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.

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 been prepared by Central building research institute, Roorkee.

1. Wall density

\[ W_d = \frac{100 \times \text{wall area}}{\text{floor area}} \]

2. Distribution of seismic force into individual wall

Uniform lateral force in wall is given by,

\[ F_{bd} = \frac{Q_{ix}}{\sigma K_{xi}} \]

\[ F_{iy} = \frac{Q_{iy}}{\sigma K_{yi}} \]

Force due to torsion in wall is given by,

\[ F_{txi} = \frac{Q_{i\theta}}{K_t} \sum K_{xi} \]

\[ F_{tyi} = \frac{Q_{i\theta}}{K_t} \sum K_{yi} \]

3. Check for overturning

\[ M_o = \frac{P_i \times h_w}{2} \quad \text{and} \quad M_r = \text{total load} \times h_w \]

So that, \( M_r / M_o \) should be greater than 1.5

4. Check for compressive stress
\[ P_{\text{comp}} = k_s \cdot f_m \]
Where, \( f_m = 0.422 \cdot f_b^{0.69} \cdot f_{m0}^{0.252} \)

Panel is considered to be safe if given criteria is fulfilled:
\[ P_{\text{comp}} > 2.6 \cdot \sigma_{dl} \]

5. Compressive strength check from wall density consideration

\[
W_d \geq \frac{f_w \, w \, n_s}{P_{\text{comp}}} 
\]

in the both X and Y direction.

5. Check for tensile stress

\[ \sigma_t = \frac{M}{S} - \sigma_{dl} \]

6. Check for shear stress

Permissible shear stress for the confined masonry wall is given as:
\[ T_u = 0.1 + \frac{\sigma_d}{6} \]

7. Shear strength check from wall density consideration

\[
W_d \geq \frac{A_3 \cdot f_w \cdot w \cdot n_s}{\tau_u} 
\]

8. Out of plane stability of wall panel

Seismic load per unit area of wall panel, \( F = A_h \cdot \gamma_{m} \cdot t_w \)
Ultimate bending moment per unit length of wall is given by, \( M_u = F \cdot h^2/8 \)
Bending stress, \( \sigma_b = M_u / S \)
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,
\[ A_{st} = (1+0.25k) \cdot P_i \cdot h_w / l_w \cdot f_y \]
Problem statement
Design confined masonry building for the G+3 storey plan shown in figure, for given detail.

Building geometry

<table>
<thead>
<tr>
<th>Component</th>
<th>Dimensions</th>
<th>Unit</th>
</tr>
</thead>
<tbody>
<tr>
<td>Plan dimension in X-dir</td>
<td>18</td>
<td>m</td>
</tr>
<tr>
<td>Plan dimension in Y-dir</td>
<td>12</td>
<td>m</td>
</tr>
<tr>
<td>Storey height</td>
<td>3.6</td>
<td>m</td>
</tr>
<tr>
<td>Building height</td>
<td>14.4</td>
<td>m</td>
</tr>
<tr>
<td>Bond beam</td>
<td>230<em>350 &amp;230</em>230</td>
<td>mm</td>
</tr>
<tr>
<td>Tie column</td>
<td>230<em>350 &amp;230</em>230</td>
<td>mm</td>
</tr>
<tr>
<td>Slab</td>
<td>120</td>
<td>mm</td>
</tr>
<tr>
<td>Door opening</td>
<td>1200*2100</td>
<td>mm</td>
</tr>
<tr>
<td>Window opening</td>
<td>1350*1200</td>
<td>mm</td>
</tr>
<tr>
<td>Wall thickness</td>
<td>350 &amp; 230</td>
<td>mm</td>
</tr>
</tbody>
</table>

Material properties

<table>
<thead>
<tr>
<th>Component</th>
<th>Parameter</th>
<th>Value</th>
<th>Unit</th>
</tr>
</thead>
<tbody>
<tr>
<td>Brick</td>
<td>Comp. strength</td>
<td>10.5</td>
<td>N/mm²</td>
</tr>
<tr>
<td>Mortar</td>
<td>Comp. strength</td>
<td>3</td>
<td>N/mm²</td>
</tr>
</tbody>
</table>
Masonry Density 20 kN/mm$^2$
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

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Zone factor (Z)</td>
<td>0.16 (zone-3)</td>
</tr>
<tr>
<td>Importance factor (I)</td>
<td>1.5</td>
</tr>
<tr>
<td>Response reduction factor (R)</td>
<td>3</td>
</tr>
<tr>
<td>Soil type</td>
<td>medium</td>
</tr>
</tbody>
</table>

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 1$^{st}$, 2$^{nd}$, and 2.6% for 3$^{rd}$, 4$^{th}$ storey in each direction.

Now $W_d$ for 1$^{st}$, 2$^{nd}$ & 3$^{rd}$, 4$^{th}$ 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.

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

From above equation, stiffness of ground floor & 1$^{st}$ 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 2$^{nd}$ & 3$^{rd}$ 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 & 1$^{st}$ floor, $\sum k = 1.2042E$
• Summation of all wall stiffness in X- direction for 2nd & 3rd floor, $\sum k = 0.7938E$
• summation of all wall stiffness in Y-direction for ground & 1st floor, $\sum k = 1.426E$
• summation of all wall stiffness in Y-direction for 2nd & 3rd floor, $\sum k = 0.9396E$

➤ Center of mass

$$X_m = \frac{\sum W_i * x_i}{\sum W_i} \quad \text{and} \quad Y_m = \frac{\sum W_i * y_i}{\sum W_i}$$

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

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

➤ Center of stiffness

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

$= 9 \text{ m}$
$= 6.14 \text{ m}$

➤ 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

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

$e_x = 0.15 \text{ m}$ and $e_y = 0.07 \text{ m}$

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 $0.15 - 0.05 * 12$

$= 0.825 \text{ m}$ or $-0.45 \text{ m}$

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

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

$= 1.5 * 0.07 + 0.05 * 18$ or $0.07 - 0.05 * 18$

$= 1.005 \text{ m}$ or $-0.83 \text{ m}$

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

➤ Calculation of confined masonry seismic weight

At first floor level:

Wall length, $lw = 118.8 \text{ m}$, $t_w = 350 \text{ mm}$, $h_w = 3.5 \text{ m}$

Weight of wall = $118.8 * 0.350 * 3.6 * 20$ = $2993.76 \text{ kN}$

10% deduction for opening, then weight = $2700 \text{ kN}$

Bond beam = $118.8 * 0.350 * 0.230 * 25$ = $240 \text{ kN}$
Tie column $= 24 \times 3.6 \times 0.350 \times 0.230 \times 25 = 173.88$ kN
Slab $= (0.120 \times 18 \times 12 \times 25) + 2 \times 18 \times 12 = 1080$ 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, $W_3 = 3051$ kN
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**

  $V_b = A_h \times W$

  Where, $A_h = \frac{Z*1+Sa}{2*R+g}$, all parameters are given at starting of problem.

  Fundamental time period, $T_a = 0.09 \times h / (d)^{0.5}$ = 0.30 sec for $X$ direction
  Fundamental time period, $T_a = 0.09 \times h / (d)^{0.5}$ = 0.37 sec for $Y$ direction

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

  $V_b = 12715 \times 0.1 = 1217$ kN in $X$ & $Y$ direction

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

due to torsion, $F_{1-2,t} = (Q_i \times e_{dy} \times Y_i \times \sum K_{xi}) / k_t$

  $= 380.07 \times 1.005 \times (12-6.14) \times 0.7938E / 56.76 E$

  $= 31.30$ 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, \( M_o = P_i * h/2 \) at ground floor level
  
  \[
  = 408.74 + 401.65 + 140.86 + 20.34
  \]
  
  \[
  = 971.59 \text{ kN.m}
  \]

  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 , \( Mr = 1108 + 1015.2 + 902.01 + 451.008 \)
  
  \[
  = 3477.018 \text{ kN.m}
  \]

  \[
  Mr / Mo = 3477.018 / 971 = 3.58 > 1.5 \text{ so ok.}
  \]

- **In plane stability of wall panel**

  1. **Check for compressive stress:**

     \[
     P_{\text{comp}} = K_s * f_{\text{m}}, \text{ where } f_{\text{m}} = 0.422 * f_b^{0.69} * f_{\text{mo}}^{0.252}
     \]

     \[
     = 2.819 \text{ N/mm}^2
     \]

     For considered wall panel, slenderness ratio, \( h/tw = 3600/350 = 10.28 \)

     From table-9 , IS:1905, the value of \( K_s = 0.88 \)

     \[
     P_{\text{comp}} = 2.819 * 0.88 = 2.48 \text{ N/mm}^2 \text{...compressive strength of wall}
     \]

     Now, \( \sigma_{dl} \) = compressive stress due to dead and live load

     Self weight of panel = 300.672 kN

     Slab load = 120.96 kN

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

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

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

     Here, \( P_{\text{comp}} > 0.8684 \text{ N/mm}^2 \), hence panel 1-2 is safe in compression.

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

     Now, \( W_d \geq f_g * w * n_s / P_{\text{comp}} \)

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

     \[
     P_{\text{comp}} \text{ required} = 2.33*0.0195 * 4 / 0.078 = 2.091 < 2.48 \text{ N/mm}^2 \)…in Y direction
So, safe in compression in both direction

3. **Check for tensile stress:**
   \[
   \sigma_t = \frac{M}{S} - \sigma_{dl} = (326.84 \times 10^6 / 75.6 \times 10^6) - 0.334
   \]
   \[
   = 0.098 \text{ N/mm}^2 < 0.25 \text{ N/mm}^2 \quad \text{...safe in tension}
   \]

4. **Check for shear stress:**
   Permissible shear stress is given as,
   \[
   t_u = \frac{0.1 + \sigma_d}{6}
   \]
   where \( \sigma_d \) is compressive stress due to dead load
   \[
   = 0.148 \text{ N/mm}^2 \quad \text{...shear strength}
   \]
   Actual shear stress acting on the panel, \( 1-2 = P/A_w = 0.144 \text{ N/mm}^2 < 0.148 \)...so safe

5. **Shear strength Check from wall density consideration:**
   \[
   W_d \geq A_h \cdot f_s \cdot w \cdot n_s / t_u,
   \]
   so that \( t_u \) min = \( 0.1 \times 1.3 \times 0.0195 \times 4 / 0.067 \) in x direction
   \[
   = 0.135 < 0.148 \text{ N/mm}^2 \quad \text{so safe in shear.}
   \]
   t_u min = \( 0.1 \times 1.3 \times 0.0195 \times 4 / 0.078 \) in Y direction
   \[
   = 0.116 \text{ N/mm}^2 < 0.148 \quad \text{so safe in shear}
   \]

- **Out of plane stability of wall panel**
  - Seismic weight per unit area of wall panel, \( F = A_h \cdot g_m \cdot t_w = 0.7 \text{ kN/m}^2 \)
  - Ultimate bending moment per unit length of wall, \( M_u = F \times h / 8 = 0.7 \times 3.6^2 / 8 = 1.134 \text{ kN.m/m} \)
  - Bending stress, \( \sigma_b = M_u / S = (1.134 \times 1000) \times 6 / 350^2 = 0.055 \text{ N/mm}^2 \)
  - Actual stress, \( \sigma_p = \sigma_b - \sigma_{dl} = 0.055 - 0.334 = -0.279 \text{ N/mm} \)
  - 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,
    \[
    A_{st} = P / f_y
    \]
  - At fourth floor,
    For bond beam of wall panel 1-2, \( A_{st} = 56.77 \times 1000 / 415 = 136 \text{ mm}^2 \)
    \[
    A_{st} \text{ min} = 0.85 \times 230 \times 230 / 415 = 108 \text{ mm}^2
    \]
    So provided \( A_{st} > A_{st} \text{ min} \)...so ok.
    Therefore, provide 4 bar of 10 mm dia.
    also, \( T_v = V_u / bd = 56770 / 230 \times 230 = 1.073 \text{ N/mm}^2 \)
    so that, \( p_t = 100 \times 314 / 230 \times 230 = 0.593\% \)
from IS: 456-2000, $T_c = 0.51 \text{ N/mm}^2$  
\[ V_{uc} = 0.51 \times 230 \times 230 = 26.97 \text{ kN}, \text{ so that } V_{us} = 56.77 - 26.97 = 29.8 \text{ kN} \]

So, take 6 mm dia bar stirrups, $A_{st} = 2 \times 0.785 \times 36 = 56.52 \text{ mm}^2$  
\[ S_v = 0.87 \times 415 \times 56.52 \times 230 / 29800 = 157 \text{ mm} \]

Or $S_v = 0.75 \times d = 0.75 \times 230 = 172 \text{ mm} \text{ or } 300 \text{ 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 stirrup at 130 c/c

### Design of tie column

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

\[ A_{st} = (1 + 0.25 \times k) \frac{P_i \times h_w}{l_w} \times f_y \]

Where, $k = \text{num of story above above the analysed storey}$

At fourth floor, $A_{st} = (1 + 0.25 \times 0) \times 56.77 \times 3.6 / 3.6 \times 415$

\[ = 136 \text{ mm}^2 \]

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

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

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

At second floor, $A_{st} = 615 \text{ mm}^2$

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

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

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

4. IS 13920 :2016 “ Ductile Design and detailing of Reinforced Concrete structures subjected to Seismic Forces”