JETIR.ORG ISSN: 2349-5162 | ESTD Year : 2014 | Monthly Issue
JOURNAL OF EMERGING TECHNOLOGIES AND INNOVATIVE RESEARCH (JETIR)
An International Scholarly Open Access, Peer-reviewed, Refereed Journal

Structural Design of Reinforced Concrete Shear wall

Gaurav Kandwal
Himgiri Zee University Dehradun.
Prof Pankaj Methani
HOD Civil Himgiri Zee University.

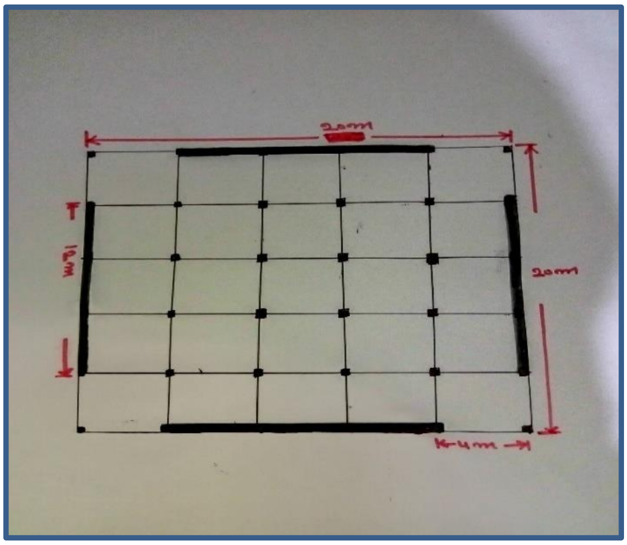
Abstract -Shear walls are uniquely designed structural walls constructed into buildings to resist lateral forces that are created by the plane of the wall due to wind, earthquakes and other forces. The term "shear wall" is somewhat misleading because such walls deal with similar flexural members. They are typically used in large high-rise buildings and have unlimited use in preventing buildings from completely collapsing due to seismic forces. It is always advisable to incorporate them in areas that are likely to experience high intensity earthquakes or strong winds. Wind walls are designed as simple concrete walls.
Key Words: Reinforced concrete shear wall, Axial force, Bending moment, Shear force, Base Moment, Base Shear, Shear strength.

1. INTRODUCTION

A shear wall is a structural element in a reinforced concrete frame structure that resists lateral forces such as wind forces. Shear walls are generally used in high-rise buildings exposed to cross wind and seismic forces. In reinforced concrete frame structures, the effects of wind forces increase with increasing height of the structure. Good practice guidelines limit horizontal movement or rocking.

2. Problem Statement

10-storey building has plan dimensions as shown in Fig. 1.Two shear walls are to be provided in each direction to resist the seismic forces. The axial load on each shearwall is 16000kN due to both dead and live loads. The height between floors is 3.5m. dead load per unit area of the floor, which consists of floor slab, finishes. etc., is 4.5kN/m^2 and the weight of partitions on floor is 3kN/m^2.The intensity of live load on each floor is3.5kN/m^2 and onroof is 1.5kN/m^2. The soil below the foundation is hard andthe building is located in Rishikesh.



3. Solution

Seismic weight of the building

According to the provisions of the code, the percentage of design load to be considered for earthquake force calculation is 25% for floors and live load for roof is not considered.

Hence, the effective weight at each floor will be

= 4.5 + 3.0 + 0.25 × 3.5 = 8.375kN/m^2

and that at the roof = 4.5kN/m^2

Weight of 60 beams, each 4 m span, on each floor and roof
= 0.3×0.6× (4×60) ×25

= 1080kN
Weight of 36 columns at each floor

= 0.3×0.6×2.4×36×25

= 388.8kN
Weight of columns at roof = 0.5x388.8= 194.4kN

Plan area of building is 20 m × 20 m = 400 m^2 Equivalent load at roof level = 4.5×400 + 1080 + 194.4
= 3074.4kN
Equivalent load at each floor = 8.375×400 + 1080 + 388.8
= 4818.8kN
Seismic weight of the building, *W* = 3074.4.4 + 4818.8×9
= 46443.6kN

3.1 Base shear

The fundamental natural period of oscillation T for buildings with shear walls is given by T = $\frac{0.09h}{\sqrt{d}}$

= (0.09x35)/ (20)$^{1/2}$(d, the plan dimension = 20m)

Building is situated in Rishikesh, i.e., in Zone IV. Zone factorZ = 0.24, importance factor I = 1, response reduction factorR = 4.0 for 5% damping and type I soil, average response acceleration coefficient $\frac{S_a}{g}$= 1.81 Design horizontal seismic

coefficient $A_h = \frac{Z}{2} \times \frac{I}{R} \times \frac{S_a}{g}$

$= \frac{0.24 \times 1 \times 1.81}{2 \times 4}$ = 0.0543

Base shear $V_b = A_h \times W$ = 0.0543×46443.6 = 2521.887kN
Lateral loads and shear forces at different floor level aregiven in Table 1.

Table 1 Calculation of lateral loads and shear

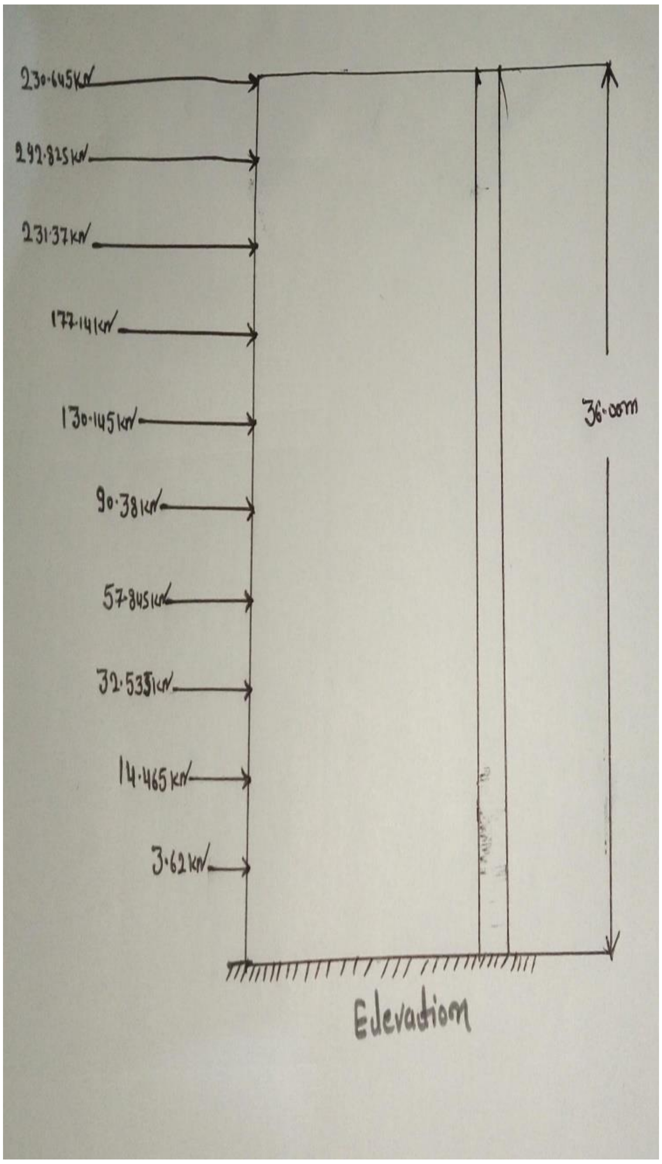
Mass No.	W(KN)	H(m)	$W_ih_i^2$	$W_ih_i^2 / \sum W_ih_i^2$	Q_i(KN)	V_i(KN)
1	3074.4	35	3766140	0.8291312	461.29	464.29
2	4818.8	31.5	4781454.3	0.23222469	585.65	1046.94
3	4818.8	28	377939.2	0.18348618	462.74	1509.68
4	4818.8	24.5	2892484.7	0.104816	354.28	1863.96
5	4818.8	21	2125090.8	0.10321097	260.29	2124.25
6	4818.8	17.5	1475757.5	0.07167429	180.76	2305.01
7	4818.8	14	944484.8	0.04587154	115.69	2420.07
8	4818.8	10.5	531272.7	0.02580274	65.071	2485.77
9	4818.8	7	236121.2	0.01146789	28.93	2514.701
10	4818.8	3.5	59030.3	0.00286697	7.24	2521.94
			$\sum W_ih_i^2$=	20589775.5		

3.2 Bending moment and shear force

Two shear walls are arranged in each direction to resist seismic forces. Therefore, the lateral forces acting on one shear wall will be half of the calculated shears and are as shown in Fig. 2.

Figure 2. Lateral Forces on shear wall

The shear wall will be designed as a cantilever fixed at thebase and free at the top.
Maximum shear force at base V = 1260.97kNMaximum bending moment at base,



M = (7.24×3.5) + (28.93×7) + (65.071×1.5) +(115.6×14) + (180.76×17.5) + (260.29×21) + (354.28×24.5) + (462.74×28) + (585.65.73×31.5) + (461.29×35

M = 67388.5905kNm

3.3 Loads and material properties

Typical shear wall stress results obtained from the above section of the analysis and are as follows:

Taking partial safety factor 1.5

Factored shear force, V_u = 1.5 × 1260.97 = 1891.455kNFactored bending moment, M_u = 1.5 × 67388.5905 = 101082.88kNm

Factored axial load, P_u = 1.5 × 16000 = 24000kNAxial force on boundary element = 2400kN

The material properties for reinforced concrete shear wall and reinforcing steel are assumed as follows: Concrete gradeM30, *fck* = 30MPa, Reinforcement grade HYSD Fe 415

4. General requirements for a shear wall

Thickness of Wall, t_w = 300mm (Minimum thickness, as perClause 10.1.2, IS13920:2016 shall be 150 mm)

Length of wall, L_w =14000mm

Since wall thickness > 200 mm. vertical as well as horizontalreinforcement shall be provided in two layers or in two curtains in the shear wall (Clause 10.17(b), IS 13920:2016)

Maximum diameter or reinforcement < $(t_w)/10$(Clause 10.1.8, IS 13920:2016)

The maximum diameter of the reinforcement should therefore be 30 mm. As 30 mm size bars are not available on the market, the maximum practical diameter of the reinforcing bar must be 28 mm.

Maximum spacing of reinforcement shall not exceed the smaller of the following:

- $L_w/5$ =14000/5 = 2800mm
- $3t_w$ = 3 × 300 = 900 mm
- 450 mm

(Clause 10.1.9, IS 13920:2016)

Hence, maximum spacing of reinforcement ≤450mm Minimum in-plane reinforcement in the longitudinal andtransverse directions in the shear wall shall be 0.25% of therespective gross sectional area of the wall.

(Clause 10.1.6, IS 13920:2016)

4.1 Shear strength requirements

Let the nominal shear stress in the wall = $\tau_v \tau_v = \frac{V_u}{t_w d_w}$(Clause 10.2.1, IS: 13920-2016) ∴

Where,

d_w= effective depth of the wall sectionFor rectangular wall sections,

d_w= 0.8 × L_w (Clause 10.2.1, IS: 13920-2016)

∴ d_w = 0.8 × 12000 = 9600mm

∴ τ_v= (1891.45x1000)/(300x9600)

= 0.656MPa

For M30 grade concrete τ_{cmax}= 3.5MPa[Table-20, IS 456:2000]

Since $\tau_v < \tau_{cmax}$, wall section is adequate for shear.

Assume the minimum 0.25% steel in the wall in the verticalas well as in the horizontal direction

For p_t = 0.25%, τ_c= 0.37MPa [Table 19, IS 456:2000]

Since $\tau_v > \tau_c$,

Shear reinforcement is required.

4.2 Horizontal shear reinforcement

Provide horizontal shear reinforcement as per 10.2.3 section c of Is 13920:2016

Assume 2-legged 12 mm diameter horizontally alignedclosed Stirrups along the height of the shear wall,

Spacing of stirrups along the height of shear wall

∴ Spacing of stirrups = $S_v = \frac{0.87 \times f_y \times A_h \times d_w}{V_{us}}$

∴ S_v= (0.87x415x2x113x9600)/[(1891.45x1000)-(0.37x300x9600)]

∴ S_v = 950 mm c/c

Minimum horizontal reinforcement = $(A_{sh})_{min}$

= 0.0025 of gross sectional area of the wall in elevation(Clause 10.1.6, IS 13920:2016)

∴ $(A_{sh})_{min}$ = 0.0025 ×300 × 1000 = 750 mm²

Hence, provide 2-legged 12 mm diameter horizontally aligned closed stirrups at 950mm c/c along the entire heightof the shear wall.

4.3 Vertical shear reinforcement

According to Clause 10.1.6 of IS 13920:2016, the vertical reinforcement, which shall be uniformly distributed in the wall section, shall not be less than the horizontal reinforcement.
Assume 2-legged 12 mm diameter vertically oriented stirrups.

Spacing of stirrups = $S_v = \frac{0.87 \times f_y \times A_h \times d_w}{V_{us}}$

∴ ∴ S_v= (0.87x415x2x113x9600)/ [(1891.45x1000) -(0.37x300x9600)]

∴ S_v = 950 mm c/c

Minimum vertical reinforcement = 0.0025 of gross sectionalarea of the wall in plan
(Clause 10.1.6, IS 13920:2016)
∴ $(A_{sv}) = 0.0025 \times 300 \times 12000 = 9000\ mm^2$
Hence, provide 2-legged 12 mm diameter vertically alignedclosed stirrups at 950mm c/c along the entire length of theshear wall. $A_{sv} > A_{sh}$, hence ok.

4.4 Check for flexural strength

With reference to Annex 'A' and Clause 10.3.1 ofIS 13920:2016

$x_u/L_w = \frac{(\Phi+\lambda)}{(2\Phi+0.36)}$

$x_u'/L_w = \frac{(0.0035)}{\left[0.0035+\left(\frac{0.87 f_y}{E_s}\right)\right]}$

$\Phi = \frac{(0.87 f_y \times \rho)}{f_{ck}}$

$\lambda = \frac{(P_u)}{f_{ck} \times t_w \times l_w}$

ρ = vertical reinforcement ratio = $\frac{A_{st}}{t_w \times l_w}$

=[(2x113x12000)/950]/(300x12000) = 0.0007928

Φ =(0.87x415x0.0007928)/30 = 0.00954

λ =(24000x10000/(30x300x12000) = 0.223

x_u/L_w=(0.00954+0.223)/(2x0.00954+0.36) = 0.6134

$x_u'/L_w = \frac{(0.0035)}{\left[0.0035+\left(\frac{0.87\times 415}{2\times 10^5}\right)\right]} = 0.6597$

Since $x_u'/L_w > x_u/L_w$, Eq.(a) of Annex "A" (Clause 10.3.1)IS 13920:2016, is applicable.

4.5 Moment of resistance of Rectangular Shear wallsection

A-1 bearing moment of a slender rectangular shear wall section with uniform vertical reinforcement can be estimated as follows:

$$\therefore \frac{M_{uv}}{f_{ck} \times t_w \times l_w^2} = \Phi\left[\left(1+\frac{\lambda}{\Phi}\right)\left(\frac{1}{2}-0.416\frac{x_u}{l_w}\right)-\left(\frac{x_u}{l_w}\right)^2\left(0.168+\frac{\beta^2}{3}\right)\right]$$

$\beta = \left(\frac{0.87\times 415}{0.0035\times 2\times 10^5}\right) = 0.5158$ & $\frac{\beta^2}{3} = 0.088678$

$\therefore \frac{M_{uv}}{f_{ck} \times t_w \times l_w^2}$ = {0.00954 × [(24.375) × (0.2448) – 0.09657]}

$\therefore \frac{M_{uv}}{f_{ck} \times t_w \times l_w^2} = 0.05600$

∴ $M_{uv} = 0.05600 \times 30 \times 300 \times 12000^2$
= 72576.0kNm
Balance moment to be resisted by the edge reinforcement ineach shear wall
= (101082.88 – 72576.0)
= 28506.88kNm

4.6 Check on boundary elements

To check the necessity of providing boundary elements inthe shear wall (Figure 3).
Gross area of wall section, A_g = 12000×300
= 3600000mm^2

I_{xx} =(300x12000^3)/12 = 43.2×10^{12} mm^4

Combined stress at edge of wall:

$\therefore \sigma = \frac{P}{A} \pm \frac{M}{I} y$

$\therefore$ σ =(24000x10^3)/(12000x300) ±[(101082.88x10^6)/(43.2x10^2)]x(12000/2)

$\therefore$ σ = 6.67±14.0392

Maximum stress = σ_{max} = 20.70MPa

According to clause 9.4.1 of IS 13920:2016, if the extreme compressive stress of fibers in the wall due to factored gravity load plus factored earthquake exceeds 0.2 fck= 0.2×30
= 6 < 20.70MPa

Therefore, boundary elements are required in the shear wall. Provide a 1400mm long by 500mm wide boundary element at each edge of the shear wall, Figure 1.

4.7 Design of boundary elements

The bounding element is essentially considered a column. Vertical reinforcement in edge members shall not be less than 0.80% nor more than 6%. (Clause 9.4.4, IS 13920
1993)

Adopt 3% vertical reinforcement in the boundary elements.$A_{sc\ provided}$ = 0.03×1400×500 = 21000mm^2

Axial compression load on the boundary element dueseismic forces
= (M_u-M_{uv})/C_w (Clause 9.4.2, IS 13920:1993)

M_u = Factored design moment on the entire shear wallsection = 278862.64kNm

M_{uv} = Moment of resistance provided by the distributedVertical reinforcement across the wall section
= 171735.56kNm

C_w = center to center distance between the boundaryelements along the two vertical edges of the shear wall
= 12.6 m

Axial compressive load =(101082.88-28506.88)/12.6

= 5760kN

Required axial load capacity of boundary element = axial load due to gravity effects + axial load due to seismic forces = 2400 + 5760 = 8160kN

As per clause 9.4.2, IS 13920:1993, the boundary element shall be assumed to behave as an axially loaded short column.

If the design strength of an axially loaded short column = P_{uD},then

$0.4f_{ck}$ A_c + $0.674f_yA_{sc}$

P_{uD} = $0.447f_{ck}$ A_g + (f_c– 0.447 f_{ck})$A_{sc}f_c$= 0.790 f_y(for Fe415 steel)

P_{uD} = 0.447×30×1400×500+ (0.790×415 –
0.447×30) ×21000
= 15990.24kN> 8160.15kN, Hence ok

Area of steel for each boundary element = 21000mm^{2}Provide 10 nos 40Φ + 12 nos 32Φ in each element

A_{sc} provided = 22217.34 mm^2>21000 mm^2 Hence, Ok

As per (Clause 9.4.5, IS13920:1993), boundary elements shall be provided with special confining reinforcement throughout their height.

Area of special confining reinforcementA_{sh} = 0.18 $S_h \frac{f_{ck}}{f_y} \times \left(\frac{A_g}{A_k} - 1\right)$

(Clause 7.4 8, IS 13920:1993)

Assume 12 mm diameter closed hoops as confiningreinforcement with a clear cover of 40 mm

The gross area of the boundary element sectionAg =1400×500 = 700000 mm^2

The size of the core = A_k= (1400-40 - 40) × (500 -40-40) =1320×420 mm

Since both dimension of the core is greater than 300mm, a cross tie will have to be used.

If the cross-tie is placed at mid-length of the longer dimension of the core then, h = 1320/2 = 660 mm and for shorter dimension, h = 420/2 = 210 mm.

The spacing of the confining hoops, S, shall not exceed the smaller of

1. $^1/_4$Minimum member dimension, i.e.$^1/_4 \times 500$ =125mm
2. 100mm.

The spacing however, need not be less than 100mm.(Clause 7.4.6 of IS 13920:1993)

Hence adopt 100mm spacing of the confining hoopsS = 100mm

A_{sh} =$0.18 \times 100 \times 660 \times \frac{30}{415} \times \left(\frac{1400\times500}{1320\times420} - 1\right)$=225.55 mm^2>
113.09 mm^2 (Area of 12 mm diameter bar)

Hence, 12 mm diameter confining reinforcement isinsufficient.

Adopt 20 mm diameter confining hoops at a spacing of 100mm c/c along the entire height of the boundary element A_{sh}

provided = 314 mm^2 > 225.55 mm^2.
Hence, ok.

REFERENCES

[1] IS 13920:1993 Ductile detailing of reinforced concrete structures subjected to seismic forces - Code of practice [CED 39: Earthquake Engineering].

[2] IS 13920:2016 Ductile detailing of reinforced concrete structures subjected to seismic forces - Code of practice [CED 39: Earthquake Engineering].

[3] IS 456 (2000): Plain and Reinforced Concrete - Code ofPractice [CED 2: Cement and Concrete]

[4] IS 1893-1 (2002): Criteria for Earthquake ResistantDesign of Structures, Part 1: General Provisions and Buildings [CED 39: Earthquake Engineering]

[5] IS:4326-1993: Earthquake Resistant Design and Construction of Buildings

[6] Research paper using IRJET.

[7] Advance R/F concrete design by P.C Varghese.

[8] SP:22(S&T)-1982: Indian Standard ExplanatoryHandbook On codes for Earthquake Engineering - IS:1893-1975 & IS:4326-1976

[9] IITK-BMTPC Earthquake Tips: Learning Seismic Designand Construction, IIT Kanpur, India.