Brick masonry infills in seismic design of RC framed buildings: Part 1 — Cost implications

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Five reinforced concrete (RC) framed buildings with brick masonry infills were designed for the same seismic hazard in accordance with the applicable provisions given in Eurocode 8, Nepal Building Code 201 and Indian seismic code (with and without ductile detailing), and the equivalent braced frame method given in the literature. The buildings designed by the Nepal Building Code 201 and the equivalent braced frame method were found to be more economical.

Reinforced concrete (RC) framed buildings with infill walls are usually analysed and designed as bare frames, without considering the strength and stiffness contributions of the infills. However, during earthquakes, these infill walls contribute to the response of the structure and the behaviour of infilled framed buildings is different from that predicted for bare frame structures. Therefore, based on the understanding of the actual response, design provisions need to be developed. Fortunately, a few countries already have codal provisions for seismic design of RC framed buildings with brick masonry infills. The present study evaluates these available provisions with a view to identify design methodologies that exploit the benefits of infills in a rational manner, for improving the contribution of these infills and for reducing the detrimental effects.

Equivalent braced frame method

Significant experimental and analytical research is reported in literature, which attempts to understand the behaviour of infilled frames. Studies show that infill walls decrease inter-storey drifts and increase stiffness and strength of a structure. Ductility of infilled structures, however, is less than that of bare structures. Quality of infill material, workmanship and quality of frame-infill interface significantly affect the behaviour of infilled frames.

Different types of analytical macro-models, based on the physical understanding of the overall behaviour of an infill panel, were developed over the years to mimic the behaviour of infilled frames. The single strut model is the most widely used, though multi-strut models are also sometimes reported to give better results. Of the available models, though the single strut model is the simplest one, it is unable to capture the local effects occurring to the frame members. But, it is evidently the most suitable one for analysis of large structures. Thus, RC frames with unreinforced masonry walls are modelled as equivalent braced frames (EBF) with infill walls replaced by "equivalent struts". The state-of-the-art indicates that the constitutive relation of the strut elements has been developed only for the single strut models. Therefore, currently only single strut idealisation can be used in rigorous non-linear pushover analyses of RC frames with infill walls. Details of these are given in the companion paper[1].

The early versions of this equivalent strut model included a pin-jointed strut with its width taken as one-third the infill diagonal[2]. This approach, with only the stiffness property of the strut to be the input, found its immediate acceptance in the modelling of infilled frames[3]. Using the theory of "beam on elastic foundation", a non-dimensional parameter was defined as the relative lateral stiffness of the infill. This method was further extended to predict the lateral stiffness and strength of multi-storey infilled frames[4]. Curves, showing the width of diagonal strut, were derived in terms of a relative infill/frame stiffness parameter[5].

Another model for representing the brick infill panel by equivalent diagonal strut was proposed[6]. The strut area, A_e, in^2, was given by the following expression:

$$A_e = w_e t \qquad ...(1)$$

where,

$$w_e = 0.175(\lambda h)^{-0.4} w' \text{ and} \qquad ...(2)$$

$$\lambda = \sqrt[4]{\frac{E_i t \sin(2\theta)}{4 E_f I_c h'}} \qquad ...(3)$$

where,

E_i = the modulus of elasticity of the infill material, ksi

E_f = the modulus of elasticity of the frame material, ksi

I_c = the moment of inertia of colum, in^4

t = the thickness of infill, in

h = the centre line height of frame

h' = the height of infill

w' = the diagonal length of infill panel

θ = the slope of infill diagonal to the horizontal.

A simple and conservative expression of the width of equivalent strut was proposed as[7]:

$$w_e = 0.25 d_m \qquad ...(4)$$

where,

d_m = the length of the infill diagonal.

The infilled frame in this model was idealised as an equivalent diagonally-braced frame with the diagonal compression struts pin-connected to the frame corners.

Codal provisions

Very few design codes have made provisions on RC frames with brick masonry infills. The current focus is to evaluate these available provisions, in that quantitatively assess how they take advantage of the presence of infills and identify the clauses that may need some modifications. Such an effort to evaluate provisions of Eurocode 8 alone in the light of experimental and analytical studies has already begun[8]. Non-linear pushover analyses of plane frames were also performed to study the vulnerability of buildings designed as per BS 8110 and the effect of the masonry infills[9,10]. Some of the codal provisions considering the contribution of the infill walls are discussed here.

Eurocode 8

Eurocode 8 (EC 8) considers brick masonry infilled RC frames as 'dual' systems, which are classified into three ductility classes, namely, high, medium and low[11]. The effect of infills is neglected for low ductility class. When asymmetrical arrangement of the infills causes severe irregularities in plan, three-dimensional models are recommended for analysis. When the irregularity is not so severe in plan, the accidental eccentricity, e_{li}, is increased by a factor of 2, where $e_{li} = \pm 0.05 b_i$ and b_i is the floor dimension perpendicular to the considered direction of the seismic action.

The design seismic action effects, except displacements, of RC frames are modified by a modification factor of $\frac{S_d(T_{ave})}{S_d(T_{bf})}$ where, $S_d(T_{ave})$ = design spectrum ordinate corresponding to the average of the natural period of the infilled and $S_d(T_{bf})$ = that corresponding to the bare frame.

The average value, T_{ave}, of the first mode period of the structure is obtained as:

$$T_{ave} = \frac{T_{bf} + T_{if}}{2} \qquad ...(5)$$

where,

T_{bf} = the first mode period of the bare structure without taking into account any stiffness of the infills

T_{if} = first mode period of bare stucture taking into account the infills as structural elements.

Empirical expressions are provided for the calculation of T_{if}.

The design base shear force, V_B, is calculated using T_{ave} and distributed over the height of the building. The design lateral force, Q_i, at the floor i is obtained as:

$$Q_i = V_B \frac{W_i h_i}{\sum_{j=1}^{N} W_j h_j} \qquad ...(6)$$

where,

W_i = the seismic weight of floor i

h_i = the height of floor i measured from the base

N = the total number of floors in the building (number of levels at which the masses are lumped).

When there is considerable irregularity in the elevation, the code recommends a local increase of seismic effects in the respective storeys. In absence of a precise model, a multiplication factor, α, for estimating the increase in the local seismic effects, is provided as a function of the total reduction ΔV_{RW} of the resistance of the masonry walls in the storey concerned compared to the more infilled storey and the sum $\sum V_{Sd}$ of the seismic shear forces acting on all structural vertical elements in that floor,

$$\alpha = 1 + \frac{\Delta V_{RW}}{\sum V_{Sd}} \qquad ...(7)$$

If α is less than 1.1, this scaling is not required.

Nepal building code 201

One particular section of Nepal National Building Code 201 (NBC 201) provides mandatory rules of thumb, which are meant only for ordinary buildings up to three-storeys in the lowest seismic zone in Nepal[12]. In higher seismic regions, adopting these thumb rules is expected to improve their performance. As per these rules, the building is designed to resist seismic forces by composite action. The design base shear force is calculated for the fundamental natural period of the bare structure and distributed over the height of the building as given by equation (6). At a particular level i, the shear force, V_{ij}, resisted by an individual load-resisting wall, j, is determined by:

$$V_{ij} = \frac{t_{eij}}{\sum_j t_{eij}} \sum_i^{Roof} Q_i \qquad ...(8)$$

where,

$\sum_i^{Roof} Q_i$ = the sum of floor loads above the particular level i

t_{eij} = the effective thickness of the particular lateral load resisting wall j at level i

$\sum_j t_{eij}$ = the sum of the effective thicknesses of the j lateral load resisting walls in level i.

The effective wall thickness, t_{eij}, including plaster is given by:

$$t_{eij} = t_i \left(1 + \frac{t_{pi} E_p}{t_i E_b}\right) \qquad ...(9)$$

where,

t_i = the thickness of the lateral load resisting masonry walls at level i

t_{pi} = the total thickness of the plaster acting with the wall at level i

E_p = the modulus of elasticity of plaster and

E_b = the modulus of elasticity of brick masonry.

If a wall does not resist lateral load, compression strut action is not considered to be formed in the particular panel.

Bare frame analysis and design, without assistance from infill walls, are done for the combined effects of the following loads:

(i) applied gravity loads including the weight of infills, and

(ii) seismic conditions obtained by superposing the effects of two sets of forces, namely:

- frame member forces arising from the horizontal seismic base shear of $0.25C_dW_t$, where C_d is the design seismic coefficient and W_t is the seismic weight (dead load plus 25 percent of live load)
- axial forces in frame members arising from the composite action of frame and walls under a horizontal seismic base shear of $0.9C_dW_t$ these axial forces are obtained by modeling infill wall panels as diagonal struts and by assuming the frame members and diagonal struts to be pin-jointed

(iii) the design shear force in a column abutting a lateral load resisting wall is $\frac{V_{ij}}{2}$, whereas the shear force in the wall is V_{ij}.

Indian seismic code

The Indian seismic code recommends linear elastic analysis of the bare structure excluding the effect of the brick infills[13]. The approximate fundamental natural period of vibration, T, (seconds) of an RC moment-resisting frame (MRF) building with brick infill panels is to be estimated by the empirical expression

$$T_{if} = \frac{0.09h}{\sqrt{d}} \qquad ...(10)$$

where,

h = the total height of the main structure, m

d = the maximum base dimension of the building along the considered direction of seismic force, m.

The code specifies a response reduction factor ($2R$), depending on the perceived seismic damage of the structure, characterised by ductile or brittle deformations. Hence, values

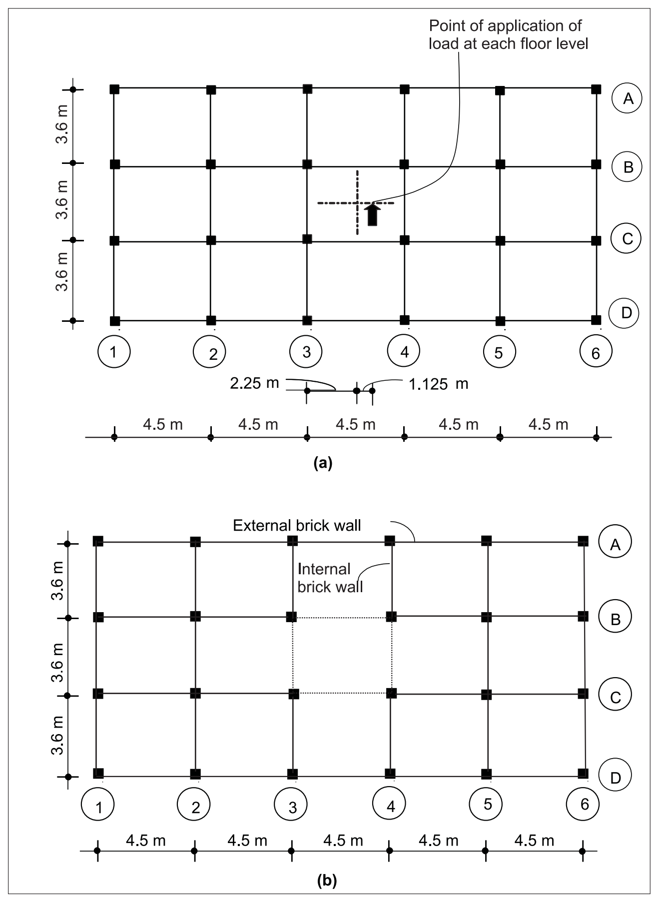

Fig 1 Plan at a typical floor of example building studied *(a)* beams, columns and point of application of pushover forces at floor and roof levels, and *(b)* in fill brick walls

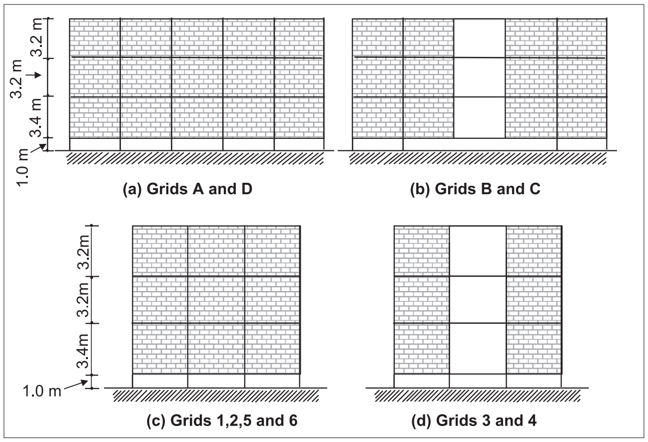

Fig 2 Typical elevations showing arrangements of bricks walls along different frame grids

of 6.0 and 10.0 are suggested for ordinary RC MRFs (those designed and detailed as per the Indian concrete code) and for special RC MRFs (those especially detailed to provide ductile behaviour as per Indian seismic detailing code), respectively[14,15]. The base shear is calculated using the first mode period of the building. To obtain the design seismic force, the elastic force corresponding to the fundamental natural period is then reduced to the actual capacity of the structure with the help of this factor. The calculated design base shear force, V_B, is then distributed over the height of the building. The design lateral force, Q_i, at the floor i is obtained by:

$$Q_i = V_B \frac{W_i h_i^2}{\sum_{j=1}^{N} W_j h_j^2} \qquad ...(11)$$

where,

W_i = the seismic weight of floor i

h_i = the height of floor i measured from the base

N = the total number of floors in the building (number of levels at which the masses are lumped).

Design of example buildings

A typical three-storey residential building, with five bays in the longitudinal direction and three in the transverse direction, is considered, *Fig* 1. The plinth beams, placed 1.0 m above the foundation level, are also modelled in the structure. The wall panel sizes are kept within the limits prescribed by the Indian masonry code for partition walls with adequate restraint at both ends and at the top[16]. The arrangement of brick walls is as shown in *Figs* 1 and 2.

The grade of concrete used is M20 and that of steel is *Fe*415. For concrete, the modulus of elasticity is taken as that recommended by IS 456, that is, $5700\sqrt{f_{ck}}$ MPa where f_{ck} is 28-day characteristic cube strength, MPa. The Poisson's ratio, unit weight and mass density for concrete are taken as 0.2, 25 kN/m^3 and 2.5 kg/m^3, respectively. For masonry, modulus of elasticity and Poisson's ratio are taken as 6,300 MPa and 0.15, respectively[17,18]. The masses of the brick walls are lumped to act at the floor levels. The floor and the roof slabs are taken as 130 mm thick. The external and the internal brick walls are taken to be 230 mm and 115 mm thick, respectively; larger thicknesses than these are provided if required from design considerations. The floor finish on floors and the weathering course on the roof are taken as 1.0 kN/m^2 and 2.25 kN/m^2, respectively. The live load on floors and that on the roof are taken as 2.0 kN/m^2 and 0.75 kN/m^2, respectively.

The following load combinations given in IS 1893 are considered: $1.5(DL + LL)$, $1.2(DL + LL^* + EL_x)$, $1.2(DL + LL^* + EL_y)$, $1.5(DL \pm EL_x)$, $1.5(DL \pm EL_y)$, $0.9DL \pm 1.5EL_x$, and $0.9DL + 1.5EL_y$, where LL^* is 25 percent of the full design live load LL on the floors and is zero on the roof. Also, when the earthquake load is considered, the seismic weight is obtained by considering 25 percent of LL.

The total design base shear, V_B, on the building is calculated as per the IS 1893, and given by $V_B = A_h W$, where W is the seismic weight of the whole building and A_h the design horizontal acceleration spectrum given by

$$A_h = \frac{ZI}{2R}\frac{S_a}{g} \qquad ...(17)$$

where,

Z = the seismic zone factor taken as 0.36 for seismic zone V

I = the importance factor taken as 1.0 for the ordinary residential building

$2R$ = the response reduction factor taken as 6.0 for ordinary RC MRFs detailed as per IS 456 and as 10.0 for special RC MRFs detailed as per the Indian seismic detailing code[14]

$\frac{S_a}{g}$ = the average response acceleration coefficient.

The fundamental natural period, T, (seconds) of the bare and infilled frames are calculated using the empirical expressions given in IS 1893. The lateral seismic forces at each floor are applied at a design eccentricity of $0.05b_i$ where b_i is the floor plan dimension of floor i perpendicular to the direction of lateral seismic force. The structure is discretised into three-dimensional frame elements. The nodes at each floor are constrained by rigid diaphragms. The frame members are designed by the limit state method given in IS 456.

The example building is analysed and designed by the design philosophies given in EC8, NBC201 and applicable provisions in Indian code (with and without ductile detailing) and also by the EBF method given in literature[8]. While doing so, a uniform seismic hazard given by IS 1893 is considered in all the five designs and the design base shear is calculated as per IS 1893. The members are designed as per IS 456 and detailed as per IS 13920. While designing as per NBC201, the shear force resisted by individual load-resisting walls are estimated and checked against the permissible shear strength as per the provisions given in IS 1905. In doing so, the strut

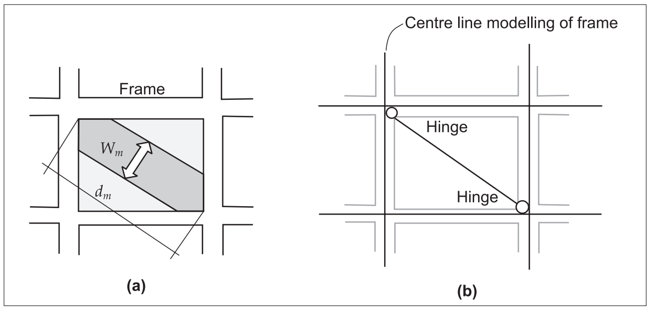

Fig 3 Idealisation of brick in fill panel as equivalent diagonal strut

properties are calculated using equations (1) to (3). Of the two buildings designed as per the Indian code, one is designed and detailed as per IS 456. And the second one is designed as per IS 456 but detailed as per IS 13920.

In the design following the EBF Method, the design base shear is calculated and then distributed over the height of the building as per IS 1893. Elastic linear analysis of the bare structure is done for the load combination involving dead and live loads only. For the other load cases, which include lateral seismic forces acting on the structure, the brick infill panels are considered in the analysis. The RC building is idealised as MRF with the brick infill panels modeled as equivalent diagonal pinned-pinned struts, *Fig* 3. The width of the struts is obtained from equation (4). The axial forces in the struts, obtained from the above analyses, are resolved into vertical and horizontal directions to obtain the vertical compressive force and the horizontal shear force in the wall panels. Before considering the infill walls as structural elements participating in resisting lateral loads, the stress values obtained from the forces mentioned above are checked against the corresponding permissible stresses recommended by IS 1905. The vertical compressive force in the wall is checked against the permissible compressive stress as prescribed in the code. The value of f_m' is taken from the results of brick prism tests conducted during another study as 4.0 MPa[17]. The permissible shear is calculated on the area of the bed joint as per IS 1905 and compared with the corresponding value obtained from analysis. The thicknesses of the walls, which failed in shear, are increased. This resulted in walls that are thicker than those provided in case of the other design methodologies previously described. The exterior walls are 345 mm thick and the interior walls are 230 mm thick. The thickness of the interior wall panels in the ground storey along grids 2 and 5 is also taken as 345 mm from structural consideration.

Concluding remarks

The quantities of concrete and steel used in the structural members of the five buildings studied are shown in *Fig* 4. Concrete quantities required in all buildings are comparable, whereas reinforcing steel required in the buildings designed by NBC 201 and EBF Method are about half of that in the other three buildings. Thus, buildings designed by these methods are economical.

The effect of brick infills on the seismic performance of these buildings needs to be well understood and based on that, design methodologies, which exploit the benefits of infills in a rational manner, need to be developed. The effect of the brick infills on the overall response of these five buildings is presented in a companion paper to understand the implications of the different design procedures[1]. Non-linear pushover analyses are performed on models of buildings designed as per the appropriate provisions in Eurocode 8, Nepal Building Code 201, Indian seismic code (with and without ductile detailing) and the equivalent braced frame method discussed in this paper. The seismic hazard level is kept same for all five buildings as that corresponding to the seismic zone V of the Indian seismic code.

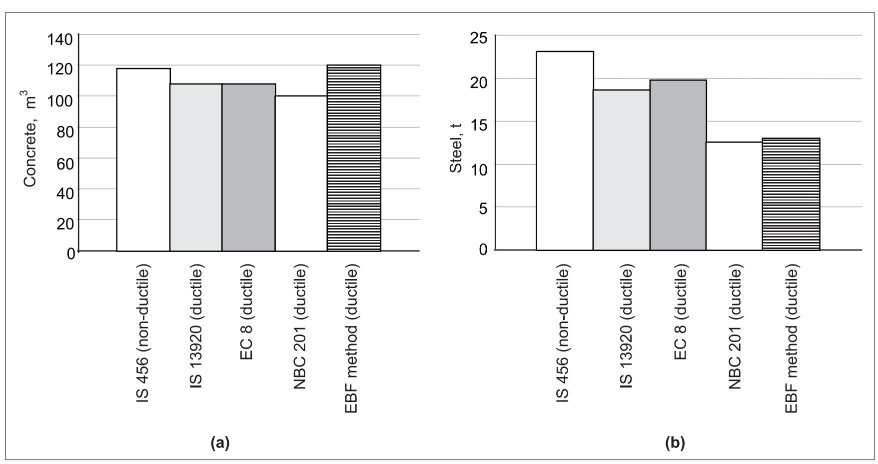

Fig 4 Quantity of concrete and steel in buildings designed by various procedures

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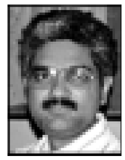

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