Analysis and design of confined masonry wall

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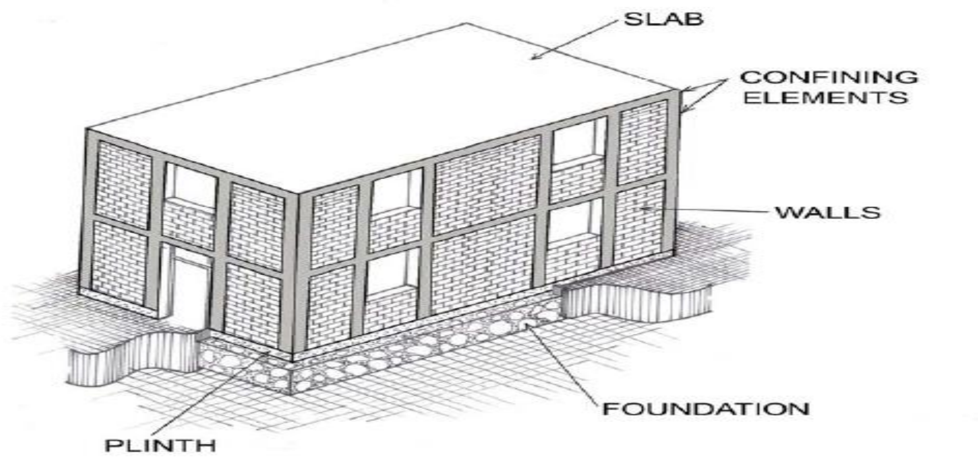
Abstract

Considering the present scenario, it is observed that the performance of the unreinforced masonry building during the earthquake conditions have resulted into significant damages in the building and subsequent life loss. The conventional method of masonry construction adopted so far utilizes the same construction material as in confined masonry, however the construction technique differs for both.The study consider analysis and design of confined unreinforced masonry wall. The difference in the method of construction and the performance of both, under the seismic conditions is considered through the study on a sample building..

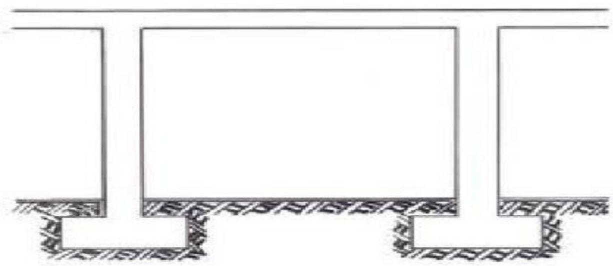
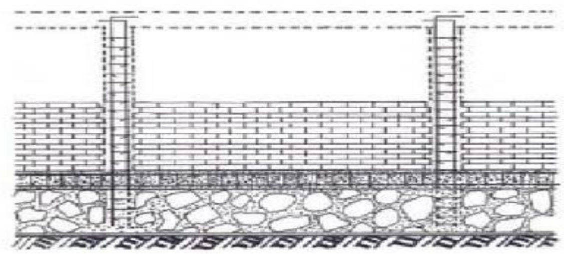
Keywords: *confined masonry, G+3 storey, seismic design, tie elements, wall density*

INTRODUCTION

- Confined masonry consist of load bearing walls, horizontal & vertical reinforced concrete confining elements, built on all four sides of the wall panel, as shown in the figure.



- The load bearing walls are built up to some height & then the RC confining tie-column elements are casted, unlike ordinary RC frame structure building, where wall acts as infilled wall only. This way, it is different from RC frame structure as shown in the figure.

- The study includes analysis and design of confined masonry by considering various design parameters as listed here: wall density, base shear distribution to wall along height, check for overturning, compressive stress check, compressive strength check from wall density, tensile stress check, shear stress check, shear strength check from wall density, out of plane stability check and design of bond beam – tie column.[1]The analysis and design of the confined masonry wall presented here is carried out using the guidelines provided by the [1]Central building research institute, Roorkee (India), [2]IS 1893-2002, "Criteria for Earthquake Resistant Design of Structures", Bureau of Indian Indian standards and [3] IS 1905-2002, "Code of Practice for Structural use of Unreinforced Masonry.

- **OBJECTIVE**
  To provide a summary of the seismic design provisions for confined masonry wall along with explanatory example.

ANALYSIS AND DESIGN PARAMETERS FOR WALL

The following parameters are referred as literature review to incorporate it in the example, the design guidelines have beenprepared by Central building research institute, Roorkee.

1. Wall density

Wd (%) =(wall area/ floor area)*100 along the considered direction

2. Distribution of seismic force into individual wall

Uniform lateral force in wall is given by ,

$$F_{lxi} = \frac{Q_{ix}}{\sigma\ K_{xi}} K_{xi} \qquad\qquad F_{lyi} = \frac{Q_{iy}}{\sigma\ K_{yi}} K_{yi}$$

Force due to torsion in wall is given by,

$$F_{txi} = \frac{Q_i e_{dy}}{K_t}\ Y'_i \sum K_{xi} \qquad\qquad F_{tyi} = \frac{Q_i e_{dx}}{K_t}\ X'_i \sum K_{yi}$$

3. Check for overturning

$M_o = P_i * h_w/2$ and M_r = total load $* h_w$

So that, Mr/Mo should be greater than 1.5

4. Check for compressive stress

$P_{comp} = k_s * f_m$
Where, $f_m = 0.422 * f_b^{0.69} * f_{mo}^{0.252}$

Panel is considered to be safe if given criteria is fulfilled:
$P_{comp} > 2.6 * \sigma_{dl}$

5. Compressive strength check from wall density consideration

$$W_d \geq \frac{f_g\, w\, n_s}{P_{comp}}$$

in the both X and Y direction.

5. Check for tensile stress

$$\sigma_t = \frac{M}{S} - \sigma_{dl}$$

6.Check for shear stress

Permissibe shear stress for the confined masonry wall is given as:
$T_u = 0.1 + \sigma_d / 6$

7. Shear strength check from wall density consideration

$$W_d \geq \frac{A_h\, f_s\, w\, n_s}{\tau_u}$$

8. Out of plane stability of wall panel

Seismic load per unit area of wall panel, $F = A_h * \gamma_m * t_w$
Ultimate bending moment per unit length of wall is given by, $Mu = F*h^2/8$
Bending stress, $\sigma_b = Mu / S$
Actual stress = $\sigma_b - \sigma_{dl}$ should be less than tensile strength of masonry

9. Design of Bond beam

Bond beam has been designed for total lateral load acting on the wall.
$A_{st} = P_i / f_y$

10. Design of tie column

Area of steel in tie column is calculated by the expression ,
$Ast = (1+0.25k) * P_i * h_w / l_w * f_y$

- **Problem statement**
- Design confined masonry building for the G+3 storey plan shown in figure, for given detail.

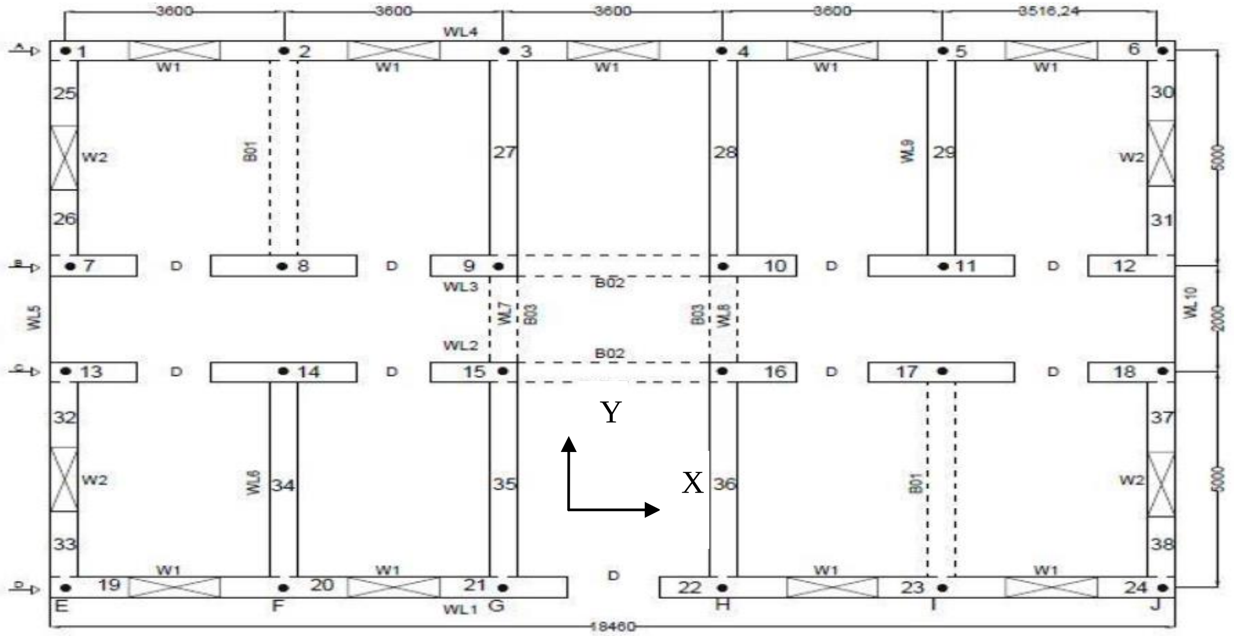


Figure 4.4 Building Plan

- **Building geometry**

Component	Dimensions	Unit
Plan dimension in X-dir	18	m
Plan dimension in Y-dir	12	m
Storey height	3.6	m
Building height	14.4	m
Bond beam	230*350 &230*230	mm
Tie column	230*350 &230*230	mm
Slab	120	mm
Door opening	1200*2100	mm
Window opening	1350*1200	mm
Wall thickness	350 & 230	mm

- **Material properties**

Component	Parameter	Value	Unit
Brick	Comp. strength	10.5	N/mm^2
Mortar	Comp. strength	3	N/mm^2

Masonry	Density	20	kN/mm^3
Masonry	Tensile strength	0.25	N/mm^2
Concrete	Density	25	N/mm^3
Concrete	Grade	20	N/mm^2
Steel	Grade	Fe-415	N/mm^2

- **Seismic parameters**

Parameter	**Value**
Zone factor(Z)	0.16(zone-3)
Importance factor(I)	1.5
Response reduction factor(R)	3
Soil type	medium

Load(slab)	**Component**	**Value(kN/m^2)**
Dead	Slab	3
Dead	Floor finish	1
Live	Imposed	4
Total	All	8

- **Wall density calculation (W_d)**

It is given as, W_d = wall plan area / floor area

For floor weight = 8 kN/m^2 and n = 4 storey,

So that, W_d required = 5.2% for 1st, 2nd and 2.6% for 3rd, 4th storeyin each direction.

Now W_d for 1st,2nd & 3rd ,4th storey in X-direction = 6.7 % & 4.5% ….so ok.

And W_d in Y direction for the same, = 7.8% & 5.1% ….so ok

- **Equivalent stiffness of wall panel**

Five types of wall panel are here.

1.With window & 3600 mm length 2. With window & 5000 mm length

3.With door & 3600 mm length 4. Solid wall of 5000 mm length

5. Solid wall of 200 mm length

Now, stiffness of wall panel is determined by following equation.

$$R_c = \frac{Et}{4(\frac{h}{d})^3 + 3(\frac{h}{d})} \ \ for\ cantilever\ wall \quad \& \quad R_f = \frac{Et}{(\frac{h}{d})^3 + 3(\frac{h}{d})} \ \ for\ solid\ wall$$

From above equation, stiffness of ground floor & 1st floor walls of 350 mm thickness is found out and shown below.

k_1 = 0.0810E, k_2= 0.134E, k_3 = 0.0528E, k_4 = 0.138E, k_5 = 0.031E

and stiffness of 2nd& 3rd floor walls of 230 mm thickness is found out and shown below.

k_1 = 0.0532E, k_2= 0.0884E, k_3 = 0.035E, k_4 = 0.091E, k_5 = 0.020E

- So that, summation of all wall stiffness in X-direction for ground & 1st floor , $\sum$ k = 1.2042E

- Summation of all wall stiffness in X- direction for 2^{nd}& 3^{rd} floor, $\sum k = 0.7938E$
- summation of all wall stiffness in Y-direction for ground & 1^{st} floor , $\sum k = 1.426E$
- summation of all wall stiffness in Y-direction for 2^{nd}& 3^{rd} floor , $\sum k = 0.9396E$

➢ **Center of mass**

$X_m = \sum W_i * x_i / \sum W_i$ and $Y_m = \sum W_i * y_i / \sum W_i$

Due to symmetry of the building plan, the center of mass in both direction is at midway.

Taking origin at 1^{st} point in plan, center of mass of typical floor (x,y) = (9.15 , 6.15) m

➢ **Center of stiffness**

$X_{cs} = \sum K_y * x_i / \sum K_i$ and $Y_{cs} = \sum K_x * y_i / \sum K_i$

=9 m = 6.14 m

➢ **Torsional stiffness of walls (G.F &F.F)**

$K_t = \sum K_{xi} * Y_i^2 + \sum K_{yi} * X_i^2$

= 86.806E

➢ **Computation of eccentricity**

$e_x = X_{cm} - X_{cs}$ and $e_y = Y_{cm} - Y_{cs}$

e_x =0.15 m and e_y = 0.07 m

so ,$(e_{di})_x = 1.5 * e_x + 0.05 * b_i$ or $e_x - 0.05 * b_i$

= 1.5 *0.15 + 0.05 * 12 or 0.15 – 0.05 *12

= 0.825 m or - 0.45 m

So that, $(e_{di})_x$= 0.825 m

,$(e_{di})_y = 1.5 * e_y + 0.05 * b_i$ or $e_y - 0.05 * b_i$

= 1.5 *0.07 + 0.05 * 18 or 0.07 – 0.05 *18

= 1.005 m or -0.83 m

So that, $(e_{di})_y$ = 1.005 m

➢ **Calculation of confined masonry seismic weight**

At first floor level:

Wall length, lw = 118.8 m, t_w = 350 mm , h_w = 3.5 m

Weight of wall = 118.8 * 0.350 * 3.6 * 20 = 2993.76 kN

10 % deduction for opening, then weight = 2700 kN

Bond beam = 118.8 * 0.350 *0.230 *25 = 240 kN

Tie column = 24 * 3.6 *0.350 *0.230 *25 = 173.88 kN

Slab = (0.120 * 18 *12 *25) + 2 *18 *12 = 1080 kN

Finally, total weight at the first floor , W_1= 4193 kN

Similarly, at second floor level, W_2 =3622 kN

At third floor level,W_3 = 3051kN

At fourth floor level,W_4 = 1309.68 kN

Total seismic weight, $W=W_1 +W_2 =W_3 +W_4$ = 12175 kN

- **Calculation of base shear**

$V_b = A_h * W$

Where, $A_h = \frac{Z*I*Sa}{2*R*g}$, all parameters are given at starting of problem.

Fundamental time period , $T_a = 0.09 * h / (d)^{0.5}$ = 0.30 sec for X direction

Fundamental time period , $T_a = 0.09 * h / (d)^{0.5}$ = 0.37 sec for Y direction

So that, A_h = 0.1 for both X & Y direction

V_b = 12715 * 0.1

= 1217 kN in X & Y direction

Floor	W_i(kN)	H_i (m)	$W_ih_i^2$	$C=W_ih_i^2/\sum W_ih_i^2$	$Q_i = V_b *c$	V_i (kN)
4	1309.68	14.4	271575.24	0.3123	380.07	380.07
3	3051	10.8	355868.64	0.4092	498	878.07
2	3622	7.2	187764.48	0.2159	262.75	1140.82
1	4193	3.6	54341.28	0.0624	75.94	1217
			$\sum$= 869549		$\sum$ = 1217	

- **Distribution of seismic force into individual panel**

Here, we consider wall panel 1- 2 , and corresponding calculations are done.

Force due to uniform lateral translation:

At fourth floor, $F_{1-2,X}= Q_i *K_{xi}/ \sum K_{xi}$= 380.07 *0.0532E /0.7938E = 25.47 kN

due to torsion, $F_{1-2,t} = (Qi*e_{dy} *Y_i'*\sum K_{xi})/ k_t$

= 380.07 *1.005* (12-6.14)*0.7938E / 56.76 E

=31.30 kN

Total force , P_{1-2} =25.47 + 31.30 = 56.77 kN…at fourth floor level

Similarly at third floor level, P_{1-2} = 74.38 kN

At second floor level, P_{1-2} = 39.13 kN

At first floor level, P_{1-2} = 11.30 kN

- **Check for overturning**

Overturning moment, $M_o = P_i * h_w/2$ at ground floor level

= 408.74 +401.65+140.86+20.34

= 971.59 kN.m

Total gravity load for wall panel 1-2 = s.w +slab load (dead+live)

at fourth floor , w_4 = 77 kN at third floor , w_3 = 94 kN

at second floor, w_2 = 125.28 kN at first floor, w_1= 125.28 kN

so, resisting moment ,Mr =1108 + 1015.2 +902.01 +451.008

= 3477.018 kN.m

Mr /Mo = 3477.018/971 = 3.58 > 1.5..so ok.

- **In plane stability of wall panel**

1. **Check for compressive stress:**

$P_{comp} = K_s * f_m$, where $f_m = 0.422*f_b^{0.69} *f_{mo}^{0.252}$

$= 2.819$ N/mm^2

For considered wall panel, slenderness ratio, h/tw = 3600/ 350 =10 .28

From table-9 , IS:1905 , the value of Ks = 0.88

P comp = 2.819 * 0.88 = 2.48 N/mm2…compressive strength of wall

Now, σ_{dl} = compressive stress due to dead and live load

Self weight of panel = 300.672 kN

Slab load = 120.96 kN

So that, σ_{dl}= (120.96+300.672)/(0.35*3.6) = 0.334N/mm^2

Now , panel is considered to be safe if following criteria is fulfilled.

$P_{comp} = 2.6 * \sigma_{dl} = 2.6 *0.334 = 0.8684$ N/mm^2

Here, P comp > 0.8684 N/mm^2,hence panel 1-2 is safe in compression.

2. **Compressive strength check from wall density consideration:**

Now, $W_d \geq f_g *w *n_s /P_{comp}$

so that, P_{comp} required = 2.33*0.0195 *4/0.067 = 2.43 < 2.48N/mm^2…in X direction

P_{comp} required = 2.33* 0.0195 *4/0.078 =2.091 < 2.48 N/mm^2…in Y direction

So, safe in compression in both direction

3. **Check for tensile stress:**

σ_t =M/S – σ_{dl} = (326.84 * 10 ^{6}/75.6*10 ^{6})- 0.334

= 0.098 N/mm^2< 0.25 N/mm^2 …safe in tension

4. **Check for shear stress:**

Permissible shear stress is given as,

t_u = (0.1 +σ_d)/6, where σ_d is compressive stress due to dead load

= 0.148 N/mm^2…shear strength

Actual shear stress acting on the panel,1-2 = P_i/A_w =0.144 N/mm^2< 0.148..so safe

5. **Shear strength Check from wall density consideration :**

Wd ≥ A_h *f_s *w* n_s/t_u , so that t_u min =0.1*1.3*0.0195*4/ 0.067 in x direction

=0.135 < 0.148 N/mm ^{2} so safe in shear.

t_u min = 0.1 *1.3 *0.0195*4 /0.078 in Y direction

= 0.116 N/mm^2 < 0.148 so safe in shear

- **Out of plane stability of wall panel**

Seismic weight per unit area of wall panel, F = A_h * ϱ_m * t_w = 0.7 kN/m^2

Ultimate bending moment per unit length of wall, Mu = F *h^2/8

= 0.7 *3.6^2 /8 = 1.134 kN.m/m

Bending stress, σ_b = M_u/S = (1.134*1000)*6/350^2 = 0.055 N/mm^2

Actual stress, $\sigma_p = \sigma_b - \sigma_{dl}$ = 0.055 – 0.334 = – 0.279 N/mm

Actual stress < tensile stress , hence wall panel is safe about out of plane stability.

- **Design of bond beam**

Bond beam has been designed for total lateral load acting on the wall,

$$A_{st} = P_i / f_y$$

At fourth floor,

For bond beam of wall panel 1-2, A_{st} = 56.77 *1000/ 415 = 136 mm2

A_{st} min = 0.85*230*230/415 = 108mm2

So provided A_{st}> A_{st} min..so ok.

Therefore, provide 4 bar of 10 mm dia.

also, $T_v = V_u$/bd = 56770/ 230*230 = 1.073 N/mm^2

so that, p_t = 100*314/230*230 =0.593%

from IS: 456-2000, T_c = 0.51 N/mm^2

V_{uc} = 0.51* 230*230 =26.97 kN, so that V_{us} = 56.77- 26.97 = 29.8 kN

So, take 6 mm dia bar stirrups, A_{st} =2*0.785*36 = 56.52 mm^2

S_v = 0.87*415*56.52*230/29800 =157 mm

Or S_v = 0.75*d = 0.75 *230 = 172 mm or 300 mm

Therefore , provide 6 mm stirrups at 150 mm c/c.

Similarly, at third floor

Provide 4 bar of 10 mm dia. and 8 mm stirrup at 180 mm c/c

At second floor

Provide 4 bar of 12 mm dia . and 8 mm stirrup at 150 mm c/c as IS 4326:2016

At first floor

Provide 4 bar 12 mm dia. and 8 mm strirrup at 130 c/c

- **Design of tie column**

Area of steel in tie column is calculated by the expression,

A_{st} = (1+ 0.25*k)P_i*h_w/l_w*f_y,

Where, k = num of story above above the analysed storey

Ao at fourth floor, Ast= (1+0.25*0)56.77*3.6/3.6*415

= 136 mm^2

Therefore, provide 4 bar of 10 mm dia (314mm^2) longitudinally

Similarly, at third floor , A_{st} = 395 mm^2

Provide 4 bar of 12 mm dia (452mm^2)

At second floor , A_{st} = 615 mm2

Provide 4 bar of 16 mm dia (803 mm^2)

At first floor, A_{st} =765 mm^2

Provide 4 bar of 16 mm dia (803mm^2)

And provide 2 legged 6 mm dia stirrup at 150 c/c as shear reinforcement in all tie columns, according to detailing criteria specified in IS 13920:2016.

- **Conclusion**

A simplified approach for the design of confined masonry building wall panel as a building element is presented here in this document, as there is not much resources available for design of confined masonry building. This document may serve as guiding material for designing confined masonry structure.

References

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