Design of Transverse:

$$d \leq \frac{1}{4} \text{ dia of main bar 6 mm}$$

Spacing:

$$\geq \frac{1}{2} \times \text{Least lateral dim}$$

Design of Shear Wall

General requirements:

Thickness of any past of the wall ≤ 150 mm

Effective extension of the flange width beyond the face of the web – lesser of the following

(a) 1/2 distance to as adjusted shear wall web

(b) 1/10 total wall height

(c) Actual width

Where the extreme flash compressive stresses in the wall due to all loads. (gravity + lateral forces)

Then the boundary elements are provided above the vertical boundaries of the wall.

Boundary elements are discontinued when the compressive stresses are less than $0.15 f_{cy}$.

Also not required the entire wall section is provided with special confining steel reinforcement.

Reinforcement

$$(A_{st})_{\min} = 0.0025 \text{ is both vest and horizontal direction}$$

$$\frac{A_s}{A_c(gns)} = P \leq 0.0025$$

In the factored shear stress $\tau(V)$

$$V > 0.25\sqrt{f_{ck}} \quad \text{The bars should be provided}$$

$$t_w > 200 \text{ mm}$$

as two mats in the plane of the wall, one on each face.

(a) $db \geq 1/10\, t_w$.

(b) $(\text{spacing})_{\max} \geq L/5,\ 3t_w,\ 450$ mm — less

(c) $(A_{st})_{\text{vest}} \geq$ horizontal steel

Shear strength requirements

Nominal shear stress $\left[\tau_V = \dfrac{V_u}{bd}\right]\left[\tau_V = \dfrac{V_u}{t_w d_w}\right]$

V_u – factored S.F.

t_w – thickness of web

d_w – effective depth of wall section

d_w – 0.8 lw — bar section.

Design Shear Strength of Concrete

$$[\tau_c]_{\max} = 0.63\sqrt{f_{ck}} \quad \text{— Table 20 IS456}$$

Shear strength of concrete τ_c

Live beam shear. Table assume 0.25% steel.

If necessary, it can be increased by the following multiplying factor. (IS456 : Clause 40.2.2)

$$\delta = \left[1 + \frac{3P_u}{A_c . P_{ck}}\right] \geq 1.5.$$

Pu – axial load; δ – multiplying factor.

∴ Shear capacity of con:

$$V_{uc} = t_c t_w d_w$$

∴ Shear capacity of steel

$$V_{us} = V_u - V_{uc}$$

$$[V_{us} = V_u - t_c t_w d_w]$$

The steel required to resist the shear

$$V_S = \frac{0.87 . f_y . A_{bd}}{S_V}$$

A_h – horizontal start

The amount of horizontal steel provided shall not be less than (≤).

$$[A_{st}]_V \leq [A_{st}]_k$$

$$\therefore \quad \left[\frac{V_{us}}{d_w} = \frac{0.87 f_y A_s}{S_y}\right]$$

Table 62 sp16 can be used for determing the dia of steel and spacing.

Adequacy of Boundary Elements

Max. axial load due to gravity load and moment

$$P = \text{Sum of factored gravity loads} + \frac{M_u - M_{uv}}{C_w}$$

M_u – Facfored deisgn moment on entire wall seet

M_{uv} – MR. by distributed vertical steel across the wall section

C_w – c/c distance between boundary elements

Load factor – 0.8 – if graivty load adds to the strength of the wall.

Per of steel in boundary element = 0.8 to 6%

upper limit – 4%

Flexural Strength:

Moment of Resistance of Rectangular Shear Wall Section

The flexural strength of a sknder rectangular shear wall section with u.d. vertical reinforcement subjected the unaxial bending and axial load may be situated.

Two formance depending on section fails influxural tension (or) flexural component are given

P_u – axial component on wall assumed as to act @ centre of wall

A_V – Area of vertical reinforcmement

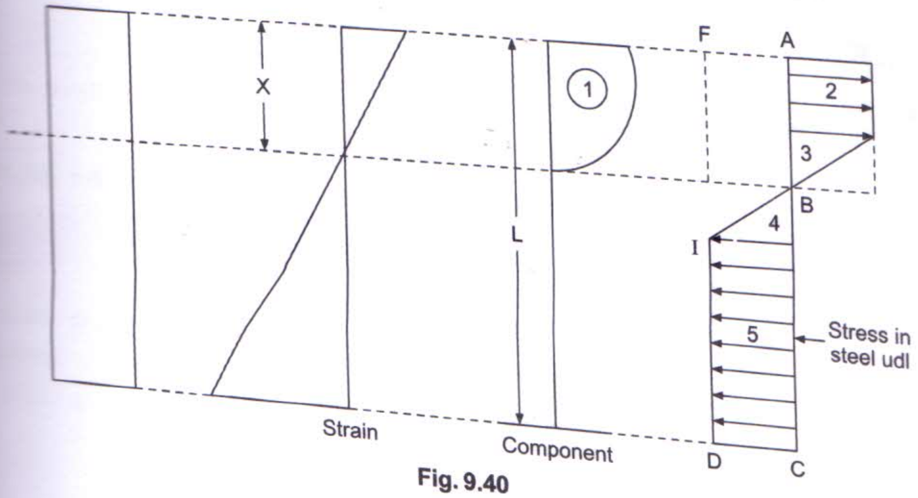

Fig. 9.40

ρ – Vertical steel ratio $\left[\frac{A_V}{t_w t_w}\right]$

$\lambda - \frac{P}{f_{cu}.t_w t_w}$

$\phi - \frac{0.87 f_y.\rho}{f_{ck}} = \frac{\rho f_y}{f_{cu}}$

M_u = M.R. of the wall which can be obtained from the diagram

For balanced failure

$$\frac{\bar{X}}{L} = \left[\frac{0.0035}{0.0035 + 0.87\frac{f_y}{ES}}\right]$$

Magnitude of the forces Acting on the Section and Distance From Tension Fibre

S.No	Force	Distance	Port
A.	Compn in cons $(0.36 f_{ck}.x.t_w)$	$L - 0.41x$	1
B.	compn steel (f_s)		
	(i) $\rho\, x\, t\, (1-\beta)t$	$L - \frac{1}{2}x(1-\beta)$	2
	(ii) $\frac{1}{2} f_3.\rho_x \beta_t$	$L - x + \frac{2}{3}$	3
C.	Tension steel		
	(i) $\frac{1}{2} f_s \rho_x \beta_t$	$L - x - \beta_x$	4
	(ii) $f_s \rho\, (L - x - p_n)t$	$\frac{1}{2}[L - x - \beta_x]$	5
D.	Extensial loads wall P_w	$\frac{1}{2}L$	

The cond to be satisfied

$$P_w = C - T;\ M_u = \text{forces about mid point}$$

$\therefore$ $\dfrac{X}{L}$ can be determined

$$P_w = 1 + 2 + 3 - 4 - 5 = 1 + 2 - 5$$

$$P_w = 1 - ACDF + 2ABEI = 0.36 f_{cu}.x.t. - f_s.Lt\,\rho + 2\rho f_s \ldots$$

$$\left[\frac{P_w}{f_{cu}.t.L} = \lambda\right] = 0.36.\frac{x}{L} - \rho\frac{f_s}{f_{cu}} + 2\rho\frac{f_s}{f_{cv}}\frac{x}{L}$$

$$\phi = \rho - \frac{f_s}{f_{cu}} = \frac{\rho 0.87 f_y}{f_{cu}}$$

$$(\lambda + \phi) = \frac{x}{L}[0.36 + 2\phi]$$

$$\left[\frac{x}{L}\right] = \left[\frac{\lambda + \phi}{0.36 + 2\phi}\right]$$

Case-1: If $\left[\dfrac{X_y}{L} < \dfrac{\overline{X}_u}{L}\right]$, $\overline{X}_u$ – balanced depth of N.A.

Tension failure occurs and the value of the M.R. flexural strength) is obtained by taking m... of for about the mid point

$$\left[\frac{M_u}{f_{cu}\,tL^2}\right] = \phi\left[1 + \frac{\lambda}{\phi}\right]\left[\frac{1}{2} - 0.42\frac{x_n}{L_w}\right] - \left(\frac{x_y}{L}\right)^2\left[0.168 + \frac{\beta^2}{3}\right]$$

$$\beta = \frac{0.87 f_y}{0.0035 E_S} = 0.516 \text{ Fe415}$$

Case-2: $\left[\dfrac{X}{L} < 0.5\right]$

$$\frac{M_u}{f_{ck}.tL^2} = \phi\left[1 + \frac{\lambda}{\phi}\right]\left[\frac{1}{2} - 0.42\frac{x_1}{L}\right]$$

$$\left[\frac{X}{L} > \frac{\overline{x}}{L} > 1.0\right]$$

we have compn failure assuming N.A. with section i.e., $\left[\dfrac{\overline{x}}{L} < 1.0\right]$

$$\therefore \quad \frac{M_u}{f_{cu}.t.L} = \alpha_1\frac{x}{L} - \alpha_3\frac{x}{L} - \alpha_3 - \frac{\lambda}{2}$$

$$\alpha_1 = \left[0.36 + \phi\left(1 - \frac{\beta}{2} - \frac{1}{2p}\right)\right]$$

$$\alpha_2 = \left[0.15 + \frac{\phi}{2}\left(1 - \beta - \frac{\beta^2}{2} - \frac{1}{3p}\right)\right]$$

$$\alpha_3 = \frac{\phi}{6\beta}\left(\frac{1}{\left(\frac{x_u}{l}\right)} - 3\right)$$

$$\alpha_1\left(\frac{x_y}{l_w}\right)^3 + \alpha_4\left(\frac{x_y}{l_w}\right) - \alpha_5 = 0$$

$$\alpha_4 = \left(\frac{\phi}{\beta} - \lambda\right)\alpha_5 = \left(\frac{\phi}{2p}\right)$$

Design of shear wall

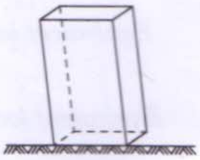

Fig. 9.41

on each shear wall – 5700 kN ($DL + u$)

Floor height – 3.0 m

$DL = 4$ kN/m^2

wt of partition = 2 kN/m^2

$LL = 3.0$ kN/m^2, Proof = 1.5

foundation – head soil Delhi

Determine – S.F. and shears at different floor levels

shear wall to resist seismic forces c/s of beams 300 × 600 mm

wt. of Building

code provision to be consider for the calculation of E.Q. is 25% for the floors and no live considered for the roof.

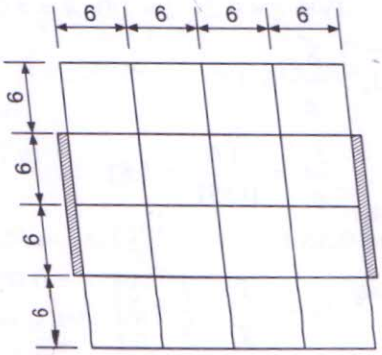

Fig. 9.42

Area of plan = 24 × 24 = 576 m^2

due to DL on each floor = 4 × 576 = 2304 kN

Partitions = 2 × 576 = 1152 kN

Total *wt* due DL = 3456 kN

Effective *wt* of due to LL = 0.25 × 3 × 576 = 432 kN

wt. of beams = [0.3 × 0.6 × 25] × 10 × 24 m = 1080 kN

wt. of column = [0.3 × 0.6 × 25 × 2.4] × 25 = 270 kN

Calculation of S.W

Dead load contribution

$$\text{Area of plan} = 24 \times 24 = 576 \text{ m}^2$$

$$\text{Dead load} = 4 \times 576 = 2304 \text{ kN}$$

$$\text{Position} = 2 \times 576 = 1152 \text{ kN}$$

$$\text{SW of beam} = (0.3 \times 0.6 \times 25) \times 24 \times 10 = 1080 \text{ kN}$$

$$\text{SW of column} = (0.3 \times 0.6 \times 2.4) \times 25 \times 25 = 270 \text{ kN}$$

$$\text{wt of column @ roof} = 270/2 = 135 \text{ kN}$$

Equivalent load at roof level

$$2304 + 1080 + 135 = 3519 \text{ kN}$$

Equivalent load @ Floor level

$$2304 + 1152 + 432 + 1080 + 270 = 5238 \text{ kN}$$

Seismic *wt* of the building

$$WS = 5238 \times 9 + 3519 = 50661 \text{ kN}$$

Base shear

The fundamental natural period (T)

For building having shear wall

$$T = \frac{0.09h}{\sqrt{d}} = \frac{0.09}{\sqrt{24}} \times 30 = 0.55\% \text{ see}$$

Seismic parameter for IS1893-2002

$$z\text{–IV}\%\ z = 0.2u;\ I = 1.0;\ R = 4.0 \text{ (Ductile)}$$

For 5% damping; type-I soil 1, $\dfrac{\delta_y}{g} = 1.8$

$$\frac{S_a}{g} = \frac{1.0}{0.551} = 1.81$$

Design of horizontal seismic coefficient

$$A_h = \left[\frac{S_a}{g} \times \frac{I}{R} \times \frac{z}{2}\right]$$

$$= \left(1.81 \times \frac{1}{4} \times \frac{0.24}{2}\right) = 0.0543$$

Base shear

$$V_B = A_h V = 0.0543 \times 50661$$

$$= 2750.8 \text{ kN}$$

Lateral load : and shear forces

Design lateral force $i = Q_i = V_B \times \dfrac{w_i h_i^2}{\Sigma w_i h_i^2}$