CHAPTER 3

INVESTIGATION OF RC FRAME WITH PARTIAL INFILL
(CONTROL SPECIMEN)

3.1 GENERAL

The results of the experimental investigation carried out on two-bay, two-storey masonry-infilled RC frame with partial infill in the bottom-storey is presented. This frame is the control frame designated as S1BF tested to identify the captive-column effect due to partial masonry infill. The load-displacement response, specimen behaviour, and the crack pattern of the frame S1BF are discussed. Various parameters like lateral deflection, strength, stiffness, ductility, and energy dissipation capacity were considered for study on the behaviour of the frame.

3.2 LOAD-DISPLACEMENT RESPONSE (P-\Delta)

The frame was subjected to lateral cyclic loads in a quasi-static pattern simulating seismic action. The history of sequence of loading for the frame S1BF is shown in Figure 3.1. The load carrying capacity of the specimen was named as ultimate load. The ultimate load of 88 kN was reached in the eighteenth cycle of loading. The load-displacement response of the frame S1BF was recorded as plotted in Figure 3.2. At the ultimate base shear,
the top-storey deflection was found to be 28.5 mm. The displacement due to rigid body rotation of the footing and the foundation block were incorporated in the calculation of net deflection.
3.3 SPECIMEN BEHAVIOUR AND CRACK PATTERN

The detailed behaviour of specimen S1BF is described in the following section. The terms front, centre, and back are used to identify the location of columns with respect to the loading end. The term front refers to the member nearest to the loading jack, while the term back refers to the member farthest from the loading end.

In the control specimen S1BF, structural cracks began to form at a base shear of 40 kN. These cracks started from the tension side of the beam-column joint in the front and central columns of the bottom-storey (Figures 3.3 and 3.4). The bottom-storey columns were made captive at 50 kN and cracks initiated in the front and central column portions adjacent to the top of partial infill. At a base shear of 60 kN, the cracks formed in the top and bottom of the column region adjacent to the gap without infill widened to form flexural hinges and diagonal shear cracks started propagating between them (Figures 3.5 and 3.6). Simultaneously separation of infill took place in the bottom-storey at the leeward end in each bay (Figure 3.7). At 65 kN, a shear crack appeared in the partial masonry infill exactly along the diagonal as shown in Figure 3.8. Figure 3.2 shows that the specimen S1BF reached a maximum lateral displacement of 28.5 mm, which corresponds to a base shear of 88 kN. Additionally, cracks developed in the back column of bottom-storey at the compression end because of diagonal strut action of the partial infill (Figure 3.9). From the failure pattern, it is observed that the captive-column condition was formed at a base shear of 50 kN and it leads to the formation of flexural hinges and shear cracks in the captive columns (front and central columns of bottom-storey). However, no cracks were developed in the columns, beams and in the infill of top-storey (Figure 3.13) clearly depicting
that the frame had failed only by shear failure of bottom-storey columns due to captive-column effect. Various stages of frame S1BF are shown in Figures 3.3 to 3.13. Figure 3.14 shows the crack pattern of the frame S1BF.

Figure 3.3 First Structural Crack in the Front Column at 40 kN

Figure 3.4 Structural Cracks in the Bottom-Storey Columns
Figure 3.5 Flexural Hinge Formation and Propagation of Diagonal Shear Crack in the Front Column

Figure 3.6 Flexural Hinge Formation in the Central Column (Rear View)
Figure 3.7 Separation of Infill at Tension Corners

Figure 3.8 Diagonal Cracks in the Partial Masonry Infill
Figure 3.9 Cracks in the Back Column of Bottom-Storey

Figure 3.10 Front Column of Bottom-Storey at Maximum Lateral Displacement of the Frame (Rear View)

Figure 3.11 Central Column of Bottom-Storey at Maximum Lateral Displacement of the Frame (Rear View)
Figure 3.12 Behaviour of Infill and Front Column at Ultimate Stage

Figure 3.13 Top-Storey without Any Cracks
3.4 LATERAL DEFLECTION, STRENGTH AND STIFFNESS

Response envelope for the specimen S1BF shown in Figure 3.15 was plotted by connecting the peak points of the lateral load-displacement curve. Response envelope curve showed the strength and stiffness characteristics of the specimen and also its general behaviour. The observed ultimate load of frame S1BF was 88 kN and the corresponding maximum lateral displacement was 28.5 mm. Base shear versus LVDT deflection at various levels of the frame S1BF is shown in Figure 3.16. In this Figure, it is observed that after the formation of captive-column condition at a base shear of 50 kN, there is a sudden increase in the deflection of LVDT 3 and LVDT 4. This shows the development of cracks in the top and bottom of the column region adjacent to the gap without infill in the bottom-storey and a sudden rotation of the frame above the partial infill.
The function of the deflection curves corresponding to LVDT1, LVDT2, LVDT3, and LVDT4 placed at various levels of the frame (Figure 2.15) are given by
\[
y = -4.9512x^6 + 67.337x^5 - 347.07x^4 + 839.9x^3 - 950.67x^2 + 434.92x \\
(R^2 = 0.9671)
\]  
(3.1)

\[
y = 0.0076x^5 - 0.2731x^4 + 3.6363x^3 - 22.207x^2 + 64.233x \\
(R^2 = 0.9324)
\]  
(3.2)

\[
y = -0.00007x^6 + 0.0046x^5 - 0.1193x^4 + 1.5555x^3 - 10.72x^2 + 39.77x \\
(R^2 = 0.9608)
\]  
(3.3)

\[
y = -0.000008x^6 + 0.0007x^5 - 0.0251x^4 + 0.45x^3 - 4.4121x^2 + 24.712x \\
(R^2 = 0.9994)
\]  
(3.4)

The relationships established can be utilised for estimating the lateral loads that can be experienced by buildings of similar nature and for estimating the deflections at various levels of such buildings. From the estimated values, the formation of captive-column condition due to partial infill can be assessed. However for obtaining a generalised relationship, studies of similar nature have to be carried out with frames of different heights.

The stiffness of test specimen at each load cycle is shown in Figure 3.17. The stiffness was calculated as the amount of base shear required for causing unit deflection at the top-storey level. The initial stiffness of the
frame S1BF was 25 kN/mm. In Figure 3.17, the stiffness was found to decrease from 25 kN/mm during the second cycle to 3.1 kN/mm during the eighteenth cycle of loading. In addition, it is observed that after the formation of captive-column condition during the tenth cycle at a base shear of 50 kN, the frame exhibited a sudden stiffness degradation resulting in critical shear failure of bottom-storey columns due to captive-column effect. The function of the stiffness curve for load cycles (x-values) ranging from 1 to 18 is given by

\[
y = -0.00005x^6 + 0.0027x^5 - 0.0571x^4 + 0.5513x^3 - 2.4498x^2 + 2.978x + 24.299
\]

\[
(R^2 = 0.9965)
\]

(3.5)
3.5 DUCTILITY

Ductility is defined as the ability of the structure or its components to sustain large inelastic deformations. In this study, displacement ductility factor at each cycle of loading was calculated as the ratio of peak displacement during the corresponding cycle to yield displacement ($\Delta_y$).

In the analysis of nonlinear structures, the force-displacement relationship most frequently adopted is the ‘bilinear model’. This model is typically used for structures or structural elements with a linear force-displacement relationship in both the elastic and the inelastic range. For concrete structures, unfortunately, the force-displacement relationship is not actually bilinear, and certain assumptions have to be made. Since the actual yield point of a concrete frame is hard to define, a method proposed in FEMA-356 (2000) for the estimation of effective stiffness of nonlinear structures was used to obtain the yield point of each frame. Based on this method, the nonlinear force-displacement relationship between base shear and displacement of the test specimen was replaced with an idealised relationship to calculate the effective lateral stiffness, $k_e$ and effective yield strength, $V_y$ of the frame. This relationship shall be bilinear with initial slope of the idealised curve as $k_e$. According to FEMA-356 (2000), the effective yield strength of the specimen was taken as the maximum base shear force along the actual curve. The effective lateral stiffness was calculated at a base shear equal to 60% of the effective yield strength of the structure using the following equation:
The yield displacement, $\Delta_y$, was calculated from

$$\Delta_y = \frac{V_y}{k_e}$$  \hspace{1cm} (3.7)

The calculated effective stiffness and yield displacement of the test specimen S1BF is given in Table 3.1. The variation of displacement ductility factors of the frame S1BF with load cycles is shown in Figure 3.18.

<table>
<thead>
<tr>
<th>Specimen</th>
<th>$V_y$ (kN)</th>
<th>$k_e$ (kN/mm)</th>
<th>$\Delta_y$ (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>S1BF</td>
<td>88</td>
<td>12.3</td>
<td>7.2</td>
</tr>
</tbody>
</table>
Cumulative ductility factor up to any point is the sum of displacement ductility factors attained in each cycle of loading up to the cycle considered. This gives an idea about the overall ductility of the laterally loaded structure. The cumulative ductility factor at the ultimate cycle for the control frame S1BF was found to be 15.56. Figure 3.19 shows the cumulative ductility factors for various load cycles. The function of the cumulative ductility factor curve is

\[ y = -0.00007x^5 + 0.0031x^4 - 0.0402x^3 + 0.2194x^2 - 0.3067x \]

\[ (R^2 = 0.9995) \]  

(3.8)
The energy dissipation was determined by calculating the areas inside the hysteretic load-displacement loops for each cycle. The energy dissipation may be considered as a measure of material damage to the specimen. The cumulative energy dissipated was calculated as the sum of the energy dissipated in consecutive cycles throughout the test. The variation of energy dissipated by the frame during each cycle is shown in Figure 3.20 and the variation of cumulative energy dissipated by the frame is shown in Figure 3.21.
The cumulative energy dissipation values were plotted against the corresponding cumulative displacement ductility factors to study the total energy dissipated by the frame with respect to ductility. The variation of cumulative energy dissipation characteristics of the test specimen S1BF is shown in Figure 3.22. From the graph, it is observed that the frame S1BF dissipated a total energy of 930.96 kN-mm which corresponds to a
displacement ductility of 13.92. The function of the cumulative energy dissipation curve with respect to ductility is

\[ y = -0.0048x^5 + 0.2535x^4 - 3.9754x^3 + 24.294x^2 - 3.3212x \]

\[ (R^2 = 0.9999) \]

(3.9)

![Figure 3.22 Cumulative Energy Dissipation – Ductility Relationship of Frame S1BF](image)

3.7 SUMMARY

This chapter presented the experimental investigations carried out on two-bay, two-storey, masonry-infilled RC frame (control specimen) with partial infill in the bottom-storey. The frame designated as S1BF was subjected to quasi-static cyclic loads simulating seismic action. The load-displacement response, specimen behaviour, crack pattern, and mode of failure
of the frame S1BF were observed. The specimen has failed due to captive-column effect with brittle shear failure in the bottom-storey columns. From the failure pattern, it is now widely recognised that the partially filled masonry infill walls used for fixing openings and ventilators in RC buildings have significantly altered their seismic response. Various parameters like ultimate capacity, lateral deflection, stiffness, ductility, and energy dissipation capacity were calculated to study the behaviour of the frame and to make a comparison with the retrofitted specimens later.