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Captive Column Effect in Cold Formed Steel Frame with Partially Infilled Cement Bonded Particle Board under Lateral Monotonic Loading

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Abstract - In recent years, for earthquake prone areas, lightweight structures are used to reduce the effect of lateral forces due to earthquakes. In latest trends, the usage of cold formed steel structures (CFS) with Cement bonded particle board (CBPB) as an infill and it is recommended in all earthquake prone areas. In partially infilled structures, column with short gap will behave as short column during earthquake and attract larger forces due to in-plane stiffness of the infills and can damage the column seriously due to excessive shear forces, which is known as captive column effect. Since it is complex to consider the contribution of strength and stiffness of these partial infills, the structure is analysed and designed as bare frames. On the other hand, the real behaviour of partial infilled structures during earthquake is like captive columns. One of the major failures of structure during earthquake is due to captive column effect.

Several literatures and research papers have been published in the area of captive column effect and its remedy in RCC frame. A detail study is required for understanding the *captive column effect* in cold formed steel columns under lateral loading as in the case of RCC framed structures. An experimental investigation is conducted in single bay two storied cold formed steel bare frame and cold formed steel frame with CBPB as partial infill in bottom storey to study the behaviour of captive column effect. This study clearly indicates that, the bare frame deflected uniformly which shows the lateral forces are distributed over the full height whereas in the frame with partial infill, the short column has attracted excessive shear forces resulting in failure Proving *Captive Column Effect.*

This study clearly indicates that the columns in lightweight structures with partial infill also behave as a short column and can be seriously damaged during an earthquake.

Keywords: Cold formed steel structures, Cement bonded particle board, Captive column effect, Partial In-fill,In-plane stiffness

I.INTRODUCTION

During past earthquake several buildings have failed predominantly due to captive column failure and soft storey. Designers have started to pay attention to avoid the similar damages in future earthquakes. Cold formed steel structures are preferred by the designers in order to reduce the impact on the building due to lateral forces. Cold formed steel framed building with infill boards are usually analysed and designed as bare frames without considering the strength and stiffness contributions of the infills. However, the actual behaviour of the structure with partial infill during an earthquake is different as shown in Fig. 1and Fig 2.

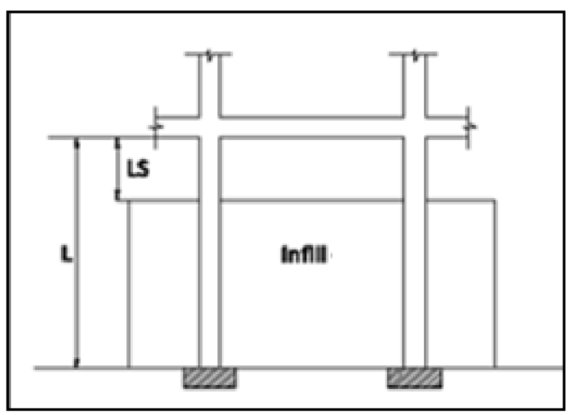

Fig.1 Frame with Partil Infill Board

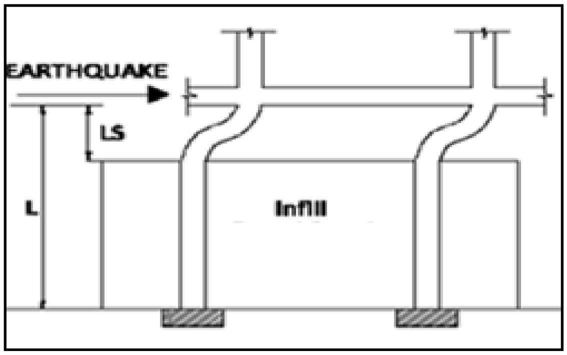

Fig.2 Short Column Behaviour

Further to this research work, an experimental and analytical investigation is carried out to study the behaviour of captive column effect in CFS frame with partial infill (Frame 2) andCFS frame without infill (Frame 1). Hence the purpose of the present study is to investigate the behaviour of bare cold formed steel structures and CFS frame with partial infill and to find out the captive column failure in lightweight structures.

II.RELATED WORKS

Few research papers on thebehaviour of cold formed steel framed structures with different infill panel boards have been reported.

Serrette(1996, 1997) studied shear strength of plywood, oriented strand board(OSB) and gypsum sheathed light weight steel framed walls through running full scale static and reversed cyclic loading test. The results show that the capacity of both side infill boards is twice the capacity of the one side board.

Gad et al. (1999b) conducted a shake table test on a single-storey test house as part of evaluating the earthquake performance of strap braced cold-formed steel wall structures. A room was adopted as the test specimen with the measurement of 2.3 m × 2.4 m and 2.4 height with a concrete slab on top simulating the roof mass. The effects of non-structural components such as plasterboard and brick veneer were also studied by including them in the test structure. The El-Centro earthquake was selected as a testing ground motion record. The dynamic shake table tests showed that yielding of the braces could take place in addition to slip and in most cases, failure of the brace connections. In general, the steel frames were able to perform well under the seismic loading and the non-structural components made a significant influence on the frame lateral bracing.

Fiorino (2003) carried out lateral monotonic as well as cyclic loading test on cold formed steel stud shear wall lateral resisting system. The results show that the walls have more strength in monotonic loading than cyclic loading.

Kim et al. (2006) carried out a full-scale one-directional shake table test on a double storey (6.4 m height) two-side strap braced structure with two framing lines imposing a real ground motion. The dynamic test showed that the thin steel strap bracing in CFS building are very tough and ductile members. The contribution of CFS columns to storey shear decreased due to the anchor deformation and existing gap between the columns and the floor-slab.

Morello (2009) investigated the behaviour of gypsum sheathed walls under lateral load through monotonic and cyclic testing of eight wall specimens and has shown that the shear walls have acceptable seismic performance.

Suresh Babu,(2011) carried out the experimental work on 2D and 3D RCC structure with partial infill and found that the failure of structure due to captive column effect. By adding a brick insert in the same structure, the capacity of the structure increases considerably, as a result of reducing the captive column effect.

DaBreo (2012) studied the performance of single storey one-sided steel sheathed CFS framed shear walls constructed with additional blocking, under combined gravity and lateral load. The results show that the blocked walls achieved nominal design resistance of 1.37 to 1.8 times higher than the identical walls without the blocking.

MoyaedAlaee et al. (2012) developed a semi-empirical method based on data obtained from fastener connection experimental tests for the prediction of the behaviors of CFS framed wood sheathed shear walls. It was shown that the predicted wall displacement and resistance agreed well with the wall experimental test results.

From the literature review, the CFS frames with boards have shown good performance for the lateral forces due to earthquake.

III. EXPERIMENTAL INVESTIGATION

A. Test Model of Frames

Experimental investigation on full scale structure is generally very difficult to be carried out and hence it can be investigated through model studies by scaling the size and properties of the real structures.Test models are fabricated to 1:3 reduced scales following the laws of similitude by scaling down the geometric and material properties of the prototype for Frame (1) and Frame (2). The scale ratio was chosen based on the capacity and dimension of the reaction frame and test floor available at the laboratory. A single bay two-storeyed cold formed steel bare frame (Frame -1) and a CFS frame with partial Cement bonded particle board(CBPB) infill of 12mm thick provided in the bottom storey and completely filled CBPB provided in the top storey (Frame-2) connected by means of metalself tapping screws at 200mm c/c shown in Fig.3 & Fig.4 are tested.

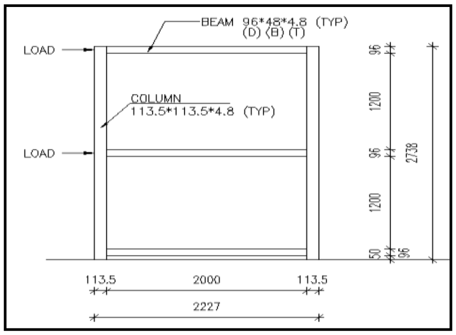

Fig.3 Frame -1 (Bare Frame)

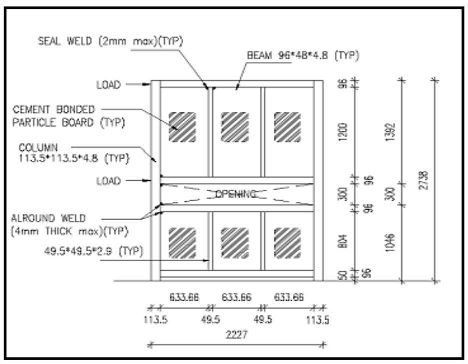

Fig.4 Frame – 2 (Partially In-filled Frame)

The models are analysed in STAAD Pro and designed for gravity loads according to IS801-1975 “Code of practice for use of cold formed light gauge steel structural member in general building construction”. The columns and beams are connected by fillet weld. The grade of cold formed steel used is YST 310 with Young’s Modulus of 2.05 x 10^5N/mm^2. The test conducted on CBPB to find the Modulus of Elasticity is shown in Fig.5. From the test results, the Load-deflection diagram for CBPB board is obtained and this diagram is used to find the Modulus of Elasticity of CBPB. The average Modulus of Elasticity of theCement bonded particle board obtained from the test result is 4005 N/mm$^{2.}$

Fig.5 Testing of CBPB specimen

B. Geometric Properties of the Test Model

The geometric properties of the prototype and model are given in Table 1 based on laws of similitude. The cross sections of the model reinforcement do not conform exactly the laws of similitude. So, the yield forces rather than the yield stresses are selected as the target to be achieved.

Table:1 Geometric properties of the Prototype and Model

Sl. No.	Property	Prototype (mm)	Model (mm)	Scale
1	Storey Height			
	Ground Floor	3600	1200	3
	First Floor	3600	1200	3
2	Frame Span (One bay)	6000	2000	3
3	Opening Height (Frame-2)	900	300	3
4	Beams	RHS- 250 x 150 x 12	RHS - 96 x 48 x 4.8	2.6
5	Columns (Frame -1 to 3)	SHS - 300 x 300 x 12	SHS - 113.5 x 113.5 x 4.8	2.6
6	Infill board supporting post	SHS - 130 x 130 x 8	SHS - 49.5 x 49.5 x 2.9	2.6
7	Cement bonded particle board	31.2	12	2.6

**RHS – Rectangular Hollow Section*

SHS – Square Hollow Section

C. Test Set-up

The models are tested as vertical cantilevers under a cyclic loading programme. The schematic diagram of test set-up is presented in Fig.6. Equivalent static lateral monotonic loading is applied at first and second storey levels in line with the beams using hydraulic jacks of capacity 500 kN. The reaction frame, which is used for loading arrangements, is rigidly fixed to the test floor. A common console controlled all the two jacks. Pressure gauges are used

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to measure the applied load. The experimental model used for testing is shown in Fig.7 and Fig.8.

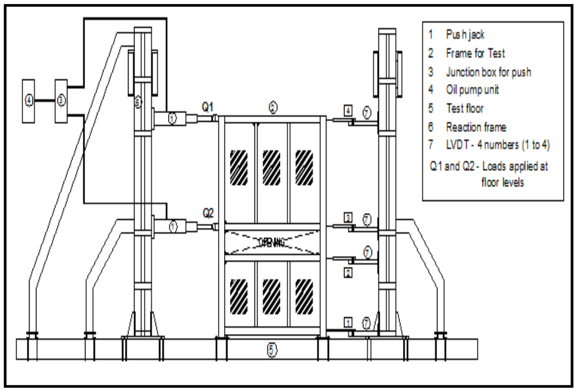


Fig.6 Schematic diagram of test –setup

Fig.7 Frame -1 (Bare Frame)

Fig.8 Frame – 2 (Partially In-filled Frame)

LVDT (Linear Variable Differential Transducer) of least count 0.01mm is used for measuring deflections at all storey levels as shown in Fig.8. An additional LVDT is placed at the top of partial infill. The rigid body rotation if any is measured by providing deflectometers on the sides of the base plate. DEMEC (Mechanical strain gauges) points (pellets) are pasted for measuring strain in columns, beams and infill panels as shown in Fig.9 and Fig.10.

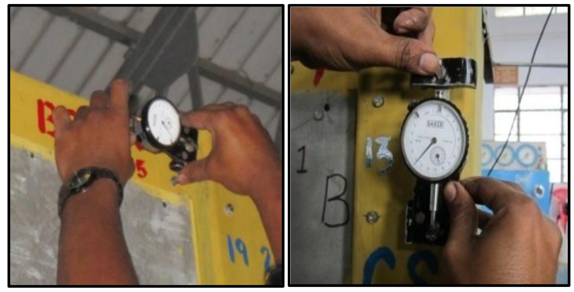


Fig.9 Measurement of strain in columns and beams

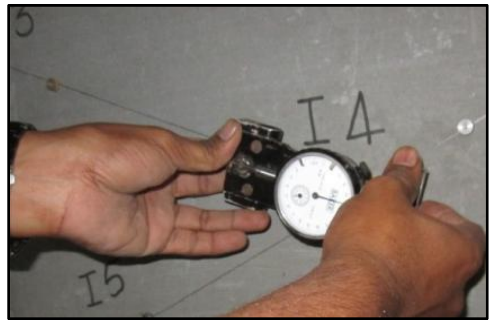


Fig.10 Measurement of strain in CBPB Infill

D. Experimental Programme

The frame was subjected to equivalent static lateral cyclic loading. The load was applied in increments of 5 kN for frame-1 and 10kN base shear for frame-2 base shear for each cycle and released to zero after each cycle. The deflections at all storey levels were measured at each increment and decrement of the load. The strain in column, beams and infill were monitored at maximum load of each cycle and at unloading conditions of frame (i.e., when the load is released fully) during all cycles of loading. The formation and propagation of infill cracks, hinge formation and column failure patterns were recorded.

IV. Results and discussions

Based on the results of the experimental investigation carried out on bare frame (Frame-1) and partially in-filled frame (Frame-2), various parameters like load-deflection behaviour, stiffness degradation, ductility factor and Energy dissipationcapacity are studied.

A. Load-Deflection Behaviour (P-Δ)

The frame was subjected to static lateral cyclic loading. The load was applied in increment of 5 kN for frame-1 and 10kN base shear for frame-2 for each cycle and released to zero after each cycle.The history of sequence of loading for the frames 1 & 2 is shown in Fig.11 and Fig.12. For frame (1) the ultimate base shear of 40kN was reached in the eighth cycle of loading and for frame (2) the ultimate base shear of 130kN was reached in the thirteenth cycle of loading. After reaching the ultimate load, post ultimate cycles were performed to study the behaviour of Cold formed Steel frame till final collapse.

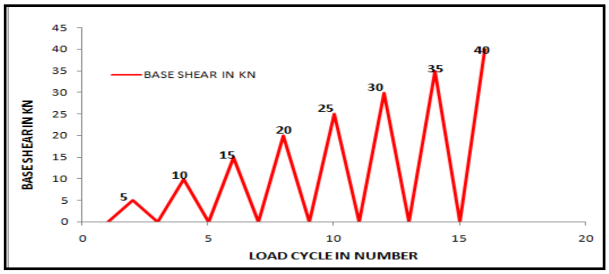


Fig.11 Sequence of Loading for Frame -1

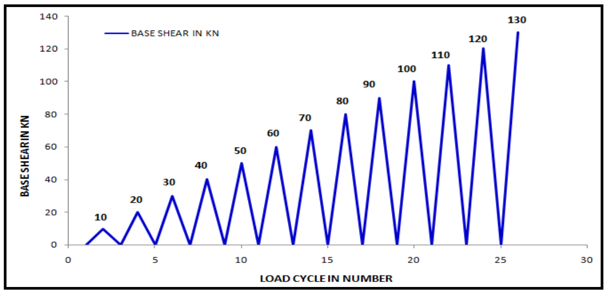


Fig.12 Sequence of Loading for Frame -2

The hysterisis loop for top storey displacement versus base shear diagram for Frame (1) and Frame (2) is represented in Fig.13 and Fig.14. The top storey versus base shear diagram for Frame (1) and Frame (2) is represented in Fig.15 and Fig.16. At the ultimate base shear, the top storey deflection for Frame (1) is found to be 2.30mm at first cycle and 75mm at eighth cycle. For Frame (2) the top storey deflection is found to be 0.25mm at first cycle and 30.50mm at thirteenth cycle.

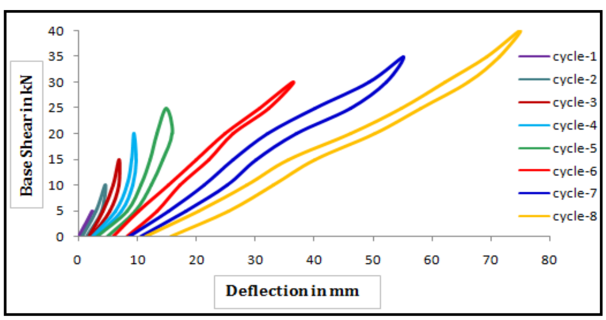
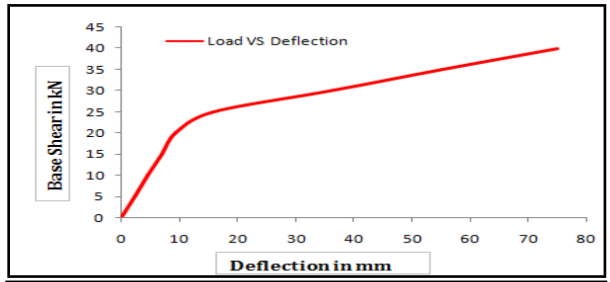


Fig.13Hysterisis Loop for Base Shear VsTop storey Deflection for Frame 1



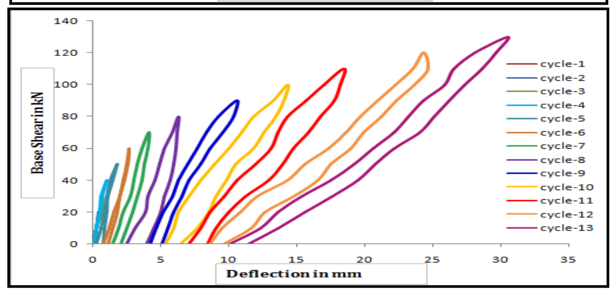


Fig.14 Hysterisis Loop for Base Shear Vs Top storey Deflection for Frame 2

Fig.15 Base Shear Vs Top storey Deflection for Frame (1)

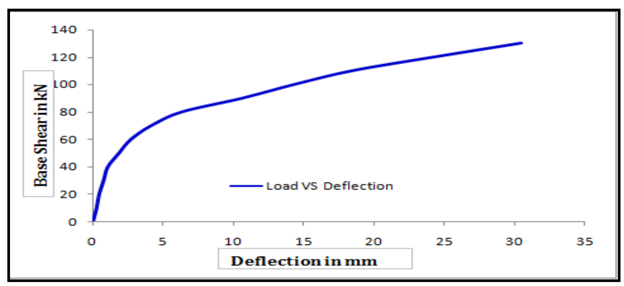


Fig.16 Base Shear Vs Top storey Deflectionfor Frame (2)

B. Ductility Factor (μ)

The ductility factor (μ) is calculated. The first yield deflection (Δy) for the assumed bi-linear load-deflection behaviour of the Frame (1) and Frame (2) is 9.34 mm and 2.7 mm respectively. The ductility factor value μ = (Δ1/Δy) for various load cycles of the frame is worked out and the variation of ductility and cumulative ductility factor with load cycles for Frame (1) is shown in Fig.17 and for Frame (2) it is shown in Fig.18. For Frame (2), the cumulative ductility factor is found to be increasing from 6.49 in the eighth cycle to 42.92 in the thirteenth cycle of loading showing the sudden reduction in stiffness because of column buckling adjacent to the opening and formation of cracks in columns adjacent to partial infill. For Frame (1), due to the absence of partial infill, the columns are deformed over the full height and it attracted lesser amount of lateral forces.

Hence there is no abrupt reduction in the stiffness and the cumulative ductility factor also increased uniformly.

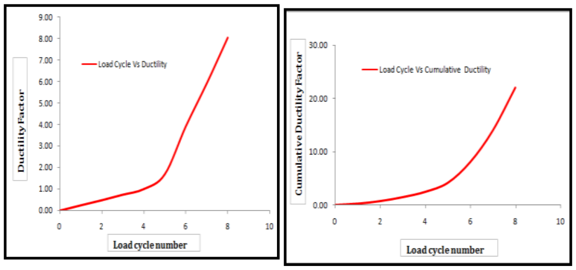


Fig.17 Ductility and Cumulative Ductility Factor for Frame (1)

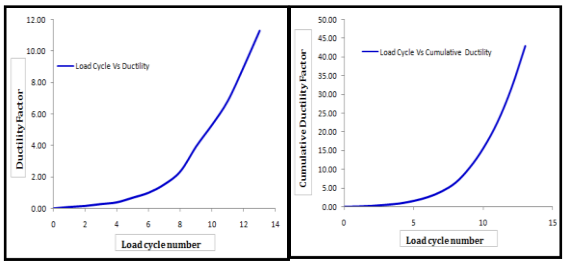


Fig.18 Ductility and Cumulative Ductility Factor for Frame (2)

C. Stiffness Degradation

The stiffness of the member is obtained from the relationship, K=P/Δ; whereK = stiffness of the member; P = Load applied in the frame in kN; Δ = Deflection in mm

The stiffness of the cold formed partially-infilled frame (Frame-2) for various load cycles is calculated and the variation of stiffness with respect to load cycles is shown in Fig.19. The decrease in stiffness of the cement bonded particle board in CFS frame is observed with 40kN/mm during first cycle to 4.26kN/mm during the thirteenth cycle of loading. This may be due to the flexural hinges at top and bottom of the short columns,buckling with cracks in the short column portion and the partial CBPB infill failure. For bare frame (Frame-1), there is not much decrease in the stiffness and it is 2.17 kN / mm during first cycle to 0.53kN/mm during the eighth cycle of loading. The stiffness of bare frame for various load cycles is calculated and the variation of stiffness with respect to load cycles is shown in Fig.20.

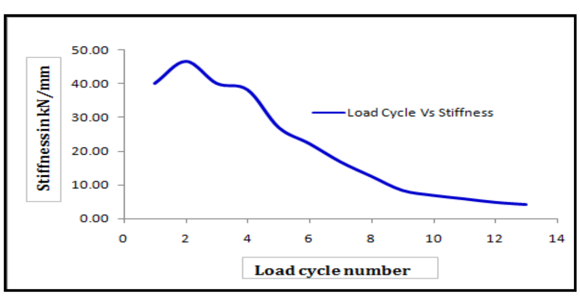


Fig.19 Stiffness Degradation Curve for Frame (2)

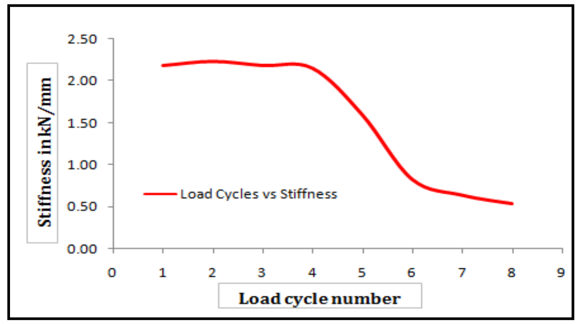


Fig.20 Stiffness Degradation Curve for Frame (1)

D. Energy Dissipation Capacity

The energy dissipation capacity of the Frames (1) and (2) during various load cycles are calculated as the area bounded by the hysteresis loops of the base shear versus top storey deflection diagram. The Energy dissipation and Cumulative Energy dissipation capacity of the Frame (1) are shown in Fig.21 and Fig.22 respectively. For Frame (1), the energy dissipation capacity during first cycle of loading is 6.53kN.mm. and that during eighth cycle is 166.97kN.mm. The Energy dissipation and Cumulative Energy dissipation capacity of the Frame (2) is shown in Fig.23 & Fig.24. For Frame (2), the energy dissipation capacity during first cycle of loading is 6.24kN.mm. and that during thirteenth cycle is 212.30kN.mm. The Cumulative Energy dissipation capacity for Frame (1) is found to be 767.44kN.mm. and for Frame (2), it is 1242.94 kN.mm. The results show more energy dissipation capacity for Frame (2) due to the presence of CBPB infill.

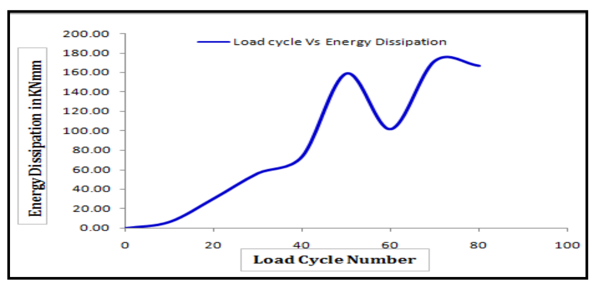


Fig.21 Energy Dissipation Capacity for Frame (1)

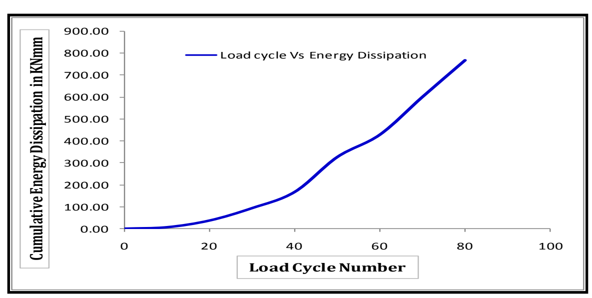


Fig.22 Cumulative Energy Dissipation Capacity for Frame (1)

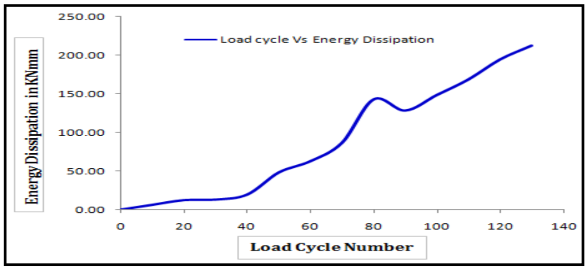


Fig.23Energy Dissipation Capacity for Frame (2)

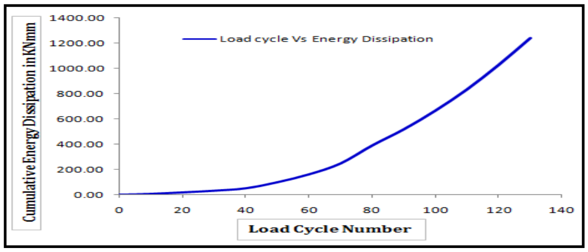


Fig.24Cumulative Energy Dissipation Capacity for Frame (2)

E. Behaviour and Mode of Failure

The deformation and failure pattern of Frame (1) and (2) is hown in Fig.25 and Fig.26. In Frame (1) where the infill is not provided the board infill's capacity is not active and only the bare frame has to resist the lateral force. In the absence of infill, the columns in both top and bottom storey has deformed over the full height and no significant damages are noticed in the bare frame (Frame 1) colums and beams.

Fig.25 Deformation of Bare Frame–Frame (1)

Fig.26 Failure Pattern of Frame (2)

It is observed that, at a base shear of 50 kN, the column buckling has started as shown in Fig.27 in the leeward column adjacent to the opening. At a base shear of 80 kN, cracks are initiated as shown in Fig.28 in the same column at the beam column junction. On further loading, the crack propagated and at the base shear of 130 kN, the column hollow section started tearing as shown in in Fig.29. The crack pattern indicates a combined effect of flexure and shear failure. After reaching the ultimate load, post ultimate cycles are performed to study the behaviour of the CFS-CBPB frame till final collapse.

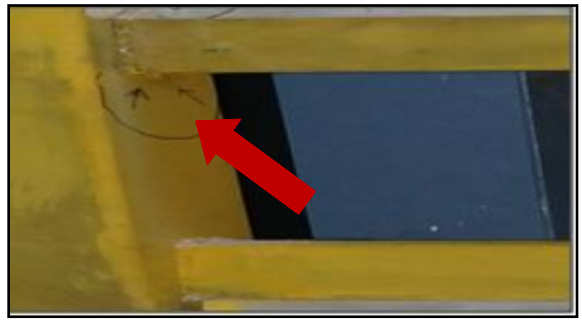


Fig.27 Column Buckling at a Base Shear of 50 kN -Frame (2)

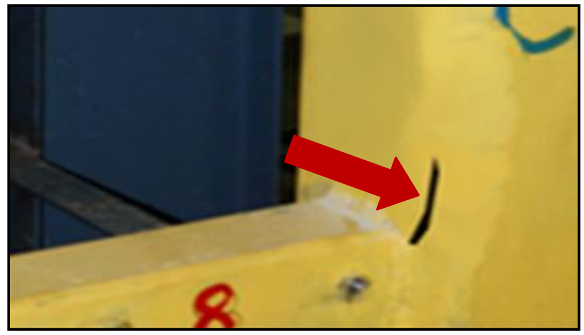


Fig.28 Column Crack at a Base Shear of 80 kN– Frame (2)

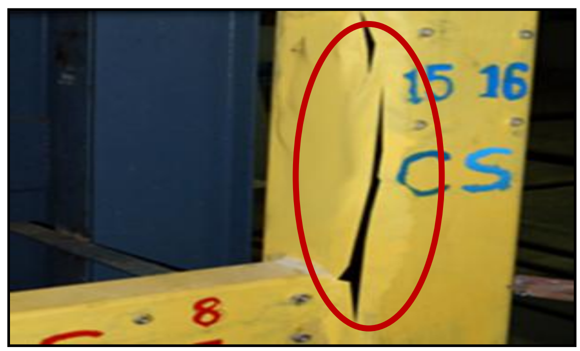


Fig.29 Tearing of Column at a Base Shear of 130 kN–Frame (2)

The localised separation of the infilled board panel from the frame in the bottom storey due to tension (Fig.30) and crushing of infilled board panel (Fig.31) in the bottom storey at one corner in the loading point side due to compression indicates the diagonal strut concept and stress flow in the line connecting the load point to the diagonallyopposite corner.

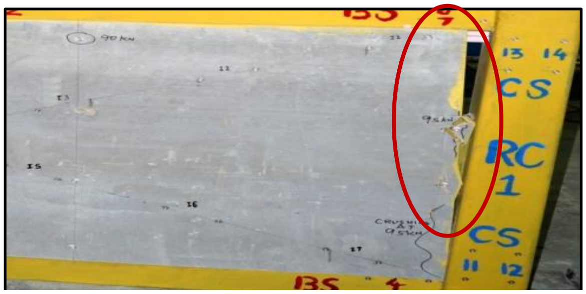


Fig.30 Separation of Infill at Tension Corner – Frame (2)

Fig.31 Crushing of Infill at Compression Corner-Frame (2)

Cement bonded particle board have significant capacity to resist against lateral forces caused by earthquake and wind. However the spacing of screws plays a vital role in lateral resistance capacity of CBPB panels. Pull out of screws and subsequent crushing of infill is noticed as shown in Fig.32 due to tension and shear in both top and bottom storey.

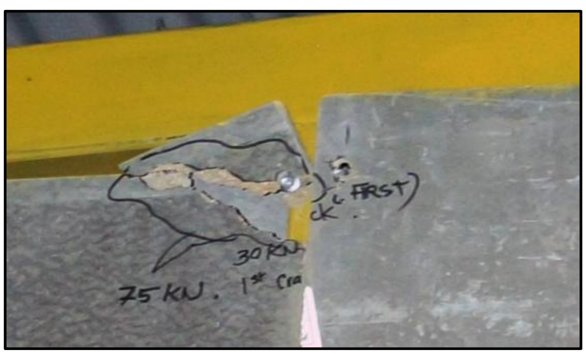


Fig.32 Pull out and Shear Failure of Screws–Frame (2)

V. ANALYTICAL INVESTIGATION

Non – Linear finite element analysis has been carried out using ANSYS -14 Software. Comparative study is made between experimental and the analytical values. The Cold formed steel frames (1) and (2) are modeled in ANSYS software as shown in Fig.33 and Fig.34 respectively.

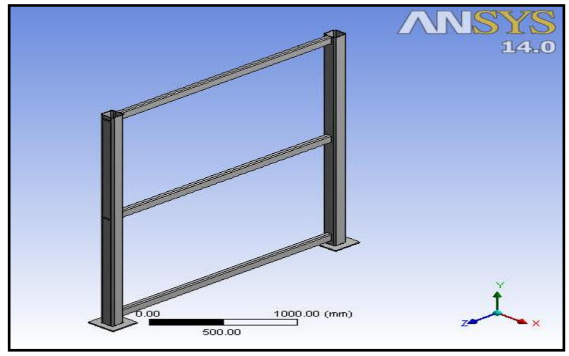


Fig.33 Frame (1) – ANSYS Model

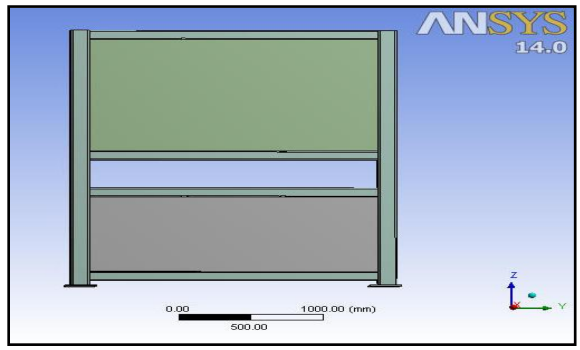


Fig.34 Frame (2) – ANSYS Model

The deformed shapes of the software model for frames (1) and (2) are presented in Fig.35 and Fig.36 respectively.

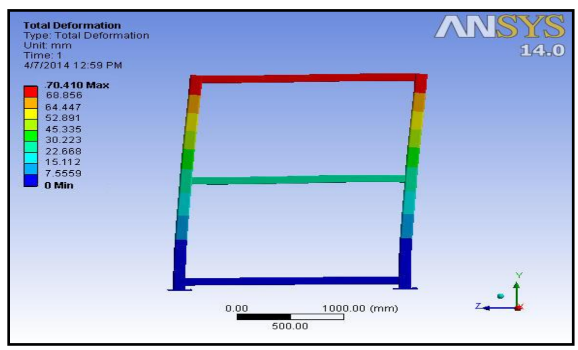


Fig.35 Frame (1) – Deformed shape

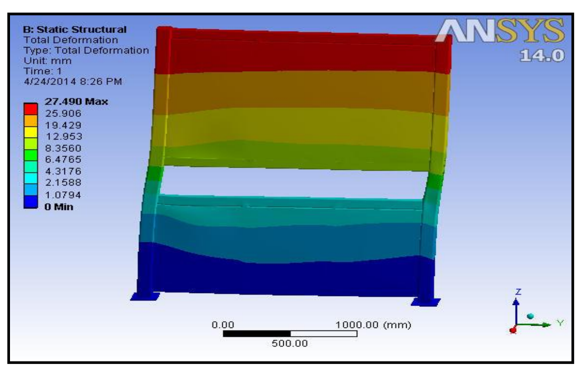


Fig.36 Frame (2) – Deformed shape

The stress patterns of frames (1) and (2) are shown in Fig.37 and Fig.38 respectively.

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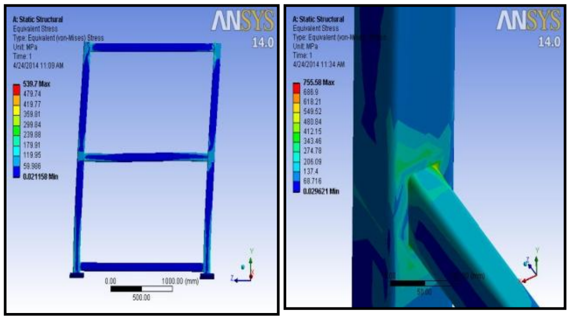

Fig.37 Frame (1) – Von-misses Stress Variation

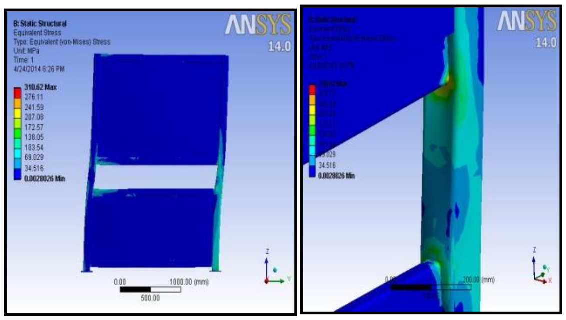

Fig.38 Frame (2) – Von-misses Stress Variation

The base shear Vs top storey deflections for both analytical and experimental results for Frame (1) and Frame (2) are shown in Fig.39 and Fig.40. The analytical results are compared with experimental results.In the analytical study, there is a sudden increase in the top storey deflection after a base shear of 80kN as observed in the experiment. This proves the initiation of the captive column effect adjacent to the gap region in comparison with the experimental values. Analytical results by ANSYS-14 under-estimates the experimental results in the range of 5to10 %.The maximum displacement with LVDT at various levels for Frames (1) and (2) are shown in Fig.41and Fig.42

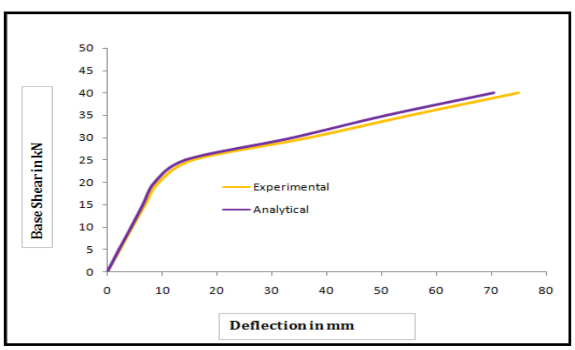

Fig.39 Comparison of Base Shear Vs Top storey Deflection for Frame (1)

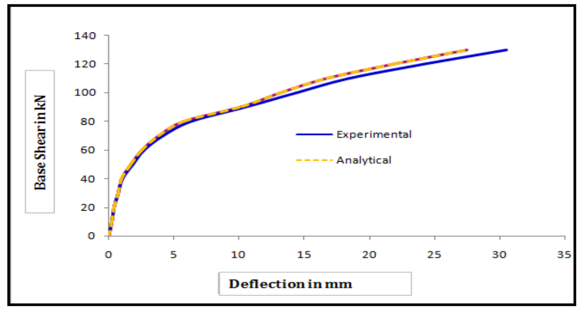

Fig.40 Comparison of Base Shear Vs Top storey Deflection for Frame (2)

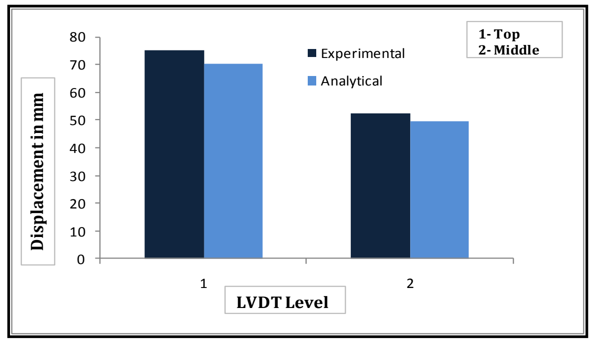

Fig.41 Comparison of Maximum displacement with LVDT at various levels - Frame (1)

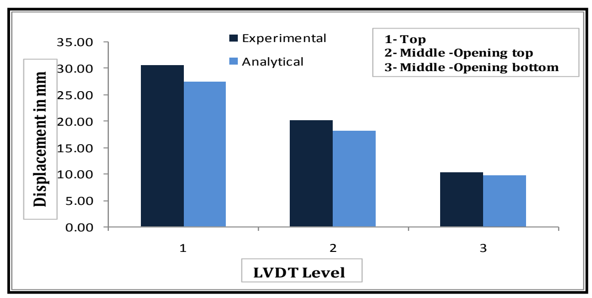

Fig.42 Comparison of Maximum displacement with LVDT at various levels - Frame (2)

VI. Conclusions

To identify the captive column effect on lightweight structures made out of cold formed steel structures with side boards as infill, an experimental work is carried out. It is observed from the experimental study that the Bare Frame (Frame-1) deformed uniformly over the full height of frame, there is no significant stress concentration at particular pointand the stresses are distributed throughout the frame. The CFS frame with CBPB partial infill (Frame-2) attracted larger forces where the columns left open in the bottom storey are unable to bend freely, behave as short column andsubsequently fail.This experimental investigation clearly shows that the frame with partial infill is subjected to captive column effect and therefore it is recommended that captive column effect is to be addressed in the design stage itself for lateral loads due to earthquake.

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