

Detailed Member Calculations

Units: N&mm

Regulation: ASCE 41-17

Calculation No. 1

column C1, Floor 1

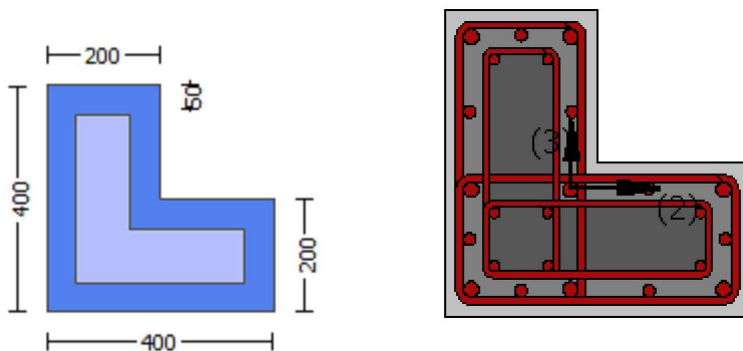
Limit State: Operational Level (data interpolation between analysis steps 1 and 2)

Analysis: Uniform +X

Check: Shear capacity V_{Rd}

Edge: Start

Local Axis: (2)



Start Of Calculation of Shear Capacity for element: column JLC1 of floor 1

At local axis: 2

Integration Section: (a)

Section Type: rcjls

Constant Properties

Knowledge Factor, $\gamma = 0.90$

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

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

Consequently:

Jacket

New material of Secondary Member: Concrete Strength, $f_c = f_{c_lower_bound} = 20.00$

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

Concrete Elasticity, $E_c = 25742.96$
Steel Elasticity, $E_s = 200000.00$
Existing Column
Existing material of Secondary Member: Concrete Strength, $f_c = f_{c_lower_bound} = 12.00$
Existing material of Secondary Member: Steel Strength, $f_s = f_{s_lower_bound} = 400.00$
Concrete Elasticity, $E_c = 19940.411$
Steel Elasticity, $E_s = 200000.00$

Note: Especially for the calculation of μ 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, $f_c = f_{cm} = 30.00$
New material: Steel Strength, $f_s = f_{sm} = 625.00$
Existing Column
Existing material: Concrete Strength, $f_c = f_{cm} = 18.00$
Existing material: Steel Strength, $f_s = f_{sm} = 500.00$

Max Height, $H_{max} = 400.00$
Min Height, $H_{min} = 200.00$
Max Width, $W_{max} = 400.00$
Min Width, $W_{min} = 200.00$
Jacket Thickness, $t_j = 50.00$
Cover Thickness, $c = 15.00$
Element Length, $L = 3000.00$
Secondary Member
Smooth Bars
Ductile Steel
Without Detailing for Earthquake Resistance (including stirrups not 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, $M_a = 8.5589E+007$
Shear Force, $V_a = -0.60572401$
EDGE -B-
Bending Moment, $M_b = -6.4291E+007$
Shear Force, $V_b = 0.60572401$
BOTH EDGES
Axial Force, $F = -2.1578E+006$
Longitudinal Reinforcement Area Distribution (in 2 divisions)
-Tension: $A_{st} = 0.00$
-Compression: $A_{sc} = 5353.274$
Longitudinal Reinforcement Area Distribution (in 3 divisions)
-Tension: $A_{st,ten} = 1137.257$
-Compression: $A_{st,com} = 2208.54$
-Middle: $A_{st,mid} = 2007.478$
Mean Diameter of Tension Reinforcement, $Db_{L,ten} = 16.80$

Existing component: From table 7-7, ASCE 41_17: Final Shear Capacity $V_R = V_n = 267030.06$
 $V_n ((10.3), ASCE 41-17) = k_n \cdot V_{CoI} = 296700.066$
 $V_{CoI} = 296700.066$
 $k_n = 1.00$
displacement_ductility_demand = 0.07831286

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: $f_c' = (f_c'_{jacket} \cdot Area_{jacket} + f_c'_{core} \cdot Area_{core}) / Area_{section} = 16.66667$, but $f_c'^{0.5} \leq$

8.3 MPa (22.5.3.1, ACI 318-14)

$$M/Vd = 4.00$$

$$\mu_u = 8.5589E+007$$

$$V_u = 0.60572401$$

$$d = 0.8 \cdot h = 320.00$$

$$N_u = 2.1578E+006$$

$$A_g = 80000.00$$

$$\text{From (11.5.4.8), ACI 318-14: } V_s = V_{s,jacket} + V_{s,core} = 376991.118$$

where:

$$V_{s,jacket} = V_{s,j1} + V_{s,j2} = 376991.118$$

$V_{s,j1} = 125663.706$ is calculated for section web jacket, with:

$$d = 160.00$$

$$A_v = 157079.633$$

$$f_y = 500.00$$

$$s = 100.00$$

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

$$s/d = 0.625$$

$V_{s,j2} = 251327.412$ is calculated for section flange jacket, with:

$$d = 320.00$$

$$A_v = 157079.633$$

$$f_y = 500.00$$

$$s = 100.00$$

$$V_{s,j2} \text{ is multiplied by } Col,j2 = 1.00$$

$$s/d = 0.3125$$

$$V_{s,core} = V_{s,c1} + V_{s,c2} = 0.00$$

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

$$d = 80.00$$

$$A_v = 100530.965$$

$$f_y = 400.00$$

$$s = 250.00$$

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

$$s/d = 3.125$$

$V_{s,c2} = 0.00$ is calculated for section flange core, with:

$$d = 240.00$$

$$A_v = 100530.965$$

$$f_y = 400.00$$

$$s = 250.00$$

$$V_{s,c2} \text{ is multiplied by } Col,c2 = 0.00$$

$$s/d = 1.04167$$

$$V_f ((11-3)-(11.4), \text{ACI 440}) = 0.00$$

$$\text{From (11-11), ACI 440: } V_s + V_f \leq 173568.578$$

$$bw = 200.00$$

displacement ductility demand is calculated as δ_u / y

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

From analysis, chord rotation $\theta_r = 0.00329273$

$$y = (M_y \cdot L_s / 3) / E_{eff} = 0.0420459 ((4.29), \text{Biskinis Phd})$$

$$M_y = 5.2455E+008$$

$$L_s = M/V \text{ (with } L_s > 0.1 \cdot L \text{ and } L_s < 2 \cdot L) = 6000.00$$

$$\text{From table 10.5, ASCE 41_17: } E_{eff} = \text{factor} \cdot E_c \cdot I_g = 2.4951E+013$$

$$\text{factor} = 0.70$$

$$A_g = 120000.00$$

$$\text{Mean concrete strength: } f'_c = (f'_{c,jacket} \cdot \text{Area}_{jacket} + f'_{c,core} \cdot \text{Area}_{core}) / \text{Area}_{section} = 25.00$$

$$N = 2.1578E+006$$

$$E_c \cdot I_g = E_{c,jacket} \cdot I_{g,jacket} + E_{c,core} \cdot I_{g,core} = 3.5645E+013$$

Calculation of Yielding Moment M_y

Calculation of δ_u and M_y according to Annex 7 -

Assuming neutral axis within flange ($y < t/d$, compression zone rectangular) with:

flange width, $b = 400.00$

web width, $b_w = 200.00$

flange thickness, $t = 200.00$

$y = \text{Min}(y_{\text{ten}}, y_{\text{com}})$

$y_{\text{ten}} = 1.4921411\text{E-}005$

with $f_y = 596.2441$

$d = 367.00$

$y = 0.45559994$

$A = 0.06111896$

$B = 0.04120454$

with $p_t = 0.00774698$

$p_c = 0.01504455$

$p_v = 0.01367492$

$N = 2.1578\text{E+}006$

$b = 400.00$

$" = 0.08991826$

$y_{\text{comp}} = 1.1520096\text{E-}005$

with $f_c = 30.00$

$E_c = 25742.96$

$y = 0.49615013$

$A = 0.00143$

$B = 0.01655203$

with $E_s = 200000.00$

CONFIRMATION: $y = 0.49615013 < t/d$

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 JLC1 of floor 1

At local axis: 2

Integration Section: (a)

Calculation No. 2

column C1, Floor 1

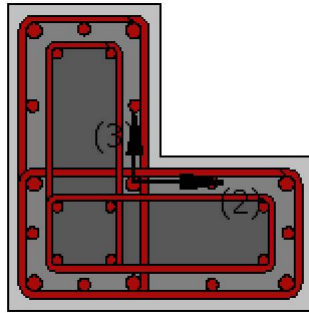
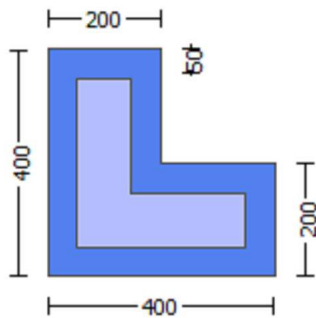
Limit State: Operational Level (data interpolation between analysis steps 1 and 2)

Analysis: Uniform +X

Check: Chord rotation capacity (θ)

Edge: Start

Local Axis: (2)



Start Of Calculation of Shear Capacity ratio for element: column JLC1 of floor 1

At Shear local axis: 3

(Bending local axis: 2)

Section Type: rcjlcs

Constant Properties

Knowledge Factor, $\gamma = 0.90$

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

Consequently:

Jacket

New material of Secondary Member: Concrete Strength, $f_c = f_{cm} = 30.00$

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

Concrete Elasticity, $E_c = 25742.96$

Steel Elasticity, $E_s = 200000.00$

Existing Column

Existing material of Secondary Member: Concrete Strength, $f_c = f_{cm} = 18.00$

Existing material of Secondary Member: Steel Strength, $f_s = f_{sm} = 500.00$

Concrete Elasticity, $E_c = 19940.411$

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, $f_s = 1.25 \cdot f_{sm} = 625.00$

#####

Max Height, $H_{max} = 400.00$

Min Height, $H_{min} = 200.00$

Max Width, $W_{max} = 400.00$

Min Width, $W_{min} = 200.00$

Jacket Thickness, $t_j = 50.00$

Cover Thickness, $c = 15.00$

Mean Confinement Factor overall section = 1.00

Element Length, $L = 3000.00$

Secondary Member

Smooth Bars

Ductile Steel

Without Detailing for Earthquake Resistance (including stirrups not 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, $V_a = -1.4594946E-005$

EDGE -B-

Shear Force, $V_b = 1.4594946E-005$

BOTH EDGES

Axial Force, $F = -2.1578\text{E}+006$

Longitudinal Reinforcement Area Distribution (in 2 divisions)

-Tension: $As_t = 0.00$

-Compression: $As_c = 5353.274$

Longitudinal Reinforcement Area Distribution (in 3 divisions)

-Tension: $As_{t,ten} = 2208.54$

-Compression: $As_{c,com} = 1137.257$

-Middle: $As_{mid} = 2007.478$

Calculation of Shear Capacity ratio, $V_e/V_r = 1.37616$

Member Controlled by Shear ($V_e/V_r > 1$)

Calculation of Shear Demand from fig. R18.6.5, ACI 318-14 $V_e = (M_{pr1} + M_{pr2})/l_n = 481543.08$
with

$M_{pr1} = \text{Max}(Mu_{1+}, Mu_{1-}) = 7.2231\text{E}+008$

$Mu_{1+} = 7.2231\text{E}+008$, is the ultimate moment strength at the edge 1 of the member in the actual moment direction which is defined for the static loading combination

$Mu_{1-} = 6.3496\text{E}+008$, 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

$M_{pr2} = \text{Max}(Mu_{2+}, Mu_{2-}) = 7.2231\text{E}+008$

$Mu_{2+} = 7.2231\text{E}+008$, 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

$Mu_{2-} = 6.3496\text{E}+008$, 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 Mu_{1+}

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

$\phi_u = 3.9059244\text{E}-005$

$M_u = 7.2231\text{E}+008$

with full section properties:

$b = 400.00$

$d = 367.00$

$d' = 33.00$

$v = 0.48996389$

$N = 2.1578\text{E}+006$

$f_c = 30.00$

ϕ_c (5A.5, TBDY) = 0.002

Final value of ϕ_u : $\phi_u^* = \text{shear_factor} * \text{Max}(\phi_u, \phi_c) = 0.00925707$

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

From (5.4b), TBDY: $\phi_u = 0.00925707$

we (5.4c) = 0.0207149

a_{se} ((5.4d), TBDY) = $(a_{se1} * A_{ext} + a_{se2} * A_{int}) / A_{sec} = 0.33836562$

$a_{se1} = \text{Max}(((A_{conf,max1} - A_{noConf1}) / A_{conf,max1}) * (A_{conf,min1} / A_{conf,max1}), 0) = 0.33836562$

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.

$A_{conf,max1} = 89600.00$ 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.

$A_{conf,min1} = 59225.00$ 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.

$A_{noConf1} = 43733.333$ is the unconfined external core area which is equal to $b_i^2/6$ as defined at (A.2).

$a_{se2} (>= a_{se1}) = \text{Max}(((A_{conf,max2} - A_{noConf2}) / A_{conf,max2}) * (A_{conf,min2} / A_{conf,max2}), 0) = 0.33836562$

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.

$A_{conf,max2} = 45264.00$ 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.

$A_{conf,min2} = 0.00$ 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.

$A_{noConf2} = 32309.333$ is the unconfined internal core area which is equal to $b_i^2/6$ as defined at (A.2).

$p_{sh,min} \cdot F_{ywe} = \text{Min}(p_{sh,x} \cdot F_{ywe}, p_{sh,y} \cdot F_{ywe}) = 6.12205$

Expression (5.4d) for $p_{sh,min} \cdot F_{ywe}$ has been multiplied by 0.3 according to 15.7.1.3 for members without earthquake detailing (90° closed stirrups)

$p_{sh,x} \cdot F_{ywe} = p_{sh1} \cdot F_{ywe1} + p_{sh2} \cdot F_{ywe2} = 6.12205$

$p_{sh1} \text{ ((5.4d), TBDY)} = L_{stir1} \cdot A_{stir1} / (A_{sec} \cdot s_1) = 0.00680678$

L_{stir1} (Length of stirrups along Y) = 1040.00

A_{stir1} (stirrups area) = 78.53982

$p_{sh2} \text{ (5.4d)} = L_{stir2} \cdot A_{stir2} / (A_{sec} \cdot s_2) = 0.0012868$

L_{stir2} (Length of stirrups along Y) = 768.00

A_{stir2} (stirrups area) = 50.26548

$p_{sh,y} \cdot F_{ywe} = p_{sh1} \cdot F_{ywe1} + p_{sh2} \cdot F_{ywe2} = 6.12205$

$p_{sh1} \text{ ((5.4d), TBDY)} = L_{stir1} \cdot A_{stir1} / (A_{sec} \cdot s_1) = 0.00680678$

L_{stir1} (Length of stirrups along X) = 1040.00

A_{stir1} (stirrups area) = 78.53982

$p_{sh2} \text{ ((5.4d), TBDY)} = L_{stir2} \cdot A_{stir2} / (A_{sec} \cdot s_2) = 0.0012868$

L_{stir2} (Length of stirrups along X) = 768.00

A_{stir2} (stirrups area) = 50.26548

$A_{sec} = 120000.00$

$s_1 = 100.00$

$s_2 = 250.00$

$f_{ywe1} = 781.25$

$f_{ywe2} = 625.00$

$f_{ce} = 30.00$

From ((5.A5), TBDY), TBDY: $c_c = 0.002$

c = confinement factor = 1.00

$y_1 = 0.0025$

$sh_1 = 0.008$

$ft_1 = 898.293$

$fy_1 = 748.5775$

$su_1 = 0.032$

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

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

$su_1 = 0.4 \cdot esu1_{nominal} \text{ ((5.5), TBDY)} = 0.032$

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

For calculation of $esu1_{nominal}$ 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 $fs_1 = (f_{sjacket} \cdot A_{sl,ten,jacket} + f_{s,core} \cdot A_{sl,ten,core}) / A_{sl,ten} = 748.5775$

with $Es_1 = (E_{sjacket} \cdot A_{sl,ten,jacket} + E_{s,core} \cdot A_{sl,ten,core}) / A_{sl,ten} = 200000.00$

$y_2 = 0.0025$

$sh_2 = 0.008$

$ft_2 = 886.7403$

$fy_2 = 738.9503$

$su_2 = 0.032$

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

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

$su_2 = 0.4 \cdot esu2_{nominal} \text{ ((5.5), TBDY)} = 0.032$

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

For calculation of $esu2_{nominal}$ 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 $\text{Min}(1, 1.25 \cdot (l_b/l_d)^{2/3})$, from 10.3.5, ASCE 41-17.

with $fs_2 = (f_{sjacket} \cdot A_{sl,com,jacket} + f_{s,core} \cdot A_{sl,com,core}) / A_{sl,com} = 738.9503$

with $Es_2 = (E_{sjacket} \cdot A_{sl,com,jacket} + E_{s,core} \cdot A_{sl,com,core}) / A_{sl,com} = 200000.00$

```

yv = 0.0025
shv = 0.008
ftv = 894.3662
fyv = 745.3052
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 = 745.3052
    with Esv = (Es,jacket*Asl,mid,jacket + Es,mid*Asl,mid,core)/Asl,mid = 200000.00
    1 = Asl,ten/(b*d)*(fs1/fc) = 0.37540035
    2 = Asl,com/(b*d)*(fs2/fc) = 0.19082108
    v = Asl,mid/(b*d)*(fsv/fc) = 0.33973286
and confined core properties:
b = 360.00
d = 347.00
d' = 13.00
fcc (5A.2, TBDY) = 30.00
cc (5A.5, TBDY) = 0.002
    c = confinement factor = 1.00
    1 = Asl,ten/(b*d)*(fs1/fc) = 0.44115251
    2 = Asl,com/(b*d)*(fs2/fc) = 0.22424379
    v = Asl,mid/(b*d)*(fsv/fc) = 0.39923778
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) = 0.64577857
MRc (4.17) = 7.2231E+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

--->

$*cu(4.11) = 0.67946111$

$M_{Ro}(4.18) = 4.2866E+008$

$M_{Ro} < 0.8*M_{Rc}$

--->

$u = cu$ (unconfined full section) = $3.9059244E-005$

$\mu_u = M_{Rc}$

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:

$u = 4.2871818E-005$

$\mu_u = 6.3496E+008$

with full section properties:

$b = 200.00$

$d = 367.00$

$d' = 33.00$

$v = 0.97992777$

$N = 2.1578E+006$

$f_c = 30.00$

α (5A.5, TBDY) = 0.002

Final value of cu : $cu^* = \text{shear_factor} * \text{Max}(cu, cc) = 0.00925707$

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

From (5.4b), TBDY: $cu = 0.00925707$

w_e (5.4c) = 0.0207149

α_{se} ((5.4d), TBDY) = $(\alpha_{se1}*A_{ext} + \alpha_{se2}*A_{int})/A_{sec} = 0.33836562$

$\alpha_{se1} = \text{Max}(((A_{conf,max1} - A_{noConf1})/A_{conf,max1}) * (A_{conf,min1}/A_{conf,max1}), 0) = 0.33836562$

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.

$A_{conf,max1} = 89600.00$ 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.

$A_{conf,min1} = 59225.00$ 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.

$A_{noConf1} = 43733.333$ is the unconfined external core area which is equal to $b_i^2/6$ as defined at (A.2).

$\alpha_{se2} (\geq \alpha_{se1}) = \text{Max}(((A_{conf,max2} - A_{noConf2})/A_{conf,max2}) * (A_{conf,min2}/A_{conf,max2}), 0) = 0.33836562$

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.

$A_{conf,max2} = 45264.00$ 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.

$A_{conf,min2} = 0.00$ 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.

$A_{noConf2} = 32309.333$ is the unconfined internal core area which is equal to $b_i^2/6$ as defined at (A.2).

$p_{sh,min}*F_{ywe} = \text{Min}(p_{sh,x}*F_{ywe}, p_{sh,y}*F_{ywe}) = 6.12205$

Expression (5.4d) for $p_{sh,min}*F_{ywe}$ has been multiplied by 0.3 according to 15.7.1.3 for members without earthquake detailing (90° closed stirrups)

$p_{sh,x}*F_{ywe} = p_{sh1}*F_{ywe1} + p_{sh2}*F_{ywe2} = 6.12205$

p_{sh1} ((5.4d), TBDY) = $L_{stir1}*A_{stir1}/(A_{sec}*s_1) = 0.00680678$

Lstir1 (Length of stirrups along Y) = 1040.00
Astir1 (stirrups area) = 78.53982
psh2 (5.4d) = Lstir2*Astir2/(Asec*s2) = 0.0012868
Lstir2 (Length of stirrups along Y) = 768.00
Astir2 (stirrups area) = 50.26548

psh_y*Fywe = psh1*Fywe1+ps2*Fywe2 = 6.12205
psh1 ((5.4d), TBDY) = Lstir1*Astir1/(Asec*s1) = 0.00680678
Lstir1 (Length of stirrups along X) = 1040.00
Astir1 (stirrups area) = 78.53982
psh2 ((5.4d), TBDY) = Lstir2*Astir2/(Asec*s2) = 0.0012868
Lstir2 (Length of stirrups along X) = 768.00
Astir2 (stirrups area) = 50.26548

Asec = 120000.00

s1 = 100.00

s2 = 250.00

fywe1 = 781.25

fywe2 = 625.00

fce = 30.00

From ((5.A5), TBDY), TBDY: cc = 0.002

c = confinement factor = 1.00

y1 = 0.0025

sh1 = 0.008

ft1 = 886.7403

fy1 = 738.9503

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 = 738.9503

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

y2 = 0.0025

sh2 = 0.008

ft2 = 898.293

fy2 = 748.5775

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 = 748.5775

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

yv = 0.0025

shv = 0.008

ftv = 894.3662

fyv = 745.3052

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  $f_{syv} = f_{sv}/1.2$ , 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  $f_{sv} = (f_{sjacket} \cdot A_{sl,mid,jacket} + f_{s,mid} \cdot A_{sl,mid,core}) / A_{sl,mid} = 745.3052$ 
with  $E_{sv} = (E_{sjacket} \cdot A_{sl,mid,jacket} + E_{s,mid} \cdot A_{sl,mid,core}) / A_{sl,mid} = 200000.00$ 
 $1 = A_{sl,ten} / (b \cdot d) \cdot (f_{s1}/f_c) = 0.38164216$ 
 $2 = A_{sl,com} / (b \cdot d) \cdot (f_{s2}/f_c) = 0.7508007$ 
 $v = A_{sl,mid} / (b \cdot d) \cdot (f_{sv}/f_c) = 0.67946571$ 
and confined core properties:
 $b = 160.00$ 
 $d = 347.00$ 
 $d' = 13.00$ 
 $f_{cc} \text{ (5A.2, TBDY)} = 30.00$ 
 $cc \text{ (5A.5, TBDY)} = 0.002$ 
 $c = \text{confinement factor} = 1.00$ 
 $1 = A_{sl,ten} / (b \cdot d) \cdot (f_{s1}/f_c) = 0.50454853$ 
 $2 = A_{sl,com} / (b \cdot d) \cdot (f_{s2}/f_c) = 0.99259314$ 
 $v = A_{sl,mid} / (b \cdot d) \cdot (f_{sv}/f_c) = 0.898285$ 
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
---->
 $c_u \text{ (4.10)} = 0.58834972$ 
 $M_{Rc} \text{ (4.17)} = 6.3496E+008$ 
---->
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:  $b_o, d_o, d'_o$ 
-  $N, 1, 2, v$  normalised to  $b_o \cdot d_o$ , instead of  $b \cdot d$ 
-  $f_{cc}, cc$  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
---->
 $c_u^* \text{ (4.11)} = 0.63603191$ 
 $M_{Ro} \text{ (4.18)} = 4.4768E+008$ 
 $M_{Ro} < 0.8 \cdot M_{Rc}$ 
---->
 $u = c_u \text{ (unconfined full section)} = 4.2871818E-005$ 
 $\mu = M_{Rc}$ 

```

Calculation of ratio l_b/l_d

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

Calculation of Mu2+

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

$$\phi_u = 3.9059244E-005$$

$$\mu_u = 7.2231E+008$$

with full section properties:

$$b = 400.00$$

$$d = 367.00$$

$$d' = 33.00$$

$$\nu = 0.48996389$$

$$N = 2.1578E+006$$

$$f_c = 30.00$$

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

$$\text{Final value of } \phi_u: \phi_u^* = \text{shear_factor} * \text{Max}(\phi_u, \phi_{co}) = 0.00925707$$

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

$$\text{From (5.4b), TBDY: } \phi_u = 0.00925707$$

$$\phi_{we} (5.4c) = 0.0207149$$

$$\phi_{ase} ((5.4d), TBDY) = (\phi_{ase1} * A_{ext} + \phi_{ase2} * A_{int}) / A_{sec} = 0.33836562$$

$$\phi_{ase1} = \text{Max}(((A_{conf,max1} - A_{noConf1}) / A_{conf,max1}) * (A_{conf,min1} / A_{conf,max1}), 0) = 0.33836562$$

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.

$A_{conf,max1} = 89600.00$ 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.

$A_{conf,min1} = 59225.00$ 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.

$A_{noConf1} = 43733.333$ is the unconfined external core area which is equal to $b_i^2/6$ as defined at (A.2).

$$\phi_{ase2} (\geq \phi_{ase1}) = \text{Max}(((A_{conf,max2} - A_{noConf2}) / A_{conf,max2}) * (A_{conf,min2} / A_{conf,max2}), 0) = 0.33836562$$

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.

$A_{conf,max2} = 45264.00$ 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.

$A_{conf,min2} = 0.00$ 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.

$A_{noConf2} = 32309.333$ is the unconfined internal core area which is equal to $b_i^2/6$ as defined at (A.2).

$$\phi_{psh,min} * F_{ywe} = \text{Min}(\phi_{psh,x} * F_{ywe}, \phi_{psh,y} * F_{ywe}) = 6.12205$$

Expression (5.4d) for $\phi_{psh,min} * F_{ywe}$ has been multiplied by 0.3 according to 15.7.1.3 for members without earthquake detailing (90° closed stirrups)

$$\phi_{psh,x} * F_{ywe} = \phi_{psh1} * F_{ywe1} + \phi_{psh2} * F_{ywe2} = 6.12205$$

$$\phi_{psh1} ((5.4d), TBDY) = L_{stir1} * A_{stir1} / (A_{sec} * s_1) = 0.00680678$$

$$L_{stir1} (\text{Length of stirrups along Y}) = 1040.00$$

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

$$\phi_{psh2} (5.4d) = L_{stir2} * A_{stir2} / (A_{sec} * s_2) = 0.0012868$$

$$L_{stir2} (\text{Length of stirrups along Y}) = 768.00$$

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

$$\phi_{psh,y} * F_{ywe} = \phi_{psh1} * F_{ywe1} + \phi_{psh2} * F_{ywe2} = 6.12205$$

$$\phi_{psh1} ((5.4d), TBDY) = L_{stir1} * A_{stir1} / (A_{sec} * s_1) = 0.00680678$$

$$L_{stir1} (\text{Length of stirrups along X}) = 1040.00$$

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

$$\phi_{psh2} ((5.4d), TBDY) = L_{stir2} * A_{stir2} / (A_{sec} * s_2) = 0.0012868$$

$$L_{stir2} (\text{Length of stirrups along X}) = 768.00$$

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

```

Asec = 120000.00
s1 = 100.00
s2 = 250.00
fywe1 = 781.25
fywe2 = 625.00
fce = 30.00
From ((5.A.5), TBDY), TBDY: cc = 0.002
c = confinement factor = 1.00
y1 = 0.0025
sh1 = 0.008
ft1 = 898.293
fy1 = 748.5775
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 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 = 748.5775
with Es1 = (Es,jacket*Asl,ten,jacket + Es,core*Asl,ten,core)/Asl,ten = 200000.00
y2 = 0.0025
sh2 = 0.008
ft2 = 886.7403
fy2 = 738.9503
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 = 738.9503
with Es2 = (Es,jacket*Asl,com,jacket + Es,core*Asl,com,core)/Asl,com = 200000.00
yv = 0.0025
shv = 0.008
ftv = 894.3662
fyv = 745.3052
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 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 = 745.3052
with Esv = (Es,jacket*Asl,mid,jacket + Es,mid*Asl,mid,core)/Asl,mid = 200000.00
1 = Asl,ten/(b*d)*(fs1/fc) = 0.37540035
2 = Asl,com/(b*d)*(fs2/fc) = 0.19082108
v = Asl,mid/(b*d)*(fsv/fc) = 0.33973286
and confined core properties:
b = 360.00
d = 347.00
d' = 13.00
fcc (5A.2, TBDY) = 30.00
cc (5A.5, TBDY) = 0.002

```

```

c = confinement factor = 1.00
1 = Asl,ten/(b*d)*(fs1/fc) = 0.44115251
2 = Asl,com/(b*d)*(fs2/fc) = 0.22424379
v = Asl,mid/(b*d)*(fsv/fc) = 0.39923778
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) = 0.64577857
MRc (4.17) = 7.2231E+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) = 0.67946111
MRo (4.18) = 4.2866E+008
MRo < 0.8*MRc
---->
u = cu (unconfined full section) = 3.9059244E-005
Mu = MRc
-----

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 = 4.2871818E-005
Mu = 6.3496E+008
-----

with full section properties:
b = 200.00

```

$d = 367.00$
 $d' = 33.00$
 $v = 0.97992777$
 $N = 2.1578E+006$
 $f_c = 30.00$
 $co (5A.5, TBDY) = 0.002$
 Final value of cu : $cu^* = shear_factor * Max(cu, cc) = 0.00925707$
 The Shear_factor is considered equal to 1 (pure moment strength)
 From (5.4b), TBDY: $cu = 0.00925707$
 $we (5.4c) = 0.0207149$
 $ase ((5.4d), TBDY) = (ase1 * A_{ext} + ase2 * A_{int}) / A_{sec} = 0.33836562$
 $ase1 = Max(((A_{conf,max1} - A_{noConf1}) / A_{conf,max1}) * (A_{conf,min1} / A_{conf,max1}), 0) = 0.33836562$
 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.
 $A_{conf,max1} = 89600.00$ 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.
 $A_{conf,min1} = 59225.00$ 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.
 $A_{noConf1} = 43733.333$ is the unconfined external core area which is equal to $b_i^2/6$ as defined at (A.2).
 $ase2 (>= ase1) = Max(((A_{conf,max2} - A_{noConf2}) / A_{conf,max2}) * (A_{conf,min2} / A_{conf,max2}), 0) = 0.33836562$
 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.
 $A_{conf,max2} = 45264.00$ 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.
 $A_{conf,min2} = 0.00$ 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.
 $A_{noConf2} = 32309.333$ is the unconfined internal core area which is equal to $b_i^2/6$ as defined at (A.2).
 $psh_{min} * F_{ywe} = Min(psh_x * F_{ywe}, psh_y * F_{ywe}) = 6.12205$
 Expression (5.4d) for $psh_{min} * F_{ywe}$ has been multiplied by 0.3 according to 15.7.1.3 for members without earthquake detailing (90° closed stirrups)

$psh_x * F_{ywe} = psh1 * F_{ywe1} + ps2 * F_{ywe2} = 6.12205$
 $psh1 ((5.4d), TBDY) = L_{stir1} * A_{stir1} / (A_{sec} * s1) = 0.00680678$
 L_{stir1} (Length of stirrups along Y) = 1040.00
 A_{stir1} (stirrups area) = 78.53982
 $psh2 (5.4d) = L_{stir2} * A_{stir2} / (A_{sec} * s2) = 0.0012868$
 L_{stir2} (Length of stirrups along Y) = 768.00
 A_{stir2} (stirrups area) = 50.26548

$psh_y * F_{ywe} = psh1 * F_{ywe1} + ps2 * F_{ywe2} = 6.12205$
 $psh1 ((5.4d), TBDY) = L_{stir1} * A_{stir1} / (A_{sec} * s1) = 0.00680678$
 L_{stir1} (Length of stirrups along X) = 1040.00
 A_{stir1} (stirrups area) = 78.53982
 $psh2 ((5.4d), TBDY) = L_{stir2} * A_{stir2} / (A_{sec} * s2) = 0.0012868$
 L_{stir2} (Length of stirrups along X) = 768.00
 A_{stir2} (stirrups area) = 50.26548

$A_{sec} = 120000.00$
 $s1 = 100.00$
 $s2 = 250.00$
 $f_{ywe1} = 781.25$
 $f_{ywe2} = 625.00$
 $f_{ce} = 30.00$
 From ((5.A5), TBDY), TBDY: $cc = 0.002$
 $c =$ confinement factor = 1.00
 $y1 = 0.0025$
 $sh1 = 0.008$
 $ft1 = 886.7403$
 $fy1 = 738.9503$

```

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 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 = 738.9503
with Es1 = (Es,jacket*Asl,ten,jacket + Es,core*Asl,ten,core)/Asl,ten = 200000.00
y2 = 0.0025
sh2 = 0.008
ft2 = 898.293
fy2 = 748.5775
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 = 748.5775
with Es2 = (Es,jacket*Asl,com,jacket + Es,core*Asl,com,core)/Asl,com = 200000.00
yv = 0.0025
shv = 0.008
ftv = 894.3662
fyv = 745.3052
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 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 = 745.3052
with Esv = (Es,jacket*Asl,mid,jacket + Es,mid*Asl,mid,core)/Asl,mid = 200000.00
1 = Asl,ten/(b*d)*(fs1/fc) = 0.38164216
2 = Asl,com/(b*d)*(fs2/fc) = 0.7508007
v = Asl,mid/(b*d)*(fsv/fc) = 0.67946571
and confined core properties:
b = 160.00
d = 347.00
d' = 13.00
fcc (5A.2, TBDY) = 30.00
cc (5A.5, TBDY) = 0.002
c = confinement factor = 1.00
1 = Asl,ten/(b*d)*(fs1/fc) = 0.50454853
2 = Asl,com/(b*d)*(fs2/fc) = 0.99259314
v = Asl,mid/(b*d)*(fsv/fc) = 0.898285
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
--->

ϕ_{cu} (4.10) = 0.58834972
 M_{Rc} (4.17) = 6.3496E+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: 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}, 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) = 0.63603191
 M_{Ro} (4.18) = 4.4768E+008
 $M_{Ro} < 0.8*M_{Rc}$

--->
 $u = \phi_{cu}$ (unconfined full section) = 4.2871818E-005
 $M_u = M_{Rc}$

Calculation of ratio I_b/I_d

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

Calculation of Shear Strength $V_r = \min(V_{r1}, V_{r2}) = 349917.706$

Calculation of Shear Strength at edge 1, $V_{r1} = 349917.706$

$V_{r1} = V_{Co1}$ ((10.3), ASCE 41-17) = $k_{nl} * V_{Co10}$
 $V_{Co10} = 349917.706$
 $k_{nl} = 1$ (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: $f'_c = (f'_c_{jacket} * Area_{jacket} + f'_c_{core} * Area_{core}) / Area_{section} = 25.00$, but $f'_c^{0.5} \leq 8.3$
MPa (22.5.3.1, ACI 318-14)
 $M/Vd = 4.00$
 $M_u = 3.4090E+006$
 $V_u = 1.4594946E-005$
 $d = 0.8 * h = 320.00$
 $N_u = 2.1578E+006$
 $A_g = 80000.00$
From (11.5.4.8), ACI 318-14: $V_s = V_{sjacket} + V_{s,core} = 471238.898$
where:
 $V_{sjacket} = V_{sj1} + V_{sj2} = 471238.898$

Vs,j1 = 314159.265 is calculated for section web jacket, with:

$$d = 320.00$$

$$A_v = 157079.633$$

$$f_y = 625.00$$

$$s = 100.00$$

Vs,j1 is multiplied by Col,j1 = 1.00

$$s/d = 0.3125$$

Vs,j2 = 157079.633 is calculated for section flange jacket, with:

$$d = 160.00$$

$$A_v = 157079.633$$

$$f_y = 625.00$$

$$s = 100.00$$

Vs,j2 is multiplied by Col,j2 = 1.00

$$s/d = 0.625$$

$$V_{s,core} = V_{s,c1} + V_{s,c2} = 0.00$$

Vs,c1 = 0.00 is calculated for section web core, with:

$$d = 240.00$$

$$A_v = 100530.965$$

$$f_y = 500.00$$

$$s = 250.00$$

Vs,c1 is multiplied by Col,c1 = 0.00

$$s/d = 1.04167$$

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

$$d = 80.00$$

$$A_v = 100530.965$$

$$f_y = 500.00$$

$$s = 250.00$$

Vs,c2 is multiplied by Col,c2 = 0.00

$$s/d = 3.125$$

$$V_f ((11-3)-(11.4), \text{ACI } 440) = 0.00$$

$$\text{From } (11-11), \text{ACI } 440: V_s + V_f \leq 212577.225$$

$$b_w = 200.00$$

Calculation of Shear Strength at edge 2, Vr2 = 349917.706

$$V_{r2} = V_{Col} ((10.3), \text{ASCE } 41-17) = k_{nl} * V_{Col0}$$

$$V_{Col0} = 349917.706$$

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

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).

$$= 1 \text{ (normal-weight concrete)}$$

$$\text{Mean concrete strength: } f'_c = (f'_c_{jacket} * \text{Area}_{jacket} + f'_c_{core} * \text{Area}_{core}) / \text{Area}_{section} = 25.00, \text{ but } f'_c^{0.5} \leq 8.3 \text{ MPa } (22.5.3.1, \text{ACI } 318-14)$$

$$M/Vd = 4.00$$

$$\mu_u = 15336.932$$

$$V_u = 1.4594946E+005$$

$$d = 0.8 * h = 320.00$$

$$N_u = 2.1578E+006$$

$$A_g = 80000.00$$

$$\text{From } (11.5.4.8), \text{ACI } 318-14: V_s = V_{sjacket} + V_{s,core} = 471238.898$$

where:

$$V_{sjacket} = V_{sj1} + V_{sj2} = 471238.898$$

Vs,j1 = 314159.265 is calculated for section web jacket, with:

$$d = 320.00$$

$$A_v = 157079.633$$

$$f_y = 625.00$$

$$s = 100.00$$

Vs,j1 is multiplied by Col,j1 = 1.00

$$s/d = 0.3125$$

Vs,j2 = 157079.633 is calculated for section flange jacket, with:

$$d = 160.00$$

$$A_v = 157079.633$$

$$f_y = 625.00$$

$$s = 100.00$$

Vs,j2 is multiplied by Col,j2 = 1.00

```

s/d = 0.625
Vs,core = Vs,c1 + Vs,c2 = 0.00
Vs,c1 = 0.00 is calculated for section web core, with:
d = 240.00
Av = 100530.965
fy = 500.00
s = 250.00
Vs,c1 is multiplied by Col,c1 = 0.00
s/d = 1.04167
Vs,c2 = 0.00 is calculated for section flange core, with:
d = 80.00
Av = 100530.965
fy = 500.00
s = 250.00
Vs,c2 is multiplied by Col,c2 = 0.00
s/d = 3.125
Vf ((11-3)-(11.4), ACI 440) = 0.00
From (11-11), ACI 440: Vs + Vf <= 212577.225
bw = 200.00
-----

End Of Calculation of Shear Capacity ratio for element: column JLC1 of floor 1
At local axis: 3
-----

Start Of Calculation of Shear Capacity ratio for element: column JLC1 of floor 1
At Shear local axis: 2
(Bending local axis: 3)
Section Type: rcjlc

Constant Properties
-----

Knowledge Factor,   = 0.90
Mean strength values are used for both shear and moment calculations.
Consequently:
Jacket
New material of Secondary Member: Concrete Strength, fc = fcm = 30.00
New material of Secondary Member: Steel Strength, fs = fsm = 625.00
Concrete Elasticity, Ec = 25742.96
Steel Elasticity, Es = 200000.00
Existing Column
Existing material of Secondary Member: Concrete Strength, fc = fcm = 18.00
Existing material of Secondary Member: Steel Strength, fs = fsm = 500.00
Concrete Elasticity, Ec = 19940.411
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 = 625.00
#####
Max Height, Hmax = 400.00
Min Height, Hmin = 200.00
Max Width, Wmax = 400.00
Min Width, Wmin = 200.00
Jacket Thickness, tj = 50.00
Cover Thickness, c = 15.00
Mean Confinement Factor overall section = 1.00
Element Length, L = 3000.00
Secondary Member
Smooth Bars
Ductile Steel
Without Detailing for Earthquake Resistance (including stirrups not 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: 2
EDGE -A-
Shear Force, $V_a = -5.7948283E-005$
EDGE -B-
Shear Force, $V_b = 5.7948283E-005$
BOTH EDGES
Axial Force, $F = -2.1578E+006$
Longitudinal Reinforcement Area Distribution (in 2 divisions)
-Tension: $As_t = 0.00$
-Compression: $As_c = 5353.274$
Longitudinal Reinforcement Area Distribution (in 3 divisions)
-Tension: $As_{t,ten} = 1137.257$
-Compression: $As_{c,com} = 2208.54$
-Middle: $As_{c,mid} = 2007.478$

Calculation of Shear Capacity ratio, $V_e/V_r = 1.10794$
Member Controlled by Shear ($V_e/V_r > 1$)
Calculation of Shear Demand from fig. R18.6.5, ACI 318-14 $V_e = (M_{pr1} + M_{pr2})/l_n = 387689.388$
with
 $M_{pr1} = \text{Max}(\mu_{u1+}, \mu_{u1-}) = 5.8153E+008$
 $\mu_{u1+} = 5.8153E+008$, is the ultimate moment strength at the edge 1 of the member in the actual moment direction which is defined for the static loading combination
 $\mu_{u1-} = 4.9554E+008$, 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
 $M_{pr2} = \text{Max}(\mu_{u2+}, \mu_{u2-}) = 5.8153E+008$
 $\mu_{u2+} = 5.8153E+008$, 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
 $\mu_{u2-} = 4.9554E+008$, 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 μ_u according to 4.1, Biskinis/Fardis 2013:
 $\mu_u = 1.1511372E-005$
 $\mu_u = 5.8153E+008$

with full section properties:

$b = 400.00$
 $d = 367.00$
 $d' = 33.00$
 $v = 0.48996389$
 $N = 2.1578E+006$
 $f_c = 30.00$
 $\alpha_1(5A.5, \text{TBDY}) = 0.002$
Final value of μ_u : $\mu_u^* = \text{shear_factor} * \text{Max}(\mu_u, \mu_c) = 0.00925707$
The Shear_factor is considered equal to 1 (pure moment strength)
From (5.4b), TBDY: $\mu_u = 0.00925707$
we (5.4c) = 0.0207149
 $\alpha_1(5.4d, \text{TBDY}) = (\alpha_1 * A_{ext} + \alpha_2 * A_{int})/A_{sec} = 0.33836562$
 $\alpha_1 = \text{Max}((A_{conf,max1} - A_{noConf1})/A_{conf,max1} * (A_{conf,min1}/A_{conf,max1}), 0) = 0.33836562$
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.

Aconf,max1 = 89600.00 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 = 59225.00 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 = 43733.333 is the unconfined external core area which is equal to $b_i^2/6$ as defined at (A.2).

ase2 (\geq ase1) = $\text{Max}(((A_{\text{conf,max2}} - A_{\text{noConf2}})/A_{\text{conf,max2}}) * (A_{\text{conf,min2}}/A_{\text{conf,max2}}), 0) = 0.33836562$

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 = 45264.00 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 = 0.00 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 = 32309.333 is the unconfined internal core area which is equal to $b_i^2/6$ as defined at (A.2).

psh,min*Fywe = $\text{Min}(psh_x * Fywe, psh_y * Fywe) = 6.12205$

Expression (5.4d) for psh,min*Fywe has been multiplied by 0.3 according to 15.7.1.3 for members without earthquake detailing (90° closed stirrups)

psh_x*Fywe = psh1*Fywe1 + ps2*Fywe2 = 6.12205

psh1 ((5.4d), TBDY) = $L_{\text{stir1}} * A_{\text{stir1}} / (A_{\text{sec}} * s_1) = 0.00680678$

Lstir1 (Length of stirrups along Y) = 1040.00

Astir1 (stirrups area) = 78.53982

psh2 (5.4d) = $L_{\text{stir2}} * A_{\text{stir2}} / (A_{\text{sec}} * s_2) = 0.0012868$

Lstir2 (Length of stirrups along Y) = 768.00

Astir2 (stirrups area) = 50.26548

psh_y*Fywe = psh1*Fywe1 + ps2*Fywe2 = 6.12205

psh1 ((5.4d), TBDY) = $L_{\text{stir1}} * A_{\text{stir1}} / (A_{\text{sec}} * s_1) = 0.00680678$

Lstir1 (Length of stirrups along X) = 1040.00

Astir1 (stirrups area) = 78.53982

psh2 ((5.4d), TBDY) = $L_{\text{stir2}} * A_{\text{stir2}} / (A_{\text{sec}} * s_2) = 0.0012868$

Lstir2 (Length of stirrups along X) = 768.00

Astir2 (stirrups area) = 50.26548

Asec = 120000.00

s1 = 100.00

s2 = 250.00

fywe1 = 781.25

fywe2 = 625.00

fce = 30.00

From ((5.A5), TBDY), TBDY: cc = 0.002

c = confinement factor = 1.00

y1 = 0.0025

sh1 = 0.008

ft1 = 886.7403

fy1 = 738.9503

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 = $(f_{s,jacket} * A_{s,ten,jacket} + f_{s,core} * A_{s,ten,core}) / A_{s,ten} = 738.9503$

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 = 898.293

fy2 = 748.5775

```

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 = 748.5775
with Es2 = (Es,jacket*Asl,com,jacket + Es,core*Asl,com,core)/Asl,com = 200000.00
yv = 0.0025
shv = 0.008
ftv = 894.3662
fyv = 745.3052
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 = 745.3052
with Esv = (Es,jacket*Asl,mid,jacket + Es,mid*Asl,mid,core)/Asl,mid = 200000.00
1 = Asl,ten/(b*d)*(fs1/fc) = 0.19082108
2 = Asl,com/(b*d)*(fs2/fc) = 0.37540035
v = Asl,mid/(b*d)*(fsv/fc) = 0.33973286
and confined core properties:
b = 360.00
d = 347.00
d' = 13.00
fcc (5A.2, TBDY) = 30.00
cc (5A.5, TBDY) = 0.002
c = confinement factor = 1.00
1 = Asl,ten/(b*d)*(fs1/fc) = 0.22424379
2 = Asl,com/(b*d)*(fs2/fc) = 0.44115251
v = Asl,mid/(b*d)*(fsv/fc) = 0.39923778
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) = 0.42533033
MRc (4.17) = 7.2277E+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) = 0.50069522

M_{Ro} (4.18) = 5.8153E+008

--->

$u = c_u$ (4.2) = 1.1511372E-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 μ_u according to 4.1, Biskinis/Fardis 2013:

$u = 3.0526570E-005$

$\mu_u = 4.9554E+008$

with full section properties:

$b = 200.00$

$d = 367.00$

$d' = 33.00$

$v = 0.97992777$

$N = 2.1578E+006$

$f_c = 30.00$

α (5A.5, TBDY) = 0.002

Final value of c_u : $c_u^* = \text{shear_factor} * \text{Max}(c_u, c_c) = 0.00925707$

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

From (5.4b), TBDY: $c_u = 0.00925707$

w_e (5.4c) = 0.0207149

α_{se} ((5.4d), TBDY) = $(\alpha_{se1} * A_{ext} + \alpha_{se2} * A_{int}) / A_{sec} = 0.33836562$

$\alpha_{se1} = \text{Max}(((A_{conf,max1} - A_{noConf1}) / A_{conf,max1}) * (A_{conf,min1} / A_{conf,max1}), 0) = 0.33836562$

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.

$A_{conf,max1} = 89600.00$ 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.

$A_{conf,min1} = 59225.00$ 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.

$A_{noConf1} = 43733.333$ is the unconfined external core area which is equal to $b_i^2/6$ as defined at (A.2).

$\alpha_{se2} (\geq \alpha_{se1}) = \text{Max}(((A_{conf,max2} - A_{noConf2}) / A_{conf,max2}) * (A_{conf,min2} / A_{conf,max2}), 0) = 0.33836562$

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.

$A_{conf,max2} = 45264.00$ 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.

$A_{conf,min2} = 0.00$ 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.

$A_{noConf2} = 32309.333$ is the unconfined internal core area which is equal to $b_i^2/6$ as defined at (A.2).

$psh_{min} \cdot F_{ywe} = \text{Min}(psh_x \cdot F_{ywe}, psh_y \cdot F_{ywe}) = 6.12205$

Expression (5.4d) for $psh_{min} \cdot F_{ywe}$ has been multiplied by 0.3 according to 15.7.1.3 for members without earthquake detailing (90° closed stirrups)

$psh_x \cdot F_{ywe} = psh1 \cdot F_{ywe1} + ps2 \cdot F_{ywe2} = 6.12205$

$psh1$ ((5.4d), TBDY) = $L_{stir1} \cdot A_{stir1} / (A_{sec} \cdot s1) = 0.00680678$

L_{stir1} (Length of stirrups along Y) = 1040.00

A_{stir1} (stirrups area) = 78.53982

$psh2$ (5.4d) = $L_{stir2} \cdot A_{stir2} / (A_{sec} \cdot s2) = 0.0012868$

L_{stir2} (Length of stirrups along Y) = 768.00

A_{stir2} (stirrups area) = 50.26548

$psh_y \cdot F_{ywe} = psh1 \cdot F_{ywe1} + ps2 \cdot F_{ywe2} = 6.12205$

$psh1$ ((5.4d), TBDY) = $L_{stir1} \cdot A_{stir1} / (A_{sec} \cdot s1) = 0.00680678$

L_{stir1} (Length of stirrups along X) = 1040.00

A_{stir1} (stirrups area) = 78.53982

$psh2$ ((5.4d), TBDY) = $L_{stir2} \cdot A_{stir2} / (A_{sec} \cdot s2) = 0.0012868$

L_{stir2} (Length of stirrups along X) = 768.00

A_{stir2} (stirrups area) = 50.26548

$A_{sec} = 120000.00$

$s1 = 100.00$

$s2 = 250.00$

$f_{ywe1} = 781.25$

$f_{ywe2} = 625.00$

$f_{ce} = 30.00$

From ((5.A5), TBDY), TBDY: $cc = 0.002$

c = confinement factor = 1.00

$y1 = 0.0025$

$sh1 = 0.008$

$ft1 = 898.293$

$fy1 = 748.5775$

$su1 = 0.032$

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

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

$su1 = 0.4 \cdot esu1_{nominal}$ ((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 $\text{Min}(1, 1.25 \cdot (lb/ld)^{2/3})$, from 10.3.5, ASCE 41-17.

with $fs1 = (fs_{jacket} \cdot A_{sl,ten,jacket} + fs_{core} \cdot A_{sl,ten,core}) / A_{sl,ten} = 748.5775$

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

$y2 = 0.0025$

$sh2 = 0.008$

$ft2 = 886.7403$

$fy2 = 738.9503$

$su2 = 0.032$

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 $fs2 = (fs_{jacket} \cdot A_{sl,com,jacket} + fs_{core} \cdot A_{sl,com,core}) / A_{sl,com} = 738.9503$

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 = 894.3662
fyv = 745.3052
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 = 745.3052
    with Esv = (Es,jacket*Asl,mid,jacket + Es,mid*Asl,mid,core)/Asl,mid = 200000.00
    1 = Asl,ten/(b*d)*(fs1/fc) = 0.7508007
    2 = Asl,com/(b*d)*(fs2/fc) = 0.38164216
    v = Asl,mid/(b*d)*(fsv/fc) = 0.67946571
and confined core properties:
b = 160.00
d = 347.00
d' = 13.00
fcc (5A.2, TBDY) = 30.00
cc (5A.5, TBDY) = 0.002
    c = confinement factor = 1.00
    1 = Asl,ten/(b*d)*(fs1/fc) = 0.99259314
    2 = Asl,com/(b*d)*(fs2/fc) = 0.50454853
    v = Asl,mid/(b*d)*(fsv/fc) = 0.898285
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) = 0.8262842
    MRc (4.18) = 4.9554E+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) = 0.90160009
MRO (4.18) = 2.4685E+008
MRO < 0.8*MRc

--->

u = cu (unconfined full section) = 3.0526570E-005
Mu = MRc

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 = 1.1511372E-005
Mu = 5.8153E+008

with full section properties:

b = 400.00
d = 367.00
d' = 33.00
v = 0.48996389
N = 2.1578E+006

fc = 30.00
co (5A.5, TBDY) = 0.002

Final value of cu: cu* = shear_factor * Max(cu, cc) = 0.00925707
The Shear_factor is considered equal to 1 (pure moment strength)
From (5.4b), TBDY: cu = 0.00925707

we (5.4c) = 0.0207149

ase ((5.4d), TBDY) = (ase1*Aext+ase2*Aint)/Asec = 0.33836562

ase1 = Max(((Aconf,max1-AnoConf1)/Aconf,max1)*(Aconf,min1/Aconf,max1),0) = 0.33836562

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 = 89600.00 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 = 59225.00 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 = 43733.333 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) = 0.33836562

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 = 45264.00 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 = 0.00 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 = 32309.333 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) = 6.12205

Expression (5.4d) for psh,min*Fywe has been multiplied by 0.3 according to 15.7.1.3 for members without earthquake detailing (90° closed stirrups)

psh_x*Fywe = psh1*Fywe1+ps2*Fywe2 = 6.12205

psh1 ((5.4d), TBDY) = Lstir1*Astir1/(Asec*s1) = 0.00680678

Lstir1 (Length of stirrups along Y) = 1040.00

Astir1 (stirrups area) = 78.53982

$psh_2 (5.4d) = Lstir_2 * Astir_2 / (Asec * s_2) = 0.0012868$
 $Lstir_2$ (Length of stirrups along Y) = 768.00
 $Astir_2$ (stirrups area) = 50.26548

$psh_y * Fywe = psh_1 * Fywe_1 + ps_2 * Fywe_2 = 6.12205$
 $psh_1 ((5.4d), TBDY) = Lstir_1 * Astir_1 / (Asec * s_1) = 0.00680678$
 $Lstir_1$ (Length of stirrups along X) = 1040.00
 $Astir_1$ (stirrups area) = 78.53982
 $psh_2 ((5.4d), TBDY) = Lstir_2 * Astir_2 / (Asec * s_2) = 0.0012868$
 $Lstir_2$ (Length of stirrups along X) = 768.00
 $Astir_2$ (stirrups area) = 50.26548

$Asec = 120000.00$

$s_1 = 100.00$

$s_2 = 250.00$

$fywe_1 = 781.25$

$fywe_2 = 625.00$

$fce = 30.00$

From ((5.A5), TBDY), TBDY: $cc = 0.002$

c = confinement factor = 1.00

$y_1 = 0.0025$

$sh_1 = 0.008$

$ft_1 = 886.7403$

$fy_1 = 738.9503$

$su_1 = 0.032$

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

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

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

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 $fs_1 = (fs_{jacket} * Asl_{ten,jacket} + fs_{core} * Asl_{ten,core}) / Asl_{ten} = 738.9503$

with $Es_1 = (Es_{jacket} * Asl_{ten,jacket} + Es_{core} * Asl_{ten,core}) / Asl_{ten} = 200000.00$

$y_2 = 0.0025$

$sh_2 = 0.008$

$ft_2 = 898.293$

$fy_2 = 748.5775$

$su_2 = 0.032$

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

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

$su_2 = 0.4 * esu_{2_nominal} ((5.5), TBDY) = 0.032$

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 $fs_2 = (fs_{jacket} * Asl_{com,jacket} + fs_{core} * Asl_{com,core}) / Asl_{com} = 748.5775$

with $Es_2 = (Es_{jacket} * Asl_{com,jacket} + Es_{core} * Asl_{com,core}) / Asl_{com} = 200000.00$

$y_v = 0.0025$

$sh_v = 0.008$

$ft_v = 894.3662$

$fy_v = 745.3052$

$suv = 0.032$

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 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 = 745.3052
with Esv = (Es,jacket*Asl,mid,jacket + Es,mid*Asl,mid,core)/Asl,mid = 200000.00
1 = Asl,ten/(b*d)*(fs1/fc) = 0.19082108
2 = Asl,com/(b*d)*(fs2/fc) = 0.37540035
v = Asl,mid/(b*d)*(fsv/fc) = 0.33973286
and confined core properties:
b = 360.00
d = 347.00
d' = 13.00
fcc (5A.2, TBDY) = 30.00
cc (5A.5, TBDY) = 0.002
c = confinement factor = 1.00
1 = Asl,ten/(b*d)*(fs1/fc) = 0.22424379
2 = Asl,com/(b*d)*(fs2/fc) = 0.44115251
v = Asl,mid/(b*d)*(fsv/fc) = 0.39923778
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) = 0.42533033
MRc (4.17) = 7.2277E+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) = 0.50069522
MRo (4.18) = 5.8153E+008
--->
u = cu (4.2) = 1.1511372E-005
Mu = MRo

```

Calculation of ratio lb/d

Adequate Lap Length: lb/d >= 1

Calculation of Mu2-

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

$$\mu = 3.0526570E-005$$

$$\mu_u = 4.9554E+008$$

with full section properties:

$$b = 200.00$$

$$d = 367.00$$

$$d' = 33.00$$

$$v = 0.97992777$$

$$N = 2.1578E+006$$

$$f_c = 30.00$$

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

$$\text{Final value of } \mu_u: \mu_u^* = \text{shear_factor} * \text{Max}(\mu_u, c_o) = 0.00925707$$

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

$$\text{From (5.4b), TBDY: } \mu_u = 0.00925707$$

$$\mu_{ue}(5.4c) = 0.0207149$$

$$\mu_{ase}((5.4d), TBDY) = (\mu_{ase1} * A_{ext} + \mu_{ase2} * A_{int}) / A_{sec} = 0.33836562$$

$$\mu_{ase1} = \text{Max}(((A_{conf,max1} - A_{noConf1}) / A_{conf,max1}) * (A_{conf,min1} / A_{conf,max1}), 0) = 0.33836562$$

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.

$A_{conf,max1} = 89600.00$ 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.

$A_{conf,min1} = 59225.00$ 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.

$A_{noConf1} = 43733.333$ is the unconfined external core area which is equal to $b_i^2/6$ as defined at (A.2).

$$\mu_{ase2} (> \mu_{ase1}) = \text{Max}(((A_{conf,max2} - A_{noConf2}) / A_{conf,max2}) * (A_{conf,min2} / A_{conf,max2}), 0) = 0.33836562$$

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.

$A_{conf,max2} = 45264.00$ 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.

$A_{conf,min2} = 0.00$ 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.

$A_{noConf2} = 32309.333$ is the unconfined internal core area which is equal to $b_i^2/6$ as defined at (A.2).

$$\mu_{psh,min} * F_{ywe} = \text{Min}(\mu_{psh,x} * F_{ywe}, \mu_{psh,y} * F_{ywe}) = 6.12205$$

Expression (5.4d) for $\mu_{psh,min} * F_{ywe}$ has been multiplied by 0.3 according to 15.7.1.3 for members without earthquake detailing (90° closed stirrups)

$$\mu_{psh,x} * F_{ywe} = \mu_{psh1} * F_{ywe1} + \mu_{psh2} * F_{ywe2} = 6.12205$$

$$\mu_{psh1}((5.4d), TBDY) = L_{stir1} * A_{stir1} / (A_{sec} * s1) = 0.00680678$$

$$L_{stir1} \text{ (Length of stirrups along Y)} = 1040.00$$

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

$$\mu_{psh2}((5.4d), TBDY) = L_{stir2} * A_{stir2} / (A_{sec} * s2) = 0.0012868$$

$$L_{stir2} \text{ (Length of stirrups along Y)} = 768.00$$

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

$$\mu_{psh,y} * F_{ywe} = \mu_{psh1} * F_{ywe1} + \mu_{psh2} * F_{ywe2} = 6.12205$$

$$\mu_{psh1}((5.4d), TBDY) = L_{stir1} * A_{stir1} / (A_{sec} * s1) = 0.00680678$$

$$L_{stir1} \text{ (Length of stirrups along X)} = 1040.00$$

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

$$\mu_{psh2}((5.4d), TBDY) = L_{stir2} * A_{stir2} / (A_{sec} * s2) = 0.0012868$$

$$L_{stir2} \text{ (Length of stirrups along X)} = 768.00$$

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

$$A_{sec} = 120000.00$$

$$s1 = 100.00$$

$s_2 = 250.00$
 $fy_{w1} = 781.25$
 $fy_{w2} = 625.00$
 $f_{ce} = 30.00$
 From ((5A.5), TBDY), TBDY: $cc = 0.002$
 $c = \text{confinement factor} = 1.00$
 $y_1 = 0.0025$
 $sh_1 = 0.008$
 $ft_1 = 898.293$
 $fy_1 = 748.5775$
 $su_1 = 0.032$
 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 $fs_1 = (fs_{\text{jacket}} * A_{sl, \text{ten, jacket}} + fs_{\text{core}} * A_{sl, \text{ten, core}}) / A_{sl, \text{ten}} = 748.5775$
 with $Es_1 = (Es_{\text{jacket}} * A_{sl, \text{ten, jacket}} + Es_{\text{core}} * A_{sl, \text{ten, core}}) / A_{sl, \text{ten}} = 200000.00$
 $y_2 = 0.0025$
 $sh_2 = 0.008$
 $ft_2 = 886.7403$
 $fy_2 = 738.9503$
 $su_2 = 0.032$
 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_2, sh_2, ft_2, fy_2 , are also multiplied by $\text{Min}(1, 1.25 * (lb/ld)^{2/3})$, from 10.3.5, ASCE 41-17.
 with $fs_2 = (fs_{\text{jacket}} * A_{sl, \text{com, jacket}} + fs_{\text{core}} * A_{sl, \text{com, core}}) / A_{sl, \text{com}} = 738.9503$
 with $Es_2 = (Es_{\text{jacket}} * A_{sl, \text{com, jacket}} + Es_{\text{core}} * A_{sl, \text{com, core}}) / A_{sl, \text{com}} = 200000.00$
 $y_v = 0.0025$
 $sh_v = 0.008$
 $ft_v = 894.3662$
 $fy_v = 745.3052$
 $suv = 0.032$
 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 $fsyv = fs_v/1.2$, 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 $\text{Min}(1, 1.25 * (lb/ld)^{2/3})$, from 10.3.5, ASCE 41-17.
 with $fs_v = (fs_{\text{jacket}} * A_{sl, \text{mid, jacket}} + fs_{\text{mid}} * A_{sl, \text{mid, core}}) / A_{sl, \text{mid}} = 745.3052$
 with $Es_v = (Es_{\text{jacket}} * A_{sl, \text{mid, jacket}} + Es_{\text{mid}} * A_{sl, \text{mid, core}}) / A_{sl, \text{mid}} = 200000.00$
 $1 = A_{sl, \text{ten}} / (b * d) * (fs_1 / f_c) = 0.7508007$
 $2 = A_{sl, \text{com}} / (b * d) * (fs_2 / f_c) = 0.38164216$
 $v = A_{sl, \text{mid}} / (b * d) * (fs_v / f_c) = 0.67946571$
 and confined core properties:
 $b = 160.00$
 $d = 347.00$
 $d' = 13.00$
 $f_{cc} \text{ (5A.2, TBDY)} = 30.00$
 $cc \text{ (5A.5, TBDY)} = 0.002$
 $c = \text{confinement factor} = 1.00$
 $1 = A_{sl, \text{ten}} / (b * d) * (fs_1 / f_c) = 0.99259314$
 $2 = A_{sl, \text{com}} / (b * d) * (fs_2 / f_c) = 0.50454853$

```

v = Asl,mid/(b*d)*(fsv/fc) = 0.898285
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) = 0.8262842
MRc (4.18) = 4.9554E+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) = 0.90160009
MRo (4.18) = 2.4685E+008
MRo < 0.8*MRc
---->
u = cu (unconfined full section) = 3.0526570E-005
Mu = MRc
-----

Calculation of ratio lb/lc
-----
Adequate Lap Length: lb/lc >= 1
-----
-----
-----
-----
Calculation of Shear Strength Vr = Min(Vr1,Vr2) = 349917.706
-----
Calculation of Shear Strength at edge 1, Vr1 = 349917.706
Vr1 = VCol ((10.3), ASCE 41-17) = knl*VCol0
VCol0 = 349917.706
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 = 25.00, but fc'^0.5 <= 8.3

```

MPa (22.5.3.1, ACI 318-14)

$$M/Vd = 4.00$$

$$\mu_u = 8.5591E+007$$

$$V_u = 5.7948283E-005$$

$$d = 0.8 \cdot h = 320.00$$

$$N_u = 2.1578E+006$$

$$A_g = 80000.00$$

$$\text{From (11.5.4.8), ACI 318-14: } V_s = V_{sjacket} + V_{s,core} = 471238.898$$

where:

$$V_{sjacket} = V_{sj1} + V_{sj2} = 471238.898$$

$V_{sj1} = 157079.633$ is calculated for section web jacket, with:

$$d = 160.00$$

$$A_v = 157079.633$$

$$f_y = 625.00$$

$$s = 100.00$$

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

$$s/d = 0.625$$

$V_{sj2} = 314159.265$ is calculated for section flange jacket, with:

$$d = 320.00$$

$$A_v = 157079.633$$

$$f_y = 625.00$$

$$s = 100.00$$

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

$$s/d = 0.3125$$

$$V_{s,core} = V_{s,c1} + V_{s,c2} = 0.00$$

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

$$d = 80.00$$

$$A_v = 100530.965$$

$$f_y = 500.00$$

$$s = 250.00$$

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

$$s/d = 3.125$$

$V_{s,c2} = 0.00$ is calculated for section flange core, with:

$$d = 240.00$$

$$A_v = 100530.965$$

$$f_y = 500.00$$

$$s = 250.00$$

$V_{s,c2}$ is multiplied by $Col,c2 = 0.00$

$$s/d = 1.04167$$

$$V_f \text{ ((11-3)-(11.4), ACI 440)} = 0.00$$

$$\text{From (11-11), ACI 440: } V_s + V_f \leq 212577.225$$

$$bw = 200.00$$

Calculation of Shear Strength at edge 2, $V_{r2} = 349917.706$

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

$$V_{Col0} = 349917.706$$

$$knl = 1 \text{ (zero step-static loading)}$$

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).

$$= 1 \text{ (normal-weight concrete)}$$

$$\text{Mean concrete strength: } f_c' = (f_c'_{jacket} \cdot Area_{jacket} + f_c'_{core} \cdot Area_{core}) / Area_{section} = 25.00, \text{ but } f_c'^{0.5} \leq 8.3$$

MPa (22.5.3.1, ACI 318-14)

$$M/Vd = 4.00$$

$$\mu_u = 6.4291E+007$$

$$V_u = 5.7948283E-005$$

$$d = 0.8 \cdot h = 320.00$$

$$N_u = 2.1578E+006$$

$$A_g = 80000.00$$

$$\text{From (11.5.4.8), ACI 318-14: } V_s = V_{sjacket} + V_{s,core} = 471238.898$$

where:

$$V_{sjacket} = V_{sj1} + V_{sj2} = 471238.898$$

$V_{sj1} = 157079.633$ is calculated for section web jacket, with:

$$d = 160.00$$

$$A_v = 157079.633$$

$f_y = 625.00$
 $s = 100.00$
 $V_{s,j1}$ is multiplied by $Col,j1 = 1.00$
 $s/d = 0.625$
 $V_{s,j2} = 314159.265$ is calculated for section flange jacket, with:
 $d = 320.00$
 $A_v = 157079.633$
 $f_y = 625.00$
 $s = 100.00$
 $V_{s,j2}$ is multiplied by $Col,j2 = 1.00$
 $s/d = 0.3125$
 $V_{s,core} = V_{s,c1} + V_{s,c2} = 0.00$
 $V_{s,c1} = 0.00$ is calculated for section web core, with:
 $d = 80.00$
 $A_v = 100530.965$
 $f_y = 500.00$
 $s = 250.00$
 $V_{s,c1}$ is multiplied by $Col,c1 = 0.00$
 $s/d = 3.125$
 $V_{s,c2} = 0.00$ is calculated for section flange core, with:
 $d = 240.00$
 $A_v = 100530.965$
 $f_y = 500.00$
 $s = 250.00$
 $V_{s,c2}$ is multiplied by $Col,c2 = 0.00$
 $s/d = 1.04167$
 $V_f ((11-3)-(11.4), ACI 440) = 0.00$
 From (11-11), ACI 440: $V_s + V_f \leq 212577.225$
 $bw = 200.00$

 End Of Calculation of Shear Capacity ratio for element: column JLC1 of floor 1
 At local axis: 2

Start Of Calculation of Chord Rotation Capacity for element: column JLC1 of floor 1
 At local axis: 2
 Integration Section: (a)
 Section Type: rcjlc

Constant Properties

 Knowledge Factor, $\phi = 0.90$
 Chord Rotation is generally considered as Deformation-Controlled Action according to Table C7-1, ASCE 41-17.
 Mean strengths are used for Deformation-Controlled Actions according to 7.5.1.3, ASCE 41-17
 Consequently:
 Jacket
 New material of Secondary Member: Concrete Strength, $f_c = f_{cm} = 30.00$
 New material of Secondary Member: Steel Strength, $f_s = f_{sm} = 625.00$
 Concrete Elasticity, $E_c = 25742.96$
 Steel Elasticity, $E_s = 200000.00$
 Existing Column
 Existing material of Secondary Member: Concrete Strength, $f_c = f_{cm} = 18.00$
 Existing material of Secondary Member: Steel Strength, $f_s = f_{sm} = 500.00$
 Concrete Elasticity, $E_c = 19940.411$
 Steel Elasticity, $E_s = 200000.00$
 Max Height, $H_{max} = 400.00$
 Min Height, $H_{min} = 200.00$
 Max Width, $W_{max} = 400.00$
 Min Width, $W_{min} = 200.00$
 Jacket Thickness, $t_j = 50.00$
 Cover Thickness, $c = 15.00$
 Element Length, $L = 3000.00$
 Secondary Member
 Smooth Bars

Ductile Steel
Without Detailing for Earthquake Resistance (including stirrups not 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, $M = -3.4089\text{E}+006$
Shear Force, $V2 = -0.60572401$
Shear Force, $V3 = -1.4589530\text{E}-005$
Axial Force, $F = -2.1578\text{E}+006$
Longitudinal Reinforcement Area Distribution (in 2 divisions)
-Tension: $As_t = 0.00$
-Compression: $As_c = 5353.274$
Longitudinal Reinforcement Area Distribution (in 3 divisions)
-Tension: $As_{t,ten} = 2208.54$
-Compression: $As_{c,com} = 1137.257$
-Middle: $As_{c,mid} = 2007.478$
Longitudinal External Reinforcement Area Distribution (in 3 divisions)
-Tension: $As_{t,ten,jacket} = 1746.726$
-Compression: $As_{c,com,jacket} = 829.3805$
-Middle: $As_{c,mid,jacket} = 1545.664$
Longitudinal Internal Reinforcement Area Distribution (in 3 divisions)
-Tension: $As_{t,ten,core} = 461.8141$
-Compression: $As_{c,com,core} = 307.8761$
-Middle: $As_{c,mid,core} = 461.8141$
Mean Diameter of Tension Reinforcement, $Db_L = 16.60$

Existing component: From table 7-7, ASCE 41_17: Final chord rotation Capacity $u_R = u = 0.03721297$
 $u = y + p = 0.04134775$

- Calculation of y -

$y = (M_y * L_s / 3) / E_{eff} = 0.04134775$ ((4.29), Biskinis Phd))
 $M_y = 5.1584\text{E}+008$
 $L_s = M/V$ (with $L_s > 0.1 * L$ and $L_s < 2 * L$) = 6000.00
From table 10.5, ASCE 41_17: $E_{eff} = \text{factor} * E_c * I_g = 2.4951\text{E}+013$
factor = 0.70
 $A_g = 120000.00$
Mean concrete strength: $f_c' = (f_c'_{jacket} * \text{Area}_{jacket} + f_c'_{core} * \text{Area}_{core}) / \text{Area}_{section} = 25.00$
 $N = 2.1578\text{E}+006$
 $E_c * I_g = E_{c,jacket} * I_{g,jacket} + E_{c,core} * I_{g,core} = 3.5645\text{E}+013$

Calculation of Yielding Moment M_y

Calculation of y 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, $b = 400.00$
web width, $b_w = 200.00$
flange thickness, $t = 200.00$

$y = \text{Min}(y_{ten}, y_{com})$
 $y_{ten} = 1.6571499\text{E}-005$
with $f_y = 596.2441$
 $d = 367.00$
 $y = 0.50980796$
 $A = 0.19238222$

B = 0.11480474
 with pt = 0.01504455
 pc = 0.00774698
 pv = 0.01367492
 N = 2.1578E+006
 b = 400.00
 " = 0.08991826
 y_comp = 9.6849368E-006
 with fc = 30.00
 Ec = 25742.96
 y = 0.5901636
 A = 0.0730043
 B = 0.06549972
 with Es = 200000.00
 CONFIRMATION: $y = 0.5901636 > t/d$

Calculation of ratio l_b/l_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 $V_y E / V_{col} E = 1.37616$

$d = d_{external} = 367.00$

$s = s_{external} = 100.00$

- $t = s_1 + s_2 + 2 * t_f / b_w * (f_{fe} / f_s) = 0.00809358$

jacket: $s_1 = A_{v1} * L_{stir1} / (s_1 * A_g) = 0.00680678$

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

$L_{stir1} = 1040.00$, is the total Length of all stirrups parallel to loading (shear) direction

$s_1 = 100.00$

core: $s_2 = A_{v2} * L_{stir2} / (s_2 * A_g) = 0.0012868$

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

$L_{stir2} = 768.00$, 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.

$NUD = 2.1578E+006$

$A_g = 120000.00$

$f_{cE} = (f_{c_jacket} * Area_jacket + f_{c_core} * Area_core) / section_area = 25.00$

$f_{yIE} = (f_{y_ext_Long_Reinf} * Area_ext_Long_Reinf + f_{y_int_Long_Reinf} * Area_int_Long_Reinf) / Area_Tot_Long_Rein = 596.2441$

$f_{yIE} = (f_{y_ext_Trans_Reinf} * s_1 + f_{y_int_Trans_Reinf} * s_2) / (s_1 + s_2) = 605.1263$

$p_l = Area_Tot_Long_Rein / (b * d) = 0.03646644$

$b = 400.00$

$d = 367.00$

$f_{cE} = 25.00$

End Of Calculation of Chord Rotation Capacity for element: column JLC1 of floor 1

At local axis: 2

Integration Section: (a)

Calculation No. 3

column C1, Floor 1

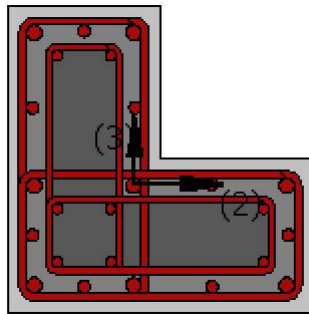
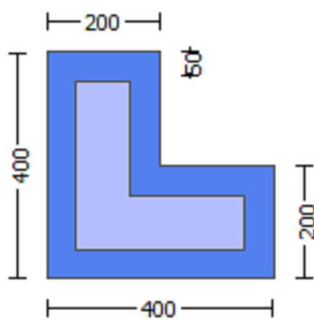
Limit State: Operational Level (data interpolation between analysis steps 1 and 2)

Analysis: Uniform +X

Check: Shear capacity VRd

Edge: Start

Local Axis: (3)



Start Of Calculation of Shear Capacity for element: column JLC1 of floor 1

At local axis: 3

Integration Section: (a)

Section Type: rcjlcs

Constant Properties

Knowledge Factor, $\gamma = 0.90$

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

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

Consequently:

Jacket

New material of Secondary Member: Concrete Strength, $f_c = f_{c_lower_bound} = 20.00$

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

Concrete Elasticity, $E_c = 25742.96$

Steel Elasticity, $E_s = 200000.00$

Existing Column

Existing material of Secondary Member: Concrete Strength, $f_c = f_{c_lower_bound} = 12.00$

Existing material of Secondary Member: Steel Strength, $f_s = f_{s_lower_bound} = 400.00$

Concrete Elasticity, $E_c = 19940.411$

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, $f_c = f_{cm} = 30.00$

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

Existing Column

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

Existing material: Steel Strength, $f_s = f_{sm} = 500.00$

#####

Max Height, Hmax = 400.00
 Min Height, Hmin = 200.00
 Max Width, Wmax = 400.00
 Min Width, Wmin = 200.00
 Jacket Thickness, tj = 50.00
 Cover Thickness, c = 15.00
 Element Length, L = 3000.00
 Secondary Member
 Smooth Bars
 Ductile Steel
 Without Detailing for Earthquake Resistance (including stirrups not 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 = -3.4089E+006
 Shear Force, Va = -1.4589530E-005
 EDGE -B-
 Bending Moment, Mb = 15336.81
 Shear Force, Vb = 1.4589530E-005
 BOTH EDGES
 Axial Force, F = -2.1578E+006
 Longitudinal Reinforcement Area Distribution (in 2 divisions)
 -Tension: Aslt = 0.00
 -Compression: Aslc = 5353.274
 Longitudinal Reinforcement Area Distribution (in 3 divisions)
 -Tension: Asl,ten = 2208.54
 -Compression: Asl,com = 1137.257
 -Middle: Asl,mid = 2007.478
 Mean Diameter of Tension Reinforcement, DbL,ten = 16.60

Existing component: From table 7-7, ASCE 41_17: Final Shear Capacity VR = $V_n = 267030.06$
 $V_n ((10.3), ASCE 41-17) = knl * V_{CoI} = 296700.066$
 $V_{CoI} = 296700.066$
 $knl = 1.00$
 $displacement_ductility_demand = 0.01268458$

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: $fc' = (fc'_{jacket} * Area_{jacket} + fc'_{core} * Area_{core}) / Area_{section} = 16.66667$, but $fc'^{0.5} \leq 8.3$ MPa (22.5.3.1, ACI 318-14)
 $M/Vd = 4.00$
 $M_u = 3.4089E+006$
 $V_u = 1.4589530E-005$
 $d = 0.8 * h = 320.00$
 $N_u = 2.1578E+006$
 $A_g = 80000.00$
 From (11.5.4.8), ACI 318-14: $V_s = V_{s,jacket} + V_{s,core} = 376991.118$
 where:
 $V_{s,jacket} = V_{s,j1} + V_{s,j2} = 376991.118$
 $V_{s,j1} = 251327.412$ is calculated for section web jacket, with:
 $d = 320.00$
 $A_v = 157079.633$
 $f_y = 500.00$
 $s = 100.00$
 $V_{s,j1}$ is multiplied by $Col,j1 = 1.00$
 $s/d = 0.3125$
 $V_{s,j2} = 125663.706$ is calculated for section flange jacket, with:

$d = 160.00$
 $A_v = 157079.633$
 $f_y = 500.00$
 $s = 100.00$
 $V_{s,j2}$ is multiplied by $Col,j2 = 1.00$
 $s/d = 0.625$
 $V_{s,core} = V_{s,c1} + V_{s,c2} = 0.00$
 $V_{s,c1} = 0.00$ is calculated for section web core, with:
 $d = 240.00$
 $A_v = 100530.965$
 $f_y = 400.00$
 $s = 250.00$
 $V_{s,c1}$ is multiplied by $Col,c1 = 0.00$
 $s/d = 1.04167$
 $V_{s,c2} = 0.00$ is calculated for section flange core, with:
 $d = 80.00$
 $A_v = 100530.965$
 $f_y = 400.00$
 $s = 250.00$
 $V_{s,c2}$ is multiplied by $Col,c2 = 0.00$
 $s/d = 3.125$
 $V_f ((11-3)-(11.4), ACI 440) = 0.00$
 From (11-11), ACI 440: $V_s + V_f \leq 173568.578$
 $bw = 200.00$

displacement ductility demand is calculated as δ_u / y

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

From analysis, chord rotation $\theta_r = 0.00052448$
 $y = (M_y * L_s / 3) / E_{eff} = 0.04134775 ((4.29), Biskinis Phd)$
 $M_y = 5.1584E+008$
 $L_s = M/V$ (with $L_s > 0.1 * L$ and $L_s < 2 * L$) = 6000.00
 From table 10.5, ASCE 41_17: $E_{eff} = factor * E_c * I_g = 2.4951E+013$
 $factor = 0.70$
 $A_g = 120000.00$
 Mean concrete strength: $f'_c = (f'_{c,jacket} * Area_{jacket} + f'_{c,core} * Area_{core}) / Area_{section} = 25.00$
 $N = 2.1578E+006$
 $E_c * I_g = E_{c,jacket} * I_{g,jacket} + E_{c,core} * I_{g,core} = 3.5645E+013$

Calculation of Yielding Moment M_y

Calculation of δ_u 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, $b = 400.00$
 web width, $bw = 200.00$
 flange thickness, $t = 200.00$

$y = \text{Min}(y_{ten}, y_{com})$
 $y_{ten} = 1.6571499E-005$
 with $f_y = 596.2441$
 $d = 367.00$
 $y = 0.50980796$
 $A = 0.19238222$
 $B = 0.11480474$
 with $pt = 0.01504455$
 $pc = 0.00774698$
 $pv = 0.01367492$
 $N = 2.1578E+006$
 $b = 400.00$

$\rho = 0.08991826$
 $y_{comp} = 9.6849368E-006$
 with $f_c = 30.00$
 $E_c = 25742.96$
 $y = 0.5901636$
 $A = 0.0730043$
 $B = 0.06549972$
 with $E_s = 200000.00$
 CONFIRMATION: $y = 0.5901636 > t/d$

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 JLC1 of floor 1

At local axis: 3

Integration Section: (a)

Calculation No. 4

column C1, Floor 1

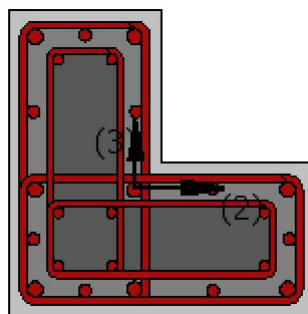
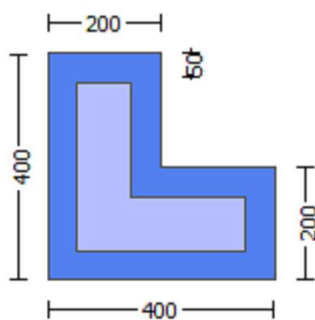
Limit State: Operational Level (data interpolation between analysis steps 1 and 2)

Analysis: Uniform +X

Check: Chord rotation capacity (ϕ)

Edge: Start

Local Axis: (3)



Start Of Calculation of Shear Capacity ratio for element: column JLC1 of floor 1

At Shear local axis: 3

(Bending local axis: 2)

Section Type: rcjlc

Constant Properties

Knowledge Factor, $\phi = 0.90$

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

Consequently:

Jacket

New material of Secondary Member: Concrete Strength, $f_c = f_{cm} = 30.00$

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

Concrete Elasticity, $E_c = 25742.96$

Steel Elasticity, $E_s = 200000.00$

Existing Column

Existing material of Secondary Member: Concrete Strength, $f_c = f_{cm} = 18.00$

Existing material of Secondary Member: Steel Strength, $f_s = f_{sm} = 500.00$

Concrete Elasticity, $E_c = 19940.411$

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, $f_s = 1.25 \cdot f_{sm} = 625.00$

#####

Max Height, $H_{max} = 400.00$

Min Height, $H_{min} = 200.00$

Max Width, $W_{max} = 400.00$

Min Width, $W_{min} = 200.00$

Jacket Thickness, $t_j = 50.00$

Cover Thickness, $c = 15.00$

Mean Confinement Factor overall section = 1.00

Element Length, $L = 3000.00$

Secondary Member

Smooth Bars

Ductile Steel

Without Detailing for Earthquake Resistance (including stirrups not 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, $V_a = -1.4594946E-005$

EDGE -B-

Shear Force, $V_b = 1.4594946E-005$

BOTH EDGES

Axial Force, $F = -2.1578E+006$

Longitudinal Reinforcement Area Distribution (in 2 divisions)

-Tension: $As_t = 0.00$

-Compression: $As_c = 5353.274$

Longitudinal Reinforcement Area Distribution (in 3 divisions)

-Tension: $As_{t,ten} = 2208.54$

-Compression: $As_{l,com} = 1137.257$

-Middle: $As_{l,mid} = 2007.478$

Calculation of Shear Capacity ratio, $V_e/V_r = 1.37616$

Member Controlled by Shear ($V_e/V_r > 1$)

Calculation of Shear Demand from fig. R18.6.5, ACI 318-14 $V_e = (M_{pr1} + M_{pr2})/l_n = 481543.08$

with

$M_{pr1} = \max(\mu_{u1+}, \mu_{u1-}) = 7.2231E+008$

$\mu_{u1+} = 7.2231E+008$, is the ultimate moment strength at the edge 1 of the member in the actual moment direction which is defined for the static loading combination

$\mu_{u1-} = 6.3496E+008$, 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

$M_{pr2} = \max(\mu_{u2+}, \mu_{u2-}) = 7.2231E+008$

$\mu_{u2+} = 7.2231E+008$, is the ultimate moment strength at the edge 2 of the member in the actual moment direction

which is defined for the the static loading combination

Mu2- = 6.3496E+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:

$$\phi_u = 3.9059244E-005$$

$$\mu_u = 7.2231E+008$$

with full section properties:

$$b = 400.00$$

$$d = 367.00$$

$$d' = 33.00$$

$$\nu = 0.48996389$$

$$N = 2.1578E+006$$

$$f_c = 30.00$$

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

$$\text{Final value of } \phi_{cu}: \phi_{cu}^* = \text{shear_factor} * \text{Max}(\phi_{cu}, \phi_{co}) = 0.00925707$$

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

$$\text{From (5.4b), TBDY: } \phi_{cu} = 0.00925707$$

$$\phi_{we} (5.4c) = 0.0207149$$

$$\phi_{ase} ((5.4d), TBDY) = (\phi_{ase1} * A_{ext} + \phi_{ase2} * A_{int}) / A_{sec} = 0.33836562$$

$$\phi_{ase1} = \text{Max}(((A_{conf,max1} - A_{noConf1}) / A_{conf,max1}) * (A_{conf,min1} / A_{conf,max1}), 0) = 0.33836562$$

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.

$A_{conf,max1} = 89600.00$ 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.

$A_{conf,min1} = 59225.00$ 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.

$A_{noConf1} = 43733.333$ is the unconfined external core area which is equal to $b_i^2/6$ as defined at (A.2).

$$\phi_{ase2} (>= \phi_{ase1}) = \text{Max}(((A_{conf,max2} - A_{noConf2}) / A_{conf,max2}) * (A_{conf,min2} / A_{conf,max2}), 0) = 0.33836562$$

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.

$A_{conf,max2} = 45264.00$ 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.

$A_{conf,min2} = 0.00$ 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.

$A_{noConf2} = 32309.333$ is the unconfined internal core area which is equal to $b_i^2/6$ as defined at (A.2).

$$\phi_{psh,min} * F_{ywe} = \text{Min}(\phi_{psh,x} * F_{ywe}, \phi_{psh,y} * F_{ywe}) = 6.12205$$

Expression (5.4d) for $\phi_{psh,min} * F_{ywe}$ has been multiplied by 0.3 according to 15.7.1.3 for members without earthquake detailing (90° closed stirrups)

$$\phi_{psh,x} * F_{ywe} = \phi_{psh1} * F_{ywe1} + \phi_{ps2} * F_{ywe2} = 6.12205$$

$$\phi_{psh1} ((5.4d), TBDY) = L_{stir1} * A_{stir1} / (A_{sec} * s_1) = 0.00680678$$

$$L_{stir1} (\text{Length of stirrups along Y}) = 1040.00$$

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

$$\phi_{ps2} (5.4d) = L_{stir2} * A_{stir2} / (A_{sec} * s_2) = 0.0012868$$

$$L_{stir2} (\text{Length of stirrups along Y}) = 768.00$$

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

$$\phi_{psh,y} * F_{ywe} = \phi_{psh1} * F_{ywe1} + \phi_{ps2} * F_{ywe2} = 6.12205$$

$$\phi_{psh1} ((5.4d), TBDY) = L_{stir1} * A_{stir1} / (A_{sec} * s_1) = 0.00680678$$

$$L_{stir1} (\text{Length of stirrups along X}) = 1040.00$$

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

$$\phi_{ps2} ((5.4d), TBDY) = L_{stir2} * A_{stir2} / (A_{sec} * s_2) = 0.0012868$$

Lstir2 (Length of stirrups along X) = 768.00
Astir2 (stirrups area) = 50.26548

Asec = 120000.00

s1 = 100.00

s2 = 250.00

fywe1 = 781.25

fywe2 = 625.00

fce = 30.00

From ((5.A5), TBDY), TBDY: cc = 0.002

c = confinement factor = 1.00

y1 = 0.0025

sh1 = 0.008

ft1 = 898.293

fy1 = 748.5775

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 = 748.5775

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

y2 = 0.0025

sh2 = 0.008

ft2 = 886.7403

fy2 = 738.9503

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 = 738.9503

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

yv = 0.0025

shv = 0.008

ftv = 894.3662

fyv = 745.3052

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 = 745.3052

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

1 = Asl,ten/(b*d)*(fs1/fc) = 0.37540035

2 = Asl,com/(b*d)*(fs2/fc) = 0.19082108

v = Asl,mid/(b*d)*(fsv/fc) = 0.33973286

and confined core properties:

b = 360.00

d = 347.00

d' = 13.00

```

fcc (5A.2, TBDY) = 30.00
cc (5A.5, TBDY) = 0.002
c = confinement factor = 1.00
1 = Asl,ten/(b*d)*(fs1/fc) = 0.44115251
2 = Asl,com/(b*d)*(fs2/fc) = 0.22424379
v = Asl,mid/(b*d)*(fsv/fc) = 0.39923778
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) = 0.64577857
MRc (4.17) = 7.2231E+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) = 0.67946111
MRo (4.18) = 4.2866E+008
MRo < 0.8*MRc
---->
u = cu (unconfined full section) = 3.9059244E-005
Mu = MRc
-----

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 = 4.2871818E-005
Mu = 6.3496E+008
-----

```

with full section properties:

$$b = 200.00$$

$$d = 367.00$$

$$d' = 33.00$$

$$v = 0.97992777$$

$$N = 2.1578E+006$$

$$f_c = 30.00$$

$$\phi (5A.5, \text{TB DY}) = 0.002$$

$$\text{Final value of } \phi u: \phi u^* = \text{shear_factor} * \text{Max}(\phi u, \phi c) = 0.00925707$$

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

$$\text{From (5.4b), TB DY: } \phi u = 0.00925707$$

$$\phi w (5.4c) = 0.0207149$$

$$\phi a_s ((5.4d), \text{TB DY}) = (\phi a_{s1} * A_{ext} + \phi a_{s2} * A_{int}) / A_{sec} = 0.33836562$$

$$\phi a_{s1} = \text{Max}(((A_{conf,max1} - A_{noConf1}) / A_{conf,max1}) * (A_{conf,min1} / A_{conf,max1}), 0) = 0.33836562$$

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.

$A_{conf,max1} = 89600.00$ 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.

$A_{conf,min1} = 59225.00$ 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.

$A_{noConf1} = 43733.333$ is the unconfined external core area which is equal to $b_i^2/6$ as defined at (A.2).

$$\phi a_{s2} (>= \phi a_{s1}) = \text{Max}(((A_{conf,max2} - A_{noConf2}) / A_{conf,max2}) * (A_{conf,min2} / A_{conf,max2}), 0) = 0.33836562$$

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.

$A_{conf,max2} = 45264.00$ 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.

$A_{conf,min2} = 0.00$ 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.

$A_{noConf2} = 32309.333$ is the unconfined internal core area which is equal to $b_i^2/6$ as defined at (A.2).

$$\phi p_{sh,min} * F_{ywe} = \text{Min}(\phi p_{sh,x} * F_{ywe}, \phi p_{sh,y} * F_{ywe}) = 6.12205$$

Expression (5.4d) for $\phi p_{sh,min} * F_{ywe}$ has been multiplied by 0.3 according to 15.7.1.3 for members without earthquake detailing (90° closed stirrups)

$$\phi p_{sh,x} * F_{ywe} = \phi p_{sh1} * F_{ywe1} + \phi p_{sh2} * F_{ywe2} = 6.12205$$

$$\phi p_{sh1} ((5.4d), \text{TB DY}) = L_{stir1} * A_{stir1} / (A_{sec} * s_1) = 0.00680678$$

$$L_{stir1} (\text{Length of stirrups along Y}) = 1040.00$$

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

$$\phi p_{sh2} (5.4d) = L_{stir2} * A_{stir2} / (A_{sec} * s_2) = 0.0012868$$

$$L_{stir2} (\text{Length of stirrups along Y}) = 768.00$$

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

$$\phi p_{sh,y} * F_{ywe} = \phi p_{sh1} * F_{ywe1} + \phi p_{sh2} * F_{ywe2} = 6.12205$$

$$\phi p_{sh1} ((5.4d), \text{TB DY}) = L_{stir1} * A_{stir1} / (A_{sec} * s_1) = 0.00680678$$

$$L_{stir1} (\text{Length of stirrups along X}) = 1040.00$$

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

$$\phi p_{sh2} ((5.4d), \text{TB DY}) = L_{stir2} * A_{stir2} / (A_{sec} * s_2) = 0.0012868$$

$$L_{stir2} (\text{Length of stirrups along X}) = 768.00$$

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

$$A_{sec} = 120000.00$$

$$s_1 = 100.00$$

$$s_2 = 250.00$$

$$f_{ywe1} = 781.25$$

$$f_{ywe2} = 625.00$$

$$f_{ce} = 30.00$$

$$\text{From ((5.A5), TB DY), TB DY: } \phi c = 0.002$$

$$c = \text{confinement factor} = 1.00$$

$$y_1 = 0.0025$$

$$sh_1 = 0.008$$

```

ft1 = 886.7403
fy1 = 738.9503
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 = 738.9503
    with Es1 = (Es,jacket*Asl,ten,jacket + Es,core*Asl,ten,core)/Asl,ten = 200000.00
y2 = 0.0025
sh2 = 0.008
ft2 = 898.293
fy2 = 748.5775
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 = 748.5775
    with Es2 = (Es,jacket*Asl,com,jacket + Es,core*Asl,com,core)/Asl,com = 200000.00
yv = 0.0025
shv = 0.008
ftv = 894.3662
fyv = 745.3052
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 = 745.3052
    with Esv = (Es,jacket*Asl,mid,jacket + Es,mid*Asl,mid,core)/Asl,mid = 200000.00
1 = Asl,ten/(b*d)*(fs1/fc) = 0.38164216
2 = Asl,com/(b*d)*(fs2/fc) = 0.7508007
v = Asl,mid/(b*d)*(fsv/fc) = 0.67946571
and confined core properties:
b = 160.00
d = 347.00
d' = 13.00
fcc (5A.2, TBDY) = 30.00
cc (5A.5, TBDY) = 0.002
    c = confinement factor = 1.00
    1 = Asl,ten/(b*d)*(fs1/fc) = 0.50454853
    2 = Asl,com/(b*d)*(fs2/fc) = 0.99259314
    v = Asl,mid/(b*d)*(fsv/fc) = 0.898285
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)

---->

$v < s, y_1$ - LHS eq.(4.7) is not satisfied

---->

$v < v_c, y_1$ - RHS eq.(4.6) is satisfied

---->

c_u (4.10) = 0.58834972

M_{Rc} (4.17) = 6.3496E+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: 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

---->

$v^* < v^* s, y_2$ - 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

---->

$v^* < v^* c, y_2$ - LHS eq.(4.6) is not satisfied

---->

$v^* < v^* c, y_1$ - RHS eq.(4.6) is not satisfied

---->

$*c_u$ (4.11) = 0.63603191

M_{Ro} (4.18) = 4.4768E+008

$M_{Ro} < 0.8 * M_{Rc}$

---->

$u = c_u$ (unconfined full section) = 4.2871818E-005

$M_u = M_{Rc}$

Calculation of ratio l_b/d

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

Calculation of M_{u2+}

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

$u = 3.9059244E-005$

$M_u = 7.2231E+008$

with full section properties:

$b = 400.00$

$d = 367.00$

$d' = 33.00$

$v = 0.48996389$

$N = 2.1578E+006$

$f_c = 30.00$

c_o (5A.5, TBDY) = 0.002

Final value of c_u : $c_u^* = \text{shear_factor} * \text{Max}(c_u, c_c) = 0.00925707$

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

From (5.4b), TBDY: $c_u = 0.00925707$

w_e (5.4c) = 0.0207149

a_{se} ((5.4d), TBDY) = $(a_{se1} * A_{ext} + a_{se2} * A_{int}) / A_{sec} = 0.33836562$

$$ase1 = \text{Max}(((A_{conf,max1} - A_{noConf1}) / A_{conf,max1}) * (A_{conf,min1} / A_{conf,max1}), 0) = 0.33836562$$

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.

$A_{conf,max1} = 89600.00$ 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.

$A_{conf,min1} = 59225.00$ 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.

$A_{noConf1} = 43733.333$ is the unconfined external core area which is equal to $b_i^2/6$ as defined at (A.2).

$$ase2 (>= ase1) = \text{Max}(((A_{conf,max2} - A_{noConf2}) / A_{conf,max2}) * (A_{conf,min2} / A_{conf,max2}), 0) = 0.33836562$$

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.

$A_{conf,max2} = 45264.00$ 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.

$A_{conf,min2} = 0.00$ 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.

$A_{noConf2} = 32309.333$ is the unconfined internal core area which is equal to $b_i^2/6$ as defined at (A.2).

$$psh_{min} * F_{ywe} = \text{Min}(psh_x * F_{ywe}, psh_y * F_{ywe}) = 6.12205$$

Expression (5.4d) for $psh_{min} * F_{ywe}$ has been multiplied by 0.3 according to 15.7.1.3 for members without earthquake detailing (90° closed stirrups)

$$psh_x * F_{ywe} = psh1 * F_{ywe1} + ps2 * F_{ywe2} = 6.12205$$

$$psh1 \text{ ((5.4d), TBDY)} = L_{stir1} * A_{stir1} / (A_{sec} * s1) = 0.00680678$$

$$L_{stir1} \text{ (Length of stirrups along Y)} = 1040.00$$

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

$$psh2 \text{ (5.4d)} = L_{stir2} * A_{stir2} / (A_{sec} * s2) = 0.0012868$$

$$L_{stir2} \text{ (Length of stirrups along Y)} = 768.00$$

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

$$psh_y * F_{ywe} = psh1 * F_{ywe1} + ps2 * F_{ywe2} = 6.12205$$

$$psh1 \text{ ((5.4d), TBDY)} = L_{stir1} * A_{stir1} / (A_{sec} * s1) = 0.00680678$$

$$L_{stir1} \text{ (Length of stirrups along X)} = 1040.00$$

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

$$psh2 \text{ ((5.4d), TBDY)} = L_{stir2} * A_{stir2} / (A_{sec} * s2) = 0.0012868$$

$$L_{stir2} \text{ (Length of stirrups along X)} = 768.00$$

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

$$A_{sec} = 120000.00$$

$$s1 = 100.00$$

$$s2 = 250.00$$

$$f_{ywe1} = 781.25$$

$$f_{ywe2} = 625.00$$

$$f_{ce} = 30.00$$

$$\text{From ((5.A5), TBDY), TBDY: } cc = 0.002$$

$$c = \text{confinement factor} = 1.00$$

$$y1 = 0.0025$$

$$sh1 = 0.008$$

$$ft1 = 898.293$$

$$fy1 = 748.5775$$

$$su1 = 0.032$$

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.

$$\text{with } fs1 = (fs_{jacket} * A_{sl,ten,jacket} + fs_{core} * A_{sl,ten,core}) / A_{sl,ten} = 748.5775$$

```

with Es1 = (Es,jacket*Asl,ten,jacket + Es,core*Asl,ten,core)/Asl,ten = 200000.00
y2 = 0.0025
sh2 = 0.008
ft2 = 886.7403
fy2 = 738.9503
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 = 738.9503
with Es2 = (Es,jacket*Asl,com,jacket + Es,core*Asl,com,core)/Asl,com = 200000.00
yv = 0.0025
shv = 0.008
ftv = 894.3662
fyv = 745.3052
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 = 745.3052
with Esv = (Es,jacket*Asl,mid,jacket + Es,mid*Asl,mid,core)/Asl,mid = 200000.00
1 = Asl,ten/(b*d)*(fs1/fc) = 0.37540035
2 = Asl,com/(b*d)*(fs2/fc) = 0.19082108
v = Asl,mid/(b*d)*(fsv/fc) = 0.33973286
and confined core properties:
b = 360.00
d = 347.00
d' = 13.00
fcc (5A.2, TBDY) = 30.00
cc (5A.5, TBDY) = 0.002
c = confinement factor = 1.00
1 = Asl,ten/(b*d)*(fs1/fc) = 0.44115251
2 = Asl,com/(b*d)*(fs2/fc) = 0.22424379
v = Asl,mid/(b*d)*(fsv/fc) = 0.39923778
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) = 0.64577857
MRc (4.17) = 7.2231E+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, ν_1 , ν_2 , ν normalised to $\nu_0 \cdot d_o$, instead of $b \cdot d$
- parameters of confined concrete, fcc, cc, used in lieu of fc, ecu

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

ν^*_{cu} (4.11) = 0.67946111

MRO (4.18) = 4.2866E+008

MRO < 0.8*MRC

$\nu_u = \nu_{cu}$ (unconfined full section) = 3.9059244E-005

$\mu_u = MRC$

Calculation of ratio I_b/I_d

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

Calculation of μ_{u2}

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

$\nu_u = 4.2871818E-005$

$\mu_u = 6.3496E+008$

with full section properties:

b = 200.00

d = 367.00

d' = 33.00

$\nu = 0.97992777$

N = 2.1578E+006

fc = 30.00

co (5A.5, TBDY) = 0.002

Final value of ν_{cu} : $\nu_{cu}^* = \text{shear_factor} \cdot \text{Max}(\nu_{cu}, \nu_{cc}) = 0.00925707$

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

From (5.4b), TBDY: $\nu_{cu} = 0.00925707$

we (5.4c) = 0.0207149

ase ((5.4d), TBDY) = $(\text{ase1} \cdot A_{ext} + \text{ase2} \cdot A_{int}) / A_{sec} = 0.33836562$

ase1 = $\text{Max}(((A_{conf,max1} - A_{noConf1}) / A_{conf,max1}) \cdot (A_{conf,min1} / A_{conf,max1}), 0) = 0.33836562$

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.

$A_{conf,max1} = 89600.00$ 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.

$A_{conf,min1} = 59225.00$ 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.

$A_{noConf1} = 43733.333$ is the unconfined external core area which is equal to $b^2/6$ as defined at (A.2).

ase2 ($\geq \text{ase1}$) = $\text{Max}(((A_{conf,max2} - A_{noConf2}) / A_{conf,max2}) \cdot (A_{conf,min2} / A_{conf,max2}), 0) = 0.33836562$

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 = 45264.00 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 = 0.00 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 = 32309.333 is the unconfined internal core area which is equal to $b_i^2/6$ as defined at (A.2).

$psh_{min} \cdot F_{ywe} = \text{Min}(psh_x \cdot F_{ywe}, psh_y \cdot F_{ywe}) = 6.12205$

Expression (5.4d) for $psh_{min} \cdot F_{ywe}$ has been multiplied by 0.3 according to 15.7.1.3 for members without earthquake detailing (90° closed stirrups)

$psh_x \cdot F_{ywe} = psh1 \cdot F_{ywe1} + ps2 \cdot F_{ywe2} = 6.12205$

$psh1$ ((5.4d), TBDY) = $L_{stir1} \cdot A_{stir1} / (A_{sec} \cdot s1) = 0.00680678$

L_{stir1} (Length of stirrups along Y) = 1040.00

A_{stir1} (stirrups area) = 78.53982

$psh2$ (5.4d) = $L_{stir2} \cdot A_{stir2} / (A_{sec} \cdot s2) = 0.0012868$

L_{stir2} (Length of stirrups along Y) = 768.00

A_{stir2} (stirrups area) = 50.26548

$psh_y \cdot F_{ywe} = psh1 \cdot F_{ywe1} + ps2 \cdot F_{ywe2} = 6.12205$

$psh1$ ((5.4d), TBDY) = $L_{stir1} \cdot A_{stir1} / (A_{sec} \cdot s1) = 0.00680678$

L_{stir1} (Length of stirrups along X) = 1040.00

A_{stir1} (stirrups area) = 78.53982

$psh2$ ((5.4d), TBDY) = $L_{stir2} \cdot A_{stir2} / (A_{sec} \cdot s2) = 0.0012868$

L_{stir2} (Length of stirrups along X) = 768.00

A_{stir2} (stirrups area) = 50.26548

$A_{sec} = 120000.00$

$s1 = 100.00$

$s2 = 250.00$

$f_{ywe1} = 781.25$

$f_{ywe2} = 625.00$

$f_{ce} = 30.00$

From ((5.A5), TBDY), TBDY: $cc = 0.002$

c = confinement factor = 1.00

$y1 = 0.0025$

$sh1 = 0.008$

$ft1 = 886.7403$

$fy1 = 738.9503$

$su1 = 0.032$

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

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

$su1 = 0.4 \cdot esu1_{nominal}$ ((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 $\text{Min}(1, 1.25 \cdot (l_b/l_d)^{2/3})$, from 10.3.5, ASCE 41-17.

with $fs1 = (f_{s,jacket} \cdot A_{s,ten,jacket} + f_{s,core} \cdot A_{s,ten,core}) / A_{s,ten} = 738.9503$

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

$y2 = 0.0025$

$sh2 = 0.008$

$ft2 = 898.293$

$fy2 = 748.5775$

$su2 = 0.032$

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

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

$su2 = 0.4 \cdot esu2_{nominal}$ ((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 $fs2 = (fs_{jacket} \cdot Asl_{com,jacket} + fs_{core} \cdot Asl_{com,core}) / Asl_{com} = 748.5775$
with $Es2 = (Es_{jacket} \cdot Asl_{com,jacket} + Es_{core} \cdot Asl_{com,core}) / Asl_{com} = 200000.00$
 $yv = 0.0025$
 $shv = 0.008$
 $ftv = 894.3662$
 $fyv = 745.3052$
 $suv = 0.032$
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 $fsv = (fs_{jacket} \cdot Asl_{mid,jacket} + fs_{mid} \cdot Asl_{mid,core}) / Asl_{mid} = 745.3052$
with $Esv = (Es_{jacket} \cdot Asl_{mid,jacket} + Es_{mid} \cdot Asl_{mid,core}) / Asl_{mid} = 200000.00$
 $1 = Asl_{ten} / (b \cdot d) \cdot (fs1 / fc) = 0.38164216$
 $2 = Asl_{com} / (b \cdot d) \cdot (fs2 / fc) = 0.7508007$
 $v = Asl_{mid} / (b \cdot d) \cdot (fsv / fc) = 0.67946571$
and confined core properties:
 $b = 160.00$
 $d = 347.00$
 $d' = 13.00$
 fcc (5A.2, TBDY) = 30.00
 cc (5A.5, TBDY) = 0.002
 c = confinement factor = 1.00
 $1 = Asl_{ten} / (b \cdot d) \cdot (fs1 / fc) = 0.50454853$
 $2 = Asl_{com} / (b \cdot d) \cdot (fs2 / fc) = 0.99259314$
 $v = Asl_{mid} / (b \cdot d) \cdot (fsv / fc) = 0.898285$
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) = 0.58834972
 MRC (4.17) = 6.3496E+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
--->
 $v^* < v^*,y2$ - 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

--->

$*c_u (4.11) = 0.63603191$

$MR_o (4.18) = 4.4768E+008$

$MR_o < 0.8*MR_c$

--->

$u = c_u$ (unconfined full section) = $4.2871818E-005$

$Mu = MR_c$

Calculation of ratio l_b/d

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

Calculation of Shear Strength $V_r = \min(V_{r1}, V_{r2}) = 349917.706$

Calculation of Shear Strength at edge 1, $V_{r1} = 349917.706$

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

$V_{Col0} = 349917.706$

$knl = 1$ (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: $f'_c = (f'_c_{jacket} * Area_{jacket} + f'_c_{core} * Area_{core}) / Area_{section} = 25.00$, but $f'_c^{0.5} \leq 8.3$ MPa (22.5.3.1, ACI 318-14)

$M/Vd = 4.00$

$Mu = 3.4090E+006$

$Vu = 1.4594946E-005$

$d = 0.8 * h = 320.00$

$Nu = 2.1578E+006$

$Ag = 80000.00$

From (11.5.4.8), ACI 318-14: $V_s = V_{sjacket} + V_{s,core} = 471238.898$

where:

$V_{sjacket} = V_{sj1} + V_{sj2} = 471238.898$

$V_{sj1} = 314159.265$ is calculated for section web jacket, with:

$d = 320.00$

$A_v = 157079.633$

$f_y = 625.00$

$s = 100.00$

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

$s/d = 0.3125$

$V_{sj2} = 157079.633$ is calculated for section flange jacket, with:

$d = 160.00$

$A_v = 157079.633$

$f_y = 625.00$

$s = 100.00$

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

$s/d = 0.625$

$V_{s,core} = V_{s,c1} + V_{s,c2} = 0.00$

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

$d = 240.00$

$A_v = 100530.965$

$f_y = 500.00$

$s = 250.00$

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

$s/d = 1.04167$

$V_{s,c2} = 0.00$ is calculated for section flange core, with:

$d = 80.00$

$$A_v = 100530.965$$

$$f_y = 500.00$$

$$s = 250.00$$

$V_{s,c2}$ is multiplied by $Col,c2 = 0.00$

$$s/d = 3.125$$

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

$$\text{From (11-11), ACI 440: } V_s + V_f \leq 212577.225$$

$$bw = 200.00$$

Calculation of Shear Strength at edge 2, $V_{r2} = 349917.706$

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

$$V_{Col0} = 349917.706$$

$$knl = 1 \text{ (zero step-static loading)}$$

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).

$$= 1 \text{ (normal-weight concrete)}$$

$$\text{Mean concrete strength: } f'_c = (f'_c_{\text{jacket}} * \text{Area}_{\text{jacket}} + f'_c_{\text{core}} * \text{Area}_{\text{core}}) / \text{Area}_{\text{section}} = 25.00, \text{ but } f'_c^{0.5} \leq 8.3 \text{ MPa (22.5.3.1, ACI 318-14)}$$

$$M/Vd = 4.00$$

$$\mu_u = 15336.932$$

$$V_u = 1.4594946E-005$$

$$d = 0.8 * h = 320.00$$

$$N_u = 2.1578E+006$$

$$A_g = 80000.00$$

$$\text{From (11.5.4.8), ACI 318-14: } V_s = V_{s,jacket} + V_{s,core} = 471238.898$$

where:

$$V_{s,jacket} = V_{sj1} + V_{sj2} = 471238.898$$

$V_{sj1} = 314159.265$ is calculated for section web jacket, with:

$$d = 320.00$$

$$A_v = 157079.633$$

$$f_y = 625.00$$

$$s = 100.00$$

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

$$s/d = 0.3125$$

$V_{sj2} = 157079.633$ is calculated for section flange jacket, with:

$$d = 160.00$$

$$A_v = 157079.633$$

$$f_y = 625.00$$

$$s = 100.00$$

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

$$s/d = 0.625$$

$$V_{s,core} = V_{s,c1} + V_{s,c2} = 0.00$$

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

$$d = 240.00$$

$$A_v = 100530.965$$

$$f_y = 500.00$$

$$s = 250.00$$

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

$$s/d = 1.04167$$

$V_{s,c2} = 0.00$ is calculated for section flange core, with:

$$d = 80.00$$

$$A_v = 100530.965$$

$$f_y = 500.00$$

$$s = 250.00$$

$V_{s,c2}$ is multiplied by $Col,c2 = 0.00$

$$s/d = 3.125$$

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

$$\text{From (11-11), ACI 440: } V_s + V_f \leq 212577.225$$

$$bw = 200.00$$

End Of Calculation of Shear Capacity ratio for element: column JLC1 of floor 1

At local axis: 3

Start Of Calculation of Shear Capacity ratio for element: column JLC1 of floor 1
 At Shear local axis: 2
 (Bending local axis: 3)
 Section Type: rcjlc
 Constant Properties
 Knowledge Factor, $\gamma = 0.90$
 Mean strength values are used for both shear and moment calculations.
 Consequently:
 Jacket
 New material of Secondary Member: Concrete Strength, $f_c = f_{cm} = 30.00$
 New material of Secondary Member: Steel Strength, $f_s = f_{sm} = 625.00$
 Concrete Elasticity, $E_c = 25742.96$
 Steel Elasticity, $E_s = 200000.00$
 Existing Column
 Existing material of Secondary Member: Concrete Strength, $f_c = f_{cm} = 18.00$
 Existing material of Secondary Member: Steel Strength, $f_s = f_{sm} = 500.00$
 Concrete Elasticity, $E_c = 19940.411$
 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, $f_s = 1.25 \cdot f_{sm} = 625.00$
 #####
 Max Height, $H_{max} = 400.00$
 Min Height, $H_{min} = 200.00$
 Max Width, $W_{max} = 400.00$
 Min Width, $W_{min} = 200.00$
 Jacket Thickness, $t_j = 50.00$
 Cover Thickness, $c = 15.00$
 Mean Confinement Factor overall section = 1.00
 Element Length, $L = 3000.00$
 Secondary Member
 Smooth Bars
 Ductile Steel
 Without Detailing for Earthquake Resistance (including stirrups not 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: 2
 EDGE -A-
 Shear Force, $V_a = -5.7948283E-005$
 EDGE -B-
 Shear Force, $V_b = 5.7948283E-005$
 BOTH EDGES
 Axial Force, $F = -2.1578E+006$
 Longitudinal Reinforcement Area Distribution (in 2 divisions)
 -Tension: $As_t = 0.00$
 -Compression: $As_c = 5353.274$
 Longitudinal Reinforcement Area Distribution (in 3 divisions)
 -Tension: $As_{t,ten} = 1137.257$
 -Compression: $As_{l,com} = 2208.54$
 -Middle: $As_{l,mid} = 2007.478$

Calculation of Shear Capacity ratio , $V_e/V_r = 1.10794$

Member Controlled by Shear ($V_e/V_r > 1$)

Calculation of Shear Demand from fig. R18.6.5, ACI 318-14 $V_e = (M_{pr1} + M_{pr2})/l_n = 387689.388$
with

$M_{pr1} = \text{Max}(M_{u1+}, M_{u1-}) = 5.8153\text{E}+008$

$M_{u1+} = 5.8153\text{E}+008$, is the ultimate moment strength at the edge 1 of the member in the actual moment direction which is defined for the static loading combination

$M_{u1-} = 4.9554\text{E}+008$, 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

$M_{pr2} = \text{Max}(M_{u2+}, M_{u2-}) = 5.8153\text{E}+008$

$M_{u2+} = 5.8153\text{E}+008$, 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

$M_{u2-} = 4.9554\text{E}+008$, 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 M_{u1+}

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

$\phi_u = 1.1511372\text{E}-005$

$M_u = 5.8153\text{E}+008$

with full section properties:

$b = 400.00$

$d = 367.00$

$d' = 33.00$

$v = 0.48996389$

$N = 2.1578\text{E}+006$

$f_c = 30.00$

ϕ_c (5A.5, TBDY) = 0.002

Final value of ϕ_u : $\phi_u^* = \text{shear_factor} * \text{Max}(\phi_u, \phi_c) = 0.00925707$

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

From (5.4b), TBDY: $\phi_u = 0.00925707$

ϕ_{ue} (5.4c) = 0.0207149

ϕ_{ase} ((5.4d), TBDY) = $(\phi_{ase1} * A_{ext} + \phi_{ase2} * A_{int}) / A_{sec} = 0.33836562$

$\phi_{ase1} = \text{Max}(((A_{conf,max1} - A_{noConf1}) / A_{conf,max1}) * (A_{conf,min1} / A_{conf,max1}), 0) = 0.33836562$

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.

$A_{conf,max1} = 89600.00$ 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.

$A_{conf,min1} = 59225.00$ 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.

$A_{noConf1} = 43733.333$ is the unconfined external core area which is equal to $b_i^2/6$ as defined at (A.2).

$\phi_{ase2} (\geq \phi_{ase1}) = \text{Max}(((A_{conf,max2} - A_{noConf2}) / A_{conf,max2}) * (A_{conf,min2} / A_{conf,max2}), 0) = 0.33836562$

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.

$A_{conf,max2} = 45264.00$ 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.

$A_{conf,min2} = 0.00$ 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.

$A_{noConf2} = 32309.333$ is the unconfined internal core area which is equal to $b_i^2/6$ as defined at (A.2).

$\phi_{psh,min} * F_{ywe} = \text{Min}(\phi_{psh,x} * F_{ywe}, \phi_{psh,y} * F_{ywe}) = 6.12205$

Expression (5.4d) for $\phi_{psh,min} * F_{ywe}$ has been multiplied by 0.3 according to 15.7.1.3 for members without earthquake detailing (90° closed stirrups)

 $\phi_{psh,x} * F_{ywe} = \phi_{psh1} * F_{ywe1} + \phi_{psh2} * F_{ywe2} = 6.12205$

$psh1 ((5.4d), TBDY) = Lstir1 * Astir1 / (Asec * s1) = 0.00680678$
 $Lstir1$ (Length of stirrups along Y) = 1040.00
 $Astir1$ (stirrups area) = 78.53982
 $psh2 (5.4d) = Lstir2 * Astir2 / (Asec * s2) = 0.0012868$
 $Lstir2$ (Length of stirrups along Y) = 768.00
 $Astir2$ (stirrups area) = 50.26548

$psh_y * Fywe = psh1 * Fywe1 + ps2 * Fywe2 = 6.12205$
 $psh1 ((5.4d), TBDY) = Lstir1 * Astir1 / (Asec * s1) = 0.00680678$
 $Lstir1$ (Length of stirrups along X) = 1040.00
 $Astir1$ (stirrups area) = 78.53982
 $psh2 ((5.4d), TBDY) = Lstir2 * Astir2 / (Asec * s2) = 0.0012868$
 $Lstir2$ (Length of stirrups along X) = 768.00
 $Astir2$ (stirrups area) = 50.26548

Asec = 120000.00

s1 = 100.00

s2 = 250.00

fywe1 = 781.25

fywe2 = 625.00

fce = 30.00

From ((5.A5), TBDY), TBDY: cc = 0.002

c = confinement factor = 1.00

y1 = 0.0025

sh1 = 0.008

ft1 = 886.7403

fy1 = 738.9503

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 = 738.9503

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

y2 = 0.0025

sh2 = 0.008

ft2 = 898.293

fy2 = 748.5775

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 = 748.5775

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

yv = 0.0025

shv = 0.008

ftv = 894.3662

fyv = 745.3052

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 = 745.3052
with Esv = (Es,jacket*Asl,mid,jacket + Es,mid*Asl,mid,core)/Asl,mid = 200000.00
1 = Asl,ten/(b*d)*(fs1/fc) = 0.19082108
2 = Asl,com/(b*d)*(fs2/fc) = 0.37540035
v = Asl,mid/(b*d)*(fsv/fc) = 0.33973286
and confined core properties:
b = 360.00
d = 347.00
d' = 13.00
fcc (5A.2, TBDY) = 30.00
cc (5A.5, TBDY) = 0.002
c = confinement factor = 1.00
1 = Asl,ten/(b*d)*(fs1/fc) = 0.22424379
2 = Asl,com/(b*d)*(fs2/fc) = 0.44115251
v = Asl,mid/(b*d)*(fsv/fc) = 0.39923778
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) = 0.42533033
MRc (4.17) = 7.2277E+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) = 0.50069522
MRo (4.18) = 5.8153E+008
---->
u = cu (4.2) = 1.1511372E-005
Mu = MRo

```

Calculation of ratio lb/ld

Adequate Lap Length: lb/ld >= 1

Calculation of Mu1-

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

$$\mu = 3.0526570E-005$$

$$\mu_u = 4.9554E+008$$

with full section properties:

$$b = 200.00$$

$$d = 367.00$$

$$d' = 33.00$$

$$\nu = 0.97992777$$

$$N = 2.1578E+006$$

$$f_c = 30.00$$

$$\alpha (5A.5, \text{TB DY}) = 0.002$$

$$\text{Final value of } \mu_u: \mu_u^* = \text{shear_factor} * \text{Max}(\mu_u, \alpha) = 0.00925707$$

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

$$\text{From (5.4b), TB DY: } \mu_u = 0.00925707$$

$$\mu_{ue} (5.4c) = 0.0207149$$

$$\alpha_{se} ((5.4d), \text{TB DY}) = (\alpha_{se1} * A_{ext} + \alpha_{se2} * A_{int}) / A_{sec} = 0.33836562$$

$$\alpha_{se1} = \text{Max}(((A_{conf,max1} - A_{noConf1}) / A_{conf,max1}) * (A_{conf,min1} / A_{conf,max1}), 0) = 0.33836562$$

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.

$A_{conf,max1} = 89600.00$ 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.

$A_{conf,min1} = 59225.00$ 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.

$A_{noConf1} = 43733.333$ is the unconfined external core area which is equal to $b_i^2/6$ as defined at (A.2).

$$\alpha_{se2} (> \alpha_{se1}) = \text{Max}(((A_{conf,max2} - A_{noConf2}) / A_{conf,max2}) * (A_{conf,min2} / A_{conf,max2}), 0) = 0.33836562$$

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.

$A_{conf,max2} = 45264.00$ 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.

$A_{conf,min2} = 0.00$ 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.

$A_{noConf2} = 32309.333$ is the unconfined internal core area which is equal to $b_i^2/6$ as defined at (A.2).

$$p_{sh,min} * F_{ywe} = \text{Min}(p_{sh,x} * F_{ywe}, p_{sh,y} * F_{ywe}) = 6.12205$$

Expression (5.4d) for $p_{sh,min} * F_{ywe}$ has been multiplied by 0.3 according to 15.7.1.3 for members without earthquake detailing (90° closed stirrups)

$$p_{sh,x} * F_{ywe} = p_{sh1} * F_{ywe1} + p_{sh2} * F_{ywe2} = 6.12205$$

$$p_{sh1} ((5.4d), \text{TB DY}) = L_{stir1} * A_{stir1} / (A_{sec} * s_1) = 0.00680678$$

$$L_{stir1} (\text{Length of stirrups along Y}) = 1040.00$$

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

$$p_{sh2} (5.4d) = L_{stir2} * A_{stir2} / (A_{sec} * s_2) = 0.0012868$$

$$L_{stir2} (\text{Length of stirrups along Y}) = 768.00$$

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

$$p_{sh,y} * F_{ywe} = p_{sh1} * F_{ywe1} + p_{sh2} * F_{ywe2} = 6.12205$$

$$p_{sh1} ((5.4d), \text{TB DY}) = L_{stir1} * A_{stir1} / (A_{sec} * s_1) = 0.00680678$$

$$L_{stir1} (\text{Length of stirrups along X}) = 1040.00$$

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

$$p_{sh2} ((5.4d), \text{TB DY}) = L_{stir2} * A_{stir2} / (A_{sec} * s_2) = 0.0012868$$

$$L_{stir2} (\text{Length of stirrups along X}) = 768.00$$

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

```

Asec = 120000.00
s1 = 100.00
s2 = 250.00
fywe1 = 781.25
fywe2 = 625.00
fce = 30.00
From ((5.A.5), TBDY), TBDY: cc = 0.002
c = confinement factor = 1.00
y1 = 0.0025
sh1 = 0.008
ft1 = 898.293
fy1 = 748.5775
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 Min(1,1.25*(lb/lb,min)^ 2/3), from 10.3.5, ASCE 41-17.
with fs1 = (fs,jacket*Asl,ten,jacket + fs,core*Asl,ten,core)/Asl,ten = 748.5775
with Es1 = (Es,jacket*Asl,ten,jacket + Es,core*Asl,ten,core)/Asl,ten = 200000.00
y2 = 0.0025
sh2 = 0.008
ft2 = 886.7403
fy2 = 738.9503
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,min)^ 2/3), from 10.3.5, ASCE 41-17.
with fs2 = (fs,jacket*Asl,com,jacket + fs,core*Asl,com,core)/Asl,com = 738.9503
with Es2 = (Es,jacket*Asl,com,jacket + Es,core*Asl,com,core)/Asl,com = 200000.00
yv = 0.0025
shv = 0.008
ftv = 894.3662
fyv = 745.3052
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 Min(1,1.25*(lb/lb,min)^ 2/3), from 10.3.5, ASCE 41-17.
with fsv = (fs,jacket*Asl,mid,jacket + fs,mid*Asl,mid,core)/Asl,mid = 745.3052
with Esv = (Es,jacket*Asl,mid,jacket + Es,mid*Asl,mid,core)/Asl,mid = 200000.00
1 = Asl,ten/(b*d)*(fs1/fc) = 0.7508007
2 = Asl,com/(b*d)*(fs2/fc) = 0.38164216
v = Asl,mid/(b*d)*(fsv/fc) = 0.67946571
and confined core properties:
b = 160.00
d = 347.00
d' = 13.00
fcc (5A.2, TBDY) = 30.00
cc (5A.5, TBDY) = 0.002

```

```

c = confinement factor = 1.00
1 = Asl,ten/(b*d)*(fs1/fc) = 0.99259314
2 = Asl,com/(b*d)*(fs2/fc) = 0.50454853
v = Asl,mid/(b*d)*(fsv/fc) = 0.898285
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) = 0.8262842
MRc (4.18) = 4.9554E+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) = 0.90160009
MRo (4.18) = 2.4685E+008
MRo < 0.8*MRc
---->
u = cu (unconfined full section) = 3.0526570E-005
Mu = MRc
-----

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 = 1.1511372E-005
Mu = 5.8153E+008
-----

with full section properties:
b = 400.00

```

$d = 367.00$
 $d' = 33.00$
 $v = 0.48996389$
 $N = 2.1578E+006$
 $f_c = 30.00$
 $co (5A.5, TBDY) = 0.002$
 Final value of cu : $cu^* = shear_factor * Max(cu, cc) = 0.00925707$
 The Shear_factor is considered equal to 1 (pure moment strength)
 From (5.4b), TBDY: $cu = 0.00925707$
 $we (5.4c) = 0.0207149$
 $ase ((5.4d), TBDY) = (ase1 * A_{ext} + ase2 * A_{int}) / A_{sec} = 0.33836562$
 $ase1 = Max(((A_{conf,max1} - A_{noConf1}) / A_{conf,max1}) * (A_{conf,min1} / A_{conf,max1}), 0) = 0.33836562$
 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.
 $A_{conf,max1} = 89600.00$ 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.
 $A_{conf,min1} = 59225.00$ 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.
 $A_{noConf1} = 43733.333$ is the unconfined external core area which is equal to $b_i^2/6$ as defined at (A.2).
 $ase2 (>= ase1) = Max(((A_{conf,max2} - A_{noConf2}) / A_{conf,max2}) * (A_{conf,min2} / A_{conf,max2}), 0) = 0.33836562$
 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.
 $A_{conf,max2} = 45264.00$ 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.
 $A_{conf,min2} = 0.00$ 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.
 $A_{noConf2} = 32309.333$ is the unconfined internal core area which is equal to $b_i^2/6$ as defined at (A.2).
 $psh_{min} * F_{ywe} = Min(psh_x * F_{ywe}, psh_y * F_{ywe}) = 6.12205$
 Expression (5.4d) for $psh_{min} * F_{ywe}$ has been multiplied by 0.3 according to 15.7.1.3 for members without earthquake detailing (90° closed stirrups)

$psh_x * F_{ywe} = psh1 * F_{ywe1} + ps2 * F_{ywe2} = 6.12205$
 $psh1 ((5.4d), TBDY) = L_{stir1} * A_{stir1} / (A_{sec} * s1) = 0.00680678$
 L_{stir1} (Length of stirrups along Y) = 1040.00
 A_{stir1} (stirrups area) = 78.53982
 $psh2 ((5.4d)) = L_{stir2} * A_{stir2} / (A_{sec} * s2) = 0.0012868$
 L_{stir2} (Length of stirrups along Y) = 768.00
 A_{stir2} (stirrups area) = 50.26548

$psh_y * F_{ywe} = psh1 * F_{ywe1} + ps2 * F_{ywe2} = 6.12205$
 $psh1 ((5.4d), TBDY) = L_{stir1} * A_{stir1} / (A_{sec} * s1) = 0.00680678$
 L_{stir1} (Length of stirrups along X) = 1040.00
 A_{stir1} (stirrups area) = 78.53982
 $psh2 ((5.4d), TBDY) = L_{stir2} * A_{stir2} / (A_{sec} * s2) = 0.0012868$
 L_{stir2} (Length of stirrups along X) = 768.00
 A_{stir2} (stirrups area) = 50.26548

$A_{sec} = 120000.00$
 $s1 = 100.00$
 $s2 = 250.00$
 $f_{ywe1} = 781.25$
 $f_{ywe2} = 625.00$
 $f_{ce} = 30.00$
 From ((5.A5), TBDY), TBDY: $cc = 0.002$
 $c =$ confinement factor = 1.00
 $y1 = 0.0025$
 $sh1 = 0.008$
 $ft1 = 886.7403$
 $fy1 = 738.9503$

```

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 = 738.9503
with Es1 = (Es,jacket*Asl,ten,jacket + Es,core*Asl,ten,core)/Asl,ten = 200000.00
y2 = 0.0025
sh2 = 0.008
ft2 = 898.293
fy2 = 748.5775
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 = 748.5775
with Es2 = (Es,jacket*Asl,com,jacket + Es,core*Asl,com,core)/Asl,com = 200000.00
yv = 0.0025
shv = 0.008
ftv = 894.3662
fyv = 745.3052
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 = 745.3052
with Esv = (Es,jacket*Asl,mid,jacket + Es,mid*Asl,mid,core)/Asl,mid = 200000.00
1 = Asl,ten/(b*d)*(fs1/fc) = 0.19082108
2 = Asl,com/(b*d)*(fs2/fc) = 0.37540035
v = Asl,mid/(b*d)*(fsv/fc) = 0.33973286
and confined core properties:
b = 360.00
d = 347.00
d' = 13.00
fcc (5A.2, TBDY) = 30.00
cc (5A.5, TBDY) = 0.002
c = confinement factor = 1.00
1 = Asl,ten/(b*d)*(fs1/fc) = 0.22424379
2 = Asl,com/(b*d)*(fs2/fc) = 0.44115251
v = Asl,mid/(b*d)*(fsv/fc) = 0.39923778
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) = 0.42533033
 M_{Rc} (4.17) = 7.2277E+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: 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, e_c

--->
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) = 0.50069522
 M_{Ro} (4.18) = 5.8153E+008
--->

$u = c_u$ (4.2) = 1.1511372E-005
 $\mu = M_{Ro}$

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:

$u = 3.0526570E-005$
 $\mu = 4.9554E+008$

with full section properties:

$b = 200.00$
 $d = 367.00$
 $d' = 33.00$
 $v = 0.97992777$
 $N = 2.1578E+006$
 $f_c = 30.00$
 c_o (5A.5, TBDY) = 0.002
Final value of c_u : $c_u^* = \text{shear_factor} * \text{Max}(c_u, c_c) = 0.00925707$
The Shear_factor is considered equal to 1 (pure moment strength)
From (5.4b), TBDY: $c_u = 0.00925707$
 w_e (5.4c) = 0.0207149
 a_{se} ((5.4d), TBDY) = $(a_{se1} * A_{ext} + a_{se2} * A_{int}) / A_{sec} = 0.33836562$
 $a_{se1} = \text{Max}(((A_{conf,max1} - A_{noConf1}) / A_{conf,max1}) * (A_{conf,min1} / A_{conf,max1}), 0) = 0.33836562$
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.

Aconf,max1 = 89600.00 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 = 59225.00 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 = 43733.333 is the unconfined external core area which is equal to $b_i^2/6$ as defined at (A.2).
ase2 (\geq ase1) = $\text{Max}(((A_{\text{conf,max2}} - A_{\text{noConf2}})/A_{\text{conf,max2}}) * (A_{\text{conf,min2}}/A_{\text{conf,max2}}), 0) = 0.33836562$

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 = 45264.00 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 = 0.00 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 = 32309.333 is the unconfined internal core area which is equal to $b_i^2/6$ as defined at (A.2).
psh,min*Fywe = $\text{Min}(psh_x * Fywe, psh_y * Fywe) = 6.12205$

Expression (5.4d) for psh,min*Fywe has been multiplied by 0.3 according to 15.7.1.3 for members without
earthquake detailing (90° closed stirrups)

psh_x*Fywe = psh1*Fywe1 + ps2*Fywe2 = 6.12205
psh1 ((5.4d), TBDY) = $L_{\text{stir1}} * A_{\text{stir1}} / (A_{\text{sec}} * s_1) = 0.00680678$
Lstir1 (Length of stirrups along Y) = 1040.00
Astir1 (stirrups area) = 78.53982
psh2 (5.4d) = $L_{\text{stir2}} * A_{\text{stir2}} / (A_{\text{sec}} * s_2) = 0.0012868$
Lstir2 (Length of stirrups along Y) = 768.00
Astir2 (stirrups area) = 50.26548

psh_y*Fywe = psh1*Fywe1 + ps2*Fywe2 = 6.12205
psh1 ((5.4d), TBDY) = $L_{\text{stir1}} * A_{\text{stir1}} / (A_{\text{sec}} * s_1) = 0.00680678$
Lstir1 (Length of stirrups along X) = 1040.00
Astir1 (stirrups area) = 78.53982
psh2 ((5.4d), TBDY) = $L_{\text{stir2}} * A_{\text{stir2}} / (A_{\text{sec}} * s_2) = 0.0012868$
Lstir2 (Length of stirrups along X) = 768.00
Astir2 (stirrups area) = 50.26548

Asec = 120000.00

s1 = 100.00

s2 = 250.00

fywe1 = 781.25

fywe2 = 625.00

fce = 30.00

From ((5.A5), TBDY), TBDY: cc = 0.002

c = confinement factor = 1.00

y1 = 0.0025

sh1 = 0.008

ft1 = 898.293

fy1 = 748.5775

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 = $0.4 * e_{s1_nominal} ((5.5), TBDY) = 0.032$

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 $\text{Min}(1, 1.25 * (lb/lb)^{2/3})$, from 10.3.5, ASCE 41-17.

with fs1 = $(f_{s,jacket} * A_{s,ten,jacket} + f_{s,core} * A_{s,ten,core}) / A_{s,ten} = 748.5775$

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 = 886.7403
fy2 = 738.9503
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/ld)^ 2/3), from 10.3.5, ASCE 41-17.
    with fs2 = (fs,jacket*Asl,com,jacket + fs,core*Asl,com,core)/Asl,com = 738.9503
    with Es2 = (Es,jacket*Asl,com,jacket + Es,core*Asl,com,core)/Asl,com = 200000.00
    yv = 0.0025
    shv = 0.008
ftv = 894.3662
fyv = 745.3052
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 = 745.3052
    with Esv = (Es,jacket*Asl,mid,jacket + Es,mid*Asl,mid,core)/Asl,mid = 200000.00
    1 = Asl,ten/(b*d)*(fs1/fc) = 0.7508007
    2 = Asl,com/(b*d)*(fs2/fc) = 0.38164216
    v = Asl,mid/(b*d)*(fsv/fc) = 0.67946571
and confined core properties:
b = 160.00
d = 347.00
d' = 13.00
fcc (5A.2, TBDY) = 30.00
cc (5A.5, TBDY) = 0.002
    c = confinement factor = 1.00
    1 = Asl,ten/(b*d)*(fs1/fc) = 0.99259314
    2 = Asl,com/(b*d)*(fs2/fc) = 0.50454853
    v = Asl,mid/(b*d)*(fsv/fc) = 0.898285
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) = 0.8262842
MRc (4.18) = 4.9554E+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, 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

---->

$*cu$ (4.11) = 0.90160009

M_{Ro} (4.18) = 2.4685E+008

$M_{Ro} < 0.8 * M_{Rc}$

---->

$u = cu$ (unconfined full section) = 3.0526570E-005

$\mu = M_{Rc}$

Calculation of ratio l_b/d

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

Calculation of Shear Strength $V_r = \min(V_{r1}, V_{r2}) = 349917.706$

Calculation of Shear Strength at edge 1, $V_{r1} = 349917.706$

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

$V_{Col0} = 349917.706$

$k_n l = 1$ (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: $f'_c = (f'_{c_jacket} * Area_jacket + f'_{c_core} * Area_core) / Area_section = 25.00$, but $f'_c^{0.5} \leq 8.3$ MPa (22.5.3.1, ACI 318-14)

$M/V_d = 4.00$

$\mu_u = 8.5591E+007$

$V_u = 5.7948283E-005$

$d = 0.8 * h = 320.00$

$N_u = 2.1578E+006$

$A_g = 80000.00$

From (11.5.4.8), ACI 318-14: $V_s = V_{sjacket} + V_{s,core} = 471238.898$

where:

$V_{sjacket} = V_{sj1} + V_{sj2} = 471238.898$

$V_{sj1} = 157079.633$ is calculated for section web jacket, with:

$d = 160.00$

$A_v = 157079.633$

$f_y = 625.00$

$s = 100.00$

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

$s/d = 0.625$

$V_{sj2} = 314159.265$ is calculated for section flange jacket, with:

$d = 320.00$

$A_v = 157079.633$

$f_y = 625.00$

$s = 100.00$

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

$s/d = 0.3125$

$V_{s,core} = V_{s,c1} + V_{s,c2} = 0.00$
 $V_{s,c1} = 0.00$ is calculated for section web core, with:
 $d = 80.00$
 $A_v = 100530.965$
 $f_y = 500.00$
 $s = 250.00$
 $V_{s,c1}$ is multiplied by $Col,c1 = 0.00$
 $s/d = 3.125$
 $V_{s,c2} = 0.00$ is calculated for section flange core, with:
 $d = 240.00$
 $A_v = 100530.965$
 $f_y = 500.00$
 $s = 250.00$
 $V_{s,c2}$ is multiplied by $Col,c2 = 0.00$
 $s/d = 1.04167$
 $V_f ((11-3)-(11.4), ACI 440) = 0.00$
 From (11-11), ACI 440: $V_s + V_f \leq 212577.225$
 $bw = 200.00$

Calculation of Shear Strength at edge 2, $V_{r2} = 349917.706$
 $V_{r2} = V_{Col} ((10.3), ASCE 41-17) = knl * V_{Col0}$
 $V_{Col0} = 349917.706$
 $knl = 1$ (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: $f'_c = (f'_c_{jacket} * Area_{jacket} + f'_c_{core} * Area_{core}) / Area_{section} = 25.00$, but $f'_c^{0.5} \leq 8.3$
 MPa (22.5.3.1, ACI 318-14)
 $M/Vd = 4.00$
 $\mu_u = 6.4291E+007$
 $V_u = 5.7948283E-005$
 $d = 0.8 * h = 320.00$
 $N_u = 2.1578E+006$
 $A_g = 80000.00$
 From (11.5.4.8), ACI 318-14: $V_s = V_{s,jacket} + V_{s,core} = 471238.898$
 where:
 $V_{s,jacket} = V_{s,j1} + V_{s,j2} = 471238.898$
 $V_{s,j1} = 157079.633$ is calculated for section web jacket, with:
 $d = 160.00$
 $A_v = 157079.633$
 $f_y = 625.00$
 $s = 100.00$
 $V_{s,j1}$ is multiplied by $Col,j1 = 1.00$
 $s/d = 0.625$
 $V_{s,j2} = 314159.265$ is calculated for section flange jacket, with:
 $d = 320.00$
 $A_v = 157079.633$
 $f_y = 625.00$
 $s = 100.00$
 $V_{s,j2}$ is multiplied by $Col,j2 = 1.00$
 $s/d = 0.3125$
 $V_{s,core} = V_{s,c1} + V_{s,c2} = 0.00$
 $V_{s,c1} = 0.00$ is calculated for section web core, with:
 $d = 80.00$
 $A_v = 100530.965$
 $f_y = 500.00$
 $s = 250.00$
 $V_{s,c1}$ is multiplied by $Col,c1 = 0.00$
 $s/d = 3.125$
 $V_{s,c2} = 0.00$ is calculated for section flange core, with:
 $d = 240.00$
 $A_v = 100530.965$
 $f_y = 500.00$
 $s = 250.00$

$V_{s,c2}$ is multiplied by $Col,c2 = 0.00$
 $s/d = 1.04167$
 $V_f ((11-3)-(11.4), ACI 440) = 0.00$
From (11-11), ACI 440: $V_s + V_f \leq 212577.225$
 $bw = 200.00$

End Of Calculation of Shear Capacity ratio for element: column JLC1 of floor 1
At local axis: 2

Start Of Calculation of Chord Rotation Capacity for element: column JLC1 of floor 1
At local axis: 3
Integration Section: (a)
Section Type: rcjlc

Constant Properties

Knowledge Factor, $\gamma = 0.90$
Chord Rotation is generally considered as Deformation-Controlled Action according to Table C7-1, ASCE 41-17.
Mean strengths are used for Deformation-Controlled Actions according to 7.5.1.3, ASCE 41-17
Consequently:
Jacket
New material of Secondary Member: Concrete Strength, $f_c = f_{cm} = 30.00$
New material of Secondary Member: Steel Strength, $f_s = f_{sm} = 625.00$
Concrete Elasticity, $E_c = 25742.96$
Steel Elasticity, $E_s = 200000.00$
Existing Column
Existing material of Secondary Member: Concrete Strength, $f_c = f_{cm} = 18.00$
Existing material of Secondary Member: Steel Strength, $f_s = f_{sm} = 500.00$
Concrete Elasticity, $E_c = 19940.411$
Steel Elasticity, $E_s = 200000.00$
Max Height, $H_{max} = 400.00$
Min Height, $H_{min} = 200.00$
Max Width, $W_{max} = 400.00$
Min Width, $W_{min} = 200.00$
Jacket Thickness, $t_j = 50.00$
Cover Thickness, $c = 15.00$
Element Length, $L = 3000.00$
Secondary Member
Smooth Bars
Ductile Steel
Without Detailing for Earthquake Resistance (including stirrups not 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, $M = 8.5589E+007$
Shear Force, $V_2 = -0.60572401$
Shear Force, $V_3 = -1.4589530E-005$
Axial Force, $F = -2.1578E+006$
Longitudinal Reinforcement Area Distribution (in 2 divisions)
-Tension: $As_t = 0.00$
-Compression: $As_c = 5353.274$
Longitudinal Reinforcement Area Distribution (in 3 divisions)
-Tension: $As_{t,ten} = 1137.257$
-Compression: $As_{l,com} = 2208.54$
-Middle: $As_{l,mid} = 2007.478$
Longitudinal External Reinforcement Area Distribution (in 3 divisions)
-Tension: $As_{l,ten,jacket} = 829.3805$
-Compression: $As_{l,com,jacket} = 1746.726$
-Middle: $As_{l,mid,jacket} = 1545.664$

Longitudinal Internal Reinforcement Area Distribution (in 3 divisions)

-Tension: $A_{sl,ten,core} = 307.8761$

-Compression: $A_{sl,com,core} = 461.8141$

-Middle: $A_{sl,mid,core} = 461.8141$

Mean Diameter of Tension Reinforcement, $D_bL = 16.80$

Existing component: From table 7-7, ASCE 41_17: Final chord rotation Capacity $u_R = u = 0.03784131$

$u = y + p = 0.0420459$

- Calculation of y -

$y = (M_y * L_s / 3) / E_{eff} = 0.0420459$ ((4.29), Biskinis Phd))

$M_y = 5.2455E+008$

$L_s = M/V$ (with $L_s > 0.1 * L$ and $L_s < 2 * L$) = 6000.00

From table 10.5, ASCE 41_17: $E_{eff} = factor * E_c * I_g = 2.4951E+013$

factor = 0.70

$A_g = 120000.00$

Mean concrete strength: $f_c' = (f_c'_{jacket} * Area_{jacket} + f_c'_{core} * Area_{core}) / Area_{section} = 25.00$

$N = 2.1578E+006$

$E_c * I_g = E_{c,jacket} * I_{g,jacket} + E_{c,core} * I_{g,core} = 3.5645E+013$

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, $b = 400.00$

web width, $b_w = 200.00$

flange thickness, $t = 200.00$

$y = \text{Min}(y_{ten}, y_{com})$

$y_{ten} = 1.4921411E-005$

with $f_y = 596.2441$

$d = 367.00$

$y = 0.45559994$

$A = 0.06111896$

$B = 0.04120454$

with $pt = 0.00774698$

$pc = 0.01504455$

$pv = 0.01367492$

$N = 2.1578E+006$

$b = 400.00$

$" = 0.08991826$

$y_{comp} = 1.1520096E-005$

with $f_c = 30.00$

$E_c = 25742.96$

$y = 0.49615013$

$A = 0.00143$

$B = 0.01655203$

with $E_s = 200000.00$

CONFIRMATION: $y = 0.49615013 < t/d$

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 $l_b/l_d \geq 1$
shear control ratio $V_y E / V_{ColOE} = 1.10794$

$d = d_{external} = 367.00$

$s = s_{external} = 100.00$

- $t = s_1 + s_2 + 2 \cdot t_f / b_w \cdot (f_{fe} / f_s) = 0.00809358$

jacket: $s_1 = A_{v1} \cdot L_{stir1} / (s_1 \cdot A_g) = 0.00680678$

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

$L_{stir1} = 1040.00$, is the total Length of all stirrups parallel to loading (shear) direction

$s_1 = 100.00$

core: $s_2 = A_{v2} \cdot L_{stir2} / (s_2 \cdot A_g) = 0.0012868$

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

$L_{stir2} = 768.00$, 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.

$NUD = 2.1578E+006$

$A_g = 120000.00$

$f_{cE} = (f_{c,jacket} \cdot Area_{jacket} + f_{c,core} \cdot Area_{core}) / section_area = 25.00$

$f_{yIE} = (f_{y,ext_Long_Reinf} \cdot Area_{ext_Long_Reinf} + f_{y,int_Long_Reinf} \cdot Area_{int_Long_Reinf}) / Area_{Tot_Long_Rein} = 596.2441$

$f_{yIE} = (f_{y,ext_Trans_Reinf} \cdot s_1 + f_{y,int_Trans_Reinf} \cdot s_2) / (s_1 + s_2) = 605.1263$

$\rho_l = Area_{Tot_Long_Rein} / (b \cdot d) = 0.03646644$

$b = 400.00$

$d = 367.00$

$f_{cE} = 25.00$

End Of Calculation of Chord Rotation Capacity for element: column JLC1 of floor 1

At local axis: 3

Integration Section: (a)

Calculation No. 5

column C1, Floor 1

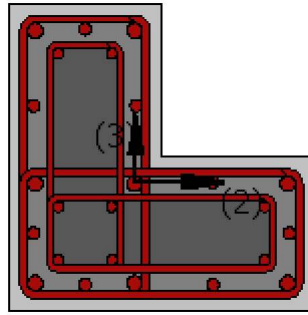
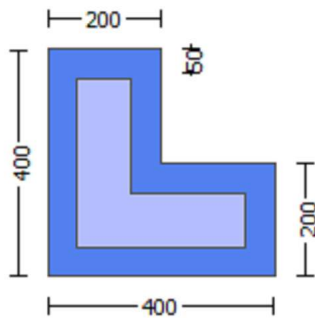
Limit State: Operational Level (data interpolation between analysis steps 1 and 2)

Analysis: Uniform +X

Check: Shear capacity V_{Rd}

Edge: End

Local Axis: (2)



Start Of Calculation of Shear Capacity for element: column JLC1 of floor 1

At local axis: 2

Integration Section: (b)

Section Type: rcjics

Constant Properties

Knowledge Factor, $\gamma = 0.90$

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

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

Consequently:

Jacket

New material of Secondary Member: Concrete Strength, $f_c = f_{c_lower_bound} = 20.00$

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

Concrete Elasticity, $E_c = 25742.96$

Steel Elasticity, $E_s = 200000.00$

Existing Column

Existing material of Secondary Member: Concrete Strength, $f_c = f_{c_lower_bound} = 12.00$

Existing material of Secondary Member: Steel Strength, $f_s = f_{s_lower_bound} = 400.00$

Concrete Elasticity, $E_c = 19940.411$

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, $f_c = f_{cm} = 30.00$

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

Existing Column

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

Existing material: Steel Strength, $f_s = f_{sm} = 500.00$

#####

Max Height, $H_{max} = 400.00$

Min Height, $H_{min} = 200.00$

Max Width, $W_{max} = 400.00$

Min Width, $W_{min} = 200.00$

Jacket Thickness, $t_j = 50.00$

Cover Thickness, $c = 15.00$

Element Length, $L = 3000.00$

Secondary Member

Smooth Bars

Ductile Steel

Without Detailing for Earthquake Resistance (including stirrups not 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, $M_a = 8.5589E+007$

Shear Force, $V_a = -0.60572401$
 EDGE -B-
 Bending Moment, $M_b = -6.4291E+007$
 Shear Force, $V_b = 0.60572401$
 BOTH EDGES
 Axial Force, $F = -2.1578E+006$
 Longitudinal Reinforcement Area Distribution (in 2 divisions)
 -Tension: $A_{st} = 0.00$
 -Compression: $A_{sc} = 5353.274$
 Longitudinal Reinforcement Area Distribution (in 3 divisions)
 -Tension: $A_{st,ten} = 1137.257$
 -Compression: $A_{sc,com} = 2208.54$
 -Middle: $A_{sc,mid} = 2007.478$
 Mean Diameter of Tension Reinforcement, $Db_{L,ten} = 16.80$

Existing component: From table 7-7, ASCE 41_17: Final Shear Capacity $V_R = \phi V_n = 267030.06$
 V_n ((10.3), ASCE 41-17) = $k_n \phi V_{CoI} = 296700.066$
 $V_{CoI} = 296700.066$
 $k_n = 1.00$
 $displacement_ductility_demand = 0.07087185$

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).

$\phi = 1$ (normal-weight concrete)
 Mean concrete strength: $f'_c = (f'_{c,jacket} \cdot Area_{jacket} + f'_{c,core} \cdot Area_{core}) / Area_{section} = 16.66667$, but $f'_c^{0.5} \leq 8.3$ MPa (22.5.3.1, ACI 318-14)
 $M/Vd = 4.00$
 $M_u = 6.4291E+007$
 $V_u = 0.60572401$
 $d = 0.8 \cdot h = 320.00$
 $N_u = 2.1578E+006$
 $A_g = 80000.00$
 From (11.5.4.8), ACI 318-14: $V_s = V_{s,jacket} + V_{s,core} = 376991.118$
 where:
 $V_{s,jacket} = V_{s,j1} + V_{s,j2} = 376991.118$
 $V_{s,j1} = 125663.706$ is calculated for section web jacket, with:
 $d = 160.00$
 $A_v = 157079.633$
 $f_y = 500.00$
 $s = 100.00$
 $V_{s,j1}$ is multiplied by $Col,j1 = 1.00$
 $s/d = 0.625$
 $V_{s,j2} = 251327.412$ is calculated for section flange jacket, with:
 $d = 320.00$
 $A_v = 157079.633$
 $f_y = 500.00$
 $s = 100.00$
 $V_{s,j2}$ is multiplied by $Col,j2 = 1.00$
 $s/d = 0.3125$
 $V_{s,core} = V_{s,c1} + V_{s,c2} = 0.00$
 $V_{s,c1} = 0.00$ is calculated for section web core, with:
 $d = 80.00$
 $A_v = 100530.965$
 $f_y = 400.00$
 $s = 250.00$
 $V_{s,c1}$ is multiplied by $Col,c1 = 0.00$
 $s/d = 3.125$
 $V_{s,c2} = 0.00$ is calculated for section flange core, with:
 $d = 240.00$
 $A_v = 100530.965$
 $f_y = 400.00$
 $s = 250.00$
 $V_{s,c2}$ is multiplied by $Col,c2 = 0.00$

$s/d = 1.04167$
 $V_f((11-3)-(11.4), ACI\ 440) = 0.00$
From $(11-11)$, ACI 440: $V_s + V_f \leq 173568.578$
 $bw = 200.00$

displacement ductility demand is calculated as ϕ / y

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

From analysis, chord rotation $\theta = 0.00297987$
 $y = (M_y * L_s / 3) / E_{eff} = 0.0420459$ ((4.29), Biskinis Phd))
 $M_y = 5.2455E+008$
 $L_s = M/V$ (with $L_s > 0.1 * L$ and $L_s < 2 * L$) = 6000.00
From table 10.5, ASCE 41_17: $E_{eff} = factor * E_c * I_g = 2.4951E+013$
 $factor = 0.70$
 $A_g = 120000.00$
Mean concrete strength: $f'_c = (f'_c_{jacket} * Area_{jacket} + f'_c_{core} * Area_{core}) / Area_{section} = 25.00$
 $N = 2.1578E+006$
 $E_c * I_g = E_c_{jacket} * I_{g_jacket} + E_c_{core} * I_{g_core} = 3.5645E+013$

Calculation of Yielding Moment M_y

Calculation of ϕ and M_y according to Annex 7 -

Assuming neutral axis within flange ($y < t/d$, compression zone rectangular) with:
flange width, $b = 400.00$
web width, $bw = 200.00$
flange thickness, $t = 200.00$

$y = \text{Min}(y_{ten}, y_{com})$
 $y_{ten} = 1.4921411E-005$
with $f_y = 596.2441$
 $d = 367.00$
 $y = 0.45559994$
 $A = 0.06111896$
 $B = 0.04120454$
with $pt = 0.00774698$
 $pc = 0.01504455$
 $pv = 0.01367492$
 $N = 2.1578E+006$
 $b = 400.00$
 $" = 0.08991826$
 $y_{comp} = 1.1520096E-005$
with $f_c = 30.00$
 $E_c = 25742.96$
 $y = 0.49615013$
 $A = 0.00143$
 $B = 0.01655203$
with $E_s = 200000.00$
CONFIRMATION: $y = 0.49615013 < t/d$

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 JLC1 of floor 1
At local axis: 2
Integration Section: (b)

Calculation No. 6

column C1, Floor 1

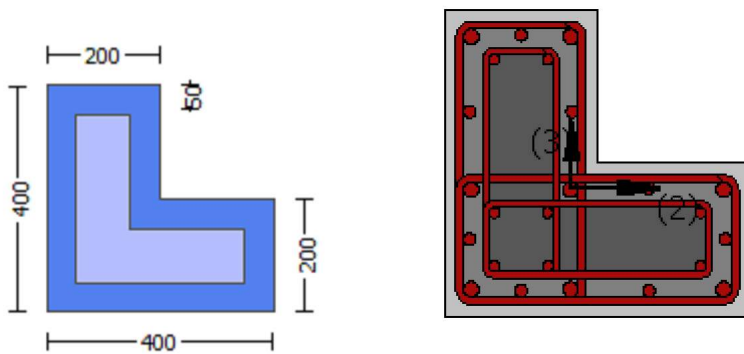
Limit State: Operational Level (data interpolation between analysis steps 1 and 2)

Analysis: Uniform +X

Check: Chord rotation capacity (μ)

Edge: End

Local Axis: (2)



Start Of Calculation of Shear Capacity ratio for element: column JLC1 of floor 1

At Shear local axis: 3

(Bending local axis: 2)

Section Type: rcjlc

Constant Properties

Knowledge Factor, $\gamma = 0.90$

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

Consequently:

Jacket

New material of Secondary Member: Concrete Strength, $f_c = f_{cm} = 30.00$

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

Concrete Elasticity, $E_c = 25742.96$

Steel Elasticity, $E_s = 200000.00$

Existing Column

Existing material of Secondary Member: Concrete Strength, $f_c = f_{cm} = 18.00$

Existing material of Secondary Member: Steel Strength, $f_s = f_{sm} = 500.00$

Concrete Elasticity, $E_c = 19940.411$

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, $f_s = 1.25 \cdot f_{sm} = 625.00$

#####

Max Height, $H_{max} = 400.00$

Min Height, $H_{min} = 200.00$

Max Width, $W_{max} = 400.00$
 Min Width, $W_{min} = 200.00$
 Jacket Thickness, $t_j = 50.00$
 Cover Thickness, $c = 15.00$
 Mean Confinement Factor overall section = 1.00
 Element Length, $L = 3000.00$
 Secondary Member
 Smooth Bars
 Ductile Steel
 Without Detailing for Earthquake Resistance (including stirrups not 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, $V_a = -1.4594946E-005$
 EDGE -B-
 Shear Force, $V_b = 1.4594946E-005$
 BOTH EDGES
 Axial Force, $F = -2.1578E+006$
 Longitudinal Reinforcement Area Distribution (in 2 divisions)
 -Tension: $As_t = 0.00$
 -Compression: $As_c = 5353.274$
 Longitudinal Reinforcement Area Distribution (in 3 divisions)
 -Tension: $As_{ten} = 2208.54$
 -Compression: $As_{com} = 1137.257$
 -Middle: $As_{mid} = 2007.478$

Calculation of Shear Capacity ratio, $V_e/V_r = 1.37616$
 Member Controlled by Shear ($V_e/V_r > 1$)
 Calculation of Shear Demand from fig. R18.6.5, ACI 318-14 $V_e = (M_{pr1} + M_{pr2})/l_n = 481543.08$
 with
 $M_{pr1} = \max(\mu_{u1+}, \mu_{u1-}) = 7.2231E+008$
 $\mu_{u1+} = 7.2231E+008$, is the ultimate moment strength at the edge 1 of the member in the actual moment direction which is defined for the static loading combination
 $\mu_{u1-} = 6.3496E+008$, 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
 $M_{pr2} = \max(\mu_{u2+}, \mu_{u2-}) = 7.2231E+008$
 $\mu_{u2+} = 7.2231E+008$, 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
 $\mu_{u2-} = 6.3496E+008$, 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:
 $\phi_u = 3.9059244E-005$
 $\mu_u = 7.2231E+008$

with full section properties:

$b = 400.00$
 $d = 367.00$
 $d' = 33.00$
 $v = 0.48996389$
 $N = 2.1578E+006$
 $f_c = 30.00$
 $\phi_c (5A.5, TBDY) = 0.002$

Final value of cu : $cu^* = \text{shear_factor} * \text{Max}(cu, cc) = 0.00925707$
The Shear_factor is considered equal to 1 (pure moment strength)
From (5.4b), TBDY: $cu = 0.00925707$
we (5.4c) = 0.0207149
 $ase((5.4d), \text{TBDY}) = (ase1 * A_{ext} + ase2 * A_{int}) / A_{sec} = 0.33836562$
 $ase1 = \text{Max}(((A_{conf,max1} - A_{noConf1}) / A_{conf,max1}) * (A_{conf,min1} / A_{conf,max1}), 0) = 0.33836562$
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.
 $A_{conf,max1} = 89600.00$ 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.
 $A_{conf,min1} = 59225.00$ 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.
 $A_{noConf1} = 43733.333$ is the unconfined external core area which is equal to $b_i^2/6$ as defined at (A.2).
 $ase2 (>= ase1) = \text{Max}(((A_{conf,max2} - A_{noConf2}) / A_{conf,max2}) * (A_{conf,min2} / A_{conf,max2}), 0) = 0.33836562$
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.
 $A_{conf,max2} = 45264.00$ 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.
 $A_{conf,min2} = 0.00$ 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.
 $A_{noConf2} = 32309.333$ is the unconfined internal core area which is equal to $b_i^2/6$ as defined at (A.2).
 $psh_{min} * F_{ywe} = \text{Min}(psh_x * F_{ywe}, psh_y * F_{ywe}) = 6.12205$
Expression (5.4d) for $psh_{min} * F_{ywe}$ has been multiplied by 0.3 according to 15.7.1.3 for members without earthquake detailing (90° closed stirrups)

 $psh_x * F_{ywe} = psh1 * F_{ywe1} + ps2 * F_{ywe2} = 6.12205$
 $psh1((5.4d), \text{TBDY}) = L_{stir1} * A_{stir1} / (A_{sec} * s1) = 0.00680678$
 L_{stir1} (Length of stirrups along Y) = 1040.00
 A_{stir1} (stirrups area) = 78.53982
 $ps2(5.4d) = L_{stir2} * A_{stir2} / (A_{sec} * s2) = 0.0012868$
 L_{stir2} (Length of stirrups along Y) = 768.00
 A_{stir2} (stirrups area) = 50.26548

$psh_y * F_{ywe} = psh1 * F_{ywe1} + ps2 * F_{ywe2} = 6.12205$
 $psh1((5.4d), \text{TBDY}) = L_{stir1} * A_{stir1} / (A_{sec} * s1) = 0.00680678$
 L_{stir1} (Length of stirrups along X) = 1040.00
 A_{stir1} (stirrups area) = 78.53982
 $ps2((5.4d), \text{TBDY}) = L_{stir2} * A_{stir2} / (A_{sec} * s2) = 0.0012868$
 L_{stir2} (Length of stirrups along X) = 768.00
 A_{stir2} (stirrups area) = 50.26548

$A_{sec} = 120000.00$
 $s1 = 100.00$
 $s2 = 250.00$
 $fywe1 = 781.25$
 $fywe2 = 625.00$
 $f_{ce} = 30.00$

From ((5.A5), TBDY), TBDY: $cc = 0.002$
 $c = \text{confinement factor} = 1.00$

$y1 = 0.0025$
 $sh1 = 0.008$
 $ft1 = 898.293$
 $fy1 = 748.5775$
 $su1 = 0.032$

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
 $lo/lo_{min} = lb/ld = 1.00$
 $su1 = 0.4 * esu1_{nominal}((5.5), \text{TBDY}) = 0.032$

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 $Min(1, 1.25 \cdot (lb/ld)^{2/3})$, from 10.3.5, ASCE 41-17.
with $fs1 = (fs_jacket \cdot Asl_ten_jacket + fs_core \cdot Asl_ten_core) / Asl_ten = 748.5775$
with $Es1 = (Es_jacket \cdot Asl_ten_jacket + Es_core \cdot Asl_ten_core) / Asl_ten = 200000.00$
 $y2 = 0.0025$
 $sh2 = 0.008$
 $ft2 = 886.7403$
 $fy2 = 738.9503$
 $su2 = 0.032$
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 $Min(1, 1.25 \cdot (lb/ld)^{2/3})$, from 10.3.5, ASCE 41-17.
with $fs2 = (fs_jacket \cdot Asl_com_jacket + fs_core \cdot Asl_com_core) / Asl_com = 738.9503$
with $Es2 = (Es_jacket \cdot Asl_com_jacket + Es_core \cdot Asl_com_core) / Asl_com = 200000.00$
 $yv = 0.0025$
 $shv = 0.008$
 $ftv = 894.3662$
 $fyv = 745.3052$
 $suv = 0.032$
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 $Min(1, 1.25 \cdot (lb/ld)^{2/3})$, from 10.3.5, ASCE 41-17.
with $fsv = (fs_jacket \cdot Asl_mid_jacket + fs_mid \cdot Asl_mid_core) / Asl_mid = 745.3052$
with $Es_v = (Es_jacket \cdot Asl_mid_jacket + Es_mid \cdot Asl_mid_core) / Asl_mid = 200000.00$
 $1 = Asl_ten / (b \cdot d) \cdot (fs1 / fc) = 0.37540035$
 $2 = Asl_com / (b \cdot d) \cdot (fs2 / fc) = 0.19082108$
 $v = Asl_mid / (b \cdot d) \cdot (fsv / fc) = 0.33973286$
and confined core properties:
 $b = 360.00$
 $d = 347.00$
 $d' = 13.00$
 $fcc (5A.2, TBDY) = 30.00$
 $cc (5A.5, TBDY) = 0.002$
 $c = \text{confinement factor} = 1.00$
 $1 = Asl_ten / (b \cdot d) \cdot (fs1 / fc) = 0.44115251$
 $2 = Asl_com / (b \cdot d) \cdot (fs2 / fc) = 0.22424379$
 $v = Asl_mid / (b \cdot d) \cdot (fsv / fc) = 0.39923778$
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) = 0.64577857
MRc (4.17) = 7.2231E+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) = 0.67946111
MRo (4.18) = 4.2866E+008
MRo < 0.8*MRc
---->
u = cu (unconfined full section) = 3.9059244E-005
Mu = MRc
-----

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 = 4.2871818E-005
Mu = 6.3496E+008
-----

with full section properties:
b = 200.00
d = 367.00
d' = 33.00
v = 0.97992777
N = 2.1578E+006
fc = 30.00
co (5A.5, TBDY) = 0.002
Final value of cu: cu* = shear_factor * Max( cu, cc) = 0.00925707
The Shear_factor is considered equal to 1 (pure moment strength)
From (5.4b), TBDY: cu = 0.00925707
we (5.4c) = 0.0207149
ase ((5.4d), TBDY) = (ase1*Aext+ase2*Aint)/Asec = 0.33836562
ase1 = Max(((Aconf,max1-AnoConf1)/Aconf,max1)*(Aconf,min1/Aconf,max1),0) = 0.33836562
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 = 89600.00 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 = 59225.00 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 = 43733.333 is the unconfined external core area which is equal to $b_i^2/6$ as defined at (A.2).
ase2 (\geq ase1) = $\text{Max}(((A_{\text{conf,max2}} - A_{\text{noConf2}})/A_{\text{conf,max2}}) * (A_{\text{conf,min2}}/A_{\text{conf,max2}}), 0) = 0.33836562$

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 = 45264.00 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 = 0.00 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 = 32309.333 is the unconfined internal core area which is equal to $b_i^2/6$ as defined at (A.2).

psh,min*Fywe = $\text{Min}(psh_x * Fywe, psh_y * Fywe) = 6.12205$

Expression (5.4d) for psh,min*Fywe has been multiplied by 0.3 according to 15.7.1.3 for members without earthquake detailing (90° closed stirrups)

psh_x*Fywe = psh1*Fywe1 + ps2*Fywe2 = 6.12205

psh1 ((5.4d), TBDY) = $L_{\text{stir1}} * A_{\text{stir1}} / (A_{\text{sec}} * s_1) = 0.00680678$

Lstir1 (Length of stirrups along Y) = 1040.00

Astir1 (stirrups area) = 78.53982

psh2 (5.4d) = $L_{\text{stir2}} * A_{\text{stir2}} / (A_{\text{sec}} * s_2) = 0.0012868$

Lstir2 (Length of stirrups along Y) = 768.00

Astir2 (stirrups area) = 50.26548

psh_y*Fywe = psh1*Fywe1 + ps2*Fywe2 = 6.12205

psh1 ((5.4d), TBDY) = $L_{\text{stir1}} * A_{\text{stir1}} / (A_{\text{sec}} * s_1) = 0.00680678$

Lstir1 (Length of stirrups along X) = 1040.00

Astir1 (stirrups area) = 78.53982

psh2 ((5.4d), TBDY) = $L_{\text{stir2}} * A_{\text{stir2}} / (A_{\text{sec}} * s_2) = 0.0012868$

Lstir2 (Length of stirrups along X) = 768.00

Astir2 (stirrups area) = 50.26548

Asec = 120000.00

s1 = 100.00

s2 = 250.00

fywe1 = 781.25

fywe2 = 625.00

fce = 30.00

From ((5.A5), TBDY), TBDY: cc = 0.002

c = confinement factor = 1.00

y1 = 0.0025

sh1 = 0.008

ft1 = 886.7403

fy1 = 738.9503

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

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 = $(f_{s,jacket} * A_{s,ten,jacket} + f_{s,core} * A_{s,ten,core}) / A_{s,ten} = 738.9503$

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 = 898.293

fy2 = 748.5775

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 = 748.5775
with Es2 = (Es,jacket*Asl,com,jacket + Es,core*Asl,com,core)/Asl,com = 200000.00
yv = 0.0025
shv = 0.008
ftv = 894.3662
fyv = 745.3052
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 = 745.3052
with Esv = (Es,jacket*Asl,mid,jacket + Es,mid*Asl,mid,core)/Asl,mid = 200000.00
1 = Asl,ten/(b*d)*(fs1/fc) = 0.38164216
2 = Asl,com/(b*d)*(fs2/fc) = 0.7508007
v = Asl,mid/(b*d)*(fsv/fc) = 0.67946571
and confined core properties:
b = 160.00
d = 347.00
d' = 13.00
fcc (5A.2, TBDY) = 30.00
cc (5A.5, TBDY) = 0.002
c = confinement factor = 1.00
1 = Asl,ten/(b*d)*(fs1/fc) = 0.50454853
2 = Asl,com/(b*d)*(fs2/fc) = 0.99259314
v = Asl,mid/(b*d)*(fsv/fc) = 0.898285
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) = 0.58834972
MRc (4.17) = 6.3496E+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*,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

σ_{cu} (4.11) = 0.63603191

M_{Ro} (4.18) = 4.4768E+008

$M_{Ro} < 0.8 \cdot M_{Rc}$

$u = u_{cu}$ (unconfined full section) = 4.2871818E-005

$\mu_u = \mu_{Rc}$

Calculation of ratio I_b/I_d

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

Calculation of μ_{u2+}

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

$\kappa_u = 3.9059244E-005$

$\mu_u = 7.2231E+008$

with full section properties:

$b = 400.00$

$d = 367.00$

$d' = 33.00$

$v = 0.48996389$

$N = 2.1578E+006$

$f_c = 30.00$

σ_{co} (5A.5, TBDY) = 0.002

Final value of σ_{cu} : $\sigma_{cu} = \text{shear_factor} \cdot \text{Max}(\sigma_{cu}, \sigma_{cc}) = 0.00925707$

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

From (5.4b), TBDY: $\sigma_{cu} = 0.00925707$

ω_e (5.4c) = 0.0207149

ω_{ase} ((5.4d), TBDY) = $(\omega_{ase1} \cdot A_{ext} + \omega_{ase2} \cdot A_{int}) / A_{sec} = 0.33836562$

$\omega_{ase1} = \text{Max}(((A_{conf,max1} - A_{noConf1}) / A_{conf,max1}) \cdot (A_{conf,min1} / A_{conf,max1}), 0) = 0.33836562$

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.

$A_{conf,max1} = 89600.00$ 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.

$A_{conf,min1} = 59225.00$ 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.

$A_{noConf1} = 43733.333$ is the unconfined external core area which is equal to $b^2/6$ as defined at (A.2).

$\omega_{ase2} (\geq \omega_{ase1}) = \text{Max}(((A_{conf,max2} - A_{noConf2}) / A_{conf,max2}) \cdot (A_{conf,min2} / A_{conf,max2}), 0) = 0.33836562$

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.

$A_{conf,max2} = 45264.00$ 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.

$A_{conf,min2} = 0.00$ 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 = 32309.333 is the unconfined internal core area which is equal to $b_i^2/6$ as defined at (A.2).

$psh_{min} \cdot Fywe = \min(psh_x \cdot Fywe, psh_y \cdot Fywe) = 6.12205$

Expression (5.4d) for $psh_{min} \cdot Fywe$ has been multiplied by 0.3 according to 15.7.1.3 for members without earthquake detailing (90° closed stirrups)

$psh_x \cdot Fywe = psh1 \cdot Fywe1 + ps2 \cdot Fywe2 = 6.12205$
 $psh1 \text{ ((5.4d), TBDY)} = Lstir1 \cdot Astir1 / (Asec \cdot s1) = 0.00680678$
 $Lstir1 \text{ (Length of stirrups along Y)} = 1040.00$
 $Astir1 \text{ (stirrups area)} = 78.53982$
 $psh2 \text{ (5.4d)} = Lstir2 \cdot Astir2 / (Asec \cdot s2) = 0.0012868$
 $Lstir2 \text{ (Length of stirrups along Y)} = 768.00$
 $Astir2 \text{ (stirrups area)} = 50.26548$

$psh_y \cdot Fywe = psh1 \cdot Fywe1 + ps2 \cdot Fywe2 = 6.12205$
 $psh1 \text{ ((5.4d), TBDY)} = Lstir1 \cdot Astir1 / (Asec \cdot s1) = 0.00680678$
 $Lstir1 \text{ (Length of stirrups along X)} = 1040.00$
 $Astir1 \text{ (stirrups area)} = 78.53982$
 $psh2 \text{ ((5.4d), TBDY)} = Lstir2 \cdot Astir2 / (Asec \cdot s2) = 0.0012868$
 $Lstir2 \text{ (Length of stirrups along X)} = 768.00$
 $Astir2 \text{ (stirrups area)} = 50.26548$

Asec = 120000.00

s1 = 100.00

s2 = 250.00

fywe1 = 781.25

fywe2 = 625.00

fce = 30.00

From ((5.A5), TBDY), TBDY: cc = 0.002

c = confinement factor = 1.00

y1 = 0.0025

sh1 = 0.008

ft1 = 898.293

fy1 = 748.5775

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 $\min(1, 1.25 \cdot (lb/lb)^{2/3})$, from 10.3.5, ASCE 41-17.

with fs1 = (fs,jacket*Asl,ten,jacket + fs,core*Asl,ten,core)/Asl,ten = 748.5775

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

y2 = 0.0025

sh2 = 0.008

ft2 = 886.7403

fy2 = 738.9503

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 $\min(1, 1.25 \cdot (lb/lb)^{2/3})$, from 10.3.5, ASCE 41-17.

with fs2 = (fs,jacket*Asl,com,jacket + fs,core*Asl,com,core)/Asl,com = 738.9503

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

yv = 0.0025

shv = 0.008

ftv = 894.3662

fyv = 745.3052

```

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 = 745.3052
with Esv = (Esjacket*Aslmid,jacket + Esmid*Aslmid,core)/Aslmid = 200000.00
1 = Aslten/(b*d)*(fs1/fc) = 0.37540035
2 = Aslcom/(b*d)*(fs2/fc) = 0.19082108
v = Aslmid/(b*d)*(fsv/fc) = 0.33973286
and confined core properties:
b = 360.00
d = 347.00
d' = 13.00
fcc (5A.2, TBDY) = 30.00
cc (5A.5, TBDY) = 0.002
c = confinement factor = 1.00
1 = Aslten/(b*d)*(fs1/fc) = 0.44115251
2 = Aslcom/(b*d)*(fs2/fc) = 0.22424379
v = Aslmid/(b*d)*(fsv/fc) = 0.39923778
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) = 0.64577857
MRc (4.17) = 7.2231E+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
- 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*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) = 0.67946111
MRo (4.18) = 4.2866E+008

```

$$M_{Ro} < 0.8 \cdot M_{Rc}$$

---->

$$u = c_u \text{ (unconfined full section)} = 3.9059244E-005$$

$$M_u = M_{Rc}$$

Calculation of ratio l_b/l_d

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

Calculation of M_{u2} -

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

$$u = 4.2871818E-005$$

$$M_u = 6.3496E+008$$

with full section properties:

$$b = 200.00$$

$$d = 367.00$$

$$d' = 33.00$$

$$v = 0.97992777$$

$$N = 2.1578E+006$$

$$f_c = 30.00$$

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

$$\text{Final value of } c_u: c_u^* = \text{shear_factor} \cdot \text{Max}(c_u, c_c) = 0.00925707$$

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

$$\text{From (5.4b), TBDY: } c_u = 0.00925707$$

$$w_e \text{ (5.4c)} = 0.0207149$$

$$a_{se} \text{ ((5.4d), TBDY)} = (a_{se1} \cdot A_{ext} + a_{se2} \cdot A_{int}) / A_{sec} = 0.33836562$$

$$a_{se1} = \text{Max}(((A_{conf,max1} - A_{noConf1}) / A_{conf,max1}) \cdot (A_{conf,min1} / A_{conf,max1}), 0) = 0.33836562$$

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.

$A_{conf,max1} = 89600.00$ 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.

$A_{conf,min1} = 59225.00$ 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.

$A_{noConf1} = 43733.333$ is the unconfined external core area which is equal to $b_i^2/6$ as defined at (A.2).

$$a_{se2} (\geq a_{se1}) = \text{Max}(((A_{conf,max2} - A_{noConf2}) / A_{conf,max2}) \cdot (A_{conf,min2} / A_{conf,max2}), 0) = 0.33836562$$

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.

$A_{conf,max2} = 45264.00$ 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.

$A_{conf,min2} = 0.00$ 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.

$A_{noConf2} = 32309.333$ is the unconfined internal core area which is equal to $b_i^2/6$ as defined at (A.2).

$$p_{sh,min} \cdot F_{ywe} = \text{Min}(p_{sh,x} \cdot F_{ywe}, p_{sh,y} \cdot F_{ywe}) = 6.12205$$

Expression (5.4d) for $p_{sh,min} \cdot F_{ywe}$ has been multiplied by 0.3 according to 15.7.1.3 for members without earthquake detailing (90° closed stirrups)

$$p_{sh,x} \cdot F_{ywe} = p_{sh1} \cdot F_{ywe1} + p_{s2} \cdot F_{ywe2} = 6.12205$$

$$p_{sh1} \text{ ((5.4d), TBDY)} = L_{stir1} \cdot A_{stir1} / (A_{sec} \cdot s_1) = 0.00680678$$

$$L_{stir1} \text{ (Length of stirrups along Y)} = 1040.00$$

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

$$p_{sh2} \text{ (5.4d)} = L_{stir2} \cdot A_{stir2} / (A_{sec} \cdot s_2) = 0.0012868$$

$$L_{stir2} \text{ (Length of stirrups along Y)} = 768.00$$

Astir2 (stirrups area) = 50.26548

psh_y*Fywe = psh1*Fywe1+ps2*Fywe2 = 6.12205
psh1 ((5.4d), TBDY) = Lstir1*Astir1/(Asec*s1) = 0.00680678
Lstir1 (Length of stirrups along X) = 1040.00
Astir1 (stirrups area) = 78.53982
psh2 ((5.4d), TBDY) = Lstir2*Astir2/(Asec*s2) = 0.0012868
Lstir2 (Length of stirrups along X) = 768.00
Astir2 (stirrups area) = 50.26548

Asec = 120000.00

s1 = 100.00

s2 = 250.00

fywe1 = 781.25

fywe2 = 625.00

fce = 30.00

From ((5.A.5), TBDY), TBDY: cc = 0.002

c = confinement factor = 1.00

y1 = 0.0025

sh1 = 0.008

ft1 = 886.7403

fy1 = 738.9503

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 = 738.9503

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

y2 = 0.0025

sh2 = 0.008

ft2 = 898.293

fy2 = 748.5775

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 = 748.5775

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

yv = 0.0025

shv = 0.008

ftv = 894.3662

fyv = 745.3052

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 = 745.3052

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

```

1 = Asl,ten/(b*d)*(fs1/fc) = 0.38164216
2 = Asl,com/(b*d)*(fs2/fc) = 0.7508007
v = Asl,mid/(b*d)*(fsv/fc) = 0.67946571
and confined core properties:
b = 160.00
d = 347.00
d' = 13.00
fcc (5A.2, TBDY) = 30.00
cc (5A.5, TBDY) = 0.002
c = confinement factor = 1.00
1 = Asl,ten/(b*d)*(fs1/fc) = 0.50454853
2 = Asl,com/(b*d)*(fs2/fc) = 0.99259314
v = Asl,mid/(b*d)*(fsv/fc) = 0.898285
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) = 0.58834972
MRc (4.17) = 6.3496E+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) = 0.63603191
MRo (4.18) = 4.4768E+008
MRo < 0.8*MRc
--->
u = cu (unconfined full section) = 4.2871818E-005
Mu = MRc

```

```

-----
Calculation of ratio lb/d
-----

```

```

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

```

Calculation of Shear Strength $V_r = \text{Min}(V_{r1}, V_{r2}) = 349917.706$

Calculation of Shear Strength at edge 1, $V_{r1} = 349917.706$

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

$V_{Col0} = 349917.706$

$knl = 1$ (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: $f_c' = (f_c'_{jacket} * Area_{jacket} + f_c'_{core} * Area_{core}) / Area_{section} = 25.00$, but $f_c'^{0.5} \leq 8.3$ MPa (22.5.3.1, ACI 318-14)

$M/Vd = 4.00$

$M_u = 3.4090E+006$

$V_u = 1.4594946E+005$

$d = 0.8 * h = 320.00$

$N_u = 2.1578E+006$

$A_g = 80000.00$

From (11.5.4.8), ACI 318-14: $V_s = V_{sjacket} + V_{s,core} = 471238.898$

where:

$V_{sjacket} = V_{sj1} + V_{sj2} = 471238.898$

$V_{sj1} = 314159.265$ is calculated for section web jacket, with:

$d = 320.00$

$A_v = 157079.633$

$f_y = 625.00$

$s = 100.00$

V_{sj1} is multiplied by $Col_{j1} = 1.00$

$s/d = 0.3125$

$V_{sj2} = 157079.633$ is calculated for section flange jacket, with:

$d = 160.00$

$A_v = 157079.633$

$f_y = 625.00$

$s = 100.00$

V_{sj2} is multiplied by $Col_{j2} = 1.00$

$s/d = 0.625$

$V_{s,core} = V_{s,c1} + V_{s,c2} = 0.00$

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

$d = 240.00$

$A_v = 100530.965$

$f_y = 500.00$

$s = 250.00$

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

$s/d = 1.04167$

$V_{s,c2} = 0.00$ is calculated for section flange core, with:

$d = 80.00$

$A_v = 100530.965$

$f_y = 500.00$

$s = 250.00$

$V_{s,c2}$ is multiplied by $Col_{c2} = 0.00$

$s/d = 3.125$

$V_f \text{ ((11-3)-(11.4), ACI 440)} = 0.00$

From (11-11), ACI 440: $V_s + V_f \leq 212577.225$

$bw = 200.00$

Calculation of Shear Strength at edge 2, $V_{r2} = 349917.706$

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

$V_{Col0} = 349917.706$

$knl = 1$ (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: $f_c' = (f_c'_{jacket} * Area_{jacket} + f_c'_{core} * Area_{core}) / Area_{section} = 25.00$, but $f_c'^{0.5} \leq 8.3$ MPa (22.5.3.1, ACI 318-14)

$M/Vd = 4.00$
 $\mu_u = 15336.932$
 $V_u = 1.4594946E-005$
 $d = 0.8 \cdot h = 320.00$
 $N_u = 2.1578E+006$
 $A_g = 80000.00$
 From (11.5.4.8), ACI 318-14: $V_s = V_{sjacket} + V_{s,core} = 471238.898$
 where:
 $V_{sjacket} = V_{sj1} + V_{sj2} = 471238.898$
 $V_{sj1} = 314159.265$ is calculated for section web jacket, with:
 $d = 320.00$
 $A_v = 157079.633$
 $f_y = 625.00$
 $s = 100.00$
 V_{sj1} is multiplied by $Col,j1 = 1.00$
 $s/d = 0.3125$
 $V_{sj2} = 157079.633$ is calculated for section flange jacket, with:
 $d = 160.00$
 $A_v = 157079.633$
 $f_y = 625.00$
 $s = 100.00$
 V_{sj2} is multiplied by $Col,j2 = 1.00$
 $s/d = 0.625$
 $V_{s,core} = V_{s,c1} + V_{s,c2} = 0.00$
 $V_{s,c1} = 0.00$ is calculated for section web core, with:
 $d = 240.00$
 $A_v = 100530.965$
 $f_y = 500.00$
 $s = 250.00$
 $V_{s,c1}$ is multiplied by $Col,c1 = 0.00$
 $s/d = 1.04167$
 $V_{s,c2} = 0.00$ is calculated for section flange core, with:
 $d = 80.00$
 $A_v = 100530.965$
 $f_y = 500.00$
 $s = 250.00$
 $V_{s,c2}$ is multiplied by $Col,c2 = 0.00$
 $s/d = 3.125$
 $V_f ((11-3)-(11.4), ACI 440) = 0.00$
 From (11-11), ACI 440: $V_s + V_f \leq 212577.225$
 $bw = 200.00$

 End Of Calculation of Shear Capacity ratio for element: column JLC1 of floor 1
 At local axis: 3

 Start Of Calculation of Shear Capacity ratio for element: column JLC1 of floor 1
 At Shear local axis: 2
 (Bending local axis: 3)
 Section Type: rcjlc3

Constant Properties

 Knowledge Factor, $\phi = 0.90$
 Mean strength values are used for both shear and moment calculations.
 Consequently:
 Jacket
 New material of Secondary Member: Concrete Strength, $f_c = f_{cm} = 30.00$
 New material of Secondary Member: Steel Strength, $f_s = f_{sm} = 625.00$
 Concrete Elasticity, $E_c = 25742.96$
 Steel Elasticity, $E_s = 200000.00$
 Existing Column
 Existing material of Secondary Member: Concrete Strength, $f_c = f_{cm} = 18.00$
 Existing material of Secondary Member: Steel Strength, $f_s = f_{sm} = 500.00$

Concrete Elasticity, $E_c = 19940.411$
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, $f_s = 1.25 \cdot f_{sm} = 625.00$

Max Height, $H_{max} = 400.00$
Min Height, $H_{min} = 200.00$
Max Width, $W_{max} = 400.00$
Min Width, $W_{min} = 200.00$
Jacket Thickness, $t_j = 50.00$
Cover Thickness, $c = 15.00$
Mean Confinement Factor overall section = 1.00
Element Length, $L = 3000.00$
Secondary Member
Smooth Bars
Ductile Steel
Without Detailing for Earthquake Resistance (including stirrups not 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, $V_a = -5.7948283E-005$
EDGE -B-
Shear Force, $V_b = 5.7948283E-005$
BOTH EDGES
Axial Force, $F = -2.1578E+006$
Longitudinal Reinforcement Area Distribution (in 2 divisions)
-Tension: $A_{slt} = 0.00$
-Compression: $A_{slc} = 5353.274$
Longitudinal Reinforcement Area Distribution (in 3 divisions)
-Tension: $A_{sl,ten} = 1137.257$
-Compression: $A_{sl,com} = 2208.54$
-Middle: $A_{sl,mid} = 2007.478$

Calculation of Shear Capacity ratio , $V_e/V_r = 1.10794$
Member Controlled by Shear ($V_e/V_r > 1$)
Calculation of Shear Demand from fig. R18.6.5, ACI 318-14 $V_e = (M_{pr1} + M_{pr2})/l_n = 387689.388$
with
 $M_{pr1} = \text{Max}(\mu_{u1+}, \mu_{u1-}) = 5.8153E+008$
 $\mu_{u1+} = 5.8153E+008$, is the ultimate moment strength at the edge 1 of the member in the actual moment direction
which is defined for the static loading combination
 $\mu_{u1-} = 4.9554E+008$, 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
 $M_{pr2} = \text{Max}(\mu_{u2+}, \mu_{u2-}) = 5.8153E+008$
 $\mu_{u2+} = 5.8153E+008$, is the ultimate moment strength at the edge 2 of the member in the actual moment direction
which is defined for the the static loading combination
 $\mu_{u2-} = 4.9554E+008$, 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:

$$\phi_u = 1.1511372E-005$$

$$\mu = 5.8153E+008$$

with full section properties:

$$b = 400.00$$

$$d = 367.00$$

$$d' = 33.00$$

$$v = 0.48996389$$

$$N = 2.1578E+006$$

$$f_c = 30.00$$

$$\phi_0 (5A.5, \text{TB DY}) = 0.002$$

$$\text{Final value of } \phi_u^* = \text{shear_factor} * \text{Max}(\phi_u, \phi_0) = 0.00925707$$

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

$$\text{From (5.4b), TB DY: } \phi_u = 0.00925707$$

$$\phi_{ue} (5.4c) = 0.0207149$$

$$\phi_{ase} ((5.4d), \text{TB DY}) = (\phi_{ase1} * A_{ext} + \phi_{ase2} * A_{int}) / A_{sec} = 0.33836562$$

$$\phi_{ase1} = \text{Max}(((A_{conf,max1} - A_{noConf1}) / A_{conf,max1}) * (A_{conf,min1} / A_{conf,max1}), 0) = 0.33836562$$

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.

$A_{conf,max1} = 89600.00$ 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.

$A_{conf,min1} = 59225.00$ 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.

$A_{noConf1} = 43733.333$ is the unconfined external core area which is equal to $b_i^2/6$ as defined at (A.2).

$$\phi_{ase2} (> \phi_{ase1}) = \text{Max}(((A_{conf,max2} - A_{noConf2}) / A_{conf,max2}) * (A_{conf,min2} / A_{conf,max2}), 0) = 0.33836562$$

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.

$A_{conf,max2} = 45264.00$ 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.

$A_{conf,min2} = 0.00$ 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.

$A_{noConf2} = 32309.333$ is the unconfined internal core area which is equal to $b_i^2/6$ as defined at (A.2).

$$\phi_{psh,min} * F_{ywe} = \text{Min}(\phi_{psh,x} * F_{ywe}, \phi_{psh,y} * F_{ywe}) = 6.12205$$

Expression (5.4d) for $\phi_{psh,min} * F_{ywe}$ has been multiplied by 0.3 according to 15.7.1.3 for members without earthquake detailing (90° closed stirrups)

$$\phi_{psh,x} * F_{ywe} = \phi_{psh1} * F_{ywe1} + \phi_{ps2} * F_{ywe2} = 6.12205$$

$$\phi_{psh1} ((5.4d), \text{TB DY}) = L_{stir1} * A_{stir1} / (A_{sec} * s_1) = 0.00680678$$

L_{stir1} (Length of stirrups along Y) = 1040.00

A_{stir1} (stirrups area) = 78.53982

$$\phi_{psh2} (5.4d) = L_{stir2} * A_{stir2} / (A_{sec} * s_2) = 0.0012868$$

L_{stir2} (Length of stirrups along Y) = 768.00

A_{stir2} (stirrups area) = 50.26548

$$\phi_{psh,y} * F_{ywe} = \phi_{psh1} * F_{ywe1} + \phi_{ps2} * F_{ywe2} = 6.12205$$

$$\phi_{psh1} ((5.4d), \text{TB DY}) = L_{stir1} * A_{stir1} / (A_{sec} * s_1) = 0.00680678$$

L_{stir1} (Length of stirrups along X) = 1040.00

A_{stir1} (stirrups area) = 78.53982

$$\phi_{psh2} ((5.4d), \text{TB DY}) = L_{stir2} * A_{stir2} / (A_{sec} * s_2) = 0.0012868$$

L_{stir2} (Length of stirrups along X) = 768.00

A_{stir2} (stirrups area) = 50.26548

$$A_{sec} = 120000.00$$

$$s_1 = 100.00$$

$$s_2 = 250.00$$

$$f_{ywe1} = 781.25$$

$$f_{ywe2} = 625.00$$

$$f_{ce} = 30.00$$

From ((5A.5), TBDY), TBDY: $cc = 0.002$
 $c = \text{confinement factor} = 1.00$
 $y1 = 0.0025$
 $sh1 = 0.008$
 $ft1 = 886.7403$
 $fy1 = 738.9503$
 $su1 = 0.032$
 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 $fs1 = (fs_jacket * Asl, \text{ten}, \text{jacket} + fs_core * Asl, \text{ten}, \text{core}) / Asl, \text{ten} = 738.9503$
 with $Es1 = (Es_jacket * Asl, \text{ten}, \text{jacket} + Es_core * Asl, \text{ten}, \text{core}) / Asl, \text{ten} = 200000.00$
 $y2 = 0.0025$
 $sh2 = 0.008$
 $ft2 = 898.293$
 $fy2 = 748.5775$
 $su2 = 0.032$
 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 $fs2 = (fs_jacket * Asl, \text{com}, \text{jacket} + fs_core * Asl, \text{com}, \text{core}) / Asl, \text{com} = 748.5775$
 with $Es2 = (Es_jacket * Asl, \text{com}, \text{jacket} + Es_core * Asl, \text{com}, \text{core}) / Asl, \text{com} = 200000.00$
 $yv = 0.0025$
 $shv = 0.008$
 $ftv = 894.3662$
 $fyv = 745.3052$
 $suv = 0.032$
 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 $fsv = (fs_jacket * Asl, \text{mid}, \text{jacket} + fs_mid * Asl, \text{mid}, \text{core}) / Asl, \text{mid} = 745.3052$
 with $Es_v = (Es_jacket * Asl, \text{mid}, \text{jacket} + Es_mid * Asl, \text{mid}, \text{core}) / Asl, \text{mid} = 200000.00$
 $1 = Asl, \text{ten} / (b * d) * (fs1 / fc) = 0.19082108$
 $2 = Asl, \text{com} / (b * d) * (fs2 / fc) = 0.37540035$
 $v = Asl, \text{mid} / (b * d) * (fsv / fc) = 0.33973286$
 and confined core properties:
 $b = 360.00$
 $d = 347.00$
 $d' = 13.00$
 $fcc (5A.2, TBDY) = 30.00$
 $cc (5A.5, TBDY) = 0.002$
 $c = \text{confinement factor} = 1.00$
 $1 = Asl, \text{ten} / (b * d) * (fs1 / fc) = 0.22424379$
 $2 = Asl, \text{com} / (b * d) * (fs2 / fc) = 0.44115251$
 $v = Asl, \text{mid} / (b * d) * (fsv / fc) = 0.39923778$
 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 - RHS eq.(4.6) is satisfied
---->
cu (4.10) = 0.42533033
MRc (4.17) = 7.2277E+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
- 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) = 0.50069522
MRo (4.18) = 5.8153E+008
---->
u = cu (4.2) = 1.1511372E-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 = 3.0526570E-005
Mu = 4.9554E+008

```

with full section properties:

```

b = 200.00
d = 367.00
d' = 33.00
v = 0.97992777
N = 2.1578E+006
fc = 30.00
co (5A.5, TBDY) = 0.002
Final value of cu: cu* = shear_factor * Max( cu, cc) = 0.00925707
The Shear_factor is considered equal to 1 (pure moment strength)

```

From (5.4b), TBDY: $c_u = 0.00925707$

w_e (5.4c) = 0.0207149

a_{se} ((5.4d), TBDY) = $(a_{se1} \cdot A_{ext} + a_{se2} \cdot A_{int}) / A_{sec} = 0.33836562$

$a_{se1} = \text{Max}(((A_{conf,max1} - A_{noConf1}) / A_{conf,max1}) \cdot (A_{conf,min1} / A_{conf,max1}), 0) = 0.33836562$

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.

$A_{conf,max1} = 89600.00$ 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.

$A_{conf,min1} = 59225.00$ 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.

$A_{noConf1} = 43733.333$ is the unconfined external core area which is equal to $b_i^2/6$ as defined at (A.2).

$a_{se2} (>= a_{se1}) = \text{Max}(((A_{conf,max2} - A_{noConf2}) / A_{conf,max2}) \cdot (A_{conf,min2} / A_{conf,max2}), 0) = 0.33836562$

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.

$A_{conf,max2} = 45264.00$ 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.

$A_{conf,min2} = 0.00$ 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.

$A_{noConf2} = 32309.333$ is the unconfined internal core area which is equal to $b_i^2/6$ as defined at (A.2).

$p_{sh,min} \cdot F_{ywe} = \text{Min}(p_{sh,x} \cdot F_{ywe}, p_{sh,y} \cdot F_{ywe}) = 6.12205$

Expression (5.4d) for $p_{sh,min} \cdot F_{ywe}$ has been multiplied by 0.3 according to 15.7.1.3 for members without earthquake detailing (90° closed stirrups)

$p_{sh,x} \cdot F_{ywe} = p_{sh1} \cdot F_{ywe1} + p_{s2} \cdot F_{ywe2} = 6.12205$

p_{sh1} ((5.4d), TBDY) = $L_{stir1} \cdot A_{stir1} / (A_{sec} \cdot s_1) = 0.00680678$

L_{stir1} (Length of stirrups along Y) = 1040.00

A_{stir1} (stirrups area) = 78.53982

p_{sh2} (5.4d) = $L_{stir2} \cdot A_{stir2} / (A_{sec} \cdot s_2) = 0.0012868$

L_{stir2} (Length of stirrups along Y) = 768.00

A_{stir2} (stirrups area) = 50.26548

$p_{sh,y} \cdot F_{ywe} = p_{sh1} \cdot F_{ywe1} + p_{s2} \cdot F_{ywe2} = 6.12205$

p_{sh1} ((5.4d), TBDY) = $L_{stir1} \cdot A_{stir1} / (A_{sec} \cdot s_1) = 0.00680678$

L_{stir1} (Length of stirrups along X) = 1040.00

A_{stir1} (stirrups area) = 78.53982

p_{sh2} ((5.4d), TBDY) = $L_{stir2} \cdot A_{stir2} / (A_{sec} \cdot s_2) = 0.0012868$

L_{stir2} (Length of stirrups along X) = 768.00

A_{stir2} (stirrups area) = 50.26548

$A_{sec} = 120000.00$

$s_1 = 100.00$

$s_2 = 250.00$

$f_{ywe1} = 781.25$

$f_{ywe2} = 625.00$

$f_{ce} = 30.00$

From ((5.A5), TBDY), TBDY: $c_c = 0.002$

c = confinement factor = 1.00

$y_1 = 0.0025$

$sh_1 = 0.008$

$ft_1 = 898.293$

$f_{y1} = 748.5775$

$su_1 = 0.032$

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, f_{y1} , 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  $f_{s1} = (f_{sjacket} \cdot A_{sl,ten,jacket} + f_{s,core} \cdot A_{sl,ten,core})/A_{sl,ten} = 748.5775$ 
with  $E_{s1} = (E_{sjacket} \cdot A_{sl,ten,jacket} + E_{s,core} \cdot A_{sl,ten,core})/A_{sl,ten} = 200000.00$ 
 $y2 = 0.0025$ 
 $sh2 = 0.008$ 
 $ft2 = 886.7403$ 
 $fy2 = 738.9503$ 
 $su2 = 0.032$ 
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
 $l_o/l_{ou,min} = l_b/l_{b,min} = 1.00$ 
 $su2 = 0.4 \cdot esu2\_nominal ((5.5), TBDY) = 0.032$ 
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  $f_{s2} = (f_{sjacket} \cdot A_{sl,com,jacket} + f_{s,core} \cdot A_{sl,com,core})/A_{sl,com} = 738.9503$ 
with  $E_{s2} = (E_{sjacket} \cdot A_{sl,com,jacket} + E_{s,core} \cdot A_{sl,com,core})/A_{sl,com} = 200000.00$ 
 $yv = 0.0025$ 
 $shv = 0.008$ 
 $ftv = 894.3662$ 
 $fyv = 745.3052$ 
 $suv = 0.032$ 
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  $yv, shv, ftv, fyv$ , 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  $f_{sv} = (f_{sjacket} \cdot A_{sl,mid,jacket} + f_{s,mid} \cdot A_{sl,mid,core})/A_{sl,mid} = 745.3052$ 
with  $E_{sv} = (E_{sjacket} \cdot A_{sl,mid,jacket} + E_{s,mid} \cdot A_{sl,mid,core})/A_{sl,mid} = 200000.00$ 
 $1 = A_{sl,ten}/(b \cdot d) \cdot (f_{s1}/f_c) = 0.7508007$ 
 $2 = A_{sl,com}/(b \cdot d) \cdot (f_{s2}/f_c) = 0.38164216$ 
 $v = A_{sl,mid}/(b \cdot d) \cdot (f_{sv}/f_c) = 0.67946571$ 
and confined core properties:
 $b = 160.00$ 
 $d = 347.00$ 
 $d' = 13.00$ 
 $f_{cc} (5A.2, TBDY) = 30.00$ 
 $cc (5A.5, TBDY) = 0.002$ 
 $c = \text{confinement factor} = 1.00$ 
 $1 = A_{sl,ten}/(b \cdot d) \cdot (f_{s1}/f_c) = 0.99259314$ 
 $2 = A_{sl,com}/(b \cdot d) \cdot (f_{s2}/f_c) = 0.50454853$ 
 $v = A_{sl,mid}/(b \cdot d) \cdot (f_{sv}/f_c) = 0.898285$ 
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
---->
 $cu (4.11) = 0.8262842$ 

```

MRC (4.18) = 4.9554E+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) = 0.90160009

MRO (4.18) = 2.4685E+008

MRO < 0.8*MRC

--->

u = cu (unconfined full section) = 3.0526570E-005

Mu = MRC

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 = 1.1511372E-005

Mu = 5.8153E+008

with full section properties:

b = 400.00

d = 367.00

d' = 33.00

v = 0.48996389

N = 2.1578E+006

fc = 30.00

co (5A.5, TBDY) = 0.002

Final value of cu: cu* = shear_factor * Max(cu, cc) = 0.00925707

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

From (5.4b), TBDY: cu = 0.00925707

we (5.4c) = 0.0207149

ase ((5.4d), TBDY) = (ase1*Aext+ase2*Aint)/Asec = 0.33836562

ase1 = Max(((Aconf,max1-AnoConf1)/Aconf,max1)*(Aconf,min1/Aconf,max1),0) = 0.33836562

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 = 89600.00 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 = 59225.00 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 = 43733.333 is the unconfined external core area which is equal to $b_i^2/6$ as defined at (A.2).
 $ase2 (>= ase1) = \text{Max}(((A_{conf,max2} - \text{AnoConf2})/A_{conf,max2}) * (A_{conf,min2}/A_{conf,max2}), 0) = 0.33836562$

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 = 45264.00 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 = 0.00 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 = 32309.333 is the unconfined internal core area which is equal to $b_i^2/6$ as defined at (A.2).

$psh_{min} * F_{ywe} = \text{Min}(psh_x * F_{ywe}, psh_y * F_{ywe}) = 6.12205$

Expression (5.4d) for $psh_{min} * F_{ywe}$ has been multiplied by 0.3 according to 15.7.1.3 for members without earthquake detailing (90° closed stirrups)

 $psh_x * F_{ywe} = psh1 * F_{ywe1} + ps2 * F_{ywe2} = 6.12205$
 $psh1 ((5.4d), TBDY) = L_{stir1} * A_{stir1} / (A_{sec} * s1) = 0.00680678$
Lstir1 (Length of stirrups along Y) = 1040.00
Astir1 (stirrups area) = 78.53982
 $psh2 ((5.4d) = L_{stir2} * A_{stir2} / (A_{sec} * s2) = 0.0012868$
Lstir2 (Length of stirrups along Y) = 768.00
Astir2 (stirrups area) = 50.26548

 $psh_y * F_{ywe} = psh1 * F_{ywe1} + ps2 * F_{ywe2} = 6.12205$
 $psh1 ((5.4d), TBDY) = L_{stir1} * A_{stir1} / (A_{sec} * s1) = 0.00680678$
Lstir1 (Length of stirrups along X) = 1040.00
Astir1 (stirrups area) = 78.53982
 $psh2 ((5.4d), TBDY) = L_{stir2} * A_{stir2} / (A_{sec} * s2) = 0.0012868$
Lstir2 (Length of stirrups along X) = 768.00
Astir2 (stirrups area) = 50.26548

Asec = 120000.00

s1 = 100.00

s2 = 250.00

fywe1 = 781.25

fywe2 = 625.00

fce = 30.00

From ((5.A5), TBDY), TBDY: cc = 0.002

c = confinement factor = 1.00

y1 = 0.0025

sh1 = 0.008

ft1 = 886.7403

fy1 = 738.9503

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,tens,jacket + fs,core * Asl,tens,core) / Asl,tens = 738.9503

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

y2 = 0.0025

sh2 = 0.008

ft2 = 898.293

fy2 = 748.5775

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 = 748.5775
with Es2 = (Es,jacket*Asl,com,jacket + Es,core*Asl,com,core)/Asl,com = 200000.00
yv = 0.0025
shv = 0.008
ftv = 894.3662
fyv = 745.3052
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 = 745.3052
with Esv = (Es,jacket*Asl,mid,jacket + Es,mid*Asl,mid,core)/Asl,mid = 200000.00
1 = Asl,ten/(b*d)*(fs1/fc) = 0.19082108
2 = Asl,com/(b*d)*(fs2/fc) = 0.37540035
v = Asl,mid/(b*d)*(fsv/fc) = 0.33973286
and confined core properties:
b = 360.00
d = 347.00
d' = 13.00
fcc (5A.2, TBDY) = 30.00
cc (5A.5, TBDY) = 0.002
c = confinement factor = 1.00
1 = Asl,ten/(b*d)*(fs1/fc) = 0.22424379
2 = Asl,com/(b*d)*(fs2/fc) = 0.44115251
v = Asl,mid/(b*d)*(fsv/fc) = 0.39923778
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) = 0.42533033
MRc (4.17) = 7.2277E+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
---->
 $\mu_{cu}$  (4.11) = 0.50069522
MRo (4.18) = 5.8153E+008
---->
 $\mu = \mu_{cu}$  (4.2) = 1.1511372E-005
Mu = 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 μ_u according to 4.1, Biskinis/Fardis 2013:

$\mu_u = 3.0526570E-005$

$\mu_u = 4.9554E+008$

with full section properties:

$b = 200.00$

$d = 367.00$

$d' = 33.00$

$v = 0.97992777$

$N = 2.1578E+006$

$f_c = 30.00$

μ_{co} (5A.5, TBDY) = 0.002

Final value of μ_{cu} : $\mu_{cu}^* = \text{shear_factor} * \text{Max}(\mu_{cu}, \mu_{cc}) = 0.00925707$

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

From (5.4b), TBDY: $\mu_{cu} = 0.00925707$

μ_{we} (5.4c) = 0.0207149

μ_{ase} ((5.4d), TBDY) = $(\mu_{ase1} * A_{ext} + \mu_{ase2} * A_{int}) / A_{sec} = 0.33836562$

$\mu_{ase1} = \text{Max}(((A_{conf,max1} - A_{noConf1}) / A_{conf,max1}) * (A_{conf,min1} / A_{conf,max1}), 0) = 0.33836562$

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.

$A_{conf,max1} = 89600.00$ 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.

$A_{conf,min1} = 59225.00$ 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.

$A_{noConf1} = 43733.333$ is the unconfined external core area which is equal to $b_i^2/6$ as defined at (A.2).

$\mu_{ase2} (\geq \mu_{ase1}) = \text{Max}(((A_{conf,max2} - A_{noConf2}) / A_{conf,max2}) * (A_{conf,min2} / A_{conf,max2}), 0) = 0.33836562$

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.

$A_{conf,max2} = 45264.00$ 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.

$A_{conf,min2} = 0.00$ 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.

$A_{noConf2} = 32309.333$ is the unconfined internal core area which is equal to $b_i^2/6$ as defined at (A.2).

$psh_{min} \cdot F_{ywe} = \text{Min}(psh_x \cdot F_{ywe}, psh_y \cdot F_{ywe}) = 6.12205$
 Expression (5.4d) for $psh_{min} \cdot F_{ywe}$ has been multiplied by 0.3 according to 15.7.1.3 for members without earthquake detailing (90° closed stirrups)

$psh_x \cdot F_{ywe} = psh1 \cdot F_{ywe1} + ps2 \cdot F_{ywe2} = 6.12205$
 $psh1$ ((5.4d), TBDY) = $Lstir1 \cdot Astir1 / (Asec \cdot s1) = 0.00680678$
 $Lstir1$ (Length of stirrups along Y) = 1040.00
 $Astir1$ (stirrups area) = 78.53982
 $psh2$ (5.4d) = $Lstir2 \cdot Astir2 / (Asec \cdot s2) = 0.0012868$
 $Lstir2$ (Length of stirrups along Y) = 768.00
 $Astir2$ (stirrups area) = 50.26548

$psh_y \cdot F_{ywe} = psh1 \cdot F_{ywe1} + ps2 \cdot F_{ywe2} = 6.12205$
 $psh1$ ((5.4d), TBDY) = $Lstir1 \cdot Astir1 / (Asec \cdot s1) = 0.00680678$
 $Lstir1$ (Length of stirrups along X) = 1040.00
 $Astir1$ (stirrups area) = 78.53982
 $psh2$ ((5.4d), TBDY) = $Lstir2 \cdot Astir2 / (Asec \cdot s2) = 0.0012868$
 $Lstir2$ (Length of stirrups along X) = 768.00
 $Astir2$ (stirrups area) = 50.26548

$Asec = 120000.00$

$s1 = 100.00$

$s2 = 250.00$

$fy_{we1} = 781.25$

$fy_{we2} = 625.00$

$f_{ce} = 30.00$

From ((5.A5), TBDY), TBDY: $cc = 0.002$

$c = \text{confinement factor} = 1.00$

$y1 = 0.0025$

$sh1 = 0.008$

$ft1 = 898.293$

$fy1 = 748.5775$

$su1 = 0.032$

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

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

$su1 = 0.4 \cdot esu1_{nominal}$ ((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 $fs1 = (fs_{jacket} \cdot Asl, \text{ten}, \text{jacket} + fs_{core} \cdot Asl, \text{ten}, \text{core}) / Asl, \text{ten} = 748.5775$

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

$y2 = 0.0025$

$sh2 = 0.008$

$ft2 = 886.7403$

$fy2 = 738.9503$

$su2 = 0.032$

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

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

$su2 = 0.4 \cdot esu2_{nominal}$ ((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 $fs2 = (fs_{jacket} \cdot Asl, \text{com}, \text{jacket} + fs_{core} \cdot Asl, \text{com}, \text{core}) / Asl, \text{com} = 738.9503$

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

$yv = 0.0025$

$shv = 0.008$

$ftv = 894.3662$

$fyv = 745.3052$

$suv = 0.032$

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 = 745.3052
with Esv = (Esjacket*Asl,mid,jacket + Es,mid*Asl,mid,core)/Asl,mid = 200000.00
1 = Asl,ten/(b*d)*(fs1/fc) = 0.7508007
2 = Asl,com/(b*d)*(fs2/fc) = 0.38164216
v = Asl,mid/(b*d)*(fsv/fc) = 0.67946571
and confined core properties:
b = 160.00
d = 347.00
d' = 13.00
fcc (5A.2, TBDY) = 30.00
cc (5A.5, TBDY) = 0.002
c = confinement factor = 1.00
1 = Asl,ten/(b*d)*(fs1/fc) = 0.99259314
2 = Asl,com/(b*d)*(fs2/fc) = 0.50454853
v = Asl,mid/(b*d)*(fsv/fc) = 0.898285
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) = 0.8262842
MRc (4.18) = 4.9554E+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) = 0.90160009
MRo (4.18) = 2.4685E+008
MRo < 0.8*MRc
---->
u = cu (unconfined full section) = 3.0526570E-005

```

Mu = MRc

Calculation of ratio lb/d

Adequate Lap Length: lb/d >= 1

Calculation of Shear Strength Vr = Min(Vr1,Vr2) = 349917.706

Calculation of Shear Strength at edge 1, Vr1 = 349917.706

Vr1 = VCol ((10.3), ASCE 41-17) = knl*VCol0

VCol0 = 349917.706

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 = 25.00, but fc'^0.5 <= 8.3 MPa (22.5.3.1, ACI 318-14)

M/Vd = 4.00

Mu = 8.5591E+007

Vu = 5.7948283E+005

d = 0.8*h = 320.00

Nu = 2.1578E+006

Ag = 80000.00

From (11.5.4.8), ACI 318-14: Vs = Vs_jacket + Vs_core = 471238.898

where:

Vs_jacket = Vs_j1 + Vs_j2 = 471238.898

Vs_j1 = 157079.633 is calculated for section web jacket, with:

d = 160.00

Av = 157079.633

fy = 625.00

s = 100.00

Vs_j1 is multiplied by Col,j1 = 1.00

s/d = 0.625

Vs_j2 = 314159.265 is calculated for section flange jacket, with:

d = 320.00

Av = 157079.633

fy = 625.00

s = 100.00

Vs_j2 is multiplied by Col,j2 = 1.00

s/d = 0.3125

Vs_core = Vs_c1 + Vs_c2 = 0.00

Vs_c1 = 0.00 is calculated for section web core, with:

d = 80.00

Av = 100530.965

fy = 500.00

s = 250.00

Vs_c1 is multiplied by Col,c1 = 0.00

s/d = 3.125

Vs_c2 = 0.00 is calculated for section flange core, with:

d = 240.00

Av = 100530.965

fy = 500.00

s = 250.00

Vs_c2 is multiplied by Col,c2 = 0.00

s/d = 1.04167

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

From (11-11), ACI 440: Vs + Vf <= 212577.225

bw = 200.00

Calculation of Shear Strength at edge 2, Vr2 = 349917.706

Vr2 = VCol ((10.3), ASCE 41-17) = knl*VCol0

VCol0 = 349917.706

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: $f'_c = (f'_{c_jacket} \cdot Area_jacket + f'_{c_core} \cdot Area_core) / Area_section = 25.00$, but $f'_c^{0.5} \leq 8.3$ MPa (22.5.3.1, ACI 318-14)

$M/Vd = 4.00$

$\mu_u = 6.4291E+007$

$\mu_u = 5.7948283E-005$

$d = 0.8 \cdot h = 320.00$

$\mu_u = 2.1578E+006$

$A_g = 80000.00$

From (11.5.4.8), ACI 318-14: $V_s = V_{s,jacket} + V_{s,core} = 471238.898$

where:

$V_{s,jacket} = V_{s,j1} + V_{s,j2} = 471238.898$

$V_{s,j1} = 157079.633$ is calculated for section web jacket, with:

$d = 160.00$

$A_v = 157079.633$

$f_y = 625.00$

$s = 100.00$

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

$s/d = 0.625$

$V_{s,j2} = 314159.265$ is calculated for section flange jacket, with:

$d = 320.00$

$A_v = 157079.633$

$f_y = 625.00$

$s = 100.00$

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

$s/d = 0.3125$

$V_{s,core} = V_{s,c1} + V_{s,c2} = 0.00$

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

$d = 80.00$

$A_v = 100530.965$

$f_y = 500.00$

$s = 250.00$

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

$s/d = 3.125$

$V_{s,c2} = 0.00$ is calculated for section flange core, with:

$d = 240.00$

$A_v = 100530.965$

$f_y = 500.00$

$s = 250.00$

$V_{s,c2}$ is multiplied by $Col,c2 = 0.00$

$s/d = 1.04167$

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

From (11-11), ACI 440: $V_s + V_f \leq 212577.225$

$b_w = 200.00$

End Of Calculation of Shear Capacity ratio for element: column JLC1 of floor 1
At local axis: 2

Start Of Calculation of Chord Rotation Capacity for element: column JLC1 of floor 1
At local axis: 2
Integration Section: (b)
Section Type: rcjics

Constant Properties

Knowledge Factor, = 0.90

Chord Rotation is generally considered as Deformation-Controlled Action according to Table C7-1, ASCE 41-17.
Mean strengths are used for Deformation-Controlled Actions according to 7.5.1.3, ASCE 41-17
Consequently:
Jacket
New material of Secondary Member: Concrete Strength, $f_c = f_{cm} = 30.00$
New material of Secondary Member: Steel Strength, $f_s = f_{sm} = 625.00$
Concrete Elasticity, $E_c = 25742.96$
Steel Elasticity, $E_s = 200000.00$
Existing Column
Existing material of Secondary Member: Concrete Strength, $f_c = f_{cm} = 18.00$
Existing material of Secondary Member: Steel Strength, $f_s = f_{sm} = 500.00$
Concrete Elasticity, $E_c = 19940.411$
Steel Elasticity, $E_s = 200000.00$
Max Height, $H_{max} = 400.00$
Min Height, $H_{min} = 200.00$
Max Width, $W_{max} = 400.00$
Min Width, $W_{min} = 200.00$
Jacket Thickness, $t_j = 50.00$
Cover Thickness, $c = 15.00$
Element Length, $L = 3000.00$
Secondary Member
Smooth Bars
Ductile Steel
Without Detailing for Earthquake Resistance (including stirrups not 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, $M = 15336.81$
Shear Force, $V_2 = 0.60572401$
Shear Force, $V_3 = 1.4589530E-005$
Axial Force, $F = -2.1578E+006$
Longitudinal Reinforcement Area Distribution (in 2 divisions)
-Tension: $As_t = 0.00$
-Compression: $As_c = 5353.274$
Longitudinal Reinforcement Area Distribution (in 3 divisions)
-Tension: $As_{t,ten} = 2208.54$
-Compression: $As_{l,com} = 1137.257$
-Middle: $As_{l,mid} = 2007.478$
Longitudinal External Reinforcement Area Distribution (in 3 divisions)
-Tension: $As_{l,ten,jacket} = 1746.726$
-Compression: $As_{l,com,jacket} = 829.3805$
-Middle: $As_{l,mid,jacket} = 1545.664$
Longitudinal Internal Reinforcement Area Distribution (in 3 divisions)
-Tension: $As_{l,ten,core} = 461.8141$
-Compression: $As_{l,com,core} = 307.8761$
-Middle: $As_{l,mid,core} = 461.8141$
Mean Diameter of Tension Reinforcement, $Db_L = 16.60$

Existing component: From table 7-7, ASCE 41_17: Final chord rotation Capacity $u_R = * u = 0.03721297$
 $u = y + p = 0.04134775$

- Calculation of y -

$y = (M_y * L_s / 3) / E_{eff} = 0.04134775$ ((4.29), Biskinis Phd))
 $M_y = 5.1584E+008$
 $L_s = M/V$ (with $L_s > 0.1 * L$ and $L_s < 2 * L$) = 6000.00
From table 10.5, ASCE 41_17: $E_{eff} = factor * E_c * I_g = 2.4951E+013$
factor = 0.70
 $A_g = 120000.00$

Mean concrete strength: $f_c' = (f_{c'}^{\text{jacket}} \cdot \text{Area}_{\text{jacket}} + f_{c'}^{\text{core}} \cdot \text{Area}_{\text{core}}) / \text{Area}_{\text{section}} = 25.00$

$N = 2.1578\text{E}+006$

$E_c \cdot I_g = E_{c_{\text{jacket}}} \cdot I_{g_{\text{jacket}}} + E_{c_{\text{core}}} \cdot I_{g_{\text{core}}} = 3.5645\text{E}+013$

Calculation of Yielding Moment M_y

Calculation of y 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, $b = 400.00$

web width, $b_w = 200.00$

flange thickness, $t = 200.00$

$y = \text{Min}(y_{\text{ten}}, y_{\text{com}})$

$y_{\text{ten}} = 1.6571499\text{E}-005$

with $f_y = 596.2441$

$d = 367.00$

$y = 0.50980796$

$A = 0.19238222$

$B = 0.11480474$

with $p_t = 0.01504455$

$p_c = 0.00774698$

$p_v = 0.01367492$

$N = 2.1578\text{E}+006$

$b = 400.00$

$\alpha = 0.08991826$

$y_{\text{comp}} = 9.6849368\text{E}-006$

with $f_c = 30.00$

$E_c = 25742.96$

$y = 0.5901636$

$A = 0.0730043$

$B = 0.06549972$

with $E_s = 200000.00$

CONFIRMATION: $y = 0.5901636 > t/d$

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 $I_b/I_d \geq 1$

shear control ratio $V_y E / V_{col} O E = 1.37616$

$d = d_{\text{external}} = 367.00$

$s = s_{\text{external}} = 100.00$

- $t = s_1 + s_2 + 2 \cdot t_f / b_w \cdot (f_{fe} / f_s) = 0.00809358$

jacket: $s_1 = A_{v1} \cdot L_{\text{stir1}} / (s_1 \cdot A_g) = 0.00680678$

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

$L_{\text{stir1}} = 1040.00$, is the total Length of all stirrups parallel to loading (shear) direction

$s_1 = 100.00$

core: $s_2 = A_{v2} \cdot L_{\text{stir2}} / (s_2 \cdot A_g) = 0.0012868$

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

$L_{\text{stir2}} = 768.00$, 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.

$N_{UD} = 2.1578\text{E}+006$

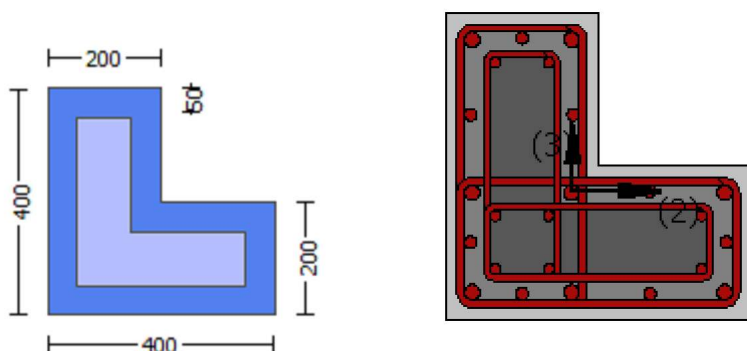
$A_g = 120000.00$

$f_{cE} = (f_{c_jacket} \cdot Area_jacket + f_{c_core} \cdot Area_core) / section_area = 25.00$
 $f_{yIE} = (f_{y_ext_Long_Reinf} \cdot Area_ext_Long_Reinf + f_{y_int_Long_Reinf} \cdot Area_int_Long_Reinf) / Area_Tot_Long_Rein = 596.2441$
 $f_{yIE} = (f_{y_ext_Trans_Reinf} \cdot s1 + f_{y_int_Trans_Reinf} \cdot s2) / (s1 + s2) = 605.1263$
 $\rho_l = Area_Tot_Long_Rein / (b \cdot d) = 0.03646644$
 $b = 400.00$
 $d = 367.00$
 $f_{cE} = 25.00$

 End Of Calculation of Chord Rotation Capacity for element: column JLC1 of floor 1
 At local axis: 2
 Integration Section: (b)

Calculation No. 7

column C1, Floor 1
 Limit State: Operational Level (data interpolation between analysis steps 1 and 2)
 Analysis: Uniform +X
 Check: Shear capacity VRd
 Edge: End
 Local Axis: (3)



Start Of Calculation of Shear Capacity for element: column JLC1 of floor 1
 At local axis: 3
 Integration Section: (b)
 Section Type: rcjls

Constant Properties

 Knowledge Factor, $\gamma = 0.90$
 Member Shear Force is generally considered as Force-Controlled Action according to Table C7-1, ASCE 41-17.
 Lower-bound strengths are used for Force-Controlled Actions according to 7.5.1.3, ASCE 41-17
 Consequently:
 Jacket

New material of Secondary Member: Concrete Strength, $f_c = f_{c_lower_bound} = 20.00$
 New material of Secondary Member: Steel Strength, $f_s = f_{s_lower_bound} = 500.00$
 Concrete Elasticity, $E_c = 25742.96$
 Steel Elasticity, $E_s = 200000.00$
 Existing Column
 Existing material of Secondary Member: Concrete Strength, $f_c = f_{c_lower_bound} = 12.00$
 Existing material of Secondary Member: Steel Strength, $f_s = f_{s_lower_bound} = 400.00$
 Concrete Elasticity, $E_c = 19940.411$
 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, $f_c = f_{cm} = 30.00$
 New material: Steel Strength, $f_s = f_{sm} = 625.00$
 Existing Column
 Existing material: Concrete Strength, $f_c = f_{cm} = 18.00$
 Existing material: Steel Strength, $f_s = f_{sm} = 500.00$
 #####
 Max Height, $H_{max} = 400.00$
 Min Height, $H_{min} = 200.00$
 Max Width, $W_{max} = 400.00$
 Min Width, $W_{min} = 200.00$
 Jacket Thickness, $t_j = 50.00$
 Cover Thickness, $c = 15.00$
 Element Length, $L = 3000.00$
 Secondary Member
 Smooth Bars
 Ductile Steel
 Without Detailing for Earthquake Resistance (including stirrups not 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, $M_a = -3.4089E+006$
 Shear Force, $V_a = -1.4589530E-005$
 EDGE -B-
 Bending Moment, $M_b = 15336.81$
 Shear Force, $V_b = 1.4589530E-005$
 BOTH EDGES
 Axial Force, $F = -2.1578E+006$
 Longitudinal Reinforcement Area Distribution (in 2 divisions)
 -Tension: $As_t = 0.00$
 -Compression: $As_c = 5353.274$
 Longitudinal Reinforcement Area Distribution (in 3 divisions)
 -Tension: $As_{t,ten} = 2208.54$
 -Compression: $As_{c,com} = 1137.257$
 -Middle: $As_{mid} = 2007.478$
 Mean Diameter of Tension Reinforcement, $Db_{L,ten} = 16.60$

Existing component: From table 7-7, ASCE 41_17: Final Shear Capacity $V_R = \phi V_n = 267030.06$
 V_n ((10.3), ASCE 41-17) = $k_n l \cdot V_{CoI} = 296700.066$
 $V_{CoI} = 296700.066$
 $k_n l = 1.00$
 $displacement_ductility_demand = 0.00885451$

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: $fc' = (fc'_{jacket} \cdot Area_{jacket} + fc'_{core} \cdot Area_{core}) / Area_{section} = 16.66667$, but $fc'^{0.5} \leq 8.3$ MPa (22.5.3.1, ACI 318-14)
 $M/Vd = 4.00$
 $\mu_u = 15336.81$
 $V_u = 1.4589530E-005$
 $d = 0.8 \cdot h = 320.00$
 $N_u = 2.1578E+006$
 $A_g = 80000.00$
From (11.5.4.8), ACI 318-14: $V_s = V_{sjacket} + V_{s,core} = 376991.118$
where:
 $V_{sjacket} = V_{sj1} + V_{sj2} = 376991.118$
 $V_{sj1} = 251327.412$ is calculated for section web jacket, with:
 $d = 320.00$
 $A_v = 157079.633$
 $f_y = 500.00$
 $s = 100.00$
 V_{sj1} is multiplied by $Col,j1 = 1.00$
 $s/d = 0.3125$
 $V_{sj2} = 125663.706$ is calculated for section flange jacket, with:
 $d = 160.00$
 $A_v = 157079.633$
 $f_y = 500.00$
 $s = 100.00$
 V_{sj2} is multiplied by $Col,j2 = 1.00$
 $s/d = 0.625$
 $V_{s,core} = V_{s,c1} + V_{s,c2} = 0.00$
 $V_{s,c1} = 0.00$ is calculated for section web core, with:
 $d = 240.00$
 $A_v = 100530.965$
 $f_y = 400.00$
 $s = 250.00$
 $V_{s,c1}$ is multiplied by $Col,c1 = 0.00$
 $s/d = 1.04167$
 $V_{s,c2} = 0.00$ is calculated for section flange core, with:
 $d = 80.00$
 $A_v = 100530.965$
 $f_y = 400.00$
 $s = 250.00$
 $V_{s,c2}$ is multiplied by $Col,c2 = 0.00$
 $s/d = 3.125$
 $V_f ((11-3)-(11.4), ACI 440) = 0.00$
From (11-11), ACI 440: $V_s + V_f \leq 173568.578$
 $bw = 200.00$

displacement_ductility_demand is calculated as ϕ / y

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

From analysis, chord rotation $\theta = 0.00036611$
 $y = (M_y \cdot L_s / 3) / E_{eff} = 0.04134775$ ((4.29), Biskinis Phd))
 $M_y = 5.1584E+008$
 $L_s = M/V$ (with $L_s > 0.1 \cdot L$ and $L_s < 2 \cdot L$) = 6000.00
From table 10.5, ASCE 41_17: $E_{eff} = factor \cdot E_c \cdot I_g = 2.4951E+013$
 $factor = 0.70$
 $A_g = 120000.00$
Mean concrete strength: $fc' = (fc'_{jacket} \cdot Area_{jacket} + fc'_{core} \cdot Area_{core}) / Area_{section} = 25.00$
 $N = 2.1578E+006$
 $E_c \cdot I_g = E_{c,jacket} \cdot I_{g,jacket} + E_{c,core} \cdot I_{g,core} = 3.5645E+013$

Calculation of Yielding Moment M_y

Calculation of y 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, $b = 400.00$
web width, $b_w = 200.00$
flange thickness, $t = 200.00$

$y = \text{Min}(y_{\text{ten}}, y_{\text{com}})$
 $y_{\text{ten}} = 1.6571499\text{E-}005$
with $f_y = 596.2441$
 $d = 367.00$
 $y = 0.50980796$
 $A = 0.19238222$
 $B = 0.11480474$
with $p_t = 0.01504455$
 $p_c = 0.00774698$
 $p_v = 0.01367492$
 $N = 2.1578\text{E+}006$
 $b = 400.00$
 $" = 0.08991826$
 $y_{\text{comp}} = 9.6849368\text{E-}006$
with $f_c = 30.00$
 $E_c = 25742.96$
 $y = 0.5901636$
 $A = 0.0730043$
 $B = 0.06549972$
with $E_s = 200000.00$
CONFIRMATION: $y = 0.5901636 > t/d$

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 JLC1 of floor 1
At local axis: 3
Integration Section: (b)

Calculation No. 8

column C1, Floor 1

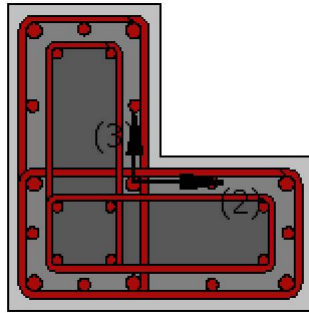
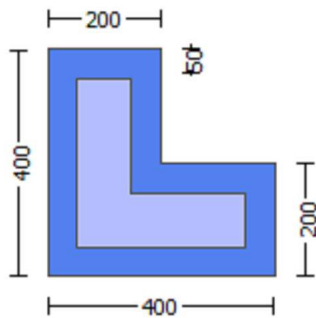
Limit State: Operational Level (data interpolation between analysis steps 1 and 2)

Analysis: Uniform +X

Check: Chord rotation capacity (ϕ)

Edge: End

Local Axis: (3)



Start Of Calculation of Shear Capacity ratio for element: column JLC1 of floor 1

At Shear local axis: 3

(Bending local axis: 2)

Section Type: rcjlcs

Constant Properties

Knowledge Factor, $\gamma = 0.90$

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

Consequently:

Jacket

New material of Secondary Member: Concrete Strength, $f_c = f_{cm} = 30.00$

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

Concrete Elasticity, $E_c = 25742.96$

Steel Elasticity, $E_s = 200000.00$

Existing Column

Existing material of Secondary Member: Concrete Strength, $f_c = f_{cm} = 18.00$

Existing material of Secondary Member: Steel Strength, $f_s = f_{sm} = 500.00$

Concrete Elasticity, $E_c = 19940.411$

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, $f_s = 1.25 \cdot f_{sm} = 625.00$

#####

Max Height, $H_{max} = 400.00$

Min Height, $H_{min} = 200.00$

Max Width, $W_{max} = 400.00$

Min Width, $W_{min} = 200.00$

Jacket Thickness, $t_j = 50.00$

Cover Thickness, $c = 15.00$

Mean Confinement Factor overall section = 1.00

Element Length, $L = 3000.00$

Secondary Member

Smooth Bars

Ductile Steel

Without Detailing for Earthquake Resistance (including stirrups not 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, $V_a = -1.4594946E-005$

EDGE -B-

Shear Force, $V_b = 1.4594946E-005$

BOTH EDGES

Axial Force, $F = -2.1578\text{E}+006$

Longitudinal Reinforcement Area Distribution (in 2 divisions)

-Tension: $As_t = 0.00$

-Compression: $As_c = 5353.274$

Longitudinal Reinforcement Area Distribution (in 3 divisions)

-Tension: $As_{t,ten} = 2208.54$

-Compression: $As_{c,com} = 1137.257$

-Middle: $As_{c,mid} = 2007.478$

Calculation of Shear Capacity ratio, $V_e/V_r = 1.37616$

Member Controlled by Shear ($V_e/V_r > 1$)

Calculation of Shear Demand from fig. R18.6.5, ACI 318-14 $V_e = (M_{pr1} + M_{pr2})/l_n = 481543.08$
with

$M_{pr1} = \text{Max}(Mu_{1+}, Mu_{1-}) = 7.2231\text{E}+008$

$Mu_{1+} = 7.2231\text{E}+008$, is the ultimate moment strength at the edge 1 of the member in the actual moment direction which is defined for the static loading combination

$Mu_{1-} = 6.3496\text{E}+008$, 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

$M_{pr2} = \text{Max}(Mu_{2+}, Mu_{2-}) = 7.2231\text{E}+008$

$Mu_{2+} = 7.2231\text{E}+008$, 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

$Mu_{2-} = 6.3496\text{E}+008$, 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 Mu_{1+}

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

$\phi_u = 3.9059244\text{E}-005$

$M_u = 7.2231\text{E}+008$

with full section properties:

$b = 400.00$

$d = 367.00$

$d' = 33.00$

$v = 0.48996389$

$N = 2.1578\text{E}+006$

$f_c = 30.00$

ϕ_c (5A.5, TBDY) = 0.002

Final value of ϕ_u : $\phi_u^* = \text{shear_factor} * \text{Max}(\phi_u, \phi_c) = 0.00925707$

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

From (5.4b), TBDY: $\phi_u = 0.00925707$

we (5.4c) = 0.0207149

a_{se} ((5.4d), TBDY) = $(a_{se1} * A_{ext} + a_{se2} * A_{int}) / A_{sec} = 0.33836562$

$a_{se1} = \text{Max}(((A_{conf,max1} - A_{noConf1}) / A_{conf,max1}) * (A_{conf,min1} / A_{conf,max1}), 0) = 0.33836562$

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.

$A_{conf,max1} = 89600.00$ 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.

$A_{conf,min1} = 59225.00$ 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.

$A_{noConf1} = 43733.333$ is the unconfined external core area which is equal to $b_i^2/6$ as defined at (A.2).

$a_{se2} (>= a_{se1}) = \text{Max}(((A_{conf,max2} - A_{noConf2}) / A_{conf,max2}) * (A_{conf,min2} / A_{conf,max2}), 0) = 0.33836562$

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.

$A_{conf,max2} = 45264.00$ 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.

$A_{conf,min2} = 0.00$ 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.

$A_{noConf2} = 32309.333$ is the unconfined internal core area which is equal to $b_i^2/6$ as defined at (A.2).

$p_{sh,min} \cdot F_{ywe} = \text{Min}(p_{sh,x} \cdot F_{ywe}, p_{sh,y} \cdot F_{ywe}) = 6.12205$

Expression (5.4d) for $p_{sh,min} \cdot F_{ywe}$ has been multiplied by 0.3 according to 15.7.1.3 for members without earthquake detailing (90° closed stirrups)

$p_{sh,x} \cdot F_{ywe} = p_{sh1} \cdot F_{ywe1} + p_{s2} \cdot F_{ywe2} = 6.12205$

$p_{sh1} \text{ ((5.4d), TBDY)} = L_{stir1} \cdot A_{stir1} / (A_{sec} \cdot s_1) = 0.00680678$

L_{stir1} (Length of stirrups along Y) = 1040.00

A_{stir1} (stirrups area) = 78.53982

$p_{sh2} \text{ (5.4d)} = L_{stir2} \cdot A_{stir2} / (A_{sec} \cdot s_2) = 0.0012868$

L_{stir2} (Length of stirrups along Y) = 768.00

A_{stir2} (stirrups area) = 50.26548

$p_{sh,y} \cdot F_{ywe} = p_{sh1} \cdot F_{ywe1} + p_{s2} \cdot F_{ywe2} = 6.12205$

$p_{sh1} \text{ ((5.4d), TBDY)} = L_{stir1} \cdot A_{stir1} / (A_{sec} \cdot s_1) = 0.00680678$

L_{stir1} (Length of stirrups along X) = 1040.00

A_{stir1} (stirrups area) = 78.53982

$p_{sh2} \text{ ((5.4d), TBDY)} = L_{stir2} \cdot A_{stir2} / (A_{sec} \cdot s_2) = 0.0012868$

L_{stir2} (Length of stirrups along X) = 768.00

A_{stir2} (stirrups area) = 50.26548

$A_{sec} = 120000.00$

$s_1 = 100.00$

$s_2 = 250.00$

$f_{ywe1} = 781.25$

$f_{ywe2} = 625.00$

$f_{ce} = 30.00$

From ((5.A5), TBDY), TBDY: $c_c = 0.002$

c = confinement factor = 1.00

$y_1 = 0.0025$

$sh_1 = 0.008$

$ft_1 = 898.293$

$fy_1 = 748.5775$

$su_1 = 0.032$

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

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

$su_1 = 0.4 \cdot esu1_{nominal} \text{ ((5.5), TBDY)} = 0.032$

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

For calculation of $esu1_{nominal}$ 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 $fs_1 = (f_{s,jacket} \cdot A_{sl,ten,jacket} + f_{s,core} \cdot A_{sl,ten,core}) / A_{sl,ten} = 748.5775$

with $Es_1 = (E_{s,jacket} \cdot A_{sl,ten,jacket} + E_{s,core} \cdot A_{sl,ten,core}) / A_{sl,ten} = 200000.00$

$y_2 = 0.0025$

$sh_2 = 0.008$

$ft_2 = 886.7403$

$fy_2 = 738.9503$

$su_2 = 0.032$

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

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

$su_2 = 0.4 \cdot esu2_{nominal} \text{ ((5.5), TBDY)} = 0.032$

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

For calculation of $esu2_{nominal}$ 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 $\text{Min}(1, 1.25 \cdot (l_b/l_d)^{2/3})$, from 10.3.5, ASCE 41-17.

with $fs_2 = (f_{s,jacket} \cdot A_{sl,com,jacket} + f_{s,core} \cdot A_{sl,com,core}) / A_{sl,com} = 738.9503$

with $Es_2 = (E_{s,jacket} \cdot A_{sl,com,jacket} + E_{s,core} \cdot A_{sl,com,core}) / A_{sl,com} = 200000.00$

```

yv = 0.0025
shv = 0.008
ftv = 894.3662
fyv = 745.3052
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 = 745.3052
    with Esv = (Es,jacket*Asl,mid,jacket + Es,mid*Asl,mid,core)/Asl,mid = 200000.00
    1 = Asl,ten/(b*d)*(fs1/fc) = 0.37540035
    2 = Asl,com/(b*d)*(fs2/fc) = 0.19082108
    v = Asl,mid/(b*d)*(fsv/fc) = 0.33973286
and confined core properties:
b = 360.00
d = 347.00
d' = 13.00
fcc (5A.2, TBDY) = 30.00
cc (5A.5, TBDY) = 0.002
    c = confinement factor = 1.00
    1 = Asl,ten/(b*d)*(fs1/fc) = 0.44115251
    2 = Asl,com/(b*d)*(fs2/fc) = 0.22424379
    v = Asl,mid/(b*d)*(fsv/fc) = 0.39923778
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) = 0.64577857
MRc (4.17) = 7.2231E+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

--->

$*cu(4.11) = 0.67946111$

$M_{Ro}(4.18) = 4.2866E+008$

$M_{Ro} < 0.8*M_{Rc}$

--->

$u = cu$ (unconfined full section) = $3.9059244E-005$

$\mu_u = M_{Rc}$

Calculation of ratio l_b/d

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

Calculation of μ_{u1} -

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

$\mu_u = 4.2871818E-005$

$\mu_u = 6.3496E+008$

with full section properties:

$b = 200.00$

$d = 367.00$

$d' = 33.00$

$v = 0.97992777$

$N = 2.1578E+006$

$f_c = 30.00$

α (5A.5, TBDY) = 0.002

Final value of μ_u : $\mu_u^* = \text{shear_factor} * \text{Max}(\mu_u, \mu_c) = 0.00925707$

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

From (5.4b), TBDY: $\mu_u = 0.00925707$

μ_{ue} (5.4c) = 0.0207149

α_{se} ((5.4d), TBDY) = $(\alpha_{se1}*A_{ext} + \alpha_{se2}*A_{int})/A_{sec} = 0.33836562$

$\alpha_{se1} = \text{Max}(((A_{conf,max1} - A_{noConf1})/A_{conf,max1}) * (A_{conf,min1}/A_{conf,max1}), 0) = 0.33836562$

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.

$A_{conf,max1} = 89600.00$ 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.

$A_{conf,min1} = 59225.00$ 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.

$A_{noConf1} = 43733.333$ is the unconfined external core area which is equal to $b^2/6$ as defined at (A.2).

$\alpha_{se2} (\geq \alpha_{se1}) = \text{Max}(((A_{conf,max2} - A_{noConf2})/A_{conf,max2}) * (A_{conf,min2}/A_{conf,max2}), 0) = 0.33836562$

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.

$A_{conf,max2} = 45264.00$ 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.

$A_{conf,min2} = 0.00$ 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.

$A_{noConf2} = 32309.333$ is the unconfined internal core area which is equal to $b^2/6$ as defined at (A.2).

$p_{sh,min}*F_{ywe} = \text{Min}(p_{sh,x}*F_{ywe}, p_{sh,y}*F_{ywe}) = 6.12205$

Expression (5.4d) for $p_{sh,min}*F_{ywe}$ has been multiplied by 0.3 according to 15.7.1.3 for members without earthquake detailing (90° closed stirrups)

$p_{sh,x}*F_{ywe} = p_{sh1}*F_{ywe1} + p_{sh2}*F_{ywe2} = 6.12205$

p_{sh1} ((5.4d), TBDY) = $L_{stir1}*A_{stir1}/(A_{sec}*s_1) = 0.00680678$

Lstir1 (Length of stirrups along Y) = 1040.00
 Astir1 (stirrups area) = 78.53982
 $psh2 (5.4d) = Lstir2 * Astir2 / (Asec * s2) = 0.0012868$
 Lstir2 (Length of stirrups along Y) = 768.00
 Astir2 (stirrups area) = 50.26548

$psh_y * Fywe = psh1 * Fywe1 + ps2 * Fywe2 = 6.12205$
 $psh1 ((5.4d), TBDY) = Lstir1 * Astir1 / (Asec * s1) = 0.00680678$
 Lstir1 (Length of stirrups along X) = 1040.00
 Astir1 (stirrups area) = 78.53982
 $psh2 ((5.4d), TBDY) = Lstir2 * Astir2 / (Asec * s2) = 0.0012868$
 Lstir2 (Length of stirrups along X) = 768.00
 Astir2 (stirrups area) = 50.26548

Asec = 120000.00

s1 = 100.00

s2 = 250.00

fywe1 = 781.25

fywe2 = 625.00

fce = 30.00

From ((5.A5), TBDY), TBDY: cc = 0.002

c = confinement factor = 1.00

y1 = 0.0025

sh1 = 0.008

ft1 = 886.7403

fy1 = 738.9503

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 = $(fs_jacket * Asl,ten,jacket + fs_core * Asl,ten,core) / Asl,ten = 738.9503$

with Es1 = $(Es_jacket * Asl,ten,jacket + Es_core * Asl,ten,core) / Asl,ten = 200000.00$

y2 = 0.0025

sh2 = 0.008

ft2 = 898.293

fy2 = 748.5775

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 = $(fs_jacket * Asl,com,jacket + fs_core * Asl,com,core) / Asl,com = 748.5775$

with Es2 = $(Es_jacket * Asl,com,jacket + Es_core * Asl,com,core) / Asl,com = 200000.00$

yv = 0.0025

shv = 0.008

ftv = 894.3662

fyv = 745.3052

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  $f_{syv} = f_{sv}/1.2$ , 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  $f_{sv} = (f_{sjacket} \cdot A_{sl,mid,jacket} + f_{s,mid} \cdot A_{sl,mid,core}) / A_{sl,mid} = 745.3052$ 
with  $E_{sv} = (E_{sjacket} \cdot A_{sl,mid,jacket} + E_{s,mid} \cdot A_{sl,mid,core}) / A_{sl,mid} = 200000.00$ 
 $1 = A_{sl,ten} / (b \cdot d) \cdot (f_{s1}/f_c) = 0.38164216$ 
 $2 = A_{sl,com} / (b \cdot d) \cdot (f_{s2}/f_c) = 0.7508007$ 
 $v = A_{sl,mid} / (b \cdot d) \cdot (f_{sv}/f_c) = 0.67946571$ 
and confined core properties:
 $b = 160.00$ 
 $d = 347.00$ 
 $d' = 13.00$ 
 $f_{cc} \text{ (5A.2, TBDY)} = 30.00$ 
 $cc \text{ (5A.5, TBDY)} = 0.002$ 
 $c = \text{confinement factor} = 1.00$ 
 $1 = A_{sl,ten} / (b \cdot d) \cdot (f_{s1}/f_c) = 0.50454853$ 
 $2 = A_{sl,com} / (b \cdot d) \cdot (f_{s2}/f_c) = 0.99259314$ 
 $v = A_{sl,mid} / (b \cdot d) \cdot (f_{sv}/f_c) = 0.898285$ 
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
--->
 $c_u \text{ (4.10)} = 0.58834972$ 
 $M_{Rc} \text{ (4.17)} = 6.3496E+008$ 
--->
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:  $b_o, d_o, d'_o$ 
-  $N, 1, 2, v$  normalised to  $b_o \cdot d_o$ , instead of  $b \cdot d$ 
-  $f_{cc}, cc$  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
--->
 $*c_u \text{ (4.11)} = 0.63603191$ 
 $M_{Ro} \text{ (4.18)} = 4.4768E+008$ 
 $M_{Ro} < 0.8 \cdot M_{Rc}$ 
--->
 $u = c_u \text{ (unconfined full section)} = 4.2871818E-005$ 
 $\mu = M_{Rc}$ 

```

Calculation of ratio l_b/l_d

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

Calculation of Mu2+

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

$$\phi_u = 3.9059244E-005$$

$$\mu = 7.2231E+008$$

with full section properties:

$$b = 400.00$$

$$d = 367.00$$

$$d' = 33.00$$

$$\nu = 0.48996389$$

$$N = 2.1578E+006$$

$$f_c = 30.00$$

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

$$\text{Final value of } \phi_u: \phi_u^* = \text{shear_factor} * \text{Max}(\phi_u, \phi_c) = 0.00925707$$

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

$$\text{From (5.4b), TBDY: } \phi_u = 0.00925707$$

$$\phi_{we} \text{ (5.4c)} = 0.0207149$$

$$\phi_{ase} \text{ ((5.4d), TBDY)} = (\phi_{ase1} * A_{ext} + \phi_{ase2} * A_{int}) / A_{sec} = 0.33836562$$

$$\phi_{ase1} = \text{Max}(((A_{conf,max1} - A_{noConf1}) / A_{conf,max1}) * (A_{conf,min1} / A_{conf,max1}), 0) = 0.33836562$$

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.

$A_{conf,max1} = 89600.00$ 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.

$A_{conf,min1} = 59225.00$ 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.

$A_{noConf1} = 43733.333$ is the unconfined external core area which is equal to $b_i^2/6$ as defined at (A.2).

$$\phi_{ase2} (\geq \phi_{ase1}) = \text{Max}(((A_{conf,max2} - A_{noConf2}) / A_{conf,max2}) * (A_{conf,min2} / A_{conf,max2}), 0) = 0.33836562$$

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.

$A_{conf,max2} = 45264.00$ 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.

$A_{conf,min2} = 0.00$ 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.

$A_{noConf2} = 32309.333$ is the unconfined internal core area which is equal to $b_i^2/6$ as defined at (A.2).

$$\phi_{psh,min} * F_{ywe} = \text{Min}(\phi_{psh,x} * F_{ywe}, \phi_{psh,y} * F_{ywe}) = 6.12205$$

Expression (5.4d) for $\phi_{psh,min} * F_{ywe}$ has been multiplied by 0.3 according to 15.7.1.3 for members without earthquake detailing (90° closed stirrups)

$$\phi_{psh,x} * F_{ywe} = \phi_{psh1} * F_{ywe1} + \phi_{psh2} * F_{ywe2} = 6.12205$$

$$\phi_{psh1} \text{ ((5.4d), TBDY)} = L_{stir1} * A_{stir1} / (A_{sec} * s_1) = 0.00680678$$

$$L_{stir1} \text{ (Length of stirrups along Y)} = 1040.00$$

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

$$\phi_{psh2} \text{ (5.4d)} = L_{stir2} * A_{stir2} / (A_{sec} * s_2) = 0.0012868$$

$$L_{stir2} \text{ (Length of stirrups along Y)} = 768.00$$

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

$$\phi_{psh,y} * F_{ywe} = \phi_{psh1} * F_{ywe1} + \phi_{psh2} * F_{ywe2} = 6.12205$$

$$\phi_{psh1} \text{ ((5.4d), TBDY)} = L_{stir1} * A_{stir1} / (A_{sec} * s_1) = 0.00680678$$

$$L_{stir1} \text{ (Length of stirrups along X)} = 1040.00$$

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

$$\phi_{psh2} \text{ ((5.4d), TBDY)} = L_{stir2} * A_{stir2} / (A_{sec} * s_2) = 0.0012868$$

$$L_{stir2} \text{ (Length of stirrups along X)} = 768.00$$

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

```

Asec = 120000.00
s1 = 100.00
s2 = 250.00
fywe1 = 781.25
fywe2 = 625.00
fce = 30.00
From ((5.A.5), TBDY), TBDY: cc = 0.002
c = confinement factor = 1.00
y1 = 0.0025
sh1 = 0.008
ft1 = 898.293
fy1 = 748.5775
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 Min(1,1.25*(lb/lb,min)^ 2/3), from 10.3.5, ASCE 41-17.
with fs1 = (fs,jacket*Asl,ten,jacket + fs,core*Asl,ten,core)/Asl,ten = 748.5775
with Es1 = (Es,jacket*Asl,ten,jacket + Es,core*Asl,ten,core)/Asl,ten = 200000.00
y2 = 0.0025
sh2 = 0.008
ft2 = 886.7403
fy2 = 738.9503
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,min)^ 2/3), from 10.3.5, ASCE 41-17.
with fs2 = (fs,jacket*Asl,com,jacket + fs,core*Asl,com,core)/Asl,com = 738.9503
with Es2 = (Es,jacket*Asl,com,jacket + Es,core*Asl,com,core)/Asl,com = 200000.00
yv = 0.0025
shv = 0.008
ftv = 894.3662
fyv = 745.3052
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 Min(1,1.25*(lb/lb,min)^ 2/3), from 10.3.5, ASCE 41-17.
with fsv = (fs,jacket*Asl,mid,jacket + fs,mid*Asl,mid,core)/Asl,mid = 745.3052
with Esv = (Es,jacket*Asl,mid,jacket + Es,mid*Asl,mid,core)/Asl,mid = 200000.00
1 = Asl,ten/(b*d)*(fs1/fc) = 0.37540035
2 = Asl,com/(b*d)*(fs2/fc) = 0.19082108
v = Asl,mid/(b*d)*(fsv/fc) = 0.33973286
and confined core properties:
b = 360.00
d = 347.00
d' = 13.00
fcc (5A.2, TBDY) = 30.00
cc (5A.5, TBDY) = 0.002

```

```

c = confinement factor = 1.00
1 = Asl,ten/(b*d)*(fs1/fc) = 0.44115251
2 = Asl,com/(b*d)*(fs2/fc) = 0.22424379
v = Asl,mid/(b*d)*(fsv/fc) = 0.39923778
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) = 0.64577857
MRc (4.17) = 7.2231E+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) = 0.67946111
MRo (4.18) = 4.2866E+008
MRo < 0.8*MRc
---->
u = cu (unconfined full section) = 3.9059244E-005
Mu = MRc
-----

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 = 4.2871818E-005
Mu = 6.3496E+008
-----

with full section properties:
b = 200.00

```

$d = 367.00$
 $d' = 33.00$
 $v = 0.97992777$
 $N = 2.1578E+006$
 $f_c = 30.00$
 $co (5A.5, TBDY) = 0.002$
 Final value of cu : $cu^* = shear_factor * Max(cu, cc) = 0.00925707$
 The Shear_factor is considered equal to 1 (pure moment strength)
 From (5.4b), TBDY: $cu = 0.00925707$
 $we (5.4c) = 0.0207149$
 $ase ((5.4d), TBDY) = (ase1 * A_{ext} + ase2 * A_{int}) / A_{sec} = 0.33836562$
 $ase1 = Max(((A_{conf,max1} - A_{noConf1}) / A_{conf,max1}) * (A_{conf,min1} / A_{conf,max1}), 0) = 0.33836562$
 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.
 $A_{conf,max1} = 89600.00$ 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.
 $A_{conf,min1} = 59225.00$ 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.
 $A_{noConf1} = 43733.333$ is the unconfined external core area which is equal to $b_i^2/6$ as defined at (A.2).
 $ase2 (>= ase1) = Max(((A_{conf,max2} - A_{noConf2}) / A_{conf,max2}) * (A_{conf,min2} / A_{conf,max2}), 0) = 0.33836562$
 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.
 $A_{conf,max2} = 45264.00$ 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.
 $A_{conf,min2} = 0.00$ 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.
 $A_{noConf2} = 32309.333$ is the unconfined internal core area which is equal to $b_i^2/6$ as defined at (A.2).
 $psh_{min} * F_{ywe} = Min(psh_x * F_{ywe}, psh_y * F_{ywe}) = 6.12205$
 Expression (5.4d) for $psh_{min} * F_{ywe}$ has been multiplied by 0.3 according to 15.7.1.3 for members without earthquake detailing (90° closed stirrups)

$psh_x * F_{ywe} = psh1 * F_{ywe1} + ps2 * F_{ywe2} = 6.12205$
 $psh1 ((5.4d), TBDY) = L_{stir1} * A_{stir1} / (A_{sec} * s1) = 0.00680678$
 L_{stir1} (Length of stirrups along Y) = 1040.00
 A_{stir1} (stirrups area) = 78.53982
 $psh2 ((5.4d)) = L_{stir2} * A_{stir2} / (A_{sec} * s2) = 0.0012868$
 L_{stir2} (Length of stirrups along Y) = 768.00
 A_{stir2} (stirrups area) = 50.26548

$psh_y * F_{ywe} = psh1 * F_{ywe1} + ps2 * F_{ywe2} = 6.12205$
 $psh1 ((5.4d), TBDY) = L_{stir1} * A_{stir1} / (A_{sec} * s1) = 0.00680678$
 L_{stir1} (Length of stirrups along X) = 1040.00
 A_{stir1} (stirrups area) = 78.53982
 $psh2 ((5.4d), TBDY) = L_{stir2} * A_{stir2} / (A_{sec} * s2) = 0.0012868$
 L_{stir2} (Length of stirrups along X) = 768.00
 A_{stir2} (stirrups area) = 50.26548

$A_{sec} = 120000.00$
 $s1 = 100.00$
 $s2 = 250.00$
 $f_{ywe1} = 781.25$
 $f_{ywe2} = 625.00$
 $f_{ce} = 30.00$
 From ((5.A5), TBDY), TBDY: $cc = 0.002$
 $c =$ confinement factor = 1.00
 $y1 = 0.0025$
 $sh1 = 0.008$
 $ft1 = 886.7403$
 $fy1 = 738.9503$

```

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 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 = 738.9503
with Es1 = (Es,jacket*Asl,ten,jacket + Es,core*Asl,ten,core)/Asl,ten = 200000.00
y2 = 0.0025
sh2 = 0.008
ft2 = 898.293
fy2 = 748.5775
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 = 748.5775
with Es2 = (Es,jacket*Asl,com,jacket + Es,core*Asl,com,core)/Asl,com = 200000.00
yv = 0.0025
shv = 0.008
ftv = 894.3662
fyv = 745.3052
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 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 = 745.3052
with Esv = (Es,jacket*Asl,mid,jacket + Es,mid*Asl,mid,core)/Asl,mid = 200000.00
1 = Asl,ten/(b*d)*(fs1/fc) = 0.38164216
2 = Asl,com/(b*d)*(fs2/fc) = 0.7508007
v = Asl,mid/(b*d)*(fsv/fc) = 0.67946571
and confined core properties:
b = 160.00
d = 347.00
d' = 13.00
fcc (5A.2, TBDY) = 30.00
cc (5A.5, TBDY) = 0.002
c = confinement factor = 1.00
1 = Asl,ten/(b*d)*(fs1/fc) = 0.50454853
2 = Asl,com/(b*d)*(fs2/fc) = 0.99259314
v = Asl,mid/(b*d)*(fsv/fc) = 0.898285
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
--->

ϕ_{cu} (4.10) = 0.58834972
 M_{Rc} (4.17) = 6.3496E+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: 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}, 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) = 0.63603191
 M_{Ro} (4.18) = 4.4768E+008
 $M_{Ro} < 0.8*M_{Rc}$

--->
 $u = \phi_{cu}$ (unconfined full section) = 4.2871818E-005
 $M_u = M_{Rc}$

Calculation of ratio I_b/I_d

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

Calculation of Shear Strength $V_r = \min(V_{r1}, V_{r2}) = 349917.706$

Calculation of Shear Strength at edge 1, $V_{r1} = 349917.706$

$V_{r1} = V_{Co1}$ ((10.3), ASCE 41-17) = $k_{nl}*V_{Co10}$
 $V_{Co10} = 349917.706$
 $k_{nl} = 1$ (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: $f'_c = (f'_c_{jacket}*Area_{jacket} + f'_c_{core}*Area_{core})/Area_{section} = 25.00$, but $f_c^{0.5} \leq 8.3$
MPa (22.5.3.1, ACI 318-14)
 $M/Vd = 4.00$
 $M_u = 3.4090E+006$
 $V_u = 1.4594946E-005$
 $d = 0.8*h = 320.00$
 $N_u = 2.1578E+006$
 $A_g = 80000.00$
From (11.5.4.8), ACI 318-14: $V_s = V_{sjacket} + V_{s,core} = 471238.898$
where:
 $V_{sjacket} = V_{sj1} + V_{sj2} = 471238.898$

Vs,j1 = 314159.265 is calculated for section web jacket, with:

$$d = 320.00$$

$$A_v = 157079.633$$

$$f_y = 625.00$$

$$s = 100.00$$

Vs,j1 is multiplied by Col,j1 = 1.00

$$s/d = 0.3125$$

Vs,j2 = 157079.633 is calculated for section flange jacket, with:

$$d = 160.00$$

$$A_v = 157079.633$$

$$f_y = 625.00$$

$$s = 100.00$$

Vs,j2 is multiplied by Col,j2 = 1.00

$$s/d = 0.625$$

$$V_{s,core} = V_{s,c1} + V_{s,c2} = 0.00$$

Vs,c1 = 0.00 is calculated for section web core, with:

$$d = 240.00$$

$$A_v = 100530.965$$

$$f_y = 500.00$$

$$s = 250.00$$

Vs,c1 is multiplied by Col,c1 = 0.00

$$s/d = 1.04167$$

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

$$d = 80.00$$

$$A_v = 100530.965$$

$$f_y = 500.00$$

$$s = 250.00$$

Vs,c2 is multiplied by Col,c2 = 0.00

$$s/d = 3.125$$

$$V_f ((11-3)-(11.4), \text{ACI } 440) = 0.00$$

$$\text{From } (11-11), \text{ACI } 440: V_s + V_f \leq 212577.225$$

$$bw = 200.00$$

Calculation of Shear Strength at edge 2, Vr2 = 349917.706

$$Vr2 = V_{Col} ((10.3), \text{ASCE } 41-17) = knl * V_{Col0}$$

$$V_{Col0} = 349917.706$$

$$knl = 1 \text{ (zero step-static loading)}$$

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).

$$= 1 \text{ (normal-weight concrete)}$$

$$\text{Mean concrete strength: } f'_c = (f'_c_{jacket} * \text{Area}_{jacket} + f'_c_{core} * \text{Area}_{core}) / \text{Area}_{section} = 25.00, \text{ but } f'_c^{0.5} \leq 8.3 \text{ MPa (22.5.3.1, ACI 318-14)}$$

$$M/Vd = 4.00$$

$$\mu_u = 15336.932$$

$$V_u = 1.4594946E+005$$

$$d = 0.8 * h = 320.00$$

$$N_u = 2.1578E+006$$

$$A_g = 80000.00$$

$$\text{From (11.5.4.8), ACI 318-14: } V_s = V_{sjacket} + V_{s,core} = 471238.898$$

where:

$$V_{sjacket} = V_{sj1} + V_{sj2} = 471238.898$$

Vs,j1 = 314159.265 is calculated for section web jacket, with:

$$d = 320.00$$

$$A_v = 157079.633$$

$$f_y = 625.00$$

$$s = 100.00$$

Vs,j1 is multiplied by Col,j1 = 1.00

$$s/d = 0.3125$$

Vs,j2 = 157079.633 is calculated for section flange jacket, with:

$$d = 160.00$$

$$A_v = 157079.633$$

$$f_y = 625.00$$

$$s = 100.00$$

Vs,j2 is multiplied by Col,j2 = 1.00

```

s/d = 0.625
Vs,core = Vs,c1 + Vs,c2 = 0.00
Vs,c1 = 0.00 is calculated for section web core, with:
d = 240.00
Av = 100530.965
fy = 500.00
s = 250.00
Vs,c1 is multiplied by Col,c1 = 0.00
s/d = 1.04167
Vs,c2 = 0.00 is calculated for section flange core, with:
d = 80.00
Av = 100530.965
fy = 500.00
s = 250.00
Vs,c2 is multiplied by Col,c2 = 0.00
s/d = 3.125
Vf ((11-3)-(11.4), ACI 440) = 0.00
From (11-11), ACI 440: Vs + Vf <= 212577.225
bw = 200.00
-----

End Of Calculation of Shear Capacity ratio for element: column JLC1 of floor 1
At local axis: 3
-----

Start Of Calculation of Shear Capacity ratio for element: column JLC1 of floor 1
At Shear local axis: 2
(Bending local axis: 3)
Section Type: rcjlc3

Constant Properties
-----

Knowledge Factor,   = 0.90
Mean strength values are used for both shear and moment calculations.
Consequently:
Jacket
New material of Secondary Member: Concrete Strength, fc = fcm = 30.00
New material of Secondary Member: Steel Strength, fs = fsm = 625.00
Concrete Elasticity, Ec = 25742.96
Steel Elasticity, Es = 200000.00
Existing Column
Existing material of Secondary Member: Concrete Strength, fc = fcm = 18.00
Existing material of Secondary Member: Steel Strength, fs = fsm = 500.00
Concrete Elasticity, Ec = 19940.411
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 = 625.00
#####
Max Height, Hmax = 400.00
Min Height, Hmin = 200.00
Max Width, Wmax = 400.00
Min Width, Wmin = 200.00
Jacket Thickness, tj = 50.00
Cover Thickness, c = 15.00
Mean Confinement Factor overall section = 1.00
Element Length, L = 3000.00
Secondary Member
Smooth Bars
Ductile Steel
Without Detailing for Earthquake Resistance (including stirrups not 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: 2
EDGE -A-
Shear Force, $V_a = -5.7948283E-005$
EDGE -B-
Shear Force, $V_b = 5.7948283E-005$
BOTH EDGES
Axial Force, $F = -2.1578E+006$
Longitudinal Reinforcement Area Distribution (in 2 divisions)
-Tension: $As_t = 0.00$
-Compression: $As_c = 5353.274$
Longitudinal Reinforcement Area Distribution (in 3 divisions)
-Tension: $As_{t,ten} = 1137.257$
-Compression: $As_{c,com} = 2208.54$
-Middle: $As_{c,mid} = 2007.478$

Calculation of Shear Capacity ratio, $V_e/V_r = 1.10794$
Member Controlled by Shear ($V_e/V_r > 1$)
Calculation of Shear Demand from fig. R18.6.5, ACI 318-14 $V_e = (M_{pr1} + M_{pr2})/l_n = 387689.388$
with
 $M_{pr1} = \text{Max}(\mu_{u1+}, \mu_{u1-}) = 5.8153E+008$
 $\mu_{u1+} = 5.8153E+008$, is the ultimate moment strength at the edge 1 of the member in the actual moment direction which is defined for the static loading combination
 $\mu_{u1-} = 4.9554E+008$, 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
 $M_{pr2} = \text{Max}(\mu_{u2+}, \mu_{u2-}) = 5.8153E+008$
 $\mu_{u2+} = 5.8153E+008$, 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
 $\mu_{u2-} = 4.9554E+008$, 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 μ_u according to 4.1, Biskinis/Fardis 2013:
 $\mu_u = 1.1511372E-005$
 $\mu_u = 5.8153E+008$

with full section properties:

$b = 400.00$
 $d = 367.00$
 $d' = 33.00$
 $v = 0.48996389$
 $N = 2.1578E+006$
 $f_c = 30.00$
 $\alpha = (5A_s, TBDY) = 0.002$
Final value of μ_u : $\mu_u^* = \text{shear_factor} * \text{Max}(\mu_u, \alpha) = 0.00925707$
The Shear_factor is considered equal to 1 (pure moment strength)
From (5.4b), TBDY: $\mu_u = 0.00925707$
we (5.4c) = 0.0207149
 $\alpha_{se} ((5.4d), TBDY) = (\alpha_{se1} * A_{ext} + \alpha_{se2} * A_{int}) / A_{sec} = 0.33836562$
 $\alpha_{se1} = \text{Max}(((A_{conf,max1} - A_{noConf1}) / A_{conf,max1}) * (A_{conf,min1} / A_{conf,max1}), 0) = 0.33836562$
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.

Aconf,max1 = 89600.00 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 = 59225.00 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 = 43733.333 is the unconfined external core area which is equal to $b_i^2/6$ as defined at (A.2).

ase2 (\geq ase1) = $\text{Max}(((A_{\text{conf,max2}} - A_{\text{noConf2}})/A_{\text{conf,max2}}) * (A_{\text{conf,min2}}/A_{\text{conf,max2}}), 0) = 0.33836562$

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 = 45264.00 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 = 0.00 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 = 32309.333 is the unconfined internal core area which is equal to $b_i^2/6$ as defined at (A.2).

psh,min*Fywe = $\text{Min}(psh_x * Fywe, psh_y * Fywe) = 6.12205$

Expression (5.4d) for psh,min*Fywe has been multiplied by 0.3 according to 15.7.1.3 for members without earthquake detailing (90° closed stirrups)

psh_x*Fywe = psh1*Fywe1 + ps2*Fywe2 = 6.12205

psh1 ((5.4d), TBDY) = $L_{\text{stir1}} * A_{\text{stir1}} / (A_{\text{sec}} * s_1) = 0.00680678$

Lstir1 (Length of stirrups along Y) = 1040.00

Astir1 (stirrups area) = 78.53982

psh2 (5.4d) = $L_{\text{stir2}} * A_{\text{stir2}} / (A_{\text{sec}} * s_2) = 0.0012868$

Lstir2 (Length of stirrups along Y) = 768.00

Astir2 (stirrups area) = 50.26548

psh_y*Fywe = psh1*Fywe1 + ps2*Fywe2 = 6.12205

psh1 ((5.4d), TBDY) = $L_{\text{stir1}} * A_{\text{stir1}} / (A_{\text{sec}} * s_1) = 0.00680678$

Lstir1 (Length of stirrups along X) = 1040.00

Astir1 (stirrups area) = 78.53982

psh2 ((5.4d), TBDY) = $L_{\text{stir2}} * A_{\text{stir2}} / (A_{\text{sec}} * s_2) = 0.0012868$

Lstir2 (Length of stirrups along X) = 768.00

Astir2 (stirrups area) = 50.26548

Asec = 120000.00

s1 = 100.00

s2 = 250.00

fywe1 = 781.25

fywe2 = 625.00

fce = 30.00

From ((5.A5), TBDY), TBDY: cc = 0.002

c = confinement factor = 1.00

y1 = 0.0025

sh1 = 0.008

ft1 = 886.7403

fy1 = 738.9503

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 = $(f_{s,\text{jacket}} * A_{s,\text{ten,jacket}} + f_{s,\text{core}} * A_{s,\text{ten,core}}) / A_{s,\text{ten}} = 738.9503$

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

y2 = 0.0025

sh2 = 0.008

ft2 = 898.293

fy2 = 748.5775

```

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 = 748.5775
with Es2 = (Es,jacket*Asl,com,jacket + Es,core*Asl,com,core)/Asl,com = 200000.00
yv = 0.0025
shv = 0.008
ftv = 894.3662
fyv = 745.3052
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 = 745.3052
with Esv = (Es,jacket*Asl,mid,jacket + Es,mid*Asl,mid,core)/Asl,mid = 200000.00
1 = Asl,ten/(b*d)*(fs1/fc) = 0.19082108
2 = Asl,com/(b*d)*(fs2/fc) = 0.37540035
v = Asl,mid/(b*d)*(fsv/fc) = 0.33973286
and confined core properties:
b = 360.00
d = 347.00
d' = 13.00
fcc (5A.2, TBDY) = 30.00
cc (5A.5, TBDY) = 0.002
c = confinement factor = 1.00
1 = Asl,ten/(b*d)*(fs1/fc) = 0.22424379
2 = Asl,com/(b*d)*(fs2/fc) = 0.44115251
v = Asl,mid/(b*d)*(fsv/fc) = 0.39923778
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) = 0.42533033
MRc (4.17) = 7.2277E+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) = 0.50069522

M_{Ro} (4.18) = 5.8153E+008

--->

u = cu (4.2) = 1.1511372E-005

Mu = M_{Ro}

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 = 3.0526570E-005

Mu = 4.9554E+008

with full section properties:

b = 200.00

d = 367.00

d' = 33.00

v = 0.97992777

N = 2.1578E+006

fc = 30.00

co (5A.5, TBDY) = 0.002

Final value of cu: cu* = shear_factor * Max(cu, cc) = 0.00925707

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

From (5.4b), TBDY: cu = 0.00925707

we (5.4c) = 0.0207149

ase ((5.4d), TBDY) = (ase1*Aext+ase2*Aint)/Asec = 0.33836562

ase1 = Max(((Aconf,max1-AnoConf1)/Aconf,max1)*(Aconf,min1/Aconf,max1),0) = 0.33836562

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 = 89600.00 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 = 59225.00 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 = 43733.333 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) = 0.33836562

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 = 45264.00 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.

$A_{conf,min2} = 0.00$ 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.

$A_{noConf2} = 32309.333$ is the unconfined internal core area which is equal to $b_i^2/6$ as defined at (A.2).

$psh_{min} \cdot F_{ywe} = \text{Min}(psh_x \cdot F_{ywe}, psh_y \cdot F_{ywe}) = 6.12205$

Expression (5.4d) for $psh_{min} \cdot F_{ywe}$ has been multiplied by 0.3 according to 15.7.1.3 for members without earthquake detailing (90° closed stirrups)

$psh_x \cdot F_{ywe} = psh1 \cdot F_{ywe1} + ps2 \cdot F_{ywe2} = 6.12205$

$psh1$ ((5.4d), TBDY) = $L_{stir1} \cdot A_{stir1} / (A_{sec} \cdot s1) = 0.00680678$

L_{stir1} (Length of stirrups along Y) = 1040.00

A_{stir1} (stirrups area) = 78.53982

$psh2$ (5.4d) = $L_{stir2} \cdot A_{stir2} / (A_{sec} \cdot s2) = 0.0012868$

L_{stir2} (Length of stirrups along Y) = 768.00

A_{stir2} (stirrups area) = 50.26548

$psh_y \cdot F_{ywe} = psh1 \cdot F_{ywe1} + ps2 \cdot F_{ywe2} = 6.12205$

$psh1$ ((5.4d), TBDY) = $L_{stir1} \cdot A_{stir1} / (A_{sec} \cdot s1) = 0.00680678$

L_{stir1} (Length of stirrups along X) = 1040.00

A_{stir1} (stirrups area) = 78.53982

$psh2$ ((5.4d), TBDY) = $L_{stir2} \cdot A_{stir2} / (A_{sec} \cdot s2) = 0.0012868$

L_{stir2} (Length of stirrups along X) = 768.00

A_{stir2} (stirrups area) = 50.26548

$A_{sec} = 120000.00$

$s1 = 100.00$

$s2 = 250.00$

$f_{ywe1} = 781.25$

$f_{ywe2} = 625.00$

$f_{ce} = 30.00$

From ((5.A5), TBDY), TBDY: $cc = 0.002$

c = confinement factor = 1.00

$y1 = 0.0025$

$sh1 = 0.008$

$ft1 = 898.293$

$fy1 = 748.5775$

$su1 = 0.032$

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

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

$su1 = 0.4 \cdot esu1_{nominal}$ ((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 $\text{Min}(1, 1.25 \cdot (lb/ld)^{2/3})$, from 10.3.5, ASCE 41-17.

with $fs1 = (fs_{jacket} \cdot A_{sl,ten,jacket} + fs_{core} \cdot A_{sl,ten,core}) / A_{sl,ten} = 748.5775$

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

$y2 = 0.0025$

$sh2 = 0.008$

$ft2 = 886.7403$

$fy2 = 738.9503$

$su2 = 0.032$

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 $fs2 = (fs_{jacket} \cdot A_{sl,com,jacket} + fs_{core} \cdot A_{sl,com,core}) / A_{sl,com} = 738.9503$

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 = 894.3662
fyv = 745.3052
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 = 745.3052
    with Esv = (Es,jacket*Asl,mid,jacket + Es,mid*Asl,mid,core)/Asl,mid = 200000.00
    1 = Asl,ten/(b*d)*(fs1/fc) = 0.7508007
    2 = Asl,com/(b*d)*(fs2/fc) = 0.38164216
    v = Asl,mid/(b*d)*(fsv/fc) = 0.67946571
and confined core properties:
    b = 160.00
    d = 347.00
    d' = 13.00
    fcc (5A.2, TBDY) = 30.00
    cc (5A.5, TBDY) = 0.002
    c = confinement factor = 1.00
    1 = Asl,ten/(b*d)*(fs1/fc) = 0.99259314
    2 = Asl,com/(b*d)*(fs2/fc) = 0.50454853
    v = Asl,mid/(b*d)*(fsv/fc) = 0.898285
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) = 0.8262842
    MRc (4.18) = 4.9554E+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) = 0.90160009
 MRO (4.18) = 2.4685E+008
 MRO < 0.8*MRc

--->
 u = cu (unconfined full section) = 3.0526570E-005
 Mu = MRc

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 = 1.1511372E-005
 Mu = 5.8153E+008

with full section properties:

b = 400.00
 d = 367.00
 d' = 33.00
 v = 0.48996389
 N = 2.1578E+006

fc = 30.00
 co (5A.5, TBDY) = 0.002

Final value of cu: cu* = shear_factor * Max(cu, cc) = 0.00925707
 The Shear_factor is considered equal to 1 (pure moment strength)
 From (5.4b), TBDY: cu = 0.00925707

we (5.4c) = 0.0207149

ase ((5.4d), TBDY) = (ase1*Aext+ase2*Aint)/Asec = 0.33836562

ase1 = Max(((Aconf,max1-AnoConf1)/Aconf,max1)*(Aconf,min1/Aconf,max1),0) = 0.33836562

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 = 89600.00 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 = 59225.00 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 = 43733.333 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) = 0.33836562

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 = 45264.00 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 = 0.00 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 = 32309.333 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) = 6.12205

Expression (5.4d) for psh,min*Fywe has been multiplied by 0.3 according to 15.7.1.3 for members without earthquake detailing (90° closed stirrups)

psh_x*Fywe = psh1*Fywe1+ps2*Fywe2 = 6.12205

psh1 ((5.4d), TBDY) = Lstir1*Astir1/(Asec*s1) = 0.00680678

Lstir1 (Length of stirrups along Y) = 1040.00

Astir1 (stirrups area) = 78.53982

$psh_2 (5.4d) = Lstir_2 * Astir_2 / (Asec * s_2) = 0.0012868$
 $Lstir_2$ (Length of stirrups along Y) = 768.00
 $Astir_2$ (stirrups area) = 50.26548

$psh_y * Fywe = psh_1 * Fywe_1 + ps_2 * Fywe_2 = 6.12205$
 $psh_1 ((5.4d), TBDY) = Lstir_1 * Astir_1 / (Asec * s_1) = 0.00680678$
 $Lstir_1$ (Length of stirrups along X) = 1040.00
 $Astir_1$ (stirrups area) = 78.53982
 $psh_2 ((5.4d), TBDY) = Lstir_2 * Astir_2 / (Asec * s_2) = 0.0012868$
 $Lstir_2$ (Length of stirrups along X) = 768.00
 $Astir_2$ (stirrups area) = 50.26548

$Asec = 120000.00$

$s_1 = 100.00$

$s_2 = 250.00$

$fywe_1 = 781.25$

$fywe_2 = 625.00$

$fce = 30.00$

From ((5.A5), TBDY), TBDY: $cc = 0.002$

$c =$ confinement factor = 1.00

$y_1 = 0.0025$

$sh_1 = 0.008$

$ft_1 = 886.7403$

$fy_1 = 738.9503$

$su_1 = 0.032$

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

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

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

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 $Min(1, 1.25 * (lb/l_d)^{2/3})$, from 10.3.5, ASCE 41-17.

with $fs_1 = (fs_{jacket} * Asl_{ten,jacket} + fs_{core} * Asl_{ten,core}) / Asl_{ten} = 738.9503$

with $Es_1 = (Es_{jacket} * Asl_{ten,jacket} + Es_{core} * Asl_{ten,core}) / Asl_{ten} = 200000.00$

$y_2 = 0.0025$

$sh_2 = 0.008$

$ft_2 = 898.293$

$fy_2 = 748.5775$

$su_2 = 0.032$

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

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

$su_2 = 0.4 * esu_{2_nominal} ((5.5), TBDY) = 0.032$

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 $Min(1, 1.25 * (lb/l_d)^{2/3})$, from 10.3.5, ASCE 41-17.

with $fs_2 = (fs_{jacket} * Asl_{com,jacket} + fs_{core} * Asl_{com,core}) / Asl_{com} = 748.5775$

with $Es_2 = (Es_{jacket} * Asl_{com,jacket} + Es_{core} * Asl_{com,core}) / Asl_{com} = 200000.00$

$y_v = 0.0025$

$sh_v = 0.008$

$ft_v = 894.3662$

$fy_v = 745.3052$

$suv = 0.032$

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

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

$suv = 0.4 * esuv_{nominal} ((5.5), TBDY) = 0.032$

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 $Min(1, 1.25 * (lb/l_d)^{2/3})$, from 10.3.5, ASCE 41-17.

```

with fsv = (fs,jacket*Asl,mid,jacket + fs,mid*Asl,mid,core)/Asl,mid = 745.3052
with Esv = (Es,jacket*Asl,mid,jacket + Es,mid*Asl,mid,core)/Asl,mid = 200000.00
1 = Asl,ten/(b*d)*(fs1/fc) = 0.19082108
2 = Asl,com/(b*d)*(fs2/fc) = 0.37540035
v = Asl,mid/(b*d)*(fsv/fc) = 0.33973286
and confined core properties:
b = 360.00
d = 347.00
d' = 13.00
fcc (5A.2, TBDY) = 30.00
cc (5A.5, TBDY) = 0.002
c = confinement factor = 1.00
1 = Asl,ten/(b*d)*(fs1/fc) = 0.22424379
2 = Asl,com/(b*d)*(fs2/fc) = 0.44115251
v = Asl,mid/(b*d)*(fsv/fc) = 0.39923778
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) = 0.42533033
MRc (4.17) = 7.2277E+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) = 0.50069522
MRo (4.18) = 5.8153E+008
--->
u = cu (4.2) = 1.1511372E-005
Mu = MRo

```

Calculation of ratio lb/d

Adequate Lap Length: lb/d >= 1

Calculation of Mu2-

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

$$\mu = 3.0526570E-005$$

$$\mu = 4.9554E+008$$

with full section properties:

$$b = 200.00$$

$$d = 367.00$$

$$d' = 33.00$$

$$v = 0.97992777$$

$$N = 2.1578E+006$$

$$f_c = 30.00$$

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

$$\text{Final value of } \mu: \mu^* = \text{shear_factor} * \text{Max}(\mu, \alpha) = 0.00925707$$

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

$$\text{From (5.4b), TBDY: } \mu = 0.00925707$$

$$\mu_e (5.4c) = 0.0207149$$

$$\alpha_e ((5.4d), TBDY) = (\alpha_e1 * A_{ext} + \alpha_e2 * A_{int}) / A_{sec} = 0.33836562$$

$$\alpha_e1 = \text{Max}(((A_{conf,max1} - A_{noConf1}) / A_{conf,max1}) * (A_{conf,min1} / A_{conf,max1}), 0) = 0.33836562$$

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.

$A_{conf,max1} = 89600.00$ 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.

$A_{conf,min1} = 59225.00$ 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.

$A_{noConf1} = 43733.333$ is the unconfined external core area which is equal to $b_i^2/6$ as defined at (A.2).

$$\alpha_e2 (> \alpha_e1) = \text{Max}(((A_{conf,max2} - A_{noConf2}) / A_{conf,max2}) * (A_{conf,min2} / A_{conf,max2}), 0) = 0.33836562$$

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.

$A_{conf,max2} = 45264.00$ 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.

$A_{conf,min2} = 0.00$ 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.

$A_{noConf2} = 32309.333$ is the unconfined internal core area which is equal to $b_i^2/6$ as defined at (A.2).

$$p_{sh,min} * F_{ywe} = \text{Min}(p_{sh,x} * F_{ywe}, p_{sh,y} * F_{ywe}) = 6.12205$$

Expression (5.4d) for $p_{sh,min} * F_{ywe}$ has been multiplied by 0.3 according to 15.7.1.3 for members without earthquake detailing (90° closed stirrups)

$$p_{sh,x} * F_{ywe} = p_{sh1} * F_{ywe1} + p_{sh2} * F_{ywe2} = 6.12205$$

$$p_{sh1} ((5.4d), TBDY) = L_{stir1} * A_{stir1} / (A_{sec} * s1) = 0.00680678$$

$$L_{stir1} \text{ (Length of stirrups along Y)} = 1040.00$$

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

$$p_{sh2} (5.4d) = L_{stir2} * A_{stir2} / (A_{sec} * s2) = 0.0012868$$

$$L_{stir2} \text{ (Length of stirrups along Y)} = 768.00$$

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

$$p_{sh,y} * F_{ywe} = p_{sh1} * F_{ywe1} + p_{sh2} * F_{ywe2} = 6.12205$$

$$p_{sh1} ((5.4d), TBDY) = L_{stir1} * A_{stir1} / (A_{sec} * s1) = 0.00680678$$

$$L_{stir1} \text{ (Length of stirrups along X)} = 1040.00$$

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

$$p_{sh2} ((5.4d), TBDY) = L_{stir2} * A_{stir2} / (A_{sec} * s2) = 0.0012868$$

$$L_{stir2} \text{ (Length of stirrups along X)} = 768.00$$

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

$$A_{sec} = 120000.00$$

$$s1 = 100.00$$

$s_2 = 250.00$
 $fy_{we1} = 781.25$
 $fy_{we2} = 625.00$
 $f_{ce} = 30.00$
 From ((5A.5), TBDY), TBDY: $cc = 0.002$
 $c = \text{confinement factor} = 1.00$
 $y_1 = 0.0025$
 $sh_1 = 0.008$
 $ft_1 = 898.293$
 $fy_1 = 748.5775$
 $su_1 = 0.032$
 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 $fs_1 = (fs_{jacket} * Asl, \text{ten, jacket} + fs_{core} * Asl, \text{ten, core}) / Asl, \text{ten} = 748.5775$
 with $Es_1 = (Es_{jacket} * Asl, \text{ten, jacket} + Es_{core} * Asl, \text{ten, core}) / Asl, \text{ten} = 200000.00$
 $y_2 = 0.0025$
 $sh_2 = 0.008$
 $ft_2 = 886.7403$
 $fy_2 = 738.9503$
 $su_2 = 0.032$
 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 $fs_2 = (fs_{jacket} * Asl, \text{com, jacket} + fs_{core} * Asl, \text{com, core}) / Asl, \text{com} = 738.9503$
 with $Es_2 = (Es_{jacket} * Asl, \text{com, jacket} + Es_{core} * Asl, \text{com, core}) / Asl, \text{com} = 200000.00$
 $y_v = 0.0025$
 $sh_v = 0.008$
 $ft_v = 894.3662$
 $fy_v = 745.3052$
 $suv = 0.032$
 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 $fsyv = fs_v/1.2$, 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_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 $fs_v = (fs_{jacket} * Asl, \text{mid, jacket} + fs_{mid} * Asl, \text{mid, core}) / Asl, \text{mid} = 745.3052$
 with $Es_v = (Es_{jacket} * Asl, \text{mid, jacket} + Es_{mid} * Asl, \text{mid, core}) / Asl, \text{mid} = 200000.00$
 $1 = Asl, \text{ten} / (b * d) * (fs_1 / f_c) = 0.7508007$
 $2 = Asl, \text{com} / (b * d) * (fs_2 / f_c) = 0.38164216$
 $v = Asl, \text{mid} / (b * d) * (fs_v / f_c) = 0.67946571$
 and confined core properties:
 $b = 160.00$
 $d = 347.00$
 $d' = 13.00$
 $f_{cc} \text{ (5A.2, TBDY)} = 30.00$
 $cc \text{ (5A.5, TBDY)} = 0.002$
 $c = \text{confinement factor} = 1.00$
 $1 = Asl, \text{ten} / (b * d) * (fs_1 / f_c) = 0.99259314$
 $2 = Asl, \text{com} / (b * d) * (fs_2 / f_c) = 0.50454853$

```

v = Asl,mid/(b*d)*(fsv/fc) = 0.898285
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) = 0.8262842
MRc (4.18) = 4.9554E+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) = 0.90160009
MRo (4.18) = 2.4685E+008
MRo < 0.8*MRc
--->
u = cu (unconfined full section) = 3.0526570E-005
Mu = MRc
-----

```

Calculation of ratio lb/l_d

Adequate Lap Length: lb/l_d >= 1

Calculation of Shear Strength $V_r = \text{Min}(V_{r1}, V_{r2}) = 349917.706$

Calculation of Shear Strength at edge 1, $V_{r1} = 349917.706$

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

$V_{Co10} = 349917.706$

$k_{nl} = 1$ (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: $f'_c = (f'_c_jacket * \text{Area_jacket} + f'_c_core * \text{Area_core}) / \text{Area_section} = 25.00$, but $f'_c^{0.5} \leq 8.3$

MPa (22.5.3.1, ACI 318-14)

$$M/Vd = 4.00$$

$$\mu_u = 8.5591E+007$$

$$V_u = 5.7948283E-005$$

$$d = 0.8 \cdot h = 320.00$$

$$N_u = 2.1578E+006$$

$$A_g = 80000.00$$

$$\text{From (11.5.4.8), ACI 318-14: } V_s = V_{sjacket} + V_{s,core} = 471238.898$$

where:

$$V_{sjacket} = V_{sj1} + V_{sj2} = 471238.898$$

$V_{sj1} = 157079.633$ is calculated for section web jacket, with:

$$d = 160.00$$

$$A_v = 157079.633$$

$$f_y = 625.00$$

$$s = 100.00$$

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

$$s/d = 0.625$$

$V_{sj2} = 314159.265$ is calculated for section flange jacket, with:

$$d = 320.00$$

$$A_v = 157079.633$$

$$f_y = 625.00$$

$$s = 100.00$$

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

$$s/d = 0.3125$$

$$V_{s,core} = V_{s,c1} + V_{s,c2} = 0.00$$

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

$$d = 80.00$$

$$A_v = 100530.965$$

$$f_y = 500.00$$

$$s = 250.00$$

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

$$s/d = 3.125$$

$V_{s,c2} = 0.00$ is calculated for section flange core, with:

$$d = 240.00$$

$$A_v = 100530.965$$

$$f_y = 500.00$$

$$s = 250.00$$

$V_{s,c2}$ is multiplied by $Col,c2 = 0.00$

$$s/d = 1.04167$$

$$V_f \text{ ((11-3)-(11.4), ACI 440)} = 0.00$$

$$\text{From (11-11), ACI 440: } V_s + V_f \leq 212577.225$$

$$bw = 200.00$$

Calculation of Shear Strength at edge 2, $V_{r2} = 349917.706$

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

$$V_{Col0} = 349917.706$$

$$knl = 1 \text{ (zero step-static loading)}$$

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).

$$= 1 \text{ (normal-weight concrete)}$$

$$\text{Mean concrete strength: } f'_c = (f'_{c,jacket} \cdot Area_{jacket} + f'_{c,core} \cdot Area_{core}) / Area_{section} = 25.00, \text{ but } f_c^{0.5} \leq 8.3$$

MPa (22.5.3.1, ACI 318-14)

$$M/Vd = 4.00$$

$$\mu_u = 6.4291E+007$$

$$V_u = 5.7948283E-005$$

$$d = 0.8 \cdot h = 320.00$$

$$N_u = 2.1578E+006$$

$$A_g = 80000.00$$

$$\text{From (11.5.4.8), ACI 318-14: } V_s = V_{sjacket} + V_{s,core} = 471238.898$$

where:

$$V_{sjacket} = V_{sj1} + V_{sj2} = 471238.898$$

$V_{sj1} = 157079.633$ is calculated for section web jacket, with:

$$d = 160.00$$

$$A_v = 157079.633$$

$f_y = 625.00$
 $s = 100.00$
 $V_{s,j1}$ is multiplied by $Col,j1 = 1.00$
 $s/d = 0.625$
 $V_{s,j2} = 314159.265$ is calculated for section flange jacket, with:
 $d = 320.00$
 $A_v = 157079.633$
 $f_y = 625.00$
 $s = 100.00$
 $V_{s,j2}$ is multiplied by $Col,j2 = 1.00$
 $s/d = 0.3125$
 $V_{s,core} = V_{s,c1} + V_{s,c2} = 0.00$
 $V_{s,c1} = 0.00$ is calculated for section web core, with:
 $d = 80.00$
 $A_v = 100530.965$
 $f_y = 500.00$
 $s = 250.00$
 $V_{s,c1}$ is multiplied by $Col,c1 = 0.00$
 $s/d = 3.125$
 $V_{s,c2} = 0.00$ is calculated for section flange core, with:
 $d = 240.00$
 $A_v = 100530.965$
 $f_y = 500.00$
 $s = 250.00$
 $V_{s,c2}$ is multiplied by $Col,c2 = 0.00$
 $s/d = 1.04167$
 $V_f ((11-3)-(11.4), ACI 440) = 0.00$
 From (11-11), ACI 440: $V_s + V_f \leq 212577.225$
 $bw = 200.00$

 End Of Calculation of Shear Capacity ratio for element: column JLC1 of floor 1
 At local axis: 2

 Start Of Calculation of Chord Rotation Capacity for element: column JLC1 of floor 1
 At local axis: 3
 Integration Section: (b)
 Section Type: rcjlc

Constant Properties

 Knowledge Factor, $\gamma = 0.90$
 Chord Rotation is generally considered as Deformation-Controlled Action according to Table C7-1, ASCE 41-17.
 Mean strengths are used for Deformation-Controlled Actions according to 7.5.1.3, ASCE 41-17
 Consequently:
 Jacket
 New material of Secondary Member: Concrete Strength, $f_c = f_{cm} = 30.00$
 New material of Secondary Member: Steel Strength, $f_s = f_{sm} = 625.00$
 Concrete Elasticity, $E_c = 25742.96$
 Steel Elasticity, $E_s = 200000.00$
 Existing Column
 Existing material of Secondary Member: Concrete Strength, $f_c = f_{cm} = 18.00$
 Existing material of Secondary Member: Steel Strength, $f_s = f_{sm} = 500.00$
 Concrete Elasticity, $E_c = 19940.411$
 Steel Elasticity, $E_s = 200000.00$
 Max Height, $H_{max} = 400.00$
 Min Height, $H_{min} = 200.00$
 Max Width, $W_{max} = 400.00$
 Min Width, $W_{min} = 200.00$
 Jacket Thickness, $t_j = 50.00$
 Cover Thickness, $c = 15.00$
 Element Length, $L = 3000.00$
 Secondary Member
 Smooth Bars

Ductile Steel
Without Detailing for Earthquake Resistance (including stirrups not 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, $M = -6.4291\text{E}+007$
Shear Force, $V2 = 0.60572401$
Shear Force, $V3 = 1.4589530\text{E}-005$
Axial Force, $F = -2.1578\text{E}+006$
Longitudinal Reinforcement Area Distribution (in 2 divisions)
-Tension: $As_t = 0.00$
-Compression: $As_c = 5353.274$
Longitudinal Reinforcement Area Distribution (in 3 divisions)
-Tension: $As_{t,ten} = 1137.257$
-Compression: $As_{c,com} = 2208.54$
-Middle: $As_{c,mid} = 2007.478$
Longitudinal External Reinforcement Area Distribution (in 3 divisions)
-Tension: $As_{t,ten,jacket} = 829.3805$
-Compression: $As_{c,com,jacket} = 1746.726$
-Middle: $As_{c,mid,jacket} = 1545.664$
Longitudinal Internal Reinforcement Area Distribution (in 3 divisions)
-Tension: $As_{t,ten,core} = 307.8761$
-Compression: $As_{c,com,core} = 461.8141$
-Middle: $As_{c,mid,core} = 461.8141$
Mean Diameter of Tension Reinforcement, $Db_L = 16.80$

Existing component: From table 7-7, ASCE 41_17: Final chord rotation Capacity $u_R = u = 0.03784131$
 $u = y + p = 0.0420459$

- Calculation of y -

$y = (M_y * L_s / 3) / E_{eff} = 0.0420459$ ((4.29), Biskinis Phd))
 $M_y = 5.2455\text{E}+008$
 $L_s = M/V$ (with $L_s > 0.1 * L$ and $L_s < 2 * L$) = 6000.00
From table 10.5, ASCE 41_17: $E_{eff} = \text{factor} * E_c * I_g = 2.4951\text{E}+013$
factor = 0.70
 $A_g = 120000.00$
Mean concrete strength: $f'_c = (f'_{c,jacket} * \text{Area}_{jacket} + f'_{c,core} * \text{Area}_{core}) / \text{Area}_{section} = 25.00$
 $N = 2.1578\text{E}+006$
 $E_c * I_g = E_{c,jacket} * I_{g,jacket} + E_{c,core} * I_{g,core} = 3.5645\text{E}+013$

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, $b = 400.00$
web width, $b_w = 200.00$
flange thickness, $t = 200.00$

$y = \text{Min}(y_{ten}, y_{com})$
 $y_{ten} = 1.4921411\text{E}-005$
with $f_y = 596.2441$
 $d = 367.00$
 $y = 0.45559994$
 $A = 0.06111896$
 $B = 0.04120454$

with $p_t = 0.00774698$
 $p_c = 0.01504455$
 $p_v = 0.01367492$
 $N = 2.1578E+006$
 $b = 400.00$
 $" = 0.08991826$
 $y_{comp} = 1.1520096E-005$
 with $f_c = 30.00$
 $E_c = 25742.96$
 $y = 0.49615013$
 $A = 0.00143$
 $B = 0.01655203$
 with $E_s = 200000.00$
 CONFIRMATION: $y = 0.49615013 < t/d$

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 $I_b/I_d \geq 1$

shear control ratio $V_y E / V_{col} E = 1.10794$

$d = d_{external} = 367.00$

$s = s_{external} = 100.00$

- $t = s_1 + s_2 + 2 * t_f / b_w * (f_{fe} / f_s) = 0.00809358$

jacket: $s_1 = A_{v1} * L_{stir1} / (s_1 * A_g) = 0.00680678$

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

$L_{stir1} = 1040.00$, is the total Length of all stirrups parallel to loading (shear) direction

$s_1 = 100.00$

core: $s_2 = A_{v2} * L_{stir2} / (s_2 * A_g) = 0.0012868$

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

$L_{stir2} = 768.00$, 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.

$N_{UD} = 2.1578E+006$

$A_g = 120000.00$

$f_{cE} = (f_{c,jacket} * Area_{jacket} + f_{c,core} * Area_{core}) / section_area = 25.00$

$f_{yE} = (f_{y,ext_Long_Reinf} * Area_{ext_Long_Reinf} + f_{y,int_Long_Reinf} * Area_{int_Long_Reinf}) / Area_{Tot_Long_Rein} = 596.2441$

$f_{yE} = (f_{y,ext_Trans_Reinf} * s_1 + f_{y,int_Trans_Reinf} * s_2) / (s_1 + s_2) = 605.1263$

$p_l = Area_{Tot_Long_Rein} / (b * d) = 0.03646644$

$b = 400.00$

$d = 367.00$

$f_{cE} = 25.00$

End Of Calculation of Chord Rotation Capacity for element: column JLC1 of floor 1

At local axis: 3

Integration Section: (b)

Calculation No. 9

column C1, Floor 1

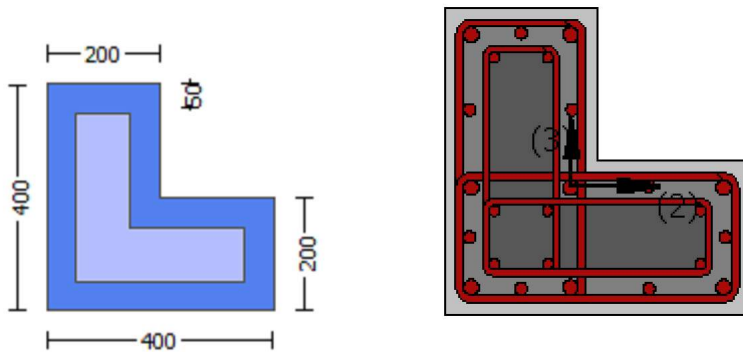
Limit State: Life Safety (data interpolation between analysis steps 1 and 2)

Analysis: Uniform +X

Check: Shear capacity VRd

Edge: Start

Local Axis: (2)



Start Of Calculation of Shear Capacity for element: column JLC1 of floor 1

At local axis: 2

Integration Section: (a)

Section Type: rcjlcs

Constant Properties

Knowledge Factor, $\gamma = 0.90$

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

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

Consequently:

Jacket

New material of Secondary Member: Concrete Strength, $f_c = f_{c_lower_bound} = 20.00$

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

Concrete Elasticity, $E_c = 25742.96$

Steel Elasticity, $E_s = 200000.00$

Existing Column

Existing material of Secondary Member: Concrete Strength, $f_c = f_{c_lower_bound} = 12.00$

Existing material of Secondary Member: Steel Strength, $f_s = f_{s_lower_bound} = 400.00$

Concrete Elasticity, $E_c = 19940.411$

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, $f_c = f_{cm} = 30.00$

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

Existing Column

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

Existing material: Steel Strength, $f_s = f_{sm} = 500.00$

#####

Max Height, Hmax = 400.00
 Min Height, Hmin = 200.00
 Max Width, Wmax = 400.00
 Min Width, Wmin = 200.00
 Jacket Thickness, tj = 50.00
 Cover Thickness, c = 15.00
 Element Length, L = 3000.00
 Secondary Member
 Smooth Bars
 Ductile Steel
 Without Detailing for Earthquake Resistance (including stirrups not 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 = 8.5589E+007
 Shear Force, Va = -0.60572401
 EDGE -B-
 Bending Moment, Mb = -6.4291E+007
 Shear Force, Vb = 0.60572401
 BOTH EDGES
 Axial Force, F = -2.1578E+006
 Longitudinal Reinforcement Area Distribution (in 2 divisions)
 -Tension: Aslt = 0.00
 -Compression: Aslc = 5353.274
 Longitudinal Reinforcement Area Distribution (in 3 divisions)
 -Tension: Asl,ten = 1137.257
 -Compression: Asl,com = 2208.54
 -Middle: Asl,mid = 2007.478
 Mean Diameter of Tension Reinforcement, DbL,ten = 16.80

Existing component: From table 7-7, ASCE 41_17: Final Shear Capacity VR = $\phi V_n = 267030.06$
 $V_n ((10.3), ASCE 41-17) = knl \cdot V_{CoI} = 296700.066$
 $V_{CoI} = 296700.066$
 $knl = 1.00$
 $displacement_ductility_demand = 0.07831286$

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: $fc' = (fc'_{jacket} \cdot Area_{jacket} + fc'_{core} \cdot Area_{core}) / Area_{section} = 16.66667$, but $fc'^{0.5} \leq 8.3$ MPa (22.5.3.1, ACI 318-14)
 $M/Vd = 4.00$
 $M_u = 8.5589E+007$
 $V_u = 0.60572401$
 $d = 0.8 \cdot h = 320.00$
 $N_u = 2.1578E+006$
 $A_g = 80000.00$
 From (11.5.4.8), ACI 318-14: $V_s = V_{s,jacket} + V_{s,core} = 376991.118$
 where:
 $V_{s,jacket} = V_{s,j1} + V_{s,j2} = 376991.118$
 $V_{s,j1} = 125663.706$ is calculated for section web jacket, with:
 $d = 160.00$
 $A_v = 157079.633$
 $f_y = 500.00$
 $s = 100.00$
 $V_{s,j1}$ is multiplied by $Col,j1 = 1.00$
 $s/d = 0.625$
 $V_{s,j2} = 251327.412$ is calculated for section flange jacket, with:

$d = 320.00$
 $A_v = 157079.633$
 $f_y = 500.00$
 $s = 100.00$
 $V_{s,j2}$ is multiplied by $Col,j2 = 1.00$
 $s/d = 0.3125$
 $V_{s,core} = V_{s,c1} + V_{s,c2} = 0.00$
 $V_{s,c1} = 0.00$ is calculated for section web core, with:
 $d = 80.00$
 $A_v = 100530.965$
 $f_y = 400.00$
 $s = 250.00$
 $V_{s,c1}$ is multiplied by $Col,c1 = 0.00$
 $s/d = 3.125$
 $V_{s,c2} = 0.00$ is calculated for section flange core, with:
 $d = 240.00$
 $A_v = 100530.965$
 $f_y = 400.00$
 $s = 250.00$
 $V_{s,c2}$ is multiplied by $Col,c2 = 0.00$
 $s/d = 1.04167$
 $V_f ((11-3)-(11.4), ACI 440) = 0.00$
 From (11-11), ACI 440: $V_s + V_f \leq 173568.578$
 $bw = 200.00$

displacement ductility demand is calculated as δ_u / y

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

From analysis, chord rotation $\theta_r = 0.00329273$
 $y = (M_y * L_s / 3) / E_{eff} = 0.0420459$ ((4.29), Biskinis Phd))
 $M_y = 5.2455E+008$
 $L_s = M/V$ (with $L_s > 0.1 * L$ and $L_s < 2 * L$) = 6000.00
 From table 10.5, ASCE 41_17: $E_{eff} = factor * E_c * I_g = 2.4951E+013$
 $factor = 0.70$
 $A_g = 120000.00$
 Mean concrete strength: $f'_c = (f'_{c,jacket} * Area_{jacket} + f'_{c,core} * Area_{core}) / Area_{section} = 25.00$
 $N = 2.1578E+006$
 $E_c * I_g = E_{c,jacket} * I_{g,jacket} + E_{c,core} * I_{g,core} = 3.5645E+013$

Calculation of Yielding Moment M_y

Calculation of δ_u and M_y according to Annex 7 -

Assuming neutral axis within flange ($y_c < t/d$, compression zone rectangular) with:
 flange width, $b = 400.00$
 web width, $bw = 200.00$
 flange thickness, $t = 200.00$

$y = \text{Min}(y_{ten}, y_{com})$
 $y_{ten} = 1.4921411E-005$
 with $f_y = 596.2441$
 $d = 367.00$
 $y = 0.45559994$
 $A = 0.06111896$
 $B = 0.04120454$
 with $pt = 0.00774698$
 $pc = 0.01504455$
 $pv = 0.01367492$
 $N = 2.1578E+006$
 $b = 400.00$
 $\rho = 0.08991826$

y_comp = 1.1520096E-005
with $f_c = 30.00$
 $E_c = 25742.96$
 $y = 0.49615013$
 $A = 0.00143$
 $B = 0.01655203$
with $E_s = 200000.00$
CONFIRMATION: $y = 0.49615013 < t/d$

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 JLC1 of floor 1

At local axis: 2

Integration Section: (a)

Calculation No. 10

column C1, Floor 1

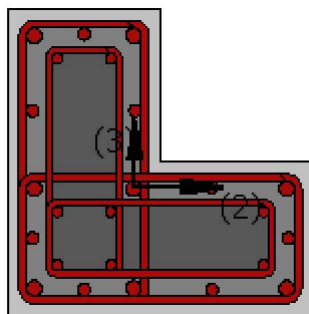
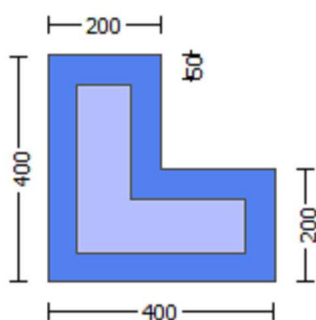
Limit State: Life Safety (data interpolation between analysis steps 1 and 2)

Analysis: Uniform +X

Check: Chord rotation capacity (ϕ)

Edge: Start

Local Axis: (2)



Start Of Calculation of Shear Capacity ratio for element: column JLC1 of floor 1

At Shear local axis: 3

(Bending local axis: 2)

Section Type: rcjlc

Constant Properties

Knowledge Factor, $\phi = 0.90$

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

Consequently:

```

Jacket
New material of Secondary Member: Concrete Strength,  $f_c = f_{cm} = 30.00$ 
New material of Secondary Member: Steel Strength,  $f_s = f_{sm} = 625.00$ 
Concrete Elasticity,  $E_c = 25742.96$ 
Steel Elasticity,  $E_s = 200000.00$ 
Existing Column
Existing material of Secondary Member: Concrete Strength,  $f_c = f_{cm} = 18.00$ 
Existing material of Secondary Member: Steel Strength,  $f_s = f_{sm} = 500.00$ 
Concrete Elasticity,  $E_c = 19940.411$ 
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,  $f_s = 1.25 \cdot f_{sm} = 625.00$ 
#####
Max Height,  $H_{max} = 400.00$ 
Min Height,  $H_{min} = 200.00$ 
Max Width,  $W_{max} = 400.00$ 
Min Width,  $W_{min} = 200.00$ 
Jacket Thickness,  $t_j = 50.00$ 
Cover Thickness,  $c = 15.00$ 
Mean Confinement Factor overall section = 1.00
Element Length,  $L = 3000.00$ 
Secondary Member
Smooth Bars
Ductile Steel
Without Detailing for Earthquake Resistance (including stirrups not 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,  $V_a = -1.4594946E-005$ 
EDGE -B-
Shear Force,  $V_b = 1.4594946E-005$ 
BOTH EDGES
Axial Force,  $F = -2.1578E+006$ 
Longitudinal Reinforcement Area Distribution (in 2 divisions)
-Tension:  $As_t = 0.00$ 
-Compression:  $As_c = 5353.274$ 
Longitudinal Reinforcement Area Distribution (in 3 divisions)
-Tension:  $As_{ten} = 2208.54$ 
-Compression:  $As_{com} = 1137.257$ 
-Middle:  $As_{mid} = 2007.478$ 
-----
-----

Calculation of Shear Capacity ratio ,  $V_e/V_r = 1.37616$ 
Member Controlled by Shear ( $V_e/V_r > 1$ )
Calculation of Shear Demand from fig. R18.6.5, ACI 318-14  $V_e = (M_{pr1} + M_{pr2})/l_n = 481543.08$ 
with
 $M_{pr1} = \text{Max}(\mu_{u1+}, \mu_{u1-}) = 7.2231E+008$ 
 $\mu_{u1+} = 7.2231E+008$ , is the ultimate moment strength at the edge 1 of the member in the actual moment direction
which is defined for the static loading combination
 $\mu_{u1-} = 6.3496E+008$ , 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
 $M_{pr2} = \text{Max}(\mu_{u2+}, \mu_{u2-}) = 7.2231E+008$ 
 $\mu_{u2+} = 7.2231E+008$ , is the ultimate moment strength at the edge 2 of the member in the actual moment direction
which is defined for the the static loading combination

```


Mu2- = 6.3496E+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:

$$\phi_u = 3.9059244E-005$$

$$M_u = 7.2231E+008$$

with full section properties:

$$b = 400.00$$

$$d = 367.00$$

$$d' = 33.00$$

$$\nu = 0.48996389$$

$$N = 2.1578E+006$$

$$f_c = 30.00$$

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

$$\text{Final value of } \phi_{cu}: \phi_{cu}^* = \text{shear_factor} * \text{Max}(\phi_{cu}, \phi_{co}) = 0.00925707$$

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

$$\text{From (5.4b), TBDY: } \phi_{cu} = 0.00925707$$

$$\phi_{we} (5.4c) = 0.0207149$$

$$\phi_{ase} ((5.4d), TBDY) = (\phi_{ase1} * A_{ext} + \phi_{ase2} * A_{int}) / A_{sec} = 0.33836562$$

$$\phi_{ase1} = \text{Max}(((A_{conf,max1} - A_{noConf1}) / A_{conf,max1}) * (A_{conf,min1} / A_{conf,max1}), 0) = 0.33836562$$

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.

$A_{conf,max1} = 89600.00$ 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.

$A_{conf,min1} = 59225.00$ 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.

$A_{noConf1} = 43733.333$ is the unconfined external core area which is equal to $b_i^2/6$ as defined at (A.2).

$$\phi_{ase2} (> \phi_{ase1}) = \text{Max}(((A_{conf,max2} - A_{noConf2}) / A_{conf,max2}) * (A_{conf,min2} / A_{conf,max2}), 0) = 0.33836562$$

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.

$A_{conf,max2} = 45264.00$ 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.

$A_{conf,min2} = 0.00$ 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.

$A_{noConf2} = 32309.333$ is the unconfined internal core area which is equal to $b_i^2/6$ as defined at (A.2).

$$\phi_{psh,min} * F_{ywe} = \text{Min}(\phi_{psh,x} * F_{ywe}, \phi_{psh,y} * F_{ywe}) = 6.12205$$

Expression (5.4d) for $\phi_{psh,min} * F_{ywe}$ has been multiplied by 0.3 according to 15.7.1.3 for members without earthquake detailing (90° closed stirrups)

$$\phi_{psh,x} * F_{ywe} = \phi_{psh1} * F_{ywe1} + \phi_{psh2} * F_{ywe2} = 6.12205$$

$$\phi_{psh1} ((5.4d), TBDY) = L_{stir1} * A_{stir1} / (A_{sec} * s_1) = 0.00680678$$

$$L_{stir1} (\text{Length of stirrups along Y}) = 1040.00$$

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

$$\phi_{psh2} (5.4d) = L_{stir2} * A_{stir2} / (A_{sec} * s_2) = 0.0012868$$

$$L_{stir2} (\text{Length of stirrups along Y}) = 768.00$$

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

$$\phi_{psh,y} * F_{ywe} = \phi_{psh1} * F_{ywe1} + \phi_{psh2} * F_{ywe2} = 6.12205$$

$$\phi_{psh1} ((5.4d), TBDY) = L_{stir1} * A_{stir1} / (A_{sec} * s_1) = 0.00680678$$

$$L_{stir1} (\text{Length of stirrups along X}) = 1040.00$$

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

$$\phi_{psh2} ((5.4d), TBDY) = L_{stir2} * A_{stir2} / (A_{sec} * s_2) = 0.0012868$$

$$L_{stir2} (\text{Length of stirrups along X}) = 768.00$$

Astir2 (stirrups area) = 50.26548

Asec = 120000.00

s1 = 100.00

s2 = 250.00

fywe1 = 781.25

fywe2 = 625.00

fce = 30.00

From ((5.A5), TBDY), TBDY: cc = 0.002

c = confinement factor = 1.00

y1 = 0.0025

sh1 = 0.008

ft1 = 898.293

fy1 = 748.5775

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 = 748.5775

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

y2 = 0.0025

sh2 = 0.008

ft2 = 886.7403

fy2 = 738.9503

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 = 738.9503

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

yv = 0.0025

shv = 0.008

ftv = 894.3662

fyv = 745.3052

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 = 745.3052

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

1 = Asl,ten/(b*d)*(fs1/fc) = 0.37540035

2 = Asl,com/(b*d)*(fs2/fc) = 0.19082108

v = Asl,mid/(b*d)*(fsv/fc) = 0.33973286

and confined core properties:

b = 360.00

d = 347.00

d' = 13.00

fcc (5A.2, TBDY) = 30.00

```

cc (5A.5, TBDY) = 0.002
c = confinement factor = 1.00
1 = Asl,ten/(b*d)*(fs1/fc) = 0.44115251
2 = Asl,com/(b*d)*(fs2/fc) = 0.22424379
v = Asl,mid/(b*d)*(fsv/fc) = 0.39923778
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) = 0.64577857
MRc (4.17) = 7.2231E+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) = 0.67946111
MRo (4.18) = 4.2866E+008
MRo < 0.8*MRc
---->
u = cu (unconfined full section) = 3.9059244E-005
Mu = MRc
-----

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 = 4.2871818E-005
Mu = 6.3496E+008
-----
with full section properties:

```

$b = 200.00$
 $d = 367.00$
 $d' = 33.00$
 $v = 0.97992777$
 $N = 2.1578E+006$
 $f_c = 30.00$
 $co (5A.5, TBDY) = 0.002$
 Final value of cu : $cu^* = shear_factor * Max(cu, cc) = 0.00925707$
 The Shear_factor is considered equal to 1 (pure moment strength)
 From (5.4b), TBDY: $cu = 0.00925707$
 $w_e (5.4c) = 0.0207149$
 $ase ((5.4d), TBDY) = (ase1 * A_{ext} + ase2 * A_{int}) / A_{sec} = 0.33836562$
 $ase1 = Max(((A_{conf,max1} - A_{noConf1}) / A_{conf,max1}) * (A_{conf,min1} / A_{conf,max1}), 0) = 0.33836562$
 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.
 $A_{conf,max1} = 89600.00$ 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.
 $A_{conf,min1} = 59225.00$ 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.
 $A_{noConf1} = 43733.333$ is the unconfined external core area which is equal to $b_i^2/6$ as defined at (A.2).
 $ase2 (>= ase1) = Max(((A_{conf,max2} - A_{noConf2}) / A_{conf,max2}) * (A_{conf,min2} / A_{conf,max2}), 0) = 0.33836562$
 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.
 $A_{conf,max2} = 45264.00$ 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.
 $A_{conf,min2} = 0.00$ 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.
 $A_{noConf2} = 32309.333$ is the unconfined internal core area which is equal to $b_i^2/6$ as defined at (A.2).
 $psh_{min} * F_{ywe} = Min(psh_x * F_{ywe}, psh_y * F_{ywe}) = 6.12205$
 Expression (5.4d) for $psh_{min} * F_{ywe}$ has been multiplied by 0.3 according to 15.7.1.3 for members without earthquake detailing (90° closed stirrups)

$psh_x * F_{ywe} = psh1 * F_{ywe1} + ps2 * F_{ywe2} = 6.12205$
 $psh1 ((5.4d), TBDY) = L_{stir1} * A_{stir1} / (A_{sec} * s1) = 0.00680678$
 L_{stir1} (Length of stirrups along Y) = 1040.00
 A_{stir1} (stirrups area) = 78.53982
 $psh2 (5.4d) = L_{stir2} * A_{stir2} / (A_{sec} * s2) = 0.0012868$
 L_{stir2} (Length of stirrups along Y) = 768.00
 A_{stir2} (stirrups area) = 50.26548

$psh_y * F_{ywe} = psh1 * F_{ywe1} + ps2 * F_{ywe2} = 6.12205$
 $psh1 ((5.4d), TBDY) = L_{stir1} * A_{stir1} / (A_{sec} * s1) = 0.00680678$
 L_{stir1} (Length of stirrups along X) = 1040.00
 A_{stir1} (stirrups area) = 78.53982
 $psh2 ((5.4d), TBDY) = L_{stir2} * A_{stir2} / (A_{sec} * s2) = 0.0012868$
 L_{stir2} (Length of stirrups along X) = 768.00
 A_{stir2} (stirrups area) = 50.26548

$A_{sec} = 120000.00$
 $s1 = 100.00$
 $s2 = 250.00$
 $f_{ywe1} = 781.25$
 $f_{ywe2} = 625.00$
 $f_{ce} = 30.00$
 From ((5.A5), TBDY), TBDY: $cc = 0.002$
 c = confinement factor = 1.00
 $y1 = 0.0025$
 $sh1 = 0.008$
 $ft1 = 886.7403$

```

fy1 = 738.9503
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 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 = 738.9503
    with Es1 = (Es,jacket*Asl,ten,jacket + Es,core*Asl,ten,core)/Asl,ten = 200000.00
y2 = 0.0025
sh2 = 0.008
ft2 = 898.293
fy2 = 748.5775
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 = 748.5775
    with Es2 = (Es,jacket*Asl,com,jacket + Es,core*Asl,com,core)/Asl,com = 200000.00
yv = 0.0025
shv = 0.008
ftv = 894.3662
fyv = 745.3052
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 = 745.3052
    with Esv = (Es,jacket*Asl,mid,jacket + Es,mid*Asl,mid,core)/Asl,mid = 200000.00
1 = Asl,ten/(b*d)*(fs1/fc) = 0.38164216
2 = Asl,com/(b*d)*(fs2/fc) = 0.7508007
v = Asl,mid/(b*d)*(fsv/fc) = 0.67946571
and confined core properties:
b = 160.00
d = 347.00
d' = 13.00
fcc (5A.2, TBDY) = 30.00
cc (5A.5, TBDY) = 0.002
    c = confinement factor = 1.00
    1 = Asl,ten/(b*d)*(fs1/fc) = 0.50454853
    2 = Asl,com/(b*d)*(fs2/fc) = 0.99259314
    v = Asl,mid/(b*d)*(fsv/fc) = 0.898285
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) = 0.58834972
 $M_{Rc}$  (4.17) = 6.3496E+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:  $b_o$ ,  $d_o$ ,  $d'_o$ 
-  $N$ ,  $1$ ,  $2$ ,  $v$  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) = 0.63603191
 $M_{Ro}$  (4.18) = 4.4768E+008
 $M_{Ro} < 0.8 * M_{Rc}$ 
---->
 $u = c_u$  (unconfined full section) = 4.2871818E-005
 $M_u = M_{Rc}$ 
-----

Calculation of ratio  $l_b/d$ 
-----
Adequate Lap Length:  $l_b/d \geq 1$ 
-----
-----
Calculation of  $M_{u2+}$ 
-----
-----
Calculation of ultimate curvature  $u$  according to 4.1, Biskinis/Fardis 2013:
 $u = 3.9059244E-005$ 
 $M_u = 7.2231E+008$ 
-----
with full section properties:
 $b = 400.00$ 
 $d = 367.00$ 
 $d' = 33.00$ 
 $v = 0.48996389$ 
 $N = 2.1578E+006$ 
 $f_c = 30.00$ 
 $c_o$  (5A.5, TBDY) = 0.002
Final value of  $c_u$ :  $c_u^* = \text{shear\_factor} * \text{Max}(c_u, c_c) = 0.00925707$ 
The Shear_factor is considered equal to 1 (pure moment strength)
From (5.4b), TBDY:  $c_u = 0.00925707$ 
 $w_e$  (5.4c) = 0.0207149
 $a_{se}$  ((5.4d), TBDY) =  $(a_{se1} * A_{ext} + a_{se2} * A_{int}) / A_{sec} = 0.33836562$ 
 $a_{se1} = \text{Max}(((A_{conf,max1} - A_{noConf1}) / A_{conf,max1}) * (A_{conf,min1} / A_{conf,max1}), 0) = 0.33836562$ 

```

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 = 89600.00 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 = 59225.00 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 = 43733.333 is the unconfined external core area which is equal to $b_i^2/6$ as defined at (A.2).

ase2 (\geq ase1) = $\text{Max}(((A_{\text{conf,max2}} - A_{\text{NoConf2}})/A_{\text{conf,max2}}) * (A_{\text{conf,min2}}/A_{\text{conf,max2}}), 0) = 0.33836562$

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 = 45264.00 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 = 0.00 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 = 32309.333 is the unconfined internal core area which is equal to $b_i^2/6$ as defined at (A.2).

psh,min*Fywe = $\text{Min}(psh_x * Fywe, psh_y * Fywe) = 6.12205$

Expression (5.4d) for psh,min*Fywe has been multiplied by 0.3 according to 15.7.1.3 for members without earthquake detailing (90° closed stirrups)

psh_x*Fywe = psh1*Fywe1 + ps2*Fywe2 = 6.12205

psh1 ((5.4d), TBDY) = $L_{\text{stir1}} * A_{\text{stir1}} / (A_{\text{sec}} * s_1) = 0.00680678$

Lstir1 (Length of stirrups along Y) = 1040.00

Astir1 (stirrups area) = 78.53982

psh2 (5.4d) = $L_{\text{stir2}} * A_{\text{stir2}} / (A_{\text{sec}} * s_2) = 0.0012868$

Lstir2 (Length of stirrups along Y) = 768.00

Astir2 (stirrups area) = 50.26548

psh_y*Fywe = psh1*Fywe1 + ps2*Fywe2 = 6.12205

psh1 ((5.4d), TBDY) = $L_{\text{stir1}} * A_{\text{stir1}} / (A_{\text{sec}} * s_1) = 0.00680678$

Lstir1 (Length of stirrups along X) = 1040.00

Astir1 (stirrups area) = 78.53982

psh2 ((5.4d), TBDY) = $L_{\text{stir2}} * A_{\text{stir2}} / (A_{\text{sec}} * s_2) = 0.0012868$

Lstir2 (Length of stirrups along X) = 768.00

Astir2 (stirrups area) = 50.26548

Asec = 120000.00

s1 = 100.00

s2 = 250.00

fywe1 = 781.25

fywe2 = 625.00

fce = 30.00

From ((5.A5), TBDY), TBDY: cc = 0.002

c = confinement factor = 1.00

y1 = 0.0025

sh1 = 0.008

ft1 = 898.293

fy1 = 748.5775

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

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 $\text{Min}(1, 1.25 * (lb/lb)^{2/3})$, from 10.3.5, ASCE 41-17.

with fs1 = $(f_{s,jacket} * A_{s,ten,jacket} + f_{s,core} * A_{s,ten,core}) / A_{s,ten} = 748.5775$

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 = 886.7403
fy2 = 738.9503
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/ld)^ 2/3), from 10.3.5, ASCE 41-17.
    with fs2 = (fs,jacket*Asl,com,jacket + fs,core*Asl,com,core)/Asl,com = 738.9503
    with Es2 = (Es,jacket*Asl,com,jacket + Es,core*Asl,com,core)/Asl,com = 200000.00
yv = 0.0025
shv = 0.008
ftv = 894.3662
fyv = 745.3052
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 = 745.3052
    with Esv = (Es,jacket*Asl,mid,jacket + Es,mid*Asl,mid,core)/Asl,mid = 200000.00
1 = Asl,ten/(b*d)*(fs1/fc) = 0.37540035
2 = Asl,com/(b*d)*(fs2/fc) = 0.19082108
v = Asl,mid/(b*d)*(fsv/fc) = 0.33973286
and confined core properties:
b = 360.00
d = 347.00
d' = 13.00
fcc (5A.2, TBDY) = 30.00
cc (5A.5, TBDY) = 0.002
c = confinement factor = 1.00
1 = Asl,ten/(b*d)*(fs1/fc) = 0.44115251
2 = Asl,com/(b*d)*(fs2/fc) = 0.22424379
v = Asl,mid/(b*d)*(fsv/fc) = 0.39923778
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) = 0.64577857
MRc (4.17) = 7.2231E+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) = 0.67946111

M_{Ro} (4.18) = 4.2866E+008

$M_{Ro} < 0.8 * M_{Rc}$

--->

$u = cu$ (unconfined full section) = 3.9059244E-005

$\mu = M_{Rc}$

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:

$u = 4.2871818E-005$

$\mu = 6.3496E+008$

with full section properties:

$b = 200.00$

$d = 367.00$

$d' = 33.00$

$v = 0.97992777$

$N = 2.1578E+006$

$f_c = 30.00$

α (5A.5, TBDY) = 0.002

Final value of cu : $cu^* = \text{shear_factor} * \text{Max}(cu, cc) = 0.00925707$

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

From (5.4b), TBDY: $cu = 0.00925707$

w_e (5.4c) = 0.0207149

α_{se} ((5.4d), TBDY) = $(\alpha_{se1} * A_{ext} + \alpha_{se2} * A_{int}) / A_{sec} = 0.33836562$

$\alpha_{se1} = \text{Max}(((A_{conf,max1} - A_{noConf1}) / A_{conf,max1}) * (A_{conf,min1} / A_{conf,max1}), 0) = 0.33836562$

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.

$A_{conf,max1} = 89600.00$ 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.

$A_{conf,min1} = 59225.00$ 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.

$A_{noConf1} = 43733.333$ is the unconfined external core area which is equal to $b^2/6$ as defined at (A.2).

$\alpha_{se2} (\geq \alpha_{se1}) = \text{Max}(((A_{conf,max2} - A_{noConf2}) / A_{conf,max2}) * (A_{conf,min2} / A_{conf,max2}), 0) = 0.33836562$

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.

$A_{conf,max2} = 45264.00$ 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.

$A_{conf,min2} = 0.00$ 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.

$A_{noConf2} = 32309.333$ is the unconfined internal core area which is equal to $b_i^2/6$ as defined at (A.2).

$p_{sh,min} \cdot F_{ywe} = \text{Min}(p_{sh,x} \cdot F_{ywe}, p_{sh,y} \cdot F_{ywe}) = 6.12205$

Expression (5.4d) for $p_{sh,min} \cdot F_{ywe}$ has been multiplied by 0.3 according to 15.7.1.3 for members without earthquake detailing (90° closed stirrups)

 $p_{sh,x} \cdot F_{ywe} = p_{sh1} \cdot F_{ywe1} + p_{s2} \cdot F_{ywe2} = 6.12205$

p_{sh1} ((5.4d), TBDY) = $L_{stir1} \cdot A_{stir1} / (A_{sec} \cdot s_1) = 0.00680678$

L_{stir1} (Length of stirrups along Y) = 1040.00

A_{stir1} (stirrups area) = 78.53982

p_{sh2} (5.4d) = $L_{stir2} \cdot A_{stir2} / (A_{sec} \cdot s_2) = 0.0012868$

L_{stir2} (Length of stirrups along Y) = 768.00

A_{stir2} (stirrups area) = 50.26548

 $p_{sh,y} \cdot F_{ywe} = p_{sh1} \cdot F_{ywe1} + p_{s2} \cdot F_{ywe2} = 6.12205$

p_{sh1} ((5.4d), TBDY) = $L_{stir1} \cdot A_{stir1} / (A_{sec} \cdot s_1) = 0.00680678$

L_{stir1} (Length of stirrups along X) = 1040.00

A_{stir1} (stirrups area) = 78.53982

p_{sh2} ((5.4d), TBDY) = $L_{stir2} \cdot A_{stir2} / (A_{sec} \cdot s_2) = 0.0012868$

L_{stir2} (Length of stirrups along X) = 768.00

A_{stir2} (stirrups area) = 50.26548

 $A_{sec} = 120000.00$

$s_1 = 100.00$

$s_2 = 250.00$

$f_{ywe1} = 781.25$

$f_{ywe2} = 625.00$

$f_{ce} = 30.00$

From ((5.A5), TBDY), TBDY: $cc = 0.002$

c = confinement factor = 1.00

$y_1 = 0.0025$

$sh_1 = 0.008$

$ft_1 = 886.7403$

$fy_1 = 738.9503$

$su_1 = 0.032$

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 $fs_1 = (f_{s,jacket} \cdot A_{s,ten,jacket} + f_{s,core} \cdot A_{s,ten,core}) / A_{s,ten} = 738.9503$

with $Es_1 = (E_{s,jacket} \cdot A_{s,ten,jacket} + E_{s,core} \cdot A_{s,ten,core}) / A_{s,ten} = 200000.00$

$y_2 = 0.0025$

$sh_2 = 0.008$

$ft_2 = 898.293$

$fy_2 = 748.5775$

$su_2 = 0.032$

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 $fsy_2 = fs_2/1.2$, 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 $fs2 = (fs_{jacket} \cdot Asl_{com,jacket} + fs_{core} \cdot Asl_{com,core}) / Asl_{com} = 748.5775$
 with $Es2 = (Es_{jacket} \cdot Asl_{com,jacket} + Es_{core} \cdot Asl_{com,core}) / Asl_{com} = 200000.00$
 $yv = 0.0025$
 $shv = 0.008$
 $ftv = 894.3662$
 $fyv = 745.3052$
 $suv = 0.032$
 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 $fsv = (fs_{jacket} \cdot Asl_{mid,jacket} + fs_{mid} \cdot Asl_{mid,core}) / Asl_{mid} = 745.3052$
 with $Esv = (Es_{jacket} \cdot Asl_{mid,jacket} + Es_{mid} \cdot Asl_{mid,core}) / Asl_{mid} = 200000.00$
 $1 = Asl_{ten} / (b \cdot d) \cdot (fs1 / fc) = 0.38164216$
 $2 = Asl_{com} / (b \cdot d) \cdot (fs2 / fc) = 0.7508007$
 $v = Asl_{mid} / (b \cdot d) \cdot (fsv / fc) = 0.67946571$

and confined core properties:

$b = 160.00$
 $d = 347.00$
 $d' = 13.00$
 $fcc (5A.2, TBDY) = 30.00$
 $cc (5A.5, TBDY) = 0.002$
 $c = \text{confinement factor} = 1.00$
 $1 = Asl_{ten} / (b \cdot d) \cdot (fs1 / fc) = 0.50454853$
 $2 = Asl_{com} / (b \cdot d) \cdot (fs2 / fc) = 0.99259314$
 $v = Asl_{mid} / (b \cdot d) \cdot (fsv / fc) = 0.898285$

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

--->

$cu (4.10) = 0.58834972$

$MRc (4.17) = 6.3496E+008$

--->

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) = 0.63603191
MRo (4.18) = 4.4768E+008
MRo < 0.8*MRc
--->
u = cu (unconfined full section) = 4.2871818E-005
Mu = MRc
-----

Calculation of ratio lb/d
-----
Adequate Lap Length: lb/d >= 1
-----
-----
-----

Calculation of Shear Strength Vr = Min(Vr1,Vr2) = 349917.706
-----
Calculation of Shear Strength at edge 1, Vr1 = 349917.706
Vr1 = VCol ((10.3), ASCE 41-17) = knl*VColO
VColO = 349917.706
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 = 25.00, but fc'^0.5 <= 8.3
MPa (22.5.3.1, ACI 318-14)
M/Vd = 4.00
Mu = 3.4090E+006
Vu = 1.4594946E-005
d = 0.8*h = 320.00
Nu = 2.1578E+006
Ag = 80000.00
From (11.5.4.8), ACI 318-14: Vs = Vs,jacket + Vs,core = 471238.898
where:
Vs,jacket = Vs,j1 + Vs,j2 = 471238.898
Vs,j1 = 314159.265 is calculated for section web jacket, with:
d = 320.00
Av = 157079.633
fy = 625.00
s = 100.00
Vs,j1 is multiplied by Col,j1 = 1.00
s/d = 0.3125
Vs,j2 = 157079.633 is calculated for section flange jacket, with:
d = 160.00
Av = 157079.633
fy = 625.00
s = 100.00
Vs,j2 is multiplied by Col,j2 = 1.00
s/d = 0.625
Vs,core = Vs,c1 + Vs,c2 = 0.00
Vs,c1 = 0.00 is calculated for section web core, with:
d = 240.00
Av = 100530.965
fy = 500.00
s = 250.00
Vs,c1 is multiplied by Col,c1 = 0.00
s/d = 1.04167
Vs,c2 = 0.00 is calculated for section flange core, with:
d = 80.00
Av = 100530.965

```

$f_y = 500.00$
 $s = 250.00$
 $V_{s,c2}$ is multiplied by $Col,c2 = 0.00$
 $s/d = 3.125$
 $V_f ((11-3)-(11.4), ACI 440) = 0.00$
 From (11-11), ACI 440: $V_s + V_f \leq 212577.225$
 $bw = 200.00$

Calculation of Shear Strength at edge 2, $V_{r2} = 349917.706$
 $V_{r2} = V_{Col} ((10.3), ASCE 41-17) = knl * V_{Col0}$
 $V_{Col0} = 349917.706$
 $knl = 1$ (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: $f_c' = (f_c'_{jacket} * Area_{jacket} + f_c'_{core} * Area_{core}) / Area_{section} = 25.00$, but $f_c'^{0.5} \leq 8.3$ MPa (22.5.3.1, ACI 318-14)
 $M/Vd = 4.00$
 $\mu_u = 15336.932$
 $V_u = 1.4594946E+005$
 $d = 0.8 * h = 320.00$
 $N_u = 2.1578E+006$
 $A_g = 80000.00$
 From (11.5.4.8), ACI 318-14: $V_s = V_{s,jacket} + V_{s,core} = 471238.898$
 where:
 $V_{s,jacket} = V_{s,j1} + V_{s,j2} = 471238.898$
 $V_{s,j1} = 314159.265$ is calculated for section web jacket, with:
 $d = 320.00$
 $A_v = 157079.633$
 $f_y = 625.00$
 $s = 100.00$
 $V_{s,j1}$ is multiplied by $Col,j1 = 1.00$
 $s/d = 0.3125$
 $V_{s,j2} = 157079.633$ is calculated for section flange jacket, with:
 $d = 160.00$
 $A_v = 157079.633$
 $f_y = 625.00$
 $s = 100.00$
 $V_{s,j2}$ is multiplied by $Col,j2 = 1.00$
 $s/d = 0.625$
 $V_{s,core} = V_{s,c1} + V_{s,c2} = 0.00$
 $V_{s,c1} = 0.00$ is calculated for section web core, with:
 $d = 240.00$
 $A_v = 100530.965$
 $f_y = 500.00$
 $s = 250.00$
 $V_{s,c1}$ is multiplied by $Col,c1 = 0.00$
 $s/d = 1.04167$
 $V_{s,c2} = 0.00$ is calculated for section flange core, with:
 $d = 80.00$
 $A_v = 100530.965$
 $f_y = 500.00$
 $s = 250.00$
 $V_{s,c2}$ is multiplied by $Col,c2 = 0.00$
 $s/d = 3.125$
 $V_f ((11-3)-(11.4), ACI 440) = 0.00$
 From (11-11), ACI 440: $V_s + V_f \leq 212577.225$
 $bw = 200.00$

End Of Calculation of Shear Capacity ratio for element: column JLC1 of floor 1
 At local axis: 3

Start Of Calculation of Shear Capacity ratio for element: column JLC1 of floor 1

At Shear local axis: 2

(Bending local axis: 3)

Section Type: rcjls

Constant Properties

Knowledge Factor, $\gamma = 0.90$

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

Consequently:

Jacket

New material of Secondary Member: Concrete Strength, $f_c = f_{cm} = 30.00$

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

Concrete Elasticity, $E_c = 25742.96$

Steel Elasticity, $E_s = 200000.00$

Existing Column

Existing material of Secondary Member: Concrete Strength, $f_c = f_{cm} = 18.00$

Existing material of Secondary Member: Steel Strength, $f_s = f_{sm} = 500.00$

Concrete Elasticity, $E_c = 19940.411$

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, $f_s = 1.25 \cdot f_{sm} = 625.00$

#####

Max Height, $H_{max} = 400.00$

Min Height, $H_{min} = 200.00$

Max Width, $W_{max} = 400.00$

Min Width, $W_{min} = 200.00$

Jacket Thickness, $t_j = 50.00$

Cover Thickness, $c = 15.00$

Mean Confinement Factor overall section = 1.00

Element Length, $L = 3000.00$

Secondary Member

Smooth Bars

Ductile Steel

Without Detailing for Earthquake Resistance (including stirrups not 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: 2

EDGE -A-

Shear Force, $V_a = -5.7948283E-005$

EDGE -B-

Shear Force, $V_b = 5.7948283E-005$

BOTH EDGES

Axial Force, $F = -2.1578E+006$

Longitudinal Reinforcement Area Distribution (in 2 divisions)

-Tension: $As_t = 0.00$

-Compression: $As_c = 5353.274$

Longitudinal Reinforcement Area Distribution (in 3 divisions)

-Tension: $As_{t,ten} = 1137.257$

-Compression: $As_{l,com} = 2208.54$

-Middle: $As_{l,mid} = 2007.478$

Calculation of Shear Capacity ratio , $V_e/V_r = 1.10794$

Member Controlled by Shear ($V_e/V_r > 1$)

Calculation of Shear Demand from fig. R18.6.5, ACI 318-14 $V_e = (M_{pr1} + M_{pr2})/l_n = 387689.388$

with

$M_{pr1} = \text{Max}(M_{u1+}, M_{u1-}) = 5.8153\text{E}+008$

$M_{u1+} = 5.8153\text{E}+008$, is the ultimate moment strength at the edge 1 of the member in the actual moment direction which is defined for the static loading combination

$M_{u1-} = 4.9554\text{E}+008$, 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

$M_{pr2} = \text{Max}(M_{u2+}, M_{u2-}) = 5.8153\text{E}+008$

$M_{u2+} = 5.8153\text{E}+008$, 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

$M_{u2-} = 4.9554\text{E}+008$, 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 M_{u1+}

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

$\phi_u = 1.1511372\text{E}-005$

$M_u = 5.8153\text{E}+008$

with full section properties:

$b = 400.00$

$d = 367.00$

$d' = 33.00$

$v = 0.48996389$

$N = 2.1578\text{E}+006$

$f_c = 30.00$

ϕ_c (5A.5, TBDY) = 0.002

Final value of ϕ_u : $\phi_u^* = \text{shear_factor} * \text{Max}(\phi_u, \phi_c) = 0.00925707$

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

From (5.4b), TBDY: $\phi_u = 0.00925707$

ϕ_{ue} (5.4c) = 0.0207149

ϕ_{ase} ((5.4d), TBDY) = $(\phi_{ase1} * A_{ext} + \phi_{ase2} * A_{int}) / A_{sec} = 0.33836562$

$\phi_{ase1} = \text{Max}(((A_{conf,max1} - A_{noConf1}) / A_{conf,max1}) * (A_{conf,min1} / A_{conf,max1}), 0) = 0.33836562$

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.

$A_{conf,max1} = 89600.00$ 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.

$A_{conf,min1} = 59225.00$ 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.

$A_{noConf1} = 43733.333$ is the unconfined external core area which is equal to $b_i^2/6$ as defined at (A.2).

$\phi_{ase2} (> \phi_{ase1}) = \text{Max}(((A_{conf,max2} - A_{noConf2}) / A_{conf,max2}) * (A_{conf,min2} / A_{conf,max2}), 0) = 0.33836562$

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.

$A_{conf,max2} = 45264.00$ 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.

$A_{conf,min2} = 0.00$ 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.

$A_{noConf2} = 32309.333$ is the unconfined internal core area which is equal to $b_i^2/6$ as defined at (A.2).

$p_{sh,min} * F_{ywe} = \text{Min}(p_{sh,x} * F_{ywe}, p_{sh,y} * F_{ywe}) = 6.12205$

Expression (5.4d) for $p_{sh,min} * F_{ywe}$ has been multiplied by 0.3 according to 15.7.1.3 for members without earthquake detailing (90° closed stirrups)

 $p_{sh,x} * F_{ywe} = p_{sh1} * F_{ywe1} + p_{sh2} * F_{ywe2} = 6.12205$

ϕ_{sh1} ((5.4d), TBDY) = $L_{stir1} * A_{stir1} / (A_{sec} * s_1) = 0.00680678$

Lstir1 (Length of stirrups along Y) = 1040.00
 Astir1 (stirrups area) = 78.53982
 psh2 (5.4d) = Lstir2*Astir2/(Asec*s2) = 0.0012868
 Lstir2 (Length of stirrups along Y) = 768.00
 Astir2 (stirrups area) = 50.26548

psh_y*Fywe = psh1*Fywe1+ps2*Fywe2 = 6.12205
 psh1 ((5.4d), TBDY) = Lstir1*Astir1/(Asec*s1) = 0.00680678
 Lstir1 (Length of stirrups along X) = 1040.00
 Astir1 (stirrups area) = 78.53982
 psh2 ((5.4d), TBDY) = Lstir2*Astir2/(Asec*s2) = 0.0012868
 Lstir2 (Length of stirrups along X) = 768.00
 Astir2 (stirrups area) = 50.26548

Asec = 120000.00

s1 = 100.00

s2 = 250.00

fywe1 = 781.25

fywe2 = 625.00

fce = 30.00

From ((5.A5), TBDY), TBDY: cc = 0.002

c = confinement factor = 1.00

y1 = 0.0025

sh1 = 0.008

ft1 = 886.7403

fy1 = 738.9503

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 = 738.9503

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

y2 = 0.0025

sh2 = 0.008

ft2 = 898.293

fy2 = 748.5775

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 = 748.5775

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

yv = 0.0025

shv = 0.008

ftv = 894.3662

fyv = 745.3052

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  $f_{sv} = f_{sv}/1.2$ , 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  $f_{sv} = (f_{s,jacket} \cdot A_{s,mid,jacket} + f_{s,mid} \cdot A_{s,mid,core}) / A_{s,mid} = 745.3052$ 
with  $E_{sv} = (E_{s,jacket} \cdot A_{s,mid,jacket} + E_{s,mid} \cdot A_{s,mid,core}) / A_{s,mid} = 200000.00$ 
 $1 = A_{s,ten} / (b \cdot d) \cdot (f_{s1}/f_c) = 0.19082108$ 
 $2 = A_{s,com} / (b \cdot d) \cdot (f_{s2}/f_c) = 0.37540035$ 
 $v = A_{s,mid} / (b \cdot d) \cdot (f_{sv}/f_c) = 0.33973286$ 
and confined core properties:
 $b = 360.00$ 
 $d = 347.00$ 
 $d' = 13.00$ 
 $f_{cc} \text{ (5A.2, TBDY)} = 30.00$ 
 $cc \text{ (5A.5, TBDY)} = 0.002$ 
 $c = \text{confinement factor} = 1.00$ 
 $1 = A_{s,ten} / (b \cdot d) \cdot (f_{s1}/f_c) = 0.22424379$ 
 $2 = A_{s,com} / (b \cdot d) \cdot (f_{s2}/f_c) = 0.44115251$ 
 $v = A_{s,mid} / (b \cdot d) \cdot (f_{sv}/f_c) = 0.39923778$ 
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
---->
 $c_u \text{ (4.10)} = 0.42533033$ 
 $M_{Rc} \text{ (4.17)} = 7.2277E+008$ 
---->
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:  $b_o, d_o, d'_o$ 
-  $N, 1, 2, v$  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
---->
 $*c_u \text{ (4.11)} = 0.50069522$ 
 $M_{Ro} \text{ (4.18)} = 5.8153E+008$ 
---->
 $u = c_u \text{ (4.2)} = 1.1511372E-005$ 
 $\mu = M_{Ro}$ 

```

Calculation of ratio l_b/l_d

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

Calculation of Mu1-

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

$$\mu = 3.0526570E-005$$

$$\mu_u = 4.9554E+008$$

with full section properties:

$$b = 200.00$$

$$d = 367.00$$

$$d' = 33.00$$

$$v = 0.97992777$$

$$N = 2.1578E+006$$

$$f_c = 30.00$$

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

$$\text{Final value of } c_u: c_u = \text{shear_factor} * \text{Max}(c_u, c_o) = 0.00925707$$

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

$$\text{From (5.4b), TBDY: } c_u = 0.00925707$$

$$w_e \text{ (5.4c)} = 0.0207149$$

$$a_{se} \text{ ((5.4d), TBDY)} = (a_{se1} * A_{ext} + a_{se2} * A_{int}) / A_{sec} = 0.33836562$$

$$a_{se1} = \text{Max}(((A_{conf,max1} - A_{noConf1}) / A_{conf,max1}) * (A_{conf,min1} / A_{conf,max1}), 0) = 0.33836562$$

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.

$A_{conf,max1} = 89600.00$ 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.

$A_{conf,min1} = 59225.00$ 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.

$A_{noConf1} = 43733.333$ is the unconfined external core area which is equal to $b_i^2/6$ as defined at (A.2).

$$a_{se2} (>= a_{se1}) = \text{Max}(((A_{conf,max2} - A_{noConf2}) / A_{conf,max2}) * (A_{conf,min2} / A_{conf,max2}), 0) = 0.33836562$$

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.

$A_{conf,max2} = 45264.00$ 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.

$A_{conf,min2} = 0.00$ 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.

$A_{noConf2} = 32309.333$ is the unconfined internal core area which is equal to $b_i^2/6$ as defined at (A.2).

$$p_{sh,min} * F_{ywe} = \text{Min}(p_{sh,x} * F_{ywe}, p_{sh,y} * F_{ywe}) = 6.12205$$

Expression (5.4d) for $p_{sh,min} * F_{ywe}$ has been multiplied by 0.3 according to 15.7.1.3 for members without earthquake detailing (90° closed stirrups)

$$p_{sh,x} * F_{ywe} = p_{sh1} * F_{ywe1} + p_{sh2} * F_{ywe2} = 6.12205$$

$$p_{sh1} \text{ ((5.4d), TBDY)} = L_{stir1} * A_{stir1} / (A_{sec} * s_1) = 0.00680678$$

$$L_{stir1} \text{ (Length of stirrups along Y)} = 1040.00$$

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

$$p_{sh2} \text{ (5.4d)} = L_{stir2} * A_{stir2} / (A_{sec} * s_2) = 0.0012868$$

$$L_{stir2} \text{ (Length of stirrups along Y)} = 768.00$$

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

$$p_{sh,y} * F_{ywe} = p_{sh1} * F_{ywe1} + p_{sh2} * F_{ywe2} = 6.12205$$

$$p_{sh1} \text{ ((5.4d), TBDY)} = L_{stir1} * A_{stir1} / (A_{sec} * s_1) = 0.00680678$$

$$L_{stir1} \text{ (Length of stirrups along X)} = 1040.00$$

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

$$p_{sh2} \text{ ((5.4d), TBDY)} = L_{stir2} * A_{stir2} / (A_{sec} * s_2) = 0.0012868$$

$$L_{stir2} \text{ (Length of stirrups along X)} = 768.00$$

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

```

Asec = 120000.00
s1 = 100.00
s2 = 250.00
fywe1 = 781.25
fywe2 = 625.00
fce = 30.00
From ((5A.5), TBDY), TBDY: cc = 0.002
c = confinement factor = 1.00
y1 = 0.0025
sh1 = 0.008
ft1 = 898.293
fy1 = 748.5775
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  $\text{Min}(1, 1.25 \cdot (lb/ld)^{2/3})$ , from 10.3.5, ASCE 41-17.
with fs1 = (fs,jacket*Asl,ten,jacket + fs,core*Asl,ten,core)/Asl,ten = 748.5775
with Es1 = (Es,jacket*Asl,ten,jacket + Es,core*Asl,ten,core)/Asl,ten = 200000.00
y2 = 0.0025
sh2 = 0.008
ft2 = 886.7403
fy2 = 738.9503
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 = 738.9503
with Es2 = (Es,jacket*Asl,com,jacket + Es,core*Asl,com,core)/Asl,com = 200000.00
yv = 0.0025
shv = 0.008
ftv = 894.3662
fyv = 745.3052
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  $\text{Min}(1, 1.25 \cdot (lb/ld)^{2/3})$ , from 10.3.5, ASCE 41-17.
with fsv = (fs,jacket*Asl,mid,jacket + fs,mid*Asl,mid,core)/Asl,mid = 745.3052
with Esv = (Es,jacket*Asl,mid,jacket + Es,mid*Asl,mid,core)/Asl,mid = 200000.00
1 = Asl,ten/(b*d)*(fs1/fc) = 0.7508007
2 = Asl,com/(b*d)*(fs2/fc) = 0.38164216
v = Asl,mid/(b*d)*(fsv/fc) = 0.67946571
and confined core properties:
b = 160.00
d = 347.00
d' = 13.00
fcc (5A.2, TBDY) = 30.00
cc (5A.5, TBDY) = 0.002
c = confinement factor = 1.00

```

```

1 = Asl,ten/(b*d)*(fs1/fc) = 0.99259314
2 = Asl,com/(b*d)*(fs2/fc) = 0.50454853
v = Asl,mid/(b*d)*(fsv/fc) = 0.898285
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) = 0.8262842
MRc (4.18) = 4.9554E+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, 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) = 0.90160009
MRo (4.18) = 2.4685E+008
MRo < 0.8*MRc
---->
u = cu (unconfined full section) = 3.0526570E-005
Mu = MRc
-----

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 = 1.1511372E-005
Mu = 5.8153E+008
-----
with full section properties:
b = 400.00
d = 367.00

```

$d' = 33.00$
 $v = 0.48996389$
 $N = 2.1578E+006$
 $f_c = 30.00$
 $\alpha (5A.5, TBDY) = 0.002$
 Final value of α : $\alpha^* = \text{shear_factor} * \text{Max}(\alpha, \alpha_c) = 0.00925707$
 The Shear_factor is considered equal to 1 (pure moment strength)
 From (5.4b), TBDY: $\alpha = 0.00925707$
 $w_e (5.4c) = 0.0207149$
 $\alpha_{se} ((5.4d), TBDY) = (\alpha_{se1} * A_{ext} + \alpha_{se2} * A_{int}) / A_{sec} = 0.33836562$
 $\alpha_{se1} = \text{Max}(((A_{conf,max1} - A_{noConf1}) / A_{conf,max1}) * (A_{conf,min1} / A_{conf,max1}), 0) = 0.33836562$
 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.
 $A_{conf,max1} = 89600.00$ 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.
 $A_{conf,min1} = 59225.00$ 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.
 $A_{noConf1} = 43733.333$ is the unconfined external core area which is equal to $b_i^2/6$ as defined at (A.2).
 $\alpha_{se2} (> \alpha_{se1}) = \text{Max}(((A_{conf,max2} - A_{noConf2}) / A_{conf,max2}) * (A_{conf,min2} / A_{conf,max2}), 0) = 0.33836562$
 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.
 $A_{conf,max2} = 45264.00$ 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.
 $A_{conf,min2} = 0.00$ 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.
 $A_{noConf2} = 32309.333$ is the unconfined internal core area which is equal to $b_i^2/6$ as defined at (A.2).
 $p_{sh,min} * F_{ywe} = \text{Min}(p_{sh,x} * F_{ywe}, p_{sh,y} * F_{ywe}) = 6.12205$
 Expression (5.4d) for $p_{sh,min} * F_{ywe}$ has been multiplied by 0.3 according to 15.7.1.3 for members without earthquake detailing (90° closed stirrups)

$p_{sh,x} * F_{ywe} = p_{sh1} * F_{ywe1} + p_{sh2} * F_{ywe2} = 6.12205$
 $p_{sh1} ((5.4d), TBDY) = L_{stir1} * A_{stir1} / (A_{sec} * s_1) = 0.00680678$
 L_{stir1} (Length of stirrups along Y) = 1040.00
 A_{stir1} (stirrups area) = 78.53982
 $p_{sh2} (5.4d) = L_{stir2} * A_{stir2} / (A_{sec} * s_2) = 0.0012868$
 L_{stir2} (Length of stirrups along Y) = 768.00
 A_{stir2} (stirrups area) = 50.26548

$p_{sh,y} * F_{ywe} = p_{sh1} * F_{ywe1} + p_{sh2} * F_{ywe2} = 6.12205$
 $p_{sh1} ((5.4d), TBDY) = L_{stir1} * A_{stir1} / (A_{sec} * s_1) = 0.00680678$
 L_{stir1} (Length of stirrups along X) = 1040.00
 A_{stir1} (stirrups area) = 78.53982
 $p_{sh2} ((5.4d), TBDY) = L_{stir2} * A_{stir2} / (A_{sec} * s_2) = 0.0012868$
 L_{stir2} (Length of stirrups along X) = 768.00
 A_{stir2} (stirrups area) = 50.26548

$A_{sec} = 120000.00$
 $s_1 = 100.00$
 $s_2 = 250.00$
 $f_{ywe1} = 781.25$
 $f_{ywe2} = 625.00$
 $f_{ce} = 30.00$
 From ((5A.5), TBDY), TBDY: $\alpha_c = 0.002$
 α_c = confinement factor = 1.00
 $y_1 = 0.0025$
 $sh_1 = 0.008$
 $ft_1 = 886.7403$
 $fy_1 = 738.9503$
 $su_1 = 0.032$

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.
 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 $fs_1 = (fs_{jacket} * A_{s,ten,jacket} + fs_{core} * A_{s,ten,core}) / A_{s,ten} = 738.9503$
 with $Es_1 = (Es_{jacket} * A_{s,ten,jacket} + Es_{core} * A_{s,ten,core}) / A_{s,ten} = 200000.00$
 $y_2 = 0.0025$
 $sh_2 = 0.008$
 $ft_2 = 898.293$
 $fy_2 = 748.5775$
 $su_2 = 0.032$
 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 $fs_2 = (fs_{jacket} * A_{s,com,jacket} + fs_{core} * A_{s,com,core}) / A_{s,com} = 748.5775$
 with $Es_2 = (Es_{jacket} * A_{s,com,jacket} + Es_{core} * A_{s,com,core}) / A_{s,com} = 200000.00$
 $y_v = 0.0025$
 $sh_v = 0.008$
 $ft_v = 894.3662$
 $fy_v = 745.3052$
 $suv = 0.032$
 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 $fsv = (fs_{jacket} * A_{s,mid,jacket} + fs_{mid} * A_{s,mid,core}) / A_{s,mid} = 745.3052$
 with $Es_v = (Es_{jacket} * A_{s,mid,jacket} + Es_{mid} * A_{s,mid,core}) / A_{s,mid} = 200000.00$
 $1 = A_{s,ten} / (b * d) * (fs_1 / fc) = 0.19082108$
 $2 = A_{s,com} / (b * d) * (fs_2 / fc) = 0.37540035$
 $v = A_{s,mid} / (b * d) * (fsv / fc) = 0.33973286$
 and confined core properties:
 $b = 360.00$
 $d = 347.00$
 $d' = 13.00$
 $f_{cc} (5A.2, TBDY) = 30.00$
 $cc (5A.5, TBDY) = 0.002$
 $c = \text{confinement factor} = 1.00$
 $1 = A_{s,ten} / (b * d) * (fs_1 / fc) = 0.22424379$
 $2 = A_{s,com} / (b * d) * (fs_2 / fc) = 0.44115251$
 $v = A_{s,mid} / (b * d) * (fsv / fc) = 0.39923778$
 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) = 0.42533033
 M_{Rc} (4.17) = 7.2277E+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: 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
 --->
 ϵ^*_{cu} (4.11) = 0.50069522
 M_{Ro} (4.18) = 5.8153E+008
 --->
 $u = \epsilon_{cu}$ (4.2) = 1.1511372E-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 ϵ_u according to 4.1, Biskinis/Fardis 2013:

$u = 3.0526570E-005$

$M_u = 4.9554E+008$

with full section properties:

$b = 200.00$

$d = 367.00$

$d' = 33.00$

$v = 0.97992777$

$N = 2.1578E+006$

$f_c = 30.00$

ϵ_{co} (5A.5, TBDY) = 0.002

Final value of ϵ_{cu} : $\epsilon_{cu}^* = \text{shear_factor} * \text{Max}(\epsilon_{cu}, \epsilon_{cc}) = 0.00925707$

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

From (5.4b), TBDY: $\epsilon_{cu} = 0.00925707$

we (5.4c) = 0.0207149

ϵ_{ase} ((5.4d), TBDY) = $(\epsilon_{ase1} * A_{ext} + \epsilon_{ase2} * A_{int}) / A_{sec} = 0.33836562$

$\epsilon_{ase1} = \text{Max}(((A_{conf,max1} - A_{noConf1}) / A_{conf,max1}) * (A_{conf,min1} / A_{conf,max1}), 0) = 0.33836562$

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.

$A_{conf,max1} = 89600.00$ 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.
 $A_{conf,min1} = 59225.00$ 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.

$A_{noConf1} = 43733.333$ is the unconfined external core area which is equal to $b_1^2/6$ as defined at (A.2).
 $a_{se2} (>= a_{se1}) = \text{Max}(((A_{conf,max2} - A_{noConf2})/A_{conf,max2}) * (A_{conf,min2}/A_{conf,max2}), 0) = 0.33836562$

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.

$A_{conf,max2} = 45264.00$ 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.

$A_{conf,min2} = 0.00$ 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.

$A_{noConf2} = 32309.333$ is the unconfined internal core area which is equal to $b_1^2/6$ as defined at (A.2).
 $p_{sh,min} * F_{ywe} = \text{Min}(p_{sh,x} * F_{ywe}, p_{sh,y} * F_{ywe}) = 6.12205$

Expression (5.4d) for $p_{sh,min} * F_{ywe}$ has been multiplied by 0.3 according to 15.7.1.3 for members without earthquake detailing (90° closed stirrups)

 $p_{sh,x} * F_{ywe} = p_{sh1} * F_{ywe1} + p_{sh2} * F_{ywe2} = 6.12205$
 $p_{sh1} \text{ ((5.4d), TBDY)} = L_{stir1} * A_{stir1} / (A_{sec} * s_1) = 0.00680678$
 L_{stir1} (Length of stirrups along Y) = 1040.00
 A_{stir1} (stirrups area) = 78.53982
 $p_{sh2} \text{ (5.4d)} = L_{stir2} * A_{stir2} / (A_{sec} * s_2) = 0.0012868$
 L_{stir2} (Length of stirrups along Y) = 768.00
 A_{stir2} (stirrups area) = 50.26548

 $p_{sh,y} * F_{ywe} = p_{sh1} * F_{ywe1} + p_{sh2} * F_{ywe2} = 6.12205$
 $p_{sh1} \text{ ((5.4d), TBDY)} = L_{stir1} * A_{stir1} / (A_{sec} * s_1) = 0.00680678$
 L_{stir1} (Length of stirrups along X) = 1040.00
 A_{stir1} (stirrups area) = 78.53982
 $p_{sh2} \text{ ((5.4d), TBDY)} = L_{stir2} * A_{stir2} / (A_{sec} * s_2) = 0.0012868$
 L_{stir2} (Length of stirrups along X) = 768.00
 A_{stir2} (stirrups area) = 50.26548

 $A_{sec} = 120000.00$

$s_1 = 100.00$

$s_2 = 250.00$

$f_{ywe1} = 781.25$

$f_{ywe2} = 625.00$

$f_{ce} = 30.00$

From ((5.A5), TBDY), TBDY: $cc = 0.002$

c = confinement factor = 1.00

$y_1 = 0.0025$

$sh_1 = 0.008$

$ft_1 = 898.293$

$fy_1 = 748.5775$

$su_1 = 0.032$

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

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

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

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 $fs_1 = (f_{s,jacket} * A_{s,ten,jacket} + f_{s,core} * A_{s,ten,core}) / A_{s,ten} = 748.5775$

with $Es_1 = (E_{s,jacket} * A_{s,ten,jacket} + E_{s,core} * A_{s,ten,core}) / A_{s,ten} = 200000.00$

$y_2 = 0.0025$

$sh_2 = 0.008$

$ft_2 = 886.7403$


```

fy2 = 738.9503
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 = 738.9503
    with Es2 = (Es,jacket*Asl,com,jacket + Es,core*Asl,com,core)/Asl,com = 200000.00
yv = 0.0025
shv = 0.008
ftv = 894.3662
fyv = 745.3052
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 = 745.3052
    with Esv = (Es,jacket*Asl,mid,jacket + Es,mid*Asl,mid,core)/Asl,mid = 200000.00
1 = Asl,ten/(b*d)*(fs1/fc) = 0.7508007
2 = Asl,com/(b*d)*(fs2/fc) = 0.38164216
v = Asl,mid/(b*d)*(fsv/fc) = 0.67946571
and confined core properties:
b = 160.00
d = 347.00
d' = 13.00
fcc (5A.2, TBDY) = 30.00
cc (5A.5, TBDY) = 0.002
    c = confinement factor = 1.00
    1 = Asl,ten/(b*d)*(fs1/fc) = 0.99259314
    2 = Asl,com/(b*d)*(fs2/fc) = 0.50454853
    v = Asl,mid/(b*d)*(fsv/fc) = 0.898285
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) = 0.8262842
MRc (4.18) = 4.9554E+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) = 0.90160009
MRo (4.18) = 2.4685E+008
MRo < 0.8*MRc
--->
u = cu (unconfined full section) = 3.0526570E-005
Mu = MRc
-----

Calculation of ratio lb/d
-----
Adequate Lap Length: lb/d >= 1
-----
-----
-----
Calculation of Shear Strength Vr = Min(Vr1,Vr2) = 349917.706
-----
Calculation of Shear Strength at edge 1, Vr1 = 349917.706
Vr1 = VCol ((10.3), ASCE 41-17) = knl*VColO
VColO = 349917.706
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 = 25.00, but fc'^0.5 <= 8.3
MPa (22.5.3.1, ACI 318-14)
M/Vd = 4.00
Mu = 8.5591E+007
Vu = 5.7948283E-005
d = 0.8*h = 320.00
Nu = 2.1578E+006
Ag = 80000.00
From (11.5.4.8), ACI 318-14: Vs = Vs,jacket + Vs,core = 471238.898
where:
Vs,jacket = Vs,j1 + Vs,j2 = 471238.898
Vs,j1 = 157079.633 is calculated for section web jacket, with:
d = 160.00
Av = 157079.633
fy = 625.00
s = 100.00
Vs,j1 is multiplied by Col,j1 = 1.00
s/d = 0.625
Vs,j2 = 314159.265 is calculated for section flange jacket, with:
d = 320.00
Av = 157079.633
fy = 625.00
s = 100.00
Vs,j2 is multiplied by Col,j2 = 1.00
s/d = 0.3125
Vs,core = Vs,c1 + Vs,c2 = 0.00

```

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

$$d = 80.00$$

$$A_v = 100530.965$$

$$f_y = 500.00$$

$$s = 250.00$$

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

$$s/d = 3.125$$

$V_{s,c2} = 0.00$ is calculated for section flange core, with:

$$d = 240.00$$

$$A_v = 100530.965$$

$$f_y = 500.00$$

$$s = 250.00$$

$V_{s,c2}$ is multiplied by $Col_{c2} = 0.00$

$$s/d = 1.04167$$

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

From (11-11), ACI 440: $V_s + V_f \leq 212577.225$

$$bw = 200.00$$

Calculation of Shear Strength at edge 2, $V_{r2} = 349917.706$

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

$$V_{Col0} = 349917.706$$

$knl = 1$ (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: $fc' = (fc'_{jacket} * Area_{jacket} + fc'_{core} * Area_{core}) / Area_{section} = 25.00$, but $fc'^{0.5} \leq 8.3$ MPa (22.5.3.1, ACI 318-14)

$$M/Vd = 4.00$$

$$\mu_u = 6.4291E+007$$

$$V_u = 5.7948283E-005$$

$$d = 0.8 * h = 320.00$$

$$N_u = 2.1578E+006$$

$$A_g = 80000.00$$

From (11.5.4.8), ACI 318-14: $V_s = V_{s,jacket} + V_{s,core} = 471238.898$

where:

$$V_{s,jacket} = V_{s,j1} + V_{s,j2} = 471238.898$$

$V_{s,j1} = 157079.633$ is calculated for section web jacket, with:

$$d = 160.00$$

$$A_v = 157079.633$$

$$f_y = 625.00$$

$$s = 100.00$$

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

$$s/d = 0.625$$

$V_{s,j2} = 314159.265$ is calculated for section flange jacket, with:

$$d = 320.00$$

$$A_v = 157079.633$$

$$f_y = 625.00$$

$$s = 100.00$$

$V_{s,j2}$ is multiplied by $Col_{j2} = 1.00$

$$s/d = 0.3125$$

$$V_{s,core} = V_{s,c1} + V_{s,c2} = 0.00$$

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

$$d = 80.00$$

$$A_v = 100530.965$$

$$f_y = 500.00$$

$$s = 250.00$$

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

$$s/d = 3.125$$

$V_{s,c2} = 0.00$ is calculated for section flange core, with:

$$d = 240.00$$

$$A_v = 100530.965$$

$$f_y = 500.00$$

$$s = 250.00$$

$V_{s,c2}$ is multiplied by $Col_{c2} = 0.00$

s/d = 1.04167
Vf ((11-3)-(11.4), ACI 440) = 0.00
From (11-11), ACI 440: Vs + Vf <= 212577.225
bw = 200.00

End Of Calculation of Shear Capacity ratio for element: column JLC1 of floor 1
At local axis: 2

Start Of Calculation of Chord Rotation Capacity for element: column JLC1 of floor 1
At local axis: 2
Integration Section: (a)
Section Type: rcjls

Constant Properties

Knowledge Factor, $\phi = 0.90$
Chord Rotation is generally considered as Deformation-Controlled Action according to Table C7-1, ASCE 41-17.
Mean strengths are used for Deformation-Controlled Actions according to 7.5.1.3, ASCE 41-17
Consequently:
Jacket
New material of Secondary Member: Concrete Strength, $f_c = f_{cm} = 30.00$
New material of Secondary Member: Steel Strength, $f_s = f_{sm} = 625.00$
Concrete Elasticity, $E_c = 25742.96$
Steel Elasticity, $E_s = 200000.00$
Existing Column
Existing material of Secondary Member: Concrete Strength, $f_c = f_{cm} = 18.00$
Existing material of Secondary Member: Steel Strength, $f_s = f_{sm} = 500.00$
Concrete Elasticity, $E_c = 19940.411$
Steel Elasticity, $E_s = 200000.00$
Max Height, $H_{max} = 400.00$
Min Height, $H_{min} = 200.00$
Max Width, $W_{max} = 400.00$
Min Width, $W_{min} = 200.00$
Jacket Thickness, $t_j = 50.00$
Cover Thickness, $c = 15.00$
Element Length, $L = 3000.00$
Secondary Member
Smooth Bars
Ductile Steel
Without Detailing for Earthquake Resistance (including stirrups not 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, $M = -3.4089E+006$
Shear Force, $V_2 = -0.60572401$
Shear Force, $V_3 = -1.4589530E-005$
Axial Force, $F = -2.1578E+006$
Longitudinal Reinforcement Area Distribution (in 2 divisions)
-Tension: $As_t = 0.00$
-Compression: $As_c = 5353.274$
Longitudinal Reinforcement Area Distribution (in 3 divisions)
-Tension: $As_{t,ten} = 2208.54$
-Compression: $As_{l,com} = 1137.257$
-Middle: $As_{l,mid} = 2007.478$
Longitudinal External Reinforcement Area Distribution (in 3 divisions)
-Tension: $As_{l,ten,jacket} = 1746.726$
-Compression: $As_{l,com,jacket} = 829.3805$
-Middle: $As_{l,mid,jacket} = 1545.664$
Longitudinal Internal Reinforcement Area Distribution (in 3 divisions)

-Tension: $Asl_{ten,core} = 461.8141$
-Compression: $Asl_{com,core} = 307.8761$
-Middle: $Asl_{mid,core} = 461.8141$

Mean Diameter of Tension Reinforcement, $DbL = 16.60$

Existing component: From table 7-7, ASCE 41_17: Final chord rotation Capacity $u_R = u = 0.03721297$
 $u = y + p = 0.04134775$

- Calculation of y -

$y = (My * Ls / 3) / Eleff = 0.04134775$ ((4.29), Biskinis Phd))
 $My = 5.1584E+008$
 $Ls = M/V$ (with $Ls > 0.1 * L$ and $Ls < 2 * L$) = 6000.00
From table 10.5, ASCE 41_17: $Eleff = factor * Ec * Ig = 2.4951E+013$
 $factor = 0.70$
 $Ag = 120000.00$
Mean concrete strength: $fc' = (fc'_{jacket} * Area_{jacket} + fc'_{core} * Area_{core}) / Area_{section} = 25.00$
 $N = 2.1578E+006$
 $Ec * Ig = Ec_{jacket} * Ig_{jacket} + Ec_{core} * Ig_{core} = 3.5645E+013$

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, $b = 400.00$
web width, $bw = 200.00$
flange thickness, $t = 200.00$

$y = \text{Min}(y_{ten}, y_{com})$
 $y_{ten} = 1.6571499E-005$
with $fy = 596.2441$
 $d = 367.00$
 $y = 0.50980796$
 $A = 0.19238222$
 $B = 0.11480474$
with $pt = 0.01504455$
 $pc = 0.00774698$
 $pv = 0.01367492$
 $N = 2.1578E+006$
 $b = 400.00$
 $" = 0.08991826$
 $y_{comp} = 9.6849368E-006$
with $fc = 30.00$
 $Ec = 25742.96$
 $y = 0.5901636$
 $A = 0.0730043$
 $B = 0.06549972$
with $Es = 200000.00$
CONFIRMATION: $y = 0.5901636 > t/d$

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 $V_y E / V_{ColOE} = 1.37616$

$d = d_{external} = 367.00$

$s = s_{external} = 100.00$

- $t = s_1 + s_2 + 2 * t_f / b_w * (f_{fe} / f_s) = 0.00809358$

jacket: $s_1 = A_{v1} * L_{stir1} / (s_1 * A_g) = 0.00680678$

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

$L_{stir1} = 1040.00$, is the total Length of all stirrups parallel to loading (shear) direction

$s_1 = 100.00$

core: $s_2 = A_{v2} * L_{stir2} / (s_2 * A_g) = 0.0012868$

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

$L_{stir2} = 768.00$, 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.

$NUD = 2.1578E+006$

$A_g = 120000.00$

$f_{cE} = (f_{c,jacket} * Area_{jacket} + f_{c,core} * Area_{core}) / section_area = 25.00$

$f_{yIE} = (f_{y,ext_Long_Reinf} * Area_{ext_Long_Reinf} + f_{y,int_Long_Reinf} * Area_{int_Long_Reinf}) / Area_{Tot_Long_Rein} = 596.2441$

$f_{yIE} = (f_{y,ext_Trans_Reinf} * s_1 + f_{y,int_Trans_Reinf} * s_2) / (s_1 + s_2) = 605.1263$

$\rho_l = Area_{Tot_Long_Rein} / (b * d) = 0.03646644$

$b = 400.00$

$d = 367.00$

$f_{cE} = 25.00$

End Of Calculation of Chord Rotation Capacity for element: column JLC1 of floor 1

At local axis: 2

Integration Section: (a)

Calculation No. 11

column C1, Floor 1

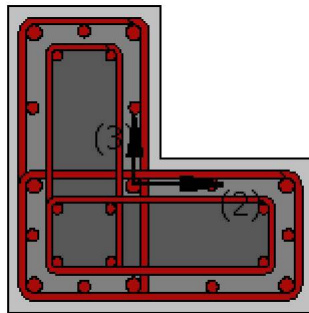
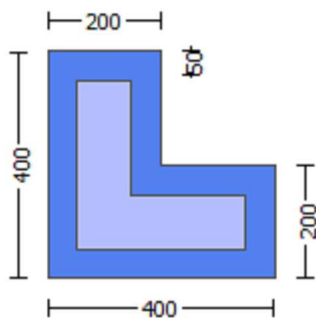
Limit State: Life Safety (data interpolation between analysis steps 1 and 2)

Analysis: Uniform +X

Check: Shear capacity V_{Rd}

Edge: Start

Local Axis: (3)



Start Of Calculation of Shear Capacity for element: column JLC1 of floor 1

At local axis: 3

Integration Section: (a)

Section Type: rcjics

Constant Properties

Knowledge Factor, $\gamma = 0.90$

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

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

Consequently:

Jacket

New material of Secondary Member: Concrete Strength, $f_c = f_{c_lower_bound} = 20.00$

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

Concrete Elasticity, $E_c = 25742.96$

Steel Elasticity, $E_s = 200000.00$

Existing Column

Existing material of Secondary Member: Concrete Strength, $f_c = f_{c_lower_bound} = 12.00$

Existing material of Secondary Member: Steel Strength, $f_s = f_{s_lower_bound} = 400.00$

Concrete Elasticity, $E_c = 19940.411$

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, $f_c = f_{cm} = 30.00$

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

Existing Column

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

Existing material: Steel Strength, $f_s = f_{sm} = 500.00$

#####

Max Height, $H_{max} = 400.00$

Min Height, $H_{min} = 200.00$

Max Width, $W_{max} = 400.00$

Min Width, $W_{min} = 200.00$

Jacket Thickness, $t_j = 50.00$

Cover Thickness, $c = 15.00$

Element Length, $L = 3000.00$

Secondary Member

Smooth Bars

Ductile Steel

Without Detailing for Earthquake Resistance (including stirrups not 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, $M_a = -3.4089E+006$

Shear Force, $V_a = -1.4589530E-005$

EDGE -B-

Bending Moment, $M_b = 15336.81$

Shear Force, $V_b = 1.4589530E-005$

BOTH EDGES

Axial Force, $F = -2.1578E+006$

Longitudinal Reinforcement Area Distribution (in 2 divisions)

-Tension: $A_{st} = 0.00$

-Compression: $A_{sc} = 5353.274$

Longitudinal Reinforcement Area Distribution (in 3 divisions)

-Tension: $A_{st,ten} = 2208.54$

-Compression: $A_{sc,com} = 1137.257$

-Middle: $A_{st,mid} = 2007.478$

Mean Diameter of Tension Reinforcement, $Db_{L,ten} = 16.60$

Existing component: From table 7-7, ASCE 41_17: Final Shear Capacity $V_R = \phi V_n = 267030.06$

V_n ((10.3), ASCE 41-17) = $k_n \phi V_{CoIO} = 296700.066$

$V_{CoI} = 296700.066$

$k_n = 1.00$

$displacement_ductility_demand = 0.01268458$

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

= 1 (normal-weight concrete)

Mean concrete strength: $f_c' = (f_c'_{jacket} \cdot Area_{jacket} + f_c'_{core} \cdot Area_{core}) / Area_{section} = 16.66667$, but $f_c'^{0.5} \leq 8.3$ MPa (22.5.3.1, ACI 318-14)

$M/Vd = 4.00$

$M_u = 3.4089E+006$

$V_u = 1.4589530E-005$

$d = 0.8 \cdot h = 320.00$

$N_u = 2.1578E+006$

$A_g = 80000.00$

From (11.5.4.8), ACI 318-14: $V_s = V_{s,jacket} + V_{s,core} = 376991.118$

where:

$V_{s,jacket} = V_{s,j1} + V_{s,j2} = 376991.118$

$V_{s,j1} = 251327.412$ is calculated for section web jacket, with:

$d = 320.00$

$A_v = 157079.633$

$f_y = 500.00$

$s = 100.00$

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

$s/d = 0.3125$

$V_{s,j2} = 125663.706$ is calculated for section flange jacket, with:

$d = 160.00$

$A_v = 157079.633$

$f_y = 500.00$

$s = 100.00$

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

$s/d = 0.625$

$V_{s,core} = V_{s,c1} + V_{s,c2} = 0.00$

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

$d = 240.00$

$A_v = 100530.965$

$f_y = 400.00$

$s = 250.00$

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

$s/d = 1.04167$

$V_{s,c2} = 0.00$ is calculated for section flange core, with:

$d = 80.00$

$A_v = 100530.965$

$f_y = 400.00$

$s = 250.00$

$V_{s,c2}$ is multiplied by $Col,c2 = 0.00$

$s/d = 3.125$
 $V_f ((11-3)-(11.4), \text{ACI } 440) = 0.00$
 From $(11-11), \text{ACI } 440: V_s + V_f \leq 173568.578$
 $b_w = 200.00$

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 $\theta = 0.00052448$
 $y = (M_y * L_s / 3) / E_{eff} = 0.04134775 ((4.29), \text{Biskinis Phd})$
 $M_y = 5.1584E+008$
 $L_s = M/V$ (with $L_s > 0.1 * L$ and $L_s < 2 * L$) = 6000.00
 From table 10.5, ASCE 41_17: $E_{eff} = \text{factor} * E_c * I_g = 2.4951E+013$
 factor = 0.70
 $A_g = 120000.00$
 Mean concrete strength: $f'_c = (f'_c_{\text{jacket}} * \text{Area}_{\text{jacket}} + f'_c_{\text{core}} * \text{Area}_{\text{core}}) / \text{Area}_{\text{section}} = 25.00$
 $N = 2.1578E+006$
 $E_c * I_g = E_{c_{\text{jacket}}} * I_{g_{\text{jacket}}} + E_{c_{\text{core}}} * I_{g_{\text{core}}} = 3.5645E+013$

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, $b = 400.00$
 web width, $b_w = 200.00$
 flange thickness, $t = 200.00$

$y = \text{Min}(y_{\text{ten}}, y_{\text{com}})$
 $y_{\text{ten}} = 1.6571499E-005$
 with $f_y = 596.2441$
 $d = 367.00$
 $y = 0.50980796$
 $A = 0.19238222$
 $B = 0.11480474$
 with $p_t = 0.01504455$
 $p_c = 0.00774698$
 $p_v = 0.01367492$
 $N = 2.1578E+006$
 $b = 400.00$
 $\lambda = 0.08991826$
 $y_{\text{comp}} = 9.6849368E-006$
 with $f_c = 30.00$
 $E_c = 25742.96$
 $y = 0.5901636$
 $A = 0.0730043$
 $B = 0.06549972$
 with $E_s = 200000.00$
 CONFIRMATION: $y = 0.5901636 > t/d$

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 JLC1 of floor 1
 At local axis: 3
 Integration Section: (a)

Calculation No. 12

column C1, Floor 1

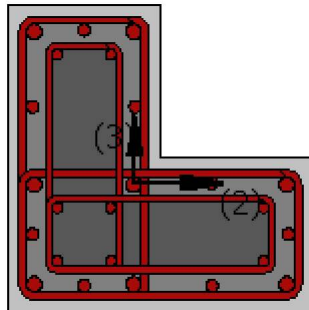
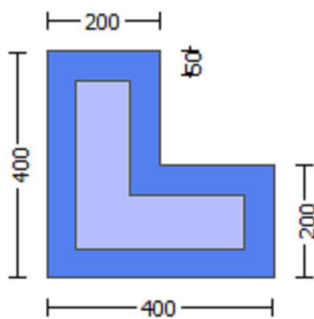
Limit State: Life Safety (data interpolation between analysis steps 1 and 2)

Analysis: Uniform +X

Check: Chord rotation capacity (θ)

Edge: Start

Local Axis: (3)



Start Of Calculation of Shear Capacity ratio for element: column JLC1 of floor 1

At Shear local axis: 3

(Bending local axis: 2)

Section Type: rcjlc

Constant Properties

Knowledge Factor, $\phi = 0.90$

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

Consequently:

Jacket

New material of Secondary Member: Concrete Strength, $f_c = f_{cm} = 30.00$

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

Concrete Elasticity, $E_c = 25742.96$

Steel Elasticity, $E_s = 200000.00$

Existing Column

Existing material of Secondary Member: Concrete Strength, $f_c = f_{cm} = 18.00$

Existing material of Secondary Member: Steel Strength, $f_s = f_{sm} = 500.00$

Concrete Elasticity, $E_c = 19940.411$

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, $f_s = 1.25 \cdot f_{sm} = 625.00$

#####

Max Height, $H_{max} = 400.00$

Min Height, $H_{min} = 200.00$
 Max Width, $W_{max} = 400.00$
 Min Width, $W_{min} = 200.00$
 Jacket Thickness, $t_j = 50.00$
 Cover Thickness, $c = 15.00$
 Mean Confinement Factor overall section = 1.00
 Element Length, $L = 3000.00$
 Secondary Member
 Smooth Bars
 Ductile Steel
 Without Detailing for Earthquake Resistance (including stirrups not 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, $V_a = -1.4594946E-005$
 EDGE -B-
 Shear Force, $V_b = 1.4594946E-005$
 BOTH EDGES
 Axial Force, $F = -2.1578E+006$
 Longitudinal Reinforcement Area Distribution (in 2 divisions)
 -Tension: $As_t = 0.00$
 -Compression: $As_c = 5353.274$
 Longitudinal Reinforcement Area Distribution (in 3 divisions)
 -Tension: $As_{t,ten} = 2208.54$
 -Compression: $As_{l,com} = 1137.257$
 -Middle: $As_{l,mid} = 2007.478$

Calculation of Shear Capacity ratio, $V_e/V_r = 1.37616$
 Member Controlled by Shear ($V_e/V_r > 1$)
 Calculation of Shear Demand from fig. R18.6.5, ACI 318-14 $V_e = (M_{pr1} + M_{pr2})/l_n = 481543.08$
 with
 $M_{pr1} = \max(\mu_{u1+}, \mu_{u1-}) = 7.2231E+008$
 $\mu_{u1+} = 7.2231E+008$, is the ultimate moment strength at the edge 1 of the member in the actual moment direction which is defined for the static loading combination
 $\mu_{u1-} = 6.3496E+008$, 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
 $M_{pr2} = \max(\mu_{u2+}, \mu_{u2-}) = 7.2231E+008$
 $\mu_{u2+} = 7.2231E+008$, 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
 $\mu_{u2-} = 6.3496E+008$, 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:
 $\phi_u = 3.9059244E-005$
 $\mu_u = 7.2231E+008$

with full section properties:

$b = 400.00$
 $d = 367.00$
 $d' = 33.00$
 $\nu = 0.48996389$
 $N = 2.1578E+006$
 $f_c = 30.00$

co (5A.5, TBDY) = 0.002
 Final value of cu : $cu^* = shear_factor * Max(cu, cc) = 0.00925707$
 The Shear_factor is considered equal to 1 (pure moment strength)
 From (5.4b), TBDY: $cu = 0.00925707$
 we (5.4c) = 0.0207149
 ase ((5.4d), TBDY) = $(ase1 * A_{ext} + ase2 * A_{int}) / A_{sec} = 0.33836562$
 $ase1 = Max(((A_{conf,max1} - A_{noConf1}) / A_{conf,max1}) * (A_{conf,min1} / A_{conf,max1}), 0) = 0.33836562$
 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.
 $A_{conf,max1} = 89600.00$ 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.
 $A_{conf,min1} = 59225.00$ 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.
 $A_{noConf1} = 43733.333$ is the unconfined external core area which is equal to $b_i^2/6$ as defined at (A.2).
 $ase2 (>= ase1) = Max(((A_{conf,max2} - A_{noConf2}) / A_{conf,max2}) * (A_{conf,min2} / A_{conf,max2}), 0) = 0.33836562$
 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.
 $A_{conf,max2} = 45264.00$ 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.
 $A_{conf,min2} = 0.00$ 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.
 $A_{noConf2} = 32309.333$ is the unconfined internal core area which is equal to $b_i^2/6$ as defined at (A.2).
 $psh_{min} * F_{ywe} = Min(psh_x * F_{ywe}, psh_y * F_{ywe}) = 6.12205$
 Expression (5.4d) for $psh_{min} * F_{ywe}$ has been multiplied by 0.3 according to 15.7.1.3 for members without earthquake detailing (90° closed stirrups)

$psh_x * F_{ywe} = psh1 * F_{ywe1} + ps2 * F_{ywe2} = 6.12205$
 $psh1$ ((5.4d), TBDY) = $L_{stir1} * A_{stir1} / (A_{sec} * s1) = 0.00680678$
 L_{stir1} (Length of stirrups along Y) = 1040.00
 A_{stir1} (stirrups area) = 78.53982
 $psh2$ (5.4d) = $L_{stir2} * A_{stir2} / (A_{sec} * s2) = 0.0012868$
 L_{stir2} (Length of stirrups along Y) = 768.00
 A_{stir2} (stirrups area) = 50.26548

$psh_y * F_{ywe} = psh1 * F_{ywe1} + ps2 * F_{ywe2} = 6.12205$
 $psh1$ ((5.4d), TBDY) = $L_{stir1} * A_{stir1} / (A_{sec} * s1) = 0.00680678$
 L_{stir1} (Length of stirrups along X) = 1040.00
 A_{stir1} (stirrups area) = 78.53982
 $psh2$ ((5.4d), TBDY) = $L_{stir2} * A_{stir2} / (A_{sec} * s2) = 0.0012868$
 L_{stir2} (Length of stirrups along X) = 768.00
 A_{stir2} (stirrups area) = 50.26548

$A_{sec} = 120000.00$
 $s1 = 100.00$
 $s2 = 250.00$
 $f_{ywe1} = 781.25$
 $f_{ywe2} = 625.00$
 $f_{ce} = 30.00$

From ((5.A5), TBDY), TBDY: $cc = 0.002$
 c = confinement factor = 1.00

$y1 = 0.0025$
 $sh1 = 0.008$
 $ft1 = 898.293$
 $fy1 = 748.5775$
 $su1 = 0.032$
 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
 $lo/lo_{u,min} = lb/d = 1.00$

```

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 = 748.5775
with Es1 = (Es,jacket*Asl,ten,jacket + Es,core*Asl,ten,core)/Asl,ten = 200000.00
y2 = 0.0025
sh2 = 0.008
ft2 = 886.7403
fy2 = 738.9503
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 = 738.9503
with Es2 = (Es,jacket*Asl,com,jacket + Es,core*Asl,com,core)/Asl,com = 200000.00
yv = 0.0025
shv = 0.008
ftv = 894.3662
fyv = 745.3052
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 = 745.3052
with Esv = (Es,jacket*Asl,mid,jacket + Es,mid*Asl,mid,core)/Asl,mid = 200000.00
1 = Asl,ten/(b*d)*(fs1/fc) = 0.37540035
2 = Asl,com/(b*d)*(fs2/fc) = 0.19082108
v = Asl,mid/(b*d)*(fsv/fc) = 0.33973286
and confined core properties:
b = 360.00
d = 347.00
d' = 13.00
fcc (5A.2, TBDY) = 30.00
cc (5A.5, TBDY) = 0.002
c = confinement factor = 1.00
1 = Asl,ten/(b*d)*(fs1/fc) = 0.44115251
2 = Asl,com/(b*d)*(fs2/fc) = 0.22424379
v = Asl,mid/(b*d)*(fsv/fc) = 0.39923778
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) = 0.64577857

M_{Rc} (4.17) = 7.2231E+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: 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) = 0.67946111

M_{Ro} (4.18) = 4.2866E+008

$M_{Ro} < 0.8*M_{Rc}$

---->

$u = c_u$ (unconfined full section) = 3.9059244E-005

$\mu = M_{Rc}$

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:

$u = 4.2871818E-005$

$\mu = 6.3496E+008$

with full section properties:

$b = 200.00$

$d = 367.00$

$d' = 33.00$

$v = 0.97992777$

$N = 2.1578E+006$

$f_c = 30.00$

ϵ_{co} (5A.5, TBDY) = 0.002

Final value of c_u : $c_u^* = \text{shear_factor} * \text{Max}(c_u, \epsilon_{cc}) = 0.00925707$

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

From (5.4b), TBDY: $c_u = 0.00925707$

we (5.4c) = 0.0207149

a_{se} ((5.4d), TBDY) = $(a_{se1}*A_{ext} + a_{se2}*A_{int})/A_{sec} = 0.33836562$

$a_{se1} = \text{Max}(((A_{conf,max1} - A_{noConf1})/A_{conf,max1}) * (A_{conf,min1}/A_{conf,max1}), 0) = 0.33836562$

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.

$A_{conf,max1} = 89600.00$ 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.
 $A_{conf,min1} = 59225.00$ 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.
 $A_{noConf1} = 43733.333$ is the unconfined external core area which is equal to $b_i^2/6$ as defined at (A.2).
 $ase2 (>=ase1) = \text{Max}(((A_{conf,max2}-A_{noConf2})/A_{conf,max2})*(A_{conf,min2}/A_{conf,max2}),0) = 0.33836562$
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.
 $A_{conf,max2} = 45264.00$ 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.
 $A_{conf,min2} = 0.00$ 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.
 $A_{noConf2} = 32309.333$ is the unconfined internal core area which is equal to $b_i^2/6$ as defined at (A.2).
 $psh_{min}*F_{ywe} = \text{Min}(psh_x*F_{ywe}, psh_y*F_{ywe}) = 6.12205$
Expression (5.4d) for $psh_{min}*F_{ywe}$ has been multiplied by 0.3 according to 15.7.1.3 for members without
earthquake detailing (90° closed stirrups)

 $psh_x*F_{ywe} = psh1*F_{ywe1} + ps2*F_{ywe2} = 6.12205$
 $psh1 ((5.4d), TBDY) = L_{stir1}*A_{stir1}/(A_{sec}*s1) = 0.00680678$
 L_{stir1} (Length of stirrups along Y) = 1040.00
 A_{stir1} (stirrups area) = 78.53982
 $psh2 (5.4d) = L_{stir2}*A_{stir2}/(A_{sec}*s2) = 0.0012868$
 L_{stir2} (Length of stirrups along Y) = 768.00
 A_{stir2} (stirrups area) = 50.26548

 $psh_y*F_{ywe} = psh1*F_{ywe1} + ps2*F_{ywe2} = 6.12205$
 $psh1 ((5.4d), TBDY) = L_{stir1}*A_{stir1}/(A_{sec}*s1) = 0.00680678$
 L_{stir1} (Length of stirrups along X) = 1040.00
 A_{stir1} (stirrups area) = 78.53982
 $psh2 ((5.4d), TBDY) = L_{stir2}*A_{stir2}/(A_{sec}*s2) = 0.0012868$
 L_{stir2} (Length of stirrups along X) = 768.00
 A_{stir2} (stirrups area) = 50.26548

 $A_{sec} = 120000.00$
 $s1 = 100.00$
 $s2 = 250.00$
 $f_{ywe1} = 781.25$
 $f_{ywe2} = 625.00$
 $f_{ce} = 30.00$
From ((5.A5), TBDY), TBDY: $cc = 0.002$
 $c = \text{confinement factor} = 1.00$
 $y1 = 0.0025$
 $sh1 = 0.008$
 $ft1 = 886.7403$
 $fy1 = 738.9503$
 $su1 = 0.032$
using (30) in Biskinis/Fardis (2013) multiplied with shear_factor
and also multiplied by the shear_factor according to 15.7.1.4, with
 $\text{Shear_factor} = 1.00$
 $l_o/l_{ou,min} = l_b/l_d = 1.00$
 $su1 = 0.4*es_{u1_nominal} ((5.5), TBDY) = 0.032$
From table 5A.1, TBDY: $es_{u1_nominal} = 0.08$,
For calculation of $es_{u1_nominal}$ and $y1, sh1, ft1, fy1$, it is considered
characteristic value $fs_{y1} = fs1/1.2$, 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 $fs1 = (f_{s,jacket}*A_{sl,ten,jacket} + f_{s,core}*A_{sl,ten,core})/A_{sl,ten} = 738.9503$
with $Es1 = (E_{s,jacket}*A_{sl,ten,jacket} + E_{s,core}*A_{sl,ten,core})/A_{sl,ten} = 200000.00$
 $y2 = 0.0025$
 $sh2 = 0.008$
 $ft2 = 898.293$
 $fy2 = 748.5775$
 $su2 = 0.032$
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 $fs_2 = (fs_{jacket} * Asl_{com,jacket} + fs_{core} * Asl_{com,core}) / Asl_{com} = 748.5775$
with $Es_2 = (Es_{jacket} * Asl_{com,jacket} + Es_{core} * Asl_{com,core}) / Asl_{com} = 200000.00$
 $y_v = 0.0025$
 $sh_v = 0.008$
 $ft_v = 894.3662$
 $fy_v = 745.3052$
 $suv = 0.032$
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 $fsv = fs_v/1.2$, from table 5.1, TBDY
For calculation of $esuv_{nominal}$ and y_v, sh_v, ft_v, fy_v , it is considered
characteristic value $fsv = fs_v/1.2$, 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 $fs_v = (fs_{jacket} * Asl_{mid,jacket} + fs_{mid} * Asl_{mid,core}) / Asl_{mid} = 745.3052$
with $Es_v = (Es_{jacket} * Asl_{mid,jacket} + Es_{mid} * Asl_{mid,core}) / Asl_{mid} = 200000.00$
 $1 = Asl_{ten} / (b * d) * (fs_1 / fc) = 0.38164216$
 $2 = Asl_{com} / (b * d) * (fs_2 / fc) = 0.7508007$
 $v = Asl_{mid} / (b * d) * (fs_v / fc) = 0.67946571$
and confined core properties:
 $b = 160.00$
 $d = 347.00$
 $d' = 13.00$
 fcc (5A.2, TBDY) = 30.00
 cc (5A.5, TBDY) = 0.002
 c = confinement factor = 1.00
 $1 = Asl_{ten} / (b * d) * (fs_1 / fc) = 0.50454853$
 $2 = Asl_{com} / (b * d) * (fs_2 / fc) = 0.99259314$
 $v = Asl_{mid} / (b * d) * (fs_v / fc) = 0.898285$
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) = 0.58834972
 M_{Rc} (4.17) = 6.3496E+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) = 0.63603191

MRO (4.18) = 4.4768E+008

MRO < 0.8*MRc

---->

u = cu (unconfined full section) = 4.2871818E-005

Mu = MRc

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 = 3.9059244E-005

Mu = 7.2231E+008

with full section properties:

b = 400.00

d = 367.00

d' = 33.00

v = 0.48996389

N = 2.1578E+006

fc = 30.00

co (5A.5, TBDY) = 0.002

Final value of cu: cu* = shear_factor * Max(cu, cc) = 0.00925707

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

From (5.4b), TBDY: cu = 0.00925707

we (5.4c) = 0.0207149

ase ((5.4d), TBDY) = (ase1*Aext+ase2*Aint)/Asec = 0.33836562

ase1 = Max(((Aconf,max1-AnoConf1)/Aconf,max1)*(Aconf,min1/Aconf,max1),0) = 0.33836562

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 = 89600.00 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 = 59225.00 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 = 43733.333 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) = 0.33836562

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 = 45264.00 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 = 0.00 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 = 32309.333 is the unconfined internal core area which is equal to $b_i^2/6$ as defined at (A.2).

psh,min*Fywe = Min(psh,x*Fywe , psh,y*Fywe) = 6.12205

Expression (5.4d) for psh,min*Fywe has been multiplied by 0.3 according to 15.7.1.3 for members without earthquake detailing (90° closed stirrups)

psh_x*Fywe = psh1*Fywe1+ps2*Fywe2 = 6.12205

psh1 ((5.4d), TBDY) = Lstir1*Astir1/(Asec*s1) = 0.00680678

Lstir1 (Length of stirrups along Y) = 1040.00

Astir1 (stirrups area) = 78.53982

psh2 (5.4d) = Lstir2*Astir2/(Asec*s2) = 0.0012868

Lstir2 (Length of stirrups along Y) = 768.00

Astir2 (stirrups area) = 50.26548

psh_y*Fywe = psh1*Fywe1+ps2*Fywe2 = 6.12205

psh1 ((5.4d), TBDY) = Lstir1*Astir1/(Asec*s1) = 0.00680678

Lstir1 (Length of stirrups along X) = 1040.00

Astir1 (stirrups area) = 78.53982

psh2 ((5.4d), TBDY) = Lstir2*Astir2/(Asec*s2) = 0.0012868

Lstir2 (Length of stirrups along X) = 768.00

Astir2 (stirrups area) = 50.26548

Asec = 120000.00

s1 = 100.00

s2 = 250.00

fywe1 = 781.25

fywe2 = 625.00

fce = 30.00

From ((5.A5), TBDY), TBDY: cc = 0.002

c = confinement factor = 1.00

y1 = 0.0025

sh1 = 0.008

ft1 = 898.293

fy1 = 748.5775

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 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 = 748.5775

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

y2 = 0.0025

sh2 = 0.008

ft2 = 886.7403

fy2 = 738.9503

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 = 738.9503

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

yv = 0.0025

shv = 0.008

ftv = 894.3662

```

fyv = 745.3052
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 = (fsjacket*Asl,mid,jacket + fs,mid*Asl,mid,core)/Asl,mid = 745.3052
    with Esv = (Es,jacket*Asl,mid,jacket + Es,mid*Asl,mid,core)/Asl,mid = 200000.00
    1 = Asl,ten/(b*d)*(fs1/fc) = 0.37540035
    2 = Asl,com/(b*d)*(fs2/fc) = 0.19082108
    v = Asl,mid/(b*d)*(fsv/fc) = 0.33973286
and confined core properties:
b = 360.00
d = 347.00
d' = 13.00
fcc (5A.2, TBDY) = 30.00
cc (5A.5, TBDY) = 0.002
    c = confinement factor = 1.00
    1 = Asl,ten/(b*d)*(fs1/fc) = 0.44115251
    2 = Asl,com/(b*d)*(fs2/fc) = 0.22424379
    v = Asl,mid/(b*d)*(fsv/fc) = 0.39923778
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) = 0.64577857
    MRc (4.17) = 7.2231E+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) = 0.67946111

```

$$M_{Ro} (4.18) = 4.2866E+008$$

$$M_{Ro} < 0.8 * M_{Rc}$$

--->

$$u = c_u \text{ (unconfined full section)} = 3.9059244E-005$$

$$M_u = M_{Rc}$$

Calculation of ratio l_b/l_d

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

Calculation of M_{u2} -

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

$$u = 4.2871818E-005$$

$$M_u = 6.3496E+008$$

with full section properties:

$$b = 200.00$$

$$d = 367.00$$

$$d' = 33.00$$

$$v = 0.97992777$$

$$N = 2.1578E+006$$

$$f_c = 30.00$$

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

$$\text{Final value of } c_u: c_u^* = \text{shear_factor} * \text{Max}(c_u, c_o) = 0.00925707$$

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

$$\text{From (5.4b), TB DY: } c_u = 0.00925707$$

$$w_e (5.4c) = 0.0207149$$

$$a_{se} ((5.4d), \text{TB DY}) = (a_{se1} * A_{ext} + a_{se2} * A_{int}) / A_{sec} = 0.33836562$$

$$a_{se1} = \text{Max}(((A_{conf,max1} - A_{noConf1}) / A_{conf,max1}) * (A_{conf,min1} / A_{conf,max1}), 0) = 0.33836562$$

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.

$A_{conf,max1} = 89600.00$ 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.

$A_{conf,min1} = 59225.00$ 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.

$A_{noConf1} = 43733.333$ is the unconfined external core area which is equal to $b_i^2/6$ as defined at (A.2).

$$a_{se2} (\geq a_{se1}) = \text{Max}(((A_{conf,max2} - A_{noConf2}) / A_{conf,max2}) * (A_{conf,min2} / A_{conf,max2}), 0) = 0.33836562$$

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.

$A_{conf,max2} = 45264.00$ 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.

$A_{conf,min2} = 0.00$ 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.

$A_{noConf2} = 32309.333$ is the unconfined internal core area which is equal to $b_i^2/6$ as defined at (A.2).

$$p_{sh,min} * F_{ywe} = \text{Min}(p_{sh,x} * F_{ywe}, p_{sh,y} * F_{ywe}) = 6.12205$$

Expression (5.4d) for $p_{sh,min} * F_{ywe}$ has been multiplied by 0.3 according to 15.7.1.3 for members without earthquake detailing (90° closed stirrups)

$$p_{sh,x} * F_{ywe} = p_{sh1} * F_{ywe1} + p_{sh2} * F_{ywe2} = 6.12205$$

$$p_{sh1} ((5.4d), \text{TB DY}) = L_{stir1} * A_{stir1} / (A_{sec} * s_1) = 0.00680678$$

$$L_{stir1} \text{ (Length of stirrups along Y)} = 1040.00$$

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

$$p_{sh2} (5.4d) = L_{stir2} * A_{stir2} / (A_{sec} * s_2) = 0.0012868$$

Lstir2 (Length of stirrups along Y) = 768.00
Astir2 (stirrups area) = 50.26548

psh_y*Fywe = psh1*Fywe1+ps2*Fywe2 = 6.12205
psh1 ((5.4d), TBDY) = Lstir1*Astir1/(Asec*s1) = 0.00680678
Lstir1 (Length of stirrups along X) = 1040.00
Astir1 (stirrups area) = 78.53982
psh2 ((5.4d), TBDY) = Lstir2*Astir2/(Asec*s2) = 0.0012868
Lstir2 (Length of stirrups along X) = 768.00
Astir2 (stirrups area) = 50.26548

Asec = 120000.00
s1 = 100.00
s2 = 250.00

fywe1 = 781.25
fywe2 = 625.00
fce = 30.00

From ((5.A5), TBDY), TBDY: cc = 0.002
c = confinement factor = 1.00

y1 = 0.0025
sh1 = 0.008
ft1 = 886.7403
fy1 = 738.9503
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 = 738.9503

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

y2 = 0.0025
sh2 = 0.008
ft2 = 898.293
fy2 = 748.5775
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 = 748.5775

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

yv = 0.0025
shv = 0.008
ftv = 894.3662
fyv = 745.3052
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 = 745.3052

```

with Esv = (Esjacket*Aslmid,jacket + Esmid*Aslmid,core)/Aslmid = 200000.00
1 = Aslten/(b*d)*(fs1/fc) = 0.38164216
2 = Aslcom/(b*d)*(fs2/fc) = 0.7508007
v = Aslmid/(b*d)*(fsv/fc) = 0.67946571
and confined core properties:
b = 160.00
d = 347.00
d' = 13.00
fcc (5A.2, TBDY) = 30.00
cc (5A.5, TBDY) = 0.002
c = confinement factor = 1.00
1 = Aslten/(b*d)*(fs1/fc) = 0.50454853
2 = Aslcom/(b*d)*(fs2/fc) = 0.99259314
v = Aslmid/(b*d)*(fsv/fc) = 0.898285
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) = 0.58834972
MRc (4.17) = 6.3496E+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
- 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*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) = 0.63603191
MRo (4.18) = 4.4768E+008
MRo < 0.8*MRc
--->
u = cu (unconfined full section) = 4.2871818E-005
Mu = MRc

```

Calculation of ratio lb/d

Adequate Lap Length: lb/d >= 1

Calculation of Shear Strength $V_r = \text{Min}(V_{r1}, V_{r2}) = 349917.706$

Calculation of Shear Strength at edge 1, $V_{r1} = 349917.706$

$V_{r1} = V_{Col} ((10.3), \text{ASCE } 41-17) = k_{nl} * V_{Col0}$

$V_{Col0} = 349917.706$

$k_{nl} = 1$ (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: $f_c' = (f_c'_{\text{jacket}} * \text{Area}_{\text{jacket}} + f_c'_{\text{core}} * \text{Area}_{\text{core}}) / \text{Area}_{\text{section}} = 25.00$, but $f_c'^{0.5} \leq 8.3$ MPa (22.5.3.1, ACI 318-14)

$M / Vd = 4.00$

$\mu_u = 3.4090E+006$

$V_u = 1.4594946E+005$

$d = 0.8 * h = 320.00$

$N_u = 2.1578E+006$

$A_g = 80000.00$

From (11.5.4.8), ACI 318-14: $V_s = V_{sjacket} + V_{s,core} = 471238.898$

where:

$V_{sjacket} = V_{sj1} + V_{sj2} = 471238.898$

$V_{sj1} = 314159.265$ is calculated for section web jacket, with:

$d = 320.00$

$A_v = 157079.633$

$f_y = 625.00$

$s = 100.00$

V_{sj1} is multiplied by $Col_{j1} = 1.00$

$s/d = 0.3125$

$V_{sj2} = 157079.633$ is calculated for section flange jacket, with:

$d = 160.00$

$A_v = 157079.633$

$f_y = 625.00$

$s = 100.00$

V_{sj2} is multiplied by $Col_{j2} = 1.00$

$s/d = 0.625$

$V_{s,core} = V_{s,c1} + V_{s,c2} = 0.00$

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

$d = 240.00$

$A_v = 100530.965$

$f_y = 500.00$

$s = 250.00$

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

$s/d = 1.04167$

$V_{s,c2} = 0.00$ is calculated for section flange core, with:

$d = 80.00$

$A_v = 100530.965$

$f_y = 500.00$

$s = 250.00$

$V_{s,c2}$ is multiplied by $Col_{c2} = 0.00$

$s/d = 3.125$

$V_f ((11-3)-(11.4), \text{ACI } 440) = 0.00$

From (11-11), ACI 440: $V_s + V_f \leq 212577.225$

$bw = 200.00$

Calculation of Shear Strength at edge 2, $V_{r2} = 349917.706$

$V_{r2} = V_{Col} ((10.3), \text{ASCE } 41-17) = k_{nl} * V_{Col0}$

$V_{Col0} = 349917.706$

$k_{nl} = 1$ (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: $f_c' = (f_c'_{\text{jacket}} * \text{Area}_{\text{jacket}} + f_c'_{\text{core}} * \text{Area}_{\text{core}}) / \text{Area}_{\text{section}} = 25.00$, but $f_c'^{0.5} \leq 8.3$

MPa (22.5.3.1, ACI 318-14)

$$M/Vd = 4.00$$

$$\mu_u = 15336.932$$

$$V_u = 1.4594946E-005$$

$$d = 0.8 \cdot h = 320.00$$

$$N_u = 2.1578E+006$$

$$A_g = 80000.00$$

$$\text{From (11.5.4.8), ACI 318-14: } V_s = V_{s,jacket} + V_{s,core} = 471238.898$$

where:

$$V_{s,jacket} = V_{s,j1} + V_{s,j2} = 471238.898$$

$V_{s,j1} = 314159.265$ is calculated for section web jacket, with:

$$d = 320.00$$

$$A_v = 157079.633$$

$$f_y = 625.00$$

$$s = 100.00$$

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

$$s/d = 0.3125$$

$V_{s,j2} = 157079.633$ is calculated for section flange jacket, with:

$$d = 160.00$$

$$A_v = 157079.633$$

$$f_y = 625.00$$

$$s = 100.00$$

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

$$s/d = 0.625$$

$$V_{s,core} = V_{s,c1} + V_{s,c2} = 0.00$$

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

$$d = 240.00$$

$$A_v = 100530.965$$

$$f_y = 500.00$$

$$s = 250.00$$

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

$$s/d = 1.04167$$

$V_{s,c2} = 0.00$ is calculated for section flange core, with:

$$d = 80.00$$

$$A_v = 100530.965$$

$$f_y = 500.00$$

$$s = 250.00$$

$V_{s,c2}$ is multiplied by $Col,c2 = 0.00$

$$s/d = 3.125$$

$$V_f ((11-3)-(11.4), \text{ACI 440}) = 0.00$$

$$\text{From (11-11), ACI 440: } V_s + V_f \leq 212577.225$$

$$bw = 200.00$$

End Of Calculation of Shear Capacity ratio for element: column JLC1 of floor 1

At local axis: 3

Start Of Calculation of Shear Capacity ratio for element: column JLC1 of floor 1

At Shear local axis: 2

(Bending local axis: 3)

Section Type: rcjlc3

Constant Properties

Knowledge Factor, $\phi = 0.90$

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

Consequently:

Jacket

New material of Secondary Member: Concrete Strength, $f_c = f_{cm} = 30.00$

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

Concrete Elasticity, $E_c = 25742.96$

Steel Elasticity, $E_s = 200000.00$

Existing Column

Existing material of Secondary Member: Concrete Strength, $f_c = f_{cm} = 18.00$

Existing material of Secondary Member: Steel Strength, $f_s = f_{sm} = 500.00$
Concrete Elasticity, $E_c = 19940.411$
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, $f_s = 1.25 \cdot f_{sm} = 625.00$

Max Height, $H_{max} = 400.00$
Min Height, $H_{min} = 200.00$
Max Width, $W_{max} = 400.00$
Min Width, $W_{min} = 200.00$
Jacket Thickness, $t_j = 50.00$
Cover Thickness, $c = 15.00$
Mean Confinement Factor overall section = 1.00
Element Length, $L = 3000.00$
Secondary Member
Smooth Bars
Ductile Steel
Without Detailing for Earthquake Resistance (including stirrups not 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, $V_a = -5.7948283E-005$
EDGE -B-
Shear Force, $V_b = 5.7948283E-005$
BOTH EDGES
Axial Force, $F = -2.1578E+006$
Longitudinal Reinforcement Area Distribution (in 2 divisions)
-Tension: $As_t = 0.00$
-Compression: $As_c = 5353.274$
Longitudinal Reinforcement Area Distribution (in 3 divisions)
-Tension: $As_{t,ten} = 1137.257$
-Compression: $As_{c,com} = 2208.54$
-Middle: $As_{c,mid} = 2007.478$

Calculation of Shear Capacity ratio , $V_e/V_r = 1.10794$
Member Controlled by Shear ($V_e/V_r > 1$)
Calculation of Shear Demand from fig. R18.6.5, ACI 318-14 $V_e = (M_{pr1} + M_{pr2})/l_n = 387689.388$
with
 $M_{pr1} = \text{Max}(Mu_{1+} , Mu_{1-}) = 5.8153E+008$
 $Mu_{1+} = 5.8153E+008$, is the ultimate moment strength at the edge 1 of the member in the actual moment direction
which is defined for the static loading combination
 $Mu_{1-} = 4.9554E+008$, 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
 $M_{pr2} = \text{Max}(Mu_{2+} , Mu_{2-}) = 5.8153E+008$
 $Mu_{2+} = 5.8153E+008$, is the ultimate moment strength at the edge 2 of the member in the actual moment direction
which is defined for the the static loading combination
 $Mu_{2-} = 4.9554E+008$, 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 Mu_{1+} -----

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

$$u = 1.1511372E-005$$

$$Mu = 5.8153E+008$$

with full section properties:

$$b = 400.00$$

$$d = 367.00$$

$$d' = 33.00$$

$$v = 0.48996389$$

$$N = 2.1578E+006$$

$$f_c = 30.00$$

$$\phi_0 (5A.5, TBDY) = 0.002$$

$$\text{Final value of } \phi_u: \phi_u^* = \text{shear_factor} * \text{Max}(\phi_u, \phi_c) = 0.00925707$$

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

$$\text{From (5.4b), TBDY: } \phi_u = 0.00925707$$

$$\phi_{we} (5.4c) = 0.0207149$$

$$\phi_{ase} ((5.4d), TBDY) = (\phi_{ase1} * A_{ext} + \phi_{ase2} * A_{int}) / A_{sec} = 0.33836562$$

$$\phi_{ase1} = \text{Max}(((A_{conf,max1} - A_{noConf1}) / A_{conf,max1}) * (A_{conf,min1} / A_{conf,max1}), 0) = 0.33836562$$

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.

$A_{conf,max1} = 89600.00$ 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.

$A_{conf,min1} = 59225.00$ 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.

$A_{noConf1} = 43733.333$ is the unconfined external core area which is equal to $b_i^2/6$ as defined at (A.2).

$$\phi_{ase2} (> \phi_{ase1}) = \text{Max}(((A_{conf,max2} - A_{noConf2}) / A_{conf,max2}) * (A_{conf,min2} / A_{conf,max2}), 0) = 0.33836562$$

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.

$A_{conf,max2} = 45264.00$ 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.

$A_{conf,min2} = 0.00$ 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.

$A_{noConf2} = 32309.333$ is the unconfined internal core area which is equal to $b_i^2/6$ as defined at (A.2).

$$p_{sh,min} * F_{ywe} = \text{Min}(p_{sh,x} * F_{ywe}, p_{sh,y} * F_{ywe}) = 6.12205$$

Expression (5.4d) for $p_{sh,min} * F_{ywe}$ has been multiplied by 0.3 according to 15.7.1.3 for members without earthquake detailing (90° closed stirrups)

$$p_{sh,x} * F_{ywe} = p_{sh1} * F_{ywe1} + p_{sh2} * F_{ywe2} = 6.12205$$

$$p_{sh1} ((5.4d), TBDY) = L_{stir1} * A_{stir1} / (A_{sec} * s_1) = 0.00680678$$

$$L_{stir1} \text{ (Length of stirrups along Y)} = 1040.00$$

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

$$p_{sh2} (5.4d) = L_{stir2} * A_{stir2} / (A_{sec} * s_2) = 0.0012868$$

$$L_{stir2} \text{ (Length of stirrups along Y)} = 768.00$$

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

$$p_{sh,y} * F_{ywe} = p_{sh1} * F_{ywe1} + p_{sh2} * F_{ywe2} = 6.12205$$

$$p_{sh1} ((5.4d), TBDY) = L_{stir1} * A_{stir1} / (A_{sec} * s_1) = 0.00680678$$

$$L_{stir1} \text{ (Length of stirrups along X)} = 1040.00$$

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

$$p_{sh2} ((5.4d), TBDY) = L_{stir2} * A_{stir2} / (A_{sec} * s_2) = 0.0012868$$

$$L_{stir2} \text{ (Length of stirrups along X)} = 768.00$$

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

$$A_{sec} = 120000.00$$

$$s_1 = 100.00$$

$$s_2 = 250.00$$

$$f_{ywe1} = 781.25$$

$$f_{ywe2} = 625.00$$

```

fce = 30.00
From ((5A.5), TBDY), TBDY: cc = 0.002
c = confinement factor = 1.00
y1 = 0.0025
sh1 = 0.008
ft1 = 886.7403
fy1 = 738.9503
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 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 = 738.9503
with Es1 = (Es,jacket*Asl,ten,jacket + Es,core*Asl,ten,core)/Asl,ten = 200000.00
y2 = 0.0025
sh2 = 0.008
ft2 = 898.293
fy2 = 748.5775
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 = 748.5775
with Es2 = (Es,jacket*Asl,com,jacket + Es,core*Asl,com,core)/Asl,com = 200000.00
yv = 0.0025
shv = 0.008
ftv = 894.3662
fyv = 745.3052
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 = 745.3052
with Esv = (Es,jacket*Asl,mid,jacket + Es,mid*Asl,mid,core)/Asl,mid = 200000.00
1 = Asl,ten/(b*d)*(fs1/fc) = 0.19082108
2 = Asl,com/(b*d)*(fs2/fc) = 0.37540035
v = Asl,mid/(b*d)*(fsv/fc) = 0.33973286
and confined core properties:
b = 360.00
d = 347.00
d' = 13.00
fcc (5A.2, TBDY) = 30.00
cc (5A.5, TBDY) = 0.002
c = confinement factor = 1.00
1 = Asl,ten/(b*d)*(fs1/fc) = 0.22424379
2 = Asl,com/(b*d)*(fs2/fc) = 0.44115251
v = Asl,mid/(b*d)*(fsv/fc) = 0.39923778
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) = 0.42533033
MRc (4.17) = 7.2277E+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) = 0.50069522
MRo (4.18) = 5.8153E+008
---->
u = cu (4.2) = 1.1511372E-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 = 3.0526570E-005
Mu = 4.9554E+008

with full section properties:

b = 200.00
d = 367.00
d' = 33.00
v = 0.97992777
N = 2.1578E+006
fc = 30.00
co (5A.5, TBDY) = 0.002
Final value of cu: cu* = shear_factor * Max(cu, cc) = 0.00925707

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

From (5.4b), TBDY: $c_u = 0.00925707$

w_e (5.4c) = 0.0207149

a_{se} ((5.4d), TBDY) = $(a_{se1} \cdot A_{ext} + a_{se2} \cdot A_{int}) / A_{sec} = 0.33836562$

$a_{se1} = \text{Max}(((A_{conf,max1} - A_{noConf1}) / A_{conf,max1}) \cdot (A_{conf,min1} / A_{conf,max1}), 0) = 0.33836562$

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.

$A_{conf,max1} = 89600.00$ 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.

$A_{conf,min1} = 59225.00$ 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.

$A_{noConf1} = 43733.333$ is the unconfined external core area which is equal to $b_i^2/6$ as defined at (A.2).

$a_{se2} (>= a_{se1}) = \text{Max}(((A_{conf,max2} - A_{noConf2}) / A_{conf,max2}) \cdot (A_{conf,min2} / A_{conf,max2}), 0) = 0.33836562$

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.

$A_{conf,max2} = 45264.00$ 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.

$A_{conf,min2} = 0.00$ 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.

$A_{noConf2} = 32309.333$ is the unconfined internal core area which is equal to $b_i^2/6$ as defined at (A.2).

$p_{sh,min} \cdot F_{ywe} = \text{Min}(p_{sh,x} \cdot F_{ywe}, p_{sh,y} \cdot F_{ywe}) = 6.12205$

Expression (5.4d) for $p_{sh,min} \cdot F_{ywe}$ has been multiplied by 0.3 according to 15.7.1.3 for members without earthquake detailing (90° closed stirrups)

$p_{sh,x} \cdot F_{ywe} = p_{sh1} \cdot F_{ywe1} + p_{s2} \cdot F_{ywe2} = 6.12205$

p_{sh1} ((5.4d), TBDY) = $L_{stir1} \cdot A_{stir1} / (A_{sec} \cdot s_1) = 0.00680678$

L_{stir1} (Length of stirrups along Y) = 1040.00

A_{stir1} (stirrups area) = 78.53982

p_{sh2} (5.4d) = $L_{stir2} \cdot A_{stir2} / (A_{sec} \cdot s_2) = 0.0012868$

L_{stir2} (Length of stirrups along Y) = 768.00

A_{stir2} (stirrups area) = 50.26548

$p_{sh,y} \cdot F_{ywe} = p_{sh1} \cdot F_{ywe1} + p_{s2} \cdot F_{ywe2} = 6.12205$

p_{sh1} ((5.4d), TBDY) = $L_{stir1} \cdot A_{stir1} / (A_{sec} \cdot s_1) = 0.00680678$

L_{stir1} (Length of stirrups along X) = 1040.00

A_{stir1} (stirrups area) = 78.53982

p_{sh2} ((5.4d), TBDY) = $L_{stir2} \cdot A_{stir2} / (A_{sec} \cdot s_2) = 0.0012868$

L_{stir2} (Length of stirrups along X) = 768.00

A_{stir2} (stirrups area) = 50.26548

$A_{sec} = 120000.00$

$s_1 = 100.00$

$s_2 = 250.00$

$f_{ywe1} = 781.25$

$f_{ywe2} = 625.00$

$f_{ce} = 30.00$

From ((5.A5), TBDY), TBDY: $c_c = 0.002$

c = confinement factor = 1.00

$y_1 = 0.0025$

$sh_1 = 0.008$

$ft_1 = 898.293$

$f_{y1} = 748.5775$

$su_1 = 0.032$

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 $esu1_nominal$ 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 $fs1 = (fs_{jacket} \cdot Asl_{ten,jacket} + fs_{core} \cdot Asl_{ten,core}) / Asl_{ten} = 748.5775$

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

$y2 = 0.0025$
 $sh2 = 0.008$
 $ft2 = 886.7403$
 $fy2 = 738.9503$
 $su2 = 0.032$

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 $fs2 = (fs_{jacket} \cdot Asl_{com,jacket} + fs_{core} \cdot Asl_{com,core}) / Asl_{com} = 738.9503$

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

$yv = 0.0025$
 $shv = 0.008$
 $ftv = 894.3662$
 $fyv = 745.3052$
 $suv = 0.032$

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 $fsv = (fs_{jacket} \cdot Asl_{mid,jacket} + fs_{mid} \cdot Asl_{mid,core}) / Asl_{mid} = 745.3052$

with $Es_v = (Es_{jacket} \cdot Asl_{mid,jacket} + Es_{mid} \cdot Asl_{mid,core}) / Asl_{mid} = 200000.00$

$1 = Asl_{ten} / (b \cdot d) \cdot (fs1 / fc) = 0.7508007$
 $2 = Asl_{com} / (b \cdot d) \cdot (fs2 / fc) = 0.38164216$
 $v = Asl_{mid} / (b \cdot d) \cdot (fsv / fc) = 0.67946571$

and confined core properties:

$b = 160.00$
 $d = 347.00$
 $d' = 13.00$
 $fcc (5A.2, TBDY) = 30.00$
 $cc (5A.5, TBDY) = 0.002$
 $c = \text{confinement factor} = 1.00$
 $1 = Asl_{ten} / (b \cdot d) \cdot (fs1 / fc) = 0.99259314$
 $2 = Asl_{com} / (b \cdot d) \cdot (fs2 / fc) = 0.50454853$
 $v = Asl_{mid} / (b \cdot d) \cdot (fsv / fc) = 0.898285$

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) = 0.8262842
MRc (4.18) = 4.9554E+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) = 0.90160009
MRo (4.18) = 2.4685E+008
MRo < 0.8*MRc
--->
u = cu (unconfined full section) = 3.0526570E-005
Mu = MRc
-----

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 = 1.1511372E-005
Mu = 5.8153E+008
-----

with full section properties:
b = 400.00
d = 367.00
d' = 33.00
v = 0.48996389
N = 2.1578E+006
fc = 30.00
co (5A.5, TBDY) = 0.002
Final value of cu: cu* = shear_factor * Max( cu, cc) = 0.00925707
The Shear_factor is considered equal to 1 (pure moment strength)
From (5.4b), TBDY: cu = 0.00925707
we (5.4c) = 0.0207149
ase ((5.4d), TBDY) = (ase1*Aext+ase2*Aint)/Asec = 0.33836562
ase1 = Max(((Aconf,max1-AnoConf1)/Aconf,max1)*(Aconf,min1/Aconf,max1),0) = 0.33836562
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 = 89600.00 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 = 59225.00 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 = 43733.333 is the unconfined external core area which is equal to $b_i^2/6$ as defined at (A.2).
 $ase2 (>=ase1) = \text{Max}(((Aconf,max2-AnoConf2)/Aconf,max2)*(Aconf,min2/Aconf,max2),0) = 0.33836562$

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 = 45264.00 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 = 0.00 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 = 32309.333 is the unconfined internal core area which is equal to $b_i^2/6$ as defined at (A.2).

$psh,min*Fywe = \text{Min}(psh,x*Fywe, psh,y*Fywe) = 6.12205$

Expression (5.4d) for $psh,min*Fywe$ has been multiplied by 0.3 according to 15.7.1.3 for members without earthquake detailing (90° closed stirrups)

 $psh_x*Fywe = psh1*Fywe1 + ps2*Fywe2 = 6.12205$

$psh1 ((5.4d), TBDY) = Lstir1*Astir1/(Asec*s1) = 0.00680678$

Lstir1 (Length of stirrups along Y) = 1040.00

Astir1 (stirrups area) = 78.53982

$psh2 ((5.4d) = Lstir2*Astir2/(Asec*s2) = 0.0012868$

Lstir2 (Length of stirrups along Y) = 768.00

Astir2 (stirrups area) = 50.26548

 $psh_y*Fywe = psh1*Fywe1 + ps2*Fywe2 = 6.12205$

$psh1 ((5.4d), TBDY) = Lstir1*Astir1/(Asec*s1) = 0.00680678$

Lstir1 (Length of stirrups along X) = 1040.00

Astir1 (stirrups area) = 78.53982

$psh2 ((5.4d), TBDY) = Lstir2*Astir2/(Asec*s2) = 0.0012868$

Lstir2 (Length of stirrups along X) = 768.00

Astir2 (stirrups area) = 50.26548

Asec = 120000.00

s1 = 100.00

s2 = 250.00

fywe1 = 781.25

fywe2 = 625.00

fce = 30.00

From ((5.A5), TBDY), TBDY: cc = 0.002

c = confinement factor = 1.00

y1 = 0.0025

sh1 = 0.008

ft1 = 886.7403

fy1 = 738.9503

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 $fs1 = (fs,jacket*Asl,ten,jacket + fs,core*Asl,ten,core)/Asl,ten = 738.9503$

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

y2 = 0.0025

sh2 = 0.008

ft2 = 898.293

fy2 = 748.5775

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 = 748.5775
with Es2 = (Es,jacket*Asl,com,jacket + Es,core*Asl,com,core)/Asl,com = 200000.00
yv = 0.0025
shv = 0.008
ftv = 894.3662
fyv = 745.3052
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 = 745.3052
with Esv = (Es,jacket*Asl,mid,jacket + Es,mid*Asl,mid,core)/Asl,mid = 200000.00
1 = Asl,ten/(b*d)*(fs1/fc) = 0.19082108
2 = Asl,com/(b*d)*(fs2/fc) = 0.37540035
v = Asl,mid/(b*d)*(fsv/fc) = 0.33973286
and confined core properties:
b = 360.00
d = 347.00
d' = 13.00
fcc (5A.2, TBDY) = 30.00
cc (5A.5, TBDY) = 0.002
c = confinement factor = 1.00
1 = Asl,ten/(b*d)*(fs1/fc) = 0.22424379
2 = Asl,com/(b*d)*(fs2/fc) = 0.44115251
v = Asl,mid/(b*d)*(fsv/fc) = 0.39923778
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) = 0.42533033
MRc (4.17) = 7.2277E+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) = 0.50069522

M_{Ro} (4.18) = 5.8153E+008

---->

$u = c_u$ (4.2) = 1.1511372E-005

$\mu = M_{Ro}$

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:

$u = 3.0526570E-005$

$\mu = 4.9554E+008$

with full section properties:

$b = 200.00$

$d = 367.00$

$d' = 33.00$

$v = 0.97992777$

$N = 2.1578E+006$

$f_c = 30.00$

α (5A.5, TBDY) = 0.002

Final value of c_u : $c_u^* = \text{shear_factor} * \text{Max}(c_u, \alpha) = 0.00925707$

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

From (5.4b), TBDY: $c_u = 0.00925707$

w_e (5.4c) = 0.0207149

a_{se} ((5.4d), TBDY) = $(a_{se1} * A_{ext} + a_{se2} * A_{int}) / A_{sec} = 0.33836562$

$a_{se1} = \text{Max}(((A_{conf,max1} - A_{noConf1}) / A_{conf,max1}) * (A_{conf,min1} / A_{conf,max1}), 0) = 0.33836562$

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.

$A_{conf,max1} = 89600.00$ 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.

$A_{conf,min1} = 59225.00$ 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.

$A_{noConf1} = 43733.333$ is the unconfined external core area which is equal to $b_i^2/6$ as defined at (A.2).

$a_{se2} (\geq a_{se1}) = \text{Max}(((A_{conf,max2} - A_{noConf2}) / A_{conf,max2}) * (A_{conf,min2} / A_{conf,max2}), 0) = 0.33836562$

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.

$A_{conf,max2} = 45264.00$ 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.

$A_{conf,min2} = 0.00$ 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 = 32309.333 is the unconfined internal core area which is equal to $b_i^2/6$ as defined at (A.2).
 $psh_{min} \cdot F_{ywe} = \min(psh_x \cdot F_{ywe}, psh_y \cdot F_{ywe}) = 6.12205$
 Expression (5.4d) for $psh_{min} \cdot F_{ywe}$ has been multiplied by 0.3 according to 15.7.1.3 for members without earthquake detailing (90° closed stirrups)

$psh_x \cdot F_{ywe} = psh1 \cdot F_{ywe1} + ps2 \cdot F_{ywe2} = 6.12205$
 $psh1$ ((5.4d), TBDY) = $L_{stir1} \cdot A_{stir1} / (A_{sec} \cdot s1) = 0.00680678$
 L_{stir1} (Length of stirrups along Y) = 1040.00
 A_{stir1} (stirrups area) = 78.53982
 $psh2$ (5.4d) = $L_{stir2} \cdot A_{stir2} / (A_{sec} \cdot s2) = 0.0012868$
 L_{stir2} (Length of stirrups along Y) = 768.00
 A_{stir2} (stirrups area) = 50.26548

$psh_y \cdot F_{ywe} = psh1 \cdot F_{ywe1} + ps2 \cdot F_{ywe2} = 6.12205$
 $psh1$ ((5.4d), TBDY) = $L_{stir1} \cdot A_{stir1} / (A_{sec} \cdot s1) = 0.00680678$
 L_{stir1} (Length of stirrups along X) = 1040.00
 A_{stir1} (stirrups area) = 78.53982
 $psh2$ ((5.4d), TBDY) = $L_{stir2} \cdot A_{stir2} / (A_{sec} \cdot s2) = 0.0012868$
 L_{stir2} (Length of stirrups along X) = 768.00
 A_{stir2} (stirrups area) = 50.26548

$A_{sec} = 120000.00$
 $s1 = 100.00$
 $s2 = 250.00$
 $f_{ywe1} = 781.25$
 $f_{ywe2} = 625.00$
 $f_{ce} = 30.00$
 From ((5.A5), TBDY), TBDY: $cc = 0.002$
 $c = \text{confinement factor} = 1.00$
 $y1 = 0.0025$
 $sh1 = 0.008$
 $ft1 = 898.293$
 $fy1 = 748.5775$
 $su1 = 0.032$
 using (30) in Biskinis/Fardis (2013) multiplied with shear_factor
 and also multiplied by the shear_factor according to 15.7.1.4, with
 $\text{Shear_factor} = 1.00$
 $lo/lou, min = lb/ld = 1.00$
 $su1 = 0.4 \cdot esu1_{nominal}$ ((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 $fs1 = (fs_{jacket} \cdot A_{sl, ten, jacket} + fs_{core} \cdot A_{sl, ten, core}) / A_{sl, ten} = 748.5775$
 with $Es1 = (Es_{jacket} \cdot A_{sl, ten, jacket} + Es_{core} \cdot A_{sl, ten, core}) / A_{sl, ten} = 200000.00$
 $y2 = 0.0025$
 $sh2 = 0.008$
 $ft2 = 886.7403$
 $fy2 = 738.9503$
 $su2 = 0.032$
 using (30) in Biskinis/Fardis (2013) multiplied with shear_factor
 and also multiplied by the shear_factor according to 15.7.1.4, with
 $\text{Shear_factor} = 1.00$
 $lo/lou, min = lb/lb, min = 1.00$
 $su2 = 0.4 \cdot esu2_{nominal}$ ((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 $fs2 = (fs_{jacket} \cdot A_{sl, com, jacket} + fs_{core} \cdot A_{sl, com, core}) / A_{sl, com} = 738.9503$
 with $Es2 = (Es_{jacket} \cdot A_{sl, com, jacket} + Es_{core} \cdot A_{sl, com, core}) / A_{sl, com} = 200000.00$
 $yv = 0.0025$
 $shv = 0.008$
 $ftv = 894.3662$
 $fyv = 745.3052$
 $suv = 0.032$
 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 = (fsjacket*Asl,mid,jacket + fs,mid*Asl,mid,core)/Asl,mid = 745.3052
with Esv = (Esjacket*Asl,mid,jacket + Es,mid*Asl,mid,core)/Asl,mid = 200000.00
1 = Asl,ten/(b*d)*(fs1/fc) = 0.7508007
2 = Asl,com/(b*d)*(fs2/fc) = 0.38164216
v = Asl,mid/(b*d)*(fsv/fc) = 0.67946571
and confined core properties:
b = 160.00
d = 347.00
d' = 13.00
fcc (5A.2, TBDY) = 30.00
cc (5A.5, TBDY) = 0.002
c = confinement factor = 1.00
1 = Asl,ten/(b*d)*(fs1/fc) = 0.99259314
2 = Asl,com/(b*d)*(fs2/fc) = 0.50454853
v = Asl,mid/(b*d)*(fsv/fc) = 0.898285
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) = 0.8262842
MRc (4.18) = 4.9554E+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) = 0.90160009
MRo (4.18) = 2.4685E+008
MRo < 0.8*MRc
---->

```

$u = cu$ (unconfined full section) = $3.0526570E-005$
 $Mu = MRc$

Calculation of ratio lb/d

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

Calculation of Shear Strength $Vr = \text{Min}(Vr1, Vr2) = 349917.706$

Calculation of Shear Strength at edge 1, $Vr1 = 349917.706$

$Vr1 = VCol$ ((10.3), ASCE 41-17) = $knl * VCol0$

$VCol0 = 349917.706$

$knl = 1$ (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: $fc' = (fc'_{jacket} * Area_{jacket} + fc'_{core} * Area_{core}) / Area_{section} = 25.00$, but $fc'^{0.5} \leq 8.3$ MPa (22.5.3.1, ACI 318-14)

$M/Vd = 4.00$

$Mu = 8.5591E+007$

$Vu = 5.7948283E-005$

$d = 0.8 * h = 320.00$

$Nu = 2.1578E+006$

$Ag = 80000.00$

From (11.5.4.8), ACI 318-14: $Vs = Vs_{jacket} + Vs_{core} = 471238.898$

where:

$Vs_{jacket} = Vs_{j1} + Vs_{j2} = 471238.898$

$Vs_{j1} = 157079.633$ is calculated for section web jacket, with:

$d = 160.00$

$Av = 157079.633$

$fy = 625.00$

$s = 100.00$

Vs_{j1} is multiplied by $Col_{j1} = 1.00$

$s/d = 0.625$

$Vs_{j2} = 314159.265$ is calculated for section flange jacket, with:

$d = 320.00$

$Av = 157079.633$

$fy = 625.00$

$s = 100.00$

Vs_{j2} is multiplied by $Col_{j2} = 1.00$

$s/d = 0.3125$

$Vs_{core} = Vs_{c1} + Vs_{c2} = 0.00$

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

$d = 80.00$

$Av = 100530.965$

$fy = 500.00$

$s = 250.00$

Vs_{c1} is multiplied by $Col_{c1} = 0.00$

$s/d = 3.125$

$Vs_{c2} = 0.00$ is calculated for section flange core, with:

$d = 240.00$

$Av = 100530.965$

$fy = 500.00$

$s = 250.00$

Vs_{c2} is multiplied by $Col_{c2} = 0.00$

$s/d = 1.04167$

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

From (11-11), ACI 440: $Vs + Vf \leq 212577.225$

$bw = 200.00$

Calculation of Shear Strength at edge 2, $V_{r2} = 349917.706$

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

$V_{Col0} = 349917.706$

$knl = 1$ (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: $f_c' = (f_c'_{jacket} * Area_{jacket} + f_c'_{core} * Area_{core}) / Area_{section} = 25.00$, but $f_c'^{0.5} \leq 8.3$ MPa (22.5.3.1, ACI 318-14)

$M / Vd = 4.00$

$\mu_u = 6.4291E+007$

$\mu_u = 5.7948283E-005$

$d = 0.8 * h = 320.00$

$N_u = 2.1578E+006$

$A_g = 80000.00$

From (11.5.4.8), ACI 318-14: $V_s = V_{sjacket} + V_{s,core} = 471238.898$

where:

$V_{sjacket} = V_{sj1} + V_{sj2} = 471238.898$

$V_{sj1} = 157079.633$ is calculated for section web jacket, with:

$d = 160.00$

$A_v = 157079.633$

$f_y = 625.00$

$s = 100.00$

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

$s/d = 0.625$

$V_{sj2} = 314159.265$ is calculated for section flange jacket, with:

$d = 320.00$

$A_v = 157079.633$

$f_y = 625.00$

$s = 100.00$

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

$s/d = 0.3125$

$V_{s,core} = V_{s,c1} + V_{s,c2} = 0.00$

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

$d = 80.00$

$A_v = 100530.965$

$f_y = 500.00$

$s = 250.00$

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

$s/d = 3.125$

$V_{s,c2} = 0.00$ is calculated for section flange core, with:

$d = 240.00$

$A_v = 100530.965$

$f_y = 500.00$

$s = 250.00$

$V_{s,c2}$ is multiplied by $Col,c2 = 0.00$

$s/d = 1.04167$

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

From (11-11), ACI 440: $V_s + V_f \leq 212577.225$

$bw = 200.00$

End Of Calculation of Shear Capacity ratio for element: column JLC1 of floor 1

At local axis: 2

Start Of Calculation of Chord Rotation Capacity for element: column JLC1 of floor 1

At local axis: 3

Integration Section: (a)

Section Type: rcjlc3

Constant Properties

Knowledge Factor, $\phi = 0.90$
 Chord Rotation is generally considered as Deformation-Controlled Action according to Table C7-1, ASCE 41-17.
 Mean strengths are used for Deformation-Controlled Actions according to 7.5.1.3, ASCE 41-17
 Consequently:
 Jacket
 New material of Secondary Member: Concrete Strength, $f_c = f_{cm} = 30.00$
 New material of Secondary Member: Steel Strength, $f_s = f_{sm} = 625.00$
 Concrete Elasticity, $E_c = 25742.96$
 Steel Elasticity, $E_s = 200000.00$
 Existing Column
 Existing material of Secondary Member: Concrete Strength, $f_c = f_{cm} = 18.00$
 Existing material of Secondary Member: Steel Strength, $f_s = f_{sm} = 500.00$
 Concrete Elasticity, $E_c = 19940.411$
 Steel Elasticity, $E_s = 200000.00$
 Max Height, $H_{max} = 400.00$
 Min Height, $H_{min} = 200.00$
 Max Width, $W_{max} = 400.00$
 Min Width, $W_{min} = 200.00$
 Jacket Thickness, $t_j = 50.00$
 Cover Thickness, $c = 15.00$
 Element Length, $L = 3000.00$
 Secondary Member
 Smooth Bars
 Ductile Steel
 Without Detailing for Earthquake Resistance (including stirrups not 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, $M = 8.5589E+007$
 Shear Force, $V_2 = -0.60572401$
 Shear Force, $V_3 = -1.4589530E-005$
 Axial Force, $F = -2.1578E+006$
 Longitudinal Reinforcement Area Distribution (in 2 divisions)
 -Tension: $As_t = 0.00$
 -Compression: $As_c = 5353.274$
 Longitudinal Reinforcement Area Distribution (in 3 divisions)
 -Tension: $As_{t,ten} = 1137.257$
 -Compression: $As_{l,com} = 2208.54$
 -Middle: $As_{l,mid} = 2007.478$
 Longitudinal External Reinforcement Area Distribution (in 3 divisions)
 -Tension: $As_{l,ten,jacket} = 829.3805$
 -Compression: $As_{l,com,jacket} = 1746.726$
 -Middle: $As_{l,mid,jacket} = 1545.664$
 Longitudinal Internal Reinforcement Area Distribution (in 3 divisions)
 -Tension: $As_{l,ten,core} = 307.8761$
 -Compression: $As_{l,com,core} = 461.8141$
 -Middle: $As_{l,mid,core} = 461.8141$
 Mean Diameter of Tension Reinforcement, $Db_L = 16.80$

Existing component: From table 7-7, ASCE 41_17: Final chord rotation Capacity $u_R = \phi \cdot u = 0.03784131$
 $u = y + p = 0.0420459$

- Calculation of y -

$y = (M_y \cdot L_s / 3) / E_{eff} = 0.0420459 ((4.29), \text{Biskinis Phd})$
 $M_y = 5.2455E+008$
 $L_s = M/V \text{ (with } L_s > 0.1 \cdot L \text{ and } L_s < 2 \cdot L) = 6000.00$
 From table 10.5, ASCE 41_17: $E_{eff} = \text{factor} \cdot E_c \cdot I_g = 2.4951E+013$
 factor = 0.70

$$A_g = 120000.00$$

$$\text{Mean concrete strength: } f_c' = (f_c'_{\text{jacket}} \cdot A_{\text{jacket}} + f_c'_{\text{core}} \cdot A_{\text{core}}) / A_{\text{section}} = 25.00$$

$$N = 2.1578\text{E}+006$$

$$E_c \cdot I_g = E_c_{\text{jacket}} \cdot I_{g_{\text{jacket}}} + E_c_{\text{core}} \cdot I_{g_{\text{core}}} = 3.5645\text{E}+013$$

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, $b = 400.00$

web width, $b_w = 200.00$

flange thickness, $t = 200.00$

$$y = \text{Min}(y_{\text{ten}}, y_{\text{com}})$$

$$y_{\text{ten}} = 1.4921411\text{E}-005$$

$$\text{with } f_y = 596.2441$$

$$d = 367.00$$

$$y = 0.45559994$$

$$A = 0.06111896$$

$$B = 0.04120454$$

$$\text{with } p_t = 0.00774698$$

$$p_c = 0.01504455$$

$$p_v = 0.01367492$$

$$N = 2.1578\text{E}+006$$

$$b = 400.00$$

$$" = 0.08991826$$

$$y_{\text{comp}} = 1.1520096\text{E}-005$$

$$\text{with } f_c = 30.00$$

$$E_c = 25742.96$$

$$y = 0.49615013$$

$$A = 0.00143$$

$$B = 0.01655203$$

$$\text{with } E_s = 200000.00$$

CONFIRMATION: $y = 0.49615013 < t/d$

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 $I_b/I_d \geq 1$
shear control ratio $V_y E / V_{col} E = 1.10794$

$$d = d_{\text{external}} = 367.00$$

$$s = s_{\text{external}} = 100.00$$

$$- t = s_1 + s_2 + 2 \cdot t_f / b_w \cdot (f_{fe} / f_s) = 0.00809358$$

$$\text{jacket: } s_1 = A_{v1} \cdot L_{\text{stir1}} / (s_1 \cdot A_g) = 0.00680678$$

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

$L_{\text{stir1}} = 1040.00$, is the total Length of all stirrups parallel to loading (shear) direction

$$s_1 = 100.00$$

$$\text{core: } s_2 = A_{v2} \cdot L_{\text{stir2}} / (s_2 \cdot A_g) = 0.0012868$$

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

$L_{\text{stir2}} = 768.00$, 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.

$$N_{UD} = 2.1578\text{E}+006$$

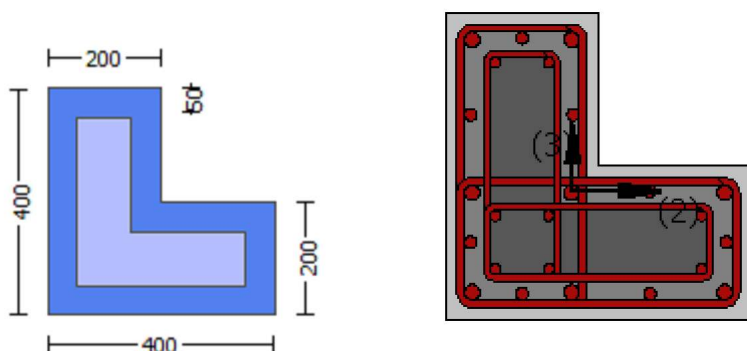
$$A_g = 120000.00$$

$f_{cE} = (f_{c_jacket} \cdot Area_jacket + f_{c_core} \cdot Area_core) / section_area = 25.00$
 $f_{yIE} = (f_{y_ext_Long_Reinf} \cdot Area_ext_Long_Reinf + f_{y_int_Long_Reinf} \cdot Area_int_Long_Reinf) / Area_Tot_Long_Rein = 596.2441$
 $f_{yIE} = (f_{y_ext_Trans_Reinf} \cdot s1 + f_{y_int_Trans_Reinf} \cdot s2) / (s1 + s2) = 605.1263$
 $\rho_l = Area_Tot_Long_Rein / (b \cdot d) = 0.03646644$
 $b = 400.00$
 $d = 367.00$
 $f_{cE} = 25.00$

 End Of Calculation of Chord Rotation Capacity for element: column JLC1 of floor 1
 At local axis: 3
 Integration Section: (a)

Calculation No. 13

column C1, Floor 1
 Limit State: Life Safety (data interpolation between analysis steps 1 and 2)
 Analysis: Uniform +X
 Check: Shear capacity V_{Rd}
 Edge: End
 Local Axis: (2)



Start Of Calculation of Shear Capacity for element: column JLC1 of floor 1
 At local axis: 2
 Integration Section: (b)
 Section Type: rcjls

Constant Properties

 Knowledge Factor, $\gamma = 0.90$
 Member Shear Force is generally considered as Force-Controlled Action according to Table C7-1, ASCE 41-17.
 Lower-bound strengths are used for Force-Controlled Actions according to 7.5.1.3, ASCE 41-17
 Consequently:
 Jacket

New material of Secondary Member: Concrete Strength, $f_c = f_{c_lower_bound} = 20.00$
 New material of Secondary Member: Steel Strength, $f_s = f_{s_lower_bound} = 500.00$
 Concrete Elasticity, $E_c = 25742.96$
 Steel Elasticity, $E_s = 200000.00$
 Existing Column
 Existing material of Secondary Member: Concrete Strength, $f_c = f_{c_lower_bound} = 12.00$
 Existing material of Secondary Member: Steel Strength, $f_s = f_{s_lower_bound} = 400.00$
 Concrete Elasticity, $E_c = 19940.411$
 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, $f_c = f_{cm} = 30.00$
 New material: Steel Strength, $f_s = f_{sm} = 625.00$
 Existing Column
 Existing material: Concrete Strength, $f_c = f_{cm} = 18.00$
 Existing material: Steel Strength, $f_s = f_{sm} = 500.00$
 #####
 Max Height, $H_{max} = 400.00$
 Min Height, $H_{min} = 200.00$
 Max Width, $W_{max} = 400.00$
 Min Width, $W_{min} = 200.00$
 Jacket Thickness, $t_j = 50.00$
 Cover Thickness, $c = 15.00$
 Element Length, $L = 3000.00$
 Secondary Member
 Smooth Bars
 Ductile Steel
 Without Detailing for Earthquake Resistance (including stirrups not 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, $M_a = 8.5589E+007$
 Shear Force, $V_a = -0.60572401$
 EDGE -B-
 Bending Moment, $M_b = -6.4291E+007$
 Shear Force, $V_b = 0.60572401$
 BOTH EDGES
 Axial Force, $F = -2.1578E+006$
 Longitudinal Reinforcement Area Distribution (in 2 divisions)
 -Tension: $As_t = 0.00$
 -Compression: $As_c = 5353.274$
 Longitudinal Reinforcement Area Distribution (in 3 divisions)
 -Tension: $As_{t,ten} = 1137.257$
 -Compression: $As_{c,com} = 2208.54$
 -Middle: $As_{mid} = 2007.478$
 Mean Diameter of Tension Reinforcement, $Db_{L,ten} = 16.80$

Existing component: From table 7-7, ASCE 41_17: Final Shear Capacity $V_R = \phi V_n = 267030.06$
 V_n ((10.3), ASCE 41-17) = $k_n l V_{CoI} = 296700.066$
 $V_{CoI} = 296700.066$
 $k_n l = 1.00$
 $displacement_ductility_demand = 0.07087185$

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: $fc' = (fc'_{jacket} \cdot Area_{jacket} + fc'_{core} \cdot Area_{core}) / Area_{section} = 16.66667$, but $fc'^{0.5} \leq 8.3$ MPa (22.5.3.1, ACI 318-14)
 $M/Vd = 4.00$
 $\mu_u = 6.4291E+007$
 $V_u = 0.60572401$
 $d = 0.8 \cdot h = 320.00$
 $N_u = 2.1578E+006$
 $A_g = 80000.00$
From (11.5.4.8), ACI 318-14: $V_s = V_{sjacket} + V_{s,core} = 376991.118$
where:
 $V_{sjacket} = V_{sj1} + V_{sj2} = 376991.118$
 $V_{sj1} = 125663.706$ is calculated for section web jacket, with:
 $d = 160.00$
 $A_v = 157079.633$
 $f_y = 500.00$
 $s = 100.00$
 V_{sj1} is multiplied by $Col,j1 = 1.00$
 $s/d = 0.625$
 $V_{sj2} = 251327.412$ is calculated for section flange jacket, with:
 $d = 320.00$
 $A_v = 157079.633$
 $f_y = 500.00$
 $s = 100.00$
 V_{sj2} is multiplied by $Col,j2 = 1.00$
 $s/d = 0.3125$
 $V_{s,core} = V_{s,c1} + V_{s,c2} = 0.00$
 $V_{s,c1} = 0.00$ is calculated for section web core, with:
 $d = 80.00$
 $A_v = 100530.965$
 $f_y = 400.00$
 $s = 250.00$
 $V_{s,c1}$ is multiplied by $Col,c1 = 0.00$
 $s/d = 3.125$
 $V_{s,c2} = 0.00$ is calculated for section flange core, with:
 $d = 240.00$
 $A_v = 100530.965$
 $f_y = 400.00$
 $s = 250.00$
 $V_{s,c2}$ is multiplied by $Col,c2 = 0.00$
 $s/d = 1.04167$
 $V_f ((11-3)-(11.4), ACI 440) = 0.00$
From (11-11), ACI 440: $V_s + V_f \leq 173568.578$
 $bw = 200.00$

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 $\theta_c = 0.00297987$
 $y = (M_y \cdot L_s / 3) / E_{eff} = 0.0420459$ ((4.29), Biskinis Phd))
 $M_y = 5.2455E+008$
 $L_s = M/V$ (with $L_s > 0.1 \cdot L$ and $L_s < 2 \cdot L$) = 6000.00
From table 10.5, ASCE 41_17: $E_{eff} = factor \cdot E_c \cdot I_g = 2.4951E+013$
 $factor = 0.70$
 $A_g = 120000.00$
Mean concrete strength: $fc' = (fc'_{jacket} \cdot Area_{jacket} + fc'_{core} \cdot Area_{core}) / Area_{section} = 25.00$
 $N = 2.1578E+006$
 $E_c \cdot I_g = E_{c,jacket} \cdot I_{g,jacket} + E_{c,core} \cdot I_{g,core} = 3.5645E+013$

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, $b = 400.00$

web width, $b_w = 200.00$

flange thickness, $t = 200.00$

$y = \text{Min}(y_{\text{ten}}, y_{\text{com}})$

$y_{\text{ten}} = 1.4921411\text{E-}005$

with $f_y = 596.2441$

$d = 367.00$

$y = 0.45559994$

$A = 0.06111896$

$B = 0.04120454$

with $p_t = 0.00774698$

$p_c = 0.01504455$

$p_v = 0.01367492$

$N = 2.1578\text{E+}006$

$b = 400.00$

" = 0.08991826

$y_{\text{comp}} = 1.1520096\text{E-}005$

with $f_c = 30.00$

$E_c = 25742.96$

$y = 0.49615013$

$A = 0.00143$

$B = 0.01655203$

with $E_s = 200000.00$

CONFIRMATION: $y = 0.49615013 < t/d$

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 JLC1 of floor 1

At local axis: 2

Integration Section: (b)

Calculation No. 14

column C1, Floor 1

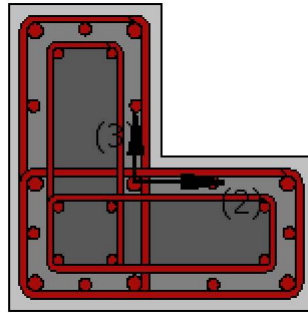
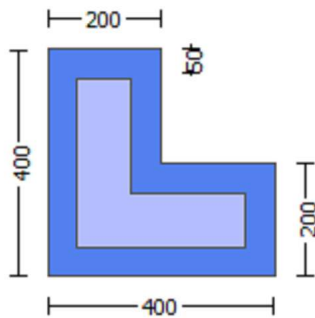
Limit State: Life Safety (data interpolation between analysis steps 1 and 2)

Analysis: Uniform +X

Check: Chord rotation capacity (θ)

Edge: End

Local Axis: (2)



Start Of Calculation of Shear Capacity ratio for element: column JLC1 of floor 1

At Shear local axis: 3

(Bending local axis: 2)

Section Type: rcjlcs

Constant Properties

Knowledge Factor, $\gamma = 0.90$

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

Consequently:

Jacket

New material of Secondary Member: Concrete Strength, $f_c = f_{cm} = 30.00$

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

Concrete Elasticity, $E_c = 25742.96$

Steel Elasticity, $E_s = 200000.00$

Existing Column

Existing material of Secondary Member: Concrete Strength, $f_c = f_{cm} = 18.00$

Existing material of Secondary Member: Steel Strength, $f_s = f_{sm} = 500.00$

Concrete Elasticity, $E_c = 19940.411$

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, $f_s = 1.25 \cdot f_{sm} = 625.00$

#####

Max Height, $H_{max} = 400.00$

Min Height, $H_{min} = 200.00$

Max Width, $W_{max} = 400.00$

Min Width, $W_{min} = 200.00$

Jacket Thickness, $t_j = 50.00$

Cover Thickness, $c = 15.00$

Mean Confinement Factor overall section = 1.00

Element Length, $L = 3000.00$

Secondary Member

Smooth Bars

Ductile Steel

Without Detailing for Earthquake Resistance (including stirrups not 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, $V_a = -1.4594946E-005$

EDGE -B-

Shear Force, $V_b = 1.4594946E-005$

BOTH EDGES

Axial Force, $F = -2.1578\text{E}+006$

Longitudinal Reinforcement Area Distribution (in 2 divisions)

-Tension: $As_t = 0.00$

-Compression: $As_c = 5353.274$

Longitudinal Reinforcement Area Distribution (in 3 divisions)

-Tension: $As_{t,ten} = 2208.54$

-Compression: $As_{c,com} = 1137.257$

-Middle: $As_{mid} = 2007.478$

Calculation of Shear Capacity ratio, $V_e/V_r = 1.37616$

Member Controlled by Shear ($V_e/V_r > 1$)

Calculation of Shear Demand from fig. R18.6.5, ACI 318-14 $V_e = (M_{pr1} + M_{pr2})/l_n = 481543.08$

with

$M_{pr1} = \text{Max}(Mu_{1+}, Mu_{1-}) = 7.2231\text{E}+008$

$Mu_{1+} = 7.2231\text{E}+008$, is the ultimate moment strength at the edge 1 of the member in the actual moment direction which is defined for the static loading combination

$Mu_{1-} = 6.3496\text{E}+008$, 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

$M_{pr2} = \text{Max}(Mu_{2+}, Mu_{2-}) = 7.2231\text{E}+008$

$Mu_{2+} = 7.2231\text{E}+008$, 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

$Mu_{2-} = 6.3496\text{E}+008$, 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 Mu_{1+}

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

$\phi_u = 3.9059244\text{E}-005$

$M_u = 7.2231\text{E}+008$

with full section properties:

$b = 400.00$

$d = 367.00$

$d' = 33.00$

$v = 0.48996389$

$N = 2.1578\text{E}+006$

$f_c = 30.00$

ϕ_c (5A.5, TBDY) = 0.002

Final value of ϕ_u : $\phi_u^* = \text{shear_factor} * \text{Max}(\phi_u, \phi_c) = 0.00925707$

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

From (5.4b), TBDY: $\phi_u = 0.00925707$

we (5.4c) = 0.0207149

a_{se} ((5.4d), TBDY) = $(a_{se1} * A_{ext} + a_{se2} * A_{int}) / A_{sec} = 0.33836562$

$a_{se1} = \text{Max}(((A_{conf,max1} - A_{noConf1}) / A_{conf,max1}) * (A_{conf,min1} / A_{conf,max1}), 0) = 0.33836562$

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.

$A_{conf,max1} = 89600.00$ 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.

$A_{conf,min1} = 59225.00$ 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.

$A_{noConf1} = 43733.333$ is the unconfined external core area which is equal to $b_i^2/6$ as defined at (A.2).

$a_{se2} (>= a_{se1}) = \text{Max}(((A_{conf,max2} - A_{noConf2}) / A_{conf,max2}) * (A_{conf,min2} / A_{conf,max2}), 0) = 0.33836562$

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.

$A_{conf,max2} = 45264.00$ 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.

$A_{conf,min2} = 0.00$ 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.

$A_{noConf2} = 32309.333$ is the unconfined internal core area which is equal to $b_i^2/6$ as defined at (A.2).

$p_{sh,min} \cdot F_{ywe} = \text{Min}(p_{sh,x} \cdot F_{ywe}, p_{sh,y} \cdot F_{ywe}) = 6.12205$

Expression (5.4d) for $p_{sh,min} \cdot F_{ywe}$ has been multiplied by 0.3 according to 15.7.1.3 for members without earthquake detailing (90° closed stirrups)

$p_{sh,x} \cdot F_{ywe} = p_{sh1} \cdot F_{ywe1} + p_{s2} \cdot F_{ywe2} = 6.12205$

p_{sh1} ((5.4d), TBDY) = $L_{stir1} \cdot A_{stir1} / (A_{sec} \cdot s_1) = 0.00680678$

L_{stir1} (Length of stirrups along Y) = 1040.00

A_{stir1} (stirrups area) = 78.53982

p_{s2} (5.4d) = $L_{stir2} \cdot A_{stir2} / (A_{sec} \cdot s_2) = 0.0012868$

L_{stir2} (Length of stirrups along Y) = 768.00

A_{stir2} (stirrups area) = 50.26548

$p_{sh,y} \cdot F_{ywe} = p_{sh1} \cdot F_{ywe1} + p_{s2} \cdot F_{ywe2} = 6.12205$

p_{sh1} ((5.4d), TBDY) = $L_{stir1} \cdot A_{stir1} / (A_{sec} \cdot s_1) = 0.00680678$

L_{stir1} (Length of stirrups along X) = 1040.00

A_{stir1} (stirrups area) = 78.53982

p_{s2} ((5.4d), TBDY) = $L_{stir2} \cdot A_{stir2} / (A_{sec} \cdot s_2) = 0.0012868$

L_{stir2} (Length of stirrups along X) = 768.00

A_{stir2} (stirrups area) = 50.26548

$A_{sec} = 120000.00$

$s_1 = 100.00$

$s_2 = 250.00$

$f_{ywe1} = 781.25$

$f_{ywe2} = 625.00$

$f_{ce} = 30.00$

From ((5.A5), TBDY), TBDY: $c_c = 0.002$

c = confinement factor = 1.00

$y_1 = 0.0025$

$sh_1 = 0.008$

$ft_1 = 898.293$

$fy_1 = 748.5775$

$su_1 = 0.032$

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

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

$su_1 = 0.4 \cdot esu1_nominal$ ((5.5), TBDY) = 0.032

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

For calculation of $esu1_nominal$ 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 $fs_1 = (f_{sjacket} \cdot A_{sl,ten,jacket} + f_{s,core} \cdot A_{sl,ten,core}) / A_{sl,ten} = 748.5775$

with $Es_1 = (E_{sjacket} \cdot A_{sl,ten,jacket} + E_{s,core} \cdot A_{sl,ten,core}) / A_{sl,ten} = 200000.00$

$y_2 = 0.0025$

$sh_2 = 0.008$

$ft_2 = 886.7403$

$fy_2 = 738.9503$

$su_2 = 0.032$

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

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

$su_2 = 0.4 \cdot esu2_nominal$ ((5.5), TBDY) = 0.032

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

For calculation of $esu2_nominal$ 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 $\text{Min}(1, 1.25 \cdot (l_b/l_d)^{2/3})$, from 10.3.5, ASCE 41-17.

with $fs_2 = (f_{sjacket} \cdot A_{sl,com,jacket} + f_{s,core} \cdot A_{sl,com,core}) / A_{sl,com} = 738.9503$

with $Es_2 = (E_{sjacket} \cdot A_{sl,com,jacket} + E_{s,core} \cdot A_{sl,com,core}) / A_{sl,com} = 200000.00$

```

yv = 0.0025
shv = 0.008
ftv = 894.3662
fyv = 745.3052
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 = 745.3052
    with Esv = (Es,jacket*Asl,mid,jacket + Es,mid*Asl,mid,core)/Asl,mid = 200000.00
    1 = Asl,ten/(b*d)*(fs1/fc) = 0.37540035
    2 = Asl,com/(b*d)*(fs2/fc) = 0.19082108
    v = Asl,mid/(b*d)*(fsv/fc) = 0.33973286
and confined core properties:
b = 360.00
d = 347.00
d' = 13.00
fcc (5A.2, TBDY) = 30.00
cc (5A.5, TBDY) = 0.002
    c = confinement factor = 1.00
    1 = Asl,ten/(b*d)*(fs1/fc) = 0.44115251
    2 = Asl,com/(b*d)*(fs2/fc) = 0.22424379
    v = Asl,mid/(b*d)*(fsv/fc) = 0.39923778
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) = 0.64577857
MRc (4.17) = 7.2231E+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

--->

$*cu(4.11) = 0.67946111$

$M_{Ro}(4.18) = 4.2866E+008$

$M_{Ro} < 0.8*M_{Rc}$

--->

$u = cu$ (unconfined full section) = $3.9059244E-005$

$\mu_u = M_{Rc}$

Calculation of ratio l_b/d

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

Calculation of μ_{u1} -

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

$\mu_u = 4.2871818E-005$

$\mu_u = 6.3496E+008$

with full section properties:

$b = 200.00$

$d = 367.00$

$d' = 33.00$

$v = 0.97992777$

$N = 2.1578E+006$

$f_c = 30.00$

α (5A.5, TBDY) = 0.002

Final value of μ_u : $\mu_u^* = \text{shear_factor} * \text{Max}(\mu_u, \mu_c) = 0.00925707$

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

From (5.4b), TBDY: $\mu_u = 0.00925707$

μ_{ue} (5.4c) = 0.0207149

α_{se} ((5.4d), TBDY) = $(\alpha_{se1}*A_{ext} + \alpha_{se2}*A_{int})/A_{sec} = 0.33836562$

$\alpha_{se1} = \text{Max}(((A_{conf,max1} - A_{noConf1})/A_{conf,max1}) * (A_{conf,min1}/A_{conf,max1}), 0) = 0.33836562$

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.

$A_{conf,max1} = 89600.00$ 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.

$A_{conf,min1} = 59225.00$ 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.

$A_{noConf1} = 43733.333$ is the unconfined external core area which is equal to $b_i^2/6$ as defined at (A.2).

$\alpha_{se2} (\geq \alpha_{se1}) = \text{Max}(((A_{conf,max2} - A_{noConf2})/A_{conf,max2}) * (A_{conf,min2}/A_{conf,max2}), 0) = 0.33836562$

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.

$A_{conf,max2} = 45264.00$ 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.

$A_{conf,min2} = 0.00$ 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.

$A_{noConf2} = 32309.333$ is the unconfined internal core area which is equal to $b_i^2/6$ as defined at (A.2).

$p_{sh,min}*F_{ywe} = \text{Min}(p_{sh,x}*F_{ywe}, p_{sh,y}*F_{ywe}) = 6.12205$

Expression (5.4d) for $p_{sh,min}*F_{ywe}$ has been multiplied by 0.3 according to 15.7.1.3 for members without earthquake detailing (90° closed stirrups)

$p_{sh,x}*F_{ywe} = p_{sh1}*F_{ywe1} + p_{sh2}*F_{ywe2} = 6.12205$

p_{sh1} ((5.4d), TBDY) = $L_{stir1}*A_{stir1}/(A_{sec}*s_1) = 0.00680678$

Lstir1 (Length of stirrups along Y) = 1040.00
Astir1 (stirrups area) = 78.53982
psh2 (5.4d) = Lstir2*Astir2/(Asec*s2) = 0.0012868
Lstir2 (Length of stirrups along Y) = 768.00
Astir2 (stirrups area) = 50.26548

psh_y*Fywe = psh1*Fywe1+ps2*Fywe2 = 6.12205
psh1 ((5.4d), TBDY) = Lstir1*Astir1/(Asec*s1) = 0.00680678
Lstir1 (Length of stirrups along X) = 1040.00
Astir1 (stirrups area) = 78.53982
psh2 ((5.4d), TBDY) = Lstir2*Astir2/(Asec*s2) = 0.0012868
Lstir2 (Length of stirrups along X) = 768.00
Astir2 (stirrups area) = 50.26548

Asec = 120000.00

s1 = 100.00

s2 = 250.00

fywe1 = 781.25

fywe2 = 625.00

fce = 30.00

From ((5.A5), TBDY), TBDY: cc = 0.002

c = confinement factor = 1.00

y1 = 0.0025

sh1 = 0.008

ft1 = 886.7403

fy1 = 738.9503

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 = 738.9503

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

y2 = 0.0025

sh2 = 0.008

ft2 = 898.293

fy2 = 748.5775

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 = 748.5775

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

yv = 0.0025

shv = 0.008

ftv = 894.3662

fyv = 745.3052

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  $f_{syv} = f_{sv}/1.2$ , 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  $f_{sv} = (f_{sjacket} \cdot A_{sl,mid,jacket} + f_{s,mid} \cdot A_{sl,mid,core}) / A_{sl,mid} = 745.3052$ 
with  $E_{sv} = (E_{sjacket} \cdot A_{sl,mid,jacket} + E_{s,mid} \cdot A_{sl,mid,core}) / A_{sl,mid} = 200000.00$ 
 $1 = A_{sl,ten} / (b \cdot d) \cdot (f_{s1}/f_c) = 0.38164216$ 
 $2 = A_{sl,com} / (b \cdot d) \cdot (f_{s2}/f_c) = 0.7508007$ 
 $v = A_{sl,mid} / (b \cdot d) \cdot (f_{sv}/f_c) = 0.67946571$ 
and confined core properties:
 $b = 160.00$ 
 $d = 347.00$ 
 $d' = 13.00$ 
 $f_{cc} \text{ (5A.2, TBDY)} = 30.00$ 
 $cc \text{ (5A.5, TBDY)} = 0.002$ 
 $c = \text{confinement factor} = 1.00$ 
 $1 = A_{sl,ten} / (b \cdot d) \cdot (f_{s1}/f_c) = 0.50454853$ 
 $2 = A_{sl,com} / (b \cdot d) \cdot (f_{s2}/f_c) = 0.99259314$ 
 $v = A_{sl,mid} / (b \cdot d) \cdot (f_{sv}/f_c) = 0.898285$ 
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
--->
 $c_u \text{ (4.10)} = 0.58834972$ 
 $M_{Rc} \text{ (4.17)} = 6.3496E+008$ 
--->
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:  $b_o, d_o, d'_o$ 
-  $N, 1, 2, v$  normalised to  $b_o \cdot d_o$ , instead of  $b \cdot d$ 
-  $f_{cc}, cc$  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
--->
 $*c_u \text{ (4.11)} = 0.63603191$ 
 $M_{Ro} \text{ (4.18)} = 4.4768E+008$ 
 $M_{Ro} < 0.8 \cdot M_{Rc}$ 
--->
 $u = c_u \text{ (unconfined full section)} = 4.2871818E-005$ 
 $\mu = M_{Rc}$ 

```

Calculation of ratio l_b/l_d

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

Calculation of Mu2+

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

$$\phi_u = 3.9059244E-005$$

$$\mu_u = 7.2231E+008$$

with full section properties:

$$b = 400.00$$

$$d = 367.00$$

$$d' = 33.00$$

$$\nu = 0.48996389$$

$$N = 2.1578E+006$$

$$f_c = 30.00$$

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

$$\text{Final value of } \phi_u: \phi_u^* = \text{shear_factor} * \text{Max}(\phi_u, \phi_{co}) = 0.00925707$$

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

$$\text{From (5.4b), TBDY: } \phi_u = 0.00925707$$

$$\phi_{we} (5.4c) = 0.0207149$$

$$\phi_{ase} ((5.4d), TBDY) = (\phi_{ase1} * A_{ext} + \phi_{ase2} * A_{int}) / A_{sec} = 0.33836562$$

$$\phi_{ase1} = \text{Max}(((A_{conf,max1} - A_{noConf1}) / A_{conf,max1}) * (A_{conf,min1} / A_{conf,max1}), 0) = 0.33836562$$

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.

$A_{conf,max1} = 89600.00$ 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.

$A_{conf,min1} = 59225.00$ 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.

$A_{noConf1} = 43733.333$ is the unconfined external core area which is equal to $b_i^2/6$ as defined at (A.2).

$$\phi_{ase2} (> \phi_{ase1}) = \text{Max}(((A_{conf,max2} - A_{noConf2}) / A_{conf,max2}) * (A_{conf,min2} / A_{conf,max2}), 0) = 0.33836562$$

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.

$A_{conf,max2} = 45264.00$ 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.

$A_{conf,min2} = 0.00$ 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.

$A_{noConf2} = 32309.333$ is the unconfined internal core area which is equal to $b_i^2/6$ as defined at (A.2).

$$\phi_{psh,min} * F_{ywe} = \text{Min}(\phi_{psh,x} * F_{ywe}, \phi_{psh,y} * F_{ywe}) = 6.12205$$

Expression (5.4d) for $\phi_{psh,min} * F_{ywe}$ has been multiplied by 0.3 according to 15.7.1.3 for members without earthquake detailing (90° closed stirrups)

$$\phi_{psh,x} * F_{ywe} = \phi_{psh1} * F_{ywe1} + \phi_{psh2} * F_{ywe2} = 6.12205$$

$$\phi_{psh1} ((5.4d), TBDY) = L_{stir1} * A_{stir1} / (A_{sec} * s_1) = 0.00680678$$

$$L_{stir1} (\text{Length of stirrups along Y}) = 1040.00$$

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

$$\phi_{psh2} (5.4d) = L_{stir2} * A_{stir2} / (A_{sec} * s_2) = 0.0012868$$

$$L_{stir2} (\text{Length of stirrups along Y}) = 768.00$$

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

$$\phi_{psh,y} * F_{ywe} = \phi_{psh1} * F_{ywe1} + \phi_{psh2} * F_{ywe2} = 6.12205$$

$$\phi_{psh1} ((5.4d), TBDY) = L_{stir1} * A_{stir1} / (A_{sec} * s_1) = 0.00680678$$

$$L_{stir1} (\text{Length of stirrups along X}) = 1040.00$$

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

$$\phi_{psh2} ((5.4d), TBDY) = L_{stir2} * A_{stir2} / (A_{sec} * s_2) = 0.0012868$$

$$L_{stir2} (\text{Length of stirrups along X}) = 768.00$$

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

```

Asec = 120000.00
s1 = 100.00
s2 = 250.00
fywe1 = 781.25
fywe2 = 625.00
fce = 30.00
From ((5.A.5), TBDY), TBDY: cc = 0.002
c = confinement factor = 1.00
y1 = 0.0025
sh1 = 0.008
ft1 = 898.293
fy1 = 748.5775
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 Min(1,1.25*(lb/lb,min)^ 2/3), from 10.3.5, ASCE 41-17.
with fs1 = (fs,jacket*Asl,ten,jacket + fs,core*Asl,ten,core)/Asl,ten = 748.5775
with Es1 = (Es,jacket*Asl,ten,jacket + Es,core*Asl,ten,core)/Asl,ten = 200000.00
y2 = 0.0025
sh2 = 0.008
ft2 = 886.7403
fy2 = 738.9503
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,min)^ 2/3), from 10.3.5, ASCE 41-17.
with fs2 = (fs,jacket*Asl,com,jacket + fs,core*Asl,com,core)/Asl,com = 738.9503
with Es2 = (Es,jacket*Asl,com,jacket + Es,core*Asl,com,core)/Asl,com = 200000.00
yv = 0.0025
shv = 0.008
ftv = 894.3662
fyv = 745.3052
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 Min(1,1.25*(lb/lb,min)^ 2/3), from 10.3.5, ASCE 41-17.
with fsv = (fs,jacket*Asl,mid,jacket + fs,mid*Asl,mid,core)/Asl,mid = 745.3052
with Esv = (Es,jacket*Asl,mid,jacket + Es,mid*Asl,mid,core)/Asl,mid = 200000.00
1 = Asl,ten/(b*d)*(fs1/fc) = 0.37540035
2 = Asl,com/(b*d)*(fs2/fc) = 0.19082108
v = Asl,mid/(b*d)*(fsv/fc) = 0.33973286
and confined core properties:
b = 360.00
d = 347.00
d' = 13.00
fcc (5A.2, TBDY) = 30.00
cc (5A.5, TBDY) = 0.002

```

```

c = confinement factor = 1.00
1 = Asl,ten/(b*d)*(fs1/fc) = 0.44115251
2 = Asl,com/(b*d)*(fs2/fc) = 0.22424379
v = Asl,mid/(b*d)*(fsv/fc) = 0.39923778
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) = 0.64577857
MRc (4.17) = 7.2231E+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) = 0.67946111
MRo (4.18) = 4.2866E+008
MRo < 0.8*MRc
---->
u = cu (unconfined full section) = 3.9059244E-005
Mu = MRc
-----

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 = 4.2871818E-005
Mu = 6.3496E+008
-----

with full section properties:
b = 200.00

```

$d = 367.00$
 $d' = 33.00$
 $v = 0.97992777$
 $N = 2.1578E+006$
 $f_c = 30.00$
 $co (5A.5, TBDY) = 0.002$
 Final value of cu : $cu^* = shear_factor * Max(cu, cc) = 0.00925707$
 The Shear_factor is considered equal to 1 (pure moment strength)
 From (5.4b), TBDY: $cu = 0.00925707$
 $we (5.4c) = 0.0207149$
 $ase ((5.4d), TBDY) = (ase1*Aext+ase2*Aint)/Asec = 0.33836562$
 $ase1 = Max(((Aconf,max1-AnoConf1)/Aconf,max1)*(Aconf,min1/Aconf,max1),0) = 0.33836562$
 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.
 $Aconf,max1 = 89600.00$ 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.
 $Aconf,min1 = 59225.00$ 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.
 $AnoConf1 = 43733.333$ is the unconfined external core area which is equal to $bi^2/6$ as defined at (A.2).
 $ase2 (>=ase1) = Max(((Aconf,max2-AnoConf2)/Aconf,max2)*(Aconf,min2/Aconf,max2),0) = 0.33836562$
 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.
 $Aconf,max2 = 45264.00$ 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.
 $Aconf,min2 = 0.00$ 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.
 $AnoConf2 = 32309.333$ is the unconfined internal core area which is equal to $bi^2/6$ as defined at (A.2).
 $psh,min*Fywe = Min(psh,x*Fywe, psh,y*Fywe) = 6.12205$
 Expression (5.4d) for $psh,min*Fywe$ has been multiplied by 0.3 according to 15.7.1.3 for members without earthquake detailing (90° closed stirrups)

$psh_x*Fywe = psh1*Fywe1+ps2*Fywe2 = 6.12205$
 $psh1 ((5.4d), TBDY) = Lstir1*Astir1/(Asec*s1) = 0.00680678$
 $Lstir1$ (Length of stirrups along Y) = 1040.00
 $Astir1$ (stirrups area) = 78.53982
 $psh2 (5.4d) = Lstir2*Astir2/(Asec*s2) = 0.0012868$
 $Lstir2$ (Length of stirrups along Y) = 768.00
 $Astir2$ (stirrups area) = 50.26548

$psh_y*Fywe = psh1*Fywe1+ps2*Fywe2 = 6.12205$
 $psh1 ((5.4d), TBDY) = Lstir1*Astir1/(Asec*s1) = 0.00680678$
 $Lstir1$ (Length of stirrups along X) = 1040.00
 $Astir1$ (stirrups area) = 78.53982
 $psh2 ((5.4d), TBDY) = Lstir2*Astir2/(Asec*s2) = 0.0012868$
 $Lstir2$ (Length of stirrups along X) = 768.00
 $Astir2$ (stirrups area) = 50.26548

$Asec = 120000.00$
 $s1 = 100.00$
 $s2 = 250.00$
 $fywe1 = 781.25$
 $fywe2 = 625.00$
 $fce = 30.00$
 From ((5.A5), TBDY), TBDY: $cc = 0.002$
 $c =$ confinement factor = 1.00
 $y1 = 0.0025$
 $sh1 = 0.008$
 $ft1 = 886.7403$
 $fy1 = 738.9503$

```

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 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 = 738.9503
with Es1 = (Es,jacket*Asl,ten,jacket + Es,core*Asl,ten,core)/Asl,ten = 200000.00
y2 = 0.0025
sh2 = 0.008
ft2 = 898.293
fy2 = 748.5775
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 = 748.5775
with Es2 = (Es,jacket*Asl,com,jacket + Es,core*Asl,com,core)/Asl,com = 200000.00
yv = 0.0025
shv = 0.008
ftv = 894.3662
fyv = 745.3052
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 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 = 745.3052
with Esv = (Es,jacket*Asl,mid,jacket + Es,mid*Asl,mid,core)/Asl,mid = 200000.00
1 = Asl,ten/(b*d)*(fs1/fc) = 0.38164216
2 = Asl,com/(b*d)*(fs2/fc) = 0.7508007
v = Asl,mid/(b*d)*(fsv/fc) = 0.67946571
and confined core properties:
b = 160.00
d = 347.00
d' = 13.00
fcc (5A.2, TBDY) = 30.00
cc (5A.5, TBDY) = 0.002
c = confinement factor = 1.00
1 = Asl,ten/(b*d)*(fs1/fc) = 0.50454853
2 = Asl,com/(b*d)*(fs2/fc) = 0.99259314
v = Asl,mid/(b*d)*(fsv/fc) = 0.898285
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
--->

ϕ_{cu} (4.10) = 0.58834972
 M_{Rc} (4.17) = 6.3496E+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: 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}, 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) = 0.63603191
 M_{Ro} (4.18) = 4.4768E+008
 $M_{Ro} < 0.8*M_{Rc}$

--->
 $u = \phi_{cu}$ (unconfined full section) = 4.2871818E-005
 $M_u = M_{Rc}$

Calculation of ratio I_b/I_d

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

Calculation of Shear Strength $V_r = \min(V_{r1}, V_{r2}) = 349917.706$

Calculation of Shear Strength at edge 1, $V_{r1} = 349917.706$
 $V_{r1} = V_{Co1}$ ((10.3), ASCE 41-17) = $k_{nl}*V_{Co1}$
 $V_{Co1} = 349917.706$
 $k_{nl} = 1$ (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: $f'_c = (f'_c_{jacket}*Area_{jacket} + f'_c_{core}*Area_{core})/Area_{section} = 25.00$, but $f_c^{0.5} \leq 8.3$
MPa (22.5.3.1, ACI 318-14)
 $M/Vd = 4.00$
 $M_u = 3.4090E+006$
 $V_u = 1.4594946E-005$
 $d = 0.8*h = 320.00$
 $N_u = 2.1578E+006$
 $A_g = 80000.00$
From (11.5.4.8), ACI 318-14: $V_s = V_{sjacket} + V_{s,core} = 471238.898$
where:
 $V_{sjacket} = V_{sj1} + V_{sj2} = 471238.898$

Vs,j1 = 314159.265 is calculated for section web jacket, with:

$$d = 320.00$$

$$A_v = 157079.633$$

$$f_y = 625.00$$

$$s = 100.00$$

Vs,j1 is multiplied by Col,j1 = 1.00

$$s/d = 0.3125$$

Vs,j2 = 157079.633 is calculated for section flange jacket, with:

$$d = 160.00$$

$$A_v = 157079.633$$

$$f_y = 625.00$$

$$s = 100.00$$

Vs,j2 is multiplied by Col,j2 = 1.00

$$s/d = 0.625$$

$$V_{s,core} = V_{s,c1} + V_{s,c2} = 0.00$$

Vs,c1 = 0.00 is calculated for section web core, with:

$$d = 240.00$$

$$A_v = 100530.965$$

$$f_y = 500.00$$

$$s = 250.00$$

Vs,c1 is multiplied by Col,c1 = 0.00

$$s/d = 1.04167$$

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

$$d = 80.00$$

$$A_v = 100530.965$$

$$f_y = 500.00$$

$$s = 250.00$$

Vs,c2 is multiplied by Col,c2 = 0.00

$$s/d = 3.125$$

$$V_f ((11-3)-(11.4), \text{ACI } 440) = 0.00$$

$$\text{From } (11-11), \text{ACI } 440: V_s + V_f \leq 212577.225$$

$$bw = 200.00$$

Calculation of Shear Strength at edge 2, Vr2 = 349917.706

$$Vr2 = V_{Col} ((10.3), \text{ASCE } 41-17) = knl * V_{Col0}$$

$$V_{Col0} = 349917.706$$

$$knl = 1 \text{ (zero step-static loading)}$$

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).

$$= 1 \text{ (normal-weight concrete)}$$

$$\text{Mean concrete strength: } f'_c = (f'_c_{jacket} * \text{Area}_{jacket} + f'_c_{core} * \text{Area}_{core}) / \text{Area}_{section} = 25.00, \text{ but } f'_c^{0.5} \leq 8.3 \text{ MPa (22.5.3.1, ACI 318-14)}$$

$$M/Vd = 4.00$$

$$\mu_u = 15336.932$$

$$V_u = 1.4594946E+005$$

$$d = 0.8 * h = 320.00$$

$$N_u = 2.1578E+006$$

$$A_g = 80000.00$$

$$\text{From (11.5.4.8), ACI 318-14: } V_s = V_{sjacket} + V_{s,core} = 471238.898$$

where:

$$V_{sjacket} = V_{sj1} + V_{sj2} = 471238.898$$

Vs,j1 = 314159.265 is calculated for section web jacket, with:

$$d = 320.00$$

$$A_v = 157079.633$$

$$f_y = 625.00$$

$$s = 100.00$$

Vs,j1 is multiplied by Col,j1 = 1.00

$$s/d = 0.3125$$

Vs,j2 = 157079.633 is calculated for section flange jacket, with:

$$d = 160.00$$

$$A_v = 157079.633$$

$$f_y = 625.00$$

$$s = 100.00$$

Vs,j2 is multiplied by Col,j2 = 1.00

```

s/d = 0.625
Vs,core = Vs,c1 + Vs,c2 = 0.00
Vs,c1 = 0.00 is calculated for section web core, with:
d = 240.00
Av = 100530.965
fy = 500.00
s = 250.00
Vs,c1 is multiplied by Col,c1 = 0.00
s/d = 1.04167
Vs,c2 = 0.00 is calculated for section flange core, with:
d = 80.00
Av = 100530.965
fy = 500.00
s = 250.00
Vs,c2 is multiplied by Col,c2 = 0.00
s/d = 3.125
Vf ((11-3)-(11.4), ACI 440) = 0.00
From (11-11), ACI 440: Vs + Vf <= 212577.225
bw = 200.00
-----

End Of Calculation of Shear Capacity ratio for element: column JLC1 of floor 1
At local axis: 3
-----

Start Of Calculation of Shear Capacity ratio for element: column JLC1 of floor 1
At Shear local axis: 2
(Bending local axis: 3)
Section Type: rcjlc

Constant Properties
-----

Knowledge Factor,   = 0.90
Mean strength values are used for both shear and moment calculations.
Consequently:
Jacket
New material of Secondary Member: Concrete Strength, fc = fcm = 30.00
New material of Secondary Member: Steel Strength, fs = fsm = 625.00
Concrete Elasticity, Ec = 25742.96
Steel Elasticity, Es = 200000.00
Existing Column
Existing material of Secondary Member: Concrete Strength, fc = fcm = 18.00
Existing material of Secondary Member: Steel Strength, fs = fsm = 500.00
Concrete Elasticity, Ec = 19940.411
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 = 625.00
#####
Max Height, Hmax = 400.00
Min Height, Hmin = 200.00
Max Width, Wmax = 400.00
Min Width, Wmin = 200.00
Jacket Thickness, tj = 50.00
Cover Thickness, c = 15.00
Mean Confinement Factor overall section = 1.00
Element Length, L = 3000.00
Secondary Member
Smooth Bars
Ductile Steel
Without Detailing for Earthquake Resistance (including stirrups not 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: 2
EDGE -A-
Shear Force, $V_a = -5.7948283E-005$
EDGE -B-
Shear Force, $V_b = 5.7948283E-005$
BOTH EDGES
Axial Force, $F = -2.1578E+006$
Longitudinal Reinforcement Area Distribution (in 2 divisions)
-Tension: $As_t = 0.00$
-Compression: $As_c = 5353.274$
Longitudinal Reinforcement Area Distribution (in 3 divisions)
-Tension: $As_{t,ten} = 1137.257$
-Compression: $As_{c,com} = 2208.54$
-Middle: $As_{c,mid} = 2007.478$

Calculation of Shear Capacity ratio, $V_e/V_r = 1.10794$
Member Controlled by Shear ($V_e/V_r > 1$)
Calculation of Shear Demand from fig. R18.6.5, ACI 318-14 $V_e = (M_{pr1} + M_{pr2})/l_n = 387689.388$
with
 $M_{pr1} = \text{Max}(\mu_{u1+}, \mu_{u1-}) = 5.8153E+008$
 $\mu_{u1+} = 5.8153E+008$, is the ultimate moment strength at the edge 1 of the member in the actual moment direction which is defined for the static loading combination
 $\mu_{u1-} = 4.9554E+008$, 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
 $M_{pr2} = \text{Max}(\mu_{u2+}, \mu_{u2-}) = 5.8153E+008$
 $\mu_{u2+} = 5.8153E+008$, 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
 $\mu_{u2-} = 4.9554E+008$, 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 μ_u according to 4.1, Biskinis/Fardis 2013:
 $\mu_u = 1.1511372E-005$
 $\mu_u = 5.8153E+008$

with full section properties:

$b = 400.00$
 $d = 367.00$
 $d' = 33.00$
 $v = 0.48996389$
 $N = 2.1578E+006$

$f_c = 30.00$

$\alpha = (5A_s, TBDY) = 0.002$

Final value of μ_u : $\mu_u^* = \text{shear_factor} * \text{Max}(\mu_u, \alpha) = 0.00925707$

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

From (5.4b), TBDY: $\mu_u = 0.00925707$

we (5.4c) = 0.0207149

$\alpha_{se} ((5.4d), TBDY) = (\alpha_{se1} * A_{ext} + \alpha_{se2} * A_{int}) / A_{sec} = 0.33836562$

$\alpha_{se1} = \text{Max}(((A_{conf,max1} - A_{noConf1}) / A_{conf,max1}) * (A_{conf,min1} / A_{conf,max1}), 0) = 0.33836562$

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.

Aconf,max1 = 89600.00 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 = 59225.00 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 = 43733.333 is the unconfined external core area which is equal to $b_i^2/6$ as defined at (A.2).

ase2 (\geq ase1) = $\text{Max}(((A_{\text{conf,max2}} - A_{\text{noConf2}})/A_{\text{conf,max2}}) * (A_{\text{conf,min2}}/A_{\text{conf,max2}}), 0) = 0.33836562$

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 = 45264.00 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 = 0.00 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 = 32309.333 is the unconfined internal core area which is equal to $b_i^2/6$ as defined at (A.2).

psh,min*Fywe = $\text{Min}(psh_x * Fywe, psh_y * Fywe) = 6.12205$

Expression (5.4d) for psh,min*Fywe has been multiplied by 0.3 according to 15.7.1.3 for members without earthquake detailing (90° closed stirrups)

psh_x*Fywe = psh1*Fywe1 + ps2*Fywe2 = 6.12205

psh1 ((5.4d), TBDY) = $L_{\text{stir1}} * A_{\text{stir1}} / (A_{\text{sec}} * s_1) = 0.00680678$

Lstir1 (Length of stirrups along Y) = 1040.00

Astir1 (stirrups area) = 78.53982

psh2 (5.4d) = $L_{\text{stir2}} * A_{\text{stir2}} / (A_{\text{sec}} * s_2) = 0.0012868$

Lstir2 (Length of stirrups along Y) = 768.00

Astir2 (stirrups area) = 50.26548

psh_y*Fywe = psh1*Fywe1 + ps2*Fywe2 = 6.12205

psh1 ((5.4d), TBDY) = $L_{\text{stir1}} * A_{\text{stir1}} / (A_{\text{sec}} * s_1) = 0.00680678$

Lstir1 (Length of stirrups along X) = 1040.00

Astir1 (stirrups area) = 78.53982

psh2 ((5.4d), TBDY) = $L_{\text{stir2}} * A_{\text{stir2}} / (A_{\text{sec}} * s_2) = 0.0012868$

Lstir2 (Length of stirrups along X) = 768.00

Astir2 (stirrups area) = 50.26548

Asec = 120000.00

s1 = 100.00

s2 = 250.00

fywe1 = 781.25

fywe2 = 625.00

fce = 30.00

From ((5.A5), TBDY), TBDY: cc = 0.002

c = confinement factor = 1.00

y1 = 0.0025

sh1 = 0.008

ft1 = 886.7403

fy1 = 738.9503

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 = $(f_{s,jacket} * A_{s,ten,jacket} + f_{s,core} * A_{s,ten,core}) / A_{s,ten} = 738.9503$

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 = 898.293

fy2 = 748.5775

```

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 = 748.5775
with Es2 = (Es,jacket*Asl,com,jacket + Es,core*Asl,com,core)/Asl,com = 200000.00
yv = 0.0025
shv = 0.008
ftv = 894.3662
fyv = 745.3052
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 = 745.3052
with Esv = (Es,jacket*Asl,mid,jacket + Es,mid*Asl,mid,core)/Asl,mid = 200000.00
1 = Asl,ten/(b*d)*(fs1/fc) = 0.19082108
2 = Asl,com/(b*d)*(fs2/fc) = 0.37540035
v = Asl,mid/(b*d)*(fsv/fc) = 0.33973286
and confined core properties:
b = 360.00
d = 347.00
d' = 13.00
fcc (5A.2, TBDY) = 30.00
cc (5A.5, TBDY) = 0.002
c = confinement factor = 1.00
1 = Asl,ten/(b*d)*(fs1/fc) = 0.22424379
2 = Asl,com/(b*d)*(fs2/fc) = 0.44115251
v = Asl,mid/(b*d)*(fsv/fc) = 0.39923778
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) = 0.42533033
MRc (4.17) = 7.2277E+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

--->

$*c_u(4.11) = 0.50069522$

$M_{Ro}(4.18) = 5.8153E+008$

--->

$u = c_u(4.2) = 1.1511372E-005$

$\mu_u = M_{Ro}$

Calculation of ratio l_b/d

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

Calculation of μ_{u1} -

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

$u = 3.0526570E-005$

$\mu_u = 4.9554E+008$

with full section properties:

$b = 200.00$

$d = 367.00$

$d' = 33.00$

$v = 0.97992777$

$N = 2.1578E+006$

$f_c = 30.00$

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

Final value of c_u : $c_u^* = \text{shear_factor} * \text{Max}(c_u, c_c) = 0.00925707$

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

From (5.4b), TBDY: $c_u = 0.00925707$

$w_e(5.4c) = 0.0207149$

$\alpha_{se}((5.4d), TBDY) = (\alpha_{se1} * A_{ext} + \alpha_{se2} * A_{int}) / A_{sec} = 0.33836562$

$\alpha_{se1} = \text{Max}(((A_{conf,max1} - A_{noConf1}) / A_{conf,max1}) * (A_{conf,min1} / A_{conf,max1}), 0) = 0.33836562$

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.

$A_{conf,max1} = 89600.00$ 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.

$A_{conf,min1} = 59225.00$ 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.

$A_{noConf1} = 43733.333$ is the unconfined external core area which is equal to $b_i^2/6$ as defined at (A.2).

$\alpha_{se2} (\geq \alpha_{se1}) = \text{Max}(((A_{conf,max2} - A_{noConf2}) / A_{conf,max2}) * (A_{conf,min2} / A_{conf,max2}), 0) = 0.33836562$

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.

$A_{conf,max2} = 45264.00$ 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.

$A_{conf,min2} = 0.00$ 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.

$A_{noConf2} = 32309.333$ is the unconfined internal core area which is equal to $b_i^2/6$ as defined at (A.2).

$psh_{min} \cdot F_{ywe} = \text{Min}(psh_x \cdot F_{ywe}, psh_y \cdot F_{ywe}) = 6.12205$

Expression (5.4d) for $psh_{min} \cdot F_{ywe}$ has been multiplied by 0.3 according to 15.7.1.3 for members without earthquake detailing (90° closed stirrups)

$psh_x \cdot F_{ywe} = psh1 \cdot F_{ywe1} + ps2 \cdot F_{ywe2} = 6.12205$

$psh1$ ((5.4d), TBDY) = $L_{stir1} \cdot A_{stir1} / (A_{sec} \cdot s1) = 0.00680678$

L_{stir1} (Length of stirrups along Y) = 1040.00

A_{stir1} (stirrups area) = 78.53982

$psh2$ (5.4d) = $L_{stir2} \cdot A_{stir2} / (A_{sec} \cdot s2) = 0.0012868$

L_{stir2} (Length of stirrups along Y) = 768.00

A_{stir2} (stirrups area) = 50.26548

$psh_y \cdot F_{ywe} = psh1 \cdot F_{ywe1} + ps2 \cdot F_{ywe2} = 6.12205$

$psh1$ ((5.4d), TBDY) = $L_{stir1} \cdot A_{stir1} / (A_{sec} \cdot s1) = 0.00680678$

L_{stir1} (Length of stirrups along X) = 1040.00

A_{stir1} (stirrups area) = 78.53982

$psh2$ ((5.4d), TBDY) = $L_{stir2} \cdot A_{stir2} / (A_{sec} \cdot s2) = 0.0012868$

L_{stir2} (Length of stirrups along X) = 768.00

A_{stir2} (stirrups area) = 50.26548

$A_{sec} = 120000.00$

$s1 = 100.00$

$s2 = 250.00$

$f_{ywe1} = 781.25$

$f_{ywe2} = 625.00$

$f_{ce} = 30.00$

From ((5.A5), TBDY), TBDY: $cc = 0.002$

c = confinement factor = 1.00

$y1 = 0.0025$

$sh1 = 0.008$

$ft1 = 898.293$

$fy1 = 748.5775$

$su1 = 0.032$

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

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

$su1 = 0.4 \cdot esu1_{nominal}$ ((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 $\text{Min}(1, 1.25 \cdot (lb/ld)^{2/3})$, from 10.3.5, ASCE 41-17.

with $fs1 = (fs_{jacket} \cdot A_{sl,ten,jacket} + fs_{core} \cdot A_{sl,ten,core}) / A_{sl,ten} = 748.5775$

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

$y2 = 0.0025$

$sh2 = 0.008$

$ft2 = 886.7403$

$fy2 = 738.9503$

$su2 = 0.032$

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 $fs2 = (fs_{jacket} \cdot A_{sl,com,jacket} + fs_{core} \cdot A_{sl,com,core}) / A_{sl,com} = 738.9503$

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 = 894.3662
fyv = 745.3052
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 = 745.3052
    with Esv = (Es,jacket*Asl,mid,jacket + Es,mid*Asl,mid,core)/Asl,mid = 200000.00
    1 = Asl,ten/(b*d)*(fs1/fc) = 0.7508007
    2 = Asl,com/(b*d)*(fs2/fc) = 0.38164216
    v = Asl,mid/(b*d)*(fsv/fc) = 0.67946571
and confined core properties:
b = 160.00
d = 347.00
d' = 13.00
fcc (5A.2, TBDY) = 30.00
cc (5A.5, TBDY) = 0.002
    c = confinement factor = 1.00
    1 = Asl,ten/(b*d)*(fs1/fc) = 0.99259314
    2 = Asl,com/(b*d)*(fs2/fc) = 0.50454853
    v = Asl,mid/(b*d)*(fsv/fc) = 0.898285
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) = 0.8262842
    MRc (4.18) = 4.9554E+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) = 0.90160009
MRo (4.18) = 2.4685E+008
MRo < 0.8*MRc

--->

u = cu (unconfined full section) = 3.0526570E-005
Mu = MRc

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 = 1.1511372E-005
Mu = 5.8153E+008

with full section properties:

b = 400.00
d = 367.00
d' = 33.00
v = 0.48996389
N = 2.1578E+006

fc = 30.00
co (5A.5, TBDY) = 0.002

Final value of cu: cu* = shear_factor * Max(cu, cc) = 0.00925707
The Shear_factor is considered equal to 1 (pure moment strength)
From (5.4b), TBDY: cu = 0.00925707

we (5.4c) = 0.0207149

ase ((5.4d), TBDY) = (ase1*Aext+ase2*Aint)/Asec = 0.33836562

ase1 = Max(((Aconf,max1-AnoConf1)/Aconf,max1)*(Aconf,min1/Aconf,max1),0) = 0.33836562

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 = 89600.00 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 = 59225.00 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 = 43733.333 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) = 0.33836562

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 = 45264.00 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 = 0.00 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 = 32309.333 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) = 6.12205

Expression (5.4d) for psh,min*Fywe has been multiplied by 0.3 according to 15.7.1.3 for members without earthquake detailing (90° closed stirrups)

psh_x*Fywe = psh1*Fywe1+ps2*Fywe2 = 6.12205

psh1 ((5.4d), TBDY) = Lstir1*Astir1/(Asec*s1) = 0.00680678

Lstir1 (Length of stirrups along Y) = 1040.00

Astir1 (stirrups area) = 78.53982

$psh2 \text{ (5.4d)} = Lstir2 * Astir2 / (Asec * s2) = 0.0012868$
 $Lstir2 \text{ (Length of stirrups along Y)} = 768.00$
 $Astir2 \text{ (stirrups area)} = 50.26548$

$psh_y * Fywe = psh1 * Fywe1 + ps2 * Fywe2 = 6.12205$
 $psh1 \text{ ((5.4d), TBDY)} = Lstir1 * Astir1 / (Asec * s1) = 0.00680678$
 $Lstir1 \text{ (Length of stirrups along X)} = 1040.00$
 $Astir1 \text{ (stirrups area)} = 78.53982$
 $psh2 \text{ ((5.4d), TBDY)} = Lstir2 * Astir2 / (Asec * s2) = 0.0012868$
 $Lstir2 \text{ (Length of stirrups along X)} = 768.00$
 $Astir2 \text{ (stirrups area)} = 50.26548$

$Asec = 120000.00$

$s1 = 100.00$

$s2 = 250.00$

$fywe1 = 781.25$

$fywe2 = 625.00$

$fce = 30.00$

From ((5.A5), TBDY), TBDY: $cc = 0.002$

$c = \text{confinement factor} = 1.00$

$y1 = 0.0025$

$sh1 = 0.008$

$ft1 = 886.7403$

$fy1 = 738.9503$

$su1 = 0.032$

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

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

$su1 = 0.4 * esu1_nominal \text{ ((5.5), TBDY)} = 0.032$

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 $fs1 = (fs_{jacket} * Asl_{ten,jacket} + fs_{core} * Asl_{ten,core}) / Asl_{ten} = 738.9503$

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

$y2 = 0.0025$

$sh2 = 0.008$

$ft2 = 898.293$

$fy2 = 748.5775$

$su2 = 0.032$

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

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

$su2 = 0.4 * esu2_nominal \text{ ((5.5), TBDY)} = 0.032$

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 $fs2 = (fs_{jacket} * Asl_{com,jacket} + fs_{core} * Asl_{com,core}) / Asl_{com} = 748.5775$

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

$yv = 0.0025$

$shv = 0.008$

$ftv = 894.3662$

$fyv = 745.3052$

$suv = 0.032$

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

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

$suv = 0.4 * esuv_nominal \text{ ((5.5), TBDY)} = 0.032$

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,mid,jacket + fs,mid*Asl,mid,core)/Asl,mid = 745.3052
with Esv = (Es,jacket*Asl,mid,jacket + Es,mid*Asl,mid,core)/Asl,mid = 200000.00
1 = Asl,ten/(b*d)*(fs1/fc) = 0.19082108
2 = Asl,com/(b*d)*(fs2/fc) = 0.37540035
v = Asl,mid/(b*d)*(fsv/fc) = 0.33973286
and confined core properties:
b = 360.00
d = 347.00
d' = 13.00
fcc (5A.2, TBDY) = 30.00
cc (5A.5, TBDY) = 0.002
c = confinement factor = 1.00
1 = Asl,ten/(b*d)*(fs1/fc) = 0.22424379
2 = Asl,com/(b*d)*(fs2/fc) = 0.44115251
v = Asl,mid/(b*d)*(fsv/fc) = 0.39923778
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) = 0.42533033
MRc (4.17) = 7.2277E+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) = 0.50069522
MRo (4.18) = 5.8153E+008
--->
u = cu (4.2) = 1.1511372E-005
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:

$$\phi_u = 3.0526570E-005$$

$$\mu = 4.9554E+008$$

with full section properties:

$$b = 200.00$$

$$d = 367.00$$

$$d' = 33.00$$

$$v = 0.97992777$$

$$N = 2.1578E+006$$

$$f_c = 30.00$$

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

$$\text{Final value of } \phi_{cu}: \phi_{cu}^* = \text{shear_factor} * \text{Max}(\phi_{cu}, \phi_{co}) = 0.00925707$$

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

$$\text{From (5.4b), TBDY: } \phi_{cu} = 0.00925707$$

$$\phi_{we} (5.4c) = 0.0207149$$

$$\phi_{ase} ((5.4d), TBDY) = (\phi_{ase1} * A_{ext} + \phi_{ase2} * A_{int}) / A_{sec} = 0.33836562$$

$$\phi_{ase1} = \text{Max}(((A_{conf,max1} - A_{noConf1}) / A_{conf,max1}) * (\phi_{conf,min1} / A_{conf,max1}), 0) = 0.33836562$$

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.

$A_{conf,max1} = 89600.00$ 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.

$A_{conf,min1} = 59225.00$ 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.

$A_{noConf1} = 43733.333$ is the unconfined external core area which is equal to $b_i^2/6$ as defined at (A.2).

$$\phi_{ase2} (> \phi_{ase1}) = \text{Max}(((A_{conf,max2} - A_{noConf2}) / A_{conf,max2}) * (\phi_{conf,min2} / A_{conf,max2}), 0) = 0.33836562$$

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.

$A_{conf,max2} = 45264.00$ 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.

$A_{conf,min2} = 0.00$ 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.

$A_{noConf2} = 32309.333$ is the unconfined internal core area which is equal to $b_i^2/6$ as defined at (A.2).

$$\phi_{psh,min} * F_{ywe} = \text{Min}(\phi_{psh,x} * F_{ywe}, \phi_{psh,y} * F_{ywe}) = 6.12205$$

Expression (5.4d) for $\phi_{psh,min} * F_{ywe}$ has been multiplied by 0.3 according to 15.7.1.3 for members without earthquake detailing (90° closed stirrups)

$$\phi_{psh,x} * F_{ywe} = \phi_{psh1} * F_{ywe1} + \phi_{psh2} * F_{ywe2} = 6.12205$$

$$\phi_{psh1} ((5.4d), TBDY) = L_{stir1} * A_{stir1} / (A_{sec} * s1) = 0.00680678$$

$$L_{stir1} \text{ (Length of stirrups along Y)} = 1040.00$$

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

$$\phi_{psh2} (5.4d) = L_{stir2} * A_{stir2} / (A_{sec} * s2) = 0.0012868$$

$$L_{stir2} \text{ (Length of stirrups along Y)} = 768.00$$

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

$$\phi_{psh,y} * F_{ywe} = \phi_{psh1} * F_{ywe1} + \phi_{psh2} * F_{ywe2} = 6.12205$$

$$\phi_{psh1} ((5.4d), TBDY) = L_{stir1} * A_{stir1} / (A_{sec} * s1) = 0.00680678$$

$$L_{stir1} \text{ (Length of stirrups along X)} = 1040.00$$

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

$$\phi_{psh2} ((5.4d), TBDY) = L_{stir2} * A_{stir2} / (A_{sec} * s2) = 0.0012868$$

$$L_{stir2} \text{ (Length of stirrups along X)} = 768.00$$

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

$$A_{sec} = 120000.00$$

$$s1 = 100.00$$

$s_2 = 250.00$
 $f_{ywe1} = 781.25$
 $f_{ywe2} = 625.00$
 $f_{ce} = 30.00$
 From ((5A.5), TBDY), TBDY: $cc = 0.002$
 $c = \text{confinement factor} = 1.00$
 $y_1 = 0.0025$
 $sh_1 = 0.008$
 $ft_1 = 898.293$
 $fy_1 = 748.5775$
 $su_1 = 0.032$
 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 $fs_1 = (fs_{jacket} * Asl, \text{ten, jacket} + fs_{core} * Asl, \text{ten, core}) / Asl, \text{ten} = 748.5775$
 with $Es_1 = (Es_{jacket} * Asl, \text{ten, jacket} + Es_{core} * Asl, \text{ten, core}) / Asl, \text{ten} = 200000.00$
 $y_2 = 0.0025$
 $sh_2 = 0.008$
 $ft_2 = 886.7403$
 $fy_2 = 738.9503$
 $su_2 = 0.032$
 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 $fs_2 = (fs_{jacket} * Asl, \text{com, jacket} + fs_{core} * Asl, \text{com, core}) / Asl, \text{com} = 738.9503$
 with $Es_2 = (Es_{jacket} * Asl, \text{com, jacket} + Es_{core} * Asl, \text{com, core}) / Asl, \text{com} = 200000.00$
 $y_v = 0.0025$
 $sh_v = 0.008$
 $ft_v = 894.3662$
 $fy_v = 745.3052$
 $suv = 0.032$
 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 $fsyv = fs_v/1.2$, 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_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 $fs_v = (fs_{jacket} * Asl, \text{mid, jacket} + fs_{mid} * Asl, \text{mid, core}) / Asl, \text{mid} = 745.3052$
 with $Es_v = (Es_{jacket} * Asl, \text{mid, jacket} + Es_{mid} * Asl, \text{mid, core}) / Asl, \text{mid} = 200000.00$
 $1 = Asl, \text{ten} / (b * d) * (fs_1 / f_c) = 0.7508007$
 $2 = Asl, \text{com} / (b * d) * (fs_2 / f_c) = 0.38164216$
 $v = Asl, \text{mid} / (b * d) * (fs_v / f_c) = 0.67946571$
 and confined core properties:
 $b = 160.00$
 $d = 347.00$
 $d' = 13.00$
 $f_{cc} (5A.2, \text{TBDY}) = 30.00$
 $cc (5A.5, \text{TBDY}) = 0.002$
 $c = \text{confinement factor} = 1.00$
 $1 = Asl, \text{ten} / (b * d) * (fs_1 / f_c) = 0.99259314$
 $2 = Asl, \text{com} / (b * d) * (fs_2 / f_c) = 0.50454853$

```

v = Asl,mid/(b*d)*(fsv/fc) = 0.898285
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) = 0.8262842
MRc (4.18) = 4.9554E+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) = 0.90160009
MRo (4.18) = 2.4685E+008
MRo < 0.8*MRc
--->
u = cu (unconfined full section) = 3.0526570E-005
Mu = MRc
-----

```

Calculation of ratio lb/l_d

Adequate Lap Length: lb/l_d >= 1

Calculation of Shear Strength $V_r = \text{Min}(V_{r1}, V_{r2}) = 349917.706$

Calculation of Shear Strength at edge 1, $V_{r1} = 349917.706$

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

$V_{Co10} = 349917.706$

$knl = 1$ (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: $fc' = (fc'_{\text{jacket}} * \text{Area}_{\text{jacket}} + fc'_{\text{core}} * \text{Area}_{\text{core}}) / \text{Area}_{\text{section}} = 25.00$, but $fc'^{0.5} \leq 8.3$

MPa (22.5.3.1, ACI 318-14)

$$M/Vd = 4.00$$

$$\mu_u = 8.5591E+007$$

$$V_u = 5.7948283E-005$$

$$d = 0.8 \cdot h = 320.00$$

$$N_u = 2.1578E+006$$

$$A_g = 80000.00$$

$$\text{From (11.5.4.8), ACI 318-14: } V_s = V_{sjacket} + V_{s,core} = 471238.898$$

where:

$$V_{sjacket} = V_{sj1} + V_{sj2} = 471238.898$$

$V_{sj1} = 157079.633$ is calculated for section web jacket, with:

$$d = 160.00$$

$$A_v = 157079.633$$

$$f_y = 625.00$$

$$s = 100.00$$

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

$$s/d = 0.625$$

$V_{sj2} = 314159.265$ is calculated for section flange jacket, with:

$$d = 320.00$$

$$A_v = 157079.633$$

$$f_y = 625.00$$

$$s = 100.00$$

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

$$s/d = 0.3125$$

$$V_{s,core} = V_{s,c1} + V_{s,c2} = 0.00$$

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

$$d = 80.00$$

$$A_v = 100530.965$$

$$f_y = 500.00$$

$$s = 250.00$$

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

$$s/d = 3.125$$

$V_{s,c2} = 0.00$ is calculated for section flange core, with:

$$d = 240.00$$

$$A_v = 100530.965$$

$$f_y = 500.00$$

$$s = 250.00$$

$$V_{s,c2} \text{ is multiplied by } Col,c2 = 0.00$$

$$s/d = 1.04167$$

$$V_f \text{ ((11-3)-(11.4), ACI 440)} = 0.00$$

$$\text{From (11-11), ACI 440: } V_s + V_f \leq 212577.225$$

$$bw = 200.00$$

Calculation of Shear Strength at edge 2, $V_{r2} = 349917.706$

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

$$V_{Col0} = 349917.706$$

$$knl = 1 \text{ (zero step-static loading)}$$

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).

$$= 1 \text{ (normal-weight concrete)}$$

$$\text{Mean concrete strength: } f_c' = (f_c'_{jacket} \cdot Area_{jacket} + f_c'_{core} \cdot Area_{core}) / Area_{section} = 25.00, \text{ but } f_c'^{0.5} \leq 8.3$$

MPa (22.5.3.1, ACI 318-14)

$$M/Vd = 4.00$$

$$\mu_u = 6.4291E+007$$

$$V_u = 5.7948283E-005$$

$$d = 0.8 \cdot h = 320.00$$

$$N_u = 2.1578E+006$$

$$A_g = 80000.00$$

$$\text{From (11.5.4.8), ACI 318-14: } V_s = V_{sjacket} + V_{s,core} = 471238.898$$

where:

$$V_{sjacket} = V_{sj1} + V_{sj2} = 471238.898$$

$V_{sj1} = 157079.633$ is calculated for section web jacket, with:

$$d = 160.00$$

$$A_v = 157079.633$$

$f_y = 625.00$
 $s = 100.00$
 $V_{s,j1}$ is multiplied by $Col,j1 = 1.00$
 $s/d = 0.625$
 $V_{s,j2} = 314159.265$ is calculated for section flange jacket, with:
 $d = 320.00$
 $A_v = 157079.633$
 $f_y = 625.00$
 $s = 100.00$
 $V_{s,j2}$ is multiplied by $Col,j2 = 1.00$
 $s/d = 0.3125$
 $V_{s,core} = V_{s,c1} + V_{s,c2} = 0.00$
 $V_{s,c1} = 0.00$ is calculated for section web core, with:
 $d = 80.00$
 $A_v = 100530.965$
 $f_y = 500.00$
 $s = 250.00$
 $V_{s,c1}$ is multiplied by $Col,c1 = 0.00$
 $s/d = 3.125$
 $V_{s,c2} = 0.00$ is calculated for section flange core, with:
 $d = 240.00$
 $A_v = 100530.965$
 $f_y = 500.00$
 $s = 250.00$
 $V_{s,c2}$ is multiplied by $Col,c2 = 0.00$
 $s/d = 1.04167$
 $V_f ((11-3)-(11.4), ACI 440) = 0.00$
 From (11-11), ACI 440: $V_s + V_f \leq 212577.225$
 $bw = 200.00$

 End Of Calculation of Shear Capacity ratio for element: column JLC1 of floor 1
 At local axis: 2

Start Of Calculation of Chord Rotation Capacity for element: column JLC1 of floor 1
 At local axis: 2
 Integration Section: (b)
 Section Type: rcjlc

Constant Properties

 Knowledge Factor, $\phi = 0.90$
 Chord Rotation is generally considered as Deformation-Controlled Action according to Table C7-1, ASCE 41-17.
 Mean strengths are used for Deformation-Controlled Actions according to 7.5.1.3, ASCE 41-17
 Consequently:
 Jacket
 New material of Secondary Member: Concrete Strength, $f_c = f_{cm} = 30.00$
 New material of Secondary Member: Steel Strength, $f_s = f_{sm} = 625.00$
 Concrete Elasticity, $E_c = 25742.96$
 Steel Elasticity, $E_s = 200000.00$
 Existing Column
 Existing material of Secondary Member: Concrete Strength, $f_c = f_{cm} = 18.00$
 Existing material of Secondary Member: Steel Strength, $f_s = f_{sm} = 500.00$
 Concrete Elasticity, $E_c = 19940.411$
 Steel Elasticity, $E_s = 200000.00$
 Max Height, $H_{max} = 400.00$
 Min Height, $H_{min} = 200.00$
 Max Width, $W_{max} = 400.00$
 Min Width, $W_{min} = 200.00$
 Jacket Thickness, $t_j = 50.00$
 Cover Thickness, $c = 15.00$
 Element Length, $L = 3000.00$
 Secondary Member
 Smooth Bars

Ductile Steel
Without Detailing for Earthquake Resistance (including stirrups not 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, $M = 15336.81$
Shear Force, $V2 = 0.60572401$
Shear Force, $V3 = 1.4589530E-005$
Axial Force, $F = -2.1578E+006$
Longitudinal Reinforcement Area Distribution (in 2 divisions)
-Tension: $As_t = 0.00$
-Compression: $As_c = 5353.274$
Longitudinal Reinforcement Area Distribution (in 3 divisions)
-Tension: $As_{ten} = 2208.54$
-Compression: $As_{com} = 1137.257$
-Middle: $As_{mid} = 2007.478$
Longitudinal External Reinforcement Area Distribution (in 3 divisions)
-Tension: $As_{ten,jacket} = 1746.726$
-Compression: $As_{com,jacket} = 829.3805$
-Middle: $As_{mid,jacket} = 1545.664$
Longitudinal Internal Reinforcement Area Distribution (in 3 divisions)
-Tension: $As_{ten,core} = 461.8141$
-Compression: $As_{com,core} = 307.8761$
-Middle: $As_{mid,core} = 461.8141$
Mean Diameter of Tension Reinforcement, $Db_L = 16.60$

Existing component: From table 7-7, ASCE 41_17: Final chord rotation Capacity $u_R = u = 0.03721297$
 $u = y + p = 0.04134775$

- Calculation of y -

$y = (M_y * L_s / 3) / E_{eff} = 0.04134775$ ((4.29), Biskinis Phd))
 $M_y = 5.1584E+008$
 $L_s = M/V$ (with $L_s > 0.1 * L$ and $L_s < 2 * L$) = 6000.00
From table 10.5, ASCE 41_17: $E_{eff} = factor * E_c * I_g = 2.4951E+013$
 $factor = 0.70$
 $A_g = 120000.00$
Mean concrete strength: $fc' = (fc'_{jacket} * Area_{jacket} + fc'_{core} * Area_{core}) / Area_{section} = 25.00$
 $N = 2.1578E+006$
 $E_c * I_g = E_{c,jacket} * I_{g,jacket} + E_{c,core} * I_{g,core} = 3.5645E+013$

Calculation of Yielding Moment M_y

Calculation of y 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, $b = 400.00$
web width, $b_w = 200.00$
flange thickness, $t = 200.00$

$y = \text{Min}(y_{ten}, y_{com})$
 $y_{ten} = 1.6571499E-005$
with $f_y = 596.2441$
 $d = 367.00$
 $y = 0.50980796$
 $A = 0.19238222$

B = 0.11480474
 with pt = 0.01504455
 pc = 0.00774698
 pv = 0.01367492
 N = 2.1578E+006
 b = 400.00
 " = 0.08991826
 y_comp = 9.6849368E-006
 with fc = 30.00
 Ec = 25742.96
 y = 0.5901636
 A = 0.0730043
 B = 0.06549972
 with Es = 200000.00
 CONFIRMATION: y = 0.5901636 > t/d

Calculation of ratio lb/d

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

shear control ratio $V_y E / V_{col} E = 1.37616$

d = d_external = 367.00

s = s_external = 100.00

- $t = s_1 + s_2 + 2 \cdot t_f / b_w \cdot (f_{fe} / f_s) = 0.00809358$

jacket: $s_1 = A_{v1} \cdot L_{stir1} / (s_1 \cdot A_g) = 0.00680678$

Av1 = 78.53982, is the area of every stirrup parallel to loading (shear) direction

Lstir1 = 1040.00, is the total Length of all stirrups parallel to loading (shear) direction

s1 = 100.00

core: $s_2 = A_{v2} \cdot L_{stir2} / (s_2 \cdot A_g) = 0.0012868$

Av2 = 50.26548, is the area of every stirrup parallel to loading (shear) direction

Lstir2 = 768.00, is the total Length of all stirrups parallel to loading (shear) direction

s2 = 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 = 2.1578E+006

Ag = 120000.00

$f_{cE} = (f_{c_jacket} \cdot Area_jacket + f_{c_core} \cdot Area_core) / section_area = 25.00$

$f_{yIE} = (f_{y_ext_Long_Reinf} \cdot Area_ext_Long_Reinf + f_{y_int_Long_Reinf} \cdot Area_int_Long_Reinf) / Area_Tot_Long_Rein = 596.2441$

$f_{yIE} = (f_{y_ext_Trans_Reinf} \cdot s_1 + f_{y_int_Trans_Reinf} \cdot s_2) / (s_1 + s_2) = 605.1263$

$p_l = Area_Tot_Long_Rein / (b \cdot d) = 0.03646644$

b = 400.00

d = 367.00

$f_{cE} = 25.00$

End Of Calculation of Chord Rotation Capacity for element: column JLC1 of floor 1

At local axis: 2

Integration Section: (b)

Calculation No. 15

column C1, Floor 1

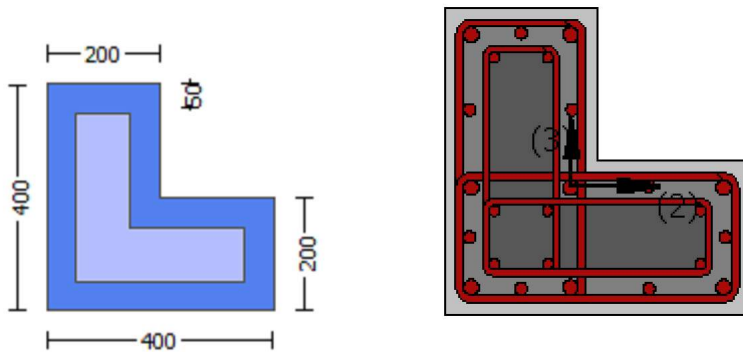
Limit State: Life Safety (data interpolation between analysis steps 1 and 2)

Analysis: Uniform +X

Check: Shear capacity VRd

Edge: End

Local Axis: (3)



Start Of Calculation of Shear Capacity for element: column JLC1 of floor 1

At local axis: 3

Integration Section: (b)

Section Type: rcjlcs

Constant Properties

Knowledge Factor, $\gamma = 0.90$

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

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

Consequently:

Jacket

New material of Secondary Member: Concrete Strength, $f_c = f_{c_lower_bound} = 20.00$

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

Concrete Elasticity, $E_c = 25742.96$

Steel Elasticity, $E_s = 200000.00$

Existing Column

Existing material of Secondary Member: Concrete Strength, $f_c = f_{c_lower_bound} = 12.00$

Existing material of Secondary Member: Steel Strength, $f_s = f_{s_lower_bound} = 400.00$

Concrete Elasticity, $E_c = 19940.411$

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, $f_c = f_{cm} = 30.00$

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

Existing Column

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

Existing material: Steel Strength, $f_s = f_{sm} = 500.00$

#####

Max Height, Hmax = 400.00
 Min Height, Hmin = 200.00
 Max Width, Wmax = 400.00
 Min Width, Wmin = 200.00
 Jacket Thickness, tj = 50.00
 Cover Thickness, c = 15.00
 Element Length, L = 3000.00
 Secondary Member
 Smooth Bars
 Ductile Steel
 Without Detailing for Earthquake Resistance (including stirrups not 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 = -3.4089E+006
 Shear Force, Va = -1.4589530E-005
 EDGE -B-
 Bending Moment, Mb = 15336.81
 Shear Force, Vb = 1.4589530E-005
 BOTH EDGES
 Axial Force, F = -2.1578E+006
 Longitudinal Reinforcement Area Distribution (in 2 divisions)
 -Tension: Aslt = 0.00
 -Compression: Aslc = 5353.274
 Longitudinal Reinforcement Area Distribution (in 3 divisions)
 -Tension: Asl,ten = 2208.54
 -Compression: Asl,com = 1137.257
 -Middle: Asl,mid = 2007.478
 Mean Diameter of Tension Reinforcement, DbL,ten = 16.60

Existing component: From table 7-7, ASCE 41_17: Final Shear Capacity VR = $V_n = 267030.06$
 $V_n ((10.3), ASCE 41-17) = knl * V_{CoI} = 296700.066$
 $V_{CoI} = 296700.066$
 $knl = 1.00$
 $displacement_ductility_demand = 0.00885451$

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: $fc' = (fc'_{jacket} * Area_{jacket} + fc'_{core} * Area_{core}) / Area_{section} = 16.66667$, but $fc'^{0.5} \leq 8.3$ MPa (22.5.3.1, ACI 318-14)
 $M/Vd = 4.00$
 $M_u = 15336.81$
 $V_u = 1.4589530E-005$
 $d = 0.8 * h = 320.00$
 $N_u = 2.1578E+006$
 $A_g = 80000.00$
 From (11.5.4.8), ACI 318-14: $V_s = V_{s,jacket} + V_{s,core} = 376991.118$
 where:
 $V_{s,jacket} = V_{s,j1} + V_{s,j2} = 376991.118$
 $V_{s,j1} = 251327.412$ is calculated for section web jacket, with:
 $d = 320.00$
 $A_v = 157079.633$
 $f_y = 500.00$
 $s = 100.00$
 $V_{s,j1}$ is multiplied by $Col,j1 = 1.00$
 $s/d = 0.3125$
 $V_{s,j2} = 125663.706$ is calculated for section flange jacket, with:

$d = 160.00$
 $A_v = 157079.633$
 $f_y = 500.00$
 $s = 100.00$
 $V_{s,j2}$ is multiplied by $Col,j2 = 1.00$
 $s/d = 0.625$
 $V_{s,core} = V_{s,c1} + V_{s,c2} = 0.00$
 $V_{s,c1} = 0.00$ is calculated for section web core, with:
 $d = 240.00$
 $A_v = 100530.965$
 $f_y = 400.00$
 $s = 250.00$
 $V_{s,c1}$ is multiplied by $Col,c1 = 0.00$
 $s/d = 1.04167$
 $V_{s,c2} = 0.00$ is calculated for section flange core, with:
 $d = 80.00$
 $A_v = 100530.965$
 $f_y = 400.00$
 $s = 250.00$
 $V_{s,c2}$ is multiplied by $Col,c2 = 0.00$
 $s/d = 3.125$
 $V_f ((11-3)-(11.4), ACI 440) = 0.00$
 From (11-11), ACI 440: $V_s + V_f \leq 173568.578$
 $bw = 200.00$

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 $\theta_r = 0.00036611$
 $y = (M_y * L_s / 3) / E_{eff} = 0.04134775 ((4.29), Biskinis Phd)$
 $M_y = 5.1584E+008$
 $L_s = M/V$ (with $L_s > 0.1 * L$ and $L_s < 2 * L$) = 6000.00
 From table 10.5, ASCE 41_17: $E_{eff} = factor * E_c * I_g = 2.4951E+013$
 $factor = 0.70$
 $A_g = 120000.00$
 Mean concrete strength: $f'_c = (f'_{c,jacket} * Area_{jacket} + f'_{c,core} * Area_{core}) / Area_{section} = 25.00$
 $N = 2.1578E+006$
 $E_c * I_g = E_{c,jacket} * I_{g,jacket} + E_{c,core} * I_{g,core} = 3.5645E+013$

Calculation of Yielding Moment M_y

Calculation of δ_u 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, $b = 400.00$
 web width, $bw = 200.00$
 flange thickness, $t = 200.00$

$y = \text{Min}(y_{ten}, y_{com})$
 $y_{ten} = 1.6571499E-005$
 with $f_y = 596.2441$
 $d = 367.00$
 $y = 0.50980796$
 $A = 0.19238222$
 $B = 0.11480474$
 with $pt = 0.01504455$
 $pc = 0.00774698$
 $pv = 0.01367492$
 $N = 2.1578E+006$
 $b = 400.00$

" = 0.08991826
y_comp = 9.6849368E-006
with fc = 30.00
Ec = 25742.96
y = 0.5901636
A = 0.0730043
B = 0.06549972
with Es = 200000.00
CONFIRMATION: y = 0.5901636 > t/d

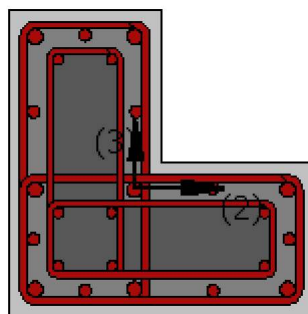
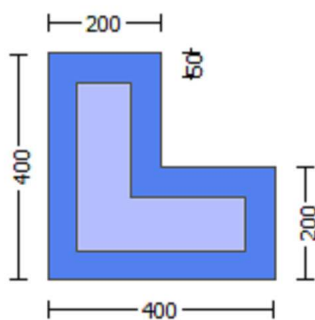
Calculation of ratio lb/d

Adequate Lap Length: lb/d >= 1

End Of Calculation of Shear Capacity for element: column JLC1 of floor 1
At local axis: 3
Integration Section: (b)

Calculation No. 16

column C1, Floor 1
Limit State: Life Safety (data interpolation between analysis steps 1 and 2)
Analysis: Uniform +X
Check: Chord rotation capacity (u)
Edge: End
Local Axis: (3)



Start Of Calculation of Shear Capacity ratio for element: column JLC1 of floor 1
At Shear local axis: 3
(Bending local axis: 2)
Section Type: rcjlc

Constant Properties

Knowledge Factor, = 0.90
Mean strength values are used for both shear and moment calculations.

Consequently:

Jacket

New material of Secondary Member: Concrete Strength, $f_c = f_{cm} = 30.00$

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

Concrete Elasticity, $E_c = 25742.96$

Steel Elasticity, $E_s = 200000.00$

Existing Column

Existing material of Secondary Member: Concrete Strength, $f_c = f_{cm} = 18.00$

Existing material of Secondary Member: Steel Strength, $f_s = f_{sm} = 500.00$

Concrete Elasticity, $E_c = 19940.411$

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, $f_s = 1.25 \cdot f_{sm} = 625.00$

#####

Max Height, $H_{max} = 400.00$

Min Height, $H_{min} = 200.00$

Max Width, $W_{max} = 400.00$

Min Width, $W_{min} = 200.00$

Jacket Thickness, $t_j = 50.00$

Cover Thickness, $c = 15.00$

Mean Confinement Factor overall section = 1.00

Element Length, $L = 3000.00$

Secondary Member

Smooth Bars

Ductile Steel

Without Detailing for Earthquake Resistance (including stirrups not 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, $V_a = -1.4594946E-005$

EDGE -B-

Shear Force, $V_b = 1.4594946E-005$

BOTH EDGES

Axial Force, $F = -2.1578E+006$

Longitudinal Reinforcement Area Distribution (in 2 divisions)

-Tension: $As_t = 0.00$

-Compression: $As_c = 5353.274$

Longitudinal Reinforcement Area Distribution (in 3 divisions)

-Tension: $As_{t,ten} = 2208.54$

-Compression: $As_{l,com} = 1137.257$

-Middle: $As_{l,mid} = 2007.478$

Calculation of Shear Capacity ratio, $V_e/V_r = 1.37616$

Member Controlled by Shear ($V_e/V_r > 1$)

Calculation of Shear Demand from fig. R18.6.5, ACI 318-14 $V_e = (M_{pr1} + M_{pr2})/l_n = 481543.08$

with

$M_{pr1} = \max(\mu_{u1+}, \mu_{u1-}) = 7.2231E+008$

$\mu_{u1+} = 7.2231E+008$, is the ultimate moment strength at the edge 1 of the member in the actual moment direction which is defined for the static loading combination

$\mu_{u1-} = 6.3496E+008$, 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

$M_{pr2} = \max(\mu_{u2+}, \mu_{u2-}) = 7.2231E+008$

$\mu_{u2+} = 7.2231E+008$, is the ultimate moment strength at the edge 2 of the member in the actual moment direction

which is defined for the the static loading combination

Mu2- = 6.3496E+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:

$$u = 3.9059244E-005$$

$$\mu = 7.2231E+008$$

with full section properties:

$$b = 400.00$$

$$d = 367.00$$

$$d' = 33.00$$

$$v = 0.48996389$$

$$N = 2.1578E+006$$

$$f_c = 30.00$$

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

$$\text{Final value of } c_u: c_u^* = \text{shear_factor} * \text{Max}(c_u, c_o) = 0.00925707$$

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

$$\text{From (5.4b), TBDY: } c_u = 0.00925707$$

$$w_e \text{ (5.4c)} = 0.0207149$$

$$a_{se} \text{ ((5.4d), TBDY)} = (a_{se1} * A_{ext} + a_{se2} * A_{int}) / A_{sec} = 0.33836562$$

$$a_{se1} = \text{Max}(((A_{conf,max1} - A_{noConf1}) / A_{conf,max1}) * (A_{conf,min1} / A_{conf,max1}), 0) = 0.33836562$$

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.

$A_{conf,max1} = 89600.00$ 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.

$A_{conf,min1} = 59225.00$ 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.

$A_{noConf1} = 43733.333$ is the unconfined external core area which is equal to $b_i^2/6$ as defined at (A.2).

$$a_{se2} (>= a_{se1}) = \text{Max}(((A_{conf,max2} - A_{noConf2}) / A_{conf,max2}) * (A_{conf,min2} / A_{conf,max2}), 0) = 0.33836562$$

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.

$A_{conf,max2} = 45264.00$ 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.

$A_{conf,min2} = 0.00$ 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.

$A_{noConf2} = 32309.333$ is the unconfined internal core area which is equal to $b_i^2/6$ as defined at (A.2).

$$p_{sh,min} * F_{ywe} = \text{Min}(p_{sh,x} * F_{ywe}, p_{sh,y} * F_{ywe}) = 6.12205$$

Expression (5.4d) for $p_{sh,min} * F_{ywe}$ has been multiplied by 0.3 according to 15.7.1.3 for members without earthquake detailing (90° closed stirrups)

$$p_{sh,x} * F_{ywe} = p_{sh1} * F_{ywe1} + p_{sh2} * F_{ywe2} = 6.12205$$

$$p_{sh1} \text{ ((5.4d), TBDY)} = L_{stir1} * A_{stir1} / (A_{sec} * s_1) = 0.00680678$$

$$L_{stir1} \text{ (Length of stirrups along Y)} = 1040.00$$

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

$$p_{sh2} \text{ (5.4d)} = L_{stir2} * A_{stir2} / (A_{sec} * s_2) = 0.0012868$$

$$L_{stir2} \text{ (Length of stirrups along Y)} = 768.00$$

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

$$p_{sh,y} * F_{ywe} = p_{sh1} * F_{ywe1} + p_{sh2} * F_{ywe2} = 6.12205$$

$$p_{sh1} \text{ ((5.4d), TBDY)} = L_{stir1} * A_{stir1} / (A_{sec} * s_1) = 0.00680678$$

$$L_{stir1} \text{ (Length of stirrups along X)} = 1040.00$$

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

$$p_{sh2} \text{ ((5.4d), TBDY)} = L_{stir2} * A_{stir2} / (A_{sec} * s_2) = 0.0012868$$

Lstir2 (Length of stirrups along X) = 768.00
Astir2 (stirrups area) = 50.26548

Asec = 120000.00

s1 = 100.00

s2 = 250.00

fywe1 = 781.25

fywe2 = 625.00

fce = 30.00

From ((5.A5), TBDY), TBDY: cc = 0.002

c = confinement factor = 1.00

y1 = 0.0025

sh1 = 0.008

ft1 = 898.293

fy1 = 748.5775

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 = 748.5775

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

y2 = 0.0025

sh2 = 0.008

ft2 = 886.7403

fy2 = 738.9503

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 = 738.9503

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

yv = 0.0025

shv = 0.008

ftv = 894.3662

fyv = 745.3052

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 = 745.3052

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

1 = Asl,ten/(b*d)*(fs1/fc) = 0.37540035

2 = Asl,com/(b*d)*(fs2/fc) = 0.19082108

v = Asl,mid/(b*d)*(fsv/fc) = 0.33973286

and confined core properties:

b = 360.00

d = 347.00

d' = 13.00

```

fcc (5A.2, TBDY) = 30.00
cc (5A.5, TBDY) = 0.002
c = confinement factor = 1.00
1 = Asl,ten/(b*d)*(fs1/fc) = 0.44115251
2 = Asl,com/(b*d)*(fs2/fc) = 0.22424379
v = Asl,mid/(b*d)*(fsv/fc) = 0.39923778
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) = 0.64577857
MRc (4.17) = 7.2231E+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) = 0.67946111
MRo (4.18) = 4.2866E+008
MRo < 0.8*MRc
---->
u = cu (unconfined full section) = 3.9059244E-005
Mu = MRc
-----

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 = 4.2871818E-005
Mu = 6.3496E+008
-----

```

with full section properties:

$$b = 200.00$$

$$d = 367.00$$

$$d' = 33.00$$

$$v = 0.97992777$$

$$N = 2.1578E+006$$

$$f_c = 30.00$$

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

$$\text{Final value of } cu: cu^* = \text{shear_factor} * \text{Max}(cu, cc) = 0.00925707$$

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

$$\text{From (5.4b), TBDY: } cu = 0.00925707$$

$$we(5.4c) = 0.0207149$$

$$ase((5.4d), TBDY) = (ase1 * A_{ext} + ase2 * A_{int}) / A_{sec} = 0.33836562$$

$$ase1 = \text{Max}(((A_{conf,max1} - A_{noConf1}) / A_{conf,max1}) * (A_{conf,min1} / A_{conf,max1}), 0) = 0.33836562$$

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.

$A_{conf,max1} = 89600.00$ 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.

$A_{conf,min1} = 59225.00$ 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.

$A_{noConf1} = 43733.333$ is the unconfined external core area which is equal to $b_i^2/6$ as defined at (A.2).

$$ase2 (>= ase1) = \text{Max}(((A_{conf,max2} - A_{noConf2}) / A_{conf,max2}) * (A_{conf,min2} / A_{conf,max2}), 0) = 0.33836562$$

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.

$A_{conf,max2} = 45264.00$ 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.

$A_{conf,min2} = 0.00$ 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.

$A_{noConf2} = 32309.333$ is the unconfined internal core area which is equal to $b_i^2/6$ as defined at (A.2).

$$psh_{min} * F_{ywe} = \text{Min}(psh_x * F_{ywe}, psh_y * F_{ywe}) = 6.12205$$

Expression (5.4d) for $psh_{min} * F_{ywe}$ has been multiplied by 0.3 according to 15.7.1.3 for members without earthquake detailing (90° closed stirrups)

$$psh_x * F_{ywe} = psh1 * F_{ywe1} + ps2 * F_{ywe2} = 6.12205$$

$$psh1((5.4d), TBDY) = L_{stir1} * A_{stir1} / (A_{sec} * s1) = 0.00680678$$

$$L_{stir1} \text{ (Length of stirrups along Y)} = 1040.00$$

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

$$psh2(5.4d) = L_{stir2} * A_{stir2} / (A_{sec} * s2) = 0.0012868$$

$$L_{stir2} \text{ (Length of stirrups along Y)} = 768.00$$

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

$$psh_y * F_{ywe} = psh1 * F_{ywe1} + ps2 * F_{ywe2} = 6.12205$$

$$psh1((5.4d), TBDY) = L_{stir1} * A_{stir1} / (A_{sec} * s1) = 0.00680678$$

$$L_{stir1} \text{ (Length of stirrups along X)} = 1040.00$$

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

$$psh2((5.4d), TBDY) = L_{stir2} * A_{stir2} / (A_{sec} * s2) = 0.0012868$$

$$L_{stir2} \text{ (Length of stirrups along X)} = 768.00$$

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

$$A_{sec} = 120000.00$$

$$s1 = 100.00$$

$$s2 = 250.00$$

$$f_{ywe1} = 781.25$$

$$f_{ywe2} = 625.00$$

$$f_{ce} = 30.00$$

$$\text{From ((5.A5), TBDY), TBDY: } cc = 0.002$$

$$c = \text{confinement factor} = 1.00$$

$$y1 = 0.0025$$

$$sh1 = 0.008$$

```

ft1 = 886.7403
fy1 = 738.9503
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 = 738.9503
    with Es1 = (Es,jacket*Asl,ten,jacket + Es,core*Asl,ten,core)/Asl,ten = 200000.00
y2 = 0.0025
sh2 = 0.008
ft2 = 898.293
fy2 = 748.5775
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 = 748.5775
    with Es2 = (Es,jacket*Asl,com,jacket + Es,core*Asl,com,core)/Asl,com = 200000.00
yv = 0.0025
shv = 0.008
ftv = 894.3662
fyv = 745.3052
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 = 745.3052
    with Esv = (Es,jacket*Asl,mid,jacket + Es,mid*Asl,mid,core)/Asl,mid = 200000.00
1 = Asl,ten/(b*d)*(fs1/fc) = 0.38164216
2 = Asl,com/(b*d)*(fs2/fc) = 0.7508007
v = Asl,mid/(b*d)*(fsv/fc) = 0.67946571
and confined core properties:
b = 160.00
d = 347.00
d' = 13.00
fcc (5A.2, TBDY) = 30.00
cc (5A.5, TBDY) = 0.002
    c = confinement factor = 1.00
    1 = Asl,ten/(b*d)*(fs1/fc) = 0.50454853
    2 = Asl,com/(b*d)*(fs2/fc) = 0.99259314
    v = Asl,mid/(b*d)*(fsv/fc) = 0.898285
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)

---->

$v < s, y_1$ - LHS eq.(4.7) is not satisfied

---->

$v < v_c, y_1$ - RHS eq.(4.6) is satisfied

---->

ϵ_{cu} (4.10) = 0.58834972

M_{Rc} (4.17) = 6.3496E+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: b_o , d_o , d'_o
- N , ϵ_1 , ϵ_2 , v normalised to $b_o \cdot d_o$, instead of $b \cdot d$
- ϵ_c - parameters of confined concrete, ϵ_{cc} , used in lieu of ϵ_c , ϵ_{cu}

---->

Subcase: Rupture of tension steel

---->

$v^* < v^* s, y_2$ - 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

---->

$v^* < v^* c, y_2$ - LHS eq.(4.6) is not satisfied

---->

$v^* < v^* c, y_1$ - RHS eq.(4.6) is not satisfied

---->

ϵ^*_{cu} (4.11) = 0.63603191

M_{Ro} (4.18) = 4.4768E+008

$M_{Ro} < 0.8 \cdot M_{Rc}$

---->

$\epsilon_u = \epsilon_{cu}$ (unconfined full section) = 4.2871818E-005

$M_u = M_{Rc}$

Calculation of ratio I_b/I_d

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

Calculation of M_{u2+}

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

$\epsilon_u = 3.9059244E-005$

$M_u = 7.2231E+008$

with full section properties:

$b = 400.00$

$d = 367.00$

$d' = 33.00$

$v = 0.48996389$

$N = 2.1578E+006$

$f_c = 30.00$

ϵ_{co} (5A.5, TBDY) = 0.002

Final value of ϵ_{cu} : $\epsilon_{cu}^* = \text{shear_factor} \cdot \text{Max}(\epsilon_{cu}, \epsilon_{cc}) = 0.00925707$

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

From (5.4b), TBDY: $\epsilon_{cu} = 0.00925707$

we (5.4c) = 0.0207149

ϵ_{ase} ((5.4d), TBDY) = $(\epsilon_{ase1} \cdot A_{ext} + \epsilon_{ase2} \cdot A_{int}) / A_{sec} = 0.33836562$

$$ase1 = \text{Max}(((A_{conf,max1} - A_{noConf1}) / A_{conf,max1}) * (A_{conf,min1} / A_{conf,max1}), 0) = 0.33836562$$

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.

$A_{conf,max1} = 89600.00$ 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.

$A_{conf,min1} = 59225.00$ 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.

$A_{noConf1} = 43733.333$ is the unconfined external core area which is equal to $b_i^2/6$ as defined at (A.2).

$$ase2 (>= ase1) = \text{Max}(((A_{conf,max2} - A_{noConf2}) / A_{conf,max2}) * (A_{conf,min2} / A_{conf,max2}), 0) = 0.33836562$$

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.

$A_{conf,max2} = 45264.00$ 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.

$A_{conf,min2} = 0.00$ 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.

$A_{noConf2} = 32309.333$ is the unconfined internal core area which is equal to $b_i^2/6$ as defined at (A.2).

$$psh_{min} * F_{ywe} = \text{Min}(psh_x * F_{ywe}, psh_y * F_{ywe}) = 6.12205$$

Expression (5.4d) for $psh_{min} * F_{ywe}$ has been multiplied by 0.3 according to 15.7.1.3 for members without earthquake detailing (90° closed stirrups)

$$psh_x * F_{ywe} = psh1 * F_{ywe1} + ps2 * F_{ywe2} = 6.12205$$

$$psh1 \text{ ((5.4d), TBDY)} = L_{stir1} * A_{stir1} / (A_{sec} * s1) = 0.00680678$$

$$L_{stir1} \text{ (Length of stirrups along Y)} = 1040.00$$

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

$$psh2 \text{ (5.4d)} = L_{stir2} * A_{stir2} / (A_{sec} * s2) = 0.0012868$$

$$L_{stir2} \text{ (Length of stirrups along Y)} = 768.00$$

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

$$psh_y * F_{ywe} = psh1 * F_{ywe1} + ps2 * F_{ywe2} = 6.12205$$

$$psh1 \text{ ((5.4d), TBDY)} = L_{stir1} * A_{stir1} / (A_{sec} * s1) = 0.00680678$$

$$L_{stir1} \text{ (Length of stirrups along X)} = 1040.00$$

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

$$psh2 \text{ ((5.4d), TBDY)} = L_{stir2} * A_{stir2} / (A_{sec} * s2) = 0.0012868$$

$$L_{stir2} \text{ (Length of stirrups along X)} = 768.00$$

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

$$A_{sec} = 120000.00$$

$$s1 = 100.00$$

$$s2 = 250.00$$

$$f_{ywe1} = 781.25$$

$$f_{ywe2} = 625.00$$

$$f_{ce} = 30.00$$

$$\text{From ((5.A5), TBDY), TBDY: } cc = 0.002$$

$$c = \text{confinement factor} = 1.00$$

$$y1 = 0.0025$$

$$sh1 = 0.008$$

$$ft1 = 898.293$$

$$fy1 = 748.5775$$

$$su1 = 0.032$$

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.

$$\text{with } fs1 = (f_{sjacket} * A_{sl,ten,jacket} + f_{s,core} * A_{sl,ten,core}) / A_{sl,ten} = 748.5775$$

```

with Es1 = (Es,jacket*Asl,ten,jacket + Es,core*Asl,ten,core)/Asl,ten = 200000.00
y2 = 0.0025
sh2 = 0.008
ft2 = 886.7403
fy2 = 738.9503
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 = 738.9503
with Es2 = (Es,jacket*Asl,com,jacket + Es,core*Asl,com,core)/Asl,com = 200000.00
yv = 0.0025
shv = 0.008
ftv = 894.3662
fyv = 745.3052
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 = 745.3052
with Esv = (Es,jacket*Asl,mid,jacket + Es,mid*Asl,mid,core)/Asl,mid = 200000.00
1 = Asl,ten/(b*d)*(fs1/fc) = 0.37540035
2 = Asl,com/(b*d)*(fs2/fc) = 0.19082108
v = Asl,mid/(b*d)*(fsv/fc) = 0.33973286
and confined core properties:
b = 360.00
d = 347.00
d' = 13.00
fcc (5A.2, TBDY) = 30.00
cc (5A.5, TBDY) = 0.002
c = confinement factor = 1.00
1 = Asl,ten/(b*d)*(fs1/fc) = 0.44115251
2 = Asl,com/(b*d)*(fs2/fc) = 0.22424379
v = Asl,mid/(b*d)*(fsv/fc) = 0.39923778
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) = 0.64577857
MRc (4.17) = 7.2231E+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, ν_1 , ν_2 , ν normalised to $\nu_0 \cdot d_o$, instead of $b \cdot d$
- parameters of confined concrete, fcc, cc, used in lieu of fc, ecu

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

ν^*_{cu} (4.11) = 0.67946111

M_{Ro} (4.18) = 4.2866E+008

M_{Ro} < 0.8*M_{Rc}

$\nu_u = \nu_{cu}$ (unconfined full section) = 3.9059244E-005

$\mu_u = M_{Rc}$

Calculation of ratio lb/d

Adequate Lap Length: lb/d >= 1

Calculation of μ_{u2}

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

$\nu_u = 4.2871818E-005$

$\mu_u = 6.3496E+008$

with full section properties:

b = 200.00

d = 367.00

d' = 33.00

$\nu = 0.97992777$

N = 2.1578E+006

fc = 30.00

co (5A.5, TBDY) = 0.002

Final value of ν_{cu} : $\nu_{cu}^* = \text{shear_factor} * \text{Max}(\nu_{cu}, \nu_{cc}) = 0.00925707$

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

From (5.4b), TBDY: $\nu_{cu} = 0.00925707$

ν_{we} (5.4c) = 0.0207149

ν_{ase} ((5.4d), TBDY) = $(\nu_{ase1} * A_{ext} + \nu_{ase2} * A_{int}) / A_{sec} = 0.33836562$

$\nu_{ase1} = \text{Max}(((A_{conf,max1} - A_{noConf1}) / A_{conf,max1}) * (A_{conf,min1} / A_{conf,max1}), 0) = 0.33836562$

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} = 89600.00 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} = 59225.00 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} = 43733.333 is the unconfined external core area which is equal to $b^2/6$ as defined at (A.2).

$\nu_{ase2} (>= \nu_{ase1}) = \text{Max}(((A_{conf,max2} - A_{noConf2}) / A_{conf,max2}) * (A_{conf,min2} / A_{conf,max2}), 0) = 0.33836562$

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 = 45264.00 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 = 0.00 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 = 32309.333 is the unconfined internal core area which is equal to $b_i^2/6$ as defined at (A.2).

$psh_{min} \cdot F_{ywe} = \text{Min}(psh_x \cdot F_{ywe}, psh_y \cdot F_{ywe}) = 6.12205$

Expression (5.4d) for $psh_{min} \cdot F_{ywe}$ has been multiplied by 0.3 according to 15.7.1.3 for members without earthquake detailing (90° closed stirrups)

$psh_x \cdot F_{ywe} = psh1 \cdot F_{ywe1} + ps2 \cdot F_{ywe2} = 6.12205$

$psh1$ ((5.4d), TBDY) = $L_{stir1} \cdot A_{stir1} / (A_{sec} \cdot s1) = 0.00680678$

L_{stir1} (Length of stirrups along Y) = 1040.00

A_{stir1} (stirrups area) = 78.53982

$psh2$ (5.4d) = $L_{stir2} \cdot A_{stir2} / (A_{sec} \cdot s2) = 0.0012868$

L_{stir2} (Length of stirrups along Y) = 768.00

A_{stir2} (stirrups area) = 50.26548

$psh_y \cdot F_{ywe} = psh1 \cdot F_{ywe1} + ps2 \cdot F_{ywe2} = 6.12205$

$psh1$ ((5.4d), TBDY) = $L_{stir1} \cdot A_{stir1} / (A_{sec} \cdot s1) = 0.00680678$

L_{stir1} (Length of stirrups along X) = 1040.00

A_{stir1} (stirrups area) = 78.53982

$psh2$ ((5.4d), TBDY) = $L_{stir2} \cdot A_{stir2} / (A_{sec} \cdot s2) = 0.0012868$

L_{stir2} (Length of stirrups along X) = 768.00

A_{stir2} (stirrups area) = 50.26548

$A_{sec} = 120000.00$

$s1 = 100.00$

$s2 = 250.00$

$f_{ywe1} = 781.25$

$f_{ywe2} = 625.00$

$f_{ce} = 30.00$

From ((5.A5), TBDY), TBDY: $cc = 0.002$

c = confinement factor = 1.00

$y1 = 0.0025$

$sh1 = 0.008$

$ft1 = 886.7403$

$fy1 = 738.9503$

$su1 = 0.032$

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

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

$su1 = 0.4 \cdot esu1_{nominal}$ ((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 $\text{Min}(1, 1.25 \cdot (l_b/l_d)^{2/3})$, from 10.3.5, ASCE 41-17.

with $fs1 = (f_{s,jacket} \cdot A_{s,ten,jacket} + f_{s,core} \cdot A_{s,ten,core}) / A_{s,ten} = 738.9503$

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

$y2 = 0.0025$

$sh2 = 0.008$

$ft2 = 898.293$

$fy2 = 748.5775$

$su2 = 0.032$

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

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

$su2 = 0.4 \cdot esu2_{nominal}$ ((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  $fs2 = (fs_{jacket} \cdot Asl_{com,jacket} + fs_{core} \cdot Asl_{com,core}) / Asl_{com} = 748.5775$ 
with  $Es2 = (Es_{jacket} \cdot Asl_{com,jacket} + Es_{core} \cdot Asl_{com,core}) / Asl_{com} = 200000.00$ 
 $yv = 0.0025$ 
 $shv = 0.008$ 
 $ftv = 894.3662$ 
 $fyv = 745.3052$ 
 $suv = 0.032$ 
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  $fsv = (fs_{jacket} \cdot Asl_{mid,jacket} + fs_{mid} \cdot Asl_{mid,core}) / Asl_{mid} = 745.3052$ 
with  $Esv = (Es_{jacket} \cdot Asl_{mid,jacket} + Es_{mid} \cdot Asl_{mid,core}) / Asl_{mid} = 200000.00$ 
 $1 = Asl_{ten} / (b \cdot d) \cdot (fs1 / fc) = 0.38164216$ 
 $2 = Asl_{com} / (b \cdot d) \cdot (fs2 / fc) = 0.7508007$ 
 $v = Asl_{mid} / (b \cdot d) \cdot (fsv / fc) = 0.67946571$ 
and confined core properties:
 $b = 160.00$ 
 $d = 347.00$ 
 $d' = 13.00$ 
 $fcc (5A.2, TBDY) = 30.00$ 
 $cc (5A.5, TBDY) = 0.002$ 
 $c = \text{confinement factor} = 1.00$ 
 $1 = Asl_{ten} / (b \cdot d) \cdot (fs1 / fc) = 0.50454853$ 
 $2 = Asl_{com} / (b \cdot d) \cdot (fs2 / fc) = 0.99259314$ 
 $v = Asl_{mid} / (b \cdot d) \cdot (fsv / fc) = 0.898285$ 
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
--->
 $cu (4.10) = 0.58834972$ 
 $MRC (4.17) = 6.3496E+008$ 
--->
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
--->
 $v^* < v^*,y2$  - 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

--->

$*c_u (4.11) = 0.63603191$

$MR_o (4.18) = 4.4768E+008$

$MR_o < 0.8*MR_c$

--->

$u = c_u$ (unconfined full section) = $4.2871818E-005$

$\mu = MR_c$

Calculation of ratio l_b/l_d

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

Calculation of Shear Strength $V_r = \min(V_{r1}, V_{r2}) = 349917.706$

Calculation of Shear Strength at edge 1, $V_{r1} = 349917.706$

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

$V_{Col0} = 349917.706$

$knl = 1$ (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: $f'_c = (f'_c_{jacket} * Area_{jacket} + f'_c_{core} * Area_{core}) / Area_{section} = 25.00$, but $f'_c^{0.5} \leq 8.3$ MPa (22.5.3.1, ACI 318-14)

$M/V_d = 4.00$

$\mu_u = 3.4090E+006$

$V_u = 1.4594946E-005$

$d = 0.8 * h = 320.00$

$N_u = 2.1578E+006$

$A_g = 80000.00$

From (11.5.4.8), ACI 318-14: $V_s = V_{sjacket} + V_{s,core} = 471238.898$

where:

$V_{sjacket} = V_{sj1} + V_{sj2} = 471238.898$

$V_{sj1} = 314159.265$ is calculated for section web jacket, with:

$d = 320.00$

$A_v = 157079.633$

$f_y = 625.00$

$s = 100.00$

V_{sj1} is multiplied by $Col_{j1} = 1.00$

$s/d = 0.3125$

$V_{sj2} = 157079.633$ is calculated for section flange jacket, with:

$d = 160.00$

$A_v = 157079.633$

$f_y = 625.00$

$s = 100.00$

V_{sj2} is multiplied by $Col_{j2} = 1.00$

$s/d = 0.625$

$V_{s,core} = V_{s,c1} + V_{s,c2} = 0.00$

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

$d = 240.00$

$A_v = 100530.965$

$f_y = 500.00$

$s = 250.00$

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

$s/d = 1.04167$

$V_{s,c2} = 0.00$ is calculated for section flange core, with:

$d = 80.00$

$$A_v = 100530.965$$

$$f_y = 500.00$$

$$s = 250.00$$

$V_{s,c2}$ is multiplied by $Col,c2 = 0.00$

$$s/d = 3.125$$

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

$$\text{From (11-11), ACI 440: } V_s + V_f \leq 212577.225$$

$$bw = 200.00$$

Calculation of Shear Strength at edge 2, $V_{r2} = 349917.706$

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

$$V_{Col0} = 349917.706$$

$$knl = 1 \text{ (zero step-static loading)}$$

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).

$$= 1 \text{ (normal-weight concrete)}$$

$$\text{Mean concrete strength: } f'_c = (f'_c_{\text{jacket}} * \text{Area}_{\text{jacket}} + f'_c_{\text{core}} * \text{Area}_{\text{core}}) / \text{Area}_{\text{section}} = 25.00, \text{ but } f'_c^{0.5} \leq 8.3 \text{ MPa (22.5.3.1, ACI 318-14)}$$

$$M/Vd = 4.00$$

$$\mu_u = 15336.932$$

$$V_u = 1.4594946E-005$$

$$d = 0.8 * h = 320.00$$

$$N_u = 2.1578E+006$$

$$A_g = 80000.00$$

$$\text{From (11.5.4.8), ACI 318-14: } V_s = V_{s,jacket} + V_{s,core} = 471238.898$$

where:

$$V_{s,jacket} = V_{sj1} + V_{sj2} = 471238.898$$

$V_{sj1} = 314159.265$ is calculated for section web jacket, with:

$$d = 320.00$$

$$A_v = 157079.633$$

$$f_y = 625.00$$

$$s = 100.00$$

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

$$s/d = 0.3125$$

$V_{sj2} = 157079.633$ is calculated for section flange jacket, with:

$$d = 160.00$$

$$A_v = 157079.633$$

$$f_y = 625.00$$

$$s = 100.00$$

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

$$s/d = 0.625$$

$$V_{s,core} = V_{s,c1} + V_{s,c2} = 0.00$$

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

$$d = 240.00$$

$$A_v = 100530.965$$

$$f_y = 500.00$$

$$s = 250.00$$

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

$$s/d = 1.04167$$

$V_{s,c2} = 0.00$ is calculated for section flange core, with:

$$d = 80.00$$

$$A_v = 100530.965$$

$$f_y = 500.00$$

$$s = 250.00$$

$V_{s,c2}$ is multiplied by $Col,c2 = 0.00$

$$s/d = 3.125$$

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

$$\text{From (11-11), ACI 440: } V_s + V_f \leq 212577.225$$

$$bw = 200.00$$

End Of Calculation of Shear Capacity ratio for element: column JLC1 of floor 1

At local axis: 3

Start Of Calculation of Shear Capacity ratio for element: column JLC1 of floor 1

At Shear local axis: 2

(Bending local axis: 3)

Section Type: rcjlc

Constant Properties

Knowledge Factor, $\gamma = 0.90$

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

Consequently:

Jacket

New material of Secondary Member: Concrete Strength, $f_c = f_{cm} = 30.00$

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

Concrete Elasticity, $E_c = 25742.96$

Steel Elasticity, $E_s = 200000.00$

Existing Column

Existing material of Secondary Member: Concrete Strength, $f_c = f_{cm} = 18.00$

Existing material of Secondary Member: Steel Strength, $f_s = f_{sm} = 500.00$

Concrete Elasticity, $E_c = 19940.411$

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, $f_s = 1.25 \cdot f_{sm} = 625.00$

#####

Max Height, $H_{max} = 400.00$

Min Height, $H_{min} = 200.00$

Max Width, $W_{max} = 400.00$

Min Width, $W_{min} = 200.00$

Jacket Thickness, $t_j = 50.00$

Cover Thickness, $c = 15.00$

Mean Confinement Factor overall section = 1.00

Element Length, $L = 3000.00$

Secondary Member

Smooth Bars

Ductile Steel

Without Detailing for Earthquake Resistance (including stirrups not 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: 2

EDGE -A-

Shear Force, $V_a = -5.7948283E-005$

EDGE -B-

Shear Force, $V_b = 5.7948283E-005$

BOTH EDGES

Axial Force, $F = -2.1578E+006$

Longitudinal Reinforcement Area Distribution (in 2 divisions)

-Tension: $As_t = 0.00$

-Compression: $As_c = 5353.274$

Longitudinal Reinforcement Area Distribution (in 3 divisions)

-Tension: $As_{t,ten} = 1137.257$

-Compression: $As_{l,com} = 2208.54$

-Middle: $As_{l,mid} = 2007.478$

Calculation of Shear Capacity ratio , $V_e/V_r = 1.10794$

Member Controlled by Shear ($V_e/V_r > 1$)

Calculation of Shear Demand from fig. R18.6.5, ACI 318-14 $V_e = (M_{pr1} + M_{pr2})/l_n = 387689.388$
with

$M_{pr1} = \text{Max}(M_{u1+}, M_{u1-}) = 5.8153\text{E}+008$

$M_{u1+} = 5.8153\text{E}+008$, is the ultimate moment strength at the edge 1 of the member in the actual moment direction which is defined for the static loading combination

$M_{u1-} = 4.9554\text{E}+008$, 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

$M_{pr2} = \text{Max}(M_{u2+}, M_{u2-}) = 5.8153\text{E}+008$

$M_{u2+} = 5.8153\text{E}+008$, 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

$M_{u2-} = 4.9554\text{E}+008$, 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 M_{u1+}

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

$\phi_u = 1.1511372\text{E}-005$

$M_u = 5.8153\text{E}+008$

with full section properties:

$b = 400.00$

$d = 367.00$

$d' = 33.00$

$v = 0.48996389$

$N = 2.1578\text{E}+006$

$f_c = 30.00$

ϕ_c (5A.5, TBDY) = 0.002

Final value of ϕ_u : $\phi_u^* = \text{shear_factor} * \text{Max}(\phi_u, \phi_c) = 0.00925707$

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

From (5.4b), TBDY: $\phi_u = 0.00925707$

ϕ_{ue} (5.4c) = 0.0207149

ϕ_{ase} ((5.4d), TBDY) = $(\phi_{ase1} * A_{ext} + \phi_{ase2} * A_{int}) / A_{sec} = 0.33836562$

$\phi_{ase1} = \text{Max}(((A_{conf,max1} - A_{noConf1}) / A_{conf,max1}) * (A_{conf,min1} / A_{conf,max1}), 0) = 0.33836562$

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.

$A_{conf,max1} = 89600.00$ 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.

$A_{conf,min1} = 59225.00$ 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.

$A_{noConf1} = 43733.333$ is the unconfined external core area which is equal to $b_i^2/6$ as defined at (A.2).

$\phi_{ase2} (> \phi_{ase1}) = \text{Max}(((A_{conf,max2} - A_{noConf2}) / A_{conf,max2}) * (A_{conf,min2} / A_{conf,max2}), 0) = 0.33836562$

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.

$A_{conf,max2} = 45264.00$ 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.

$A_{conf,min2} = 0.00$ 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.

$A_{noConf2} = 32309.333$ is the unconfined internal core area which is equal to $b_i^2/6$ as defined at (A.2).

$\phi_{psh,min} * F_{ywe} = \text{Min}(\phi_{psh,x} * F_{ywe}, \phi_{psh,y} * F_{ywe}) = 6.12205$

Expression (5.4d) for $\phi_{psh,min} * F_{ywe}$ has been multiplied by 0.3 according to 15.7.1.3 for members without earthquake detailing (90° closed stirrups)

 $\phi_{psh,x} * F_{ywe} = \phi_{psh1} * F_{ywe1} + \phi_{psh2} * F_{ywe2} = 6.12205$

$psh1 ((5.4d), TBDY) = Lstir1 * Astir1 / (Asec * s1) = 0.00680678$
 $Lstir1$ (Length of stirrups along Y) = 1040.00
 $Astir1$ (stirrups area) = 78.53982
 $psh2 (5.4d) = Lstir2 * Astir2 / (Asec * s2) = 0.0012868$
 $Lstir2$ (Length of stirrups along Y) = 768.00
 $Astir2$ (stirrups area) = 50.26548

$psh_y * Fywe = psh1 * Fywe1 + ps2 * Fywe2 = 6.12205$
 $psh1 ((5.4d), TBDY) = Lstir1 * Astir1 / (Asec * s1) = 0.00680678$
 $Lstir1$ (Length of stirrups along X) = 1040.00
 $Astir1$ (stirrups area) = 78.53982
 $psh2 ((5.4d), TBDY) = Lstir2 * Astir2 / (Asec * s2) = 0.0012868$
 $Lstir2$ (Length of stirrups along X) = 768.00
 $Astir2$ (stirrups area) = 50.26548

Asec = 120000.00

s1 = 100.00

s2 = 250.00

fywe1 = 781.25

fywe2 = 625.00

fce = 30.00

From ((5.A5), TBDY), TBDY: cc = 0.002

c = confinement factor = 1.00

y1 = 0.0025

sh1 = 0.008

ft1 = 886.7403

fy1 = 738.9503

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 = 738.9503

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

y2 = 0.0025

sh2 = 0.008

ft2 = 898.293

fy2 = 748.5775

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 = 748.5775

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

yv = 0.0025

shv = 0.008

ftv = 894.3662

fyv = 745.3052

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 = 745.3052
with Esv = (Es,jacket*Asl,mid,jacket + Es,mid*Asl,mid,core)/Asl,mid = 200000.00
1 = Asl,ten/(b*d)*(fs1/fc) = 0.19082108
2 = Asl,com/(b*d)*(fs2/fc) = 0.37540035
v = Asl,mid/(b*d)*(fsv/fc) = 0.33973286
and confined core properties:
b = 360.00
d = 347.00
d' = 13.00
fcc (5A.2, TBDY) = 30.00
cc (5A.5, TBDY) = 0.002
c = confinement factor = 1.00
1 = Asl,ten/(b*d)*(fs1/fc) = 0.22424379
2 = Asl,com/(b*d)*(fs2/fc) = 0.44115251
v = Asl,mid/(b*d)*(fsv/fc) = 0.39923778
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) = 0.42533033
MRc (4.17) = 7.2277E+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) = 0.50069522
MRo (4.18) = 5.8153E+008
---->
u = cu (4.2) = 1.1511372E-005
Mu = MRo

```

Calculation of ratio lb/ld

Adequate Lap Length: lb/ld >= 1

Calculation of Mu1-

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

$$\mu = 3.0526570E-005$$

$$\mu_u = 4.9554E+008$$

with full section properties:

$$b = 200.00$$

$$d = 367.00$$

$$d' = 33.00$$

$$v = 0.97992777$$

$$N = 2.1578E+006$$

$$f_c = 30.00$$

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

$$\text{Final value of } \mu_u: \mu_u^* = \text{shear_factor} * \text{Max}(\mu_u, \alpha) = 0.00925707$$

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

$$\text{From (5.4b), TBDY: } \mu_u = 0.00925707$$

$$\mu_{ue} (5.4c) = 0.0207149$$

$$\alpha_{se} ((5.4d), TBDY) = (\alpha_{se1} * A_{ext} + \alpha_{se2} * A_{int}) / A_{sec} = 0.33836562$$

$$\alpha_{se1} = \text{Max}(((A_{conf,max1} - A_{noConf1}) / A_{conf,max1}) * (A_{conf,min1} / A_{conf,max1}), 0) = 0.33836562$$

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.

$A_{conf,max1} = 89600.00$ 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.

$A_{conf,min1} = 59225.00$ 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.

$A_{noConf1} = 43733.333$ is the unconfined external core area which is equal to $b_i^2/6$ as defined at (A.2).

$$\alpha_{se2} (> \alpha_{se1}) = \text{Max}(((A_{conf,max2} - A_{noConf2}) / A_{conf,max2}) * (A_{conf,min2} / A_{conf,max2}), 0) = 0.33836562$$

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.

$A_{conf,max2} = 45264.00$ 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.

$A_{conf,min2} = 0.00$ 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.

$A_{noConf2} = 32309.333$ is the unconfined internal core area which is equal to $b_i^2/6$ as defined at (A.2).

$$p_{sh,min} * F_{ywe} = \text{Min}(p_{sh,x} * F_{ywe}, p_{sh,y} * F_{ywe}) = 6.12205$$

Expression (5.4d) for $p_{sh,min} * F_{ywe}$ has been multiplied by 0.3 according to 15.7.1.3 for members without earthquake detailing (90° closed stirrups)

$$p_{sh,x} * F_{ywe} = p_{sh1} * F_{ywe1} + p_{sh2} * F_{ywe2} = 6.12205$$

$$p_{sh1} ((5.4d), TBDY) = L_{stir1} * A_{stir1} / (A_{sec} * s_1) = 0.00680678$$

$$L_{stir1} (\text{Length of stirrups along Y}) = 1040.00$$

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

$$p_{sh2} (5.4d) = L_{stir2} * A_{stir2} / (A_{sec} * s_2) = 0.0012868$$

$$L_{stir2} (\text{Length of stirrups along Y}) = 768.00$$

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

$$p_{sh,y} * F_{ywe} = p_{sh1} * F_{ywe1} + p_{sh2} * F_{ywe2} = 6.12205$$

$$p_{sh1} ((5.4d), TBDY) = L_{stir1} * A_{stir1} / (A_{sec} * s_1) = 0.00680678$$

$$L_{stir1} (\text{Length of stirrups along X}) = 1040.00$$

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

$$p_{sh2} ((5.4d), TBDY) = L_{stir2} * A_{stir2} / (A_{sec} * s_2) = 0.0012868$$

$$L_{stir2} (\text{Length of stirrups along X}) = 768.00$$

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

```

Asec = 120000.00
s1 = 100.00
s2 = 250.00
fywe1 = 781.25
fywe2 = 625.00
fce = 30.00
From ((5.A.5), TBDY), TBDY: cc = 0.002
c = confinement factor = 1.00
y1 = 0.0025
sh1 = 0.008
ft1 = 898.293
fy1 = 748.5775
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 Min(1,1.25*(lb/lb,min)^ 2/3), from 10.3.5, ASCE 41-17.
with fs1 = (fs,jacket*Asl,ten,jacket + fs,core*Asl,ten,core)/Asl,ten = 748.5775
with Es1 = (Es,jacket*Asl,ten,jacket + Es,core*Asl,ten,core)/Asl,ten = 200000.00
y2 = 0.0025
sh2 = 0.008
ft2 = 886.7403
fy2 = 738.9503
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,min)^ 2/3), from 10.3.5, ASCE 41-17.
with fs2 = (fs,jacket*Asl,com,jacket + fs,core*Asl,com,core)/Asl,com = 738.9503
with Es2 = (Es,jacket*Asl,com,jacket + Es,core*Asl,com,core)/Asl,com = 200000.00
yv = 0.0025
shv = 0.008
ftv = 894.3662
fyv = 745.3052
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 Min(1,1.25*(lb/lb,min)^ 2/3), from 10.3.5, ASCE 41-17.
with fsv = (fs,jacket*Asl,mid,jacket + fs,mid*Asl,mid,core)/Asl,mid = 745.3052
with Esv = (Es,jacket*Asl,mid,jacket + Es,mid*Asl,mid,core)/Asl,mid = 200000.00
1 = Asl,ten/(b*d)*(fs1/fc) = 0.7508007
2 = Asl,com/(b*d)*(fs2/fc) = 0.38164216
v = Asl,mid/(b*d)*(fsv/fc) = 0.67946571
and confined core properties:
b = 160.00
d = 347.00
d' = 13.00
fcc (5A.2, TBDY) = 30.00
cc (5A.5, TBDY) = 0.002

```

```

c = confinement factor = 1.00
1 = Asl,ten/(b*d)*(fs1/fc) = 0.99259314
2 = Asl,com/(b*d)*(fs2/fc) = 0.50454853
v = Asl,mid/(b*d)*(fsv/fc) = 0.898285
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) = 0.8262842
MRc (4.18) = 4.9554E+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) = 0.90160009
MRo (4.18) = 2.4685E+008
MRo < 0.8*MRc
---->
u = cu (unconfined full section) = 3.0526570E-005
Mu = MRc
-----

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 = 1.1511372E-005
Mu = 5.8153E+008
-----

with full section properties:
b = 400.00

```

$d = 367.00$
 $d' = 33.00$
 $v = 0.48996389$
 $N = 2.1578E+006$
 $f_c = 30.00$
 $co (5A.5, TBDY) = 0.002$
 Final value of cu : $cu^* = shear_factor * Max(cu, cc) = 0.00925707$
 The Shear_factor is considered equal to 1 (pure moment strength)
 From (5.4b), TBDY: $cu = 0.00925707$
 $we (5.4c) = 0.0207149$
 $ase ((5.4d), TBDY) = (ase1 * A_{ext} + ase2 * A_{int}) / A_{sec} = 0.33836562$
 $ase1 = Max(((A_{conf,max1} - A_{noConf1}) / A_{conf,max1}) * (A_{conf,min1} / A_{conf,max1}), 0) = 0.33836562$
 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.
 $A_{conf,max1} = 89600.00$ 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.
 $A_{conf,min1} = 59225.00$ 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.
 $A_{noConf1} = 43733.333$ is the unconfined external core area which is equal to $b_i^2/6$ as defined at (A.2).
 $ase2 (>= ase1) = Max(((A_{conf,max2} - A_{noConf2}) / A_{conf,max2}) * (A_{conf,min2} / A_{conf,max2}), 0) = 0.33836562$
 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.
 $A_{conf,max2} = 45264.00$ 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.
 $A_{conf,min2} = 0.00$ 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.
 $A_{noConf2} = 32309.333$ is the unconfined internal core area which is equal to $b_i^2/6$ as defined at (A.2).
 $psh_{min} * F_{ywe} = Min(psh_x * F_{ywe}, psh_y * F_{ywe}) = 6.12205$
 Expression (5.4d) for $psh_{min} * F_{ywe}$ has been multiplied by 0.3 according to 15.7.1.3 for members without earthquake detailing (90° closed stirrups)

$psh_x * F_{ywe} = psh1 * F_{ywe1} + ps2 * F_{ywe2} = 6.12205$
 $psh1 ((5.4d), TBDY) = L_{stir1} * A_{stir1} / (A_{sec} * s1) = 0.00680678$
 L_{stir1} (Length of stirrups along Y) = 1040.00
 A_{stir1} (stirrups area) = 78.53982
 $psh2 (5.4d) = L_{stir2} * A_{stir2} / (A_{sec} * s2) = 0.0012868$
 L_{stir2} (Length of stirrups along Y) = 768.00
 A_{stir2} (stirrups area) = 50.26548

$psh_y * F_{ywe} = psh1 * F_{ywe1} + ps2 * F_{ywe2} = 6.12205$
 $psh1 ((5.4d), TBDY) = L_{stir1} * A_{stir1} / (A_{sec} * s1) = 0.00680678$
 L_{stir1} (Length of stirrups along X) = 1040.00
 A_{stir1} (stirrups area) = 78.53982
 $psh2 ((5.4d), TBDY) = L_{stir2} * A_{stir2} / (A_{sec} * s2) = 0.0012868$
 L_{stir2} (Length of stirrups along X) = 768.00
 A_{stir2} (stirrups area) = 50.26548

$A_{sec} = 120000.00$
 $s1 = 100.00$
 $s2 = 250.00$
 $f_{ywe1} = 781.25$
 $f_{ywe2} = 625.00$
 $f_{ce} = 30.00$
 From ((5.A5), TBDY), TBDY: $cc = 0.002$
 $c =$ confinement factor = 1.00
 $y1 = 0.0025$
 $sh1 = 0.008$
 $ft1 = 886.7403$
 $fy1 = 738.9503$

```

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 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 = 738.9503
with Es1 = (Es,jacket*Asl,ten,jacket + Es,core*Asl,ten,core)/Asl,ten = 200000.00
y2 = 0.0025
sh2 = 0.008
ft2 = 898.293
fy2 = 748.5775
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 = 748.5775
with Es2 = (Es,jacket*Asl,com,jacket + Es,core*Asl,com,core)/Asl,com = 200000.00
yv = 0.0025
shv = 0.008
ftv = 894.3662
fyv = 745.3052
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 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 = 745.3052
with Esv = (Es,jacket*Asl,mid,jacket + Es,mid*Asl,mid,core)/Asl,mid = 200000.00
1 = Asl,ten/(b*d)*(fs1/fc) = 0.19082108
2 = Asl,com/(b*d)*(fs2/fc) = 0.37540035
v = Asl,mid/(b*d)*(fsv/fc) = 0.33973286
and confined core properties:
b = 360.00
d = 347.00
d' = 13.00
fcc (5A.2, TBDY) = 30.00
cc (5A.5, TBDY) = 0.002
c = confinement factor = 1.00
1 = Asl,ten/(b*d)*(fs1/fc) = 0.22424379
2 = Asl,com/(b*d)*(fs2/fc) = 0.44115251
v = Asl,mid/(b*d)*(fsv/fc) = 0.39923778
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) = 0.42533033
 M_{Rc} (4.17) = 7.2277E+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: 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, e_c

--->
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) = 0.50069522
 M_{Ro} (4.18) = 5.8153E+008
--->

$u = c_u$ (4.2) = 1.1511372E-005
 $\mu = M_{Ro}$

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:

$u = 3.0526570E-005$
 $\mu = 4.9554E+008$

with full section properties:

$b = 200.00$
 $d = 367.00$
 $d' = 33.00$
 $v = 0.97992777$
 $N = 2.1578E+006$

$f_c = 30.00$

c_o (5A.5, TBDY) = 0.002

Final value of c_u : $c_u^* = \text{shear_factor} * \text{Max}(c_u, c_c) = 0.00925707$

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

From (5.4b), TBDY: $c_u = 0.00925707$

w_e (5.4c) = 0.0207149

a_{se} ((5.4d), TBDY) = $(a_{se1} * A_{ext} + a_{se2} * A_{int}) / A_{sec} = 0.33836562$

$a_{se1} = \text{Max}(((A_{conf,max1} - A_{noConf1}) / A_{conf,max1}) * (A_{conf,min1} / A_{conf,max1}), 0) = 0.33836562$

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.

Aconf,max1 = 89600.00 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 = 59225.00 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 = 43733.333 is the unconfined external core area which is equal to $b_i^2/6$ as defined at (A.2).
ase2 (\geq ase1) = $\text{Max}(((A_{\text{conf,max2}} - A_{\text{noConf2}})/A_{\text{conf,max2}}) * (A_{\text{conf,min2}}/A_{\text{conf,max2}}), 0) = 0.33836562$

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 = 45264.00 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 = 0.00 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 = 32309.333 is the unconfined internal core area which is equal to $b_i^2/6$ as defined at (A.2).
psh,min*Fywe = $\text{Min}(psh_x * Fywe, psh_y * Fywe) = 6.12205$

Expression (5.4d) for psh,min*Fywe has been multiplied by 0.3 according to 15.7.1.3 for members without
earthquake detailing (90° closed stirrups)

psh_x*Fywe = psh1*Fywe1 + ps2*Fywe2 = 6.12205
psh1 ((5.4d), TBDY) = $L_{\text{stir1}} * A_{\text{stir1}} / (A_{\text{sec}} * s1) = 0.00680678$
Lstir1 (Length of stirrups along Y) = 1040.00
Astir1 (stirrups area) = 78.53982
psh2 ((5.4d), TBDY) = $L_{\text{stir2}} * A_{\text{stir2}} / (A_{\text{sec}} * s2) = 0.0012868$
Lstir2 (Length of stirrups along Y) = 768.00
Astir2 (stirrups area) = 50.26548

psh_y*Fywe = psh1*Fywe1 + ps2*Fywe2 = 6.12205
psh1 ((5.4d), TBDY) = $L_{\text{stir1}} * A_{\text{stir1}} / (A_{\text{sec}} * s1) = 0.00680678$
Lstir1 (Length of stirrups along X) = 1040.00
Astir1 (stirrups area) = 78.53982
psh2 ((5.4d), TBDY) = $L_{\text{stir2}} * A_{\text{stir2}} / (A_{\text{sec}} * s2) = 0.0012868$
Lstir2 (Length of stirrups along X) = 768.00
Astir2 (stirrups area) = 50.26548

Asec = 120000.00
s1 = 100.00
s2 = 250.00
fywe1 = 781.25
fywe2 = 625.00
fce = 30.00

From ((5.A5), TBDY), TBDY: cc = 0.002
c = confinement factor = 1.00

y1 = 0.0025
sh1 = 0.008
ft1 = 898.293
fy1 = 748.5775
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/lb = 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 $\text{Min}(1, 1.25 * (lb/lb)^{2/3})$, from 10.3.5, ASCE 41-17.

with fs1 = $(f_{s,jacket} * A_{s,ten,jacket} + f_{s,core} * A_{s,ten,core}) / A_{s,ten} = 748.5775$

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 = 886.7403
fy2 = 738.9503
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/ld)^ 2/3), from 10.3.5, ASCE 41-17.
    with fs2 = (fs,jacket*Asl,com,jacket + fs,core*Asl,com,core)/Asl,com = 738.9503
    with Es2 = (Es,jacket*Asl,com,jacket + Es,core*Asl,com,core)/Asl,com = 200000.00
    yv = 0.0025
    shv = 0.008
ftv = 894.3662
fyv = 745.3052
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 = 745.3052
    with Esv = (Es,jacket*Asl,mid,jacket + Es,mid*Asl,mid,core)/Asl,mid = 200000.00
    1 = Asl,ten/(b*d)*(fs1/fc) = 0.7508007
    2 = Asl,com/(b*d)*(fs2/fc) = 0.38164216
    v = Asl,mid/(b*d)*(fsv/fc) = 0.67946571
and confined core properties:
b = 160.00
d = 347.00
d' = 13.00
fcc (5A.2, TBDY) = 30.00
cc (5A.5, TBDY) = 0.002
    c = confinement factor = 1.00
    1 = Asl,ten/(b*d)*(fs1/fc) = 0.99259314
    2 = Asl,com/(b*d)*(fs2/fc) = 0.50454853
    v = Asl,mid/(b*d)*(fsv/fc) = 0.898285
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) = 0.8262842
MRc (4.18) = 4.9554E+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, 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

---->

$*cu$ (4.11) = 0.90160009

M_{Ro} (4.18) = 2.4685E+008

$M_{Ro} < 0.8 * M_{Rc}$

---->

$u = cu$ (unconfined full section) = 3.0526570E-005

$\mu = M_{Rc}$

Calculation of ratio l_b/d

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

Calculation of Shear Strength $V_r = \min(V_{r1}, V_{r2}) = 349917.706$

Calculation of Shear Strength at edge 1, $V_{r1} = 349917.706$

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

$V_{Col0} = 349917.706$

$k_{nl} = 1$ (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: $f'_c = (f'_{c_jacket} * Area_jacket + f'_{c_core} * Area_core) / Area_section = 25.00$, but $f'_c^{0.5} \leq 8.3$ MPa (22.5.3.1, ACI 318-14)

$M/Vd = 4.00$

$\mu_u = 8.5591E+007$

$V_u = 5.7948283E-005$

$d = 0.8 * h = 320.00$

$N_u = 2.1578E+006$

$A_g = 80000.00$

From (11.5.4.8), ACI 318-14: $V_s = V_{sjacket} + V_{s,core} = 471238.898$

where:

$V_{sjacket} = V_{sj1} + V_{sj2} = 471238.898$

$V_{sj1} = 157079.633$ is calculated for section web jacket, with:

$d = 160.00$

$A_v = 157079.633$

$f_y = 625.00$

$s = 100.00$

V_{sj1} is multiplied by $Col_{j1} = 1.00$

$s/d = 0.625$

$V_{sj2} = 314159.265$ is calculated for section flange jacket, with:

$d = 320.00$

$A_v = 157079.633$

$f_y = 625.00$

$s = 100.00$

V_{sj2} is multiplied by $Col_{j2} = 1.00$

$s/d = 0.3125$

$V_{s,core} = V_{s,c1} + V_{s,c2} = 0.00$
 $V_{s,c1} = 0.00$ is calculated for section web core, with:
 $d = 80.00$
 $A_v = 100530.965$
 $f_y = 500.00$
 $s = 250.00$
 $V_{s,c1}$ is multiplied by $Col,c1 = 0.00$
 $s/d = 3.125$
 $V_{s,c2} = 0.00$ is calculated for section flange core, with:
 $d = 240.00$
 $A_v = 100530.965$
 $f_y = 500.00$
 $s = 250.00$
 $V_{s,c2}$ is multiplied by $Col,c2 = 0.00$
 $s/d = 1.04167$
 $V_f ((11-3)-(11.4), ACI 440) = 0.00$
 From (11-11), ACI 440: $V_s + V_f \leq 212577.225$
 $bw = 200.00$

Calculation of Shear Strength at edge 2, $V_{r2} = 349917.706$
 $V_{r2} = V_{Col} ((10.3), ASCE 41-17) = knl * V_{Col0}$
 $V_{Col0} = 349917.706$
 $knl = 1$ (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: $f'_c = (f'_c_jacket * Area_jacket + f'_c_core * Area_core) / Area_section = 25.00$, but $f'_c^{0.5} \leq 8.3$
 MPa (22.5.3.1, ACI 318-14)
 $M/Vd = 4.00$
 $\mu_u = 6.4291E+007$
 $\nu_u = 5.7948283E-005$
 $d = 0.8 * h = 320.00$
 $N_u = 2.1578E+006$
 $A_g = 80000.00$
 From (11.5.4.8), ACI 318-14: $V_s = V_{s,jacket} + V_{s,core} = 471238.898$
 where:
 $V_{s,jacket} = V_{s,j1} + V_{s,j2} = 471238.898$
 $V_{s,j1} = 157079.633$ is calculated for section web jacket, with:
 $d = 160.00$
 $A_v = 157079.633$
 $f_y = 625.00$
 $s = 100.00$
 $V_{s,j1}$ is multiplied by $Col,j1 = 1.00$
 $s/d = 0.625$
 $V_{s,j2} = 314159.265$ is calculated for section flange jacket, with:
 $d = 320.00$
 $A_v = 157079.633$
 $f_y = 625.00$
 $s = 100.00$
 $V_{s,j2}$ is multiplied by $Col,j2 = 1.00$
 $s/d = 0.3125$
 $V_{s,core} = V_{s,c1} + V_{s,c2} = 0.00$
 $V_{s,c1} = 0.00$ is calculated for section web core, with:
 $d = 80.00$
 $A_v = 100530.965$
 $f_y = 500.00$
 $s = 250.00$
 $V_{s,c1}$ is multiplied by $Col,c1 = 0.00$
 $s/d = 3.125$
 $V_{s,c2} = 0.00$ is calculated for section flange core, with:
 $d = 240.00$
 $A_v = 100530.965$
 $f_y = 500.00$
 $s = 250.00$

$V_{s,c2}$ is multiplied by $Col,c2 = 0.00$
 $s/d = 1.04167$
 $V_f ((11-3)-(11.4), ACI 440) = 0.00$
From (11-11), ACI 440: $V_s + V_f \leq 212577.225$
 $bw = 200.00$

End Of Calculation of Shear Capacity ratio for element: column JLC1 of floor 1
At local axis: 2

Start Of Calculation of Chord Rotation Capacity for element: column JLC1 of floor 1
At local axis: 3
Integration Section: (b)
Section Type: rcjlc3

Constant Properties

Knowledge Factor, $\gamma = 0.90$
Chord Rotation is generally considered as Deformation-Controlled Action according to Table C7-1, ASCE 41-17.
Mean strengths are used for Deformation-Controlled Actions according to 7.5.1.3, ASCE 41-17
Consequently:
Jacket
New material of Secondary Member: Concrete Strength, $f_c = f_{cm} = 30.00$
New material of Secondary Member: Steel Strength, $f_s = f_{sm} = 625.00$
Concrete Elasticity, $E_c = 25742.96$
Steel Elasticity, $E_s = 200000.00$
Existing Column
Existing material of Secondary Member: Concrete Strength, $f_c = f_{cm} = 18.00$
Existing material of Secondary Member: Steel Strength, $f_s = f_{sm} = 500.00$
Concrete Elasticity, $E_c = 19940.411$
Steel Elasticity, $E_s = 200000.00$
Max Height, $H_{max} = 400.00$
Min Height, $H_{min} = 200.00$
Max Width, $W_{max} = 400.00$
Min Width, $W_{min} = 200.00$
Jacket Thickness, $t_j = 50.00$
Cover Thickness, $c = 15.00$
Element Length, $L = 3000.00$
Secondary Member
Smooth Bars
Ductile Steel
Without Detailing for Earthquake Resistance (including stirrups not 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, $M = -6.4291E+007$
Shear Force, $V_2 = 0.60572401$
Shear Force, $V_3 = 1.4589530E-005$
Axial Force, $F = -2.1578E+006$
Longitudinal Reinforcement Area Distribution (in 2 divisions)
-Tension: $As_t = 0.00$
-Compression: $As_c = 5353.274$
Longitudinal Reinforcement Area Distribution (in 3 divisions)
-Tension: $As_{t,ten} = 1137.257$
-Compression: $As_{l,com} = 2208.54$
-Middle: $As_{l,mid} = 2007.478$
Longitudinal External Reinforcement Area Distribution (in 3 divisions)
-Tension: $As_{l,ten,jacket} = 829.3805$
-Compression: $As_{l,com,jacket} = 1746.726$
-Middle: $As_{l,mid,jacket} = 1545.664$

Longitudinal Internal Reinforcement Area Distribution (in 3 divisions)

-Tension: $A_{sl,ten,core} = 307.8761$

-Compression: $A_{sl,com,core} = 461.8141$

-Middle: $A_{sl,mid,core} = 461.8141$

Mean Diameter of Tension Reinforcement, $DbL = 16.80$

Existing component: From table 7-7, ASCE 41_17: Final chord rotation Capacity $u_R = u = 0.03784131$

$u = y + p = 0.0420459$

- Calculation of y -

$y = (M_y * L_s / 3) / E_{eff} = 0.0420459$ ((4.29), Biskinis Phd))

$M_y = 5.2455E+008$

$L_s = M/V$ (with $L_s > 0.1 * L$ and $L_s < 2 * L$) = 6000.00

From table 10.5, ASCE 41_17: $E_{eff} = factor * E_c * I_g = 2.4951E+013$

factor = 0.70

$A_g = 120000.00$

Mean concrete strength: $fc' = (fc'_{jacket} * Area_{jacket} + fc'_{core} * Area_{core}) / Area_{section} = 25.00$

$N = 2.1578E+006$

$E_c * I_g = E_{c,jacket} * I_{g,jacket} + E_{c,core} * I_{g,core} = 3.5645E+013$

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, $b = 400.00$

web width, $b_w = 200.00$

flange thickness, $t = 200.00$

$y = \text{Min}(y_{ten}, y_{com})$

$y_{ten} = 1.4921411E-005$

with $f_y = 596.2441$

$d = 367.00$

$y = 0.45559994$

$A = 0.06111896$

$B = 0.04120454$

with $pt = 0.00774698$

$pc = 0.01504455$

$pv = 0.01367492$

$N = 2.1578E+006$

$b = 400.00$

$" = 0.08991826$

$y_{comp} = 1.1520096E-005$

with $fc = 30.00$

$E_c = 25742.96$

$y = 0.49615013$

$A = 0.00143$

$B = 0.01655203$

with $E_s = 200000.00$

CONFIRMATION: $y = 0.49615013 < t/d$

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 $l_b/l_d \geq 1$
shear control ratio $V_yE/V_{ColOE} = 1.10794$

$$d = d_{\text{external}} = 367.00$$

$$s = s_{\text{external}} = 100.00$$

- $t = s_1 + s_2 + 2 \cdot t_f/bw \cdot (f_{fe}/f_s) = 0.00809358$

$$\text{jacket: } s_1 = A_{v1} \cdot L_{\text{stir1}} / (s_1 \cdot A_g) = 0.00680678$$

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

$L_{\text{stir1}} = 1040.00$, is the total Length of all stirrups parallel to loading (shear) direction

$$s_1 = 100.00$$

$$\text{core: } s_2 = A_{v2} \cdot L_{\text{stir2}} / (s_2 \cdot A_g) = 0.0012868$$

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

$L_{\text{stir2}} = 768.00$, 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.

$$N_{UD} = 2.1578E+006$$

$$A_g = 120000.00$$

$$f_{cE} = (f_{c,jacket} \cdot \text{Area}_{jacket} + f_{c,core} \cdot \text{Area}_{core}) / \text{section_area} = 25.00$$

$$f_{yIE} = (f_{y,ext_Long_Reinf} \cdot \text{Area}_{ext_Long_Reinf} + f_{y,int_Long_Reinf} \cdot \text{Area}_{int_Long_Reinf}) / \text{Area}_{Tot_Long_Rein} = 596.2441$$

$$f_{ytE} = (f_{y,ext_Trans_Reinf} \cdot s_1 + f_{y,int_Trans_Reinf} \cdot s_2) / (s_1 + s_2) = 605.1263$$

$$\rho_l = \text{Area}_{Tot_Long_Rein} / (b \cdot d) = 0.03646644$$

$$b = 400.00$$

$$d = 367.00$$

$$f_{cE} = 25.00$$

End Of Calculation of Chord Rotation Capacity for element: column JLC1 of floor 1

At local axis: 3

Integration Section: (b)
