Effect of weak and soft storeys on seismic performance of reinforced concrete frames with unreinforced brick infills

Discussion by N. Subramanian

Replies by Patnala V.S. Neelima and R. Pradeep Kumar

This has reference to the paper titled 'Effect of weak and soft storeys on seismic performance of reinforced concrete frames with unreinforced brick infills', authored by Patnala V.S. Neelima and R. Pradeep Kumar, published in The Indian Concrete Journal (Vol. 90, No. 2, pp. 19-26).

Several past earthquakes have demonstrated that the open ground storey is vulnerable during earthquakes and is the cause of total failure of the structure, as shown in Figure 15. From this figure, it is clearly seen that the failure is confined to the columns and beam-column joints in the ground floor (GF) soft storey and all the other elements in the higher stories are unaffected (GF columns are imposed to large deformation and also plastic hinges are formed at top and bottom of the GF columns). In spite of this weakness, this kind of arrangement is still preferred in several of our big cities by architects and developers, since it is convenient to provide covered car park area in the ground floor. Thus the basic requirement of safety is compromised in these structures during earthquake. Hence they should be banned at least in Seismic zones VI and V.

(a) Soft storey failure during the M 7.7 earthquake, Gujarat India, Jan. 26, 2001

(b) Soft storey failure in M 7.4 earthquake, Turkey, Aug. 17,1999

Figure 15. Soft storey failures in past earthquakes

Incidentally, after observing the string of recent earthquakes and volcanic activity in the 'ring of fire' region, especially at Ecuador and Japan, Los Angeles released publically the addresses of 13,500 soft-storey non-ductile apartments and condos, which need to be retrofitted to resist earthquake forces (Los Angeles building officials went through numerous city records and walked block-to-block to identify these structures. Owners of these building were informed individually, and a number of them have already begun the retrofitting process. These retrofits are explained in FEMA 547,2006, and may cost as much as $130,000). Similar initiative was taken by the City of San Francisco in 2013. Such initiatives, to strengthen existing soft-storey non-ductile buildings, should be taken in India also, in order to save human lives and eliminate/reduce property damage during earthquakes.

In order to avoid failures due to soft-storey effect, an ad hoc Clause 7.10.3 was introduced in IS 1893(part 1):2002, without much research. It is interesting to note that this paper under discussion disputes this clause and concludes that the columns designed for 2.5 times the storey shears and moments will not perform satisfactorily during earthquakes.

The authors seem to have designed only the columns in the soft storey for the increased storey shears and moments, whereas Clause 7.10.3(a) of IS 1893(part 1):2002 suggests that columns as well as beams in the soft storey are to be designed for 2.5 times the storey shears and moments.

Replies from the authors

Firstly, the authors are deeply thankful to the reader for going through the published paper thoroughly and rigorously. The authors appreciate the interest shown by the reader in understanding the paper. We put our full effort for responding to the queries.

According to the clause 7.10.3(a) of IS 1893:2002, the design moments in both the columns and beams are to be increased by 2.5 times, but increase in design moments of beams may lead to strong beam and weak column which is not a suggestible solution in earthquake resistant design of RC buildings. Hence according to "Proposed draft provisions and commentary on Indian Seismic Code IS 1893 (Part 1), Document No. IITK-GSDMA-EQ05-V4.0", Clause 7.11.1.2 and commentary C7.11.1.2, the columns are only designed for increased moments.

Also there is a sub-clause (b) which says that *'besides the columns designed and detailed for the calculated storey shears and moments, shear walls should be placed symmetrically on both directions of the building as far away from the centre of the building as feasible, and designed exclusively for 1.5 times the lateral calculated storey shear force"*. Thus, considering Clause 7.10.3(a) of IS 1893(part 1):2002 alone, without considering Clause 7.10.3(b) should be considered as a mistake, and surely will not result in a safe structure.

Reply: The issue raised is true but in practice, there are hardly very few constructed buildings in any city which will allow for the provision of shear walls in open parking area. The analysis is performed keeping in mind the practical issues taking place in site.

The code, however, does not specify the stiffness of columns or shear walls, which will compensate for the loss of stiffness due to the elimination of infill walls in the ground floor. The discusser is of the opinion that we should provide sufficient stiffness, in order to satisfy the storey drift limitation of 0.004 times the storey height as specified in Clause 7.11.1 of IS 1893 (part 1):2002. It is not sure whether the increased column dimensions as per the codal provisions, which were provided by the authors in the reply to the discussions and published in the March 2016 issue of the Journal, satisfied the storey drift limitation specified in Clause 7.11.1 of IS 1893(part 1):2002.

Reply: According to the existing Indian code, Clause 4.20, the definition of soft storey is given as "It is the one in which the lateral stiffness is less than 70 percent of that in the storey above or less than 80 percent of the average lateral stiffness of the three storeys above". Hence the frame with and without soft storey differ by the criteria of stiffness. In any recommendation given on soft storey building in code, the main objective is to match the three parameters of capacity of the building; stiffness, strength and ductility. Hence to compensate the loss of stiffness due to elimination of infill walls, the open ground storey columns are designed for higher design forces and moments. But the stiffness need not provide the storey drift limitation of 0.004 times the storey height as specified in the code. Rather the load carrying capacity and seismic behaviour of the improved building should match or atleast should be comparable with the building with infill walls.

Moreover, in such soft storey GF columns, the confinement reinforcement in the form of shear ties should be provided as per IS 13920:1993, at closer intervals throughout the height of the column (and also in the beam-column joint) and should have 135^{o} hooks (Subramanian, 2011). How this aspect is considered in the analytical model by the authors? It has to be emphasized that earthquake detailing is very important and provides ductility to vulnerable columns and beam-column joints.

Reply: The shear reinforcement provided in the columns is designed according to the requirements following the IS 456:2000. The shear stirrups provided in the column are extended through the beam column joint. As the 135° hook is provided for giving sufficient confinement effect to the member, this effect is modeled numerically by adding the additional stiffness to the elements present near the stirrups.

One more problem is that in the current practice, only the load due to the infill walls are taken in the analysis and the frames (either 2D or 3D) are analyzed as bare frames only. Of course, it may not be economical to do a pushover analysis in design offices. It has to be kept in mind that even the pushover analysis is not very accurate and is based on several assumptions. Tabeshpour, et al. (2012) provides a discussion on the different types of modeling and their issues.

Reply: Non linear static pushover analysis may not be the finest analysis for understanding the seismic behavior of buildings but it is a better alternative over the linear analysis. Moreover, the present work comments on the design provisions of buildings subjected to lateral forces for which

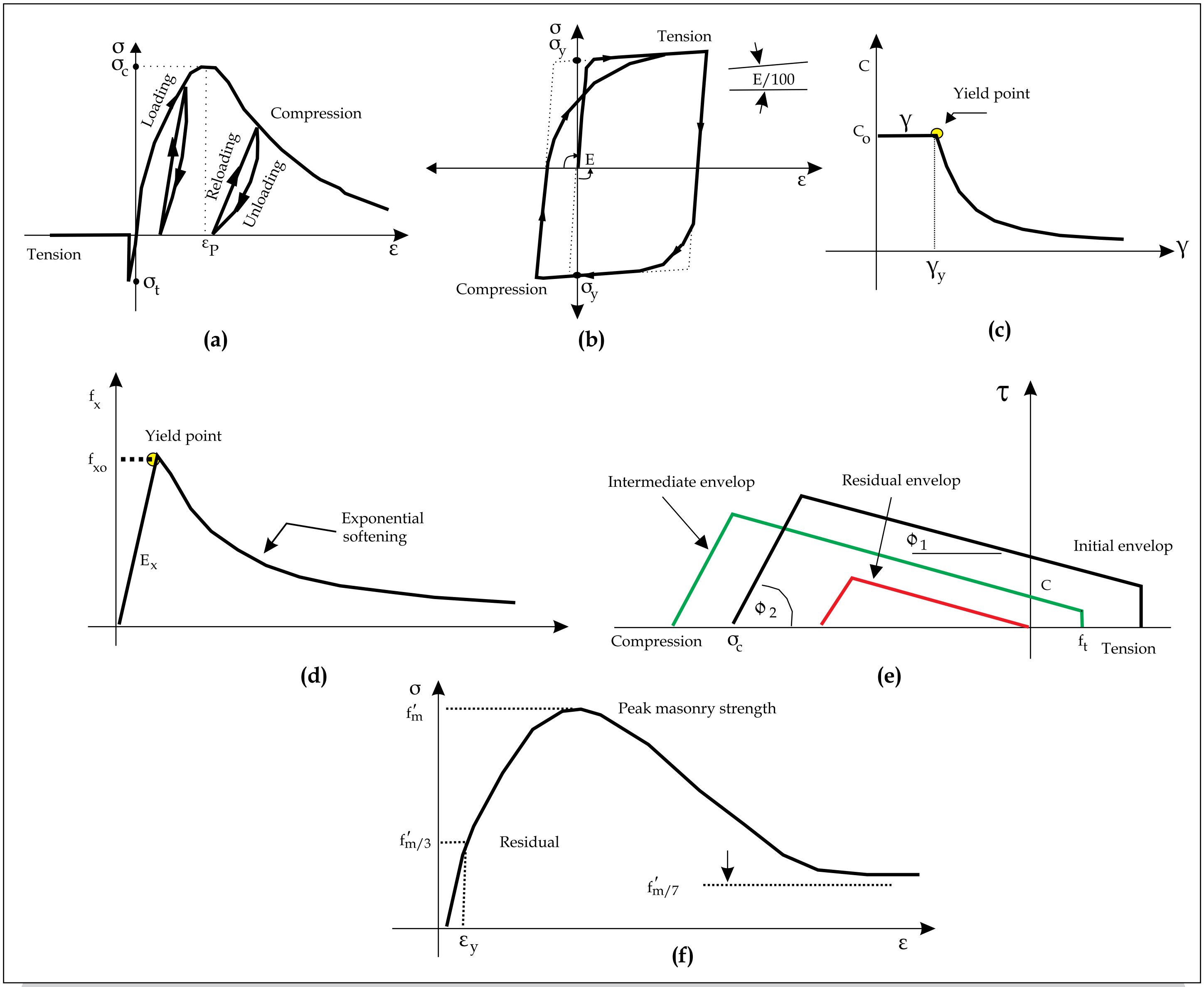

Figure A. (a) Nonlinear stress strain relation for concrete (b) Bilinear stress strain relation for steel (c) Cohesion degradation for brick masonry joints (d) Bond degradation for brick masonry joints (e) Failure envelope for brick masonry joints (f) Hardening and softening model for brick masonry joints the capacity of the building under lateral forces is required which can be obtained by pushover analysis rather from other advanced analyses like nonlinear time history analysis, incremental dynamic analysis etc.

The discusser is not conversant with the Applied Element Method (AEM) used by the authors. However, it may be interesting if the authors explain the interface condition, between the infill brickwork and the concrete frame, considered in the analysis. It is usually better to conduct experimental work to confirm the assumption made in the complex computer analysis.

Reply: For brick masonry, the material model used was a composite model that takes into account the brick and mortar with their respective constitutive relation with elastic and plastic behavior of hardening and softening. Brick springs were assumed to follow principal stress failure criteria with linear elastic behavior. Once the bricks reach elastic limit, normal and shear stresses are not transferred through cracked surface in tensile state. Failure modes that come from joint participation of unit and mortar in high compressive stress is considered by liberalized compression cap. The cohesion and bond values are constant till the stress first time when stress exceeds the respective failure envelopes. Figure 1 (a), (b) shows the material models used for concrete [10] and steel [10], (c), (d), (e) and (f) show the cohesion degradation [1], bond degradation [1], failure envelope [1] and softening model [11] defined at the brick masonry joint. For further details on the method please refer to the research paper in references [5] in the paper under discussion.

Several analytical and experimental investigations have been reported in the literature and show that separation of frame and infill takes place along one diagonal and a compression strut forms along the other and the structural load transfer mechanism is changed from frame action to predominant truss action and the columns will have increased axial forces but with reduced bending moments and shear forces (Murty and Jain, 2000). Hence, it may be easier to find the behaviour of these frames, by substituting the infill wall by diagonal struts. Again, several formulae are available for idealizing these brick struts. One such idealization is given in Clause B-3.3 of the IS 800:2007.

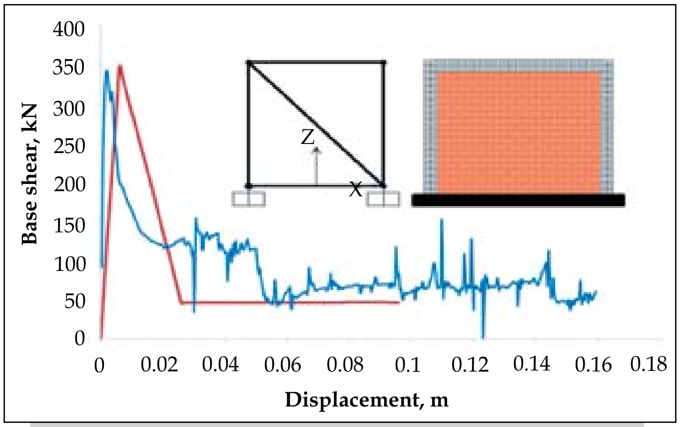

Figure B. Comparison of pushover curves for structures modeled with infill walls using FEM and AEM

Reply: As the finite element method is based on the assumption that the material is continuum, limitations exist in representation of cracks and separation between elements. The finite element strut modeling assumes the infill wall as a single strut by which only few parameters responsible for the seismic behavior can be assessed. Figure 2 show the comparison between the strut modeling and the AEM modeling.

As discussed by the undersigned in the discussion of an earlier paper published in ICJ on a similar topic, the discusser listed some practical problems while including the effect of infill brickwork in the analysis (Subramanian, 2013a). A few of them are:

Location and size of openings will affect the effect of the infill : Most of the internal partition walls in multi-storey buildings are half – brick thick (115 mm thickness) walls. They cannot be assumed to provide any stiffness, as they will fail immediately during an earthquake. During the 2001 Bhuj earthquake (Gujarat, India), even banded half brick walls collapsed and caused huge loss of lives in residential flats. A reliable method to strengthen these walls, so that they will perform better during earthquakes, has not been found yet. Although resulting in increased cost, gunited ferrocement, on both sides of the wall, may prevent these walls from breaking up. Even in the outer walls, which have 230 mm thickness, window and door openings, will weaken the walls, unless RC bands are provided. Even if the opening are properly modelled and incorporated, it is impossible to anticipate the changes that may be occur during the life time of the structure. In fact, the openings in brick walls, if not properly placed, will even result in short column effect and affect the performance of RC columns during earthquakes (Subramanian, 2013b).

Reply: As the present study is carried out in 2d analysis, the out of plane behavior of the infill wall cannot be simulated. Hence 3d analysis of the same study has to be carried. As suggested by the discusser, the effect of opening on seismic behavior of RC frames with masonry infill walls will be dealt in the future work.

Dynamic load and inelastic behaviour: It is well known that behaviour of the infill is brittle in nature and once it cracks, the frame stiffness will drop drastically. Also, during the first few cycles of earthquake, the infill brick wall will collapse completely (in fact many injuries are attributed to the flying debris of brick walls).

Reply: Same as the previous reply to above query.

Infills may result in non-ductile performance: The increased stiffness of the building due to the presence of infills may reduce the ability of the frame to flex and deform. In ductile RC frames, masonry infills may prevent the primary frame elements (i.e., columns and beams) from responding in a ductile manner - instead, such structures may show a non-ductile (brittle) performance. This may culminate with a sudden and dramatic failure (Murty et al, 2006)

Reply: Though the discussers view is valid, infill walls are the better elements for dissipation of energy input to the buildings. Hence the seismic performance of brick infill walls have to clearly understood in relation with the RC frames.

Infill material property variation: There are many infill material available these days - clay bricks, flyash bricks, solid concrete blocks, hollow concrete blocks, Aerated Concrete (AAC) Blocks, Cellular Lightweight Concrete (CLC) blocks, Perforated Clay Blocks, Compressed Stabilized Earth blocks, etc. Each has different material properties. Considerable research is required to identify their behaviour under cyclic loading from earthquake and also the effects of openings in such walls.

Reply: As suggested by the discusser, the effect of infill material property variation on seismic behavior of RC frames with masonry infill walls will be dealt in the future work.

Quality of material and workmanship: Some of the blocks such as AAC and CLC are manufactured in the factory and have better quality control than others like clay bricks and flyash bricks. Very poor quality bricks are found to be the cause of the failure of infilled walls on several occasions. Similarly, workmanship may also vary considerably, resulting in weak joints, susceptible for cracks and ultimate collapse.

Reply: As workmanship cannot be modeled, this is the limitation of the numerical modeling

Richer mortar has to be used: Richer cement-sand mortar of 1: 4 mixture (1 part cement by 4 parts of sand) makes the masonry stronger against earthquake shaking as compared with the usual 1:6 mortar used in such construction, by a factor of 2.5 to 3.0

Reply: As suggested by the discusser, the effect of mortar richness on seismic behavior of RC frames with masonry infill walls will be dealt in the future work.

Some more additional queries on the paper are:

Their analysis shows that there is huge increase in load carrying capacity of the structure, if the structure is having

(a) (b)

Figure 16. Examples of pan caking failure of buildings with soft storey at different levels

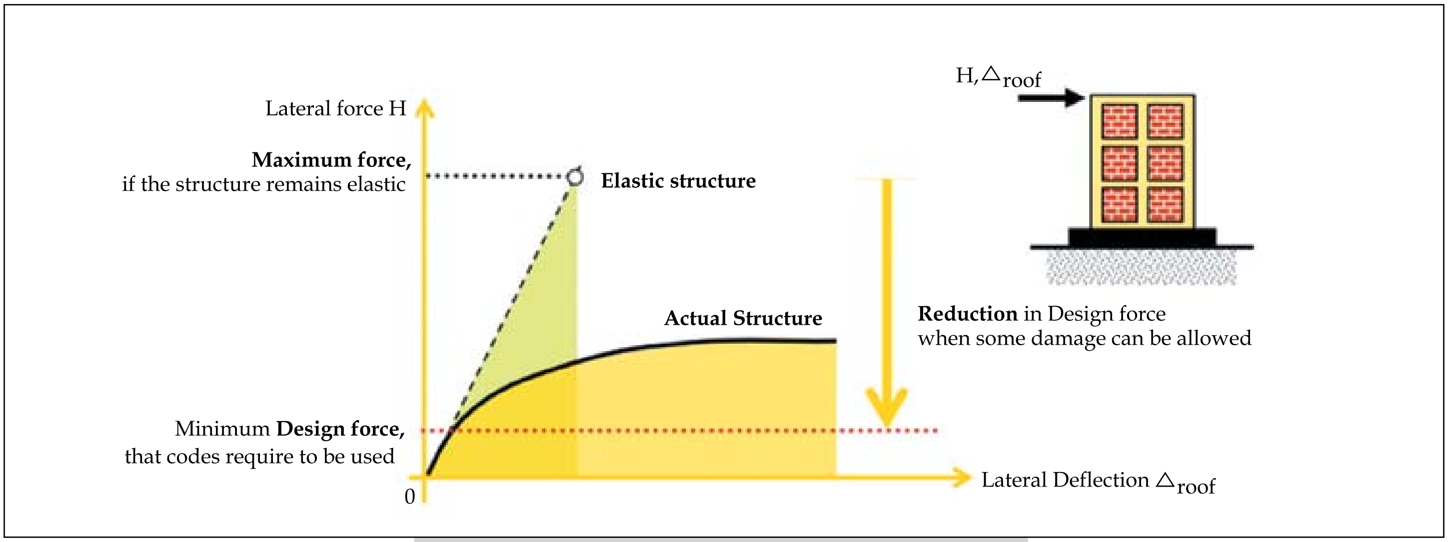

Figure C. Basic strategy of earthquake design: Calculate maximum elastic force and reduce by a factor to obtain design forces (Ref: "Some Concepts in Earthquake Behavior of Buildings", by C V R Murty at al, GSDMA, Govt. of Gujarat)

weak storey in the first floor, when compared to the ground floor. However, in actual earthquakes, weak stories present in the structure, irrespective of the level, is considered to cause pan caking failure (see Figure 16).

Reply: Definitely, the load carrying capacity of the building with weak storey at first floor is more when compared to the building with weak storey at the ground floor. The comparison is relative. Moreover, the presence of infill gives a higher load carrying capacity only in the initial stages of loading but very soon the load carrying capacity decreases suddenly. This can be seen in any of the pushover curves for buildings with infill walls. The interpretation drawn from the analyses carried out is that with the change in weak storey from ground floor to the top floor, the likelihood of the collapse of the structure or the concentrated failure of one floor (i.e., pan cake collapse) can be decreased.

In their analysis have they considered reduced stiffness due to cracked column and beam section properties, which will affect the results considerably (see Section 10.10.4 of ACI 318-11)?

Reply: Yes. As the Applied Element Method (AEM) is continuum modeling, the reduction in stiffness due to failure of each and every finite element is considered in the analysis and the new stiffness is used in every step for finding the load carrying capacity. AEM uses distributed plasticity model which is different from concentrated plasticity model used in FEM.

Which formula was used to calculate the fundamental natural period of the structure?

Reply: According to IS 1893:2002, the formula provided for RC buildings with infill walls is used.

In Table 2, they have shown that the maximum base shear for the structure with soft storey has a maximum base shear of only 160 kN, as compared to 4673 kN for the structure without soft storey. Similarly, the modified structure considering 2.5 times the shear and moments, as per Clause 7.10.3(a) of IS 1893(part 1):2002, has a maximum base shear of only 578 kN. Design seismic base shear is dependent on A_h (design horizontal acceleration spectrum value) and W (seismic weight of the building) only. Will the removal of infill wall only in the GF, result in so much reduction in the design seismic base shear?

Reply: According to the code, the formula for base shear represents the design base shear but not the maximum base shear. Moreover, the design base shear given in code is obtained from linear analysis. The nonlinearity of the building is represented by a factor 'R' response reduction factor. Whereas the maximum base shear of the building is obtained from non linear static pushover analysis. This can be clearly illustrated in Figure 3.

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