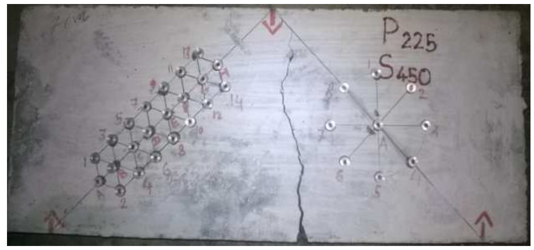
APPENDIX –I

PHOTOGRAPHS OF MODERATE DEEP BEAMS

1 PLAIN CONCRETE BEAM (1P)

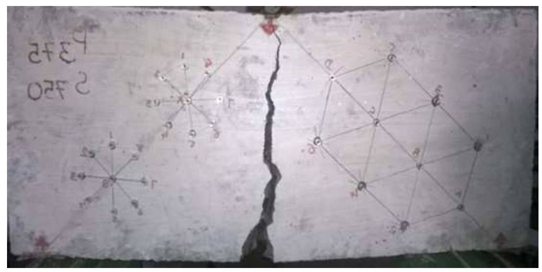
Beam Notation 14-1 - P150 S300
Ultimate Load – 2.5 Ton
Mode of Failure : Flexure

Beam Notation 14-2 – P225 S450
Ultimate Load – 3.6 Ton
Mode of Failure : Flexure



Beam Notation 14-3– P300 S600
Ultimate Load – 3.1 Ton
Mode of Failure : Flexure

Beam Notation 14-4 – P375 S750
Ultimate Load – 6 Ton
Mode of Failure : Flexure



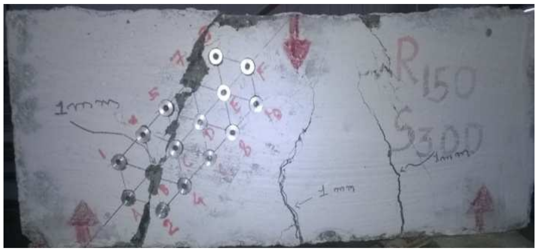
Beam Notation 14-5– P450 S900
Ultimate Load – 5.Ton
Mode of Failure : Flexure

Beam Notation 14-6 – P525 S1050
Ultimate Load – 8 Ton
Mode of Failure : Flexure

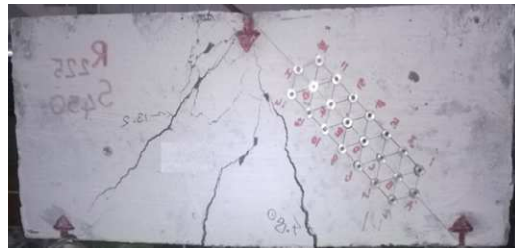
Beam Notation 14-7 – P600 S1200
Ultimate Load – 6.5 Ton
Mode of Failure : Flexure

2 RCC SERIES (1P)

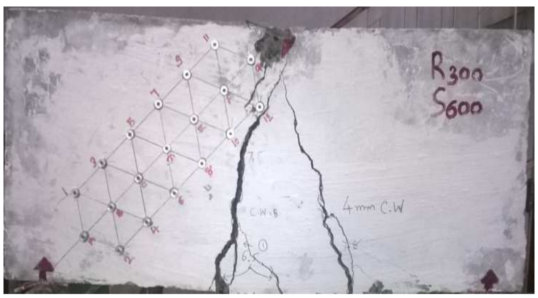
Beam Notation 14-8– R150 S300
First Crack Load – 3 Ton
Ultimate Load – 10 Ton
Mode of Failure : Flexure shear



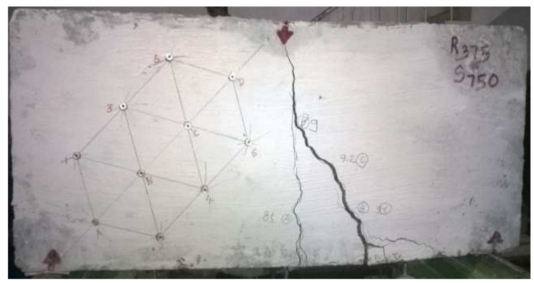
Beam Notation 14-9– R225 S450
First Crack Load – 4.8 Ton
Ultimate Load – 13.6 Ton
Mode of Failure : Flexure shear



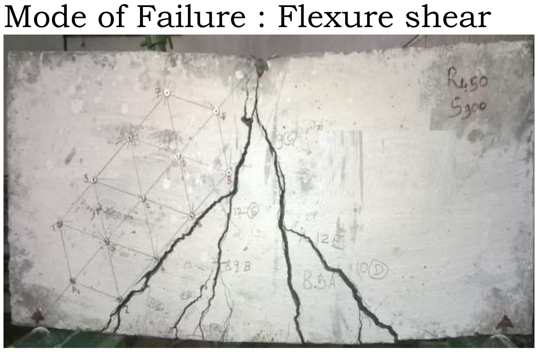
Beam Notation 14-10– R300 S600
First Crack Load – 6.5 Ton
Ultimate Load – 14 Ton
Mode of Failure : Flexure shear



Beam Notation 14-11– R375 S750
First Crack Load – 8 Ton
Ultimate Load – 15 Ton
Mode of Failure : Flexure shear

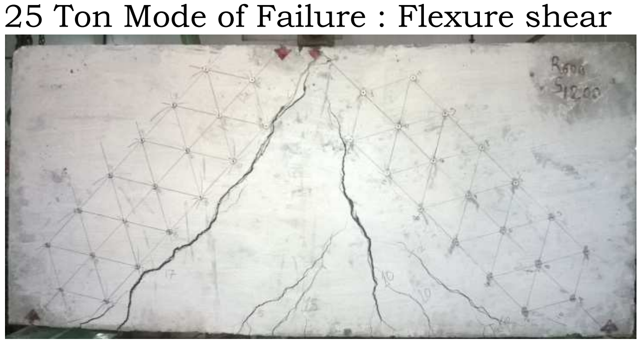


Beam Notation14-12 - R450 S900
First Crack Load – 8.5 Ton
Ultimate Load – 16.5 Ton
Mode of Failure : Flexure shear



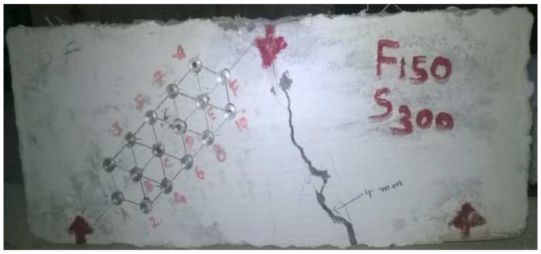
Beam Notation14-13– R525 S1050
First Crack Load – 12 Ton
Ultimate Load – 19.4 Ton
Mode of Failure : Flexure shear

Beam Notation 14-14– R600 S1200
First Crack Load – 10 Ton
Ultimate Load – 25 Ton Mode of Failure : Flexure shear

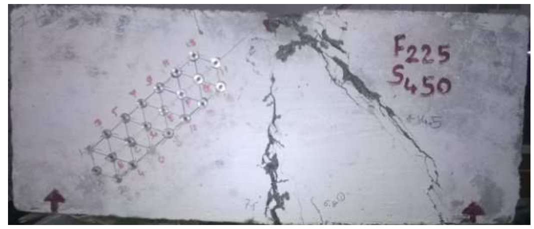


3 FIBROUS SERIES (1P)

Beam Notation 14-15– F150 S300
First Crack Load – 4 Ton
Ultimate Load – 12 Ton
Mode of Failure : Flexure shear



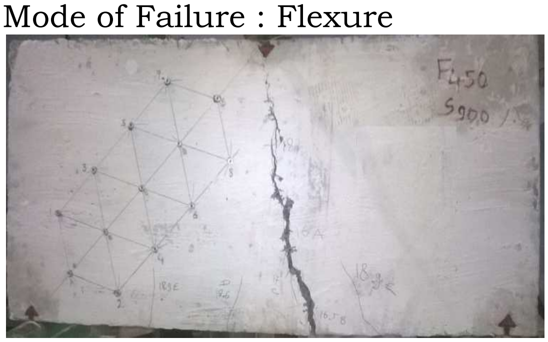
Beam Notation14-16-F225 S450First Crack Load – 11.3 Ton
Ultimate Load – 20.6 Ton
Mode of Failure : Flexure shear



Beam Notation 14-17- F300 S600
First Crack Load – 8.5 Ton
Ultimate Load – 16.4 Ton
Mode of Failure : Flexure

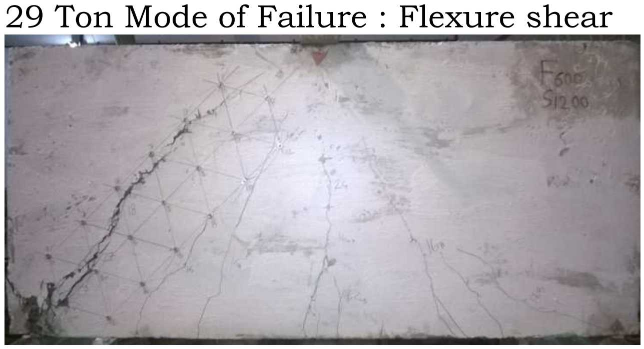
Beam Notation 14-18– F375 S750
First Crack Load – 10 Ton
Ultimate Load – 19.2Ton
Mode of Failure : Flexure

Beam Notation 14-19- F450 S900
First Crack Load – 15.3 Ton
Ultimate Load – 23 Ton
Mode of Failure : Flexure



Beam Notation 14-20- F525 S1050
First Crack Load – 12.6 Ton
Ultimate Load – 28.1 Ton
Mode of Failure : Flexure shear

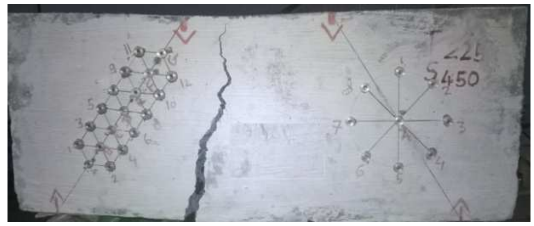
Beam Notation 14-21 F600 S1200
First Crack Load – 11.2 Ton
Ultimate Load – 29 Ton Mode of Failure : Flexure shear



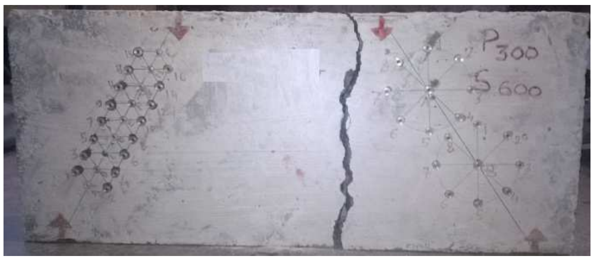
4 PLAIN CONCRETE BEAM (2P)

Beam Notation 14-22- P150 S300
Ultimate Load – 21.7 Ton
Mode of Failure : Flexure

Beam Notation 14-23-P225 S450Ultimate Load – 3.8 Ton
Mode of Failure : Flexure



Beam Notation 14-24– P300 S600
Ultimate Load – 5.9 Ton
Mode of Failure : Flexure



Beam Notation14-25– P375 S750
Ultimate Load – 6.9 Ton
Mode of Failure : Flexure

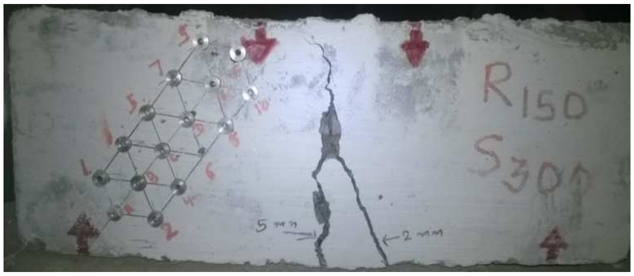
Beam Notation14-26– P450 S900
Ultimate Load – 6.8 Ton
Mode of Failure : Flexure

BeamNotation14-27–P525 S1050
Ultimate Load – 8 Ton
Mode of Failure : Flexure

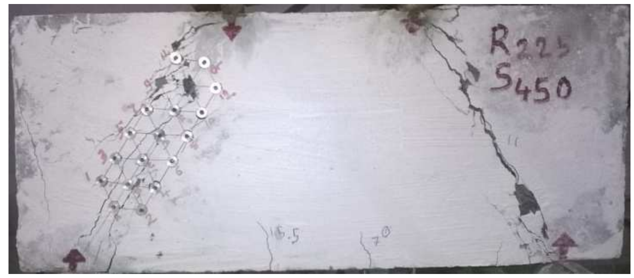
Beam Notation14-28– P600 S1200
Ultimate Load – 8.5 Ton
Mode of Failure : Flexure

5 RCC SERIES (2P)

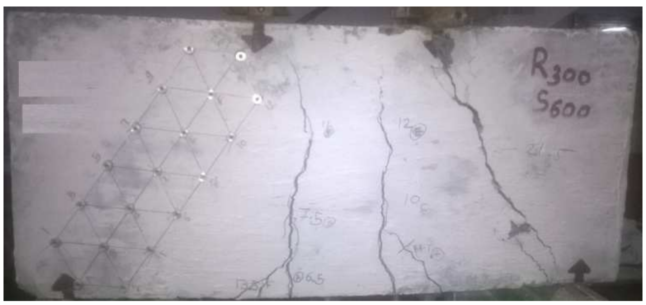
Beam Notation 14-29– R150 S300
First Crack Load – 3 Ton
Ultimate Load – 9 Ton
Mode of Failure : Flexure shear



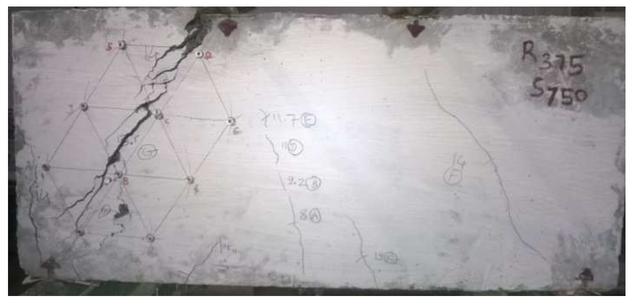
Beam Notation14-30– R225 S450
First Crack Load – 7 Ton
Ultimate Load – 18 Ton
Mode of Failure : Flexure shear



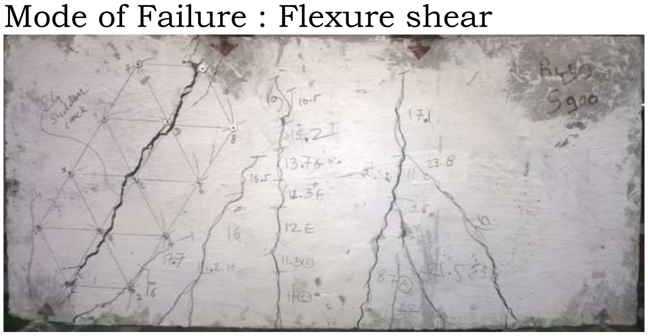
Beam Notation14-31– R300 S600
First Crack Load – 6.5 Ton
Ultimate Load – 21.9 Ton
Mode of Failure : Flexure shear



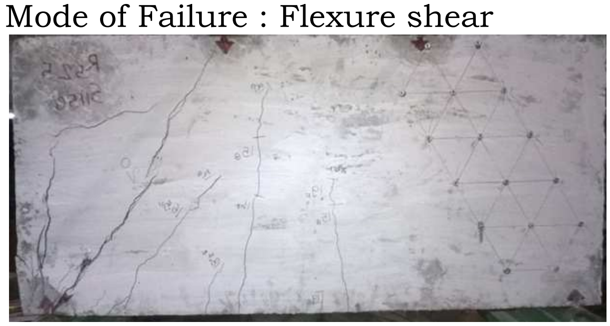
Beam Notation14-32– R375 S750
First Crack Load – 8 Ton
Ultimate Load – 21.3 Ton
Mode of Failure : Flexure shear



Beam Notation14-33– R450 S900
First Crack Load – 8.7 Ton
Ultimate Load – 24.1 Ton
Mode of Failure : Flexure shear



Beam Notation14-34– R525 S1050
First Crack Load – 12 Ton
Ultimate Load – 22 Ton
Mode of Failure : Flexure shear

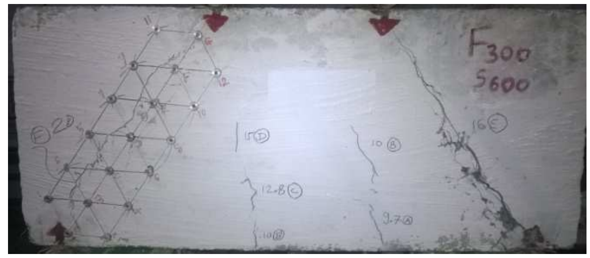


Beam Notation 14-35– R600 S1200
First Crack Load – 13 Ton
Ultimate Load – 26 Ton
Mode of Failure : Flexure shear

6 FIBROUS SERIES (2P)

Beam Notation 14-36– F225 S450
First Crack Load – 11.3 Ton
Ultimate Load – 20.6 Ton
Mode of Failure : Flexure shear

Beam Notation14-37– F300 S600
First Crack Load – 9.7 Ton
Ultimate Load – 22 Ton
Mode of Failure : Flexure shear

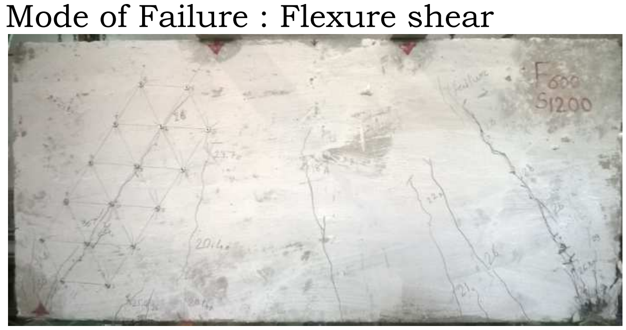


Beam Notation14-38– F375 S750
First Crack Load – 15 Ton
Ultimate Load – 25.7 Ton
Mode of Failure : Flexure shear

Beam Notation14-39– F450 S900
First Crack Load – 11.5 Ton
Ultimate Load – 28.6 Ton
Mode of Failure : Flexure shear

Beam Notation14-40– F525 S1050
First Crack Load – 15 Ton
Ultimate Load – 29 Ton
Mode of Failure : Flexure shear

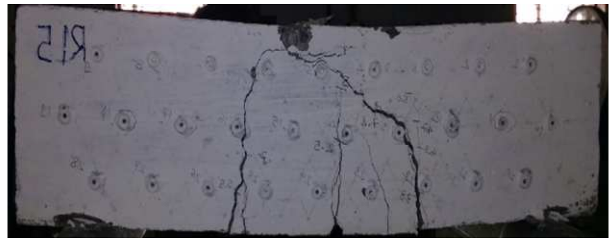
Beam Notation 14-41– F600 S1200
First Crack Load – 15.7 Ton
Ultimate Load – 31 Ton
Mode of Failure : Flexure shear



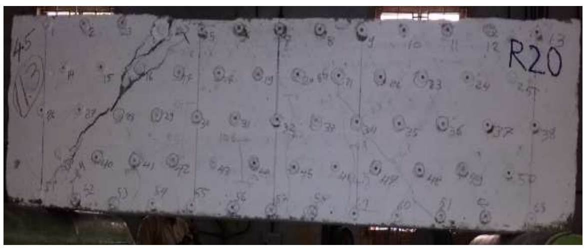
7 RCC BEAM SERIES (1P)

Beam Notation 14-42: R10
First Crack Load: 2.5 Ton
Ultimate Load: 8.1 Ton
Mode Of Failure: Flexure-Shear

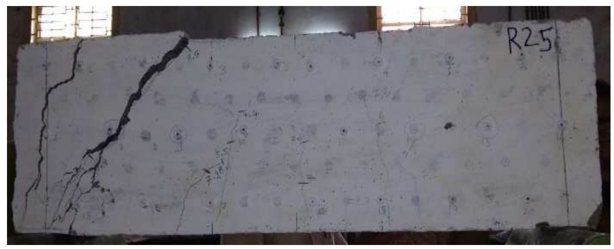
Beam Notation 14-43: R15
First Crack Load: 2.8 Ton
Ultimate Load: 8.5 Ton
Mode Of Failure: Flexure shear



Beam Notation 14-44: R20
First Crack Load: 4.5 Ton
Ultimate Load: 11.7 Ton
Mode Of Failure: Flexure-Shear



Beam Notation 14-45: R25
First Crack Load: 5.5 Ton
Ultimate Load: 11.9 Ton
Mode Of Failure: Flexure-Shear



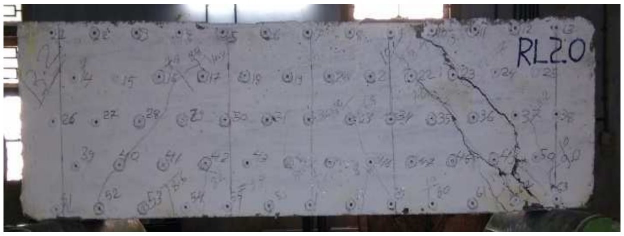
Beam Notation 14-46: R30
First Crack Load: 4 Ton
Ultimate Load: 12 Ton
Mode Of Failure: Flexure-Shear

Beam Notation 14-47: R35
First Crack Load: 6 Tom
Ultimate Load: 14.7 Ton
Mode Of Failure: Flexure-Shear

Beam Notation 14-48: R40
First Crack Load: 7.5 Ton
Ultimate Load: 15.5 Ton
Mode Of Failure: Flexure-Shear

8 LAYERED RCC SERIES

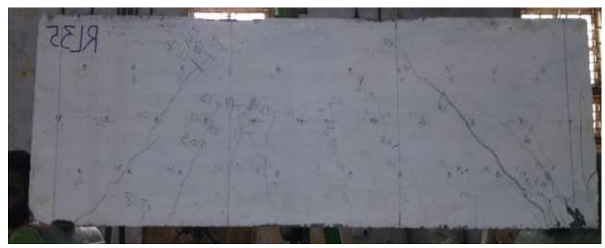
Beam Notation 14-49: RL20
First Crack Load: 4.5 Ton
Ultimate Load: 13.6 Ton
Mode Of Failure: Flexure-Shear



Beam Notation 14-50: RL25
First Crack Load: 6 Ton
Ultimate Load: 11.5 Ton
Mode Of Failure: Flexure-Shear

Beam Notation 14-51: RL30
First Crack Load: 4.8 Ton
Ultimate Load: 16 Ton
Mode Of Failure: Flexure-Shear

Beam Notation 14-52: RL35
First Crack Load: 6.5 Ton
Ultimate Load: 17.8 Ton
Mode Of Failure: Flexure-Shear

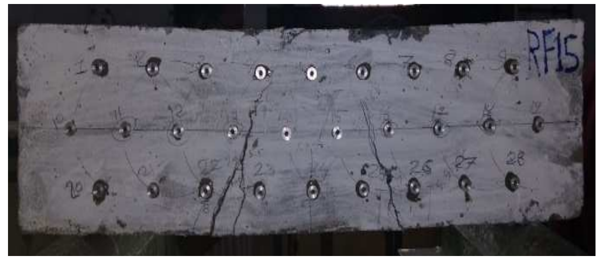


Beam Notation 14-53: RL40
First Crack Load: 8 Ton
Ultimate Load: 15.6 Ton
Mode Of Failure: Flexure-Shear

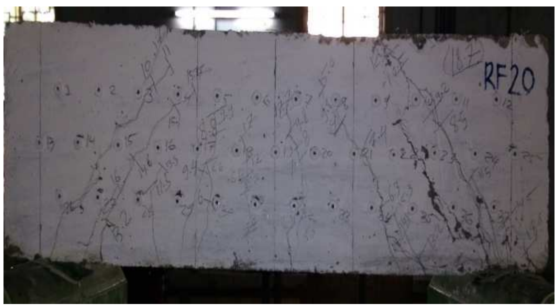
9 FIBEROUS RCC SERIES

T Beam Notation 14-54: RF10
First Crack Load: 3.5 Ton
Ultimate Load: 10 Ton
Mode Of Failure: Flexure

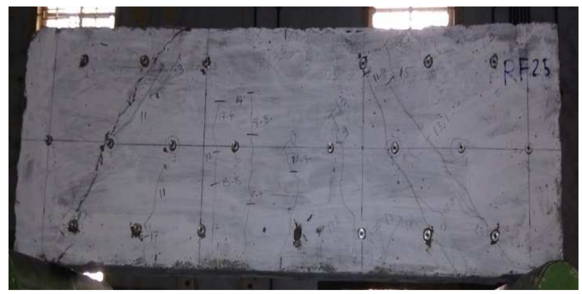
Beam Notation 14-55: RF15
First Crack Load: 3.8 Ton
Ultimate Load: 11.6 Ton
Mode Of Failure: Flexure-Shear



Beam Notation 14-56: RF20
First Crack Load: 5.4 Ton
Ultimate Load: 14.1 Ton
Mode Of Failure: Flexure-Shear



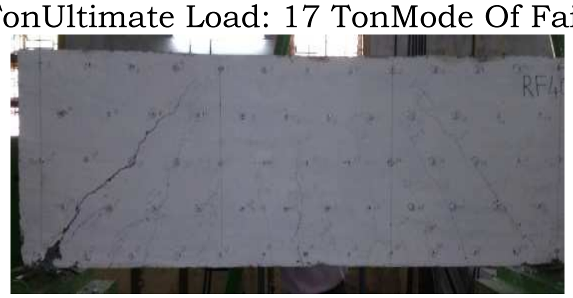
Beam Notation 14-57: RF25
First Crack Load: 6.7 Ton
Ultimate Load: 13.6 Ton
Mode Of Failure: Flexure-Shear



Beam Notation 14-58: RF30
First Crack Load: 6.7 Ton
Ultimate Load: 14.5 Ton
Mode Of Failure: Flexure

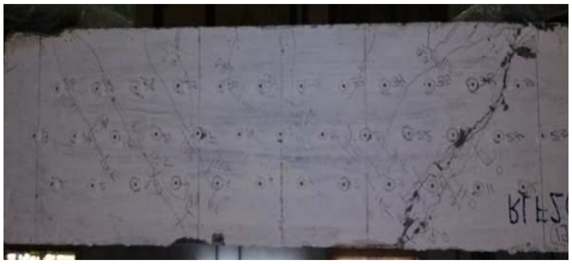
Beam Notation 14-59: RF35
First Crack Load: 7.5 Ton
Ultimate Load: 17.7 Ton
Mode Of Failure: Flexure-Shear

Beam Notation 14-60: RF40
First Crack Load: 8 TonUltimate Load: 17 TonMode Of Failure: Flexure-Shear

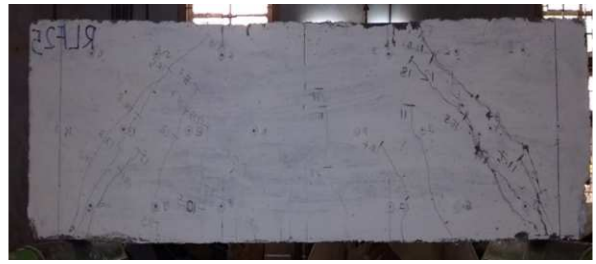


10 FIBEROUS LAYERED RCC SERIES

Beam Notation 14-61: RLF 20
First Crack Load: 8.2 Ton
Ultimate Load: 15.5 Ton
Mode Of Failure: Flexure-Shear



Beam Notation 14-62: RLF 25
First Crack Load: 8.5 Ton
Ultimate Load: 16.6 Ton
Mode Of Failure: Flexure-Shear



Beam Notation 14-63: RLF 30
First Crack Load: 6 Ton
Ultimate Load: 26.7 Ton
Mode Of Failure: Flexure-Shear

Beam Notation 14-64: RLF 35
First Crack Load: 8 Ton
Ultimate Load: 19.8 Ton
Mode Of Failure: Flexure-Shear

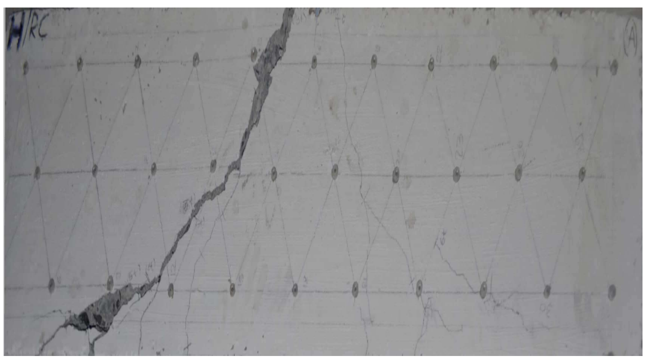
Beam Notation 14-65: RLF 40
First Crack Load: 10 Ton
Ultimate Load: 20 Ton
Mode Of Failure: Flexure-Shear

11 **RCC SERIES (1P)**

Beam notation 14-66: F/RCC
First crack load:2.70 Ton
Ultimate load:7.10 Ton
Mode of failure:Flexure-Shear Failure

Beam notation 14-67:A1/RCC
First crack load:3.40 Ton
Ultimate load:7.30 Ton
Mode of failure:Flexure-Shear Failure

Beam notation 14-68:H/RCC
First crack load: 3.70 Ton
Ultimate load:8.20 Ton
Mode of Failure :Flexure-Shear Failure



Beam notation 14-69:I/RCC
First crack load:3.80 Ton
Ultimate load:9.00 Ton
Mode of Failure :Flexure-Shear Failure

Beam notation 14-70:J/RCC
First crack load: 3.90 Ton
Ultimate load:9.40 Ton
Mode of Failure : Flexure-Shear Failure

12 **PFRC SERIES(1P)**

Beam Notation 14-71: A1/PFRC
First Crack Load: 3.60 Ton
Ultimate Load: 7.80 Ton
Mode ofFailure:Flexure-Shear Failure

Beam Notation 14-72: H/PFRC
First Crack Load: 3.70 Ton
Ultimate Load: 9.70 Ton
Mode ofFailure: Flexure-Shear Failure

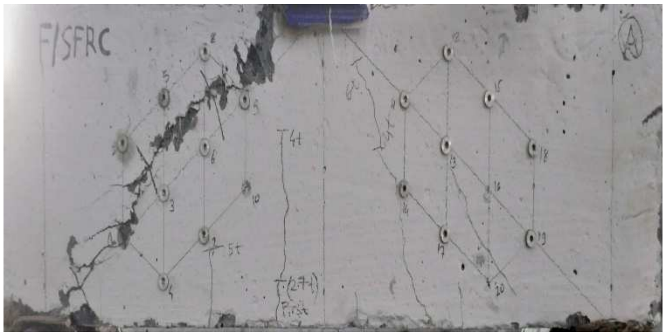
Beam Notation 14-73: I/PFRC
First Crack Load: 4.00 Ton
Ultimate Load:10.10 Ton
Mode ofFailure: Flexure-Shear Failure

Beam Notation 14-74: H/PFRC
First Crack Load:3.70 Ton
Ultimate Load:9.70 Ton
Mode of Failure:Flexure-Shear Failure

Beam Notation 14-75: J/PFRC
First Crack Load: 4.20 Ton
Ultimate Load: 11.20 Ton
Mode ofFailure: Flexure-Shear Failure

13 SFRC SERIES(1P)

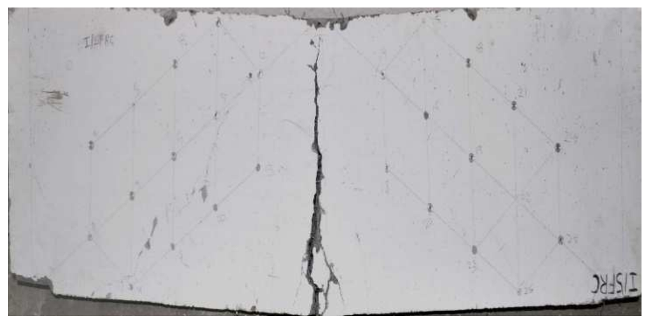
Beam Notation 14-76: F/SFRC
First Crack Load:3.10 Ton
Ultimate Load:7.30 Ton
Mode Of Failure:Flexure-Shear Failure



Beam Notation 14-77: A1/SFRC
First Crack Load: 3.70 Ton
Ultimate Load: 7.90 Ton
Mode Of Failure:Flexure-Shear Failure

Beam Notation 14-78:H/SFRC
First Crack Load:3.90 Ton
Ultimate Load:12.20 Ton
Mode Of Failure:Flexure Failure

Beam Notation 14-79: I/SFRC
First Crack Load: 4.30 Ton
Ultimate Load:12.50 Ton
Mode Of Failure:Flexure Failure



Beam Notation 14-80: J/SFRC
First Crack Load: 4.50 Ton
Ultimate Load: 13.60 Ton
Mode ofFailure: Flexure Failure

APPENDIX II

MODIFICATION OF MR P J ROBBIN'S FORMULA

1 SAMPLE OF CALCULATION OF ULTIMATE SHEAR STRENGTH USING VARIOUS EQUATION

1.1 SAMPLE OF CALCULATION OF ULTIMATE SHEAR STRENGTH USING APPA RAO EQUATION:

1.1.1 Sample Calculation of beam R450 x S900: (1P)

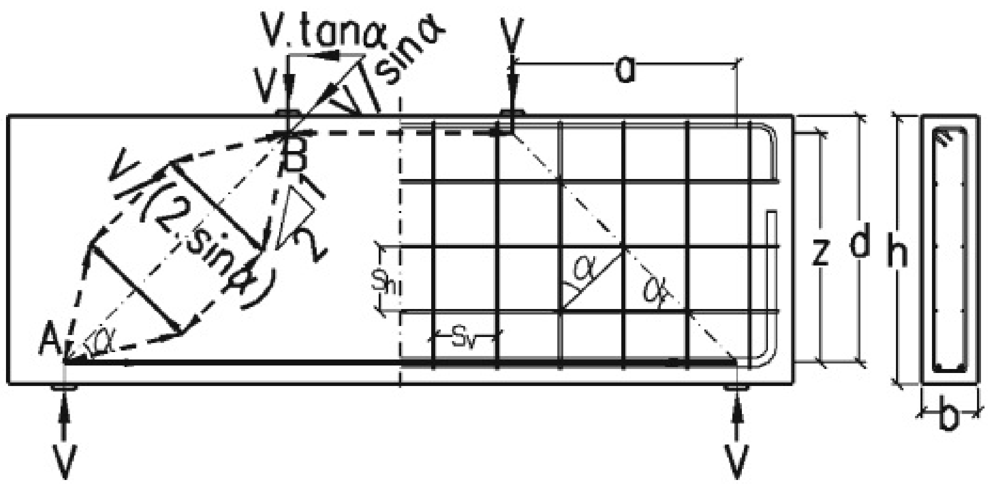


Figure 15.1 Refined strut-and-tie model for RC deep beam

$$v_n = \frac{11.40\rho^{0.35}\sqrt{f'_c}}{1+2\left(\frac{a}{d}\right)}\left(0.38 + \frac{1}{\sqrt{1+\left(\frac{d}{25d_a}\right)}}\right) + 0.02\rho^{-0.08}\rho_h f_y\left(\frac{d}{a}\right) + 0.31\rho_v f_y\left(\frac{a}{d}\right)$$

Where,

vn = Nominal shear stress of the beam

ρ = *0.3%*

f'_c =36 MPa

a = *450*

d = 425 mm

da = 20 mm

ρh = *0*

ρv = *0*

$$v_n = \frac{11.40\ 0.003^{0.35}\sqrt{36}}{1+2\left(\frac{450}{425}\right)}\left(0.38+\frac{1}{\sqrt{1+\left(\frac{425}{25\ x\ 20}\right)}}\right)+0+0$$

$V_n = \frac{8.95}{3.11}(1.11)$

$= 3.20\ N/mm^2$

Ultimate shear capacity= V_nx b x d = 3.20 x 75 x 425

= 10.2 Ton

Ultimate load W_u = 100.2 x 2 = 20.4 Ton

1.2 SAMPLE OF CALCULATION OF ULTIMATE SHEAR STRENGTH USING CHENG AND TANG EQUATION:

1.2.1 Sample Calculation of beam R450 x S900: (1P)

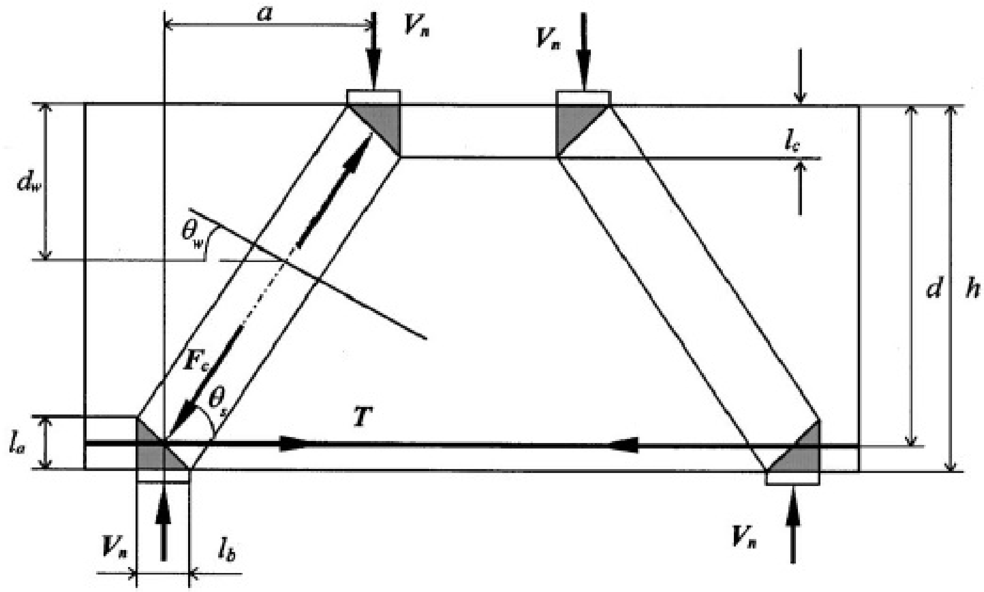


Figure 15-2 Strut-and-tie model for simply supported deep beams

$$V_N = \frac{1}{\frac{\sin(2\theta_S)}{A_c f_t}+\frac{1}{f'_t A_{str}\sin(\theta_S)}}$$

Where,

$$f_t = \frac{2A_s f_y sin\theta_s}{A_c/sin\theta_s}+\sum\frac{2A_w f_{yw}\sin(\theta_s+\theta_w)}{A_c/sin\theta_s}\cdot\frac{d_w}{d}+0.5\sqrt{f'_c}$$

$$\theta_s = tan^{-1}D/_a = tan^{-1}450/_{450} = 45^o$$

$A_c = 450\ x\ 75 = 33750\ mm^2$

$A_s = 113\ mm^2$

$l_b = 85\ mm$

$$l_a = \frac{l_b}{\tan\theta_s} = \frac{85}{\tan 45} = 85$$

$$l = \frac{D - l_a}{\sin\theta_s} = \frac{450 - 85}{\sin 45} = 516.18$$

$f'_c = 28$

$A_{str} = l\ x\ b = 516.18\ x\ 75 = 38713.5\ mm^2$

$$f_t = \frac{2(113)(415)(0.707)}{33750/0.707} + \sum 0\ .+0.5\sqrt{28}$$

$f_t = 1.38 + 2.64 = 4.02$

$$V_N = \frac{1}{\frac{(1)}{33750\ x\ 4.02} + \frac{1}{(28)(38713.5)(0.707)}} = 12.015\ Ton$$

Ultimate Load = 2 x 12.015 = 24.03 Ton

1.3 SAMPLE OF CALCULATION OF ULTIMATE SHEAR STRENGTH OF DERIVED EQUATION:

1.3.1 Sample Calculation of beam R450 x S900: (1P)

Modified P. J. ROBBINS formula as,

V = P. J. ROBBINS formula x BAZANT's size effect factor

$$V = [C_1\left(1 - 0.35\ \frac{x}{D}\right) f_t bD + C_2 \sum^n \left\{A\left(\frac{y_i}{D}\right) sin^2\alpha_i\right\}]$$

$$x\ ((1+d/\lambda_o d_a)^{(-0.5)})$$

V = ultimate shear strength of the beam (N)

C1 = 1.4

C2 = 300 N/mm^2

f_t = 3.55 N/mm^2

b = 75 mm

D = 450 mm

A = 113.04 mm^2

y_i = 425 mm

Clear Shear Span (x) = 365 mm

λ_o = 13.68

d_a = 20 mm (Max. size of aggregate)

$\sin^2\alpha_i = \sin^2(\tan^{-1}(D/x)) = \sin^2(\tan^{-1}(450/365)) = 0.6032$

$$V = [1.4\left(1 - 0.35\ \frac{365}{450}\right) 3.55\ (75)\ (450) + (300) \sum \left\{(113.04)\left(\frac{425}{450}\right)(0.6032)\right\}]$$

$$\times\ ((1+(425)/(13.68)(20))^{(-0.5)})$$

$$= (120118.68 + 19319.28) \times (0.63)$$

V = 87845.91 N = 8.78Ton

So, Ultimate Load = 2 x V = 2 x 8.78 = 17.56 Ton

1.3.2 Sample Calculation of beam F450 x S900: (1P)

Modified P. J. ROBBINS formula as,

V = P. J. ROBBINS formula x BAZANT's size effect factor

$$V = [C_1\left(1 - 0.35\ \frac{x}{D}\right) f_t bD + C_2 \sum^n \left\{A\left(\frac{y_i}{D}\right) sin^2\alpha_i\right\}]$$

$$\times\ ((1+d/\lambda_o d_a)^{(-0.5)})$$

V = ultimate shear strength of the beam (N)

C1 = 1.4

C2 = 300 N/mm^2

f_t =4.7 N/mm^2

b = 75 mm

D = 450 mm

A = 113.04 mm^2

yi = 425 mm

Clear Shear Span (x) = 365 mm

λ_o = 13.68

d_a = 20 mm (Max. size of aggregate)

$\sin^2\alpha_i = \sin^2(\tan^{-1}(D/x)) = \sin^2(\tan^{-1}(450/365)) = 0.6032$

$$V = [1.4\left(1 - 0.35\,\frac{365}{450}\right) 4.7\,(75)\,(450) +$$

$$(300)\sum \left\{(113.04)\left(\frac{425}{450}\right)(0.6032)\right\}]$$

$\times\ ((1+(425)/(13.68)(20))^{(-0.5)})$

$= (159030.37 + 19319.28) \times (0.63)$

V = 112360.28 N = 11.23Ton

So, Ultimate Load = 2 x V = 2 x 11.23 = 22.47Ton

1.4 SAMPLE OF CALCULATION OF ULTIMATE SHEAR STRENGTH USING VARIOUS EQUATION

1.4 SAMPLE OF CALCULATION OF ULTIMATE SHEAR STRENGTH USING APPA RAO EQUATION:

1.4.1 Sample Calculation of beam R450 x S900:(2P)

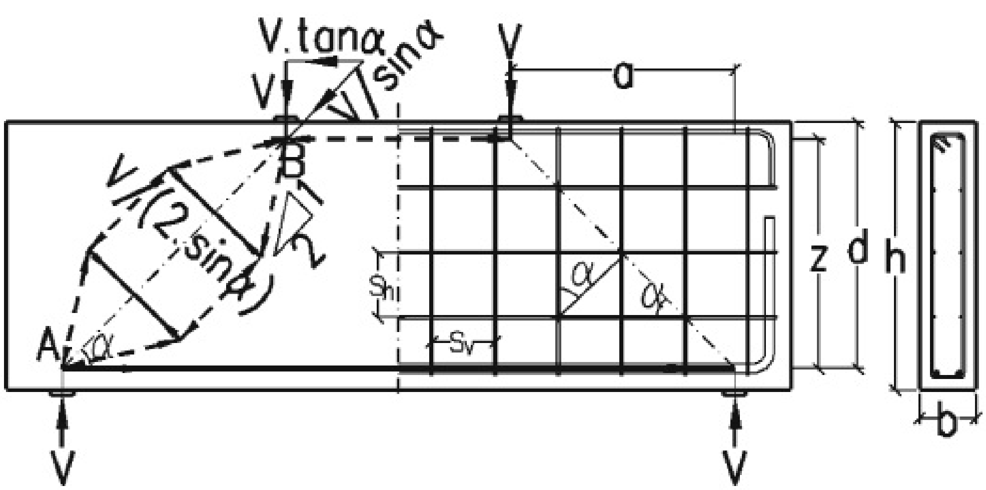


Figure 15-3 Refined strut-and-tie model for RC deep beam

$$v_n = \frac{11.40\rho^{0.35}\sqrt{f'_c}}{1+2\left(\frac{a}{d}\right)}\left(0.38+\frac{1}{\sqrt{1+\left(\frac{d}{25d_a}\right)}}\right)+0.02\rho^{-0.08}\rho_h f_y\left(\frac{d}{a}\right)+0.31\rho_v f_y\left(\frac{a}{d}\right)$$

Where,

νn = Nominal shear stress of the beam

ρ = 0.3%

f'_c =36 MPa

a = 300

d = 425 mm

da = 20 mm

ρh = 0

ρv = 0

$$v_n = \frac{11.40\ 0.003^{0.35}\sqrt{36}}{1+2\left(\frac{300}{425}\right)}\left(0.38+\frac{1}{\sqrt{1+\left(\frac{425}{25\ \mathrm{x}\ 20}\right)}}\right)+0+0$$

$V_n = \frac{8.95}{2.41}$ (1.11)

= 4.12 N/mm²

Ultimate shear capacity= V_nx b x d = 4.12 x 75 x 425

= 13.13 Ton

Ultimate load W_u = 13.13 x 2 = 26.26 Ton

1.5 SAMPLE OF CALCULATION OF ULTIMATE SHEAR STRENGTH USING CHENG AND TANG EQUATION:

1.5.1 Sample Calculation of beam R450 x S900:(2P)

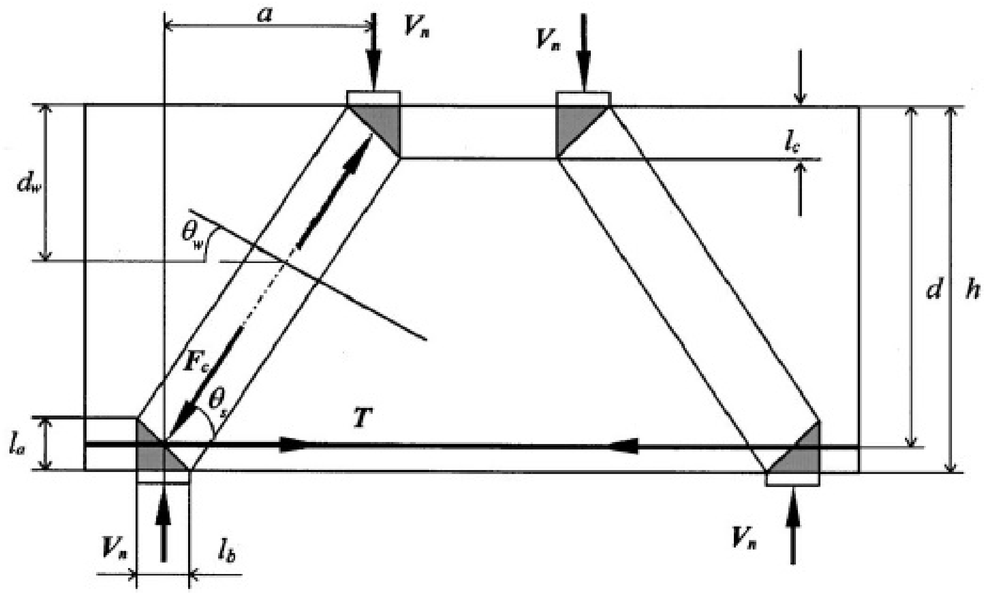


Figure 15-4 Strut-And-Tie Model For Simply Supported Deep Beams

$$V_N = \frac{1}{\frac{\sin(2\theta_s)}{A_c f_t} + \frac{1}{f'_t A_{str} \sin(\theta_s)}}$$

Where,

$$f_t = \frac{2A_s f_y \sin\theta_s}{A_c/\sin\theta_s} + \sum \frac{2A_w f_{yw} \sin(\theta_s + \theta_w)}{A_c/\sin\theta_s} \cdot \frac{d_w}{d} + 0.5\sqrt{f'_c}$$

$$\theta_s = \tan^{-1} D/a = \tan^{-1} 450/300 = 56.30^o$$

$$A_c = 450 \text{ x } 75 = 33750 \text{ mm}^2$$

$$A_s = 113 \text{ mm}^2$$

$$l_b = 85 \text{ mm}$$

$$l_a = \frac{l_b}{\tan\theta_s} = \frac{85}{\tan 56.30} = 56.66$$

$$l = \frac{D - l_a}{\sin\theta_s} = \frac{450 - 56.66}{\sin 56.30} = 472.79$$

$$f'_c = 28$$

$$A_{str} = l \text{ x } b = 472.3 \text{ x } 75 = 35459.25 \text{ mm}^2$$

$$f_t = \frac{2(113)(415)(0.83)}{33750/0.83} + \sum 0\ . + 0.5\sqrt{28}$$

$$f_t = 1.91 + 2.64 = 4.55$$

$$V_N = \frac{1}{\frac{(0.92)}{33750\ \times 4.55} + \frac{1}{(28)(35459.25)(0.83)}} = 13.88\ \text{Ton}$$

Ultimate Load = 2 x 13.88 = 27.76 Ton

1.6 SAMPLE OF CALCULATION OF ULTIMATE SHEAR STRENGTH OF DERIVED EQUATION:

1.6.1 Sample Calculation of beam R450 x S900:(2P)

Modified P. J. ROBBINS formula as,

V = P. J. ROBBINS formula x BAZANT's size effect factor

$$V = \left[C_1\left(1 - 0.35\ \frac{x}{D}\right) f_t\ b\ D + C_2 \sum^n \left\{A\left(\frac{y_i}{D}\right)\sin^2\alpha_i\right\}\right]$$

$$\times\ ((1+d/\lambda_o d_a)^{(-0.5)})$$

V = ultimate shear strength of the beam (N)

C1 = 1.4

C2 = 300 N/mm^2

ft = 3.7 N/mm^2

b = 75 mm

D = 450 mm

A = 113.04 mm^2

yi = 425 mm

Clear Shear Span (x) = 215 mm

λ_o = 13.68

d_a = 20 mm (Max. size of aggregate)

$\sin^2\alpha_i = \sin^2(\tan^{-1}(D/x)) = \sin^2(\tan^{-1}(450/215)) = 0.8142$

$$V = [1.4\left(1 - 0.35\,\frac{215}{450}\right) 3.7\,(75)\,(450) +$$

$$(300) \sum \left\{(113.04)\left(\frac{425}{450}\right)(0.8142)\right\}]$$

x $((1+(425)/(13.68)(20))^{(-0.5)})$

= (145590.37 + 26077.19) x (0.63)

V = 108150.56 N = 10.81 Ton

So, Ultimate Load = 2 x V = 2 x 10.81 = 21.62 Ton

1.6.2 Sample Calculation of beam F450 x S900: (2P)

Modified P. J. ROBBINS formula as,

V = P. J. ROBBINS formula x BAZANT's size effect factor

$$V = [C_1\left(1 - 0.35\,\frac{x}{D}\right) f_t\, b\, D + C_2 \sum^{n} \left\{A\left(\frac{y_i}{D}\right)\sin^2\alpha_i\right\}]$$

x $((1+d/\lambda_o d_a)^{(-0.5)})$

V = Ultimate shear strength of the beam (N)

C1 = 1.4

C2 = 300 N/mm^2

f_t = 4.6 N/mm^2

b = 75 mm

D = 450 mm

A = 113.04 mm^2

yi = 425 mm

Clear Shear Span (x) = 215 mm

λ_o = 13.68

d_a = 20 mm (Max. size of aggregate)

$\sin^2\alpha_i = \sin^2(\tan^{-1}(D/x)) = \sin^2(\tan^{-1}(450/215)) = 0.8142$

$$V = [1.4\left(1 - 0.35\,\frac{215}{450}\right)4.6\,(75)\,(450) +$$

$$(300)\sum \left\{(113.04)\left(\frac{425}{450}\right)(0.8142)\right\}]$$

$$x\ ((1+(425)/(13.68)(20))^{(-0.5)})$$

$$= (181004.25 + 26077.19) \times (0.63)$$

V = 130461.30 N = 13.04 Ton

So, Ultimate Load = 2 x V = 2 x 13.04 = 26.08 Ton

REGRESSION ANALYSIS

Experimental Ultimate Load = 2 x P. J. ROBBINS formula x BAZANT's size effect factor

$$W_u = 2\,x\,[C_1\left(1 - 0.35\,\frac{x}{D}\right)f_t bD + C_2\sum^n\left\{A\left(\frac{y_i}{D}\right)sin^2\alpha_i\right\}]$$

$$x\ ((1+d/\lambda_o d_a)^{(-0.5)})$$

$$\frac{1}{(1+\frac{d}{\lambda oda})^{(0.5)}} = \frac{W_u}{2\,x\,[C_1\left(1 - 0.35\,\frac{x}{D}\right)f_t bD +\ C_2\sum^n\left\{A\left(\frac{y_i}{D}\right)sin^2\alpha_i\right\}]}$$

$$\frac{d}{\lambda oda} = \sqrt{\frac{2\,x\,[C_1\left(1 - 0.35\,\frac{x}{D}\right)f_t bD +\ C_2\sum^n\left\{A\left(\frac{y_i}{D}\right)sin^2\alpha_i\right\}]}{W_u}} - 1$$

$$\frac{d}{da[\sqrt{\frac{2\,x\,[C_1\left(1-0.35\,\frac{x}{D}\right)f_t bD+\ C_2\sum^n\left\{A\left(\frac{y_i}{D}\right)sin^2\alpha_i\right\}]}{W_u}} - 1]} = \lambda o$$

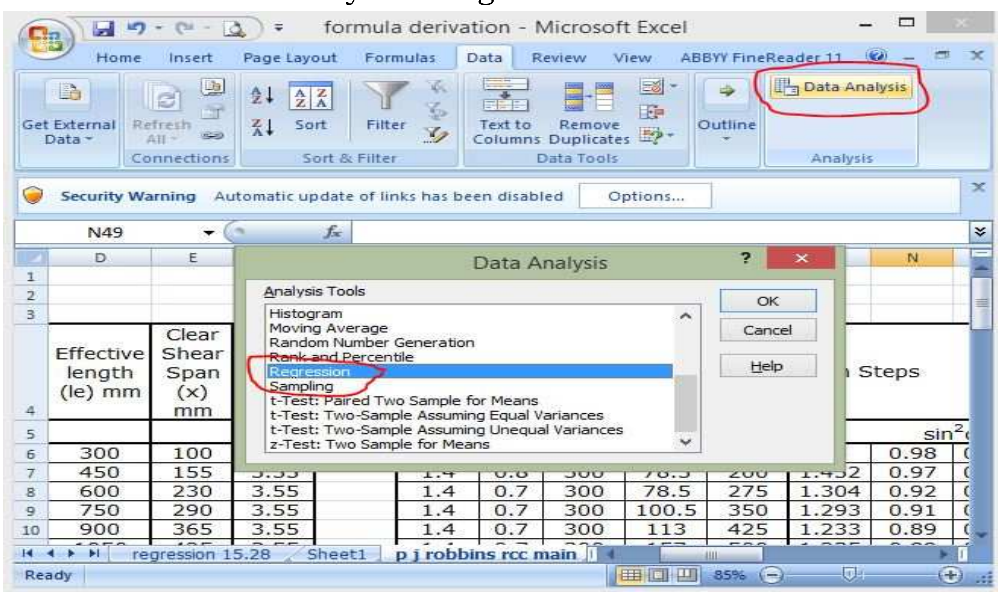
λ_o is calculated for RCC using the above equation. The result of $\lambda_o$are given as,

For RCC 1P		For RCC 2P
Depth	λo	λo
150	1.466	19.45
225	6.315	2.566

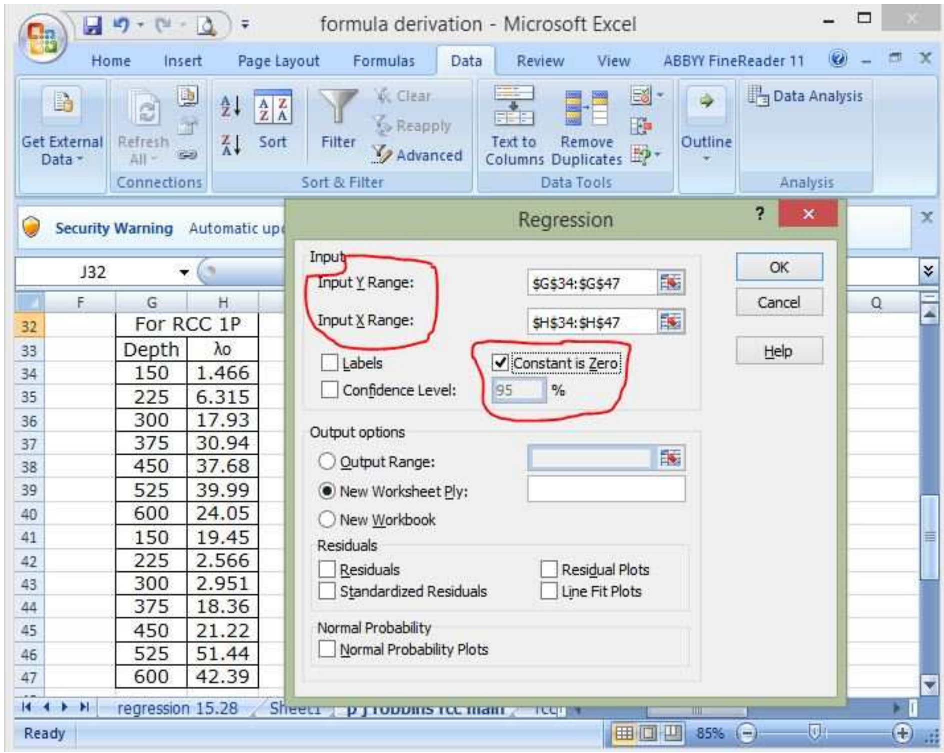
300	17.93	2.951
375	30.94	18.36
450	37.68	21.22
525	39.99	51.44
600	24.05	42.39

1.7 Regression procedure

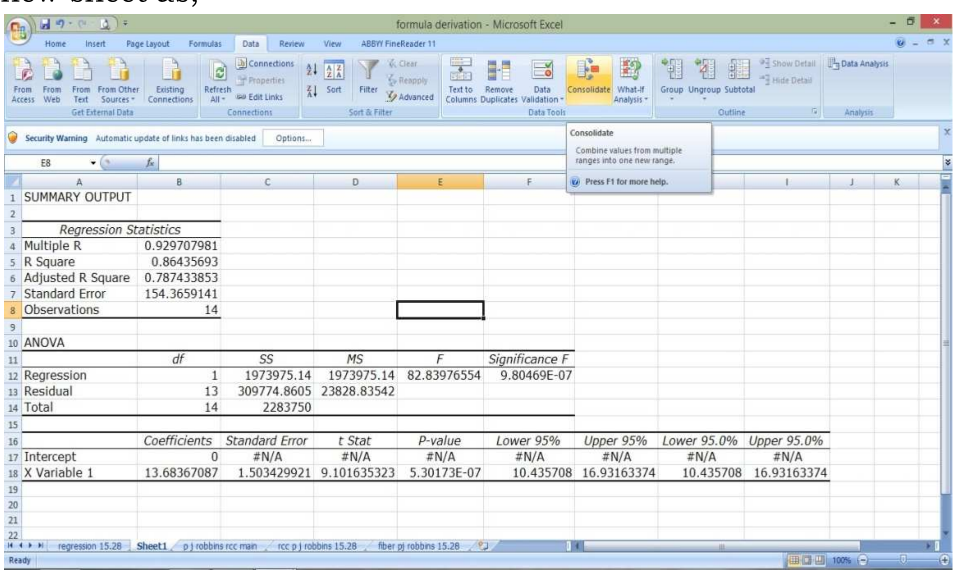
1. Define the dependent variable and independent variable
   - Dependent variable = Depth
   - Independent variable = λo
2. Go to Data > Data Analysis > Regression



3. Select the dependent variable in Y range and Independent variable in X range.



4. In this step taking constant equal to Zero (0) to gain the intercept Zero (0) in Regression. Here confidence level is taken as 95% so P-value and significance F should be less than 5% e.g.<0.05.
5. After clicking OK in Third step we get the regression output in new sheet as,



SUMMARY OUTPUT

Regression Statistics	
Multiple R	0.929707981
R Square	0.86435693
Adjusted R Square	0.787433853
Standard Error	154.3659141
Observations	14

ANOVA

	df	SS	MS	F	Significance F
Regression	1	1973975.14	1973975.14	82.83976554	9.80469E-07
Residual	13	309774.8605	23828.83542		
Total	14	2283750			

	Coefficients	Standard Error	t Stat	P-value	Lower 95%	Upper 95%	Lower 95.0%	Upper 95.0%
Intercept	0	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A
X Variable 1	13.68367087	1.503429921	9.101635323	5.30173E-07	10.435708	16.93163374	10.435708	16.93163374

6. From output here we get the co-efficient X variable as 13.68.

λ_o is calculated for RCC equal to 13.68 from Regression Analysis.

APPENDIX-III

MODIFICATION OF S.N.PATEL & S.K.DAMALE'S EQUATION

1 SAMPLE OF CALCULATION OF ULTIMATE SHEAR STRENGTH USING CHENG AND TANG EQUATION:

1.1 Sample Calculation for R30

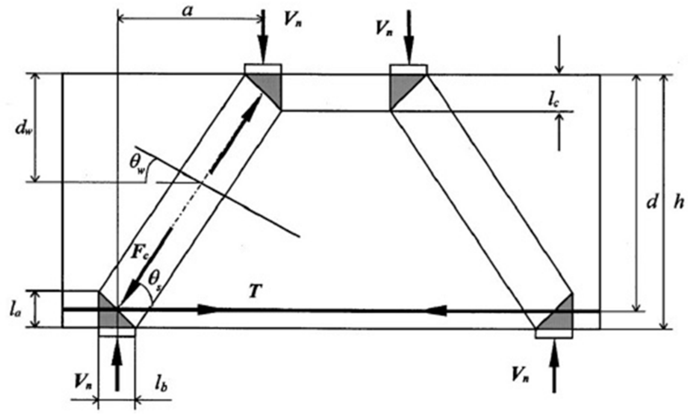


Fig. 16.1 : Strut-and-tie model for simply supported deep beams

$$V_N = \frac{1}{\frac{\sin(2\theta_s)}{A_c f_t} + \frac{1}{f'_t A_{str} \sin(\theta_s)}}$$

Where,

$$f_t = \frac{2A_s f_y \sin\theta_s}{A_c/\sin\theta_s} + \sum \frac{2A_w f_{yw} \sin(\theta_s + \theta_w)}{A_c/\sin\theta_s} \cdot \frac{d_w}{d} + 0.5\sqrt{f'_c}$$

$$\theta_s = \tan^{-1}\frac{D}{a} = \tan^{-1}\frac{300}{300} = 45^o$$

a = 300 mmb = 75 mmD = 300 mm

a = 300 mmF_c'= 44.02 N/mm^2F_y = 500 N/mm^2

A_s = 100.48 mm^2

$$l_b = 75 \text{ mm}$$

$$l_a = \frac{l_b}{\tan\theta_s} = \frac{75}{\tan 45} = 75\text{mm}$$

$$l = \frac{D - l_a}{\sin\theta_s} = \frac{300 - 75}{\sin 50} = 318.20 \text{ mm}$$

$$A_{str} = l \times b = 318.20 \times 75 = 23864.85 \text{ mm}^2$$

$$f_t = \frac{2(100.53)(500)(0.79)}{22500/0.79} + \sum 0\ . + 0.5\sqrt{44.021}$$

$$f_t = 2.23 + 3.32 = 5.55\ \text{N/mm}^2$$

$$V_N = \frac{1}{\frac{(1)}{22500\ x\ 5.55} + \frac{1}{(44.02)(23864.85)(0.707)}} = 106927.98\ \text{N.}$$

$$= 10.9\ \textbf{Tonne}$$

Ultimate Load = 2 x 10.9 = 21.8 Tonne

1.2 SAMPLE OF CALCULATION OF ULTIMATE SHEAR STRENGTH USING NEHDI OPTIMIZED EQUATION:

1.2.1 Sample Calculation for R15

$$Vc = 2.1\left(\frac{f'c\ \rho fl\ d}{a}\frac{Efl}{Es}\right)^{0.23} b\ d$$

here,

$$\text{If } \frac{a}{d} < 2.5$$

$$\text{So,} \quad Vc = 2.1\left(\frac{f'c\ \ \rho fl\ \ d}{a}\ \frac{Efl}{Es}\right)^{0.23}\ b\ \ d\ \times\ \frac{2.5\ d}{a}$$

Here,

a = 150 mm

d = 130 mm

b = 75 mm

$f'c = 41.41$ N/mm²

$\rho fl = 0.447\ \%$

$E_{fl} = E_s = 210000\ \text{N/mm}^2$

$$Vc = 2.1 \times \left(\frac{41.41\ \times 0.0045 \times 130\ \ d}{150}\right)^{0.23} \times 75\ \times 130\ \times\ \frac{2.5\ \times 130}{150}$$

V_c= 29118.79 N

Wc = 2 x 29118.79 = 58237.59 N

Wc = 5.938 Tonne

1.3 SAMPLE OF CALCULATION OF ULTIMATE SHEAR STRENGTH USING S N PATEL AND DR S. K. DAMLE'S EQUATION (ORIGINAL EQUATION)

1.3.1 Sample Calculation of beam R15

$$Wu = 2Vu = \frac{3.ft.b.d}{\sqrt{1+0.75(\frac{a}{d})^2}} + \frac{fy}{\sqrt{1+(\frac{a}{d})^2}} . \sum_0^i (\frac{yi}{d} + \mathbf{0.4}). \boldsymbol{Asi.sin}\ (\boldsymbol{\alpha 1} + \boldsymbol{\theta}) \ .....(1)$$

- f_t =Tensile strength of concrete = 3.47 N/mm^2
- b = width of beam= 75 mm
- d = effective depth of beam = 130 mm
- a = shear span = 150 mm
- f_y = yield strength of reinforcement = 500N/mm^2
- y_i = depth of reinforcement layer from top of beam 130 mm
- A_{si} = area of steel in ith level = 50.26 mm^2
- $\propto \mathbf{1} = \frac{3}{\sqrt{1+0.75\left(\frac{a}{d}\right)^2}}, = \frac{150}{\sqrt{1+0.75\left(\frac{150}{130}\right)^2}}, = 2.122$
- $\boldsymbol{\theta}$ == 45 = 0.785

$$Wu = 2Vu = \frac{3\times75\times130}{\sqrt{1+0.75(\frac{150}{130})^2}} + \frac{500}{\sqrt{1+(\frac{150}{130})^2}} . \sum_0^i (\frac{130}{130} + \mathbf{0.4}) \times \mathbf{50.26} \times \boldsymbol{sin}\ (\mathbf{2.12} + \mathbf{0.785}\ )$$

Wu = 87489.80391 N

Wu = 8.92 Tonne

1.4 SAMPLE OF CALCULATION OF ULTIMATE SHEAR STRENGTH USING MODIFIED EQUATION

1.4.1 Sample Calculation of beam R15

$$Wu = 2Vu = \frac{3.ft.b.d}{\sqrt{1+0.75(\frac{a}{d})^2}} S_f + \frac{fy}{\sqrt{1+(\frac{a}{d})^2}} . \sum_0^i (\frac{yi}{d} + \mathbf{0.4}). \boldsymbol{Asi.sin}(\propto \mathbf{1} + \boldsymbol{\theta})$$

S_f = 3.38 X (d)$^{-1/3}$ X (fck)$^{1/3}$$_{-1.27}$

=3.38 X (130)$^{-1/3}$X(41.41)$^{1/3}$ -1.27

= 1.038

$$Wu = (\frac{3\times75\times130}{\sqrt{1+0.75(\frac{150}{130})^2}} \times \mathbf{1.038}\ ) + \frac{500}{\sqrt{1+(\frac{150}{130})^2}} . \sum_0^i (\frac{130}{130} + \mathbf{0.4}) \times \mathbf{50.26} \times \boldsymbol{sin}\ (\mathbf{2.12} + \mathbf{0.785}\ )$$

Wu = 9.20 Tonne

1.5 SAMPLE OF CALCULATION OF ULTIMATE SHEAR STRENGTH USING ORIGINAL P J ROBINS EQUATION:

1.5.1 Sample Calculation of beam RF35:

Vu = P. J. ROBINS formula

$$\mathbf{Vu} = [C_1\left(1 - 0.35\,\frac{x}{D}\right) f_t\, b\, D + C_2 \sum^n \left\{A\left(\frac{y_i}{D}\right)\sin^2\alpha_i\right\}]$$

Vu= ultimate shear strength of the beam (N)

C1 = 1.4

C2 = 300 N/mm^2

ft = 4.58 N/mm^2

b = 75 mm

D = 350 mm

As= 157.08 mm^2

yi = 425 mm

Clear Shear Span (x) = 350mm

da = 20 mm (Max. size of aggregate)

$Sin^2\alpha_i$= $Sin^2(tan^{-1}(D/x))$ = $Sin^2(tan^{-1}(350/350))$ = 0.66

$$\mathbf{Vu} = [C_1\left(1 - 0.35\,\frac{x}{D}\right) f_t\, b\, D + C_2 \sum^n \left\{A\left(\frac{y_i}{D}\right)\sin^2\alpha_i\right\}]$$

$$\mathbf{Vu} = [\mathbf{1.41.4} \times \left(\mathbf{1} - \mathbf{0.35}\,\frac{\mathbf{350}}{\mathbf{350}}\right) \times \mathbf{4.58}\,(\mathbf{75})\,(\mathbf{350})$$

$$+(\mathbf{300}) \sum \left\{(\mathbf{157.08})\left(\frac{\mathbf{330}}{\mathbf{350}}\right)(\mathbf{0.66})\right\}]$$

Vu = 155609.5 N

Vu = 15.87 31.74 Tonne

So, Ultimate Load = 2 x Vu = 2 x 15.87 = 31.74 Tonne

1.6 SAMPLE OF CALCULATION OF ULTIMATE SHEAR STRENGTH USING MODIFIED P. J. ROBINS FORMULA

V = P. J. ROBINS formula x BAZANT's size effect factor

$$V = \left[C_1\left(1-0.35\,\frac{x}{D}\right)f_t\,b\,D + C_2\sum^{n}\left\{A\left(\frac{y_i}{D}\right)\sin^2\alpha_i\right\}\right] \times \left(\left(1+d/\lambda_o d_a\right)^{(-0.5)}\right)$$

Vu= ultimate shear strength of the beam (N)

C1 = 1.4

C2 = 300

ft = 4.58 N/mm^2

b = 75 mm

D = 350 mm

As= 157.08 mm^2

yi = 425 mm

Clear Shear Span (x) = 350mm

da = 20 mm (Max. size of aggregate)

$\sin^2\alpha_i$= sin^2(tan^{-1}(D/x)) = Sin^2(tan^{-1}(350/350)) = 0.66

λo = 13.68

$$V = Vu = \left[1.41.4 \times \left(1-0.35\,\frac{350}{350}\right) \times 4.58\,(75)\,(350) + (300)\sum\left\{(157.08)\left(\frac{330}{350}\right)(0.66)\right\}\right] \times \left(\left(1+(330)/(13.68)(20)\right)^{(-0.5)}\right)$$

= (181004.25 + 26077.19) x (0.673261)

Vu =10.68 Tonne

So, Ultimate Load = 2 x Vu = 21.36616Tonne

1.7 SAMPLE OF CALCULATION OF ULTIMATE SHEAR STRENGTH USING DR. S. N. PATEL AND DR. S. K. DAMLE (ORIGINAL EQUATION IT WITHOUT SIZE FACTOR) EQUATION:

1.7.1 Sample Calculation of beam R15

$$\textbf{Wu = 2Vu} = \frac{3.ft.b.d}{\sqrt{1+0.75(\frac{a}{d})^2}} + \frac{fy}{\sqrt{1+(\frac{a}{d})^2}} \cdot \sum_0^i\left(\frac{yi}{d} + 0.4\right).\mathrm{Asi}.\sin(\alpha 1 + \theta) \quad .....(1)$$

- ft =Tensile strength of concrete = 3.47
- b = width of beam= 75
- d = effective depth of beam = 130
- a = shear span 150
- fy = yield strength of reinforcement 500
- yi = depth of reinforcement layer from top of beam 130
- Asi = area of steel in ith level = 50.26

- $\propto 1 = \frac{3}{\sqrt{1+0.75\left(\frac{a}{d}\right)^2}}, = \frac{150}{\sqrt{1+0.75\left(\frac{150}{130}\right)^2}}, =2.122$
- $\theta = \tan^{-1}\frac{D}{a}$ = ten $^{-1}\frac{150}{150}$ = 45° = 0.785

Wu = 2Vu = $\frac{3\times75\times130}{\sqrt{1+0.75(\frac{150}{130})^2}} + \frac{500}{\sqrt{1+(\frac{150}{130})^2}} \cdot \sum_0^i\left(\frac{130}{130} + 0.4\right) \times 50.26 \times \sin(2.12 + 0.785)$

Wu = 87489.80391 N

Wu = 8.92 Tonne

1.8 SAMPLE OF CALCULATION OF ULTIMATE SHEAR STRENGTH USING USING MODIFIED EQUATION

1.8.1 Sample Calculation of beam R15

$$\textbf{Wu = 2Vu} = \frac{3.ft.b.d}{\sqrt{1+0.75(\frac{a}{d})^2}} S_f + \frac{fy}{\sqrt{1+(\frac{a}{d})^2}} \cdot \sum_0^i\left(\frac{yi}{d} + 0.4\right).\mathrm{Asi}.\sin(\propto 1 + \theta)$$

Sf(2nd) = $57.64.\,d^{\frac{-2}{3}}$-9.65 $d^{\frac{-1}{3}}$+ 0.61

=57.67 X (130)-2/3-9.65 X (130)-1/3 +0.61

= 0.9492

$$Wu = \left(\frac{3\times75\times130}{\sqrt{1+0.75\left(\frac{150}{130}\right)^2}}\times 0.9492\right) + \frac{500}{\sqrt{1+\left(\frac{150}{130}\right)^2}} \cdot \sum_0^i \left(\frac{130}{130}+0.4\right)\times 50.26\times \sin(2.12 + 0.785)$$

Wu = 83847.36 N = 8.55 Tonne

1.9 SAMPLE OF CALCULATION OF ULTIMATE SHEAR STRENGTH MODIFIED IS

1.9.1 Sample Calculation of beam R15

$$Vc = k1\ k2\ \tau c\ b\ d$$

$$\tau c = \frac{0.85 \times \sqrt{0.8\,fck} \times (\sqrt{1+5\beta} - 1)}{6 \times \beta}$$

$$\beta = \frac{0.8 \times fck}{45.55 \times \rho t} > 1$$

Were,

k1 is a function of shear span to depth (a/d) ratio

factor k2 accounts for the size effect of beams

$$k1 = 2.2 \times \frac{d}{a} + 0.12 \quad \text{When } \frac{a}{d} < 2.5$$

$$k1 = 1 \quad \text{When } \frac{a}{d} \geq 2.5$$

$$k2 = 1 \quad \text{When } d \leq 300\ mm$$

$$k2 = \frac{750}{450 + d} \quad \text{When } d > 300\ mm$$

Fck = 41.41 N/mm^2 Pt = 0.4468%

b = 75 mm d = 130 mm

a = 150 mm D = 150 mm

$$\beta = \frac{0.8 \times fck}{45.55 \times \rho t} > 1$$

$$\beta = \frac{0.8 \times 41.41}{45.55 \times 0.4468} = 1.6275 > 1$$

$$\beta = 1.6275$$

$$\tau c = \frac{0.85 \times \sqrt{0.8\,fck} \times \left(\sqrt{1+5\beta} - 1\right)}{6 \times \beta}$$

$$\tau c = \frac{0.85 \times \sqrt{0.8 \times 41.41} \times (\sqrt{1+(5\times1.6275)} - 1)}{6 \times 1.6275} = 1.0133\ N/mm^2$$

$$\mathbf{k1 = 2.2 \times \frac{d}{a} + 0.12 \quad When \ \frac{a}{d} \ < 2.5}$$

$$\mathbf{\frac{a}{d} \ = \ \frac{150}{130} = 1.15 \ < \ 2.5}$$

$$\mathbf{k1 = 2.2 \times \frac{130}{150} + 0.12 = 2.03}$$

$$\mathbf{k2 = 1 \quad When \ \ d \leq 300 \, mm}$$

$$\mathbf{Vc = k1 \, k2 \, \tau c \, b \, d}$$

$$\mathbf{Vc = 2.03 \ \times 1 \ 1.0133 \ \times 75 \ \times 130 = 20028.46 \, N}$$

$$\mathbf{Wc = Vc \ \times 2}$$

$$\mathbf{Wc = 40048.92 \ \ N}$$

$$\mathbf{Wc = 4.08 \ \ Tonne}$$

ANNEXTURE IV

MODIFICATION OF SHEAR STRENGTH FORMULA OF EUROCODE

1. SAMPLE OF CALCULATION OF ULTIMATE SHEAR STRENGTH USING MODIFIED EUROCODE-02 EQUATION:

1.1. Sample Calculation Of RCC Beam

Name of Beam	:	F/RCC
Depth, D	:	175 mm
Width, b	:	75 mm
Length, L	:	700 mm
Effective Length, l	:	600 mm
Effective Depth, d	:	150 mm
Compressive Strength of concrete f_{ck}	:	31.11 Mpa

- **Area of Steel**

Two bars of 8 mm dia. is provided as flexure reinforcement.

Diameter of bar d_b= 8 mm

$A_{st}= \frac{\pi}{4} d_b^2$

$= 50.26\ mm^2$

- **Percentage of Reinforcement**

$pt = \frac{Ast}{bd}$ x 100%

$= 0.4468\%$

- **Shear span to Depth ratio**

Shear Span, a= Effective depth / 2

= 600/2

= 300 mm

Shear Span to depth ratio = a/d

= 300/150

= 2

➢ **Shear Strength calculation**

$v_c = 0.84(1 + \sqrt{\frac{200}{d}})(\sqrt[3]{ptfck})(\frac{a}{d})^{-1.033}$ bd.

$= 0.84 \times (1 + \sqrt{\frac{200}{150}})(\sqrt[3]{0.4468 \times 31.11})(2)^{-1.033} \times 75 \times 150.$

= 6.15 ton

1.1.1. Sample Calculation Of PFRC Beam

Name of Beam	:	F/PFRC
Depth, D	:	175 mm
Width, b	:	75 mm
Length, L	:	700 mm
Effective Length, l	:	600 mm
Effective Depth, d	:	150 mm
Compressive Strength of concrete f_{ck}	:	34.25 Mpa
Type of Fibre	:	Polypropylene
Content of Fibre	:	0.7% of concrete volume
Length of Fibre	:	6.0 mm
Diameter of Fibre	:	0.03 mm

- **Area of Steel**

Two bars of 8 mm dia. is provided as flexure reinforcement.

Diameter of bar d_b= 8 mm

$A_{st}= \frac{\pi}{4} d_b^2$

$= 50.26\ mm^2$

- **Percentage of Reinforcement**

$pt = \frac{Ast}{bd}$ x 100%

$= 0.4468\%$

- **Shear span to Depth ratio**

Shear Span, a= Effective depth / 2

= 600/2

= 300 mm

Shear Span to depth ratio = a/d

= 300/150

= 2

- **Aspect Ratio of Fibre**

$\frac{lf}{df}$= Length of Fibre/Dia. of Fibre

= 6.0 / 0.03

= 200

- **Shear Strength calculation**

v_c= $0.84((1 + \sqrt{\frac{200}{d}})(\sqrt[3]{ptfck}) (\frac{a}{d})^{-1.033} bd + \beta v_f (\frac{lf}{df}))$;

Here; $\beta = 0.07\sqrt{fck}$ for monofilament polypropylene fibers

$Pt = 100\frac{Ast}{bwd}$ Reinforcement ratio

$= 0.84 \times 75 \times 150 \times ((1 + \sqrt{\frac{200}{150}})(\sqrt[3]{0.4468 \times 34.25})(2)^{-1.033})+(0.07 \times \sqrt{34.25} \times 200)$

= 7.23 ton

1.1.2. Sample Calculation Of SFRC Beam

Name of Beam	:	F/SFRC
Depth, D	:	175 mm
Width, b	:	75 mm
Length, L	:	700 mm
Effective Length, l	:	600 mm
Effective Depth, d	:	150 mm
Compressive Strength of concrete f_{ck}	:	31.78 Mpa
Type of Fibre	:	End hook type Steel fibre
Content of Fibre	:	0.7% of concrete volume
Length of Fibre	:	30 mm
Diameter of Fibre	:	0.6 mm

- **Area of Steel**

Two bars of 8 mm dia. is provided as flexure reinforcement.

Diameter of bar d_b= 8 mm

$A_{st}= \frac{\pi}{4} d_b^2$

$= 50.26 \text{ mm}^2$

- **Percentage of Reinforcement**

$pt = \frac{Ast}{bd} \text{ x } 100\%$

$= 0.4468\%$

Shear span to Depth ratio

Shear Span, a= Effective depth / 2

= 600/2

= 300 mm

Shear Span to depth ratio = a/d

= 300/150

= 2

Aspect Ratio of Fibre

$\frac{lf}{df}$= Length of Fibre/Dia. of Fibre

= 30 / 0.6

= 50

Shear Strength calculation

$v_c = 0.84((1 + \sqrt{\frac{200}{d}})(\sqrt[3]{ptfck})\ (\frac{a}{d})^{-1.033}\ bd + \beta v_f\ (\frac{lf}{df}))$;

Here; $\beta = 1.17\sqrt{fck}$ for end hook type steel fibers

$Pt = 100\frac{Ast}{bwd}$ Reinforcement ratio

$= 0.84 \times 75 \times 150 \times ((1 + \sqrt{\frac{200}{150}})(\sqrt[3]{0.4468 \times 31.78})(2)^{-1.033}) + (1.17 \times \sqrt{31.78} \times 50)$

= 8.83 ton

1.2. EUROCODE 02-2004

(2) The shear resistance of a member with shear reinforcement is equal to:

$$V_{Rd} = V_{Rd,s} + V_{ccd} + V_{td} \quad (6.1)$$

(3) In regions of the member where $V_{Ed} \leq V_{Rd,c}$ no calculated shear reinforcement is necessary. V_{Ed} is the design shear force in the section considered resulting from external loading and prestressing (bonded or unbonded).

(4) When, on the basis of the design shear calculation, no shear reinforcement is required, minimum shear reinforcement should nevertheless be provided according to 9.2.2. The minimum shear reinforcement may be omitted in members such as slabs (solid, ribbed or hollow core slabs) where transverse redistribution of loads is possible. Minimum reinforcement may also be omitted in members of minor importance (e.g. lintels with span ≤ 2 m) which do not contribute significantly to the overall resistance and stability of the structure.

(5) In regions where $V_{Ed} > V_{Rd,c}$ according to Expression (6.2), sufficient shear reinforcement should be provided in order that $V_{Ed} \leq V_{Rd}$ (see Expression (6.8)).

(6) The sum of the design shear force and the contributions of the flanges, $V_{Ed} - V_{ccd} - V_{td}$, should not exceed the permitted maximum value $V_{Rd,max}$ (see 6.2.3), anywhere in the member.

(7) The longitudinal tension reinforcement should be able to resist the additional tensile force caused by shear (see 6.2.3 (7)).

(8) For members subject to predominantly uniformly distributed loading the design shear force need not to be checked at a distance less than d from the face of the support. Any shear reinforcement required should continue to the support. In addition it should be verified that the shear at the support does not exceed $V_{Rd,max}$ (see also 6.2.2 (6) and 6.2.3 (8).

(9) Where a load is applied near the bottom of a section, sufficient vertical reinforcement to carry the load to the top of the section should be provided in addition to any reinforcement required to resist shear.

6.2.2 Members not requiring design shear reinforcement

(1) The design value for the shear resistance $V_{Rd,c}$ is given by:

$$V_{Rd,c} = [C_{Rd,c}k(100\,\rho_l f_{ck})^{1/3} + k_1\,\sigma_{cp}]\,b_w d \quad (6.2.a)$$

with a minimum of

$$V_{Rd,c} = (v_{min} + k_1\sigma_{cp})\,b_w d \quad (6.2.b)$$

where:

f_{ck} is in MPa

$k \quad = 1+\sqrt{\frac{200}{d}} \leq 2,0$ with d in mm

$\rho_l \quad = \frac{A_{sl}}{b_w d} \leq 0,02$

A_{sl} is the area of the tensile reinforcement, which extends $\geq (l_{bd} + d)$ beyond the section considered (see Figure 6.3).

b_w is the smallest width of the cross-section in the tensile area [mm]
σ_{cp} = N_{Ed}/A_c < 0,2 f_{cd} [MPa]
N_{Ed} is the axial force in the cross-section due to loading or prestressing [in N] (N_{Ed}>0 for compression). The influence of imposed deformations on N_E may be ignored.
A_C is the area of concrete cross section [mm²]
$V_{Rd,c}$ is [N]

Note: The values of $C_{Rd,c}$, v_{min} and k_1 for use in a Country may be found in its National Annex. The recommended value for $C_{Rd,c}$ is $0,18/\gamma_c$, that for v_{min} is given by Expression (6.3N) and that for k_1 is 0,15.

$$v_{min} = 0,035\, k^{3/2} \cdot f_{ck}^{1/2} \quad (6.3N)$$

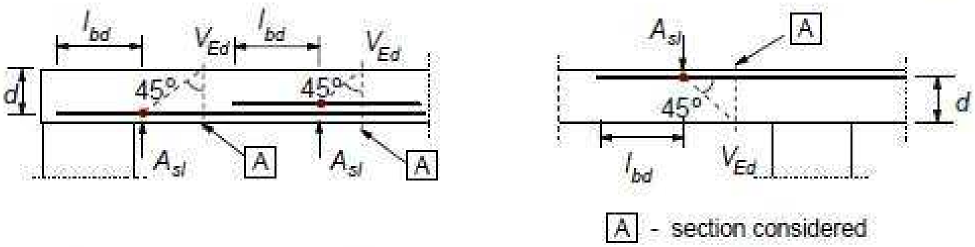


Figure 6.3: Definition of A_{sl} in Expression (6.2)

(2) In prestressed single span members without shear reinforcement, the shear resistance of the regions cracked in bending may be calculated using Expression (6.2a). In regions uncracked in bending (where the flexural tensile stress is smaller than $f_{ctk,0,05}/\gamma_c$) the shear resistance should be limited by the tensile strength of the concrete. In these regions the shear resistance is given by:

$$V_{Rd,c} = \frac{I \cdot b_w}{S} \sqrt{(f_{ctd})^2 + \alpha_l \sigma_{cp} f_{ctd}} \quad (6.4)$$

where
I is the second moment of area
b_w is the width of the cross-section at the centroidal axis, allowing for the presence of ducts in accordance with Expressions (6.16) and (6.17)
S is the first moment of area above and about the centroidal axis
α_l = l_x/l_{pt2} ≤ 1,0 for pretensioned tendons
= 1,0 for other types of prestressing
l_x is the distance of section considered from the starting point of the transmission length
l_{pt2} is the upper bound value of the transmission length of the prestressing element according to Expression (8.18).
σ_{cp} is the concrete compressive stress at the centroidal axis due to axial loading and/or prestressing ($\sigma_{cp} = N_{Ed}/A_c$ in MPa, N_{Ed} > 0 in compression)

For cross-sections where the width varies over the height, the maximum principal stress may occur on an axis other than the centroidal axis. In such a case the minimum value of the shear resistance should be found by calculating $V_{Rd,c}$ at various axes in the cross-section.

APPENDIX V

MODIFICATION OF SHEAR STRENGTH EQUATION OF CSA CODE

1. SHEAR STRENGTH CALCULATION AS PER CANADIAN STANDARDS-ASSOCIATION CSA-A23.3-04:

As per chapter -11 Shear and torsion (page no.53) the following procedure adopted for find out shear strength of concrete:

- As per **clause 11.3.3** Factored shear resistance

  The factored shear resistance shall be determined by:

  $$V_r = V_c + V_s + V_p$$

  However, V_r shall not exceed

  $V_{r\,max} = 0.25\varphi_c f_c' b_w \mathrm{D} + V_p$

- As per **clause 11.3.4:**

  The value $V_c$of shall be computed from:

  $V_c = \beta \text{ x} \sqrt{f_c'} \text{ x } \lambda \text{ x} \phi_c \text{x } b_w \text{ x D}$

  Where β is determined as specified in Clause 11.3.6.

  In the determination of V_c, the term shall not be taken greater than 8 MPa.

- As per **clause 11.3.6.3 determination of β:**

  In lieu of more accurate calculations in accordance with Clause 11.3.6.4, and provided that the specified yield strength of the longitudinal steel reinforcement does not exceed 400 MPa and the specified concrete strength does not exceed 60 MPa, θ shall be taken as 35° and β shall be determined as follows:

  (a) If the section contains at least the minimum transverse reinforcement as specified by Equation (11-1),

  β Shall be taken as 0.18.

  (b) If the section contains no transverse reinforcement and the specified nominal maximum size of coarse aggregate is not less than 20 mm, β shall be taken as

$$\boldsymbol{\beta} = \left(\frac{\mathbf{230}}{\mathbf{1000} + \boldsymbol{d}}\right)$$

(c) Alternatively, the value of β for sections containing no transverse reinforcement may be determined for all aggregate sizes by replacing the parameter d in above Equation by the equivalent crack spacing parameter, s_{ze}, where

$$s_{ze} = \frac{35s_z}{15 + a_g}$$

Where,

s_z= Spacing between longitudinal steel bars

a_g= Nominal maximum size of aggregate

11.3 Design for shear and torsion in flexural regions

11.3.1 Required shear resistance

Members subjected to shear shall be proportioned so that

$$V_r \geq V_f \qquad \textbf{(11-3)}$$

11.3.2 Sections near supports

Sections located less than a distance d_v from the face of the support may be designed for the same shear, V_f, as that computed at a distance d_v, provided that

(a) the reaction force in the direction of applied shear introduces compression into the member; and

(b) no concentrated load that causes a shear force greater than $0.3\lambda\phi_c\sqrt{f_c'}b_w d_v$ is applied within the distance d_v from the face of the support.

11.3.3 Factored shear resistance

The factored shear resistance shall be determined by

$$V_r = V_c + V_s + V_p \qquad \textbf{(11-4)}$$

However, V_r shall not exceed

$$V_{r,max} = 0.25\phi_c f_c' b_w d_v + V_p \qquad \textbf{(11-5)}$$

11.3.4 Determination of V_c

The value of V_c shall be computed from

$$V_c = \phi_c \lambda \beta \sqrt{f_c'} b_w d_v \qquad \textbf{(11-6)}$$

where β is determined as specified in Clause 11.3.6.

In the determination of V_c, the term $\sqrt{f_c'}$ shall not be taken greater than 8 MPa.

11.3.5 Determination of V_s

11.3.5.1

For members with transverse reinforcement perpendicular to the longitudinal axis, V_s shall be computed from

$$V_s = \frac{\phi_s A_v f_y d_v \cot\theta}{s} \qquad \textbf{(11-7)}$$

where θ is determined as specified in Clause 11.3.6.

11.3.5.2

For members with transverse reinforcement inclined at an angle α to the longitudinal axis, V_s shall be computed from

$$V_s = \frac{\phi_s A_v f_y d_v (\cot\theta + \cot\alpha)\sin\alpha}{s} \qquad \textbf{(11-8)}$$

where θ is determined as specified in Clause 11.3.6.

11.3.6 Determination of β and θ

11.3.6.1 Members subjected to significant axial tension

For members subjected to significant axial tension, the values of β and θ shall be determined as specified in Clause 11.3.6.4.

11.3.6.2 Values for special member types

Unless otherwise permitted by Clause 11.3.6.3 or Clause 11.3.6.4, the value of β shall be taken as 0.21 and θ shall be taken as 42° for any of the following member types:

(a) slabs or footings with an overall thickness not greater than 350 mm;
(b) footings in which the distance from the point of zero shear to the face of the column, pedestal, or wall is less than three times the effective shear depth of the footing;
(c) beams with an overall thickness not greater than 250 mm;
(d) concrete joist construction defined by Clause 10.4; and
(e) beams cast integrally with slabs where the depth of the beam below the slab is not greater than one-half the width of web or 350 mm.

11.3.6.3 Simplified method

In lieu of more accurate calculations in accordance with Clause 11.3.6.4, and provided that the specified yield strength of the longitudinal steel reinforcement does not exceed 400 MPa and the specified concrete strength does not exceed 60 MPa, θ shall be taken as 35° and β shall be determined as follows:

(a) If the section contains at least the minimum transverse reinforcement as specified by Equation (11-1), β shall be taken as 0.18.
(b) If the section contains no transverse reinforcement and the specified nominal maximum size of coarse aggregate is not less than 20 mm, β shall be taken as

$$\beta = \frac{230}{(1000 + d_v)} \qquad \textbf{(11-9)}$$

(c) Alternatively, the value of β for sections containing no transverse reinforcement may be determined for all aggregate sizes by replacing the parameter d_v in Equation (11-9) by the equivalent crack spacing parameter, s_{ze}, where

$$s_{ze} = \frac{35 s_z}{15 + a_g} \qquad \textbf{(11-10)}$$

However, s_{ze} shall not be taken as less than $0.85 s_z$. The crack spacing parameter, s_z, shall be taken as d_v or as the maximum distance between layers of distributed longitudinal reinforcement, whichever is less. Each layer of such reinforcement shall have an area at least equal to $0.003 b_w s_z$ (see Figure 11.2).

When the simplified method specified in this Clause is used, all other clauses of Clause 11 shall apply, except Clause 11.3.6.4. Accordingly, this simplified method shall not be used for members subjected to significant tension, and the longitudinal reinforcement for all members shall be proportioned as specified in Clause 11.3.9.

APPENDIX VI

CRACK WIDTH CALCULATION

1. EXAMPLE OF RCC MODERATE DEEP

Calculate maximum crack width for RCC moderate deep beam for the given data.

Effective span	L	= 900 mm
Width of beam	b	= 75 mm
Overall depth of beam	D,h	= 275 mm
Effective depth of beam	d_{eff}	= 250 mm
Concrete clear cover	d_c	= 25 mm
Shear span	a	= 250 mm
Characteristics compressive strength	F_{ck}	= 25 N/mm^2
Modulus of elasticity of steel bars	E_s	= 200000 N/mm^2
Diameter of bars	Φ	= 10 mm
Number of bars	n	= 2
Maximum bar spacing	S	= 45 mm
Ultimate Load	P	= 17.8 ton

Calculation:

<u>Area of longitudinal reinforcement (A_{st})</u>

$A_{st} = n \times \frac{\pi}{4} \Phi^2$

$= 2 \times \frac{\pi}{4} 10^2$

$= 157$ mm^2

<u>Depth of neutral axis (X)</u>

Modulus of elasticity of concrete (Ec)

$= 5000 \sqrt[2]{F_{ck}} = 25000$ N/mm^2

Modular ration (m)

$= \frac{E_s}{E_c}$

$= \frac{200000}{25000} = 8$

Steel reinforcement ratio (ρ)

$= \frac{A_s}{bd}$

$= \frac{157}{75 \times 250} = 0.00837$

Depth of N.A

$\frac{X}{d} = m\rho\left(\sqrt{1 + \frac{2}{m\rho}} - 1\right)$

$= 8 \times 0.00837\left(\sqrt{1 + \frac{2}{8 \times 0.00837}} - 1\right) \times 250$

= 76.26 mm

Tensile stress in reinforcement (fs)

Maximum bending moment (M)

$M = \frac{p}{2} \times 10 \times \frac{a}{1000} \times 10^6$

$= \frac{17.8}{2} \times 10 \times \frac{250}{1000} \times 10^6$

= 22250000 N.mm

Lever arm (z)

$Z = d - \frac{X}{3}$

$= 250 - \frac{76.26}{3}$

= 224.574 mm

Steel stress (fs) $f_s = \frac{M}{A_s z}$

$= \frac{22.25 \times 10^6}{157 \times 224.574}$

= 631.06 N/mm^2

Effective area of concrete around steel bars (Ao)

$A_0 \quad = \frac{A_e}{n} = \frac{2d_c b}{n}$

$= \frac{2 \times 25 \times 75}{2}$

$= 1875\ mm^2$

<u>Factor β</u>

$\beta \quad = \frac{h-x}{d-x} = \frac{275-76.26}{250-76.26} = 1.14$

<u>Φ/ρ ratio</u>

$= \frac{10}{0.00837}$

$= 1194.74$

<u>Le/D ratio</u>

$= \frac{900}{275}$

$= 3.27$

<u>Maximum crack width</u>

$$W_{max} = \frac{0.03217\beta f_s \sqrt[3]{d_c A_o}\left(\frac{\Phi}{\rho}\right)^{\left(0.052*^{Le}/_{D}\right)}}{1+\left(\frac{L_e}{D}\right)^{-1.245}} \times 10^{-3}$$

$$= \frac{0.03217 \times 1.14 \times 631.06 \times \sqrt[3]{25 \times 1875}\left(\frac{10}{0.00837}\right)^{\left(0.052*^{900}/_{275}\right)}}{1+\left(\frac{900}{275}\right)^{-1.245}} \times 10^{-3}$$

$= 2.28$ mm

<u>Error</u> = ((theo – exp)/theo)×100

$= \frac{2.28-2.52}{2.28} \times 100 = 10.52\ \%$

1.1. EXAMPLE OF PFRC MODERATE DEEP BEAM

Calculate maximum crack width for PFRC moderate deep beam for the given data

Effective span	L	= 900 mm
Width of beam	b	= 75 mm

Overall depth of beam	D,h	= 275 mm
Effective depth of beam	d_{eff}	= 250 mm
Concrete clear cover	d_c	= 25 mm
Shear span	a	= 250 mm
Characteristics compressive strength	F_{ck}	= 25 N/mm^2
Modulus of elasticity of steel bars	E_s	= 200000 N/mm^2
Diameter of bars	Φ	= 10 mm
Number of bars	n	= 2
Maximum bar spacing	S	= 45 mm
Ultimate Load	P	= 18.1 tn
Fiber content	V_f	= 0.7%
Length of fiber	l_f	= 6 mm
Diameter of fiber	d_f	= 0.03 mm

Calculation:

Area of longitudinal reinforcement (A_{st})

$A_{st} = n \times \frac{\pi}{4}\Phi^2$

$= 2 \times \frac{\pi}{4}10^2$

$= 157$ mm^2

Depth of neutral axis (X)

Modulus of elasticity of concrete (Ec)

$= 5000 \sqrt[2]{F_{ck}} = 25000$ N/mm^2

Modular ration (m)

$= \frac{E_s}{E_c}$

$= \frac{200000}{25000} = 8$

Steel reinforcement ratio (ρ)

$= \frac{A_s}{bd}$

$= \frac{157}{75 \times 250} = 0.00837$

Depth of N.A

$\frac{X}{d} \quad = m\rho\left(\sqrt{1+\frac{2}{m\rho}}-1\right)$

$= 8 \times 0.00837\left(\sqrt{1+\frac{2}{8 \times 0.00837}}-1\right) \times 250$

= 76.26 mm

<u>Tensile stress in reinforcement (fs)</u>

Maximum bending moment (M)

$M \quad = \frac{p}{2} \times 10 \times \frac{a}{1000} \times 10^6$

$= \frac{18.1}{2} \times 10 \times \frac{250}{1000} \times 10^6$

= 22625000 N.mm

Lever arm (z)

$Z \quad = d - \frac{X}{3}$

$= 250 - \frac{76.26}{3} = 224.574$ mm

Steel stress (fs)

$f_s \quad = \frac{M}{A_s z} = \frac{22.625 \times 10^6}{157 \times 224.574}$

= 641.70 N/mm^2

Effective area of concrete around steel bars (Ao)

$A_0 \quad = \frac{A_e}{n} = \frac{2 d_c b}{n}$

$= \frac{2 \times 25 \times 75}{2}$

= 1875 mm^2

Factor β

β $= \frac{h-x}{d-x} = \frac{275-76.26}{250-76.26} = 1.14$

Φ/ρ ratio

$= \frac{10}{0.00837}$

= 1194.74

Le/D ratio $= \frac{900}{275}$ = 3.27

Maximum crack width

$$W_{max} = \frac{0.03217 \times 10^{-3} \beta f_s \sqrt[3]{d_c A_o} \left(\frac{\Phi}{\rho}\right)^{(0.052 * {Le}/{D})}}{1 + \left(\frac{L_e}{D}\right)^{-1.245}} - \frac{2}{3} \% V_f \sqrt[2]{{l_f}/{d_f}}$$

$$= \frac{0.03217 \times 10^{-3} \times 1.14 \times 641.70 \times \sqrt[3]{25 \times 1875} \left(\frac{10}{0.00837}\right)^{(0.052 * {900}/{275})}}{1 + \left(\frac{900}{275}\right)^{-1.245}} - \frac{2}{3} * 0.007 * \sqrt[2]{6/0.03}$$

= 2.25 mm

Error

= ((theo – exp)/theo)×100

$= \frac{2.25-2.49}{2.25} \times 100 = -10.75\ \%$

1.2. EXAMPLE OF SFRC MODERATE DEEP BEAM

Calculate maximum crack width for SFRC moderate deep beam for the given data

Effective span	L	= 900 mm
Width of beam	b	= 75 mm
Overall depth of beam	D,h	= 275 mm
Effective depth of beam	d_{eff}	= 250 mm
Concrete clear cover	d_c	= 25 mm
Shear span	a	= 250 mm

Characteristics compressive strength	F_{ck}	= 25 N/mm^2
Modulus of elasticity of steel bars	E_s	= 200000 N/mm^2
Diameter of bars	Φ	= 10 mm
Number of bars	n	= 2
Maximum bar spacing	S	= 45 mm
Ultimate Load	P	= 19.6 ton
Fiber content	V_f	= 0.7%
Length of fiber	l_f	= 30 mm
Diameter of fiber	d_f	= 0.6 mm

Calculation:

Area of longitudinal reinforcement (A_{st})

A_{st} $= n \times \frac{\pi}{4}\Phi^2$

$= 2 \times \frac{\pi}{4}10^2$

= 157 mm^2

Depth of neutral axis (X)

Modulus of elasticity of concrete (Ec)

$= 5000 \sqrt[2]{F_{ck}}$ = 25000 N/mm^2

Modular ration (m)

$= \frac{E_s}{E_c}$

$= \frac{200000}{25000} = 8$

Steel reinforcement ratio (ρ)

$= \frac{A_s}{bd}$

$= \frac{157}{75 \times 250} = 0.00837$

Depth of N.A

$\frac{X}{d} = m\rho\left(\sqrt{1+\frac{2}{m\rho}}-1\right)$

$$= 8 \times 0.00837\left(\sqrt{1+\frac{2}{8\times0.00837}}-1\right)\times 250$$

$= 76.26$ mm

Tensile stress in reinforcement (fs)

Maximum bending moment (M)

$$M = \frac{p}{2}\times 10 \times \frac{a}{1000}\times 10^6$$

$$= \frac{19.6}{2}\times 10 \times \frac{250}{1000}\times 10^6$$

$= 24500000$ N.mm

Lever arm (z)

$$Z = d - \frac{x}{3}$$

$$= 250 - \frac{76.26}{3}$$

$= 224.574$ mm

Steel stress (fs)

$$f_s = \frac{M}{A_s z} = \frac{24.5\times10^6}{157\times224.574} = 694.875 \text{ N/mm}^2$$

Effective area of concrete around steel bars (Ao)

$$A_0 = \frac{A_e}{n} = \frac{2d_c b}{n}$$

$$= \frac{2\times25\times75}{2}$$

$= 1875$ mm^2

Factor β

$$\beta = \frac{h-x}{d-x} = \frac{275-76.26}{250-76.26} = 1.14$$

Φ/ρ ratio

$$= \frac{10}{0.00837} = 1194.74$$

Le/D ratio $= \frac{900}{275} = 3.27$

Maximum crack width

$$W_{max} = \frac{0.03217 \times 10^{-3} \beta f_s \sqrt[3]{d_c A_o} \left(\frac{\Phi}{\rho}\right)^{(0.052 * Le/D)}}{1 + \left(\frac{L_e}{D}\right)^{-1.245}} - 3.616\% V_f \sqrt[2]{l_f / d_f}$$

$$= \frac{0.03217 \times 10^{-3} \times 1.14 \times 694.87 \times \sqrt[3]{25 \times 1875} \left(\frac{10}{0.00837}\right)^{(0.052 * 900/275)}}{1 + \left(\frac{900}{275}\right)^{-1.245}} - 3.616 * 0.007 * \sqrt[2]{30/0.6}$$

= 2.33 mm

Error

= ((theo – exp)/theo)×100

$= \frac{2.33-2.50}{2.33} \times 100 = -7.42\ \%$

1.3. DEPENDENCE OF CRACK WIDTH ON Φ/P RATIO:

The crack spacing in reinforced concrete structures (without fibers) can be calculated using the following expression presented in Eurocode 2:

$$S_{r\,max} = K_1 c + K_2 K_3 K_4 \frac{\phi}{\rho_{s,eff}}$$

Where:

C is the concrete cover

Φ is the diameter of bar

ρ_{seff} is the effective reinforcement ratio, $\rho_{s\,eff} = A_s / A_{c,eff}$ and $A_{c,eff}$ is the effective area ofconcrete in tension surrounding the reinforcement.

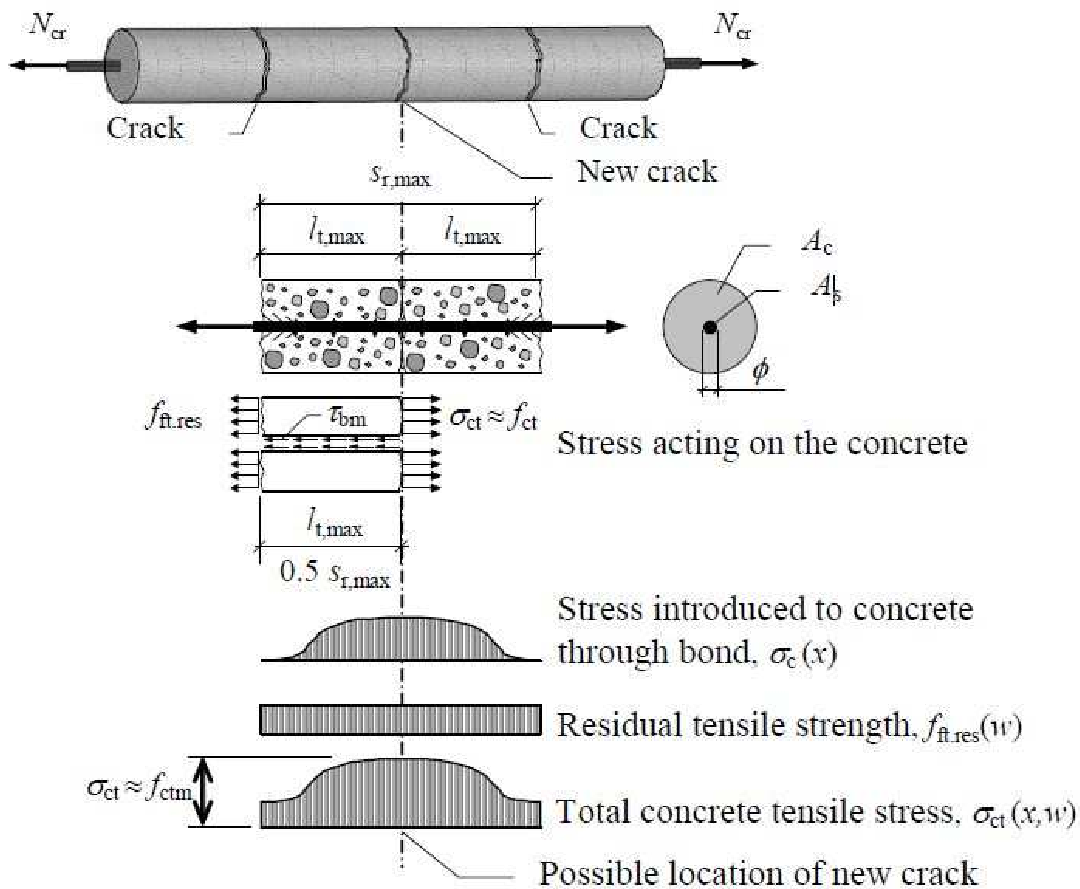
K_1 = 0.8 for high bond bars and 1.6 for bars with an effectively plain surface

K_2= 0.5 for bending, 1.0 for pure tension or for eccentric tension or $(\varepsilon_1 + \varepsilon_2 / 2\varepsilon_1)$ for eccentric tension

K_3 = 3.4

K_4 = 0.425

For a section with combined reinforcement a similar expression, which takes into account the contribution from the fiber reinforcement, can be derived. Consider a reinforced tension rod loaded with the crack load, N_{cr}, according to Figure 3. The rod is reinforced with a centrally placed reinforcement bar, with an area of A_s, and fibers. The force equilibrium in the region between two cracks with the maximum crack distance $S_{rmax} = 2 \cdot l_{t,max}$ is analyzed. At the crack the fiber reinforced concrete transfers a stress $f_{ft.res}$. At the midpoint between the two cracks the concrete is about to crack and the stress is thus $\sigma_{ct} \approx f_{ctm}$. The increase of stress is a result of stresses being transferred from the reinforcement to the concrete through bond.



The bond stress τ_b varies along the transmission length and has an average value of τ_{bm} which can be calculated as:

$$\tau_{bm} = \frac{\int_0^{l_{t,max}} \tau b(x) dx}{l_{t,max}}$$

If the tension rod is cut in the middle between the two cracks and along the interface betweenthe reinforcement and concrete the following equilibrium condition can be formulated:

$$\tau_{bm} \cdot \pi \cdot \phi \cdot (0.5 \cdot S_{rmax}) + f_{ft.res} A_c = f_{ctm} A_c$$

The concrete gross cross-sectional area can be formulated as:

$$A_c = A_s \frac{A_c}{A_s} = \frac{A_s}{\rho_s}$$

Thus,

$$\tau_{bm} \cdot \pi \cdot \phi \cdot (0.5 \cdot S_{rmax}) = \frac{\pi \phi^2}{4\rho_s} \left(f_{ct} - f_{ft.res}\right)$$

$$S_{r.min} = \frac{1}{2} . \frac{(f_{ctm} - f_{ft.res})}{\tau_{bm}} \frac{\phi}{\rho_s}$$

The minimum crack spacing is equal to half the maximum crack spacing. Accordingly, the minimum crack spacing can be calculated as:

$$S_{r.min} = \frac{1}{4} . \frac{(f_{ctm} - f_{ft.res})}{\tau_{bm}} \frac{\phi}{\rho_s}$$

Crack width is a function of crack spacing and spacing depends on $\frac{\phi}{\rho_s}$ ratio.