INFLUENCE OF BUILT-IN STAIRCASES ON THE SEISMIC RESPONSE OF REINFORCED CONCRETE BUILDINGS

Faizan Ul Haq Mir¹ and Durgesh C. Rai²

ABSTRACT

Staircases are essential building components that serve as the primary vertical routes in most buildings and are critical for exit and access in emergency situations. Past earthquakes, in many instances, have demonstrated the poor performance of commonly used built-in staircases wherein local damages, failure of staircase elements and the surrounding RC framing elements have been observed owing to the increase in seismic demands caused by the large lateral stiffness of such staircases. The torsional eccentricities introduced in buildings due to asymmetric location of such staircases further add to their vulnerabilities as suggested by past studies. In this study, two RC frame masonry infilled buildings of IIT Kanpur campus with different placement of built-in staircases in the building plan were selected and their dynamic characteristics were estimated from ambient and forced vibration measurements. The finite element (FE) models of these two structures were calibrated using the field measurements and subsequently investigated for the effects of staircases on the structural response under lateral loads. Further, the codal guidelines suggesting provision of masonry infills or enclosure walls around built-in staircases were investigated and the importance of proper shear design of landing beams and associated elements in cases where enclosure walls are not provided was underscored. Approaches to estimate the required shear demand in such cases were proposed. It was proposed that the stairwell should be conservatively designed for a drift of 0.8 percent or for a drift of $0.07h_b$ where $h_b$ is the depth of the beam in the same bay as the stair.

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Faizan Ul Haq Mir\textsuperscript{1} and Durgesh C. Rai\textsuperscript{2}

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Staircases are essential building components that serve as the primary vertical routes in most buildings and are critical for exit in emergency situations. Past earthquakes, in many instances, have demonstrated the poor performance of commonly used built-in staircases wherein local damages, failure of staircase elements and the surrounding RC framing elements have been observed owing to the increase in seismic demands caused by the large lateral stiffness of such staircases. The torsional eccentricities introduced in buildings due to asymmetric location of such staircases further add to their vulnerabilities as suggested by past studies. In this study, two RC frame masonry infilled buildings of IIT Kanpur campus with different placement of built-in staircases in the building plan were selected and their dynamic characteristics were estimated from ambient and forced vibration measurements. The finite element (FE) models of these two structures were calibrated using the field measurements and subsequently investigated for the effects of staircases on the structural response under lateral loads. Further, the codal guidelines suggesting provision of masonry infills or enclosure walls around built in staircases were investigated and the importance of proper shear design of landing beams and associated elements in cases where enclosure walls are not provided was underscored. Approaches to estimate the required shear demand in such cases were proposed. It was proposed that the stairwell should conservatively be designed for a drift of 0.8 percent or for a drift of $0.07h_b$ where $h_b$ is the depth of the beam in the same bay as the stair.

Introduction

Staircases serve as the primary means of egress in most buildings and are critical for exit in emergency situations particularly earthquakes. For satisfactory performance, a staircase should remain fit for use even after severe or moderate shaking. Past earthquakes in many instances, however, have demonstrated otherwise. Damages have occurred in structures because of the interaction with staircases and in staircases because of inadequate design. On one hand, staircases can introduce undesirable torsional eccentricities in buildings that may lead to the failure of primary structural elements and on the other hand, owing to their inherently high stiffness, they can attract large seismic forces leading to the failure of local elements like landing beams or

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supporting columns. While the codal provisions in India pertaining to staircases present a rather simple approach wherein the effects of built-in staircases on the overall or local behavior are not addressed explicitly, studies in the past have focused and highlighted such interactions and effects but most of them have relied, almost completely, on analytical models of buildings. Thus there is a need to bring out some clarity on whether the approaches given in the standards ensure safety or not. This study aims at revisiting such global effects while relating the analytical models of two RC frame masonry infilled buildings of IIT Kanpur campus with experimental results of the same. Ambient and forced vibration measurements were conducted on the two buildings and their natural frequencies and mode shapes determined for calibration of FE models. Further, the role of infills in reducing the demands on staircase elements is highlighted. While the codal provisions require that built-in staircases be provided with enclosure walls, architectural and functional requirements often entail otherwise. Approaches for estimation of demands in such cases are suggested.

Vibration Surveys for Dynamic Characterization

Two buildings, SBRA Building and Hall-XII Building, on the campus of IIT Kanpur were selected for testing and further analysis. These buildings are seven and six story tall and serve as residential and hostel buildings respectively. For dynamic characterization, ambient and forced vibration surveys of both the two structures were conducted separately. Since the number of available sensors was less than the number of locations where response was to be measured, multiple test setups had to be used in both the cases. Peak Picking procedure was used to identify the natural frequencies at the site and Frequency Domain Decomposition (FDD) [1] was used later for modeshape estimation. Mode shape estimates from different test setups were glued together using the Post Global Estimation Rescaling (PoGER) approach [2]. Owing to the presence of some degree of non-proportional damping in actual structures, complex mode shapes were encountered while analyzing the vibration data. The approach suggested by Niedbal (1984) was used for extraction of real modes [3].

The ambient testing of SBRA building yielded three modes at frequencies of 2.11 Hz, 2.42 Hz and 2.92 Hz while the forced tests yielded similar modes at slightly lower frequencies of 2.05 Hz, 2.37 Hz and 2.82 Hz. The three frequencies correspond to translation along x (Mode A), translation along y (Mode B) and torsion (Mode C). Fig. 1 shows the auto spectral density (ASD) plots for the ambient recordings and the sweep test recordings corresponding to the initial test setup in the SBRA Building. It is worth mentioning here that in the sweep test, due to the eccentric location of the shaker all three modes could be identified from a single run.

Two modes could be identified from the ambient testing of Hall-XII building at frequencies of 2.90 Hz and 3.11 Hz while three modes were identified from the forced tests at 2.81 Hz, 3.05 Hz and 3.42 Hz. The predominant nature of these three modes was identified to be translational along y (Mode A), translation along x (Mode B) and torsion (Mode C) respectively. Fig. 2 shows the ASD plot for the sweep test corresponding to the initial setup for Hall-XII Building. This corresponds to the shaker placed eccentrically along x. It can be seen that all three modes are excited in this configuration thus ruling out any pure translational modes along y. The estimated modeshapes are shown in the next section.

The correlation between the modeshapes estimated from ambient (AVT) and forced tests (FVT) is given in Table 1 along with a summary of the vibration tests. The modeshapes are compared using MAC [4]. On a side note, the fundamental frequencies calculated using the equation given in IS-1893:2002 (Cl. 7.6.2) are also listed. The equation predicts the frequencies reasonably well for the SBRA building with errors less than 8 percent, but no so well for the translational Mode A of Hall-XII building. This may be due to the fact, that in this building, the
columns are invariably placed with their larger section dimension parallel to y-direction leading to a higher frequency in that direction than that predicted by the empirical relation in IS-1893.

![Graph](image1)

(a) SBRA Building: (a) Test setup at terrace (b) ASD plots for ambient recordings (c) ASD plots for sweep test recordings

![Graph](image2)

(a) Hall-XII Building: (a) Test setup at terrace (b) ASD plots for sweep test recordings

| Table 1. Summary of AVT and FVT results |
|-------------------------------|-------------------|-------------------|-------------------|-------------------|-------------------|-------------------|
|                               | SBRA Building     | Hall-XII Building |
| Nature                       | Mode A            | Mode B            | Mode C            | Mode A            | Mode B            | Mode C            |
| AVT                          | 2.11 Hz           | 2.42 Hz           | 2.92 Hz           | 2.90 Hz           | 3.11 Hz           | -                 |
| FVT                          | 2.05 Hz           | 2.37 Hz           | 2.82 Hz           | 2.81 Hz           | 3.05 Hz           | 3.42 Hz           |
| MAC                          | 0.99              | 0.99              | 0.97              | 0.99              | 0.99              | -                 |
| IS 1893                      | 2.20 Hz [7.31]*   | 2.26 Hz [-4.64]*  | -                 | 1.97 Hz [-29.89]* | 3.10 Hz [1.62]*  | -                 |

**Finite Element Modelling and Calibration**

The computer program SAP2000 [CSI], developed by Computers and Structures Inc., Berkeley, which is capable of performing linear, non-linear, static and dynamic analyses, was used to develop the FE models of the two buildings. The three-dimensional mode shapes and frequencies can be identified using eigenvector or Ritz vector analysis. In this study these modal parameters were evaluated using eigenvector analysis. Beams and columns in both the buildings were modelled as two-noded frame elements with six degrees of freedom – three translational and three rotational at each node, based on the center line dimensions. No rigid offsets were used to model the beam-to-beam, column-to-column or beam-to-column connections. Masonry infills were modelled as equivalent diagonal struts with moment releases at both the ends. The single strut model was used
because of its simplicity and less computational requirements [5]. The thickness of the strut elements was taken equal to the actual thickness of the masonry. While several expressions are available for calculating the width of the equivalent diagonal strut, in this study, it was considered as one-fourth the diagonal length of the wall panel [6]. For walls with openings, the width of the strut was reduced using a reduction factor \( \rho_w \) given as:

\[
\rho_w = 1 - 2.6A_r, \quad 0.05 < A_r < 0.4
\]

where \( A_r \) is the ratio of the opening area to that of the infill.

A number of assumptions were made in building the FE models of the buildings. The base in both the structures was assumed to be fixed and soil structure interaction was neglected. The beam-to-column connections were modelled as fixed – that is, it was assumed that all frames are moment resisting. Shear walls around lifts and floor slabs were modelled as shell elements. The diagonal masonry struts were assumed to be weightless inherently. Their weight was calculated and applied as a distributed line load on the supporting beams. Live loads for different floors were distributed over the floor diaphragms.

For concrete members the stiffness as given in SAP2000 for Indian Standard M20 and M30 concrete was used initially for the two buildings respectively. The program uses the following equation for stiffness calculation:

\[
E_c = 5000\sqrt{f_{ck}}
\]

where \( E_c \) is the short term static modulus of elasticity in MPa and \( f_{ck} \) is the specified compressive strength of concrete. The effect of the presence of reinforcement on the overall elastic modulus of the frame members was taken into account using the following equation for calculating the average modulus of reinforced concrete:

\[
E_{rc} = E_e(1 - \rho) + E_s\rho
\]

where \( E_{rc} \), \( E_e \), \( E_s \) and \( \rho \) are the static modulus of reinforced concrete, the elastic modulus of concrete, the elastic modulus of steel and the steel ratio respectively [7]. For masonry struts, the elastic modulus for masonry, \( E_m \) was calculated as

\[
E_m = 550f_{im}'
\]

where \( f_{im}' \) is the compressive prism strength of masonry in MPa. The results of an experimental study were used and the value of \( f_{im}' \) proposed for masonry with mortar having intermediate strength was used [8]. That is, \( f_{im}' = 6.6 \) MPa was used in this study. Fig. 3 shows the standard views of the two FE models in SAP2000. For clarity, the diagonal struts haven’t been shown.

Once the full frame models were built using the discussed approach, modal analyses were carried out. The natural frequencies of the preliminary models were found to be different from those obtained experimentally. Previous studies have shown that natural frequencies depend on the material properties such as modulus of elasticity while the associated mode shapes and mass participation factors are more dependent on geometry and boundary conditions and not as much on material properties [9]. The moduli of elasticity of the various components of the models were thus updated in a manner such that the first mode frequencies of the models matched those obtained from the vibration surveys. While the frequencies obtained from forced and ambient tests were slightly different, the values from the forced tests were used owing to the fact that there is lack of control on the actual forcing in ambient tests. Thus, all \( E \) values in a model were multiplied by a factor \( \Delta \) defined in Table 2 where \( f_{tar} \) is the target frequency – the experimental first mode frequency and \( f_{old} \) is the first mode frequency as obtained from the modal analysis of a full frame
model. The procedure was continued till a close match was obtained. Table 2 shows the iterations involved in the process.

![Figure 3. FE models in SAP2000: (a) SBRA Building (b) Hall-XII Building](image)

<table>
<thead>
<tr>
<th>Model</th>
<th>Fundamental Frequency</th>
<th>Total % Change in ‘E’</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>AVT</td>
<td>FVT</td>
</tr>
<tr>
<td>SBRA</td>
<td>Initial FE Model</td>
<td>2.11</td>
</tr>
<tr>
<td></td>
<td>Updated FE Model-1</td>
<td></td>
</tr>
<tr>
<td>Hall-XII</td>
<td>Initial FE Model</td>
<td>2.90</td>
</tr>
<tr>
<td></td>
<td>Updated FE Model-1</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Updated FE Model-2</td>
<td></td>
</tr>
</tbody>
</table>

For the FE model of SBRA, an increase of 8% in all the moduli values was needed and for the Hall-XII building a reduction of 24% was needed for calibration with respect to the first mode frequencies. Table 3 shows the comparison of experimental and analytical modes for the two structures after calibration. The FVT result has been used for percentage error calculation in case of frequencies. The mode shapes are compared using MAC. Figs. 4 and 5 show a comparison of the analytical and experimental modes for the two structures.

![Table 3. Comparison of experimental and analytical modes](table)

<table>
<thead>
<tr>
<th>Mode</th>
<th>AVT (Hz)</th>
<th>FVT (Hz)</th>
<th>Analytical (Hz)</th>
<th>% Error</th>
<th>MAC</th>
</tr>
</thead>
<tbody>
<tr>
<td>SBRA Building</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>A</td>
<td>2.11</td>
<td>2.05</td>
<td>2.05</td>
<td>-</td>
<td>0.99</td>
</tr>
<tr>
<td>B</td>
<td>2.42</td>
<td>2.37</td>
<td>2.47</td>
<td>4.22%</td>
<td>0.97</td>
</tr>
<tr>
<td>C</td>
<td>2.92</td>
<td>2.82</td>
<td>2.51</td>
<td>-11.00%</td>
<td>0.94</td>
</tr>
<tr>
<td>Hall-XII Building</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>A</td>
<td>2.90</td>
<td>2.81</td>
<td>2.81</td>
<td>-</td>
<td>0.95</td>
</tr>
<tr>
<td>B</td>
<td>3.11</td>
<td>3.05</td>
<td>3.14</td>
<td>2.95%</td>
<td>0.97</td>
</tr>
<tr>
<td>C</td>
<td>-</td>
<td>3.42</td>
<td>3.38</td>
<td>-1.12%</td>
<td>0.02</td>
</tr>
</tbody>
</table>
Figure 4. SBRA Building - comparison of experimental and analytical modes: (a) Mode A (b) Mode B (c) Mode C

Figure 5. SBRA Building - comparison of experimental and analytical modes: (a) Mode A (b) Mode B (c) Mode C

It is worth noting here that for Mode C in case of Hall-XII building, the experimentally obtained shape in addition to being ‘torsional’ in nature also has a ‘translational component’ along $y$ and hence the MAC value for the modeshape is almost zero. Once the models were calibrated, three different modifications were carried out for both the models and the resulting dynamic characteristics analyzed and compared with those of the calibrated model. In the first instance, the staircases were removed from the calibrated model. In the second one, the staircases were kept in place and infills were removed. And in the third one, both staircases and infills were removed. The third modification corresponds, essentially, to the bare frame case. This way, a comparison can be made between the contribution of staircases and infills to the overall lateral stiffness of these structures. Overall, there are four models for each structure – the calibrated full frame model (CFF), model without stairs – that is stairs removed (SR), models without infills – that is infills removed (IR) and the bare frame model (BF). Tables 4 and 5 show the resulting frequencies for the first three modes obtained from the eigenvalue analysis of the different models of SBRA Building and Hall-XII Building respectively. For the mode corresponding to ‘$Y$-Translation’ for
the IR and BF models of Hall-XII, the MAC value has not been calculated. This is because the obtained modeshapes for these two cases have a twist in the opposite sense.

Table 4. SBRA Building: Comparison of experimental and analytical modes

<table>
<thead>
<tr>
<th>Vibration Description</th>
<th>Exp. Mode</th>
<th>Building Models</th>
<th>FVT Result</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>CFF</td>
<td>SR</td>
</tr>
<tr>
<td>X-Translation</td>
<td>Mode A</td>
<td>2.05 (0.99)*</td>
<td>2.02 (0.99)*</td>
</tr>
<tr>
<td>Y-Translation</td>
<td>Mode B</td>
<td>2.47 (0.97)*</td>
<td>2.46 (0.98)*</td>
</tr>
<tr>
<td>Torsion</td>
<td>Mode C</td>
<td>2.51 (0.94)*</td>
<td>2.50 (0.94)*</td>
</tr>
</tbody>
</table>

Table 5. Hall-XII Building: Comparison of experimental and analytical modes

<table>
<thead>
<tr>
<th>Vibration Description</th>
<th>Exp. Mode</th>
<th>Building Models</th>
<th>FVT Result</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>CFF</td>
<td>SR</td>
</tr>
<tr>
<td>Y-Translation</td>
<td>Mode A</td>
<td>2.81 (0.95)*</td>
<td>2.80 (0.94)*</td>
</tr>
<tr>
<td>X-Translation</td>
<td>Mode B</td>
<td>3.14 (0.97)*</td>
<td>3.13 (0.97)*</td>
</tr>
<tr>
<td>Torsion</td>
<td>Mode C</td>
<td>3.38 (-)</td>
<td>3.36 (-)</td>
</tr>
</tbody>
</table>

*Values in parentheses represent the MAC values

It is observed that in going from CFF to the SR case, there is not much change in the natural frequencies or in the nature or order of occurrence of modes. However, in going from CFF to IR case, there is a significant reduction in the natural frequencies. The order of the fundamental mode is changed in the IR model. The original first and second modes of CFF reverse order in the IR model. Also, in going from IR to BF, there is not much change. This indicates that staircases, for these two structures, do not have much effect on the global dynamic characteristics. In both the structures, the primary sources of lateral stiffness are the columns and the infills. It is worth mentioning here that some of the column sections used in these structures are as large as 300 mm × 750 mm in the case of SBRA Building or as large as 300 mm × 830 mm in case of Hall-XII Building which explains why the relative contribution of a staircase with a 150 mm thick waist slab is so small.

Role of Infills in Reducing Force Demands and Avoiding Common Failure Modes

Non-linear static pushover analyses were conducted on an isolated stairwell for a better understanding of the relative contribution of staircases and infills (along the staircase axes) towards the stiffness and also of the failure mechanisms in the presence or absence of infills. Four cases were considered – a full frame with both infills as well as staircases (FF), a frame with infills but no staircases (SR), a frame with stairs but no infills (IR) and a bare frame (BF) case. (*R* indicates removed.) In the models considered, the infills were modelled as diagonal struts as discussed earlier and for ease in modelling the non-linearity in the staircase slabs, they were also modelled as beam elements.

In SAP2000, the plasticity is lumped at predefined locations on the members using defined hinges. Three different types of hinges were used in the analysis – moment hinges, shear hinges and axial hinges. Moment and axial hinges were defined to be deformation controlled (ductile)
while shear hinges were defined as load controlled (brittle) as they are expected to lose the entire load capacity after reaching the shear strength. For realistic modelling, shear hinges were assigned in the beams at the level of landing and in the inclined beams at locations where the shear from a separate linear analysis was found to be high – that is at the ends and at the center of the landing beam and at the ends of the beam elements used to model the flights. Fig. 6 shows the full frame (FF) model and the hinge locations for various members. For simulating the shear behaviour at the mid of the landing beam, it was modelled using two beam elements, equal in length, with moment and shear hinges at their ends. A load pattern with equal horizontal loads at the upper corner points was chosen for the displacement controlled pushover case.

Beam and column sections – 300 mm × 300 mm were designed using the section designer in SAP2000. All the beams and columns were assigned moment hinges (M-3) as per FEMA356 at their ends. The interaction with axial forces was neglected. Force controlled shear hinges were assigned to the beams and columns in addition to moment hinges. The shear hinges were defined to have a shear capacity \( V_u \) calculated as the summation of the strengths provided by an assumed nominal shear reinforcement \( V_s \) and concrete \( V_c \). That is,

\[
V_u = V_s + V_c
\]

Figure 6. Hinge locations and types for FF Model

Nominal shear reinforcement comprising 2 legged stirrups of 8mm diameter at a spacing of 0.75\( d_e \) (where \( d_e \) is the effective depth of section) was considered and the corresponding shear resistance \( V_s \) was calculated using the following relation:

\[
V_s = 1.25 f_y A_{sv} \frac{d_e}{S_v}
\]

where \( f_y \) and \( A_{sv} \) are the yield strength and area of the shear reinforcement respectively, and \( S_v \) is the spacing of shear reinforcement. This relation is obtained by replacing ‘0.87\( f_y \)’ by ‘1.25\( f_y \)’, for the actual strain hardened concrete, in the relation given in Cl. 40.4 of IS 456:2000. The contribution of concrete towards the shear strength \( V_c \), neglecting axial forces, was calculated as:

\[
V_c = \tau_c b d_e
\]

where \( \tau_c \) is the shear strength of concrete, \( b \) and \( d_e \) are the width and the effective depth of the section. Axial hinges in diagonal struts were defined using the non-linear stress-strain curve for intermediate strength masonry proposed in an experimental study [8]. The behaviour in tension and compression was assumed similar. A comparison of the pushover plots is shown in Fig. 7(a)
wherein it can be observed that in the presence of infills the capacity is increased manifold compared to the BF or the IR and the failure in these cases, marked by a drop in the curves, takes place due to failure of infills. The failure mode in the BF case is gradual - as evident from the flat post-peak curve. This corresponds to yielding of moment hinges. The introduction of the staircase as seen in the IR case leads to a sudden failure – this corresponds to shear failure. The locations at which hinges form in this case for the first six steps of the pushover analysis are shown in Fig. 7(b)-(g). It is worth noting that the shear hinges on the landing beam form in the first step itself. Perhaps for this reason— the shear failure of landing beam in the IR case - IS 4326:1993 recommends providing enclosure walls (infills), at least one brick thick, around built-in staircases in a building. However, owing to architectural or functional requirements, provision of such walls may not be possible always. Approaches to estimate the demands in such cases are presented here.

Approach 1: In the conventional strong column-weak beam philosophy of design, plastic hinges are supposed to form in the beams first. Considering a single story stairwell, loaded along the axes of the stair, the sudden shear failure can be avoided if the moment hinges in the beams precede the failure of the shear hinges in the landing beam, that is, if moment hinge formation at locations shown in green in Fig. 8 precede shear hinge failures at locations shown in red.
the order of $0.035h_b$ where $h_b$ is the depth of the beam [10]. Considering the common case of two hinges forming in beams that run in a direction along the staircase axes the maximum deformation that can occur is around $0.07h_b$. The shear capacity of the landing beams should thus be enough to sustain this amount of drift without failure. The required shear strength can thus be calculated by analyzing the stairwell for an elastic drift of $0.07h_b$.

**Approach 2:** The maximum drift of 0.4 percent as per IS 1893:2002 should be scaled up by a factor of 2. This can be reasoned in two ways – one, that the design of egress structures should be done for the maximum considered earthquake (MCE) and not for the design basis earthquake (DBE) owing to the critical importance of such structures after an earthquake and two, that since it is difficult to provide enough ductility in the landing beams the response reduction factor should be halved. Thus, the design should be done for a drift of 0.8 percent. A similar approach has been suggested for estimation of drift demands in sliding staircases in a past study [11].

The drift values from the two approaches are comparable for common beam sizes and story heights, as is the behaviour in pushover. Results of pushover analysis carried out with capacity of landing beam shear hinges increased and the capacity of other hinges remaining unchanged are shown in Fig. 9(a).

It can be seen that with an increase in the shear capacity of the shear hinges at the landing level, the sudden drops in the pushover curve of the IR case are avoided. The locations at which hinges form in this case for the first six steps of the pushover analysis are shown in Fig. 9(b) –(g). Also, unlike the results shown in Fig. 7, in this case the hinges don’t form in the landing beam even up to the third step. Those that do after the third step, as can be seen in Fig. 9(e), are moment hinges and not shear hinges as verified from the hinge results in SAP2000.

It is worth noting here that the analysis for $0.07h_b$ drift assumes hinge formation till failure only in the beams while in reality, the associated columns also start developing hinges. The formation of such hinges leads to a reduced shear demand in the landing beam. As an example, the initially considered shear capacity of the landing beam in the preceding section was 136 kN. On analysing the IR model for $0.07h_b$ drift at the top, the required shear strength in the landing beam comes to be around 270 kN. For 0.8 percent drift, the value is around 300 kN. However, the results of the analyses considering 270 kN or 300 kN are similar indicating that beyond a point –
which is not known, an increase in the shear capacity does not help in improving the behaviour. This indicates that the analysis for $0.07h_b$ drift or 0.8 percent drift should give an upper bound to the required strength.

**Conclusions**

An FE study on models of two buildings calibrated using results from field vibration tests showed that the primary contributors towards global dynamic characteristics of the two structures were the infills and the moment resisting frames. The staircases were found to have minimal effects on the global dynamic characteristics in both cases. The existing provisions of the Indian codes pertaining to period/natural frequency estimation or design of built-in staircases were found to be adequate. For the unaddressed case of no enclosure walls around the staircases, it was proposed that the stairwell should conservatively be designed for a drift of 0.8 percent or for a drift of $0.07h_b$ where $h_b$ is the depth of the beam in the same bay as the stair.

**References**


