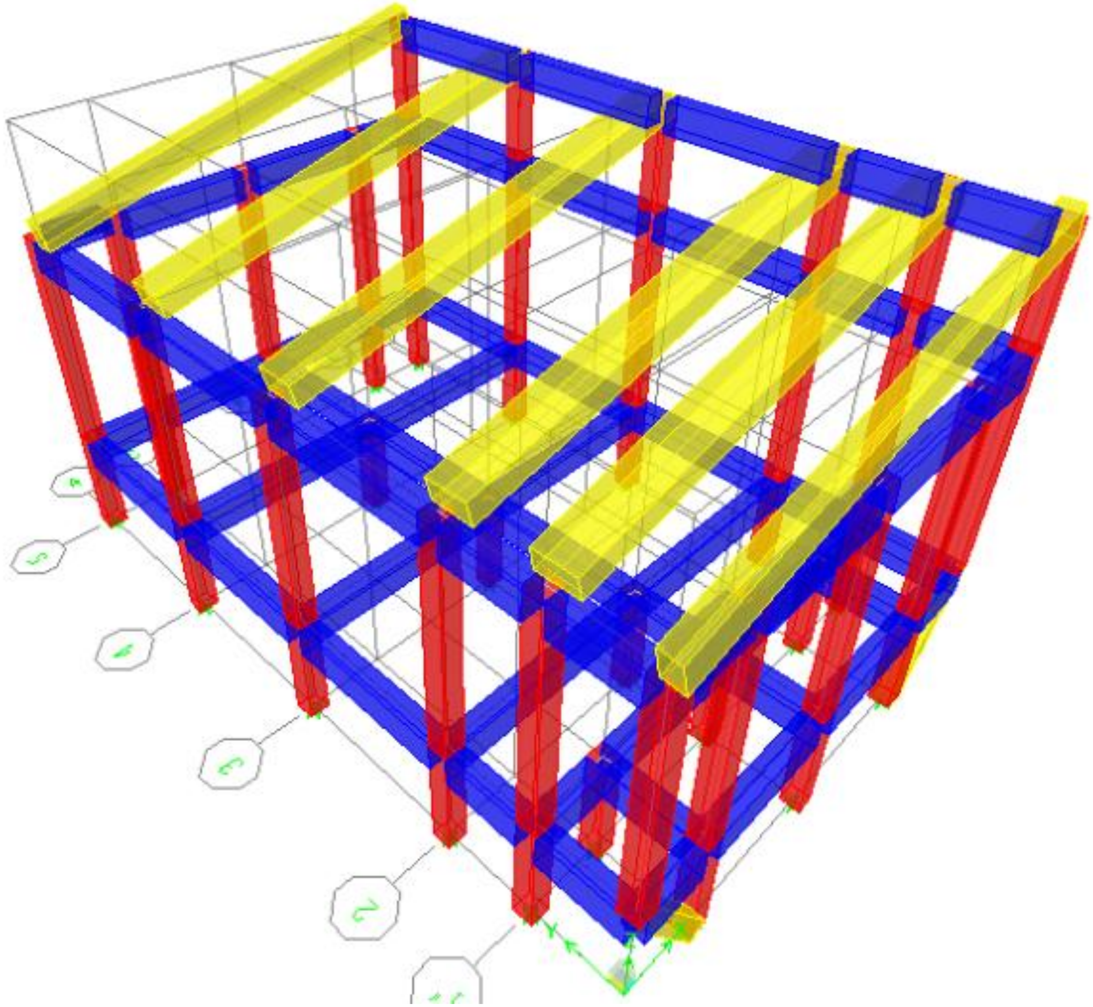
CHAPTER 5 DESIGN FOR EARTHQUAKE RESISTANCE(FRAME EXAMPLE)

1.0 STRUCTURAL SYSTEM



2.0 GEOMETRICAL CONSTRAINTS FOR MEDIUM DUCTILITY CLASS

2.1.2 Grade Beams (b/h = 250/400 mm)

2.1.2.1 Geometrical constraints for DCM

The width b_w of a primary seismic beam shall satisfy the expression:

$b_w \leq \min\{b_c + h_w \,;\, 2b_c\}$ (EN 1998-1-1, 5.4.1.2.1)[1]

$\rightarrow b_w \leq \min\{250 + 400 \,;\, 2 \times 250\} = \min\{650 \,;\, 500\}$

$\rightarrow$ **b_w = 250 mm ≤500 mm$\rightarrow$ (ok)**

2.0 GEOMETRICAL CONSTRAINTS FOR MEDIUM DUCTILITY CLASS

2.4 Columns

2.4.1 Geometrical constraints

Unless $\theta \leq 0.1$, the cross-sectional dimensions of primary seismic columns should not be smaller than one tenth of the larger distance between the

2.0 GEOMETRICAL CONSTRAINTS FOR MEDIUM DUCTILITY CLASS

point of contraflexure and the ends of the column, for bending within a plane parallel to the column dimension considered.

$\theta \leq 0.1 \rightarrow$ ok

2.4.2 Resistances

In primary seismic columns the value of the normalized axial force ν_d shall not exceed 0.65.

$$\nu_d = N_{Ed}/(f_{cd}bh) = (\cancel{198.3}\times10^3/(11.33\times250^2)) = \cancel{0.28} < 0.65 \rightarrow \text{ok}$$

105.0 0.148

2.0 GEOMETRICAL CONSTRAINTS FOR MEDIUM DUCTILITY CLASS

2.5 Beam-column joints

(a) Provide hoops in joints of primary seismic beams with columns, s = 96 mm

(b) Provide at least one intermediate (between column corner bars) vertical bars. Provided are 8ϕ16 and 8ϕ20 with 4 intermediate bars$\rightarrow$ ok

3.1 DETAILING OF COLUMNS FOR LOCAL DUCTILITY (CODE EXTRACT)

5.4.3.2.2 Detailing of primary seismic columns for local ductility

(1)P The total longitudinal reinforcement ratio ρ_l shall be not less than 0,01 and not more than 0,04. In symmetrical cross-sections symmetrical reinforcement should be provided ($\rho = \rho'$).

(2)P At least one intermediate bar shall be provided between corner bars along each column side, to ensure the integrity of the beam-column joints.

(3)P The regions up to a distance l_{cr} from both end sections of a primary seismic column shall be considered as being critical regions.

(4) In the absence of more precise information, the length of the critical region l_{cr} (in metres) may be computed from the following expression:

$$l_{cr} = \max\{h_c;\ l_{cl}/6;\ 0,45\} \qquad (5.14)$$

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3.1 DETAILING OF COLUMNS FOR LOCAL DUCTILITY

(a) $0.01 < \rho < 0.04 \rightarrow \rho = (8 \times 314)/250^2 = 0.04 \rightarrow$ ok

(b) At least one intermediate bar shall be provided $\rightarrow$ ok

(c) $l_{cr} = \max\{h_c;\ l_{cl}/6;\ 0.45) \rightarrow l_{cr} = \max\{250;\ 2650/6 = 441.7;\ 0.45\} \rightarrow l_{cr} = 0.45$ m

(d) In the critical region at the base of primary seismic columns a value the curvature ductility factor μ_ϕ , should be provided at least equal to that given in Clause 5.2.3.4(3)[1]. $\rightarrow \mu_\phi = 1 + 2(q_o - 1)T_c/T_1 = 1 + 2\times(3.3 - 1)\times 0.5/0.287 =$ 9.01.

If for the specified value of μ_ϕ a concrete strain larger than $\varepsilon_{cu2} = 0.0035$ is needed anywhere in the cross-section, compensation for the loss of resistance due to spalling of the concrete shall be achieved by means of adequate confinement of the concrete core.

3.1 DETAILING OF COLUMNS FOR LOCAL DUCTILITY

The requirements in (d) are deemed to be satisfied if:

$\alpha\omega_{wd} \geq 30\mu_{\varphi}\nu_d \times \varepsilon_{sy,d} \times (b_c/b_o)$ - 0.$\cancel{0}$035, where:

ω_{wd} is the $\boxed{\text{mechanical volumetric ratio}}$ of confining hoops within the critical regions equal to:

$$\omega_{wd} = \frac{\text{volume of confining hoops}}{\text{volume of concrete core}} \cdot \frac{f_{yd}}{f_{cd}} = \frac{46623}{192^2 \times 96} \times \frac{347.8}{11.33} = 0.4044$$

α is the confinement effectiveness factor, equal to $\alpha = \alpha_n \times \alpha_s$:

$\alpha_n = 1 - \Sigma b_i^2 / 6b_o h_o = 1-(192^2+192^2+192^2+192^2)/(6\times192\times192) = 0.333$

$\alpha_s = (1 - s/2b_o)(1-s/2h_o) = (1-96/(2\times192))(1-96/(2\times192)) = 0.5625$

$\rightarrow \alpha = 0.333 \times 0.5625 = 0.1875$

105.0 0.148

$\nu_d = 149.2\times10^3/(11.33\times250^2) = 0.\cancel{2}1$ (calculated with the maximum column force for the seismic design situation)

0.148 0.192

$\rightarrow 0.1875 \times 0.4044 \geq 30\times9.01\times0.\cancel{2}1\times0.001739\times(250/2\cancel{5}0) - 0.035$

$\rightarrow 0.076 \geq 0.\cancel{0}64 \rightarrow$ **The columns possess adequate ductility**

0.056

3.1 DETAILING OF COLUMNS FOR LOCAL DUCTILITY

(e) min ω_{wd} = 0.08 should be provided within the critical region at the base of the primary seismic columns → ok $\omega_{wd} = 0.4044$

(f) Within the critical regions of the primary seismic columns, hoops and cross-ties, of at least 6 mm in diameter, shall be provided at a spacing such that a minimum ductility is ensured and local buckling of longitudinal bars is prevented.

The minimum conditions in (f) are deemed to be satisfied if:

(i) The spacing, s, of hoops (in mm) does not exceed:
$s = \min\{b_o/2;\ 175;\ 8d_{bl}\} = \min\{192/2;\ 175;\ 8\times20\} \rightarrow s = 96$ mm

(ii) $s_{max} = 200$ mm

Summary: →Provide 8 ϕ16 in storey 1→$A_{s,avail} = 1608$ mm^2

→Provide 8 ϕ20 in storey 2→$A_{s,avail} = 2512$ mm^2

3.1 DETAILING OF COLUMNS FOR LOCAL DUCTILITY

Provide ϕ 8 ties c/c 96 mmin the critical zone with 135° hooks (l =80 mm) anchored into the column core.

Provide ϕ 8 ties c/c 120 mm in the middle with 135° hooks (l =80 mm) anchored into the column core

3.2 BEAM DESIGN FROM ANALYSIS RESULT FOR SEISMIC DESIGN SITUATION

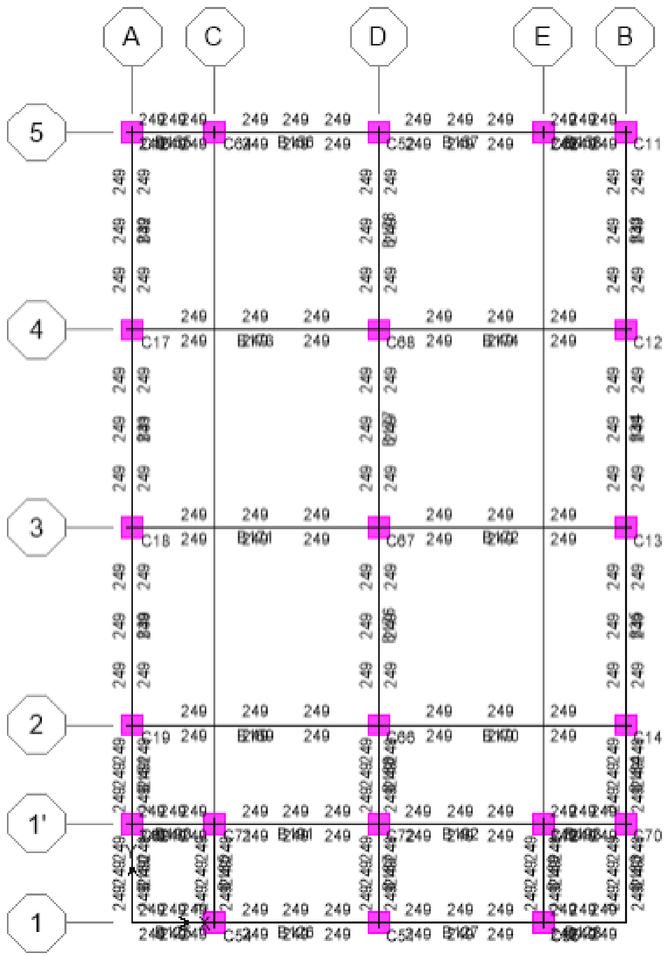


Fig. 3.4.1 (a) Ground floor beams reinforcement

3.2 BEAM DESIGN FROM ANALYSIS RESULT FOR SEISMIC DESIGN SITUATION

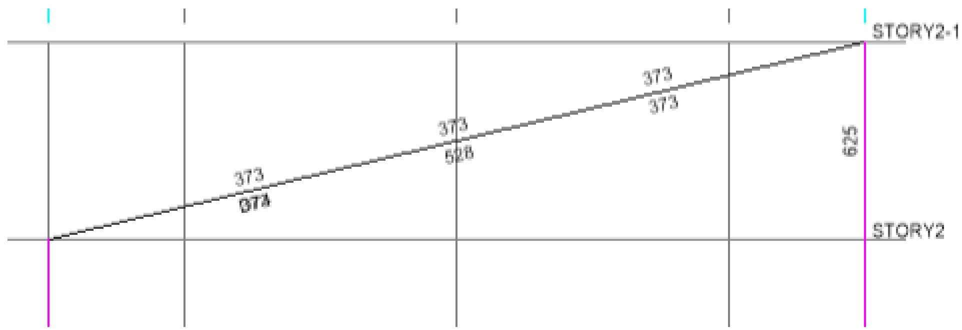


Fig 3.4.1 (d) Inclined beam reinforcement – Axis 1’

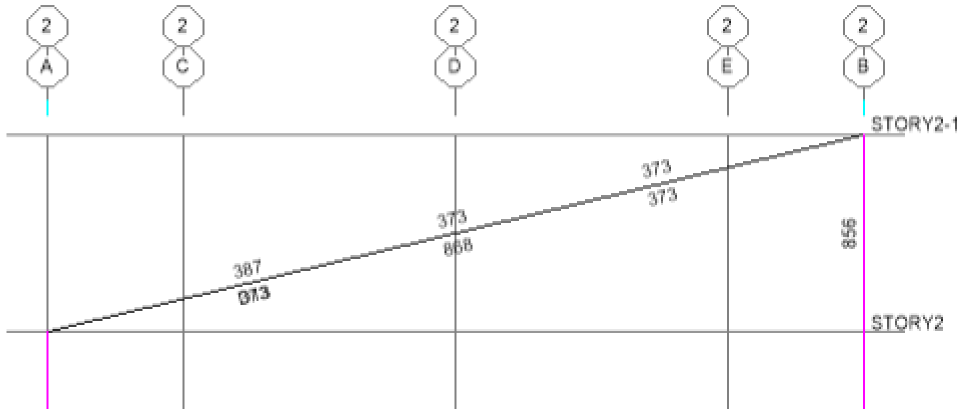


Fig 3.4.1 (e) Inclined beam reinforcement – Axis 2

3.3 DETAILING OF BEAMS FOR LOCAL DUCTILITY

3.4.2 Detailing of beams for local ductility

- **Grade beams (b_w/h_w = 250/ 400 mm) (Level ± 0.00)**

(1) Reinforcement at the compression zone is 2ϕ16 ≥half of the reinforcement (2ϕ16) provided at the tension zone.→**ok**

(2) The reinforcement ratio of the tension zone ρ does not exceed a value ρ_{max} equal to:

$$\rho_{max} = \rho' + \frac{0.0018}{\mu_\varphi \varepsilon_{sy,d}} \cdot \frac{f_{cd}}{f_{yd}}$$

where μ_φ is the curvature ductility factor approximated by

3.3 DETAILING OF BEAMS FOR LOCAL DUCTILITY

$\mu_\varphi = 1 + 2(q_o - 1)T_C / T_1$ if $T_1 < T_C$

$T_C = 0.5$ s

$T_1 = 0.287$ s (fundamental period of vibration)

$q_o = 3.3$ for DCM for frame systems (see section 2.6.2.2)

$\rightarrow \mu_\varphi = 1 + 2(3.3 - 1)0.5/\ 0.287 = 9.01$

$\rho' = 0.0046$

$$\rightarrow \rho_{\max} = 0.0046 + \frac{0.0018}{\mu_\varphi \varepsilon_{sy,d}} \cdot \frac{f_{cd}}{f_{yd}}$$

$$\rightarrow \rho_{\max} = 0.0046 + \frac{0.0018}{9.01 \times 0.001739} \times \frac{11.33}{347.8} = 0.0083$$

$\rightarrow \rho_{\max} = 0.0083 \geq \rho_{avail} = 0.0046 \rightarrow$ **ok**

3.3 DETAILING OF BEAMS FOR LOCAL DUCTILITY

(3) The reinforcement ratio of the tension zone ρ shall not be less than the minimum value ρ_{min} equal to:

$$\rho_{\min} = 0.5\left(\frac{f_{ctm}}{f_{yk}}\right)$$

$\rho_{min} = 0.5(2.2/400) = 0.00275 \leq \rho_{avail} = 0.0046 \rightarrow$ **ok**

(4) Spacing of hoops within the critical regions:

$$s = \min\{h_w/4;\ 24d_{bw};\ 225;\ 8d_{bL}\}$$

$$= \min\{400/4;\ 24\times8;\ 225;\ 8\times16\}$$

$\rightarrow s = 100$ mm

3.3 DETAILING OF BEAMS FOR LOCAL DUCTILITY

3.4.3 Anchorage of Reinforcement

Use closed stirrups with 135° hooks and extensions of length $10d_w$ (= 10×8 = 80 mm) for hoops as transverse reinforcement in beams.

To prevent bond failure the diameter of the beam longitudinal bars passing through the beam-column joints, d_{bl}, shall be limited in accordance with the following expressions:

3.4 SHEAR DESIGN, GLOBAL DUCTILITY VIA STRONG COLUMNS

3.4.4 Shear Design

In primary seismic beams the design shear forces shall be determined in accordance with the **capacity design rule**, on the basis of the equilibrium of the beam under:

3.4 SHEAR DESIGN, GLOBAL DUCTILITY VIA STRONG COLUMNS

(a) The transverse load acting on it in the seismic design situation and

(b) End moments $M_{i,d}$ (with i = 1,2 denoting the end sections of the beam), corresponding to plastic hinge formation for positive and negative directions of seismic loading.

$$M_{i,d} = \gamma_{Rd} M_{Rb,i} \cdot \min\left(1, \frac{\sum M_{Rc}}{\sum M_{Rb}}\right)$$

$\gamma_{R.d}$ is the overstrength factor due to stain hardening which is equal to 1.0 for ductility class of DCM.

3.4 SHEAR DESIGN, GLOBAL DUCTILITY VIA STRONG COLUMNS

$M_{Rb,i}$ is the design value of the beam moment of resistance at end i in the sense of the seismic bending moment under the considered sense of the seismic action (i.e. +ve moment on the left end and –ve moment on the right end for frame swaying to the right and so on)

ΣM_{Rc} and ΣM_{Rb} are the sum of the design values of the moments of resistance of the columns and the sum of the design values of the moments of resistance of the beams framing into the joint, respectively.

→**Determine $M_{Rb,i}$, ΣM_{Rc} and ΣM_{Rb}**

3.4 SHEAR DESIGN, GLOBAL DUCTILITY VIA STRONG COLUMNS

→**Determine $M_{Rb,i}$, ΣM_{Rc} and ΣM_{Rb}**

Moment resistances of beams, $M_{Rb,i}$

Beam types	Section (b_w/h_w)	rebars	$M_{Rb,i}$ (kNm)
1	250/400	2ϕ16 top and bottom	-46.73 (+46.73)
2	250/400	2ϕ16 top and bottom (span 3ϕ16 (2ϕ16))	-46.73 (+46.73)
3 (joist)	500/300	4ϕ16 top and 3ϕ16 bottomon Axes 3 and	-65.7 (+51.3)

3.4 SHEAR DESIGN, GLOBAL DUCTILITY VIA STRONG COLUMNS

		4 (span 6ϕ16)[1)]	
4 (joist)	500/300	3ϕ16 top and bottom on Axes 1' and 2 (span 5ϕ16)[1)]	-51.3 (+51.3)

Note:

[1)]Out of the 6ϕ16 (5ϕ16)only 3ϕ16 is developed to give the required +ve moment resistance at the beam-column joint and similarly for the other joist beams.

3.4 SHEAR DESIGN, GLOBAL DUCTILITY VIA STRONG COLUMNS

Moment resistances of columns, M_{Rc}

According to EN 1998-1: 2004[1], the following condition should be satisfied at all joints of primary seismic beams with primary seismic columns to satisfy capacity design provisions in order to obtain hierarchy of resistance of the various structural components necessary for ensuring the intended configuration of plastic hinges and for avoiding brittle failure modes.

$$\Sigma M_{Rc} \geq 1.3 \Sigma M_{Rb}$$

3.4 SHEAR DESIGN, GLOBAL DUCTILITY VIA STRONG COLUMNS

→ΣM_{Rc} = 1.3 × 65.7 = 85.4 kNm (Roof slab beam-column joint on **Axes 3 and 4**). The value of ΣM_{Rc} should correspond to the column axial forces in the seismic design situation for the considered sense of the seismic action.

→ Design column A/3 for $M_{Ed,z}$ = 85.4 kNm, axial loads and moment about the y-axis determined from analysis for the governing seismic design situation.

Max/min N_{Ed} = -81.9 kN/ ~~-114.6 kN~~

-105.0 kN

3.4 SHEAR DESIGN, GLOBAL DUCTILITY VIA STRONG COLUMNS

$M_{Ed,y}$ = 4.4 KNm

Section design resistance using 8ϕ20 is (MASQUE)[4]:

N_{Rd}/ $M_{Rd,y}$/ $M_{Rd,z}$ = -81.7 kN/ 6.5 kNm/ 70.9 kNm

→ Strong column - weak beam conditions Is not be met for two columns at roof level

Increase reinforcement in columns. Exercise as an assignment. Assume uniaxial bending.

3.4 SHEAR DESIGN, GLOBAL DUCTILITY VIA STRONG COLUMNS

Other columns:

$\rightarrow \Sigma M_{Rc} = 1.3 \times 51.3 = 66.7$ kNm

$\rightarrow$ Design column A/2 for $M_{Ed,z} = 66.7$ kNm, axial loads and moment about the y-axis determined from analysis for the governing seismic design situation.

Max/min $N_{Ed} = -71.6$ kN

$M_{Ed,y} = 6.3$ KNm

Governing section design resistance using 8ϕ20 is (MASQUE)[4]:

$N_{Rd}/ M_{Rd,y}/ M_{Rd,z} = -68.8$ kN/ 10.2 kNm/ 68.8 kNm

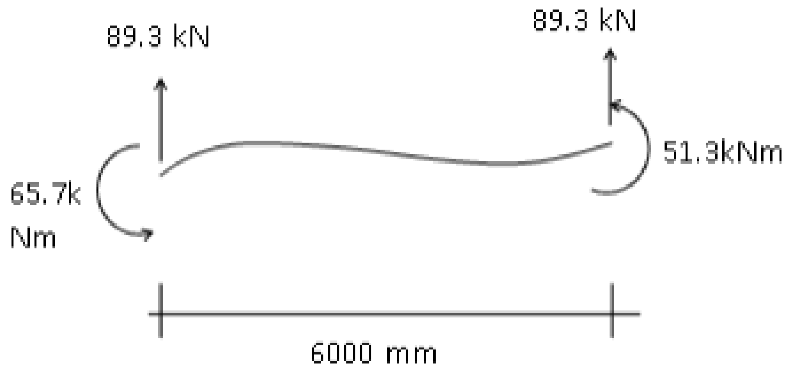
3.4 SHEAR DESIGN, GLOBAL DUCTILITY VIA STRONG COLUMNS

Shear design based on capacity design rule:

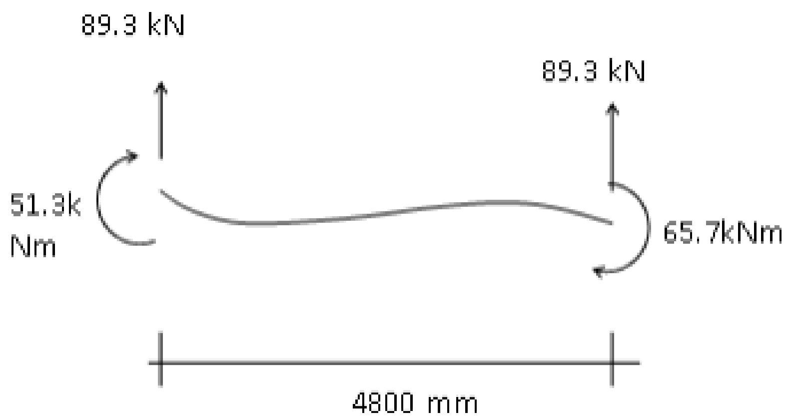
(i) Frame swaying to the left (frame on Axis 3):

Factored shear force from gravity loading $V_{Ed,L} = V_{ED,R} = 89.3$ kN

3.4 SHEAR DESIGN, GLOBAL DUCTILITY VIA STRONG COLUMNS



(ii) Frame swaying to the right:



3.4 Shear Design, Global Ductility via Strong Columns

Governing Design Shear force:

$V_{Ed} = (51.3 + 65.7)/\cancel{2} + 89.3 = 58.5 + 88.2 = \cancel{147.3}\text{ kN}$

4.8

113.7 kN

Design for shear:

The smallest possible inclination angle of the struts is:

With $z = 0.9 \times 259 = 233.1$ mm

$V_{Rd,c} = \beta_{ct} \times 0.1 \times \eta_1 \times f_{ck}^{1/3} \times (1 + 1.2 \times \sigma_{cd}/f_{cd}) \times b_w \times z =$

$2.4 \times 0.1 \times 1.0 \times 20^{1/3} \times 500 \times 233.1 = 75.9$ kN

$$0.58 \le \cot\theta \le \frac{1.2 - 1.4\dfrac{\sigma_{cd}}{f_{cd}}}{1 - \dfrac{V_{Rd,c}}{V_{Ed}}} = \frac{1.2}{1 - \dfrac{75.9}{147.3}} = 2.476 \le 3.0$$

$\rightarrow \theta = 22.0^{\circ}$

3.4 SHEAR DESIGN, GLOBAL DUCTILITY VIA STRONG COLUMNS

Check the struts:

$\alpha_c = 0.75\times \eta_1 = 0.75\times 1.0 = 0.75$

$f_{cd} = 11.33$ MPa

$$V_{Rd,\max} = \frac{b_w \times z \times \alpha_c \times f_{cd}}{\cot\theta + \tan\theta} = \frac{500\times 233.1\times 0.75\times 11.33}{\cot 22 + \tan 22} = 343.9 \text{ kN}$$

V_{Ed} = ~~147.3~~ kN< 343.9 kN→**ok** 113.7 kN

Required amount of stirrups:

With $\alpha = 90^o$ 113.7 kN

$$a_{sw} = \frac{147.3\times 10^3}{\cot 22\times 347.8\times 233.1} = 0.734 \text{ mm}^2/\text{mm} = 734 \text{ mm}^2/\text{m}$$

Available stirrups to satisfy local ductility requirement → three legged ϕ8 c/c 75

→$a_{sw,available}$ = 2000 mm^2/ m →**ok**