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# Seismic behavior of retrofitted RC frame with prefabricated RC wall panel under cyclic loading

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## ABSTRACT

*The purpose of this study is to investigate the seismic behaviour of both the control frame and retrofitted RC Frame with a prefabricated RC wall panel subjected to lateral cyclic loading. For this purpose, two 1:3 scale, two-dimensional single-bay two-storey RC Frame with precast wall panel is used. The first test frame (control frame) is subjected to lateral cyclic loads and its responses are noted. The second frame is analytically modelled by incorporating the TRM technique. The experimental values are used for calculating the various seismic parameters like load-deflection, maximum load carrying capacity of the frame, stiffness degradation and the ductility and cumulative ductility, energy dissipation and cumulative energy dissipation capacity and the drift ratio for the first frame. Finally, the test results and analytical modelling are compared for two test frames under cyclic loading. The response of the frames under lateral cyclic loading shows the complete behaviours of the RC frame with prefabricated wall panels. The results confirm that the RC wall panel is stiffer in comparison with the RC frame members, resulting in early failure of the framing elements. However, the use of the TRM retrofitting method proves to be effective, which increases the load carrying capacity of the frame with reduced damage to the structure. The experimental results were analogous with the analytically modelled frames.*

***Key Words:*** Prefabricated RC wall panel, retrofitting, cyclic loading, push over analysis, load-deflection, strength, stiffness, analytical modelling.

## 1. INTRODUCTION

The evolution of earthquake engineering in India has started long back after the 1897 Assam earthquake where a new earthquake-resistant type of housing was developed which is still prevalent in northeast India. Since then, several kinds of research have been carried out to understand the dynamic behavior of the buildings subjected to an earthquake. Studying these behavior helps the engineers to design and detail an earthquake-resistant structures.

Prashant Motwani et.al [1] theoretically modelled a G+1 structure using ANSYS software to understand the behavior of brick masonry. During stiffness analysis, it was observed that infill panels increase the stiffness of the bare frame by 2.54 times. The result obtained from the strength analysis showed that the provision of infill frames changes the structural behavior from flexural action into axial action. By stiffening the frame with infill masonry, the natural time period of vibration is decreased due to an increase in weight. The time period for a framed building is 2 to 3 times higher than an infill frame composite building. From the past research, it is evident that the presence of an infill wall greatly influences the seismic behavior of the building. This paved the way for new infill materials and strengthening the existing brick wall with new methodologies for seismic resistance.

N Ganesan et.al [2] experimentally conducted tests to investigate the strength and behavior of reinforced concrete (RC) frames with ferrocement infills. The frame showed better performance in lateral strength, stiffness, energy dissipation capacity and ductility characteristics. The newly built structures following the Indian seismic codes perform better during an earthquake, whereas the existing building suffered severe damages. This led to strengthening and retrofitting of the existing buildings.

C Lakshmi Anuhya et.al [3] studied the effectiveness of the relatively new construction material Textile Reinforced Concrete (TRC), in strengthening the brick masonry prism. Among all strengthening methods mortar bonded TRC laminae exhibited enhancement in terms of compressive strength.

Seung-Ho Choi et.al [4] performed the seismic strengthening using externally anchored precast wall panels (EPCW) and cyclic loading tests were then conducted to examine seismic performances of RC frame specimens. The RC frame specimens strengthened using the EPCWs showed less structural damages in the column members of the existing frame structures with non-seismic details, thus showing a more stable lateral behavior. Strengthening was further improved using prefabricated elements to the existing structure.

In this research work, a reinforced cement concrete frame consisting of one bay and two-storey structure with a reinforced cement concrete precast wall panel is studied for the seismic behavior. The study is carried out on two such RCC frames, where one is wholly on seismic behavior and the other frame is for retrofitted seismic behavior. The retrofitting is done using Textile Reinforced Mortar (TRM). This study incorporates prefabricated wall panels in the place of conventional masonry type of infills to know about its behaviors under cyclic loading. The frames are primarily analyzed and designed to resist earthquake as per IS 1893-2016(Part1). The seismic response of the structure is analyzed based on the Pushover analysis of the frames. Pushover analysis is a static nonlinear procedure using a simplified nonlinear technique to estimate seismic structural deformations. It is an incremental static analysis used to determine the force-displacement relationship or the capacity curve for a structure or the structural element. Finally, the experimental values are compared with the analytical results for both the frames.

## 2. EXPERIMENTAL INVESTIGATION

### 2.1 Materials

Materials were used in this research are Portland Pozzolana Cement (PPC) purchased from Chettinad Cement (Flyash based). The cement used was found to be confirming to various specifications of IS: 1489 (Part 1) 1991. Crushed granite angular aggregate of size 12 mm nominal size as coarse aggregate having specific gravity of 2.71. Manufactured river sand having specific gravity of 2.63 was used as fine aggregate. Locally available potable water confirming to IS 456 was used. Mix design for M20 concrete was done as per IS 10262 [2009]. Special mortar Conbextra GP2 is used for precision grouting where it is essential to withstand static and dynamic loads. It conforms to ASTM C1107. Steel rebar of grade Fe 415 was used in different sizes such as 6,8,10 and 12mm. The retrofitting is done using Textile Reinforced Mortar (TRM).

### 2.2 Details of the frame model

**Table -1:** Dimensions of the frame model

| Details | Size (mm) |
|---|---|
| All floor Columns | 100mm x 150mm x 1000mm |
| All floor Beams | 100mm x 150mm x 1250mm |
| All Wall Panels | 1230mm x 100mm x 980mm |
| Foundation | 1850mm x 150mm x 610mm |

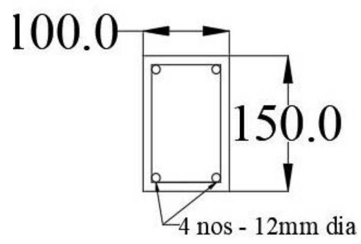


**Fig -1:** Cross-section of column

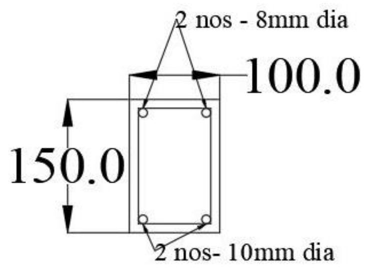


**Fig -2:** Cross-section of beam

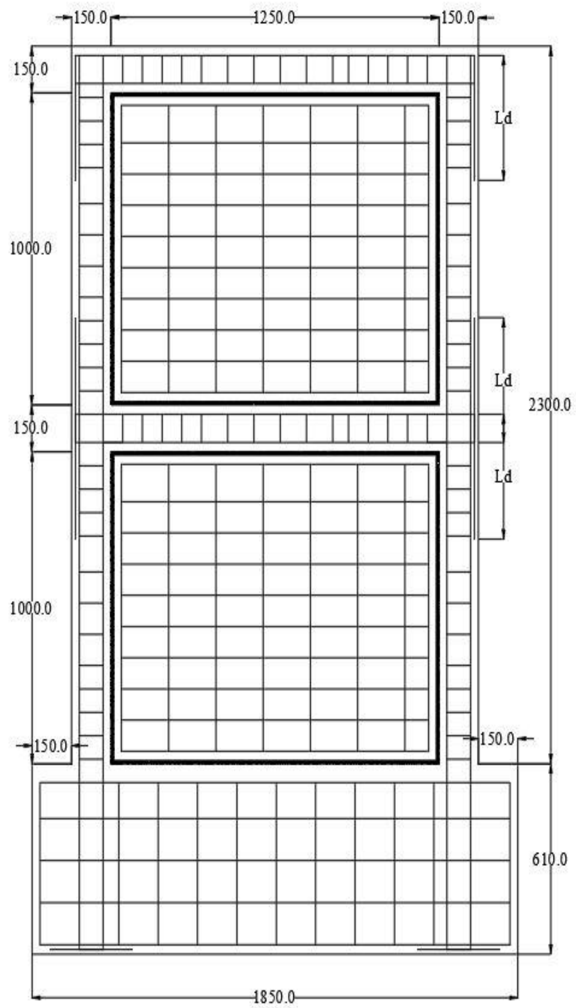


**Fig -3:** Schematic diagram of reinforcement details

**Table -2:** Reinforcement of the frame model

| Details | Flexural reinforcement | Shear reinforcement |
|---|---|---|
| All floor Columns | 4 Nos. of 12 mm ϕ - 2 Nos. on either side. | Ties<br>8 mm ϕ @ 75 mm c/c<br>8 mm ϕ @ 100mm c/c |
| All floor Beams | Top - 2 Nos. of 8 mm ϕ<br>Bottom - 2 Nos. of 10 mm ϕ | Stirrups<br>8 mm ϕ @ 75 mm c/c<br>8 mm ϕ @ 100mm c/c |
| Wall Panel | Top face - 6 mm ϕ @ 100 mm c/c<br>Bottom face - 6 mm ϕ @ 100 mm c/c | Stirrups<br>6 mm ϕ @ 180 mm c/c |
| Foundation | Top face - 10 mm ϕ @ 150 mm c/c<br>Bottom face - 10 mm ϕ @ 150 mm c/c | Stirrups<br>8 mm ϕ @ 150mm c/c |

### 2.3 Test setup

Load cell of 1000KN and 50KN are fixed at the top and bottom storey respectively. Instrument for measuring the deflection (LVDT) is also fixed opposite to the load cell arrangement. Rigid body rotation of foundation block is measured using dial gauges placed on both sides of foundation. Strain measurements are taken using Demec strain gauge. Display units such as LVDT and Load cell units are placed. Hydraulic pumps are used for the application of load.

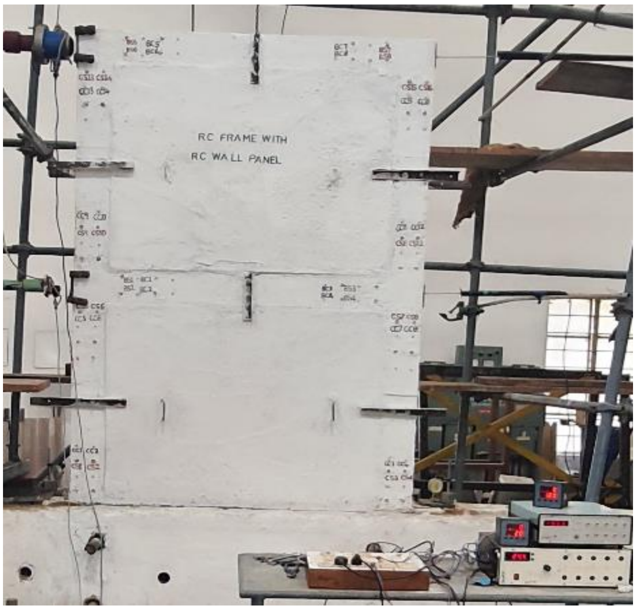


**Fig -4:** Test set up

## 3. EXPERIMENTAL RESULTS

### 3.1 Load-deflection behavior

The RC Frame has been subjected to an experimental investigation (cyclic loading) with an interval of loads. The RC wall panel frame attained its ultimate loading capacity in the (19th cycle) with the maximum load of 90KN and displacement of 156.9mm. Thereafter, the post cyclic loading is carried out in two cycles. At the end of the post cyclic loading, the displacement of the frame reached 166.8mm. The top storey deflection versus applied load is shown in chart -1 and chart -2.

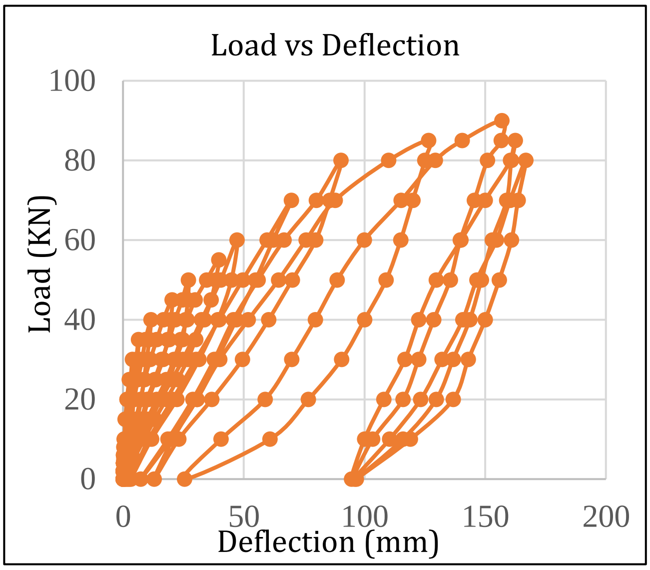


**Chart -1:** Cyclic load vs Deflection

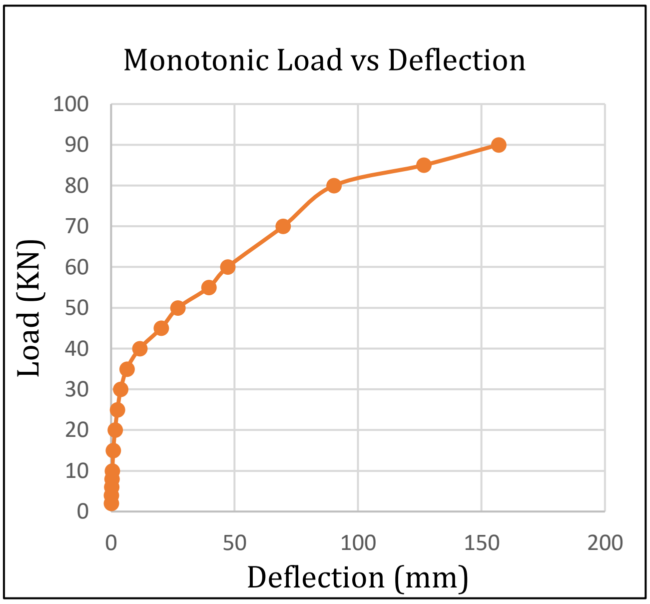


**Chart -2:** Monotonic load vs deflection

### 3.2 Stiffness degradation

The stiffness tends to decrease due to the column crushing, bond failure and energy dissipation. Stiffness degradation occurs from 33.33 to 0.57 KN/mm for about 19 cycles.

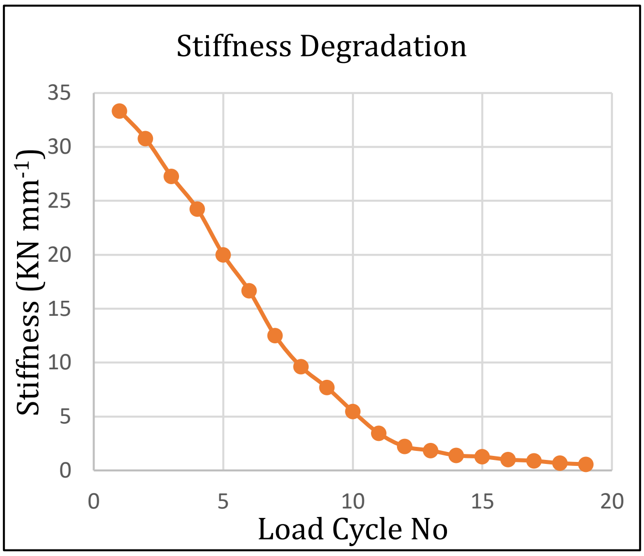


**Chart -3:** Stiffness Degradation

### 3.3 Ductility characteristics

The ductility of the structure is its ability to undergo increasing deformation beyond the initial yield deformation while sustaining load. The ductility is the ratio of the maximum deflection of a cycle to the first yield deflection.

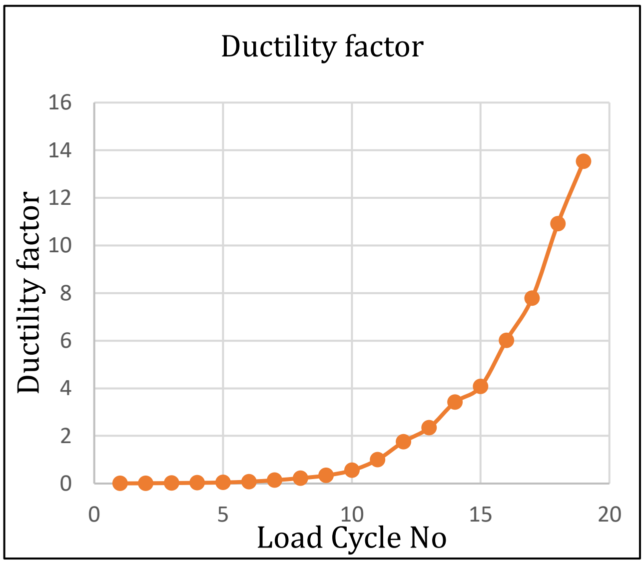


**Chart -4:** Ductility Factor

Ductility Factor ($\mu$) = $\Delta/\Delta y$

Where, $\Delta y$ is the first yield deflection.

$\Delta$ is the maximum deflection.

The variation of ductility factors with load cycles is presented in chart -4. The ductility factor during the first cycle of loading was 0.005 and 13.52 was during the last cycle.

### 3.4 Cumulative ductility

When a structure is subjected to cyclic loading, cumulative ductility up to any load point is defined as the sum of ductility at maximum load level attained in each cycle up to cycle considered. The variation of cumulative ductility with respect to load cycle is presented in the chart-5. The cumulative ductility was found to increase from 0.005 during the first cycle to 52.236 during the last cycle of loading.

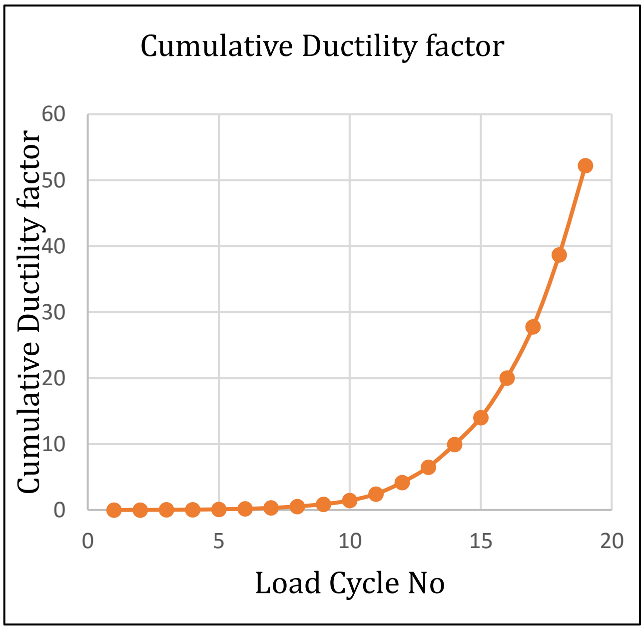


**Chart -5:** Cumulative Ductility factor

### 3.5 Energy dissipation capacity

The energy dissipation capacity of the frame during various load cycles was calculated as the sum of the area under the hysteresis loops from the load versus the top storey deflection diagram obtained. The energy dissipation capacity during the first cycle of loading was 0 KN mm and that during the 19th cycle was 3973 KN mm. The energy dissipation capacity values are calculated for all cycles and the variation of energy dissipated by the frame during each cycle is shown in chart -6.

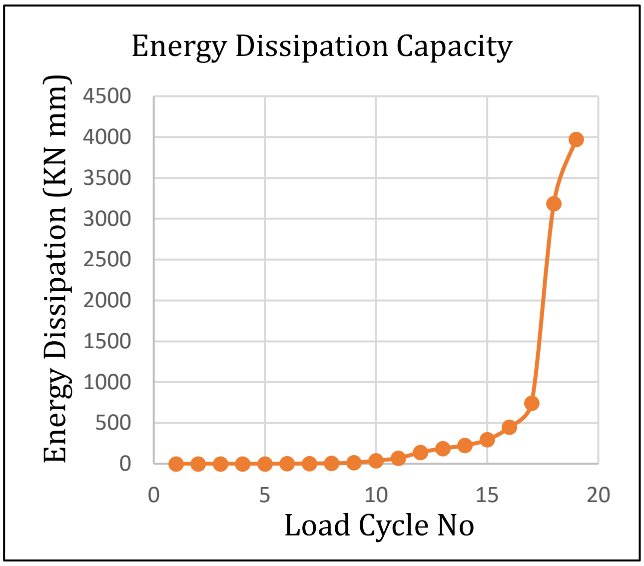


**Chart -6:** Energy Dissipation Capacity

### 3.6 Cumulative energy dissipation capacity

When a structure is subjected to cyclic loading, cumulative energy dissipation capacity up to any load point is defined as the sum of energy dissipation at maximum load level attained in each cycle up to cycle considered. The variation of cumulative energy dissipation with respect to load cycle is presented in chart-7. The cumulative energy dissipation was found to increase from 0 KN mm during the first cycle to 9331.655 KN mm during the last cycle of loading.

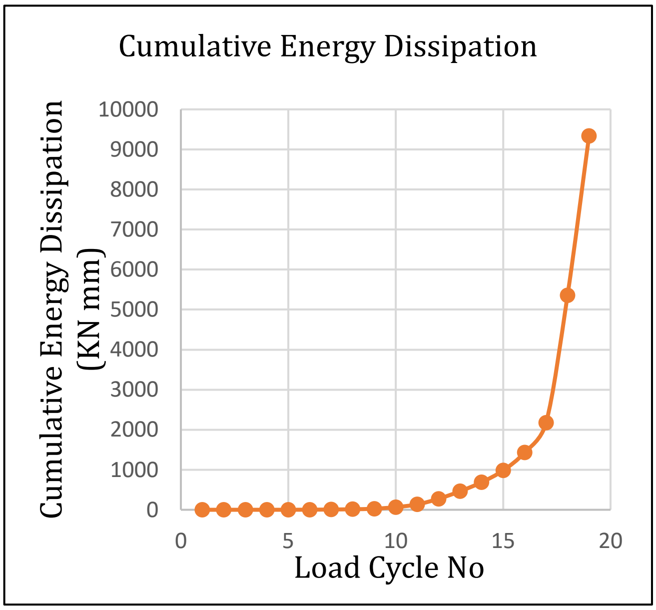


**Chart -7:** Cumulative Energy Dissipation Capacity

### 3.7 Drift ratio

The permissible storey drift is limited to 0.004 times the storey height, so that minimum damage would take place during an earthquake and pose less psychological fear. The drift ratio is the ratio of is the storey displacement divided by the storey height. In this test, the drift ratio of the 1st cycle is 0.003 and it gradually increases to 6.822 at the 19th cycle of loading. The maximum drift ratio is below the prescribed drift limit at the peak cycle loading.

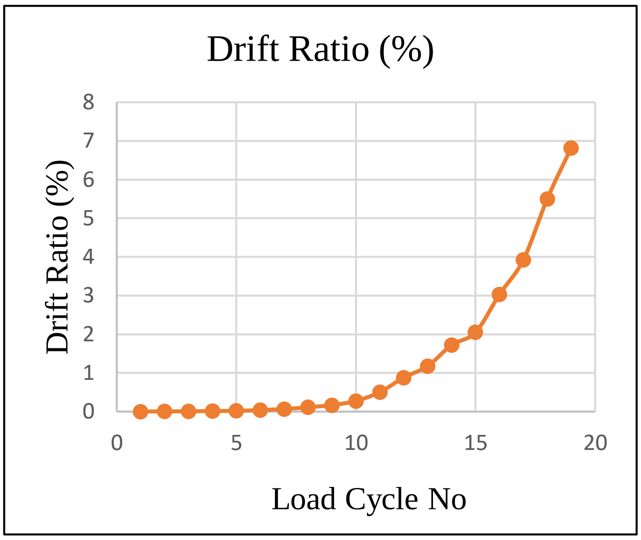


**Chart -8:** Drift ratio

## 4. FINITE ELEMENT ANALYSIS

Non-linear static analysis is carried out in ANSYS Workbench. The concrete is modelled using SOLID 65 and the reinforcements are modelled using BEAM 188 element. The contact elements used are CONTA174 and TARGE170.

### a) Frame 1 (normal frame)

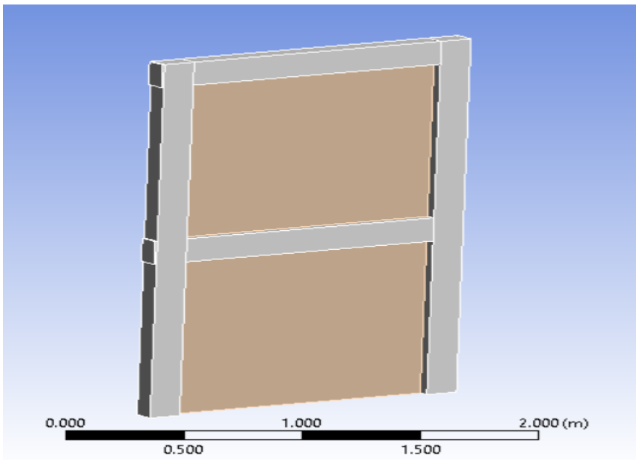


**Fig -5:** 3D view of the model

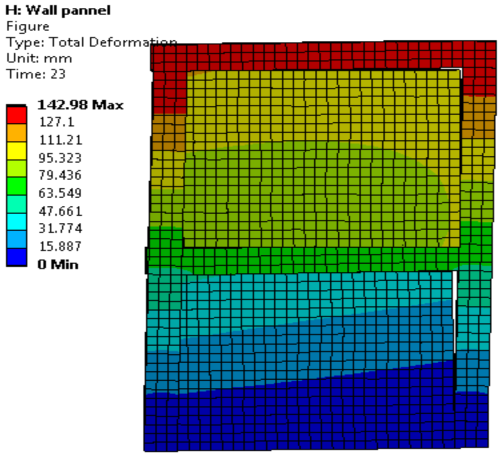


**Fig -6:** Total Deformation of the frame

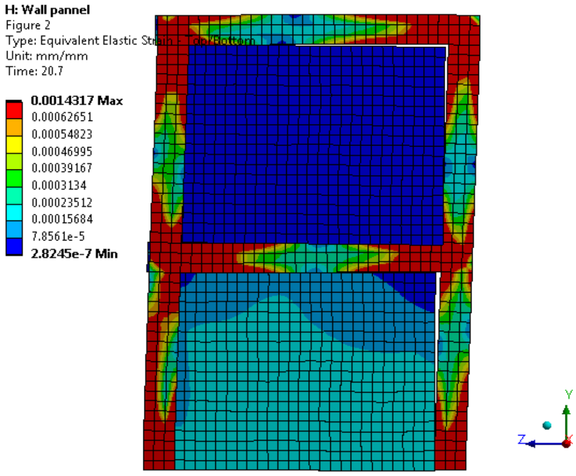


**Fig -7:** Equivalent elastic strain of the frame

## b) Frame 2 (retrofitted frame)

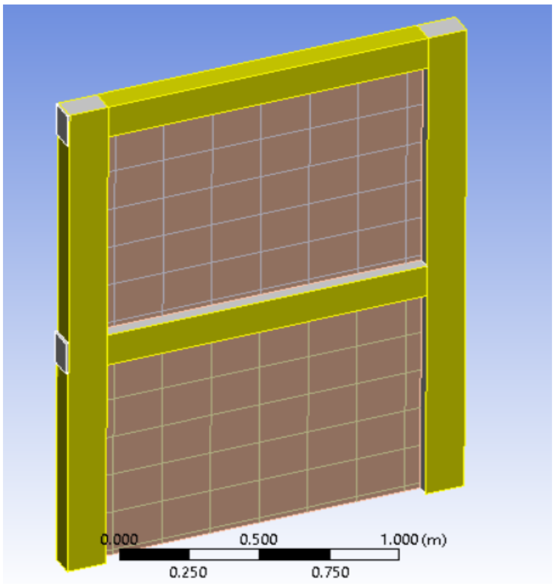


**Fig -8:** 3D view of the model

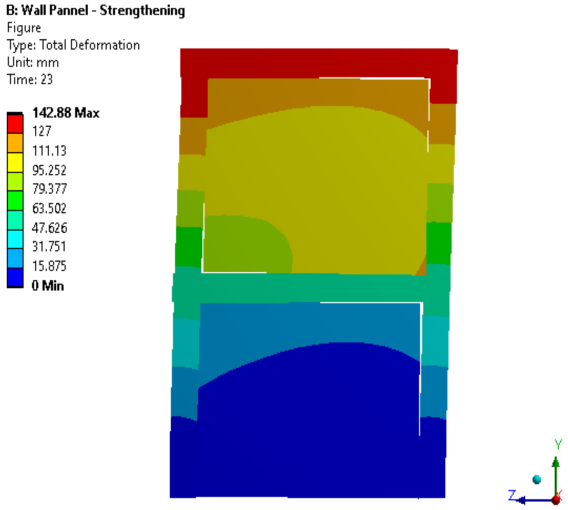


**Fig -9:** Total Deformation of the retrofitted frame

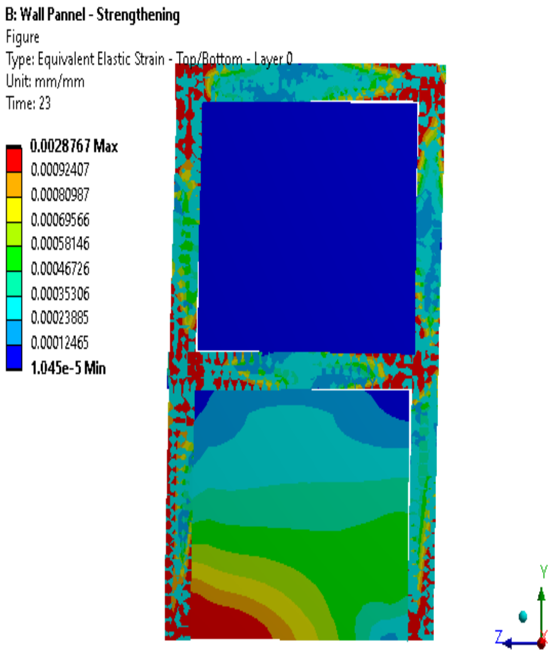


**Fig -10:** Equivalent elastic strain of the frame

## 5. CRACK STUDY

### 5.1 General

One–third full size models of two-storey, one-bay, reinforced concrete frame was tested under static lateral cyclic loading and its load carrying capacity, stiffness, ductility factor, cumulative ductility factor, energy dissipation & cumulative energy dissipation capacity and drift ratio have been studied experimentally. Theoretical analysis for both the retrofitted and normal RC frame have been carried out using frame analysis in ANSYS. The experimental results of the frame have been compared with those theoretical analysis results.

### 5.2 Response of the frame at each load step

- The RC frame with prefabricated RC wall panel has been subjected to an experimental investigation (Cyclic Loading) with an increment of 2KN up to 5 load cycles and from 6th load cycle and increment of 5KN is applied.
- At 25 KN, horizontal cracks have been identified at the bottom story panel connection. The mortar cracks have been identified.
- At 35 KN, a similar kind of mortar joint cracking has been found at the top story wall panel connection.
- At 40 KN, horizontal cracks appeared at the windward column of the frame. This indicates the yielding of concrete occurred at the 12th Load cycle. Due to yielding the concrete attained its maximum strain after which the readings are not noted.
- At 55 KN, a horizontal crack has been found at the corners of the windward column.
- At 60 KN, further cracking of the bottom storey windward column occurred. Several minute cracks have been found at the bottom story beam.
- At 70 KN, several minute cracks appeared at the windward column at 600mm from the foundation. Due to diagonal compression strut action, the bottom storey panel to frame crushing has initiated. Similarly, the top storey panel exerts compression behavior on the beam-column joint of the fist storey.
- At 80 KN, several cracks are found at the windward column and appear throughout the column width.
- At 85 KN, the top story cover crushing has initiated, and minute cracks were found at the windward top story column.
- At 90 KN, the top story concrete crushing occurred which further increased diagonal cracking at the first storey beam-column joint. This is the peak load of the frame which caused a deflection of 156.9 mm.
- After this post cyclic load is applied to the frame and further deformation is obtained.
- At 85 KN, the frame yields to a deformation of 162.4 mm.
- At 80 KN, the frame yields to a maximum deformation of 166.8 mm. At this load the beam-column joint of the first story cover has separated from the core frame. After the cover separation, the testing is stopped, and the final deformations are noted.

### 5.3 Failure mechanism

The failure pattern confirms that the stiffness of the wall panel is more compared to the stiffness of the RC frame. This had led to the early failure of the structure before reaching its ultimate capacity. The stiffness of the prefabricated wall panel can be reduced by decreasing the thickness of the panel to have a better interaction between the frame and the panel. As the in-plane loading on the panel continues, there is a separation initiated at the interface of the wall panel and the frame members (beam and column) at the off-diagonal corners. Once a gap is formed, the stresses at the tensile corners are relieved while those near the compressive corners are increased. As the load continues to increase, further separation occurs between the wall panel and the frame, resulting in contact only at the frame sections near the loaded corners. Due to this behavior of the RC wall frame, it resembles a braced frame with one diagonal member(compression). This compression strut action increases the shear demand more on the beam-column joints causing the early failure of the structure without reaching its maximum potential. The main reason for the debonding of the panel from the frame on the off-diagonal corners was due to the greater strength and stiffness of the wall panel compared to the frame. Thus, the intended purpose of providing the wall panel to resist the seismic forces is not met out. Another main failure pattern observed is the top storey cover crushing due to increased compressive load on the corners of the frame. Due to this effect, further load transfer becomes difficult and thereafter the concrete core starts taking the load. Once the frame reaches its peak load, the load carrying capacity reduces gradually but the frame starts yielding at a higher rate. The test is stopped when the bottom storey beam-column cover got separated from the concrete core.

**Fig -11:** Corner crushing


**Fig – 12:** Maximum deflection of RC frame with RC wall panel


**Fig -13:** Joint failure


**Fig -14:** Top storey cover crushing


**Fig -15**: Debonding of wall panel

### 5.4 Comparison of ANSYS model with experimental results

**Table -3:** Load carrying capacity - Frame 1

| DETAILS | LOAD | DEFLECTION |
|---|---|---|
| Experiment | 90 | 156.9 |
| Analytical | 82.2 | 142.1 |

**Table -4:** Load carrying capacity - Frame 2

| DETAILS | LOAD | DEFLECTION |
|---|---|---|
| Analytical | 121.5 | 142.88 |

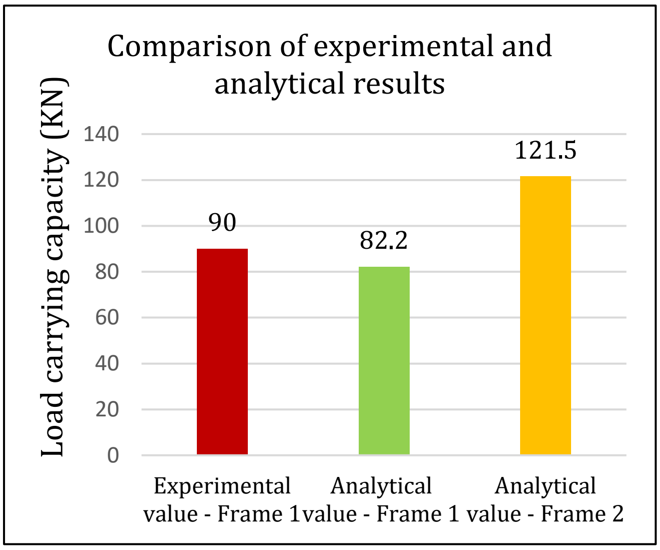


**Chart -9:** Strength comparisons of frames

### 5.5 Comparison of failure pattern of analytical and experimental results

The analytically obtained results were in accordance with the experimentally conducted test. The results obtained for the two frames are discussed below.

### a) Frame – 1 (normal frame)

As per the analytical modelling, the maximum strains are concentrated at the beam-column joints and minimum on the wall panels. This effect is analogous to that of the experiment as the major cracks are formed on the joints. Also added to that there is a debonding of the wall panel from the frame as the load cycle increases. The load carrying capacity and the deformation obtained in the analytical procedure are lesser compared to the experimental values. This may be due to minor variations while performing the test.

### b) Frame – 2 (retrofitted frame)

The retrofitting process is done by adding three layers only over the RC frame members. It is evident from the failure patterns of the first frame, that the RC wall panels are not affected by seismic forces. The textile reinforced mortar is used for the retrofitting of the frame. Initially, the first layer is modelled as special mortar which is specifically used for the repair purposes due to its enhanced properties. Next, the actual textile reinforced glass mesh is placed over the mortar layer. Finally, the special mortar is laid over the fibre. The overall thickness of the layers is 8.6mm. The thickness is limited to 10mm to avoid delamination of the layers. The retrofitted frame has a higher load carrying capacity of 121.5KN with a lateral deformation of 142.88mm. Also, the frame members have lesser strain concentration in the beam-column joints compared to the normal frame. This proves that the retrofitting process yields higher strength and lesser damage to the structure.

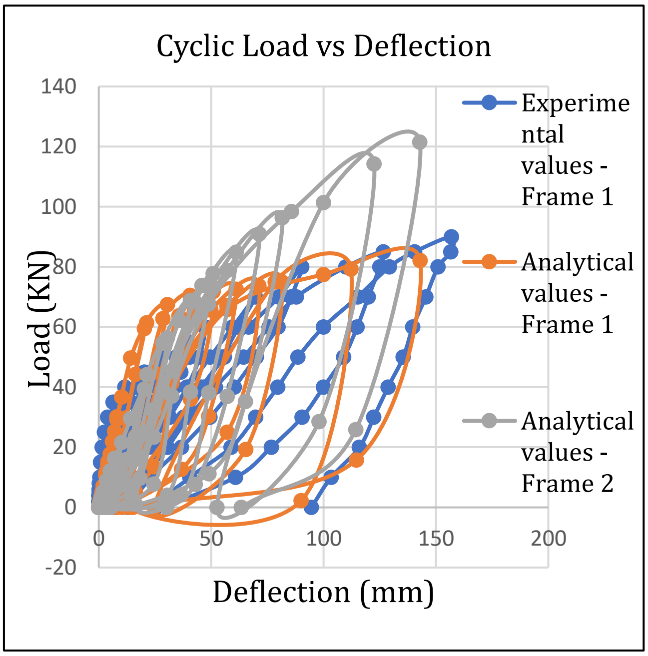


**Chart -10:** Cyclic load vs deflection behavior of frames

### 6. CONCLUSIONS

An experimental investigation was done on the RC frame with a prefabricated RC wall panel to study the seismic behavior of the structure subjected to lateral cyclic loading. Further, the study is done analytically in ANSYS software for both frames. The load-displacement behavior was studied in two frames and the results were compared. The normal RC wall panel frame attained its ultimate loading capacity in the (19th cycle) with the maximum load of 90KN and displacement of 156.9mm. After 90KN load, the post cyclic loading is done where the concrete transferred the load to steel reinforcements and steel started yielding which leads to more deflection afterwards. In the retrofitted frame, the maximum peak load reached up to 121.5 kN and the deflection was similar to that of the normal frame. This indicates that the load carrying capacity increased due to retrofitting. Added to that, the damage is greatly reduced by the retrofitting process. The stiffness of the frame was 33.33 kN/mm at the first load cycle which gradually degraded to 0.57 kN/mm for about 19 cycles. The ductility factor during the first cycle of loading was 0.005 and 13.52

during the last cycle. The cumulative ductility was found to increase from 0.005 during the first cycle to 52.236 during the last cycle of loading. The energy dissipation capacity during the first cycle of loading was 0 KN mm and that during the 19th cycle was 3973 KN mm. The cumulative energy dissipation was found to increase from 0 KN mm during the first cycle to 9331.655 KN mm during the last cycle of loading. In this test, the drift ratio of the frame for the 1st cycle is 0.003 and it gradually increases to 6.822 at the 19th cycle of loading. The frame developed beam and column hinging near beam-column interfaces before they reached their maximum storey shear force and they eventually failed due to joint shear, exhibiting successive strength drops. The leeward column experienced shear and in addition to it, compression occurred due to the diagonal strut effect of the wall panel which initiated the final collapse of the frame. After the joint failure of the frame and wall panel connection in the off-diagonal corners, only the compression strut action was predominant and the rest of the wall panel was inactive. Thus, the use of prefabricated wall panels leads to the damage of the primary framing elements. This is due to the higher stiffness of the panel in comparison with the RC framed structure. Also, mortar failure is observed at the early loading stages of the frame. Therefore, this research concluded that the use of RC wall panel of same thickness as that of the RC frame elements reduced the seismic performance of the structure. But the TRM retrofitting enhanced the strength and performance of the structure.

## 6.1 Suggestions

Reducing the wall panel thickness greatly helps in the involvement of the panel under seismic forces. Panel to frame interaction can be improved by providing shear connectors. Out-of-plane failure must be considered for the complete behavior of the prefabricated wall panel under cyclic loading. Reverse cyclic loading can be applied instead of cyclic loading. 3D frames with prefabricated wall panels can be studied under static lateral cyclic loading.

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