = fs2/1.2$, from table 5.1, TBDY.
 $y1, sh1, ft1, fy1$, are also multiplied by $\text{Min}(1, 1.25 \cdot (lb/ld)^{2/3})$, from 10.3.5, ASCE 41-17.
 with $fs2 = (fs_{jacket} \cdot A_{sl,com,jacket} + fs_{core} \cdot A_{sl,com,core}) / A_{sl,com} = 756.25$
 with $Es2 = (Es_{jacket} \cdot A_{sl,com,jacket} + Es_{core} \cdot A_{sl,com,core}) / A_{sl,com} = 200000.00$
 $yv = 0.0025$
 $shv = 0.008$
 $ftv = 899.1522$
 $fyv = 749.2935$
 $suv = 0.032$
 using (30) in Biskinis/Fardis (2013) multiplied with shear_factor
 and also multiplied by the shear_factor according to 15.7.1.4, with
 Shear_factor = 1.00
 $lo/lou, min = lb/ld = 1.00$
 $suv = 0.4 \cdot esuv_nominal \cdot ((5.5), TBDY) = 0.032$
 From table 5A.1, TBDY: $esuv_nominal = 0.08$,
 considering characteristic value $fsyv = fsv/1.2$, from table 5.1, TBDY
 For calculation of $esuv_nominal$ and yv, shv, ftv, fyv , it is considered
 characteristic value $fsyv = fsv/1.2$, from table 5.1, TBDY.
 $y1, sh1, ft1, fy1$, are also multiplied by $\text{Min}(1, 1.25 \cdot (lb/ld)^{2/3})$, from 10.3.5, ASCE 41-17.
 with $fsv = (fs_{jacket} \cdot A_{sl,mid,jacket} + fs_{mid} \cdot A_{sl,mid,core}) / A_{sl,mid} = 749.2935$
 with $Es_v = (Es_{jacket} \cdot A_{sl,mid,jacket} + Es_{mid} \cdot A_{sl,mid,core}) / A_{sl,mid} = 200000.00$
 $1 = A_{sl,ten} / (b \cdot d) \cdot (fs1 / fc) = 0.15845977$
 $2 = A_{sl,com} / (b \cdot d) \cdot (fs2 / fc) = 0.15845977$
 $v = A_{sl,mid} / (b \cdot d) \cdot (fsv / fc) = 0.36847443$
 and confined core properties:
 $b = 390.00$
 $d = 877.00$
 $d' = 13.00$
 $fcc \text{ (5A.2, TBDY)} = 25.58352$
 $cc \text{ (5A.5, TBDY)} = 0.00621307$
 $c = \text{confinement factor} = 1.42131$
 $1 = A_{sl,ten} / (b \cdot d) \cdot (fs1 / fc) = 0.18909264$
 $2 = A_{sl,com} / (b \cdot d) \cdot (fs2 / fc) = 0.18909264$
 $v = A_{sl,mid} / (b \cdot d) \cdot (fsv / fc) = 0.43970658$
 Case/Assumption: Unconfined full section - Steel rupture
 ' satisfies Eq. (4.3)
 --->
 $v < v_{s,y2}$ - LHS eq.(4.5) is not satisfied
 --->
 $v < v_{s,c}$ - RHS eq.(4.5) is not satisfied
 --->
 Case/Assumption Rejected.
 --->
 New Case/Assumption: Unconfined full section - Spalling of concrete cover
 ' satisfies Eq. (4.4)
 --->
 $v < s_{y1}$ - LHS eq.(4.7) is not satisfied
 --->
 $v < v_{c,y1}$ - RHS eq.(4.6) is satisfied
 --->
 $cu \text{ (4.10)} = 0.83934343$
 $MRC \text{ (4.17)} = 1.8439E+009$

```

---->
New Case/Assumption: Ultimate curvature of confined core after spalling of concrete cover
In expressions below, the following modifications have been made
- b, d, d' replaced by geometric parameters of the core: bo, do, d'o
- N,  $\phi$ ,  $\rho$ ,  $\nu$  normalised to  $b_o*d_o$ , instead of  $b*d$ 
-  $\rho$ ,  $\nu$  parameters of confined concrete,  $\rho_{cc}$ ,  $\nu_{cc}$ , used in lieu of  $\rho_c$ ,  $\nu_c$ 
---->
Subcase: Rupture of tension steel
---->
 $\nu^* < \nu^*_{s,y2}$  - LHS eq.(4.5) is not satisfied
---->
 $\nu^* < \nu^*_{s,c}$  - LHS eq.(4.5) is not satisfied
---->
Subcase rejected
---->
New Subcase: Failure of compression zone
---->
 $\nu^* < \nu^*_{c,y2}$  - LHS eq.(4.6) is not satisfied
---->
 $\nu^* < \nu^*_{c,y1}$  - RHS eq.(4.6) is not satisfied
---->
 $\phi_{cu}$  (4.11) = 0.89983732
MRo (4.18) = 1.8431E+009
---->
 $\phi_u = \phi_{cu}$  (4.2) = 7.8730391E-006
Mu = MRo
-----

Calculation of ratio lb/d
-----
Adequate Lap Length: lb/d >= 1
-----
-----
-----
Calculation of Shear Strength  $V_r = \text{Min}(V_{r1}, V_{r2}) = 1.4186E+006$ 
-----
Calculation of Shear Strength at edge 1,  $V_{r1} = 1.4186E+006$ 
 $V_{r1} = V_{col}$  ((10.3), ASCE 41-17) =  $k_n I^* V_{col0}$ 
 $V_{col0} = 1.4186E+006$ 
 $k_n = 1$  (zero step-static loading)
-----
NOTE: In expression (10-3) ' $V_s = A_v f_y d/s$ ' is replaced by ' $V_s + \phi^* V_f$ '
where  $V_f$  is the contribution of FRPs (11.3), ACI 440).
-----
= 1 (normal-weight concrete)
Mean concrete strength:  $f'_c = (f'_{c,jacket} * \text{Area}_{jacket} + f'_{c,core} * \text{Area}_{core}) / \text{Area}_{section} = 15.20$ , but  $f'_c^{0.5} \leq 8.3$ 
MPa (22.5.3.1, ACI 318-14)
M/Vd = 4.00
Mu = 2.6919E+008
Vu = 137.4894
d = 0.8*h = 760.00
Nu = 7.6839E+006
Ag = 427500.00
From (11.5.4.8), ACI 318-14:  $V_s = V_{sjacket} + V_{s,core} = 1.2262E+006$ 
where:
 $V_{sjacket} = V_{sj1} + V_{sj2} = 1.0996E+006$ 
 $V_{sj1} = 353429.174$  is calculated for section web jacket, with:
d = 360.00
Av = 157079.633
fy = 625.00
s = 100.00
 $V_{sj1}$  is multiplied by  $\phi_{col,j1} = 1.00$ 
s/d = 0.27777778
 $V_{sj2} = 746128.255$  is calculated for section flange jacket, with:
d = 760.00
Av = 157079.633

```

$f_y = 625.00$
 $s = 100.00$
 $V_{s,j2}$ is multiplied by $Col,j2 = 1.00$
 $s/d = 0.13157895$
 $V_{s,core} = V_{s,c1} + V_{s,c2} = 126669.016$
 $V_{s,c1} = 0.00$ is calculated for section web core, with:
 $d = 200.00$
 $A_v = 100530.965$
 $f_y = 525.00$
 $s = 250.00$
 $V_{s,c1}$ is multiplied by $Col,c1 = 0.00$
 $s/d = 1.25$
 $V_{s,c2} = 126669.016$ is calculated for section flange core, with:
 $d = 600.00$
 $A_v = 100530.965$
 $f_y = 525.00$
 $s = 250.00$
 $V_{s,c2}$ is multiplied by $Col,c2 = 1.00$
 $s/d = 0.41666667$
 $V_f ((11-3)-(11.4), ACI 440) = 0.00$
 From (11-11), ACI 440: $V_s + V_f \leq 885757.128$
 $bw = 450.00$

Calculation of Shear Strength at edge 2, $V_{r2} = 1.4186E+006$

$V_{r2} = V_{Col} ((10.3), ASCE 41-17) = knl * V_{Col0}$

$V_{Col0} = 1.4186E+006$

$knl = 1$ (zero step-static loading)

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

= 1 (normal-weight concrete)

Mean concrete strength: $fc' = (fc'_{jacket} * Area_{jacket} + fc'_{core} * Area_{core}) / Area_{section} = 15.20$, but $fc'^{0.5} \leq 8.3$ MPa (22.5.3.1, ACI 318-14)

$M/Vd = 4.00$

$\mu_u = 2.5135E+008$

$V_u = 137.4894$

$d = 0.8 * h = 760.00$

$N_u = 7.6839E+006$

$A_g = 427500.00$

From (11.5.4.8), ACI 318-14: $V_s = V_{sjacket} + V_{s,core} = 1.2262E+006$

where:

$V_{sjacket} = V_{sj1} + V_{sj2} = 1.0996E+006$

$V_{sj1} = 353429.174$ is calculated for section web jacket, with:

$d = 360.00$

$A_v = 157079.633$

$f_y = 625.00$

$s = 100.00$

V_{sj1} is multiplied by $Col,j1 = 1.00$

$s/d = 0.27777778$

$V_{sj2} = 746128.255$ is calculated for section flange jacket, with:

$d = 760.00$

$A_v = 157079.633$

$f_y = 625.00$

$s = 100.00$

V_{sj2} is multiplied by $Col,j2 = 1.00$

$s/d = 0.13157895$

$V_{s,core} = V_{s,c1} + V_{s,c2} = 126669.016$

$V_{s,c1} = 0.00$ is calculated for section web core, with:

$d = 200.00$

$A_v = 100530.965$

$f_y = 525.00$

$s = 250.00$

$V_{s,c1}$ is multiplied by $Col,c1 = 0.00$

$s/d = 1.25$

$V_{s,c2} = 126669.016$ is calculated for section flange core, with:

d = 600.00
Av = 100530.965
fy = 525.00
s = 250.00
Vs,c2 is multiplied by Col,c2 = 1.00
s/d = 0.41666667
Vf ((11-3)-(11.4), ACI 440) = 0.00
From (11-11), ACI 440: Vs + Vf <= 885757.128
bw = 450.00

End Of Calculation of Shear Capacity ratio for element: column JTC1 of floor 1
At local axis: 2

Start Of Calculation of Chord Rotation Capacity for element: column JTC1 of floor 1
At local axis: 2
Integration Section: (a)
Section Type: rcjtcs

Constant Properties

Knowledge Factor, γ = 0.80
Chord Rotation is generally considered as Deformation-Controlled Action according to Table C7-1, ASCE 41-17.
Mean strengths are used for Deformation-Controlled Actions according to 7.5.1.3, ASCE 41-17
Consequently:
Jacket
New material of Primary Member: Concrete Strength, $f_c = f_{cm} = 18.00$
New material of Primary Member: Steel Strength, $f_s = f_{sm} = 625.00$
Concrete Elasticity, $E_c = 19940.411$
Steel Elasticity, $E_s = 200000.00$
Existing Column
Existing material of Primary Member: Concrete Strength, $f_c = f_{cm} = 12.00$
Existing material of Primary Member: Steel Strength, $f_s = f_{sm} = 525.00$
Concrete Elasticity, $E_c = 16281.278$
Steel Elasticity, $E_s = 200000.00$
Max Height, $H_{max} = 750.00$
Min Height, $H_{min} = 450.00$
Max Width, $W_{max} = 950.00$
Min Width, $W_{min} = 450.00$
Eccentricity, $E_{cc} = 250.00$
Jacket Thickness, $t_j = 100.00$
Cover Thickness, $c = 25.00$
Element Length, $L = 3000.00$
Primary Member
Ribbed Bars
Ductile Steel
With Detailing for Earthquake Resistance (including stirrups closed at 135°)
Longitudinal Bars With Ends Lapped Starting at the End Sections
Adequate Lap Length ($l_b/l_d \geq 1$)
No FRP Wrapping

Stepwise Properties

Bending Moment, $M = -1.0497E+009$
Shear Force, $V_2 = -455948.704$
Shear Force, $V_3 = 29.64437$
Axial Force, $F = -7.6755E+006$
Longitudinal Reinforcement Area Distribution (in 2 divisions)
-Tension: $As_t = 2522.699$
-Compression: $As_c = 4168.893$
Longitudinal Reinforcement Area Distribution (in 3 divisions)
-Tension: $As_{l,ten} = 1539.38$
-Compression: $As_{l,com} = 2475.575$

-Middle: $Asl_{mid} = 2676.637$
 Longitudinal External Reinforcement Area Distribution (in 3 divisions)
 -Tension: $Asl_{ten,jacket} = 1231.504$
 -Compression: $Asl_{com,jacket} = 1859.823$
 -Middle: $Asl_{mid,jacket} = 2060.885$
 Longitudinal Internal Reinforcement Area Distribution (in 3 divisions)
 -Tension: $Asl_{ten,core} = 307.8761$
 -Compression: $Asl_{com,core} = 615.7522$
 -Middle: $Asl_{mid,core} = 615.7522$
 Mean Diameter of Tension Reinforcement, $DbL = 16.57143$

Existing component: From table 7-7, ASCE 41_17: Final chord rotation Capacity $u_R = \phi \cdot u = 0.01081173$
 $u = y + p = 0.01351466$

- Calculation of y -

$y = (My \cdot L_s / 3) / E_{eff} = 0.01351466 ((4.29), \text{Biskinis Phd})$
 $My = 2.0395E+009$
 $L_s = M/V \text{ (with } L_s > 0.1 \cdot L \text{ and } L_s < 2 \cdot L) = 6000.00$
 From table 10.5, ASCE 41_17: $E_{eff} = factor \cdot E_c \cdot I_g = 3.0181E+014$
 $factor = 0.70$
 $Ag = 562500.00$
 Mean concrete strength: $fc' = (fc'_{jacket} \cdot Area_{jacket} + fc'_{core} \cdot Area_{core}) / Area_{section} = 15.20$
 $N = 7.6755E+006$
 $E_c \cdot I_g = E_{c,jacket} \cdot I_{g,jacket} + E_{c,core} \cdot I_{g,core} = 4.3116E+014$

Calculation of Yielding Moment My

Calculation of y and My according to Annex 7 -

Assuming neutral axis out of flange ($y > t/d$, compression zone NOT rectangular)
 extended expressions (2.1)-(2.7) of Biskinis-Fardis (2009) are adopted with:
 flange width, $b = 950.00$
 web width, $bw = 450.00$
 flange thickness, $t = 450.00$

$y = \text{Min}(y_{ten}, y_{com})$
 $y_{ten} = 7.9835200E-006$
 with $f_y = 601.9953$
 $d = 707.00$
 $y = 0.46672739$
 $A = 0.13161899$
 $B = 0.07228946$
 with $p_t = 0.00229194$
 $p_c = 0.00368581$
 $p_v = 0.00398517$
 $N = 7.6755E+006$
 $b = 950.00$
 $\alpha = 0.06082037$
 $y_{comp} = 3.5421048E-006$
 with $fc = 18.00$
 $E_c = 19940.411$
 $y = 0.64882874$
 $A = 0.01730459$
 $B = 0.03221394$
 with $E_s = 200000.00$
 CONFIRMATION: $y = 0.64882874 > t/d$

Calculation of ratio I_b/I_d

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

- Calculation of p -

From table 10-8: $p = 0.00$

with:

- Columns not controlled by inadequate development or splicing along the clear height because $l_b/l_d \geq 1$
shear control ratio $V_{yE}/V_{ColOE} = 1.74773$

$d = d_{\text{external}} = 707.00$

$s = s_{\text{external}} = 50.00$

- $t = s_1 + s_2 + 2 \cdot t_f/bw \cdot (f_{fe}/f_s) = 0.0035764$

jacket: $s_1 = A_{v1} \cdot L_{\text{stir1}}/(s_1 \cdot A_g) = 0.00301593$

$A_{v1} = 78.53982$, is the area of every stirrup parallel to loading (shear) direction

$L_{\text{stir1}} = 2160.00$, is the total Length of all stirrups parallel to loading (shear) direction

$s_1 = 100.00$

core: $s_2 = A_{v2} \cdot L_{\text{stir2}}/(s_2 \cdot A_g) = 0.00056047$

$A_{v2} = 50.26548$, is the area of every stirrup parallel to loading (shear) direction

$L_{\text{stir2}} = 1568.00$, is the total Length of all stirrups parallel to loading (shear) direction

$s_2 = 250.00$

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

where $f = 2 \cdot t_f/bw$ is FRP ratio (EC8 - 3, A.4.4.3(6)) and f_{fe}/f_s normalises f to steel strength

All these variables have already been given in Shear control ratio calculation.

For the normalisation f_s of jacket is used.

$NUD = 7.6755E+006$

$A_g = 562500.00$

$f_{cE} = (f_{c_jacket} \cdot \text{Area_jacket} + f_{c_core} \cdot \text{Area_core})/\text{section_area} = 15.20$

$f_{yE} = (f_{y_ext_Long_Reinf} \cdot \text{Area_ext_Long_Reinf} + f_{y_int_Long_Reinf} \cdot \text{Area_int_Long_Reinf})/\text{Area_Tot_Long_Rein} = 601.9953$

$f_{yE} = (f_{y_ext_Trans_Reinf} \cdot s_1 + f_{y_int_Trans_Reinf} \cdot s_2)/(s_1 + s_2) = 609.3286$

$p_l = \text{Area_Tot_Long_Rein}/(b \cdot d) = 0.00996292$

$b = 950.00$

$d = 707.00$

$f_{cE} = 15.20$

End Of Calculation of Chord Rotation Capacity for element: column JTC1 of floor 1

At local axis: 2

Integration Section: (a)

Calculation No. 3

column C1, Floor 1

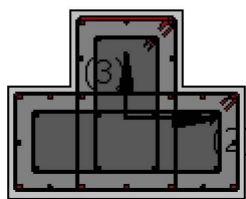
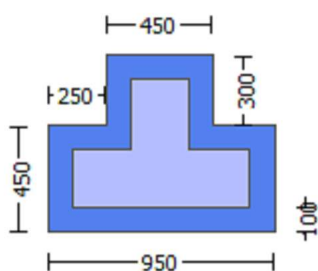
Limit State: Operational Level (data interpolation between analysis steps 39 and 40)

Analysis: Uniform +X

Check: Shear capacity V_{Rd}

Edge: Start

Local Axis: (3)



Start Of Calculation of Shear Capacity for element: column JTC1 of floor 1

At local axis: 3

Integration Section: (a)

Section Type: rcjtcs

Constant Properties

Knowledge Factor, $\gamma = 0.80$

Member Shear Force is generally considered as Force-Controlled Action according to Table C7-1, ASCE 41-17.

Lower-bound strengths are used for Force-Controlled Actions according to 7.5.1.3, ASCE 41-17

Consequently:

Jacket

New material of Primary Member: Concrete Strength, $f_c = f_{c_lower_bound} = 12.00$

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

Concrete Elasticity, $E_c = 19940.411$

Steel Elasticity, $E_s = 200000.00$

Existing Column

Existing material of Primary Member: Concrete Strength, $f_c = f_{c_lower_bound} = 8.00$

Existing material of Primary Member: Steel Strength, $f_s = f_{s_lower_bound} = 420.00$

Concrete Elasticity, $E_c = 16281.278$

Steel Elasticity, $E_s = 200000.00$

#####

Note: Especially for the calculation of γ for displacement ductility demand, the expected (mean value) strengths are used (7.5.1.3, ASCE 41-17) because bending is considered as Deformation-Controlled Action (Table C7-1, ASCE 41-17).

Jacket

New material: Concrete Strength, $f_c = f_{cm} = 18.00$

New material: Steel Strength, $f_s = f_{sm} = 625.00$

Existing Column

Existing material: Concrete Strength, $f_c = f_{cm} = 12.00$

Existing material: Steel Strength, $f_s = f_{sm} = 525.00$

#####

Max Height, $H_{max} = 750.00$

Min Height, $H_{min} = 450.00$

Max Width, $W_{max} = 950.00$

Min Width, $W_{min} = 450.00$

Eccentricity, $Ecc = 250.00$

Jacket Thickness, $t_j = 100.00$

Cover Thickness, $c = 25.00$

Element Length, $L = 3000.00$

Primary Member

Ribbed Bars

Ductile Steel

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

Longitudinal Bars With Ends Lapped Starting at the End Sections

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

No FRP Wrapping

Stepwise Properties

EDGE -A-

Bending Moment, $M_a = -1.0497E+009$
 Shear Force, $V_a = 29.64437$
 EDGE -B-
 Bending Moment, $M_b = 6.8967E+008$
 Shear Force, $V_b = -29.64437$
 BOTH EDGES
 Axial Force, $F = -7.6755E+006$
 Longitudinal Reinforcement Area Distribution (in 2 divisions)
 -Tension: $A_{sl,t} = 2522.699$
 -Compression: $A_{sl,c} = 4168.893$
 Longitudinal Reinforcement Area Distribution (in 3 divisions)
 -Tension: $A_{sl,ten} = 1539.38$
 -Compression: $A_{sl,com} = 2475.575$
 -Middle: $A_{sl,mid} = 2676.637$
 Mean Diameter of Tension Reinforcement, $Db_{L,ten} = 16.57143$

Existing component: From table 7-7, ASCE 41_17: Final Shear Capacity $V_R = V_n = 792832.491$
 V_n ((10.3), ASCE 41-17) = $k_n I V_{CoI} = 991040.614$
 $V_{CoI} = 991040.614$
 $k_n = 1.00$
 displacement_ductility_demand = 1.15591

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

= 1 (normal-weight concrete)
 Mean concrete strength: $f'_c = (f'_{c,jacket} \cdot Area_{jacket} + f'_{c,core} \cdot Area_{core}) / Area_{section} = 10.13333$, but $f'_c^{0.5} \leq 8.3$ MPa (22.5.3.1, ACI 318-14)
 $M/Vd = 4.00$
 $\mu_u = 1.0497E+009$
 $V_u = 29.64437$
 $d = 0.8 \cdot h = 600.00$
 $N_u = 7.6755E+006$
 $A_g = 337500.00$
 From (11.5.4.8), ACI 318-14: $V_s = V_{s,jacket} + V_{s,core} = 828294.726$
 where:
 $V_{s,jacket} = V_{sj1} + V_{sj2} = 753982.237$
 $V_{sj1} = 471238.898$ is calculated for section web jacket, with:
 $d = 600.00$
 $A_v = 157079.633$
 $f_y = 500.00$
 $s = 100.00$
 V_{sj1} is multiplied by $Col,j1 = 1.00$
 $s/d = 0.16666667$
 $V_{sj2} = 282743.339$ is calculated for section flange jacket, with:
 $d = 360.00$
 $A_v = 157079.633$
 $f_y = 500.00$
 $s = 100.00$
 V_{sj2} is multiplied by $Col,j2 = 1.00$
 $s/d = 0.27777778$
 $V_{s,core} = V_{sc1} + V_{sc2} = 74312.489$
 $V_{sc1} = 74312.489$ is calculated for section web core, with:
 $d = 440.00$
 $A_v = 100530.965$
 $f_y = 420.00$
 $s = 250.00$
 V_{sc1} is multiplied by $Col,c1 = 1.00$
 $s/d = 0.56818182$
 $V_{sc2} = 0.00$ is calculated for section flange core, with:
 $d = 200.00$
 $A_v = 100530.965$
 $f_y = 420.00$
 $s = 250.00$

$V_{s,c2}$ is multiplied by $Col,c2 = 0.00$
 $s/d = 1.25$
 $V_f ((11-3)-(11.4), ACI 440) = 0.00$
 From (11-11), ACI 440: $V_s + V_f \leq 570961.316$
 $bw = 450.00$

displacement ductility demand is calculated as ϕ / y

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

From analysis, chord rotation $\theta = 0.01562176$
 $y = (M_y * L_s / 3) / E_{eff} = 0.01351466 ((4.29), Biskinis Phd)$
 $M_y = 2.0395E+009$
 $L_s = M/V$ (with $L_s > 0.1 * L$ and $L_s < 2 * L$) = 6000.00
 From table 10.5, ASCE 41_17: $E_{eff} = factor * E_c * I_g = 3.0181E+014$
 $factor = 0.70$
 $A_g = 562500.00$
 Mean concrete strength: $f_c' = (f_c'_{jacket} * Area_{jacket} + f_c'_{core} * Area_{core}) / Area_{section} = 15.20$
 $N = 7.6755E+006$
 $E_c * I_g = E_{c,jacket} * I_{g,jacket} + E_{c,core} * I_{g,core} = 4.3116E+014$

Calculation of Yielding Moment M_y

Calculation of ϕ and M_y according to Annex 7 -

Assuming neutral axis out of flange ($y > t/d$, compression zone NOT rectangular)
 extended expressions (2.1)-(2.7) of Biskinis-Fardis (2009) are adopted with:
 flange width, $b = 950.00$
 web width, $bw = 450.00$
 flange thickness, $t = 450.00$

$y = \text{Min}(y_{ten}, y_{com})$
 $y_{ten} = 7.9835200E-006$
 with $f_y = 601.9953$
 $d = 707.00$
 $y = 0.46672739$
 $A = 0.13161899$
 $B = 0.07228946$
 with $pt = 0.00229194$
 $pc = 0.00368581$
 $pv = 0.00398517$
 $N = 7.6755E+006$
 $b = 950.00$
 $" = 0.06082037$
 $y_{comp} = 3.5421048E-006$
 with $f_c = 18.00$
 $E_c = 19940.411$
 $y = 0.64882874$
 $A = 0.01730459$
 $B = 0.03221394$
 with $E_s = 200000.00$
 CONFIRMATION: $y = 0.64882874 > t/d$

Calculation of ratio I_b/I_d

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

End Of Calculation of Shear Capacity for element: column JTC1 of floor 1
 At local axis: 3
 Integration Section: (a)

Calculation No. 4

column C1, Floor 1

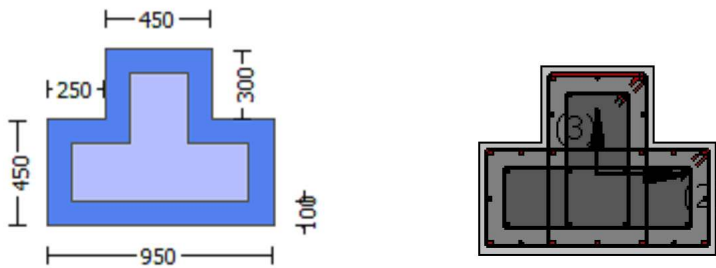
Limit State: Operational Level (data interpolation between analysis steps 39 and 40)

Analysis: Uniform +X

Check: Chord rotation capacity (ϕ)

Edge: Start

Local Axis: (3)



Start Of Calculation of Shear Capacity ratio for element: column JTC1 of floor 1

At Shear local axis: 3

(Bending local axis: 2)

Section Type: rcjtcs

Constant Properties

Knowledge Factor, $\gamma = 0.80$

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

Consequently:

Jacket

New material of Primary Member: Concrete Strength, $f_c = f_{cm} = 18.00$

New material of Primary Member: Steel Strength, $f_s = f_{sm} = 625.00$

Concrete Elasticity, $E_c = 19940.411$

Steel Elasticity, $E_s = 200000.00$

Existing Column

Existing material of Primary Member: Concrete Strength, $f_c = f_{cm} = 12.00$

Existing material of Primary Member: Steel Strength, $f_s = f_{sm} = 525.00$

Concrete Elasticity, $E_c = 16281.278$

Steel Elasticity, $E_s = 200000.00$

#####

Note: Especially for the calculation of moment strengths,

the above steel re-bar strengths are multiplied by 1.25 according to R18.6.5, ACI 318-14

Jacket

New material: Steel Strength, $f_s = 1.25 \cdot f_{sm} = 781.25$

Existing Column

Existing material: Steel Strength, $f_s = 1.25 \cdot f_{sm} = 656.25$

#####

Max Height, Hmax = 750.00
 Min Height, Hmin = 450.00
 Max Width, Wmax = 950.00
 Min Width, Wmin = 450.00
 Eccentricity, Ecc = 250.00
 Jacket Thickness, tj = 100.00
 Cover Thickness, c = 25.00
 Mean Confinement Factor overall section = 1.42131
 Element Length, L = 3000.00
 Primary Member
 Ribbed Bars
 Ductile Steel
 With Detailing for Earthquake Resistance (including stirrups closed at 135°)
 Longitudinal Bars With Ends Lapped Starting at the End Sections
 Adequate Lap Length ($l_o/l_{o,u,min} \geq 1$)
 No FRP Wrapping

Stepwise Properties

At local axis: 3
 EDGE -A-
 Shear Force, Va = -227.4778
 EDGE -B-
 Shear Force, Vb = 227.4778
 BOTH EDGES
 Axial Force, F = -7.6839E+006
 Longitudinal Reinforcement Area Distribution (in 2 divisions)
 -Tension: Aslt = 314.1593
 -Compression: Aslc = 6176.371
 Longitudinal Reinforcement Area Distribution (in 3 divisions)
 -Tension: Asl,ten = 1539.38
 -Compression: Asl,com = 2475.575
 -Middle: Asl,mid = 2676.637

Calculation of Shear Capacity ratio, $V_e/V_r = 1.74773$
 Member Controlled by Shear ($V_e/V_r > 1$)
 Calculation of Shear Demand from fig. R18.6.5, ACI 318-14 $V_e = (M_{pr1} + M_{pr2})/l_n = 2.0410E+006$
 with
 $M_{pr1} = \text{Max}(\mu_{u1+}, \mu_{u1-}) = 3.0615E+009$
 $\mu_{u1+} = 3.0615E+009$, is the ultimate moment strength at the edge 1 of the member in the actual moment direction
 which is defined for the static loading combination
 $\mu_{u1-} = 8.9683E+008$, is the ultimate moment strength at the edge 1 of the member in the opposite moment
 direction which is defined for the static loading combination
 $M_{pr2} = \text{Max}(\mu_{u2+}, \mu_{u2-}) = 3.0615E+009$
 $\mu_{u2+} = 3.0615E+009$, is the ultimate moment strength at the edge 2 of the member in the actual moment direction
 which is defined for the the static loading combination
 $\mu_{u2-} = 8.9683E+008$, 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 μ_{u1+}

Calculation of ultimate curvature ϕ_u according to 4.1, Biskinis/Fardis 2013:
 $\phi_u = 1.3930706E-005$
 $M_u = 3.0615E+009$

with full section properties:

b = 950.00
 d = 707.00
 d' = 43.00
 v = 0.63557247

$N = 7.6839E+006$
 $f_c = 18.00$
 $\alpha (5A.5, TBDY) = 0.002$
 Final value of α : $\alpha^* = \text{shear_factor} * \text{Max}(\alpha, \alpha_c) = 0.01486841$
 The Shear_factor is considered equal to 1 (pure moment strength)
 From (5.4b), TBDY: $\alpha_c = 0.01486841$
 $\alpha_{se} (5.4c) = 0.08077545$
 $\alpha_{se} ((5.4d), TBDY) = (\alpha_{se1} * A_{ext} + \alpha_{se2} * A_{int}) / A_{sec} = 0.53375773$
 $\alpha_{se1} = \text{Max}(((A_{conf,max1} - A_{noConf1}) / A_{conf,max1}) * (A_{conf,min1} / A_{conf,max1}), 0) = 0.53375773$
 The definitions of A_{noConf} , $A_{conf,min}$ and $A_{conf,max}$ are derived from generalization of the rectangular sections confinement, which is expressed by (5.4d).
 The generalization is done according to Mander, J., Priestley, M., and Park, R. (1988) "Theoretical Stress-Strain Model for Confined Concrete."
 J. Struct. Eng., 10.1061/(ASCE)0733-9445(1988)114:8(1804), 1804-1826.
 $A_{conf,max1} = 464100.00$ is the confined core area at levels of member with hoops and is calculated as the area of core enclosed by the center lines of the external perimeter hoops.
 $A_{conf,min1} = 395025.00$ is the confined core area at midway between the levels of hoops and is calculated by reducing all the dimensions of the area $A_{conf,max1}$ by a length equal to half the clear spacing between external hoops.
 $A_{noConf1} = 173066.667$ is the unconfined external core area which is equal to $b_i^2/6$ as defined at (A.2).
 $\alpha_{se2} (\geq \alpha_{se1}) = \text{Max}(((A_{conf,max2} - A_{noConf2}) / A_{conf,max2}) * (A_{conf,min2} / A_{conf,max2}), 0) = 0.53375773$
 The definitions of A_{noConf} , $A_{conf,min}$ and $A_{conf,max}$ are derived from generalization of the rectangular sections confinement, which is expressed by (5.4d).
 The generalization is done according to Mander, J., Priestley, M., and Park, R. (1988) "Theoretical Stress-Strain Model for Confined Concrete."
 J. Struct. Eng., 10.1061/(ASCE)0733-9445(1988)114:8(1804), 1804-1826.
 $A_{conf,max2} = 252164.00$ is the confined core area at levels of member with hoops and is calculated as the area of core enclosed by the center lines of the internal perimeter hoops.
 $A_{conf,min2} = 111441.00$ is the confined core area at midway between the levels of hoops and is calculated by reducing all the dimensions of the area $A_{conf,max2}$ by a length equal to half the clear spacing between internal hoops.
 $A_{noConf2} = 110709.333$ is the unconfined internal core area which is equal to $b_i^2/6$ as defined at (A.2).
 $p_{sh,min} * F_{ywe} = \text{Min}(p_{sh,x} * F_{ywe}, p_{sh,y} * F_{ywe}) = 2.724$

$p_{sh,x} * F_{ywe} = p_{sh1} * F_{ywe1} + p_{s2} * F_{ywe2} = 2.724$
 $p_{sh1} ((5.4d), TBDY) = L_{stir1} * A_{stir1} / (A_{sec} * s_1) = 0.00301593$
 L_{stir1} (Length of stirrups along Y) = 2160.00
 A_{stir1} (stirrups area) = 78.53982
 $p_{sh2} (5.4d) = L_{stir2} * A_{stir2} / (A_{sec} * s_2) = 0.00056047$
 L_{stir2} (Length of stirrups along Y) = 1568.00
 A_{stir2} (stirrups area) = 50.26548

$p_{sh,y} * F_{ywe} = p_{sh1} * F_{ywe1} + p_{s2} * F_{ywe2} = 3.25416$
 $p_{sh1} ((5.4d), TBDY) = L_{stir1} * A_{stir1} / (A_{sec} * s_1) = 0.00357443$
 L_{stir1} (Length of stirrups along X) = 2560.00
 A_{stir1} (stirrups area) = 78.53982
 $p_{sh2} ((5.4d), TBDY) = L_{stir2} * A_{stir2} / (A_{sec} * s_2) = 0.00070345$
 L_{stir2} (Length of stirrups along X) = 1968.00
 A_{stir2} (stirrups area) = 50.26548

$A_{sec} = 562500.00$
 $s_1 = 100.00$
 $s_2 = 250.00$
 $f_{ywe1} = 781.25$
 $f_{ywe2} = 656.25$
 $f_{ce} = 18.00$
 From ((5.A5), TBDY), TBDY: $\alpha_c = 0.00621307$
 $\alpha_c = \text{confinement factor} = 1.42131$
 $\gamma_1 = 0.0025$
 $\gamma_{sh} = 0.008$
 $f_{t1} = 907.50$
 $f_{y1} = 756.25$
 $\gamma_{su1} = 0.032$
 using (30) in Biskinis/Fardis (2013) multiplied with shear_factor and also multiplied by the shear_factor according to 15.7.1.4, with Shear_factor = 1.00
 $l_o/l_{ou,min} = l_b/d = 1.00$

```

su1 = 0.4*esu1_nominal ((5.5), TBDY) = 0.032
From table 5A.1, TBDY: esu1_nominal = 0.08,
For calculation of esu1_nominal and y1, sh1,ft1,fy1, it is considered
characteristic value fsy1 = fs1/1.2, from table 5.1, TBDY.
y1, sh1,ft1,fy1, are also multiplied by Min(1,1.25*(lb/ld)^ 2/3), from 10.3.5, ASCE 41-17.
with fs1 = (fs,jacket*Asl,ten,jacket + fs,core*Asl,ten,core)/Asl,ten = 756.25
with Es1 = (Es,jacket*Asl,ten,jacket + Es,core*Asl,ten,core)/Asl,ten = 200000.00
y2 = 0.0025
sh2 = 0.008
ft2 = 900.1904
fy2 = 750.1586
su2 = 0.032
using (30) in Biskinis/Fardis (2013) multiplied with shear_factor
and also multiplied by the shear_factor according to 15.7.1.4, with
Shear_factor = 1.00
lo/lou,min = lb/lb,min = 1.00
su2 = 0.4*esu2_nominal ((5.5), TBDY) = 0.032
From table 5A.1, TBDY: esu2_nominal = 0.08,
For calculation of esu2_nominal and y2, sh2,ft2,fy2, it is considered
characteristic value fsy2 = fs2/1.2, from table 5.1, TBDY.
y1, sh1,ft1,fy1, are also multiplied by Min(1,1.25*(lb/ld)^ 2/3), from 10.3.5, ASCE 41-17.
with fs2 = (fs,jacket*Asl,com,jacket + fs,core*Asl,com,core)/Asl,com = 750.1586
with Es2 = (Es,jacket*Asl,com,jacket + Es,core*Asl,com,core)/Asl,com = 200000.00
yv = 0.0025
shv = 0.008
ftv = 902.993
fyv = 752.4941
suv = 0.032
using (30) in Biskinis/Fardis (2013) multiplied with shear_factor
and also multiplied by the shear_factor according to 15.7.1.4, with
Shear_factor = 1.00
lo/lou,min = lb/ld = 1.00
suv = 0.4*esuv_nominal ((5.5), TBDY) = 0.032
From table 5A.1, TBDY: esuv_nominal = 0.08,
considering characteristic value fsyv = fsv/1.2, from table 5.1, TBDY
For calculation of esuv_nominal and yv, shv,ftv,fyv, it is considered
characteristic value fsyv = fsv/1.2, from table 5.1, TBDY.
y1, sh1,ft1,fy1, are also multiplied by Min(1,1.25*(lb/ld)^ 2/3), from 10.3.5, ASCE 41-17.
with fsv = (fs,jacket*Asl,mid,jacket + fs,mid*Asl,mid,core)/Asl,mid = 752.4941
with Esv = (Es,jacket*Asl,mid,jacket + Es,mid*Asl,mid,core)/Asl,mid = 200000.00
1 = Asl,ten/(b*d)*(fs1/fc) = 0.09629324
2 = Asl,com/(b*d)*(fs2/fc) = 0.15360794
v = Asl,mid/(b*d)*(fsv/fc) = 0.16660079
and confined core properties:
b = 890.00
d = 677.00
d' = 13.00
fcc (5A.2, TBDY) = 25.58352
cc (5A.5, TBDY) = 0.00621307
c = confinement factor = 1.42131
1 = Asl,ten/(b*d)*(fs1/fc) = 0.10733965
2 = Asl,com/(b*d)*(fs2/fc) = 0.17122928
v = Asl,mid/(b*d)*(fsv/fc) = 0.18571261
Case/Assumption: Unconfined full section - Steel rupture
' satisfies Eq. (4.3)
--->
v < vs,y2 - LHS eq.(4.5) is not satisfied
--->
v < vs,c - RHS eq.(4.5) is not satisfied
--->
Case/Assumption Rejected.
--->
New Case/Assumption: Unconfined full section - Spalling of concrete cover
' satisfies Eq. (4.4)
--->
v < s,y1 - LHS eq.(4.7) is not satisfied
--->

```

$v < v_{c,y1}$ - RHS eq.(4.6) is satisfied

---->

$c_u(4.10) = 0.58509201$

$M_{Rc}(4.17) = 2.3801E+009$

---->

New Case/Assumption: Ultimate curvature of confined core after spalling of concrete cover

In expressions below, the following modifications have been made

- b, d, d' replaced by geometric parameters of the core: b_o, d_o, d'_o
- $N, 1, 2, v$ normalised to b_o*d_o , instead of $b*d$
- parameters of confined concrete, f_{cc}, ϵ_{cc} , used in lieu of f_c, ϵ_{cu}

---->

Subcase: Rupture of tension steel

---->

$v^* < v^*_{s,y2}$ - LHS eq.(4.5) is not satisfied

---->

$v^* < v^*_{s,c}$ - LHS eq.(4.5) is not satisfied

---->

Subcase rejected

---->

New Subcase: Failure of compression zone

---->

$v^* < v^*_{c,y2}$ - LHS eq.(4.6) is not satisfied

---->

$v^* < v^*_{c,y1}$ - RHS eq.(4.6) is not satisfied

---->

$*c_u(4.11) = 0.65878578$

$M_{Ro}(4.18) = 3.0615E+009$

---->

$u = c_u(4.2) = 1.3930706E-005$

$\mu_u = M_{Ro}$

Calculation of ratio l_b/d

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

Calculation of μ_{u1} -

Calculation of ultimate curvature u according to 4.1, Biskinis/Fardis 2013:

$u = 9.1773508E-006$

$\mu_u = 8.9683E+008$

with full section properties:

$b = 450.00$

$d = 707.00$

$d' = 43.00$

$v = 1.34176$

$N = 7.6839E+006$

$f_c = 18.00$

$\epsilon_{co}(5A.5, TBDY) = 0.002$

Final value of c_u : $c_u^* = \text{shear_factor} * \text{Max}(c_u, \epsilon_{cc}) = 0.01486841$

The Shear_factor is considered equal to 1 (pure moment strength)

From (5.4b), TBDY: $c_u = 0.01486841$

$w_e(5.4c) = 0.08077545$

$a_{se}((5.4d), TBDY) = (a_{se1} * A_{ext} + a_{se2} * A_{int}) / A_{sec} = 0.53375773$

$a_{se1} = \text{Max}(((A_{conf,max1} - A_{noConf1}) / A_{conf,max1}) * (A_{conf,min1} / A_{conf,max1}), 0) = 0.53375773$

The definitions of A_{noConf} , $A_{conf,min}$ and $A_{conf,max}$ are derived from generalization of the rectangular sections confinement, which is expressed by (5.4d).

The generalization is done according to Mander, J., Priestley, M., and Park, R. (1988)

"Theoretical Stress-Strain Model for Confined Concrete."

J. Struct. Eng., 10.1061/(ASCE)0733-9445(1988)114:8(1804), 1804-1826.

$A_{conf,max1} = 464100.00$ is the confined core area at levels of member with hoops and

is calculated as the area of core enclosed by the center lines of the external perimeter hoops.

Aconf,min1 = 395025.00 is the confined core area at midway between the levels of hoops and is calculated by reducing all the dimensions of the area Aconf,max1 by a length equal to half the clear spacing between external hoops.

AnoConf1 = 173066.667 is the unconfined external core area which is equal to $b_i^2/6$ as defined at (A.2).
ase2 (\geq ase1) = $\text{Max}(((A_{\text{conf,max2}} - A_{\text{noConf2}})/A_{\text{conf,max2}}) * (A_{\text{conf,min2}}/A_{\text{conf,max2}}), 0) = 0.53375773$

The definitions of AnoConf, Aconf,min and Aconf,max are derived from generalization of the rectangular sections confinement, which is expressed by (5.4d).

The generalization is done according to Mander, J., Priestley, M., and Park, R. (1988)

"Theoretical Stress-Strain Model for Confined Concrete."

J. Struct. Eng., 10.1061/(ASCE)0733-9445(1988)114:8(1804), 1804-1826.

Aconf,max2 = 252164.00 is the confined core area at levels of member with hoops and is calculated as the area of core enclosed by the center lines of the internal perimeter hoops.

Aconf,min2 = 111441.00 is the confined core area at midway between the levels of hoops and is calculated by reducing all the dimensions of the area Aconf,max2 by a length equal to half the clear spacing between internal hoops.

AnoConf2 = 110709.333 is the unconfined internal core area which is equal to $b_i^2/6$ as defined at (A.2).
psh,min*Fywe = $\text{Min}(psh_x * Fywe, psh_y * Fywe) = 2.724$

psh_x*Fywe = psh1*Fywe1 + ps2*Fywe2 = 2.724
psh1 ((5.4d), TBDY) = $L_{\text{stir1}} * A_{\text{stir1}} / (A_{\text{sec}} * s1) = 0.00301593$
Lstir1 (Length of stirrups along Y) = 2160.00
Astir1 (stirrups area) = 78.53982
psh2 ((5.4d)) = $L_{\text{stir2}} * A_{\text{stir2}} / (A_{\text{sec}} * s2) = 0.00056047$
Lstir2 (Length of stirrups along Y) = 1568.00
Astir2 (stirrups area) = 50.26548

psh_y*Fywe = psh1*Fywe1 + ps2*Fywe2 = 3.25416
psh1 ((5.4d), TBDY) = $L_{\text{stir1}} * A_{\text{stir1}} / (A_{\text{sec}} * s1) = 0.00357443$
Lstir1 (Length of stirrups along X) = 2560.00
Astir1 (stirrups area) = 78.53982
psh2 ((5.4d), TBDY) = $L_{\text{stir2}} * A_{\text{stir2}} / (A_{\text{sec}} * s2) = 0.00070345$
Lstir2 (Length of stirrups along X) = 1968.00
Astir2 (stirrups area) = 50.26548

Asec = 562500.00

s1 = 100.00

s2 = 250.00

fywe1 = 781.25

fywe2 = 656.25

fce = 18.00

From ((5.A5), TBDY), TBDY: cc = 0.00621307

c = confinement factor = 1.42131

y1 = 0.0025

sh1 = 0.008

ft1 = 900.1904

fy1 = 750.1586

su1 = 0.032

using (30) in Biskinis/Fardis (2013) multiplied with shear_factor and also multiplied by the shear_factor according to 15.7.1.4, with Shear_factor = 1.00

lo/lo,min = lb/ld = 1.00

su1 = $0.4 * esu1_{\text{nominal}} ((5.5), \text{TBDY}) = 0.032$

From table 5A.1, TBDY: esu1_nominal = 0.08,

For calculation of esu1_nominal and y1, sh1, ft1, fy1, it is considered characteristic value fsy1 = fs1/1.2, from table 5.1, TBDY.

y1, sh1, ft1, fy1, are also multiplied by $\text{Min}(1, 1.25 * (lb/ld)^{2/3})$, from 10.3.5, ASCE 41-17.

with fs1 = $(f_{s,jacket} * A_{s,ten,jacket} + f_{s,core} * A_{s,ten,core}) / A_{s,ten} = 750.1586$

with Es1 = $(E_{s,jacket} * A_{s,ten,jacket} + E_{s,core} * A_{s,ten,core}) / A_{s,ten} = 200000.00$

y2 = 0.0025

sh2 = 0.008

ft2 = 907.50

fy2 = 756.25

su2 = 0.032

using (30) in Biskinis/Fardis (2013) multiplied with shear_factor and also multiplied by the shear_factor according to 15.7.1.4, with Shear_factor = 1.00

lo/lo,min = lb/lb,min = 1.00

```

su2 = 0.4*esu2_nominal ((5.5), TBDY) = 0.032
From table 5A.1, TBDY: esu2_nominal = 0.08,
For calculation of esu2_nominal and y2, sh2,ft2,fy2, it is considered
characteristic value fsy2 = fs2/1.2, from table 5.1, TBDY.
y1, sh1,ft1,fy1, are also multiplied by Min(1,1.25*(lb/d)^ 2/3), from 10.3.5, ASCE 41-17.
with fs2 = (fsjacket*Asl,com,jacket + fs,core*Asl,com,core)/Asl,com = 756.25
with Es2 = (Es,jacket*Asl,com,jacket + Es,core*Asl,com,core)/Asl,com = 200000.00
yv = 0.0025
shv = 0.008
ftv = 902.993
fyv = 752.4941
suv = 0.032
using (30) in Biskinis/Fardis (2013) multiplied with shear_factor
and also multiplied by the shear_factor according to 15.7.1.4, with
Shear_factor = 1.00
lo/lou,min = lb/d = 1.00
suv = 0.4*esuv_nominal ((5.5), TBDY) = 0.032
From table 5A.1, TBDY: esuv_nominal = 0.08,
considering characteristic value fsyv = fsv/1.2, from table 5.1, TBDY
For calculation of esuv_nominal and yv, shv,ftv,fyv, it is considered
characteristic value fsyv = fsv/1.2, from table 5.1, TBDY.
y1, sh1,ft1,fy1, are also multiplied by Min(1,1.25*(lb/d)^ 2/3), from 10.3.5, ASCE 41-17.
with fsv = (fsjacket*Asl,mid,jacket + fs,mid*Asl,mid,core)/Asl,mid = 752.4941
with Esv = (Es,jacket*Asl,mid,jacket + Es,mid*Asl,mid,core)/Asl,mid = 200000.00
1 = Asl,ten/(b*d)*(fs1/fc) = 0.32428344
2 = Asl,com/(b*d)*(fs2/fc) = 0.20328574
v = Asl,mid/(b*d)*(fsv/fc) = 0.35171278
and confined core properties:
b = 390.00
d = 677.00
d' = 13.00
fcc (5A.2, TBDY) = 25.58352
cc (5A.5, TBDY) = 0.00621307
c = confinement factor = 1.42131
1 = Asl,ten/(b*d)*(fs1/fc) = 0.39075399
2 = Asl,com/(b*d)*(fs2/fc) = 0.24495458
v = Asl,mid/(b*d)*(fsv/fc) = 0.42380571
Case/Assumption: Unconfined full section - Steel rupture
' satisfies Eq. (4.3)
--->
v < vs,y2 - LHS eq.(4.5) is not satisfied
--->
v < vs,c - RHS eq.(4.5) is not satisfied
--->
Case/Assumption Rejected.
--->
New Case/Assumption: Unconfined full section - Spalling of concrete cover
' satisfies Eq. (4.4)
--->
v < s,y1 - LHS eq.(4.7) is not satisfied
--->
v < vc,y1 - RHS eq.(4.6) is not satisfied
--->
cu (4.11) = 0.95920575
MRc (4.18) = 7.7321E+008
--->
New Case/Assumption: Ultimate curvature of confined core after spalling of concrete cover
In expressions below, the following modifications have been made
- b, d, d' replaced by geometric parameters of the core: bo, do, d'o
- N, 1, 2, v normalised to bo*do, instead of b*d
- - parameters of confined concrete, fcc, cc, used in lieu of fc, ecu
--->
Subcase: Rupture of tension steel
--->
v* < v*s,y2 - LHS eq.(4.5) is not satisfied
--->
v* < v*s,c - LHS eq.(4.5) is not satisfied

```

```

---->
Subcase rejected
---->
New Subcase: Failure of compression zone
---->
 $v^* < v^*c,y2$  - LHS eq.(4.6) is not satisfied
---->
 $v^* < v^*c,y1$  - RHS eq.(4.6) is not satisfied
---->
 $*c_u$  (4.11) = 1.00
MRO (4.18) = 8.9683E+008
---->
 $u = c_u$  (4.2) = 9.1773508E-006
Mu = MRO

```

Calculation of ratio l_b/l_d

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

Calculation of μ_{2+}

Calculation of ultimate curvature μ according to 4.1, Biskinis/Fardis 2013:

$u = 1.3930706E-005$

$\mu = 3.0615E+009$

with full section properties:

$b = 950.00$

$d = 707.00$

$d' = 43.00$

$v = 0.63557247$

$N = 7.6839E+006$

$f_c = 18.00$

α (5A.5, TBDY) = 0.002

Final value of c_u : $c_u^* = \text{shear_factor} * \text{Max}(c_u, c_c) = 0.01486841$

The Shear_factor is considered equal to 1 (pure moment strength)

From (5.4b), TBDY: $c_u = 0.01486841$

w_e (5.4c) = 0.08077545

α_{se} ((5.4d), TBDY) = $(\alpha_{se1} * A_{ext} + \alpha_{se2} * A_{int}) / A_{sec} = 0.53375773$

$\alpha_{se1} = \text{Max}(((A_{conf,max1} - A_{noConf1}) / A_{conf,max1}) * (A_{conf,min1} / A_{conf,max1}), 0) = 0.53375773$

The definitions of A_{noConf} , $A_{conf,min}$ and $A_{conf,max}$ are derived from generalization of the rectangular sections confinement, which is expressed by (5.4d).

The generalization is done according to Mander, J., Priestley, M., and Park, R. (1988)

"Theoretical Stress-Strain Model for Confined Concrete."

J. Struct. Eng., 10.1061/(ASCE)0733-9445(1988)114:8(1804), 1804-1826.

$A_{conf,max1} = 464100.00$ is the confined core area at levels of member with hoops and

is calculated as the area of core enclosed by the center lines of the external perimeter hoops.

$A_{conf,min1} = 395025.00$ is the confined core area at midway between the levels of hoops and

is calculated by reducing all the dimensions of the area $A_{conf,max1}$ by a length equal to half the clear spacing between external hoops.

$A_{noConf1} = 173066.667$ is the unconfined external core area which is equal to $b_i^2/6$ as defined at (A.2).

$\alpha_{se2} (\geq \alpha_{se1}) = \text{Max}(((A_{conf,max2} - A_{noConf2}) / A_{conf,max2}) * (A_{conf,min2} / A_{conf,max2}), 0) = 0.53375773$

The definitions of A_{noConf} , $A_{conf,min}$ and $A_{conf,max}$ are derived from generalization of the rectangular sections confinement, which is expressed by (5.4d).

The generalization is done according to Mander, J., Priestley, M., and Park, R. (1988)

"Theoretical Stress-Strain Model for Confined Concrete."

J. Struct. Eng., 10.1061/(ASCE)0733-9445(1988)114:8(1804), 1804-1826.

$A_{conf,max2} = 252164.00$ is the confined core area at levels of member with hoops and

is calculated as the area of core enclosed by the center lines of the internal perimeter hoops.

$A_{conf,min2} = 111441.00$ is the confined core area at midway between the levels of hoops and

is calculated by reducing all the dimensions of the area $A_{conf,max2}$ by a length equal to half the clear spacing between internal hoops.

$A_{noConf2} = 110709.333$ is the unconfined internal core area which is equal to $b_i^2/6$ as defined at (A.2).

$$psh,min*Fywe = \text{Min}(psh,x*Fywe , psh,y*Fywe) = 2.724$$

$$psh_x*Fywe = psh1*Fywe1+ps2*Fywe2 = 2.724$$

$$psh1 ((5.4d), TBDY) = Lstir1*Astir1/(Asec*s1) = 0.00301593$$

$$Lstir1 \text{ (Length of stirrups along Y)} = 2160.00$$

$$Astir1 \text{ (stirrups area)} = 78.53982$$

$$psh2 (5.4d) = Lstir2*Astir2/(Asec*s2) = 0.00056047$$

$$Lstir2 \text{ (Length of stirrups along Y)} = 1568.00$$

$$Astir2 \text{ (stirrups area)} = 50.26548$$

$$psh_y*Fywe = psh1*Fywe1+ps2*Fywe2 = 3.25416$$

$$psh1 ((5.4d), TBDY) = Lstir1*Astir1/(Asec*s1) = 0.00357443$$

$$Lstir1 \text{ (Length of stirrups along X)} = 2560.00$$

$$Astir1 \text{ (stirrups area)} = 78.53982$$

$$psh2 ((5.4d), TBDY) = Lstir2*Astir2/(Asec*s2) = 0.00070345$$

$$Lstir2 \text{ (Length of stirrups along X)} = 1968.00$$

$$Astir2 \text{ (stirrups area)} = 50.26548$$

$$Asec = 562500.00$$

$$s1 = 100.00$$

$$s2 = 250.00$$

$$fywe1 = 781.25$$

$$fywe2 = 656.25$$

$$fce = 18.00$$

$$\text{From } ((5.A5), TBDY), TBDY: cc = 0.00621307$$

$$c = \text{confinement factor} = 1.42131$$

$$y1 = 0.0025$$

$$sh1 = 0.008$$

$$ft1 = 907.50$$

$$fy1 = 756.25$$

$$su1 = 0.032$$

using (30) in Biskinis/Fardis (2013) multiplied with shear_factor
and also multiplied by the shear_factor according to 15.7.1.4, with
Shear_factor = 1.00

$$lo/lou,min = lb/l_d = 1.00$$

$$su1 = 0.4*esu1_nominal ((5.5), TBDY) = 0.032$$

$$\text{From table 5A.1, TBDY: } esu1_nominal = 0.08,$$

For calculation of esu1_nominal and y1, sh1,ft1,fy1, it is considered
characteristic value fsy1 = fs1/1.2, from table 5.1, TBDY.

$$y1, sh1,ft1,fy1, \text{ are also multiplied by } \text{Min}(1,1.25*(lb/l_d)^{2/3}), \text{ from 10.3.5, ASCE 41-17.}$$

$$\text{with } fs1 = (fs,jacket*Asl,ten,jacket + fs,core*Asl,ten,core)/Asl,ten = 756.25$$

$$\text{with } Es1 = (Es,jacket*Asl,ten,jacket + Es,core*Asl,ten,core)/Asl,ten = 200000.00$$

$$y2 = 0.0025$$

$$sh2 = 0.008$$

$$ft2 = 900.1904$$

$$fy2 = 750.1586$$

$$su2 = 0.032$$

using (30) in Biskinis/Fardis (2013) multiplied with shear_factor
and also multiplied by the shear_factor according to 15.7.1.4, with
Shear_factor = 1.00

$$lo/lou,min = lb/lb,min = 1.00$$

$$su2 = 0.4*esu2_nominal ((5.5), TBDY) = 0.032$$

$$\text{From table 5A.1, TBDY: } esu2_nominal = 0.08,$$

For calculation of esu2_nominal and y2, sh2,ft2,fy2, it is considered
characteristic value fsy2 = fs2/1.2, from table 5.1, TBDY.

$$y1, sh1,ft1,fy1, \text{ are also multiplied by } \text{Min}(1,1.25*(lb/l_d)^{2/3}), \text{ from 10.3.5, ASCE 41-17.}$$

$$\text{with } fs2 = (fs,jacket*Asl,com,jacket + fs,core*Asl,com,core)/Asl,com = 750.1586$$

$$\text{with } Es2 = (Es,jacket*Asl,com,jacket + Es,core*Asl,com,core)/Asl,com = 200000.00$$

$$yv = 0.0025$$

$$shv = 0.008$$

$$ftv = 902.993$$

$$fyv = 752.4941$$

$$suv = 0.032$$

using (30) in Biskinis/Fardis (2013) multiplied with shear_factor
and also multiplied by the shear_factor according to 15.7.1.4, with
Shear_factor = 1.00

$$lo/lou,min = lb/l_d = 1.00$$

```

suv = 0.4*esuv_nominal ((5.5), TBDY) = 0.032
From table 5A.1, TBDY: esuv_nominal = 0.08,
considering characteristic value fsyv = fsv/1.2, from table 5.1, TBDY
For calculation of esuv_nominal and yv, shv,ftv,fyv, it is considered
characteristic value fsyv = fsv/1.2, from table 5.1, TBDY.
y1, sh1,ft1,fy1, are also multiplied by Min(1,1.25*(lb/d)^2/3), from 10.3.5, ASCE 41-17.
with fsv = (fs,jacket*Asl,mid,jacket + fs,mid*Asl,mid,core)/Asl,mid = 752.4941
with Esv = (Es,jacket*Asl,mid,jacket + Es,mid*Asl,mid,core)/Asl,mid = 200000.00
1 = Asl,ten/(b*d)*(fs1/fc) = 0.09629324
2 = Asl,com/(b*d)*(fs2/fc) = 0.15360794
v = Asl,mid/(b*d)*(fsv/fc) = 0.16660079
and confined core properties:
b = 890.00
d = 677.00
d' = 13.00
fcc (5A.2, TBDY) = 25.58352
cc (5A.5, TBDY) = 0.00621307
c = confinement factor = 1.42131
1 = Asl,ten/(b*d)*(fs1/fc) = 0.10733965
2 = Asl,com/(b*d)*(fs2/fc) = 0.17122928
v = Asl,mid/(b*d)*(fsv/fc) = 0.18571261
Case/Assumption: Unconfined full section - Steel rupture
' satisfies Eq. (4.3)
---->
v < vs,y2 - LHS eq.(4.5) is not satisfied
---->
v < vs,c - RHS eq.(4.5) is not satisfied
---->
Case/Assumption Rejected.
---->
New Case/Assumption: Unconfined full section - Spalling of concrete cover
' satisfies Eq. (4.4)
---->
v < sy1 - LHS eq.(4.7) is not satisfied
---->
v < vc,y1 - RHS eq.(4.6) is satisfied
---->
cu (4.10) = 0.58509201
MRc (4.17) = 2.3801E+009
---->
New Case/Assumption: Ultimate curvature of confined core after spalling of concrete cover
In expressions below, the following modifications have been made
- b, d, d' replaced by geometric parameters of the core: bo, do, d'o
- N, 1, 2, v normalised to bo*do, instead of b*d
- parameters of confined concrete, fcc, cc, used in lieu of fc, ecu
---->
Subcase: Rupture of tension steel
---->
v* < v*sy2 - LHS eq.(4.5) is not satisfied
---->
v* < v*sc - LHS eq.(4.5) is not satisfied
---->
Subcase rejected
---->
New Subcase: Failure of compression zone
---->
v* < v*cy2 - LHS eq.(4.6) is not satisfied
---->
v* < v*cy1 - RHS eq.(4.6) is not satisfied
---->
*cu (4.11) = 0.65878578
MRo (4.18) = 3.0615E+009
---->
u = cu (4.2) = 1.3930706E-005
Mu = MRo
-----

```


Calculation of ratio l_b/d

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

Calculation of μ_u

Calculation of ultimate curvature μ_u according to 4.1, Biskinis/Fardis 2013:

$$\mu_u = 9.1773508E-006$$

$$\mu_u = 8.9683E+008$$

with full section properties:

$$b = 450.00$$

$$d = 707.00$$

$$d' = 43.00$$

$$\nu = 1.34176$$

$$N = 7.6839E+006$$

$$f_c = 18.00$$

$$c_o \text{ (5A.5, TBDY)} = 0.002$$

$$\text{Final value of } \mu_u: \mu_u = \text{shear_factor} * \text{Max}(\mu_u, c_o) = 0.01486841$$

The Shear_factor is considered equal to 1 (pure moment strength)

$$\text{From (5.4b), TBDY: } \mu_u = 0.01486841$$

$$\mu_u \text{ (5.4c)} = 0.08077545$$

$$\mu_u \text{ ((5.4d), TBDY)} = (\mu_{u1} * A_{ext} + \mu_{u2} * A_{int}) / A_{sec} = 0.53375773$$

$$\mu_{u1} = \text{Max}(((A_{conf,max1} - A_{noConf1}) / A_{conf,max1}) * (A_{conf,min1} / A_{conf,max1}), 0) = 0.53375773$$

The definitions of A_{noConf} , $A_{conf,min}$ and $A_{conf,max}$ are derived from generalization of the rectangular sections confinement, which is expressed by (5.4d).

The generalization is done according to Mander, J., Priestley, M., and Park, R. (1988)

"Theoretical Stress-Strain Model for Confined Concrete."

J. Struct. Eng., 10.1061/(ASCE)0733-9445(1988)114:8(1804), 1804-1826.

$A_{conf,max1} = 464100.00$ is the confined core area at levels of member with hoops and

is calculated as the area of core enclosed by the center lines of the external perimeter hoops.

$A_{conf,min1} = 395025.00$ is the confined core area at midway between the levels of hoops and

is calculated by reducing all the dimensions of the area $A_{conf,max1}$ by a length

equal to half the clear spacing between external hoops.

$A_{noConf1} = 173066.667$ is the unconfined external core area which is equal to $b_i^2/6$ as defined at (A.2).

$$\mu_{u2} (\geq \mu_{u1}) = \text{Max}(((A_{conf,max2} - A_{noConf2}) / A_{conf,max2}) * (A_{conf,min2} / A_{conf,max2}), 0) = 0.53375773$$

The definitions of A_{noConf} , $A_{conf,min}$ and $A_{conf,max}$ are derived from generalization of the rectangular sections confinement, which is expressed by (5.4d).

The generalization is done according to Mander, J., Priestley, M., and Park, R. (1988)

"Theoretical Stress-Strain Model for Confined Concrete."

J. Struct. Eng., 10.1061/(ASCE)0733-9445(1988)114:8(1804), 1804-1826.

$A_{conf,max2} = 252164.00$ is the confined core area at levels of member with hoops and

is calculated as the area of core enclosed by the center lines of the internal perimeter hoops.

$A_{conf,min2} = 111441.00$ is the confined core area at midway between the levels of hoops and

is calculated by reducing all the dimensions of the area $A_{conf,max2}$ by a length

equal to half the clear spacing between internal hoops.

$A_{noConf2} = 110709.333$ is the unconfined internal core area which is equal to $b_i^2/6$ as defined at (A.2).

$$p_{sh,min} * F_{ywe} = \text{Min}(p_{sh,x} * F_{ywe}, p_{sh,y} * F_{ywe}) = 2.724$$

$$p_{sh,x} * F_{ywe} = p_{sh1} * F_{ywe1} + p_{sh2} * F_{ywe2} = 2.724$$

$$p_{sh1} \text{ ((5.4d), TBDY)} = L_{stir1} * A_{stir1} / (A_{sec} * s_1) = 0.00301593$$

$$L_{stir1} \text{ (Length of stirrups along Y)} = 2160.00$$

$$A_{stir1} \text{ (stirrups area)} = 78.53982$$

$$p_{sh2} \text{ (5.4d)} = L_{stir2} * A_{stir2} / (A_{sec} * s_2) = 0.00056047$$

$$L_{stir2} \text{ (Length of stirrups along Y)} = 1568.00$$

$$A_{stir2} \text{ (stirrups area)} = 50.26548$$

$$p_{sh,y} * F_{ywe} = p_{sh1} * F_{ywe1} + p_{sh2} * F_{ywe2} = 3.25416$$

$$p_{sh1} \text{ ((5.4d), TBDY)} = L_{stir1} * A_{stir1} / (A_{sec} * s_1) = 0.00357443$$

$$L_{stir1} \text{ (Length of stirrups along X)} = 2560.00$$

$$A_{stir1} \text{ (stirrups area)} = 78.53982$$

$$p_{sh2} \text{ ((5.4d), TBDY)} = L_{stir2} * A_{stir2} / (A_{sec} * s_2) = 0.00070345$$

$$L_{stir2} \text{ (Length of stirrups along X)} = 1968.00$$

Astir2 (stirrups area) = 50.26548

Asec = 562500.00

s1 = 100.00

s2 = 250.00

fywe1 = 781.25

fywe2 = 656.25

fce = 18.00

From ((5.A5), TBDY), TBDY: cc = 0.00621307

c = confinement factor = 1.42131

y1 = 0.0025

sh1 = 0.008

ft1 = 900.1904

fy1 = 750.1586

su1 = 0.032

using (30) in Biskinis/Fardis (2013) multiplied with shear_factor
and also multiplied by the shear_factor according to 15.7.1.4, with
Shear_factor = 1.00

lo/lou,min = lb/lb,min = 1.00

su1 = 0.4*esu1_nominal ((5.5), TBDY) = 0.032

From table 5A.1, TBDY: esu1_nominal = 0.08,

For calculation of esu1_nominal and y1, sh1,ft1,fy1, it is considered
characteristic value fsy1 = fs1/1.2, from table 5.1, TBDY.

y1, sh1,ft1,fy1, are also multiplied by $\text{Min}(1, 1.25 \cdot (lb/lb)^{2/3})$, from 10.3.5, ASCE 41-17.

with fs1 = (fs,jacket*Asl,ten,jacket + fs,core*Asl,ten,core)/Asl,ten = 750.1586

with Es1 = (Es,jacket*Asl,ten,jacket + Es,core*Asl,ten,core)/Asl,ten = 200000.00

y2 = 0.0025

sh2 = 0.008

ft2 = 907.50

fy2 = 756.25

su2 = 0.032

using (30) in Biskinis/Fardis (2013) multiplied with shear_factor
and also multiplied by the shear_factor according to 15.7.1.4, with
Shear_factor = 1.00

lo/lou,min = lb/lb,min = 1.00

su2 = 0.4*esu2_nominal ((5.5), TBDY) = 0.032

From table 5A.1, TBDY: esu2_nominal = 0.08,

For calculation of esu2_nominal and y2, sh2,ft2,fy2, it is considered
characteristic value fsy2 = fs2/1.2, from table 5.1, TBDY.

y1, sh1,ft1,fy1, are also multiplied by $\text{Min}(1, 1.25 \cdot (lb/lb)^{2/3})$, from 10.3.5, ASCE 41-17.

with fs2 = (fs,jacket*Asl,com,jacket + fs,core*Asl,com,core)/Asl,com = 756.25

with Es2 = (Es,jacket*Asl,com,jacket + Es,core*Asl,com,core)/Asl,com = 200000.00

yv = 0.0025

shv = 0.008

ftv = 902.993

fyv = 752.4941

suv = 0.032

using (30) in Biskinis/Fardis (2013) multiplied with shear_factor
and also multiplied by the shear_factor according to 15.7.1.4, with
Shear_factor = 1.00

lo/lou,min = lb/lb,min = 1.00

suv = 0.4*esuv_nominal ((5.5), TBDY) = 0.032

From table 5A.1, TBDY: esuv_nominal = 0.08,

considering characteristic value fsyv = fsv/1.2, from table 5.1, TBDY

For calculation of esuv_nominal and yv, shv,ftv,fyv, it is considered
characteristic value fsyv = fsv/1.2, from table 5.1, TBDY.

y1, sh1,ft1,fy1, are also multiplied by $\text{Min}(1, 1.25 \cdot (lb/lb)^{2/3})$, from 10.3.5, ASCE 41-17.

with fsv = (fs,jacket*Asl,mid,jacket + fs,mid*Asl,mid,core)/Asl,mid = 752.4941

with Esv = (Es,jacket*Asl,mid,jacket + Es,mid*Asl,mid,core)/Asl,mid = 200000.00

1 = Asl,ten/(b*d)*(fs1/fc) = 0.32428344

2 = Asl,com/(b*d)*(fs2/fc) = 0.20328574

v = Asl,mid/(b*d)*(fsv/fc) = 0.35171278

and confined core properties:

b = 390.00

d = 677.00

d' = 13.00

fcc (5A.2, TBDY) = 25.58352

```

cc (5A.5, TBDY) = 0.00621307
c = confinement factor = 1.42131
1 = Asl,ten/(b*d)*(fs1/fc) = 0.39075399
2 = Asl,com/(b*d)*(fs2/fc) = 0.24495458
v = Asl,mid/(b*d)*(fsv/fc) = 0.42380571
Case/Assumption: Unconfined full section - Steel rupture
'satisfies Eq. (4.3)
--->
v < vs,y2 - LHS eq.(4.5) is not satisfied
--->
v < vs,c - RHS eq.(4.5) is not satisfied
--->
Case/Assumption Rejected.
--->
New Case/Assumption: Unconfined full section - Spalling of concrete cover
'satisfies Eq. (4.4)
--->
v < s,y1 - LHS eq.(4.7) is not satisfied
--->
v < vc,y1 - RHS eq.(4.6) is not satisfied
--->
cu (4.11) = 0.95920575
MRc (4.18) = 7.7321E+008
--->
New Case/Assumption: Ultimate curvature of confined core after spalling of concrete cover
In expressions below, the following modifications have been made
- b, d, d' replaced by geometric parameters of the core: bo, do, d'o
- N, 1, 2, v normalised to bo*do, instead of b*d
- - parameters of confined concrete, fcc, cc, used in lieu of fc, ecu
--->
Subcase: Rupture of tension steel
--->
v* < v*s,y2 - LHS eq.(4.5) is not satisfied
--->
v* < v*s,c - LHS eq.(4.5) is not satisfied
--->
Subcase rejected
--->
New Subcase: Failure of compression zone
--->
v* < v*c,y2 - LHS eq.(4.6) is not satisfied
--->
v* < v*c,y1 - RHS eq.(4.6) is not satisfied
--->
*cu (4.11) = 1.00
MRo (4.18) = 8.9683E+008
--->
u = cu (4.2) = 9.1773508E-006
Mu = MRo

```

Calculation of ratio lb/l_d

Adequate Lap Length: lb/l_d >= 1

Calculation of Shear Strength $V_r = \text{Min}(V_{r1}, V_{r2}) = 1.1678\text{E}+006$

Calculation of Shear Strength at edge 1, $V_{r1} = 1.1678\text{E}+006$

$V_{r1} = V_{Co1} \text{ ((10.3), ASCE 41-17)} = k_{nl} * V_{Co10}$

$V_{Co10} = 1.1678\text{E}+006$

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

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

= 1 (normal-weight concrete)
 Mean concrete strength: $fc' = (fc'_{jacket} \cdot Area_{jacket} + fc'_{core} \cdot Area_{core}) / Area_{section} = 15.20$, but $fc'^{0.5} \leq 8.3$ MPa (22.5.3.1, ACI 318-14)
 $M/Vd = 4.00$
 $\mu_u = 7.6104E+008$
 $V_u = 227.4778$
 $d = 0.8 \cdot h = 600.00$
 $N_u = 7.6839E+006$
 $A_g = 337500.00$
 From (11.5.4.8), ACI 318-14: $V_s = V_{sjacket} + V_{s,core} = 1.0354E+006$
 where:
 $V_{sjacket} = V_{sj1} + V_{sj2} = 942477.796$
 $V_{sj1} = 589048.623$ is calculated for section web jacket, with:
 $d = 600.00$
 $A_v = 157079.633$
 $f_y = 625.00$
 $s = 100.00$
 V_{sj1} is multiplied by $Col,j1 = 1.00$
 $s/d = 0.16666667$
 $V_{sj2} = 353429.174$ is calculated for section flange jacket, with:
 $d = 360.00$
 $A_v = 157079.633$
 $f_y = 625.00$
 $s = 100.00$
 V_{sj2} is multiplied by $Col,j2 = 1.00$
 $s/d = 0.27777778$
 $V_{s,core} = V_{s,c1} + V_{s,c2} = 92890.612$
 $V_{s,c1} = 92890.612$ is calculated for section web core, with:
 $d = 440.00$
 $A_v = 100530.965$
 $f_y = 525.00$
 $s = 250.00$
 $V_{s,c1}$ is multiplied by $Col,c1 = 1.00$
 $s/d = 0.56818182$
 $V_{s,c2} = 0.00$ is calculated for section flange core, with:
 $d = 200.00$
 $A_v = 100530.965$
 $f_y = 525.00$
 $s = 250.00$
 $V_{s,c2}$ is multiplied by $Col,c2 = 0.00$
 $s/d = 1.25$
 $V_f ((11-3)-(11.4), ACI 440) = 0.00$
 From (11-11), ACI 440: $V_s + V_f \leq 699281.943$
 $bw = 450.00$

Calculation of Shear Strength at edge 2, $V_{r2} = 1.1678E+006$
 $V_{r2} = V_{Col} ((10.3), ASCE 41-17) = knl \cdot V_{Col0}$
 $V_{Col0} = 1.1678E+006$
 $knl = 1$ (zero step-static loading)

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

= 1 (normal-weight concrete)
 Mean concrete strength: $fc' = (fc'_{jacket} \cdot Area_{jacket} + fc'_{core} \cdot Area_{core}) / Area_{section} = 15.20$, but $fc'^{0.5} \leq 8.3$ MPa (22.5.3.1, ACI 318-14)
 $M/Vd = 4.00$
 $\mu_u = 6.8991E+008$
 $V_u = 227.4778$
 $d = 0.8 \cdot h = 600.00$
 $N_u = 7.6839E+006$
 $A_g = 337500.00$
 From (11.5.4.8), ACI 318-14: $V_s = V_{sjacket} + V_{s,core} = 1.0354E+006$
 where:
 $V_{sjacket} = V_{sj1} + V_{sj2} = 942477.796$

Vs,j1 = 589048.623 is calculated for section web jacket, with:

d = 600.00

Av = 157079.633

fy = 625.00

s = 100.00

Vs,j1 is multiplied by Col,j1 = 1.00

s/d = 0.16666667

Vs,j2 = 353429.174 is calculated for section flange jacket, with:

d = 360.00

Av = 157079.633

fy = 625.00

s = 100.00

Vs,j2 is multiplied by Col,j2 = 1.00

s/d = 0.27777778

Vs,core = Vs,c1 + Vs,c2 = 92890.612

Vs,c1 = 92890.612 is calculated for section web core, with:

d = 440.00

Av = 100530.965

fy = 525.00

s = 250.00

Vs,c1 is multiplied by Col,c1 = 1.00

s/d = 0.56818182

Vs,c2 = 0.00 is calculated for section flange core, with:

d = 200.00

Av = 100530.965

fy = 525.00

s = 250.00

Vs,c2 is multiplied by Col,c2 = 0.00

s/d = 1.25

Vf ((11-3)-(11.4), ACI 440) = 0.00

From (11-11), ACI 440: Vs + Vf <= 699281.943

bw = 450.00

End Of Calculation of Shear Capacity ratio for element: column JTC1 of floor 1
At local axis: 3

Start Of Calculation of Shear Capacity ratio for element: column JTC1 of floor 1
At Shear local axis: 2
(Bending local axis: 3)
Section Type: rcjtcs

Constant Properties

Knowledge Factor, = 0.80

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

Consequently:

Jacket

New material of Primary Member: Concrete Strength, fc = fcm = 18.00

New material of Primary Member: Steel Strength, fs = fsm = 625.00

Concrete Elasticity, Ec = 19940.411

Steel Elasticity, Es = 200000.00

Existing Column

Existing material of Primary Member: Concrete Strength, fc = fcm = 12.00

Existing material of Primary Member: Steel Strength, fs = fsm = 525.00

Concrete Elasticity, Ec = 16281.278

Steel Elasticity, Es = 200000.00

#####

Note: Especially for the calculation of moment strengths,

the above steel re-bar strengths are multiplied by 1.25 according to R18.6.5, ACI 318-14

Jacket

New material: Steel Strength, fs = 1.25*fsm = 781.25

Existing Column

Existing material: Steel Strength, fs = 1.25*fsm = 656.25

#####

Max Height, Hmax = 750.00
Min Height, Hmin = 450.00
Max Width, Wmax = 950.00
Min Width, Wmin = 450.00
Eccentricity, Ecc = 250.00
Jacket Thickness, tj = 100.00
Cover Thickness, c = 25.00
Mean Confinement Factor overall section = 1.42131
Element Length, L = 3000.00
Primary Member
Ribbed Bars
Ductile Steel
With Detailing for Earthquake Resistance (including stirrups closed at 135°)
Longitudinal Bars With Ends Lapped Starting at the End Sections
Adequate Lap Length ($l_o/l_{ou}, \min \geq 1$)
No FRP Wrapping

Stepwise Properties

At local axis: 2
EDGE -A-
Shear Force, Va = -137.4894
EDGE -B-
Shear Force, Vb = 137.4894
BOTH EDGES
Axial Force, F = -7.6839E+006
Longitudinal Reinforcement Area Distribution (in 2 divisions)
-Tension: Aslt = 314.1593
-Compression: Aslc = 6176.371
Longitudinal Reinforcement Area Distribution (in 3 divisions)
-Tension: Asl,ten = 1539.38
-Compression: Asl,com = 1539.38
-Middle: Asl,mid = 3612.832

Calculation of Shear Capacity ratio , $V_e/V_r = 0.86614729$
Member Controlled by Flexure ($V_e/V_r < 1$)
Calculation of Shear Demand from fig. R18.6.5, ACI 318-14 $V_e = (M_{pr1} + M_{pr2})/l_n = 1.2287E+006$
with
 $M_{pr1} = \max(\mu_{u1+}, \mu_{u1-}) = 1.8431E+009$
 $\mu_{u1+} = 1.8431E+009$, is the ultimate moment strength at the edge 1 of the member in the actual moment direction
which is defined for the static loading combination
 $\mu_{u1-} = 1.8431E+009$, is the ultimate moment strength at the edge 1 of the member in the opposite moment
direction which is defined for the static loading combination
 $M_{pr2} = \max(\mu_{u2+}, \mu_{u2-}) = 1.8431E+009$
 $\mu_{u2+} = 1.8431E+009$, is the ultimate moment strength at the edge 2 of the member in the actual moment direction
which is defined for the static loading combination
 $\mu_{u2-} = 1.8431E+009$, is the ultimate moment strength at the edge 2 of the member in the opposite moment
direction which is defined for the static loading combination

Calculation of μ_{u1+}

Calculation of ultimate curvature μ_u according to 4.1, Biskinis/Fardis 2013:
 $\mu_u = 7.8730391E-006$
 $\mu_u = 1.8431E+009$

with full section properties:

b = 450.00
d = 907.00
d' = 43.00

$v = 1.0459$
 $N = 7.6839E+006$
 $f_c = 18.00$
 $\alpha (5A.5, TBDY) = 0.002$
 Final value of α : $\alpha^* = \text{shear_factor} * \text{Max}(\alpha, \alpha_c) = 0.01486841$
 The Shear_factor is considered equal to 1 (pure moment strength)
 From (5.4b), TBDY: $\alpha = 0.01486841$
 $w_e (5.4c) = 0.08077545$
 $\alpha_{se} ((5.4d), TBDY) = (\alpha_{se1} * A_{ext} + \alpha_{se2} * A_{int}) / A_{sec} = 0.53375773$
 $\alpha_{se1} = \text{Max}(((A_{conf,max1} - A_{noConf1}) / A_{conf,max1}) * (A_{conf,min1} / A_{conf,max1}), 0) = 0.53375773$
 The definitions of A_{noConf} , $A_{conf,min}$ and $A_{conf,max}$ are derived from generalization of the rectangular sections confinement, which is expressed by (5.4d).
 The generalization is done according to Mander, J., Priestley, M., and Park, R. (1988) "Theoretical Stress-Strain Model for Confined Concrete."
 J. Struct. Eng., 10.1061/(ASCE)0733-9445(1988)114:8(1804), 1804-1826.
 $A_{conf,max1} = 464100.00$ is the confined core area at levels of member with hoops and is calculated as the area of core enclosed by the center lines of the external perimeter hoops.
 $A_{conf,min1} = 395025.00$ is the confined core area at midway between the levels of hoops and is calculated by reducing all the dimensions of the area $A_{conf,max1}$ by a length equal to half the clear spacing between external hoops.
 $A_{noConf1} = 173066.667$ is the unconfined external core area which is equal to $b_i^2/6$ as defined at (A.2).
 $\alpha_{se2} (>= \alpha_{se1}) = \text{Max}(((A_{conf,max2} - A_{noConf2}) / A_{conf,max2}) * (A_{conf,min2} / A_{conf,max2}), 0) = 0.53375773$
 The definitions of A_{noConf} , $A_{conf,min}$ and $A_{conf,max}$ are derived from generalization of the rectangular sections confinement, which is expressed by (5.4d).
 The generalization is done according to Mander, J., Priestley, M., and Park, R. (1988) "Theoretical Stress-Strain Model for Confined Concrete."
 J. Struct. Eng., 10.1061/(ASCE)0733-9445(1988)114:8(1804), 1804-1826.
 $A_{conf,max2} = 252164.00$ is the confined core area at levels of member with hoops and is calculated as the area of core enclosed by the center lines of the internal perimeter hoops.
 $A_{conf,min2} = 111441.00$ is the confined core area at midway between the levels of hoops and is calculated by reducing all the dimensions of the area $A_{conf,max2}$ by a length equal to half the clear spacing between internal hoops.
 $A_{noConf2} = 110709.333$ is the unconfined internal core area which is equal to $b_i^2/6$ as defined at (A.2).
 $p_{sh,min} * F_{ywe} = \text{Min}(p_{sh,x} * F_{ywe}, p_{sh,y} * F_{ywe}) = 2.724$

$p_{sh,x} * F_{ywe} = p_{sh1} * F_{ywe1} + p_{sh2} * F_{ywe2} = 2.724$
 $p_{sh1} ((5.4d), TBDY) = L_{stir1} * A_{stir1} / (A_{sec} * s_1) = 0.00301593$
 L_{stir1} (Length of stirrups along Y) = 2160.00
 A_{stir1} (stirrups area) = 78.53982
 $p_{sh2} (5.4d) = L_{stir2} * A_{stir2} / (A_{sec} * s_2) = 0.00056047$
 L_{stir2} (Length of stirrups along Y) = 1568.00
 A_{stir2} (stirrups area) = 50.26548

$p_{sh,y} * F_{ywe} = p_{sh1} * F_{ywe1} + p_{sh2} * F_{ywe2} = 3.25416$
 $p_{sh1} ((5.4d), TBDY) = L_{stir1} * A_{stir1} / (A_{sec} * s_1) = 0.00357443$
 L_{stir1} (Length of stirrups along X) = 2560.00
 A_{stir1} (stirrups area) = 78.53982
 $p_{sh2} ((5.4d), TBDY) = L_{stir2} * A_{stir2} / (A_{sec} * s_2) = 0.00070345$
 L_{stir2} (Length of stirrups along X) = 1968.00
 A_{stir2} (stirrups area) = 50.26548

$A_{sec} = 562500.00$
 $s_1 = 100.00$
 $s_2 = 250.00$
 $f_{ywe1} = 781.25$
 $f_{ywe2} = 656.25$
 $f_{ce} = 18.00$
 From ((5A5), TBDY), TBDY: $\alpha_c = 0.00621307$
 $c = \text{confinement factor} = 1.42131$
 $\gamma_1 = 0.0025$
 $\gamma_{sh1} = 0.008$
 $f_{t1} = 907.50$
 $f_{y1} = 756.25$
 $\gamma_{su1} = 0.032$
 using (30) in Biskinis/Fardis (2013) multiplied with shear_factor and also multiplied by the shear_factor according to 15.7.1.4, with Shear_factor = 1.00

$l_o/l_{ou,min} = l_b/l_d = 1.00$
 $su_1 = 0.4 * esu_{1,nominal} ((5.5), TBDY) = 0.032$
 From table 5A.1, TBDY: $esu_{1,nominal} = 0.08$,
 For calculation of $esu_{1,nominal}$ and y_1, sh_1, ft_1, fy_1 , it is considered
 characteristic value $fsy_1 = fs_1/1.2$, from table 5.1, TBDY.
 y_1, sh_1, ft_1, fy_1 , are also multiplied by $Min(1, 1.25 * (l_b/l_d)^{2/3})$, from 10.3.5, ASCE 41-17.
 with $fs_1 = (fs_{jacket} * A_{sl,ten,jacket} + fs_{core} * A_{sl,ten,core}) / A_{sl,ten} = 756.25$
 with $Es_1 = (Es_{jacket} * A_{sl,ten,jacket} + Es_{core} * A_{sl,ten,core}) / A_{sl,ten} = 200000.00$
 $y_2 = 0.0025$
 $sh_2 = 0.008$
 $ft_2 = 907.50$
 $fy_2 = 756.25$
 $su_2 = 0.032$
 using (30) in Biskinis/Fardis (2013) multiplied with shear_factor
 and also multiplied by the shear_factor according to 15.7.1.4, with
 Shear_factor = 1.00
 $l_o/l_{ou,min} = l_b/l_{b,min} = 1.00$
 $su_2 = 0.4 * esu_{2,nominal} ((5.5), TBDY) = 0.032$
 From table 5A.1, TBDY: $esu_{2,nominal} = 0.08$,
 For calculation of $esu_{2,nominal}$ and y_2, sh_2, ft_2, fy_2 , it is considered
 characteristic value $fsy_2 = fs_2/1.2$, from table 5.1, TBDY.
 y_1, sh_1, ft_1, fy_1 , are also multiplied by $Min(1, 1.25 * (l_b/l_d)^{2/3})$, from 10.3.5, ASCE 41-17.
 with $fs_2 = (fs_{jacket} * A_{sl,com,jacket} + fs_{core} * A_{sl,com,core}) / A_{sl,com} = 756.25$
 with $Es_2 = (Es_{jacket} * A_{sl,com,jacket} + Es_{core} * A_{sl,com,core}) / A_{sl,com} = 200000.00$
 $y_v = 0.0025$
 $sh_v = 0.008$
 $ft_v = 899.1522$
 $fy_v = 749.2935$
 $suv = 0.032$
 using (30) in Biskinis/Fardis (2013) multiplied with shear_factor
 and also multiplied by the shear_factor according to 15.7.1.4, with
 Shear_factor = 1.00
 $l_o/l_{ou,min} = l_b/l_d = 1.00$
 $suv = 0.4 * esuv_{nominal} ((5.5), TBDY) = 0.032$
 From table 5A.1, TBDY: $esuv_{nominal} = 0.08$,
 considering characteristic value $fsyv = fsv/1.2$, from table 5.1, TBDY
 For calculation of $esuv_{nominal}$ and y_v, sh_v, ft_v, fy_v , it is considered
 characteristic value $fsyv = fsv/1.2$, from table 5.1, TBDY.
 y_1, sh_1, ft_1, fy_1 , are also multiplied by $Min(1, 1.25 * (l_b/l_d)^{2/3})$, from 10.3.5, ASCE 41-17.
 with $fsv = (fs_{jacket} * A_{sl,mid,jacket} + fs_{mid} * A_{sl,mid,core}) / A_{sl,mid} = 749.2935$
 with $Es_v = (Es_{jacket} * A_{sl,mid,jacket} + Es_{mid} * A_{sl,mid,core}) / A_{sl,mid} = 200000.00$
 $1 = A_{sl,ten} / (b * d) * (fs_1 / fc) = 0.15845977$
 $2 = A_{sl,com} / (b * d) * (fs_2 / fc) = 0.15845977$
 $v = A_{sl,mid} / (b * d) * (fsv / fc) = 0.36847443$
 and confined core properties:
 $b = 390.00$
 $d = 877.00$
 $d' = 13.00$
 $f_{cc} (5A.2, TBDY) = 25.58352$
 $cc (5A.5, TBDY) = 0.00621307$
 $c = \text{confinement factor} = 1.42131$
 $1 = A_{sl,ten} / (b * d) * (fs_1 / fc) = 0.18909264$
 $2 = A_{sl,com} / (b * d) * (fs_2 / fc) = 0.18909264$
 $v = A_{sl,mid} / (b * d) * (fsv / fc) = 0.43970658$
 Case/Assumption: Unconfined full section - Steel rupture
 ' satisfies Eq. (4.3)
 --->
 $v < v_{s,y2}$ - LHS eq.(4.5) is not satisfied
 --->
 $v < v_{s,c}$ - RHS eq.(4.5) is not satisfied
 --->
 Case/Assumption Rejected.
 --->
 New Case/Assumption: Unconfined full section - Spalling of concrete cover
 ' satisfies Eq. (4.4)
 --->
 $v < s_{y1}$ - LHS eq.(4.7) is not satisfied


```

---->
v < vc,y1 - RHS eq.(4.6) is satisfied
---->
cu (4.10) = 0.83934343
MRc (4.17) = 1.8439E+009
---->
New Case/Assumption: Ultimate curvature of confined core after spalling of concrete cover
In expressions below, the following modifications have been made
- b, d, d' replaced by geometric parameters of the core: bo, do, d'o
- N, 1, 2, v normalised to bo*do, instead of b*d
- - parameters of confined concrete, fcc, cc, used in lieu of fc, ecu

```

```

---->
Subcase: Rupture of tension steel

```

```

---->
v* < v*s,y2 - LHS eq.(4.5) is not satisfied
---->
v* < v*s,c - LHS eq.(4.5) is not satisfied

```

```

---->
Subcase rejected

```

```

---->
New Subcase: Failure of compression zone

```

```

---->
v* < v*c,y2 - LHS eq.(4.6) is not satisfied
---->
v* < v*c,y1 - RHS eq.(4.6) is not satisfied

```

```

---->
*cu (4.11) = 0.89983732
MRo (4.18) = 1.8431E+009

```

```

---->
u = cu (4.2) = 7.8730391E-006
Mu = MRo

```

```

-----
Calculation of ratio lb/d

```

```

-----
Adequate Lap Length: lb/d >= 1

```

```

-----
Calculation of Mu1-

```

```

-----
Calculation of ultimate curvature u according to 4.1, Biskinis/Fardis 2013:

```

```

u = 7.8730391E-006
Mu = 1.8431E+009

```

```

-----
with full section properties:

```

```

b = 450.00
d = 907.00
d' = 43.00
v = 1.0459
N = 7.6839E+006

```

```

fc = 18.00
co (5A.5, TBDY) = 0.002

```

```

Final value of cu: cu* = shear_factor * Max( cu, cc) = 0.01486841
The Shear_factor is considered equal to 1 (pure moment strength)
From (5.4b), TBDY: cu = 0.01486841

```

```

we (5.4c) = 0.08077545

```

```

ase ((5.4d), TBDY) = (ase1*Aext+ase2*Aint)/Asec = 0.53375773

```

```

ase1 = Max(((Aconf,max1-AnoConf1)/Aconf,max1)*(Aconf,min1/Aconf,max1),0) = 0.53375773

```

```

The definitions of AnoConf, Aconf,min and Aconf,max are derived from generalization
of the rectangular sections confinement, which is expressed by (5.4d).

```

```

The generalization is done according to Mander, J., Priestley, M., and Park, R. (1988)

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"Theoretical Stress-Strain Model for Confined Concrete."

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J. Struct. Eng., 10.1061/(ASCE)0733-9445(1988)114:8(1804), 1804-1826.

```

```

Aconf,max1 = 464100.00 is the confined core area at levels of member with hoops and

```

is calculated as the area of core enclosed by the center lines of the external perimeter hoops.

Aconf,min1 = 395025.00 is the confined core area at midway between the levels of hoops and

is calculated by reducing all the dimensions of the area Aconf,max1 by a length equal to half the clear spacing between external hoops.

AnoConf1 = 173066.667 is the unconfined external core area which is equal to $b_i^2/6$ as defined at (A.2).
ase2 (\geq ase1) = $\text{Max}(((A_{\text{conf,max2}} - A_{\text{noConf2}})/A_{\text{conf,max2}}) * (A_{\text{conf,min2}}/A_{\text{conf,max2}}), 0) = 0.53375773$

The definitions of AnoConf, Aconf,min and Aconf,max are derived from generalization of the rectangular sections confinement, which is expressed by (5.4d).

The generalization is done according to Mander, J., Priestley, M., and Park, R. (1988)

"Theoretical Stress-Strain Model for Confined Concrete."

J. Struct. Eng., 10.1061/(ASCE)0733-9445(1988)114:8(1804), 1804-1826.

Aconf,max2 = 252164.00 is the confined core area at levels of member with hoops and

is calculated as the area of core enclosed by the center lines of the internal perimeter hoops.

Aconf,min2 = 111441.00 is the confined core area at midway between the levels of hoops and

is calculated by reducing all the dimensions of the area Aconf,max2 by a length equal to half the clear spacing between internal hoops.

AnoConf2 = 110709.333 is the unconfined internal core area which is equal to $b_i^2/6$ as defined at (A.2).
 $psh_{\text{min}} * F_{ywe} = \text{Min}(psh_x * F_{ywe}, psh_y * F_{ywe}) = 2.724$

 $psh_x * F_{ywe} = psh1 * F_{ywe1} + ps2 * F_{ywe2} = 2.724$

$psh1$ ((5.4d), TBDY) = $L_{stir1} * A_{stir1} / (A_{sec} * s1) = 0.00301593$

L_{stir1} (Length of stirrups along Y) = 2160.00

A_{stir1} (stirrups area) = 78.53982

$psh2$ ((5.4d)) = $L_{stir2} * A_{stir2} / (A_{sec} * s2) = 0.00056047$

L_{stir2} (Length of stirrups along Y) = 1568.00

A_{stir2} (stirrups area) = 50.26548

 $psh_y * F_{ywe} = psh1 * F_{ywe1} + ps2 * F_{ywe2} = 3.25416$

$psh1$ ((5.4d), TBDY) = $L_{stir1} * A_{stir1} / (A_{sec} * s1) = 0.00357443$

L_{stir1} (Length of stirrups along X) = 2560.00

A_{stir1} (stirrups area) = 78.53982

$psh2$ ((5.4d), TBDY) = $L_{stir2} * A_{stir2} / (A_{sec} * s2) = 0.00070345$

L_{stir2} (Length of stirrups along X) = 1968.00

A_{stir2} (stirrups area) = 50.26548

 $A_{sec} = 562500.00$

$s1 = 100.00$

$s2 = 250.00$

$f_{ywe1} = 781.25$

$f_{ywe2} = 656.25$

$f_{ce} = 18.00$

From ((5.A5), TBDY), TBDY: $cc = 0.00621307$

c = confinement factor = 1.42131

$y1 = 0.0025$

$sh1 = 0.008$

$ft1 = 907.50$

$fy1 = 756.25$

$su1 = 0.032$

using (30) in Biskinis/Fardis (2013) multiplied with shear_factor
and also multiplied by the shear_factor according to 15.7.1.4, with
Shear_factor = 1.00

$l_o/l_{ou,min} = l_b/l_d = 1.00$

$su1 = 0.4 * esu1_{\text{nominal}}$ ((5.5), TBDY) = 0.032

From table 5A.1, TBDY: $esu1_{\text{nominal}} = 0.08$,

For calculation of $esu1_{\text{nominal}}$ and $y1, sh1, ft1, fy1$, it is considered
characteristic value $fsy1 = fs1/1.2$, from table 5.1, TBDY.

$y1, sh1, ft1, fy1$, are also multiplied by $\text{Min}(1, 1.25 * (l_b/l_d)^{2/3})$, from 10.3.5, ASCE 41-17.

with $fs1 = (f_{s,jacket} * A_{s1,ten,jacket} + f_{s,core} * A_{s1,ten,core}) / A_{s1,ten} = 756.25$

with $Es1 = (E_{s,jacket} * A_{s1,ten,jacket} + E_{s,core} * A_{s1,ten,core}) / A_{s1,ten} = 200000.00$

$y2 = 0.0025$

$sh2 = 0.008$

$ft2 = 907.50$

$fy2 = 756.25$

$su2 = 0.032$

using (30) in Biskinis/Fardis (2013) multiplied with shear_factor
and also multiplied by the shear_factor according to 15.7.1.4, with
Shear_factor = 1.00

```

lo/lou,min = lb/lbmin = 1.00
su2 = 0.4*esu2,nominal ((5.5), TBDY) = 0.032
From table 5A.1, TBDY: esu2,nominal = 0.08,
For calculation of esu2,nominal and y2, sh2,ft2,fy2, it is considered
characteristic value fsy2 = fs2/1.2, from table 5.1, TBDY.
y1, sh1,ft1,fy1, are also multiplied by Min(1,1.25*(lb/ld)2/3), from 10.3.5, ASCE 41-17.
with fs2 = (fsjacket*Aslcom,jacket + fscore*Aslcom,core)/Aslcom = 756.25
with Es2 = (Esjacket*Aslcom,jacket + Escore*Aslcom,core)/Aslcom = 200000.00
yv = 0.0025
shv = 0.008
ftv = 899.1522
fyv = 749.2935
suv = 0.032
using (30) in Biskinis/Fardis (2013) multiplied with shear_factor
and also multiplied by the shear_factor according to 15.7.1.4, with
Shear_factor = 1.00
lo/lou,min = lb/ld = 1.00
suv = 0.4*esuvnominal ((5.5), TBDY) = 0.032
From table 5A.1, TBDY: esuvnominal = 0.08,
considering characteristic value fsyv = fsv/1.2, from table 5.1, TBDY
For calculation of esuvnominal and yv, shv,ftv,fyv, it is considered
characteristic value fsyv = fsv/1.2, from table 5.1, TBDY.
y1, sh1,ft1,fy1, are also multiplied by Min(1,1.25*(lb/ld)2/3), from 10.3.5, ASCE 41-17.
with fsv = (fsjacket*Aslmid,jacket + fsmid*Aslmid,core)/Aslmid = 749.2935
with Esv = (Esjacket*Aslmid,jacket + Esmid*Aslmid,core)/Aslmid = 200000.00
1 = Aslten/(b*d)*(fs1/fc) = 0.15845977
2 = Aslcom/(b*d)*(fs2/fc) = 0.15845977
v = Aslmid/(b*d)*(fsv/fc) = 0.36847443
and confined core properties:
b = 390.00
d = 877.00
d' = 13.00
fcc (5A.2, TBDY) = 25.58352
cc (5A.5, TBDY) = 0.00621307
c = confinement factor = 1.42131
1 = Aslten/(b*d)*(fs1/fc) = 0.18909264
2 = Aslcom/(b*d)*(fs2/fc) = 0.18909264
v = Aslmid/(b*d)*(fsv/fc) = 0.43970658
Case/Assumption: Unconfined full section - Steel rupture
' satisfies Eq. (4.3)
---->
v < vs,y2 - LHS eq.(4.5) is not satisfied
---->
v < vs,c - RHS eq.(4.5) is not satisfied
---->
Case/Assumption Rejected.
---->
New Case/Assumption: Unconfined full section - Spalling of concrete cover
' satisfies Eq. (4.4)
---->
v < s,y1 - LHS eq.(4.7) is not satisfied
---->
v < vc,y1 - RHS eq.(4.6) is satisfied
---->
cu (4.10) = 0.83934343
MRc (4.17) = 1.8439E+009
---->
New Case/Assumption: Ultimate curvature of confined core after spalling of concrete cover
In expressions below, the following modifications have been made
- b, d, d' replaced by geometric parameters of the core: bo, do, d'o
- N1, N2, v normalised to bo*do, instead of b*d
- parameters of confined concrete, fcc, cc, used in lieu of fc, ecu
---->
Subcase: Rupture of tension steel
---->
v* < v*s,y2 - LHS eq.(4.5) is not satisfied
---->

```

$v^* < v^*_{s,c}$ - LHS eq.(4.5) is not satisfied

---->

Subcase rejected

---->

New Subcase: Failure of compression zone

---->

$v^* < v^*_{c,y2}$ - LHS eq.(4.6) is not satisfied

---->

$v^* < v^*_{c,y1}$ - RHS eq.(4.6) is not satisfied

---->

$*c_u$ (4.11) = 0.89983732

M_{Ro} (4.18) = 1.8431E+009

---->

$u = c_u$ (4.2) = 7.8730391E-006

$\mu = M_{Ro}$

Calculation of ratio l_b/l_d

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

Calculation of μ_{2+}

Calculation of ultimate curvature u according to 4.1, Biskinis/Fardis 2013:

$u = 7.8730391E-006$

$\mu = 1.8431E+009$

with full section properties:

$b = 450.00$

$d = 907.00$

$d' = 43.00$

$v = 1.0459$

$N = 7.6839E+006$

$f_c = 18.00$

α (5A.5, TBDY) = 0.002

Final value of c_u : $c_u^* = \text{shear_factor} * \text{Max}(c_u, c_c) = 0.01486841$

The Shear_factor is considered equal to 1 (pure moment strength)

From (5.4b), TBDY: $c_u = 0.01486841$

w_e (5.4c) = 0.08077545

a_{se} ((5.4d), TBDY) = $(a_{se1} * A_{ext} + a_{se2} * A_{int}) / A_{sec} = 0.53375773$

$a_{se1} = \text{Max}(((A_{conf,max1} - A_{noConf1}) / A_{conf,max1}) * (A_{conf,min1} / A_{conf,max1}), 0) = 0.53375773$

The definitions of A_{noConf} , $A_{conf,min}$ and $A_{conf,max}$ are derived from generalization of the rectangular sections confinement, which is expressed by (5.4d).

The generalization is done according to Mander, J., Priestley, M., and Park, R. (1988)

"Theoretical Stress-Strain Model for Confined Concrete."

J. Struct. Eng., 10.1061/(ASCE)0733-9445(1988)114:8(1804), 1804-1826.

$A_{conf,max1} = 464100.00$ is the confined core area at levels of member with hoops and is calculated as the area of core enclosed by the center lines of the external perimeter hoops.

$A_{conf,min1} = 395025.00$ is the confined core area at midway between the levels of hoops and is calculated by reducing all the dimensions of the area $A_{conf,max1}$ by a length equal to half the clear spacing between external hoops.

$A_{noConf1} = 173066.667$ is the unconfined external core area which is equal to $b_i^2/6$ as defined at (A.2).

$a_{se2} (\geq a_{se1}) = \text{Max}(((A_{conf,max2} - A_{noConf2}) / A_{conf,max2}) * (A_{conf,min2} / A_{conf,max2}), 0) = 0.53375773$

The definitions of A_{noConf} , $A_{conf,min}$ and $A_{conf,max}$ are derived from generalization of the rectangular sections confinement, which is expressed by (5.4d).

The generalization is done according to Mander, J., Priestley, M., and Park, R. (1988)

"Theoretical Stress-Strain Model for Confined Concrete."

J. Struct. Eng., 10.1061/(ASCE)0733-9445(1988)114:8(1804), 1804-1826.

$A_{conf,max2} = 252164.00$ is the confined core area at levels of member with hoops and is calculated as the area of core enclosed by the center lines of the internal perimeter hoops.

$A_{conf,min2} = 111441.00$ is the confined core area at midway between the levels of hoops and is calculated by reducing all the dimensions of the area $A_{conf,max2}$ by a length equal to half the clear spacing between internal hoops.

AnoConf2 = 110709.333 is the unconfined internal core area which is equal to $b_i^2/6$ as defined at (A.2).
 $psh_{min} \cdot F_{ywe} = \min(psh_x \cdot F_{ywe}, psh_y \cdot F_{ywe}) = 2.724$

$psh_x \cdot F_{ywe} = psh1 \cdot F_{ywe1} + ps2 \cdot F_{ywe2} = 2.724$
 $psh1 \text{ ((5.4d), TBDY)} = Lstir1 \cdot Astir1 / (Asec \cdot s1) = 0.00301593$
 $Lstir1 \text{ (Length of stirrups along Y)} = 2160.00$
 $Astir1 \text{ (stirrups area)} = 78.53982$
 $psh2 \text{ (5.4d)} = Lstir2 \cdot Astir2 / (Asec \cdot s2) = 0.00056047$
 $Lstir2 \text{ (Length of stirrups along Y)} = 1568.00$
 $Astir2 \text{ (stirrups area)} = 50.26548$

$psh_y \cdot F_{ywe} = psh1 \cdot F_{ywe1} + ps2 \cdot F_{ywe2} = 3.25416$
 $psh1 \text{ ((5.4d), TBDY)} = Lstir1 \cdot Astir1 / (Asec \cdot s1) = 0.00357443$
 $Lstir1 \text{ (Length of stirrups along X)} = 2560.00$
 $Astir1 \text{ (stirrups area)} = 78.53982$
 $psh2 \text{ ((5.4d), TBDY)} = Lstir2 \cdot Astir2 / (Asec \cdot s2) = 0.00070345$
 $Lstir2 \text{ (Length of stirrups along X)} = 1968.00$
 $Astir2 \text{ (stirrups area)} = 50.26548$

$Asec = 562500.00$
 $s1 = 100.00$
 $s2 = 250.00$
 $fy_{we1} = 781.25$
 $fy_{we2} = 656.25$
 $f_{ce} = 18.00$

From ((5.A5), TBDY), TBDY: $cc = 0.00621307$
 $c = \text{confinement factor} = 1.42131$

$y1 = 0.0025$
 $sh1 = 0.008$
 $ft1 = 907.50$
 $fy1 = 756.25$
 $su1 = 0.032$

using (30) in Biskinis/Fardis (2013) multiplied with shear_factor
and also multiplied by the shear_factor according to 15.7.1.4, with
 $\text{Shear_factor} = 1.00$

$lo/lou_{min} = lb/ld = 1.00$
 $su1 = 0.4 \cdot esu1_{nominal} \text{ ((5.5), TBDY)} = 0.032$

From table 5A.1, TBDY: $esu1_{nominal} = 0.08$,

For calculation of $esu1_{nominal}$ and $y1, sh1, ft1, fy1$, it is considered
characteristic value $fsy1 = fs1/1.2$, from table 5.1, TBDY.

$y1, sh1, ft1, fy1$, are also multiplied by $\min(1, 1.25 \cdot (lb/ld)^{2/3})$, from 10.3.5, ASCE 41-17.

with $fs1 = (fs_{jacket} \cdot A_{sl,ten,jacket} + fs_{core} \cdot A_{sl,ten,core}) / A_{sl,ten} = 756.25$

with $Es1 = (Es_{jacket} \cdot A_{sl,ten,jacket} + Es_{core} \cdot A_{sl,ten,core}) / A_{sl,ten} = 200000.00$

$y2 = 0.0025$
 $sh2 = 0.008$
 $ft2 = 907.50$
 $fy2 = 756.25$
 $su2 = 0.032$

using (30) in Biskinis/Fardis (2013) multiplied with shear_factor
and also multiplied by the shear_factor according to 15.7.1.4, with
 $\text{Shear_factor} = 1.00$

$lo/lou_{min} = lb/lb_{min} = 1.00$
 $su2 = 0.4 \cdot esu2_{nominal} \text{ ((5.5), TBDY)} = 0.032$

From table 5A.1, TBDY: $esu2_{nominal} = 0.08$,

For calculation of $esu2_{nominal}$ and $y2, sh2, ft2, fy2$, it is considered
characteristic value $fsy2 = fs2/1.2$, from table 5.1, TBDY.

$y1, sh1, ft1, fy1$, are also multiplied by $\min(1, 1.25 \cdot (lb/ld)^{2/3})$, from 10.3.5, ASCE 41-17.

with $fs2 = (fs_{jacket} \cdot A_{sl,com,jacket} + fs_{core} \cdot A_{sl,com,core}) / A_{sl,com} = 756.25$

with $Es2 = (Es_{jacket} \cdot A_{sl,com,jacket} + Es_{core} \cdot A_{sl,com,core}) / A_{sl,com} = 200000.00$

$yv = 0.0025$
 $shv = 0.008$
 $ftv = 899.1522$
 $fyv = 749.2935$
 $suv = 0.032$

using (30) in Biskinis/Fardis (2013) multiplied with shear_factor
and also multiplied by the shear_factor according to 15.7.1.4, with
 $\text{Shear_factor} = 1.00$

```

lo/lou,min = lb/ld = 1.00
suv = 0.4*esuv_nominal ((5.5), TBDY) = 0.032
From table 5A.1, TBDY: esuv_nominal = 0.08,
considering characteristic value fsyv = fsv/1.2, from table 5.1, TBDY
For calculation of esuv_nominal and yv, shv,ftv,fyv, it is considered
characteristic value fsyv = fsv/1.2, from table 5.1, TBDY.
y1, sh1,ft1,fy1, are also multiplied by Min(1,1.25*(lb/ld)^ 2/3), from 10.3.5, ASCE 41-17.
with fsv = (fsjacket*Aslmid,jacket + fsmid*Aslmid,core)/Aslmid = 749.2935
with Esv = (Esjacket*Aslmid,jacket + Esmid*Aslmid,core)/Aslmid = 200000.00
1 = Aslten/(b*d)*(fs1/fc) = 0.15845977
2 = Aslcom/(b*d)*(fs2/fc) = 0.15845977
v = Aslmid/(b*d)*(fsv/fc) = 0.36847443
and confined core properties:
b = 390.00
d = 877.00
d' = 13.00
fcc (5A.2, TBDY) = 25.58352
cc (5A.5, TBDY) = 0.00621307
c = confinement factor = 1.42131
1 = Aslten/(b*d)*(fs1/fc) = 0.18909264
2 = Aslcom/(b*d)*(fs2/fc) = 0.18909264
v = Aslmid/(b*d)*(fsv/fc) = 0.43970658
Case/Assumption: Unconfined full section - Steel rupture
' satisfies Eq. (4.3)
---->
v < vs,y2 - LHS eq.(4.5) is not satisfied
---->
v < vs,c - RHS eq.(4.5) is not satisfied
---->
Case/Assumption Rejected.
---->
New Case/Assumption: Unconfined full section - Spalling of concrete cover
' satisfies Eq. (4.4)
---->
v < sy1 - LHS eq.(4.7) is not satisfied
---->
v < vc,y1 - RHS eq.(4.6) is satisfied
---->
cu (4.10) = 0.83934343
MRc (4.17) = 1.8439E+009
---->
New Case/Assumption: Ultimate curvature of confined core after spalling of concrete cover
In expressions below, the following modifications have been made
- b, d, d' replaced by geometric parameters of the core: bo, do, d'o
- N, 1, 2, v normalised to bo*d'o, instead of b*d
- parameters of confined concrete, fcc, cc, used in lieu of fc, ecu
---->
Subcase: Rupture of tension steel
---->
v* < v*sy2 - LHS eq.(4.5) is not satisfied
---->
v* < v*sc - LHS eq.(4.5) is not satisfied
---->
Subcase rejected
---->
New Subcase: Failure of compression zone
---->
v* < v*cy2 - LHS eq.(4.6) is not satisfied
---->
v* < v*cy1 - RHS eq.(4.6) is not satisfied
---->
*cu (4.11) = 0.89983732
MRo (4.18) = 1.8431E+009
---->
u = cu (4.2) = 7.8730391E-006
Mu = MRo
-----

```

Calculation of ratio l_b/d

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

Calculation of μ_u -

Calculation of ultimate curvature μ_u according to 4.1, Biskinis/Fardis 2013:

$$\mu_u = 7.8730391E-006$$

$$\mu_u = 1.8431E+009$$

with full section properties:

$$b = 450.00$$

$$d = 907.00$$

$$d' = 43.00$$

$$v = 1.0459$$

$$N = 7.6839E+006$$

$$f_c = 18.00$$

$$\phi (5A.5, \text{TBDY}) = 0.002$$

$$\text{Final value of } \mu_u: \mu_u^* = \text{shear_factor} * \text{Max}(\mu_u, \phi) = 0.01486841$$

The Shear_factor is considered equal to 1 (pure moment strength)

$$\text{From (5.4b), TBDY: } \mu_u = 0.01486841$$

$$\mu_u (5.4c) = 0.08077545$$

$$\mu_{ase} ((5.4d), \text{TBDY}) = (\mu_{ase1} * A_{ext} + \mu_{ase2} * A_{int}) / A_{sec} = 0.53375773$$

$$\mu_{ase1} = \text{Max}(((A_{conf,max1} - A_{noConf1}) / A_{conf,max1}) * (A_{conf,min1} / A_{conf,max1}), 0) = 0.53375773$$

The definitions of A_{noConf} , $A_{conf,min}$ and $A_{conf,max}$ are derived from generalization of the rectangular sections confinement, which is expressed by (5.4d).

The generalization is done according to Mander, J., Priestley, M., and Park, R. (1988)

"Theoretical Stress-Strain Model for Confined Concrete."

J. Struct. Eng., 10.1061/(ASCE)0733-9445(1988)114:8(1804), 1804-1826.

$A_{conf,max1} = 464100.00$ is the confined core area at levels of member with hoops and

is calculated as the area of core enclosed by the center lines of the external perimeter hoops.

$A_{conf,min1} = 395025.00$ is the confined core area at midway between the levels of hoops and

is calculated by reducing all the dimensions of the area $A_{conf,max1}$ by a length equal to half the clear spacing between external hoops.

$A_{noConf1} = 173066.667$ is the unconfined external core area which is equal to $b_i^2/6$ as defined at (A.2).

$$\mu_{ase2} (\geq \mu_{ase1}) = \text{Max}(((A_{conf,max2} - A_{noConf2}) / A_{conf,max2}) * (A_{conf,min2} / A_{conf,max2}), 0) = 0.53375773$$

The definitions of A_{noConf} , $A_{conf,min}$ and $A_{conf,max}$ are derived from generalization of the rectangular sections confinement, which is expressed by (5.4d).

The generalization is done according to Mander, J., Priestley, M., and Park, R. (1988)

"Theoretical Stress-Strain Model for Confined Concrete."

J. Struct. Eng., 10.1061/(ASCE)0733-9445(1988)114:8(1804), 1804-1826.

$A_{conf,max2} = 252164.00$ is the confined core area at levels of member with hoops and

is calculated as the area of core enclosed by the center lines of the internal perimeter hoops.

$A_{conf,min2} = 111441.00$ is the confined core area at midway between the levels of hoops and

is calculated by reducing all the dimensions of the area $A_{conf,max2}$ by a length equal to half the clear spacing between internal hoops.

$A_{noConf2} = 110709.333$ is the unconfined internal core area which is equal to $b_i^2/6$ as defined at (A.2).

$$\text{psh}_{min} * F_{ywe} = \text{Min}(\text{psh}_x * F_{ywe}, \text{psh}_y * F_{ywe}) = 2.724$$

$$\text{psh}_x * F_{ywe} = \text{psh}_1 * F_{ywe1} + \text{ps}_2 * F_{ywe2} = 2.724$$

$$\text{psh}_1 ((5.4d), \text{TBDY}) = L_{stir1} * A_{stir1} / (A_{sec} * s_1) = 0.00301593$$

$$L_{stir1} (\text{Length of stirrups along Y}) = 2160.00$$

$$A_{stir1} (\text{stirrups area}) = 78.53982$$

$$\text{psh}_2 (5.4d) = L_{stir2} * A_{stir2} / (A_{sec} * s_2) = 0.00056047$$

$$L_{stir2} (\text{Length of stirrups along Y}) = 1568.00$$

$$A_{stir2} (\text{stirrups area}) = 50.26548$$

$$\text{psh}_y * F_{ywe} = \text{psh}_1 * F_{ywe1} + \text{ps}_2 * F_{ywe2} = 3.25416$$

$$\text{psh}_1 ((5.4d), \text{TBDY}) = L_{stir1} * A_{stir1} / (A_{sec} * s_1) = 0.00357443$$

$$L_{stir1} (\text{Length of stirrups along X}) = 2560.00$$

$$A_{stir1} (\text{stirrups area}) = 78.53982$$

$$\text{psh}_2 ((5.4d), \text{TBDY}) = L_{stir2} * A_{stir2} / (A_{sec} * s_2) = 0.00070345$$

Lstir2 (Length of stirrups along X) = 1968.00
Astir2 (stirrups area) = 50.26548

Asec = 562500.00
s1 = 100.00
s2 = 250.00
fywe1 = 781.25
fywe2 = 656.25
fce = 18.00

From ((5A5), TBDY), TBDY: cc = 0.00621307
c = confinement factor = 1.42131

y1 = 0.0025
sh1 = 0.008
ft1 = 907.50
fy1 = 756.25
su1 = 0.032

using (30) in Biskinis/Fardis (2013) multiplied with shear_factor
and also multiplied by the shear_factor according to 15.7.1.4, with
Shear_factor = 1.00

lo/lou,min = lb/ld = 1.00

su1 = 0.4*esu1_nominal ((5.5), TBDY) = 0.032

From table 5A.1, TBDY: esu1_nominal = 0.08,

For calculation of esu1_nominal and y1, sh1,ft1,fy1, it is considered
characteristic value fsy1 = fs1/1.2, from table 5.1, TBDY.

y1, sh1,ft1,fy1, are also multiplied by $\text{Min}(1, 1.25 \cdot (lb/ld)^{2/3})$, from 10.3.5, ASCE 41-17.

with fs1 = (fs,jacket*Asl,ten,jacket + fs,core*Asl,ten,core)/Asl,ten = 756.25

with Es1 = (Es,jacket*Asl,ten,jacket + Es,core*Asl,ten,core)/Asl,ten = 200000.00

y2 = 0.0025
sh2 = 0.008
ft2 = 907.50
fy2 = 756.25
su2 = 0.032

using (30) in Biskinis/Fardis (2013) multiplied with shear_factor
and also multiplied by the shear_factor according to 15.7.1.4, with
Shear_factor = 1.00

lo/lou,min = lb/lb,min = 1.00

su2 = 0.4*esu2_nominal ((5.5), TBDY) = 0.032

From table 5A.1, TBDY: esu2_nominal = 0.08,

For calculation of esu2_nominal and y2, sh2,ft2,fy2, it is considered
characteristic value fsy2 = fs2/1.2, from table 5.1, TBDY.

y1, sh1,ft1,fy1, are also multiplied by $\text{Min}(1, 1.25 \cdot (lb/ld)^{2/3})$, from 10.3.5, ASCE 41-17.

with fs2 = (fs,jacket*Asl,com,jacket + fs,core*Asl,com,core)/Asl,com = 756.25

with Es2 = (Es,jacket*Asl,com,jacket + Es,core*Asl,com,core)/Asl,com = 200000.00

yv = 0.0025
shv = 0.008
ftv = 899.1522
fyv = 749.2935
suv = 0.032

using (30) in Biskinis/Fardis (2013) multiplied with shear_factor
and also multiplied by the shear_factor according to 15.7.1.4, with
Shear_factor = 1.00

lo/lou,min = lb/ld = 1.00

suv = 0.4*esuv_nominal ((5.5), TBDY) = 0.032

From table 5A.1, TBDY: esuv_nominal = 0.08,

considering characteristic value fsyv = fsv/1.2, from table 5.1, TBDY

For calculation of esuv_nominal and yv, shv,ftv,fyv, it is considered
characteristic value fsyv = fsv/1.2, from table 5.1, TBDY.

y1, sh1,ft1,fy1, are also multiplied by $\text{Min}(1, 1.25 \cdot (lb/ld)^{2/3})$, from 10.3.5, ASCE 41-17.

with fsv = (fs,jacket*Asl,mid,jacket + fs,mid*Asl,mid,core)/Asl,mid = 749.2935

with Esv = (Es,jacket*Asl,mid,jacket + Es,mid*Asl,mid,core)/Asl,mid = 200000.00

1 = Asl,ten/(b*d)*(fs1/fc) = 0.15845977

2 = Asl,com/(b*d)*(fs2/fc) = 0.15845977

v = Asl,mid/(b*d)*(fsv/fc) = 0.36847443

and confined core properties:

b = 390.00
d = 877.00
d' = 13.00


```

fcc (5A.2, TBDY) = 25.58352
cc (5A.5, TBDY) = 0.00621307
c = confinement factor = 1.42131
1 = Asl,ten/(b*d)*(fs1/fc) = 0.18909264
2 = Asl,com/(b*d)*(fs2/fc) = 0.18909264
v = Asl,mid/(b*d)*(fsv/fc) = 0.43970658
Case/Assumption: Unconfined full section - Steel rupture
' satisfies Eq. (4.3)
---->
v < vs,y2 - LHS eq.(4.5) is not satisfied
---->
v < vs,c - RHS eq.(4.5) is not satisfied
---->
Case/Assumption Rejected.
---->
New Case/Assumption: Unconfined full section - Spalling of concrete cover
' satisfies Eq. (4.4)
---->
v < s,y1 - LHS eq.(4.7) is not satisfied
---->
v < vc,y1 - RHS eq.(4.6) is satisfied
---->
cu (4.10) = 0.83934343
MRc (4.17) = 1.8439E+009
---->
New Case/Assumption: Ultimate curvature of confined core after spalling of concrete cover
In expressions below, the following modifications have been made
- b, d, d' replaced by geometric parameters of the core: bo, do, d'o
- N, 1, 2, v normalised to bo*do, instead of b*d
- - parameters of confined concrete, fcc, cc, used in lieu of fc, ecu
---->
Subcase: Rupture of tension steel
---->
v* < v*s,y2 - LHS eq.(4.5) is not satisfied
---->
v* < v*s,c - LHS eq.(4.5) is not satisfied
---->
Subcase rejected
---->
New Subcase: Failure of compression zone
---->
v* < v*c,y2 - LHS eq.(4.6) is not satisfied
---->
v* < v*c,y1 - RHS eq.(4.6) is not satisfied
---->
*cu (4.11) = 0.89983732
MRo (4.18) = 1.8431E+009
---->
u = cu (4.2) = 7.8730391E-006
Mu = MRo
-----

Calculation of ratio lb/d
-----
Adequate Lap Length: lb/d >= 1
-----
-----
-----
-----
Calculation of Shear Strength Vr = Min(Vr1,Vr2) = 1.4186E+006
-----
Calculation of Shear Strength at edge 1, Vr1 = 1.4186E+006
Vr1 = VCol ((10.3), ASCE 41-17) = knl*VColO
VColO = 1.4186E+006
knl = 1 (zero step-static loading)
-----
NOTE: In expression (10-3) 'Vs = Av*fy*d/s' is replaced by 'Vs+ f*VF'

```

where V_f is the contribution of FRPs (11.3), ACI 440).

= 1 (normal-weight concrete)

Mean concrete strength: $f'_c = (f'_{c,jacket} \cdot Area_{jacket} + f'_{c,core} \cdot Area_{core}) / Area_{section} = 15.20$, but $f'_c^{0.5} \leq 8.3$ MPa (22.5.3.1, ACI 318-14)

$M/Vd = 4.00$

$\mu_u = 2.6919E+008$

$V_u = 137.4894$

$d = 0.8 \cdot h = 760.00$

$N_u = 7.6839E+006$

$A_g = 427500.00$

From (11.5.4.8), ACI 318-14: $V_s = V_{s,jacket} + V_{s,core} = 1.2262E+006$

where:

$V_{s,jacket} = V_{s,j1} + V_{s,j2} = 1.0996E+006$

$V_{s,j1} = 353429.174$ is calculated for section web jacket, with:

$d = 360.00$

$A_v = 157079.633$

$f_y = 625.00$

$s = 100.00$

$V_{s,j1}$ is multiplied by $Col,j1 = 1.00$

$s/d = 0.27777778$

$V_{s,j2} = 746128.255$ is calculated for section flange jacket, with:

$d = 760.00$

$A_v = 157079.633$

$f_y = 625.00$

$s = 100.00$

$V_{s,j2}$ is multiplied by $Col,j2 = 1.00$

$s/d = 0.13157895$

$V_{s,core} = V_{s,c1} + V_{s,c2} = 126669.016$

$V_{s,c1} = 0.00$ is calculated for section web core, with:

$d = 200.00$

$A_v = 100530.965$

$f_y = 525.00$

$s = 250.00$

$V_{s,c1}$ is multiplied by $Col,c1 = 0.00$

$s/d = 1.25$

$V_{s,c2} = 126669.016$ is calculated for section flange core, with:

$d = 600.00$

$A_v = 100530.965$

$f_y = 525.00$

$s = 250.00$

$V_{s,c2}$ is multiplied by $Col,c2 = 1.00$

$s/d = 0.41666667$

V_f ((11-3)-(11.4), ACI 440) = 0.00

From (11-11), ACI 440: $V_s + V_f \leq 885757.128$

$bw = 450.00$

Calculation of Shear Strength at edge 2, $V_{r2} = 1.4186E+006$

$V_{r2} = V_{Col} \text{ ((10.3), ASCE 41-17)} = knl \cdot V_{Col0}$

$V_{Col0} = 1.4186E+006$

$knl = 1$ (zero step-static loading)

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

= 1 (normal-weight concrete)

Mean concrete strength: $f'_c = (f'_{c,jacket} \cdot Area_{jacket} + f'_{c,core} \cdot Area_{core}) / Area_{section} = 15.20$, but $f'_c^{0.5} \leq 8.3$ MPa (22.5.3.1, ACI 318-14)

$M/Vd = 4.00$

$\mu_u = 2.5135E+008$

$V_u = 137.4894$

$d = 0.8 \cdot h = 760.00$

$N_u = 7.6839E+006$

$A_g = 427500.00$

From (11.5.4.8), ACI 318-14: $V_s = V_{s,jacket} + V_{s,core} = 1.2262E+006$

where:

$$V_{sjacket} = V_{sj1} + V_{sj2} = 1.0996E+006$$

$V_{sj1} = 353429.174$ is calculated for section web jacket, with:

$$d = 360.00$$

$$A_v = 157079.633$$

$$f_y = 625.00$$

$$s = 100.00$$

V_{sj1} is multiplied by $Col,j1 = 1.00$

$$s/d = 0.27777778$$

$V_{sj2} = 746128.255$ is calculated for section flange jacket, with:

$$d = 760.00$$

$$A_v = 157079.633$$

$$f_y = 625.00$$

$$s = 100.00$$

V_{sj2} is multiplied by $Col,j2 = 1.00$

$$s/d = 0.13157895$$

$$V_{s,core} = V_{s,c1} + V_{s,c2} = 126669.016$$

$V_{s,c1} = 0.00$ is calculated for section web core, with:

$$d = 200.00$$

$$A_v = 100530.965$$

$$f_y = 525.00$$

$$s = 250.00$$

$V_{s,c1}$ is multiplied by $Col,c1 = 0.00$

$$s/d = 1.25$$

$V_{s,c2} = 126669.016$ is calculated for section flange core, with:

$$d = 600.00$$

$$A_v = 100530.965$$

$$f_y = 525.00$$

$$s = 250.00$$

$V_{s,c2}$ is multiplied by $Col,c2 = 1.00$

$$s/d = 0.41666667$$

$$V_f ((11-3)-(11.4), ACI 440) = 0.00$$

$$\text{From } (11-11), ACI 440: V_s + V_f \leq 885757.128$$

$$bw = 450.00$$

End Of Calculation of Shear Capacity ratio for element: column JTC1 of floor 1
At local axis: 2

Start Of Calculation of Chord Rotation Capacity for element: column JTC1 of floor 1
At local axis: 3
Integration Section: (a)
Section Type: rcjtcs

Constant Properties

Knowledge Factor, $\phi = 0.80$

Chord Rotation is generally considered as Deformation-Controlled Action according to Table C7-1, ASCE 41-17.

Mean strengths are used for Deformation-Controlled Actions according to 7.5.1.3, ASCE 41-17

Consequently:

Jacket

New material of Primary Member: Concrete Strength, $f_c = f_{cm} = 18.00$

New material of Primary Member: Steel Strength, $f_s = f_{sm} = 625.00$

Concrete Elasticity, $E_c = 19940.411$

Steel Elasticity, $E_s = 200000.00$

Existing Column

Existing material of Primary Member: Concrete Strength, $f_c = f_{cm} = 12.00$

Existing material of Primary Member: Steel Strength, $f_s = f_{sm} = 525.00$

Concrete Elasticity, $E_c = 16281.278$

Steel Elasticity, $E_s = 200000.00$

Max Height, $H_{max} = 750.00$

Min Height, $H_{min} = 450.00$

Max Width, $W_{max} = 950.00$

Min Width, $W_{min} = 450.00$

Eccentricity, $E_{cc} = 250.00$

Jacket Thickness, $t_j = 100.00$
 Cover Thickness, $c = 25.00$
 Element Length, $L = 3000.00$
 Primary Member
 Ribbed Bars
 Ductile Steel
 With Detailing for Earthquake Resistance (including stirrups closed at 135°)
 Longitudinal Bars With Ends Lapped Starting at the End Sections
 Adequate Lap Length ($l_b/l_d \geq 1$)
 No FRP Wrapping

Stepwise Properties

Bending Moment, $M = -1.2771E+009$
 Shear Force, $V_2 = -455948.704$
 Shear Force, $V_3 = 29.64437$
 Axial Force, $F = -7.6755E+006$
 Longitudinal Reinforcement Area Distribution (in 2 divisions)
 -Tension: $A_{st} = 2522.699$
 -Compression: $A_{sc} = 4168.893$
 Longitudinal Reinforcement Area Distribution (in 3 divisions)
 -Tension: $A_{st,ten} = 1539.38$
 -Compression: $A_{st,com} = 1539.38$
 -Middle: $A_{st,mid} = 3612.832$
 Longitudinal External Reinforcement Area Distribution (in 3 divisions)
 -Tension: $A_{st,ten,jacket} = 1231.504$
 -Compression: $A_{st,com,jacket} = 1231.504$
 -Middle: $A_{st,mid,jacket} = 2689.203$
 Longitudinal Internal Reinforcement Area Distribution (in 3 divisions)
 -Tension: $A_{st,ten,core} = 307.8761$
 -Compression: $A_{st,com,core} = 307.8761$
 -Middle: $A_{st,mid,core} = 923.6282$
 Mean Diameter of Tension Reinforcement, $Db_L = 16.57143$

Existing component: From table 7-7, ASCE 41_17: Final chord rotation Capacity $u_R = u = 0.00232693$
 $u = y + p = 0.00290867$

- Calculation of y -

$y = (M_y * L_s / 3) / E_{eff} = 0.00290867$ ((4.29), Biskinis Phd))
 $M_y = 1.4240E+009$
 $L_s = M/V$ (with $L_s > 0.1 * L$ and $L_s < 2 * L$) = 2800.911
 From table 10.5, ASCE 41_17: $E_{eff} = factor * E_c * I_g = 4.5707E+014$
 $factor = 0.70$
 $A_g = 562500.00$
 Mean concrete strength: $fc' = (fc'_{jacket} * Area_{jacket} + fc'_{core} * Area_{core}) / Area_{section} = 15.20$
 $N = 7.6755E+006$
 $E_c * I_g = E_{c,jacket} * I_{g,jacket} + E_{c,core} * I_{g,core} = 6.5295E+014$

Calculation of Yielding Moment M_y

Calculation of y and M_y according to Annex 7 -

$y = \text{Min}(y_{ten}, y_{com})$
 $y_{ten} = 7.1477677E-006$
 with $f_y = 601.9953$
 $d = 907.00$
 $y = 0.53571421$
 $A = 0.04763352$
 $B = 0.03982468$

with $p_t = 0.0037716$
 $p_c = 0.0037716$
 $p_v = 0.00885172$
 $N = 7.6755E+006$
 $b = 450.00$
 $" = 0.04740904$
 $y_{comp} = 1.7850636E-006$
 with $f_c = 18.00$
 $E_c = 19940.411$
 $y = 1.00358$
 $A = -0.04147374$
 $B = 0.0085861$
 with $E_s = 200000.00$

Calculation of ratio I_b/I_d

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

- Calculation of p -

From table 10-8: $p = 0.00$

with:

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

shear control ratio $V_y E / V_{col} E = 0.86614729$

$d = d_{external} = 907.00$

$s = s_{external} = 50.00$

- $t = s_1 + s_2 + 2 \cdot t_f / b_w \cdot (f_{fe} / f_s) = 0.00427788$

jacket: $s_1 = A_{v1} \cdot L_{stir1} / (s_1 \cdot A_g) = 0.00357443$

$A_{v1} = 78.53982$, is the area of every stirrup parallel to loading (shear) direction

$L_{stir1} = 2560.00$, is the total Length of all stirrups parallel to loading (shear) direction

$s_1 = 100.00$

core: $s_2 = A_{v2} \cdot L_{stir2} / (s_2 \cdot A_g) = 0.00070345$

$A_{v2} = 50.26548$, is the area of every stirrup parallel to loading (shear) direction

$L_{stir2} = 1968.00$, is the total Length of all stirrups parallel to loading (shear) direction

$s_2 = 250.00$

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

where $f = 2 \cdot t_f / b_w$ is FRP ratio (EC8 - 3, A.4.4.3(6)) and f_{fe} / f_s normalises f to steel strength

All these variables have already been given in Shear control ratio calculation.

For the normalisation f_s of jacket is used.

$N_{UD} = 7.6755E+006$

$A_g = 562500.00$

$f_{cE} = (f_{c,jacket} \cdot Area_{jacket} + f_{c,core} \cdot Area_{core}) / section_area = 15.20$

$f_{yIE} = (f_{y,ext_Long_Reinf} \cdot Area_{ext_Long_Reinf} + f_{y,int_Long_Reinf} \cdot Area_{int_Long_Reinf}) / Area_{Tot_Long_Rein} = 601.9953$

$f_{yIE} = (f_{y,ext_Trans_Reinf} \cdot s_1 + f_{y,int_Trans_Reinf} \cdot s_2) / (s_1 + s_2) = 608.5561$

$p_l = Area_{Tot_Long_Rein} / (b \cdot d) = 0.01639493$

$b = 450.00$

$d = 907.00$

$f_{cE} = 15.20$

End Of Calculation of Chord Rotation Capacity for element: column JTC1 of floor 1

At local axis: 3

Integration Section: (a)

Calculation No. 5

column C1, Floor 1

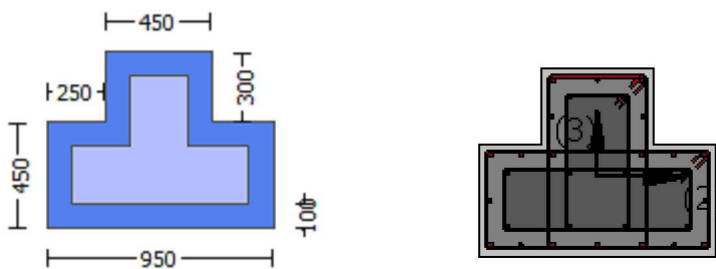
Limit State: Operational Level (data interpolation between analysis steps 39 and 40)

Analysis: Uniform +X

Check: Shear capacity VR_d

Edge: End

Local Axis: (2)



Start Of Calculation of Shear Capacity for element: column JTC1 of floor 1

At local axis: 2

Integration Section: (b)

Section Type: rcjtcs

Constant Properties

Knowledge Factor, $\gamma = 0.80$

Member Shear Force is generally considered as Force-Controlled Action according to Table C7-1, ASCE 41-17.

Lower-bound strengths are used for Force-Controlled Actions according to 7.5.1.3, ASCE 41-17

Consequently:

Jacket

New material of Primary Member: Concrete Strength, $f_c = f_{c_lower_bound} = 12.00$

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

Concrete Elasticity, $E_c = 19940.411$

Steel Elasticity, $E_s = 200000.00$

Existing Column

Existing material of Primary Member: Concrete Strength, $f_c = f_{c_lower_bound} = 8.00$

Existing material of Primary Member: Steel Strength, $f_s = f_{s_lower_bound} = 420.00$

Concrete Elasticity, $E_c = 16281.278$

Steel Elasticity, $E_s = 200000.00$

#####

Note: Especially for the calculation of γ for displacement ductility demand, the expected (mean value) strengths are used (7.5.1.3, ASCE 41-17) because bending is considered as Deformation-Controlled Action (Table C7-1, ASCE 41-17).

Jacket

New material: Concrete Strength, $f_c = f_{cm} = 18.00$

New material: Steel Strength, $f_s = f_{sm} = 625.00$

Existing Column

Existing material: Concrete Strength, $f_c = f_{cm} = 12.00$

Existing material: Steel Strength, $f_s = f_{sm} = 525.00$

#####

Max Height, Hmax = 750.00
 Min Height, Hmin = 450.00
 Max Width, Wmax = 950.00
 Min Width, Wmin = 450.00
 Eccentricity, Ecc = 250.00
 Jacket Thickness, tj = 100.00
 Cover Thickness, c = 25.00
 Element Length, L = 3000.00
 Primary Member
 Ribbed Bars
 Ductile Steel
 With Detailing for Earthquake Resistance (including stirrups closed at 135°)
 Longitudinal Bars With Ends Lapped Starting at the End Sections
 Adequate Lap Length ($l_o/l_{o,u,min} = l_b/l_d \geq 1$)
 No FRP Wrapping

Stepwise Properties

EDGE -A-
 Bending Moment, Ma = -1.2771E+009
 Shear Force, Va = -455948.704
 EDGE -B-
 Bending Moment, Mb = -4.0305E+008
 Shear Force, Vb = 455948.704
 BOTH EDGES
 Axial Force, F = -7.6755E+006
 Longitudinal Reinforcement Area Distribution (in 2 divisions)
 -Tension: Aslt = 314.1593
 -Compression: Aslc = 6377.433
 Longitudinal Reinforcement Area Distribution (in 3 divisions)
 -Tension: Asl,ten = 1539.38
 -Compression: Asl,com = 1539.38
 -Middle: Asl,mid = 3612.832
 Mean Diameter of Tension Reinforcement, DbL,ten = 16.57143

Existing component: From table 7-7, ASCE 41_17: Final Shear Capacity VR = $V_n = 1.3416E+006$
 $V_n ((10.3), ASCE 41-17) = knl * V_{ColO} = 1.6770E+006$
 $V_{Col} = 1.6770E+006$
 $knl = 1.00$
 $displacement_ductility_demand = 0.1412762$

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

$= 1$ (normal-weight concrete)
 Mean concrete strength: $f_c' = (f_c'_{jacket} * Area_{jacket} + f_c'_{core} * Area_{core}) / Area_{section} = 10.13333$, but $f_c'^{0.5} \leq 8.3$ MPa (22.5.3.1, ACI 318-14)
 $M/Vd = 2.00$
 $M_u = 4.0305E+008$
 $V_u = 455948.704$
 $d = 0.8 * h = 760.00$
 $N_u = 7.6755E+006$
 $A_g = 427500.00$
 From (11.5.4.8), ACI 318-14: $V_s = V_{s,jacket} + V_{s,core} = 980981.156$
 where:
 $V_{s,jacket} = V_{s,j1} + V_{s,j2} = 879645.943$
 $V_{s,j1} = 282743.339$ is calculated for section web jacket, with:
 $d = 360.00$
 $A_v = 157079.633$
 $f_y = 500.00$
 $s = 100.00$
 $V_{s,j1}$ is multiplied by $Col,j1 = 1.00$
 $s/d = 0.27777778$

$V_{s,j2} = 596902.604$ is calculated for section flange jacket, with:

$$d = 760.00$$

$$A_v = 157079.633$$

$$f_y = 500.00$$

$$s = 100.00$$

$V_{s,j2}$ is multiplied by $Col,j2 = 1.00$

$$s/d = 0.13157895$$

$$V_{s,core} = V_{s,c1} + V_{s,c2} = 101335.213$$

$V_{s,c1} = 0.00$ is calculated for section web core, with:

$$d = 200.00$$

$$A_v = 100530.965$$

$$f_y = 420.00$$

$$s = 250.00$$

$V_{s,c1}$ is multiplied by $Col,c1 = 0.00$

$$s/d = 1.25$$

$V_{s,c2} = 101335.213$ is calculated for section flange core, with:

$$d = 600.00$$

$$A_v = 100530.965$$

$$f_y = 420.00$$

$$s = 250.00$$

$V_{s,c2}$ is multiplied by $Col,c2 = 1.00$

$$s/d = 0.41666667$$

$$V_f ((11-3)-(11.4), ACI 440) = 0.00$$

$$\text{From } (11-11), ACI 440: V_s + V_f \leq 723217.666$$

$$bw = 450.00$$

displacement ductility demand is calculated as δ / y

- Calculation of δ / y for END B -

for rotation axis 3 and integ. section (b)

From analysis, chord rotation $\theta = 0.00012969$

$$y = (M_y * L_s / 3) / E_{eff} = 0.00091799 \text{ ((4.29), Biskinis Phd)}$$

$$M_y = 1.4240E+009$$

$$L_s = M/V \text{ (with } L_s > 0.1 * L \text{ and } L_s < 2 * L) = 883.982$$

$$\text{From table 10.5, ASCE 41_17: } E_{eff} = \text{factor} * E_c * I_g = 4.5707E+014$$

$$\text{factor} = 0.70$$

$$A_g = 562500.00$$

$$\text{Mean concrete strength: } f'_c = (f'_{c,jacket} * Area_{jacket} + f'_{c,core} * Area_{core}) / Area_{section} = 15.20$$

$$N = 7.6755E+006$$

$$E_c * I_g = E_{c,jacket} * I_{g,jacket} + E_{c,core} * I_{g,core} = 6.5295E+014$$

Calculation of Yielding Moment M_y

Calculation of δ / y and M_y according to Annex 7 -

$$y = \text{Min}(y_{ten}, y_{com})$$

$$y_{ten} = 7.1477677E-006$$

$$\text{with } f_y = 601.9953$$

$$d = 907.00$$

$$y = 0.53571421$$

$$A = 0.04763352$$

$$B = 0.03982468$$

$$\text{with } p_t = 0.0037716$$

$$p_c = 0.0037716$$

$$p_v = 0.00885172$$

$$N = 7.6755E+006$$

$$b = 450.00$$

$$\epsilon = 0.04740904$$

$$y_{comp} = 1.7850636E-006$$

$$\text{with } f_c = 18.00$$

$$E_c = 19940.411$$

$$y = 1.00358$$

A = -0.04147374
B = 0.0085861
with Es = 200000.00

Calculation of ratio I_b/I_d

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

End Of Calculation of Shear Capacity for element: column JTC1 of floor 1

At local axis: 2

Integration Section: (b)

Calculation No. 6

column C1, Floor 1

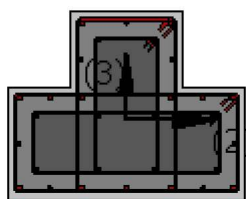
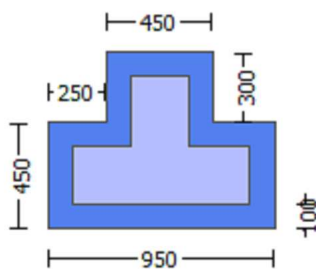
Limit State: Operational Level (data interpolation between analysis steps 39 and 40)

Analysis: Uniform +X

Check: Chord rotation capacity (ϕ)

Edge: End

Local Axis: (2)



Start Of Calculation of Shear Capacity ratio for element: column JTC1 of floor 1

At Shear local axis: 3

(Bending local axis: 2)

Section Type: rcjtcs

Constant Properties

Knowledge Factor, $\gamma = 0.80$

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

Consequently:

Jacket

New material of Primary Member: Concrete Strength, $f_c = f_{cm} = 18.00$

New material of Primary Member: Steel Strength, $f_s = f_{sm} = 625.00$

Concrete Elasticity, $E_c = 19940.411$

Steel Elasticity, $E_s = 200000.00$

```

Existing Column
Existing material of Primary Member: Concrete Strength,  $f_c = f_{cm} = 12.00$ 
Existing material of Primary Member: Steel Strength,  $f_s = f_{sm} = 525.00$ 
Concrete Elasticity,  $E_c = 16281.278$ 
Steel Elasticity,  $E_s = 200000.00$ 
#####
Note: Especially for the calculation of moment strengths,
the above steel re-bar strengths are multiplied by 1.25 according to R18.6.5, ACI 318-14
Jacket
New material: Steel Strength,  $f_s = 1.25 \cdot f_{sm} = 781.25$ 
Existing Column
Existing material: Steel Strength,  $f_s = 1.25 \cdot f_{sm} = 656.25$ 
#####
Max Height,  $H_{max} = 750.00$ 
Min Height,  $H_{min} = 450.00$ 
Max Width,  $W_{max} = 950.00$ 
Min Width,  $W_{min} = 450.00$ 
Eccentricity,  $E_{cc} = 250.00$ 
Jacket Thickness,  $t_j = 100.00$ 
Cover Thickness,  $c = 25.00$ 
Mean Confinement Factor overall section = 1.42131
Element Length,  $L = 3000.00$ 
Primary Member
Ribbed Bars
Ductile Steel
With Detailing for Earthquake Resistance (including stirrups closed at 135°)
Longitudinal Bars With Ends Lapped Starting at the End Sections
Adequate Lap Length ( $l_o/l_{ou}, \min > 1$ )
No FRP Wrapping
-----

Stepwise Properties
-----
At local axis: 3
EDGE -A-
Shear Force,  $V_a = -227.4778$ 
EDGE -B-
Shear Force,  $V_b = 227.4778$ 
BOTH EDGES
Axial Force,  $F = -7.6839E+006$ 
Longitudinal Reinforcement Area Distribution (in 2 divisions)
  -Tension:  $A_{sl,t} = 314.1593$ 
  -Compression:  $A_{sl,c} = 6176.371$ 
Longitudinal Reinforcement Area Distribution (in 3 divisions)
  -Tension:  $A_{sl,ten} = 1539.38$ 
  -Compression:  $A_{sl,com} = 2475.575$ 
  -Middle:  $A_{sl,mid} = 2676.637$ 
-----
-----

Calculation of Shear Capacity ratio ,  $V_e/V_r = 1.74773$ 
Member Controlled by Shear ( $V_e/V_r > 1$ )
Calculation of Shear Demand from fig. R18.6.5, ACI 318-14  $V_e = (M_{pr1} + M_{pr2})/l_n = 2.0410E+006$ 
with
 $M_{pr1} = \max(\mu_{u1+}, \mu_{u1-}) = 3.0615E+009$ 
 $\mu_{u1+} = 3.0615E+009$ , is the ultimate moment strength at the edge 1 of the member in the actual moment direction
which is defined for the static loading combination
 $\mu_{u1-} = 8.9683E+008$ , is the ultimate moment strength at the edge 1 of the member in the opposite moment
direction which is defined for the static loading combination
 $M_{pr2} = \max(\mu_{u2+}, \mu_{u2-}) = 3.0615E+009$ 
 $\mu_{u2+} = 3.0615E+009$ , is the ultimate moment strength at the edge 2 of the member in the actual moment direction
which is defined for the the static loading combination
 $\mu_{u2-} = 8.9683E+008$ , 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 curvature ϕ_u according to 4.1, Biskinis/Fardis 2013:

$$u = 1.3930706E-005$$

$$\mu = 3.0615E+009$$

with full section properties:

$$b = 950.00$$

$$d = 707.00$$

$$d' = 43.00$$

$$v = 0.63557247$$

$$N = 7.6839E+006$$

$$f_c = 18.00$$

$$\phi_0 (5A.5, TBDY) = 0.002$$

$$\text{Final value of } \phi_u: \phi_u^* = \text{shear_factor} * \text{Max}(\phi_u, \phi_0) = 0.01486841$$

The Shear_factor is considered equal to 1 (pure moment strength)

$$\text{From (5.4b), TBDY: } \phi_u = 0.01486841$$

$$\phi_{ue} (5.4c) = 0.08077545$$

$$\phi_{ase} ((5.4d), TBDY) = (\phi_{ase1} * A_{ext} + \phi_{ase2} * A_{int}) / A_{sec} = 0.53375773$$

$$\phi_{ase1} = \text{Max}(((A_{conf,max1} - A_{noConf1}) / A_{conf,max1}) * (A_{conf,min1} / A_{conf,max1}), 0) = 0.53375773$$

The definitions of A_{noConf} , $A_{conf,min}$ and $A_{conf,max}$ are derived from generalization of the rectangular sections confinement, which is expressed by (5.4d).

The generalization is done according to Mander, J., Priestley, M., and Park, R. (1988)

"Theoretical Stress-Strain Model for Confined Concrete."

J. Struct. Eng., 10.1061/(ASCE)0733-9445(1988)114:8(1804), 1804-1826.

$A_{conf,max1} = 464100.00$ is the confined core area at levels of member with hoops and is calculated as the area of core enclosed by the center lines of the external perimeter hoops.

$A_{conf,min1} = 395025.00$ is the confined core area at midway between the levels of hoops and is calculated by reducing all the dimensions of the area $A_{conf,max1}$ by a length equal to half the clear spacing between external hoops.

$A_{noConf1} = 173066.667$ is the unconfined external core area which is equal to $b_i^2/6$ as defined at (A.2).

$$\phi_{ase2} (> \phi_{ase1}) = \text{Max}(((A_{conf,max2} - A_{noConf2}) / A_{conf,max2}) * (A_{conf,min2} / A_{conf,max2}), 0) = 0.53375773$$

The definitions of A_{noConf} , $A_{conf,min}$ and $A_{conf,max}$ are derived from generalization of the rectangular sections confinement, which is expressed by (5.4d).

The generalization is done according to Mander, J., Priestley, M., and Park, R. (1988)

"Theoretical Stress-Strain Model for Confined Concrete."

J. Struct. Eng., 10.1061/(ASCE)0733-9445(1988)114:8(1804), 1804-1826.

$A_{conf,max2} = 252164.00$ is the confined core area at levels of member with hoops and is calculated as the area of core enclosed by the center lines of the internal perimeter hoops.

$A_{conf,min2} = 111441.00$ is the confined core area at midway between the levels of hoops and is calculated by reducing all the dimensions of the area $A_{conf,max2}$ by a length equal to half the clear spacing between internal hoops.

$A_{noConf2} = 110709.333$ is the unconfined internal core area which is equal to $b_i^2/6$ as defined at (A.2).

$$\phi_{psh,min} * F_{ywe} = \text{Min}(\phi_{psh,x} * F_{ywe}, \phi_{psh,y} * F_{ywe}) = 2.724$$

$$\phi_{psh,x} * F_{ywe} = \phi_{psh1} * F_{ywe1} + \phi_{psh2} * F_{ywe2} = 2.724$$

$$\phi_{psh1} ((5.4d), TBDY) = L_{stir1} * A_{stir1} / (A_{sec} * s_1) = 0.00301593$$

$$L_{stir1} \text{ (Length of stirrups along Y)} = 2160.00$$

$$A_{stir1} \text{ (stirrups area)} = 78.53982$$

$$\phi_{psh2} (5.4d) = L_{stir2} * A_{stir2} / (A_{sec} * s_2) = 0.00056047$$

$$L_{stir2} \text{ (Length of stirrups along Y)} = 1568.00$$

$$A_{stir2} \text{ (stirrups area)} = 50.26548$$

$$\phi_{psh,y} * F_{ywe} = \phi_{psh1} * F_{ywe1} + \phi_{psh2} * F_{ywe2} = 3.25416$$

$$\phi_{psh1} ((5.4d), TBDY) = L_{stir1} * A_{stir1} / (A_{sec} * s_1) = 0.00357443$$

$$L_{stir1} \text{ (Length of stirrups along X)} = 2560.00$$

$$A_{stir1} \text{ (stirrups area)} = 78.53982$$

$$\phi_{psh2} ((5.4d), TBDY) = L_{stir2} * A_{stir2} / (A_{sec} * s_2) = 0.00070345$$

$$L_{stir2} \text{ (Length of stirrups along X)} = 1968.00$$

$$A_{stir2} \text{ (stirrups area)} = 50.26548$$

$$A_{sec} = 562500.00$$

$$s_1 = 100.00$$

$$s_2 = 250.00$$

$$f_{ywe1} = 781.25$$

```

fywe2 = 656.25
fce = 18.00
From ((5A.5), TBDY), TBDY: cc = 0.00621307
c = confinement factor = 1.42131
y1 = 0.0025
sh1 = 0.008
ft1 = 907.50
fy1 = 756.25
su1 = 0.032
using (30) in Biskinis/Fardis (2013) multiplied with shear_factor
and also multiplied by the shear_factor according to 15.7.1.4, with
Shear_factor = 1.00
lo/lou,min = lb/ld = 1.00
su1 = 0.4*esu1_nominal ((5.5), TBDY) = 0.032
From table 5A.1, TBDY: esu1_nominal = 0.08,
For calculation of esu1_nominal and y1, sh1,ft1,fy1, it is considered
characteristic value fsy1 = fs1/1.2, from table 5.1, TBDY.
y1, sh1,ft1,fy1, are also multiplied by Min(1,1.25*(lb/ld)^ 2/3), from 10.3.5, ASCE 41-17.
with fs1 = (fs,jacket*Asl,ten,jacket + fs,core*Asl,ten,core)/Asl,ten = 756.25
with Es1 = (Es,jacket*Asl,ten,jacket + Es,core*Asl,ten,core)/Asl,ten = 200000.00
y2 = 0.0025
sh2 = 0.008
ft2 = 900.1904
fy2 = 750.1586
su2 = 0.032
using (30) in Biskinis/Fardis (2013) multiplied with shear_factor
and also multiplied by the shear_factor according to 15.7.1.4, with
Shear_factor = 1.00
lo/lou,min = lb/lb,min = 1.00
su2 = 0.4*esu2_nominal ((5.5), TBDY) = 0.032
From table 5A.1, TBDY: esu2_nominal = 0.08,
For calculation of esu2_nominal and y2, sh2,ft2,fy2, it is considered
characteristic value fsy2 = fs2/1.2, from table 5.1, TBDY.
y1, sh1,ft1,fy1, are also multiplied by Min(1,1.25*(lb/ld)^ 2/3), from 10.3.5, ASCE 41-17.
with fs2 = (fs,jacket*Asl,com,jacket + fs,core*Asl,com,core)/Asl,com = 750.1586
with Es2 = (Es,jacket*Asl,com,jacket + Es,core*Asl,com,core)/Asl,com = 200000.00
yv = 0.0025
shv = 0.008
ftv = 902.993
fyv = 752.4941
suv = 0.032
using (30) in Biskinis/Fardis (2013) multiplied with shear_factor
and also multiplied by the shear_factor according to 15.7.1.4, with
Shear_factor = 1.00
lo/lou,min = lb/ld = 1.00
suv = 0.4*esuv_nominal ((5.5), TBDY) = 0.032
From table 5A.1, TBDY: esuv_nominal = 0.08,
considering characteristic value fsyv = fsv/1.2, from table 5.1, TBDY
For calculation of esuv_nominal and yv, shv,ftv,fyv, it is considered
characteristic value fsyv = fsv/1.2, from table 5.1, TBDY.
y1, sh1,ft1,fy1, are also multiplied by Min(1,1.25*(lb/ld)^ 2/3), from 10.3.5, ASCE 41-17.
with fsv = (fs,jacket*Asl,mid,jacket + fs,mid*Asl,mid,core)/Asl,mid = 752.4941
with Esv = (Es,jacket*Asl,mid,jacket + Es,mid*Asl,mid,core)/Asl,mid = 200000.00
1 = Asl,ten/(b*d)*(fs1/fc) = 0.09629324
2 = Asl,com/(b*d)*(fs2/fc) = 0.15360794
v = Asl,mid/(b*d)*(fsv/fc) = 0.16660079
and confined core properties:
b = 890.00
d = 677.00
d' = 13.00
fcc (5A.2, TBDY) = 25.58352
cc (5A.5, TBDY) = 0.00621307
c = confinement factor = 1.42131
1 = Asl,ten/(b*d)*(fs1/fc) = 0.10733965
2 = Asl,com/(b*d)*(fs2/fc) = 0.17122928
v = Asl,mid/(b*d)*(fsv/fc) = 0.18571261
Case/Assumption: Unconfinedsd full section - Steel rupture

```

```

' satisfies Eq. (4.3)
---->
v < vs,y2 - LHS eq.(4.5) is not satisfied
---->
v < vs,c - RHS eq.(4.5) is not satisfied
---->
Case/Assumption Rejected.
---->
New Case/Assumption: Unconfined full section - Spalling of concrete cover
' satisfies Eq. (4.4)
---->
v < s,y1 - LHS eq.(4.7) is not satisfied
---->
v < vc,y1 - RHS eq.(4.6) is satisfied
---->
cu (4.10) = 0.58509201
MRc (4.17) = 2.3801E+009
---->
New Case/Assumption: Ultimate curvature of confined core after spalling of concrete cover
In expressions below, the following modifications have been made
- b, d, d' replaced by geometric parameters of the core: bo, do, d'o
- N, 1, 2, v normalised to bo*do, instead of b*d
- parameters of confined concrete, fcc, cc, used in lieu of fc, ecu
---->
Subcase: Rupture of tension steel
---->
v* < v*s,y2 - LHS eq.(4.5) is not satisfied
---->
v* < v*s,c - LHS eq.(4.5) is not satisfied
---->
Subcase rejected
---->
New Subcase: Failure of compression zone
---->
v* < v*c,y2 - LHS eq.(4.6) is not satisfied
---->
v* < v*c,y1 - RHS eq.(4.6) is not satisfied
---->
*cu (4.11) = 0.65878578
MRo (4.18) = 3.0615E+009
---->
u = cu (4.2) = 1.3930706E-005
Mu = MRo

```

Calculation of ratio lb/d

Adequate Lap Length: lb/d >= 1

Calculation of Mu1-

Calculation of ultimate curvature u according to 4.1, Biskinis/Fardis 2013:

u = 9.1773508E-006
Mu = 8.9683E+008

with full section properties:

b = 450.00
d = 707.00
d' = 43.00
v = 1.34176
N = 7.6839E+006
fc = 18.00
co (5A.5, TBDY) = 0.002

Final value of c_u : $c_u^* = \text{shear_factor} * \text{Max}(c_u, c_c) = 0.01486841$

The Shear_factor is considered equal to 1 (pure moment strength)

From (5.4b), TBDY: $c_u = 0.01486841$

w_e (5.4c) = 0.08077545

a_{se} ((5.4d), TBDY) = $(a_{se1} * A_{ext} + a_{se2} * A_{int}) / A_{sec} = 0.53375773$

$a_{se1} = \text{Max}(((A_{conf,max1} - A_{noConf1}) / A_{conf,max1}) * (A_{conf,min1} / A_{conf,max1}), 0) = 0.53375773$

The definitions of A_{noConf} , $A_{conf,min}$ and $A_{conf,max}$ are derived from generalization of the rectangular sections confinement, which is expressed by (5.4d).

The generalization is done according to Mander, J., Priestley, M., and Park, R. (1988)

"Theoretical Stress-Strain Model for Confined Concrete."

J. Struct. Eng., 10.1061/(ASCE)0733-9445(1988)114:8(1804), 1804-1826.

$A_{conf,max1} = 464100.00$ is the confined core area at levels of member with hoops and

is calculated as the area of core enclosed by the center lines of the external perimeter hoops.

$A_{conf,min1} = 395025.00$ is the confined core area at midway between the levels of hoops and

is calculated by reducing all the dimensions of the area $A_{conf,max1}$ by a length

equal to half the clear spacing between external hoops.

$A_{noConf1} = 173066.667$ is the unconfined external core area which is equal to $b_i^2/6$ as defined at (A.2).

$a_{se2} (>= a_{se1}) = \text{Max}(((A_{conf,max2} - A_{noConf2}) / A_{conf,max2}) * (A_{conf,min2} / A_{conf,max2}), 0) = 0.53375773$

The definitions of A_{noConf} , $A_{conf,min}$ and $A_{conf,max}$ are derived from generalization of the rectangular sections confinement, which is expressed by (5.4d).

The generalization is done according to Mander, J., Priestley, M., and Park, R. (1988)

"Theoretical Stress-Strain Model for Confined Concrete."

J. Struct. Eng., 10.1061/(ASCE)0733-9445(1988)114:8(1804), 1804-1826.

$A_{conf,max2} = 252164.00$ is the confined core area at levels of member with hoops and

is calculated as the area of core enclosed by the center lines of the internal perimeter hoops.

$A_{conf,min2} = 111441.00$ is the confined core area at midway between the levels of hoops and

is calculated by reducing all the dimensions of the area $A_{conf,max2}$ by a length

equal to half the clear spacing between internal hoops.

$A_{noConf2} = 110709.333$ is the unconfined internal core area which is equal to $b_i^2/6$ as defined at (A.2).

$p_{sh,min} * F_{ywe} = \text{Min}(p_{sh,x} * F_{ywe}, p_{sh,y} * F_{ywe}) = 2.724$

$p_{sh,x} * F_{ywe} = p_{sh1} * F_{ywe1} + p_{s2} * F_{ywe2} = 2.724$

p_{sh1} ((5.4d), TBDY) = $L_{stir1} * A_{stir1} / (A_{sec} * s_1) = 0.00301593$

L_{stir1} (Length of stirrups along Y) = 2160.00

A_{stir1} (stirrups area) = 78.53982

p_{sh2} (5.4d) = $L_{stir2} * A_{stir2} / (A_{sec} * s_2) = 0.00056047$

L_{stir2} (Length of stirrups along Y) = 1568.00

A_{stir2} (stirrups area) = 50.26548

$p_{sh,y} * F_{ywe} = p_{sh1} * F_{ywe1} + p_{s2} * F_{ywe2} = 3.25416$

p_{sh1} ((5.4d), TBDY) = $L_{stir1} * A_{stir1} / (A_{sec} * s_1) = 0.00357443$

L_{stir1} (Length of stirrups along X) = 2560.00

A_{stir1} (stirrups area) = 78.53982

p_{sh2} ((5.4d), TBDY) = $L_{stir2} * A_{stir2} / (A_{sec} * s_2) = 0.00070345$

L_{stir2} (Length of stirrups along X) = 1968.00

A_{stir2} (stirrups area) = 50.26548

$A_{sec} = 562500.00$

$s_1 = 100.00$

$s_2 = 250.00$

$f_{ywe1} = 781.25$

$f_{ywe2} = 656.25$

$f_{ce} = 18.00$

From ((5.A5), TBDY), TBDY: $c_c = 0.00621307$

$c = \text{confinement factor} = 1.42131$

$y_1 = 0.0025$

$sh_1 = 0.008$

$ft_1 = 900.1904$

$fy_1 = 750.1586$

$su_1 = 0.032$

using (30) in Biskinis/Fardis (2013) multiplied with shear_factor

and also multiplied by the shear_factor according to 15.7.1.4, with

Shear_factor = 1.00

$l_o/l_{ou,min} = l_b/l_d = 1.00$

$su_1 = 0.4 * esu_{1,nominal}$ ((5.5), TBDY) = 0.032

From table 5A.1, TBDY: $esu_{1,nominal} = 0.08$,

For calculation of $esu_{1,nominal}$ and y_1 , sh_1 , ft_1 , fy_1 , it is considered

characteristic value $f_{sy1} = f_{s1}/1.2$, from table 5.1, TBDY.
 y_1, sh_1, ft_1, fy_1 , are also multiplied by $\text{Min}(1, 1.25 \cdot (l_b/l_d)^{2/3})$, from 10.3.5, ASCE 41-17.
with $f_{s1} = (f_{sjacket} \cdot A_{sl,ten,jacket} + f_{s,core} \cdot A_{sl,ten,core})/A_{sl,ten} = 750.1586$
with $E_{s1} = (E_{sjacket} \cdot A_{sl,ten,jacket} + E_{s,core} \cdot A_{sl,ten,core})/A_{sl,ten} = 200000.00$
 $y_2 = 0.0025$
 $sh_2 = 0.008$
 $ft_2 = 907.50$
 $fy_2 = 756.25$
 $su_2 = 0.032$
using (30) in Biskinis/Fardis (2013) multiplied with shear_factor
and also multiplied by the shear_factor according to 15.7.1.4, with
Shear_factor = 1.00
 $l_o/l_{ou,min} = l_b/l_{b,min} = 1.00$
 $su_2 = 0.4 \cdot esu_{2,nominal} ((5.5), TBDY) = 0.032$
From table 5A.1, TBDY: $esu_{2,nominal} = 0.08$,
For calculation of $esu_{2,nominal}$ and y_2, sh_2, ft_2, fy_2 , it is considered
characteristic value $f_{sy2} = f_{s2}/1.2$, from table 5.1, TBDY.
 y_1, sh_1, ft_1, fy_1 , are also multiplied by $\text{Min}(1, 1.25 \cdot (l_b/l_d)^{2/3})$, from 10.3.5, ASCE 41-17.
with $f_{s2} = (f_{sjacket} \cdot A_{sl,com,jacket} + f_{s,core} \cdot A_{sl,com,core})/A_{sl,com} = 756.25$
with $E_{s2} = (E_{sjacket} \cdot A_{sl,com,jacket} + E_{s,core} \cdot A_{sl,com,core})/A_{sl,com} = 200000.00$
 $y_v = 0.0025$
 $sh_v = 0.008$
 $ft_v = 902.993$
 $fy_v = 752.4941$
 $suv = 0.032$
using (30) in Biskinis/Fardis (2013) multiplied with shear_factor
and also multiplied by the shear_factor according to 15.7.1.4, with
Shear_factor = 1.00
 $l_o/l_{ou,min} = l_b/l_d = 1.00$
 $suv = 0.4 \cdot esuv_{nominal} ((5.5), TBDY) = 0.032$
From table 5A.1, TBDY: $esuv_{nominal} = 0.08$,
considering characteristic value $f_{syv} = f_{sv}/1.2$, from table 5.1, TBDY
For calculation of $esuv_{nominal}$ and y_v, sh_v, ft_v, fy_v , it is considered
characteristic value $f_{syv} = f_{sv}/1.2$, from table 5.1, TBDY.
 y_1, sh_1, ft_1, fy_1 , are also multiplied by $\text{Min}(1, 1.25 \cdot (l_b/l_d)^{2/3})$, from 10.3.5, ASCE 41-17.
with $f_{sv} = (f_{sjacket} \cdot A_{sl,mid,jacket} + f_{s,mid} \cdot A_{sl,mid,core})/A_{sl,mid} = 752.4941$
with $E_{sv} = (E_{sjacket} \cdot A_{sl,mid,jacket} + E_{s,mid} \cdot A_{sl,mid,core})/A_{sl,mid} = 200000.00$
 $1 = A_{sl,ten}/(b \cdot d) \cdot (f_{s1}/f_c) = 0.32428344$
 $2 = A_{sl,com}/(b \cdot d) \cdot (f_{s2}/f_c) = 0.20328574$
 $v = A_{sl,mid}/(b \cdot d) \cdot (f_{sv}/f_c) = 0.35171278$
and confined core properties:
 $b = 390.00$
 $d = 677.00$
 $d' = 13.00$
 $f_{cc} (5A.2, TBDY) = 25.58352$
 $cc (5A.5, TBDY) = 0.00621307$
 $c = \text{confinement factor} = 1.42131$
 $1 = A_{sl,ten}/(b \cdot d) \cdot (f_{s1}/f_c) = 0.39075399$
 $2 = A_{sl,com}/(b \cdot d) \cdot (f_{s2}/f_c) = 0.24495458$
 $v = A_{sl,mid}/(b \cdot d) \cdot (f_{sv}/f_c) = 0.42380571$
Case/Assumption: Unconfined full section - Steel rupture
' satisfies Eq. (4.3)
--->
 $v < v_{s,y2}$ - LHS eq.(4.5) is not satisfied
--->
 $v < v_{s,c}$ - RHS eq.(4.5) is not satisfied
--->
Case/Assumption Rejected.
--->
New Case/Assumption: Unconfined full section - Spalling of concrete cover
' satisfies Eq. (4.4)
--->
 $v < s_{y1}$ - LHS eq.(4.7) is not satisfied
--->
 $v < v_{c,y1}$ - RHS eq.(4.6) is not satisfied
--->
 $cu (4.11) = 0.95920575$

MRC (4.18) = 7.7321E+008

---->

New Case/Assumption: Ultimate curvature of confined core after spalling of concrete cover

In expressions below, the following modifications have been made

- b, d, d' replaced by geometric parameters of the core: bo, do, d'o
- N, ϵ_s , ϵ_c , ϵ_{cs} normalised to b_o*d_o , instead of $b*d$
- ϵ_{cs} parameters of confined concrete, ϵ_{cc} , ϵ_{cs} , used in lieu of ϵ_c , ϵ_{cs}

---->

Subcase: Rupture of tension steel

---->

$\epsilon_s^* < \epsilon_{sy}$ - LHS eq.(4.5) is not satisfied

---->

$\epsilon_s^* < \epsilon_{sc}$ - LHS eq.(4.5) is not satisfied

---->

Subcase rejected

---->

New Subcase: Failure of compression zone

---->

$\epsilon_s^* < \epsilon_{sc}$ - LHS eq.(4.6) is not satisfied

---->

$\epsilon_s^* < \epsilon_{cs}$ - RHS eq.(4.6) is not satisfied

---->

ϵ_{cu} (4.11) = 1.00

MRO (4.18) = 8.9683E+008

---->

$\epsilon_u = \epsilon_{cu}$ (4.2) = 9.1773508E-006

$\mu_u = MRO$

Calculation of ratio I_b/I_d

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

Calculation of μ_{u2+}

Calculation of ultimate curvature ϵ_u according to 4.1, Biskinis/Fardis 2013:

$\epsilon_u = 1.3930706E-005$

$\mu_u = 3.0615E+009$

with full section properties:

$b = 950.00$

$d = 707.00$

$d' = 43.00$

$\nu = 0.63557247$

$N = 7.6839E+006$

$f_c = 18.00$

α (5A.5, TBDY) = 0.002

Final value of ϵ_{cu} : $\epsilon_{cu}^* = \text{shear_factor} * \text{Max}(\epsilon_{cu}, \epsilon_{cc}) = 0.01486841$

The Shear_factor is considered equal to 1 (pure moment strength)

From (5.4b), TBDY: $\epsilon_{cu} = 0.01486841$

ϵ_{we} (5.4c) = 0.08077545

ϵ_{ase} ((5.4d), TBDY) = $(\epsilon_{ase1} * A_{ext} + \epsilon_{ase2} * A_{int}) / A_{sec} = 0.53375773$

$\epsilon_{ase1} = \text{Max}(((A_{conf,max1} - A_{noConf1}) / A_{conf,max1}) * (A_{conf,min1} / A_{conf,max1}), 0) = 0.53375773$

The definitions of A_{noConf} , $A_{conf,min}$ and $A_{conf,max}$ are derived from generalization of the rectangular sections confinement, which is expressed by (5.4d).

The generalization is done according to Mander, J., Priestley, M., and Park, R. (1988)

"Theoretical Stress-Strain Model for Confined Concrete."

J. Struct. Eng., 10.1061/(ASCE)0733-9445(1988)114:8(1804), 1804-1826.

$A_{conf,max1} = 464100.00$ is the confined core area at levels of member with hoops and

is calculated as the area of core enclosed by the center lines of the external perimeter hoops.

$A_{conf,min1} = 395025.00$ is the confined core area at midway between the levels of hoops and

is calculated by reducing all the dimensions of the area $A_{conf,max1}$ by a length equal to half the clear spacing between external hoops.

AnoConf1 = 173066.667 is the unconfined external core area which is equal to $b_i^2/6$ as defined at (A.2).
 $ase2 (>=ase1) = \text{Max}(((A_{conf,max2}-AnoConf2)/A_{conf,max2})*(A_{conf,min2}/A_{conf,max2}),0) = 0.53375773$
The definitions of AnoConf, Aconf,min and Aconf,max are derived from generalization of the rectangular sections confinement, which is expressed by (5.4d).
The generalization is done according to Mander, J., Priestley, M., and Park, R. (1988) "Theoretical Stress-Strain Model for Confined Concrete."
J. Struct. Eng., 10.1061/(ASCE)0733-9445(1988)114:8(1804), 1804-1826.
Aconf,max2 = 252164.00 is the confined core area at levels of member with hoops and is calculated as the area of core enclosed by the center lines of the internal perimeter hoops.
Aconf,min2 = 111441.00 is the confined core area at midway between the levels of hoops and is calculated by reducing all the dimensions of the area Aconf,max2 by a length equal to half the clear spacing between internal hoops.
AnoConf2 = 110709.333 is the unconfined internal core area which is equal to $b_i^2/6$ as defined at (A.2).
 $psh,min*Fywe = \text{Min}(psh,x*Fywe, psh,y*Fywe) = 2.724$

$psh_x*Fywe = psh1*Fywe1 + ps2*Fywe2 = 2.724$
 $psh1 ((5.4d), TBDY) = Lstir1*Astir1/(Asec*s1) = 0.00301593$
Lstir1 (Length of stirrups along Y) = 2160.00
Astir1 (stirrups area) = 78.53982
 $psh2 ((5.4d)) = Lstir2*Astir2/(Asec*s2) = 0.00056047$
Lstir2 (Length of stirrups along Y) = 1568.00
Astir2 (stirrups area) = 50.26548

$psh_y*Fywe = psh1*Fywe1 + ps2*Fywe2 = 3.25416$
 $psh1 ((5.4d), TBDY) = Lstir1*Astir1/(Asec*s1) = 0.00357443$
Lstir1 (Length of stirrups along X) = 2560.00
Astir1 (stirrups area) = 78.53982
 $psh2 ((5.4d), TBDY) = Lstir2*Astir2/(Asec*s2) = 0.00070345$
Lstir2 (Length of stirrups along X) = 1968.00
Astir2 (stirrups area) = 50.26548

Asec = 562500.00

s1 = 100.00

s2 = 250.00

fywe1 = 781.25

fywe2 = 656.25

fce = 18.00

From ((5.A5), TBDY), TBDY: cc = 0.00621307

c = confinement factor = 1.42131

y1 = 0.0025

sh1 = 0.008

ft1 = 907.50

fy1 = 756.25

su1 = 0.032

using (30) in Biskinis/Fardis (2013) multiplied with shear_factor and also multiplied by the shear_factor according to 15.7.1.4, with Shear_factor = 1.00

lo/lou,min = lb/ld = 1.00

su1 = 0.4*esu1_nominal ((5.5), TBDY) = 0.032

From table 5A.1, TBDY: esu1_nominal = 0.08,

For calculation of esu1_nominal and y1, sh1,ft1,fy1, it is considered characteristic value fsy1 = fs1/1.2, from table 5.1, TBDY.

y1, sh1,ft1,fy1, are also multiplied by $\text{Min}(1, 1.25*(lb/ld)^{2/3})$, from 10.3.5, ASCE 41-17.

with fs1 = (fs,jacket*Asl,ten,jacket + fs,core*Asl,ten,core)/Asl,ten = 756.25

with Es1 = (Es,jacket*Asl,ten,jacket + Es,core*Asl,ten,core)/Asl,ten = 200000.00

y2 = 0.0025

sh2 = 0.008

ft2 = 900.1904

fy2 = 750.1586

su2 = 0.032

using (30) in Biskinis/Fardis (2013) multiplied with shear_factor and also multiplied by the shear_factor according to 15.7.1.4, with Shear_factor = 1.00

lo/lou,min = lb/lb,min = 1.00

su2 = 0.4*esu2_nominal ((5.5), TBDY) = 0.032

From table 5A.1, TBDY: esu2_nominal = 0.08,

For calculation of esu2_nominal and y2, sh2,ft2,fy2, it is considered

characteristic value $f_{sy2} = f_{s2}/1.2$, from table 5.1, TBDY.
 y_1, sh_1, ft_1, f_{y1} , are also multiplied by $\text{Min}(1, 1.25 \cdot (l_b/l_d)^{2/3})$, from 10.3.5, ASCE 41-17.
with $f_{s2} = (f_{sjacket} \cdot A_{sl,com,jacket} + f_{s,core} \cdot A_{sl,com,core}) / A_{sl,com} = 750.1586$
with $E_{s2} = (E_{sjacket} \cdot A_{sl,com,jacket} + E_{s,core} \cdot A_{sl,com,core}) / A_{sl,com} = 200000.00$
 $y_v = 0.0025$
 $sh_v = 0.008$
 $ft_v = 902.993$
 $f_{yv} = 752.4941$
 $suv = 0.032$
using (30) in Biskinis/Fardis (2013) multiplied with shear_factor
and also multiplied by the shear_factor according to 15.7.1.4, with
Shear_factor = 1.00
 $l_o/l_{ou,min} = l_b/l_d = 1.00$
 $suv = 0.4 \cdot esuv_{nominal} ((5.5), TBDY) = 0.032$
From table 5A.1, TBDY: $esuv_{nominal} = 0.08$,
considering characteristic value $f_{syv} = f_{sv}/1.2$, from table 5.1, TBDY
For calculation of $esuv_{nominal}$ and y_v, sh_v, ft_v, f_{yv} , it is considered
characteristic value $f_{syv} = f_{sv}/1.2$, from table 5.1, TBDY.
 y_1, sh_1, ft_1, f_{y1} , are also multiplied by $\text{Min}(1, 1.25 \cdot (l_b/l_d)^{2/3})$, from 10.3.5, ASCE 41-17.
with $f_{sv} = (f_{sjacket} \cdot A_{sl,mid,jacket} + f_{s,mid} \cdot A_{sl,mid,core}) / A_{sl,mid} = 752.4941$
with $E_{sv} = (E_{sjacket} \cdot A_{sl,mid,jacket} + E_{s,mid} \cdot A_{sl,mid,core}) / A_{sl,mid} = 200000.00$
 $1 = A_{sl,ten} / (b \cdot d) \cdot (f_{s1}/f_c) = 0.09629324$
 $2 = A_{sl,com} / (b \cdot d) \cdot (f_{s2}/f_c) = 0.15360794$
 $v = A_{sl,mid} / (b \cdot d) \cdot (f_{sv}/f_c) = 0.16660079$
and confined core properties:
 $b = 890.00$
 $d = 677.00$
 $d' = 13.00$
 $f_{cc} (5A.2, TBDY) = 25.58352$
 $cc (5A.5, TBDY) = 0.00621307$
 $c = \text{confinement factor} = 1.42131$
 $1 = A_{sl,ten} / (b \cdot d) \cdot (f_{s1}/f_c) = 0.10733965$
 $2 = A_{sl,com} / (b \cdot d) \cdot (f_{s2}/f_c) = 0.17122928$
 $v = A_{sl,mid} / (b \cdot d) \cdot (f_{sv}/f_c) = 0.18571261$
Case/Assumption: Unconfined full section - Steel rupture
' satisfies Eq. (4.3)
--->
 $v < v_{s,y2}$ - LHS eq.(4.5) is not satisfied
--->
 $v < v_{s,c}$ - RHS eq.(4.5) is not satisfied
--->
Case/Assumption Rejected.
--->
New Case/Assumption: Unconfined full section - Spalling of concrete cover
' satisfies Eq. (4.4)
--->
 $v < s_{y1}$ - LHS eq.(4.7) is not satisfied
--->
 $v < v_{c,y1}$ - RHS eq.(4.6) is satisfied
--->
 $cu (4.10) = 0.58509201$
 $M_{Rc} (4.17) = 2.3801E+009$
--->
New Case/Assumption: Ultimate curvature of confined core after spalling of concrete cover
In expressions below, the following modifications have been made
- b, d, d' replaced by geometric parameters of the core: b_o, d_o, d'_o
- $N, 1, 2, v$ normalised to $b_o \cdot d_o$, instead of $b \cdot d$
- parameters of confined concrete, f_{cc}, cc , used in lieu of f_c, ec_u
--->
Subcase: Rupture of tension steel
--->
 $v^* < v^*_{s,y2}$ - LHS eq.(4.5) is not satisfied
--->
 $v^* < v^*_{s,c}$ - LHS eq.(4.5) is not satisfied
--->
Subcase rejected
--->

New Subcase: Failure of compression zone

--->

$v^* < v^*_{c,y2}$ - LHS eq.(4.6) is not satisfied

--->

$v^* < v^*_{c,y1}$ - RHS eq.(4.6) is not satisfied

--->

$*cu(4.11) = 0.65878578$

$M_{Ro}(4.18) = 3.0615E+009$

--->

$u = cu(4.2) = 1.3930706E-005$

$\mu = M_{Ro}$

Calculation of ratio l_b/d

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

Calculation of μ_2 -

Calculation of ultimate curvature u according to 4.1, Biskinis/Fardis 2013:

$u = 9.1773508E-006$

$\mu = 8.9683E+008$

with full section properties:

$b = 450.00$

$d = 707.00$

$d' = 43.00$

$v = 1.34176$

$N = 7.6839E+006$

$f_c = 18.00$

$\alpha(5A.5, TBDY) = 0.002$

Final value of cu : $cu^* = \text{shear_factor} * \text{Max}(cu, cc) = 0.01486841$

The Shear_factor is considered equal to 1 (pure moment strength)

From (5.4b), TBDY: $cu = 0.01486841$

$w_e(5.4c) = 0.08077545$

$ase((5.4d), TBDY) = (ase1 * A_{ext} + ase2 * A_{int}) / A_{sec} = 0.53375773$

$ase1 = \text{Max}(((A_{conf,max1} - A_{noConf1}) / A_{conf,max1}) * (A_{conf,min1} / A_{conf,max1}), 0) = 0.53375773$

The definitions of A_{noConf} , $A_{conf,min}$ and $A_{conf,max}$ are derived from generalization of the rectangular sections confinement, which is expressed by (5.4d).

The generalization is done according to Mander, J., Priestley, M., and Park, R. (1988)

"Theoretical Stress-Strain Model for Confined Concrete."

J. Struct. Eng., 10.1061/(ASCE)0733-9445(1988)114:8(1804), 1804-1826.

$A_{conf,max1} = 464100.00$ is the confined core area at levels of member with hoops and is calculated as the area of core enclosed by the center lines of the external perimeter hoops.

$A_{conf,min1} = 395025.00$ is the confined core area at midway between the levels of hoops and is calculated by reducing all the dimensions of the area $A_{conf,max1}$ by a length equal to half the clear spacing between external hoops.

$A_{noConf1} = 173066.667$ is the unconfined external core area which is equal to $b_i^2/6$ as defined at (A.2).

$ase2 (\geq ase1) = \text{Max}(((A_{conf,max2} - A_{noConf2}) / A_{conf,max2}) * (A_{conf,min2} / A_{conf,max2}), 0) = 0.53375773$

The definitions of A_{noConf} , $A_{conf,min}$ and $A_{conf,max}$ are derived from generalization of the rectangular sections confinement, which is expressed by (5.4d).

The generalization is done according to Mander, J., Priestley, M., and Park, R. (1988)

"Theoretical Stress-Strain Model for Confined Concrete."

J. Struct. Eng., 10.1061/(ASCE)0733-9445(1988)114:8(1804), 1804-1826.

$A_{conf,max2} = 252164.00$ is the confined core area at levels of member with hoops and is calculated as the area of core enclosed by the center lines of the internal perimeter hoops.

$A_{conf,min2} = 111441.00$ is the confined core area at midway between the levels of hoops and is calculated by reducing all the dimensions of the area $A_{conf,max2}$ by a length equal to half the clear spacing between internal hoops.

$A_{noConf2} = 110709.333$ is the unconfined internal core area which is equal to $b_i^2/6$ as defined at (A.2).

$p_{sh,min} * F_{ywe} = \text{Min}(p_{sh,x} * F_{ywe}, p_{sh,y} * F_{ywe}) = 2.724$

$p_{sh_x} * F_{ywe} = p_{sh1} * F_{ywe1} + p_{sh2} * F_{ywe2} = 2.724$

$psh1 ((5.4d), TBDY) = Lstir1 * Astir1 / (Asec * s1) = 0.00301593$
 $Lstir1$ (Length of stirrups along Y) = 2160.00
 $Astir1$ (stirrups area) = 78.53982
 $psh2 (5.4d) = Lstir2 * Astir2 / (Asec * s2) = 0.00056047$
 $Lstir2$ (Length of stirrups along Y) = 1568.00
 $Astir2$ (stirrups area) = 50.26548

$psh_y * Fywe = psh1 * Fywe1 + ps2 * Fywe2 = 3.25416$
 $psh1 ((5.4d), TBDY) = Lstir1 * Astir1 / (Asec * s1) = 0.00357443$
 $Lstir1$ (Length of stirrups along X) = 2560.00
 $Astir1$ (stirrups area) = 78.53982
 $psh2 ((5.4d), TBDY) = Lstir2 * Astir2 / (Asec * s2) = 0.00070345$
 $Lstir2$ (Length of stirrups along X) = 1968.00
 $Astir2$ (stirrups area) = 50.26548

Asec = 562500.00

s1 = 100.00

s2 = 250.00

fywe1 = 781.25

fywe2 = 656.25

fce = 18.00

From ((5.A5), TBDY), TBDY: cc = 0.00621307

c = confinement factor = 1.42131

y1 = 0.0025

sh1 = 0.008

ft1 = 900.1904

fy1 = 750.1586

su1 = 0.032

using (30) in Biskinis/Fardis (2013) multiplied with shear_factor
and also multiplied by the shear_factor according to 15.7.1.4, with
Shear_factor = 1.00

lo/lou,min = lb/ld = 1.00

su1 = 0.4*esu1_nominal ((5.5), TBDY) = 0.032

From table 5A.1, TBDY: esu1_nominal = 0.08,

For calculation of esu1_nominal and y1, sh1, ft1, fy1, it is considered
characteristic value fsy1 = fs1/1.2, from table 5.1, TBDY.

y1, sh1, ft1, fy1, are also multiplied by $\text{Min}(1, 1.25 * (lb/ld)^{2/3})$, from 10.3.5, ASCE 41-17.

with fs1 = (fs,jacket*Asl,ten,jacket + fs,core*Asl,ten,core)/Asl,ten = 750.1586

with Es1 = (Es,jacket*Asl,ten,jacket + Es,core*Asl,ten,core)/Asl,ten = 200000.00

y2 = 0.0025

sh2 = 0.008

ft2 = 907.50

fy2 = 756.25

su2 = 0.032

using (30) in Biskinis/Fardis (2013) multiplied with shear_factor
and also multiplied by the shear_factor according to 15.7.1.4, with
Shear_factor = 1.00

lo/lou,min = lb/lb,min = 1.00

su2 = 0.4*esu2_nominal ((5.5), TBDY) = 0.032

From table 5A.1, TBDY: esu2_nominal = 0.08,

For calculation of esu2_nominal and y2, sh2, ft2, fy2, it is considered
characteristic value fsy2 = fs2/1.2, from table 5.1, TBDY.

y1, sh1, ft1, fy1, are also multiplied by $\text{Min}(1, 1.25 * (lb/ld)^{2/3})$, from 10.3.5, ASCE 41-17.

with fs2 = (fs,jacket*Asl,com,jacket + fs,core*Asl,com,core)/Asl,com = 756.25

with Es2 = (Es,jacket*Asl,com,jacket + Es,core*Asl,com,core)/Asl,com = 200000.00

yv = 0.0025

shv = 0.008

ftv = 902.993

fyv = 752.4941

suv = 0.032

using (30) in Biskinis/Fardis (2013) multiplied with shear_factor
and also multiplied by the shear_factor according to 15.7.1.4, with
Shear_factor = 1.00

lo/lou,min = lb/ld = 1.00

suv = 0.4*esuv_nominal ((5.5), TBDY) = 0.032

From table 5A.1, TBDY: esuv_nominal = 0.08,

considering characteristic value fsyv = fsv/1.2, from table 5.1, TBDY

```

For calculation of esuv_nominal and yv, shv,ftv,fyv, it is considered
characteristic value fsyv = fsv/1.2, from table 5.1, TBDY.
y1, sh1,ft1,fy1, are also multiplied by Min(1,1.25*(lb/ld)^ 2/3), from 10.3.5, ASCE 41-17.
with fsv = (fsjacket*Asl,mid,jacket + fs,mid*Asl,mid,core)/Asl,mid = 752.4941
with Esv = (Es,jacket*Asl,mid,jacket + Es,mid*Asl,mid,core)/Asl,mid = 200000.00
1 = Asl,ten/(b*d)*(fs1/fc) = 0.32428344
2 = Asl,com/(b*d)*(fs2/fc) = 0.20328574
v = Asl,mid/(b*d)*(fsv/fc) = 0.35171278
and confined core properties:
b = 390.00
d = 677.00
d' = 13.00
fcc (5A.2, TBDY) = 25.58352
cc (5A.5, TBDY) = 0.00621307
c = confinement factor = 1.42131
1 = Asl,ten/(b*d)*(fs1/fc) = 0.39075399
2 = Asl,com/(b*d)*(fs2/fc) = 0.24495458
v = Asl,mid/(b*d)*(fsv/fc) = 0.42380571
Case/Assumption: Unconfined full section - Steel rupture
' satisfies Eq. (4.3)
---->
v < vs,y2 - LHS eq.(4.5) is not satisfied
---->
v < vs,c - RHS eq.(4.5) is not satisfied
---->
Case/Assumption Rejected.
---->
New Case/Assumption: Unconfined full section - Spalling of concrete cover
' satisfies Eq. (4.4)
---->
v < s,y1 - LHS eq.(4.7) is not satisfied
---->
v < vc,y1 - RHS eq.(4.6) is not satisfied
---->
cu (4.11) = 0.95920575
MRc (4.18) = 7.7321E+008
---->
New Case/Assumption: Ultimate curvature of confined core after spalling of concrete cover
In expressions below, the following modifications have been made
- b, d, d' replaced by geometric parameters of the core: bo, do, d'o
- N, 1, 2, v normalised to bo*do, instead of b*d
- - parameters of confined concrete, fcc, cc, used in lieu of fc, ecu
---->
Subcase: Rupture of tension steel
---->
v* < v*s,y2 - LHS eq.(4.5) is not satisfied
---->
v* < v*s,c - LHS eq.(4.5) is not satisfied
---->
Subcase rejected
---->
New Subcase: Failure of compression zone
---->
v* < v*c,y2 - LHS eq.(4.6) is not satisfied
---->
v* < v*c,y1 - RHS eq.(4.6) is not satisfied
---->
*cu (4.11) = 1.00
MRo (4.18) = 8.9683E+008
---->
u = cu (4.2) = 9.1773508E-006
Mu = MRo

```

Calculation of ratio lb/ld

Adequate Lap Length: lb/ld >= 1

Calculation of Shear Strength $V_r = \text{Min}(V_{r1}, V_{r2}) = 1.1678\text{E}+006$

Calculation of Shear Strength at edge 1, $V_{r1} = 1.1678\text{E}+006$

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

$V_{\text{ColO}} = 1.1678\text{E}+006$

$\text{knl} = 1$ (zero step-static loading)

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

$= 1$ (normal-weight concrete)

Mean concrete strength: $f'_c = (f'_c_{\text{jacket}} * \text{Area}_{\text{jacket}} + f'_c_{\text{core}} * \text{Area}_{\text{core}}) / \text{Area}_{\text{section}} = 15.20$, but $f'_c^{0.5} \leq 8.3$ MPa (22.5.3.1, ACI 318-14)

$M / Vd = 4.00$

$\mu_u = 7.6104\text{E}+008$

$V_u = 227.4778$

$d = 0.8 * h = 600.00$

$N_u = 7.6839\text{E}+006$

$A_g = 337500.00$

From (11.5.4.8), ACI 318-14: $V_s = V_{s,\text{jacket}} + V_{s,\text{core}} = 1.0354\text{E}+006$

where:

$V_{s,\text{jacket}} = V_{s,j1} + V_{s,j2} = 942477.796$

$V_{s,j1} = 589048.623$ is calculated for section web jacket, with:

$d = 600.00$

$A_v = 157079.633$

$f_y = 625.00$

$s = 100.00$

$V_{s,j1}$ is multiplied by $\text{Col},j1 = 1.00$

$s/d = 0.16666667$

$V_{s,j2} = 353429.174$ is calculated for section flange jacket, with:

$d = 360.00$

$A_v = 157079.633$

$f_y = 625.00$

$s = 100.00$

$V_{s,j2}$ is multiplied by $\text{Col},j2 = 1.00$

$s/d = 0.27777778$

$V_{s,\text{core}} = V_{s,c1} + V_{s,c2} = 92890.612$

$V_{s,c1} = 92890.612$ is calculated for section web core, with:

$d = 440.00$

$A_v = 100530.965$

$f_y = 525.00$

$s = 250.00$

$V_{s,c1}$ is multiplied by $\text{Col},c1 = 1.00$

$s/d = 0.56818182$

$V_{s,c2} = 0.00$ is calculated for section flange core, with:

$d = 200.00$

$A_v = 100530.965$

$f_y = 525.00$

$s = 250.00$

$V_{s,c2}$ is multiplied by $\text{Col},c2 = 0.00$

$s/d = 1.25$

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

From (11-11), ACI 440: $V_s + V_f \leq 699281.943$

$b_w = 450.00$

Calculation of Shear Strength at edge 2, $V_{r2} = 1.1678\text{E}+006$

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

$V_{\text{ColO}} = 1.1678\text{E}+006$

$\text{knl} = 1$ (zero step-static loading)

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

= 1 (normal-weight concrete)
 Mean concrete strength: $f_c' = (f_c'_{\text{jacket}} \cdot \text{Area}_{\text{jacket}} + f_c'_{\text{core}} \cdot \text{Area}_{\text{core}}) / \text{Area}_{\text{section}} = 15.20$, but $f_c'^{0.5} \leq 8.3$ MPa (22.5.3.1, ACI 318-14)
 $M/Vd = 4.00$
 $\mu_u = 6.8991\text{E}+008$
 $V_u = 227.4778$
 $d = 0.8 \cdot h = 600.00$
 $N_u = 7.6839\text{E}+006$
 $A_g = 337500.00$
 From (11.5.4.8), ACI 318-14: $V_s = V_{s,\text{jacket}} + V_{s,\text{core}} = 1.0354\text{E}+006$
 where:
 $V_{s,\text{jacket}} = V_{s,j1} + V_{s,j2} = 942477.796$
 $V_{s,j1} = 589048.623$ is calculated for section web jacket, with:
 $d = 600.00$
 $A_v = 157079.633$
 $f_y = 625.00$
 $s = 100.00$
 $V_{s,j1}$ is multiplied by $\text{Col},j1 = 1.00$
 $s/d = 0.16666667$
 $V_{s,j2} = 353429.174$ is calculated for section flange jacket, with:
 $d = 360.00$
 $A_v = 157079.633$
 $f_y = 625.00$
 $s = 100.00$
 $V_{s,j2}$ is multiplied by $\text{Col},j2 = 1.00$
 $s/d = 0.27777778$
 $V_{s,\text{core}} = V_{s,c1} + V_{s,c2} = 92890.612$
 $V_{s,c1} = 92890.612$ is calculated for section web core, with:
 $d = 440.00$
 $A_v = 100530.965$
 $f_y = 525.00$
 $s = 250.00$
 $V_{s,c1}$ is multiplied by $\text{Col},c1 = 1.00$
 $s/d = 0.56818182$
 $V_{s,c2} = 0.00$ is calculated for section flange core, with:
 $d = 200.00$
 $A_v = 100530.965$
 $f_y = 525.00$
 $s = 250.00$
 $V_{s,c2}$ is multiplied by $\text{Col},c2 = 0.00$
 $s/d = 1.25$
 $V_f ((11-3)-(11.4), \text{ACI } 440) = 0.00$
 From (11-11), ACI 440: $V_s + V_f \leq 699281.943$
 $bw = 450.00$

 End Of Calculation of Shear Capacity ratio for element: column JTC1 of floor 1
 At local axis: 3

 Start Of Calculation of Shear Capacity ratio for element: column JTC1 of floor 1
 At Shear local axis: 2
 (Bending local axis: 3)
 Section Type: rcjtc

Constant Properties

 Knowledge Factor, $\phi = 0.80$
 Mean strength values are used for both shear and moment calculations.
 Consequently:
 Jacket
 New material of Primary Member: Concrete Strength, $f_c = f_{cm} = 18.00$
 New material of Primary Member: Steel Strength, $f_s = f_{sm} = 625.00$
 Concrete Elasticity, $E_c = 19940.411$

Steel Elasticity, $E_s = 200000.00$
 Existing Column
 Existing material of Primary Member: Concrete Strength, $f_c = f_{cm} = 12.00$
 Existing material of Primary Member: Steel Strength, $f_s = f_{sm} = 525.00$
 Concrete Elasticity, $E_c = 16281.278$
 Steel Elasticity, $E_s = 200000.00$
 #####
 Note: Especially for the calculation of moment strengths,
 the above steel re-bar strengths are multiplied by 1.25 according to R18.6.5, ACI 318-14
 Jacket
 New material: Steel Strength, $f_s = 1.25 \cdot f_{sm} = 781.25$
 Existing Column
 Existing material: Steel Strength, $f_s = 1.25 \cdot f_{sm} = 656.25$
 #####
 Max Height, $H_{max} = 750.00$
 Min Height, $H_{min} = 450.00$
 Max Width, $W_{max} = 950.00$
 Min Width, $W_{min} = 450.00$
 Eccentricity, $E_{cc} = 250.00$
 Jacket Thickness, $t_j = 100.00$
 Cover Thickness, $c = 25.00$
 Mean Confinement Factor overall section = 1.42131
 Element Length, $L = 3000.00$
 Primary Member
 Ribbed Bars
 Ductile Steel
 With Detailing for Earthquake Resistance (including stirrups closed at 135°)
 Longitudinal Bars With Ends Lapped Starting at the End Sections
 Adequate Lap Length ($l_o/l_{ou}, \min > 1$)
 No FRP Wrapping

Stepwise Properties

At local axis: 2
 EDGE -A-
 Shear Force, $V_a = -137.4894$
 EDGE -B-
 Shear Force, $V_b = 137.4894$
 BOTH EDGES
 Axial Force, $F = -7.6839E+006$
 Longitudinal Reinforcement Area Distribution (in 2 divisions)
 -Tension: $A_{sl} = 314.1593$
 -Compression: $A_{slc} = 6176.371$
 Longitudinal Reinforcement Area Distribution (in 3 divisions)
 -Tension: $A_{sl,ten} = 1539.38$
 -Compression: $A_{sl,com} = 1539.38$
 -Middle: $A_{sl,mid} = 3612.832$

Calculation of Shear Capacity ratio, $V_e/V_r = 0.86614729$
 Member Controlled by Flexure ($V_e/V_r < 1$)
 Calculation of Shear Demand from fig. R18.6.5, ACI 318-14 $V_e = (M_{pr1} + M_{pr2})/l_n = 1.2287E+006$
 with
 $M_{pr1} = \max(\mu_{u1+}, \mu_{u1-}) = 1.8431E+009$
 $\mu_{u1+} = 1.8431E+009$, is the ultimate moment strength at the edge 1 of the member in the actual moment direction
 which is defined for the static loading combination
 $\mu_{u1-} = 1.8431E+009$, is the ultimate moment strength at the edge 1 of the member in the opposite moment
 direction which is defined for the static loading combination
 $M_{pr2} = \max(\mu_{u2+}, \mu_{u2-}) = 1.8431E+009$
 $\mu_{u2+} = 1.8431E+009$, is the ultimate moment strength at the edge 2 of the member in the actual moment direction
 which is defined for the the static loading combination
 $\mu_{u2-} = 1.8431E+009$, 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 μ_{1+}

Calculation of ultimate curvature μ according to 4.1, Biskinis/Fardis 2013:

$$\mu = 7.8730391\text{E-}006$$

$$\mu = 1.8431\text{E+}009$$

with full section properties:

$$b = 450.00$$

$$d = 907.00$$

$$d' = 43.00$$

$$v = 1.0459$$

$$N = 7.6839\text{E+}006$$

$$f_c = 18.00$$

$$c_o(5A.5, \text{TBDY}) = 0.002$$

$$\text{Final value of } \mu: \mu^* = \text{shear_factor} * \text{Max}(\mu, c_o) = 0.01486841$$

The Shear_factor is considered equal to 1 (pure moment strength)

$$\text{From (5.4b), TBDY: } \mu = 0.01486841$$

$$\mu_e(5.4c) = 0.08077545$$

$$\mu_{se}((5.4d), \text{TBDY}) = (\mu_{se1} * A_{ext} + \mu_{se2} * A_{int}) / A_{sec} = 0.53375773$$

$$\mu_{se1} = \text{Max}(((A_{conf,max1} - A_{noConf1}) / A_{conf,max1}) * (A_{conf,min1} / A_{conf,max1}), 0) = 0.53375773$$

The definitions of A_{noConf} , $A_{conf,min}$ and $A_{conf,max}$ are derived from generalization of the rectangular sections confinement, which is expressed by (5.4d).

The generalization is done according to Mander, J., Priestley, M., and Park, R. (1988)

"Theoretical Stress-Strain Model for Confined Concrete."

J. Struct. Eng., 10.1061/(ASCE)0733-9445(1988)114:8(1804), 1804-1826.

$A_{conf,max1} = 464100.00$ is the confined core area at levels of member with hoops and is calculated as the area of core enclosed by the center lines of the external perimeter hoops.

$A_{conf,min1} = 395025.00$ is the confined core area at midway between the levels of hoops and is calculated by reducing all the dimensions of the area $A_{conf,max1}$ by a length equal to half the clear spacing between external hoops.

$A_{noConf1} = 173066.667$ is the unconfined external core area which is equal to $b_i^2/6$ as defined at (A.2).

$$\mu_{se2} (>= \mu_{se1}) = \text{Max}(((A_{conf,max2} - A_{noConf2}) / A_{conf,max2}) * (A_{conf,min2} / A_{conf,max2}), 0) = 0.53375773$$

The definitions of A_{noConf} , $A_{conf,min}$ and $A_{conf,max}$ are derived from generalization of the rectangular sections confinement, which is expressed by (5.4d).

The generalization is done according to Mander, J., Priestley, M., and Park, R. (1988)

"Theoretical Stress-Strain Model for Confined Concrete."

J. Struct. Eng., 10.1061/(ASCE)0733-9445(1988)114:8(1804), 1804-1826.

$A_{conf,max2} = 252164.00$ is the confined core area at levels of member with hoops and is calculated as the area of core enclosed by the center lines of the internal perimeter hoops.

$A_{conf,min2} = 111441.00$ is the confined core area at midway between the levels of hoops and is calculated by reducing all the dimensions of the area $A_{conf,max2}$ by a length equal to half the clear spacing between internal hoops.

$A_{noConf2} = 110709.333$ is the unconfined internal core area which is equal to $b_i^2/6$ as defined at (A.2).

$$p_{sh,min} * F_{ywe} = \text{Min}(p_{sh,x} * F_{ywe}, p_{sh,y} * F_{ywe}) = 2.724$$

$$p_{sh,x} * F_{ywe} = p_{sh1} * F_{ywe1} + p_{sh2} * F_{ywe2} = 2.724$$

$$p_{sh1}((5.4d), \text{TBDY}) = L_{stir1} * A_{stir1} / (A_{sec} * s_1) = 0.00301593$$

$$L_{stir1} (\text{Length of stirrups along Y}) = 2160.00$$

$$A_{stir1} (\text{stirrups area}) = 78.53982$$

$$p_{sh2}((5.4d)) = L_{stir2} * A_{stir2} / (A_{sec} * s_2) = 0.00056047$$

$$L_{stir2} (\text{Length of stirrups along Y}) = 1568.00$$

$$A_{stir2} (\text{stirrups area}) = 50.26548$$

$$p_{sh,y} * F_{ywe} = p_{sh1} * F_{ywe1} + p_{sh2} * F_{ywe2} = 3.25416$$

$$p_{sh1}((5.4d), \text{TBDY}) = L_{stir1} * A_{stir1} / (A_{sec} * s_1) = 0.00357443$$

$$L_{stir1} (\text{Length of stirrups along X}) = 2560.00$$

$$A_{stir1} (\text{stirrups area}) = 78.53982$$

$$p_{sh2}((5.4d), \text{TBDY}) = L_{stir2} * A_{stir2} / (A_{sec} * s_2) = 0.00070345$$

$$L_{stir2} (\text{Length of stirrups along X}) = 1968.00$$

$$A_{stir2} (\text{stirrups area}) = 50.26548$$

$$A_{sec} = 562500.00$$

$$s_1 = 100.00$$

$$s_2 = 250.00$$

$fy_{we1} = 781.25$
 $fy_{we2} = 656.25$
 $f_{ce} = 18.00$
 From ((5.A.5), TBDY), TBDY: $cc = 0.00621307$
 $c = \text{confinement factor} = 1.42131$
 $y1 = 0.0025$
 $sh1 = 0.008$
 $ft1 = 907.50$
 $fy1 = 756.25$
 $su1 = 0.032$
 using (30) in Biskinis/Fardis (2013) multiplied with shear_factor
 and also multiplied by the shear_factor according to 15.7.1.4, with
 $Shear_factor = 1.00$
 $lo/lou,min = lb/ld = 1.00$
 $su1 = 0.4 * esu1_nominal ((5.5), TBDY) = 0.032$
 From table 5A.1, TBDY: $esu1_nominal = 0.08$,
 For calculation of $esu1_nominal$ and $y1, sh1, ft1, fy1$, it is considered
 characteristic value $fsy1 = fs1/1.2$, from table 5.1, TBDY.
 $y1, sh1, ft1, fy1$, are also multiplied by $Min(1, 1.25 * (lb/ld)^{2/3})$, from 10.3.5, ASCE 41-17.
 with $fs1 = (fs_{jacket} * Asl_{ten,jacket} + fs_{core} * Asl_{ten,core}) / Asl_{ten} = 756.25$
 with $Es1 = (Es_{jacket} * Asl_{ten,jacket} + Es_{core} * Asl_{ten,core}) / Asl_{ten} = 200000.00$
 $y2 = 0.0025$
 $sh2 = 0.008$
 $ft2 = 907.50$
 $fy2 = 756.25$
 $su2 = 0.032$
 using (30) in Biskinis/Fardis (2013) multiplied with shear_factor
 and also multiplied by the shear_factor according to 15.7.1.4, with
 $Shear_factor = 1.00$
 $lo/lou,min = lb/lb,min = 1.00$
 $su2 = 0.4 * esu2_nominal ((5.5), TBDY) = 0.032$
 From table 5A.1, TBDY: $esu2_nominal = 0.08$,
 For calculation of $esu2_nominal$ and $y2, sh2, ft2, fy2$, it is considered
 characteristic value $fsy2 = fs2/1.2$, from table 5.1, TBDY.
 $y1, sh1, ft1, fy1$, are also multiplied by $Min(1, 1.25 * (lb/ld)^{2/3})$, from 10.3.5, ASCE 41-17.
 with $fs2 = (fs_{jacket} * Asl_{com,jacket} + fs_{core} * Asl_{com,core}) / Asl_{com} = 756.25$
 with $Es2 = (Es_{jacket} * Asl_{com,jacket} + Es_{core} * Asl_{com,core}) / Asl_{com} = 200000.00$
 $yv = 0.0025$
 $shv = 0.008$
 $ftv = 899.1522$
 $fyv = 749.2935$
 $suv = 0.032$
 using (30) in Biskinis/Fardis (2013) multiplied with shear_factor
 and also multiplied by the shear_factor according to 15.7.1.4, with
 $Shear_factor = 1.00$
 $lo/lou,min = lb/ld = 1.00$
 $suv = 0.4 * esuv_nominal ((5.5), TBDY) = 0.032$
 From table 5A.1, TBDY: $esuv_nominal = 0.08$,
 considering characteristic value $fsyv = fsv/1.2$, from table 5.1, TBDY
 For calculation of $esuv_nominal$ and yv, shv, ftv, fyv , it is considered
 characteristic value $fsyv = fsv/1.2$, from table 5.1, TBDY.
 $y1, sh1, ft1, fy1$, are also multiplied by $Min(1, 1.25 * (lb/ld)^{2/3})$, from 10.3.5, ASCE 41-17.
 with $fsv = (fs_{jacket} * Asl_{mid,jacket} + fs_{mid} * Asl_{mid,core}) / Asl_{mid} = 749.2935$
 with $Es_v = (Es_{jacket} * Asl_{mid,jacket} + Es_{mid} * Asl_{mid,core}) / Asl_{mid} = 200000.00$
 $1 = Asl_{ten} / (b * d) * (fs1 / f_c) = 0.15845977$
 $2 = Asl_{com} / (b * d) * (fs2 / f_c) = 0.15845977$
 $v = Asl_{mid} / (b * d) * (fsv / f_c) = 0.36847443$
 and confined core properties:
 $b = 390.00$
 $d = 877.00$
 $d' = 13.00$
 $f_{cc} (5A.2, TBDY) = 25.58352$
 $cc (5A.5, TBDY) = 0.00621307$
 $c = \text{confinement factor} = 1.42131$
 $1 = Asl_{ten} / (b * d) * (fs1 / f_c) = 0.18909264$
 $2 = Asl_{com} / (b * d) * (fs2 / f_c) = 0.18909264$
 $v = Asl_{mid} / (b * d) * (fsv / f_c) = 0.43970658$

Case/Assumption: Unconfined full section - Steel rupture
' satisfies Eq. (4.3)

--->
 $v < v_{s,y2}$ - LHS eq.(4.5) is not satisfied

--->
 $v < v_{s,c}$ - RHS eq.(4.5) is not satisfied

--->
Case/Assumption Rejected.

--->
New Case/Assumption: Unconfined full section - Spalling of concrete cover
' satisfies Eq. (4.4)

--->
 $v < s_{y1}$ - LHS eq.(4.7) is not satisfied

--->
 $v < v_{c,y1}$ - RHS eq.(4.6) is satisfied

--->
 ϵ_{cu} (4.10) = 0.83934343
 M_{Rc} (4.17) = 1.8439E+009

--->
New Case/Assumption: Ultimate curvature of confined core after spalling of concrete cover
In expressions below, the following modifications have been made
- b , d , d' replaced by geometric parameters of the core: b_o , d_o , d'_o
- N , ϵ_{s1} , ϵ_{s2} , v normalised to $b_o d_o$, instead of $b d$
- parameters of confined concrete, f_{cc} , ϵ_{cc} , used in lieu of f_c , ϵ_{cu}

--->
Subcase: Rupture of tension steel

--->
 $v^* < v^* s_{y2}$ - LHS eq.(4.5) is not satisfied

--->
 $v^* < v^* s_{c}$ - LHS eq.(4.5) is not satisfied

--->
Subcase rejected

--->
New Subcase: Failure of compression zone

--->
 $v^* < v^* \epsilon_{c,y2}$ - LHS eq.(4.6) is not satisfied

--->
 $v^* < v^* \epsilon_{c,y1}$ - RHS eq.(4.6) is not satisfied

--->
 ϵ_{cu}^* (4.11) = 0.89983732
 M_{Ro} (4.18) = 1.8431E+009

--->
 $u = \epsilon_{cu}$ (4.2) = 7.8730391E-006
 $M_u = M_{Ro}$

Calculation of ratio I_b/I_d

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

Calculation of M_{u1} -

Calculation of ultimate curvature u according to 4.1, Biskinis/Fardis 2013:
 $u = 7.8730391E-006$
 $M_u = 1.8431E+009$

with full section properties:

$b = 450.00$
 $d = 907.00$
 $d' = 43.00$
 $v = 1.0459$
 $N = 7.6839E+006$
 $f_c = 18.00$

co (5A.5, TBDY) = 0.002
 Final value of cu : $cu^* = shear_factor * Max(cu, cc) = 0.01486841$
 The Shear_factor is considered equal to 1 (pure moment strength)
 From (5.4b), TBDY: $cu = 0.01486841$
 we (5.4c) = 0.08077545
 ase ((5.4d), TBDY) = $(ase1 * Aext + ase2 * Aint) / Asec = 0.53375773$
 $ase1 = Max(((Aconf,max1 - AnoConf1) / Aconf,max1) * (Aconf,min1 / Aconf,max1), 0) = 0.53375773$
 The definitions of $AnoConf$, $Aconf,min$ and $Aconf,max$ are derived from generalization of the rectangular sections confinement, which is expressed by (5.4d).
 The generalization is done according to Mander, J., Priestley, M., and Park, R. (1988) "Theoretical Stress-Strain Model for Confined Concrete."
 J. Struct. Eng., 10.1061/(ASCE)0733-9445(1988)114:8(1804), 1804-1826.
 $Aconf,max1 = 464100.00$ is the confined core area at levels of member with hoops and is calculated as the area of core enclosed by the center lines of the external perimeter hoops.
 $Aconf,min1 = 395025.00$ is the confined core area at midway between the levels of hoops and is calculated by reducing all the dimensions of the area $Aconf,max1$ by a length equal to half the clear spacing between external hoops.
 $AnoConf1 = 173066.667$ is the unconfined external core area which is equal to $bi^2/6$ as defined at (A.2).
 $ase2 (>= ase1) = Max(((Aconf,max2 - AnoConf2) / Aconf,max2) * (Aconf,min2 / Aconf,max2), 0) = 0.53375773$
 The definitions of $AnoConf$, $Aconf,min$ and $Aconf,max$ are derived from generalization of the rectangular sections confinement, which is expressed by (5.4d).
 The generalization is done according to Mander, J., Priestley, M., and Park, R. (1988) "Theoretical Stress-Strain Model for Confined Concrete."
 J. Struct. Eng., 10.1061/(ASCE)0733-9445(1988)114:8(1804), 1804-1826.
 $Aconf,max2 = 252164.00$ is the confined core area at levels of member with hoops and is calculated as the area of core enclosed by the center lines of the internal perimeter hoops.
 $Aconf,min2 = 111441.00$ is the confined core area at midway between the levels of hoops and is calculated by reducing all the dimensions of the area $Aconf,max2$ by a length equal to half the clear spacing between internal hoops.
 $AnoConf2 = 110709.333$ is the unconfined internal core area which is equal to $bi^2/6$ as defined at (A.2).
 $psh,min * Fywe = Min(psh,x * Fywe, psh,y * Fywe) = 2.724$

$psh_x * Fywe = psh1 * Fywe1 + ps2 * Fywe2 = 2.724$
 $psh1$ ((5.4d), TBDY) = $Lstir1 * Astir1 / (Asec * s1) = 0.00301593$
 $Lstir1$ (Length of stirrups along Y) = 2160.00
 $Astir1$ (stirrups area) = 78.53982
 $psh2$ (5.4d) = $Lstir2 * Astir2 / (Asec * s2) = 0.00056047$
 $Lstir2$ (Length of stirrups along Y) = 1568.00
 $Astir2$ (stirrups area) = 50.26548

$psh_y * Fywe = psh1 * Fywe1 + ps2 * Fywe2 = 3.25416$
 $psh1$ ((5.4d), TBDY) = $Lstir1 * Astir1 / (Asec * s1) = 0.00357443$
 $Lstir1$ (Length of stirrups along X) = 2560.00
 $Astir1$ (stirrups area) = 78.53982
 $psh2$ ((5.4d), TBDY) = $Lstir2 * Astir2 / (Asec * s2) = 0.00070345$
 $Lstir2$ (Length of stirrups along X) = 1968.00
 $Astir2$ (stirrups area) = 50.26548

$Asec = 562500.00$
 $s1 = 100.00$
 $s2 = 250.00$
 $fywe1 = 781.25$
 $fywe2 = 656.25$
 $fce = 18.00$
 From ((5.A5), TBDY), TBDY: $cc = 0.00621307$
 c = confinement factor = 1.42131
 $y1 = 0.0025$
 $sh1 = 0.008$
 $ft1 = 907.50$
 $fy1 = 756.25$
 $su1 = 0.032$
 using (30) in Biskinis/Fardis (2013) multiplied with shear_factor and also multiplied by the shear_factor according to 15.7.1.4, with Shear_factor = 1.00
 $lo/lou,min = lb/d = 1.00$
 $su1 = 0.4 * esu1_nominal$ ((5.5), TBDY) = 0.032
 From table 5A.1, TBDY: $esu1_nominal = 0.08$,

For calculation of $esu1_nominal$ and $y1, sh1, ft1, fy1$, it is considered characteristic value $fsy1 = fs1/1.2$, from table 5.1, TBDY.

$y1, sh1, ft1, fy1$, are also multiplied by $\text{Min}(1, 1.25 \cdot (lb/ld)^{2/3})$, from 10.3.5, ASCE 41-17.

with $fs1 = (fs_{jacket} \cdot Asl_{ten,jacket} + fs_{core} \cdot Asl_{ten,core}) / Asl_{ten} = 756.25$

with $Es1 = (Es_{jacket} \cdot Asl_{ten,jacket} + Es_{core} \cdot Asl_{ten,core}) / Asl_{ten} = 200000.00$

$y2 = 0.0025$
 $sh2 = 0.008$
 $ft2 = 907.50$
 $fy2 = 756.25$
 $su2 = 0.032$

using (30) in Biskinis/Fardis (2013) multiplied with shear_factor and also multiplied by the shear_factor according to 15.7.1.4, with Shear_factor = 1.00

$lo/lou, min = lb/lb, min = 1.00$
 $su2 = 0.4 \cdot esu2_nominal ((5.5), TBDY) = 0.032$

From table 5A.1, TBDY: $esu2_nominal = 0.08$

For calculation of $esu2_nominal$ and $y2, sh2, ft2, fy2$, it is considered characteristic value $fsy2 = fs2/1.2$, from table 5.1, TBDY.

$y1, sh1, ft1, fy1$, are also multiplied by $\text{Min}(1, 1.25 \cdot (lb/ld)^{2/3})$, from 10.3.5, ASCE 41-17.

with $fs2 = (fs_{jacket} \cdot Asl_{com,jacket} + fs_{core} \cdot Asl_{com,core}) / Asl_{com} = 756.25$

with $Es2 = (Es_{jacket} \cdot Asl_{com,jacket} + Es_{core} \cdot Asl_{com,core}) / Asl_{com} = 200000.00$

$yv = 0.0025$
 $shv = 0.008$
 $ftv = 899.1522$
 $fyv = 749.2935$
 $suv = 0.032$

using (30) in Biskinis/Fardis (2013) multiplied with shear_factor and also multiplied by the shear_factor according to 15.7.1.4, with Shear_factor = 1.00

$lo/lou, min = lb/ld = 1.00$
 $suv = 0.4 \cdot esuv_nominal ((5.5), TBDY) = 0.032$

From table 5A.1, TBDY: $esuv_nominal = 0.08$

considering characteristic value $fsyv = fsv/1.2$, from table 5.1, TBDY

For calculation of $esuv_nominal$ and yv, shv, ftv, fyv , it is considered characteristic value $fsyv = fsv/1.2$, from table 5.1, TBDY.

$y1, sh1, ft1, fy1$, are also multiplied by $\text{Min}(1, 1.25 \cdot (lb/ld)^{2/3})$, from 10.3.5, ASCE 41-17.

with $fsv = (fs_{jacket} \cdot Asl_{mid,jacket} + fs_{mid} \cdot Asl_{mid,core}) / Asl_{mid} = 749.2935$

with $Es_v = (Es_{jacket} \cdot Asl_{mid,jacket} + Es_{mid} \cdot Asl_{mid,core}) / Asl_{mid} = 200000.00$

$1 = Asl_{ten} / (b \cdot d) \cdot (fs1 / fc) = 0.15845977$
 $2 = Asl_{com} / (b \cdot d) \cdot (fs2 / fc) = 0.15845977$
 $v = Asl_{mid} / (b \cdot d) \cdot (fsv / fc) = 0.36847443$

and confined core properties:

$b = 390.00$
 $d = 877.00$
 $d' = 13.00$
 $fcc (5A.2, TBDY) = 25.58352$
 $cc (5A.5, TBDY) = 0.00621307$
 $c = \text{confinement factor} = 1.42131$
 $1 = Asl_{ten} / (b \cdot d) \cdot (fs1 / fc) = 0.18909264$
 $2 = Asl_{com} / (b \cdot d) \cdot (fs2 / fc) = 0.18909264$
 $v = Asl_{mid} / (b \cdot d) \cdot (fsv / fc) = 0.43970658$

Case/Assumption: Unconfined full section - Steel rupture
 ' satisfies Eq. (4.3)

--->

$v < vs, y2$ - LHS eq.(4.5) is not satisfied

--->

$v < vs, c$ - RHS eq.(4.5) is not satisfied

--->

Case/Assumption Rejected.

--->

New Case/Assumption: Unconfined full section - Spalling of concrete cover
 ' satisfies Eq. (4.4)

--->

$v < s, y1$ - LHS eq.(4.7) is not satisfied

--->

$v < vc, y1$ - RHS eq.(4.6) is satisfied

--->

cu (4.10) = 0.83934343
MRc (4.17) = 1.8439E+009

--->

New Case/Assumption: Ultimate curvature of confined core after spalling of concrete cover
In expressions below, the following modifications have been made

- b, d, d' replaced by geometric parameters of the core: bo, do, d'o
- N, 1, 2, v normalised to bo*do, instead of b*d
- - parameters of confined concrete, fcc, cc, used in lieu of fc, ecu

--->

Subcase: Rupture of tension steel

--->

v* < v*s,y2 - LHS eq.(4.5) is not satisfied

--->

v* < v*s,c - LHS eq.(4.5) is not satisfied

--->

Subcase rejected

--->

New Subcase: Failure of compression zone

--->

v* < v*c,y2 - LHS eq.(4.6) is not satisfied

--->

v* < v*c,y1 - RHS eq.(4.6) is not satisfied

--->

*cu (4.11) = 0.89983732

MRo (4.18) = 1.8431E+009

--->

u = cu (4.2) = 7.8730391E-006

Mu = MRo

Calculation of ratio lb/ld

Adequate Lap Length: lb/ld >= 1

Calculation of Mu2+

Calculation of ultimate curvature u according to 4.1, Biskinis/Fardis 2013:

u = 7.8730391E-006

Mu = 1.8431E+009

with full section properties:

b = 450.00

d = 907.00

d' = 43.00

v = 1.0459

N = 7.6839E+006

fc = 18.00

co (5A.5, TBDY) = 0.002

Final value of cu: cu* = shear_factor * Max(cu, cc) = 0.01486841

The Shear_factor is considered equal to 1 (pure moment strength)

From (5.4b), TBDY: cu = 0.01486841

we (5.4c) = 0.08077545

ase ((5.4d), TBDY) = (ase1*Aext+ase2*Aint)/Asec = 0.53375773

ase1 = Max(((Aconf,max1-AnoConf1)/Aconf,max1)*(Aconf,min1/Aconf,max1),0) = 0.53375773

The definitions of AnoConf, Aconf,min and Aconf,max are derived from generalization of the rectangular sections confinement, which is expressed by (5.4d).

The generalization is done according to Mander, J., Priestley, M., and Park, R. (1988)

"Theoretical Stress-Strain Model for Confined Concrete."

J. Struct. Eng., 10.1061/(ASCE)0733-9445(1988)114:8(1804), 1804-1826.

Aconf,max1 = 464100.00 is the confined core area at levels of member with hoops and

is calculated as the area of core enclosed by the center lines of the external perimeter hoops.

Aconf,min1 = 395025.00 is the confined core area at midway between the levels of hoops and

is calculated by reducing all the dimensions of the area Aconf,max1 by a length

equal to half the clear spacing between external hoops.

AnoConf1 = 173066.667 is the unconfined external core area which is equal to $b_i^2/6$ as defined at (A.2).
 $ase2 (>=ase1) = \text{Max}(((A_{conf,max2}-AnoConf2)/A_{conf,max2})*(A_{conf,min2}/A_{conf,max2}),0) = 0.53375773$

The definitions of AnoConf, Aconf,min and Aconf,max are derived from generalization of the rectangular sections confinement, which is expressed by (5.4d).

The generalization is done according to Mander, J., Priestley, M., and Park, R. (1988)

"Theoretical Stress-Strain Model for Confined Concrete."

J. Struct. Eng., 10.1061/(ASCE)0733-9445(1988)114:8(1804), 1804-1826.

Aconf,max2 = 252164.00 is the confined core area at levels of member with hoops and

is calculated as the area of core enclosed by the center lines of the internal perimeter hoops.

Aconf,min2 = 111441.00 is the confined core area at midway between the levels of hoops and

is calculated by reducing all the dimensions of the area Aconf,max2 by a length

equal to half the clear spacing between internal hoops.

AnoConf2 = 110709.333 is the unconfined internal core area which is equal to $b_i^2/6$ as defined at (A.2).

$psh,min*Fywe = \text{Min}(psh,x*Fywe, psh,y*Fywe) = 2.724$

$psh_x*Fywe = psh1*Fywe1 + ps2*Fywe2 = 2.724$

$psh1 ((5.4d), TBDY) = Lstir1*Astir1/(Asec*s1) = 0.00301593$

Lstir1 (Length of stirrups along Y) = 2160.00

Astir1 (stirrups area) = 78.53982

$psh2 ((5.4d)) = Lstir2*Astir2/(Asec*s2) = 0.00056047$

Lstir2 (Length of stirrups along Y) = 1568.00

Astir2 (stirrups area) = 50.26548

$psh_y*Fywe = psh1*Fywe1 + ps2*Fywe2 = 3.25416$

$psh1 ((5.4d), TBDY) = Lstir1*Astir1/(Asec*s1) = 0.00357443$

Lstir1 (Length of stirrups along X) = 2560.00

Astir1 (stirrups area) = 78.53982

$psh2 ((5.4d), TBDY) = Lstir2*Astir2/(Asec*s2) = 0.00070345$

Lstir2 (Length of stirrups along X) = 1968.00

Astir2 (stirrups area) = 50.26548

Asec = 562500.00

s1 = 100.00

s2 = 250.00

fywe1 = 781.25

fywe2 = 656.25

fce = 18.00

From ((5.A5), TBDY), TBDY: cc = 0.00621307

c = confinement factor = 1.42131

y1 = 0.0025

sh1 = 0.008

ft1 = 907.50

fy1 = 756.25

su1 = 0.032

using (30) in Biskinis/Fardis (2013) multiplied with shear_factor

and also multiplied by the shear_factor according to 15.7.1.4, with

Shear_factor = 1.00

lo/lou,min = lb/ld = 1.00

su1 = 0.4*esu1_nominal ((5.5), TBDY) = 0.032

From table 5A.1, TBDY: esu1_nominal = 0.08,

For calculation of esu1_nominal and y1, sh1,ft1,fy1, it is considered characteristic value fsy1 = fs1/1.2, from table 5.1, TBDY.

y1, sh1,ft1,fy1, are also multiplied by $\text{Min}(1,1.25*(lb/ld)^{2/3})$, from 10.3.5, ASCE 41-17.

with fs1 = (fs,jacket*Asl,ten,jacket + fs,core*Asl,ten,core)/Asl,ten = 756.25

with Es1 = (Es,jacket*Asl,ten,jacket + Es,core*Asl,ten,core)/Asl,ten = 200000.00

y2 = 0.0025

sh2 = 0.008

ft2 = 907.50

fy2 = 756.25

su2 = 0.032

using (30) in Biskinis/Fardis (2013) multiplied with shear_factor

and also multiplied by the shear_factor according to 15.7.1.4, with

Shear_factor = 1.00

lo/lou,min = lb/lb,min = 1.00

su2 = 0.4*esu2_nominal ((5.5), TBDY) = 0.032

From table 5A.1, TBDY: esu2_nominal = 0.08,

For calculation of $es_{u2_nominal}$ and y_2 , sh_2 , ft_2 , fy_2 , it is considered characteristic value $fs_{y2} = fs_2/1.2$, from table 5.1, TBDY.

y_1 , sh_1 , ft_1 , fy_1 , are also multiplied by $\text{Min}(1, 1.25 \cdot (l_b/l_d)^{2/3})$, from 10.3.5, ASCE 41-17.

with $fs_2 = (fs_{jacket} \cdot A_{sl,com,jacket} + fs_{core} \cdot A_{sl,com,core}) / A_{sl,com} = 756.25$

with $Es_2 = (Es_{jacket} \cdot A_{sl,com,jacket} + Es_{core} \cdot A_{sl,com,core}) / A_{sl,com} = 200000.00$

$y_v = 0.0025$
 $sh_v = 0.008$
 $ft_v = 899.1522$
 $fy_v = 749.2935$
 $s_{uv} = 0.032$

using (30) in Biskinis/Fardis (2013) multiplied with shear_factor and also multiplied by the shear_factor according to 15.7.1.4, with Shear_factor = 1.00

$l_o/l_{ou,min} = l_b/l_d = 1.00$
 $s_{uv} = 0.4 \cdot es_{uv_nominal} ((5.5), TBDY) = 0.032$

From table 5A.1, TBDY: $es_{uv_nominal} = 0.08$, considering characteristic value $fs_{yv} = f_{sv}/1.2$, from table 5.1, TBDY

For calculation of $es_{uv_nominal}$ and y_v , sh_v , ft_v , fy_v , it is considered characteristic value $fs_{yv} = f_{sv}/1.2$, from table 5.1, TBDY.

y_1 , sh_1 , ft_1 , fy_1 , are also multiplied by $\text{Min}(1, 1.25 \cdot (l_b/l_d)^{2/3})$, from 10.3.5, ASCE 41-17.

with $f_{sv} = (fs_{jacket} \cdot A_{sl,mid,jacket} + fs_{mid} \cdot A_{sl,mid,core}) / A_{sl,mid} = 749.2935$

with $Es_v = (Es_{jacket} \cdot A_{sl,mid,jacket} + Es_{mid} \cdot A_{sl,mid,core}) / A_{sl,mid} = 200000.00$

$1 = A_{sl,ten} / (b \cdot d) \cdot (f_{s1}/f_c) = 0.15845977$
 $2 = A_{sl,com} / (b \cdot d) \cdot (f_{s2}/f_c) = 0.15845977$
 $v = A_{sl,mid} / (b \cdot d) \cdot (f_{sv}/f_c) = 0.36847443$

and confined core properties:

$b = 390.00$
 $d = 877.00$
 $d' = 13.00$
 $f_{cc} (5A.2, TBDY) = 25.58352$
 $cc (5A.5, TBDY) = 0.00621307$
 $c = \text{confinement factor} = 1.42131$
 $1 = A_{sl,ten} / (b \cdot d) \cdot (f_{s1}/f_c) = 0.18909264$
 $2 = A_{sl,com} / (b \cdot d) \cdot (f_{s2}/f_c) = 0.18909264$
 $v = A_{sl,mid} / (b \cdot d) \cdot (f_{sv}/f_c) = 0.43970658$

Case/Assumption: Unconfined full section - Steel rupture
'satisfies Eq. (4.3)

--->

$v < v_{s,y2}$ - LHS eq.(4.5) is not satisfied

--->

$v < v_{s,c}$ - RHS eq.(4.5) is not satisfied

--->

Case/Assumption Rejected.

--->

New Case/Assumption: Unconfined full section - Spalling of concrete cover
'satisfies Eq. (4.4)

--->

$v < s_{y1}$ - LHS eq.(4.7) is not satisfied

--->

$v < v_{c,y1}$ - RHS eq.(4.6) is satisfied

--->

$cu (4.10) = 0.83934343$
 $M_{Rc} (4.17) = 1.8439E+009$

--->

New Case/Assumption: Ultimate curvature of confined core after spalling of concrete cover

In expressions below, the following modifications have been made

- b , d , d' replaced by geometric parameters of the core: b_o , d_o , d'_o
- N_1 , N_2 , v normalised to $b_o \cdot d_o$, instead of $b \cdot d$
- parameters of confined concrete, f_{cc} , cc , used in lieu of f_c , ecu

--->

Subcase: Rupture of tension steel

--->

$v^* < v^*_{s,y2}$ - LHS eq.(4.5) is not satisfied

--->

$v^* < v^*_{s,c}$ - LHS eq.(4.5) is not satisfied

--->

Subcase rejected

--->
New Subcase: Failure of compression zone

--->
 $v^* < v^*_{c,y2}$ - LHS eq.(4.6) is not satisfied

--->
 $v^* < v^*_{c,y1}$ - RHS eq.(4.6) is not satisfied

--->
 $\mu_{cu} (4.11) = 0.89983732$
 $M_{Ro} (4.18) = 1.8431E+009$

--->
 $\mu = \mu_{cu} (4.2) = 7.8730391E-006$
 $\mu_u = M_{Ro}$

Calculation of ratio I_b/I_d

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

Calculation of μ_{u2}

Calculation of ultimate curvature μ_u according to 4.1, Biskinis/Fardis 2013:

$\mu_u = 7.8730391E-006$
 $\mu_u = 1.8431E+009$

with full section properties:

$b = 450.00$
 $d = 907.00$
 $d' = 43.00$
 $v = 1.0459$
 $N = 7.6839E+006$

$f_c = 18.00$
 $\alpha (5A.5, TBDY) = 0.002$

Final value of μ_{cu} : $\mu_{cu}^* = \text{shear_factor} * \text{Max}(\mu_{cu}, \mu_{cc}) = 0.01486841$

The Shear_factor is considered equal to 1 (pure moment strength)

From (5.4b), TBDY: $\mu_{cu} = 0.01486841$

$\mu_{we} (5.4c) = 0.08077545$

$\mu_{ase} ((5.4d), TBDY) = (\mu_{ase1} * A_{ext} + \mu_{ase2} * A_{int}) / A_{sec} = 0.53375773$

$\mu_{ase1} = \text{Max}(((A_{conf,max1} - A_{noConf1}) / A_{conf,max1}) * (A_{conf,min1} / A_{conf,max1}), 0) = 0.53375773$

The definitions of A_{noConf} , $A_{conf,min}$ and $A_{conf,max}$ are derived from generalization of the rectangular sections confinement, which is expressed by (5.4d).

The generalization is done according to Mander, J., Priestley, M., and Park, R. (1988)

"Theoretical Stress-Strain Model for Confined Concrete."

J. Struct. Eng., 10.1061/(ASCE)0733-9445(1988)114:8(1804), 1804-1826.

$A_{conf,max1} = 464100.00$ is the confined core area at levels of member with hoops and is calculated as the area of core enclosed by the center lines of the external perimeter hoops.

$A_{conf,min1} = 395025.00$ is the confined core area at midway between the levels of hoops and is calculated by reducing all the dimensions of the area $A_{conf,max1}$ by a length equal to half the clear spacing between external hoops.

$A_{noConf1} = 173066.667$ is the unconfined external core area which is equal to $b_i^2/6$ as defined at (A.2).

$\mu_{ase2} (\geq \mu_{ase1}) = \text{Max}(((A_{conf,max2} - A_{noConf2}) / A_{conf,max2}) * (A_{conf,min2} / A_{conf,max2}), 0) = 0.53375773$

The definitions of A_{noConf} , $A_{conf,min}$ and $A_{conf,max}$ are derived from generalization of the rectangular sections confinement, which is expressed by (5.4d).

The generalization is done according to Mander, J., Priestley, M., and Park, R. (1988)

"Theoretical Stress-Strain Model for Confined Concrete."

J. Struct. Eng., 10.1061/(ASCE)0733-9445(1988)114:8(1804), 1804-1826.

$A_{conf,max2} = 252164.00$ is the confined core area at levels of member with hoops and is calculated as the area of core enclosed by the center lines of the internal perimeter hoops.

$A_{conf,min2} = 111441.00$ is the confined core area at midway between the levels of hoops and is calculated by reducing all the dimensions of the area $A_{conf,max2}$ by a length equal to half the clear spacing between internal hoops.

$A_{noConf2} = 110709.333$ is the unconfined internal core area which is equal to $b_i^2/6$ as defined at (A.2).

$\mu_{psh,min} * F_{ywe} = \text{Min}(\mu_{psh,x} * F_{ywe}, \mu_{psh,y} * F_{ywe}) = 2.724$

$psh_x * Fywe = psh1 * Fywe1 + ps2 * Fywe2 = 2.724$
 $psh1 ((5.4d), TBDY) = Lstir1 * Astir1 / (Asec * s1) = 0.00301593$
 $Lstir1 \text{ (Length of stirrups along Y)} = 2160.00$
 $Astir1 \text{ (stirrups area)} = 78.53982$
 $psh2 (5.4d) = Lstir2 * Astir2 / (Asec * s2) = 0.00056047$
 $Lstir2 \text{ (Length of stirrups along Y)} = 1568.00$
 $Astir2 \text{ (stirrups area)} = 50.26548$

$psh_y * Fywe = psh1 * Fywe1 + ps2 * Fywe2 = 3.25416$
 $psh1 ((5.4d), TBDY) = Lstir1 * Astir1 / (Asec * s1) = 0.00357443$
 $Lstir1 \text{ (Length of stirrups along X)} = 2560.00$
 $Astir1 \text{ (stirrups area)} = 78.53982$
 $psh2 ((5.4d), TBDY) = Lstir2 * Astir2 / (Asec * s2) = 0.00070345$
 $Lstir2 \text{ (Length of stirrups along X)} = 1968.00$
 $Astir2 \text{ (stirrups area)} = 50.26548$

$Asec = 562500.00$
 $s1 = 100.00$
 $s2 = 250.00$
 $fywe1 = 781.25$
 $fywe2 = 656.25$
 $fce = 18.00$

From ((5.A5), TBDY), TBDY: $cc = 0.00621307$
 $c = \text{confinement factor} = 1.42131$

$y1 = 0.0025$
 $sh1 = 0.008$
 $ft1 = 907.50$
 $fy1 = 756.25$
 $su1 = 0.032$

using (30) in Biskinis/Fardis (2013) multiplied with shear_factor
 and also multiplied by the shear_factor according to 15.7.1.4, with
 $Shear_factor = 1.00$

$lo/lou, min = lb/ld = 1.00$

$su1 = 0.4 * esu1_nominal ((5.5), TBDY) = 0.032$

From table 5A.1, TBDY: $esu1_nominal = 0.08$,

For calculation of $esu1_nominal$ and $y1, sh1, ft1, fy1$, it is considered
 characteristic value $fsy1 = fs1/1.2$, from table 5.1, TBDY.

$y1, sh1, ft1, fy1$, are also multiplied by $\text{Min}(1, 1.25 * (lb/ld)^{2/3})$, from 10.3.5, ASCE 41-17.

with $fs1 = (fs_jacket * Asl, ten, jacket + fs_core * Asl, ten, core) / Asl, ten = 756.25$

with $Es1 = (Es_jacket * Asl, ten, jacket + Es_core * Asl, ten, core) / Asl, ten = 200000.00$

$y2 = 0.0025$
 $sh2 = 0.008$
 $ft2 = 907.50$
 $fy2 = 756.25$
 $su2 = 0.032$

using (30) in Biskinis/Fardis (2013) multiplied with shear_factor
 and also multiplied by the shear_factor according to 15.7.1.4, with
 $Shear_factor = 1.00$

$lo/lou, min = lb/lb, min = 1.00$

$su2 = 0.4 * esu2_nominal ((5.5), TBDY) = 0.032$

From table 5A.1, TBDY: $esu2_nominal = 0.08$,

For calculation of $esu2_nominal$ and $y2, sh2, ft2, fy2$, it is considered
 characteristic value $fsy2 = fs2/1.2$, from table 5.1, TBDY.

$y2, sh2, ft2, fy2$, are also multiplied by $\text{Min}(1, 1.25 * (lb/ld)^{2/3})$, from 10.3.5, ASCE 41-17.

with $fs2 = (fs_jacket * Asl, com, jacket + fs_core * Asl, com, core) / Asl, com = 756.25$

with $Es2 = (Es_jacket * Asl, com, jacket + Es_core * Asl, com, core) / Asl, com = 200000.00$

$yv = 0.0025$
 $shv = 0.008$
 $ftv = 899.1522$
 $fyv = 749.2935$
 $suv = 0.032$

using (30) in Biskinis/Fardis (2013) multiplied with shear_factor
 and also multiplied by the shear_factor according to 15.7.1.4, with
 $Shear_factor = 1.00$

$lo/lou, min = lb/ld = 1.00$

$suv = 0.4 * esuv_nominal ((5.5), TBDY) = 0.032$

From table 5A.1, TBDY: $esuv_nominal = 0.08$,

considering characteristic value $f_{sv} = f_{sv}/1.2$, from table 5.1, TBDY
For calculation of $e_{suv_nominal}$ and y_v , sh_v , ft_v , fy_v , it is considered
characteristic value $f_{sv} = f_{sv}/1.2$, from table 5.1, TBDY.
 y_1 , sh_1 , ft_1 , fy_1 , are also multiplied by $\text{Min}(1, 1.25 \cdot (l_b/l_d)^{2/3})$, from 10.3.5, ASCE 41-17.
with $f_{sv} = (f_{s,jacket} \cdot A_{s,mid,jacket} + f_{s,mid} \cdot A_{s,mid,core}) / A_{s,mid} = 749.2935$
with $E_{sv} = (E_{s,jacket} \cdot A_{s,mid,jacket} + E_{s,mid} \cdot A_{s,mid,core}) / A_{s,mid} = 200000.00$
 $1 = A_{s,ten} / (b \cdot d) \cdot (f_{s1} / f_c) = 0.15845977$
 $2 = A_{s,com} / (b \cdot d) \cdot (f_{s2} / f_c) = 0.15845977$
 $v = A_{s,mid} / (b \cdot d) \cdot (f_{sv} / f_c) = 0.36847443$
and confined core properties:
 $b = 390.00$
 $d = 877.00$
 $d' = 13.00$
 $f_{cc} \text{ (5A.2, TBDY)} = 25.58352$
 $cc \text{ (5A.5, TBDY)} = 0.00621307$
 $c = \text{confinement factor} = 1.42131$
 $1 = A_{s,ten} / (b \cdot d) \cdot (f_{s1} / f_c) = 0.18909264$
 $2 = A_{s,com} / (b \cdot d) \cdot (f_{s2} / f_c) = 0.18909264$
 $v = A_{s,mid} / (b \cdot d) \cdot (f_{sv} / f_c) = 0.43970658$
Case/Assumption: Unconfined full section - Steel rupture
'satisfies Eq. (4.3)
--->
 $v < v_{s,y2}$ - LHS eq.(4.5) is not satisfied
--->
 $v < v_{s,c}$ - RHS eq.(4.5) is not satisfied
--->
Case/Assumption Rejected.
--->
New Case/Assumption: Unconfined full section - Spalling of concrete cover
'satisfies Eq. (4.4)
--->
 $v < s_{y1}$ - LHS eq.(4.7) is not satisfied
--->
 $v < v_{c,y1}$ - RHS eq.(4.6) is satisfied
--->
 $\phi_{cu} \text{ (4.10)} = 0.83934343$
 $M_{Rc} \text{ (4.17)} = 1.8439E+009$
--->
New Case/Assumption: Ultimate curvature of confined core after spalling of concrete cover
In expressions below, the following modifications have been made
- b , d , d' replaced by geometric parameters of the core: b_o , d_o , d'_o
- N_1 , N_2 , v normalised to $b_o \cdot d_o$, instead of $b \cdot d$
- parameters of confined concrete, f_{cc} , cc , used in lieu of f_c , e_{cu}
--->
Subcase: Rupture of tension steel
--->
 $v^* < v_{s,y2}$ - LHS eq.(4.5) is not satisfied
--->
 $v^* < v_{s,c}$ - LHS eq.(4.5) is not satisfied
--->
Subcase rejected
--->
New Subcase: Failure of compression zone
--->
 $v^* < v_{c,y2}$ - LHS eq.(4.6) is not satisfied
--->
 $v^* < v_{c,y1}$ - RHS eq.(4.6) is not satisfied
--->
 $\phi_{cu} \text{ (4.11)} = 0.89983732$
 $M_{Ro} \text{ (4.18)} = 1.8431E+009$
--->
 $u = \phi_{cu} \text{ (4.2)} = 7.8730391E-006$
 $\mu_u = M_{Ro}$

Calculation of ratio l_b/l_d

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

Calculation of Shear Strength $V_r = \text{Min}(V_{r1}, V_{r2}) = 1.4186\text{E}+006$

Calculation of Shear Strength at edge 1, $V_{r1} = 1.4186\text{E}+006$

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

$V_{\text{ColO}} = 1.4186\text{E}+006$

$k_n l = 1$ (zero step-static loading)

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

$= 1$ (normal-weight concrete)

Mean concrete strength: $f'_c = (f'_c \text{ jacket} \cdot \text{Area}_{\text{jacket}} + f'_c \text{ core} \cdot \text{Area}_{\text{core}}) / \text{Area}_{\text{section}} = 15.20$, but $f'_c^{0.5} \leq 8.3$ MPa (22.5.3.1, ACI 318-14)

$M/Vd = 4.00$

$\mu_u = 2.6919\text{E}+008$

$V_u = 137.4894$

$d = 0.8 \cdot h = 760.00$

$N_u = 7.6839\text{E}+006$

$A_g = 427500.00$

From (11.5.4.8), ACI 318-14: $V_s = V_{s,\text{jacket}} + V_{s,\text{core}} = 1.2262\text{E}+006$

where:

$V_{s,\text{jacket}} = V_{s,j1} + V_{s,j2} = 1.0996\text{E}+006$

$V_{s,j1} = 353429.174$ is calculated for section web jacket, with:

$d = 360.00$

$A_v = 157079.633$

$f_y = 625.00$

$s = 100.00$

$V_{s,j1}$ is multiplied by $\text{Col},j1 = 1.00$

$s/d = 0.27777778$

$V_{s,j2} = 746128.255$ is calculated for section flange jacket, with:

$d = 760.00$

$A_v = 157079.633$

$f_y = 625.00$

$s = 100.00$

$V_{s,j2}$ is multiplied by $\text{Col},j2 = 1.00$

$s/d = 0.13157895$

$V_{s,\text{core}} = V_{s,c1} + V_{s,c2} = 126669.016$

$V_{s,c1} = 0.00$ is calculated for section web core, with:

$d = 200.00$

$A_v = 100530.965$

$f_y = 525.00$

$s = 250.00$

$V_{s,c1}$ is multiplied by $\text{Col},c1 = 0.00$

$s/d = 1.25$

$V_{s,c2} = 126669.016$ is calculated for section flange core, with:

$d = 600.00$

$A_v = 100530.965$

$f_y = 525.00$

$s = 250.00$

$V_{s,c2}$ is multiplied by $\text{Col},c2 = 1.00$

$s/d = 0.41666667$

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

From (11-11), ACI 440: $V_s + V_f \leq 885757.128$

$b_w = 450.00$

Calculation of Shear Strength at edge 2, $V_{r2} = 1.4186\text{E}+006$

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

$V_{\text{ColO}} = 1.4186\text{E}+006$

$k_n l = 1$ (zero step-static loading)

NOTE: In expression (10-3) ' $V_s = A_v f_y d/s$ ' is replaced by ' $V_s + f V_f$ '

where V_f is the contribution of FRPs (11.3), ACI 440).

= 1 (normal-weight concrete)

Mean concrete strength: $f'_c = (f'_c_{\text{jacket}} \cdot \text{Area}_{\text{jacket}} + f'_c_{\text{core}} \cdot \text{Area}_{\text{core}}) / \text{Area}_{\text{section}} = 15.20$, but $f'_c^{0.5} \leq 8.3$ MPa (22.5.3.1, ACI 318-14)

$M/Vd = 4.00$

$\mu_u = 2.5135 \times 10^8$

$V_u = 137.4894$

$d = 0.8 \cdot h = 760.00$

$N_u = 7.6839 \times 10^6$

$A_g = 427500.00$

From (11.5.4.8), ACI 318-14: $V_s = V_{s,\text{jacket}} + V_{s,\text{core}} = 1.2262 \times 10^6$

where:

$V_{s,\text{jacket}} = V_{s,j1} + V_{s,j2} = 1.0996 \times 10^6$

$V_{s,j1} = 353429.174$ is calculated for section web jacket, with:

$d = 360.00$

$A_v = 157079.633$

$f_y = 625.00$

$s = 100.00$

$V_{s,j1}$ is multiplied by $\text{Col},j1 = 1.00$

$s/d = 0.27777778$

$V_{s,j2} = 746128.255$ is calculated for section flange jacket, with:

$d = 760.00$

$A_v = 157079.633$

$f_y = 625.00$

$s = 100.00$

$V_{s,j2}$ is multiplied by $\text{Col},j2 = 1.00$

$s/d = 0.13157895$

$V_{s,\text{core}} = V_{s,c1} + V_{s,c2} = 126669.016$

$V_{s,c1} = 0.00$ is calculated for section web core, with:

$d = 200.00$

$A_v = 100530.965$

$f_y = 525.00$

$s = 250.00$

$V_{s,c1}$ is multiplied by $\text{Col},c1 = 0.00$

$s/d = 1.25$

$V_{s,c2} = 126669.016$ is calculated for section flange core, with:

$d = 600.00$

$A_v = 100530.965$

$f_y = 525.00$

$s = 250.00$

$V_{s,c2}$ is multiplied by $\text{Col},c2 = 1.00$

$s/d = 0.41666667$

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

From (11-11), ACI 440: $V_s + V_f \leq 885757.128$

$b_w = 450.00$

End Of Calculation of Shear Capacity ratio for element: column JTC1 of floor 1

At local axis: 2

Start Of Calculation of Chord Rotation Capacity for element: column JTC1 of floor 1

At local axis: 2

Integration Section: (b)

Section Type: rcjtcs

Constant Properties

Knowledge Factor, $\gamma = 0.80$

Chord Rotation is generally considered as Deformation-Controlled Action according to Table C7-1, ASCE 41-17.

Mean strengths are used for Deformation-Controlled Actions according to 7.5.1.3, ASCE 41-17

Consequently:

Jacket

New material of Primary Member: Concrete Strength, $f_c = f_{cm} = 18.00$

New material of Primary Member: Steel Strength, $f_s = f_{sm} = 625.00$
 Concrete Elasticity, $E_c = 19940.411$
 Steel Elasticity, $E_s = 200000.00$
 Existing Column
 Existing material of Primary Member: Concrete Strength, $f_c = f_{cm} = 12.00$
 Existing material of Primary Member: Steel Strength, $f_s = f_{sm} = 525.00$
 Concrete Elasticity, $E_c = 16281.278$
 Steel Elasticity, $E_s = 200000.00$
 Max Height, $H_{max} = 750.00$
 Min Height, $H_{min} = 450.00$
 Max Width, $W_{max} = 950.00$
 Min Width, $W_{min} = 450.00$
 Eccentricity, $Ecc = 250.00$
 Jacket Thickness, $t_j = 100.00$
 Cover Thickness, $c = 25.00$
 Element Length, $L = 3000.00$
 Primary Member
 Ribbed Bars
 Ductile Steel
 With Detailing for Earthquake Resistance (including stirrups closed at 135°)
 Longitudinal Bars With Ends Lapped Starting at the End Sections
 Adequate Lap Length ($l_b/d \geq 1$)
 No FRP Wrapping

Stepwise Properties

Bending Moment, $M = 6.8967E+008$
 Shear Force, $V_2 = 455948.704$
 Shear Force, $V_3 = -29.64437$
 Axial Force, $F = -7.6755E+006$
 Longitudinal Reinforcement Area Distribution (in 2 divisions)
 -Tension: $A_{st} = 314.1593$
 -Compression: $A_{sc} = 6377.433$
 Longitudinal Reinforcement Area Distribution (in 3 divisions)
 -Tension: $A_{st,ten} = 1539.38$
 -Compression: $A_{sc,com} = 2475.575$
 -Middle: $A_{sc,mid} = 2676.637$
 Longitudinal External Reinforcement Area Distribution (in 3 divisions)
 -Tension: $A_{st,ten,jacket} = 1231.504$
 -Compression: $A_{sc,com,jacket} = 1859.823$
 -Middle: $A_{sc,mid,jacket} = 2060.885$
 Longitudinal Internal Reinforcement Area Distribution (in 3 divisions)
 -Tension: $A_{st,ten,core} = 307.8761$
 -Compression: $A_{sc,com,core} = 615.7522$
 -Middle: $A_{sc,mid,core} = 615.7522$
 Mean Diameter of Tension Reinforcement, $Db_L = 16.57143$

Existing component: From table 7-7, ASCE 41_17: Final chord rotation Capacity $u_{R} = * u = 0.01081173$
 $u = y + p = 0.01351466$

- Calculation of y -

$y = (M_y * L_s / 3) / E_{eff} = 0.01351466$ ((4.29), Biskinis Phd))
 $M_y = 2.0395E+009$
 $L_s = M/V$ (with $L_s > 0.1 * L$ and $L_s < 2 * L$) = 6000.00
 From table 10.5, ASCE 41_17: $E_{eff} = factor * E_c * I_g = 3.0181E+014$
 $factor = 0.70$
 $A_g = 562500.00$
 Mean concrete strength: $f_c' = (f_c'_{jacket} * Area_{jacket} + f_c'_{core} * Area_{core}) / Area_{section} = 15.20$
 $N = 7.6755E+006$
 $E_c * I_g = E_{c,jacket} * I_{g,jacket} + E_{c,core} * I_{g,core} = 4.3116E+014$

Calculation of Yielding Moment M_y

Calculation of ρ_y and M_y according to Annex 7 -

Assuming neutral axis out of flange ($\rho_y > t/d$, compression zone NOT rectangular)
extended expressions (2.1)-(2.7) of Biskinis-Fardis (2009) are adopted with:

flange width, $b = 950.00$

web width, $b_w = 450.00$

flange thickness, $t = 450.00$

$y = \text{Min}(y_{\text{ten}}, y_{\text{com}})$

$y_{\text{ten}} = 7.9835200\text{E}-006$

with $f_y = 601.9953$

$d = 707.00$

$y = 0.46672739$

$A = 0.13161899$

$B = 0.07228946$

with $p_t = 0.00229194$

$p_c = 0.00368581$

$p_v = 0.00398517$

$N = 7.6755\text{E}+006$

$b = 950.00$

$\rho = 0.06082037$

$y_{\text{comp}} = 3.5421048\text{E}-006$

with $f_c = 18.00$

$E_c = 19940.411$

$y = 0.64882874$

$A = 0.01730459$

$B = 0.03221394$

with $E_s = 200000.00$

CONFIRMATION: $y = 0.64882874 > t/d$

Calculation of ratio l_b/d

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

- Calculation of ρ_p -

From table 10-8: $\rho_p = 0.00$

with:

- Columns not controlled by inadequate development or splicing along the clear height because $l_b/d \geq 1$
shear control ratio $V_y E / V_{CoI} E = 1.74773$

$d = d_{\text{external}} = 707.00$

$s = s_{\text{external}} = 50.00$

- $t = s_1 + s_2 + 2 \cdot t_f / b_w \cdot (f_{fe} / f_s) = 0.0035764$

jacket: $s_1 = A_{v1} \cdot L_{\text{stir1}} / (s_1 \cdot A_g) = 0.00301593$

$A_{v1} = 78.53982$, is the area of every stirrup parallel to loading (shear) direction

$L_{\text{stir1}} = 2160.00$, is the total Length of all stirrups parallel to loading (shear) direction

$s_1 = 100.00$

core: $s_2 = A_{v2} \cdot L_{\text{stir2}} / (s_2 \cdot A_g) = 0.00056047$

$A_{v2} = 50.26548$, is the area of every stirrup parallel to loading (shear) direction

$L_{\text{stir2}} = 1568.00$, is the total Length of all stirrups parallel to loading (shear) direction

$s_2 = 250.00$

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

where $f = 2 \cdot t_f / b_w$ is FRP ratio (EC8 - 3, A.4.4.3(6)) and f_{fe} / f_s normalises f to steel strength

All these variables have already been given in Shear control ratio calculation.

For the normalisation f_s of jacket is used.

$N_{UD} = 7.6755\text{E}+006$

$A_g = 562500.00$

$f_{cE} = (f_{c,jacket} \cdot \text{Area}_{jacket} + f_{c,core} \cdot \text{Area}_{core}) / \text{section_area} = 15.20$

$f_{yE} = (f_{y,ext_Long_Reinf} \cdot \text{Area}_{ext_Long_Reinf} + f_{y,int_Long_Reinf} \cdot \text{Area}_{int_Long_Reinf}) / \text{Area}_{Tot_Long_Rein} = 601.9953$

$f_{tE} = (f_{y,ext_Trans_Reinf} \cdot s_1 + f_{y,int_Trans_Reinf} \cdot s_2) / (s_1 + s_2) = 609.3286$

$pl = \text{Area_Tot_Long_Rein}/(b*d) = 0.00996292$
 $b = 950.00$
 $d = 707.00$
 $f_{cE} = 15.20$

End Of Calculation of Chord Rotation Capacity for element: column JTC1 of floor 1

At local axis: 2

Integration Section: (b)

Calculation No. 7

column C1, Floor 1

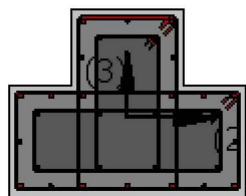
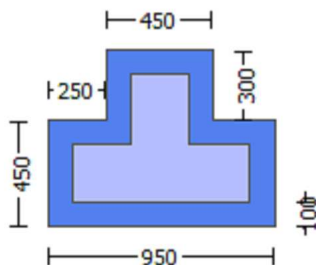
Limit State: Operational Level (data interpolation between analysis steps 39 and 40)

Analysis: Uniform +X

Check: Shear capacity VR_d

Edge: End

Local Axis: (3)



Start Of Calculation of Shear Capacity for element: column JTC1 of floor 1

At local axis: 3

Integration Section: (b)

Section Type: rcjtcs

Constant Properties

Knowledge Factor, $\gamma = 0.80$

Member Shear Force is generally considered as Force-Controlled Action according to Table C7-1, ASCE 41-17.

Lower-bound strengths are used for Force-Controlled Actions according to 7.5.1.3, ASCE 41-17

Consequently:

Jacket

New material of Primary Member: Concrete Strength, $f_c = f_{c_lower_bound} = 12.00$

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

Concrete Elasticity, $E_c = 19940.411$

Steel Elasticity, $E_s = 200000.00$


```

Existing Column
Existing material of Primary Member: Concrete Strength, fc = fc_lower_bound = 8.00
Existing material of Primary Member: Steel Strength, fs = fs_lower_bound = 420.00
Concrete Elasticity, Ec = 16281.278
Steel Elasticity, Es = 200000.00
#####
Note: Especially for the calculation of  $\mu_y$  for displacement ductility demand,
the expected (mean value) strengths are used (7.5.1.3, ASCE 41-17) because bending is considered as
Deformation-Controlled Action (Table C7-1, ASCE 41-17).
Jacket
New material: Concrete Strength, fc = fcm = 18.00
New material: Steel Strength, fs = fsm = 625.00
Existing Column
Existing material: Concrete Strength, fc = fcm = 12.00
Existing material: Steel Strength, fs = fsm = 525.00
#####
Max Height, Hmax = 750.00
Min Height, Hmin = 450.00
Max Width, Wmax = 950.00
Min Width, Wmin = 450.00
Eccentricity, Ecc = 250.00
Jacket Thickness, tj = 100.00
Cover Thickness, c = 25.00
Element Length, L = 3000.00
Primary Member
Ribbed Bars
Ductile Steel
With Detailing for Earthquake Resistance (including stirrups closed at 135°)
Longitudinal Bars With Ends Lapped Starting at the End Sections
Adequate Lap Length ( $l_o/l_{ou,min} = l_b/l_d \geq 1$ )
No FRP Wrapping
-----

Stepwise Properties
-----
EDGE -A-
Bending Moment, Ma = -1.0497E+009
Shear Force, Va = 29.64437
EDGE -B-
Bending Moment, Mb = 6.8967E+008
Shear Force, Vb = -29.64437
BOTH EDGES
Axial Force, F = -7.6755E+006
Longitudinal Reinforcement Area Distribution (in 2 divisions)
-Tension: Aslt = 314.1593
-Compression: Aslc = 6377.433
Longitudinal Reinforcement Area Distribution (in 3 divisions)
-Tension: Asl,ten = 1539.38
-Compression: Asl,com = 2475.575
-Middle: Asl,mid = 2676.637
Mean Diameter of Tension Reinforcement, DbL,ten = 16.57143
-----

Existing component: From table 7-7, ASCE 41_17: Final Shear Capacity VR =  $\phi V_n = 792832.491$ 
 $V_n$  ((10.3), ASCE 41-17) = knl*VCol = 991040.614
VCol = 991040.614
knl = 1.00
displacement_ductility_demand = 0.24893868
-----
NOTE: In expression (10-3) ' $V_s = A_v \cdot f_y \cdot d/s$ ' is replaced by ' $V_s + \phi \cdot V_f$ '
where  $V_f$  is the contribution of FRPs (11.3), ACI 440).
-----
= 1 (normal-weight concrete)
Mean concrete strength:  $f'_c = (f'_c\_jacket \cdot Area\_jacket + f'_c\_core \cdot Area\_core) / Area\_section = 10.13333$ , but  $f'_c^{0.5} \leq$ 
8.3 MPa (22.5.3.1, ACI 318-14)

```

$M/Vd = 4.00$
 $\mu_u = 6.8967E+008$
 $V_u = 29.64437$
 $d = 0.8 \cdot h = 600.00$
 $N_u = 7.6755E+006$
 $A_g = 337500.00$
 From (11.5.4.8), ACI 318-14: $V_s = V_{sjacket} + V_{s,core} = 828294.726$
 where:
 $V_{sjacket} = V_{sj1} + V_{sj2} = 753982.237$
 $V_{sj1} = 471238.898$ is calculated for section web jacket, with:
 $d = 600.00$
 $A_v = 157079.633$
 $f_y = 500.00$
 $s = 100.00$
 V_{sj1} is multiplied by $Col,j1 = 1.00$
 $s/d = 0.16666667$
 $V_{sj2} = 282743.339$ is calculated for section flange jacket, with:
 $d = 360.00$
 $A_v = 157079.633$
 $f_y = 500.00$
 $s = 100.00$
 V_{sj2} is multiplied by $Col,j2 = 1.00$
 $s/d = 0.27777778$
 $V_{s,core} = V_{s,c1} + V_{s,c2} = 74312.489$
 $V_{s,c1} = 74312.489$ is calculated for section web core, with:
 $d = 440.00$
 $A_v = 100530.965$
 $f_y = 420.00$
 $s = 250.00$
 $V_{s,c1}$ is multiplied by $Col,c1 = 1.00$
 $s/d = 0.56818182$
 $V_{s,c2} = 0.00$ is calculated for section flange core, with:
 $d = 200.00$
 $A_v = 100530.965$
 $f_y = 420.00$
 $s = 250.00$
 $V_{s,c2}$ is multiplied by $Col,c2 = 0.00$
 $s/d = 1.25$
 $V_f ((11-3)-(11.4), ACI 440) = 0.00$
 From (11-11), ACI 440: $V_s + V_f \leq 570961.316$
 $bw = 450.00$

displacement ductility demand is calculated as δ_u / y

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

From analysis, chord rotation $\theta_r = 0.00336432$
 $y = (M_y \cdot L_s / 3) / E_{eff} = 0.01351466$ ((4.29), Biskinis Phd))
 $M_y = 2.0395E+009$
 $L_s = M/V$ (with $L_s > 0.1 \cdot L$ and $L_s < 2 \cdot L$) = 6000.00
 From table 10.5, ASCE 41_17: $E_{eff} = factor \cdot E_c \cdot I_g = 3.0181E+014$
 $factor = 0.70$
 $A_g = 562500.00$
 Mean concrete strength: $f'_c = (f'_{c,jacket} \cdot Area_{jacket} + f'_{c,core} \cdot Area_{core}) / Area_{section} = 15.20$
 $N = 7.6755E+006$
 $E_c \cdot I_g = E_{c,jacket} \cdot I_{g,jacket} + E_{c,core} \cdot I_{g,core} = 4.3116E+014$

Calculation of Yielding Moment M_y

Calculation of δ_u and M_y according to Annex 7 -

Assuming neutral axis out of flange ($\delta_u > t/d$, compression zone NOT rectangular)

extended expressions (2.1)-(2.7) of Biskinis-Fardis (2009) are adopted with:

flange width, $b = 950.00$

web width, $b_w = 450.00$

flange thickness, $t = 450.00$

$y = \text{Min}(y_{\text{ten}}, y_{\text{com}})$

$y_{\text{ten}} = 7.9835200\text{E-}006$

with $f_y = 601.9953$

$d = 707.00$

$y = 0.46672739$

$A = 0.13161899$

$B = 0.07228946$

with $p_t = 0.00229194$

$p_c = 0.00368581$

$p_v = 0.00398517$

$N = 7.6755\text{E+}006$

$b = 950.00$

" = 0.06082037

$y_{\text{comp}} = 3.5421048\text{E-}006$

with $f_c = 18.00$

$E_c = 19940.411$

$y = 0.64882874$

$A = 0.01730459$

$B = 0.03221394$

with $E_s = 200000.00$

CONFIRMATION: $y = 0.64882874 > t/d$

Calculation of ratio l_b/d

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

End Of Calculation of Shear Capacity for element: column JTC1 of floor 1

At local axis: 3

Integration Section: (b)

Calculation No. 8

column C1, Floor 1

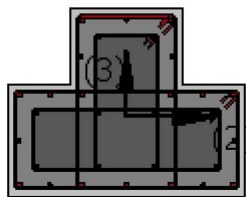
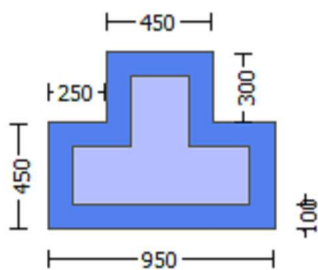
Limit State: Operational Level (data interpolation between analysis steps 39 and 40)

Analysis: Uniform +X

Check: Chord rotation capacity (ϕ)

Edge: End

Local Axis: (3)



Start Of Calculation of Shear Capacity ratio for element: column JTC1 of floor 1

At Shear local axis: 3

(Bending local axis: 2)

Section Type: rcjtcs

Constant Properties

Knowledge Factor, $\gamma = 0.80$

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

Consequently:

Jacket

New material of Primary Member: Concrete Strength, $f_c = f_{cm} = 18.00$

New material of Primary Member: Steel Strength, $f_s = f_{sm} = 625.00$

Concrete Elasticity, $E_c = 19940.411$

Steel Elasticity, $E_s = 200000.00$

Existing Column

Existing material of Primary Member: Concrete Strength, $f_c = f_{cm} = 12.00$

Existing material of Primary Member: Steel Strength, $f_s = f_{sm} = 525.00$

Concrete Elasticity, $E_c = 16281.278$

Steel Elasticity, $E_s = 200000.00$

#####

Note: Especially for the calculation of moment strengths,

the above steel re-bar strengths are multiplied by 1.25 according to R18.6.5, ACI 318-14

Jacket

New material: Steel Strength, $f_s = 1.25 \cdot f_{sm} = 781.25$

Existing Column

Existing material: Steel Strength, $f_s = 1.25 \cdot f_{sm} = 656.25$

#####

Max Height, $H_{max} = 750.00$

Min Height, $H_{min} = 450.00$

Max Width, $W_{max} = 950.00$

Min Width, $W_{min} = 450.00$

Eccentricity, $E_{cc} = 250.00$

Jacket Thickness, $t_j = 100.00$

Cover Thickness, $c = 25.00$

Mean Confinement Factor overall section = 1.42131

Element Length, $L = 3000.00$

Primary Member

Ribbed Bars

Ductile Steel

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

Longitudinal Bars With Ends Lapped Starting at the End Sections

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

No FRP Wrapping

Stepwise Properties

At local axis: 3

EDGE -A-

Shear Force, $V_a = -227.4778$

EDGE -B-

Shear Force, $V_b = 227.4778$
 BOTH EDGES
 Axial Force, $F = -7.6839E+006$
 Longitudinal Reinforcement Area Distribution (in 2 divisions)
 -Tension: $As_t = 314.1593$
 -Compression: $As_c = 6176.371$
 Longitudinal Reinforcement Area Distribution (in 3 divisions)
 -Tension: $As_{t,ten} = 1539.38$
 -Compression: $As_{c,com} = 2475.575$
 -Middle: $As_{c,mid} = 2676.637$

Calculation of Shear Capacity ratio, $V_e/V_r = 1.74773$
 Member Controlled by Shear ($V_e/V_r > 1$)
 Calculation of Shear Demand from fig. R18.6.5, ACI 318-14 $V_e = (M_{pr1} + M_{pr2})/l_n = 2.0410E+006$
 with
 $M_{pr1} = \text{Max}(\mu_{u1+}, \mu_{u1-}) = 3.0615E+009$
 $\mu_{u1+} = 3.0615E+009$, is the ultimate moment strength at the edge 1 of the member in the actual moment direction which is defined for the static loading combination
 $\mu_{u1-} = 8.9683E+008$, is the ultimate moment strength at the edge 1 of the member in the opposite moment direction which is defined for the static loading combination
 $M_{pr2} = \text{Max}(\mu_{u2+}, \mu_{u2-}) = 3.0615E+009$
 $\mu_{u2+} = 3.0615E+009$, is the ultimate moment strength at the edge 2 of the member in the actual moment direction which is defined for the static loading combination
 $\mu_{u2-} = 8.9683E+008$, is the ultimate moment strength at the edge 2 of the member in the opposite moment direction which is defined for the static loading combination

Calculation of μ_{u1+}

Calculation of ultimate curvature ϕ_u according to 4.1, Biskinis/Fardis 2013:
 $\phi_u = 1.3930706E-005$
 $\mu_u = 3.0615E+009$

with full section properties:

$b = 950.00$
 $d = 707.00$
 $d' = 43.00$
 $v = 0.63557247$
 $N = 7.6839E+006$
 $f_c = 18.00$
 $\alpha = (5A_s, TBDY) = 0.002$

Final value of ϕ_u : $\phi_u^* = \text{shear_factor} * \text{Max}(\phi_u, \alpha) = 0.01486841$

The Shear_factor is considered equal to 1 (pure moment strength)

From (5.4b), TBDY: $\phi_u = 0.01486841$

we (5.4c) $= 0.08077545$

$\alpha_{se}((5.4d), TBDY) = (\alpha_{se1} * A_{ext} + \alpha_{se2} * A_{int}) / A_{sec} = 0.53375773$

$\alpha_{se1} = \text{Max}(((A_{conf,max1} - A_{noConf1}) / A_{conf,max1}) * (A_{conf,min1} / A_{conf,max1}), 0) = 0.53375773$

The definitions of A_{noConf} , $A_{conf,min}$ and $A_{conf,max}$ are derived from generalization of the rectangular sections confinement, which is expressed by (5.4d).

The generalization is done according to Mander, J., Priestley, M., and Park, R. (1988)

"Theoretical Stress-Strain Model for Confined Concrete."

J. Struct. Eng., 10.1061/(ASCE)0733-9445(1988)114:8(1804), 1804-1826.

$A_{conf,max1} = 464100.00$ is the confined core area at levels of member with hoops and is calculated as the area of core enclosed by the center lines of the external perimeter hoops.

$A_{conf,min1} = 395025.00$ is the confined core area at midway between the levels of hoops and is calculated by reducing all the dimensions of the area $A_{conf,max1}$ by a length equal to half the clear spacing between external hoops.

$A_{noConf1} = 173066.667$ is the unconfined external core area which is equal to $b_i^2/6$ as defined at (A.2).

$\alpha_{se2} (>= \alpha_{se1}) = \text{Max}(((A_{conf,max2} - A_{noConf2}) / A_{conf,max2}) * (A_{conf,min2} / A_{conf,max2}), 0) = 0.53375773$

The definitions of A_{noConf} , $A_{conf,min}$ and $A_{conf,max}$ are derived from generalization of the rectangular sections confinement, which is expressed by (5.4d).

The generalization is done according to Mander, J., Priestley, M., and Park, R. (1988)

"Theoretical Stress-Strain Model for Confined Concrete."

J. Struct. Eng., 10.1061/(ASCE)0733-9445(1988)114:8(1804), 1804-1826.

$A_{conf,max2} = 252164.00$ is the confined core area at levels of member with hoops and is calculated as the area of core enclosed by the center lines of the internal perimeter hoops.

$A_{conf,min2} = 111441.00$ is the confined core area at midway between the levels of hoops and is calculated by reducing all the dimensions of the area $A_{conf,max2}$ by a length equal to half the clear spacing between internal hoops.

$A_{noConf2} = 110709.333$ is the unconfined internal core area which is equal to $b_i^2/6$ as defined at (A.2).
 $psh_{min} \cdot F_{ywe} = \text{Min}(psh_x \cdot F_{ywe}, psh_y \cdot F_{ywe}) = 2.724$

$psh_x \cdot F_{ywe} = psh1 \cdot F_{ywe1} + ps2 \cdot F_{ywe2} = 2.724$

$psh1 ((5.4d), TBDY) = L_{stir1} \cdot A_{stir1} / (A_{sec} \cdot s1) = 0.00301593$

L_{stir1} (Length of stirrups along Y) = 2160.00

A_{stir1} (stirrups area) = 78.53982

$psh2 (5.4d) = L_{stir2} \cdot A_{stir2} / (A_{sec} \cdot s2) = 0.00056047$

L_{stir2} (Length of stirrups along Y) = 1568.00

A_{stir2} (stirrups area) = 50.26548

$psh_y \cdot F_{ywe} = psh1 \cdot F_{ywe1} + ps2 \cdot F_{ywe2} = 3.25416$

$psh1 ((5.4d), TBDY) = L_{stir1} \cdot A_{stir1} / (A_{sec} \cdot s1) = 0.00357443$

L_{stir1} (Length of stirrups along X) = 2560.00

A_{stir1} (stirrups area) = 78.53982

$psh2 ((5.4d), TBDY) = L_{stir2} \cdot A_{stir2} / (A_{sec} \cdot s2) = 0.00070345$

L_{stir2} (Length of stirrups along X) = 1968.00

A_{stir2} (stirrups area) = 50.26548

$A_{sec} = 562500.00$

$s1 = 100.00$

$s2 = 250.00$

$f_{ywe1} = 781.25$

$f_{ywe2} = 656.25$

$f_{ce} = 18.00$

From ((5.A5), TBDY), TBDY: $cc = 0.00621307$

c = confinement factor = 1.42131

$y1 = 0.0025$

$sh1 = 0.008$

$ft1 = 907.50$

$fy1 = 756.25$

$su1 = 0.032$

using (30) in Biskinis/Fardis (2013) multiplied with shear_factor and also multiplied by the shear_factor according to 15.7.1.4, with

Shear_factor = 1.00

$lo/lo_{min} = lb/l_d = 1.00$

$su1 = 0.4 \cdot esu1_{nominal} ((5.5), TBDY) = 0.032$

From table 5A.1, TBDY: $esu1_{nominal} = 0.08$,

For calculation of $esu1_{nominal}$ and $y1, sh1, ft1, fy1$, it is considered characteristic value $fsy1 = fs1/1.2$, from table 5.1, TBDY.

$y1, sh1, ft1, fy1$, are also multiplied by $\text{Min}(1, 1.25 \cdot (lb/l_d)^{2/3})$, from 10.3.5, ASCE 41-17.

with $fs1 = (fs_{jacket} \cdot A_{sl,ten,jacket} + fs_{core} \cdot A_{sl,ten,core}) / A_{sl,ten} = 756.25$

with $Es1 = (Es_{jacket} \cdot A_{sl,ten,jacket} + Es_{core} \cdot A_{sl,ten,core}) / A_{sl,ten} = 200000.00$

$y2 = 0.0025$

$sh2 = 0.008$

$ft2 = 900.1904$

$fy2 = 750.1586$

$su2 = 0.032$

using (30) in Biskinis/Fardis (2013) multiplied with shear_factor and also multiplied by the shear_factor according to 15.7.1.4, with Shear_factor = 1.00

$lo/lo_{min} = lb/l_b, min = 1.00$

$su2 = 0.4 \cdot esu2_{nominal} ((5.5), TBDY) = 0.032$

From table 5A.1, TBDY: $esu2_{nominal} = 0.08$,

For calculation of $esu2_{nominal}$ and $y2, sh2, ft2, fy2$, it is considered characteristic value $fsy2 = fs2/1.2$, from table 5.1, TBDY.

$y1, sh1, ft1, fy1$, are also multiplied by $\text{Min}(1, 1.25 \cdot (lb/l_d)^{2/3})$, from 10.3.5, ASCE 41-17.

with $fs2 = (fs_{jacket} \cdot A_{sl,com,jacket} + fs_{core} \cdot A_{sl,com,core}) / A_{sl,com} = 750.1586$

with $Es2 = (Es_{jacket} \cdot A_{sl,com,jacket} + Es_{core} \cdot A_{sl,com,core}) / A_{sl,com} = 200000.00$

$yv = 0.0025$

```

shv = 0.008
ftv = 902.993
fyv = 752.4941
suv = 0.032
    using (30) in Biskinis/Fardis (2013) multiplied with shear_factor
    and also multiplied by the shear_factor according to 15.7.1.4, with
    Shear_factor = 1.00
    lo/lou,min = lb/ld = 1.00
    suv = 0.4*esuv_nominal ((5.5), TBDY) = 0.032
    From table 5A.1, TBDY: esuv_nominal = 0.08,
    considering characteristic value fsyv = fsv/1.2, from table 5.1, TBDY
    For calculation of esuv_nominal and yv, shv,ftv,fyv, it is considered
    characteristic value fsyv = fsv/1.2, from table 5.1, TBDY.
    y1, sh1,ft1,fy1, are also multiplied by Min(1,1.25*(lb/ld)^2/3), from 10.3.5, ASCE 41-17.
    with fsv = (fs,jacket*Asl,mid,jacket + fs,mid*Asl,mid,core)/Asl,mid = 752.4941
    with Esv = (Es,jacket*Asl,mid,jacket + Es,mid*Asl,mid,core)/Asl,mid = 200000.00
    1 = Asl,ten/(b*d)*(fs1/fc) = 0.09629324
    2 = Asl,com/(b*d)*(fs2/fc) = 0.15360794
    v = Asl,mid/(b*d)*(fsv/fc) = 0.16660079
and confined core properties:
b = 890.00
d = 677.00
d' = 13.00
fcc (5A.2, TBDY) = 25.58352
cc (5A.5, TBDY) = 0.00621307
    c = confinement factor = 1.42131
    1 = Asl,ten/(b*d)*(fs1/fc) = 0.10733965
    2 = Asl,com/(b*d)*(fs2/fc) = 0.17122928
    v = Asl,mid/(b*d)*(fsv/fc) = 0.18571261
Case/Assumption: Unconfined full section - Steel rupture
' satisfies Eq. (4.3)
---->
v < vs,y2 - LHS eq.(4.5) is not satisfied
---->
v < vs,c - RHS eq.(4.5) is not satisfied
---->
Case/Assumption Rejected.
---->
New Case/Assumption: Unconfined full section - Spalling of concrete cover
' satisfies Eq. (4.4)
---->
v < s,y1 - LHS eq.(4.7) is not satisfied
---->
v < vc,y1 - RHS eq.(4.6) is satisfied
---->
cu (4.10) = 0.58509201
MRc (4.17) = 2.3801E+009
---->
New Case/Assumption: Ultimate curvature of confined core after spalling of concrete cover
In expressions below, the following modifications have been made
- b, d, d' replaced by geometric parameters of the core: bo, do, d'o
- N, 1, 2, v normalised to bo*do, instead of b*d
- parameters of confined concrete, fcc, cc, used in lieu of fc, ecu
---->
Subcase: Rupture of tension steel
---->
v* < v*s,y2 - LHS eq.(4.5) is not satisfied
---->
v* < v*s,c - LHS eq.(4.5) is not satisfied
---->
Subcase rejected
---->
New Subcase: Failure of compression zone
---->
v* < v*c,y2 - LHS eq.(4.6) is not satisfied
---->
v* < v*c,y1 - RHS eq.(4.6) is not satisfied

```

```

--->
*cu (4.11) = 0.65878578
MRo (4.18) = 3.0615E+009
--->
u = cu (4.2) = 1.3930706E-005
Mu = MRo
-----

Calculation of ratio lb/ld
-----

Adequate Lap Length: lb/ld >= 1
-----

Calculation of Mu1-
-----

Calculation of ultimate curvature u according to 4.1, Biskinis/Fardis 2013:
u = 9.1773508E-006
Mu = 8.9683E+008
-----

with full section properties:
b = 450.00
d = 707.00
d' = 43.00
v = 1.34176
N = 7.6839E+006
fc = 18.00
co (5A.5, TBDY) = 0.002
Final value of cu: cu* = shear_factor * Max( cu, cc) = 0.01486841
The Shear_factor is considered equal to 1 (pure moment strength)
From (5.4b), TBDY: cu = 0.01486841
we (5.4c) = 0.08077545
ase ((5.4d), TBDY) = (ase1*Aext+ase2*Aint)/Asec = 0.53375773
ase1 = Max(((Aconf,max1-AnoConf1)/Aconf,max1)*(Aconf,min1/Aconf,max1),0) = 0.53375773
The definitions of AnoConf, Aconf,min and Aconf,max are derived from generalization
of the rectangular sections confinement, which is expressed by (5.4d).
The generalization is done according to Mander, J., Priestley, M., and Park, R. (1988)
"Theoretical Stress-Strain Model for Confined Concrete."
J. Struct. Eng., 10.1061/(ASCE)0733-9445(1988)114:8(1804), 1804-1826.
Aconf,max1 = 464100.00 is the confined core area at levels of member with hoops and
is calculated as the area of core enclosed by the center lines of the external perimeter hoops.
Aconf,min1 = 395025.00 is the confined core area at midway between the levels of hoops and
is calculated by reducing all the dimensions of the area Aconf,max1 by a length
equal to half the clear spacing between external hoops.
AnoConf1 = 173066.667 is the unconfined external core area which is equal to bi2/6 as defined at (A.2).
ase2 (>=ase1) = Max(((Aconf,max2-AnoConf2)/Aconf,max2)*(Aconf,min2/Aconf,max2),0) = 0.53375773
The definitions of AnoConf, Aconf,min and Aconf,max are derived from generalization
of the rectangular sections confinement, which is expressed by (5.4d).
The generalization is done according to Mander, J., Priestley, M., and Park, R. (1988)
"Theoretical Stress-Strain Model for Confined Concrete."
J. Struct. Eng., 10.1061/(ASCE)0733-9445(1988)114:8(1804), 1804-1826.
Aconf,max2 = 252164.00 is the confined core area at levels of member with hoops and
is calculated as the area of core enclosed by the center lines of the internal perimeter hoops.
Aconf,min2 = 111441.00 is the confined core area at midway between the levels of hoops and
is calculated by reducing all the dimensions of the area Aconf,max2 by a length
equal to half the clear spacing between internal hoops.
AnoConf2 = 110709.333 is the unconfined internal core area which is equal to bi2/6 as defined at (A.2).
psh,min*Fywe = Min(psh,x*Fywe , psh,y*Fywe) = 2.724
-----

psh_x*Fywe = psh1*Fywe1+ps2*Fywe2 = 2.724
psh1 ((5.4d), TBDY) = Lstir1*Astir1/(Asec*s1) = 0.00301593
Lstir1 (Length of stirrups along Y) = 2160.00
Astir1 (stirrups area) = 78.53982
psh2 (5.4d) = Lstir2*Astir2/(Asec*s2) = 0.00056047
Lstir2 (Length of stirrups along Y) = 1568.00

```


Astir2 (stirrups area) = 50.26548

psh_y*Fywe = psh1*Fywe1+ps2*Fywe2 = 3.25416
psh1 ((5.4d), TBDY) = Lstir1*Astir1/(Asec*s1) = 0.00357443
Lstir1 (Length of stirrups along X) = 2560.00
Astir1 (stirrups area) = 78.53982
psh2 ((5.4d), TBDY) = Lstir2*Astir2/(Asec*s2) = 0.00070345
Lstir2 (Length of stirrups along X) = 1968.00
Astir2 (stirrups area) = 50.26548

Asec = 562500.00

s1 = 100.00

s2 = 250.00

fywe1 = 781.25

fywe2 = 656.25

fce = 18.00

From ((5.A.5), TBDY), TBDY: cc = 0.00621307

c = confinement factor = 1.42131

y1 = 0.0025

sh1 = 0.008

ft1 = 900.1904

fy1 = 750.1586

su1 = 0.032

using (30) in Biskinis/Fardis (2013) multiplied with shear_factor
and also multiplied by the shear_factor according to 15.7.1.4, with
Shear_factor = 1.00

lo/lou,min = lb/ld = 1.00

su1 = 0.4*esu1_nominal ((5.5), TBDY) = 0.032

From table 5A.1, TBDY: esu1_nominal = 0.08,

For calculation of esu1_nominal and y1, sh1,ft1,fy1, it is considered
characteristic value fsy1 = fs1/1.2, from table 5.1, TBDY.

y1, sh1,ft1,fy1, are also multiplied by Min(1,1.25*(lb/ld)^ 2/3), from 10.3.5, ASCE 41-17.

with fs1 = (fs,jacket*Asl,ten,jacket + fs,core*Asl,ten,core)/Asl,ten = 750.1586

with Es1 = (Es,jacket*Asl,ten,jacket + Es,core*Asl,ten,core)/Asl,ten = 200000.00

y2 = 0.0025

sh2 = 0.008

ft2 = 907.50

fy2 = 756.25

su2 = 0.032

using (30) in Biskinis/Fardis (2013) multiplied with shear_factor
and also multiplied by the shear_factor according to 15.7.1.4, with
Shear_factor = 1.00

lo/lou,min = lb/lb,min = 1.00

su2 = 0.4*esu2_nominal ((5.5), TBDY) = 0.032

From table 5A.1, TBDY: esu2_nominal = 0.08,

For calculation of esu2_nominal and y2, sh2,ft2,fy2, it is considered
characteristic value fsy2 = fs2/1.2, from table 5.1, TBDY.

y1, sh1,ft1,fy1, are also multiplied by Min(1,1.25*(lb/ld)^ 2/3), from 10.3.5, ASCE 41-17.

with fs2 = (fs,jacket*Asl,com,jacket + fs,core*Asl,com,core)/Asl,com = 756.25

with Es2 = (Es,jacket*Asl,com,jacket + Es,core*Asl,com,core)/Asl,com = 200000.00

yv = 0.0025

shv = 0.008

ftv = 902.993

fyv = 752.4941

suv = 0.032

using (30) in Biskinis/Fardis (2013) multiplied with shear_factor
and also multiplied by the shear_factor according to 15.7.1.4, with
Shear_factor = 1.00

lo/lou,min = lb/ld = 1.00

suv = 0.4*esuv_nominal ((5.5), TBDY) = 0.032

From table 5A.1, TBDY: esuv_nominal = 0.08,

considering characteristic value fsyv = fsv/1.2, from table 5.1, TBDY

For calculation of esuv_nominal and yv, shv,ftv,fyv, it is considered
characteristic value fsyv = fsv/1.2, from table 5.1, TBDY.

y1, sh1,ft1,fy1, are also multiplied by Min(1,1.25*(lb/ld)^ 2/3), from 10.3.5, ASCE 41-17.

with fsv = (fs,jacket*Asl,mid,jacket + fs,mid*Asl,mid,core)/Asl,mid = 752.4941

with Esv = (Es,jacket*Asl,mid,jacket + Es,mid*Asl,mid,core)/Asl,mid = 200000.00

```

1 = Asl,ten/(b*d)*(fs1/fc) = 0.32428344
2 = Asl,com/(b*d)*(fs2/fc) = 0.20328574
v = Asl,mid/(b*d)*(fsv/fc) = 0.35171278
and confined core properties:
b = 390.00
d = 677.00
d' = 13.00
fcc (5A.2, TBDY) = 25.58352
cc (5A.5, TBDY) = 0.00621307
c = confinement factor = 1.42131
1 = Asl,ten/(b*d)*(fs1/fc) = 0.39075399
2 = Asl,com/(b*d)*(fs2/fc) = 0.24495458
v = Asl,mid/(b*d)*(fsv/fc) = 0.42380571
Case/Assumption: Unconfined full section - Steel rupture
' satisfies Eq. (4.3)
---->
v < vs,y2 - LHS eq.(4.5) is not satisfied
---->
v < vs,c - RHS eq.(4.5) is not satisfied
---->
Case/Assumption Rejected.
---->
New Case/Assumption: Unconfined full section - Spalling of concrete cover
' satisfies Eq. (4.4)
---->
v < s,y1 - LHS eq.(4.7) is not satisfied
---->
v < vc,y1 - RHS eq.(4.6) is not satisfied
---->
cu (4.11) = 0.95920575
MRc (4.18) = 7.7321E+008
---->
New Case/Assumption: Ultimate curvature of confined core after spalling of concrete cover
In expressions below, the following modifications have been made
- b, d, d' replaced by geometric parameters of the core: bo, do, d'o
- N, 1, 2, v normalised to bo*do, instead of b*d
- - parameters of confined concrete, fcc, cc, used in lieu of fc, ecu
---->
Subcase: Rupture of tension steel
---->
v* < v*s,y2 - LHS eq.(4.5) is not satisfied
---->
v* < v*s,c - LHS eq.(4.5) is not satisfied
---->
Subcase rejected
---->
New Subcase: Failure of compression zone
---->
v* < v*c,y2 - LHS eq.(4.6) is not satisfied
---->
v* < v*c,y1 - RHS eq.(4.6) is not satisfied
---->
*cu (4.11) = 1.00
MRo (4.18) = 8.9683E+008
---->
u = cu (4.2) = 9.1773508E-006
Mu = MRo

```

Calculation of ratio lb/d

Adequate Lap Length: lb/d >= 1

Calculation of Mu2+

Calculation of ultimate curvature ϕ_u according to 4.1, Biskinis/Fardis 2013:

$$\phi_u = 1.3930706E-005$$

$$\mu_u = 3.0615E+009$$

with full section properties:

$$b = 950.00$$

$$d = 707.00$$

$$d' = 43.00$$

$$\nu = 0.63557247$$

$$N = 7.6839E+006$$

$$f_c = 18.00$$

$$\phi_{co} (5A.5, TBDY) = 0.002$$

$$\text{Final value of } \phi_{cu}: \phi_{cu}^* = \text{shear_factor} * \text{Max}(\phi_{cu}, \phi_{cc}) = 0.01486841$$

The Shear_factor is considered equal to 1 (pure moment strength)

$$\text{From (5.4b), TBDY: } \phi_{cu} = 0.01486841$$

$$\phi_{we} (5.4c) = 0.08077545$$

$$\phi_{ase} ((5.4d), TBDY) = (\phi_{ase1} * A_{ext} + \phi_{ase2} * A_{int}) / A_{sec} = 0.53375773$$

$$\phi_{ase1} = \text{Max}(((A_{conf,max1} - A_{noConf1}) / A_{conf,max1}) * (A_{conf,min1} / A_{conf,max1}), 0) = 0.53375773$$

The definitions of A_{noConf} , $A_{conf,min}$ and $A_{conf,max}$ are derived from generalization of the rectangular sections confinement, which is expressed by (5.4d).

The generalization is done according to Mander, J., Priestley, M., and Park, R. (1988)

"Theoretical Stress-Strain Model for Confined Concrete."

J. Struct. Eng., 10.1061/(ASCE)0733-9445(1988)114:8(1804), 1804-1826.

$A_{conf,max1} = 464100.00$ is the confined core area at levels of member with hoops and is calculated as the area of core enclosed by the center lines of the external perimeter hoops.

$A_{conf,min1} = 395025.00$ is the confined core area at midway between the levels of hoops and is calculated by reducing all the dimensions of the area $A_{conf,max1}$ by a length equal to half the clear spacing between external hoops.

$A_{noConf1} = 173066.667$ is the unconfined external core area which is equal to $b_i^2/6$ as defined at (A.2).

$$\phi_{ase2} (> \phi_{ase1}) = \text{Max}(((A_{conf,max2} - A_{noConf2}) / A_{conf,max2}) * (A_{conf,min2} / A_{conf,max2}), 0) = 0.53375773$$

The definitions of A_{noConf} , $A_{conf,min}$ and $A_{conf,max}$ are derived from generalization of the rectangular sections confinement, which is expressed by (5.4d).

The generalization is done according to Mander, J., Priestley, M., and Park, R. (1988)

"Theoretical Stress-Strain Model for Confined Concrete."

J. Struct. Eng., 10.1061/(ASCE)0733-9445(1988)114:8(1804), 1804-1826.

$A_{conf,max2} = 252164.00$ is the confined core area at levels of member with hoops and is calculated as the area of core enclosed by the center lines of the internal perimeter hoops.

$A_{conf,min2} = 111441.00$ is the confined core area at midway between the levels of hoops and is calculated by reducing all the dimensions of the area $A_{conf,max2}$ by a length equal to half the clear spacing between internal hoops.

$A_{noConf2} = 110709.333$ is the unconfined internal core area which is equal to $b_i^2/6$ as defined at (A.2).

$$\phi_{psh,min} * F_{ywe} = \text{Min}(\phi_{psh,x} * F_{ywe}, \phi_{psh,y} * F_{ywe}) = 2.724$$

$$\phi_{psh,x} * F_{ywe} = \phi_{psh1} * F_{ywe1} + \phi_{ps2} * F_{ywe2} = 2.724$$

$$\phi_{psh1} ((5.4d), TBDY) = L_{stir1} * A_{stir1} / (A_{sec} * s_1) = 0.00301593$$

$$L_{stir1} \text{ (Length of stirrups along Y)} = 2160.00$$

$$A_{stir1} \text{ (stirrups area)} = 78.53982$$

$$\phi_{psh2} (5.4d) = L_{stir2} * A_{stir2} / (A_{sec} * s_2) = 0.00056047$$

$$L_{stir2} \text{ (Length of stirrups along Y)} = 1568.00$$

$$A_{stir2} \text{ (stirrups area)} = 50.26548$$

$$\phi_{psh,y} * F_{ywe} = \phi_{psh1} * F_{ywe1} + \phi_{ps2} * F_{ywe2} = 3.25416$$

$$\phi_{psh1} ((5.4d), TBDY) = L_{stir1} * A_{stir1} / (A_{sec} * s_1) = 0.00357443$$

$$L_{stir1} \text{ (Length of stirrups along X)} = 2560.00$$

$$A_{stir1} \text{ (stirrups area)} = 78.53982$$

$$\phi_{psh2} ((5.4d), TBDY) = L_{stir2} * A_{stir2} / (A_{sec} * s_2) = 0.00070345$$

$$L_{stir2} \text{ (Length of stirrups along X)} = 1968.00$$

$$A_{stir2} \text{ (stirrups area)} = 50.26548$$

$$A_{sec} = 562500.00$$

$$s_1 = 100.00$$

$$s_2 = 250.00$$

$$F_{ywe1} = 781.25$$

$$F_{ywe2} = 656.25$$

$$f_{ce} = 18.00$$

From ((5.A.5), TBDY), TBDY: $cc = 0.00621307$
 $c = \text{confinement factor} = 1.42131$
 $y1 = 0.0025$
 $sh1 = 0.008$
 $ft1 = 907.50$
 $fy1 = 756.25$
 $su1 = 0.032$
 using (30) in Biskinis/Fardis (2013) multiplied with shear_factor
 and also multiplied by the shear_factor according to 15.7.1.4, with
 $\text{Shear_factor} = 1.00$
 $lo/lou, \min = lb/ld = 1.00$
 $su1 = 0.4 * esu1_nominal ((5.5), TBDY) = 0.032$
 From table 5A.1, TBDY: $esu1_nominal = 0.08$,
 For calculation of $esu1_nominal$ and $y1, sh1, ft1, fy1$, it is considered
 characteristic value $fsy1 = fs1/1.2$, from table 5.1, TBDY.
 $y1, sh1, ft1, fy1$, are also multiplied by $\text{Min}(1, 1.25 * (lb/ld)^{2/3})$, from 10.3.5, ASCE 41-17.
 with $fs1 = (fs_jacket * Asl, \text{ten}, \text{jacket} + fs_core * Asl, \text{ten}, \text{core}) / Asl, \text{ten} = 756.25$
 with $Es1 = (Es_jacket * Asl, \text{ten}, \text{jacket} + Es_core * Asl, \text{ten}, \text{core}) / Asl, \text{ten} = 200000.00$
 $y2 = 0.0025$
 $sh2 = 0.008$
 $ft2 = 900.1904$
 $fy2 = 750.1586$
 $su2 = 0.032$
 using (30) in Biskinis/Fardis (2013) multiplied with shear_factor
 and also multiplied by the shear_factor according to 15.7.1.4, with
 $\text{Shear_factor} = 1.00$
 $lo/lou, \min = lb/lb, \min = 1.00$
 $su2 = 0.4 * esu2_nominal ((5.5), TBDY) = 0.032$
 From table 5A.1, TBDY: $esu2_nominal = 0.08$,
 For calculation of $esu2_nominal$ and $y2, sh2, ft2, fy2$, it is considered
 characteristic value $fsy2 = fs2/1.2$, from table 5.1, TBDY.
 $y1, sh1, ft1, fy1$, are also multiplied by $\text{Min}(1, 1.25 * (lb/ld)^{2/3})$, from 10.3.5, ASCE 41-17.
 with $fs2 = (fs_jacket * Asl, \text{com}, \text{jacket} + fs_core * Asl, \text{com}, \text{core}) / Asl, \text{com} = 750.1586$
 with $Es2 = (Es_jacket * Asl, \text{com}, \text{jacket} + Es_core * Asl, \text{com}, \text{core}) / Asl, \text{com} = 200000.00$
 $yv = 0.0025$
 $shv = 0.008$
 $ftv = 902.993$
 $fyv = 752.4941$
 $suv = 0.032$
 using (30) in Biskinis/Fardis (2013) multiplied with shear_factor
 and also multiplied by the shear_factor according to 15.7.1.4, with
 $\text{Shear_factor} = 1.00$
 $lo/lou, \min = lb/ld = 1.00$
 $suv = 0.4 * esuv_nominal ((5.5), TBDY) = 0.032$
 From table 5A.1, TBDY: $esuv_nominal = 0.08$,
 considering characteristic value $fsyv = fsv/1.2$, from table 5.1, TBDY
 For calculation of $esuv_nominal$ and yv, shv, ftv, fyv , it is considered
 characteristic value $fsyv = fsv/1.2$, from table 5.1, TBDY.
 $y1, sh1, ft1, fy1$, are also multiplied by $\text{Min}(1, 1.25 * (lb/ld)^{2/3})$, from 10.3.5, ASCE 41-17.
 with $fsv = (fs_jacket * Asl, \text{mid}, \text{jacket} + fs_mid * Asl, \text{mid}, \text{core}) / Asl, \text{mid} = 752.4941$
 with $Es_v = (Es_jacket * Asl, \text{mid}, \text{jacket} + Es_mid * Asl, \text{mid}, \text{core}) / Asl, \text{mid} = 200000.00$
 $1 = Asl, \text{ten} / (b * d) * (fs1 / fc) = 0.09629324$
 $2 = Asl, \text{com} / (b * d) * (fs2 / fc) = 0.15360794$
 $v = Asl, \text{mid} / (b * d) * (fsv / fc) = 0.16660079$
 and confined core properties:
 $b = 890.00$
 $d = 677.00$
 $d' = 13.00$
 $fcc (5A.2, TBDY) = 25.58352$
 $cc (5A.5, TBDY) = 0.00621307$
 $c = \text{confinement factor} = 1.42131$
 $1 = Asl, \text{ten} / (b * d) * (fs1 / fc) = 0.10733965$
 $2 = Asl, \text{com} / (b * d) * (fs2 / fc) = 0.17122928$
 $v = Asl, \text{mid} / (b * d) * (fsv / fc) = 0.18571261$
 Case/Assumption: Unconfined full section - Steel rupture
 ' satisfies Eq. (4.3)

$v < v_{s,y2}$ - LHS eq.(4.5) is not satisfied
 --->
 $v < v_{s,c}$ - RHS eq.(4.5) is not satisfied
 --->
 Case/Assumption Rejected.
 --->
 New Case/Assumption: Unconfined full section - Spalling of concrete cover
 ' satisfies Eq. (4.4)
 --->
 $v < v_{s,y1}$ - LHS eq.(4.7) is not satisfied
 --->
 $v < v_{c,y1}$ - RHS eq.(4.6) is satisfied
 --->
 ϕ_{cu} (4.10) = 0.58509201
 M_{Rc} (4.17) = 2.3801E+009
 --->
 New Case/Assumption: Ultimate curvature of confined core after spalling of concrete cover
 In expressions below, the following modifications have been made
 - b, d, d' replaced by geometric parameters of the core: b_o, d_o, d'_o
 - N_1, N_2, v normalised to $b_o \cdot d_o$, instead of $b \cdot d$
 - f_{cc}, ϕ_{cc} parameters of confined concrete, f_{cc}, ϕ_{cc} used in lieu of f_c, ϕ_{cu}
 --->
 Subcase: Rupture of tension steel
 --->
 $v^* < v^*_{s,y2}$ - LHS eq.(4.5) is not satisfied
 --->
 $v^* < v^*_{s,c}$ - LHS eq.(4.5) is not satisfied
 --->
 Subcase rejected
 --->
 New Subcase: Failure of compression zone
 --->
 $v^* < v^*_{c,y2}$ - LHS eq.(4.6) is not satisfied
 --->
 $v^* < v^*_{c,y1}$ - RHS eq.(4.6) is not satisfied
 --->
 ϕ^*_{cu} (4.11) = 0.65878578
 M_{Ro} (4.18) = 3.0615E+009
 --->
 $u = \phi_{cu}$ (4.2) = 1.3930706E-005
 $M_u = M_{Ro}$

Calculation of ratio I_b/I_d

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

Calculation of M_{u2}

Calculation of ultimate curvature ϕ_u according to 4.1, Biskinis/Fardis 2013:

$\phi_u = 9.1773508E-006$

$M_u = 8.9683E+008$

with full section properties:

$b = 450.00$

$d = 707.00$

$d' = 43.00$

$v = 1.34176$

$N = 7.6839E+006$

$f_c = 18.00$

ϕ_{co} (5A.5, TBDY) = 0.002

Final value of ϕ_{cu} : $\phi_{cu}^* = \text{shear_factor} * \text{Max}(\phi_{cu}, \phi_{cc}) = 0.01486841$

The Shear_factor is considered equal to 1 (pure moment strength)

From (5.4b), TBDY: $c_u = 0.01486841$

w_e (5.4c) = 0.08077545

a_{se} ((5.4d), TBDY) = $(a_{se1} \cdot A_{ext} + a_{se2} \cdot A_{int}) / A_{sec} = 0.53375773$

$a_{se1} = \text{Max}(((A_{conf,max1} - A_{noConf1}) / A_{conf,max1}) \cdot (A_{conf,min1} / A_{conf,max1}), 0) = 0.53375773$

The definitions of A_{noConf} , $A_{conf,min}$ and $A_{conf,max}$ are derived from generalization of the rectangular sections confinement, which is expressed by (5.4d).

The generalization is done according to Mander, J., Priestley, M., and Park, R. (1988)

"Theoretical Stress-Strain Model for Confined Concrete."

J. Struct. Eng., 10.1061/(ASCE)0733-9445(1988)114:8(1804), 1804-1826.

$A_{conf,max1} = 464100.00$ is the confined core area at levels of member with hoops and

is calculated as the area of core enclosed by the center lines of the external perimeter hoops.

$A_{conf,min1} = 395025.00$ is the confined core area at midway between the levels of hoops and

is calculated by reducing all the dimensions of the area $A_{conf,max1}$ by a length equal to half the clear spacing between external hoops.

$A_{noConf1} = 173066.667$ is the unconfined external core area which is equal to $b_i^2/6$ as defined at (A.2).

$a_{se2} (>= a_{se1}) = \text{Max}(((A_{conf,max2} - A_{noConf2}) / A_{conf,max2}) \cdot (A_{conf,min2} / A_{conf,max2}), 0) = 0.53375773$

The definitions of A_{noConf} , $A_{conf,min}$ and $A_{conf,max}$ are derived from generalization of the rectangular sections confinement, which is expressed by (5.4d).

The generalization is done according to Mander, J., Priestley, M., and Park, R. (1988)

"Theoretical Stress-Strain Model for Confined Concrete."

J. Struct. Eng., 10.1061/(ASCE)0733-9445(1988)114:8(1804), 1804-1826.

$A_{conf,max2} = 252164.00$ is the confined core area at levels of member with hoops and

is calculated as the area of core enclosed by the center lines of the internal perimeter hoops.

$A_{conf,min2} = 111441.00$ is the confined core area at midway between the levels of hoops and

is calculated by reducing all the dimensions of the area $A_{conf,max2}$ by a length equal to half the clear spacing between internal hoops.

$A_{noConf2} = 110709.333$ is the unconfined internal core area which is equal to $b_i^2/6$ as defined at (A.2).

$p_{sh,min} \cdot F_{ywe} = \text{Min}(p_{sh,x} \cdot F_{ywe}, p_{sh,y} \cdot F_{ywe}) = 2.724$

$p_{sh,x} \cdot F_{ywe} = p_{sh1} \cdot F_{ywe1} + p_{sh2} \cdot F_{ywe2} = 2.724$

p_{sh1} ((5.4d), TBDY) = $L_{stir1} \cdot A_{stir1} / (A_{sec} \cdot s_1) = 0.00301593$

L_{stir1} (Length of stirrups along Y) = 2160.00

A_{stir1} (stirrups area) = 78.53982

p_{sh2} (5.4d) = $L_{stir2} \cdot A_{stir2} / (A_{sec} \cdot s_2) = 0.00056047$

L_{stir2} (Length of stirrups along Y) = 1568.00

A_{stir2} (stirrups area) = 50.26548

$p_{sh,y} \cdot F_{ywe} = p_{sh1} \cdot F_{ywe1} + p_{sh2} \cdot F_{ywe2} = 3.25416$

p_{sh1} ((5.4d), TBDY) = $L_{stir1} \cdot A_{stir1} / (A_{sec} \cdot s_1) = 0.00357443$

L_{stir1} (Length of stirrups along X) = 2560.00

A_{stir1} (stirrups area) = 78.53982

p_{sh2} ((5.4d), TBDY) = $L_{stir2} \cdot A_{stir2} / (A_{sec} \cdot s_2) = 0.00070345$

L_{stir2} (Length of stirrups along X) = 1968.00

A_{stir2} (stirrups area) = 50.26548

$A_{sec} = 562500.00$

$s_1 = 100.00$

$s_2 = 250.00$

$f_{ywe1} = 781.25$

$f_{ywe2} = 656.25$

$f_{ce} = 18.00$

From ((5.A5), TBDY), TBDY: $c_c = 0.00621307$

c = confinement factor = 1.42131

$y_1 = 0.0025$

$sh_1 = 0.008$

$ft_1 = 900.1904$

$fy_1 = 750.1586$

$su_1 = 0.032$

using (30) in Biskinis/Fardis (2013) multiplied with shear_factor

and also multiplied by the shear_factor according to 15.7.1.4, with

Shear_factor = 1.00

$l_o/l_{ou,min} = l_b/l_d = 1.00$

$su_1 = 0.4 \cdot esu_1_{nominal}$ ((5.5), TBDY) = 0.032

From table 5A.1, TBDY: $esu_1_{nominal} = 0.08$,

For calculation of $esu_1_{nominal}$ and y_1 , sh_1 , ft_1 , fy_1 , it is considered characteristic value $fsy_1 = fs_1/1.2$, from table 5.1, TBDY.

y_1 , sh_1 , ft_1 , fy_1 , are also multiplied by $\text{Min}(1, 1.25 \cdot (l_b/l_d)^{2/3})$, from 10.3.5, ASCE 41-17.

```

with fs1 = (fs,jacket*Asl,ten,jacket + fs,core*Asl,ten,core)/Asl,ten = 750.1586
with Es1 = (Es,jacket*Asl,ten,jacket + Es,core*Asl,ten,core)/Asl,ten = 200000.00
y2 = 0.0025
sh2 = 0.008
ft2 = 907.50
fy2 = 756.25
su2 = 0.032
using (30) in Biskinis/Fardis (2013) multiplied with shear_factor
and also multiplied by the shear_factor according to 15.7.1.4, with
Shear_factor = 1.00
lo/lou,min = lb/lb,min = 1.00
su2 = 0.4*esu2_nominal ((5.5), TBDY) = 0.032
From table 5A.1, TBDY: esu2_nominal = 0.08,
For calculation of esu2_nominal and y2, sh2,ft2,fy2, it is considered
characteristic value fsy2 = fs2/1.2, from table 5.1, TBDY.
y1, sh1,ft1,fy1, are also multiplied by Min(1,1.25*(lb/lb)^2/3), from 10.3.5, ASCE 41-17.
with fs2 = (fs,jacket*Asl,com,jacket + fs,core*Asl,com,core)/Asl,com = 756.25
with Es2 = (Es,jacket*Asl,com,jacket + Es,core*Asl,com,core)/Asl,com = 200000.00
yv = 0.0025
shv = 0.008
ftv = 902.993
fyv = 752.4941
suv = 0.032
using (30) in Biskinis/Fardis (2013) multiplied with shear_factor
and also multiplied by the shear_factor according to 15.7.1.4, with
Shear_factor = 1.00
lo/lou,min = lb/lb = 1.00
suv = 0.4*esuv_nominal ((5.5), TBDY) = 0.032
From table 5A.1, TBDY: esuv_nominal = 0.08,
considering characteristic value fsyv = fsv/1.2, from table 5.1, TBDY
For calculation of esuv_nominal and yv, shv,ftv,fyv, it is considered
characteristic value fsyv = fsv/1.2, from table 5.1, TBDY.
y1, sh1,ft1,fy1, are also multiplied by Min(1,1.25*(lb/lb)^2/3), from 10.3.5, ASCE 41-17.
with fsv = (fs,jacket*Asl,mid,jacket + fs,mid*Asl,mid,core)/Asl,mid = 752.4941
with Esv = (Es,jacket*Asl,mid,jacket + Es,mid*Asl,mid,core)/Asl,mid = 200000.00
1 = Asl,ten/(b*d)*(fs1/fc) = 0.32428344
2 = Asl,com/(b*d)*(fs2/fc) = 0.20328574
v = Asl,mid/(b*d)*(fsv/fc) = 0.35171278
and confined core properties:
b = 390.00
d = 677.00
d' = 13.00
fcc (5A.2, TBDY) = 25.58352
cc (5A.5, TBDY) = 0.00621307
c = confinement factor = 1.42131
1 = Asl,ten/(b*d)*(fs1/fc) = 0.39075399
2 = Asl,com/(b*d)*(fs2/fc) = 0.24495458
v = Asl,mid/(b*d)*(fsv/fc) = 0.42380571
Case/Assumption: Unconfined full section - Steel rupture
' satisfies Eq. (4.3)
---->
v < vs,y2 - LHS eq.(4.5) is not satisfied
---->
v < vs,c - RHS eq.(4.5) is not satisfied
---->
Case/Assumption Rejected.
---->
New Case/Assumption: Unconfined full section - Spalling of concrete cover
' satisfies Eq. (4.4)
---->
v < s,y1 - LHS eq.(4.7) is not satisfied
---->
v < vc,y1 - RHS eq.(4.6) is not satisfied
---->
cu (4.11) = 0.95920575
MRc (4.18) = 7.7321E+008
---->

```

New Case/Assumption: Ultimate curvature of confined core after spalling of concrete cover

In expressions below, the following modifications have been made

- b, d, d' replaced by geometric parameters of the core: bo, do, d'o
- N, 1, 2, v normalised to bo*do, instead of b*d
- parameters of confined concrete, fcc, cc, used in lieu of fc, ecu

--->

Subcase: Rupture of tension steel

--->

$v^* < v^*s_y2$ - LHS eq.(4.5) is not satisfied

--->

$v^* < v^*s_c$ - LHS eq.(4.5) is not satisfied

--->

Subcase rejected

--->

New Subcase: Failure of compression zone

--->

$v^* < v^*c_y2$ - LHS eq.(4.6) is not satisfied

--->

$v^* < v^*c_y1$ - RHS eq.(4.6) is not satisfied

--->

ϕ_{cu} (4.11) = 1.00

M_{Ro} (4.18) = 8.9683E+008

--->

$\mu = \phi_{cu}$ (4.2) = 9.1773508E-006

$\mu_u = M_{Ro}$

Calculation of ratio lb/d

Adequate Lap Length: lb/d >= 1

Calculation of Shear Strength $V_r = \min(V_{r1}, V_{r2}) = 1.1678E+006$

Calculation of Shear Strength at edge 1, $V_{r1} = 1.1678E+006$

$V_{r1} = V_{col}$ ((10.3), ASCE 41-17) = knl*V_{col0}

V_{col0} = 1.1678E+006

knl = 1 (zero step-static loading)

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

= 1 (normal-weight concrete)

Mean concrete strength: $f'_c = (f'_{c,jacket} \cdot Area_{jacket} + f'_{c,core} \cdot Area_{core}) / Area_{section} = 15.20$, but $f'_c^{0.5} \leq 8.3$
MPa (22.5.3.1, ACI 318-14)

M/Vd = 4.00

$\mu_u = 7.6104E+008$

$V_u = 227.4778$

d = 0.8*h = 600.00

$N_u = 7.6839E+006$

Ag = 337500.00

From (11.5.4.8), ACI 318-14: $V_s = V_{s,jacket} + V_{s,core} = 1.0354E+006$

where:

$V_{s,jacket} = V_{s,j1} + V_{s,j2} = 942477.796$

$V_{s,j1} = 589048.623$ is calculated for section web jacket, with:

d = 600.00

$A_v = 157079.633$

$f_y = 625.00$

s = 100.00

$V_{s,j1}$ is multiplied by Col,j1 = 1.00

s/d = 0.16666667

$V_{s,j2} = 353429.174$ is calculated for section flange jacket, with:

d = 360.00

$A_v = 157079.633$

$f_y = 625.00$

$s = 100.00$
 V_{sj2} is multiplied by $Col,j2 = 1.00$
 $s/d = 0.27777778$
 $V_{s,core} = V_{s,c1} + V_{s,c2} = 92890.612$
 $V_{s,c1} = 92890.612$ is calculated for section web core, with:
 $d = 440.00$
 $A_v = 100530.965$
 $f_y = 525.00$
 $s = 250.00$
 $V_{s,c1}$ is multiplied by $Col,c1 = 1.00$
 $s/d = 0.56818182$
 $V_{s,c2} = 0.00$ is calculated for section flange core, with:
 $d = 200.00$
 $A_v = 100530.965$
 $f_y = 525.00$
 $s = 250.00$
 $V_{s,c2}$ is multiplied by $Col,c2 = 0.00$
 $s/d = 1.25$
 $V_f ((11-3)-(11.4), ACI 440) = 0.00$
 From (11-11), ACI 440: $V_s + V_f \leq 699281.943$
 $bw = 450.00$

Calculation of Shear Strength at edge 2, $V_{r2} = 1.1678E+006$
 $V_{r2} = V_{Col} ((10.3), ASCE 41-17) = knl * V_{Col0}$
 $V_{Col0} = 1.1678E+006$
 $knl = 1$ (zero step-static loading)

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

$= 1$ (normal-weight concrete)
 Mean concrete strength: $fc' = (fc'_{jacket} * Area_{jacket} + fc'_{core} * Area_{core}) / Area_{section} = 15.20$, but $fc'^{0.5} \leq 8.3$
 MPa (22.5.3.1, ACI 318-14)
 $M/Vd = 4.00$
 $M_u = 6.8991E+008$
 $V_u = 227.4778$
 $d = 0.8 * h = 600.00$
 $N_u = 7.6839E+006$
 $Ag = 337500.00$
 From (11.5.4.8), ACI 318-14: $V_s = V_{sjacket} + V_{s,core} = 1.0354E+006$
 where:
 $V_{sjacket} = V_{sj1} + V_{sj2} = 942477.796$
 $V_{sj1} = 589048.623$ is calculated for section web jacket, with:
 $d = 600.00$
 $A_v = 157079.633$
 $f_y = 625.00$
 $s = 100.00$
 V_{sj1} is multiplied by $Col,j1 = 1.00$
 $s/d = 0.16666667$
 $V_{sj2} = 353429.174$ is calculated for section flange jacket, with:
 $d = 360.00$
 $A_v = 157079.633$
 $f_y = 625.00$
 $s = 100.00$
 V_{sj2} is multiplied by $Col,j2 = 1.00$
 $s/d = 0.27777778$
 $V_{s,core} = V_{s,c1} + V_{s,c2} = 92890.612$
 $V_{s,c1} = 92890.612$ is calculated for section web core, with:
 $d = 440.00$
 $A_v = 100530.965$
 $f_y = 525.00$
 $s = 250.00$
 $V_{s,c1}$ is multiplied by $Col,c1 = 1.00$
 $s/d = 0.56818182$
 $V_{s,c2} = 0.00$ is calculated for section flange core, with:
 $d = 200.00$

Av = 100530.965

fy = 525.00

s = 250.00

Vs,c2 is multiplied by Col,c2 = 0.00

s/d = 1.25

Vf ((11-3)-(11.4), ACI 440) = 0.00

From (11-11), ACI 440: Vs + Vf <= 699281.943

bw = 450.00

End Of Calculation of Shear Capacity ratio for element: column JTC1 of floor 1

At local axis: 3

Start Of Calculation of Shear Capacity ratio for element: column JTC1 of floor 1

At Shear local axis: 2

(Bending local axis: 3)

Section Type: rcjtcs

Constant Properties

Knowledge Factor, $\phi = 0.80$

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

Consequently:

Jacket

New material of Primary Member: Concrete Strength, $f_c = f_{cm} = 18.00$

New material of Primary Member: Steel Strength, $f_s = f_{sm} = 625.00$

Concrete Elasticity, $E_c = 19940.411$

Steel Elasticity, $E_s = 200000.00$

Existing Column

Existing material of Primary Member: Concrete Strength, $f_c = f_{cm} = 12.00$

Existing material of Primary Member: Steel Strength, $f_s = f_{sm} = 525.00$

Concrete Elasticity, $E_c = 16281.278$

Steel Elasticity, $E_s = 200000.00$

#####

Note: Especially for the calculation of moment strengths,

the above steel re-bar strengths are multiplied by 1.25 according to R18.6.5, ACI 318-14

Jacket

New material: Steel Strength, $f_s = 1.25 \cdot f_{sm} = 781.25$

Existing Column

Existing material: Steel Strength, $f_s = 1.25 \cdot f_{sm} = 656.25$

#####

Max Height, $H_{max} = 750.00$

Min Height, $H_{min} = 450.00$

Max Width, $W_{max} = 950.00$

Min Width, $W_{min} = 450.00$

Eccentricity, $E_{cc} = 250.00$

Jacket Thickness, $t_j = 100.00$

Cover Thickness, $c = 25.00$

Mean Confinement Factor overall section = 1.42131

Element Length, $L = 3000.00$

Primary Member

Ribbed Bars

Ductile Steel

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

Longitudinal Bars With Ends Lapped Starting at the End Sections

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

No FRP Wrapping

Stepwise Properties

At local axis: 2

EDGE -A-

Shear Force, $V_a = -137.4894$

EDGE -B-

Shear Force, $V_b = 137.4894$

BOTH EDGES

Axial Force, $F = -7.6839E+006$

Longitudinal Reinforcement Area Distribution (in 2 divisions)

-Tension: $As_t = 314.1593$

-Compression: $As_c = 6176.371$

Longitudinal Reinforcement Area Distribution (in 3 divisions)

-Tension: $As_{t,ten} = 1539.38$

-Compression: $As_{c,com} = 1539.38$

-Middle: $As_{mid} = 3612.832$

Calculation of Shear Capacity ratio, $V_e/V_r = 0.86614729$

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

Calculation of Shear Demand from fig. R18.6.5, ACI 318-14 $V_e = (M_{pr1} + M_{pr2})/l_n = 1.2287E+006$ with

$M_{pr1} = \text{Max}(Mu_{1+}, Mu_{1-}) = 1.8431E+009$

$Mu_{1+} = 1.8431E+009$, is the ultimate moment strength at the edge 1 of the member in the actual moment direction which is defined for the static loading combination

$Mu_{1-} = 1.8431E+009$, is the ultimate moment strength at the edge 1 of the member in the opposite moment direction which is defined for the static loading combination

$M_{pr2} = \text{Max}(Mu_{2+}, Mu_{2-}) = 1.8431E+009$

$Mu_{2+} = 1.8431E+009$, is the ultimate moment strength at the edge 2 of the member in the actual moment direction which is defined for the static loading combination

$Mu_{2-} = 1.8431E+009$, is the ultimate moment strength at the edge 2 of the member in the opposite moment direction which is defined for the static loading combination

Calculation of Mu_{1+}

Calculation of ultimate curvature ϕ_u according to 4.1, Biskinis/Fardis 2013:

$\phi_u = 7.8730391E-006$

$M_u = 1.8431E+009$

with full section properties:

$b = 450.00$

$d = 907.00$

$d' = 43.00$

$v = 1.0459$

$N = 7.6839E+006$

$f_c = 18.00$

$\phi_c(5A.5, \text{TBDY}) = 0.002$

Final value of ϕ_u : $\phi_u^* = \text{shear_factor} * \text{Max}(\phi_u, \phi_c) = 0.01486841$

The Shear_factor is considered equal to 1 (pure moment strength)

From (5.4b), TBDY: $\phi_u = 0.01486841$

$\phi_{we}(5.4c) = 0.08077545$

$\phi_{ase}((5.4d), \text{TBDY}) = (\phi_{ase1} * A_{ext} + \phi_{ase2} * A_{int}) / A_{sec} = 0.53375773$

$\phi_{ase1} = \text{Max}(((A_{conf,max1} - A_{noConf1}) / A_{conf,max1}) * (A_{conf,min1} / A_{conf,max1}), 0) = 0.53375773$

The definitions of A_{noConf} , $A_{conf,min}$ and $A_{conf,max}$ are derived from generalization of the rectangular sections confinement, which is expressed by (5.4d).

The generalization is done according to Mander, J., Priestley, M., and Park, R. (1988)

"Theoretical Stress-Strain Model for Confined Concrete."

J. Struct. Eng., 10.1061/(ASCE)0733-9445(1988)114:8(1804), 1804-1826.

$A_{conf,max1} = 464100.00$ is the confined core area at levels of member with hoops and is calculated as the area of core enclosed by the center lines of the external perimeter hoops.

$A_{conf,min1} = 395025.00$ is the confined core area at midway between the levels of hoops and is calculated by reducing all the dimensions of the area $A_{conf,max1}$ by a length equal to half the clear spacing between external hoops.

$A_{noConf1} = 173066.667$ is the unconfined external core area which is equal to $b_i^2/6$ as defined at (A.2).

$\phi_{ase2} (> \phi_{ase1}) = \text{Max}(((A_{conf,max2} - A_{noConf2}) / A_{conf,max2}) * (A_{conf,min2} / A_{conf,max2}), 0) = 0.53375773$

The definitions of A_{noConf} , $A_{conf,min}$ and $A_{conf,max}$ are derived from generalization of the rectangular sections confinement, which is expressed by (5.4d).

The generalization is done according to Mander, J., Priestley, M., and Park, R. (1988)
"Theoretical Stress-Strain Model for Confined Concrete."

J. Struct. Eng., 10.1061/(ASCE)0733-9445(1988)114:8(1804), 1804-1826.

$A_{conf,max2} = 252164.00$ is the confined core area at levels of member with hoops and
is calculated as the area of core enclosed by the center lines of the internal perimeter hoops.

$A_{conf,min2} = 111441.00$ is the confined core area at midway between the levels of hoops and
is calculated by reducing all the dimensions of the area $A_{conf,max2}$ by a length
equal to half the clear spacing between internal hoops.

$A_{noConf2} = 110709.333$ is the unconfined internal core area which is equal to $b_i^2/6$ as defined at (A.2).
 $psh_{min} \cdot F_{ywe} = \text{Min}(psh_x \cdot F_{ywe}, psh_y \cdot F_{ywe}) = 2.724$

$psh_x \cdot F_{ywe} = psh1 \cdot F_{ywe1} + ps2 \cdot F_{ywe2} = 2.724$

$psh1 \text{ ((5.4d), TBDY)} = L_{stir1} \cdot A_{stir1} / (A_{sec} \cdot s1) = 0.00301593$

L_{stir1} (Length of stirrups along Y) = 2160.00

A_{stir1} (stirrups area) = 78.53982

$psh2 \text{ (5.4d)} = L_{stir2} \cdot A_{stir2} / (A_{sec} \cdot s2) = 0.00056047$

L_{stir2} (Length of stirrups along Y) = 1568.00

A_{stir2} (stirrups area) = 50.26548

$psh_y \cdot F_{ywe} = psh1 \cdot F_{ywe1} + ps2 \cdot F_{ywe2} = 3.25416$

$psh1 \text{ ((5.4d), TBDY)} = L_{stir1} \cdot A_{stir1} / (A_{sec} \cdot s1) = 0.00357443$

L_{stir1} (Length of stirrups along X) = 2560.00

A_{stir1} (stirrups area) = 78.53982

$psh2 \text{ ((5.4d), TBDY)} = L_{stir2} \cdot A_{stir2} / (A_{sec} \cdot s2) = 0.00070345$

L_{stir2} (Length of stirrups along X) = 1968.00

A_{stir2} (stirrups area) = 50.26548

$A_{sec} = 562500.00$

$s1 = 100.00$

$s2 = 250.00$

$f_{ywe1} = 781.25$

$f_{ywe2} = 656.25$

$f_{ce} = 18.00$

From ((5.A5), TBDY), TBDY: $cc = 0.00621307$

c = confinement factor = 1.42131

$y1 = 0.0025$

$sh1 = 0.008$

$ft1 = 907.50$

$fy1 = 756.25$

$su1 = 0.032$

using (30) in Biskinis/Fardis (2013) multiplied with shear_factor
and also multiplied by the shear_factor according to 15.7.1.4, with
Shear_factor = 1.00

$lo/lo_{u,min} = lb/ld = 1.00$

$su1 = 0.4 \cdot esu1_{nominal} \text{ ((5.5), TBDY)} = 0.032$

From table 5A.1, TBDY: $esu1_{nominal} = 0.08$,

For calculation of $esu1_{nominal}$ and $y1, sh1, ft1, fy1$, it is considered
characteristic value $fsy1 = fs1/1.2$, from table 5.1, TBDY.

$y1, sh1, ft1, fy1$, are also multiplied by $\text{Min}(1, 1.25 \cdot (lb/ld)^{2/3})$, from 10.3.5, ASCE 41-17.

with $fs1 = (fs_{jacket} \cdot A_{sl,ten,jacket} + fs_{core} \cdot A_{sl,ten,core}) / A_{sl,ten} = 756.25$

with $Es1 = (Es_{jacket} \cdot A_{sl,ten,jacket} + Es_{core} \cdot A_{sl,ten,core}) / A_{sl,ten} = 200000.00$

$y2 = 0.0025$

$sh2 = 0.008$

$ft2 = 907.50$

$fy2 = 756.25$

$su2 = 0.032$

using (30) in Biskinis/Fardis (2013) multiplied with shear_factor
and also multiplied by the shear_factor according to 15.7.1.4, with
Shear_factor = 1.00

$lo/lo_{u,min} = lb/lb_{min} = 1.00$

$su2 = 0.4 \cdot esu2_{nominal} \text{ ((5.5), TBDY)} = 0.032$

From table 5A.1, TBDY: $esu2_{nominal} = 0.08$,

For calculation of $esu2_{nominal}$ and $y2, sh2, ft2, fy2$, it is considered
characteristic value $fsy2 = fs2/1.2$, from table 5.1, TBDY.

$y1, sh1, ft1, fy1$, are also multiplied by $\text{Min}(1, 1.25 \cdot (lb/ld)^{2/3})$, from 10.3.5, ASCE 41-17.

with $fs2 = (fs_{jacket} \cdot A_{sl,com,jacket} + fs_{core} \cdot A_{sl,com,core}) / A_{sl,com} = 756.25$

with $Es2 = (Es_{jacket} \cdot A_{sl,com,jacket} + Es_{core} \cdot A_{sl,com,core}) / A_{sl,com} = 200000.00$

```

yv = 0.0025
shv = 0.008
ftv = 899.1522
fyv = 749.2935
suv = 0.032
    using (30) in Biskinis/Fardis (2013) multiplied with shear_factor
    and also multiplied by the shear_factor according to 15.7.1.4, with
    Shear_factor = 1.00
    lo/lou,min = lb/ld = 1.00
    suv = 0.4*esuv_nominal ((5.5), TBDY) = 0.032
    From table 5A.1, TBDY: esuv_nominal = 0.08,
    considering characteristic value fsyv = fsv/1.2, from table 5.1, TBDY
    For calculation of esuv_nominal and yv, shv,ftv,fyv, it is considered
    characteristic value fsyv = fsv/1.2, from table 5.1, TBDY.
    y1, sh1,ft1,fy1, are also multiplied by Min(1,1.25*(lb/ld)^2/3), from 10.3.5, ASCE 41-17.
    with fsv = (fs,jacket*Asl,mid,jacket + fs,mid*Asl,mid,core)/Asl,mid = 749.2935
    with Esv = (Es,jacket*Asl,mid,jacket + Es,mid*Asl,mid,core)/Asl,mid = 200000.00
    1 = Asl,ten/(b*d)*(fs1/fc) = 0.15845977
    2 = Asl,com/(b*d)*(fs2/fc) = 0.15845977
    v = Asl,mid/(b*d)*(fsv/fc) = 0.36847443
and confined core properties:
b = 390.00
d = 877.00
d' = 13.00
fcc (5A.2, TBDY) = 25.58352
cc (5A.5, TBDY) = 0.00621307
    c = confinement factor = 1.42131
    1 = Asl,ten/(b*d)*(fs1/fc) = 0.18909264
    2 = Asl,com/(b*d)*(fs2/fc) = 0.18909264
    v = Asl,mid/(b*d)*(fsv/fc) = 0.43970658
Case/Assumption: Unconfined full section - Steel rupture
' satisfies Eq. (4.3)
--->
v < vs,y2 - LHS eq.(4.5) is not satisfied
--->
v < vs,c - RHS eq.(4.5) is not satisfied
--->
Case/Assumption Rejected.
--->
New Case/Assumption: Unconfined full section - Spalling of concrete cover
' satisfies Eq. (4.4)
--->
v < s,y1 - LHS eq.(4.7) is not satisfied
--->
v < vc,y1 - RHS eq.(4.6) is satisfied
--->
cu (4.10) = 0.83934343
MRc (4.17) = 1.8439E+009
--->
New Case/Assumption: Ultimate curvature of confined core after spalling of concrete cover
In expressions below, the following modifications have been made
- b, d, d' replaced by geometric parameters of the core: bo, do, d'o
- N, 1, 2, v normalised to bo*do, instead of b*d
- parameters of confined concrete, fcc, cc, used in lieu of fc, ecu
--->
Subcase: Rupture of tension steel
--->
v* < v*s,y2 - LHS eq.(4.5) is not satisfied
--->
v* < v*s,c - LHS eq.(4.5) is not satisfied
--->
Subcase rejected
--->
New Subcase: Failure of compression zone
--->
v* < v*c,y2 - LHS eq.(4.6) is not satisfied
--->

```

$v^* < v^*_{c,y1}$ - RHS eq.(4.6) is not satisfied

---->

$$*cu(4.11) = 0.89983732$$

$$MRo(4.18) = 1.8431E+009$$

---->

$$u = cu(4.2) = 7.8730391E-006$$

$$Mu = MRo$$

Calculation of ratio lb/ld

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

Calculation of $Mu1$ -

Calculation of ultimate curvature u according to 4.1, Biskinis/Fardis 2013:

$$u = 7.8730391E-006$$

$$Mu = 1.8431E+009$$

with full section properties:

$$b = 450.00$$

$$d = 907.00$$

$$d' = 43.00$$

$$v = 1.0459$$

$$N = 7.6839E+006$$

$$fc = 18.00$$

$$co(5A.5, TBDY) = 0.002$$

$$\text{Final value of } cu: cu^* = \text{shear_factor} * \text{Max}(cu, cc) = 0.01486841$$

The Shear_factor is considered equal to 1 (pure moment strength)

$$\text{From (5.4b), TBDY: } cu = 0.01486841$$

$$we(5.4c) = 0.08077545$$

$$ase((5.4d), TBDY) = (ase1 * A_{ext} + ase2 * A_{int}) / A_{sec} = 0.53375773$$

$$ase1 = \text{Max}(((A_{conf,max1} - A_{noConf1}) / A_{conf,max1}) * (A_{conf,min1} / A_{conf,max1}), 0) = 0.53375773$$

The definitions of A_{noConf} , $A_{conf,min}$ and $A_{conf,max}$ are derived from generalization of the rectangular sections confinement, which is expressed by (5.4d).

The generalization is done according to Mander, J., Priestley, M., and Park, R. (1988)

"Theoretical Stress-Strain Model for Confined Concrete."

J. Struct. Eng., 10.1061/(ASCE)0733-9445(1988)114:8(1804), 1804-1826.

$A_{conf,max1} = 464100.00$ is the confined core area at levels of member with hoops and is calculated as the area of core enclosed by the center lines of the external perimeter hoops.

$A_{conf,min1} = 395025.00$ is the confined core area at midway between the levels of hoops and is calculated by reducing all the dimensions of the area $A_{conf,max1}$ by a length equal to half the clear spacing between external hoops.

$A_{noConf1} = 173066.667$ is the unconfined external core area which is equal to $b_i^2/6$ as defined at (A.2).

$$ase2 (\geq ase1) = \text{Max}(((A_{conf,max2} - A_{noConf2}) / A_{conf,max2}) * (A_{conf,min2} / A_{conf,max2}), 0) = 0.53375773$$

The definitions of A_{noConf} , $A_{conf,min}$ and $A_{conf,max}$ are derived from generalization of the rectangular sections confinement, which is expressed by (5.4d).

The generalization is done according to Mander, J., Priestley, M., and Park, R. (1988)

"Theoretical Stress-Strain Model for Confined Concrete."

J. Struct. Eng., 10.1061/(ASCE)0733-9445(1988)114:8(1804), 1804-1826.

$A_{conf,max2} = 252164.00$ is the confined core area at levels of member with hoops and is calculated as the area of core enclosed by the center lines of the internal perimeter hoops.

$A_{conf,min2} = 111441.00$ is the confined core area at midway between the levels of hoops and is calculated by reducing all the dimensions of the area $A_{conf,max2}$ by a length equal to half the clear spacing between internal hoops.

$A_{noConf2} = 110709.333$ is the unconfined internal core area which is equal to $b_i^2/6$ as defined at (A.2).

$$psh_{min} * Fy_{we} = \text{Min}(psh_x * Fy_{we}, psh_y * Fy_{we}) = 2.724$$

$$psh_x * Fy_{we} = psh1 * Fy_{we1} + psh2 * Fy_{we2} = 2.724$$

$$psh1((5.4d), TBDY) = L_{stir1} * A_{stir1} / (A_{sec} * s1) = 0.00301593$$

$$L_{stir1} (\text{Length of stirrups along } Y) = 2160.00$$

$$A_{stir1} (\text{stirrups area}) = 78.53982$$

$$psh2(5.4d) = L_{stir2} * A_{stir2} / (A_{sec} * s2) = 0.00056047$$

Lstir2 (Length of stirrups along Y) = 1568.00
Astir2 (stirrups area) = 50.26548

psh_y*Fywe = psh1*Fywe1+ps2*Fywe2 = 3.25416
psh1 ((5.4d), TBDY) = Lstir1*Astir1/(Asec*s1) = 0.00357443
Lstir1 (Length of stirrups along X) = 2560.00
Astir1 (stirrups area) = 78.53982
psh2 ((5.4d), TBDY) = Lstir2*Astir2/(Asec*s2) = 0.00070345
Lstir2 (Length of stirrups along X) = 1968.00
Astir2 (stirrups area) = 50.26548

Asec = 562500.00
s1 = 100.00
s2 = 250.00

fywe1 = 781.25
fywe2 = 656.25
fce = 18.00

From ((5.A5), TBDY), TBDY: cc = 0.00621307
c = confinement factor = 1.42131

y1 = 0.0025
sh1 = 0.008
ft1 = 907.50
fy1 = 756.25
su1 = 0.032

using (30) in Biskinis/Fardis (2013) multiplied with shear_factor
and also multiplied by the shear_factor according to 15.7.1.4, with
Shear_factor = 1.00

lo/lou,min = lb/lb = 1.00

su1 = 0.4*esu1_nominal ((5.5), TBDY) = 0.032

From table 5A.1, TBDY: esu1_nominal = 0.08,

For calculation of esu1_nominal and y1, sh1,ft1,fy1, it is considered
characteristic value fsy1 = fs1/1.2, from table 5.1, TBDY.

y1, sh1,ft1,fy1, are also multiplied by $\text{Min}(1, 1.25 \cdot (lb/lb)^{2/3})$, from 10.3.5, ASCE 41-17.

with fs1 = (fs,jacket*Asl,ten,jacket + fs,core*Asl,ten,core)/Asl,ten = 756.25

with Es1 = (Es,jacket*Asl,ten,jacket + Es,core*Asl,ten,core)/Asl,ten = 200000.00

y2 = 0.0025
sh2 = 0.008
ft2 = 907.50
fy2 = 756.25
su2 = 0.032

using (30) in Biskinis/Fardis (2013) multiplied with shear_factor
and also multiplied by the shear_factor according to 15.7.1.4, with
Shear_factor = 1.00

lo/lou,min = lb/lb,min = 1.00

su2 = 0.4*esu2_nominal ((5.5), TBDY) = 0.032

From table 5A.1, TBDY: esu2_nominal = 0.08,

For calculation of esu2_nominal and y2, sh2,ft2,fy2, it is considered
characteristic value fsy2 = fs2/1.2, from table 5.1, TBDY.

y1, sh1,ft1,fy1, are also multiplied by $\text{Min}(1, 1.25 \cdot (lb/lb)^{2/3})$, from 10.3.5, ASCE 41-17.

with fs2 = (fs,jacket*Asl,com,jacket + fs,core*Asl,com,core)/Asl,com = 756.25

with Es2 = (Es,jacket*Asl,com,jacket + Es,core*Asl,com,core)/Asl,com = 200000.00

yv = 0.0025
shv = 0.008
ftv = 899.1522
fyv = 749.2935
suv = 0.032

using (30) in Biskinis/Fardis (2013) multiplied with shear_factor
and also multiplied by the shear_factor according to 15.7.1.4, with
Shear_factor = 1.00

lo/lou,min = lb/lb = 1.00

suv = 0.4*esuv_nominal ((5.5), TBDY) = 0.032

From table 5A.1, TBDY: esuv_nominal = 0.08,

considering characteristic value fsyv = fsv/1.2, from table 5.1, TBDY

For calculation of esuv_nominal and yv, shv,ftv,fyv, it is considered
characteristic value fsyv = fsv/1.2, from table 5.1, TBDY.

y1, sh1,ft1,fy1, are also multiplied by $\text{Min}(1, 1.25 \cdot (lb/lb)^{2/3})$, from 10.3.5, ASCE 41-17.

with fsv = (fs,jacket*Asl,mid,jacket + fs,mid*Asl,mid,core)/Asl,mid = 749.2935

```

with Esv = (Esjacket*Aslmid,jacket + Esmid*Aslmid,core)/Aslmid = 200000.00
1 = Aslten/(b*d)*(fs1/fc) = 0.15845977
2 = Aslcom/(b*d)*(fs2/fc) = 0.15845977
v = Aslmid/(b*d)*(fsv/fc) = 0.36847443
and confined core properties:
b = 390.00
d = 877.00
d' = 13.00
fcc (5A.2, TBDY) = 25.58352
cc (5A.5, TBDY) = 0.00621307
c = confinement factor = 1.42131
1 = Aslten/(b*d)*(fs1/fc) = 0.18909264
2 = Aslcom/(b*d)*(fs2/fc) = 0.18909264
v = Aslmid/(b*d)*(fsv/fc) = 0.43970658
Case/Assumption: Unconfined full section - Steel rupture
' satisfies Eq. (4.3)
--->
v < vs,y2 - LHS eq.(4.5) is not satisfied
--->
v < vs,c - RHS eq.(4.5) is not satisfied
--->
Case/Assumption Rejected.
--->
New Case/Assumption: Unconfined full section - Spalling of concrete cover
' satisfies Eq. (4.4)
--->
v < sy1 - LHS eq.(4.7) is not satisfied
--->
v < vc,y1 - RHS eq.(4.6) is satisfied
--->
cu (4.10) = 0.83934343
MRc (4.17) = 1.8439E+009
--->
New Case/Assumption: Ultimate curvature of confined core after spalling of concrete cover
In expressions below, the following modifications have been made
- b, d, d' replaced by geometric parameters of the core: bo, do, d'o
- N1, 1, 2, v normalised to bo*do, instead of b*d
- - parameters of confined concrete, fcc, cc, used in lieu of fc, ecu
--->
Subcase: Rupture of tension steel
--->
v* < v*sy2 - LHS eq.(4.5) is not satisfied
--->
v* < v*sc - LHS eq.(4.5) is not satisfied
--->
Subcase rejected
--->
New Subcase: Failure of compression zone
--->
v* < v*cy2 - LHS eq.(4.6) is not satisfied
--->
v* < v*cy1 - RHS eq.(4.6) is not satisfied
--->
*cu (4.11) = 0.89983732
MRo (4.18) = 1.8431E+009
--->
u = cu (4.2) = 7.8730391E-006
Mu = MRo

```

Calculation of ratio lb/d

Adequate Lap Length: lb/d >= 1

Calculation of Mu2+

Calculation of ultimate curvature ϕ_u according to 4.1, Biskinis/Fardis 2013:

$$u = 7.8730391E-006$$

$$\mu = 1.8431E+009$$

with full section properties:

$$b = 450.00$$

$$d = 907.00$$

$$d' = 43.00$$

$$v = 1.0459$$

$$N = 7.6839E+006$$

$$f_c = 18.00$$

$$\alpha (5A.5, \text{TBDY}) = 0.002$$

$$\text{Final value of } \phi_u: \phi_u^* = \text{shear_factor} * \text{Max}(\phi_u, \phi_c) = 0.01486841$$

The Shear_factor is considered equal to 1 (pure moment strength)

$$\text{From (5.4b), TBDY: } \phi_u = 0.01486841$$

$$\phi_{ue} (5.4c) = 0.08077545$$

$$\alpha_{se} ((5.4d), \text{TBDY}) = (\alpha_{se1} * A_{ext} + \alpha_{se2} * A_{int}) / A_{sec} = 0.53375773$$

$$\alpha_{se1} = \text{Max}(((A_{conf,max1} - A_{noConf1}) / A_{conf,max1}) * (A_{conf,min1} / A_{conf,max1}), 0) = 0.53375773$$

The definitions of A_{noConf} , $A_{conf,min}$ and $A_{conf,max}$ are derived from generalization of the rectangular sections confinement, which is expressed by (5.4d).

The generalization is done according to Mander, J., Priestley, M., and Park, R. (1988)

"Theoretical Stress-Strain Model for Confined Concrete."

J. Struct. Eng., 10.1061/(ASCE)0733-9445(1988)114:8(1804), 1804-1826.

$A_{conf,max1} = 464100.00$ is the confined core area at levels of member with hoops and is calculated as the area of core enclosed by the center lines of the external perimeter hoops.

$A_{conf,min1} = 395025.00$ is the confined core area at midway between the levels of hoops and is calculated by reducing all the dimensions of the area $A_{conf,max1}$ by a length equal to half the clear spacing between external hoops.

$A_{noConf1} = 173066.667$ is the unconfined external core area which is equal to $b_i^2/6$ as defined at (A.2).

$$\alpha_{se2} (> \alpha_{se1}) = \text{Max}(((A_{conf,max2} - A_{noConf2}) / A_{conf,max2}) * (A_{conf,min2} / A_{conf,max2}), 0) = 0.53375773$$

The definitions of A_{noConf} , $A_{conf,min}$ and $A_{conf,max}$ are derived from generalization of the rectangular sections confinement, which is expressed by (5.4d).

The generalization is done according to Mander, J., Priestley, M., and Park, R. (1988)

"Theoretical Stress-Strain Model for Confined Concrete."

J. Struct. Eng., 10.1061/(ASCE)0733-9445(1988)114:8(1804), 1804-1826.

$A_{conf,max2} = 252164.00$ is the confined core area at levels of member with hoops and is calculated as the area of core enclosed by the center lines of the internal perimeter hoops.

$A_{conf,min2} = 111441.00$ is the confined core area at midway between the levels of hoops and is calculated by reducing all the dimensions of the area $A_{conf,max2}$ by a length equal to half the clear spacing between internal hoops.

$A_{noConf2} = 110709.333$ is the unconfined internal core area which is equal to $b_i^2/6$ as defined at (A.2).

$$\phi_{sh,min} * F_{ywe} = \text{Min}(\phi_{sh,x} * F_{ywe}, \phi_{sh,y} * F_{ywe}) = 2.724$$

$$\phi_{sh,x} * F_{ywe} = \phi_{sh1} * F_{ywe1} + \phi_{sh2} * F_{ywe2} = 2.724$$

$$\phi_{sh1} ((5.4d), \text{TBDY}) = L_{stir1} * A_{stir1} / (A_{sec} * s_1) = 0.00301593$$

$$L_{stir1} \text{ (Length of stirrups along Y)} = 2160.00$$

$$A_{stir1} \text{ (stirrups area)} = 78.53982$$

$$\phi_{sh2} (5.4d) = L_{stir2} * A_{stir2} / (A_{sec} * s_2) = 0.00056047$$

$$L_{stir2} \text{ (Length of stirrups along Y)} = 1568.00$$

$$A_{stir2} \text{ (stirrups area)} = 50.26548$$

$$\phi_{sh,y} * F_{ywe} = \phi_{sh1} * F_{ywe1} + \phi_{sh2} * F_{ywe2} = 3.25416$$

$$\phi_{sh1} ((5.4d), \text{TBDY}) = L_{stir1} * A_{stir1} / (A_{sec} * s_1) = 0.00357443$$

$$L_{stir1} \text{ (Length of stirrups along X)} = 2560.00$$

$$A_{stir1} \text{ (stirrups area)} = 78.53982$$

$$\phi_{sh2} ((5.4d), \text{TBDY}) = L_{stir2} * A_{stir2} / (A_{sec} * s_2) = 0.00070345$$

$$L_{stir2} \text{ (Length of stirrups along X)} = 1968.00$$

$$A_{stir2} \text{ (stirrups area)} = 50.26548$$

$$A_{sec} = 562500.00$$

$$s_1 = 100.00$$

$$s_2 = 250.00$$

$$f_{ywe1} = 781.25$$

$$f_{ywe2} = 656.25$$

$f_{ce} = 18.00$
 From ((5.A5), TBDY), TBDY: $cc = 0.00621307$
 $c = \text{confinement factor} = 1.42131$
 $y_1 = 0.0025$
 $sh_1 = 0.008$
 $ft_1 = 907.50$
 $fy_1 = 756.25$
 $su_1 = 0.032$
 using (30) in Biskinis/Fardis (2013) multiplied with shear_factor
 and also multiplied by the shear_factor according to 15.7.1.4, with
 $\text{Shear_factor} = 1.00$
 $lo/lou, \min = lb/ld = 1.00$
 $su_1 = 0.4 * esu_1, \text{nominal} ((5.5), \text{TBDY}) = 0.032$
 From table 5A.1, TBDY: $esu_1, \text{nominal} = 0.08$,
 For calculation of $esu_1, \text{nominal}$ and y_1, sh_1, ft_1, fy_1 , it is considered
 characteristic value $fsy_1 = fs_1/1.2$, from table 5.1, TBDY.
 y_1, sh_1, ft_1, fy_1 , are also multiplied by $\text{Min}(1, 1.25 * (lb/ld)^{2/3})$, from 10.3.5, ASCE 41-17.
 with $fs_1 = (fs, \text{jacket} * Asl, \text{ten, jacket} + fs, \text{core} * Asl, \text{ten, core}) / Asl, \text{ten} = 756.25$
 with $Es_1 = (Es, \text{jacket} * Asl, \text{ten, jacket} + Es, \text{core} * Asl, \text{ten, core}) / Asl, \text{ten} = 200000.00$
 $y_2 = 0.0025$
 $sh_2 = 0.008$
 $ft_2 = 907.50$
 $fy_2 = 756.25$
 $su_2 = 0.032$
 using (30) in Biskinis/Fardis (2013) multiplied with shear_factor
 and also multiplied by the shear_factor according to 15.7.1.4, with
 $\text{Shear_factor} = 1.00$
 $lo/lou, \min = lb/lb, \min = 1.00$
 $su_2 = 0.4 * esu_2, \text{nominal} ((5.5), \text{TBDY}) = 0.032$
 From table 5A.1, TBDY: $esu_2, \text{nominal} = 0.08$,
 For calculation of $esu_2, \text{nominal}$ and y_2, sh_2, ft_2, fy_2 , it is considered
 characteristic value $fsy_2 = fs_2/1.2$, from table 5.1, TBDY.
 y_1, sh_1, ft_1, fy_1 , are also multiplied by $\text{Min}(1, 1.25 * (lb/ld)^{2/3})$, from 10.3.5, ASCE 41-17.
 with $fs_2 = (fs, \text{jacket} * Asl, \text{com, jacket} + fs, \text{core} * Asl, \text{com, core}) / Asl, \text{com} = 756.25$
 with $Es_2 = (Es, \text{jacket} * Asl, \text{com, jacket} + Es, \text{core} * Asl, \text{com, core}) / Asl, \text{com} = 200000.00$
 $y_v = 0.0025$
 $sh_v = 0.008$
 $ft_v = 899.1522$
 $fy_v = 749.2935$
 $suv = 0.032$
 using (30) in Biskinis/Fardis (2013) multiplied with shear_factor
 and also multiplied by the shear_factor according to 15.7.1.4, with
 $\text{Shear_factor} = 1.00$
 $lo/lou, \min = lb/ld = 1.00$
 $suv = 0.4 * esuv, \text{nominal} ((5.5), \text{TBDY}) = 0.032$
 From table 5A.1, TBDY: $esuv, \text{nominal} = 0.08$,
 considering characteristic value $fsyv = fsv/1.2$, from table 5.1, TBDY
 For calculation of $esuv, \text{nominal}$ and y_v, sh_v, ft_v, fy_v , it is considered
 characteristic value $fsyv = fsv/1.2$, from table 5.1, TBDY.
 y_1, sh_1, ft_1, fy_1 , are also multiplied by $\text{Min}(1, 1.25 * (lb/ld)^{2/3})$, from 10.3.5, ASCE 41-17.
 with $fsv = (fs, \text{jacket} * Asl, \text{mid, jacket} + fs, \text{mid} * Asl, \text{mid, core}) / Asl, \text{mid} = 749.2935$
 with $Es_v = (Es, \text{jacket} * Asl, \text{mid, jacket} + Es, \text{mid} * Asl, \text{mid, core}) / Asl, \text{mid} = 200000.00$
 $1 = Asl, \text{ten} / (b * d) * (fs_1 / f_c) = 0.15845977$
 $2 = Asl, \text{com} / (b * d) * (fs_2 / f_c) = 0.15845977$
 $v = Asl, \text{mid} / (b * d) * (fsv / f_c) = 0.36847443$
 and confined core properties:
 $b = 390.00$
 $d = 877.00$
 $d' = 13.00$
 $f_{cc} (5A.2, \text{TBDY}) = 25.58352$
 $cc (5A.5, \text{TBDY}) = 0.00621307$
 $c = \text{confinement factor} = 1.42131$
 $1 = Asl, \text{ten} / (b * d) * (fs_1 / f_c) = 0.18909264$
 $2 = Asl, \text{com} / (b * d) * (fs_2 / f_c) = 0.18909264$
 $v = Asl, \text{mid} / (b * d) * (fsv / f_c) = 0.43970658$
 Case/Assumption: Unconfined full section - Steel rupture
 ' satisfies Eq. (4.3)

```

---->
v < vs,y2 - LHS eq.(4.5) is not satisfied
---->
v < vs,c - RHS eq.(4.5) is not satisfied
---->
Case/Assumption Rejected.
---->
New Case/Assumption: Unconfined full section - Spalling of concrete cover
'satisfies Eq. (4.4)
---->
v < s,y1 - LHS eq.(4.7) is not satisfied
---->
v < vc,y1 - RHS eq.(4.6) is satisfied
---->
cu (4.10) = 0.83934343
MRc (4.17) = 1.8439E+009
---->
New Case/Assumption: Ultimate curvature of confined core after spalling of concrete cover
In expressions below, the following modifications have been made
- b, d, d' replaced by geometric parameters of the core: bo, do, d'o
- N, 1, 2, v normalised to bo*do, instead of b*d
- parameters of confined concrete, fcc, cc, used in lieu of fc, ecu
---->
Subcase: Rupture of tension steel
---->
v* < v*s,y2 - LHS eq.(4.5) is not satisfied
---->
v* < v*s,c - LHS eq.(4.5) is not satisfied
---->
Subcase rejected
---->
New Subcase: Failure of compression zone
---->
v* < v*c,y2 - LHS eq.(4.6) is not satisfied
---->
v* < v*c,y1 - RHS eq.(4.6) is not satisfied
---->
*cu (4.11) = 0.89983732
MRo (4.18) = 1.8431E+009
---->
u = cu (4.2) = 7.8730391E-006
Mu = MRo

```

Calculation of ratio lb/d

Adequate Lap Length: lb/d >= 1

Calculation of Mu2-

Calculation of ultimate curvature u according to 4.1, Biskinis/Fardis 2013:

u = 7.8730391E-006
Mu = 1.8431E+009

with full section properties:

b = 450.00
d = 907.00
d' = 43.00
v = 1.0459
N = 7.6839E+006
fc = 18.00
co (5A.5, TBDY) = 0.002
Final value of cu: cu* = shear_factor * Max(cu, cc) = 0.01486841

The Shear_factor is considered equal to 1 (pure moment strength)

From (5.4b), TBDY: $c_u = 0.01486841$

w_e (5.4c) = 0.08077545

a_{se} ((5.4d), TBDY) = $(a_{se1} * A_{ext} + a_{se2} * A_{int}) / A_{sec} = 0.53375773$

$a_{se1} = \text{Max}(((A_{conf,max1} - A_{noConf1}) / A_{conf,max1}) * (A_{conf,min1} / A_{conf,max1}), 0) = 0.53375773$

The definitions of A_{noConf} , $A_{conf,min}$ and $A_{conf,max}$ are derived from generalization of the rectangular sections confinement, which is expressed by (5.4d).

The generalization is done according to Mander, J., Priestley, M., and Park, R. (1988)

"Theoretical Stress-Strain Model for Confined Concrete."

J. Struct. Eng., 10.1061/(ASCE)0733-9445(1988)114:8(1804), 1804-1826.

$A_{conf,max1} = 464100.00$ is the confined core area at levels of member with hoops and is calculated as the area of core enclosed by the center lines of the external perimeter hoops.

$A_{conf,min1} = 395025.00$ is the confined core area at midway between the levels of hoops and is calculated by reducing all the dimensions of the area $A_{conf,max1}$ by a length equal to half the clear spacing between external hoops.

$A_{noConf1} = 173066.667$ is the unconfined external core area which is equal to $b_i^2/6$ as defined at (A.2).

$a_{se2} (>= a_{se1}) = \text{Max}(((A_{conf,max2} - A_{noConf2}) / A_{conf,max2}) * (A_{conf,min2} / A_{conf,max2}), 0) = 0.53375773$

The definitions of A_{noConf} , $A_{conf,min}$ and $A_{conf,max}$ are derived from generalization of the rectangular sections confinement, which is expressed by (5.4d).

The generalization is done according to Mander, J., Priestley, M., and Park, R. (1988)

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$A_{conf,max2} = 252164.00$ is the confined core area at levels of member with hoops and is calculated as the area of core enclosed by the center lines of the internal perimeter hoops.

$A_{conf,min2} = 111441.00$ is the confined core area at midway between the levels of hoops and is calculated by reducing all the dimensions of the area $A_{conf,max2}$ by a length equal to half the clear spacing between internal hoops.

$A_{noConf2} = 110709.333$ is the unconfined internal core area which is equal to $b_i^2/6$ as defined at (A.2).

$p_{sh,min} * F_{ywe} = \text{Min}(p_{sh,x} * F_{ywe}, p_{sh,y} * F_{ywe}) = 2.724$

$p_{sh,x} * F_{ywe} = p_{sh1} * F_{ywe1} + p_{sh2} * F_{ywe2} = 2.724$

p_{sh1} ((5.4d), TBDY) = $L_{stir1} * A_{stir1} / (A_{sec} * s_1) = 0.00301593$

L_{stir1} (Length of stirrups along Y) = 2160.00

A_{stir1} (stirrups area) = 78.53982

p_{sh2} (5.4d) = $L_{stir2} * A_{stir2} / (A_{sec} * s_2) = 0.00056047$

L_{stir2} (Length of stirrups along Y) = 1568.00

A_{stir2} (stirrups area) = 50.26548

$p_{sh,y} * F_{ywe} = p_{sh1} * F_{ywe1} + p_{sh2} * F_{ywe2} = 3.25416$

p_{sh1} ((5.4d), TBDY) = $L_{stir1} * A_{stir1} / (A_{sec} * s_1) = 0.00357443$

L_{stir1} (Length of stirrups along X) = 2560.00

A_{stir1} (stirrups area) = 78.53982

p_{sh2} ((5.4d), TBDY) = $L_{stir2} * A_{stir2} / (A_{sec} * s_2) = 0.00070345$

L_{stir2} (Length of stirrups along X) = 1968.00

A_{stir2} (stirrups area) = 50.26548

$A_{sec} = 562500.00$

$s_1 = 100.00$

$s_2 = 250.00$

$f_{ywe1} = 781.25$

$f_{ywe2} = 656.25$

$f_{ce} = 18.00$

From ((5.A5), TBDY), TBDY: $c_c = 0.00621307$

c = confinement factor = 1.42131

$y_1 = 0.0025$

$sh_1 = 0.008$

$ft_1 = 907.50$

$fy_1 = 756.25$

$su_1 = 0.032$

using (30) in Biskinis/Fardis (2013) multiplied with shear_factor

and also multiplied by the shear_factor according to 15.7.1.4, with

Shear_factor = 1.00

$l_o/l_{ou,min} = l_b/l_d = 1.00$

$su_1 = 0.4 * esu_{1,nominal}$ ((5.5), TBDY) = 0.032

From table 5A.1, TBDY: $esu_{1,nominal} = 0.08$,

For calculation of $esu_{1,nominal}$ and y_1 , sh_1 , ft_1 , fy_1 , it is considered characteristic value $fsy_1 = fs_1/1.2$, from table 5.1, TBDY.

$y1, sh1, ft1, fy1$, are also multiplied by $\text{Min}(1, 1.25 \cdot (lb/ld)^{2/3})$, from 10.3.5, ASCE 41-17.
 with $fs1 = (fs_{jacket} \cdot Asl_{ten,jacket} + fs_{core} \cdot Asl_{ten,core}) / Asl_{ten} = 756.25$
 with $Es1 = (Es_{jacket} \cdot Asl_{ten,jacket} + Es_{core} \cdot Asl_{ten,core}) / Asl_{ten} = 200000.00$
 $y2 = 0.0025$
 $sh2 = 0.008$
 $ft2 = 907.50$
 $fy2 = 756.25$
 $su2 = 0.032$
 using (30) in Biskinis/Fardis (2013) multiplied with shear_factor
 and also multiplied by the shear_factor according to 15.7.1.4, with
 Shear_factor = 1.00
 $lo/lou, min = lb/lb, min = 1.00$
 $su2 = 0.4 \cdot esu2_{nominal} ((5.5), TBDY) = 0.032$
 From table 5A.1, TBDY: $esu2_{nominal} = 0.08$,
 For calculation of $esu2_{nominal}$ and $y2, sh2, ft2, fy2$, it is considered
 characteristic value $fsy2 = fs2/1.2$, from table 5.1, TBDY.
 $y1, sh1, ft1, fy1$, are also multiplied by $\text{Min}(1, 1.25 \cdot (lb/ld)^{2/3})$, from 10.3.5, ASCE 41-17.
 with $fs2 = (fs_{jacket} \cdot Asl_{com,jacket} + fs_{core} \cdot Asl_{com,core}) / Asl_{com} = 756.25$
 with $Es2 = (Es_{jacket} \cdot Asl_{com,jacket} + Es_{core} \cdot Asl_{com,core}) / Asl_{com} = 200000.00$
 $yv = 0.0025$
 $shv = 0.008$
 $ftv = 899.1522$
 $fyv = 749.2935$
 $suv = 0.032$
 using (30) in Biskinis/Fardis (2013) multiplied with shear_factor
 and also multiplied by the shear_factor according to 15.7.1.4, with
 Shear_factor = 1.00
 $lo/lou, min = lb/ld = 1.00$
 $suv = 0.4 \cdot esuv_{nominal} ((5.5), TBDY) = 0.032$
 From table 5A.1, TBDY: $esuv_{nominal} = 0.08$,
 considering characteristic value $fsyv = fsv/1.2$, from table 5.1, TBDY
 For calculation of $esuv_{nominal}$ and yv, shv, ftv, fyv , it is considered
 characteristic value $fsyv = fsv/1.2$, from table 5.1, TBDY.
 $y1, sh1, ft1, fy1$, are also multiplied by $\text{Min}(1, 1.25 \cdot (lb/ld)^{2/3})$, from 10.3.5, ASCE 41-17.
 with $fsv = (fs_{jacket} \cdot Asl_{mid,jacket} + fs_{mid} \cdot Asl_{mid,core}) / Asl_{mid} = 749.2935$
 with $Es_v = (Es_{jacket} \cdot Asl_{mid,jacket} + Es_{mid} \cdot Asl_{mid,core}) / Asl_{mid} = 200000.00$
 $1 = Asl_{ten} / (b \cdot d) \cdot (fs1 / fc) = 0.15845977$
 $2 = Asl_{com} / (b \cdot d) \cdot (fs2 / fc) = 0.15845977$
 $v = Asl_{mid} / (b \cdot d) \cdot (fsv / fc) = 0.36847443$
 and confined core properties:
 $b = 390.00$
 $d = 877.00$
 $d' = 13.00$
 $fcc (5A.2, TBDY) = 25.58352$
 $cc (5A.5, TBDY) = 0.00621307$
 $c = \text{confinement factor} = 1.42131$
 $1 = Asl_{ten} / (b \cdot d) \cdot (fs1 / fc) = 0.18909264$
 $2 = Asl_{com} / (b \cdot d) \cdot (fs2 / fc) = 0.18909264$
 $v = Asl_{mid} / (b \cdot d) \cdot (fsv / fc) = 0.43970658$
 Case/Assumption: Unconfined full section - Steel rupture
 ' satisfies Eq. (4.3)
 --->
 $v < v_{s,y2}$ - LHS eq.(4.5) is not satisfied
 --->
 $v < v_{s,c}$ - RHS eq.(4.5) is not satisfied
 --->
 Case/Assumption Rejected.
 --->
 New Case/Assumption: Unconfined full section - Spalling of concrete cover
 ' satisfies Eq. (4.4)
 --->
 $v < s_{y1}$ - LHS eq.(4.7) is not satisfied
 --->
 $v < v_{c,y1}$ - RHS eq.(4.6) is satisfied
 --->
 $cu (4.10) = 0.83934343$
 $MRC (4.17) = 1.8439E+009$

```

---->
New Case/Assumption: Ultimate curvature of confined core after spalling of concrete cover
In expressions below, the following modifications have been made
- b, d, d' replaced by geometric parameters of the core: bo, do, d'o
- N,  $\phi$ ,  $\rho$ ,  $\nu$  normalised to  $b_o*d_o$ , instead of  $b*d$ 
-  $\rho$ ,  $\rho_c$  parameters of confined concrete,  $\rho_{cc}$ ,  $\rho_{cc}$ , used in lieu of  $\rho_c$ ,  $\rho_{cc}$ 
---->
Subcase: Rupture of tension steel
---->
 $\nu^* < \nu^*s_y2$  - LHS eq.(4.5) is not satisfied
---->
 $\nu^* < \nu^*s_c$  - LHS eq.(4.5) is not satisfied
---->
Subcase rejected
---->
New Subcase: Failure of compression zone
---->
 $\nu^* < \nu^*c_y2$  - LHS eq.(4.6) is not satisfied
---->
 $\nu^* < \nu^*c_y1$  - RHS eq.(4.6) is not satisfied
---->
 $\phi_{cu}$  (4.11) = 0.89983732
MRo (4.18) = 1.8431E+009
---->
 $\phi_u = \phi_{cu}$  (4.2) = 7.8730391E-006
Mu = MRo
-----

Calculation of ratio lb/d
-----
Adequate Lap Length: lb/d >= 1
-----
-----
-----
Calculation of Shear Strength  $V_r = \text{Min}(V_{r1}, V_{r2}) = 1.4186E+006$ 
-----
Calculation of Shear Strength at edge 1,  $V_{r1} = 1.4186E+006$ 
 $V_{r1} = V_{col}$  ((10.3), ASCE 41-17) =  $k_n l^* V_{col0}$ 
 $V_{col0} = 1.4186E+006$ 
 $k_n l = 1$  (zero step-static loading)
-----
NOTE: In expression (10-3) ' $V_s = A_v f_y d/s$ ' is replaced by ' $V_s + \phi^* V_f$ '
where  $V_f$  is the contribution of FRPs (11.3), ACI 440).
-----
= 1 (normal-weight concrete)
Mean concrete strength:  $f'_c = (f'_c \text{ jacket} * \text{Area jacket} + f'_c \text{ core} * \text{Area core}) / \text{Area section} = 15.20$ , but  $f'_c^{0.5} \leq 8.3$ 
MPa (22.5.3.1, ACI 318-14)
M/Vd = 4.00
Mu = 2.6919E+008
Vu = 137.4894
d = 0.8*h = 760.00
Nu = 7.6839E+006
Ag = 427500.00
From (11.5.4.8), ACI 318-14:  $V_s = V_{sjacket} + V_{s,core} = 1.2262E+006$ 
where:
 $V_{sjacket} = V_{sj1} + V_{sj2} = 1.0996E+006$ 
 $V_{sj1} = 353429.174$  is calculated for section web jacket, with:
d = 360.00
 $A_v = 157079.633$ 
 $f_y = 625.00$ 
s = 100.00
 $V_{sj1}$  is multiplied by  $\phi_{col,j1} = 1.00$ 
s/d = 0.27777778
 $V_{sj2} = 746128.255$  is calculated for section flange jacket, with:
d = 760.00
 $A_v = 157079.633$ 

```

$f_y = 625.00$
 $s = 100.00$
 $V_{s,j2}$ is multiplied by $Col,j2 = 1.00$
 $s/d = 0.13157895$
 $V_{s,core} = V_{s,c1} + V_{s,c2} = 126669.016$
 $V_{s,c1} = 0.00$ is calculated for section web core, with:
 $d = 200.00$
 $A_v = 100530.965$
 $f_y = 525.00$
 $s = 250.00$
 $V_{s,c1}$ is multiplied by $Col,c1 = 0.00$
 $s/d = 1.25$
 $V_{s,c2} = 126669.016$ is calculated for section flange core, with:
 $d = 600.00$
 $A_v = 100530.965$
 $f_y = 525.00$
 $s = 250.00$
 $V_{s,c2}$ is multiplied by $Col,c2 = 1.00$
 $s/d = 0.41666667$
 $V_f ((11-3)-(11.4), ACI 440) = 0.00$
 From (11-11), ACI 440: $V_s + V_f \leq 885757.128$
 $bw = 450.00$

Calculation of Shear Strength at edge 2, $V_{r2} = 1.4186E+006$

$V_{r2} = V_{Col} ((10.3), ASCE 41-17) = knl * V_{Col0}$

$V_{Col0} = 1.4186E+006$

$knl = 1$ (zero step-static loading)

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

$= 1$ (normal-weight concrete)

Mean concrete strength: $f'_c = (f'_c_jacket * Area_jacket + f'_c_core * Area_core) / Area_section = 15.20$, but $f'_c^{0.5} \leq 8.3$ MPa (22.5.3.1, ACI 318-14)

$M/Vd = 4.00$

$\mu_u = 2.5135E+008$

$V_u = 137.4894$

$d = 0.8 * h = 760.00$

$N_u = 7.6839E+006$

$A_g = 427500.00$

From (11.5.4.8), ACI 318-14: $V_s = V_{s,jacket} + V_{s,core} = 1.2262E+006$

where:

$V_{s,jacket} = V_{s,j1} + V_{s,j2} = 1.0996E+006$

$V_{s,j1} = 353429.174$ is calculated for section web jacket, with:

$d = 360.00$

$A_v = 157079.633$

$f_y = 625.00$

$s = 100.00$

$V_{s,j1}$ is multiplied by $Col,j1 = 1.00$

$s/d = 0.27777778$

$V_{s,j2} = 746128.255$ is calculated for section flange jacket, with:

$d = 760.00$

$A_v = 157079.633$

$f_y = 625.00$

$s = 100.00$

$V_{s,j2}$ is multiplied by $Col,j2 = 1.00$

$s/d = 0.13157895$

$V_{s,core} = V_{s,c1} + V_{s,c2} = 126669.016$

$V_{s,c1} = 0.00$ is calculated for section web core, with:

$d = 200.00$

$A_v = 100530.965$

$f_y = 525.00$

$s = 250.00$

$V_{s,c1}$ is multiplied by $Col,c1 = 0.00$

$s/d = 1.25$

$V_{s,c2} = 126669.016$ is calculated for section flange core, with:

d = 600.00
Av = 100530.965
fy = 525.00
s = 250.00
Vs,c2 is multiplied by Col,c2 = 1.00
s/d = 0.41666667
Vf ((11-3)-(11.4), ACI 440) = 0.00
From (11-11), ACI 440: Vs + Vf <= 885757.128
bw = 450.00

End Of Calculation of Shear Capacity ratio for element: column JTC1 of floor 1
At local axis: 2

Start Of Calculation of Chord Rotation Capacity for element: column JTC1 of floor 1
At local axis: 3
Integration Section: (b)
Section Type: rcjtcs

Constant Properties

Knowledge Factor, γ = 0.80
Chord Rotation is generally considered as Deformation-Controlled Action according to Table C7-1, ASCE 41-17.
Mean strengths are used for Deformation-Controlled Actions according to 7.5.1.3, ASCE 41-17
Consequently:
Jacket
New material of Primary Member: Concrete Strength, $f_c = f_{cm} = 18.00$
New material of Primary Member: Steel Strength, $f_s = f_{sm} = 625.00$
Concrete Elasticity, $E_c = 19940.411$
Steel Elasticity, $E_s = 200000.00$
Existing Column
Existing material of Primary Member: Concrete Strength, $f_c = f_{cm} = 12.00$
Existing material of Primary Member: Steel Strength, $f_s = f_{sm} = 525.00$
Concrete Elasticity, $E_c = 16281.278$
Steel Elasticity, $E_s = 200000.00$
Max Height, $H_{max} = 750.00$
Min Height, $H_{min} = 450.00$
Max Width, $W_{max} = 950.00$
Min Width, $W_{min} = 450.00$
Eccentricity, $E_{cc} = 250.00$
Jacket Thickness, $t_j = 100.00$
Cover Thickness, $c = 25.00$
Element Length, $L = 3000.00$
Primary Member
Ribbed Bars
Ductile Steel
With Detailing for Earthquake Resistance (including stirrups closed at 135°)
Longitudinal Bars With Ends Lapped Starting at the End Sections
Adequate Lap Length ($l_b/l_d >= 1$)
No FRP Wrapping

Stepwise Properties

Bending Moment, $M = -4.0305E+008$
Shear Force, $V_2 = 455948.704$
Shear Force, $V_3 = -29.64437$
Axial Force, $F = -7.6755E+006$
Longitudinal Reinforcement Area Distribution (in 2 divisions)
-Tension: $As_t = 314.1593$
-Compression: $As_c = 6377.433$
Longitudinal Reinforcement Area Distribution (in 3 divisions)
-Tension: $As_{l,ten} = 1539.38$
-Compression: $As_{l,com} = 1539.38$

-Middle: $Asl_{mid} = 3612.832$
 Longitudinal External Reinforcement Area Distribution (in 3 divisions)
 -Tension: $Asl_{ten,jacket} = 1231.504$
 -Compression: $Asl_{com,jacket} = 1231.504$
 -Middle: $Asl_{mid,jacket} = 2689.203$
 Longitudinal Internal Reinforcement Area Distribution (in 3 divisions)
 -Tension: $Asl_{ten,core} = 307.8761$
 -Compression: $Asl_{com,core} = 307.8761$
 -Middle: $Asl_{mid,core} = 923.6282$
 Mean Diameter of Tension Reinforcement, $DbL = 16.57143$

Existing component: From table 7-7, ASCE 41_17: Final chord rotation Capacity $u_R = \frac{1}{2} u = 0.00073439$
 $u = y + p = 0.00091799$

- Calculation of y -

$y = (My * Ls / 3) / Eleff = 0.00091799$ ((4.29), Biskinis Phd))
 $My = 1.4240E+009$
 $Ls = M/V$ (with $Ls > 0.1 * L$ and $Ls < 2 * L$) = 883.982
 From table 10.5, ASCE 41_17: $Eleff = factor * Ec * Ig = 4.5707E+014$
 $factor = 0.70$
 $Ag = 562500.00$
 Mean concrete strength: $fc' = (fc'_{jacket} * Area_{jacket} + fc'_{core} * Area_{core}) / Area_{section} = 15.20$
 $N = 7.6755E+006$
 $Ec * Ig = Ec_{jacket} * Ig_{jacket} + Ec_{core} * Ig_{core} = 6.5295E+014$

Calculation of Yielding Moment My

Calculation of y and My according to Annex 7 -

$y = \text{Min}(y_{ten}, y_{com})$
 $y_{ten} = 7.1477677E-006$
 with $fy = 601.9953$
 $d = 907.00$
 $y = 0.53571421$
 $A = 0.04763352$
 $B = 0.03982468$
 with $pt = 0.0037716$
 $pc = 0.0037716$
 $pv = 0.00885172$
 $N = 7.6755E+006$
 $b = 450.00$
 $" = 0.04740904$
 $y_{comp} = 1.7850636E-006$
 with $fc = 18.00$
 $Ec = 19940.411$
 $y = 1.00358$
 $A = -0.04147374$
 $B = 0.0085861$
 with $Es = 200000.00$

Calculation of ratio lb/d

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

- Calculation of p -

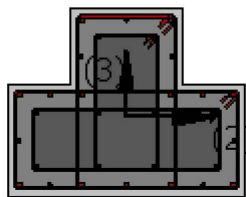
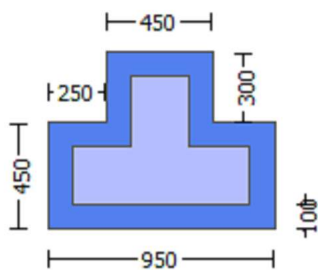
From table 10-8: $p = 0.00$
 with:

- Columns not controlled by inadequate development or splicing along the clear height because $l_b/l_d \geq 1$
shear control ratio $V_y E / V_{ColOE} = 0.86614729$
 $d = d_{external} = 907.00$
 $s = s_{external} = 50.00$
- $t = s_1 + s_2 + 2 * t_f / b_w * (f_{fe} / f_s) = 0.00427788$
jacket: $s_1 = A_{v1} * L_{stir1} / (s_1 * A_g) = 0.00357443$
 $A_{v1} = 78.53982$, is the area of every stirrup parallel to loading (shear) direction
 $L_{stir1} = 2560.00$, is the total Length of all stirrups parallel to loading (shear) direction
 $s_1 = 100.00$
core: $s_2 = A_{v2} * L_{stir2} / (s_2 * A_g) = 0.00070345$
 $A_{v2} = 50.26548$, is the area of every stirrup parallel to loading (shear) direction
 $L_{stir2} = 1968.00$, is the total Length of all stirrups parallel to loading (shear) direction
 $s_2 = 250.00$
The term $2 * t_f / b_w * (f_{fe} / f_s)$ is implemented to account for FRP contribution
where $f = 2 * t_f / b_w$ is FRP ratio (EC8 - 3, A.4.4.3(6)) and f_{fe} / f_s normalises f to steel strength
All these variables have already been given in Shear control ratio calculation.
For the normalisation f_s of jacket is used.
 $NUD = 7.6755E+006$
 $A_g = 562500.00$
 $f_{cE} = (f_{c,jacket} * Area_{jacket} + f_{c,core} * Area_{core}) / section_area = 15.20$
 $f_{yIE} = (f_{y,ext_Long_Reinf} * Area_{ext_Long_Reinf} + f_{y,int_Long_Reinf} * Area_{int_Long_Reinf}) / Area_{Tot_Long_Rein} = 601.9953$
 $f_{yIE} = (f_{y,ext_Trans_Reinf} * s_1 + f_{y,int_Trans_Reinf} * s_2) / (s_1 + s_2) = 608.5561$
 $\rho_l = Area_{Tot_Long_Rein} / (b * d) = 0.01639493$
 $b = 450.00$
 $d = 907.00$
 $f_{cE} = 15.20$

End Of Calculation of Chord Rotation Capacity for element: column JTC1 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 50 and 51)
Analysis: Uniform +X
Check: Shear capacity V_{Rd}
Edge: Start
Local Axis: (2)



Start Of Calculation of Shear Capacity for element: column JTC1 of floor 1

At local axis: 2

Integration Section: (a)

Section Type: rcjtcs

Constant Properties

Knowledge Factor, $\gamma = 0.80$

Member Shear Force is generally considered as Force-Controlled Action according to Table C7-1, ASCE 41-17.

Lower-bound strengths are used for Force-Controlled Actions according to 7.5.1.3, ASCE 41-17

Consequently:

Jacket

New material of Primary Member: Concrete Strength, $f_c = f_{c_lower_bound} = 12.00$

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

Concrete Elasticity, $E_c = 19940.411$

Steel Elasticity, $E_s = 200000.00$

Existing Column

Existing material of Primary Member: Concrete Strength, $f_c = f_{c_lower_bound} = 8.00$

Existing material of Primary Member: Steel Strength, $f_s = f_{s_lower_bound} = 420.00$

Concrete Elasticity, $E_c = 16281.278$

Steel Elasticity, $E_s = 200000.00$

#####

Note: Especially for the calculation of γ for displacement ductility demand, the expected (mean value) strengths are used (7.5.1.3, ASCE 41-17) because bending is considered as Deformation-Controlled Action (Table C7-1, ASCE 41-17).

Jacket

New material: Concrete Strength, $f_c = f_{cm} = 18.00$

New material: Steel Strength, $f_s = f_{sm} = 625.00$

Existing Column

Existing material: Concrete Strength, $f_c = f_{cm} = 12.00$

Existing material: Steel Strength, $f_s = f_{sm} = 525.00$

#####

Max Height, $H_{max} = 750.00$

Min Height, $H_{min} = 450.00$

Max Width, $W_{max} = 950.00$

Min Width, $W_{min} = 450.00$

Eccentricity, $Ecc = 250.00$

Jacket Thickness, $t_j = 100.00$

Cover Thickness, $c = 25.00$

Element Length, $L = 3000.00$

Primary Member

Ribbed Bars

Ductile Steel

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

Longitudinal Bars With Ends Lapped Starting at the End Sections

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

No FRP Wrapping

Stepwise Properties

EDGE -A-

Bending Moment, $M_a = -6.8503E+008$
 Shear Force, $V_a = -212214.235$
 EDGE -B-
 Bending Moment, $M_b = -4.7511E+008$
 Shear Force, $V_b = 212214.235$
 BOTH EDGES
 Axial Force, $F = -7.0682E+006$
 Longitudinal Reinforcement Area Distribution (in 2 divisions)
 -Tension: $A_{sl,t} = 2522.699$
 -Compression: $A_{sl,c} = 4168.893$
 Longitudinal Reinforcement Area Distribution (in 3 divisions)
 -Tension: $A_{sl,ten} = 1539.38$
 -Compression: $A_{sl,com} = 1539.38$
 -Middle: $A_{sl,mid} = 3612.832$
 Mean Diameter of Tension Reinforcement, $Db_{L,ten} = 16.57143$

Existing component: From table 7-7, ASCE 41_17: Final Shear Capacity $V_R = V_n = 662172.296$
 $V_n ((10.3), ASCE 41-17) = knl * V_{ColO} = 827715.37$
 $V_{Col} = 1.1825E+006$
 $knl = 0.70$
 $displacement_ductility_demand = 6.86799$

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

$= 1$ (normal-weight concrete)
 Mean concrete strength: $f'_c = (f'_{c,jacket} * Area_{jacket} + f'_{c,core} * Area_{core}) / Area_{section} = 10.13333$, but $f'_c^{0.5} \leq 8.3$ MPa (22.5.3.1, ACI 318-14)
 $M/Vd = 4.00$
 $M_u = 6.8503E+008$
 $V_u = 212214.235$
 $d = 0.8 * h = 760.00$
 $N_u = 7.0682E+006$
 $A_g = 427500.00$
 From (11.5.4.8), ACI 318-14: $V_s = V_{s,jacket} + V_{s,core} = 980981.156$
 where:
 $V_{s,jacket} = V_{sj1} + V_{sj2} = 879645.943$
 $V_{sj1} = 282743.339$ is calculated for section web jacket, with:
 $d = 360.00$
 $A_v = 157079.633$
 $f_y = 500.00$
 $s = 100.00$
 V_{sj1} is multiplied by $Col_{j1} = 1.00$
 $s/d = 0.27777778$
 $V_{sj2} = 596902.604$ is calculated for section flange jacket, with:
 $d = 760.00$
 $A_v = 157079.633$
 $f_y = 500.00$
 $s = 100.00$
 V_{sj2} is multiplied by $Col_{j2} = 1.00$
 $s/d = 0.13157895$
 $V_{s,core} = V_{sc1} + V_{sc2} = 101335.213$
 $V_{sc1} = 0.00$ is calculated for section web core, with:
 $d = 200.00$
 $A_v = 100530.965$
 $f_y = 420.00$
 $s = 250.00$
 V_{sc1} is multiplied by $Col_{c1} = 0.00$
 $s/d = 1.25$
 $V_{sc2} = 101335.213$ is calculated for section flange core, with:
 $d = 600.00$
 $A_v = 100530.965$
 $f_y = 420.00$
 $s = 250.00$

$V_{s,c2}$ is multiplied by $Col_{c2} = 1.00$
 $s/d = 0.41666667$
 $V_f((11-3)-(11.4), ACI 440) = 0.00$
From (11-11), ACI 440: $V_s + V_f \leq 723217.666$
 $bw = 450.00$

displacement ductility demand is calculated as ϕ / y

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

From analysis, chord rotation $\theta = 0.02430692$
 $y = (M_y * L_s / 3) / E_{eff} = 0.00353916$ ((4.29), Biskinis Phd))
 $M_y = 1.5034E+009$
 $L_s = M/V$ (with $L_s > 0.1 * L$ and $L_s < 2 * L$) = 3228.019
From table 10.5, ASCE 41_17: $E_{eff} = factor * E_c * I_g = 4.5707E+014$
factor = 0.70
 $A_g = 562500.00$
Mean concrete strength: $f'_c = (f'_{c,jacket} * Area_{jacket} + f'_{c,core} * Area_{core}) / Area_{section} = 15.20$
 $N = 7.0682E+006$
 $E_c * I_g = E_{c,jacket} * I_{g,jacket} + E_{c,core} * I_{g,core} = 6.5295E+014$

Calculation of Yielding Moment M_y

Calculation of ϕ and M_y according to Annex 7 -

$y = \min(\phi_{ten}, \phi_{com})$
 $\phi_{ten} = 6.9719165E-006$
with $f_y = 601.9953$
 $d = 907.00$
 $y = 0.52400363$
 $A = 0.04516194$
 $B = 0.0373531$
with $p_t = 0.0037716$
 $p_c = 0.0037716$
 $p_v = 0.00885172$
 $N = 7.0682E+006$
 $b = 450.00$
 $\phi = 0.04740904$
 $\phi_{comp} = 1.9344180E-006$
with $f_c = 18.00$
 $E_c = 19940.411$
 $y = 0.92609018$
 $A = -0.0368952$
 $B = 0.0085861$
with $E_s = 200000.00$

Calculation of ratio I_b/I_d

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

End Of Calculation of Shear Capacity for element: column JTC1 of floor 1
At local axis: 2
Integration Section: (a)

Calculation No. 10

column C1, Floor 1

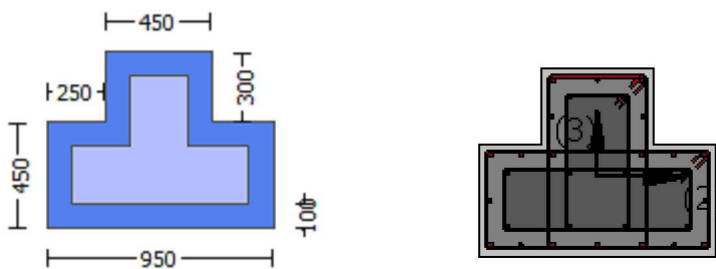
Limit State: Life Safety (data interpolation between analysis steps 50 and 51)

Analysis: Uniform +X

Check: Chord rotation capacity (μ)

Edge: Start

Local Axis: (2)



Start Of Calculation of Shear Capacity ratio for element: column JTC1 of floor 1

At Shear local axis: 3

(Bending local axis: 2)

Section Type: rcjtcs

Constant Properties

Knowledge Factor, $\gamma = 0.80$

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

Consequently:

Jacket

New material of Primary Member: Concrete Strength, $f_c = f_{cm} = 18.00$

New material of Primary Member: Steel Strength, $f_s = f_{sm} = 625.00$

Concrete Elasticity, $E_c = 19940.411$

Steel Elasticity, $E_s = 200000.00$

Existing Column

Existing material of Primary Member: Concrete Strength, $f_c = f_{cm} = 12.00$

Existing material of Primary Member: Steel Strength, $f_s = f_{sm} = 525.00$

Concrete Elasticity, $E_c = 16281.278$

Steel Elasticity, $E_s = 200000.00$

#####

Note: Especially for the calculation of moment strengths,

the above steel re-bar strengths are multiplied by 1.25 according to R18.6.5, ACI 318-14

Jacket

New material: Steel Strength, $f_s = 1.25 \cdot f_{sm} = 781.25$

Existing Column

Existing material: Steel Strength, $f_s = 1.25 \cdot f_{sm} = 656.25$

#####

Max Height, $H_{max} = 750.00$

Min Height, $H_{min} = 450.00$

Max Width, $W_{max} = 950.00$

Min Width, $W_{min} = 450.00$

Eccentricity, Ecc = 250.00
 Jacket Thickness, tj = 100.00
 Cover Thickness, c = 25.00
 Mean Confinement Factor overall section = 1.42131
 Element Length, L = 3000.00
 Primary Member
 Ribbed Bars
 Ductile Steel
 With Detailing for Earthquake Resistance (including stirrups closed at 135°)
 Longitudinal Bars With Ends Lapped Starting at the End Sections
 Adequate Lap Length ($l_o/l_{o,u}, \min > 1$)
 No FRP Wrapping

Stepwise Properties

At local axis: 3
 EDGE -A-
 Shear Force, $V_a = -227.4778$
 EDGE -B-
 Shear Force, $V_b = 227.4778$
 BOTH EDGES
 Axial Force, $F = -7.6839E+006$
 Longitudinal Reinforcement Area Distribution (in 2 divisions)
 -Tension: $A_{st} = 314.1593$
 -Compression: $A_{sc} = 6176.371$
 Longitudinal Reinforcement Area Distribution (in 3 divisions)
 -Tension: $A_{st,ten} = 1539.38$
 -Compression: $A_{st,com} = 2475.575$
 -Middle: $A_{st,mid} = 2676.637$

Calculation of Shear Capacity ratio, $V_e/V_r = 1.74773$
 Member Controlled by Shear ($V_e/V_r > 1$)
 Calculation of Shear Demand from fig. R18.6.5, ACI 318-14 $V_e = (M_{pr1} + M_{pr2})/l_n = 2.0410E+006$
 with
 $M_{pr1} = \max(\mu_{u1+}, \mu_{u1-}) = 3.0615E+009$
 $\mu_{u1+} = 3.0615E+009$, is the ultimate moment strength at the edge 1 of the member in the actual moment direction which is defined for the static loading combination
 $\mu_{u1-} = 8.9683E+008$, is the ultimate moment strength at the edge 1 of the member in the opposite moment direction which is defined for the static loading combination
 $M_{pr2} = \max(\mu_{u2+}, \mu_{u2-}) = 3.0615E+009$
 $\mu_{u2+} = 3.0615E+009$, is the ultimate moment strength at the edge 2 of the member in the actual moment direction which is defined for the static loading combination
 $\mu_{u2-} = 8.9683E+008$, is the ultimate moment strength at the edge 2 of the member in the opposite moment direction which is defined for the static loading combination

Calculation of μ_{u1+}

Calculation of ultimate curvature ϕ_u according to 4.1, Biskinis/Fardis 2013:
 $\phi_u = 1.3930706E-005$
 $\mu_u = 3.0615E+009$

with full section properties:

$b = 950.00$
 $d = 707.00$
 $d' = 43.00$
 $\nu = 0.63557247$
 $N = 7.6839E+006$
 $f_c = 18.00$
 $\alpha (5A.5, \text{TB DY}) = 0.002$
 Final value of ϕ_u : $\phi_u^* = \text{shear_factor} * \max(\phi_u, \phi_c) = 0.01486841$

The Shear_factor is considered equal to 1 (pure moment strength)

From (5.4b), TBDY: $c_u = 0.01486841$

w_e (5.4c) = 0.08077545

a_{se} ((5.4d), TBDY) = $(a_{se1} \cdot A_{ext} + a_{se2} \cdot A_{int}) / A_{sec} = 0.53375773$

$a_{se1} = \text{Max}(((A_{conf,max1} - A_{noConf1}) / A_{conf,max1}) \cdot (A_{conf,min1} / A_{conf,max1}), 0) = 0.53375773$

The definitions of A_{noConf} , $A_{conf,min}$ and $A_{conf,max}$ are derived from generalization of the rectangular sections confinement, which is expressed by (5.4d).

The generalization is done according to Mander, J., Priestley, M., and Park, R. (1988)

"Theoretical Stress-Strain Model for Confined Concrete."

J. Struct. Eng., 10.1061/(ASCE)0733-9445(1988)114:8(1804), 1804-1826.

$A_{conf,max1} = 464100.00$ is the confined core area at levels of member with hoops and is calculated as the area of core enclosed by the center lines of the external perimeter hoops.

$A_{conf,min1} = 395025.00$ is the confined core area at midway between the levels of hoops and is calculated by reducing all the dimensions of the area $A_{conf,max1}$ by a length

equal to half the clear spacing between external hoops.

$A_{noConf1} = 173066.667$ is the unconfined external core area which is equal to $b_i^2/6$ as defined at (A.2).

$a_{se2} (>= a_{se1}) = \text{Max}(((A_{conf,max2} - A_{noConf2}) / A_{conf,max2}) \cdot (A_{conf,min2} / A_{conf,max2}), 0) = 0.53375773$

The definitions of A_{noConf} , $A_{conf,min}$ and $A_{conf,max}$ are derived from generalization of the rectangular sections confinement, which is expressed by (5.4d).

The generalization is done according to Mander, J., Priestley, M., and Park, R. (1988)

"Theoretical Stress-Strain Model for Confined Concrete."

J. Struct. Eng., 10.1061/(ASCE)0733-9445(1988)114:8(1804), 1804-1826.

$A_{conf,max2} = 252164.00$ is the confined core area at levels of member with hoops and is calculated as the area of core enclosed by the center lines of the internal perimeter hoops.

$A_{conf,min2} = 111441.00$ is the confined core area at midway between the levels of hoops and is calculated by reducing all the dimensions of the area $A_{conf,max2}$ by a length

equal to half the clear spacing between internal hoops.

$A_{noConf2} = 110709.333$ is the unconfined internal core area which is equal to $b_i^2/6$ as defined at (A.2).

$p_{sh,min} \cdot F_{ywe} = \text{Min}(p_{sh,x} \cdot F_{ywe}, p_{sh,y} \cdot F_{ywe}) = 2.724$

$p_{sh,x} \cdot F_{ywe} = p_{sh1} \cdot F_{ywe1} + p_{sh2} \cdot F_{ywe2} = 2.724$

p_{sh1} ((5.4d), TBDY) = $L_{stir1} \cdot A_{stir1} / (A_{sec} \cdot s_1) = 0.00301593$

L_{stir1} (Length of stirrups along Y) = 2160.00

A_{stir1} (stirrups area) = 78.53982

p_{sh2} (5.4d) = $L_{stir2} \cdot A_{stir2} / (A_{sec} \cdot s_2) = 0.00056047$

L_{stir2} (Length of stirrups along Y) = 1568.00

A_{stir2} (stirrups area) = 50.26548

$p_{sh,y} \cdot F_{ywe} = p_{sh1} \cdot F_{ywe1} + p_{sh2} \cdot F_{ywe2} = 3.25416$

p_{sh1} ((5.4d), TBDY) = $L_{stir1} \cdot A_{stir1} / (A_{sec} \cdot s_1) = 0.00357443$

L_{stir1} (Length of stirrups along X) = 2560.00

A_{stir1} (stirrups area) = 78.53982

p_{sh2} ((5.4d), TBDY) = $L_{stir2} \cdot A_{stir2} / (A_{sec} \cdot s_2) = 0.00070345$

L_{stir2} (Length of stirrups along X) = 1968.00

A_{stir2} (stirrups area) = 50.26548

$A_{sec} = 562500.00$

$s_1 = 100.00$

$s_2 = 250.00$

$f_{ywe1} = 781.25$

$f_{ywe2} = 656.25$

$f_{ce} = 18.00$

From ((5.A5), TBDY), TBDY: $c_c = 0.00621307$

c = confinement factor = 1.42131

$y_1 = 0.0025$

$sh_1 = 0.008$

$ft_1 = 907.50$

$fy_1 = 756.25$

$su_1 = 0.032$

using (30) in Biskinis/Fardis (2013) multiplied with shear_factor

and also multiplied by the shear_factor according to 15.7.1.4, with

Shear_factor = 1.00

$l_o/l_{ou,min} = l_b/l_d = 1.00$

$su_1 = 0.4 \cdot esu_{1,nominal}$ ((5.5), TBDY) = 0.032

From table 5A.1, TBDY: $esu_{1,nominal} = 0.08$,

For calculation of $esu_{1,nominal}$ and y_1 , sh_1 , ft_1 , fy_1 , it is considered characteristic value $fsy_1 = fs_1/1.2$, from table 5.1, TBDY.

$y1, sh1, ft1, fy1$, are also multiplied by $\text{Min}(1, 1.25 \cdot (lb/ld)^{2/3})$, from 10.3.5, ASCE 41-17.
 with $fs1 = (fs_{jacket} \cdot Asl_{ten,jacket} + fs_{core} \cdot Asl_{ten,core}) / Asl_{ten} = 756.25$
 with $Es1 = (Es_{jacket} \cdot Asl_{ten,jacket} + Es_{core} \cdot Asl_{ten,core}) / Asl_{ten} = 200000.00$
 $y2 = 0.0025$
 $sh2 = 0.008$
 $ft2 = 900.1904$
 $fy2 = 750.1586$
 $su2 = 0.032$
 using (30) in Biskinis/Fardis (2013) multiplied with shear_factor
 and also multiplied by the shear_factor according to 15.7.1.4, with
 Shear_factor = 1.00
 $lo/lou, min = lb/lb, min = 1.00$
 $su2 = 0.4 \cdot esu2_{nominal} ((5.5), TBDY) = 0.032$
 From table 5A.1, TBDY: $esu2_{nominal} = 0.08$,
 For calculation of $esu2_{nominal}$ and $y2, sh2, ft2, fy2$, it is considered
 characteristic value $fsy2 = fs2/1.2$, from table 5.1, TBDY.
 $y1, sh1, ft1, fy1$, are also multiplied by $\text{Min}(1, 1.25 \cdot (lb/ld)^{2/3})$, from 10.3.5, ASCE 41-17.
 with $fs2 = (fs_{jacket} \cdot Asl_{com,jacket} + fs_{core} \cdot Asl_{com,core}) / Asl_{com} = 750.1586$
 with $Es2 = (Es_{jacket} \cdot Asl_{com,jacket} + Es_{core} \cdot Asl_{com,core}) / Asl_{com} = 200000.00$
 $yv = 0.0025$
 $shv = 0.008$
 $ftv = 902.993$
 $fyv = 752.4941$
 $suv = 0.032$
 using (30) in Biskinis/Fardis (2013) multiplied with shear_factor
 and also multiplied by the shear_factor according to 15.7.1.4, with
 Shear_factor = 1.00
 $lo/lou, min = lb/ld = 1.00$
 $suv = 0.4 \cdot esuv_{nominal} ((5.5), TBDY) = 0.032$
 From table 5A.1, TBDY: $esuv_{nominal} = 0.08$,
 considering characteristic value $fsyv = fsv/1.2$, from table 5.1, TBDY
 For calculation of $esuv_{nominal}$ and yv, shv, ftv, fyv , it is considered
 characteristic value $fsyv = fsv/1.2$, from table 5.1, TBDY.
 $y1, sh1, ft1, fy1$, are also multiplied by $\text{Min}(1, 1.25 \cdot (lb/ld)^{2/3})$, from 10.3.5, ASCE 41-17.
 with $fsv = (fs_{jacket} \cdot Asl_{mid,jacket} + fs_{mid} \cdot Asl_{mid,core}) / Asl_{mid} = 752.4941$
 with $Es_v = (Es_{jacket} \cdot Asl_{mid,jacket} + Es_{mid} \cdot Asl_{mid,core}) / Asl_{mid} = 200000.00$
 $1 = Asl_{ten} / (b \cdot d) \cdot (fs1 / fc) = 0.09629324$
 $2 = Asl_{com} / (b \cdot d) \cdot (fs2 / fc) = 0.15360794$
 $v = Asl_{mid} / (b \cdot d) \cdot (fsv / fc) = 0.16660079$
 and confined core properties:
 $b = 890.00$
 $d = 677.00$
 $d' = 13.00$
 $fcc (5A.2, TBDY) = 25.58352$
 $cc (5A.5, TBDY) = 0.00621307$
 $c = \text{confinement factor} = 1.42131$
 $1 = Asl_{ten} / (b \cdot d) \cdot (fs1 / fc) = 0.10733965$
 $2 = Asl_{com} / (b \cdot d) \cdot (fs2 / fc) = 0.17122928$
 $v = Asl_{mid} / (b \cdot d) \cdot (fsv / fc) = 0.18571261$
 Case/Assumption: Unconfined full section - Steel rupture
 ' satisfies Eq. (4.3)
 --->
 $v < v_{s,y2}$ - LHS eq.(4.5) is not satisfied
 --->
 $v < v_{s,c}$ - RHS eq.(4.5) is not satisfied
 --->
 Case/Assumption Rejected.
 --->
 New Case/Assumption: Unconfined full section - Spalling of concrete cover
 ' satisfies Eq. (4.4)
 --->
 $v < s_{y1}$ - LHS eq.(4.7) is not satisfied
 --->
 $v < v_{c,y1}$ - RHS eq.(4.6) is satisfied
 --->
 $cu (4.10) = 0.58509201$
 $M_{Rc} (4.17) = 2.3801E+009$

```

---->
New Case/Assumption: Ultimate curvature of confined core after spalling of concrete cover
In expressions below, the following modifications have been made
- b, d, d' replaced by geometric parameters of the core: bo, do, d'o
- N, 1, 2, v normalised to bo*do, instead of b*d
- - parameters of confined concrete, fcc, cc, used in lieu of fc, ec
---->
Subcase: Rupture of tension steel
---->
v* < v*s,y2 - LHS eq.(4.5) is not satisfied
---->
v* < v*s,c - LHS eq.(4.5) is not satisfied
---->
Subcase rejected
---->
New Subcase: Failure of compression zone
---->
v* < v*c,y2 - LHS eq.(4.6) is not satisfied
---->
v* < v*c,y1 - RHS eq.(4.6) is not satisfied
---->
*cu (4.11) = 0.65878578
MRo (4.18) = 3.0615E+009
---->
u = cu (4.2) = 1.3930706E-005
Mu = MRo
-----

Calculation of ratio lb/d
-----
Adequate Lap Length: lb/d >= 1
-----
-----

Calculation of Mu1-
-----
-----

Calculation of ultimate curvature u according to 4.1, Biskinis/Fardis 2013:
u = 9.1773508E-006
Mu = 8.9683E+008
-----

with full section properties:
b = 450.00
d = 707.00
d' = 43.00
v = 1.34176
N = 7.6839E+006
fc = 18.00
co (5A.5, TBDY) = 0.002
Final value of cu: cu* = shear_factor * Max( cu, cc) = 0.01486841
The Shear_factor is considered equal to 1 (pure moment strength)
From (5.4b), TBDY: cu = 0.01486841
we (5.4c) = 0.08077545
ase ((5.4d), TBDY) = (ase1*Aext+ase2*Aint)/Asec = 0.53375773
ase1 = Max(((Aconf,max1-AnoConf1)/Aconf,max1)*(Aconf,min1/Aconf,max1),0) = 0.53375773
The definitions of AnoConf, Aconf,min and Aconf,max are derived from generalization
of the rectangular sections confinement, which is expressed by (5.4d).
The generalization is done according to Mander, J., Priestley, M., and Park, R. (1988)
"Theoretical Stress-Strain Model for Confined Concrete."
J. Struct. Eng., 10.1061/(ASCE)0733-9445(1988)114:8(1804), 1804-1826.
Aconf,max1 = 464100.00 is the confined core area at levels of member with hoops and
is calculated as the area of core enclosed by the center lines of the external perimeter hoops.
Aconf,min1 = 395025.00 is the confined core area at midway between the levels of hoops and
is calculated by reducing all the dimensions of the area Aconf,max1 by a length
equal to half the clear spacing between external hoops.
AnoConf1 = 173066.667 is the unconfined external core area which is equal to bi2/6 as defined at (A.2).

```

$$ase2 (>=ase1) = \text{Max}(((A_{conf,max2}-A_{noConf2})/A_{conf,max2}) * (A_{conf,min2}/A_{conf,max2}), 0) = 0.53375773$$

The definitions of A_{noConf} , $A_{conf,min}$ and $A_{conf,max}$ are derived from generalization of the rectangular sections confinement, which is expressed by (5.4d).

The generalization is done according to Mander, J., Priestley, M., and Park, R. (1988) "Theoretical Stress-Strain Model for Confined Concrete."

J. Struct. Eng., 10.1061/(ASCE)0733-9445(1988)114:8(1804), 1804-1826.

$A_{conf,max2} = 252164.00$ is the confined core area at levels of member with hoops and is calculated as the area of core enclosed by the center lines of the internal perimeter hoops.

$A_{conf,min2} = 111441.00$ is the confined core area at midway between the levels of hoops and is calculated by reducing all the dimensions of the area $A_{conf,max2}$ by a length equal to half the clear spacing between internal hoops.

$A_{noConf2} = 110709.333$ is the unconfined internal core area which is equal to $b_i^2/6$ as defined at (A.2).
 $psh_{min} * F_{ywe} = \text{Min}(psh_x * F_{ywe}, psh_y * F_{ywe}) = 2.724$

$$psh_x * F_{ywe} = psh1 * F_{ywe1} + ps2 * F_{ywe2} = 2.724$$

$$psh1 \text{ ((5.4d), TBDY)} = L_{stir1} * A_{stir1} / (A_{sec} * s1) = 0.00301593$$

L_{stir1} (Length of stirrups along Y) = 2160.00

A_{stir1} (stirrups area) = 78.53982

$$psh2 \text{ (5.4d)} = L_{stir2} * A_{stir2} / (A_{sec} * s2) = 0.00056047$$

L_{stir2} (Length of stirrups along Y) = 1568.00

A_{stir2} (stirrups area) = 50.26548

$$psh_y * F_{ywe} = psh1 * F_{ywe1} + ps2 * F_{ywe2} = 3.25416$$

$$psh1 \text{ ((5.4d), TBDY)} = L_{stir1} * A_{stir1} / (A_{sec} * s1) = 0.00357443$$

L_{stir1} (Length of stirrups along X) = 2560.00

A_{stir1} (stirrups area) = 78.53982

$$psh2 \text{ ((5.4d), TBDY)} = L_{stir2} * A_{stir2} / (A_{sec} * s2) = 0.00070345$$

L_{stir2} (Length of stirrups along X) = 1968.00

A_{stir2} (stirrups area) = 50.26548

$$A_{sec} = 562500.00$$

$$s1 = 100.00$$

$$s2 = 250.00$$

$$f_{ywe1} = 781.25$$

$$f_{ywe2} = 656.25$$

$$f_{ce} = 18.00$$

From ((5.A5), TBDY), TBDY: $cc = 0.00621307$

c = confinement factor = 1.42131

$$y1 = 0.0025$$

$$sh1 = 0.008$$

$$ft1 = 900.1904$$

$$fy1 = 750.1586$$

$$su1 = 0.032$$

using (30) in Biskinis/Fardis (2013) multiplied with shear_factor and also multiplied by the shear_factor according to 15.7.1.4, with Shear_factor = 1.00

$$l_o/l_{ou,min} = l_b/l_d = 1.00$$

$$su1 = 0.4 * esu1_{nominal} \text{ ((5.5), TBDY)} = 0.032$$

From table 5A.1, TBDY: $esu1_{nominal} = 0.08$,

For calculation of $esu1_{nominal}$ and $y1$, $sh1$, $ft1$, $fy1$, it is considered characteristic value $fsy1 = fs1/1.2$, from table 5.1, TBDY.

$y1$, $sh1$, $ft1$, $fy1$, are also multiplied by $\text{Min}(1, 1.25 * (l_b/l_d)^{2/3})$, from 10.3.5, ASCE 41-17.

$$\text{with } fs1 = (f_{s,jacket} * A_{sl,ten,jacket} + f_{s,core} * A_{sl,ten,core}) / A_{sl,ten} = 750.1586$$

$$\text{with } Es1 = (E_{s,jacket} * A_{sl,ten,jacket} + E_{s,core} * A_{sl,ten,core}) / A_{sl,ten} = 200000.00$$

$$y2 = 0.0025$$

$$sh2 = 0.008$$

$$ft2 = 907.50$$

$$fy2 = 756.25$$

$$su2 = 0.032$$

using (30) in Biskinis/Fardis (2013) multiplied with shear_factor and also multiplied by the shear_factor according to 15.7.1.4, with Shear_factor = 1.00

$$l_o/l_{ou,min} = l_b/l_{b,min} = 1.00$$

$$su2 = 0.4 * esu2_{nominal} \text{ ((5.5), TBDY)} = 0.032$$

From table 5A.1, TBDY: $esu2_{nominal} = 0.08$,

For calculation of $esu2_{nominal}$ and $y2$, $sh2$, $ft2$, $fy2$, it is considered characteristic value $fsy2 = fs2/1.2$, from table 5.1, TBDY.

$y1, sh1, ft1, fy1$, are also multiplied by $\text{Min}(1, 1.25 \cdot (lb/ld)^{2/3})$, from 10.3.5, ASCE 41-17.
 with $fs2 = (fs_{jacket} \cdot Asl_{com,jacket} + fs_{core} \cdot Asl_{com,core}) / Asl_{com} = 756.25$
 with $Es2 = (Es_{jacket} \cdot Asl_{com,jacket} + Es_{core} \cdot Asl_{com,core}) / Asl_{com} = 200000.00$
 $yv = 0.0025$
 $shv = 0.008$
 $ftv = 902.993$
 $fyv = 752.4941$
 $suv = 0.032$
 using (30) in Biskinis/Fardis (2013) multiplied with shear_factor
 and also multiplied by the shear_factor according to 15.7.1.4, with
 Shear_factor = 1.00
 $lo/lou, min = lb/ld = 1.00$
 $suv = 0.4 \cdot esuv_{nominal} ((5.5), TBDY) = 0.032$
 From table 5A.1, TBDY: $esuv_{nominal} = 0.08$,
 considering characteristic value $fsyv = fsv/1.2$, from table 5.1, TBDY
 For calculation of $esuv_{nominal}$ and yv, shv, ftv, fyv , it is considered
 characteristic value $fsyv = fsv/1.2$, from table 5.1, TBDY.
 $y1, sh1, ft1, fy1$, are also multiplied by $\text{Min}(1, 1.25 \cdot (lb/ld)^{2/3})$, from 10.3.5, ASCE 41-17.
 with $fsv = (fs_{jacket} \cdot Asl_{mid,jacket} + fs_{mid} \cdot Asl_{mid,core}) / Asl_{mid} = 752.4941$
 with $Esv = (Es_{jacket} \cdot Asl_{mid,jacket} + Es_{mid} \cdot Asl_{mid,core}) / Asl_{mid} = 200000.00$
 $1 = Asl_{ten} / (b \cdot d) \cdot (fs1 / fc) = 0.32428344$
 $2 = Asl_{com} / (b \cdot d) \cdot (fs2 / fc) = 0.20328574$
 $v = Asl_{mid} / (b \cdot d) \cdot (fsv / fc) = 0.35171278$
 and confined core properties:
 $b = 390.00$
 $d = 677.00$
 $d' = 13.00$
 $fcc (5A.2, TBDY) = 25.58352$
 $cc (5A.5, TBDY) = 0.00621307$
 $c = \text{confinement factor} = 1.42131$
 $1 = Asl_{ten} / (b \cdot d) \cdot (fs1 / fc) = 0.39075399$
 $2 = Asl_{com} / (b \cdot d) \cdot (fs2 / fc) = 0.24495458$
 $v = Asl_{mid} / (b \cdot d) \cdot (fsv / fc) = 0.42380571$
 Case/Assumption: Unconfined full section - Steel rupture
 ' satisfies Eq. (4.3)
 --->
 $v < vs,y2$ - LHS eq.(4.5) is not satisfied
 --->
 $v < vs,c$ - RHS eq.(4.5) is not satisfied
 --->
 Case/Assumption Rejected.
 --->
 New Case/Assumption: Unconfined full section - Spalling of concrete cover
 ' satisfies Eq. (4.4)
 --->
 $v < s,y1$ - LHS eq.(4.7) is not satisfied
 --->
 $v < vc,y1$ - RHS eq.(4.6) is not satisfied
 --->
 $cu (4.11) = 0.95920575$
 $MRC (4.18) = 7.7321E+008$
 --->
 New Case/Assumption: Ultimate curvature of confined core after spalling of concrete cover
 In expressions below, the following modifications have been made
 - b, d, d' replaced by geometric parameters of the core: $bo, do, d'o$
 - $N, 1, 2, v$ normalised to $bo \cdot do$, instead of $b \cdot d$
 - parameters of confined concrete, fcc, cc , used in lieu of fc, ecu
 --->
 Subcase: Rupture of tension steel
 --->
 $v^* < v^*s,y2$ - LHS eq.(4.5) is not satisfied
 --->
 $v^* < v^*s,c$ - LHS eq.(4.5) is not satisfied
 --->
 Subcase rejected
 --->
 New Subcase: Failure of compression zone

```

---->
v* < v*c,y2 - LHS eq.(4.6) is not satisfied
---->
v* < v*c,y1 - RHS eq.(4.6) is not satisfied
---->
*cu (4.11) = 1.00
MRo (4.18) = 8.9683E+008
---->
u = cu (4.2) = 9.1773508E-006
Mu = MRo

```

Calculation of ratio lb/d

Adequate Lap Length: lb/d >= 1

Calculation of Mu2+

Calculation of ultimate curvature u according to 4.1, Biskinis/Fardis 2013:
u = 1.3930706E-005
Mu = 3.0615E+009

with full section properties:

```

b = 950.00
d = 707.00
d' = 43.00
v = 0.63557247
N = 7.6839E+006
fc = 18.00
co (5A.5, TBDY) = 0.002
Final value of cu: cu* = shear_factor * Max( cu, cc) = 0.01486841
The Shear_factor is considered equal to 1 (pure moment strength)
From (5.4b), TBDY: cu = 0.01486841
we (5.4c) = 0.08077545
ase ((5.4d), TBDY) = (ase1*Aext+ase2*Aint)/Asec = 0.53375773
ase1 = Max(((Aconf,max1-AnoConf1)/Aconf,max1)*(Aconf,min1/Aconf,max1),0) = 0.53375773
The definitions of AnoConf, Aconf,min and Aconf,max are derived from generalization
of the rectangular sections confinement, which is expressed by (5.4d).
The generalization is done according to Mander, J., Priestley, M., and Park, R. (1988)
"Theoretical Stress-Strain Model for Confined Concrete."
J. Struct. Eng., 10.1061/(ASCE)0733-9445(1988)114:8(1804), 1804-1826.
Aconf,max1 = 464100.00 is the confined core area at levels of member with hoops and
is calculated as the area of core enclosed by the center lines of the external perimeter hoops.
Aconf,min1 = 395025.00 is the confined core area at midway between the levels of hoops and
is calculated by reducing all the dimensions of the area Aconf,max1 by a length
equal to half the clear spacing between external hoops.
AnoConf1 = 173066.667 is the unconfined external core area which is equal to bi2/6 as defined at (A.2).
ase2 (>=ase1) = Max(((Aconf,max2-AnoConf2)/Aconf,max2)*(Aconf,min2/Aconf,max2),0) = 0.53375773
The definitions of AnoConf, Aconf,min and Aconf,max are derived from generalization
of the rectangular sections confinement, which is expressed by (5.4d).
The generalization is done according to Mander, J., Priestley, M., and Park, R. (1988)
"Theoretical Stress-Strain Model for Confined Concrete."
J. Struct. Eng., 10.1061/(ASCE)0733-9445(1988)114:8(1804), 1804-1826.
Aconf,max2 = 252164.00 is the confined core area at levels of member with hoops and
is calculated as the area of core enclosed by the center lines of the internal perimeter hoops.
Aconf,min2 = 111441.00 is the confined core area at midway between the levels of hoops and
is calculated by reducing all the dimensions of the area Aconf,max2 by a length
equal to half the clear spacing between internal hoops.
AnoConf2 = 110709.333 is the unconfined internal core area which is equal to bi2/6 as defined at (A.2).
psh,min*Fywe = Min(psh,x*Fywe , psh,y*Fywe) = 2.724
psh_x*Fywe = psh1*Fywe1+ps2*Fywe2 = 2.724
psh1 ((5.4d), TBDY) = Lstir1*Astir1/(Asec*s1) = 0.00301593

```

Lstir1 (Length of stirrups along Y) = 2160.00
 Astir1 (stirrups area) = 78.53982
 $psh2 (5.4d) = Lstir2 * Astir2 / (Asec * s2) = 0.00056047$
 Lstir2 (Length of stirrups along Y) = 1568.00
 Astir2 (stirrups area) = 50.26548

$psh_y * Fywe = psh1 * Fywe1 + ps2 * Fywe2 = 3.25416$
 $psh1 ((5.4d), TBDY) = Lstir1 * Astir1 / (Asec * s1) = 0.00357443$
 Lstir1 (Length of stirrups along X) = 2560.00
 Astir1 (stirrups area) = 78.53982
 $psh2 ((5.4d), TBDY) = Lstir2 * Astir2 / (Asec * s2) = 0.00070345$
 Lstir2 (Length of stirrups along X) = 1968.00
 Astir2 (stirrups area) = 50.26548

Asec = 562500.00

s1 = 100.00

s2 = 250.00

fywe1 = 781.25

fywe2 = 656.25

fce = 18.00

From ((5.A5), TBDY), TBDY: cc = 0.00621307

c = confinement factor = 1.42131

y1 = 0.0025

sh1 = 0.008

ft1 = 907.50

fy1 = 756.25

su1 = 0.032

using (30) in Biskinis/Fardis (2013) multiplied with shear_factor
 and also multiplied by the shear_factor according to 15.7.1.4, with
 Shear_factor = 1.00

lo/lou,min = lb/ld = 1.00

su1 = 0.4*esu1_nominal ((5.5), TBDY) = 0.032

From table 5A.1, TBDY: esu1_nominal = 0.08,

For calculation of esu1_nominal and y1, sh1,ft1,fy1, it is considered
 characteristic value fsy1 = fs1/1.2, from table 5.1, TBDY.

y1, sh1,ft1,fy1, are also multiplied by $\text{Min}(1, 1.25 * (lb/ld)^{2/3})$, from 10.3.5, ASCE 41-17.

with fs1 = (fs,jacket*Asl,ten,jacket + fs,core*Asl,ten,core)/Asl,ten = 756.25

with Es1 = (Es,jacket*Asl,ten,jacket + Es,core*Asl,ten,core)/Asl,ten = 200000.00

y2 = 0.0025

sh2 = 0.008

ft2 = 900.1904

fy2 = 750.1586

su2 = 0.032

using (30) in Biskinis/Fardis (2013) multiplied with shear_factor
 and also multiplied by the shear_factor according to 15.7.1.4, with
 Shear_factor = 1.00

lo/lou,min = lb/lb,min = 1.00

su2 = 0.4*esu2_nominal ((5.5), TBDY) = 0.032

From table 5A.1, TBDY: esu2_nominal = 0.08,

For calculation of esu2_nominal and y2, sh2,ft2,fy2, it is considered
 characteristic value fsy2 = fs2/1.2, from table 5.1, TBDY.

y1, sh1,ft1,fy1, are also multiplied by $\text{Min}(1, 1.25 * (lb/ld)^{2/3})$, from 10.3.5, ASCE 41-17.

with fs2 = (fs,jacket*Asl,com,jacket + fs,core*Asl,com,core)/Asl,com = 750.1586

with Es2 = (Es,jacket*Asl,com,jacket + Es,core*Asl,com,core)/Asl,com = 200000.00

yv = 0.0025

shv = 0.008

ftv = 902.993

fyv = 752.4941

suv = 0.032

using (30) in Biskinis/Fardis (2013) multiplied with shear_factor
 and also multiplied by the shear_factor according to 15.7.1.4, with
 Shear_factor = 1.00

lo/lou,min = lb/ld = 1.00

suv = 0.4*esuv_nominal ((5.5), TBDY) = 0.032

From table 5A.1, TBDY: esuv_nominal = 0.08,

considering characteristic value fsyv = fsv/1.2, from table 5.1, TBDY
 For calculation of esuv_nominal and yv, shv,ftv,fyv, it is considered

```

characteristic value  $f_{sv} = f_{sv}/1.2$ , from table 5.1, TBDY.
 $y_1, sh_1, ft_1, fy_1$ , are also multiplied by  $\text{Min}(1, 1.25 \cdot (l_b/l_d)^{2/3})$ , from 10.3.5, ASCE 41-17.
with  $f_{sv} = (f_{s,jacket} \cdot A_{s,mid,jacket} + f_{s,mid} \cdot A_{s,mid,core}) / A_{s,mid} = 752.4941$ 
with  $E_{sv} = (E_{s,jacket} \cdot A_{s,mid,jacket} + E_{s,mid} \cdot A_{s,mid,core}) / A_{s,mid} = 200000.00$ 
 $1 = A_{s,ten} / (b \cdot d) \cdot (f_{s1}/f_c) = 0.09629324$ 
 $2 = A_{s,com} / (b \cdot d) \cdot (f_{s2}/f_c) = 0.15360794$ 
 $v = A_{s,mid} / (b \cdot d) \cdot (f_{sv}/f_c) = 0.16660079$ 
and confined core properties:
 $b = 890.00$ 
 $d = 677.00$ 
 $d' = 13.00$ 
 $f_{cc} \text{ (5A.2, TBDY)} = 25.58352$ 
 $cc \text{ (5A.5, TBDY)} = 0.00621307$ 
 $c = \text{confinement factor} = 1.42131$ 
 $1 = A_{s,ten} / (b \cdot d) \cdot (f_{s1}/f_c) = 0.10733965$ 
 $2 = A_{s,com} / (b \cdot d) \cdot (f_{s2}/f_c) = 0.17122928$ 
 $v = A_{s,mid} / (b \cdot d) \cdot (f_{sv}/f_c) = 0.18571261$ 
Case/Assumption: Unconfined full section - Steel rupture
' satisfies Eq. (4.3)
--->
 $v < v_{s,y2}$  - LHS eq.(4.5) is not satisfied
--->
 $v < v_{s,c}$  - RHS eq.(4.5) is not satisfied
--->
Case/Assumption Rejected.
--->
New Case/Assumption: Unconfined full section - Spalling of concrete cover
' satisfies Eq. (4.4)
--->
 $v < s_{y1}$  - LHS eq.(4.7) is not satisfied
--->
 $v < v_{c,y1}$  - RHS eq.(4.6) is satisfied
--->
 $c_u \text{ (4.10)} = 0.58509201$ 
 $M_{Rc} \text{ (4.17)} = 2.3801E+009$ 
--->
New Case/Assumption: Ultimate curvature of confined core after spalling of concrete cover
In expressions below, the following modifications have been made
-  $b, d, d'$  replaced by geometric parameters of the core:  $b_o, d_o, d'_o$ 
-  $N, 1, 2, v$  normalised to  $b_o \cdot d_o$ , instead of  $b \cdot d$ 
- parameters of confined concrete,  $f_{cc}, cc$ , used in lieu of  $f_c, e_{cu}$ 
--->
Subcase: Rupture of tension steel
--->
 $v^* < v^*_{s,y2}$  - LHS eq.(4.5) is not satisfied
--->
 $v^* < v^*_{s,c}$  - LHS eq.(4.5) is not satisfied
--->
Subcase rejected
--->
New Subcase: Failure of compression zone
--->
 $v^* < v^*_{c,y2}$  - LHS eq.(4.6) is not satisfied
--->
 $v^* < v^*_{c,y1}$  - RHS eq.(4.6) is not satisfied
--->
 $*c_u \text{ (4.11)} = 0.65878578$ 
 $M_{Ro} \text{ (4.18)} = 3.0615E+009$ 
--->
 $u = c_u \text{ (4.2)} = 1.3930706E-005$ 
 $\mu = M_{Ro}$ 

```

Calculation of ratio l_b/l_d

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

Calculation of Mu2-

Calculation of ultimate curvature μ according to 4.1, Biskinis/Fardis 2013:

$$\mu = 9.1773508E-006$$

$$Mu = 8.9683E+008$$

with full section properties:

$$b = 450.00$$

$$d = 707.00$$

$$d' = 43.00$$

$$v = 1.34176$$

$$N = 7.6839E+006$$

$$f_c = 18.00$$

$$c_o \text{ (5A.5, TBDY)} = 0.002$$

$$\text{Final value of } c_u: c_u^* = \text{shear_factor} * \text{Max}(c_u, c_o) = 0.01486841$$

The Shear_factor is considered equal to 1 (pure moment strength)

$$\text{From (5.4b), TBDY: } c_u = 0.01486841$$

$$w_e \text{ (5.4c)} = 0.08077545$$

$$a_{se} \text{ ((5.4d), TBDY)} = (a_{se1} * A_{ext} + a_{se2} * A_{int}) / A_{sec} = 0.53375773$$

$$a_{se1} = \text{Max}(((A_{conf,max1} - A_{noConf1}) / A_{conf,max1}) * (A_{conf,min1} / A_{conf,max1}), 0) = 0.53375773$$

The definitions of A_{noConf} , $A_{conf,min}$ and $A_{conf,max}$ are derived from generalization

of the rectangular sections confinement, which is expressed by (5.4d).

The generalization is done according to Mander, J., Priestley, M., and Park, R. (1988)

"Theoretical Stress-Strain Model for Confined Concrete."

J. Struct. Eng., 10.1061/(ASCE)0733-9445(1988)114:8(1804), 1804-1826.

$A_{conf,max1} = 464100.00$ is the confined core area at levels of member with hoops and

is calculated as the area of core enclosed by the center lines of the external perimeter hoops.

$A_{conf,min1} = 395025.00$ is the confined core area at midway between the levels of hoops and

is calculated by reducing all the dimensions of the area $A_{conf,max1}$ by a length

equal to half the clear spacing between external hoops.

$A_{noConf1} = 173066.667$ is the unconfined external core area which is equal to $b_i^2/6$ as defined at (A.2).

$$a_{se2} (>= a_{se1}) = \text{Max}(((A_{conf,max2} - A_{noConf2}) / A_{conf,max2}) * (A_{conf,min2} / A_{conf,max2}), 0) = 0.53375773$$

The definitions of A_{noConf} , $A_{conf,min}$ and $A_{conf,max}$ are derived from generalization

of the rectangular sections confinement, which is expressed by (5.4d).

The generalization is done according to Mander, J., Priestley, M., and Park, R. (1988)

"Theoretical Stress-Strain Model for Confined Concrete."

J. Struct. Eng., 10.1061/(ASCE)0733-9445(1988)114:8(1804), 1804-1826.

$A_{conf,max2} = 252164.00$ is the confined core area at levels of member with hoops and

is calculated as the area of core enclosed by the center lines of the internal perimeter hoops.

$A_{conf,min2} = 111441.00$ is the confined core area at midway between the levels of hoops and

is calculated by reducing all the dimensions of the area $A_{conf,max2}$ by a length

equal to half the clear spacing between internal hoops.

$A_{noConf2} = 110709.333$ is the unconfined internal core area which is equal to $b_i^2/6$ as defined at (A.2).

$$p_{sh,min} * F_{ywe} = \text{Min}(p_{sh,x} * F_{ywe}, p_{sh,y} * F_{ywe}) = 2.724$$

$$p_{sh,x} * F_{ywe} = p_{sh1} * F_{ywe1} + p_{sh2} * F_{ywe2} = 2.724$$

$$p_{sh1} \text{ ((5.4d), TBDY)} = L_{stir1} * A_{stir1} / (A_{sec} * s_1) = 0.00301593$$

$$L_{stir1} \text{ (Length of stirrups along Y)} = 2160.00$$

$$A_{stir1} \text{ (stirrups area)} = 78.53982$$

$$p_{sh2} \text{ (5.4d)} = L_{stir2} * A_{stir2} / (A_{sec} * s_2) = 0.00056047$$

$$L_{stir2} \text{ (Length of stirrups along Y)} = 1568.00$$

$$A_{stir2} \text{ (stirrups area)} = 50.26548$$

$$p_{sh,y} * F_{ywe} = p_{sh1} * F_{ywe1} + p_{sh2} * F_{ywe2} = 3.25416$$

$$p_{sh1} \text{ ((5.4d), TBDY)} = L_{stir1} * A_{stir1} / (A_{sec} * s_1) = 0.00357443$$

$$L_{stir1} \text{ (Length of stirrups along X)} = 2560.00$$

$$A_{stir1} \text{ (stirrups area)} = 78.53982$$

$$p_{sh2} \text{ ((5.4d), TBDY)} = L_{stir2} * A_{stir2} / (A_{sec} * s_2) = 0.00070345$$

$$L_{stir2} \text{ (Length of stirrups along X)} = 1968.00$$

$$A_{stir2} \text{ (stirrups area)} = 50.26548$$

$$A_{sec} = 562500.00$$

$$s_1 = 100.00$$

$s_2 = 250.00$
 $fy_{we1} = 781.25$
 $fy_{we2} = 656.25$
 $f_{ce} = 18.00$
 From ((5A.5), TBDY), TBDY: $cc = 0.00621307$
 $c = \text{confinement factor} = 1.42131$
 $y_1 = 0.0025$
 $sh_1 = 0.008$
 $ft_1 = 900.1904$
 $fy_1 = 750.1586$
 $su_1 = 0.032$
 using (30) in Biskinis/Fardis (2013) multiplied with shear_factor
 and also multiplied by the shear_factor according to 15.7.1.4, with
 $\text{Shear_factor} = 1.00$
 $lo/lou, \min = lb/d = 1.00$
 $su_1 = 0.4 * esu_1 \text{ nominal ((5.5), TBDY)} = 0.032$
 From table 5A.1, TBDY: $esu_1 \text{ nominal} = 0.08$,
 For calculation of $esu_1 \text{ nominal}$ and y_1, sh_1, ft_1, fy_1 , it is considered
 characteristic value $fsy_1 = fs_1/1.2$, from table 5.1, TBDY.
 y_1, sh_1, ft_1, fy_1 , are also multiplied by $\text{Min}(1, 1.25 * (lb/d)^{2/3})$, from 10.3.5, ASCE 41-17.
 with $fs_1 = (fs_{jacket} * Asl, \text{ten, jacket} + fs_{core} * Asl, \text{ten, core}) / Asl, \text{ten} = 750.1586$
 with $Es_1 = (Es_{jacket} * Asl, \text{ten, jacket} + Es_{core} * Asl, \text{ten, core}) / Asl, \text{ten} = 200000.00$
 $y_2 = 0.0025$
 $sh_2 = 0.008$
 $ft_2 = 907.50$
 $fy_2 = 756.25$
 $su_2 = 0.032$
 using (30) in Biskinis/Fardis (2013) multiplied with shear_factor
 and also multiplied by the shear_factor according to 15.7.1.4, with
 $\text{Shear_factor} = 1.00$
 $lo/lou, \min = lb/lb, \min = 1.00$
 $su_2 = 0.4 * esu_2 \text{ nominal ((5.5), TBDY)} = 0.032$
 From table 5A.1, TBDY: $esu_2 \text{ nominal} = 0.08$,
 For calculation of $esu_2 \text{ nominal}$ and y_2, sh_2, ft_2, fy_2 , it is considered
 characteristic value $fsy_2 = fs_2/1.2$, from table 5.1, TBDY.
 y_2, sh_2, ft_2, fy_2 , are also multiplied by $\text{Min}(1, 1.25 * (lb/d)^{2/3})$, from 10.3.5, ASCE 41-17.
 with $fs_2 = (fs_{jacket} * Asl, \text{com, jacket} + fs_{core} * Asl, \text{com, core}) / Asl, \text{com} = 756.25$
 with $Es_2 = (Es_{jacket} * Asl, \text{com, jacket} + Es_{core} * Asl, \text{com, core}) / Asl, \text{com} = 200000.00$
 $y_v = 0.0025$
 $sh_v = 0.008$
 $ft_v = 902.993$
 $fy_v = 752.4941$
 $suv = 0.032$
 using (30) in Biskinis/Fardis (2013) multiplied with shear_factor
 and also multiplied by the shear_factor according to 15.7.1.4, with
 $\text{Shear_factor} = 1.00$
 $lo/lou, \min = lb/d = 1.00$
 $suv = 0.4 * esuv \text{ nominal ((5.5), TBDY)} = 0.032$
 From table 5A.1, TBDY: $esuv \text{ nominal} = 0.08$,
 considering characteristic value $fsyv = fs_v/1.2$, from table 5.1, TBDY
 For calculation of $esuv \text{ nominal}$ and y_v, sh_v, ft_v, fy_v , it is considered
 characteristic value $fsyv = fs_v/1.2$, from table 5.1, TBDY.
 y_v, sh_v, ft_v, fy_v , are also multiplied by $\text{Min}(1, 1.25 * (lb/d)^{2/3})$, from 10.3.5, ASCE 41-17.
 with $fs_v = (fs_{jacket} * Asl, \text{mid, jacket} + fs_{mid} * Asl, \text{mid, core}) / Asl, \text{mid} = 752.4941$
 with $Es_v = (Es_{jacket} * Asl, \text{mid, jacket} + Es_{mid} * Asl, \text{mid, core}) / Asl, \text{mid} = 200000.00$
 $1 = Asl, \text{ten} / (b * d) * (fs_1 / f_c) = 0.32428344$
 $2 = Asl, \text{com} / (b * d) * (fs_2 / f_c) = 0.20328574$
 $v = Asl, \text{mid} / (b * d) * (fs_v / f_c) = 0.35171278$
 and confined core properties:
 $b = 390.00$
 $d = 677.00$
 $d' = 13.00$
 $f_{cc} \text{ (5A.2, TBDY)} = 25.58352$
 $cc \text{ (5A.5, TBDY)} = 0.00621307$
 $c = \text{confinement factor} = 1.42131$
 $1 = Asl, \text{ten} / (b * d) * (fs_1 / f_c) = 0.39075399$
 $2 = Asl, \text{com} / (b * d) * (fs_2 / f_c) = 0.24495458$

```

v = Asl,mid/(b*d)*(fsv/fc) = 0.42380571
Case/Assumption: Unconfined full section - Steel rupture
'satisfies Eq. (4.3)
--->
v < vs,y2 - LHS eq.(4.5) is not satisfied
--->
v < vs,c - RHS eq.(4.5) is not satisfied
--->
Case/Assumption Rejected.
--->
New Case/Assumption: Unconfined full section - Spalling of concrete cover
'satisfies Eq. (4.4)
--->
v < s,y1 - LHS eq.(4.7) is not satisfied
--->
v < vc,y1 - RHS eq.(4.6) is not satisfied
--->
cu (4.11) = 0.95920575
MRc (4.18) = 7.7321E+008
--->
New Case/Assumption: Ultimate curvature of confined core after spalling of concrete cover
In expressions below, the following modifications have been made
- b, d, d' replaced by geometric parameters of the core: bo, do, d'o
- N, 1, 2, v normalised to bo*do, instead of b*d
- - parameters of confined concrete, fcc, cc, used in lieu of fc, ecu
--->
Subcase: Rupture of tension steel
--->
v* < v*s,y2 - LHS eq.(4.5) is not satisfied
--->
v* < v*s,c - LHS eq.(4.5) is not satisfied
--->
Subcase rejected
--->
New Subcase: Failure of compression zone
--->
v* < v*c,y2 - LHS eq.(4.6) is not satisfied
--->
v* < v*c,y1 - RHS eq.(4.6) is not satisfied
--->
*cu (4.11) = 1.00
MRo (4.18) = 8.9683E+008
--->
u = cu (4.2) = 9.1773508E-006
Mu = MRo
-----

Calculation of ratio lb/lc
-----
Adequate Lap Length: lb/lc >= 1
-----
-----
-----
-----
Calculation of Shear Strength Vr = Min(Vr1,Vr2) = 1.1678E+006
-----
Calculation of Shear Strength at edge 1, Vr1 = 1.1678E+006
Vr1 = VCol ((10.3), ASCE 41-17) = knl*VCol0
VCol0 = 1.1678E+006
knl = 1 (zero step-static loading)
-----
NOTE: In expression (10-3) 'Vs = Av*fy*d/s' is replaced by 'Vs+ f*Vf'
where Vf is the contribution of FRPs (11.3), ACI 440).
-----
= 1 (normal-weight concrete)
Mean concrete strength: fc' = (fc'_jacket*Area_jacket + fc'_core*Area_core)/Area_section = 15.20, but fc'^0.5 <= 8.3
MPa (22.5.3.1, ACI 318-14)

```

$M/Vd = 4.00$
 $\mu_u = 7.6104E+008$
 $V_u = 227.4778$
 $d = 0.8 \cdot h = 600.00$
 $N_u = 7.6839E+006$
 $A_g = 337500.00$
 From (11.5.4.8), ACI 318-14: $V_s = V_{sjacket} + V_{s,core} = 1.0354E+006$
 where:
 $V_{sjacket} = V_{sj1} + V_{sj2} = 942477.796$
 $V_{sj1} = 589048.623$ is calculated for section web jacket, with:
 $d = 600.00$
 $A_v = 157079.633$
 $f_y = 625.00$
 $s = 100.00$
 V_{sj1} is multiplied by $Col,j1 = 1.00$
 $s/d = 0.16666667$
 $V_{sj2} = 353429.174$ is calculated for section flange jacket, with:
 $d = 360.00$
 $A_v = 157079.633$
 $f_y = 625.00$
 $s = 100.00$
 V_{sj2} is multiplied by $Col,j2 = 1.00$
 $s/d = 0.27777778$
 $V_{s,core} = V_{s,c1} + V_{s,c2} = 92890.612$
 $V_{s,c1} = 92890.612$ is calculated for section web core, with:
 $d = 440.00$
 $A_v = 100530.965$
 $f_y = 525.00$
 $s = 250.00$
 $V_{s,c1}$ is multiplied by $Col,c1 = 1.00$
 $s/d = 0.56818182$
 $V_{s,c2} = 0.00$ is calculated for section flange core, with:
 $d = 200.00$
 $A_v = 100530.965$
 $f_y = 525.00$
 $s = 250.00$
 $V_{s,c2}$ is multiplied by $Col,c2 = 0.00$
 $s/d = 1.25$
 $V_f ((11-3)-(11.4), ACI 440) = 0.00$
 From (11-11), ACI 440: $V_s + V_f \leq 699281.943$
 $bw = 450.00$

Calculation of Shear Strength at edge 2, $V_{r2} = 1.1678E+006$
 $V_{r2} = V_{col} ((10.3), ASCE 41-17) = knl \cdot V_{col0}$
 $V_{col0} = 1.1678E+006$
 $knl = 1$ (zero step-static loading)

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

$= 1$ (normal-weight concrete)
 Mean concrete strength: $fc' = (fc'_{jacket} \cdot Area_{jacket} + fc'_{core} \cdot Area_{core}) / Area_{section} = 15.20$, but $fc'^{0.5} \leq 8.3$
 MPa (22.5.3.1, ACI 318-14)

$M/Vd = 4.00$
 $\mu_u = 6.8991E+008$
 $V_u = 227.4778$
 $d = 0.8 \cdot h = 600.00$
 $N_u = 7.6839E+006$
 $A_g = 337500.00$
 From (11.5.4.8), ACI 318-14: $V_s = V_{sjacket} + V_{s,core} = 1.0354E+006$
 where:
 $V_{sjacket} = V_{sj1} + V_{sj2} = 942477.796$
 $V_{sj1} = 589048.623$ is calculated for section web jacket, with:
 $d = 600.00$
 $A_v = 157079.633$
 $f_y = 625.00$

$s = 100.00$
 $V_{s,j1}$ is multiplied by $Col,j1 = 1.00$
 $s/d = 0.16666667$
 $V_{s,j2} = 353429.174$ is calculated for section flange jacket, with:
 $d = 360.00$
 $A_v = 157079.633$
 $f_y = 625.00$
 $s = 100.00$
 $V_{s,j2}$ is multiplied by $Col,j2 = 1.00$
 $s/d = 0.27777778$
 $V_{s,core} = V_{s,c1} + V_{s,c2} = 92890.612$
 $V_{s,c1} = 92890.612$ is calculated for section web core, with:
 $d = 440.00$
 $A_v = 100530.965$
 $f_y = 525.00$
 $s = 250.00$
 $V_{s,c1}$ is multiplied by $Col,c1 = 1.00$
 $s/d = 0.56818182$
 $V_{s,c2} = 0.00$ is calculated for section flange core, with:
 $d = 200.00$
 $A_v = 100530.965$
 $f_y = 525.00$
 $s = 250.00$
 $V_{s,c2}$ is multiplied by $Col,c2 = 0.00$
 $s/d = 1.25$
 $V_f ((11-3)-(11.4), ACI 440) = 0.00$
 From (11-11), ACI 440: $V_s + V_f \leq 699281.943$
 $bw = 450.00$

 End Of Calculation of Shear Capacity ratio for element: column JTC1 of floor 1
 At local axis: 3

Start Of Calculation of Shear Capacity ratio for element: column JTC1 of floor 1
 At Shear local axis: 2
 (Bending local axis: 3)
 Section Type: rcjtcs

Constant Properties

 Knowledge Factor, $\phi = 0.80$
 Mean strength values are used for both shear and moment calculations.
 Consequently:
 Jacket
 New material of Primary Member: Concrete Strength, $f_c = f_{cm} = 18.00$
 New material of Primary Member: Steel Strength, $f_s = f_{sm} = 625.00$
 Concrete Elasticity, $E_c = 19940.411$
 Steel Elasticity, $E_s = 200000.00$
 Existing Column
 Existing material of Primary Member: Concrete Strength, $f_c = f_{cm} = 12.00$
 Existing material of Primary Member: Steel Strength, $f_s = f_{sm} = 525.00$
 Concrete Elasticity, $E_c = 16281.278$
 Steel Elasticity, $E_s = 200000.00$
 #####
 Note: Especially for the calculation of moment strengths,
 the above steel re-bar strengths are multiplied by 1.25 according to R18.6.5, ACI 318-14
 Jacket
 New material: Steel Strength, $f_s = 1.25 * f_{sm} = 781.25$
 Existing Column
 Existing material: Steel Strength, $f_s = 1.25 * f_{sm} = 656.25$
 #####
 Max Height, $H_{max} = 750.00$
 Min Height, $H_{min} = 450.00$
 Max Width, $W_{max} = 950.00$

Min Width, $W_{min} = 450.00$
 Eccentricity, $Ecc = 250.00$
 Jacket Thickness, $t_j = 100.00$
 Cover Thickness, $c = 25.00$
 Mean Confinement Factor overall section = 1.42131
 Element Length, $L = 3000.00$
 Primary Member
 Ribbed Bars
 Ductile Steel
 With Detailing for Earthquake Resistance (including stirrups closed at 135°)
 Longitudinal Bars With Ends Lapped Starting at the End Sections
 Adequate Lap Length ($l_o/l_{ou,min} \geq 1$)
 No FRP Wrapping

Stepwise Properties

At local axis: 2
 EDGE -A-
 Shear Force, $V_a = -137.4894$
 EDGE -B-
 Shear Force, $V_b = 137.4894$
 BOTH EDGES
 Axial Force, $F = -7.6839E+006$
 Longitudinal Reinforcement Area Distribution (in 2 divisions)
 -Tension: $As_t = 314.1593$
 -Compression: $As_c = 6176.371$
 Longitudinal Reinforcement Area Distribution (in 3 divisions)
 -Tension: $As_{ten} = 1539.38$
 -Compression: $As_{com} = 1539.38$
 -Middle: $As_{mid} = 3612.832$

Calculation of Shear Capacity ratio, $V_e/V_r = 0.86614729$
 Member Controlled by Flexure ($V_e/V_r < 1$)
 Calculation of Shear Demand from fig. R18.6.5, ACI 318-14 $V_e = (M_{pr1} + M_{pr2})/l_n = 1.2287E+006$
 with
 $M_{pr1} = \max(\mu_{u1+}, \mu_{u1-}) = 1.8431E+009$
 $\mu_{u1+} = 1.8431E+009$, is the ultimate moment strength at the edge 1 of the member in the actual moment direction which is defined for the static loading combination
 $\mu_{u1-} = 1.8431E+009$, is the ultimate moment strength at the edge 1 of the member in the opposite moment direction which is defined for the static loading combination
 $M_{pr2} = \max(\mu_{u2+}, \mu_{u2-}) = 1.8431E+009$
 $\mu_{u2+} = 1.8431E+009$, is the ultimate moment strength at the edge 2 of the member in the actual moment direction which is defined for the static loading combination
 $\mu_{u2-} = 1.8431E+009$, is the ultimate moment strength at the edge 2 of the member in the opposite moment direction which is defined for the static loading combination

Calculation of μ_{u1+}

Calculation of ultimate curvature ϕ_u according to 4.1, Biskinis/Fardis 2013:
 $\phi_u = 7.8730391E-006$
 $\mu_u = 1.8431E+009$

with full section properties:

$b = 450.00$
 $d = 907.00$
 $d' = 43.00$
 $v = 1.0459$
 $N = 7.6839E+006$
 $f_c = 18.00$
 $\alpha (5A.5, TBDY) = 0.002$

Final value of c_u : $c_u^* = \text{shear_factor} * \text{Max}(c_u, c_c) = 0.01486841$

The Shear_factor is considered equal to 1 (pure moment strength)

From (5.4b), TBDY: $c_u = 0.01486841$

w_e (5.4c) = 0.08077545

a_{se} ((5.4d), TBDY) = $(a_{se1} * A_{ext} + a_{se2} * A_{int}) / A_{sec} = 0.53375773$

$a_{se1} = \text{Max}(((A_{conf,max1} - A_{noConf1}) / A_{conf,max1}) * (A_{conf,min1} / A_{conf,max1}), 0) = 0.53375773$

The definitions of A_{noConf} , $A_{conf,min}$ and $A_{conf,max}$ are derived from generalization of the rectangular sections confinement, which is expressed by (5.4d).

The generalization is done according to Mander, J., Priestley, M., and Park, R. (1988)

"Theoretical Stress-Strain Model for Confined Concrete."

J. Struct. Eng., 10.1061/(ASCE)0733-9445(1988)114:8(1804), 1804-1826.

$A_{conf,max1} = 464100.00$ is the confined core area at levels of member with hoops and

is calculated as the area of core enclosed by the center lines of the external perimeter hoops.

$A_{conf,min1} = 395025.00$ is the confined core area at midway between the levels of hoops and

is calculated by reducing all the dimensions of the area $A_{conf,max1}$ by a length

equal to half the clear spacing between external hoops.

$A_{noConf1} = 173066.667$ is the unconfined external core area which is equal to $b_i^2/6$ as defined at (A.2).

$a_{se2} (>= a_{se1}) = \text{Max}(((A_{conf,max2} - A_{noConf2}) / A_{conf,max2}) * (A_{conf,min2} / A_{conf,max2}), 0) = 0.53375773$

The definitions of A_{noConf} , $A_{conf,min}$ and $A_{conf,max}$ are derived from generalization of the rectangular sections confinement, which is expressed by (5.4d).

The generalization is done according to Mander, J., Priestley, M., and Park, R. (1988)

"Theoretical Stress-Strain Model for Confined Concrete."

J. Struct. Eng., 10.1061/(ASCE)0733-9445(1988)114:8(1804), 1804-1826.

$A_{conf,max2} = 252164.00$ is the confined core area at levels of member with hoops and

is calculated as the area of core enclosed by the center lines of the internal perimeter hoops.

$A_{conf,min2} = 111441.00$ is the confined core area at midway between the levels of hoops and

is calculated by reducing all the dimensions of the area $A_{conf,max2}$ by a length

equal to half the clear spacing between internal hoops.

$A_{noConf2} = 110709.333$ is the unconfined internal core area which is equal to $b_i^2/6$ as defined at (A.2).

$p_{sh,min} * F_{ywe} = \text{Min}(p_{sh,x} * F_{ywe}, p_{sh,y} * F_{ywe}) = 2.724$

$p_{sh,x} * F_{ywe} = p_{sh1} * F_{ywe1} + p_{s2} * F_{ywe2} = 2.724$

p_{sh1} ((5.4d), TBDY) = $L_{stir1} * A_{stir1} / (A_{sec} * s_1) = 0.00301593$

L_{stir1} (Length of stirrups along Y) = 2160.00

A_{stir1} (stirrups area) = 78.53982

p_{sh2} (5.4d) = $L_{stir2} * A_{stir2} / (A_{sec} * s_2) = 0.00056047$

L_{stir2} (Length of stirrups along Y) = 1568.00

A_{stir2} (stirrups area) = 50.26548

$p_{sh,y} * F_{ywe} = p_{sh1} * F_{ywe1} + p_{s2} * F_{ywe2} = 3.25416$

p_{sh1} ((5.4d), TBDY) = $L_{stir1} * A_{stir1} / (A_{sec} * s_1) = 0.00357443$

L_{stir1} (Length of stirrups along X) = 2560.00

A_{stir1} (stirrups area) = 78.53982

p_{sh2} ((5.4d), TBDY) = $L_{stir2} * A_{stir2} / (A_{sec} * s_2) = 0.00070345$

L_{stir2} (Length of stirrups along X) = 1968.00

A_{stir2} (stirrups area) = 50.26548

$A_{sec} = 562500.00$

$s_1 = 100.00$

$s_2 = 250.00$

$f_{ywe1} = 781.25$

$f_{ywe2} = 656.25$

$f_{ce} = 18.00$

From ((5.A5), TBDY), TBDY: $c_c = 0.00621307$

$c = \text{confinement factor} = 1.42131$

$y_1 = 0.0025$

$sh_1 = 0.008$

$ft_1 = 907.50$

$fy_1 = 756.25$

$su_1 = 0.032$

using (30) in Biskinis/Fardis (2013) multiplied with shear_factor

and also multiplied by the shear_factor according to 15.7.1.4, with

Shear_factor = 1.00

$l_o/l_{ou,min} = l_b/l_d = 1.00$

$su_1 = 0.4 * esu_{1,nominal}$ ((5.5), TBDY) = 0.032

From table 5A.1, TBDY: $esu_{1,nominal} = 0.08$,

For calculation of $esu_{1,nominal}$ and y_1 , sh_1 , ft_1 , fy_1 , it is considered

characteristic value $fsy1 = fs1/1.2$, from table 5.1, TBDY.
 $y1, sh1, ft1, fy1$, are also multiplied by $\text{Min}(1, 1.25 \cdot (lb/ld)^{2/3})$, from 10.3.5, ASCE 41-17.
with $fs1 = (fs_{jacket} \cdot Asl_{ten,jacket} + fs_{core} \cdot Asl_{ten,core}) / Asl_{ten} = 756.25$
with $Es1 = (Es_{jacket} \cdot Asl_{ten,jacket} + Es_{core} \cdot Asl_{ten,core}) / Asl_{ten} = 200000.00$
 $y2 = 0.0025$
 $sh2 = 0.008$
 $ft2 = 907.50$
 $fy2 = 756.25$
 $su2 = 0.032$
using (30) in Biskinis/Fardis (2013) multiplied with shear_factor
and also multiplied by the shear_factor according to 15.7.1.4, with
Shear_factor = 1.00
 $lo/lou, min = lb/lb, min = 1.00$
 $su2 = 0.4 \cdot esu2_{nominal} ((5.5), TBDY) = 0.032$
From table 5A.1, TBDY: $esu2_{nominal} = 0.08$,
For calculation of $esu2_{nominal}$ and $y2, sh2, ft2, fy2$, it is considered
characteristic value $fsy2 = fs2/1.2$, from table 5.1, TBDY.
 $y1, sh1, ft1, fy1$, are also multiplied by $\text{Min}(1, 1.25 \cdot (lb/ld)^{2/3})$, from 10.3.5, ASCE 41-17.
with $fs2 = (fs_{jacket} \cdot Asl_{com,jacket} + fs_{core} \cdot Asl_{com,core}) / Asl_{com} = 756.25$
with $Es2 = (Es_{jacket} \cdot Asl_{com,jacket} + Es_{core} \cdot Asl_{com,core}) / Asl_{com} = 200000.00$
 $yv = 0.0025$
 $shv = 0.008$
 $ftv = 899.1522$
 $fyv = 749.2935$
 $suv = 0.032$
using (30) in Biskinis/Fardis (2013) multiplied with shear_factor
and also multiplied by the shear_factor according to 15.7.1.4, with
Shear_factor = 1.00
 $lo/lou, min = lb/ld = 1.00$
 $suv = 0.4 \cdot esuv_{nominal} ((5.5), TBDY) = 0.032$
From table 5A.1, TBDY: $esuv_{nominal} = 0.08$,
considering characteristic value $fsyv = fsv/1.2$, from table 5.1, TBDY
For calculation of $esuv_{nominal}$ and yv, shv, ftv, fyv , it is considered
characteristic value $fsyv = fsv/1.2$, from table 5.1, TBDY.
 $y1, sh1, ft1, fy1$, are also multiplied by $\text{Min}(1, 1.25 \cdot (lb/ld)^{2/3})$, from 10.3.5, ASCE 41-17.
with $fsv = (fs_{jacket} \cdot Asl_{mid,jacket} + fs_{mid} \cdot Asl_{mid,core}) / Asl_{mid} = 749.2935$
with $Es_v = (Es_{jacket} \cdot Asl_{mid,jacket} + Es_{mid} \cdot Asl_{mid,core}) / Asl_{mid} = 200000.00$
 $1 = Asl_{ten} / (b \cdot d) \cdot (fs1 / fc) = 0.15845977$
 $2 = Asl_{com} / (b \cdot d) \cdot (fs2 / fc) = 0.15845977$
 $v = Asl_{mid} / (b \cdot d) \cdot (fsv / fc) = 0.36847443$
and confined core properties:
 $b = 390.00$
 $d = 877.00$
 $d' = 13.00$
 $fcc (5A.2, TBDY) = 25.58352$
 $cc (5A.5, TBDY) = 0.00621307$
 $c = \text{confinement factor} = 1.42131$
 $1 = Asl_{ten} / (b \cdot d) \cdot (fs1 / fc) = 0.18909264$
 $2 = Asl_{com} / (b \cdot d) \cdot (fs2 / fc) = 0.18909264$
 $v = Asl_{mid} / (b \cdot d) \cdot (fsv / fc) = 0.43970658$
Case/Assumption: Unconfined full section - Steel rupture
' satisfies Eq. (4.3)
--->
 $v < vs,y2$ - LHS eq.(4.5) is not satisfied
--->
 $v < vs,c$ - RHS eq.(4.5) is not satisfied
--->
Case/Assumption Rejected.
--->
New Case/Assumption: Unconfined full section - Spalling of concrete cover
' satisfies Eq. (4.4)
--->
 $v < s,y1$ - LHS eq.(4.7) is not satisfied
--->
 $v < vc,y1$ - RHS eq.(4.6) is satisfied
--->
 $cu (4.10) = 0.83934343$

MRC (4.17) = 1.8439E+009

---->

New Case/Assumption: Ultimate curvature of confined core after spalling of concrete cover

In expressions below, the following modifications have been made

- b, d, d' replaced by geometric parameters of the core: bo, do, d'o
- N, ϵ_1 , ϵ_2 , ν normalised to b_o*d_o , instead of $b*d$
- ϵ_c , ϵ_{cc} parameters of confined concrete, ϵ_{cc} , used in lieu of ϵ_c , ϵ_{cc}

---->

Subcase: Rupture of tension steel

---->

$\nu^* < \nu^*_{s,y2}$ - LHS eq.(4.5) is not satisfied

---->

$\nu^* < \nu^*_{s,c}$ - LHS eq.(4.5) is not satisfied

---->

Subcase rejected

---->

New Subcase: Failure of compression zone

---->

$\nu^* < \nu^*_{c,y2}$ - LHS eq.(4.6) is not satisfied

---->

$\nu^* < \nu^*_{c,y1}$ - RHS eq.(4.6) is not satisfied

---->

ϵ_{cu} (4.11) = 0.89983732

MRO (4.18) = 1.8431E+009

---->

$\epsilon_u = \epsilon_{cu}$ (4.2) = 7.8730391E-006

$M_u = M_{RO}$

Calculation of ratio I_b/I_d

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

Calculation of M_{u1} -

Calculation of ultimate curvature ϵ_u according to 4.1, Biskinis/Fardis 2013:

$\epsilon_u = 7.8730391E-006$

$M_u = 1.8431E+009$

with full section properties:

$b = 450.00$

$d = 907.00$

$d' = 43.00$

$\nu = 1.0459$

$N = 7.6839E+006$

$f_c = 18.00$

ϵ_{co} (5A.5, TBDY) = 0.002

Final value of ϵ_{cu} : $\epsilon_{cu}^* = \text{shear_factor} * \text{Max}(\epsilon_{cu}, \epsilon_{cc}) = 0.01486841$

The Shear_factor is considered equal to 1 (pure moment strength)

From (5.4b), TBDY: $\epsilon_{cu} = 0.01486841$

ϵ_{we} (5.4c) = 0.08077545

ϵ_{ase} ((5.4d), TBDY) = $(\epsilon_{ase1} * A_{ext} + \epsilon_{ase2} * A_{int}) / A_{sec} = 0.53375773$

$\epsilon_{ase1} = \text{Max}(((A_{conf,max1} - A_{noConf1}) / A_{conf,max1}) * (A_{conf,min1} / A_{conf,max1}), 0) = 0.53375773$

The definitions of A_{noConf} , $A_{conf,min}$ and $A_{conf,max}$ are derived from generalization of the rectangular sections confinement, which is expressed by (5.4d).

The generalization is done according to Mander, J., Priestley, M., and Park, R. (1988)

"Theoretical Stress-Strain Model for Confined Concrete."

J. Struct. Eng., 10.1061/(ASCE)0733-9445(1988)114:8(1804), 1804-1826.

$A_{conf,max1} = 464100.00$ is the confined core area at levels of member with hoops and

is calculated as the area of core enclosed by the center lines of the external perimeter hoops.

$A_{conf,min1} = 395025.00$ is the confined core area at midway between the levels of hoops and

is calculated by reducing all the dimensions of the area $A_{conf,max1}$ by a length equal to half the clear spacing between external hoops.

AnoConf1 = 173066.667 is the unconfined external core area which is equal to $b_i^2/6$ as defined at (A.2).
 $ase2 (>=ase1) = \text{Max}(((A_{conf,max2}-AnoConf2)/A_{conf,max2}) * (A_{conf,min2}/A_{conf,max2}), 0) = 0.53375773$
The definitions of AnoConf, Aconf,min and Aconf,max are derived from generalization of the rectangular sections confinement, which is expressed by (5.4d).
The generalization is done according to Mander, J., Priestley, M., and Park, R. (1988) "Theoretical Stress-Strain Model for Confined Concrete."
J. Struct. Eng., 10.1061/(ASCE)0733-9445(1988)114:8(1804), 1804-1826.
Aconf,max2 = 252164.00 is the confined core area at levels of member with hoops and is calculated as the area of core enclosed by the center lines of the internal perimeter hoops.
Aconf,min2 = 111441.00 is the confined core area at midway between the levels of hoops and is calculated by reducing all the dimensions of the area Aconf,max2 by a length equal to half the clear spacing between internal hoops.
AnoConf2 = 110709.333 is the unconfined internal core area which is equal to $b_i^2/6$ as defined at (A.2).
 $psh,min * Fywe = \text{Min}(psh,x * Fywe, psh,y * Fywe) = 2.724$

$psh,x * Fywe = psh1 * Fywe1 + ps2 * Fywe2 = 2.724$
 $psh1 ((5.4d), TBDY) = Lstir1 * Astir1 / (Asec * s1) = 0.00301593$
Lstir1 (Length of stirrups along Y) = 2160.00
Astir1 (stirrups area) = 78.53982
 $psh2 ((5.4d)) = Lstir2 * Astir2 / (Asec * s2) = 0.00056047$
Lstir2 (Length of stirrups along Y) = 1568.00
Astir2 (stirrups area) = 50.26548

$psh,y * Fywe = psh1 * Fywe1 + ps2 * Fywe2 = 3.25416$
 $psh1 ((5.4d), TBDY) = Lstir1 * Astir1 / (Asec * s1) = 0.00357443$
Lstir1 (Length of stirrups along X) = 2560.00
Astir1 (stirrups area) = 78.53982
 $psh2 ((5.4d), TBDY) = Lstir2 * Astir2 / (Asec * s2) = 0.00070345$
Lstir2 (Length of stirrups along X) = 1968.00
Astir2 (stirrups area) = 50.26548

Asec = 562500.00

s1 = 100.00

s2 = 250.00

fywe1 = 781.25

fywe2 = 656.25

fce = 18.00

From ((5.A5), TBDY), TBDY: cc = 0.00621307

c = confinement factor = 1.42131

y1 = 0.0025

sh1 = 0.008

ft1 = 907.50

fy1 = 756.25

su1 = 0.032

using (30) in Biskinis/Fardis (2013) multiplied with shear_factor and also multiplied by the shear_factor according to 15.7.1.4, with Shear_factor = 1.00

lo/lou,min = lb/lb = 1.00

su1 = 0.4 * esu1_nominal ((5.5), TBDY) = 0.032

From table 5A.1, TBDY: esu1_nominal = 0.08,

For calculation of esu1_nominal and y1, sh1, ft1, fy1, it is considered characteristic value fsy1 = fs1/1.2, from table 5.1, TBDY.

y1, sh1, ft1, fy1, are also multiplied by $\text{Min}(1, 1.25 * (lb/lb)^{2/3})$, from 10.3.5, ASCE 41-17.

with fs1 = (fs,jacket * Asl,ten,jacket + fs,core * Asl,ten,core) / Asl,ten = 756.25

with Es1 = (Es,jacket * Asl,ten,jacket + Es,core * Asl,ten,core) / Asl,ten = 200000.00

y2 = 0.0025

sh2 = 0.008

ft2 = 907.50

fy2 = 756.25

su2 = 0.032

using (30) in Biskinis/Fardis (2013) multiplied with shear_factor and also multiplied by the shear_factor according to 15.7.1.4, with Shear_factor = 1.00

lo/lou,min = lb/lb,min = 1.00

su2 = 0.4 * esu2_nominal ((5.5), TBDY) = 0.032

From table 5A.1, TBDY: esu2_nominal = 0.08,

For calculation of esu2_nominal and y2, sh2, ft2, fy2, it is considered

characteristic value $fsy2 = fs2/1.2$, from table 5.1, TBDY.
 $y1, sh1, ft1, fy1$, are also multiplied by $\text{Min}(1, 1.25 \cdot (lb/ld)^{2/3})$, from 10.3.5, ASCE 41-17.
 with $fs2 = (fs_{jacket} \cdot Asl_{com,jacket} + fs_{core} \cdot Asl_{com,core}) / Asl_{com} = 756.25$
 with $Es2 = (Es_{jacket} \cdot Asl_{com,jacket} + Es_{core} \cdot Asl_{com,core}) / Asl_{com} = 200000.00$
 $yv = 0.0025$
 $shv = 0.008$
 $ftv = 899.1522$
 $fyv = 749.2935$
 $suv = 0.032$
 using (30) in Biskinis/Fardis (2013) multiplied with shear_factor
 and also multiplied by the shear_factor according to 15.7.1.4, with
 Shear_factor = 1.00
 $lo/lou, min = lb/ld = 1.00$
 $suv = 0.4 \cdot esuv_{nominal} ((5.5), TBDY) = 0.032$
 From table 5A.1, TBDY: $esuv_{nominal} = 0.08$,
 considering characteristic value $fsyv = fsv/1.2$, from table 5.1, TBDY
 For calculation of $esuv_{nominal}$ and yv, shv, ftv, fyv , it is considered
 characteristic value $fsyv = fsv/1.2$, from table 5.1, TBDY.
 $y1, sh1, ft1, fy1$, are also multiplied by $\text{Min}(1, 1.25 \cdot (lb/ld)^{2/3})$, from 10.3.5, ASCE 41-17.
 with $fsv = (fs_{jacket} \cdot Asl_{mid,jacket} + fs_{mid} \cdot Asl_{mid,core}) / Asl_{mid} = 749.2935$
 with $Esv = (Es_{jacket} \cdot Asl_{mid,jacket} + Es_{mid} \cdot Asl_{mid,core}) / Asl_{mid} = 200000.00$
 $1 = Asl_{ten} / (b \cdot d) \cdot (fs1 / fc) = 0.15845977$
 $2 = Asl_{com} / (b \cdot d) \cdot (fs2 / fc) = 0.15845977$
 $v = Asl_{mid} / (b \cdot d) \cdot (fsv / fc) = 0.36847443$
 and confined core properties:
 $b = 390.00$
 $d = 877.00$
 $d' = 13.00$
 $fcc (5A.2, TBDY) = 25.58352$
 $cc (5A.5, TBDY) = 0.00621307$
 $c = \text{confinement factor} = 1.42131$
 $1 = Asl_{ten} / (b \cdot d) \cdot (fs1 / fc) = 0.18909264$
 $2 = Asl_{com} / (b \cdot d) \cdot (fs2 / fc) = 0.18909264$
 $v = Asl_{mid} / (b \cdot d) \cdot (fsv / fc) = 0.43970658$
 Case/Assumption: Unconfined full section - Steel rupture
 ' satisfies Eq. (4.3)
 --->
 $v < vs,y2$ - LHS eq.(4.5) is not satisfied
 --->
 $v < vs,c$ - RHS eq.(4.5) is not satisfied
 --->
 Case/Assumption Rejected.
 --->
 New Case/Assumption: Unconfined full section - Spalling of concrete cover
 ' satisfies Eq. (4.4)
 --->
 $v < s,y1$ - LHS eq.(4.7) is not satisfied
 --->
 $v < vc,y1$ - RHS eq.(4.6) is satisfied
 --->
 $cu (4.10) = 0.83934343$
 $MRC (4.17) = 1.8439E+009$
 --->
 New Case/Assumption: Ultimate curvature of confined core after spalling of concrete cover
 In expressions below, the following modifications have been made
 - b, d, d' replaced by geometric parameters of the core: $bo, do, d'o$
 - $N, 1, 2, v$ normalised to $bo \cdot do$, instead of $b \cdot d$
 - parameters of confined concrete, fcc, cc , used in lieu of fc, ecu
 --->
 Subcase: Rupture of tension steel
 --->
 $v^* < v^*,y2$ - LHS eq.(4.5) is not satisfied
 --->
 $v^* < v^*,c$ - LHS eq.(4.5) is not satisfied
 --->
 Subcase rejected
 --->

New Subcase: Failure of compression zone

--->

$v^* < v^*_{c,y2}$ - LHS eq.(4.6) is not satisfied

--->

$v^* < v^*_{c,y1}$ - RHS eq.(4.6) is not satisfied

--->

$*cu(4.11) = 0.89983732$

$M_{Ro}(4.18) = 1.8431E+009$

--->

$u = cu(4.2) = 7.8730391E-006$

$\mu = M_{Ro}$

Calculation of ratio l_b/d

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

Calculation of μ_{2+}

Calculation of ultimate curvature u according to 4.1, Biskinis/Fardis 2013:

$u = 7.8730391E-006$

$\mu = 1.8431E+009$

with full section properties:

$b = 450.00$

$d = 907.00$

$d' = 43.00$

$v = 1.0459$

$N = 7.6839E+006$

$f_c = 18.00$

$\alpha(5A.5, TBDY) = 0.002$

Final value of cu : $cu^* = \text{shear_factor} * \text{Max}(cu, cc) = 0.01486841$

The Shear_factor is considered equal to 1 (pure moment strength)

From (5.4b), TBDY: $cu = 0.01486841$

$w_e(5.4c) = 0.08077545$

$ase((5.4d), TBDY) = (ase1 * A_{ext} + ase2 * A_{int}) / A_{sec} = 0.53375773$

$ase1 = \text{Max}(((A_{conf,max1} - A_{noConf1}) / A_{conf,max1}) * (A_{conf,min1} / A_{conf,max1}), 0) = 0.53375773$

The definitions of A_{noConf} , $A_{conf,min}$ and $A_{conf,max}$ are derived from generalization of the rectangular sections confinement, which is expressed by (5.4d).

The generalization is done according to Mander, J., Priestley, M., and Park, R. (1988)

"Theoretical Stress-Strain Model for Confined Concrete."

J. Struct. Eng., 10.1061/(ASCE)0733-9445(1988)114:8(1804), 1804-1826.

$A_{conf,max1} = 464100.00$ is the confined core area at levels of member with hoops and is calculated as the area of core enclosed by the center lines of the external perimeter hoops.

$A_{conf,min1} = 395025.00$ is the confined core area at midway between the levels of hoops and is calculated by reducing all the dimensions of the area $A_{conf,max1}$ by a length equal to half the clear spacing between external hoops.

$A_{noConf1} = 173066.667$ is the unconfined external core area which is equal to $b_i^2/6$ as defined at (A.2).

$ase2 (\geq ase1) = \text{Max}(((A_{conf,max2} - A_{noConf2}) / A_{conf,max2}) * (A_{conf,min2} / A_{conf,max2}), 0) = 0.53375773$

The definitions of A_{noConf} , $A_{conf,min}$ and $A_{conf,max}$ are derived from generalization of the rectangular sections confinement, which is expressed by (5.4d).

The generalization is done according to Mander, J., Priestley, M., and Park, R. (1988)

"Theoretical Stress-Strain Model for Confined Concrete."

J. Struct. Eng., 10.1061/(ASCE)0733-9445(1988)114:8(1804), 1804-1826.

$A_{conf,max2} = 252164.00$ is the confined core area at levels of member with hoops and is calculated as the area of core enclosed by the center lines of the internal perimeter hoops.

$A_{conf,min2} = 111441.00$ is the confined core area at midway between the levels of hoops and is calculated by reducing all the dimensions of the area $A_{conf,max2}$ by a length equal to half the clear spacing between internal hoops.

$A_{noConf2} = 110709.333$ is the unconfined internal core area which is equal to $b_i^2/6$ as defined at (A.2).

$p_{sh,min} * F_{ywe} = \text{Min}(p_{sh,x} * F_{ywe}, p_{sh,y} * F_{ywe}) = 2.724$

$p_{sh,x} * F_{ywe} = p_{sh1} * F_{ywe1} + p_{sh2} * F_{ywe2} = 2.724$

$psh1 ((5.4d), TBDY) = Lstir1 * Astir1 / (Asec * s1) = 0.00301593$
 $Lstir1$ (Length of stirrups along Y) = 2160.00
 $Astir1$ (stirrups area) = 78.53982
 $psh2 (5.4d) = Lstir2 * Astir2 / (Asec * s2) = 0.00056047$
 $Lstir2$ (Length of stirrups along Y) = 1568.00
 $Astir2$ (stirrups area) = 50.26548

$psh_y * Fywe = psh1 * Fywe1 + ps2 * Fywe2 = 3.25416$
 $psh1 ((5.4d), TBDY) = Lstir1 * Astir1 / (Asec * s1) = 0.00357443$
 $Lstir1$ (Length of stirrups along X) = 2560.00
 $Astir1$ (stirrups area) = 78.53982
 $psh2 ((5.4d), TBDY) = Lstir2 * Astir2 / (Asec * s2) = 0.00070345$
 $Lstir2$ (Length of stirrups along X) = 1968.00
 $Astir2$ (stirrups area) = 50.26548

Asec = 562500.00

s1 = 100.00

s2 = 250.00

fywe1 = 781.25

fywe2 = 656.25

fce = 18.00

From ((5.A5), TBDY), TBDY: cc = 0.00621307

c = confinement factor = 1.42131

y1 = 0.0025

sh1 = 0.008

ft1 = 907.50

fy1 = 756.25

su1 = 0.032

using (30) in Biskinis/Fardis (2013) multiplied with shear_factor
and also multiplied by the shear_factor according to 15.7.1.4, with
Shear_factor = 1.00

lo/lou,min = lb/ld = 1.00

su1 = 0.4*esu1_nominal ((5.5), TBDY) = 0.032

From table 5A.1, TBDY: esu1_nominal = 0.08,

For calculation of esu1_nominal and y1, sh1, ft1, fy1, it is considered
characteristic value fsy1 = fs1/1.2, from table 5.1, TBDY.

y1, sh1, ft1, fy1, are also multiplied by $\text{Min}(1, 1.25 * (lb/ld)^{2/3})$, from 10.3.5, ASCE 41-17.

with fs1 = (fs,jacket*Asl,ten,jacket + fs,core*Asl,ten,core)/Asl,ten = 756.25

with Es1 = (Es,jacket*Asl,ten,jacket + Es,core*Asl,ten,core)/Asl,ten = 200000.00

y2 = 0.0025

sh2 = 0.008

ft2 = 907.50

fy2 = 756.25

su2 = 0.032

using (30) in Biskinis/Fardis (2013) multiplied with shear_factor
and also multiplied by the shear_factor according to 15.7.1.4, with
Shear_factor = 1.00

lo/lou,min = lb/lb,min = 1.00

su2 = 0.4*esu2_nominal ((5.5), TBDY) = 0.032

From table 5A.1, TBDY: esu2_nominal = 0.08,

For calculation of esu2_nominal and y2, sh2, ft2, fy2, it is considered
characteristic value fsy2 = fs2/1.2, from table 5.1, TBDY.

y1, sh1, ft1, fy1, are also multiplied by $\text{Min}(1, 1.25 * (lb/ld)^{2/3})$, from 10.3.5, ASCE 41-17.

with fs2 = (fs,jacket*Asl,com,jacket + fs,core*Asl,com,core)/Asl,com = 756.25

with Es2 = (Es,jacket*Asl,com,jacket + Es,core*Asl,com,core)/Asl,com = 200000.00

yv = 0.0025

shv = 0.008

ftv = 899.1522

fyv = 749.2935

suv = 0.032

using (30) in Biskinis/Fardis (2013) multiplied with shear_factor
and also multiplied by the shear_factor according to 15.7.1.4, with
Shear_factor = 1.00

lo/lou,min = lb/ld = 1.00

suv = 0.4*esuv_nominal ((5.5), TBDY) = 0.032

From table 5A.1, TBDY: esuv_nominal = 0.08,

considering characteristic value fsyv = fsv/1.2, from table 5.1, TBDY

```

For calculation of esuv_nominal and yv, shv,ftv,fyv, it is considered
characteristic value fsyv = fsv/1.2, from table 5.1, TBDY.
y1, sh1,ft1,fy1, are also multiplied by Min(1,1.25*(lb/ld)^ 2/3), from 10.3.5, ASCE 41-17.
with fsv = (fsjacket*Asl,mid,jacket + fs,mid*Asl,mid,core)/Asl,mid = 749.2935
with Esv = (Es,jacket*Asl,mid,jacket + Es,mid*Asl,mid,core)/Asl,mid = 200000.00
1 = Asl,ten/(b*d)*(fs1/fc) = 0.15845977
2 = Asl,com/(b*d)*(fs2/fc) = 0.15845977
v = Asl,mid/(b*d)*(fsv/fc) = 0.36847443
and confined core properties:
b = 390.00
d = 877.00
d' = 13.00
fcc (5A.2, TBDY) = 25.58352
cc (5A.5, TBDY) = 0.00621307
c = confinement factor = 1.42131
1 = Asl,ten/(b*d)*(fs1/fc) = 0.18909264
2 = Asl,com/(b*d)*(fs2/fc) = 0.18909264
v = Asl,mid/(b*d)*(fsv/fc) = 0.43970658
Case/Assumption: Unconfined full section - Steel rupture
' satisfies Eq. (4.3)
---->
v < vs,y2 - LHS eq.(4.5) is not satisfied
---->
v < vs,c - RHS eq.(4.5) is not satisfied
---->
Case/Assumption Rejected.
---->
New Case/Assumption: Unconfined full section - Spalling of concrete cover
' satisfies Eq. (4.4)
---->
v < s,y1 - LHS eq.(4.7) is not satisfied
---->
v < vc,y1 - RHS eq.(4.6) is satisfied
---->
cu (4.10) = 0.83934343
MRc (4.17) = 1.8439E+009
---->
New Case/Assumption: Ultimate curvature of confined core after spalling of concrete cover
In expressions below, the following modifications have been made
- b, d, d' replaced by geometric parameters of the core: bo, do, d'o
- N, 1, 2, v normalised to bo*do, instead of b*d
- parameters of confined concrete, fcc, cc, used in lieu of fc, ecu
---->
Subcase: Rupture of tension steel
---->
v* < v*s,y2 - LHS eq.(4.5) is not satisfied
---->
v* < v*s,c - LHS eq.(4.5) is not satisfied
---->
Subcase rejected
---->
New Subcase: Failure of compression zone
---->
v* < v*c,y2 - LHS eq.(4.6) is not satisfied
---->
v* < v*c,y1 - RHS eq.(4.6) is not satisfied
---->
*cu (4.11) = 0.89983732
MRo (4.18) = 1.8431E+009
---->
u = cu (4.2) = 7.8730391E-006
Mu = MRo

```

Calculation of ratio lb/ld

Adequate Lap Length: lb/ld >= 1

Calculation of Mu2-

Calculation of ultimate curvature μ according to 4.1, Biskinis/Fardis 2013:

$$\mu = 7.8730391\text{E-}006$$

$$\mu_u = 1.8431\text{E+}009$$

with full section properties:

$$b = 450.00$$

$$d = 907.00$$

$$d' = 43.00$$

$$\nu = 1.0459$$

$$N = 7.6839\text{E+}006$$

$$f_c = 18.00$$

$$\alpha (5A.5, \text{TB DY}) = 0.002$$

$$\text{Final value of } \mu_u: \mu_u^* = \text{shear_factor} * \text{Max}(\mu_u, \alpha) = 0.01486841$$

The Shear_factor is considered equal to 1 (pure moment strength)

$$\text{From (5.4b), TB DY: } \mu_u = 0.01486841$$

$$\mu_{ue} (5.4c) = 0.08077545$$

$$\mu_{ase} ((5.4d), \text{TB DY}) = (\mu_{ase1} * A_{ext} + \mu_{ase2} * A_{int}) / A_{sec} = 0.53375773$$

$$\mu_{ase1} = \text{Max}(((A_{conf,max1} - A_{noConf1}) / A_{conf,max1}) * (A_{conf,min1} / A_{conf,max1}), 0) = 0.53375773$$

The definitions of A_{noConf} , $A_{conf,min}$ and $A_{conf,max}$ are derived from generalization of the rectangular sections confinement, which is expressed by (5.4d).

The generalization is done according to Mander, J., Priestley, M., and Park, R. (1988)

"Theoretical Stress-Strain Model for Confined Concrete."

J. Struct. Eng., 10.1061/(ASCE)0733-9445(1988)114:8(1804), 1804-1826.

$A_{conf,max1} = 464100.00$ is the confined core area at levels of member with hoops and

is calculated as the area of core enclosed by the center lines of the external perimeter hoops.

$A_{conf,min1} = 395025.00$ is the confined core area at midway between the levels of hoops and

is calculated by reducing all the dimensions of the area $A_{conf,max1}$ by a length equal to half the clear spacing between external hoops.

$A_{noConf1} = 173066.667$ is the unconfined external core area which is equal to $b_i^2/6$ as defined at (A.2).

$$\mu_{ase2} (> \mu_{ase1}) = \text{Max}(((A_{conf,max2} - A_{noConf2}) / A_{conf,max2}) * (A_{conf,min2} / A_{conf,max2}), 0) = 0.53375773$$

The definitions of A_{noConf} , $A_{conf,min}$ and $A_{conf,max}$ are derived from generalization of the rectangular sections confinement, which is expressed by (5.4d).

The generalization is done according to Mander, J., Priestley, M., and Park, R. (1988)

"Theoretical Stress-Strain Model for Confined Concrete."

J. Struct. Eng., 10.1061/(ASCE)0733-9445(1988)114:8(1804), 1804-1826.

$A_{conf,max2} = 252164.00$ is the confined core area at levels of member with hoops and

is calculated as the area of core enclosed by the center lines of the internal perimeter hoops.

$A_{conf,min2} = 111441.00$ is the confined core area at midway between the levels of hoops and

is calculated by reducing all the dimensions of the area $A_{conf,max2}$ by a length equal to half the clear spacing between internal hoops.

$A_{noConf2} = 110709.333$ is the unconfined internal core area which is equal to $b_i^2/6$ as defined at (A.2).

$$\text{psh}_{min} * F_{ywe} = \text{Min}(\text{psh}_x * F_{ywe}, \text{psh}_y * F_{ywe}) = 2.724$$

$$\text{psh}_x * F_{ywe} = \text{psh}_1 * F_{ywe1} + \text{psh}_2 * F_{ywe2} = 2.724$$

$$\text{psh}_1 ((5.4d), \text{TB DY}) = L_{stir1} * A_{stir1} / (A_{sec} * s_1) = 0.00301593$$

$$L_{stir1} (\text{Length of stirrups along Y}) = 2160.00$$

$$A_{stir1} (\text{stirrups area}) = 78.53982$$

$$\text{psh}_2 (5.4d) = L_{stir2} * A_{stir2} / (A_{sec} * s_2) = 0.00056047$$

$$L_{stir2} (\text{Length of stirrups along Y}) = 1568.00$$

$$A_{stir2} (\text{stirrups area}) = 50.26548$$

$$\text{psh}_y * F_{ywe} = \text{psh}_1 * F_{ywe1} + \text{psh}_2 * F_{ywe2} = 3.25416$$

$$\text{psh}_1 ((5.4d), \text{TB DY}) = L_{stir1} * A_{stir1} / (A_{sec} * s_1) = 0.00357443$$

$$L_{stir1} (\text{Length of stirrups along X}) = 2560.00$$

$$A_{stir1} (\text{stirrups area}) = 78.53982$$

$$\text{psh}_2 ((5.4d), \text{TB DY}) = L_{stir2} * A_{stir2} / (A_{sec} * s_2) = 0.00070345$$

$$L_{stir2} (\text{Length of stirrups along X}) = 1968.00$$

$$A_{stir2} (\text{stirrups area}) = 50.26548$$

$$A_{sec} = 562500.00$$

$s1 = 100.00$
 $s2 = 250.00$
 $fyw1 = 781.25$
 $fyw2 = 656.25$
 $fce = 18.00$
 From ((5A5), TBDY), TBDY: $cc = 0.00621307$
 $c = \text{confinement factor} = 1.42131$
 $y1 = 0.0025$
 $sh1 = 0.008$
 $ft1 = 907.50$
 $fy1 = 756.25$
 $su1 = 0.032$
 using (30) in Biskinis/Fardis (2013) multiplied with shear_factor
 and also multiplied by the shear_factor according to 15.7.1.4, with
 $\text{Shear_factor} = 1.00$
 $lo/lou, \min = lb/ld = 1.00$
 $su1 = 0.4 * esu1_nominal ((5.5), TBDY) = 0.032$
 From table 5A.1, TBDY: $esu1_nominal = 0.08$,
 For calculation of $esu1_nominal$ and $y1, sh1, ft1, fy1$, it is considered
 characteristic value $fsy1 = fs1/1.2$, from table 5.1, TBDY.
 $y1, sh1, ft1, fy1$, are also multiplied by $\text{Min}(1, 1.25 * (lb/ld)^{2/3})$, from 10.3.5, ASCE 41-17.
 with $fs1 = (fs_jacket * Asl, \text{ten}, \text{jacket} + fs_core * Asl, \text{ten}, \text{core}) / Asl, \text{ten} = 756.25$
 with $Es1 = (Es_jacket * Asl, \text{ten}, \text{jacket} + Es_core * Asl, \text{ten}, \text{core}) / Asl, \text{ten} = 200000.00$
 $y2 = 0.0025$
 $sh2 = 0.008$
 $ft2 = 907.50$
 $fy2 = 756.25$
 $su2 = 0.032$
 using (30) in Biskinis/Fardis (2013) multiplied with shear_factor
 and also multiplied by the shear_factor according to 15.7.1.4, with
 $\text{Shear_factor} = 1.00$
 $lo/lou, \min = lb/lb, \min = 1.00$
 $su2 = 0.4 * esu2_nominal ((5.5), TBDY) = 0.032$
 From table 5A.1, TBDY: $esu2_nominal = 0.08$,
 For calculation of $esu2_nominal$ and $y2, sh2, ft2, fy2$, it is considered
 characteristic value $fsy2 = fs2/1.2$, from table 5.1, TBDY.
 $y1, sh1, ft1, fy1$, are also multiplied by $\text{Min}(1, 1.25 * (lb/ld)^{2/3})$, from 10.3.5, ASCE 41-17.
 with $fs2 = (fs_jacket * Asl, \text{com}, \text{jacket} + fs_core * Asl, \text{com}, \text{core}) / Asl, \text{com} = 756.25$
 with $Es2 = (Es_jacket * Asl, \text{com}, \text{jacket} + Es_core * Asl, \text{com}, \text{core}) / Asl, \text{com} = 200000.00$
 $yv = 0.0025$
 $shv = 0.008$
 $ftv = 899.1522$
 $fyv = 749.2935$
 $suv = 0.032$
 using (30) in Biskinis/Fardis (2013) multiplied with shear_factor
 and also multiplied by the shear_factor according to 15.7.1.4, with
 $\text{Shear_factor} = 1.00$
 $lo/lou, \min = lb/ld = 1.00$
 $suv = 0.4 * esuv_nominal ((5.5), TBDY) = 0.032$
 From table 5A.1, TBDY: $esuv_nominal = 0.08$,
 considering characteristic value $fsyv = fsv/1.2$, from table 5.1, TBDY
 For calculation of $esuv_nominal$ and yv, shv, ftv, fyv , it is considered
 characteristic value $fsyv = fsv/1.2$, from table 5.1, TBDY.
 $y1, sh1, ft1, fy1$, are also multiplied by $\text{Min}(1, 1.25 * (lb/ld)^{2/3})$, from 10.3.5, ASCE 41-17.
 with $fsv = (fs_jacket * Asl, \text{mid}, \text{jacket} + fs_mid * Asl, \text{mid}, \text{core}) / Asl, \text{mid} = 749.2935$
 with $Es_v = (Es_jacket * Asl, \text{mid}, \text{jacket} + Es_mid * Asl, \text{mid}, \text{core}) / Asl, \text{mid} = 200000.00$
 $1 = Asl, \text{ten} / (b * d) * (fs1 / fc) = 0.15845977$
 $2 = Asl, \text{com} / (b * d) * (fs2 / fc) = 0.15845977$
 $v = Asl, \text{mid} / (b * d) * (fsv / fc) = 0.36847443$
 and confined core properties:
 $b = 390.00$
 $d = 877.00$
 $d' = 13.00$
 $fcc (5A.2, TBDY) = 25.58352$
 $cc (5A.5, TBDY) = 0.00621307$
 $c = \text{confinement factor} = 1.42131$
 $1 = Asl, \text{ten} / (b * d) * (fs1 / fc) = 0.18909264$

```

2 = Asl,com/(b*d)*(fs2/fc) = 0.18909264
v = Asl,mid/(b*d)*(fsv/fc) = 0.43970658
Case/Assumption: Unconfined full section - Steel rupture
' satisfies Eq. (4.3)
---->
v < vs,y2 - LHS eq.(4.5) is not satisfied
---->
v < vs,c - RHS eq.(4.5) is not satisfied
---->
Case/Assumption Rejected.
---->
New Case/Assumption: Unconfined full section - Spalling of concrete cover
' satisfies Eq. (4.4)
---->
v < s,y1 - LHS eq.(4.7) is not satisfied
---->
v < vc,y1 - RHS eq.(4.6) is satisfied
---->
cu (4.10) = 0.83934343
MRc (4.17) = 1.8439E+009
---->
New Case/Assumption: Ultimate curvature of confined core after spalling of concrete cover
In expressions below, the following modifications have been made
- b, d, d' replaced by geometric parameters of the core: bo, do, d'o
- N, 1, 2, v normalised to bo*do, instead of b*d
- - parameters of confined concrete, fcc, cc, used in lieu of fc, ecu
---->
Subcase: Rupture of tension steel
---->
v* < v*s,y2 - LHS eq.(4.5) is not satisfied
---->
v* < v*s,c - LHS eq.(4.5) is not satisfied
---->
Subcase rejected
---->
New Subcase: Failure of compression zone
---->
v* < v*c,y2 - LHS eq.(4.6) is not satisfied
---->
v* < v*c,y1 - RHS eq.(4.6) is not satisfied
---->
*cu (4.11) = 0.89983732
MRo (4.18) = 1.8431E+009
---->
u = cu (4.2) = 7.8730391E-006
Mu = MRo
-----

Calculation of ratio lb/d
-----
Adequate Lap Length: lb/d >= 1
-----
-----
-----
-----
Calculation of Shear Strength Vr = Min(Vr1,Vr2) = 1.4186E+006
-----
Calculation of Shear Strength at edge 1, Vr1 = 1.4186E+006
Vr1 = VCol ((10.3), ASCE 41-17) = knl*VCol0
VCol0 = 1.4186E+006
knl = 1 (zero step-static loading)
-----
NOTE: In expression (10-3) 'Vs = Av*fy*d/s' is replaced by 'Vs+ f*Vf'
where Vf is the contribution of FRPs (11.3), ACI 440).
-----
= 1 (normal-weight concrete)
Mean concrete strength: fc' = (fc'_jacket*Area_jacket + fc'_core*Area_core)/Area_section = 15.20, but fc'^0.5 <= 8.3

```


MPa (22.5.3.1, ACI 318-14)

$$M/Vd = 4.00$$

$$\mu_u = 2.6919E+008$$

$$V_u = 137.4894$$

$$d = 0.8 \cdot h = 760.00$$

$$N_u = 7.6839E+006$$

$$A_g = 427500.00$$

$$\text{From (11.5.4.8), ACI 318-14: } V_s = V_{sjacket} + V_{s,core} = 1.2262E+006$$

where:

$$V_{sjacket} = V_{sj1} + V_{sj2} = 1.0996E+006$$

$V_{sj1} = 353429.174$ is calculated for section web jacket, with:

$$d = 360.00$$

$$A_v = 157079.633$$

$$f_y = 625.00$$

$$s = 100.00$$

V_{sj1} is multiplied by $Col,j1 = 1.00$

$$s/d = 0.27777778$$

$V_{sj2} = 746128.255$ is calculated for section flange jacket, with:

$$d = 760.00$$

$$A_v = 157079.633$$

$$f_y = 625.00$$

$$s = 100.00$$

V_{sj2} is multiplied by $Col,j2 = 1.00$

$$s/d = 0.13157895$$

$$V_{s,core} = V_{s,c1} + V_{s,c2} = 126669.016$$

$V_{s,c1} = 0.00$ is calculated for section web core, with:

$$d = 200.00$$

$$A_v = 100530.965$$

$$f_y = 525.00$$

$$s = 250.00$$

$V_{s,c1}$ is multiplied by $Col,c1 = 0.00$

$$s/d = 1.25$$

$V_{s,c2} = 126669.016$ is calculated for section flange core, with:

$$d = 600.00$$

$$A_v = 100530.965$$

$$f_y = 525.00$$

$$s = 250.00$$

$V_{s,c2}$ is multiplied by $Col,c2 = 1.00$

$$s/d = 0.41666667$$

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

$$\text{From (11-11), ACI 440: } V_s + V_f \leq 885757.128$$

$$bw = 450.00$$

Calculation of Shear Strength at edge 2, $V_{r2} = 1.4186E+006$

$$V_{r2} = V_{Col} ((10.3), \text{ASCE 41-17}) = knl \cdot V_{Col0}$$

$$V_{Col0} = 1.4186E+006$$

$$knl = 1 \text{ (zero step-static loading)}$$

NOTE: In expression (10-3) ' $V_s = A_v \cdot f_y \cdot d / s$ ' is replaced by ' $V_s + f \cdot V_f$ '

where V_f is the contribution of FRPs (11.3), ACI 440).

$$= 1 \text{ (normal-weight concrete)}$$

$$\text{Mean concrete strength: } f_c' = (f_c'_{jacket} \cdot \text{Area}_{jacket} + f_c'_{core} \cdot \text{Area}_{core}) / \text{Area}_{section} = 15.20, \text{ but } f_c'^{0.5} \leq 8.3$$

MPa (22.5.3.1, ACI 318-14)

$$M/Vd = 4.00$$

$$\mu_u = 2.5135E+008$$

$$V_u = 137.4894$$

$$d = 0.8 \cdot h = 760.00$$

$$N_u = 7.6839E+006$$

$$A_g = 427500.00$$

$$\text{From (11.5.4.8), ACI 318-14: } V_s = V_{sjacket} + V_{s,core} = 1.2262E+006$$

where:

$$V_{sjacket} = V_{sj1} + V_{sj2} = 1.0996E+006$$

$V_{sj1} = 353429.174$ is calculated for section web jacket, with:

$$d = 360.00$$

$$A_v = 157079.633$$

$f_y = 625.00$
 $s = 100.00$
 $V_{s,j1}$ is multiplied by $Col,j1 = 1.00$
 $s/d = 0.27777778$
 $V_{s,j2} = 746128.255$ is calculated for section flange jacket, with:
 $d = 760.00$
 $A_v = 157079.633$
 $f_y = 625.00$
 $s = 100.00$
 $V_{s,j2}$ is multiplied by $Col,j2 = 1.00$
 $s/d = 0.13157895$
 $V_{s,core} = V_{s,c1} + V_{s,c2} = 126669.016$
 $V_{s,c1} = 0.00$ is calculated for section web core, with:
 $d = 200.00$
 $A_v = 100530.965$
 $f_y = 525.00$
 $s = 250.00$
 $V_{s,c1}$ is multiplied by $Col,c1 = 0.00$
 $s/d = 1.25$
 $V_{s,c2} = 126669.016$ is calculated for section flange core, with:
 $d = 600.00$
 $A_v = 100530.965$
 $f_y = 525.00$
 $s = 250.00$
 $V_{s,c2}$ is multiplied by $Col,c2 = 1.00$
 $s/d = 0.41666667$
 $V_f ((11-3)-(11.4), ACI 440) = 0.00$
 From (11-11), ACI 440: $V_s + V_f \leq 885757.128$
 $bw = 450.00$

 End Of Calculation of Shear Capacity ratio for element: column JTC1 of floor 1
 At local axis: 2

Start Of Calculation of Chord Rotation Capacity for element: column JTC1 of floor 1
 At local axis: 2
 Integration Section: (a)
 Section Type: rcjtcs

Constant Properties

 Knowledge Factor, $\phi = 0.80$
 Chord Rotation is generally considered as Deformation-Controlled Action according to Table C7-1, ASCE 41-17.
 Mean strengths are used for Deformation-Controlled Actions according to 7.5.1.3, ASCE 41-17
 Consequently:
 Jacket
 New material of Primary Member: Concrete Strength, $f_c = f_{cm} = 18.00$
 New material of Primary Member: Steel Strength, $f_s = f_{sm} = 625.00$
 Concrete Elasticity, $E_c = 19940.411$
 Steel Elasticity, $E_s = 200000.00$
 Existing Column
 Existing material of Primary Member: Concrete Strength, $f_c = f_{cm} = 12.00$
 Existing material of Primary Member: Steel Strength, $f_s = f_{sm} = 525.00$
 Concrete Elasticity, $E_c = 16281.278$
 Steel Elasticity, $E_s = 200000.00$
 Max Height, $H_{max} = 750.00$
 Min Height, $H_{min} = 450.00$
 Max Width, $W_{max} = 950.00$
 Min Width, $W_{min} = 450.00$
 Eccentricity, $Ecc = 250.00$
 Jacket Thickness, $t_j = 100.00$
 Cover Thickness, $c = 25.00$
 Element Length, $L = 3000.00$
 Primary Member

Ribbed Bars
Ductile Steel
With Detailing for Earthquake Resistance (including stirrups closed at 135°)
Longitudinal Bars With Ends Lapped Starting at the End Sections
Adequate Lap Length ($l_b/l_d \geq 1$)
No FRP Wrapping

Stepwise Properties

Bending Moment, $M = -1.4174E+009$
Shear Force, $V2 = -212214.235$
Shear Force, $V3 = -90512.847$
Axial Force, $F = -7.0682E+006$
Longitudinal Reinforcement Area Distribution (in 2 divisions)
-Tension: $As_t = 2522.699$
-Compression: $As_c = 4168.893$
Longitudinal Reinforcement Area Distribution (in 3 divisions)
-Tension: $As_{t,ten} = 1539.38$
-Compression: $As_{c,com} = 2475.575$
-Middle: $As_{mid} = 2676.637$
Longitudinal External Reinforcement Area Distribution (in 3 divisions)
-Tension: $As_{t,ten,jacket} = 1231.504$
-Compression: $As_{c,com,jacket} = 1859.823$
-Middle: $As_{mid,jacket} = 2060.885$
Longitudinal Internal Reinforcement Area Distribution (in 3 divisions)
-Tension: $As_{t,ten,core} = 307.8761$
-Compression: $As_{c,com,core} = 615.7522$
-Middle: $As_{mid,core} = 615.7522$
Mean Diameter of Tension Reinforcement, $Db_L = 16.57143$

Existing component: From table 7-7, ASCE 41_17: Final chord rotation Capacity $u_R = u = 0.01035216$
 $u = y + p = 0.0129402$

- Calculation of y -

$y = (M_y * L_s / 3) / E_{eff} = 0.0129402$ ((4.29), Biskinis Phd))
 $M_y = 1.9528E+009$
 $L_s = M/V$ (with $L_s > 0.1 * L$ and $L_s < 2 * L$) = 6000.00
From table 10.5, ASCE 41_17: $E_{eff} = factor * E_c * I_g = 3.0181E+014$
factor = 0.70
 $A_g = 562500.00$
Mean concrete strength: $fc' = (fc'_{jacket} * Area_{jacket} + fc'_{core} * Area_{core}) / Area_{section} = 15.20$
 $N = 7.0682E+006$
 $E_c * I_g = E_{c,jacket} * I_{g,jacket} + E_{c,core} * I_{g,core} = 4.3116E+014$

Calculation of Yielding Moment M_y

Calculation of y and M_y according to Annex 7 -

Assuming neutral axis within flange ($y < t/d$, compression zone rectangular) with:
flange width, $b = 950.00$
web width, $b_w = 450.00$
flange thickness, $t = 450.00$

$y = \text{Min}(y_{ten}, y_{com})$
 $y_{ten} = 7.6761198E-006$
with $f_y = 601.9953$
 $d = 707.00$
 $y = 0.44537179$
 $A = 0.02744412$

$B = 0.02211109$
 with $p_t = 0.00229194$
 $p_c = 0.00368581$
 $p_v = 0.00398517$
 $N = 7.0682E+006$
 $b = 950.00$
 $" = 0.06082037$
 $y_{comp} = 3.8074256E-006$
 with $f_c = 18.00$
 $E_c = 19940.411$
 $y = 0.60361504$
 $A = -0.02242057$
 $B = 0.00462988$
 with $E_s = 200000.00$
 CONFIRMATION: $y = 0.60361504 < t/d$

Calculation of ratio l_b/d

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

- Calculation of p -

From table 10-8: $p = 0.00$

with:

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

shear control ratio $V_y E / V_{Co} I E = 1.74773$

$d = d_{external} = 707.00$

$s = s_{external} = 50.00$

- $t = s_1 + s_2 + 2 \cdot t_f / b_w \cdot (f_{fe} / f_s) = 0.0035764$

jacket: $s_1 = A_{v1} \cdot L_{stir1} / (s_1 \cdot A_g) = 0.00301593$

$A_{v1} = 78.53982$, is the area of every stirrup parallel to loading (shear) direction

$L_{stir1} = 2160.00$, is the total Length of all stirrups parallel to loading (shear) direction

$s_1 = 100.00$

core: $s_2 = A_{v2} \cdot L_{stir2} / (s_2 \cdot A_g) = 0.00056047$

$A_{v2} = 50.26548$, is the area of every stirrup parallel to loading (shear) direction

$L_{stir2} = 1568.00$, is the total Length of all stirrups parallel to loading (shear) direction

$s_2 = 250.00$

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

where $f = 2 \cdot t_f / b_w$ is FRP ratio (EC8 - 3, A.4.4.3(6)) and f_{fe} / f_s normalises f to steel strength

All these variables have already been given in Shear control ratio calculation.

For the normalisation f_s of jacket is used.

$NUD = 7.0682E+006$

$A_g = 562500.00$

$f_{cE} = (f_{c,jacket} \cdot Area_{jacket} + f_{c,core} \cdot Area_{core}) / section_area = 15.20$

$f_{yIE} = (f_{y,ext_Long_Reinf} \cdot Area_{ext_Long_Reinf} + f_{y,int_Long_Reinf} \cdot Area_{int_Long_Reinf}) / Area_{Tot_Long_Rein} =$

601.9953

$f_{yIE} = (f_{y,ext_Trans_Reinf} \cdot s_1 + f_{y,int_Trans_Reinf} \cdot s_2) / (s_1 + s_2) = 609.3286$

$p_l = Area_{Tot_Long_Rein} / (b \cdot d) = 0.00996292$

$b = 950.00$

$d = 707.00$

$f_{cE} = 15.20$

End Of Calculation of Chord Rotation Capacity for element: column JTC1 of floor 1

At local axis: 2

Integration Section: (a)

Calculation No. 11

column C1, Floor 1

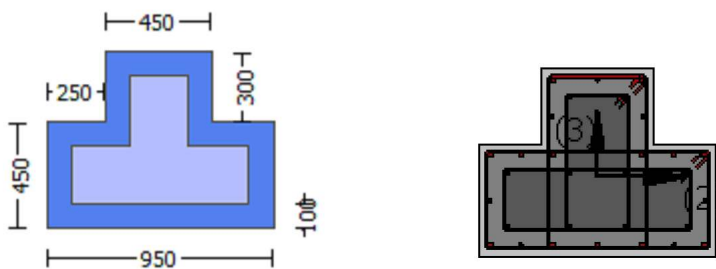
Limit State: Life Safety (data interpolation between analysis steps 50 and 51)

Analysis: Uniform +X

Check: Shear capacity VRd

Edge: Start

Local Axis: (3)



Start Of Calculation of Shear Capacity for element: column JTC1 of floor 1

At local axis: 3

Integration Section: (a)

Section Type: rcjtcs

Constant Properties

Knowledge Factor, $\gamma = 0.80$

Member Shear Force is generally considered as Force-Controlled Action according to Table C7-1, ASCE 41-17.

Lower-bound strengths are used for Force-Controlled Actions according to 7.5.1.3, ASCE 41-17

Consequently:

Jacket

New material of Primary Member: Concrete Strength, $f_c = f_{c_lower_bound} = 12.00$

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

Concrete Elasticity, $E_c = 19940.411$

Steel Elasticity, $E_s = 200000.00$

Existing Column

Existing material of Primary Member: Concrete Strength, $f_c = f_{c_lower_bound} = 8.00$

Existing material of Primary Member: Steel Strength, $f_s = f_{s_lower_bound} = 420.00$

Concrete Elasticity, $E_c = 16281.278$

Steel Elasticity, $E_s = 200000.00$

#####

Note: Especially for the calculation of γ for displacement ductility demand, the expected (mean value) strengths are used (7.5.1.3, ASCE 41-17) because bending is considered as Deformation-Controlled Action (Table C7-1, ASCE 41-17).

Jacket

New material: Concrete Strength, $f_c = f_{cm} = 18.00$

New material: Steel Strength, $f_s = f_{sm} = 625.00$

Existing Column

Existing material: Concrete Strength, $f_c = f_{cm} = 12.00$

Existing material: Steel Strength, $f_s = f_{sm} = 525.00$

#####

Max Height, Hmax = 750.00
 Min Height, Hmin = 450.00
 Max Width, Wmax = 950.00
 Min Width, Wmin = 450.00
 Eccentricity, Ecc = 250.00
 Jacket Thickness, tj = 100.00
 Cover Thickness, c = 25.00
 Element Length, L = 3000.00
 Primary Member
 Ribbed Bars
 Ductile Steel
 With Detailing for Earthquake Resistance (including stirrups closed at 135°)
 Longitudinal Bars With Ends Lapped Starting at the End Sections
 Adequate Lap Length ($l_o/l_{o,u,min} = l_b/l_d \geq 1$)
 No FRP Wrapping

Stepwise Properties

EDGE -A-
 Bending Moment, Ma = -1.4174E+009
 Shear Force, Va = -90512.847
 EDGE -B-
 Bending Moment, Mb = 8.0135E+008
 Shear Force, Vb = 90512.847
 BOTH EDGES
 Axial Force, F = -7.0682E+006
 Longitudinal Reinforcement Area Distribution (in 2 divisions)
 -Tension: Aslt = 2522.699
 -Compression: Aslc = 4168.893
 Longitudinal Reinforcement Area Distribution (in 3 divisions)
 -Tension: Asl,ten = 1539.38
 -Compression: Asl,com = 2475.575
 -Middle: Asl,mid = 2676.637
 Mean Diameter of Tension Reinforcement, DbL,ten = 16.57143

Existing component: From table 7-7, ASCE 41_17: Final Shear Capacity VR = $V_n = 713026.474$
 V_n ((10.3), ASCE 41-17) = $knl \cdot V_{ColO} = 891283.093$
 $V_{Col} = 975211.13$
 $knl = 0.9139386$
 displacement_ductility_demand = 3.14749

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

= 1 (normal-weight concrete)
 Mean concrete strength: $f'_c = (f'_{c,jacket} \cdot Area_{jacket} + f'_{c,core} \cdot Area_{core}) / Area_{section} = 10.13333$, but $f'_c^{0.5} \leq 8.3$ MPa (22.5.3.1, ACI 318-14)
 $M/Vd = 4.00$
 $M_u = 1.4174E+009$
 $V_u = 90512.847$
 $d = 0.8 \cdot h = 600.00$
 $N_u = 7.0682E+006$
 $A_g = 337500.00$
 From (11.5.4.8), ACI 318-14: $V_s = V_{s,jacket} + V_{s,core} = 828294.726$
 where:
 $V_{s,jacket} = V_{s,j1} + V_{s,j2} = 753982.237$
 $V_{s,j1} = 471238.898$ is calculated for section web jacket, with:
 $d = 600.00$
 $A_v = 157079.633$
 $f_y = 500.00$
 $s = 100.00$
 $V_{s,j1}$ is multiplied by $Col,j1 = 1.00$
 $s/d = 0.16666667$

$V_{sj2} = 282743.339$ is calculated for section flange jacket, with:

$$d = 360.00$$

$$A_v = 157079.633$$

$$f_y = 500.00$$

$$s = 100.00$$

V_{sj2} is multiplied by $Col,j2 = 1.00$

$$s/d = 0.27777778$$

$$V_{s,core} = V_{s,c1} + V_{s,c2} = 74312.489$$

$V_{s,c1} = 74312.489$ is calculated for section web core, with:

$$d = 440.00$$

$$A_v = 100530.965$$

$$f_y = 420.00$$

$$s = 250.00$$

$V_{s,c1}$ is multiplied by $Col,c1 = 1.00$

$$s/d = 0.56818182$$

$V_{s,c2} = 0.00$ is calculated for section flange core, with:

$$d = 200.00$$

$$A_v = 100530.965$$

$$f_y = 420.00$$

$$s = 250.00$$

$V_{s,c2}$ is multiplied by $Col,c2 = 0.00$

$$s/d = 1.25$$

$$V_f ((11-3)-(11.4), ACI 440) = 0.00$$

$$\text{From } (11-11), ACI 440: V_s + V_f \leq 570961.316$$

$$bw = 450.00$$

displacement ductility demand is calculated as δ / y

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

From analysis, chord rotation $\theta = 0.04072908$

$$y = (M_y * L_s / 3) / E_{eff} = 0.0129402 ((4.29), \text{Biskinis Phd})$$

$$M_y = 1.9528E+009$$

$$L_s = M/V \text{ (with } L_s > 0.1 * L \text{ and } L_s < 2 * L) = 6000.00$$

$$\text{From table 10.5, ASCE 41_17: } E_{eff} = \text{factor} * E_c * I_g = 3.0181E+014$$

$$\text{factor} = 0.70$$

$$A_g = 562500.00$$

$$\text{Mean concrete strength: } f'_c = (f'_{c,jacket} * \text{Area}_{jacket} + f'_{c,core} * \text{Area}_{core}) / \text{Area}_{section} = 15.20$$

$$N = 7.0682E+006$$

$$E_c * I_g = E_{c,jacket} * I_{g,jacket} + E_{c,core} * I_{g,core} = 4.3116E+014$$

Calculation of Yielding Moment M_y

Calculation of δ / y and M_y according to Annex 7 -

Assuming neutral axis within flange ($y < t/d$, compression zone rectangular) with:

$$\text{flange width, } b = 950.00$$

$$\text{web width, } bw = 450.00$$

$$\text{flange thickness, } t = 450.00$$

$$y = \text{Min}(y_{ten}, y_{com})$$

$$y_{ten} = 7.6761198E-006$$

$$\text{with } f_y = 601.9953$$

$$d = 707.00$$

$$y = 0.44537179$$

$$A = 0.02744412$$

$$B = 0.02211109$$

$$\text{with } pt = 0.00229194$$

$$pc = 0.00368581$$

$$pv = 0.00398517$$

$$N = 7.0682E+006$$

$$b = 950.00$$

" = 0.06082037
y_comp = 3.8074256E-006
with fc = 18.00
Ec = 19940.411
y = 0.60361504
A = -0.02242057
B = 0.00462988
with Es = 200000.00
CONFIRMATION: y = 0.60361504 < t/d

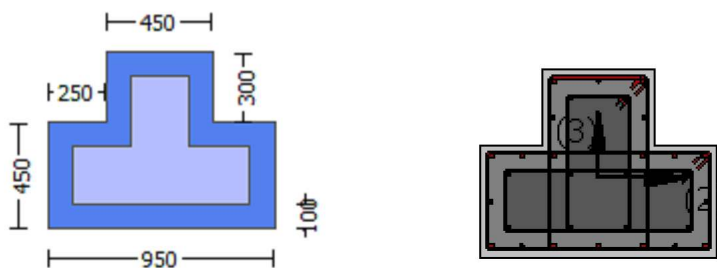
Calculation of ratio lb/ld

Adequate Lap Length: lb/ld >= 1

End Of Calculation of Shear Capacity for element: column JTC1 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 50 and 51)
Analysis: Uniform +X
Check: Chord rotation capacity (u)
Edge: Start
Local Axis: (3)



Start Of Calculation of Shear Capacity ratio for element: column JTC1 of floor 1
At Shear local axis: 3
(Bending local axis: 2)
Section Type: rcjtcs

Constant Properties

Knowledge Factor, = 0.80
Mean strength values are used for both shear and moment calculations.

Consequently:
 Jacket
 New material of Primary Member: Concrete Strength, $f_c = f_{cm} = 18.00$
 New material of Primary Member: Steel Strength, $f_s = f_{sm} = 625.00$
 Concrete Elasticity, $E_c = 19940.411$
 Steel Elasticity, $E_s = 200000.00$
 Existing Column
 Existing material of Primary Member: Concrete Strength, $f_c = f_{cm} = 12.00$
 Existing material of Primary Member: Steel Strength, $f_s = f_{sm} = 525.00$
 Concrete Elasticity, $E_c = 16281.278$
 Steel Elasticity, $E_s = 200000.00$
 #####
 Note: Especially for the calculation of moment strengths,
 the above steel re-bar strengths are multiplied by 1.25 according to R18.6.5, ACI 318-14
 Jacket
 New material: Steel Strength, $f_s = 1.25 \cdot f_{sm} = 781.25$
 Existing Column
 Existing material: Steel Strength, $f_s = 1.25 \cdot f_{sm} = 656.25$
 #####
 Max Height, $H_{max} = 750.00$
 Min Height, $H_{min} = 450.00$
 Max Width, $W_{max} = 950.00$
 Min Width, $W_{min} = 450.00$
 Eccentricity, $Ecc = 250.00$
 Jacket Thickness, $t_j = 100.00$
 Cover Thickness, $c = 25.00$
 Mean Confinement Factor overall section = 1.42131
 Element Length, $L = 3000.00$
 Primary Member
 Ribbed Bars
 Ductile Steel
 With Detailing for Earthquake Resistance (including stirrups closed at 135°)
 Longitudinal Bars With Ends Lapped Starting at the End Sections
 Adequate Lap Length ($l_o/l_{ou,min} \geq 1$)
 No FRP Wrapping

Stepwise Properties

At local axis: 3
 EDGE -A-
 Shear Force, $V_a = -227.4778$
 EDGE -B-
 Shear Force, $V_b = 227.4778$
 BOTH EDGES
 Axial Force, $F = -7.6839E+006$
 Longitudinal Reinforcement Area Distribution (in 2 divisions)
 -Tension: $As_t = 314.1593$
 -Compression: $As_c = 6176.371$
 Longitudinal Reinforcement Area Distribution (in 3 divisions)
 -Tension: $As_{t,ten} = 1539.38$
 -Compression: $As_{l,com} = 2475.575$
 -Middle: $As_{l,mid} = 2676.637$

Calculation of Shear Capacity ratio, $V_e/V_r = 1.74773$
 Member Controlled by Shear ($V_e/V_r > 1$)
 Calculation of Shear Demand from fig. R18.6.5, ACI 318-14 $V_e = (M_{pr1} + M_{pr2})/l_n = 2.0410E+006$
 with
 $M_{pr1} = \max(\mu_{u1+}, \mu_{u1-}) = 3.0615E+009$
 $\mu_{u1+} = 3.0615E+009$, is the ultimate moment strength at the edge 1 of the member in the actual moment direction
 which is defined for the static loading combination
 $\mu_{u1-} = 8.9683E+008$, is the ultimate moment strength at the edge 1 of the member in the opposite moment
 direction which is defined for the static loading combination
 $M_{pr2} = \max(\mu_{u2+}, \mu_{u2-}) = 3.0615E+009$

Mu2+ = 3.0615E+009, is the ultimate moment strength at the edge 2 of the member in the actual moment direction which is defined for the static loading combination

Mu2- = 8.9683E+008, is the ultimate moment strength at the edge 2 of the member in the opposite moment direction which is defined for the static loading combination

Calculation of Mu1+

Calculation of ultimate curvature ϕ_u according to 4.1, Biskinis/Fardis 2013:

$$\phi_u = 1.3930706E-005$$

$$M_u = 3.0615E+009$$

with full section properties:

$$b = 950.00$$

$$d = 707.00$$

$$d' = 43.00$$

$$\nu = 0.63557247$$

$$N = 7.6839E+006$$

$$f_c = 18.00$$

$$\phi_{co} (5A.5, TBDY) = 0.002$$

$$\text{Final value of } \phi_u: \phi_u^* = \text{shear_factor} * \text{Max}(\phi_u, \phi_{co}) = 0.01486841$$

The Shear_factor is considered equal to 1 (pure moment strength)

$$\text{From (5.4b), TBDY: } \phi_u = 0.01486841$$

$$\phi_{we} (5.4c) = 0.08077545$$

$$\phi_{ase} ((5.4d), TBDY) = (\phi_{ase1} * A_{ext} + \phi_{ase2} * A_{int}) / A_{sec} = 0.53375773$$

$$\phi_{ase1} = \text{Max}(((A_{conf,max1} - A_{noConf1}) / A_{conf,max1}) * (\phi_{conf,min1} / A_{conf,max1}), 0) = 0.53375773$$

The definitions of A_{noConf} , $A_{conf,min}$ and $A_{conf,max}$ are derived from generalization of the rectangular sections confinement, which is expressed by (5.4d).

The generalization is done according to Mander, J., Priestley, M., and Park, R. (1988)

"Theoretical Stress-Strain Model for Confined Concrete."

J. Struct. Eng., 10.1061/(ASCE)0733-9445(1988)114:8(1804), 1804-1826.

$A_{conf,max1} = 464100.00$ is the confined core area at levels of member with hoops and

is calculated as the area of core enclosed by the center lines of the external perimeter hoops.

$A_{conf,min1} = 395025.00$ is the confined core area at midway between the levels of hoops and

is calculated by reducing all the dimensions of the area $A_{conf,max1}$ by a length

equal to half the clear spacing between external hoops.

$A_{noConf1} = 173066.667$ is the unconfined external core area which is equal to $b_i^2/6$ as defined at (A.2).

$$\phi_{ase2} (>= \phi_{ase1}) = \text{Max}(((A_{conf,max2} - A_{noConf2}) / A_{conf,max2}) * (\phi_{conf,min2} / A_{conf,max2}), 0) = 0.53375773$$

The definitions of A_{noConf} , $A_{conf,min}$ and $A_{conf,max}$ are derived from generalization of the rectangular sections confinement, which is expressed by (5.4d).

The generalization is done according to Mander, J., Priestley, M., and Park, R. (1988)

"Theoretical Stress-Strain Model for Confined Concrete."

J. Struct. Eng., 10.1061/(ASCE)0733-9445(1988)114:8(1804), 1804-1826.

$A_{conf,max2} = 252164.00$ is the confined core area at levels of member with hoops and

is calculated as the area of core enclosed by the center lines of the internal perimeter hoops.

$A_{conf,min2} = 111441.00$ is the confined core area at midway between the levels of hoops and

is calculated by reducing all the dimensions of the area $A_{conf,max2}$ by a length

equal to half the clear spacing between internal hoops.

$A_{noConf2} = 110709.333$ is the unconfined internal core area which is equal to $b_i^2/6$ as defined at (A.2).

$$\phi_{psh,min} * F_{ywe} = \text{Min}(\phi_{psh,x} * F_{ywe}, \phi_{psh,y} * F_{ywe}) = 2.724$$

$$\phi_{psh,x} * F_{ywe} = \phi_{psh1} * F_{ywe1} + \phi_{psh2} * F_{ywe2} = 2.724$$

$$\phi_{psh1} ((5.4d), TBDY) = L_{stir1} * A_{stir1} / (A_{sec} * s_1) = 0.00301593$$

$$L_{stir1} (\text{Length of stirrups along Y}) = 2160.00$$

$$A_{stir1} (\text{stirrups area}) = 78.53982$$

$$\phi_{psh2} (5.4d) = L_{stir2} * A_{stir2} / (A_{sec} * s_2) = 0.00056047$$

$$L_{stir2} (\text{Length of stirrups along Y}) = 1568.00$$

$$A_{stir2} (\text{stirrups area}) = 50.26548$$

$$\phi_{psh,y} * F_{ywe} = \phi_{psh1} * F_{ywe1} + \phi_{psh2} * F_{ywe2} = 3.25416$$

$$\phi_{psh1} ((5.4d), TBDY) = L_{stir1} * A_{stir1} / (A_{sec} * s_1) = 0.00357443$$

$$L_{stir1} (\text{Length of stirrups along X}) = 2560.00$$

$$A_{stir1} (\text{stirrups area}) = 78.53982$$

$$\phi_{psh2} ((5.4d), TBDY) = L_{stir2} * A_{stir2} / (A_{sec} * s_2) = 0.00070345$$

$$L_{stir2} (\text{Length of stirrups along X}) = 1968.00$$

Astir2 (stirrups area) = 50.26548

Asec = 562500.00

s1 = 100.00

s2 = 250.00

fywe1 = 781.25

fywe2 = 656.25

fce = 18.00

From ((5.A5), TBDY), TBDY: cc = 0.00621307

c = confinement factor = 1.42131

y1 = 0.0025

sh1 = 0.008

ft1 = 907.50

fy1 = 756.25

su1 = 0.032

using (30) in Biskinis/Fardis (2013) multiplied with shear_factor
and also multiplied by the shear_factor according to 15.7.1.4, with
Shear_factor = 1.00

lo/lou,min = lb/lb,min = 1.00

su1 = 0.4*esu1_nominal ((5.5), TBDY) = 0.032

From table 5A.1, TBDY: esu1_nominal = 0.08,

For calculation of esu1_nominal and y1, sh1,ft1,fy1, it is considered
characteristic value fsy1 = fs1/1.2, from table 5.1, TBDY.

y1, sh1,ft1,fy1, are also multiplied by $\text{Min}(1, 1.25 \cdot (lb/lb)^{2/3})$, from 10.3.5, ASCE 41-17.

with fs1 = (fs,jacket*Asl,ten,jacket + fs,core*Asl,ten,core)/Asl,ten = 756.25

with Es1 = (Es,jacket*Asl,ten,jacket + Es,core*Asl,ten,core)/Asl,ten = 200000.00

y2 = 0.0025

sh2 = 0.008

ft2 = 900.1904

fy2 = 750.1586

su2 = 0.032

using (30) in Biskinis/Fardis (2013) multiplied with shear_factor
and also multiplied by the shear_factor according to 15.7.1.4, with
Shear_factor = 1.00

lo/lou,min = lb/lb,min = 1.00

su2 = 0.4*esu2_nominal ((5.5), TBDY) = 0.032

From table 5A.1, TBDY: esu2_nominal = 0.08,

For calculation of esu2_nominal and y2, sh2,ft2,fy2, it is considered
characteristic value fsy2 = fs2/1.2, from table 5.1, TBDY.

y1, sh1,ft1,fy1, are also multiplied by $\text{Min}(1, 1.25 \cdot (lb/lb)^{2/3})$, from 10.3.5, ASCE 41-17.

with fs2 = (fs,jacket*Asl,com,jacket + fs,core*Asl,com,core)/Asl,com = 750.1586

with Es2 = (Es,jacket*Asl,com,jacket + Es,core*Asl,com,core)/Asl,com = 200000.00

yv = 0.0025

shv = 0.008

ftv = 902.993

fyv = 752.4941

suv = 0.032

using (30) in Biskinis/Fardis (2013) multiplied with shear_factor
and also multiplied by the shear_factor according to 15.7.1.4, with
Shear_factor = 1.00

lo/lou,min = lb/lb,min = 1.00

suv = 0.4*esuv_nominal ((5.5), TBDY) = 0.032

From table 5A.1, TBDY: esuv_nominal = 0.08,

considering characteristic value fsyv = fsv/1.2, from table 5.1, TBDY

For calculation of esuv_nominal and yv, shv,ftv,fyv, it is considered
characteristic value fsyv = fsv/1.2, from table 5.1, TBDY.

y1, sh1,ft1,fy1, are also multiplied by $\text{Min}(1, 1.25 \cdot (lb/lb)^{2/3})$, from 10.3.5, ASCE 41-17.

with fsv = (fs,jacket*Asl,mid,jacket + fs,mid*Asl,mid,core)/Asl,mid = 752.4941

with Esv = (Es,jacket*Asl,mid,jacket + Es,mid*Asl,mid,core)/Asl,mid = 200000.00

1 = Asl,ten/(b*d)*(fs1/fc) = 0.09629324

2 = Asl,com/(b*d)*(fs2/fc) = 0.15360794

v = Asl,mid/(b*d)*(fsv/fc) = 0.16660079

and confined core properties:

b = 890.00

d = 677.00

d' = 13.00

fcc (5A.2, TBDY) = 25.58352

```

cc (5A.5, TBDY) = 0.00621307
c = confinement factor = 1.42131
1 = Asl,ten/(b*d)*(fs1/fc) = 0.10733965
2 = Asl,com/(b*d)*(fs2/fc) = 0.17122928
v = Asl,mid/(b*d)*(fsv/fc) = 0.18571261
Case/Assumption: Unconfined full section - Steel rupture
'satisfies Eq. (4.3)
---->
v < vs,y2 - LHS eq.(4.5) is not satisfied
---->
v < vs,c - RHS eq.(4.5) is not satisfied
---->
Case/Assumption Rejected.
---->
New Case/Assumption: Unconfined full section - Spalling of concrete cover
'satisfies Eq. (4.4)
---->
v < s,y1 - LHS eq.(4.7) is not satisfied
---->
v < vc,y1 - RHS eq.(4.6) is satisfied
---->
cu (4.10) = 0.58509201
MRc (4.17) = 2.3801E+009
---->
New Case/Assumption: Ultimate curvature of confined core after spalling of concrete cover
In expressions below, the following modifications have been made
- b, d, d' replaced by geometric parameters of the core: bo, do, d'o
- N, 1, 2, v normalised to bo*do, instead of b*d
- - parameters of confined concrete, fcc, cc, used in lieu of fc, ecu
---->
Subcase: Rupture of tension steel
---->
v* < v*s,y2 - LHS eq.(4.5) is not satisfied
---->
v* < v*s,c - LHS eq.(4.5) is not satisfied
---->
Subcase rejected
---->
New Subcase: Failure of compression zone
---->
v* < v*c,y2 - LHS eq.(4.6) is not satisfied
---->
v* < v*c,y1 - RHS eq.(4.6) is not satisfied
---->
*cu (4.11) = 0.65878578
MRo (4.18) = 3.0615E+009
---->
u = cu (4.2) = 1.3930706E-005
Mu = MRo
-----

Calculation of ratio lb/ld
-----
Adequate Lap Length: lb/ld >= 1
-----

Calculation of Mu1-
-----

-----

Calculation of ultimate curvature u according to 4.1, Biskinis/Fardis 2013:
u = 9.1773508E-006
Mu = 8.9683E+008
-----

with full section properties:
b = 450.00

```

$d = 707.00$
 $d' = 43.00$
 $v = 1.34176$
 $N = 7.6839E+006$
 $f_c = 18.00$
 $co (5A.5, TBDY) = 0.002$
 Final value of cu : $cu^* = shear_factor * Max(cu, cc) = 0.01486841$
 The Shear_factor is considered equal to 1 (pure moment strength)
 From (5.4b), TBDY: $cu = 0.01486841$
 $we (5.4c) = 0.08077545$
 $ase ((5.4d), TBDY) = (ase1 * A_{ext} + ase2 * A_{int}) / A_{sec} = 0.53375773$
 $ase1 = Max(((A_{conf,max1} - A_{noConf1}) / A_{conf,max1}) * (A_{conf,min1} / A_{conf,max1}), 0) = 0.53375773$
 The definitions of A_{noConf} , $A_{conf,min}$ and $A_{conf,max}$ are derived from generalization of the rectangular sections confinement, which is expressed by (5.4d).
 The generalization is done according to Mander, J., Priestley, M., and Park, R. (1988) "Theoretical Stress-Strain Model for Confined Concrete."
 J. Struct. Eng., 10.1061/(ASCE)0733-9445(1988)114:8(1804), 1804-1826.
 $A_{conf,max1} = 464100.00$ is the confined core area at levels of member with hoops and is calculated as the area of core enclosed by the center lines of the external perimeter hoops.
 $A_{conf,min1} = 395025.00$ is the confined core area at midway between the levels of hoops and is calculated by reducing all the dimensions of the area $A_{conf,max1}$ by a length equal to half the clear spacing between external hoops.
 $A_{noConf1} = 173066.667$ is the unconfined external core area which is equal to $b_i^2/6$ as defined at (A.2).
 $ase2 (>= ase1) = Max(((A_{conf,max2} - A_{noConf2}) / A_{conf,max2}) * (A_{conf,min2} / A_{conf,max2}), 0) = 0.53375773$
 The definitions of A_{noConf} , $A_{conf,min}$ and $A_{conf,max}$ are derived from generalization of the rectangular sections confinement, which is expressed by (5.4d).
 The generalization is done according to Mander, J., Priestley, M., and Park, R. (1988) "Theoretical Stress-Strain Model for Confined Concrete."
 J. Struct. Eng., 10.1061/(ASCE)0733-9445(1988)114:8(1804), 1804-1826.
 $A_{conf,max2} = 252164.00$ is the confined core area at levels of member with hoops and is calculated as the area of core enclosed by the center lines of the internal perimeter hoops.
 $A_{conf,min2} = 111441.00$ is the confined core area at midway between the levels of hoops and is calculated by reducing all the dimensions of the area $A_{conf,max2}$ by a length equal to half the clear spacing between internal hoops.
 $A_{noConf2} = 110709.333$ is the unconfined internal core area which is equal to $b_i^2/6$ as defined at (A.2).
 $psh,min * F_{ywe} = Min(psh,x * F_{ywe}, psh,y * F_{ywe}) = 2.724$

$psh,x * F_{ywe} = psh1 * F_{ywe1} + ps2 * F_{ywe2} = 2.724$
 $psh1 ((5.4d), TBDY) = L_{stir1} * A_{stir1} / (A_{sec} * s1) = 0.00301593$
 L_{stir1} (Length of stirrups along Y) = 2160.00
 A_{stir1} (stirrups area) = 78.53982
 $psh2 (5.4d) = L_{stir2} * A_{stir2} / (A_{sec} * s2) = 0.00056047$
 L_{stir2} (Length of stirrups along Y) = 1568.00
 A_{stir2} (stirrups area) = 50.26548

$psh,y * F_{ywe} = psh1 * F_{ywe1} + ps2 * F_{ywe2} = 3.25416$
 $psh1 ((5.4d), TBDY) = L_{stir1} * A_{stir1} / (A_{sec} * s1) = 0.00357443$
 L_{stir1} (Length of stirrups along X) = 2560.00
 A_{stir1} (stirrups area) = 78.53982
 $psh2 ((5.4d), TBDY) = L_{stir2} * A_{stir2} / (A_{sec} * s2) = 0.00070345$
 L_{stir2} (Length of stirrups along X) = 1968.00
 A_{stir2} (stirrups area) = 50.26548

$A_{sec} = 562500.00$
 $s1 = 100.00$
 $s2 = 250.00$
 $f_{ywe1} = 781.25$
 $f_{ywe2} = 656.25$
 $f_{ce} = 18.00$
 From ((5.A5), TBDY), TBDY: $cc = 0.00621307$
 $c =$ confinement factor = 1.42131
 $y1 = 0.0025$
 $sh1 = 0.008$
 $ft1 = 900.1904$
 $fy1 = 750.1586$
 $su1 = 0.032$
 using (30) in Biskinis/Fardis (2013) multiplied with shear_factor

and also multiplied by the shear_factor according to 15.7.1.4, with
Shear_factor = 1.00
 $l_o/l_{ou,min} = l_b/l_d = 1.00$
 $su_1 = 0.4 * esu_{1,nominal} ((5.5), TBDY) = 0.032$
From table 5A.1, TBDY: $esu_{1,nominal} = 0.08$,
For calculation of $esu_{1,nominal}$ and y_1, sh_1, ft_1, fy_1 , it is considered
characteristic value $fsy_1 = fs_1/1.2$, from table 5.1, TBDY.
 y_1, sh_1, ft_1, fy_1 , are also multiplied by $Min(1, 1.25 * (l_b/l_d)^{2/3})$, from 10.3.5, ASCE 41-17.
with $fs_1 = (fs_{jacket} * Asl_{ten,jacket} + fs_{core} * Asl_{ten,core}) / Asl_{ten} = 750.1586$
with $Es_1 = (Es_{jacket} * Asl_{ten,jacket} + Es_{core} * Asl_{ten,core}) / Asl_{ten} = 200000.00$
 $y_2 = 0.0025$
 $sh_2 = 0.008$
 $ft_2 = 907.50$
 $fy_2 = 756.25$
 $su_2 = 0.032$
using (30) in Biskinis/Fardis (2013) multiplied with shear_factor
and also multiplied by the shear_factor according to 15.7.1.4, with
Shear_factor = 1.00
 $l_o/l_{ou,min} = l_b/l_{b,min} = 1.00$
 $su_2 = 0.4 * esu_{2,nominal} ((5.5), TBDY) = 0.032$
From table 5A.1, TBDY: $esu_{2,nominal} = 0.08$,
For calculation of $esu_{2,nominal}$ and y_2, sh_2, ft_2, fy_2 , it is considered
characteristic value $fsy_2 = fs_2/1.2$, from table 5.1, TBDY.
 y_2, sh_2, ft_2, fy_2 , are also multiplied by $Min(1, 1.25 * (l_b/l_d)^{2/3})$, from 10.3.5, ASCE 41-17.
with $fs_2 = (fs_{jacket} * Asl_{com,jacket} + fs_{core} * Asl_{com,core}) / Asl_{com} = 756.25$
with $Es_2 = (Es_{jacket} * Asl_{com,jacket} + Es_{core} * Asl_{com,core}) / Asl_{com} = 200000.00$
 $y_v = 0.0025$
 $sh_v = 0.008$
 $ft_v = 902.993$
 $fy_v = 752.4941$
 $suv = 0.032$
using (30) in Biskinis/Fardis (2013) multiplied with shear_factor
and also multiplied by the shear_factor according to 15.7.1.4, with
Shear_factor = 1.00
 $l_o/l_{ou,min} = l_b/l_d = 1.00$
 $suv = 0.4 * esuv_{nominal} ((5.5), TBDY) = 0.032$
From table 5A.1, TBDY: $esuv_{nominal} = 0.08$,
considering characteristic value $fsyv = fsv/1.2$, from table 5.1, TBDY
For calculation of $esuv_{nominal}$ and y_v, sh_v, ft_v, fy_v , it is considered
characteristic value $fsyv = fsv/1.2$, from table 5.1, TBDY.
 y_1, sh_1, ft_1, fy_1 , are also multiplied by $Min(1, 1.25 * (l_b/l_d)^{2/3})$, from 10.3.5, ASCE 41-17.
with $fsv = (fs_{jacket} * Asl_{mid,jacket} + fs_{mid} * Asl_{mid,core}) / Asl_{mid} = 752.4941$
with $Es_v = (Es_{jacket} * Asl_{mid,jacket} + Es_{mid} * Asl_{mid,core}) / Asl_{mid} = 200000.00$
 $1 = Asl_{ten} / (b * d) * (fs_1 / fc) = 0.32428344$
 $2 = Asl_{com} / (b * d) * (fs_2 / fc) = 0.20328574$
 $v = Asl_{mid} / (b * d) * (fsv / fc) = 0.35171278$
and confined core properties:
 $b = 390.00$
 $d = 677.00$
 $d' = 13.00$
 $f_{cc} (5A.2, TBDY) = 25.58352$
 $cc (5A.5, TBDY) = 0.00621307$
 $c = \text{confinement factor} = 1.42131$
 $1 = Asl_{ten} / (b * d) * (fs_1 / fc) = 0.39075399$
 $2 = Asl_{com} / (b * d) * (fs_2 / fc) = 0.24495458$
 $v = Asl_{mid} / (b * d) * (fsv / fc) = 0.42380571$
Case/Assumption: Unconfined full section - Steel rupture
' satisfies Eq. (4.3)
--->
 $v < v_{s,y2}$ - LHS eq.(4.5) is not satisfied
--->
 $v < v_{s,c}$ - RHS eq.(4.5) is not satisfied
--->
Case/Assumption Rejected.
--->
New Case/Assumption: Unconfined full section - Spalling of concrete cover
' satisfies Eq. (4.4)

```

---->
v < s,y1 - LHS eq.(4.7) is not satisfied
---->
v < vc,y1 - RHS eq.(4.6) is not satisfied
---->
cu (4.11) = 0.95920575
MRc (4.18) = 7.7321E+008
---->
New Case/Assumption: Ultimate curvature of confined core after spalling of concrete cover
In expressions below, the following modifications have been made
- b, d, d' replaced by geometric parameters of the core: bo, do, d'o
- N, 1, 2, v normalised to bo*do, instead of b*d
- - parameters of confined concrete, fcc, cc, used in lieu of fc, ecu

```

```

---->
Subcase: Rupture of tension steel
---->
v* < v*s,y2 - LHS eq.(4.5) is not satisfied
---->
v* < v*s,c - LHS eq.(4.5) is not satisfied
---->
Subcase rejected
---->
New Subcase: Failure of compression zone
---->
v* < v*c,y2 - LHS eq.(4.6) is not satisfied
---->
v* < v*c,y1 - RHS eq.(4.6) is not satisfied
---->
*cu (4.11) = 1.00
MRo (4.18) = 8.9683E+008
---->
u = cu (4.2) = 9.1773508E-006
Mu = MRo

```

Calculation of ratio lb/d

Adequate Lap Length: lb/d >= 1

Calculation of Mu2+

Calculation of ultimate curvature u according to 4.1, Biskinis/Fardis 2013:

```

u = 1.3930706E-005
Mu = 3.0615E+009

```

with full section properties:

```

b = 950.00
d = 707.00
d' = 43.00
v = 0.63557247
N = 7.6839E+006
fc = 18.00
co (5A.5, TBDY) = 0.002
Final value of cu: cu* = shear_factor * Max( cu, cc) = 0.01486841
The Shear_factor is considered equal to 1 (pure moment strength)
From (5.4b), TBDY: cu = 0.01486841
we (5.4c) = 0.08077545
ase ((5.4d), TBDY) = (ase1*Aext+ase2*Aint)/Asec = 0.53375773
ase1 = Max(((Aconf,max1-AnoConf1)/Aconf,max1)*(Aconf,min1/Aconf,max1),0) = 0.53375773
The definitions of AnoConf, Aconf,min and Aconf,max are derived from generalization
of the rectangular sections confinement, which is expressed by (5.4d).
The generalization is done according to Mander, J., Priestley, M., and Park, R. (1988)
"Theoretical Stress-Strain Model for Confined Concrete."

```

J. Struct. Eng., 10.1061/(ASCE)0733-9445(1988)114:8(1804), 1804-1826.

Aconf,max1 = 464100.00 is the confined core area at levels of member with hoops and is calculated as the area of core enclosed by the center lines of the external perimeter hoops.
Aconf,min1 = 395025.00 is the confined core area at midway between the levels of hoops and is calculated by reducing all the dimensions of the area Aconf,max1 by a length equal to half the clear spacing between external hoops.

AnoConf1 = 173066.667 is the unconfined external core area which is equal to $b_i^2/6$ as defined at (A.2).
ase2 (\geq ase1) = $\text{Max}(((A_{\text{conf,max2}} - A_{\text{noConf2}})/A_{\text{conf,max2}}) * (A_{\text{conf,min2}}/A_{\text{conf,max2}}), 0) = 0.53375773$

The definitions of AnoConf, Aconf,min and Aconf,max are derived from generalization of the rectangular sections confinement, which is expressed by (5.4d).

The generalization is done according to Mander, J., Priestley, M., and Park, R. (1988)

"Theoretical Stress-Strain Model for Confined Concrete."

J. Struct. Eng., 10.1061/(ASCE)0733-9445(1988)114:8(1804), 1804-1826.

Aconf,max2 = 252164.00 is the confined core area at levels of member with hoops and is calculated as the area of core enclosed by the center lines of the internal perimeter hoops.

Aconf,min2 = 111441.00 is the confined core area at midway between the levels of hoops and is calculated by reducing all the dimensions of the area Aconf,max2 by a length equal to half the clear spacing between internal hoops.

AnoConf2 = 110709.333 is the unconfined internal core area which is equal to $b_i^2/6$ as defined at (A.2).
psh,min*Fywe = $\text{Min}(psh,x*Fywe, psh,y*Fywe) = 2.724$

psh_x*Fywe = psh1*Fywe1 + ps2*Fywe2 = 2.724
psh1 ((5.4d), TBDY) = $L_{\text{stir1}} * A_{\text{stir1}} / (A_{\text{sec}} * s_1) = 0.00301593$

Lstir1 (Length of stirrups along Y) = 2160.00

Astir1 (stirrups area) = 78.53982

psh2 (5.4d) = $L_{\text{stir2}} * A_{\text{stir2}} / (A_{\text{sec}} * s_2) = 0.00056047$

Lstir2 (Length of stirrups along Y) = 1568.00

Astir2 (stirrups area) = 50.26548

psh_y*Fywe = psh1*Fywe1 + ps2*Fywe2 = 3.25416
psh1 ((5.4d), TBDY) = $L_{\text{stir1}} * A_{\text{stir1}} / (A_{\text{sec}} * s_1) = 0.00357443$

Lstir1 (Length of stirrups along X) = 2560.00

Astir1 (stirrups area) = 78.53982

psh2 ((5.4d), TBDY) = $L_{\text{stir2}} * A_{\text{stir2}} / (A_{\text{sec}} * s_2) = 0.00070345$

Lstir2 (Length of stirrups along X) = 1968.00

Astir2 (stirrups area) = 50.26548

Asec = 562500.00

s1 = 100.00

s2 = 250.00

fywe1 = 781.25

fywe2 = 656.25

fce = 18.00

From ((5.A5), TBDY), TBDY: cc = 0.00621307

c = confinement factor = 1.42131

y1 = 0.0025

sh1 = 0.008

ft1 = 907.50

fy1 = 756.25

su1 = 0.032

using (30) in Biskinis/Fardis (2013) multiplied with shear_factor and also multiplied by the shear_factor according to 15.7.1.4, with Shear_factor = 1.00

lo/lo,min = lb/ld = 1.00

su1 = $0.4 * e_{s1_nominal} ((5.5), TBDY) = 0.032$

From table 5A.1, TBDY: esu1_nominal = 0.08,

For calculation of esu1_nominal and y1, sh1, ft1, fy1, it is considered characteristic value fsy1 = fs1/1.2, from table 5.1, TBDY.

y1, sh1, ft1, fy1, are also multiplied by $\text{Min}(1, 1.25 * (lb/ld)^{2/3})$, from 10.3.5, ASCE 41-17.

with fs1 = $(f_{s,jacket} * A_{s,ten,jacket} + f_{s,core} * A_{s,ten,core}) / A_{s,ten} = 756.25$

with Es1 = $(E_{s,jacket} * A_{s,ten,jacket} + E_{s,core} * A_{s,ten,core}) / A_{s,ten} = 200000.00$

y2 = 0.0025

sh2 = 0.008

ft2 = 900.1904

fy2 = 750.1586

su2 = 0.032

using (30) in Biskinis/Fardis (2013) multiplied with shear_factor

and also multiplied by the shear_factor according to 15.7.1.4, with
Shear_factor = 1.00
 $l_o/l_{o,min} = l_b/l_{b,min} = 1.00$
 $su_2 = 0.4 * esu_{2,nominal} ((5.5), TBDY) = 0.032$
From table 5A.1, TBDY: $esu_{2,nominal} = 0.08$,
For calculation of $esu_{2,nominal}$ and y_2, sh_2, ft_2, fy_2 , it is considered
characteristic value $fsy_2 = fs_2/1.2$, from table 5.1, TBDY.
 y_1, sh_1, ft_1, fy_1 , are also multiplied by $Min(1, 1.25 * (l_b/l_d)^{2/3})$, from 10.3.5, ASCE 41-17.
with $fs_2 = (fs_{jacket} * A_{sl,com,jacket} + fs_{core} * A_{sl,com,core}) / A_{sl,com} = 750.1586$
with $Es_2 = (Es_{jacket} * A_{sl,com,jacket} + Es_{core} * A_{sl,com,core}) / A_{sl,com} = 200000.00$
 $y_v = 0.0025$
 $sh_v = 0.008$
 $ft_v = 902.993$
 $fy_v = 752.4941$
 $suv = 0.032$
using (30) in Biskinis/Fardis (2013) multiplied with shear_factor
and also multiplied by the shear_factor according to 15.7.1.4, with
Shear_factor = 1.00
 $l_o/l_{o,min} = l_b/l_d = 1.00$
 $suv = 0.4 * esuv_{nominal} ((5.5), TBDY) = 0.032$
From table 5A.1, TBDY: $esuv_{nominal} = 0.08$,
considering characteristic value $fsyv = fsv/1.2$, from table 5.1, TBDY
For calculation of $esuv_{nominal}$ and y_v, sh_v, ft_v, fy_v , it is considered
characteristic value $fsyv = fsv/1.2$, from table 5.1, TBDY.
 y_1, sh_1, ft_1, fy_1 , are also multiplied by $Min(1, 1.25 * (l_b/l_d)^{2/3})$, from 10.3.5, ASCE 41-17.
with $fsv = (fs_{jacket} * A_{sl,mid,jacket} + fs_{mid} * A_{sl,mid,core}) / A_{sl,mid} = 752.4941$
with $Es_v = (Es_{jacket} * A_{sl,mid,jacket} + Es_{mid} * A_{sl,mid,core}) / A_{sl,mid} = 200000.00$
 $1 = A_{sl,ten} / (b * d) * (fs_1 / f_c) = 0.09629324$
 $2 = A_{sl,com} / (b * d) * (fs_2 / f_c) = 0.15360794$
 $v = A_{sl,mid} / (b * d) * (fsv / f_c) = 0.16660079$
and confined core properties:
 $b = 890.00$
 $d = 677.00$
 $d' = 13.00$
 $f_{cc} (5A.2, TBDY) = 25.58352$
 $cc (5A.5, TBDY) = 0.00621307$
 $c = \text{confinement factor} = 1.42131$
 $1 = A_{sl,ten} / (b * d) * (fs_1 / f_c) = 0.10733965$
 $2 = A_{sl,com} / (b * d) * (fs_2 / f_c) = 0.17122928$
 $v = A_{sl,mid} / (b * d) * (fsv / f_c) = 0.18571261$
Case/Assumption: Unconfined full section - Steel rupture
' satisfies Eq. (4.3)
---->
 $v < v_{s,y2}$ - LHS eq.(4.5) is not satisfied
---->
 $v < v_{s,c}$ - RHS eq.(4.5) is not satisfied
---->
Case/Assumption Rejected.
---->
New Case/Assumption: Unconfined full section - Spalling of concrete cover
' satisfies Eq. (4.4)
---->
 $v < s_{y1}$ - LHS eq.(4.7) is not satisfied
---->
 $v < v_{c,y1}$ - RHS eq.(4.6) is satisfied
---->
 $cu (4.10) = 0.58509201$
 $M_{Rc} (4.17) = 2.3801E+009$
---->
New Case/Assumption: Ultimate curvature of confined core after spalling of concrete cover
In expressions below, the following modifications have been made
- b, d, d' replaced by geometric parameters of the core: b_o, d_o, d'_o
- $N, 1, 2, v$ normalised to $b_o * d_o$, instead of $b * d$
- parameters of confined concrete, f_{cc}, cc , used in lieu of f_c, ec_u
---->
Subcase: Rupture of tension steel
---->

$v^* < v^*_{s,y2}$ - LHS eq.(4.5) is not satisfied

--->

$v^* < v^*_{s,c}$ - LHS eq.(4.5) is not satisfied

--->

Subcase rejected

--->

New Subcase: Failure of compression zone

--->

$v^* < v^*_{c,y2}$ - LHS eq.(4.6) is not satisfied

--->

$v^* < v^*_{c,y1}$ - RHS eq.(4.6) is not satisfied

--->

*cu (4.11) = 0.65878578

MRo (4.18) = 3.0615E+009

--->

u = cu (4.2) = 1.3930706E-005

Mu = MRo

Calculation of ratio lb/l_d

Adequate Lap Length: lb/l_d >= 1

Calculation of Mu2-

Calculation of ultimate curvature u according to 4.1, Biskinis/Fardis 2013:

u = 9.1773508E-006

Mu = 8.9683E+008

with full section properties:

b = 450.00

d = 707.00

d' = 43.00

v = 1.34176

N = 7.6839E+006

fc = 18.00

co (5A.5, TBDY) = 0.002

Final value of cu: cu* = shear_factor * Max(cu, cc) = 0.01486841

The Shear_factor is considered equal to 1 (pure moment strength)

From (5.4b), TBDY: cu = 0.01486841

we (5.4c) = 0.08077545

ase ((5.4d), TBDY) = (ase1*Aext+ase2*Aint)/Asec = 0.53375773

ase1 = Max(((Aconf,max1-AnoConf1)/Aconf,max1)*(Aconf,min1/Aconf,max1),0) = 0.53375773

The definitions of AnoConf, Aconf,min and Aconf,max are derived from generalization of the rectangular sections confinement, which is expressed by (5.4d).

The generalization is done according to Mander, J., Priestley, M., and Park, R. (1988)

"Theoretical Stress-Strain Model for Confined Concrete."

J. Struct. Eng., 10.1061/(ASCE)0733-9445(1988)114:8(1804), 1804-1826.

Aconf,max1 = 464100.00 is the confined core area at levels of member with hoops and is calculated as the area of core enclosed by the center lines of the external perimeter hoops.

Aconf,min1 = 395025.00 is the confined core area at midway between the levels of hoops and is calculated by reducing all the dimensions of the area Aconf,max1 by a length equal to half the clear spacing between external hoops.

AnoConf1 = 173066.667 is the unconfined external core area which is equal to bi²/6 as defined at (A.2).

ase2 (>=ase1) = Max(((Aconf,max2-AnoConf2)/Aconf,max2)*(Aconf,min2/Aconf,max2),0) = 0.53375773

The definitions of AnoConf, Aconf,min and Aconf,max are derived from generalization of the rectangular sections confinement, which is expressed by (5.4d).

The generalization is done according to Mander, J., Priestley, M., and Park, R. (1988)

"Theoretical Stress-Strain Model for Confined Concrete."

J. Struct. Eng., 10.1061/(ASCE)0733-9445(1988)114:8(1804), 1804-1826.

Aconf,max2 = 252164.00 is the confined core area at levels of member with hoops and is calculated as the area of core enclosed by the center lines of the internal perimeter hoops.

Aconf,min2 = 111441.00 is the confined core area at midway between the levels of hoops and

is calculated by reducing all the dimensions of the area Aconf,max2 by a length equal to half the clear spacing between internal hoops.

AnoConf2 = 110709.333 is the unconfined internal core area which is equal to $b_i^2/6$ as defined at (A.2).
 $psh,min*Fywe = \text{Min}(psh,x*Fywe, psh,y*Fywe) = 2.724$

$psh_x*Fywe = psh1*Fywe1 + ps2*Fywe2 = 2.724$
 $psh1 \text{ ((5.4d), TBDY)} = Lstir1*Astir1/(Asec*s1) = 0.00301593$
 $Lstir1 \text{ (Length of stirrups along Y)} = 2160.00$
 $Astir1 \text{ (stirrups area)} = 78.53982$
 $psh2 \text{ (5.4d)} = Lstir2*Astir2/(Asec*s2) = 0.00056047$
 $Lstir2 \text{ (Length of stirrups along Y)} = 1568.00$
 $Astir2 \text{ (stirrups area)} = 50.26548$

$psh_y*Fywe = psh1*Fywe1 + ps2*Fywe2 = 3.25416$
 $psh1 \text{ ((5.4d), TBDY)} = Lstir1*Astir1/(Asec*s1) = 0.00357443$
 $Lstir1 \text{ (Length of stirrups along X)} = 2560.00$
 $Astir1 \text{ (stirrups area)} = 78.53982$
 $psh2 \text{ ((5.4d), TBDY)} = Lstir2*Astir2/(Asec*s2) = 0.00070345$
 $Lstir2 \text{ (Length of stirrups along X)} = 1968.00$
 $Astir2 \text{ (stirrups area)} = 50.26548$

$Asec = 562500.00$
 $s1 = 100.00$
 $s2 = 250.00$
 $fywe1 = 781.25$
 $fywe2 = 656.25$
 $fce = 18.00$

From ((5.A5), TBDY), TBDY: $cc = 0.00621307$
 $c = \text{confinement factor} = 1.42131$

$y1 = 0.0025$
 $sh1 = 0.008$
 $ft1 = 900.1904$
 $fy1 = 750.1586$
 $su1 = 0.032$

using (30) in Biskinis/Fardis (2013) multiplied with shear_factor
and also multiplied by the shear_factor according to 15.7.1.4, with
 $\text{Shear_factor} = 1.00$

$lo/lou,min = lb/ld = 1.00$

$su1 = 0.4*esu1_nominal \text{ ((5.5), TBDY)} = 0.032$

From table 5A.1, TBDY: $esu1_nominal = 0.08$,

For calculation of $esu1_nominal$ and $y1, sh1, ft1, fy1$, it is considered
characteristic value $fsy1 = fs1/1.2$, from table 5.1, TBDY.

$y1, sh1, ft1, fy1$, are also multiplied by $\text{Min}(1, 1.25*(lb/ld)^{2/3})$, from 10.3.5, ASCE 41-17.

with $fs1 = (fs,jacket*Asl,ten,jacket + fs,core*Asl,ten,core)/Asl,ten = 750.1586$

with $Es1 = (Es,jacket*Asl,ten,jacket + Es,core*Asl,ten,core)/Asl,ten = 200000.00$

$y2 = 0.0025$
 $sh2 = 0.008$
 $ft2 = 907.50$
 $fy2 = 756.25$
 $su2 = 0.032$

using (30) in Biskinis/Fardis (2013) multiplied with shear_factor
and also multiplied by the shear_factor according to 15.7.1.4, with
 $\text{Shear_factor} = 1.00$

$lo/lou,min = lb/lb,min = 1.00$

$su2 = 0.4*esu2_nominal \text{ ((5.5), TBDY)} = 0.032$

From table 5A.1, TBDY: $esu2_nominal = 0.08$,

For calculation of $esu2_nominal$ and $y2, sh2, ft2, fy2$, it is considered
characteristic value $fsy2 = fs2/1.2$, from table 5.1, TBDY.

$y1, sh1, ft1, fy1$, are also multiplied by $\text{Min}(1, 1.25*(lb/ld)^{2/3})$, from 10.3.5, ASCE 41-17.

with $fs2 = (fs,jacket*Asl,com,jacket + fs,core*Asl,com,core)/Asl,com = 756.25$

with $Es2 = (Es,jacket*Asl,com,jacket + Es,core*Asl,com,core)/Asl,com = 200000.00$

$yv = 0.0025$
 $shv = 0.008$
 $ftv = 902.993$
 $fyv = 752.4941$
 $suv = 0.032$

using (30) in Biskinis/Fardis (2013) multiplied with shear_factor

and also multiplied by the shear_factor according to 15.7.1.4, with
Shear_factor = 1.00
 $l_o/l_{ou,min} = l_b/l_d = 1.00$
 $suv = 0.4 * esuv_{nominal} ((5.5), TBDY) = 0.032$
From table 5A.1, TBDY: $esuv_{nominal} = 0.08$,
considering characteristic value $fsyv = fsv/1.2$, from table 5.1, TBDY
For calculation of $esuv_{nominal}$ and yv, shv, ftv, fyv , it is considered
characteristic value $fsyv = fsv/1.2$, from table 5.1, TBDY.
 $y1, sh1, ft1, fy1$, are also multiplied by $Min(1, 1.25 * (l_b/l_d)^{2/3})$, from 10.3.5, ASCE 41-17.
with $fsv = (fs_{jacket} * Asl_{mid,jacket} + fs_{mid} * Asl_{mid,core}) / Asl_{mid} = 752.4941$
with $Esv = (Es_{jacket} * Asl_{mid,jacket} + Es_{mid} * Asl_{mid,core}) / Asl_{mid} = 200000.00$
 $1 = Asl_{ten} / (b * d) * (fs1 / fc) = 0.32428344$
 $2 = Asl_{com} / (b * d) * (fs2 / fc) = 0.20328574$
 $v = Asl_{mid} / (b * d) * (fsv / fc) = 0.35171278$
and confined core properties:
 $b = 390.00$
 $d = 677.00$
 $d' = 13.00$
 $fcc (5A.2, TBDY) = 25.58352$
 $cc (5A.5, TBDY) = 0.00621307$
 $c = \text{confinement factor} = 1.42131$
 $1 = Asl_{ten} / (b * d) * (fs1 / fc) = 0.39075399$
 $2 = Asl_{com} / (b * d) * (fs2 / fc) = 0.24495458$
 $v = Asl_{mid} / (b * d) * (fsv / fc) = 0.42380571$
Case/Assumption: Unconfined full section - Steel rupture
' satisfies Eq. (4.3)
--->
 $v < v_{s,y2}$ - LHS eq.(4.5) is not satisfied
--->
 $v < v_{s,c}$ - RHS eq.(4.5) is not satisfied
--->
Case/Assumption Rejected.
--->
New Case/Assumption: Unconfined full section - Spalling of concrete cover
' satisfies Eq. (4.4)
--->
 $v < s_{y1}$ - LHS eq.(4.7) is not satisfied
--->
 $v < v_{c,y1}$ - RHS eq.(4.6) is not satisfied
--->
 $cu (4.11) = 0.95920575$
 $MRC (4.18) = 7.7321E+008$
--->
New Case/Assumption: Ultimate curvature of confined core after spalling of concrete cover
In expressions below, the following modifications have been made
- b, d, d' replaced by geometric parameters of the core: $bo, do, d'o$
- $N, 1, 2, v$ normalised to $bo * do$, instead of $b * d$
- $parameters of confined concrete, fcc, cc, used in lieu of fc, ecu$
--->
Subcase: Rupture of tension steel
--->
 $v^* < v^*_{s,y2}$ - LHS eq.(4.5) is not satisfied
--->
 $v^* < v^*_{s,c}$ - LHS eq.(4.5) is not satisfied
--->
Subcase rejected
--->
New Subcase: Failure of compression zone
--->
 $v^* < v^*_{c,y2}$ - LHS eq.(4.6) is not satisfied
--->
 $v^* < v^*_{c,y1}$ - RHS eq.(4.6) is not satisfied
--->
 $*cu (4.11) = 1.00$
 $MRO (4.18) = 8.9683E+008$
--->
 $u = cu (4.2) = 9.1773508E-006$

Mu = M_{Ro}

Calculation of ratio lb/d

Adequate Lap Length: lb/d >= 1

Calculation of Shear Strength $V_r = \text{Min}(V_{r1}, V_{r2}) = 1.1678\text{E}+006$

Calculation of Shear Strength at edge 1, $V_{r1} = 1.1678\text{E}+006$

$V_{r1} = V_{\text{Col}} ((10.3), \text{ASCE } 41-17) = k_{nl} * V_{\text{Col}0}$

$V_{\text{Col}0} = 1.1678\text{E}+006$

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

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

= 1 (normal-weight concrete)

Mean concrete strength: $f_c' = (f_c'_{\text{jacket}} * \text{Area}_{\text{jacket}} + f_c'_{\text{core}} * \text{Area}_{\text{core}}) / \text{Area}_{\text{section}} = 15.20$, but $f_c'^{0.5} \leq 8.3$ MPa (22.5.3.1, ACI 318-14)

$M/V_d = 4.00$

$M_u = 7.6104\text{E}+008$

$V_u = 227.4778$

$d = 0.8 * h = 600.00$

$N_u = 7.6839\text{E}+006$

$A_g = 337500.00$

From (11.5.4.8), ACI 318-14: $V_s = V_{s\text{jacket}} + V_{s\text{core}} = 1.0354\text{E}+006$

where:

$V_{s\text{jacket}} = V_{s\text{j1}} + V_{s\text{j2}} = 942477.796$

$V_{s\text{j1}} = 589048.623$ is calculated for section web jacket, with:

$d = 600.00$

$A_v = 157079.633$

$f_y = 625.00$

$s = 100.00$

$V_{s\text{j1}}$ is multiplied by $\text{Col}_{\text{j1}} = 1.00$

$s/d = 0.16666667$

$V_{s\text{j2}} = 353429.174$ is calculated for section flange jacket, with:

$d = 360.00$

$A_v = 157079.633$

$f_y = 625.00$

$s = 100.00$

$V_{s\text{j2}}$ is multiplied by $\text{Col}_{\text{j2}} = 1.00$

$s/d = 0.27777778$

$V_{s\text{core}} = V_{s\text{c1}} + V_{s\text{c2}} = 92890.612$

$V_{s\text{c1}} = 92890.612$ is calculated for section web core, with:

$d = 440.00$

$A_v = 100530.965$

$f_y = 525.00$

$s = 250.00$

$V_{s\text{c1}}$ is multiplied by $\text{Col}_{\text{c1}} = 1.00$

$s/d = 0.56818182$

$V_{s\text{c2}} = 0.00$ is calculated for section flange core, with:

$d = 200.00$

$A_v = 100530.965$

$f_y = 525.00$

$s = 250.00$

$V_{s\text{c2}}$ is multiplied by $\text{Col}_{\text{c2}} = 0.00$

$s/d = 1.25$

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

From (11-11), ACI 440: $V_s + V_f \leq 699281.943$

$b_w = 450.00$

Calculation of Shear Strength at edge 2, $V_{r2} = 1.1678\text{E}+006$

$Vr2 = VCol \text{ ((10.3), ASCE 41-17)} = knl * VCol0$

$VCol0 = 1.1678E+006$

$knl = 1$ (zero step-static loading)

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

$= 1$ (normal-weight concrete)

Mean concrete strength: $fc' = (fc'_{jacket} * Area_{jacket} + fc'_{core} * Area_{core}) / Area_{section} = 15.20$, but $fc'^{0.5} \leq 8.3$ MPa (22.5.3.1, ACI 318-14)

$M/Vd = 4.00$

$Mu = 6.8991E+008$

$Vu = 227.4778$

$d = 0.8 * h = 600.00$

$Nu = 7.6839E+006$

$Ag = 337500.00$

From (11.5.4.8), ACI 318-14: $Vs = Vs_{jacket} + Vs_{core} = 1.0354E+006$

where:

$Vs_{jacket} = Vs_{j1} + Vs_{j2} = 942477.796$

$Vs_{j1} = 589048.623$ is calculated for section web jacket, with:

$d = 600.00$

$Av = 157079.633$

$fy = 625.00$

$s = 100.00$

Vs_{j1} is multiplied by $Col_{j1} = 1.00$

$s/d = 0.16666667$

$Vs_{j2} = 353429.174$ is calculated for section flange jacket, with:

$d = 360.00$

$Av = 157079.633$

$fy = 625.00$

$s = 100.00$

Vs_{j2} is multiplied by $Col_{j2} = 1.00$

$s/d = 0.27777778$

$Vs_{core} = Vs_{c1} + Vs_{c2} = 92890.612$

$Vs_{c1} = 92890.612$ is calculated for section web core, with:

$d = 440.00$

$Av = 100530.965$

$fy = 525.00$

$s = 250.00$

Vs_{c1} is multiplied by $Col_{c1} = 1.00$

$s/d = 0.56818182$

$Vs_{c2} = 0.00$ is calculated for section flange core, with:

$d = 200.00$

$Av = 100530.965$

$fy = 525.00$

$s = 250.00$

Vs_{c2} is multiplied by $Col_{c2} = 0.00$

$s/d = 1.25$

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

From (11-11), ACI 440: $Vs + Vf \leq 699281.943$

$bw = 450.00$

End Of Calculation of Shear Capacity ratio for element: column JTC1 of floor 1
At local axis: 3

Start Of Calculation of Shear Capacity ratio for element: column JTC1 of floor 1
At Shear local axis: 2
(Bending local axis: 3)
Section Type: rcjtcs

Constant Properties

Knowledge Factor, $= 0.80$

Mean strength values are used for both shear and moment calculations.
 Consequently:
 Jacket
 New material of Primary Member: Concrete Strength, $f_c = f_{cm} = 18.00$
 New material of Primary Member: Steel Strength, $f_s = f_{sm} = 625.00$
 Concrete Elasticity, $E_c = 19940.411$
 Steel Elasticity, $E_s = 200000.00$
 Existing Column
 Existing material of Primary Member: Concrete Strength, $f_c = f_{cm} = 12.00$
 Existing material of Primary Member: Steel Strength, $f_s = f_{sm} = 525.00$
 Concrete Elasticity, $E_c = 16281.278$
 Steel Elasticity, $E_s = 200000.00$
 #####
 Note: Especially for the calculation of moment strengths,
 the above steel re-bar strengths are multiplied by 1.25 according to R18.6.5, ACI 318-14
 Jacket
 New material: Steel Strength, $f_s = 1.25 \cdot f_{sm} = 781.25$
 Existing Column
 Existing material: Steel Strength, $f_s = 1.25 \cdot f_{sm} = 656.25$
 #####
 Max Height, $H_{max} = 750.00$
 Min Height, $H_{min} = 450.00$
 Max Width, $W_{max} = 950.00$
 Min Width, $W_{min} = 450.00$
 Eccentricity, $Ecc = 250.00$
 Jacket Thickness, $t_j = 100.00$
 Cover Thickness, $c = 25.00$
 Mean Confinement Factor overall section = 1.42131
 Element Length, $L = 3000.00$
 Primary Member
 Ribbed Bars
 Ductile Steel
 With Detailing for Earthquake Resistance (including stirrups closed at 135°)
 Longitudinal Bars With Ends Lapped Starting at the End Sections
 Adequate Lap Length ($l_o/l_{ou,min} \geq 1$)
 No FRP Wrapping

Stepwise Properties

At local axis: 2
 EDGE -A-
 Shear Force, $V_a = -137.4894$
 EDGE -B-
 Shear Force, $V_b = 137.4894$
 BOTH EDGES
 Axial Force, $F = -7.6839E+006$
 Longitudinal Reinforcement Area Distribution (in 2 divisions)
 -Tension: $As_t = 314.1593$
 -Compression: $As_c = 6176.371$
 Longitudinal Reinforcement Area Distribution (in 3 divisions)
 -Tension: $As_{t,ten} = 1539.38$
 -Compression: $As_{l,com} = 1539.38$
 -Middle: $As_{l,mid} = 3612.832$

Calculation of Shear Capacity ratio, $V_e/V_r = 0.86614729$
 Member Controlled by Flexure ($V_e/V_r < 1$)
 Calculation of Shear Demand from fig. R18.6.5, ACI 318-14 $V_e = (M_{pr1} + M_{pr2})/l_n = 1.2287E+006$
 with
 $M_{pr1} = \max(M_{u1+}, M_{u1-}) = 1.8431E+009$
 $M_{u1+} = 1.8431E+009$, is the ultimate moment strength at the edge 1 of the member in the actual moment direction
 which is defined for the static loading combination
 $M_{u1-} = 1.8431E+009$, is the ultimate moment strength at the edge 1 of the member in the opposite moment
 direction which is defined for the static loading combination

$$M_{pr2} = \text{Max}(M_{u2+}, M_{u2-}) = 1.8431\text{E}+009$$

$M_{u2+} = 1.8431\text{E}+009$, is the ultimate moment strength at the edge 2 of the member in the actual moment direction which is defined for the static loading combination

$M_{u2-} = 1.8431\text{E}+009$, is the ultimate moment strength at the edge 2 of the member in the opposite moment direction which is defined for the static loading combination

Calculation of M_{u1+}

Calculation of ultimate curvature ϕ_u according to 4.1, Biskinis/Fardis 2013:

$$\phi_u = 7.8730391\text{E}-006$$

$$M_u = 1.8431\text{E}+009$$

with full section properties:

$$b = 450.00$$

$$d = 907.00$$

$$d' = 43.00$$

$$\nu = 1.0459$$

$$N = 7.6839\text{E}+006$$

$$f_c = 18.00$$

$$\phi_0 \text{ (5A.5, TBDY)} = 0.002$$

$$\text{Final value of } \phi_{cu}^* = \text{shear_factor} * \text{Max}(\phi_{cu}, \phi_{cc}) = 0.01486841$$

The Shear_factor is considered equal to 1 (pure moment strength)

$$\text{From (5.4b), TBDY: } \phi_{cu} = 0.01486841$$

$$\text{we (5.4c)} = 0.08077545$$

$$\text{ase ((5.4d), TBDY)} = (\text{ase1} * A_{ext} + \text{ase2} * A_{int}) / A_{sec} = 0.53375773$$

$$\text{ase1} = \text{Max}(((A_{conf,max1} - A_{noConf1}) / A_{conf,max1}) * (A_{conf,min1} / A_{conf,max1}), 0) = 0.53375773$$

The definitions of A_{noConf} , $A_{conf,min}$ and $A_{conf,max}$ are derived from generalization of the rectangular sections confinement, which is expressed by (5.4d).

The generalization is done according to Mander, J., Priestley, M., and Park, R. (1988)

"Theoretical Stress-Strain Model for Confined Concrete."

J. Struct. Eng., 10.1061/(ASCE)0733-9445(1988)114:8(1804), 1804-1826.

$A_{conf,max1} = 464100.00$ is the confined core area at levels of member with hoops and

is calculated as the area of core enclosed by the center lines of the external perimeter hoops.

$A_{conf,min1} = 395025.00$ is the confined core area at midway between the levels of hoops and

is calculated by reducing all the dimensions of the area $A_{conf,max1}$ by a length equal to half the clear spacing between external hoops.

$A_{noConf1} = 173066.667$ is the unconfined external core area which is equal to $b_i^2/6$ as defined at (A.2).

$$\text{ase2} (\geq \text{ase1}) = \text{Max}(((A_{conf,max2} - A_{noConf2}) / A_{conf,max2}) * (A_{conf,min2} / A_{conf,max2}), 0) = 0.53375773$$

The definitions of A_{noConf} , $A_{conf,min}$ and $A_{conf,max}$ are derived from generalization of the rectangular sections confinement, which is expressed by (5.4d).

The generalization is done according to Mander, J., Priestley, M., and Park, R. (1988)

"Theoretical Stress-Strain Model for Confined Concrete."

J. Struct. Eng., 10.1061/(ASCE)0733-9445(1988)114:8(1804), 1804-1826.

$A_{conf,max2} = 252164.00$ is the confined core area at levels of member with hoops and

is calculated as the area of core enclosed by the center lines of the internal perimeter hoops.

$A_{conf,min2} = 111441.00$ is the confined core area at midway between the levels of hoops and

is calculated by reducing all the dimensions of the area $A_{conf,max2}$ by a length equal to half the clear spacing between internal hoops.

$A_{noConf2} = 110709.333$ is the unconfined internal core area which is equal to $b_i^2/6$ as defined at (A.2).

$$\text{psh}_{min} * F_{ywe} = \text{Min}(\text{psh}_x * F_{ywe}, \text{psh}_y * F_{ywe}) = 2.724$$

$$\text{psh}_x * F_{ywe} = \text{psh1} * F_{ywe1} + \text{ps2} * F_{ywe2} = 2.724$$

$$\text{psh1 ((5.4d), TBDY)} = L_{stir1} * A_{stir1} / (A_{sec} * s1) = 0.00301593$$

$$L_{stir1} \text{ (Length of stirrups along Y)} = 2160.00$$

$$A_{stir1} \text{ (stirrups area)} = 78.53982$$

$$\text{psh2 (5.4d)} = L_{stir2} * A_{stir2} / (A_{sec} * s2) = 0.00056047$$

$$L_{stir2} \text{ (Length of stirrups along Y)} = 1568.00$$

$$A_{stir2} \text{ (stirrups area)} = 50.26548$$

$$\text{psh}_y * F_{ywe} = \text{psh1} * F_{ywe1} + \text{ps2} * F_{ywe2} = 3.25416$$

$$\text{psh1 ((5.4d), TBDY)} = L_{stir1} * A_{stir1} / (A_{sec} * s1) = 0.00357443$$

$$L_{stir1} \text{ (Length of stirrups along X)} = 2560.00$$

$$A_{stir1} \text{ (stirrups area)} = 78.53982$$

$$\text{psh2 ((5.4d), TBDY)} = L_{stir2} * A_{stir2} / (A_{sec} * s2) = 0.00070345$$

Lstir2 (Length of stirrups along X) = 1968.00
Astir2 (stirrups area) = 50.26548

Asec = 562500.00

s1 = 100.00

s2 = 250.00

fywe1 = 781.25

fywe2 = 656.25

fce = 18.00

From ((5A5), TBDY), TBDY: cc = 0.00621307

c = confinement factor = 1.42131

y1 = 0.0025

sh1 = 0.008

ft1 = 907.50

fy1 = 756.25

su1 = 0.032

using (30) in Biskinis/Fardis (2013) multiplied with shear_factor
and also multiplied by the shear_factor according to 15.7.1.4, with
Shear_factor = 1.00

lo/lou,min = lb/lb,min = 1.00

su1 = 0.4*esu1_nominal ((5.5), TBDY) = 0.032

From table 5A.1, TBDY: esu1_nominal = 0.08,

For calculation of esu1_nominal and y1, sh1,ft1,fy1, it is considered
characteristic value fsy1 = fs1/1.2, from table 5.1, TBDY.

y1, sh1,ft1,fy1, are also multiplied by $\text{Min}(1, 1.25 \cdot (lb/lb)^{2/3})$, from 10.3.5, ASCE 41-17.

with fs1 = (fs,jacket*Asl,ten,jacket + fs,core*Asl,ten,core)/Asl,ten = 756.25

with Es1 = (Es,jacket*Asl,ten,jacket + Es,core*Asl,ten,core)/Asl,ten = 200000.00

y2 = 0.0025

sh2 = 0.008

ft2 = 907.50

fy2 = 756.25

su2 = 0.032

using (30) in Biskinis/Fardis (2013) multiplied with shear_factor
and also multiplied by the shear_factor according to 15.7.1.4, with
Shear_factor = 1.00

lo/lou,min = lb/lb,min = 1.00

su2 = 0.4*esu2_nominal ((5.5), TBDY) = 0.032

From table 5A.1, TBDY: esu2_nominal = 0.08,

For calculation of esu2_nominal and y2, sh2,ft2,fy2, it is considered
characteristic value fsy2 = fs2/1.2, from table 5.1, TBDY.

y1, sh1,ft1,fy1, are also multiplied by $\text{Min}(1, 1.25 \cdot (lb/lb)^{2/3})$, from 10.3.5, ASCE 41-17.

with fs2 = (fs,jacket*Asl,com,jacket + fs,core*Asl,com,core)/Asl,com = 756.25

with Es2 = (Es,jacket*Asl,com,jacket + Es,core*Asl,com,core)/Asl,com = 200000.00

yv = 0.0025

shv = 0.008

ftv = 899.1522

fyv = 749.2935

suv = 0.032

using (30) in Biskinis/Fardis (2013) multiplied with shear_factor
and also multiplied by the shear_factor according to 15.7.1.4, with
Shear_factor = 1.00

lo/lou,min = lb/lb,min = 1.00

suv = 0.4*esuv_nominal ((5.5), TBDY) = 0.032

From table 5A.1, TBDY: esuv_nominal = 0.08,

considering characteristic value fsyv = fsv/1.2, from table 5.1, TBDY

For calculation of esuv_nominal and yv, shv,ftv,fyv, it is considered
characteristic value fsyv = fsv/1.2, from table 5.1, TBDY.

y1, sh1,ft1,fy1, are also multiplied by $\text{Min}(1, 1.25 \cdot (lb/lb)^{2/3})$, from 10.3.5, ASCE 41-17.

with fsv = (fs,jacket*Asl,mid,jacket + fs,mid*Asl,mid,core)/Asl,mid = 749.2935

with Esv = (Es,jacket*Asl,mid,jacket + Es,mid*Asl,mid,core)/Asl,mid = 200000.00

1 = Asl,ten/(b*d)*(fs1/fc) = 0.15845977

2 = Asl,com/(b*d)*(fs2/fc) = 0.15845977

v = Asl,mid/(b*d)*(fsv/fc) = 0.36847443

and confined core properties:

b = 390.00

d = 877.00

d' = 13.00

```

fcc (5A.2, TBDY) = 25.58352
cc (5A.5, TBDY) = 0.00621307
c = confinement factor = 1.42131
1 = Asl,ten/(b*d)*(fs1/fc) = 0.18909264
2 = Asl,com/(b*d)*(fs2/fc) = 0.18909264
v = Asl,mid/(b*d)*(fsv/fc) = 0.43970658
Case/Assumption: Unconfined full section - Steel rupture
' satisfies Eq. (4.3)
---->
v < vs,y2 - LHS eq.(4.5) is not satisfied
---->
v < vs,c - RHS eq.(4.5) is not satisfied
---->
Case/Assumption Rejected.
---->
New Case/Assumption: Unconfined full section - Spalling of concrete cover
' satisfies Eq. (4.4)
---->
v < s,y1 - LHS eq.(4.7) is not satisfied
---->
v < vc,y1 - RHS eq.(4.6) is satisfied
---->
cu (4.10) = 0.83934343
MRc (4.17) = 1.8439E+009
---->
New Case/Assumption: Ultimate curvature of confined core after spalling of concrete cover
In expressions below, the following modifications have been made
- b, d, d' replaced by geometric parameters of the core: bo, do, d'o
- N, 1, 2, v normalised to bo*do, instead of b*d
- - parameters of confined concrete, fcc, cc, used in lieu of fc, ecu
---->
Subcase: Rupture of tension steel
---->
v* < v*s,y2 - LHS eq.(4.5) is not satisfied
---->
v* < v*s,c - LHS eq.(4.5) is not satisfied
---->
Subcase rejected
---->
New Subcase: Failure of compression zone
---->
v* < v*c,y2 - LHS eq.(4.6) is not satisfied
---->
v* < v*c,y1 - RHS eq.(4.6) is not satisfied
---->
*cu (4.11) = 0.89983732
MRo (4.18) = 1.8431E+009
---->
u = cu (4.2) = 7.8730391E-006
Mu = MRo
-----

Calculation of ratio lb/d
-----
Adequate Lap Length: lb/d >= 1
-----
-----
Calculation of Mu1-
-----
-----
Calculation of ultimate curvature u according to 4.1, Biskinis/Fardis 2013:
u = 7.8730391E-006
Mu = 1.8431E+009
-----
with full section properties:

```

$b = 450.00$
 $d = 907.00$
 $d' = 43.00$
 $v = 1.0459$
 $N = 7.6839E+006$
 $f_c = 18.00$
 $\phi (5A.5, TBDY) = 0.002$
 Final value of ϕ : $\phi^* = \text{shear_factor} * \text{Max}(\phi, \phi_c) = 0.01486841$
 The Shear_factor is considered equal to 1 (pure moment strength)
 From (5.4b), TBDY: $\phi = 0.01486841$
 $\phi_w (5.4c) = 0.08077545$
 $\phi_{se} ((5.4d), TBDY) = (\phi_{se1} * A_{ext} + \phi_{se2} * A_{int}) / A_{sec} = 0.53375773$
 $\phi_{se1} = \text{Max}(((A_{conf,max1} - A_{noConf1}) / A_{conf,max1}) * (A_{conf,min1} / A_{conf,max1}), 0) = 0.53375773$
 The definitions of A_{noConf} , $A_{conf,min}$ and $A_{conf,max}$ are derived from generalization of the rectangular sections confinement, which is expressed by (5.4d).
 The generalization is done according to Mander, J., Priestley, M., and Park, R. (1988) "Theoretical Stress-Strain Model for Confined Concrete."
 J. Struct. Eng., 10.1061/(ASCE)0733-9445(1988)114:8(1804), 1804-1826.
 $A_{conf,max1} = 464100.00$ is the confined core area at levels of member with hoops and is calculated as the area of core enclosed by the center lines of the external perimeter hoops.
 $A_{conf,min1} = 395025.00$ is the confined core area at midway between the levels of hoops and is calculated by reducing all the dimensions of the area $A_{conf,max1}$ by a length equal to half the clear spacing between external hoops.
 $A_{noConf1} = 173066.667$ is the unconfined external core area which is equal to $b_i d_i / 6$ as defined at (A.2).
 $\phi_{se2} (> \phi_{se1}) = \text{Max}(((A_{conf,max2} - A_{noConf2}) / A_{conf,max2}) * (A_{conf,min2} / A_{conf,max2}), 0) = 0.53375773$
 The definitions of A_{noConf} , $A_{conf,min}$ and $A_{conf,max}$ are derived from generalization of the rectangular sections confinement, which is expressed by (5.4d).
 The generalization is done according to Mander, J., Priestley, M., and Park, R. (1988) "Theoretical Stress-Strain Model for Confined Concrete."
 J. Struct. Eng., 10.1061/(ASCE)0733-9445(1988)114:8(1804), 1804-1826.
 $A_{conf,max2} = 252164.00$ is the confined core area at levels of member with hoops and is calculated as the area of core enclosed by the center lines of the internal perimeter hoops.
 $A_{conf,min2} = 111441.00$ is the confined core area at midway between the levels of hoops and is calculated by reducing all the dimensions of the area $A_{conf,max2}$ by a length equal to half the clear spacing between internal hoops.
 $A_{noConf2} = 110709.333$ is the unconfined internal core area which is equal to $b_i d_i / 6$ as defined at (A.2).
 $\phi_{sh,min} * \phi_{wy} = \text{Min}(\phi_{sh,x} * \phi_{wy}, \phi_{sh,y} * \phi_{wy}) = 2.724$

$\phi_{sh,x} * \phi_{wy} = \phi_{sh1} * \phi_{wy1} + \phi_{sh2} * \phi_{wy2} = 2.724$
 $\phi_{sh1} ((5.4d), TBDY) = L_{stir1} * A_{stir1} / (A_{sec} * s_1) = 0.00301593$
 L_{stir1} (Length of stirrups along Y) = 2160.00
 A_{stir1} (stirrups area) = 78.53982
 $\phi_{sh2} (5.4d) = L_{stir2} * A_{stir2} / (A_{sec} * s_2) = 0.00056047$
 L_{stir2} (Length of stirrups along Y) = 1568.00
 A_{stir2} (stirrups area) = 50.26548

$\phi_{sh,y} * \phi_{wy} = \phi_{sh1} * \phi_{wy1} + \phi_{sh2} * \phi_{wy2} = 3.25416$
 $\phi_{sh1} ((5.4d), TBDY) = L_{stir1} * A_{stir1} / (A_{sec} * s_1) = 0.00357443$
 L_{stir1} (Length of stirrups along X) = 2560.00
 A_{stir1} (stirrups area) = 78.53982
 $\phi_{sh2} ((5.4d), TBDY) = L_{stir2} * A_{stir2} / (A_{sec} * s_2) = 0.00070345$
 L_{stir2} (Length of stirrups along X) = 1968.00
 A_{stir2} (stirrups area) = 50.26548

$A_{sec} = 562500.00$
 $s_1 = 100.00$
 $s_2 = 250.00$
 $\phi_{wy1} = 781.25$
 $\phi_{wy2} = 656.25$
 $\phi_{ce} = 18.00$
 From ((5.A.5), TBDY), TBDY: $\phi_c = 0.00621307$
 ϕ_c = confinement factor = 1.42131
 $\phi_1 = 0.0025$
 $\phi_{sh1} = 0.008$
 $\phi_{t1} = 907.50$
 $\phi_{y1} = 756.25$
 $\phi_{u1} = 0.032$

using (30) in Biskinis/Fardis (2013) multiplied with shear_factor
 and also multiplied by the shear_factor according to 15.7.1.4, with
 Shear_factor = 1.00
 $l_o/l_{ou,min} = l_b/l_d = 1.00$
 $su_1 = 0.4 * esu_{1,nominal} ((5.5), TBDY) = 0.032$
 From table 5A.1, TBDY: $esu_{1,nominal} = 0.08$,
 For calculation of $esu_{1,nominal}$ and y_1, sh_1, ft_1, fy_1 , it is considered
 characteristic value $fsy_1 = fs_1/1.2$, from table 5.1, TBDY.
 y_1, sh_1, ft_1, fy_1 , are also multiplied by $Min(1, 1.25 * (l_b/l_d)^{2/3})$, from 10.3.5, ASCE 41-17.
 with $fs_1 = (fs_{jacket} * A_{sl,ten,jacket} + fs_{core} * A_{sl,ten,core}) / A_{sl,ten} = 756.25$
 with $Es_1 = (Es_{jacket} * A_{sl,ten,jacket} + Es_{core} * A_{sl,ten,core}) / A_{sl,ten} = 200000.00$
 $y_2 = 0.0025$
 $sh_2 = 0.008$
 $ft_2 = 907.50$
 $fy_2 = 756.25$
 $su_2 = 0.032$
 using (30) in Biskinis/Fardis (2013) multiplied with shear_factor
 and also multiplied by the shear_factor according to 15.7.1.4, with
 Shear_factor = 1.00
 $l_o/l_{ou,min} = l_b/l_{b,min} = 1.00$
 $su_2 = 0.4 * esu_{2,nominal} ((5.5), TBDY) = 0.032$
 From table 5A.1, TBDY: $esu_{2,nominal} = 0.08$,
 For calculation of $esu_{2,nominal}$ and y_2, sh_2, ft_2, fy_2 , it is considered
 characteristic value $fsy_2 = fs_2/1.2$, from table 5.1, TBDY.
 y_2, sh_2, ft_2, fy_2 , are also multiplied by $Min(1, 1.25 * (l_b/l_d)^{2/3})$, from 10.3.5, ASCE 41-17.
 with $fs_2 = (fs_{jacket} * A_{sl,com,jacket} + fs_{core} * A_{sl,com,core}) / A_{sl,com} = 756.25$
 with $Es_2 = (Es_{jacket} * A_{sl,com,jacket} + Es_{core} * A_{sl,com,core}) / A_{sl,com} = 200000.00$
 $y_v = 0.0025$
 $sh_v = 0.008$
 $ft_v = 899.1522$
 $fy_v = 749.2935$
 $suv = 0.032$
 using (30) in Biskinis/Fardis (2013) multiplied with shear_factor
 and also multiplied by the shear_factor according to 15.7.1.4, with
 Shear_factor = 1.00
 $l_o/l_{ou,min} = l_b/l_d = 1.00$
 $suv = 0.4 * esuv_{nominal} ((5.5), TBDY) = 0.032$
 From table 5A.1, TBDY: $esuv_{nominal} = 0.08$,
 considering characteristic value $fsyv = fsv/1.2$, from table 5.1, TBDY
 For calculation of $esuv_{nominal}$ and y_v, sh_v, ft_v, fy_v , it is considered
 characteristic value $fsyv = fsv/1.2$, from table 5.1, TBDY.
 y_1, sh_1, ft_1, fy_1 , are also multiplied by $Min(1, 1.25 * (l_b/l_d)^{2/3})$, from 10.3.5, ASCE 41-17.
 with $fsv = (fs_{jacket} * A_{sl,mid,jacket} + fs_{mid} * A_{sl,mid,core}) / A_{sl,mid} = 749.2935$
 with $Esv = (Es_{jacket} * A_{sl,mid,jacket} + Es_{mid} * A_{sl,mid,core}) / A_{sl,mid} = 200000.00$
 $1 = A_{sl,ten} / (b * d) * (fs_1 / fc) = 0.15845977$
 $2 = A_{sl,com} / (b * d) * (fs_2 / fc) = 0.15845977$
 $v = A_{sl,mid} / (b * d) * (fsv / fc) = 0.36847443$
 and confined core properties:
 $b = 390.00$
 $d = 877.00$
 $d' = 13.00$
 $f_{cc} (5A.2, TBDY) = 25.58352$
 $cc (5A.5, TBDY) = 0.00621307$
 $c = \text{confinement factor} = 1.42131$
 $1 = A_{sl,ten} / (b * d) * (fs_1 / fc) = 0.18909264$
 $2 = A_{sl,com} / (b * d) * (fs_2 / fc) = 0.18909264$
 $v = A_{sl,mid} / (b * d) * (fsv / fc) = 0.43970658$
 Case/Assumption: Unconfined full section - Steel rupture
 ' satisfies Eq. (4.3)
 --->
 $v < v_{s,y2}$ - LHS eq.(4.5) is not satisfied
 --->
 $v < v_{s,c}$ - RHS eq.(4.5) is not satisfied
 --->
 Case/Assumption Rejected.
 --->
 New Case/Assumption: Unconfined full section - Spalling of concrete cover

' satisfies Eq. (4.4)
 --->
 $v < s_y1$ - LHS eq.(4.7) is not satisfied
 --->
 $v < v_c y1$ - RHS eq.(4.6) is satisfied
 --->
 c_u (4.10) = 0.83934343
 M_{Rc} (4.17) = 1.8439E+009
 --->
 New Case/Assumption: Ultimate curvature of confined core after spalling of concrete cover
 In expressions below, the following modifications have been made
 - b, d, d' replaced by geometric parameters of the core: b_o, d_o, d'_o
 - $N, 1, 2, v$ normalised to $b_o d_o$, instead of $b d$
 - - parameters of confined concrete, f_{cc}, c_c , used in lieu of f_c, c_u
 --->
 Subcase: Rupture of tension steel
 --->
 $v^* < v^* s_y2$ - LHS eq.(4.5) is not satisfied
 --->
 $v^* < v^* s_c$ - LHS eq.(4.5) is not satisfied
 --->
 Subcase rejected
 --->
 New Subcase: Failure of compression zone
 --->
 $v^* < v^* c_y2$ - LHS eq.(4.6) is not satisfied
 --->
 $v^* < v^* c_y1$ - RHS eq.(4.6) is not satisfied
 --->
 $*c_u$ (4.11) = 0.89983732
 M_{Ro} (4.18) = 1.8431E+009
 --->
 $u = c_u$ (4.2) = 7.8730391E-006
 $\mu = M_{Ro}$

 Calculation of ratio I_b/I_d

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

Calculation of μ_{2+}

 Calculation of ultimate curvature u according to 4.1, Biskinis/Fardis 2013:

$u = 7.8730391E-006$

$\mu = 1.8431E+009$

with full section properties:

$b = 450.00$

$d = 907.00$

$d' = 43.00$

$v = 1.0459$

$N = 7.6839E+006$

$f_c = 18.00$

c_o (5A.5, TBDY) = 0.002

Final value of c_u : $c_u^* = \text{shear_factor} * \text{Max}(c_u, c_c) = 0.01486841$

The Shear_factor is considered equal to 1 (pure moment strength)

From (5.4b), TBDY: $c_u = 0.01486841$

w_e (5.4c) = 0.08077545

a_{se} ((5.4d), TBDY) = $(a_{se1} * A_{ext} + a_{se2} * A_{int}) / A_{sec} = 0.53375773$

$a_{se1} = \text{Max}(((A_{conf,max1} - A_{noConf1}) / A_{conf,max1}) * (A_{conf,min1} / A_{conf,max1}), 0) = 0.53375773$

The definitions of A_{noConf} , $A_{conf,min}$ and $A_{conf,max}$ are derived from generalization of the rectangular sections confinement, which is expressed by (5.4d).

The generalization is done according to Mander, J., Priestley, M., and Park, R. (1988)

"Theoretical Stress-Strain Model for Confined Concrete."

J. Struct. Eng., 10.1061/(ASCE)0733-9445(1988)114:8(1804), 1804-1826.

$A_{conf,max1} = 464100.00$ is the confined core area at levels of member with hoops and is calculated as the area of core enclosed by the center lines of the external perimeter hoops.
 $A_{conf,min1} = 395025.00$ is the confined core area at midway between the levels of hoops and is calculated by reducing all the dimensions of the area $A_{conf,max1}$ by a length equal to half the clear spacing between external hoops.

$A_{noConf1} = 173066.667$ is the unconfined external core area which is equal to $b_i^2/6$ as defined at (A.2).
 $a_{se2} (\geq a_{se1}) = \text{Max}(((A_{conf,max2} - A_{noConf2})/A_{conf,max2}) * (A_{conf,min2}/A_{conf,max2}), 0) = 0.53375773$

The definitions of A_{noConf} , $A_{conf,min}$ and $A_{conf,max}$ are derived from generalization of the rectangular sections confinement, which is expressed by (5.4d).

The generalization is done according to Mander, J., Priestley, M., and Park, R. (1988)

"Theoretical Stress-Strain Model for Confined Concrete."

J. Struct. Eng., 10.1061/(ASCE)0733-9445(1988)114:8(1804), 1804-1826.

$A_{conf,max2} = 252164.00$ is the confined core area at levels of member with hoops and is calculated as the area of core enclosed by the center lines of the internal perimeter hoops.

$A_{conf,min2} = 111441.00$ is the confined core area at midway between the levels of hoops and is calculated by reducing all the dimensions of the area $A_{conf,max2}$ by a length equal to half the clear spacing between internal hoops.

$A_{noConf2} = 110709.333$ is the unconfined internal core area which is equal to $b_i^2/6$ as defined at (A.2).
 $p_{sh,min} * F_{ywe} = \text{Min}(p_{sh,x} * F_{ywe}, p_{sh,y} * F_{ywe}) = 2.724$

 $p_{sh,x} * F_{ywe} = p_{sh1} * F_{ywe1} + p_{s2} * F_{ywe2} = 2.724$

p_{sh1} ((5.4d), TBDY) = $L_{stir1} * A_{stir1} / (A_{sec} * s_1) = 0.00301593$

L_{stir1} (Length of stirrups along Y) = 2160.00

A_{stir1} (stirrups area) = 78.53982

p_{sh2} (5.4d) = $L_{stir2} * A_{stir2} / (A_{sec} * s_2) = 0.00056047$

L_{stir2} (Length of stirrups along Y) = 1568.00

A_{stir2} (stirrups area) = 50.26548

 $p_{sh,y} * F_{ywe} = p_{sh1} * F_{ywe1} + p_{s2} * F_{ywe2} = 3.25416$

p_{sh1} ((5.4d), TBDY) = $L_{stir1} * A_{stir1} / (A_{sec} * s_1) = 0.00357443$

L_{stir1} (Length of stirrups along X) = 2560.00

A_{stir1} (stirrups area) = 78.53982

p_{sh2} ((5.4d), TBDY) = $L_{stir2} * A_{stir2} / (A_{sec} * s_2) = 0.00070345$

L_{stir2} (Length of stirrups along X) = 1968.00

A_{stir2} (stirrups area) = 50.26548

 $A_{sec} = 562500.00$

$s_1 = 100.00$

$s_2 = 250.00$

$f_{ywe1} = 781.25$

$f_{ywe2} = 656.25$

$f_{ce} = 18.00$

From ((5.A5), TBDY), TBDY: $cc = 0.00621307$

c = confinement factor = 1.42131

$y_1 = 0.0025$

$sh_1 = 0.008$

$ft_1 = 907.50$

$fy_1 = 756.25$

$su_1 = 0.032$

using (30) in Biskinis/Fardis (2013) multiplied with shear_factor and also multiplied by the shear_factor according to 15.7.1.4, with Shear_factor = 1.00

$l_0/l_{ou,min} = l_b/l_d = 1.00$

$su_1 = 0.4 * esu_{1,nominal}$ ((5.5), TBDY) = 0.032

From table 5A.1, TBDY: $esu_{1,nominal} = 0.08$,

For calculation of $esu_{1,nominal}$ and y_1, sh_1, ft_1, fy_1 , it is considered characteristic value $fsy_1 = fs_1/1.2$, from table 5.1, TBDY.

y_1, sh_1, ft_1, fy_1 , are also multiplied by $\text{Min}(1, 1.25 * (l_b/l_d)^{2/3})$, from 10.3.5, ASCE 41-17.

with $fs_1 = (f_{s,jacket} * A_{sl,ten,jacket} + f_{s,core} * A_{sl,ten,core}) / A_{sl,ten} = 756.25$

with $Es_1 = (E_{s,jacket} * A_{sl,ten,jacket} + E_{s,core} * A_{sl,ten,core}) / A_{sl,ten} = 200000.00$

$y_2 = 0.0025$

$sh_2 = 0.008$

$ft_2 = 907.50$

$fy_2 = 756.25$

$su_2 = 0.032$

```

using (30) in Biskinis/Fardis (2013) multiplied with shear_factor
and also multiplied by the shear_factor according to 15.7.1.4, with
Shear_factor = 1.00
lo/lou,min = lb/lb,min = 1.00
su2 = 0.4*esu2_nominal ((5.5), TBDY) = 0.032
From table 5A.1, TBDY: esu2_nominal = 0.08,
For calculation of esu2_nominal and y2, sh2,ft2,fy2, it is considered
characteristic value fsy2 = fs2/1.2, from table 5.1, TBDY.
y1, sh1,ft1,fy1, are also multiplied by Min(1,1.25*(lb/lb)^ 2/3), from 10.3.5, ASCE 41-17.
with fs2 = (fs,jacket*Asl,com,jacket + fs,core*Asl,com,core)/Asl,com = 756.25
with Es2 = (Es,jacket*Asl,com,jacket + Es,core*Asl,com,core)/Asl,com = 200000.00
yv = 0.0025
shv = 0.008
ftv = 899.1522
fyv = 749.2935
suv = 0.032
using (30) in Biskinis/Fardis (2013) multiplied with shear_factor
and also multiplied by the shear_factor according to 15.7.1.4, with
Shear_factor = 1.00
lo/lou,min = lb/lb = 1.00
suv = 0.4*esuv_nominal ((5.5), TBDY) = 0.032
From table 5A.1, TBDY: esuv_nominal = 0.08,
considering characteristic value fsyv = fsv/1.2, from table 5.1, TBDY
For calculation of esuv_nominal and yv, shv,ftv,fyv, it is considered
characteristic value fsyv = fsv/1.2, from table 5.1, TBDY.
y1, sh1,ft1,fy1, are also multiplied by Min(1,1.25*(lb/lb)^ 2/3), from 10.3.5, ASCE 41-17.
with fsv = (fs,jacket*Asl,mid,jacket + fs,mid*Asl,mid,core)/Asl,mid = 749.2935
with Esv = (Es,jacket*Asl,mid,jacket + Es,mid*Asl,mid,core)/Asl,mid = 200000.00
1 = Asl,ten/(b*d)*(fs1/fc) = 0.15845977
2 = Asl,com/(b*d)*(fs2/fc) = 0.15845977
v = Asl,mid/(b*d)*(fsv/fc) = 0.36847443
and confined core properties:
b = 390.00
d = 877.00
d' = 13.00
fcc (5A.2, TBDY) = 25.58352
cc (5A.5, TBDY) = 0.00621307
c = confinement factor = 1.42131
1 = Asl,ten/(b*d)*(fs1/fc) = 0.18909264
2 = Asl,com/(b*d)*(fs2/fc) = 0.18909264
v = Asl,mid/(b*d)*(fsv/fc) = 0.43970658
Case/Assumption: Unconfined full section - Steel rupture
' satisfies Eq. (4.3)
---->
v < vs,y2 - LHS eq.(4.5) is not satisfied
---->
v < vs,c - RHS eq.(4.5) is not satisfied
---->
Case/Assumption Rejected.
---->
New Case/Assumption: Unconfined full section - Spalling of concrete cover
' satisfies Eq. (4.4)
---->
v < s,y1 - LHS eq.(4.7) is not satisfied
---->
v < vc,y1 - RHS eq.(4.6) is satisfied
---->
cu (4.10) = 0.83934343
MRc (4.17) = 1.8439E+009
---->
New Case/Assumption: Ultimate curvature of confined core after spalling of concrete cover
In expressions below, the following modifications have been made
- b, d, d' replaced by geometric parameters of the core: bo, do, d'o
- N, 1, 2, v normalised to bo*do, instead of b*d
- parameters of confined concrete, fcc, cc, used in lieu of fc, ecu
---->
Subcase: Rupture of tension steel

```

```

---->
v* < v*s,y2 - LHS eq.(4.5) is not satisfied
---->
v* < v*s,c - LHS eq.(4.5) is not satisfied
---->
Subcase rejected
---->
New Subcase: Failure of compression zone
---->
v* < v*c,y2 - LHS eq.(4.6) is not satisfied
---->
v* < v*c,y1 - RHS eq.(4.6) is not satisfied
---->
*cu (4.11) = 0.89983732
MRo (4.18) = 1.8431E+009
---->
u = cu (4.2) = 7.8730391E-006
Mu = MRo

```

Calculation of ratio lb/ld

Adequate Lap Length: lb/ld >= 1

Calculation of Mu2-

Calculation of ultimate curvature u according to 4.1, Biskinis/Fardis 2013:
u = 7.8730391E-006
Mu = 1.8431E+009

with full section properties:

```

b = 450.00
d = 907.00
d' = 43.00
v = 1.0459
N = 7.6839E+006
fc = 18.00
co (5A.5, TBDY) = 0.002
Final value of cu: cu* = shear_factor * Max( cu, cc) = 0.01486841
The Shear_factor is considered equal to 1 (pure moment strength)
From (5.4b), TBDY: cu = 0.01486841
we (5.4c) = 0.08077545
ase ((5.4d), TBDY) = (ase1*Aext+ase2*Aint)/Asec = 0.53375773
ase1 = Max(((Aconf,max1-AnoConf1)/Aconf,max1)*(Aconf,min1/Aconf,max1),0) = 0.53375773
The definitions of AnoConf, Aconf,min and Aconf,max are derived from generalization
of the rectangular sections confinement, which is expressed by (5.4d).
The generalization is done according to Mander, J., Priestley, M., and Park, R. (1988)
"Theoretical Stress-Strain Model for Confined Concrete."
J. Struct. Eng., 10.1061/(ASCE)0733-9445(1988)114:8(1804), 1804-1826.
Aconf,max1 = 464100.00 is the confined core area at levels of member with hoops and
is calculated as the area of core enclosed by the center lines of the external perimeter hoops.
Aconf,min1 = 395025.00 is the confined core area at midway between the levels of hoops and
is calculated by reducing all the dimensions of the area Aconf,max1 by a length
equal to half the clear spacing between external hoops.
AnoConf1 = 173066.667 is the unconfined external core area which is equal to bi2/6 as defined at (A.2).
ase2 (>=ase1) = Max(((Aconf,max2-AnoConf2)/Aconf,max2)*(Aconf,min2/Aconf,max2),0) = 0.53375773
The definitions of AnoConf, Aconf,min and Aconf,max are derived from generalization
of the rectangular sections confinement, which is expressed by (5.4d).
The generalization is done according to Mander, J., Priestley, M., and Park, R. (1988)
"Theoretical Stress-Strain Model for Confined Concrete."
J. Struct. Eng., 10.1061/(ASCE)0733-9445(1988)114:8(1804), 1804-1826.
Aconf,max2 = 252164.00 is the confined core area at levels of member with hoops and
is calculated as the area of core enclosed by the center lines of the internal perimeter hoops.

```


Aconf,min2 = 111441.00 is the confined core area at midway between the levels of hoops and is calculated by reducing all the dimensions of the area Aconf,max2 by a length equal to half the clear spacing between internal hoops.

AnoConf2 = 110709.333 is the unconfined internal core area which is equal to $b_i^2/6$ as defined at (A.2).
 $psh,min*Fywe = \text{Min}(psh,x*Fywe, psh,y*Fywe) = 2.724$

$psh_x*Fywe = psh1*Fywe1 + ps2*Fywe2 = 2.724$
 $psh1 \text{ ((5.4d), TBDY)} = Lstir1*Astir1/(Asec*s1) = 0.00301593$
 $Lstir1 \text{ (Length of stirrups along Y)} = 2160.00$
 $Astir1 \text{ (stirrups area)} = 78.53982$
 $psh2 \text{ (5.4d)} = Lstir2*Astir2/(Asec*s2) = 0.00056047$
 $Lstir2 \text{ (Length of stirrups along Y)} = 1568.00$
 $Astir2 \text{ (stirrups area)} = 50.26548$

$psh_y*Fywe = psh1*Fywe1 + ps2*Fywe2 = 3.25416$
 $psh1 \text{ ((5.4d), TBDY)} = Lstir1*Astir1/(Asec*s1) = 0.00357443$
 $Lstir1 \text{ (Length of stirrups along X)} = 2560.00$
 $Astir1 \text{ (stirrups area)} = 78.53982$
 $psh2 \text{ ((5.4d), TBDY)} = Lstir2*Astir2/(Asec*s2) = 0.00070345$
 $Lstir2 \text{ (Length of stirrups along X)} = 1968.00$
 $Astir2 \text{ (stirrups area)} = 50.26548$

$Asec = 562500.00$
 $s1 = 100.00$
 $s2 = 250.00$
 $fywe1 = 781.25$
 $fywe2 = 656.25$
 $fce = 18.00$

From ((5.A5), TBDY), TBDY: $cc = 0.00621307$
 $c = \text{confinement factor} = 1.42131$

$y1 = 0.0025$
 $sh1 = 0.008$
 $ft1 = 907.50$
 $fy1 = 756.25$
 $su1 = 0.032$

using (30) in Biskinis/Fardis (2013) multiplied with shear_factor and also multiplied by the shear_factor according to 15.7.1.4, with Shear_factor = 1.00

$lo/lou,min = lb/ld = 1.00$

$su1 = 0.4*esu1_nominal \text{ ((5.5), TBDY)} = 0.032$

From table 5A.1, TBDY: $esu1_nominal = 0.08$,

For calculation of $esu1_nominal$ and $y1, sh1, ft1, fy1$, it is considered characteristic value $fsy1 = fs1/1.2$, from table 5.1, TBDY.

$y1, sh1, ft1, fy1$, are also multiplied by $\text{Min}(1, 1.25*(lb/ld)^{2/3})$, from 10.3.5, ASCE 41-17.

with $fs1 = (fs,jacket*Asl,ten,jacket + fs,core*Asl,ten,core)/Asl,ten = 756.25$

with $Es1 = (Es,jacket*Asl,ten,jacket + Es,core*Asl,ten,core)/Asl,ten = 200000.00$

$y2 = 0.0025$
 $sh2 = 0.008$
 $ft2 = 907.50$
 $fy2 = 756.25$
 $su2 = 0.032$

using (30) in Biskinis/Fardis (2013) multiplied with shear_factor and also multiplied by the shear_factor according to 15.7.1.4, with Shear_factor = 1.00

$lo/lou,min = lb/lb,min = 1.00$

$su2 = 0.4*esu2_nominal \text{ ((5.5), TBDY)} = 0.032$

From table 5A.1, TBDY: $esu2_nominal = 0.08$,

For calculation of $esu2_nominal$ and $y2, sh2, ft2, fy2$, it is considered characteristic value $fsy2 = fs2/1.2$, from table 5.1, TBDY.

$y1, sh1, ft1, fy1$, are also multiplied by $\text{Min}(1, 1.25*(lb/ld)^{2/3})$, from 10.3.5, ASCE 41-17.

with $fs2 = (fs,jacket*Asl,com,jacket + fs,core*Asl,com,core)/Asl,com = 756.25$

with $Es2 = (Es,jacket*Asl,com,jacket + Es,core*Asl,com,core)/Asl,com = 200000.00$

$yv = 0.0025$
 $shv = 0.008$
 $ftv = 899.1522$
 $fyv = 749.2935$
 $suv = 0.032$

```

using (30) in Biskinis/Fardis (2013) multiplied with shear_factor
and also multiplied by the shear_factor according to 15.7.1.4, with
Shear_factor = 1.00
lo/lo,min = lb/ld = 1.00
suv = 0.4*esuv_nominal ((5.5), TBDY) = 0.032
From table 5A.1, TBDY: esuv_nominal = 0.08,
considering characteristic value fsyv = fsv/1.2, from table 5.1, TBDY
For calculation of esuv_nominal and yv, shv,ftv,fyv, it is considered
characteristic value fsyv = fsv/1.2, from table 5.1, TBDY.
y1, sh1,ft1,fy1, are also multiplied by Min(1,1.25*(lb/ld)^ 2/3), from 10.3.5, ASCE 41-17.
with fsv = (fs,jacket*Asl,mid,jacket + fs,mid*Asl,mid,core)/Asl,mid = 749.2935
with Esv = (Es,jacket*Asl,mid,jacket + Es,mid*Asl,mid,core)/Asl,mid = 200000.00
1 = Asl,ten/(b*d)*(fs1/fc) = 0.15845977
2 = Asl,com/(b*d)*(fs2/fc) = 0.15845977
v = Asl,mid/(b*d)*(fsv/fc) = 0.36847443
and confined core properties:
b = 390.00
d = 877.00
d' = 13.00
fcc (5A.2, TBDY) = 25.58352
cc (5A.5, TBDY) = 0.00621307
c = confinement factor = 1.42131
1 = Asl,ten/(b*d)*(fs1/fc) = 0.18909264
2 = Asl,com/(b*d)*(fs2/fc) = 0.18909264
v = Asl,mid/(b*d)*(fsv/fc) = 0.43970658
Case/Assumption: Unconfined full section - Steel rupture
' satisfies Eq. (4.3)
---->
v < vs,y2 - LHS eq.(4.5) is not satisfied
---->
v < vs,c - RHS eq.(4.5) is not satisfied
---->
Case/Assumption Rejected.
---->
New Case/Assumption: Unconfined full section - Spalling of concrete cover
' satisfies Eq. (4.4)
---->
v < s,y1 - LHS eq.(4.7) is not satisfied
---->
v < vc,y1 - RHS eq.(4.6) is satisfied
---->
cu (4.10) = 0.83934343
MRc (4.17) = 1.8439E+009
---->
New Case/Assumption: Ultimate curvature of confined core after spalling of concrete cover
In expressions below, the following modifications have been made
- b, d, d' replaced by geometric parameters of the core: bo, do, d'o
- N, 1, 2, v normalised to bo*do, instead of b*d
- parameters of confined concrete, fcc, cc, used in lieu of fc, ecu
---->
Subcase: Rupture of tension steel
---->
v* < v*s,y2 - LHS eq.(4.5) is not satisfied
---->
v* < v*s,c - LHS eq.(4.5) is not satisfied
---->
Subcase rejected
---->
New Subcase: Failure of compression zone
---->
v* < v*c,y2 - LHS eq.(4.6) is not satisfied
---->
v* < v*c,y1 - RHS eq.(4.6) is not satisfied
---->
*cu (4.11) = 0.89983732
MRo (4.18) = 1.8431E+009
---->

```

$u = c_u(4.2) = 7.8730391E-006$
 $\mu = \mu_{Ro}$

Calculation of ratio l_b/l_d

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

Calculation of Shear Strength $V_r = \min(V_{r1}, V_{r2}) = 1.4186E+006$

Calculation of Shear Strength at edge 1, $V_{r1} = 1.4186E+006$
 $V_{r1} = V_{Col}((10.3), ASCE 41-17) = k_n l * V_{Col0}$
 $V_{Col0} = 1.4186E+006$
 $k_n l = 1$ (zero step-static loading)

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

$= 1$ (normal-weight concrete)
Mean concrete strength: $f_c' = (f_c'_{jacket} * Area_{jacket} + f_c'_{core} * Area_{core}) / Area_{section} = 15.20$, but $f_c'^{0.5} \leq 8.3$ MPa (22.5.3.1, ACI 318-14)
 $M/Vd = 4.00$
 $\mu_u = 2.6919E+008$
 $V_u = 137.4894$
 $d = 0.8 * h = 760.00$
 $N_u = 7.6839E+006$
 $A_g = 427500.00$
From (11.5.4.8), ACI 318-14: $V_s = V_{sjacket} + V_{s,core} = 1.2262E+006$
where:
 $V_{sjacket} = V_{sj1} + V_{sj2} = 1.0996E+006$
 $V_{sj1} = 353429.174$ is calculated for section web jacket, with:
 $d = 360.00$
 $A_v = 157079.633$
 $f_y = 625.00$
 $s = 100.00$
 V_{sj1} is multiplied by $Col_{j1} = 1.00$
 $s/d = 0.27777778$
 $V_{sj2} = 746128.255$ is calculated for section flange jacket, with:
 $d = 760.00$
 $A_v = 157079.633$
 $f_y = 625.00$
 $s = 100.00$
 V_{sj2} is multiplied by $Col_{j2} = 1.00$
 $s/d = 0.13157895$
 $V_{s,core} = V_{s,c1} + V_{s,c2} = 126669.016$
 $V_{s,c1} = 0.00$ is calculated for section web core, with:
 $d = 200.00$
 $A_v = 100530.965$
 $f_y = 525.00$
 $s = 250.00$
 $V_{s,c1}$ is multiplied by $Col_{c1} = 0.00$
 $s/d = 1.25$
 $V_{s,c2} = 126669.016$ is calculated for section flange core, with:
 $d = 600.00$
 $A_v = 100530.965$
 $f_y = 525.00$
 $s = 250.00$
 $V_{s,c2}$ is multiplied by $Col_{c2} = 1.00$
 $s/d = 0.41666667$
 $V_f((11-3)-(11.4), ACI 440) = 0.00$
From (11-11), ACI 440: $V_s + V_f \leq 885757.128$
 $b_w = 450.00$

Calculation of Shear Strength at edge 2, $V_{r2} = 1.4186E+006$

$V_{r2} = V_{Col} ((10.3), ASCE 41-17) = knl * V_{Col0}$

$V_{Col0} = 1.4186E+006$

$knl = 1$ (zero step-static loading)

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

$= 1$ (normal-weight concrete)

Mean concrete strength: $f_c' = (f_c'_{jacket} * Area_{jacket} + f_c'_{core} * Area_{core}) / Area_{section} = 15.20$, but $f_c'^{0.5} \leq 8.3$ MPa (22.5.3.1, ACI 318-14)

$M / Vd = 4.00$

$\mu_u = 2.5135E+008$

$V_u = 137.4894$

$d = 0.8 * h = 760.00$

$N_u = 7.6839E+006$

$A_g = 427500.00$

From (11.5.4.8), ACI 318-14: $V_s = V_{sjacket} + V_{s,core} = 1.2262E+006$

where:

$V_{sjacket} = V_{sj1} + V_{sj2} = 1.0996E+006$

$V_{sj1} = 353429.174$ is calculated for section web jacket, with:

$d = 360.00$

$A_v = 157079.633$

$f_y = 625.00$

$s = 100.00$

V_{sj1} is multiplied by $Col,j1 = 1.00$

$s/d = 0.27777778$

$V_{sj2} = 746128.255$ is calculated for section flange jacket, with:

$d = 760.00$

$A_v = 157079.633$

$f_y = 625.00$

$s = 100.00$

V_{sj2} is multiplied by $Col,j2 = 1.00$

$s/d = 0.13157895$

$V_{s,core} = V_{s,c1} + V_{s,c2} = 126669.016$

$V_{s,c1} = 0.00$ is calculated for section web core, with:

$d = 200.00$

$A_v = 100530.965$

$f_y = 525.00$

$s = 250.00$

$V_{s,c1}$ is multiplied by $Col,c1 = 0.00$

$s/d = 1.25$

$V_{s,c2} = 126669.016$ is calculated for section flange core, with:

$d = 600.00$

$A_v = 100530.965$

$f_y = 525.00$

$s = 250.00$

$V_{s,c2}$ is multiplied by $Col,c2 = 1.00$

$s/d = 0.41666667$

$V_f ((11-3)-(11.4), ACI 440) = 0.00$

From (11-11), ACI 440: $V_s + V_f \leq 885757.128$

$bw = 450.00$

End Of Calculation of Shear Capacity ratio for element: column JTC1 of floor 1

At local axis: 2

Start Of Calculation of Chord Rotation Capacity for element: column JTC1 of floor 1

At local axis: 3

Integration Section: (a)

Section Type: rcjtcs

Constant Properties

Knowledge Factor, $\phi = 0.80$
Chord Rotation is generally considered as Deformation-Controlled Action according to Table C7-1, ASCE 41-17.
Mean strengths are used for Deformation-Controlled Actions according to 7.5.1.3, ASCE 41-17
Consequently:
Jacket
New material of Primary Member: Concrete Strength, $f_c = f_{cm} = 18.00$
New material of Primary Member: Steel Strength, $f_s = f_{sm} = 625.00$
Concrete Elasticity, $E_c = 19940.411$
Steel Elasticity, $E_s = 200000.00$
Existing Column
Existing material of Primary Member: Concrete Strength, $f_c = f_{cm} = 12.00$
Existing material of Primary Member: Steel Strength, $f_s = f_{sm} = 525.00$
Concrete Elasticity, $E_c = 16281.278$
Steel Elasticity, $E_s = 200000.00$
Max Height, $H_{max} = 750.00$
Min Height, $H_{min} = 450.00$
Max Width, $W_{max} = 950.00$
Min Width, $W_{min} = 450.00$
Eccentricity, $Ecc = 250.00$
Jacket Thickness, $t_j = 100.00$
Cover Thickness, $c = 25.00$
Element Length, $L = 3000.00$
Primary Member
Ribbed Bars
Ductile Steel
With Detailing for Earthquake Resistance (including stirrups closed at 135°)
Longitudinal Bars With Ends Lapped Starting at the End Sections
Adequate Lap Length ($l_b/l_d \geq 1$)
No FRP Wrapping

Stepwise Properties

Bending Moment, $M = -6.8503E+008$
Shear Force, $V_2 = -212214.235$
Shear Force, $V_3 = -90512.847$
Axial Force, $F = -7.0682E+006$
Longitudinal Reinforcement Area Distribution (in 2 divisions)
-Tension: $As_t = 2522.699$
-Compression: $As_c = 4168.893$
Longitudinal Reinforcement Area Distribution (in 3 divisions)
-Tension: $As_{t,ten} = 1539.38$
-Compression: $As_{l,com} = 1539.38$
-Middle: $As_{l,mid} = 3612.832$
Longitudinal External Reinforcement Area Distribution (in 3 divisions)
-Tension: $As_{t,ten,jacket} = 1231.504$
-Compression: $As_{l,com,jacket} = 1231.504$
-Middle: $As_{l,mid,jacket} = 2689.203$
Longitudinal Internal Reinforcement Area Distribution (in 3 divisions)
-Tension: $As_{t,ten,core} = 307.8761$
-Compression: $As_{l,com,core} = 307.8761$
-Middle: $As_{l,mid,core} = 923.6282$
Mean Diameter of Tension Reinforcement, $Db_L = 16.57143$

Existing component: From table 7-7, ASCE 41_17: Final chord rotation Capacity $u_R = \phi \cdot u = 0.00283133$
 $u = y + p = 0.00353916$

- Calculation of y -

$y = (M_y \cdot L_s / 3) / E_{eff} = 0.00353916$ ((4.29), Biskinis Phd))
 $M_y = 1.5034E+009$
 $L_s = M/V$ (with $L_s > 0.1 \cdot L$ and $L_s < 2 \cdot L$) = 3228.019
From table 10.5, ASCE 41_17: $E_{eff} = factor \cdot E_c \cdot I_g = 4.5707E+014$

factor = 0.70
 $A_g = 562500.00$
Mean concrete strength: $f_c' = (f_{c'}^{\text{jacket}} \cdot \text{Area}_{\text{jacket}} + f_{c'}^{\text{core}} \cdot \text{Area}_{\text{core}}) / \text{Area}_{\text{section}} = 15.20$
 $N = 7.0682\text{E}+006$
 $E_c \cdot I_g = E_{c_{\text{jacket}}} \cdot I_{g_{\text{jacket}}} + E_{c_{\text{core}}} \cdot I_{g_{\text{core}}} = 6.5295\text{E}+014$

Calculation of Yielding Moment M_y

Calculation of y and M_y according to Annex 7 -

$y = \text{Min}(y_{\text{ten}}, y_{\text{com}})$
 $y_{\text{ten}} = 6.9719165\text{E}-006$
with $f_y = 601.9953$
 $d = 907.00$
 $y = 0.52400363$
 $A = 0.04516194$
 $B = 0.0373531$
with $p_t = 0.0037716$
 $p_c = 0.0037716$
 $p_v = 0.00885172$
 $N = 7.0682\text{E}+006$
 $b = 450.00$
 $" = 0.04740904$
 $y_{\text{comp}} = 1.9344180\text{E}-006$
with $f_c = 18.00$
 $E_c = 19940.411$
 $y = 0.92609018$
 $A = -0.0368952$
 $B = 0.0085861$
with $E_s = 200000.00$

Calculation of ratio I_b/I_d

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

- Calculation of p -

From table 10-8: $p = 0.00$

with:

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

shear control ratio $V_y E / V_{col} E = 0.86614729$

$d = d_{\text{external}} = 907.00$

$s = s_{\text{external}} = 50.00$

- $t = s_1 + s_2 + 2 \cdot t_f / b_w \cdot (f_{fe} / f_s) = 0.00427788$

jacket: $s_1 = A_{v1} \cdot L_{\text{stir1}} / (s_1 \cdot A_g) = 0.00357443$

$A_{v1} = 78.53982$, is the area of every stirrup parallel to loading (shear) direction

$L_{\text{stir1}} = 2560.00$, is the total Length of all stirrups parallel to loading (shear) direction

$s_1 = 100.00$

core: $s_2 = A_{v2} \cdot L_{\text{stir2}} / (s_2 \cdot A_g) = 0.00070345$

$A_{v2} = 50.26548$, is the area of every stirrup parallel to loading (shear) direction

$L_{\text{stir2}} = 1968.00$, is the total Length of all stirrups parallel to loading (shear) direction

$s_2 = 250.00$

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

where $f = 2 \cdot t_f / b_w$ is FRP ratio (EC8 - 3, A.4.4.3(6)) and f_{fe} / f_s normalises f to steel strength

All these variables have already been given in Shear control ratio calculation.

For the normalisation f_s of jacket is used.

$N_{UD} = 7.0682\text{E}+006$

$A_g = 562500.00$

$f_{cE} = (f_{c'}^{\text{jacket}} \cdot \text{Area}_{\text{jacket}} + f_{c'}^{\text{core}} \cdot \text{Area}_{\text{core}}) / \text{section_area} = 15.20$

$f_{yIE} = (f_{y_{\text{ext_Long_Reinf}}} \cdot \text{Area}_{\text{ext_Long_Reinf}} + f_{y_{\text{int_Long_Reinf}}} \cdot \text{Area}_{\text{int_Long_Reinf}}) / \text{Area}_{\text{Tot_Long_Rein}} = 601.9953$

$f_{yIE} = (f_{y_{\text{ext_Trans_Reinf}}} \cdot s_1 + f_{y_{\text{int_Trans_Reinf}}} \cdot s_2) / (s_1 + s_2) = 608.5561$

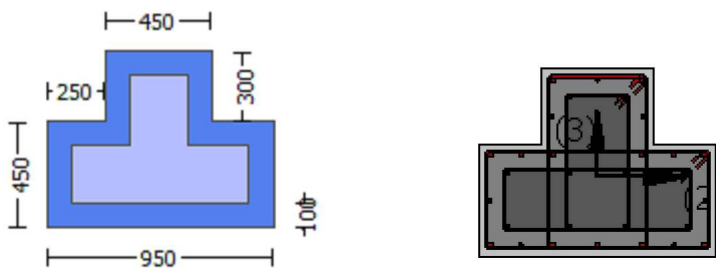
$p_l = \text{Area}_{\text{Tot_Long_Rein}} / (b \cdot d) = 0.01639493$

b = 450.00
d = 907.00
f_{cE} = 15.20

End Of Calculation of Chord Rotation Capacity for element: column JTC1 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 50 and 51)
Analysis: Uniform +X
Check: Shear capacity VR_d
Edge: End
Local Axis: (2)



Start Of Calculation of Shear Capacity for element: column JTC1 of floor 1
At local axis: 2
Integration Section: (b)
Section Type: rcjtcs

Constant Properties

Knowledge Factor, $\gamma = 0.80$
Member Shear Force is generally considered as Force-Controlled Action according to Table C7-1, ASCE 41-17.
Lower-bound strengths are used for Force-Controlled Actions according to 7.5.1.3, ASCE 41-17
Consequently:
Jacket
New material of Primary Member: Concrete Strength, $f_c = f_{c_lower_bound} = 12.00$
New material of Primary Member: Steel Strength, $f_s = f_{s_lower_bound} = 500.00$
Concrete Elasticity, $E_c = 19940.411$
Steel Elasticity, $E_s = 200000.00$
Existing Column

Existing material of Primary Member: Concrete Strength, $f_c = f_{c_lower_bound} = 8.00$
 Existing material of Primary Member: Steel Strength, $f_s = f_{s_lower_bound} = 420.00$
 Concrete Elasticity, $E_c = 16281.278$
 Steel Elasticity, $E_s = 200000.00$
 #####
 Note: Especially for the calculation of γ for displacement ductility demand,
 the expected (mean value) strengths are used (7.5.1.3, ASCE 41-17) because bending is considered as
 Deformation-Controlled Action (Table C7-1, ASCE 41-17).
 Jacket
 New material: Concrete Strength, $f_c = f_{cm} = 18.00$
 New material: Steel Strength, $f_s = f_{sm} = 625.00$
 Existing Column
 Existing material: Concrete Strength, $f_c = f_{cm} = 12.00$
 Existing material: Steel Strength, $f_s = f_{sm} = 525.00$
 #####
 Max Height, $H_{max} = 750.00$
 Min Height, $H_{min} = 450.00$
 Max Width, $W_{max} = 950.00$
 Min Width, $W_{min} = 450.00$
 Eccentricity, $Ecc = 250.00$
 Jacket Thickness, $t_j = 100.00$
 Cover Thickness, $c = 25.00$
 Element Length, $L = 3000.00$
 Primary Member
 Ribbed Bars
 Ductile Steel
 With Detailing for Earthquake Resistance (including stirrups closed at 135°)
 Longitudinal Bars With Ends Lapped Starting at the End Sections
 Adequate Lap Length ($l_o/l_{ou,min} = l_b/l_d \geq 1$)
 No FRP Wrapping

Stepwise Properties

EDGE -A-
 Bending Moment, $M_a = -6.8503E+008$
 Shear Force, $V_a = -212214.235$
 EDGE -B-
 Bending Moment, $M_b = -4.7511E+008$
 Shear Force, $V_b = 212214.235$
 BOTH EDGES
 Axial Force, $F = -7.0682E+006$
 Longitudinal Reinforcement Area Distribution (in 2 divisions)
 -Tension: $A_{st} = 515.2212$
 -Compression: $A_{sc} = 6176.371$
 Longitudinal Reinforcement Area Distribution (in 3 divisions)
 -Tension: $A_{st,ten} = 1539.38$
 -Compression: $A_{st,com} = 1539.38$
 -Middle: $A_{st,mid} = 3612.832$
 Mean Diameter of Tension Reinforcement, $Db_{L,ten} = 16.57143$

Existing component: From table 7-7, ASCE 41_17: Final Shear Capacity $V_R = V_n = 1.0774E+006$
 V_n ((10.3), ASCE 41-17) = $k_n l V_{CoIO} = 1.3468E+006$
 $V_{CoI} = 1.3468E+006$
 $k_n l = 1.00$
 $displacement_ductility_demand = 0.16836581$

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

$\gamma = 1$ (normal-weight concrete)
 Mean concrete strength: $f'_c = (f'_c_jacket * Area_jacket + f'_c_core * Area_core) / Area_section = 10.13333$, but $f'_c^{0.5} \leq 8.3$ MPa (22.5.3.1, ACI 318-14)
 $M/Vd = 2.9458$

$\mu_u = 4.7511E+008$
 $V_u = 212214.235$
 $d = 0.8 \cdot h = 760.00$
 $N_u = 7.0682E+006$
 $A_g = 427500.00$
 From (11.5.4.8), ACI 318-14: $V_s = V_{s,jacket} + V_{s,core} = 980981.156$
 where:
 $V_{s,jacket} = V_{s,j1} + V_{s,j2} = 879645.943$
 $V_{s,j1} = 282743.339$ is calculated for section web jacket, with:
 $d = 360.00$
 $A_v = 157079.633$
 $f_y = 500.00$
 $s = 100.00$
 $V_{s,j1}$ is multiplied by $Col,j1 = 1.00$
 $s/d = 0.27777778$
 $V_{s,j2} = 596902.604$ is calculated for section flange jacket, with:
 $d = 760.00$
 $A_v = 157079.633$
 $f_y = 500.00$
 $s = 100.00$
 $V_{s,j2}$ is multiplied by $Col,j2 = 1.00$
 $s/d = 0.13157895$
 $V_{s,core} = V_{s,c1} + V_{s,c2} = 101335.213$
 $V_{s,c1} = 0.00$ is calculated for section web core, with:
 $d = 200.00$
 $A_v = 100530.965$
 $f_y = 420.00$
 $s = 250.00$
 $V_{s,c1}$ is multiplied by $Col,c1 = 0.00$
 $s/d = 1.25$
 $V_{s,c2} = 101335.213$ is calculated for section flange core, with:
 $d = 600.00$
 $A_v = 100530.965$
 $f_y = 420.00$
 $s = 250.00$
 $V_{s,c2}$ is multiplied by $Col,c2 = 1.00$
 $s/d = 0.41666667$
 $V_f ((11-3)-(11.4), ACI 440) = 0.00$
 From (11-11), ACI 440: $V_s + V_f \leq 723217.666$
 $bw = 450.00$

displacement_ductility_demand is calculated as δ_u / y

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

From analysis, chord rotation $\theta_r = 0.00041327$
 $y = (M_y \cdot L_s / 3) / E_{eff} = 0.00245461$ ((4.29), Biskinis Phd))
 $M_y = 1.5034E+009$
 $L_s = M/V$ (with $L_s > 0.1 \cdot L$ and $L_s < 2 \cdot L$) = 2238.811
 From table 10.5, ASCE 41_17: $E_{eff} = factor \cdot E_c \cdot I_g = 4.5707E+014$
 $factor = 0.70$
 $A_g = 562500.00$
 Mean concrete strength: $f'_c = (f'_{c,jacket} \cdot Area_{jacket} + f'_{c,core} \cdot Area_{core}) / Area_{section} = 15.20$
 $N = 7.0682E+006$
 $E_c \cdot I_g = E_{c,jacket} \cdot I_{g,jacket} + E_{c,core} \cdot I_{g,core} = 6.5295E+014$

Calculation of Yielding Moment M_y

Calculation of δ_u and M_y according to Annex 7 -

$y = \min(y_{ten}, y_{com})$
 $y_{ten} = 6.9719165E-006$

with $f_y = 601.9953$
 $d = 907.00$
 $y = 0.52400363$
 $A = 0.04516194$
 $B = 0.0373531$
 with $p_t = 0.0037716$
 $p_c = 0.0037716$
 $p_v = 0.00885172$
 $N = 7.0682E+006$
 $b = 450.00$
 $" = 0.04740904$
 $y_{comp} = 1.9344180E-006$
 with $f_c = 18.00$
 $E_c = 19940.411$
 $y = 0.92609018$
 $A = -0.0368952$
 $B = 0.0085861$
 with $E_s = 200000.00$

Calculation of ratio I_b/I_d

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

End Of Calculation of Shear Capacity for element: column JTC1 of floor 1

At local axis: 2

Integration Section: (b)

Calculation No. 14

column C1, Floor 1

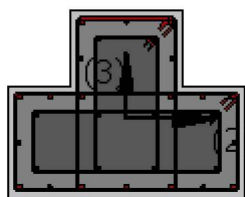
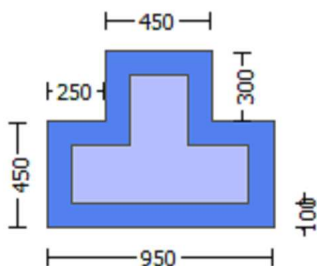
Limit State: Life Safety (data interpolation between analysis steps 50 and 51)

Analysis: Uniform +X

Check: Chord rotation capacity (ϕ)

Edge: End

Local Axis: (2)



```

Start Of Calculation of Shear Capacity ratio for element: column JTC1 of floor 1
At Shear local axis: 3
(Bending local axis: 2)
Section Type: rcjcs

Constant Properties
-----
Knowledge Factor,  $\phi = 0.80$ 
Mean strength values are used for both shear and moment calculations.
Consequently:
Jacket
New material of Primary Member: Concrete Strength,  $f_c = f_{cm} = 18.00$ 
New material of Primary Member: Steel Strength,  $f_s = f_{sm} = 625.00$ 
Concrete Elasticity,  $E_c = 19940.411$ 
Steel Elasticity,  $E_s = 200000.00$ 
Existing Column
Existing material of Primary Member: Concrete Strength,  $f_c = f_{cm} = 12.00$ 
Existing material of Primary Member: Steel Strength,  $f_s = f_{sm} = 525.00$ 
Concrete Elasticity,  $E_c = 16281.278$ 
Steel Elasticity,  $E_s = 200000.00$ 
#####
Note: Especially for the calculation of moment strengths,
the above steel re-bar strengths are multiplied by 1.25 according to R18.6.5, ACI 318-14
Jacket
New material: Steel Strength,  $f_s = 1.25 \cdot f_{sm} = 781.25$ 
Existing Column
Existing material: Steel Strength,  $f_s = 1.25 \cdot f_{sm} = 656.25$ 
#####
Max Height,  $H_{max} = 750.00$ 
Min Height,  $H_{min} = 450.00$ 
Max Width,  $W_{max} = 950.00$ 
Min Width,  $W_{min} = 450.00$ 
Eccentricity,  $E_{cc} = 250.00$ 
Jacket Thickness,  $t_j = 100.00$ 
Cover Thickness,  $c = 25.00$ 
Mean Confinement Factor overall section = 1.42131
Element Length,  $L = 3000.00$ 
Primary Member
Ribbed Bars
Ductile Steel
With Detailing for Earthquake Resistance (including stirrups closed at 135°)
Longitudinal Bars With Ends Lapped Starting at the End Sections
Adequate Lap Length ( $l_o/l_{ou,min} \geq 1$ )
No FRP Wrapping
-----

Stepwise Properties
-----
At local axis: 3
EDGE -A-
Shear Force,  $V_a = -227.4778$ 
EDGE -B-
Shear Force,  $V_b = 227.4778$ 
BOTH EDGES
Axial Force,  $F = -7.6839E+006$ 
Longitudinal Reinforcement Area Distribution (in 2 divisions)
-Tension:  $A_{st} = 314.1593$ 
-Compression:  $A_{sc} = 6176.371$ 
Longitudinal Reinforcement Area Distribution (in 3 divisions)
-Tension:  $A_{st,ten} = 1539.38$ 
-Compression:  $A_{st,com} = 2475.575$ 
-Middle:  $A_{st,mid} = 2676.637$ 
-----
-----

Calculation of Shear Capacity ratio ,  $V_e/V_r = 1.74773$ 

```

Member Controlled by Shear ($V_e/V_r > 1$)

Calculation of Shear Demand from fig. R18.6.5, ACI 318-14 $V_e = (M_{pr1} + M_{pr2})/l_n = 2.0410E+006$ with

$M_{pr1} = \text{Max}(\mu_{1+}, \mu_{1-}) = 3.0615E+009$

$\mu_{1+} = 3.0615E+009$, is the ultimate moment strength at the edge 1 of the member in the actual moment direction which is defined for the static loading combination

$\mu_{1-} = 8.9683E+008$, is the ultimate moment strength at the edge 1 of the member in the opposite moment direction which is defined for the static loading combination

$M_{pr2} = \text{Max}(\mu_{2+}, \mu_{2-}) = 3.0615E+009$

$\mu_{2+} = 3.0615E+009$, is the ultimate moment strength at the edge 2 of the member in the actual moment direction which is defined for the static loading combination

$\mu_{2-} = 8.9683E+008$, is the ultimate moment strength at the edge 2 of the member in the opposite moment direction which is defined for the static loading combination

Calculation of μ_{1+}

Calculation of ultimate curvature μ according to 4.1, Biskinis/Fardis 2013:

$\mu = 1.3930706E-005$

$\mu_u = 3.0615E+009$

with full section properties:

$b = 950.00$

$d = 707.00$

$d' = 43.00$

$v = 0.63557247$

$N = 7.6839E+006$

$f_c = 18.00$

α_1 (5A.5, TBDY) = 0.002

Final value of μ_u : $\mu_u^* = \text{shear_factor} * \text{Max}(\mu_u, \mu_c) = 0.01486841$

The Shear_factor is considered equal to 1 (pure moment strength)

From (5.4b), TBDY: $\mu_u = 0.01486841$

μ_{ue} (5.4c) = 0.08077545

α_{se} ((5.4d), TBDY) = $(\alpha_{se1} * A_{ext} + \alpha_{se2} * A_{int}) / A_{sec} = 0.53375773$

$\alpha_{se1} = \text{Max}(((A_{conf,max1} - A_{noConf1}) / A_{conf,max1}) * (A_{conf,min1} / A_{conf,max1}), 0) = 0.53375773$

The definitions of A_{noConf} , $A_{conf,min}$ and $A_{conf,max}$ are derived from generalization of the rectangular sections confinement, which is expressed by (5.4d).

The generalization is done according to Mander, J., Priestley, M., and Park, R. (1988)

"Theoretical Stress-Strain Model for Confined Concrete."

J. Struct. Eng., 10.1061/(ASCE)0733-9445(1988)114:8(1804), 1804-1826.

$A_{conf,max1} = 464100.00$ is the confined core area at levels of member with hoops and is calculated as the area of core enclosed by the center lines of the external perimeter hoops.

$A_{conf,min1} = 395025.00$ is the confined core area at midway between the levels of hoops and is calculated by reducing all the dimensions of the area $A_{conf,max1}$ by a length equal to half the clear spacing between external hoops.

$A_{noConf1} = 173066.667$ is the unconfined external core area which is equal to $b_i^2/6$ as defined at (A.2).

$\alpha_{se2} (> \alpha_{se1}) = \text{Max}(((A_{conf,max2} - A_{noConf2}) / A_{conf,max2}) * (A_{conf,min2} / A_{conf,max2}), 0) = 0.53375773$

The definitions of A_{noConf} , $A_{conf,min}$ and $A_{conf,max}$ are derived from generalization of the rectangular sections confinement, which is expressed by (5.4d).

The generalization is done according to Mander, J., Priestley, M., and Park, R. (1988)

"Theoretical Stress-Strain Model for Confined Concrete."

J. Struct. Eng., 10.1061/(ASCE)0733-9445(1988)114:8(1804), 1804-1826.

$A_{conf,max2} = 252164.00$ is the confined core area at levels of member with hoops and is calculated as the area of core enclosed by the center lines of the internal perimeter hoops.

$A_{conf,min2} = 111441.00$ is the confined core area at midway between the levels of hoops and is calculated by reducing all the dimensions of the area $A_{conf,max2}$ by a length equal to half the clear spacing between internal hoops.

$A_{noConf2} = 110709.333$ is the unconfined internal core area which is equal to $b_i^2/6$ as defined at (A.2).

$p_{sh,min} * F_{ywe} = \text{Min}(p_{sh,x} * F_{ywe}, p_{sh,y} * F_{ywe}) = 2.724$

$p_{sh,x} * F_{ywe} = p_{sh1} * F_{ywe1} + p_{sh2} * F_{ywe2} = 2.724$

p_{sh1} ((5.4d), TBDY) = $L_{stir1} * A_{stir1} / (A_{sec} * s_1) = 0.00301593$

L_{stir1} (Length of stirrups along Y) = 2160.00

A_{stir1} (stirrups area) = 78.53982

p_{sh2} (5.4d) = $L_{stir2} * A_{stir2} / (A_{sec} * s_2) = 0.00056047$

Lstir2 (Length of stirrups along Y) = 1568.00
Astir2 (stirrups area) = 50.26548

psh_y*Fywe = psh₁*Fywe₁+ps₂*Fywe₂ = 3.25416
psh₁ ((5.4d), TBDY) = Lstir₁*Astir₁/(Asec*s₁) = 0.00357443
Lstir₁ (Length of stirrups along X) = 2560.00
Astir₁ (stirrups area) = 78.53982
psh₂ ((5.4d), TBDY) = Lstir₂*Astir₂/(Asec*s₂) = 0.00070345
Lstir₂ (Length of stirrups along X) = 1968.00
Astir₂ (stirrups area) = 50.26548

Asec = 562500.00
s₁ = 100.00
s₂ = 250.00

fywe₁ = 781.25
fywe₂ = 656.25
fce = 18.00

From ((5.A5), TBDY), TBDY: cc = 0.00621307
c = confinement factor = 1.42131

y₁ = 0.0025
sh₁ = 0.008
ft₁ = 907.50
fy₁ = 756.25
su₁ = 0.032

using (30) in Biskinis/Fardis (2013) multiplied with shear_factor
and also multiplied by the shear_factor according to 15.7.1.4, with
Shear_factor = 1.00

lo/lou,min = lb/l_d = 1.00

su₁ = 0.4*esu_{1_nominal} ((5.5), TBDY) = 0.032

From table 5A.1, TBDY: esu_{1_nominal} = 0.08,

For calculation of esu_{1_nominal} and y₁, sh₁,ft₁,fy₁, it is considered
characteristic value fsy₁ = fs₁/1.2, from table 5.1, TBDY.

y₁, sh₁,ft₁,fy₁, are also multiplied by Min(1,1.25*(lb/l_d)^{2/3}), from 10.3.5, ASCE 41-17.

with fs₁ = (fs_{jacket}*Asl_{ten,jacket} + fs_{core}*Asl_{ten,core})/Asl_{ten} = 756.25

with Es₁ = (Es_{jacket}*Asl_{ten,jacket} + Es_{core}*Asl_{ten,core})/Asl_{ten} = 200000.00

y₂ = 0.0025
sh₂ = 0.008

ft₂ = 900.1904

fy₂ = 750.1586

su₂ = 0.032

using (30) in Biskinis/Fardis (2013) multiplied with shear_factor
and also multiplied by the shear_factor according to 15.7.1.4, with
Shear_factor = 1.00

lo/lou,min = lb/l_{b,min} = 1.00

su₂ = 0.4*esu_{2_nominal} ((5.5), TBDY) = 0.032

From table 5A.1, TBDY: esu_{2_nominal} = 0.08,

For calculation of esu_{2_nominal} and y₂, sh₂,ft₂,fy₂, it is considered
characteristic value fsy₂ = fs₂/1.2, from table 5.1, TBDY.

y₁, sh₁,ft₁,fy₁, are also multiplied by Min(1,1.25*(lb/l_d)^{2/3}), from 10.3.5, ASCE 41-17.

with fs₂ = (fs_{jacket}*Asl_{com,jacket} + fs_{core}*Asl_{com,core})/Asl_{com} = 750.1586

with Es₂ = (Es_{jacket}*Asl_{com,jacket} + Es_{core}*Asl_{com,core})/Asl_{com} = 200000.00

y_v = 0.0025
sh_v = 0.008

ft_v = 902.993

fy_v = 752.4941

suv = 0.032

using (30) in Biskinis/Fardis (2013) multiplied with shear_factor
and also multiplied by the shear_factor according to 15.7.1.4, with
Shear_factor = 1.00

lo/lou,min = lb/l_d = 1.00

suv = 0.4*esuv_{nominal} ((5.5), TBDY) = 0.032

From table 5A.1, TBDY: esuv_{nominal} = 0.08,

considering characteristic value fsy_v = fs_v/1.2, from table 5.1, TBDY

For calculation of esuv_{nominal} and y_v, sh_v,ft_v,fy_v, it is considered
characteristic value fsy_v = fs_v/1.2, from table 5.1, TBDY.

y₁, sh₁,ft₁,fy₁, are also multiplied by Min(1,1.25*(lb/l_d)^{2/3}), from 10.3.5, ASCE 41-17.

with fs_v = (fs_{jacket}*Asl_{mid,jacket} + fs_{mid}*Asl_{mid,core})/Asl_{mid} = 752.4941

```

with Esv = (Esjacket*Asl,mid,jacket + Esmid*Asl,mid,core)/Asl,mid = 200000.00
1 = Asl,ten/(b*d)*(fs1/fc) = 0.09629324
2 = Asl,com/(b*d)*(fs2/fc) = 0.15360794
v = Asl,mid/(b*d)*(fsv/fc) = 0.16660079
and confined core properties:
b = 890.00
d = 677.00
d' = 13.00
fcc (5A.2, TBDY) = 25.58352
cc (5A.5, TBDY) = 0.00621307
c = confinement factor = 1.42131
1 = Asl,ten/(b*d)*(fs1/fc) = 0.10733965
2 = Asl,com/(b*d)*(fs2/fc) = 0.17122928
v = Asl,mid/(b*d)*(fsv/fc) = 0.18571261
Case/Assumption: Unconfined full section - Steel rupture
' satisfies Eq. (4.3)
--->
v < vs,y2 - LHS eq.(4.5) is not satisfied
--->
v < vs,c - RHS eq.(4.5) is not satisfied
--->
Case/Assumption Rejected.
--->
New Case/Assumption: Unconfined full section - Spalling of concrete cover
' satisfies Eq. (4.4)
--->
v < sy1 - LHS eq.(4.7) is not satisfied
--->
v < vc,y1 - RHS eq.(4.6) is satisfied
--->
cu (4.10) = 0.58509201
MRc (4.17) = 2.3801E+009
--->
New Case/Assumption: Ultimate curvature of confined core after spalling of concrete cover
In expressions below, the following modifications have been made
- b, d, d' replaced by geometric parameters of the core: bo, do, d'o
- N1, 1, 2, v normalised to bo*do, instead of b*d
- - parameters of confined concrete, fcc, cc, used in lieu of fc, ecu
--->
Subcase: Rupture of tension steel
--->
v* < v*sy2 - LHS eq.(4.5) is not satisfied
--->
v* < v*sc - LHS eq.(4.5) is not satisfied
--->
Subcase rejected
--->
New Subcase: Failure of compression zone
--->
v* < v*cy2 - LHS eq.(4.6) is not satisfied
--->
v* < v*cy1 - RHS eq.(4.6) is not satisfied
--->
*cu (4.11) = 0.65878578
MRo (4.18) = 3.0615E+009
--->
u = cu (4.2) = 1.3930706E-005
Mu = MRo
-----

```

Calculation of ratio lb/d

Adequate Lap Length: lb/d >= 1

Calculation of Mu1-

Calculation of ultimate curvature ϕ_u according to 4.1, Biskinis/Fardis 2013:

$$u = 9.1773508E-006$$

$$\mu = 8.9683E+008$$

with full section properties:

$$b = 450.00$$

$$d = 707.00$$

$$d' = 43.00$$

$$v = 1.34176$$

$$N = 7.6839E+006$$

$$f_c = 18.00$$

$$\alpha (5A.5, \text{TBDY}) = 0.002$$

$$\text{Final value of } \phi_u: \phi_u^* = \text{shear_factor} * \text{Max}(\phi_u, \phi_c) = 0.01486841$$

The Shear_factor is considered equal to 1 (pure moment strength)

$$\text{From (5.4b), TBDY: } \phi_u = 0.01486841$$

$$\phi_{se} (5.4c) = 0.08077545$$

$$\phi_{ase} ((5.4d), \text{TBDY}) = (\phi_{ase1} * A_{ext} + \phi_{ase2} * A_{int}) / A_{sec} = 0.53375773$$

$$\phi_{ase1} = \text{Max}(((A_{conf,max1} - A_{noConf1}) / A_{conf,max1}) * (A_{conf,min1} / A_{conf,max1}), 0) = 0.53375773$$

The definitions of A_{noConf} , $A_{conf,min}$ and $A_{conf,max}$ are derived from generalization of the rectangular sections confinement, which is expressed by (5.4d).

The generalization is done according to Mander, J., Priestley, M., and Park, R. (1988)

"Theoretical Stress-Strain Model for Confined Concrete."

J. Struct. Eng., 10.1061/(ASCE)0733-9445(1988)114:8(1804), 1804-1826.

$A_{conf,max1} = 464100.00$ is the confined core area at levels of member with hoops and is calculated as the area of core enclosed by the center lines of the external perimeter hoops.

$A_{conf,min1} = 395025.00$ is the confined core area at midway between the levels of hoops and is calculated by reducing all the dimensions of the area $A_{conf,max1}$ by a length equal to half the clear spacing between external hoops.

$A_{noConf1} = 173066.667$ is the unconfined external core area which is equal to $b_i^2/6$ as defined at (A.2).

$$\phi_{ase2} (> \phi_{ase1}) = \text{Max}(((A_{conf,max2} - A_{noConf2}) / A_{conf,max2}) * (A_{conf,min2} / A_{conf,max2}), 0) = 0.53375773$$

The definitions of A_{noConf} , $A_{conf,min}$ and $A_{conf,max}$ are derived from generalization of the rectangular sections confinement, which is expressed by (5.4d).

The generalization is done according to Mander, J., Priestley, M., and Park, R. (1988)

"Theoretical Stress-Strain Model for Confined Concrete."

J. Struct. Eng., 10.1061/(ASCE)0733-9445(1988)114:8(1804), 1804-1826.

$A_{conf,max2} = 252164.00$ is the confined core area at levels of member with hoops and is calculated as the area of core enclosed by the center lines of the internal perimeter hoops.

$A_{conf,min2} = 111441.00$ is the confined core area at midway between the levels of hoops and is calculated by reducing all the dimensions of the area $A_{conf,max2}$ by a length equal to half the clear spacing between internal hoops.

$A_{noConf2} = 110709.333$ is the unconfined internal core area which is equal to $b_i^2/6$ as defined at (A.2).

$$\phi_{psh,min} * F_{ywe} = \text{Min}(\phi_{psh,x} * F_{ywe}, \phi_{psh,y} * F_{ywe}) = 2.724$$

$$\phi_{psh,x} * F_{ywe} = \phi_{psh1} * F_{ywe1} + \phi_{ps2} * F_{ywe2} = 2.724$$

$$\phi_{psh1} ((5.4d), \text{TBDY}) = L_{stir1} * A_{stir1} / (A_{sec} * s_1) = 0.00301593$$

$$L_{stir1} \text{ (Length of stirrups along Y)} = 2160.00$$

$$A_{stir1} \text{ (stirrups area)} = 78.53982$$

$$\phi_{psh2} (5.4d) = L_{stir2} * A_{stir2} / (A_{sec} * s_2) = 0.00056047$$

$$L_{stir2} \text{ (Length of stirrups along Y)} = 1568.00$$

$$A_{stir2} \text{ (stirrups area)} = 50.26548$$

$$\phi_{psh,y} * F_{ywe} = \phi_{psh1} * F_{ywe1} + \phi_{ps2} * F_{ywe2} = 3.25416$$

$$\phi_{psh1} ((5.4d), \text{TBDY}) = L_{stir1} * A_{stir1} / (A_{sec} * s_1) = 0.00357443$$

$$L_{stir1} \text{ (Length of stirrups along X)} = 2560.00$$

$$A_{stir1} \text{ (stirrups area)} = 78.53982$$

$$\phi_{psh2} ((5.4d), \text{TBDY}) = L_{stir2} * A_{stir2} / (A_{sec} * s_2) = 0.00070345$$

$$L_{stir2} \text{ (Length of stirrups along X)} = 1968.00$$

$$A_{stir2} \text{ (stirrups area)} = 50.26548$$

$$A_{sec} = 562500.00$$

$$s_1 = 100.00$$

$$s_2 = 250.00$$

$$f_{ywe1} = 781.25$$

$$f_{ywe2} = 656.25$$

$f_{ce} = 18.00$
 From ((5.A.5), TBDY), TBDY: $cc = 0.00621307$
 $c = \text{confinement factor} = 1.42131$
 $y_1 = 0.0025$
 $sh_1 = 0.008$
 $ft_1 = 900.1904$
 $fy_1 = 750.1586$
 $su_1 = 0.032$
 using (30) in Biskinis/Fardis (2013) multiplied with shear_factor
 and also multiplied by the shear_factor according to 15.7.1.4, with
 Shear_factor = 1.00
 $lo/lou, min = lb/ld = 1.00$
 $su_1 = 0.4 * esu_1, nominal ((5.5), TBDY) = 0.032$
 From table 5A.1, TBDY: $esu_1, nominal = 0.08$,
 For calculation of $esu_1, nominal$ and y_1, sh_1, ft_1, fy_1 , it is considered
 characteristic value $fsy_1 = fs_1/1.2$, from table 5.1, TBDY.
 y_1, sh_1, ft_1, fy_1 , are also multiplied by $Min(1, 1.25 * (lb/ld)^{2/3})$, from 10.3.5, ASCE 41-17.
 with $fs_1 = (fs, jacket * Asl, ten, jacket + fs, core * Asl, ten, core) / Asl, ten = 750.1586$
 with $Es_1 = (Es, jacket * Asl, ten, jacket + Es, core * Asl, ten, core) / Asl, ten = 200000.00$
 $y_2 = 0.0025$
 $sh_2 = 0.008$
 $ft_2 = 907.50$
 $fy_2 = 756.25$
 $su_2 = 0.032$
 using (30) in Biskinis/Fardis (2013) multiplied with shear_factor
 and also multiplied by the shear_factor according to 15.7.1.4, with
 Shear_factor = 1.00
 $lo/lou, min = lb/lb, min = 1.00$
 $su_2 = 0.4 * esu_2, nominal ((5.5), TBDY) = 0.032$
 From table 5A.1, TBDY: $esu_2, nominal = 0.08$,
 For calculation of $esu_2, nominal$ and y_2, sh_2, ft_2, fy_2 , it is considered
 characteristic value $fsy_2 = fs_2/1.2$, from table 5.1, TBDY.
 y_1, sh_1, ft_1, fy_1 , are also multiplied by $Min(1, 1.25 * (lb/ld)^{2/3})$, from 10.3.5, ASCE 41-17.
 with $fs_2 = (fs, jacket * Asl, com, jacket + fs, core * Asl, com, core) / Asl, com = 756.25$
 with $Es_2 = (Es, jacket * Asl, com, jacket + Es, core * Asl, com, core) / Asl, com = 200000.00$
 $y_v = 0.0025$
 $sh_v = 0.008$
 $ft_v = 902.993$
 $fy_v = 752.4941$
 $suv = 0.032$
 using (30) in Biskinis/Fardis (2013) multiplied with shear_factor
 and also multiplied by the shear_factor according to 15.7.1.4, with
 Shear_factor = 1.00
 $lo/lou, min = lb/ld = 1.00$
 $suv = 0.4 * esuv, nominal ((5.5), TBDY) = 0.032$
 From table 5A.1, TBDY: $esuv, nominal = 0.08$,
 considering characteristic value $fsyv = fsv/1.2$, from table 5.1, TBDY
 For calculation of $esuv, nominal$ and y_v, sh_v, ft_v, fy_v , it is considered
 characteristic value $fsyv = fsv/1.2$, from table 5.1, TBDY.
 y_1, sh_1, ft_1, fy_1 , are also multiplied by $Min(1, 1.25 * (lb/ld)^{2/3})$, from 10.3.5, ASCE 41-17.
 with $fsv = (fs, jacket * Asl, mid, jacket + fs, mid * Asl, mid, core) / Asl, mid = 752.4941$
 with $Es_v = (Es, jacket * Asl, mid, jacket + Es, mid * Asl, mid, core) / Asl, mid = 200000.00$
 $1 = Asl, ten / (b * d) * (fs_1 / f_c) = 0.32428344$
 $2 = Asl, com / (b * d) * (fs_2 / f_c) = 0.20328574$
 $v = Asl, mid / (b * d) * (fsv / f_c) = 0.35171278$
 and confined core properties:
 $b = 390.00$
 $d = 677.00$
 $d' = 13.00$
 $f_{cc} (5A.2, TBDY) = 25.58352$
 $cc (5A.5, TBDY) = 0.00621307$
 $c = \text{confinement factor} = 1.42131$
 $1 = Asl, ten / (b * d) * (fs_1 / f_c) = 0.39075399$
 $2 = Asl, com / (b * d) * (fs_2 / f_c) = 0.24495458$
 $v = Asl, mid / (b * d) * (fsv / f_c) = 0.42380571$
 Case/Assumption: Unconfined full section - Steel rupture
 ' satisfies Eq. (4.3)


```

---->
v < vs,y2 - LHS eq.(4.5) is not satisfied
---->
v < vs,c - RHS eq.(4.5) is not satisfied
---->
Case/Assumption Rejected.
---->
New Case/Assumption: Unconfined full section - Spalling of concrete cover
'satisfies Eq. (4.4)
---->
v < s,y1 - LHS eq.(4.7) is not satisfied
---->
v < vc,y1 - RHS eq.(4.6) is not satisfied
---->
cu (4.11) = 0.95920575
MRc (4.18) = 7.7321E+008
---->
New Case/Assumption: Ultimate curvature of confined core after spalling of concrete cover
In expressions below, the following modifications have been made
- b, d, d' replaced by geometric parameters of the core: bo, do, d'o
- N, 1, 2, v normalised to bo*do, instead of b*d
- parameters of confined concrete, fcc, cc, used in lieu of fc, ecu
---->
Subcase: Rupture of tension steel
---->
v* < v*s,y2 - LHS eq.(4.5) is not satisfied
---->
v* < v*s,c - LHS eq.(4.5) is not satisfied
---->
Subcase rejected
---->
New Subcase: Failure of compression zone
---->
v* < v*c,y2 - LHS eq.(4.6) is not satisfied
---->
v* < v*c,y1 - RHS eq.(4.6) is not satisfied
---->
*cu (4.11) = 1.00
MRo (4.18) = 8.9683E+008
---->
u = cu (4.2) = 9.1773508E-006
Mu = MRo

```

Calculation of ratio lb/d

Adequate Lap Length: lb/d >= 1

Calculation of Mu2+

Calculation of ultimate curvature u according to 4.1, Biskinis/Fardis 2013:

u = 1.3930706E-005

Mu = 3.0615E+009

with full section properties:

b = 950.00

d = 707.00

d' = 43.00

v = 0.63557247

N = 7.6839E+006

fc = 18.00

co (5A.5, TBDY) = 0.002

Final value of cu: cu* = shear_factor * Max(cu, cc) = 0.01486841

The Shear_factor is considered equal to 1 (pure moment strength)

From (5.4b), TBDY: $c_u = 0.01486841$

w_e (5.4c) = 0.08077545

a_{se} ((5.4d), TBDY) = $(a_{se1} * A_{ext} + a_{se2} * A_{int}) / A_{sec} = 0.53375773$

$a_{se1} = \text{Max}(((A_{conf,max1} - A_{noConf1}) / A_{conf,max1}) * (A_{conf,min1} / A_{conf,max1}), 0) = 0.53375773$

The definitions of A_{noConf} , $A_{conf,min}$ and $A_{conf,max}$ are derived from generalization of the rectangular sections confinement, which is expressed by (5.4d).

The generalization is done according to Mander, J., Priestley, M., and Park, R. (1988)

"Theoretical Stress-Strain Model for Confined Concrete."

J. Struct. Eng., 10.1061/(ASCE)0733-9445(1988)114:8(1804), 1804-1826.

$A_{conf,max1} = 464100.00$ is the confined core area at levels of member with hoops and is calculated as the area of core enclosed by the center lines of the external perimeter hoops.

$A_{conf,min1} = 395025.00$ is the confined core area at midway between the levels of hoops and is calculated by reducing all the dimensions of the area $A_{conf,max1}$ by a length equal to half the clear spacing between external hoops.

$A_{noConf1} = 173066.667$ is the unconfined external core area which is equal to $b_i^2/6$ as defined at (A.2).

$a_{se2} (>= a_{se1}) = \text{Max}(((A_{conf,max2} - A_{noConf2}) / A_{conf,max2}) * (A_{conf,min2} / A_{conf,max2}), 0) = 0.53375773$

The definitions of A_{noConf} , $A_{conf,min}$ and $A_{conf,max}$ are derived from generalization of the rectangular sections confinement, which is expressed by (5.4d).

The generalization is done according to Mander, J., Priestley, M., and Park, R. (1988)

"Theoretical Stress-Strain Model for Confined Concrete."

J. Struct. Eng., 10.1061/(ASCE)0733-9445(1988)114:8(1804), 1804-1826.

$A_{conf,max2} = 252164.00$ is the confined core area at levels of member with hoops and is calculated as the area of core enclosed by the center lines of the internal perimeter hoops.

$A_{conf,min2} = 111441.00$ is the confined core area at midway between the levels of hoops and is calculated by reducing all the dimensions of the area $A_{conf,max2}$ by a length equal to half the clear spacing between internal hoops.

$A_{noConf2} = 110709.333$ is the unconfined internal core area which is equal to $b_i^2/6$ as defined at (A.2).

$p_{sh,min} * F_{ywe} = \text{Min}(p_{sh,x} * F_{ywe}, p_{sh,y} * F_{ywe}) = 2.724$

$p_{sh,x} * F_{ywe} = p_{sh1} * F_{ywe1} + p_{s2} * F_{ywe2} = 2.724$

p_{sh1} ((5.4d), TBDY) = $L_{stir1} * A_{stir1} / (A_{sec} * s_1) = 0.00301593$

L_{stir1} (Length of stirrups along Y) = 2160.00

A_{stir1} (stirrups area) = 78.53982

p_{sh2} (5.4d) = $L_{stir2} * A_{stir2} / (A_{sec} * s_2) = 0.00056047$

L_{stir2} (Length of stirrups along Y) = 1568.00

A_{stir2} (stirrups area) = 50.26548

$p_{sh,y} * F_{ywe} = p_{sh1} * F_{ywe1} + p_{s2} * F_{ywe2} = 3.25416$

p_{sh1} ((5.4d), TBDY) = $L_{stir1} * A_{stir1} / (A_{sec} * s_1) = 0.00357443$

L_{stir1} (Length of stirrups along X) = 2560.00

A_{stir1} (stirrups area) = 78.53982

p_{sh2} ((5.4d), TBDY) = $L_{stir2} * A_{stir2} / (A_{sec} * s_2) = 0.00070345$

L_{stir2} (Length of stirrups along X) = 1968.00

A_{stir2} (stirrups area) = 50.26548

$A_{sec} = 562500.00$

$s_1 = 100.00$

$s_2 = 250.00$

$f_{ywe1} = 781.25$

$f_{ywe2} = 656.25$

$f_{ce} = 18.00$

From ((5.A5), TBDY), TBDY: $c_c = 0.00621307$

c = confinement factor = 1.42131

$y_1 = 0.0025$

$sh_1 = 0.008$

$ft_1 = 907.50$

$fy_1 = 756.25$

$su_1 = 0.032$

using (30) in Biskinis/Fardis (2013) multiplied with shear_factor

and also multiplied by the shear_factor according to 15.7.1.4, with

Shear_factor = 1.00

$l_o/l_{ou,min} = l_b/l_d = 1.00$

$su_1 = 0.4 * esu_{1,nominal}$ ((5.5), TBDY) = 0.032

From table 5A.1, TBDY: $esu_{1,nominal} = 0.08$,

For calculation of $esu_{1,nominal}$ and y_1 , sh_1 , ft_1 , fy_1 , it is considered characteristic value $fsy_1 = fs_1/1.2$, from table 5.1, TBDY.

$y1, sh1, ft1, fy1$, are also multiplied by $\text{Min}(1, 1.25 \cdot (lb/ld)^{2/3})$, from 10.3.5, ASCE 41-17.
 with $fs1 = (fs_{jacket} \cdot Asl_{ten,jacket} + fs_{core} \cdot Asl_{ten,core}) / Asl_{ten} = 756.25$
 with $Es1 = (Es_{jacket} \cdot Asl_{ten,jacket} + Es_{core} \cdot Asl_{ten,core}) / Asl_{ten} = 200000.00$
 $y2 = 0.0025$
 $sh2 = 0.008$
 $ft2 = 900.1904$
 $fy2 = 750.1586$
 $su2 = 0.032$
 using (30) in Biskinis/Fardis (2013) multiplied with shear_factor
 and also multiplied by the shear_factor according to 15.7.1.4, with
 Shear_factor = 1.00
 $lo/lou, min = lb/lb, min = 1.00$
 $su2 = 0.4 \cdot esu2_{nominal} ((5.5), TBDY) = 0.032$
 From table 5A.1, TBDY: $esu2_{nominal} = 0.08$,
 For calculation of $esu2_{nominal}$ and $y2, sh2, ft2, fy2$, it is considered
 characteristic value $fsy2 = fs2/1.2$, from table 5.1, TBDY.
 $y1, sh1, ft1, fy1$, are also multiplied by $\text{Min}(1, 1.25 \cdot (lb/ld)^{2/3})$, from 10.3.5, ASCE 41-17.
 with $fs2 = (fs_{jacket} \cdot Asl_{com,jacket} + fs_{core} \cdot Asl_{com,core}) / Asl_{com} = 750.1586$
 with $Es2 = (Es_{jacket} \cdot Asl_{com,jacket} + Es_{core} \cdot Asl_{com,core}) / Asl_{com} = 200000.00$
 $yv = 0.0025$
 $shv = 0.008$
 $ftv = 902.993$
 $fyv = 752.4941$
 $suv = 0.032$
 using (30) in Biskinis/Fardis (2013) multiplied with shear_factor
 and also multiplied by the shear_factor according to 15.7.1.4, with
 Shear_factor = 1.00
 $lo/lou, min = lb/ld = 1.00$
 $suv = 0.4 \cdot esuv_{nominal} ((5.5), TBDY) = 0.032$
 From table 5A.1, TBDY: $esuv_{nominal} = 0.08$,
 considering characteristic value $fsyv = fsv/1.2$, from table 5.1, TBDY
 For calculation of $esuv_{nominal}$ and yv, shv, ftv, fyv , it is considered
 characteristic value $fsyv = fsv/1.2$, from table 5.1, TBDY.
 $y1, sh1, ft1, fy1$, are also multiplied by $\text{Min}(1, 1.25 \cdot (lb/ld)^{2/3})$, from 10.3.5, ASCE 41-17.
 with $fsv = (fs_{jacket} \cdot Asl_{mid,jacket} + fs_{mid} \cdot Asl_{mid,core}) / Asl_{mid} = 752.4941$
 with $Es_v = (Es_{jacket} \cdot Asl_{mid,jacket} + Es_{mid} \cdot Asl_{mid,core}) / Asl_{mid} = 200000.00$
 $1 = Asl_{ten} / (b \cdot d) \cdot (fs1 / fc) = 0.09629324$
 $2 = Asl_{com} / (b \cdot d) \cdot (fs2 / fc) = 0.15360794$
 $v = Asl_{mid} / (b \cdot d) \cdot (fsv / fc) = 0.16660079$
 and confined core properties:
 $b = 890.00$
 $d = 677.00$
 $d' = 13.00$
 $fcc (5A.2, TBDY) = 25.58352$
 $cc (5A.5, TBDY) = 0.00621307$
 $c = \text{confinement factor} = 1.42131$
 $1 = Asl_{ten} / (b \cdot d) \cdot (fs1 / fc) = 0.10733965$
 $2 = Asl_{com} / (b \cdot d) \cdot (fs2 / fc) = 0.17122928$
 $v = Asl_{mid} / (b \cdot d) \cdot (fsv / fc) = 0.18571261$
 Case/Assumption: Unconfined full section - Steel rupture
 ' satisfies Eq. (4.3)
 --->
 $v < v_{s,y2}$ - LHS eq.(4.5) is not satisfied
 --->
 $v < v_{s,c}$ - RHS eq.(4.5) is not satisfied
 --->
 Case/Assumption Rejected.
 --->
 New Case/Assumption: Unconfined full section - Spalling of concrete cover
 ' satisfies Eq. (4.4)
 --->
 $v < s_{y1}$ - LHS eq.(4.7) is not satisfied
 --->
 $v < v_{c,y1}$ - RHS eq.(4.6) is satisfied
 --->
 $cu (4.10) = 0.58509201$
 $MRC (4.17) = 2.3801E+009$

```

---->
New Case/Assumption: Ultimate curvature of confined core after spalling of concrete cover
In expressions below, the following modifications have been made
- b, d, d' replaced by geometric parameters of the core: bo, do, d'o
- N, 1, 2, v normalised to bo*do, instead of b*d
- - parameters of confined concrete, fcc, cc, used in lieu of fc, ec
---->
Subcase: Rupture of tension steel
---->
v* < v*s,y2 - LHS eq.(4.5) is not satisfied
---->
v* < v*s,c - LHS eq.(4.5) is not satisfied
---->
Subcase rejected
---->
New Subcase: Failure of compression zone
---->
v* < v*c,y2 - LHS eq.(4.6) is not satisfied
---->
v* < v*c,y1 - RHS eq.(4.6) is not satisfied
---->
*cu (4.11) = 0.65878578
MRo (4.18) = 3.0615E+009
---->
u = cu (4.2) = 1.3930706E-005
Mu = MRo
-----

Calculation of ratio lb/d
-----
Adequate Lap Length: lb/d >= 1
-----
-----

Calculation of Mu2-
-----
-----

Calculation of ultimate curvature u according to 4.1, Biskinis/Fardis 2013:
u = 9.1773508E-006
Mu = 8.9683E+008
-----

with full section properties:
b = 450.00
d = 707.00
d' = 43.00
v = 1.34176
N = 7.6839E+006
fc = 18.00
co (5A.5, TBDY) = 0.002
Final value of cu: cu* = shear_factor * Max( cu, cc) = 0.01486841
The Shear_factor is considered equal to 1 (pure moment strength)
From (5.4b), TBDY: cu = 0.01486841
we (5.4c) = 0.08077545
ase ((5.4d), TBDY) = (ase1*Aext+ase2*Aint)/Asec = 0.53375773
ase1 = Max(((Aconf,max1-AnoConf1)/Aconf,max1)*(Aconf,min1/Aconf,max1),0) = 0.53375773
The definitions of AnoConf, Aconf,min and Aconf,max are derived from generalization
of the rectangular sections confinement, which is expressed by (5.4d).
The generalization is done according to Mander, J., Priestley, M., and Park, R. (1988)
"Theoretical Stress-Strain Model for Confined Concrete."
J. Struct. Eng., 10.1061/(ASCE)0733-9445(1988)114:8(1804), 1804-1826.
Aconf,max1 = 464100.00 is the confined core area at levels of member with hoops and
is calculated as the area of core enclosed by the center lines of the external perimeter hoops.
Aconf,min1 = 395025.00 is the confined core area at midway between the levels of hoops and
is calculated by reducing all the dimensions of the area Aconf,max1 by a length
equal to half the clear spacing between external hoops.
AnoConf1 = 173066.667 is the unconfined external core area which is equal to bi2/6 as defined at (A.2).

```

$$ase2 (>=ase1) = \text{Max}(((A_{conf,max2}-A_{noConf2})/A_{conf,max2}) * (A_{conf,min2}/A_{conf,max2}), 0) = 0.53375773$$

The definitions of A_{noConf} , $A_{conf,min}$ and $A_{conf,max}$ are derived from generalization of the rectangular sections confinement, which is expressed by (5.4d).

The generalization is done according to Mander, J., Priestley, M., and Park, R. (1988) "Theoretical Stress-Strain Model for Confined Concrete."

J. Struct. Eng., 10.1061/(ASCE)0733-9445(1988)114:8(1804), 1804-1826.

$A_{conf,max2} = 252164.00$ is the confined core area at levels of member with hoops and is calculated as the area of core enclosed by the center lines of the internal perimeter hoops.

$A_{conf,min2} = 111441.00$ is the confined core area at midway between the levels of hoops and is calculated by reducing all the dimensions of the area $A_{conf,max2}$ by a length equal to half the clear spacing between internal hoops.

$A_{noConf2} = 110709.333$ is the unconfined internal core area which is equal to $b_i^2/6$ as defined at (A.2).
 $psh_{min} * F_{ywe} = \text{Min}(psh_x * F_{ywe}, psh_y * F_{ywe}) = 2.724$

$$psh_x * F_{ywe} = psh1 * F_{ywe1} + ps2 * F_{ywe2} = 2.724$$

$$psh1 \text{ ((5.4d), TBDY)} = L_{stir1} * A_{stir1} / (A_{sec} * s1) = 0.00301593$$

L_{stir1} (Length of stirrups along Y) = 2160.00

A_{stir1} (stirrups area) = 78.53982

$$psh2 \text{ (5.4d)} = L_{stir2} * A_{stir2} / (A_{sec} * s2) = 0.00056047$$

L_{stir2} (Length of stirrups along Y) = 1568.00

A_{stir2} (stirrups area) = 50.26548

$$psh_y * F_{ywe} = psh1 * F_{ywe1} + ps2 * F_{ywe2} = 3.25416$$

$$psh1 \text{ ((5.4d), TBDY)} = L_{stir1} * A_{stir1} / (A_{sec} * s1) = 0.00357443$$

L_{stir1} (Length of stirrups along X) = 2560.00

A_{stir1} (stirrups area) = 78.53982

$$psh2 \text{ ((5.4d), TBDY)} = L_{stir2} * A_{stir2} / (A_{sec} * s2) = 0.00070345$$

L_{stir2} (Length of stirrups along X) = 1968.00

A_{stir2} (stirrups area) = 50.26548

$$A_{sec} = 562500.00$$

$$s1 = 100.00$$

$$s2 = 250.00$$

$$f_{ywe1} = 781.25$$

$$f_{ywe2} = 656.25$$

$$f_{ce} = 18.00$$

From ((5.A5), TBDY), TBDY: $cc = 0.00621307$

c = confinement factor = 1.42131

$$y1 = 0.0025$$

$$sh1 = 0.008$$

$$ft1 = 900.1904$$

$$fy1 = 750.1586$$

$$su1 = 0.032$$

using (30) in Biskinis/Fardis (2013) multiplied with shear_factor and also multiplied by the shear_factor according to 15.7.1.4, with Shear_factor = 1.00

$$l_o/l_{ou,min} = l_b/l_d = 1.00$$

$$su1 = 0.4 * esu1_{nominal} \text{ ((5.5), TBDY)} = 0.032$$

From table 5A.1, TBDY: $esu1_{nominal} = 0.08$,

For calculation of $esu1_{nominal}$ and $y1, sh1, ft1, fy1$, it is considered characteristic value $fsy1 = fs1/1.2$, from table 5.1, TBDY.

$y1, sh1, ft1, fy1$, are also multiplied by $\text{Min}(1, 1.25 * (l_b/l_d)^{2/3})$, from 10.3.5, ASCE 41-17.

$$\text{with } fs1 = (f_{s,jacket} * A_{sl,ten,jacket} + f_{s,core} * A_{sl,ten,core}) / A_{sl,ten} = 750.1586$$

$$\text{with } Es1 = (E_{s,jacket} * A_{sl,ten,jacket} + E_{s,core} * A_{sl,ten,core}) / A_{sl,ten} = 200000.00$$

$$y2 = 0.0025$$

$$sh2 = 0.008$$

$$ft2 = 907.50$$

$$fy2 = 756.25$$

$$su2 = 0.032$$

using (30) in Biskinis/Fardis (2013) multiplied with shear_factor and also multiplied by the shear_factor according to 15.7.1.4, with Shear_factor = 1.00

$$l_o/l_{ou,min} = l_b/l_{b,min} = 1.00$$

$$su2 = 0.4 * esu2_{nominal} \text{ ((5.5), TBDY)} = 0.032$$

From table 5A.1, TBDY: $esu2_{nominal} = 0.08$,

For calculation of $esu2_{nominal}$ and $y2, sh2, ft2, fy2$, it is considered characteristic value $fsy2 = fs2/1.2$, from table 5.1, TBDY.

$y1, sh1, ft1, fy1$, are also multiplied by $\text{Min}(1, 1.25 \cdot (lb/ld)^{2/3})$, from 10.3.5, ASCE 41-17.
 with $fs2 = (fs_{jacket} \cdot Asl_{com,jacket} + fs_{core} \cdot Asl_{com,core}) / Asl_{com} = 756.25$
 with $Es2 = (Es_{jacket} \cdot Asl_{com,jacket} + Es_{core} \cdot Asl_{com,core}) / Asl_{com} = 200000.00$
 $yv = 0.0025$
 $shv = 0.008$
 $ftv = 902.993$
 $fyv = 752.4941$
 $suv = 0.032$
 using (30) in Biskinis/Fardis (2013) multiplied with shear_factor
 and also multiplied by the shear_factor according to 15.7.1.4, with
 Shear_factor = 1.00
 $lo/lou, min = lb/ld = 1.00$
 $suv = 0.4 \cdot esuv_{nominal} ((5.5), TBDY) = 0.032$
 From table 5A.1, TBDY: $esuv_{nominal} = 0.08$,
 considering characteristic value $fsyv = fsv/1.2$, from table 5.1, TBDY
 For calculation of $esuv_{nominal}$ and yv, shv, ftv, fyv , it is considered
 characteristic value $fsyv = fsv/1.2$, from table 5.1, TBDY.
 $y1, sh1, ft1, fy1$, are also multiplied by $\text{Min}(1, 1.25 \cdot (lb/ld)^{2/3})$, from 10.3.5, ASCE 41-17.
 with $fsv = (fs_{jacket} \cdot Asl_{mid,jacket} + fs_{mid} \cdot Asl_{mid,core}) / Asl_{mid} = 752.4941$
 with $Esv = (Es_{jacket} \cdot Asl_{mid,jacket} + Es_{mid} \cdot Asl_{mid,core}) / Asl_{mid} = 200000.00$
 $1 = Asl_{ten} / (b \cdot d) \cdot (fs1 / fc) = 0.32428344$
 $2 = Asl_{com} / (b \cdot d) \cdot (fs2 / fc) = 0.20328574$
 $v = Asl_{mid} / (b \cdot d) \cdot (fsv / fc) = 0.35171278$
 and confined core properties:
 $b = 390.00$
 $d = 677.00$
 $d' = 13.00$
 $fcc (5A.2, TBDY) = 25.58352$
 $cc (5A.5, TBDY) = 0.00621307$
 $c = \text{confinement factor} = 1.42131$
 $1 = Asl_{ten} / (b \cdot d) \cdot (fs1 / fc) = 0.39075399$
 $2 = Asl_{com} / (b \cdot d) \cdot (fs2 / fc) = 0.24495458$
 $v = Asl_{mid} / (b \cdot d) \cdot (fsv / fc) = 0.42380571$
 Case/Assumption: Unconfined full section - Steel rupture
 ' satisfies Eq. (4.3)
 --->
 $v < vs,y2$ - LHS eq.(4.5) is not satisfied
 --->
 $v < vs,c$ - RHS eq.(4.5) is not satisfied
 --->
 Case/Assumption Rejected.
 --->
 New Case/Assumption: Unconfined full section - Spalling of concrete cover
 ' satisfies Eq. (4.4)
 --->
 $v < s,y1$ - LHS eq.(4.7) is not satisfied
 --->
 $v < vc,y1$ - RHS eq.(4.6) is not satisfied
 --->
 $cu (4.11) = 0.95920575$
 $MRC (4.18) = 7.7321E+008$
 --->
 New Case/Assumption: Ultimate curvature of confined core after spalling of concrete cover
 In expressions below, the following modifications have been made
 - b, d, d' replaced by geometric parameters of the core: $bo, do, d'o$
 - $N, 1, 2, v$ normalised to $bo \cdot do$, instead of $b \cdot d$
 - parameters of confined concrete, fcc, cc , used in lieu of fc, ecu
 --->
 Subcase: Rupture of tension steel
 --->
 $v^* < v^*s,y2$ - LHS eq.(4.5) is not satisfied
 --->
 $v^* < v^*s,c$ - LHS eq.(4.5) is not satisfied
 --->
 Subcase rejected
 --->
 New Subcase: Failure of compression zone

```

---->
v* < v*c,y2 - LHS eq.(4.6) is not satisfied
---->
v* < v*c,y1 - RHS eq.(4.6) is not satisfied
---->
*cu (4.11) = 1.00
MRo (4.18) = 8.9683E+008
---->
u = cu (4.2) = 9.1773508E-006
Mu = MRo

```

Calculation of ratio lb/d

Adequate Lap Length: lb/d >= 1

Calculation of Shear Strength $V_r = \text{Min}(V_{r1}, V_{r2}) = 1.1678E+006$

Calculation of Shear Strength at edge 1, $V_{r1} = 1.1678E+006$

$V_{r1} = V_{Col} ((10.3), \text{ASCE } 41-17) = k_{nl} * V_{Col0}$

$V_{Col0} = 1.1678E+006$

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

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

= 1 (normal-weight concrete)

Mean concrete strength: $f_c' = (f_c'_{\text{jacket}} * \text{Area}_{\text{jacket}} + f_c'_{\text{core}} * \text{Area}_{\text{core}}) / \text{Area}_{\text{section}} = 15.20$, but $f_c'^{0.5} \leq 8.3$ MPa (22.5.3.1, ACI 318-14)

$M/Vd = 4.00$

$\mu_u = 7.6104E+008$

$V_u = 227.4778$

$d = 0.8 * h = 600.00$

$N_u = 7.6839E+006$

$A_g = 337500.00$

From (11.5.4.8), ACI 318-14: $V_s = V_{s,jacket} + V_{s,core} = 1.0354E+006$

where:

$V_{s,jacket} = V_{sj1} + V_{sj2} = 942477.796$

$V_{sj1} = 589048.623$ is calculated for section web jacket, with:

$d = 600.00$

$A_v = 157079.633$

$f_y = 625.00$

$s = 100.00$

V_{sj1} is multiplied by $Col,j1 = 1.00$

$s/d = 0.16666667$

$V_{sj2} = 353429.174$ is calculated for section flange jacket, with:

$d = 360.00$

$A_v = 157079.633$

$f_y = 625.00$

$s = 100.00$

V_{sj2} is multiplied by $Col,j2 = 1.00$

$s/d = 0.27777778$

$V_{s,core} = V_{s,c1} + V_{s,c2} = 92890.612$

$V_{s,c1} = 92890.612$ is calculated for section web core, with:

$d = 440.00$

$A_v = 100530.965$

$f_y = 525.00$

$s = 250.00$

$V_{s,c1}$ is multiplied by $Col,c1 = 1.00$

$s/d = 0.56818182$

$V_{s,c2} = 0.00$ is calculated for section flange core, with:

$d = 200.00$

$A_v = 100530.965$

$f_y = 525.00$

$s = 250.00$
 $V_{s,c2}$ is multiplied by $Col,c2 = 0.00$
 $s/d = 1.25$
 $V_f ((11-3)-(11.4), ACI 440) = 0.00$
 From (11-11), ACI 440: $V_s + V_f \leq 699281.943$
 $bw = 450.00$

Calculation of Shear Strength at edge 2, $V_{r2} = 1.1678E+006$
 $V_{r2} = V_{Col} ((10.3), ASCE 41-17) = knl * V_{Col0}$
 $V_{Col0} = 1.1678E+006$
 $knl = 1$ (zero step-static loading)

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

$= 1$ (normal-weight concrete)
 Mean concrete strength: $f'_c = (f'_{c,jacket} * Area_{jacket} + f'_{c,core} * Area_{core}) / Area_{section} = 15.20$, but $f'_c^{0.5} \leq 8.3$ MPa (22.5.3.1, ACI 318-14)
 $M/Vd = 4.00$
 $\mu_u = 6.8991E+008$
 $V_u = 227.4778$
 $d = 0.8 * h = 600.00$
 $N_u = 7.6839E+006$
 $A_g = 337500.00$
 From (11.5.4.8), ACI 318-14: $V_s = V_{s,jacket} + V_{s,core} = 1.0354E+006$
 where:
 $V_{s,jacket} = V_{s,j1} + V_{s,j2} = 942477.796$
 $V_{s,j1} = 589048.623$ is calculated for section web jacket, with:
 $d = 600.00$
 $A_v = 157079.633$
 $f_y = 625.00$
 $s = 100.00$
 $V_{s,j1}$ is multiplied by $Col,j1 = 1.00$
 $s/d = 0.16666667$
 $V_{s,j2} = 353429.174$ is calculated for section flange jacket, with:
 $d = 360.00$
 $A_v = 157079.633$
 $f_y = 625.00$
 $s = 100.00$
 $V_{s,j2}$ is multiplied by $Col,j2 = 1.00$
 $s/d = 0.27777778$
 $V_{s,core} = V_{s,c1} + V_{s,c2} = 92890.612$
 $V_{s,c1} = 92890.612$ is calculated for section web core, with:
 $d = 440.00$
 $A_v = 100530.965$
 $f_y = 525.00$
 $s = 250.00$
 $V_{s,c1}$ is multiplied by $Col,c1 = 1.00$
 $s/d = 0.56818182$
 $V_{s,c2} = 0.00$ is calculated for section flange core, with:
 $d = 200.00$
 $A_v = 100530.965$
 $f_y = 525.00$
 $s = 250.00$
 $V_{s,c2}$ is multiplied by $Col,c2 = 0.00$
 $s/d = 1.25$
 $V_f ((11-3)-(11.4), ACI 440) = 0.00$
 From (11-11), ACI 440: $V_s + V_f \leq 699281.943$
 $bw = 450.00$

End Of Calculation of Shear Capacity ratio for element: column JTC1 of floor 1
 At local axis: 3

Start Of Calculation of Shear Capacity ratio for element: column JTC1 of floor 1

At Shear local axis: 2

(Bending local axis: 3)

Section Type: rcjctcs

Constant Properties

Knowledge Factor, $\phi = 0.80$

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

Consequently:

Jacket

New material of Primary Member: Concrete Strength, $f_c = f_{cm} = 18.00$

New material of Primary Member: Steel Strength, $f_s = f_{sm} = 625.00$

Concrete Elasticity, $E_c = 19940.411$

Steel Elasticity, $E_s = 200000.00$

Existing Column

Existing material of Primary Member: Concrete Strength, $f_c = f_{cm} = 12.00$

Existing material of Primary Member: Steel Strength, $f_s = f_{sm} = 525.00$

Concrete Elasticity, $E_c = 16281.278$

Steel Elasticity, $E_s = 200000.00$

#####

Note: Especially for the calculation of moment strengths,

the above steel re-bar strengths are multiplied by 1.25 according to R18.6.5, ACI 318-14

Jacket

New material: Steel Strength, $f_s = 1.25 \cdot f_{sm} = 781.25$

Existing Column

Existing material: Steel Strength, $f_s = 1.25 \cdot f_{sm} = 656.25$

#####

Max Height, $H_{max} = 750.00$

Min Height, $H_{min} = 450.00$

Max Width, $W_{max} = 950.00$

Min Width, $W_{min} = 450.00$

Eccentricity, $E_{cc} = 250.00$

Jacket Thickness, $t_j = 100.00$

Cover Thickness, $c = 25.00$

Mean Confinement Factor overall section = 1.42131

Element Length, $L = 3000.00$

Primary Member

Ribbed Bars

Ductile Steel

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

Longitudinal Bars With Ends Lapped Starting at the End Sections

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

No FRP Wrapping

Stepwise Properties

At local axis: 2

EDGE -A-

Shear Force, $V_a = -137.4894$

EDGE -B-

Shear Force, $V_b = 137.4894$

BOTH EDGES

Axial Force, $F = -7.6839E+006$

Longitudinal Reinforcement Area Distribution (in 2 divisions)

-Tension: $As_t = 314.1593$

-Compression: $As_c = 6176.371$

Longitudinal Reinforcement Area Distribution (in 3 divisions)

-Tension: $As_{t,ten} = 1539.38$

-Compression: $As_{l,com} = 1539.38$

-Middle: $As_{l,mid} = 3612.832$

Calculation of Shear Capacity ratio , $V_e/V_r = 0.86614729$

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

Calculation of Shear Demand from fig. R18.6.5, ACI 318-14 $V_e = (M_{pr1} + M_{pr2})/l_n = 1.2287E+006$

with

$M_{pr1} = \text{Max}(\mu_{1+}, \mu_{1-}) = 1.8431E+009$

$\mu_{1+} = 1.8431E+009$, is the ultimate moment strength at the edge 1 of the member in the actual moment direction which is defined for the static loading combination

$\mu_{1-} = 1.8431E+009$, is the ultimate moment strength at the edge 1 of the member in the opposite moment direction which is defined for the static loading combination

$M_{pr2} = \text{Max}(\mu_{2+}, \mu_{2-}) = 1.8431E+009$

$\mu_{2+} = 1.8431E+009$, is the ultimate moment strength at the edge 2 of the member in the actual moment direction which is defined for the static loading combination

$\mu_{2-} = 1.8431E+009$, is the ultimate moment strength at the edge 2 of the member in the opposite moment direction which is defined for the static loading combination

Calculation of μ_{1+}

Calculation of ultimate curvature ϕ_u according to 4.1, Biskinis/Fardis 2013:

$\phi_u = 7.8730391E-006$

$\mu_u = 1.8431E+009$

with full section properties:

$b = 450.00$

$d = 907.00$

$d' = 43.00$

$\nu = 1.0459$

$N = 7.6839E+006$

$f_c = 18.00$

ϕ_c (5A.5, TBDY) = 0.002

Final value of ϕ_u : $\phi_u^* = \text{shear_factor} * \text{Max}(\phi_u, \phi_c) = 0.01486841$

The Shear_factor is considered equal to 1 (pure moment strength)

From (5.4b), TBDY: $\phi_u = 0.01486841$

ϕ_{ue} (5.4c) = 0.08077545

ϕ_{ase} ((5.4d), TBDY) = $(\phi_{ase1} * A_{ext} + \phi_{ase2} * A_{int}) / A_{sec} = 0.53375773$

$\phi_{ase1} = \text{Max}(((A_{conf,max1} - A_{noConf1}) / A_{conf,max1}) * (A_{conf,min1} / A_{conf,max1}), 0) = 0.53375773$

The definitions of A_{noConf} , $A_{conf,min}$ and $A_{conf,max}$ are derived from generalization of the rectangular sections confinement, which is expressed by (5.4d).

The generalization is done according to Mander, J., Priestley, M., and Park, R. (1988)

"Theoretical Stress-Strain Model for Confined Concrete."

J. Struct. Eng., 10.1061/(ASCE)0733-9445(1988)114:8(1804), 1804-1826.

$A_{conf,max1} = 464100.00$ is the confined core area at levels of member with hoops and

is calculated as the area of core enclosed by the center lines of the external perimeter hoops.

$A_{conf,min1} = 395025.00$ is the confined core area at midway between the levels of hoops and

is calculated by reducing all the dimensions of the area $A_{conf,max1}$ by a length equal to half the clear spacing between external hoops.

$A_{noConf1} = 173066.667$ is the unconfined external core area which is equal to $b_i^2/6$ as defined at (A.2).

$\phi_{ase2} (> \phi_{ase1}) = \text{Max}(((A_{conf,max2} - A_{noConf2}) / A_{conf,max2}) * (A_{conf,min2} / A_{conf,max2}), 0) = 0.53375773$

The definitions of A_{noConf} , $A_{conf,min}$ and $A_{conf,max}$ are derived from generalization of the rectangular sections confinement, which is expressed by (5.4d).

The generalization is done according to Mander, J., Priestley, M., and Park, R. (1988)

"Theoretical Stress-Strain Model for Confined Concrete."

J. Struct. Eng., 10.1061/(ASCE)0733-9445(1988)114:8(1804), 1804-1826.

$A_{conf,max2} = 252164.00$ is the confined core area at levels of member with hoops and

is calculated as the area of core enclosed by the center lines of the internal perimeter hoops.

$A_{conf,min2} = 111441.00$ is the confined core area at midway between the levels of hoops and

is calculated by reducing all the dimensions of the area $A_{conf,max2}$ by a length equal to half the clear spacing between internal hoops.

$A_{noConf2} = 110709.333$ is the unconfined internal core area which is equal to $b_i^2/6$ as defined at (A.2).

$p_{sh,min} * F_{ywe} = \text{Min}(p_{sh,x} * F_{ywe}, p_{sh,y} * F_{ywe}) = 2.724$

$p_{sh,x} * F_{ywe} = p_{sh1} * F_{ywe1} + p_{sh2} * F_{ywe2} = 2.724$

p_{sh1} ((5.4d), TBDY) = $L_{stir1} * A_{stir1} / (A_{sec} * s_1) = 0.00301593$

L_{stir1} (Length of stirrups along Y) = 2160.00

A_{stir1} (stirrups area) = 78.53982

$psh_2 (5.4d) = Lstir_2 * Astir_2 / (Asec * s_2) = 0.00056047$
 $Lstir_2$ (Length of stirrups along Y) = 1568.00
 $Astir_2$ (stirrups area) = 50.26548

$psh_y * Fywe = psh_1 * Fywe_1 + ps_2 * Fywe_2 = 3.25416$
 $psh_1 ((5.4d), TBDY) = Lstir_1 * Astir_1 / (Asec * s_1) = 0.00357443$
 $Lstir_1$ (Length of stirrups along X) = 2560.00
 $Astir_1$ (stirrups area) = 78.53982
 $psh_2 ((5.4d), TBDY) = Lstir_2 * Astir_2 / (Asec * s_2) = 0.00070345$
 $Lstir_2$ (Length of stirrups along X) = 1968.00
 $Astir_2$ (stirrups area) = 50.26548

$Asec = 562500.00$

$s_1 = 100.00$

$s_2 = 250.00$

$fywe_1 = 781.25$

$fywe_2 = 656.25$

$fce = 18.00$

From ((5.A5), TBDY), TBDY: $cc = 0.00621307$

$c =$ confinement factor = 1.42131

$y_1 = 0.0025$

$sh_1 = 0.008$

$ft_1 = 907.50$

$fy_1 = 756.25$

$su_1 = 0.032$

using (30) in Biskinis/Fardis (2013) multiplied with shear_factor
and also multiplied by the shear_factor according to 15.7.1.4, with
Shear_factor = 1.00

$lo/lo_{u,min} = lb/ld = 1.00$

$su_1 = 0.4 * esu_{1,nominal} ((5.5), TBDY) = 0.032$

From table 5A.1, TBDY: $esu_{1,nominal} = 0.08$,

For calculation of $esu_{1,nominal}$ and y_1, sh_1, ft_1, fy_1 , it is considered
characteristic value $fsy_1 = fs_1/1.2$, from table 5.1, TBDY.

y_1, sh_1, ft_1, fy_1 , are also multiplied by $Min(1, 1.25 * (lb/ld)^{2/3})$, from 10.3.5, ASCE 41-17.

with $fs_1 = (fs_{jacket} * Asl_{ten,jacket} + fs_{core} * Asl_{ten,core}) / Asl_{ten} = 756.25$

with $Es_1 = (Es_{jacket} * Asl_{ten,jacket} + Es_{core} * Asl_{ten,core}) / Asl_{ten} = 200000.00$

$y_2 = 0.0025$

$sh_2 = 0.008$

$ft_2 = 907.50$

$fy_2 = 756.25$

$su_2 = 0.032$

using (30) in Biskinis/Fardis (2013) multiplied with shear_factor
and also multiplied by the shear_factor according to 15.7.1.4, with
Shear_factor = 1.00

$lo/lo_{u,min} = lb/lb_{min} = 1.00$

$su_2 = 0.4 * esu_{2,nominal} ((5.5), TBDY) = 0.032$

From table 5A.1, TBDY: $esu_{2,nominal} = 0.08$,

For calculation of $esu_{2,nominal}$ and y_2, sh_2, ft_2, fy_2 , it is considered
characteristic value $fsy_2 = fs_2/1.2$, from table 5.1, TBDY.

y_1, sh_1, ft_1, fy_1 , are also multiplied by $Min(1, 1.25 * (lb/ld)^{2/3})$, from 10.3.5, ASCE 41-17.

with $fs_2 = (fs_{jacket} * Asl_{com,jacket} + fs_{core} * Asl_{com,core}) / Asl_{com} = 756.25$

with $Es_2 = (Es_{jacket} * Asl_{com,jacket} + Es_{core} * Asl_{com,core}) / Asl_{com} = 200000.00$

$y_v = 0.0025$

$sh_v = 0.008$

$ft_v = 899.1522$

$fy_v = 749.2935$

$suv = 0.032$

using (30) in Biskinis/Fardis (2013) multiplied with shear_factor
and also multiplied by the shear_factor according to 15.7.1.4, with
Shear_factor = 1.00

$lo/lo_{u,min} = lb/ld = 1.00$

$suv = 0.4 * esuv_{nominal} ((5.5), TBDY) = 0.032$

From table 5A.1, TBDY: $esuv_{nominal} = 0.08$,

considering characteristic value $fsyv = fsv/1.2$, from table 5.1, TBDY
For calculation of $esuv_{nominal}$ and y_v, sh_v, ft_v, fy_v , it is considered

characteristic value $fsyv = fsv/1.2$, from table 5.1, TBDY.

y_1, sh_1, ft_1, fy_1 , are also multiplied by $Min(1, 1.25 * (lb/ld)^{2/3})$, from 10.3.5, ASCE 41-17.

```

with fsv = (fs,jacket*Asl,mid,jacket + fs,mid*Asl,mid,core)/Asl,mid = 749.2935
with Esv = (Es,jacket*Asl,mid,jacket + Es,mid*Asl,mid,core)/Asl,mid = 200000.00
1 = Asl,ten/(b*d)*(fs1/fc) = 0.15845977
2 = Asl,com/(b*d)*(fs2/fc) = 0.15845977
v = Asl,mid/(b*d)*(fsv/fc) = 0.36847443
and confined core properties:
b = 390.00
d = 877.00
d' = 13.00
fcc (5A.2, TBDY) = 25.58352
cc (5A.5, TBDY) = 0.00621307
c = confinement factor = 1.42131
1 = Asl,ten/(b*d)*(fs1/fc) = 0.18909264
2 = Asl,com/(b*d)*(fs2/fc) = 0.18909264
v = Asl,mid/(b*d)*(fsv/fc) = 0.43970658
Case/Assumption: Unconfined full section - Steel rupture
' satisfies Eq. (4.3)
--->
v < vs,y2 - LHS eq.(4.5) is not satisfied
--->
v < vs,c - RHS eq.(4.5) is not satisfied
--->
Case/Assumption Rejected.
--->
New Case/Assumption: Unconfined full section - Spalling of concrete cover
' satisfies Eq. (4.4)
--->
v < s,y1 - LHS eq.(4.7) is not satisfied
--->
v < vc,y1 - RHS eq.(4.6) is satisfied
--->
cu (4.10) = 0.83934343
MRc (4.17) = 1.8439E+009
--->
New Case/Assumption: Ultimate curvature of confined core after spalling of concrete cover
In expressions below, the following modifications have been made
- b, d, d' replaced by geometric parameters of the core: bo, do, d'o
- N, 1, 2, v normalised to bo*do, instead of b*d
- - parameters of confined concrete, fcc, cc, used in lieu of fc, ecu
--->
Subcase: Rupture of tension steel
--->
v* < v*s,y2 - LHS eq.(4.5) is not satisfied
--->
v* < v*s,c - LHS eq.(4.5) is not satisfied
--->
Subcase rejected
--->
New Subcase: Failure of compression zone
--->
v* < v*c,y2 - LHS eq.(4.6) is not satisfied
--->
v* < v*c,y1 - RHS eq.(4.6) is not satisfied
--->
*cu (4.11) = 0.89983732
MRo (4.18) = 1.8431E+009
--->
u = cu (4.2) = 7.8730391E-006
Mu = MRo

```

Calculation of ratio lb/d

Adequate Lap Length: lb/d >= 1

Calculation of Mu1-

Calculation of ultimate curvature ϕ_u according to 4.1, Biskinis/Fardis 2013:

$$\phi_u = 7.8730391E-006$$

$$M_u = 1.8431E+009$$

with full section properties:

$$b = 450.00$$

$$d = 907.00$$

$$d' = 43.00$$

$$v = 1.0459$$

$$N = 7.6839E+006$$

$$f_c = 18.00$$

$$\phi_{co} (5A.5, TBDY) = 0.002$$

$$\text{Final value of } \phi_{cu}: \phi_{cu}^* = \text{shear_factor} * \text{Max}(\phi_{cu}, \phi_{cc}) = 0.01486841$$

The Shear_factor is considered equal to 1 (pure moment strength)

$$\text{From (5.4b), TBDY: } \phi_{cu} = 0.01486841$$

$$\phi_{we} (5.4c) = 0.08077545$$

$$\phi_{ase} ((5.4d), TBDY) = (\phi_{ase1} * A_{ext} + \phi_{ase2} * A_{int}) / A_{sec} = 0.53375773$$

$$\phi_{ase1} = \text{Max}(((A_{conf,max1} - A_{noConf1}) / A_{conf,max1}) * (A_{conf,min1} / A_{conf,max1}), 0) = 0.53375773$$

The definitions of A_{noConf} , $A_{conf,min}$ and $A_{conf,max}$ are derived from generalization of the rectangular sections confinement, which is expressed by (5.4d).

The generalization is done according to Mander, J., Priestley, M., and Park, R. (1988)

"Theoretical Stress-Strain Model for Confined Concrete."

J. Struct. Eng., 10.1061/(ASCE)0733-9445(1988)114:8(1804), 1804-1826.

$A_{conf,max1} = 464100.00$ is the confined core area at levels of member with hoops and is calculated as the area of core enclosed by the center lines of the external perimeter hoops.

$A_{conf,min1} = 395025.00$ is the confined core area at midway between the levels of hoops and is calculated by reducing all the dimensions of the area $A_{conf,max1}$ by a length equal to half the clear spacing between external hoops.

$A_{noConf1} = 173066.667$ is the unconfined external core area which is equal to $b_i^2/6$ as defined at (A.2).

$$\phi_{ase2} (> \phi_{ase1}) = \text{Max}(((A_{conf,max2} - A_{noConf2}) / A_{conf,max2}) * (A_{conf,min2} / A_{conf,max2}), 0) = 0.53375773$$

The definitions of A_{noConf} , $A_{conf,min}$ and $A_{conf,max}$ are derived from generalization of the rectangular sections confinement, which is expressed by (5.4d).

The generalization is done according to Mander, J., Priestley, M., and Park, R. (1988)

"Theoretical Stress-Strain Model for Confined Concrete."

J. Struct. Eng., 10.1061/(ASCE)0733-9445(1988)114:8(1804), 1804-1826.

$A_{conf,max2} = 252164.00$ is the confined core area at levels of member with hoops and is calculated as the area of core enclosed by the center lines of the internal perimeter hoops.

$A_{conf,min2} = 111441.00$ is the confined core area at midway between the levels of hoops and is calculated by reducing all the dimensions of the area $A_{conf,max2}$ by a length equal to half the clear spacing between internal hoops.

$A_{noConf2} = 110709.333$ is the unconfined internal core area which is equal to $b_i^2/6$ as defined at (A.2).

$$\phi_{psh,min} * F_{ywe} = \text{Min}(\phi_{psh,x} * F_{ywe}, \phi_{psh,y} * F_{ywe}) = 2.724$$

$$\phi_{psh,x} * F_{ywe} = \phi_{psh1} * F_{ywe1} + \phi_{psh2} * F_{ywe2} = 2.724$$

$$\phi_{psh1} ((5.4d), TBDY) = L_{stir1} * A_{stir1} / (A_{sec} * s_1) = 0.00301593$$

$$L_{stir1} (\text{Length of stirrups along Y}) = 2160.00$$

$$A_{stir1} (\text{stirrups area}) = 78.53982$$

$$\phi_{psh2} (5.4d) = L_{stir2} * A_{stir2} / (A_{sec} * s_2) = 0.00056047$$

$$L_{stir2} (\text{Length of stirrups along Y}) = 1568.00$$

$$A_{stir2} (\text{stirrups area}) = 50.26548$$

$$\phi_{psh,y} * F_{ywe} = \phi_{psh1} * F_{ywe1} + \phi_{psh2} * F_{ywe2} = 3.25416$$

$$\phi_{psh1} ((5.4d), TBDY) = L_{stir1} * A_{stir1} / (A_{sec} * s_1) = 0.00357443$$

$$L_{stir1} (\text{Length of stirrups along X}) = 2560.00$$

$$A_{stir1} (\text{stirrups area}) = 78.53982$$

$$\phi_{psh2} ((5.4d), TBDY) = L_{stir2} * A_{stir2} / (A_{sec} * s_2) = 0.00070345$$

$$L_{stir2} (\text{Length of stirrups along X}) = 1968.00$$

$$A_{stir2} (\text{stirrups area}) = 50.26548$$

$$A_{sec} = 562500.00$$

$$s_1 = 100.00$$

$$s_2 = 250.00$$

$$f_{ywe1} = 781.25$$

```

fywe2 = 656.25
fce = 18.00
From ((5A.5), TBDY), TBDY: cc = 0.00621307
c = confinement factor = 1.42131
y1 = 0.0025
sh1 = 0.008
ft1 = 907.50
fy1 = 756.25
su1 = 0.032
using (30) in Biskinis/Fardis (2013) multiplied with shear_factor
and also multiplied by the shear_factor according to 15.7.1.4, with
Shear_factor = 1.00
lo/lou,min = lb/d = 1.00
su1 = 0.4*esu1_nominal ((5.5), TBDY) = 0.032
From table 5A.1, TBDY: esu1_nominal = 0.08,
For calculation of esu1_nominal and y1, sh1,ft1,fy1, it is considered
characteristic value fsy1 = fs1/1.2, from table 5.1, TBDY.
y1, sh1,ft1,fy1, are also multiplied by Min(1,1.25*(lb/d)^ 2/3), from 10.3.5, ASCE 41-17.
with fs1 = (fs,jacket*Asl,ten,jacket + fs,core*Asl,ten,core)/Asl,ten = 756.25
with Es1 = (Es,jacket*Asl,ten,jacket + Es,core*Asl,ten,core)/Asl,ten = 200000.00
y2 = 0.0025
sh2 = 0.008
ft2 = 907.50
fy2 = 756.25
su2 = 0.032
using (30) in Biskinis/Fardis (2013) multiplied with shear_factor
and also multiplied by the shear_factor according to 15.7.1.4, with
Shear_factor = 1.00
lo/lou,min = lb/lb,min = 1.00
su2 = 0.4*esu2_nominal ((5.5), TBDY) = 0.032
From table 5A.1, TBDY: esu2_nominal = 0.08,
For calculation of esu2_nominal and y2, sh2,ft2,fy2, it is considered
characteristic value fsy2 = fs2/1.2, from table 5.1, TBDY.
y1, sh1,ft1,fy1, are also multiplied by Min(1,1.25*(lb/d)^ 2/3), from 10.3.5, ASCE 41-17.
with fs2 = (fs,jacket*Asl,com,jacket + fs,core*Asl,com,core)/Asl,com = 756.25
with Es2 = (Es,jacket*Asl,com,jacket + Es,core*Asl,com,core)/Asl,com = 200000.00
yv = 0.0025
shv = 0.008
ftv = 899.1522
fyv = 749.2935
suv = 0.032
using (30) in Biskinis/Fardis (2013) multiplied with shear_factor
and also multiplied by the shear_factor according to 15.7.1.4, with
Shear_factor = 1.00
lo/lou,min = lb/d = 1.00
suv = 0.4*esuv_nominal ((5.5), TBDY) = 0.032
From table 5A.1, TBDY: esuv_nominal = 0.08,
considering characteristic value fsyv = fsv/1.2, from table 5.1, TBDY
For calculation of esuv_nominal and yv, shv,ftv,fyv, it is considered
characteristic value fsyv = fsv/1.2, from table 5.1, TBDY.
y1, sh1,ft1,fy1, are also multiplied by Min(1,1.25*(lb/d)^ 2/3), from 10.3.5, ASCE 41-17.
with fsv = (fs,jacket*Asl,mid,jacket + fs,mid*Asl,mid,core)/Asl,mid = 749.2935
with Esv = (Es,jacket*Asl,mid,jacket + Es,mid*Asl,mid,core)/Asl,mid = 200000.00
1 = Asl,ten/(b*d)*(fs1/fc) = 0.15845977
2 = Asl,com/(b*d)*(fs2/fc) = 0.15845977
v = Asl,mid/(b*d)*(fsv/fc) = 0.36847443
and confined core properties:
b = 390.00
d = 877.00
d' = 13.00
fcc (5A.2, TBDY) = 25.58352
cc (5A.5, TBDY) = 0.00621307
c = confinement factor = 1.42131
1 = Asl,ten/(b*d)*(fs1/fc) = 0.18909264
2 = Asl,com/(b*d)*(fs2/fc) = 0.18909264
v = Asl,mid/(b*d)*(fsv/fc) = 0.43970658
Case/Assumption: Unconfinedsd full section - Steel rupture

```

```

' satisfies Eq. (4.3)
---->
v < vs,y2 - LHS eq.(4.5) is not satisfied
---->
v < vs,c - RHS eq.(4.5) is not satisfied
---->
Case/Assumption Rejected.
---->
New Case/Assumption: Unconfined full section - Spalling of concrete cover
' satisfies Eq. (4.4)
---->
v < s,y1 - LHS eq.(4.7) is not satisfied
---->
v < vc,y1 - RHS eq.(4.6) is satisfied
---->
cu (4.10) = 0.83934343
MRc (4.17) = 1.8439E+009
---->
New Case/Assumption: Ultimate curvature of confined core after spalling of concrete cover
In expressions below, the following modifications have been made
- b, d, d' replaced by geometric parameters of the core: bo, do, d'o
- N, 1, 2, v normalised to bo*do, instead of b*d
- parameters of confined concrete, fcc, cc, used in lieu of fc, ecu
---->
Subcase: Rupture of tension steel
---->
v* < v*s,y2 - LHS eq.(4.5) is not satisfied
---->
v* < v*s,c - LHS eq.(4.5) is not satisfied
---->
Subcase rejected
---->
New Subcase: Failure of compression zone
---->
v* < v*c,y2 - LHS eq.(4.6) is not satisfied
---->
v* < v*c,y1 - RHS eq.(4.6) is not satisfied
---->
*cu (4.11) = 0.89983732
MRo (4.18) = 1.8431E+009
---->
u = cu (4.2) = 7.8730391E-006
Mu = MRo

```

Calculation of ratio lb/d

Adequate Lap Length: lb/d >= 1

Calculation of Mu2+

Calculation of ultimate curvature u according to 4.1, Biskinis/Fardis 2013:

u = 7.8730391E-006
Mu = 1.8431E+009

with full section properties:

b = 450.00
d = 907.00
d' = 43.00
v = 1.0459
N = 7.6839E+006
fc = 18.00
co (5A.5, TBDY) = 0.002

Final value of c_u : $c_u^* = \text{shear_factor} * \text{Max}(c_u, c_c) = 0.01486841$

The Shear_factor is considered equal to 1 (pure moment strength)

From (5.4b), TBDY: $c_u = 0.01486841$

w_e (5.4c) = 0.08077545

a_{se} ((5.4d), TBDY) = $(a_{se1} * A_{ext} + a_{se2} * A_{int}) / A_{sec} = 0.53375773$

$a_{se1} = \text{Max}(((A_{conf,max1} - A_{noConf1}) / A_{conf,max1}) * (A_{conf,min1} / A_{conf,max1}), 0) = 0.53375773$

The definitions of A_{noConf} , $A_{conf,min}$ and $A_{conf,max}$ are derived from generalization of the rectangular sections confinement, which is expressed by (5.4d).

The generalization is done according to Mander, J., Priestley, M., and Park, R. (1988)

"Theoretical Stress-Strain Model for Confined Concrete."

J. Struct. Eng., 10.1061/(ASCE)0733-9445(1988)114:8(1804), 1804-1826.

$A_{conf,max1} = 464100.00$ is the confined core area at levels of member with hoops and

is calculated as the area of core enclosed by the center lines of the external perimeter hoops.

$A_{conf,min1} = 395025.00$ is the confined core area at midway between the levels of hoops and

is calculated by reducing all the dimensions of the area $A_{conf,max1}$ by a length

equal to half the clear spacing between external hoops.

$A_{noConf1} = 173066.667$ is the unconfined external core area which is equal to $b_i^2/6$ as defined at (A.2).

$a_{se2} (>= a_{se1}) = \text{Max}(((A_{conf,max2} - A_{noConf2}) / A_{conf,max2}) * (A_{conf,min2} / A_{conf,max2}), 0) = 0.53375773$

The definitions of A_{noConf} , $A_{conf,min}$ and $A_{conf,max}$ are derived from generalization of the rectangular sections confinement, which is expressed by (5.4d).

The generalization is done according to Mander, J., Priestley, M., and Park, R. (1988)

"Theoretical Stress-Strain Model for Confined Concrete."

J. Struct. Eng., 10.1061/(ASCE)0733-9445(1988)114:8(1804), 1804-1826.

$A_{conf,max2} = 252164.00$ is the confined core area at levels of member with hoops and

is calculated as the area of core enclosed by the center lines of the internal perimeter hoops.

$A_{conf,min2} = 111441.00$ is the confined core area at midway between the levels of hoops and

is calculated by reducing all the dimensions of the area $A_{conf,max2}$ by a length

equal to half the clear spacing between internal hoops.

$A_{noConf2} = 110709.333$ is the unconfined internal core area which is equal to $b_i^2/6$ as defined at (A.2).

$p_{sh,min} * F_{ywe} = \text{Min}(p_{sh,x} * F_{ywe}, p_{sh,y} * F_{ywe}) = 2.724$

$p_{sh,x} * F_{ywe} = p_{sh1} * F_{ywe1} + p_{s2} * F_{ywe2} = 2.724$

p_{sh1} ((5.4d), TBDY) = $L_{stir1} * A_{stir1} / (A_{sec} * s_1) = 0.00301593$

L_{stir1} (Length of stirrups along Y) = 2160.00

A_{stir1} (stirrups area) = 78.53982

p_{sh2} (5.4d) = $L_{stir2} * A_{stir2} / (A_{sec} * s_2) = 0.00056047$

L_{stir2} (Length of stirrups along Y) = 1568.00

A_{stir2} (stirrups area) = 50.26548

$p_{sh,y} * F_{ywe} = p_{sh1} * F_{ywe1} + p_{s2} * F_{ywe2} = 3.25416$

p_{sh1} ((5.4d), TBDY) = $L_{stir1} * A_{stir1} / (A_{sec} * s_1) = 0.00357443$

L_{stir1} (Length of stirrups along X) = 2560.00

A_{stir1} (stirrups area) = 78.53982

p_{sh2} ((5.4d), TBDY) = $L_{stir2} * A_{stir2} / (A_{sec} * s_2) = 0.00070345$

L_{stir2} (Length of stirrups along X) = 1968.00

A_{stir2} (stirrups area) = 50.26548

$A_{sec} = 562500.00$

$s_1 = 100.00$

$s_2 = 250.00$

$f_{ywe1} = 781.25$

$f_{ywe2} = 656.25$

$f_{ce} = 18.00$

From ((5.A5), TBDY), TBDY: $c_c = 0.00621307$

c = confinement factor = 1.42131

$y_1 = 0.0025$

$sh_1 = 0.008$

$ft_1 = 907.50$

$fy_1 = 756.25$

$su_1 = 0.032$

using (30) in Biskinis/Fardis (2013) multiplied with shear_factor

and also multiplied by the shear_factor according to 15.7.1.4, with

Shear_factor = 1.00

$l_o/l_{ou,min} = l_b/l_d = 1.00$

$su_1 = 0.4 * esu_{1,nominal}$ ((5.5), TBDY) = 0.032

From table 5A.1, TBDY: $esu_{1,nominal} = 0.08$,

For calculation of $esu_{1,nominal}$ and y_1 , sh_1 , ft_1 , fy_1 , it is considered

characteristic value $f_{sy1} = f_{s1}/1.2$, from table 5.1, TBDY.
 $y1, sh1, ft1, fy1$, are also multiplied by $\text{Min}(1, 1.25 \cdot (l_b/l_d)^{2/3})$, from 10.3.5, ASCE 41-17.
with $f_{s1} = (f_{sjacket} \cdot A_{sl,ten,jacket} + f_{s,core} \cdot A_{sl,ten,core})/A_{sl,ten} = 756.25$
with $E_{s1} = (E_{sjacket} \cdot A_{sl,ten,jacket} + E_{s,core} \cdot A_{sl,ten,core})/A_{sl,ten} = 200000.00$
 $y2 = 0.0025$
 $sh2 = 0.008$
 $ft2 = 907.50$
 $fy2 = 756.25$
 $su2 = 0.032$
using (30) in Biskinis/Fardis (2013) multiplied with shear_factor
and also multiplied by the shear_factor according to 15.7.1.4, with
Shear_factor = 1.00
 $l_o/l_{ou,min} = l_b/l_{b,min} = 1.00$
 $su2 = 0.4 \cdot esu2_nominal \text{ ((5.5), TBDY)} = 0.032$
From table 5A.1, TBDY: $esu2_nominal = 0.08$,
For calculation of $esu2_nominal$ and $y2, sh2, ft2, fy2$, it is considered
characteristic value $f_{sy2} = f_{s2}/1.2$, from table 5.1, TBDY.
 $y1, sh1, ft1, fy1$, are also multiplied by $\text{Min}(1, 1.25 \cdot (l_b/l_d)^{2/3})$, from 10.3.5, ASCE 41-17.
with $f_{s2} = (f_{sjacket} \cdot A_{sl,com,jacket} + f_{s,core} \cdot A_{sl,com,core})/A_{sl,com} = 756.25$
with $E_{s2} = (E_{sjacket} \cdot A_{sl,com,jacket} + E_{s,core} \cdot A_{sl,com,core})/A_{sl,com} = 200000.00$
 $yv = 0.0025$
 $shv = 0.008$
 $ftv = 899.1522$
 $fyv = 749.2935$
 $suv = 0.032$
using (30) in Biskinis/Fardis (2013) multiplied with shear_factor
and also multiplied by the shear_factor according to 15.7.1.4, with
Shear_factor = 1.00
 $l_o/l_{ou,min} = l_b/l_d = 1.00$
 $suv = 0.4 \cdot esuv_nominal \text{ ((5.5), TBDY)} = 0.032$
From table 5A.1, TBDY: $esuv_nominal = 0.08$,
considering characteristic value $f_{syv} = f_{sv}/1.2$, from table 5.1, TBDY
For calculation of $esuv_nominal$ and yv, shv, ftv, fyv , it is considered
characteristic value $f_{syv} = f_{sv}/1.2$, from table 5.1, TBDY.
 $y1, sh1, ft1, fy1$, are also multiplied by $\text{Min}(1, 1.25 \cdot (l_b/l_d)^{2/3})$, from 10.3.5, ASCE 41-17.
with $f_{sv} = (f_{sjacket} \cdot A_{sl,mid,jacket} + f_{s,mid} \cdot A_{sl,mid,core})/A_{sl,mid} = 749.2935$
with $E_{sv} = (E_{sjacket} \cdot A_{sl,mid,jacket} + E_{s,mid} \cdot A_{sl,mid,core})/A_{sl,mid} = 200000.00$
 $1 = A_{sl,ten}/(b \cdot d) \cdot (f_{s1}/f_c) = 0.15845977$
 $2 = A_{sl,com}/(b \cdot d) \cdot (f_{s2}/f_c) = 0.15845977$
 $v = A_{sl,mid}/(b \cdot d) \cdot (f_{sv}/f_c) = 0.36847443$
and confined core properties:
 $b = 390.00$
 $d = 877.00$
 $d' = 13.00$
 $f_{cc} \text{ (5A.2, TBDY)} = 25.58352$
 $cc \text{ (5A.5, TBDY)} = 0.00621307$
 $c = \text{confinement factor} = 1.42131$
 $1 = A_{sl,ten}/(b \cdot d) \cdot (f_{s1}/f_c) = 0.18909264$
 $2 = A_{sl,com}/(b \cdot d) \cdot (f_{s2}/f_c) = 0.18909264$
 $v = A_{sl,mid}/(b \cdot d) \cdot (f_{sv}/f_c) = 0.43970658$
Case/Assumption: Unconfined full section - Steel rupture
' satisfies Eq. (4.3)
--->
 $v < v_{s,y2}$ - LHS eq.(4.5) is not satisfied
--->
 $v < v_{s,c}$ - RHS eq.(4.5) is not satisfied
--->
Case/Assumption Rejected.
--->
New Case/Assumption: Unconfined full section - Spalling of concrete cover
' satisfies Eq. (4.4)
--->
 $v < s_{y1}$ - LHS eq.(4.7) is not satisfied
--->
 $v < v_{c,y1}$ - RHS eq.(4.6) is satisfied
--->
 $cu \text{ (4.10)} = 0.83934343$

MRC (4.17) = 1.8439E+009

---->

New Case/Assumption: Ultimate curvature of confined core after spalling of concrete cover

In expressions below, the following modifications have been made

- b, d, d' replaced by geometric parameters of the core: bo, do, d'o
- N, ϵ_s , ϵ_c , ϵ_{cs} normalised to b_o*d_o , instead of $b*d$
- ϵ_{cs} parameters of confined concrete, ϵ_{cc} , ϵ_{cs} , used in lieu of ϵ_c , ϵ_{cs}

---->

Subcase: Rupture of tension steel

---->

$\epsilon_s^* < \epsilon_{sy}$ - LHS eq.(4.5) is not satisfied

---->

$\epsilon_s^* < \epsilon_{sc}$ - LHS eq.(4.5) is not satisfied

---->

Subcase rejected

---->

New Subcase: Failure of compression zone

---->

$\epsilon_s^* < \epsilon_{cy}$ - LHS eq.(4.6) is not satisfied

---->

$\epsilon_s^* < \epsilon_{cy}$ - RHS eq.(4.6) is not satisfied

---->

ϵ_{cu} (4.11) = 0.89983732

MRO (4.18) = 1.8431E+009

---->

$\epsilon_u = \epsilon_{cu}$ (4.2) = 7.8730391E-006

$\mu_u = MRO$

Calculation of ratio I_b/I_d

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

Calculation of μ_u -

Calculation of ultimate curvature ϵ_u according to 4.1, Biskinis/Fardis 2013:

$\epsilon_u = 7.8730391E-006$

$\mu_u = 1.8431E+009$

with full section properties:

b = 450.00

d = 907.00

d' = 43.00

$\epsilon_s = 1.0459$

N = 7.6839E+006

$\epsilon_c = 18.00$

ϵ_{cs} (5A.5, TBDY) = 0.002

Final value of ϵ_{cs} : $\epsilon_{cs}^* = \text{shear_factor} * \text{Max}(\epsilon_{cs}, \epsilon_{cc}) = 0.01486841$

The Shear_factor is considered equal to 1 (pure moment strength)

From (5.4b), TBDY: $\epsilon_{cs} = 0.01486841$

ϵ_{cs} (5.4c) = 0.08077545

ϵ_{cs} ((5.4d), TBDY) = $(\epsilon_{cs1} * A_{ext} + \epsilon_{cs2} * A_{int}) / A_{sec} = 0.53375773$

$\epsilon_{cs1} = \text{Max}(((A_{conf,max1} - A_{noConf1}) / A_{conf,max1}) * (A_{conf,min1} / A_{conf,max1}), 0) = 0.53375773$

The definitions of A_{noConf} , $A_{conf,min}$ and $A_{conf,max}$ are derived from generalization of the rectangular sections confinement, which is expressed by (5.4d).

The generalization is done according to Mander, J., Priestley, M., and Park, R. (1988)

"Theoretical Stress-Strain Model for Confined Concrete."

J. Struct. Eng., 10.1061/(ASCE)0733-9445(1988)114:8(1804), 1804-1826.

$A_{conf,max1} = 464100.00$ is the confined core area at levels of member with hoops and

is calculated as the area of core enclosed by the center lines of the external perimeter hoops.

$A_{conf,min1} = 395025.00$ is the confined core area at midway between the levels of hoops and

is calculated by reducing all the dimensions of the area $A_{conf,max1}$ by a length equal to half the clear spacing between external hoops.

AnoConf1 = 173066.667 is the unconfined external core area which is equal to $b_i^2/6$ as defined at (A.2).
 $ase2 (>=ase1) = \text{Max}(((A_{conf,max2}-AnoConf2)/A_{conf,max2}) * (A_{conf,min2}/A_{conf,max2}), 0) = 0.53375773$
The definitions of AnoConf, Aconf,min and Aconf,max are derived from generalization of the rectangular sections confinement, which is expressed by (5.4d).
The generalization is done according to Mander, J., Priestley, M., and Park, R. (1988) "Theoretical Stress-Strain Model for Confined Concrete."
J. Struct. Eng., 10.1061/(ASCE)0733-9445(1988)114:8(1804), 1804-1826.
Aconf,max2 = 252164.00 is the confined core area at levels of member with hoops and is calculated as the area of core enclosed by the center lines of the internal perimeter hoops.
Aconf,min2 = 111441.00 is the confined core area at midway between the levels of hoops and is calculated by reducing all the dimensions of the area Aconf,max2 by a length equal to half the clear spacing between internal hoops.
AnoConf2 = 110709.333 is the unconfined internal core area which is equal to $b_i^2/6$ as defined at (A.2).
 $psh,min * Fywe = \text{Min}(psh,x * Fywe, psh,y * Fywe) = 2.724$

$psh,x * Fywe = psh1 * Fywe1 + ps2 * Fywe2 = 2.724$
 $psh1 ((5.4d), TBDY) = Lstir1 * Astir1 / (Asec * s1) = 0.00301593$
Lstir1 (Length of stirrups along Y) = 2160.00
Astir1 (stirrups area) = 78.53982
 $psh2 ((5.4d)) = Lstir2 * Astir2 / (Asec * s2) = 0.00056047$
Lstir2 (Length of stirrups along Y) = 1568.00
Astir2 (stirrups area) = 50.26548

$psh,y * Fywe = psh1 * Fywe1 + ps2 * Fywe2 = 3.25416$
 $psh1 ((5.4d), TBDY) = Lstir1 * Astir1 / (Asec * s1) = 0.00357443$
Lstir1 (Length of stirrups along X) = 2560.00
Astir1 (stirrups area) = 78.53982
 $psh2 ((5.4d), TBDY) = Lstir2 * Astir2 / (Asec * s2) = 0.00070345$
Lstir2 (Length of stirrups along X) = 1968.00
Astir2 (stirrups area) = 50.26548

Asec = 562500.00
s1 = 100.00
s2 = 250.00
fywe1 = 781.25
fywe2 = 656.25
fce = 18.00

From ((5.A5), TBDY), TBDY: cc = 0.00621307
c = confinement factor = 1.42131

y1 = 0.0025
sh1 = 0.008
ft1 = 907.50
fy1 = 756.25
su1 = 0.032

using (30) in Biskinis/Fardis (2013) multiplied with shear_factor and also multiplied by the shear_factor according to 15.7.1.4, with Shear_factor = 1.00

lo/lou,min = lb/lb = 1.00

su1 = 0.4 * esu1_nominal ((5.5), TBDY) = 0.032

From table 5A.1, TBDY: esu1_nominal = 0.08,

For calculation of esu1_nominal and y1, sh1, ft1, fy1, it is considered characteristic value fsy1 = fs1/1.2, from table 5.1, TBDY.

y1, sh1, ft1, fy1, are also multiplied by $\text{Min}(1, 1.25 * (lb/lb)^{2/3})$, from 10.3.5, ASCE 41-17.

with fs1 = (fs,jacket * Asl,ten,jacket + fs,core * Asl,ten,core) / Asl,ten = 756.25

with Es1 = (Es,jacket * Asl,ten,jacket + Es,core * Asl,ten,core) / Asl,ten = 200000.00

y2 = 0.0025
sh2 = 0.008
ft2 = 907.50
fy2 = 756.25
su2 = 0.032

using (30) in Biskinis/Fardis (2013) multiplied with shear_factor and also multiplied by the shear_factor according to 15.7.1.4, with Shear_factor = 1.00

lo/lou,min = lb/lb,min = 1.00

su2 = 0.4 * esu2_nominal ((5.5), TBDY) = 0.032

From table 5A.1, TBDY: esu2_nominal = 0.08,

For calculation of esu2_nominal and y2, sh2, ft2, fy2, it is considered

characteristic value $fsy2 = fs2/1.2$, from table 5.1, TBDY.
 $y1, sh1, ft1, fy1$, are also multiplied by $\text{Min}(1, 1.25 \cdot (lb/ld)^{2/3})$, from 10.3.5, ASCE 41-17.
 with $fs2 = (fs_{jacket} \cdot Asl_{com,jacket} + fs_{core} \cdot Asl_{com,core}) / Asl_{com} = 756.25$
 with $Es2 = (Es_{jacket} \cdot Asl_{com,jacket} + Es_{core} \cdot Asl_{com,core}) / Asl_{com} = 200000.00$
 $yv = 0.0025$
 $shv = 0.008$
 $ftv = 899.1522$
 $fyv = 749.2935$
 $suv = 0.032$
 using (30) in Biskinis/Fardis (2013) multiplied with shear_factor
 and also multiplied by the shear_factor according to 15.7.1.4, with
 Shear_factor = 1.00
 $lo/lou, min = lb/ld = 1.00$
 $suv = 0.4 \cdot esuv_{nominal} ((5.5), TBDY) = 0.032$
 From table 5A.1, TBDY: $esuv_{nominal} = 0.08$,
 considering characteristic value $fsyv = fsv/1.2$, from table 5.1, TBDY
 For calculation of $esuv_{nominal}$ and yv, shv, ftv, fyv , it is considered
 characteristic value $fsyv = fsv/1.2$, from table 5.1, TBDY.
 $y1, sh1, ft1, fy1$, are also multiplied by $\text{Min}(1, 1.25 \cdot (lb/ld)^{2/3})$, from 10.3.5, ASCE 41-17.
 with $fsv = (fs_{jacket} \cdot Asl_{mid,jacket} + fs_{mid} \cdot Asl_{mid,core}) / Asl_{mid} = 749.2935$
 with $Esv = (Es_{jacket} \cdot Asl_{mid,jacket} + Es_{mid} \cdot Asl_{mid,core}) / Asl_{mid} = 200000.00$
 $1 = Asl_{ten} / (b \cdot d) \cdot (fs1 / fc) = 0.15845977$
 $2 = Asl_{com} / (b \cdot d) \cdot (fs2 / fc) = 0.15845977$
 $v = Asl_{mid} / (b \cdot d) \cdot (fsv / fc) = 0.36847443$
 and confined core properties:
 $b = 390.00$
 $d = 877.00$
 $d' = 13.00$
 $fcc (5A.2, TBDY) = 25.58352$
 $cc (5A.5, TBDY) = 0.00621307$
 $c = \text{confinement factor} = 1.42131$
 $1 = Asl_{ten} / (b \cdot d) \cdot (fs1 / fc) = 0.18909264$
 $2 = Asl_{com} / (b \cdot d) \cdot (fs2 / fc) = 0.18909264$
 $v = Asl_{mid} / (b \cdot d) \cdot (fsv / fc) = 0.43970658$
 Case/Assumption: Unconfined full section - Steel rupture
 ' satisfies Eq. (4.3)
 --->
 $v < vs,y2$ - LHS eq.(4.5) is not satisfied
 --->
 $v < vs,c$ - RHS eq.(4.5) is not satisfied
 --->
 Case/Assumption Rejected.
 --->
 New Case/Assumption: Unconfined full section - Spalling of concrete cover
 ' satisfies Eq. (4.4)
 --->
 $v < s,y1$ - LHS eq.(4.7) is not satisfied
 --->
 $v < vc,y1$ - RHS eq.(4.6) is satisfied
 --->
 $cu (4.10) = 0.83934343$
 $MRC (4.17) = 1.8439E+009$
 --->
 New Case/Assumption: Ultimate curvature of confined core after spalling of concrete cover
 In expressions below, the following modifications have been made
 - b, d, d' replaced by geometric parameters of the core: $bo, do, d'o$
 - $N, 1, 2, v$ normalised to $bo \cdot do$, instead of $b \cdot d$
 - parameters of confined concrete, fcc, cc , used in lieu of fc, ecu
 --->
 Subcase: Rupture of tension steel
 --->
 $v^* < v^*,y2$ - LHS eq.(4.5) is not satisfied
 --->
 $v^* < v^*,c$ - LHS eq.(4.5) is not satisfied
 --->
 Subcase rejected
 --->

New Subcase: Failure of compression zone

--->

$v^* < v^*_{c,y2}$ - LHS eq.(4.6) is not satisfied

--->

$v^* < v^*_{c,y1}$ - RHS eq.(4.6) is not satisfied

--->

$*c_u$ (4.11) = 0.89983732

M_{Ro} (4.18) = 1.8431E+009

--->

$u = c_u$ (4.2) = 7.8730391E-006

$\mu = M_{Ro}$

Calculation of ratio l_b/d

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

Calculation of Shear Strength $V_r = \min(V_{r1}, V_{r2}) = 1.4186E+006$

Calculation of Shear Strength at edge 1, $V_{r1} = 1.4186E+006$

$V_{r1} = V_{Col}$ ((10.3), ASCE 41-17) = $k_{nl} * V_{Col0}$

$V_{Col0} = 1.4186E+006$

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

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

= 1 (normal-weight concrete)

Mean concrete strength: $f'_c = (f'_{c,jacket} * Area_{jacket} + f'_{c,core} * Area_{core}) / Area_{section} = 15.20$, but $f'_c^{0.5} \leq 8.3$ MPa (22.5.3.1, ACI 318-14)

$M/Vd = 4.00$

$\mu_u = 2.6919E+008$

$V_u = 137.4894$

$d = 0.8 * h = 760.00$

$N_u = 7.6839E+006$

$A_g = 427500.00$

From (11.5.4.8), ACI 318-14: $V_s = V_{sjacket} + V_{s,core} = 1.2262E+006$

where:

$V_{sjacket} = V_{sj1} + V_{sj2} = 1.0996E+006$

$V_{sj1} = 353429.174$ is calculated for section web jacket, with:

$d = 360.00$

$A_v = 157079.633$

$f_y = 625.00$

$s = 100.00$

V_{sj1} is multiplied by $Col,j1 = 1.00$

$s/d = 0.27777778$

$V_{sj2} = 746128.255$ is calculated for section flange jacket, with:

$d = 760.00$

$A_v = 157079.633$

$f_y = 625.00$

$s = 100.00$

V_{sj2} is multiplied by $Col,j2 = 1.00$

$s/d = 0.13157895$

$V_{s,core} = V_{s,c1} + V_{s,c2} = 126669.016$

$V_{s,c1} = 0.00$ is calculated for section web core, with:

$d = 200.00$

$A_v = 100530.965$

$f_y = 525.00$

$s = 250.00$

$V_{s,c1}$ is multiplied by $Col,c1 = 0.00$

$s/d = 1.25$

$V_{s,c2} = 126669.016$ is calculated for section flange core, with:

$d = 600.00$

$A_v = 100530.965$

$f_y = 525.00$
 $s = 250.00$
 $V_{s,c2}$ is multiplied by $Col,c2 = 1.00$
 $s/d = 0.41666667$
 $V_f ((11-3)-(11.4), ACI 440) = 0.00$
 From (11-11), ACI 440: $V_s + V_f \leq 885757.128$
 $bw = 450.00$

Calculation of Shear Strength at edge 2, $V_{r2} = 1.4186E+006$
 $V_{r2} = V_{Col} ((10.3), ASCE 41-17) = knl * V_{Col0}$
 $V_{Col0} = 1.4186E+006$
 $knl = 1$ (zero step-static loading)

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

$= 1$ (normal-weight concrete)
 Mean concrete strength: $f_c' = (f_c'_{jacket} * Area_{jacket} + f_c'_{core} * Area_{core}) / Area_{section} = 15.20$, but $f_c'^{0.5} \leq 8.3$ MPa (22.5.3.1, ACI 318-14)
 $M/Vd = 4.00$
 $\mu_u = 2.5135E+008$
 $V_u = 137.4894$
 $d = 0.8 * h = 760.00$
 $N_u = 7.6839E+006$
 $A_g = 427500.00$
 From (11.5.4.8), ACI 318-14: $V_s = V_{s,jacket} + V_{s,core} = 1.2262E+006$
 where:
 $V_{s,jacket} = V_{s,j1} + V_{s,j2} = 1.0996E+006$
 $V_{s,j1} = 353429.174$ is calculated for section web jacket, with:
 $d = 360.00$
 $A_v = 157079.633$
 $f_y = 625.00$
 $s = 100.00$
 $V_{s,j1}$ is multiplied by $Col,j1 = 1.00$
 $s/d = 0.27777778$
 $V_{s,j2} = 746128.255$ is calculated for section flange jacket, with:
 $d = 760.00$
 $A_v = 157079.633$
 $f_y = 625.00$
 $s = 100.00$
 $V_{s,j2}$ is multiplied by $Col,j2 = 1.00$
 $s/d = 0.13157895$
 $V_{s,core} = V_{s,c1} + V_{s,c2} = 126669.016$
 $V_{s,c1} = 0.00$ is calculated for section web core, with:
 $d = 200.00$
 $A_v = 100530.965$
 $f_y = 525.00$
 $s = 250.00$
 $V_{s,c1}$ is multiplied by $Col,c1 = 0.00$
 $s/d = 1.25$
 $V_{s,c2} = 126669.016$ is calculated for section flange core, with:
 $d = 600.00$
 $A_v = 100530.965$
 $f_y = 525.00$
 $s = 250.00$
 $V_{s,c2}$ is multiplied by $Col,c2 = 1.00$
 $s/d = 0.41666667$
 $V_f ((11-3)-(11.4), ACI 440) = 0.00$
 From (11-11), ACI 440: $V_s + V_f \leq 885757.128$
 $bw = 450.00$

End Of Calculation of Shear Capacity ratio for element: column JTC1 of floor 1
 At local axis: 2

Start Of Calculation of Chord Rotation Capacity for element: column JTC1 of floor 1

At local axis: 2

Integration Section: (b)

Section Type: rcjtcs

Constant Properties

Knowledge Factor, $\gamma = 0.80$

Chord Rotation is generally considered as Deformation-Controlled Action according to Table C7-1, ASCE 41-17.

Mean strengths are used for Deformation-Controlled Actions according to 7.5.1.3, ASCE 41-17

Consequently:

Jacket

New material of Primary Member: Concrete Strength, $f_c = f_{cm} = 18.00$

New material of Primary Member: Steel Strength, $f_s = f_{sm} = 625.00$

Concrete Elasticity, $E_c = 19940.411$

Steel Elasticity, $E_s = 200000.00$

Existing Column

Existing material of Primary Member: Concrete Strength, $f_c = f_{cm} = 12.00$

Existing material of Primary Member: Steel Strength, $f_s = f_{sm} = 525.00$

Concrete Elasticity, $E_c = 16281.278$

Steel Elasticity, $E_s = 200000.00$

Max Height, $H_{max} = 750.00$

Min Height, $H_{min} = 450.00$

Max Width, $W_{max} = 950.00$

Min Width, $W_{min} = 450.00$

Eccentricity, $E_{cc} = 250.00$

Jacket Thickness, $t_j = 100.00$

Cover Thickness, $c = 25.00$

Element Length, $L = 3000.00$

Primary Member

Ribbed Bars

Ductile Steel

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

Longitudinal Bars With Ends Lapped Starting at the End Sections

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

No FRP Wrapping

Stepwise Properties

Bending Moment, $M = 8.0135E+008$

Shear Force, $V_2 = 212214.235$

Shear Force, $V_3 = 90512.847$

Axial Force, $F = -7.0682E+006$

Longitudinal Reinforcement Area Distribution (in 2 divisions)

-Tension: $A_{sl,t} = 515.2212$

-Compression: $A_{sl,c} = 6176.371$

Longitudinal Reinforcement Area Distribution (in 3 divisions)

-Tension: $A_{sl,ten} = 1539.38$

-Compression: $A_{sl,com} = 2475.575$

-Middle: $A_{sl,mid} = 2676.637$

Longitudinal External Reinforcement Area Distribution (in 3 divisions)

-Tension: $A_{sl,ten,jacket} = 1231.504$

-Compression: $A_{sl,com,jacket} = 1859.823$

-Middle: $A_{sl,mid,jacket} = 2060.885$

Longitudinal Internal Reinforcement Area Distribution (in 3 divisions)

-Tension: $A_{sl,ten,core} = 307.8761$

-Compression: $A_{sl,com,core} = 615.7522$

-Middle: $A_{sl,mid,core} = 615.7522$

Mean Diameter of Tension Reinforcement, $Db_L = 16.57143$

Existing component: From table 7-7, ASCE 41_17: Final chord rotation Capacity $\phi_{u,R} = \phi_u = 0.01035216$

$$u = y + p = 0.0129402$$

- Calculation of y -

$$y = (M_y * L_s / 3) / E_{eff} = 0.0129402 \text{ ((4.29), Biskinis Phd)}$$

$$M_y = 1.9528E+009$$

$$L_s = M/V \text{ (with } L_s > 0.1 * L \text{ and } L_s < 2 * L) = 6000.00$$

$$\text{From table 10.5, ASCE 41_17: } E_{eff} = \text{factor} * E_c * I_g = 3.0181E+014$$

$$\text{factor} = 0.70$$

$$A_g = 562500.00$$

$$\text{Mean concrete strength: } f_c' = (f_c'_{\text{jacket}} * A_{\text{jacket}} + f_c'_{\text{core}} * A_{\text{core}}) / A_{\text{section}} = 15.20$$

$$N = 7.0682E+006$$

$$E_c * I_g = E_{c_{\text{jacket}}} * I_{g_{\text{jacket}}} + E_{c_{\text{core}}} * I_{g_{\text{core}}} = 4.3116E+014$$

Calculation of Yielding Moment M_y

Calculation of y and M_y according to Annex 7 -

Assuming neutral axis within flange ($y < t/d$, compression zone rectangular) with:

$$\text{flange width, } b = 950.00$$

$$\text{web width, } b_w = 450.00$$

$$\text{flange thickness, } t = 450.00$$

$$y = \text{Min}(y_{\text{ten}}, y_{\text{com}})$$

$$y_{\text{ten}} = 7.6761198E-006$$

$$\text{with } f_y = 601.9953$$

$$d = 707.00$$

$$y = 0.44537179$$

$$A = 0.02744412$$

$$B = 0.02211109$$

$$\text{with } p_t = 0.00229194$$

$$p_c = 0.00368581$$

$$p_v = 0.00398517$$

$$N = 7.0682E+006$$

$$b = 950.00$$

$$" = 0.06082037$$

$$y_{\text{comp}} = 3.8074256E-006$$

$$\text{with } f_c = 18.00$$

$$E_c = 19940.411$$

$$y = 0.60361504$$

$$A = -0.02242057$$

$$B = 0.00462988$$

$$\text{with } E_s = 200000.00$$

$$\text{CONFIRMATION: } y = 0.60361504 < t/d$$

Calculation of ratio I_b/I_d

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

- Calculation of p -

From table 10-8: $p = 0.00$

with:

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

$$\text{shear control ratio } V_y E / V_{col} E = 1.74773$$

$$d = d_{\text{external}} = 707.00$$

$$s = s_{\text{external}} = 50.00$$

$$- t = s_1 + s_2 + 2 * t_f / b_w * (f_{fe} / f_s) = 0.0035764$$

$$\text{jacket: } s_1 = A_{v1} * L_{\text{stir1}} / (s_1 * A_g) = 0.00301593$$

$$A_{v1} = 78.53982, \text{ is the area of every stirrup parallel to loading (shear) direction}$$

$$L_{\text{stir1}} = 2160.00, \text{ is the total Length of all stirrups parallel to loading (shear) direction}$$

$$s_1 = 100.00$$

core: $s_2 = A_{v2} \cdot L_{stir2} / (s_2 \cdot A_g) = 0.00056047$

$A_{v2} = 50.26548$, is the area of every stirrup parallel to loading (shear) direction

$L_{stir2} = 1568.00$, is the total Length of all stirrups parallel to loading (shear) direction

$s_2 = 250.00$

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

where $f = 2 \cdot t_f / b_w$ is FRP ratio (EC8 - 3, A.4.4.3(6)) and f_{fe} / f_s normalises f to steel strength

All these variables have already been given in Shear control ratio calculation.

For the normalisation f_s of jacket is used.

$N_{UD} = 7.0682E+006$

$A_g = 562500.00$

$f_{cE} = (f_{c,jacket} \cdot Area_{jacket} + f_{c,core} \cdot Area_{core}) / section_area = 15.20$

$f_{yIE} = (f_{y,ext_Long_Reinf} \cdot Area_{ext_Long_Reinf} + f_{y,int_Long_Reinf} \cdot Area_{int_Long_Reinf}) / Area_{Tot_Long_Rein} = 601.9953$

$f_{yIE} = (f_{y,ext_Trans_Reinf} \cdot s_1 + f_{y,int_Trans_Reinf} \cdot s_2) / (s_1 + s_2) = 609.3286$

$\rho_l = Area_{Tot_Long_Rein} / (b \cdot d) = 0.00996292$

$b = 950.00$

$d = 707.00$

$f_{cE} = 15.20$

End Of Calculation of Chord Rotation Capacity for element: column JTC1 of floor 1

At local axis: 2

Integration Section: (b)

Calculation No. 15

column C1, Floor 1

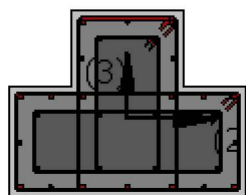
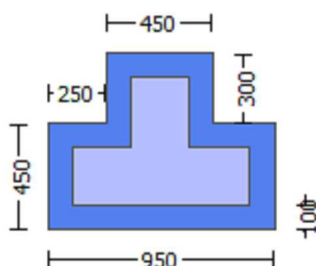
Limit State: Life Safety (data interpolation between analysis steps 50 and 51)

Analysis: Uniform +X

Check: Shear capacity V_{Rd}

Edge: End

Local Axis: (3)



Start Of Calculation of Shear Capacity for element: column JTC1 of floor 1

At local axis: 3

Integration Section: (b)

Section Type: rcjtcs

Constant Properties

Knowledge Factor, $\gamma = 0.80$

Member Shear Force is generally considered as Force-Controlled Action according to Table C7-1, ASCE 41-17.

Lower-bound strengths are used for Force-Controlled Actions according to 7.5.1.3, ASCE 41-17

Consequently:

Jacket

New material of Primary Member: Concrete Strength, $f_c = f_{c_lower_bound} = 12.00$

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

Concrete Elasticity, $E_c = 19940.411$

Steel Elasticity, $E_s = 200000.00$

Existing Column

Existing material of Primary Member: Concrete Strength, $f_c = f_{c_lower_bound} = 8.00$

Existing material of Primary Member: Steel Strength, $f_s = f_{s_lower_bound} = 420.00$

Concrete Elasticity, $E_c = 16281.278$

Steel Elasticity, $E_s = 200000.00$

#####

Note: Especially for the calculation of γ for displacement ductility demand,

the expected (mean value) strengths are used (7.5.1.3, ASCE 41-17) because bending is considered as

Deformation-Controlled Action (Table C7-1, ASCE 41-17).

Jacket

New material: Concrete Strength, $f_c = f_{cm} = 18.00$

New material: Steel Strength, $f_s = f_{sm} = 625.00$

Existing Column

Existing material: Concrete Strength, $f_c = f_{cm} = 12.00$

Existing material: Steel Strength, $f_s = f_{sm} = 525.00$

#####

Max Height, $H_{max} = 750.00$

Min Height, $H_{min} = 450.00$

Max Width, $W_{max} = 950.00$

Min Width, $W_{min} = 450.00$

Eccentricity, $E_{cc} = 250.00$

Jacket Thickness, $t_j = 100.00$

Cover Thickness, $c = 25.00$

Element Length, $L = 3000.00$

Primary Member

Ribbed Bars

Ductile Steel

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

Longitudinal Bars With Ends Lapped Starting at the End Sections

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

No FRP Wrapping

Stepwise Properties

EDGE -A-

Bending Moment, $M_a = -1.4174E+009$

Shear Force, $V_a = -90512.847$

EDGE -B-

Bending Moment, $M_b = 8.0135E+008$

Shear Force, $V_b = 90512.847$

BOTH EDGES

Axial Force, $F = -7.0682E+006$

Longitudinal Reinforcement Area Distribution (in 2 divisions)

-Tension: $A_{sl,t} = 515.2212$

-Compression: $A_{sl,c} = 6176.371$

Longitudinal Reinforcement Area Distribution (in 3 divisions)

-Tension: $A_{sl,ten} = 1539.38$

-Compression: $A_{sl,com} = 2475.575$

-Middle: $A_{sl,mid} = 2676.637$

Mean Diameter of Tension Reinforcement, $Db_{L,ten} = 16.57143$

Existing component: From table 7-7, ASCE 41-17: Final Shear Capacity $V_R = V_n = 780168.904$
 V_n ((10.3), ASCE 41-17) = $k_n V_{Col} = 975211.13$
 $V_{Col} = 975211.13$
 $k_n = 1.00$
 $displacement_ductility_demand = 0.31138445$

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

= 1 (normal-weight concrete)
 Mean concrete strength: $f_c' = (f_c'_{jacket} Area_{jacket} + f_c'_{core} Area_{core}) / Area_{section} = 10.13333$, but $f_c'^{0.5} \leq 8.3$ MPa (22.5.3.1, ACI 318-14)
 $M/Vd = 4.00$
 $\mu_u = 8.0135E+008$
 $V_u = 90512.847$
 $d = 0.8 \cdot h = 600.00$
 $N_u = 7.0682E+006$
 $A_g = 337500.00$
 From (11.5.4.8), ACI 318-14: $V_s = V_{sjacket} + V_{s,core} = 828294.726$
 where:
 $V_{sjacket} = V_{sj1} + V_{sj2} = 753982.237$
 $V_{sj1} = 471238.898$ is calculated for section web jacket, with:
 $d = 600.00$
 $A_v = 157079.633$
 $f_y = 500.00$
 $s = 100.00$
 V_{sj1} is multiplied by $Col,j1 = 1.00$
 $s/d = 0.16666667$
 $V_{sj2} = 282743.339$ is calculated for section flange jacket, with:
 $d = 360.00$
 $A_v = 157079.633$
 $f_y = 500.00$
 $s = 100.00$
 V_{sj2} is multiplied by $Col,j2 = 1.00$
 $s/d = 0.27777778$
 $V_{s,core} = V_{s,c1} + V_{s,c2} = 74312.489$
 $V_{s,c1} = 74312.489$ is calculated for section web core, with:
 $d = 440.00$
 $A_v = 100530.965$
 $f_y = 420.00$
 $s = 250.00$
 $V_{s,c1}$ is multiplied by $Col,c1 = 1.00$
 $s/d = 0.56818182$
 $V_{s,c2} = 0.00$ is calculated for section flange core, with:
 $d = 200.00$
 $A_v = 100530.965$
 $f_y = 420.00$
 $s = 250.00$
 $V_{s,c2}$ is multiplied by $Col,c2 = 0.00$
 $s/d = 1.25$
 V_f ((11-3)-(11.4), ACI 440) = 0.00
 From (11-11), ACI 440: $V_s + V_f \leq 570961.316$
 $bw = 450.00$

$displacement_ductility_demand$ is calculated as δ / y

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

From analysis, chord rotation $\theta = 0.00402938$
 $y = (M_y \cdot L_s / 3) / E_{eff} = 0.0129402$ ((4.29), Biskinis Phd))
 $M_y = 1.9528E+009$
 $L_s = M/V$ (with $L_s > 0.1 \cdot L$ and $L_s < 2 \cdot L$) = 6000.00

From table 10.5, ASCE 41_17: $E_{eff} = factor * E_c * I_g = 3.0181E+014$

factor = 0.70

$A_g = 562500.00$

Mean concrete strength: $f_c' = (f_c'_{jacket} * Area_{jacket} + f_c'_{core} * Area_{core}) / Area_{section} = 15.20$

$N = 7.0682E+006$

$E_c * I_g = E_{c_jacket} * I_{g_jacket} + E_{c_core} * I_{g_core} = 4.3116E+014$

Calculation of Yielding Moment M_y

Calculation of y and M_y according to Annex 7 -

Assuming neutral axis within flange ($y < t/d$, compression zone rectangular) with:

flange width, $b = 950.00$

web width, $b_w = 450.00$

flange thickness, $t = 450.00$

$y = \text{Min}(y_{ten}, y_{com})$

$y_{ten} = 7.6761198E-006$

with $f_y = 601.9953$

$d = 707.00$

$y = 0.44537179$

$A = 0.02744412$

$B = 0.02211109$

with $pt = 0.00229194$

$pc = 0.00368581$

$pv = 0.00398517$

$N = 7.0682E+006$

$b = 950.00$

" = 0.06082037

$y_{comp} = 3.8074256E-006$

with $f_c = 18.00$

$E_c = 19940.411$

$y = 0.60361504$

$A = -0.02242057$

$B = 0.00462988$

with $E_s = 200000.00$

CONFIRMATION: $y = 0.60361504 < t/d$

Calculation of ratio I_b/I_d

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

End Of Calculation of Shear Capacity for element: column JTC1 of floor 1

At local axis: 3

Integration Section: (b)