Discussion Forum

Seismic performance of conventional multi-storey building with open ground floors for vehicular parking

The discusser would like to congratulate the authors — Mr Ravi and Vasant Kanitkar — for throwing light on a very practical topic, which is very important to the design of multi-storey buildings of our country[1]. The draft code of IS 1893, which was under circulation for more than 2 to 3 years, did not contain clause 7.10. This clause was introduced in the final code only after witnessing the failure of several multi-storey buildings during the 2001 Bhuj earthquake. To my knowledge the factor of 2.5 specified in clause 7.10.3(a) and the factor of 1.5 specified in clause 7.10.3(b) of the code were chosen arbitrarily without any scientific investigations. Hence this paper, which compares the base shears based on this approach given by the code with the results of an inelastic analysis, assumes greater importance.

Still, we have not fully understood the behaviour of masonry infilled concrete frames subjected to earthquake forces. The failure of such buildings with soft storey at the ground floor level may be due to the following reasons/practices adopted in India.

(i) Due to the stiffening effect of masonry in the top floors, more than 90 percent of the deflections occur in the ground floor columns (shear type deformation, as rightly pointed out by the authors), which result in the formation of plastic hinges at the base and top portion of the ground floor columns and subsequent failure.

(ii) Normally, the length of ground floor columns will be larger than the other floor columns due to the raised ground level. Many builders either do not provide plinth beams or design and detail the plinth beam properly. (Incidentally, the ACI code suggests that grade beams designed to act as horizontal ties between pile caps or footings shall have a depth equal to or greater than the span / 20, but need not be greater than 450 mm. It also suggests that closed ties shall be provided at a spacing not to exceed the lesser of one half the smallest orthogonal cross – sectional dimension or 300 mm) [2]. In most of the cases the ground floor columns have to be designed as long columns but often they are designed as short columns only.

(iii) The amount of lateral ties in the columns is usually inadequate in diameter and spacing and more often not provided with 135^{o} hooks and not detailed near the joints for earthquake forces.

(iv) In many cases, the beam-column joint reinforcements are not provided, even if specified by the designers, due to the difficulty of providing them[2]. Also as indicated by the authors, while providing concrete for the beams, slabs and column tops (especially when the depth of beams are different) together, weak joints are formed at locations of maximum seismic demands.

(v) Many designers do not carry out three-dimensional analysis of the frames and design even the corner columns for uni-axial bending only.

(vi) Even though ready-mixed concrete may be used for slabs and beams, more often site mixed (without proper quality control) concrete is used for column, due to the small quantity of concrete involved. On

many occasions, concrete is not vibrated properly, resulting in honey combing of concrete. In several sites, there is no control over the water-cement ratio (many do not even use designed mixes), which has profound effect on the strength and durability of concrete.

(vii) Most of the architects do not have knowledge about earthquake-resistant design and hence often plan the buildings, which are unsymmetrical or have non-uniform mass and stiffness distribution either in plan or in elevation. Due to their planning and insistence, many design engineers provide floating columns and concealed beams. Floating columns and concealed beams should not be allowed, since they will weaken any building. Especially the floating columns at the tip of cantilever beams as indicated in Fig 2 should be avoided at any cost.

Unless these practices are corrected (especially in numerous small projects handled by small companies) buildings will continue to have failures, especially during earthquakes.

The authors suggest that for multi-storey buildings with stilts, one can adopt R=2.5 with earthquake detailing for the frames and omit the scale factor of 2.5 specified in the code for moments and shears. By adopting this approach, the bending moments, axial forces and shear forces will be increased uniformly in all the floors. But as per the behaviour illustrated in *Figs* 1 and 6 of the paper, the columns in the stilt floor are affected greatly and fail in the event of an earthquake. Also, by adopting R=2.5, the cost of the structure will increase considerably, since it will affect the design of columns and beams in all the floors.

Several researchers have conducted analytical and experimental investigations on infilled frames under static and lateral loads [4,5,6] and suggested that one can adopt a reduced bending moment of $wl^2/50$ (which is also suggested in IS:2911 for the design of grade beams supporting masonry) for the design of beams due to the arching action of masonry. But in normal practice, only the load due to masonry is considered, and do not consider the composite action is ignored. Can we consider this action and reduce the bending moments in beams supporting the masonry infills? What will be the effect of openings (windows and doors)?

As suggested by the authors, it is practically difficult to isolate the behaviour of masonry from the frames, unless the precast facades are directly attached, as done in many developed countries. But since masonry is brittle, the infill will fail after a few cycles. Hence the frame will behave as an infilled frame for a few cycles only and after that there will not be any infill effect. Have they modelled the behaviour accordingly? Of course, the best practice will be to have reinforced infill, and include its behaviour in the analysis and design so that one can achieve safety and economy.

It is believed that in tall multi-storey buildings (beyond 15-20 floors), the wind load will govern the design more than the earthquake load. In these cases also do the stilt floor columns need to be strengthened considering 2.5 times the earthquake shear?

It will be interesting if the authors made a comparison of the provisions of codes of other countries with this particular provision in the Indian code (clause 7.10 of IS:1893).

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References

1. Kanitkar, Ravi and Kanitkar, Vasant Seismic performance of conventional multi-storey building with open ground floors for vehicular parking, *The Indian Concrete Journal*, February 2004, Vol 78, No 2, pp. 9-104.
2. ______*Building code requirements for structural concrete*, ACI 318 M-02, American Concrete Institute, Farmington Hills, Michigan, USA, 2002.
3. Subramanian, N. and Prakash Rao, D.S. Seismic design of joints in RC structures, A review, *The Indian Concrete Journal*, February 2003, Vol 77, No 2, pp. 883-892.
4. Govindan,P. and Santhakumar, A.R. Composite action of reinforced concrete beams with plain masonry infills, *The Indian Concrete Journal*, August 1985, Vol 59, No 8, pp. 204-208, 222.
5. Govindan,P., Lakshmipathy, M. and Santhakumar, A.R. Behaviour of infilled beams under cyclic loading, *The Indian Concrete Journal*, May 1987, Vol 61, No 3, pp. 124-128.
6. Achintya, Dayaratnam, P. and Jain.S.K., Behaviour of brick infilled RC frame under lateral load, *The Indian Concrete Journal*, Vol 65, No 9, September 1991, pp. 453-457.

The author's reply:

The authors wish to express their appreciation to the discusser for writing an illuminating discussion on our paper. The inclusion of the factors in section 7.10.3 in IS 1893 : 2002, although arbitrary to some extent and obviously based on a last-minute decision, was definitely a step in the right direction towards addressing the collapses seen in the Bhuj earthquake. We concur with the discusser that at this stage we have not fully understood the behaviour of masonry infilled concrete frames subjected to seismic forces.

Since writing the paper, we have been actively researching building codes from the USA and Europe to better understand their approach to such structures. We hope to broaden this survey to codes applicable in areas such as Taiwan and New Zealand. Our initial research indicates an on-going discussion regarding masonry infilled concrete frames, at least within the European region. We hope to explore this issue further and we fully intend to write a companion paper within the near future.

The discusser has raised the issue of column failures within the stilt floor, even with the use of a lower *R* factor. Our intent was to use the lower *R* factor in conjunction with explicit modelling of the effect of the infill. Simplistically, this could be done by using wall elements or even cross-bracing to model the rigidity of the infill. Although very rudimentary (since infill strength is not always easily quantifiable), such an approach would be applicable within the elastic range of frame and infill performance and could be used effectively to guide the design of stilt buildings. Such an analysis would reduce inter-storey displacements at the upper floors and concentrate a majority of the displacement and rotation demands within the stilt floor columns. Thus, the seismic force distribution would not be uniform over the height of the building. We believe that such an analysis will not lead to over-designing of the upper floor members, since the inter-storey displacements would be very small at these upper floors (an inherent effect of infill). The use of the lower *R* factor is intended to address the poor ductility, redundancy and energy dissipation characteristics of the stilt type buildings (as shown in *Figs* 6 and 7 of the paper), in keeping with the spirit of the code.

In our non-linear analysis effort we explicitly modelled the infill within the concrete frames. However, we did not see any significant failure within the masonry prior to inelastic action within the frame (concentrated at the stilt level). If the strength of the frames is increased, we may see the brittle infill show some signs of failure. We have not performed such parametric analyses yet. The main issue with such parametric studies would be to gauge the strength and workmanship of the infill. It is anticipated that infill strength will have a significant effect on the performance of infilled concrete frames. In the Indian context, the variation in the construction of the infill (type of brick and mortar, quality of construction, etc) would make it rather difficult to quantify infill strength and energy dissipation properties. If all infill conformed to some standard (such as hollow concrete block with determinable properties), this process would be greatly simplified. Still, we hope to include such parametric studies in our future work.

As far as wind load is concerned, the approach has to be dual. Wind loads are resisted by structures without resorting to inelastic action (yielding, plastic hinges, etc). Codal seismic forces account for such inelastic effects inherently (use of the *R* factor). Thus, multi-storey buildings have to be designed for both wind and seismic effects, with the appropriate governing demands dictating the design at a member level, rather than the global building level.

As mentioned earlier, we hope to compile a comparison of various national and international code provisions for such types of buildings. We definitely intend to submit such work for publication in the *ICJ*.

The discusser has also pointed out several detrimental construction practices commonly employed in India, which if remedied, would go a long way towards making concrete frame buildings more reliable under earthquake forces. We concur with his views and hope for more discussion on the subject.

Lastly, we thank the discusser for his insightful comments on the subject at hand.

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