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"The Impact of Shear Wall Modeling Techniques on the Structural Response of Masonry Structures"; a Concise Comparative Discourse

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I. Abstract:

The local structural engineering industry customarily treats masonry infill walls as evenly distributed dead load applied on relevant peripheral beams. Alternately, such minimally reinforced walls are normally modeled as shell elements or occasionally neglected altogether. While the mentioned practice maybe satisfactory when dealing with gravity loads, it may lead to erroneous and misleading results when considering lateral forces such as wind or seismic excitations. In such instances, the lateral stiffness of the walls is neglected while they inevitably add to the mass of the structure. This potentially leads to adverse consequences resulting from the decrease in the associated fundamental frequency. Since seismic design is a requirement imposed by local governments and since Palestine lies on an active earthquake prone zone, the subject of adequate wall modeling within masonry structures deserves focused scrutiny. Moreover, in-fill masonry walls, whether reinforced or of plain concrete, characterize Palestinian vernacular construction. This considerably affects frame action. In a broad sense, concrete walls are considered as the first line of defense against seismic excitation. Furthermore, it is emphasized that structural modeling procedures profoundly impact, negatively or positively, the end result of any structural design undertaking. The following study brings such arguments into the lime light by focusing on a six-story single bay structure in which concrete walls are modeled in five different techniques, in addition to the reference bare frame model. The study is comparative in nature and limited to a strict numerical discourse targeting the response due to seismic excitation. Furthermore, a non-linear Pushover Analysis is carried out for two modeling techniques incorporated into a bare frame structure.

Keywords: Masonry Walls; Response Spectrum Analysis; Equivalent Struts; Base Shear; Performance Point. Performance Based Design.

II. Introduction and Problem Statement:

Masonry shear walls i.e. stone clad unreinforced infill concrete walls are rarely included in frame analysis and design although they profoundly influence the later; it is rather obvious to conclude that neglecting masonry walls in analytical models is not a realistic approach. Stone clad concrete walls are traditionally treated as nonstructural elements and excluded altogether from numerical models; they are traditionally treated as distributed dead loads on periphery beams. It is a general knowledge that stone clad concrete walls enjoy high stiffness and play a pivotal role in resisting lateral forces. The complexity of their inclusion often encourages the omission tendency in mundane structural system assessment undertakings. The noted complexity is due to material, construction methods and their associated behavior. The purpose of the present study is to examine the differences in the structural response associated with the complete or partial representation of masonry infill walls versus the representation by various form arrangement of struts. The two approaches referred to in literature are, the micro approach and the macro approach. The macro approach is favored over the micro approach due to its modeling simplicity and faster computing time. Here a masonry infill wall is replaced by an equivalent diagonal strut system. Equivalent struts replace the walls in the macro approach; in the micro approach, partial wall segments replace the masonry walls. In the present study, various infill walls modeling techniques are explored in a six-storey single-bay framed structure. The assemblage is further analyzed following a nonlinear static Pushover Analysis procedure. A comparative discourse is thus established.

III. Structural Shear Wall Modeling Techniques:

For the present study, a 2D Reinforced Concrete frame is considered. It is comprised of a single bay and six storeys; the storey height is set at four meters with a bay width of six meters. The dimensions selected are in line with vernacular building architectural modules. Furthermore, since concrete infill walls may constitute effective elements for resisting lateral forces, they are scrutinized analytically through the following comparative models constructed using ETABS 2016. In the present study, both the micro method and the macro method are investigated. Several investigators have examined the issue of selecting a proper strut width in the case of the macro method or the wall bandwidth for the micro method. Diana Samoila gave an extensive literature review [4] of all suggested modeling schemes available in the literature; the consensus is that Pauley and Priestly model has the edge. For the macro method and for the case of a one strut system, they have suggested the width of the diagonal strut to be equal to one fourth of the diagonal length of the infill wall. In the case of three struts the middle strut would be equal to one eighth of the diagonal length and the off diagonal ones eqaul to one sixteenth of the diagonal length. The location of the off diagonal struts is at α distance from one end of the beam.

$$\alpha\ (\mathrm{mm}) = \frac{\pi}{2}\sqrt[4]{\frac{E_c I_c h_m}{E_m t \sin(2\theta)}}$$

In which,

E_c is the modulus of elasticity in MPa for the column material
E_m is the modulus of elasticity in MPa for the masonry material
I_c is the moment of inertia of the column section in mm^4
h_m is the height of the masonry infill wall
t is the thickness of the masonry infill wall
θ is the angle in degrees of the inclination of the equivalent diagonal strut with the horizontal

The study begins by comparing the lateral displacements and the fundamental frequencies of the different numerical models of the structure comprised of columns, beams and the masonry walls. The frames' topology and material are shown in Table 1. To prevent the transfer of moment from the frame to the struts; moment releases are appropriately assigned to the ends. In the micro method, the in-fill walls are modeled as shell segments. However, in order to diminish resistance to out of plane bending a stiffness modifier of 0.1 is assigned to m_{11}, m_{12} and m_{22}. The following is a brief description of the models:

Model 1: A single equivalent strut i.e. a 2-node frame element with hinges at both ends represents the masonry wall; moment releases somehow remove coupling between the struts and the associated frames. The strut has minimal reinforcement.
Model 2: Three diagonal struts represent the masonry wall; each strut is a two-node element with a moment release at the ends. The struts have minimal reinforcement.
Model 3: The masonry wall is represented partially - one-half of the wall width.
Model 4: The masonry wall is represented partially - one quarter of the wall width.
Model 5: The masonry wall is modeled in its entirety.

Model 1 and Model 2 are of a macro nature while Model 3 and Model 4 are of micro nature. The strut widths are based on the suggestions presented by Pauley and Priestly. For a single strut $d_{if}/4$ =7.2/4 =1.8 m. For a three-strut arrangement, the width is 0.90 m for the middle strut while the width is 0.45 m for the other two struts. The side struts act at α = 1.1 meters from the edge. Sectional properties and relevant data used in the present exercise are shown in Table 1.

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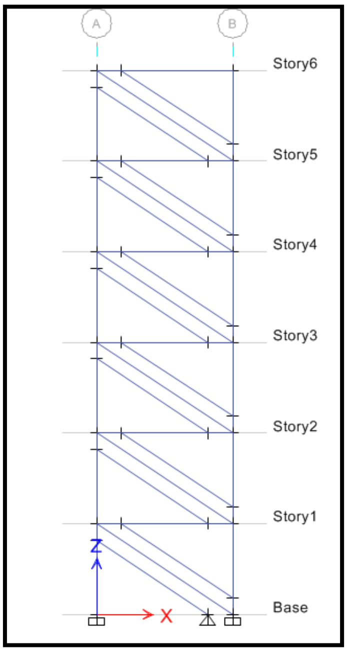
Story height	4 m
Beam size	40x25
Column size	50x25
Concrete Grade f'_c	28 MPa
Reinforcement Bars Grade	410 MPa
PGA	0.2
S_s	0.5
S_1	0.25
Response Reduction Factor	3.5
Modulus of Elasticity /Concrete [KN/m^2]	30x10^{6}MPa
Modulus of Elasticity /Masonry Material [KN/m^2]	4.5x10^{6}MPa
Poisson Ratio of Concrete	0.20
Poisson Ratio of Masonry	0.19

Table1: Basic Structural and Material Parameters



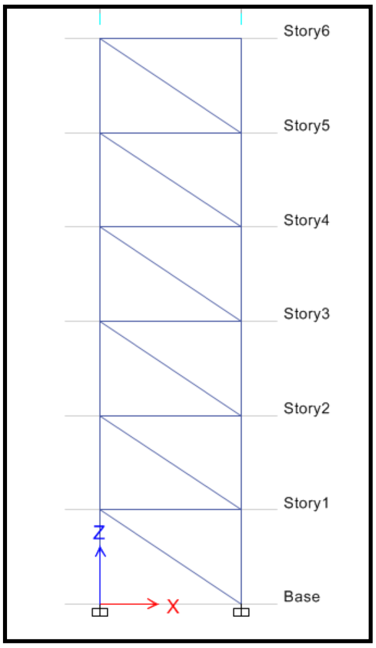


Figure 1: Model 1 and Model 2

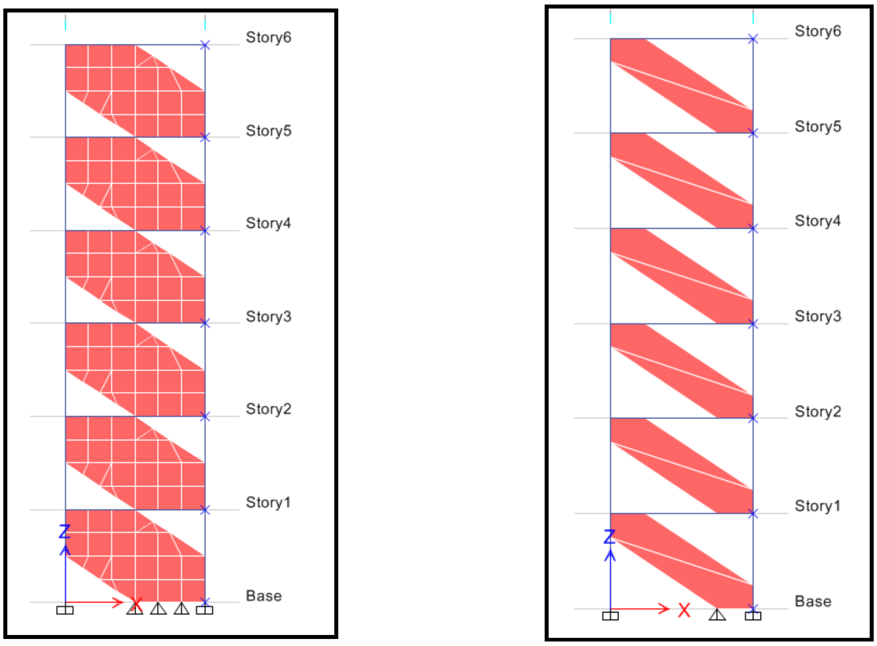
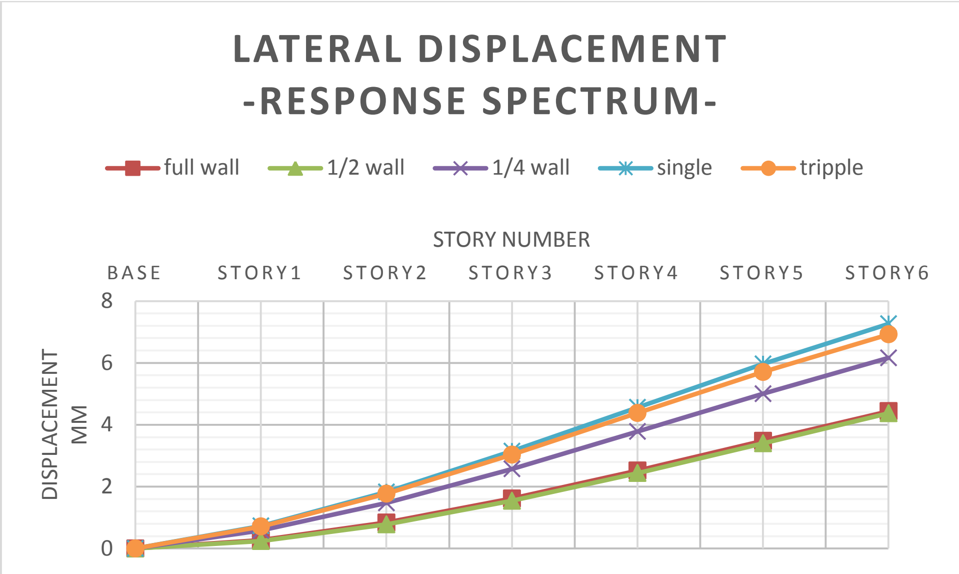


Figure 2: Model 3 and Model 4

IV. The Response Spectrum Analysis Procedure and Results:

The study proceeds by the Standard Response Spectrum procedure; this includes extracting the mode shapes, setting a mass source, defining a response spectrum function, selecting a proper scaling factor and finally carrying out a modal superposition. Twelve mode shapes are selected and the CQC method is set for modal combination while a SRSS is set for directional combination. Figure 3 shows a graphical comparison between the resulting roof displacements for five different numerical models. It is readily noticed that the model with a half width wall and that of the full width wall show almost identical results. This is because lateral forces tend to split the walls from the adjoining periphery beams; the effect continues with increasing lateral force magnitude until a quarter wall presents itself. Moreover, the single strut model and the triple struts model show quite comparable values. The results clearly show that at the macro level, a single strut is satisfactory while on the micro level the partial wall of one-quarter width of the span is adequate. Table 2 supports this conclusion. Computer time reduction and ease of analysis follow from the transformation of the system into a truss action.
Furthermore, for an added scrutiny of the possible modeling techniques the following is a behavioral comparison of selected two models using Performance Based Analysis.



Figures 3: A Comparison of the Roof Displacements in the Five Models

	Bare frame	Full wall	Single strut	Triple struts	Quarter wall	Half wall
Fundamental Period (sec)	1.56	0.274	0.368	0.37	0.271	0.368
Fundamental Frequency (Hz)	0.641	3.648	2.716	2.702	3.683	2.716
Maximum (mm) Displacement - RSX	50.8	4.439	7.26	6.923	4.371	6.16

Table2: Basic Structural Parameters of the Various Models

V. Pushover Analysis Method and Procedure:

Pushover Analysis is a static non-linear method that is essentially based on the Performance Based Design philosophy. The method is emerging as a popular method for the evaluation of the performance of existing or the newly designed structures; whence the structures are analyzed under permanent loads or monotonically increasing lateral loads which in fact represent, albeit approximately, seismically induced forces. With an increase in the applied loads, certain elements within the structure start yielding consecutively. The process results in the loss of stiffness and continues until failure. Detailed information is therefore possible concerning collapse and ductility of the structure itself and about its various components; this is achieved by assigning the so-called nonlinear hinges that have well defined backbone characteristics to the elements of the structure that are expected to show signs of yield. Hence, the real strength or weakness in a structure is adequately identified, located and eventually addressed.

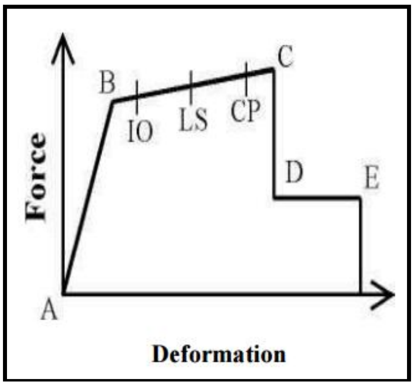


Figure 4: A Typical Hinge Load Deformation Backbone Curve

The five points A, B, C, D and E shown on the hinge backbone curve, Figure 4, define the force deflection behavior of the hinge while the three points IO, LS, and CP define the acceptance criteria for the hinge. Point A signifies the unloaded structure; point B shows the first yield of the element; point C shows the place where significant degradation begins, i.e. its nominal strength. The drop from C to D signifies the initial drop in strength of the element. The space between D and E allows the element to sustain gravity loads. Point E is the point where the maximum deformation takes place and no longer can sustain gravity loads. IO stands for Immediate Occupancy; LS stands for Life Safety; CP stands for Collapse Prevention. According to the ATC-40 methodology, a Performance Point is to be determined. This is the point where the capacity curve intersects the demand curve on the ADRS [Acceleration Displacement Response Spectra]. Such a plot merges the base shear versus displacement of a point at the roof with the Response Spectrum curve. If the point of intersection happens to be near the elastic range, the structure is judged satisfactory. If the intersection point leaves a modest reserve of capacity then the structure is pronounced weak and therefore would behave poorly under a strong seismic action. For the realistic prediction of the Performance Point, it is customary to subject the structure to earthquakes stronger than normally expected.

The Pushover displacement control procedure follows the protocol set by the ATC-40 and FEMA-273, which include the relevant modeling parameters and the acceptance criteria. In a well-designed structure, nonlinear hinges are located on all member ends. For the present exercise, the hinges are assigned at 5% and 95% of the length of all beams and columns. ETABS 2016 recommends an uncoupled moment hinge; i.e. M_3 hinges, for beams and an uncoupled axial force and biaxial bending moment hinge; i.e. PMM hinge, for columns. A control node is identified at the roof of the structure close to its center of rigidity, the displacement of which versus the base shear is the capacity curve. Moreover, an initial static non-linear PUSHDOWN load case under the full dead load is defined in order to form the initial condition for all subsequent Pushover load cases.

A crucial point in the procedure is the definition of the target displacement, which represents the roof displacement. The value forms an estimate for the expected global displacement the structure is expected to experience during a potential seismic action. The assigned target displacement for the present undertaking is 1000mm. A lateral pushover load case in the form of accelerations representing the force that would be experienced during a seismic action is applied to the structure in the x- direction, PUSHX. The lateral force is increased until some members yield. A plot of roof displacement versus base shear, i.e. the so-called capacity pushover curve, hence represents the capacity of the structure.
The outlined Pushover Analysis Procedure is applied to one macro model and one micro model in order to compliment the subject under investigation. The arrows in Figure 5 point to the Performance points.

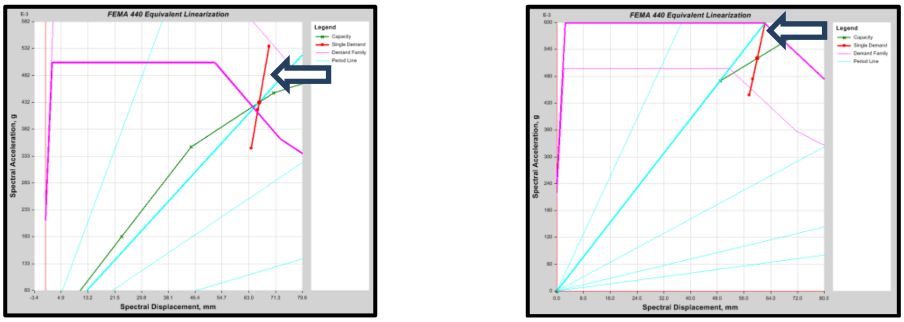


Figure 5: The SASD Curves for the Single Strut and the Partial Shell Models

VI. Conclusion:

The comparative study conducted for the various modeling schemes leads to the conclusion that for a strict linear analysis, replacing the concrete masonry infill wall by a single strut having sections with the described properties is judged as adequate representation and sufficiently accurate; it has a clear comparative benefits in terms of analysis time and modeling effort. However, from a nonlinear Pushover Analysis perspective, two exactly similar numerical models are investigated, a micro model and a macro model. Both models present parity in their general response and are equivalent to each other albeit the single strut enjoys a slight edge for having the Performance Point happening earlier on the FEMA 440 Equivalent Linearization, SASD curve. The Performance Point of the micro model is (316.7, 81.8) while that for in the macro model is (362.29, 75).This is in addition to reducing the entire infill concrete wall assemblage into a simple truss action.

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