TN175_EC2_implementation_040409

EUROPEAN CODE REQUIREMENTS FOR DESIGN OF CONCRETE FLOORS, INCLUDING POST-TENSIONING[1]

This Technical Note details the requirements of the European Code EC2 (ENV 1992-1-1:2004) for the design of concrete floor systems, including post-tensioning, and the implementation of these requirements in the Builder Platform programs.

The implementation follows the EC2 's procedure of calculating a "Demand," referred to as "design value" for each design section, and a "Resistance," for the same section, referred to as "design capacity." "Design value" and "design capacity" are generic terms that apply to displacements as well as actions. For each loading condition, or instance defined in EC2, the design is achieved by making the "resistance" exceed the associated demand "Design Value". Where necessary, reinforcement is added to meet this condition.

The implementation is broken down into the following steps:

- Serviceability limit state (SLS)
  - Check for computed stresses in concrete
  - Check for cracking and crack reinforcement
  - Minimum reinforcement
    - Based on crack width
    - Based on geometry
  - Deflection check

- Strength limit state (ULS)
  - Bending of section
    - With or without prestressing
    - With or without axial loading
  - Punching shear (two-way shear)
  - Beam shear (one-way shear)

- Initial condition (transfer of prestressing) (ILS)
  - Check for computed stresses in concrete
  - Provide rebar

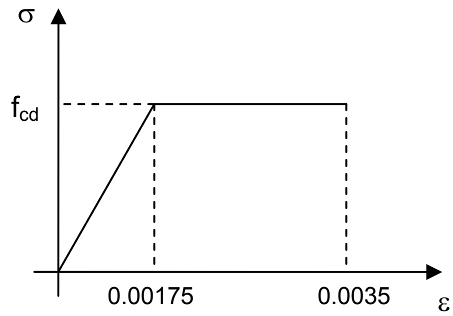
In the following, the values in square brackets "[ ]" are defaults of the program. They can be changed by the user.

MATERIAL AND MATERIAL FACTORS

Concrete

- Cylinder strength at 28 days, as specified by the user
  f_{ck} = characteristic compressive cylinder strength at 28 days;

- Bilinear stress/strain diagram with the horizontal branch at f_{cd} ; maximum strain at 0.0035; strain at limit of proportionality 0.00175



- Modulus of elasticity of concrete is automatically calculated and displayed by the program using f_{ck}, and the relationship (2.1-15)[2] of the code given below. User is given the option to override the code value and specify a user defined substitute.

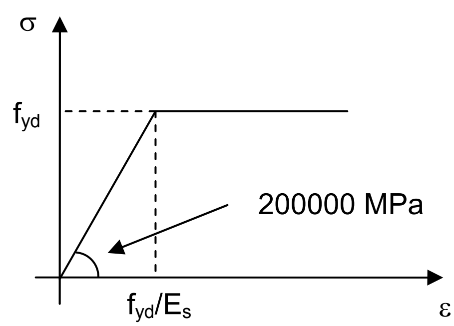
$$E_{ci} = E_{co}[(f_{ck}+\Delta f)/f_{cmo}]^{1/3}$$

where,

E_{ci}	=	modulus of elasticity at 28 days
E_{co}	=	2.15×10^4 MPa
f_{ck}	=	characteristic cylinder strength at 28 days
Δf	=	8 MPa
f_{cmo}	=	10 MPa

Nonprestressed Steel

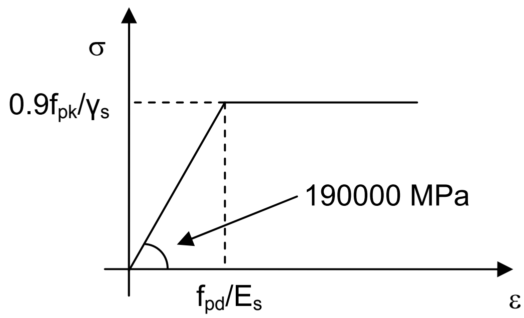
- Bilinear stress/strain diagram with the horizontal branch at $f_{yd}=f_{yk}/\gamma_s$
- Modulus of elasticity is user defined [200000 MPa]
- No limit on tensile strain is imposed



Prestressing Steel

- Bilinear stress/strain diagram with the horizontal branch at $(0.90f_{pk})/\gamma_s$
- Modulus of elasticity is user defined [190000 MPa]
- No limit on tensile strain is imposed

[2] CEB-FIP MODEL CODE 1990



Material Factors

- Concrete $\gamma_s = 1.50$
- Nonprestressed steel $\gamma_s = 1.15$
- Prestressing steel $\gamma_s = 1.15$

LOAD COMBINATIONS

The program automatically generates the following load combinations and performs the associated design checks. The default load combinations of the program can be edited by the user. Also, users can define additional load combinations.

- Service (quasi-permanent)
- Service (frequent)
- Strength condition
- Initial (transfer)

In addition to the above, the program has a “No code check” option for combinations, when a user is interested in the response of the floor system to a user defined loading, as opposed to performing a code check.

- No code check

The parts and factors of the program’s automatically generated load cases and load combinations are listed below. Except for the initial (transfer) condition, which is not explicitly defined in the code, the remainder of the combinations follows EC2 stipulations.

- Service (quasi-permanent)
  1.00 x Selfweight + 1.00 x Dead load + 0.30 x Live load + 1.00 x Prestressing

- Service (frequent)
  1.00 x Selfweight + 1.00 x Dead load + 0.50 x Live load + 1.00 x Prestressing

- Strength
  1.35 x Selfweight + 1.35 x Dead load + 1.50 x Live load + 1.00 x Hyperstatic

- Initial (transfer)
  1.00 x Selfweight + 1.15 x Prestressing
  The factors of the initial condition are based upon the common practice of engineers in the USA.

DESIGN FOR FLEXURE WITH OR WITHOUT AXIAL LOAD

Serviceability Check

- For “frequent” load condition the code stipulated stress limitations explained below are used as default. However, user can edit the default values.
  - Concrete
    - Maximum compressive stress[3] 0.60 f_{ck}. If calculated stress at any location exceeds the allowable, the program identifies the location graphically on the screen and notes it in its tabular reports.
    - The maximum allowable hypothetical tensile stress[4] used is (fct,eff). Where calculated values exceed this threshold, the program provided reinforcement to control cracking.
  - Nonprestressed Reinforcement
    - The maximum allowable stress (0.80 f_{yk}) given in the code is used. If the calculated stress exceeds the allowable value, the program automatically increases the area of steel to lower the calculated stress to the code specified limit.
  - Prestressing steel
    The maximum allowable stress under service condition (0.75 f_{yk}) given in the code is used. If this value exceeds, the program report it to the user on the computer screen.

- Stress limitations used for the “quasi-permanent” load combination are as follows:
  - Concrete
    - Maximum compressive stress 0.45 f_{ck} . If stress at any location exceeds, the program displays that location with a change in color (or broken lines for black and white display), along with a note on the text output.
    - The maximum allowable hypothetical tensile stress used is (fct,eff). Where calculated values exceed this threshold, the program provides more reinforcement to limit the crack width to the code specified limit and to control cracking.
  - Nonprestressed Reinforcement
    - None required – no check made
  - Prestressing steel
    - None required - no check made

- Cracking and reinforcement for crack control
  The minimum reinforcement for crack control (A_{smin}) is based on section 7.3.2 of the code. The following values are used in the evaluation of A_{smin}.

$$A_{s\,min} = \frac{k_c \times k \times f_{ct,\,eff} \times A_{ct}}{\sigma_s}$$

where,

[3] EN 1992-1-1:2004(E), Section 7.2(2)
[4] EN 1992-1-1:2004(E), Section 7.3.2(4)

k = for members with least dimension <= 300 mm $k = 1.0$
for members with least dimension >= 800 mm $k = 0.65$
for other members, linear interpolation is used.

k_c = is determined based on the maximum fiber stresses as follows:

- For pure tension $k_c = 1.0$
- For bending or bending combined with axial forces:

  - For rectangular sections and webs of box sections and T-sections:

$$k_c = 0.4\left[1 - \frac{\sigma_c}{k_1(h/h^*)f_{ct,eff}}\right] \text{, but not greater than 1}$$

  - For flanges of box sections and T-sections:

$$k_c = 0.9\frac{F_{cr}}{A_{ct}f_{ct,eff}} \geq 0.5$$

where

$\sigma_c = \dfrac{N_{ED}}{bh}$; average precompression

N_{ED} = Axial force at the serviceability limit state.
h^* = h for h <1.0 m
= 1.0m   for h ≥1.0 m

k_1 = 1.5   if N_{ED} is a compressive force
= $2h^*/3h$ if N_{ED} is a tensile force

F_{cr} = Absolute value of the tensile force within the flange due to the cracking moment calculated with $f_{ct,eff.}$

$f_{ct,eff}$ = tensile strength of concrete at time of crack formation, f_{ctm}, but not less than 3 MPa

f_{ctm} = mean axial tensile strength according to Table 3.1 of the code

- Minimum Overall Reinforcement
  Each design section is checked to satisfy the minimum overall reinforcement (A_s), using Section 9.2.1.1 of the code.

A_s >= (0.26 b_t d f_{ctm}/ f_{yk}) , but not less than 0.0013 b_t d

where

d = depth to the centroid of the mild steel. If no mild steel is required, the cover specified by the user and the user's choice of reinforcement is used to calculate "d"

b_t = mean width of the tension zone.

f_{pk} is used in lieu of f_{yk} , when section is prestressed.

If both prestressing and nonprestressed steel are present, weighted average of their characteristic strengths is used.

$$A_s = A_s + A_{ps} \times \frac{f_{pk}}{f_{yk}}$$

- Crack Width Limitation
  Sections are assumed cracked, if the flexural tensile stress exceeds $f_{ct,eff}$ (7.1 (2)). For crack width calculations, $f_{ct,eff}$ may be assumed to be equal to f_{ctm}.

  Calculation is based on section 7.3.4[5].

  Design crack width, $w_k = s_{r,max}\left(\varepsilon_{sm} - \varepsilon_{cm}\right)$

  Where,

  $s_{r,max}$ = maximum crack spacing
  ε_{sm} = mean strain in the reinforcement under the relevant combination of loads, including the effect of imposed deformations and taking into account the effects of tension stiffening. Only the additional tensile strain beyond the state of zero strain of concrete at the same level is considered.
  ε_{cm} = mean strain in the concrete between cracks

- $$\varepsilon_{sm} - \varepsilon_{cm} = \frac{\sigma_s - k_t \dfrac{f_{ct,eff}}{\rho_{p,eff}}\left(1 + \alpha_e \rho_{p,eff}\right)}{E_s} \geq 0.6\frac{\sigma_s}{E_s}$$

  Where,

  σ_s = the stress in the tension reinforcement calculated on the basis of a cracked section [conservatively assumed f_{yk}];
  α_e = E_s/E_{cm} ;;
  E_s = modulus of elasticity of steel;
  E_{cm} = modulus of elasticity of concrete (secant modulus) [Eci] ;

  $\rho_{p,eff}$ = $(A_s+\xi_1^2 A_p')/A_{c,eff}$;
  A_s = area of nonprestressed reinforcement in tension zone;
  $A_{p'}$ = area of tendons within $A_{c,eff}$;
  $A_{c,eff}$ = effective area of concrete in tension surrounding the reinforcement or prestressing tendons of depth $h_{c,ef}$ [=$h_{c,ef}$ * (bar spacing)]

  $f_{ct,eff}$ = mean value of the tensile strength of concrete [f_{ctm}];

  f_{ctm} = mean tensile strength of concrete $f_{ctm} = 0.30xf_{ck}^{(2/3)}$ for f_{ck} <50 MPa (Table 3.1), but not less than 3 MPa;

  $h_{c,ef}$ = lesser of 2.5(h-d), (h-x)/3 or h/2
  h = depth of the member
  x = depth of neutral axis from the compression fiber

  The following figure reproduced from EC2 explains the preceding parameters graphically

[5] EN 1992-1-1:2004(E), Section 7.2(2).

1992-1-1:2004 (E)

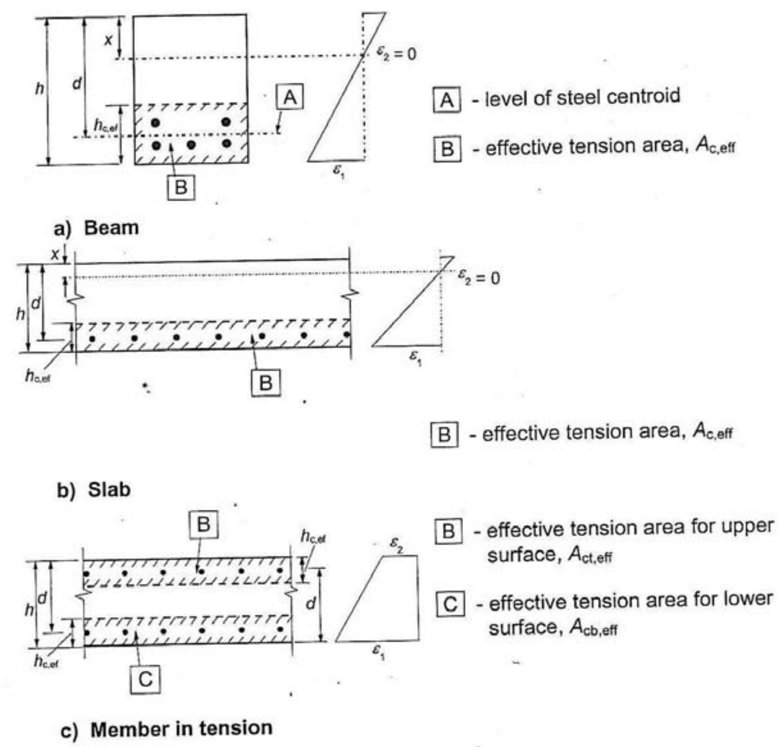


Figure 7.1: Effective tension area (typical cases)

ξ_1 $= \sqrt{\xi * \frac{\phi_s}{\phi_p}}$; adjusted ratio of bond strength taking into account the different diameters of prestressing and nonprestressed steel;

ξ = ratio of bond strength of prestressing and non-prestressed reinforcement steel according to Table 6.2 of EC2, as reproduced below:

For strands:
ξ = 0.5 for f_{ck} =< 50; and 0.25 for f_{ck} => 70 MPa

For for bars and wires:
ξ = 0.3 for f_{ck} =< 50; and 0.15 for f_{ck} => 70 MPa

Interpolate for intermediate values.

Φ_s = largest diameter of non-prestressed steel used;
Φ_p = diameter, or equivalent diameter of prestressing steel;

k_t = factor dependent on the duration of the load
= 0.6 for short term loading

= 0.4 for long term loading

- $s_{r,max}$= 1.3(h-x)

Using these parameters, the program calculates the design crack width (w_k) of each design section. If the calculated value exceeds the allowable, reinforcement is added to that section, in order to reduce the crack width to within the allowable value given below. The allowable crack width depends on the "Exposure" classification.

- Crack width for nonprestressed concrete – Exposure 2 - 4
  - Width is limited to 0.3 mm.
- Crack width for prestressed concrete – Exposure 1 - 2
  - Width is limited to 0.2 mm for frequent load combination.
  - Width is limited to 0.3 mm for quasi-permanent load combination.

The program uses the above values for allowable crack width for prestressed (grouted and unbonded systems) and nonprestressed structures independent of the exposure classes.

Strength Check in Bending

- Plane sections remain plane. Strain compatibility is used to determine the forces on a section.
- Maximum concrete strain in compression is limited to 0.0035.
- Maximum allowable value for the neutral axis "x" is determined based on concrete strength of the section f_{ck} .
  - o For f_{ck} <= 35 MPa x/d <= 0.45
  - o For f_{ck} > 35 MPa x/d <= 0.35

  Where necessary, compression reinforcement is added to enforce the above requirement.

- If a section is made up of more than one concrete material, the entire section is designed using the concrete properties of lowest strength in that section
- For prestressed and nonprestressed reinforcement that cross a design section at an angle other than 90 degrees, the change in strain of the reinforcement due to flexing of the design section about its own axis considered to contribute to the design capacity is calculated from the following relationship:

  - o Contributory change of strain in reinforcement =
    (strain normal to the design section at location of rebar)*$\cos^2\theta$

    where θ is the angle between the normal to the design section and the reinforcement. The strain in each reinforcement is calculated separately, based on the location reinforcement on the cross-section and the angle it makes with the normal to the design section.
- Additional considerations for strain in prestressing steel
  - o If bonded, strain is calculated using the stress-strain curve and the angle of strand with the design section
  - o If unbonded, the strain is calculated using the same procedure as bonded tendons, but the calculated strain is adjusted as follows:
    - The stress increase is reduced by the factor (L_s/L_T)

  - The stress increase is limited to $(L_s/L_T)100$ MPa
- Stress in nonprestressed steel is based on stress-strain relationship assumed
- Rectangular concrete block is used with maximum stress equal to ηf_{cd}[6]

$\eta=1$ for $f_{ck} \leq 50$MPa,

$\eta=1-(f_{ck}-50)/200$ for $50 < f_{ck} \leq 90$MPa

- For flanged sections, the following procedure is adopted:

  - If x is within the flange, the section is treated as a rectangle
  - If x exceeds the flange thickness, uniform compression is assumed over the flange. The stem is treated as a rectangular section

One-Way Shear Check

The design is based on the following (section 6.2 of the code):

$V_{Sd} <= V_{Rd}$

where,

V_{Sd} = design value; and

V_{Rd} = design resistance (capacity).

Three resistance values are calculated and checked against the design values. These are:

V_{Rd1} = design shear resistance without shear reinforcement

V_{Rd2} = maximum design shear resistance that can be carried without crushing of notional concrete compressive struts

V_{Rd3} = design shear resistance of section with shear reinforcement

Code formulas are used to calculate the above values. In using the code formulas, the following considerations are observed.

$v_{sd}= V_{Sd} / b_w d$

- For b_w the smallest width of the section is used
- Shear due to the vertical component of prestressing tendons is ignored
- Loss of cross-sectional area due to post-tensioning ducts is ignored
- Code provision for increase in shear capacity for sections that are close to a support is not implemented
- All available tension reinforcement is included in the calculation of A_{s1}
- Curtailment lengths for longitudinal bars are not calculated
- The Stirrup spacing are calculated based on the required shear reinforcement and is limited to minimum of (0.75(d+cotα), 300mm), where α is the angle between the shear reinforcement and longitudinal axis of the beam/slab.

Punching Shear

Categorization of columns:

No criterion is mentioned in EC2 regarding categorizations of columns for punching shear check. The program uses ACI-318 criteria as detailed below.

Based on the geometry of the floor slab at the vicinity of a column, each column is categorized into to one of the following options:

[6] EN 1992-1-1:2004(E), Eq 3.21 and 3.22

1. Interior column
   Each face of the column is at least four times the slab thickness away from a slab edge

2. Edge column
   One side of the column normal to the axis of the moment is less than four times the slab thickness away from the slab edge

3. Corner column
   Two adjacent sides of the column are less than four times the slab thickness from slab edges parallel to each

4. End column
   One side of the column parallel to the axis of the moment is less than four times the slab thickness from a slab edge

In cases 2, 3 and 4, column is assumed to be at the edge of the slab. The overhang of the slab beyond the face of the column is not included in the calculations. Hence, the analysis performed is somewhat conservative.

Design Stress

Several critical perimeters around each column are considered. For each critical perimeter, a representative design shear stress (v_u) is calculated, using a combination of the direct shear and moment:

$$v_u = \frac{\beta\, V_u}{A}$$

where V_u is the absolute value of the direct shear and β is an amplification factor of shear to take into account the effect of moments and eccentricities.

β = 1.15 for interior columns

β = 1.40 for edge and end columns

β = 1.50 for corner columns

For a column with dimensions “a” and “b,” or drop cap with dimensions “a” and “b,” and a critical section which is at distance “c” from the face of column or drop cap, “A” is given by:

1. Interior column:
   $A = 2(a+b) + 2\pi\ c$

2. Edge column: (a is parallel to the axis of moment)

   $A = 2a + b + \pi\ c$

3. Corner Column:
   $A = a + b + \frac{\pi}{2} c$

4. End column: (a is parallel to the axis of moment)

$$A = a + 2b + \pi\ c$$

Allowable stress:

Allowable stress of a section is calculated based on the following equation:

$$v_{Rd,c} = v_{min} + k_1\sigma_{cp}$$

where,

$$v_{min} = 0.035k^{3/2}f_{ck}^{1/2}$$

$k_1 = 0.1$
$k = 1 + (200/d)^{1/2} \leq 2.0$, d in mm

$$\sigma_{cp} = \frac{N_{ED}}{A_c}$$

σ_{cp} = Normal concrete stress in the critical section (MPa, positive if compression)
N_{ED} = Longitudinal force across the control section (in N).
A_c = Area of the concrete

Critical sections

The critical sections for stress check are:

(1) at the face of column;
(2) at *2*d from the face of the column, where d is the effective depth of the slab; and
(3) additional sections at 0.75d intervals, where required.

If a drop cap is present, depending on the size of the cap, the stresses are checked both within the drop cap, and beyond the perimeter of the drop cap. However, if the drop cap is too small to be effective to resist punching shear, the stress check is performed outside the perimeter of the drop cap only. For a drop cap to be considered effective, its horizontal extension from the face of the column (l_H) should exceed twice the projection of the cap below the slab soffit ($2h_H$)[7]. The first critical section is at r_{cont}[8] from the center of the column, and the subsequent sections are at 0.75d intervals. Generally, critical sections both within the drop cap and beyond it will be checked.

Stress check:

Stresses are calculated at the critical sections and compared against the allowable values:

If $v_u < v_{Rd,c}$ no punching shear reinforcement is required
if $v_u > v_{max}$ at the face of the column, punching stress is excessive; the section should be revised. The program displays graphically and reports in table the locations that need revision.

[7] EN 1992-1-1:2004, Section 6.4.2 (8)
[8] EN 1992-1-1:2004, Eqn 6.34-6.37

where
v_{max} = maximum allowable shear stress for a perimeter of the column is

$v_{max} = 0.5vf_{cd}$

where
$v = 0.6[1-(f_{ck}/250)]$ Flow Chart of TR43 Report for Post-Tensioning Design

If $v_u > v_{Rd,c}$ provide punching shear reinforcement

Stress check is performed until no shear reinforcement is needed. Where drop caps exist, stresses are checked within the drop cap until the design stress is less than permissible, then in a similar manner the stresses are checked outside the drop cap.

Shear reinforcement:

Where needed, shear reinforcement is provided according to the following:

$$A_s = \frac{\left(v_u - 0.75v_{Rd,c}\right)\ u \times d \times s_r}{1.5 \times d \times f_{ywd,ef} \sin(\alpha)}$$

$$A_{s,min} = \frac{0.08\sqrt{f_{ck}} \times s_r \times s_t}{\left(1.5\sin\alpha + \cos\alpha\right) f_{yk}}$$

where,
$f_{ywd,ef} = 250 + 0.25d \leq f_{ywd}$
f_{ywd} = design strength of punching shear reinforcement
s_r = spacing of shear links in the radial direction
s_t = spacing of shear links in the tangential direction
u = perimeter of the critical section
α = the angle of shear reinforcement with the plane of slab
d = effective depth

Arrangement of shear reinforcements:

Shear reinforcement can be in the form of shear studs or shear stirrups (links). In case of shear links, the number of shear links (N_{shear_links}) in a critical section and distance between the links ($Dist_{shear\_links}$) are given by:

$$N_{shear\_links} = \frac{A_s}{A_{shear\_link}}$$

$$Dist_{shear\_links} = \frac{u}{N_{shear\_links}}$$

where, $A_{shear-link}$ = area of the single shear link

The calculated distance will be compared with the maximum allowable by the code and will be adjusted accordingly.

If shear studs are used, the number of shear studs per rail (N_{shear_studs}) and the distance between the studs ($Dist_{shear\_studs}$) are given by:

$$N_{shear\_studs} = \frac{A_s}{A_{shear\_stud} \times N_{rails}}$$

$$Dist_{shear\_studs} = \frac{s}{N_{shear\_studs}}$$

where , $A_{shear\text{-}stud}$ = area of the shear stud
s = spacing between the critical sections.

Shear reinforcement is provided in three layers (perimeters) from the face of support including the first critical section, i.e., within the distance 2d from the face of support.

Initial Condition (Transfer of prestressing)

- Stress limitations used for the "initial" load combination are as follows:
    - Concrete
        - Maximum compressive stress[9] 0.60 f_{ci}.
        - Since the EC2 code is not specific about the allowable tensile stress limit, program uses the ACI-05 colde values (0.25√fci) as a default. But the user has the option to override those values. Where calculated values exceed this threshold, the program provided reinforcement to control cracking.

- Reinforcement
  Reinforcement will be provided for initial condition if tensile stress exceeds allowable stress. Rebar is provided based on ACI code and will be placed on tension side:

  As=T/(0.5Fy)
  Where:
  As: Area of reinforcement
  T: total tensile force on tension block
  Fy: Yield Stress of the steel but not more that 60 ksi

NOTATION

As = area of the reinforcement;

f_{cd} = design value of concrete cylinder compression strength;

[9] EN 1992-1-1:2004(E), Section 5.10.2.2(5)

f_{ck} = characteristic compressive cylinder strength at 28 days;

$f_{ct,eff}$ = mean value of the tensile strength of concrete;

f_{ctm} = mean tensile strength of concrete $f_{ctm} = 0.30xf_{ck}^{(2/3)}$ for f_{ck} <50 MPa (Table 3.1), but not less than 3 MPa;

f_{pk} = characteristic tensile strength of prestressing steel [1860 MPa];

f_{yk} = characteristic yield strength of steel, [460 MPa];

$f_{ywd,ef}$ = effective design strength of the punching shear reinforcement;

k_1 = a coefficient that takes account of the bond properties of the bars;

k_2 = a coefficient that takes account of the form of the strain distribution;

L_s = span length of tendon;

L_T = total length of tendon;

r_{cont} = distance of critical section from the center of the column, if drop cap exists;

s = spacing between successive critical sections;

v_{sd} = design shear stress;

V_{Sd} = design shear force;

V_{Rd} = design shear resistance;

x = depth of neutral axis; and

w_k = design (computed) crack width.