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Behaviour of Masonry Infilled Walls with RC Frames under In-Plane Loads

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Abstract: The analysis of in filled masonry walls with RC frames has been a major area of study in the past. Typically these walls are considered as non-structural components while designing the frame due to the complexity involved. The performance of such structures under an earthquake scenario has attracted major attention. In this paper we look into the analysis of these type of structures using equivalent diagonal strut concept. A comparison is made with the experimental results of a single bay frame performed by Mehrabi(1996)[1] et al and non-linear push over analysis performed in SAP2000 using different equivalent diagonal strut widths suggested by Mainstone(1962)[5] and Holmes(1961)[4]. This paper also compares the results obtained for a diagonal single strut model with a three strut diagonal model suggested by Chrysostomou(1991)[15] for the same widths. In general The SAP2000 analysis showed that all the models considered overestimated the lateral peak load capacity of the masonry in-filled frames.

I. INTRODUCTION

RC frames with masonry in-filled walls have been a major construction practice in many countries throughout the world including countries like India, Pakistan, and Turkey etc.

A majority of these buildings are built in seismic zones. Generally these structures are designed considering the in-fill as a non-structural component. But from past cases of earthquakes it was found that these infill walls have a major role to play in the response of the structure and that the behavior of these infill walls cannot be ignored. The strengths of these walls are not negligible and they will interact with the bounding frame under in plane loads induced by an earthquake. The infill walls increase the lateral stiffness of the frames resulting in a possible increase in the seismic demand because of a reduction in the natural period of the system.

There are different techniques to model these structures for the purpose of analyzing them. These can be divided into local or micro models where the structure is divided into numerous elements to take into account the local effects in detail. Finite element approach is one such micro modelling technique. The other is the simplified or macro model and is based on a physical understanding of the behavior of infill panel's .In this method, a macro modelling approach has been used to represent the infill panel. The equivalent diagonal strut concept falls under this category.

II. BACKGROUND/LITERATURE REVIEW

A. Equivalent Diagonal Strut

The equivalent diagonal strut concept is based on the idea that as the lateral load increases, the infill tends to partially separate from the bounding frame and form a compression strut mechanism. This kind of mechanism is assumed to cause diagonal cracking in the infill which can eventually lead to corner crushing failure in the frame members. The equivalent diagonal strut model does not consider other modes of failure in the infill.

Extensive research has been conducted in this area and it is observed from experimental observations that these kind of structures exhibit a highly non-linear inelastic behavior. The most important factor contributing to the non- linear behavior is the concrete and masonry material non-linearity.

The interaction between the infill and the frame also plays a major role in the response of an in-filled frame subjected to in-plane loads. In most instances, the lateral resistance of the structure is not equal to the sum of the lateral resistance provided by the infill and the frame. The equivalent diagonal strut generally does not take into consideration the interaction between the infill and the frame, but is a simple and capable way of representing the influence of the masonry panel in the global sense. The diagonal strut is usually modeled as a compression only truss member between the corners of the frames.

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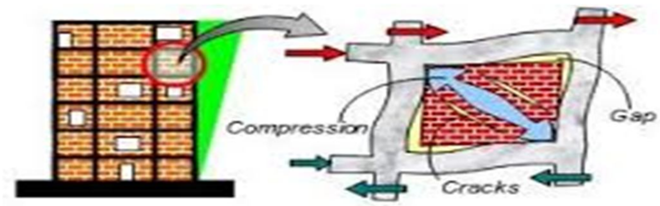

Fig1-Compression Strut Mechanism

Over the years various researchers have come up with different expressions for the width of the equivalent diagonal strut. In the early 1960's Holmes (1961)[1] came up with a width of the equivalent strut as

$$w=d/3 \qquad (1)$$

Where w=width of the equivalent strut, d=diagonal length of the strut. He said that the width of an equivalent strut is based on the thickness and aspect ratio of an infill but there was no experimental data is confirm it. Later Stafford Smith (1966, 1967)[9, 10] conducted a series of experiments and came up with a parameter which takes into account the relative stiffness of the structure. Generally the stiffness of the structure as mentioned earlier cannot be added up directly. Stafford Smith through his experiments came with

$$\lambda_h = \sqrt[4]{\frac{E_z t \sin 2\theta}{4 E_b I_s H}}$$

Where t and H are the thickness and height of the infill respectively, Θ is the inclination of the diagonal of the panel, Ez and Eb are the modulus of elasticity of the masonry and concrete respectively and Ic is the moment of inertia of the columns. He came up with a width based on this parameter but he found that his theory tends to overestimate the width of the strut.
Later Mainstone (1962, 1971)[6.5] proposed the width of a strut based on the relative stiffness parameter proposed by Stafford Smith**.**

$$w=0.16\,\lambda_h^{-0.3}\,d \qquad (2)$$

Several other widths for the equivalent diagonal strut have been propped over the years. In this study only the widths proposed by Holmes and Mainstone have been used.

B. Multiple Strut Models

As research progressed, it was soon realized that a single strut could not effectively explain the complex interactions between the frame and the in-fills. The transfer of the bending moments and shear forces from the frame to the infill could not be adequately represented by a single diagonal strut connecting only the corners. So the multiple strut model was proposed.
Thiruvengadam(1985) [16]was the first to propose the use the of multiple struts to model the infill panels. He suggested the use of multiple diagonal, horizontal and vertical pin jointed struts to represent the shear, axial and vertical stiffness of the structure. It was a very complex method to use, although it gave satisfactory results.
Many other researchers gave various multiple strut models like the 5 strut model (Syrmakezis and Vratsonou (1986)[17] etc. but in this study the 3 diagonal strut model as suggested by Chrysostomou(1991)[15] has been used.
In this model instead of using a single strut to represent the masonry infill three diagonal struts are used.
The area of the central diagonal strut is considered to be half the area considered for the single strut model and the two off diagonal struts areas were assumed to be one fourth of the area used for the single strut model. The total area of the strut used for both the model remains the same but the difference remains in the case that three struts are used in this model which acts as a better representation of how the structure behaves.
In this model, the contact length is governed by the relative stiffness parameter, λ_h and the distance between the struts along the columns was given by z (Stafford smith) [9, 10] which can be approximated to

$$z=\pi H/2\,\lambda_h$$

Where H is the height of the infill and z is described in the figure below. This z basically takes into account the contact length of the struts. The contact length along the beam and the column is same.

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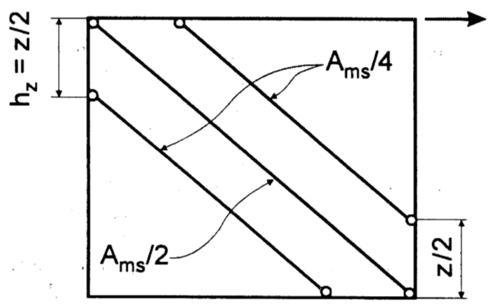


Crisafulli et al(2000)

C. Analysis

The experimental results considered in this paper were taken from the experiments performed by **Mehrabi et al (1996)[1].** The specimen 9 from the experiment was taken into consideration for comparing the results with the SAP models. The figure 3 below shows the dimensions of the specimen used. In particular, in specimen 9 strong masonry infill was used.

This one bay one story specimen was modelled in SAP2000. The model was run for non-linear static pushover analysis and the results were compared with the experimental results for the lateral peak load vs displacement relations. In the experiment the specimen was subjected to quasi-static lateral loads applied by an actuator and the peak load was observed for different specimens with weak and strong infill masonry panels. In our case we are considering only specimen 9 which had a strong infill.

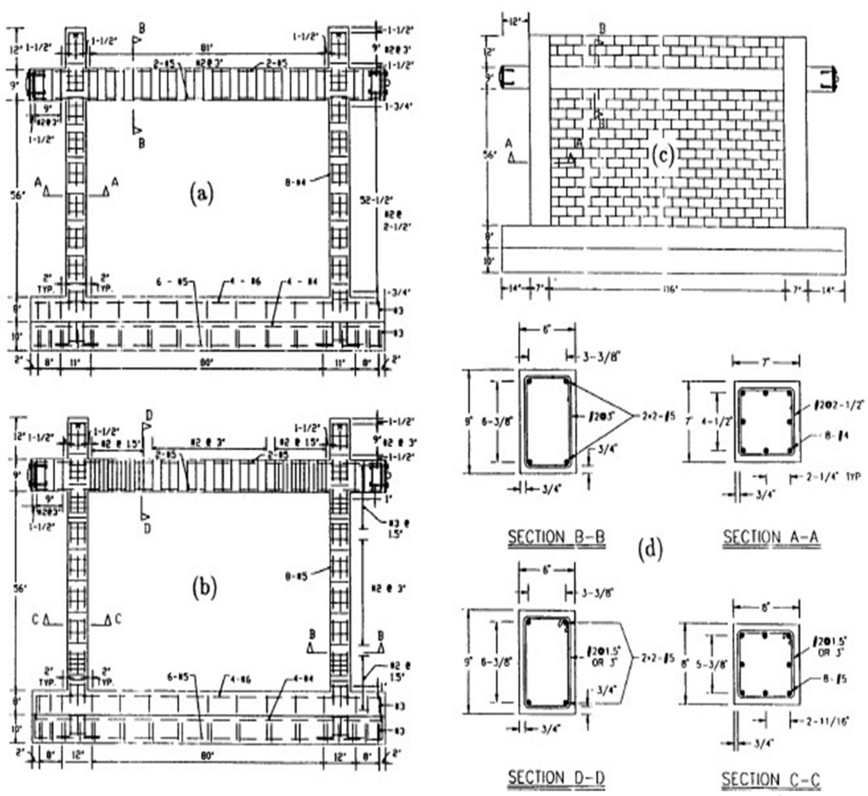


Fig3-Specimen 9(b) model used by Mehrabi (Mehrabi 1996)[9]

In order to run the non-linear analysis in SAP2000 the material non-linear properties have to be defined accurately to get the correct results. All the material properties used have been taken from the experimental results obtained by **Mehrabi et al (1996)[1]** .The concrete material properties that were defined in SAP are :- Elastic Modulus was taken as 2500ksi and the compressive strength was taken as 3.89ksi. The concrete material properties were defined as shown in the figure.

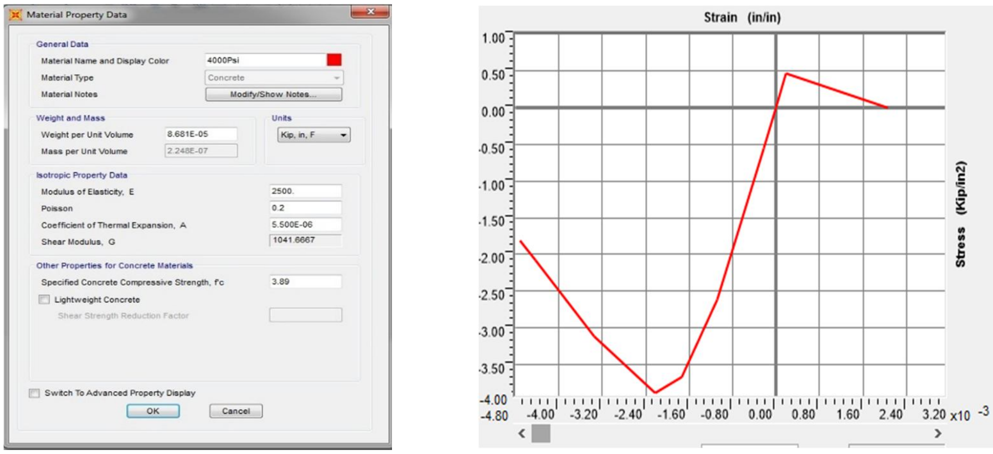


Fig4-Concrete Material Definition

The non-linear stress strain curve for concrete was taken as default from SAP2000. The material property for masonry was also defined similarly. In the case of masonry the Elastic modulus was taken as 1195ksi and the compressive strength as 2.06ksi. The non-linear stress-strain plots for masonry was as shown in the figure below [Kaushik 2006][13,14].

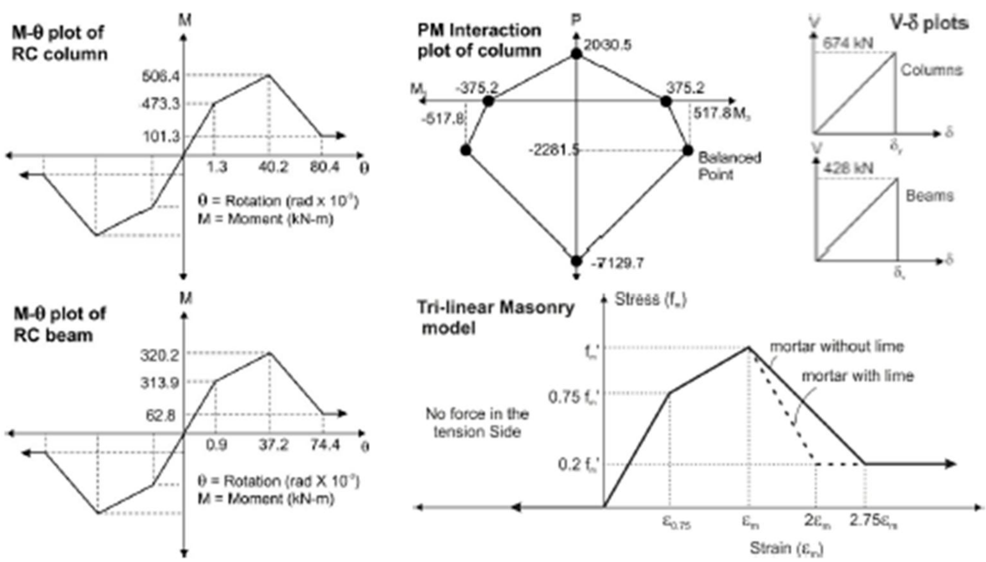


Figure 5. Typical plastic hinge properties of RC members and masonry infill

(kaushik et al (2006)[13,14]

The widths of the struts that were taken into consideration were those proposed by Holmes and Mainstone. The width calculated by Holmes equation (1) was found to be 33.63 inches and that calculated according to the Mainstone equation (2) was found to be 11.01inches.

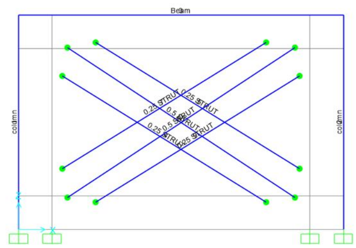
Non-linear pushover analysis requires assumption of probable hinge locations in the structure. We are assuming that hinges will be formed in the top and bottom of the columns. The hinges for the struts were assumed to form at the center of the struts. The hinge lengths for the columns were calculated according to the below given equation[4].

$$l_p = 0.08L+0.022d_bf_y$$

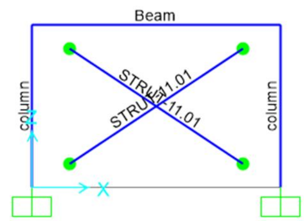
Where L is the length of the member in m, d_b is the diameter of the longitudinal steel in m and f_y is the yield strength of the longitudinal steel in Mpa. The hinge length for the strut was assumed to be three fourth of the length of the strut. The hinges properties were defined in SAP2000.The fiber P-M2-M3 model was used to define the hinges in the columns and an axial hinge was used for the struts. The hinge length was calculated as 75 inches for the struts and 10.2 inches as for the columns.

In the SAP model the columns and the beams were designed as 2-noded line elements. The column center to center distance was taken to be the length of the beam which was 92 in. The column height was taken from the base to the center of the beam which was 60.5in. The beams and the columns details were defined as described in the Mehrabi experiment shown in Fig3. The strut was created in the section designer and the width was defined accordingly and the thickness was taken as the thickness of the masonry wall which was 8 in.

Frame models for the Single and Three diagonal Struts as modelled in sap2000

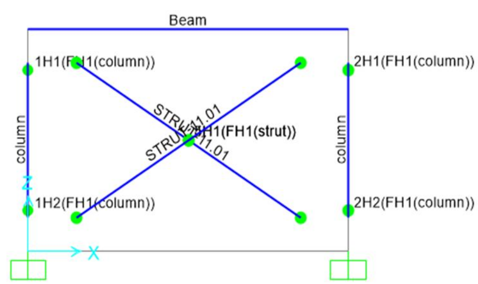


3 strut model



1 strut model

Hinge assignations for the RC members and the struts



D. Experimental Results (Mehrabi(1996)[1]

Figure below shows the experimental lateral load vs displacement relation for the masonry infilled frames as obtained by Mehrabi et al (1996)[1]

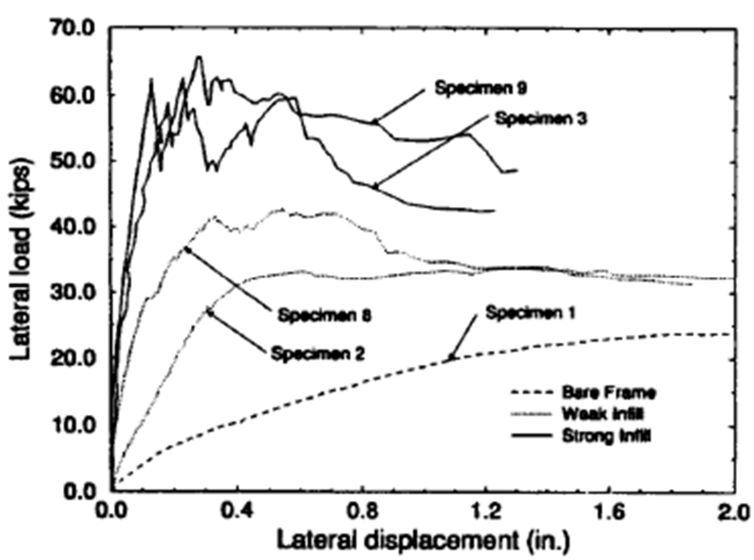


Figure 6: Lateral load vs displacement relationship for specimen 9 as obtained from experimental results (Mehrabi 1996)

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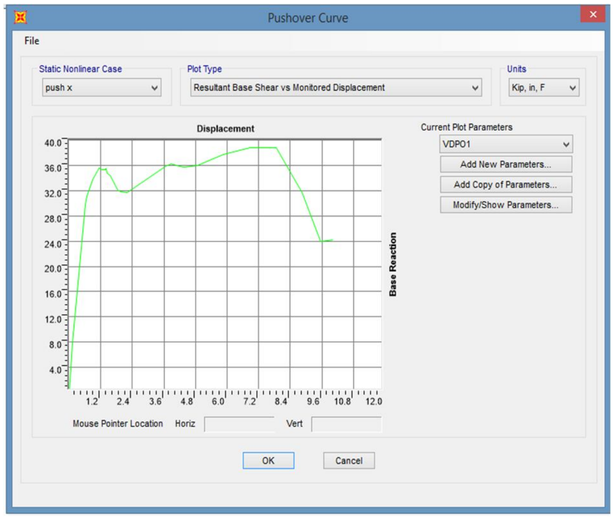
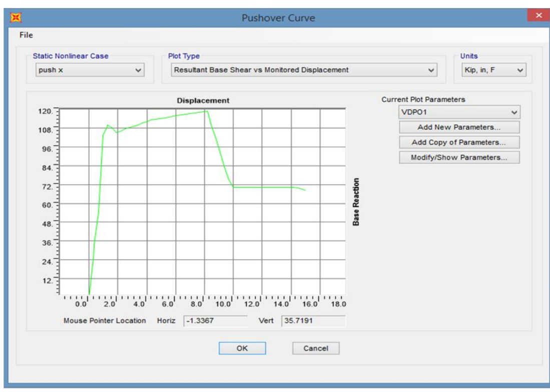
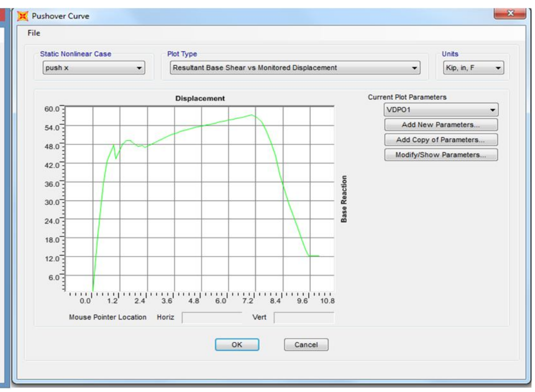
III. RESULTS OF THE PUSH OVER ANALYSIS

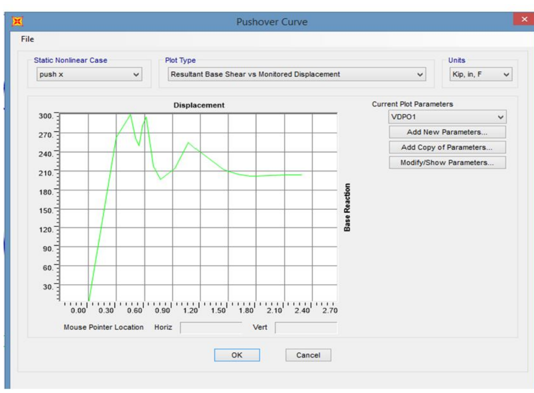


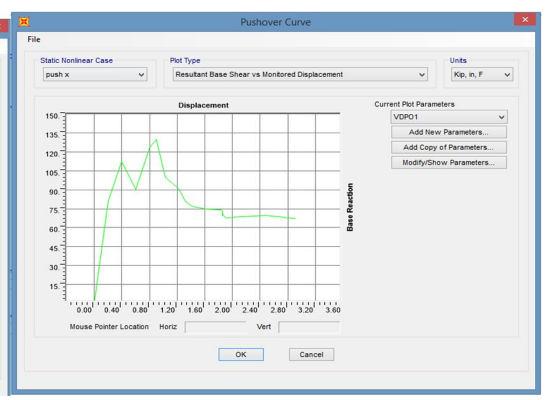
Model 1





Model 2 Model 3





Model 4 Model 5

Key

Model 1:- Concrete frame only model

Model 2:- Strut Width w = D/3 (Holmes 1961)[4]

Model 3:- Strut width $w = 0.16\ \lambda_h^{-0.3}d$ (Mainstone 1971)[5, 6]

Model 4:- 3 Strut model with w = D/3 (Holmes 1961)[4]

Model 5:- 3 Strut model with $w = 0.16\ \lambda_h^{-0.3}d$ (Mainstone 1971)[5, 6]. **All values are in Kips and inches**

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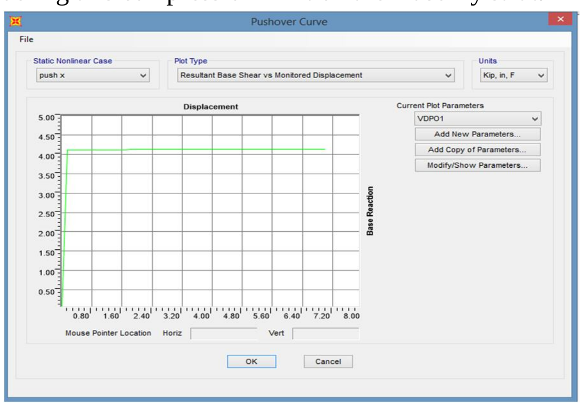
IV. SUMMARY AND CONCLUSIONS

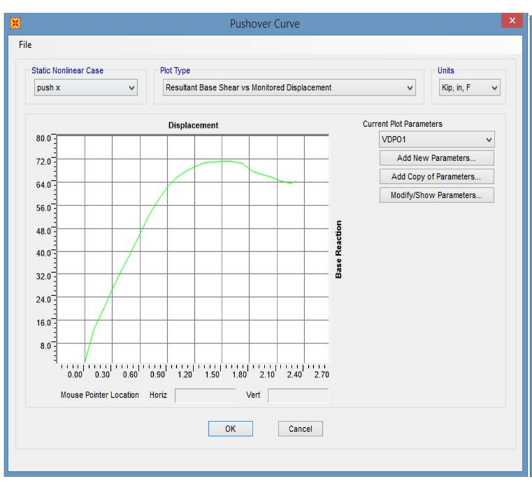
1) *Model 1:* This was the bare frame only model which was subjected to pushover analysis. As can be seen, the curve initially exhibit's the linear elastic behavior of concrete and then as the yield strength is reached it exhibits concrete plastic behavior.
2) *Model 2:* The model overestimates the lateral load capacity as compared to experimental results. As can be seen from the curve there is an initial peak in the curve, this maybe because the diagonal strut which is of considerable width is taking most of the lateral load as compared to the RC columns initially then as the yield point of the strut is reached there is a drop in the load and then due to plastic behavior of masonry together with the stiffness of the columns lateral load increases till the columns reach plastic state. The failure hinges were formed at mid-point of the diagonal strut and at the bottom of the columns.
3) *Model 3:* The pushover curve exhibits linear behavior for the most part of the displacement. The loads are in close relation to the experimental results.
4) *Model 4:* The lateral load carrying capacity was very large as compared to the experimental results. This may be due to the 3 struts which delay the hinge formation in the columns. The position of the struts along the column also largely effects the lateral load capacity. Although, the stiffness at first break was found to be similar to the practical results. The failure hinges were formed at the midpoint of the bottom diagonal strut and at the bottom of the columns.
5) *Model 5:* Similar results as model 4. The 3 strut model when modelled in SAP2000 overestimates the lateral load capacity and the stiffness values for the masonry in filled frames irrespective of change in width.

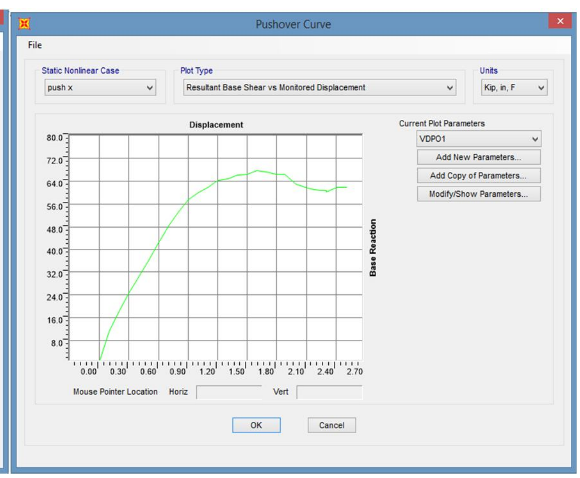
An interesting observation was made while modelling these strut models. Using the option of compression limit inbuilt in SAP200 we assigned the compression limit to our masonry strut and applied a pushover axial load on the strut element only. The following pushover curve was obtained which suggested that masonry strut behaved like an elasto-plastic material. Then push over analysis was performed for all the models considering this compression limit on the masonry strut.



Strut only model

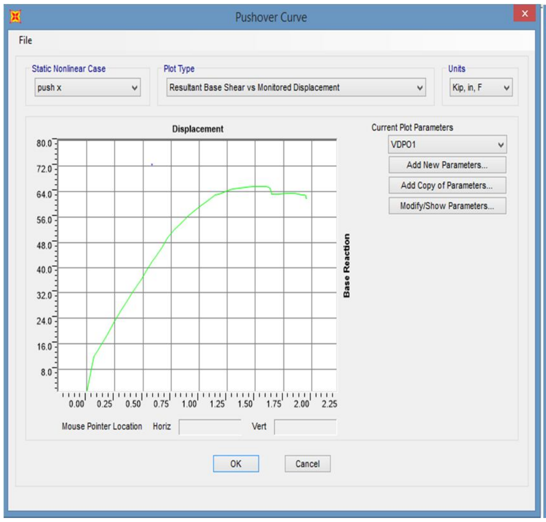
The following pushover curves were obtained for the single strut models



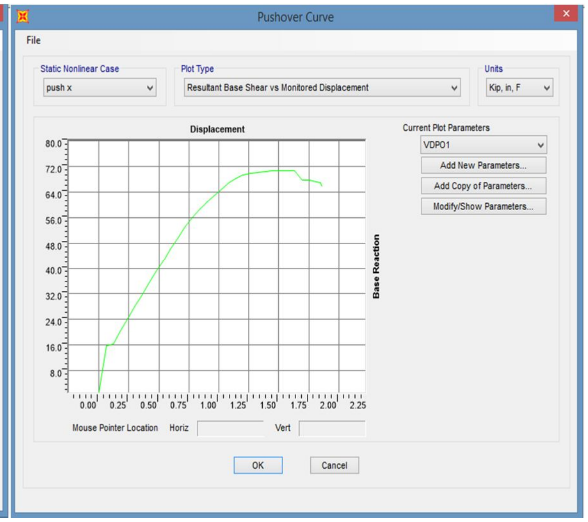


Model 2 Model 3

The following pushover curves for the three strut model was observed



Model 4



Model 5

It can be concluded that although masonry does not behave in this manner as we can observe from the non-linear stress strain curve for a masonry prism where it reaches its peak strength and reduces after that point, in the case of the modelling of the in-filled wall there might be various interactions happening which could not be accounted in our strut model. We can say that because the results shown above has a closer correlation to the experimental result, but we cannot confirm this without further research.

V. CONCLUSIONS

In general, it was found that both the single strut model and the 3 strut model when modelled in SAP2000 for the 2 widths considered overestimated the lateral load capacity as well as the stiffness of the in-filled frame when compared with the experimental results. The behavior of the in-filled frame under the lateral push over load also does not match the experimental observations. We can conclude that SAP modelling and analysis may not be a very reliable method to follow for the analysis of masonry in-filled walls in an RC Frame.

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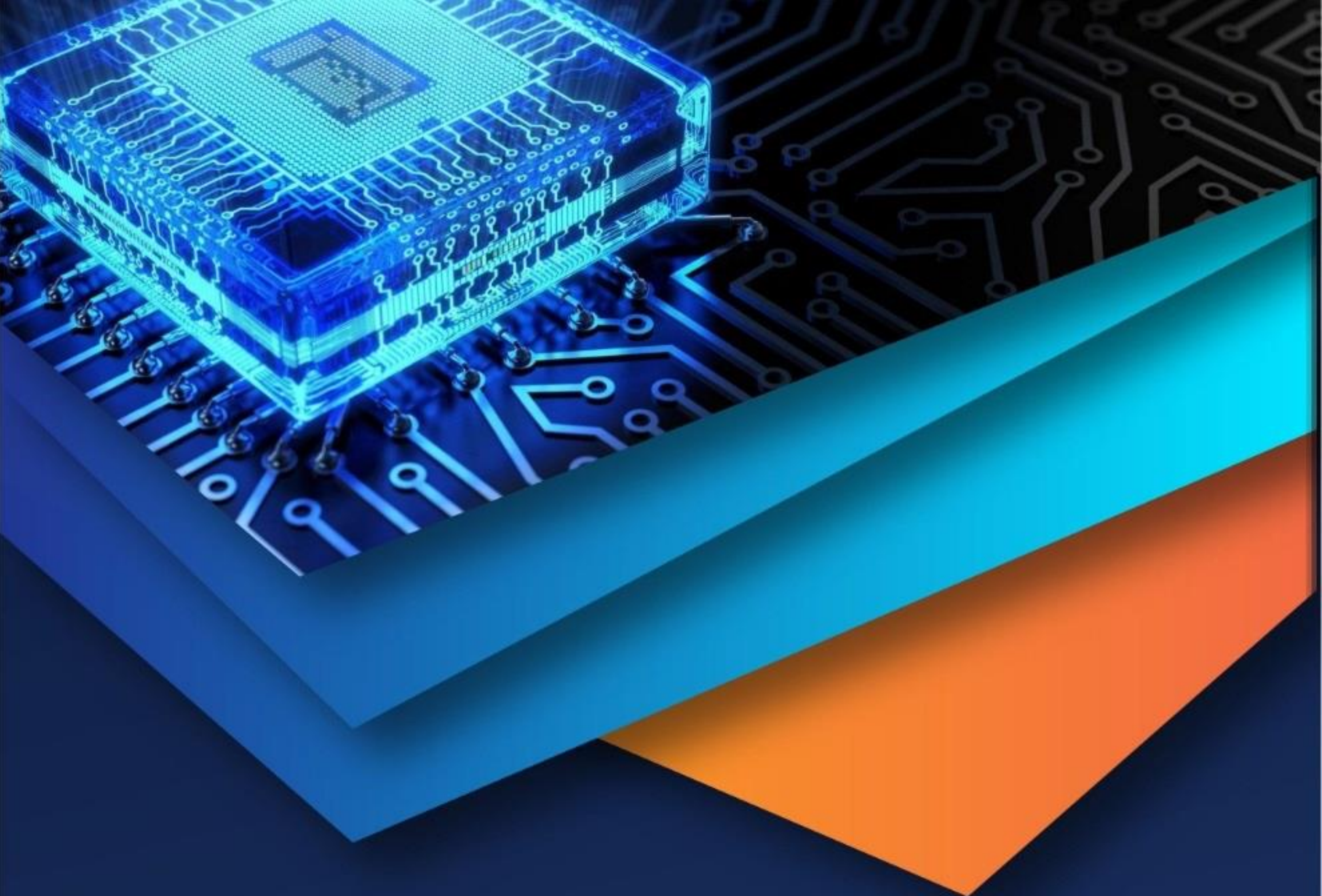

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