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SEISMIC EVALUATION OF UN-REINFORCED MASONRY STRUCTURES

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***Abstract*— The objective of this paper is to study the methodology available in literature to evaluate the seismic vulnerability of un-reinforced masonry building and to check the applicability of these procedures through experimental studies. Sixteen wall panels of varying dimensions with and without openings were tested for in-plane monotonic lateral loads. The experimental results are compared with the results of existing pushover analysis method (ASCE/SEI 41-06) for URM building. The comparisons show that ASCE/SEI 41-06 method consistently overestimates the strength and stiffness of the URM buildings. A set of modification is proposed for the pushover analysis of URM building based on the experimental investigation. These proposed modifications show consistently good performance in comparison with the existing method (ASCE/SEI 41-06) of pushover analysis.**

***Keywords*— Seismic evaluation, un-reinforced masonry, pushover analysis, plastic hinge, shear stress.**

I. *Introduction*

It is well known that masonry buildings suffer a great deal of damage during earthquakes, leading to significant loss of lives. Almost 75% of the fatalities, attributed to earthquake in last century, is caused by collapse of buildings of which the greatest portion (more than 70%) is due to collapse of masonry buildings. A majority of the tenements in India are Unreinforced Masonry (URM) buildings that are weak and vulnerable even under moderate earthquakes. On the other hand, a cursory glance through the literature on earthquake resistant structures reveals that a bulk of research efforts is on RC structures. Clearly there is a great need to expend more effort in understanding masonry buildings subjected to earthquake induced dynamic loads. With this background the main objectives of this paper is defined as to assess pushover analysis methodology prescribed in ASCE/SEI 41-06 for unreinforced masonry buildings through experimental investigation and to propose improvement if required. To achieve this objective an experimental program has been carried out as part of this research. Sixteen wall panels of varying dimensions were tested for in-plane monotonic lateral loads. For each specimen a constant axial compressive load was maintained during testing.

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A window opening at prescribed location of the test specimen was provided for eight of the sixteen specimens and its in-plane monotonic lateral load behaviour was studied. Four additional specimens with a door opening in combination with a window opening were tested for their in-plane monotonic lateral load behaviour. Four solid walls without any opening were also tested and compared with the behaviour of similar panels with openings.

The experimental results are compared with the results of existing pushover analysis method (ASCE/SEI 41-06) for URM building. The comparisons show that ASCE/SEI 41-06 method consistently overestimates the strength and stiffness of the URM buildings. A set of modification is proposed for the pushover analysis of URM building based on the experimental investigation. These proposed modifications show consistently good performance in comparison with the existing method (ASCE/SEI 41-06) of pushover analysis.

II. *Experimental Program*

The experimental program was planned to study the effect of the presence of openings on the wall behaviour, when subjected to monotonic lateral loading. Details of the test specimens are shown in the Table I. In the table, 'S' stands for Solid Wall specimen whereas 'W' and 'D' denotes for the window and door opening respectively. Location and sizes for the openings in the specimen is considered as per the construction industry practice.

TABLE I. DETAILS OF TEST SPECIMENS

Specimen ID	Opening	Length, L (m)	Height, H(m)	Thickness, t (m)
W-S1	S	1.5	2.25	0.250
W-S2	S	2.0	0.80	0.250
W-S3	S	1.5	2.25	0.125
W-S4	S	2.0	0.80	0.125
W-01	W	1.5	1.50	0.250
W-02	W	1.5	2.25	0.250
W-03	W	2.0	1.50	0.250
W-04	W	2.0	0.80	0.250
W-05	W	1.5	1.50	0.125
W-06	W	1.5	2.25	0.125
W-07	W	2.0	1.50	0.125
W-08	W	2.0	0.80	0.125
W-09	D & W	1.5	1.50	0.250
W-10	D & W	1.5	2.25	0.250
W-11	D & W	2.0	1.50	0.250
W-12	D & W	2.0	0.80	0.250

Each wall panels were placed on a 300mm thick foundation. An ISMC 300 channel attached rigidly with the foundation was connected with the strong floor of the

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laboratory to transfer the load to the ground and a fixed support condition is achieved. Fig. 1 shows the details of the experimental set-up. Burnt clay brick of 230×110×70 mm sizes were used with cement-sand mortar of 1:5 proportion to make the wall specimens. Average compressive strength of the unit brick was found to be 13 MPa after 28 days of curing. Average compressive strength of cement-sand mortar was found to be 6.3 MPa after 28 days of curing. Three mild steel bars of 10 mm diameter were provided at the top of door and window opening to transfer the load.

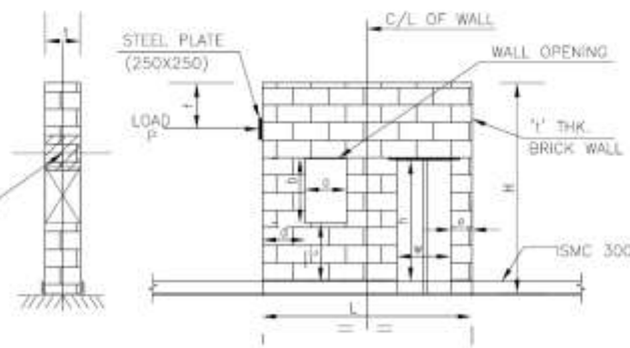

Figure 1. Details of the test set-up

Sand bags of required weight were used uniformly to apply constant axial load at the top of each test specimens. In-plane monotonic lateral loads were applied to the test specimens by hydraulic load cell attached to the vertical rigid wall.

III. *Results of In-Plane Monotonic Lateral Load Test*

Figs. 2 presents the load deformation behaviour of the a typical wall panel. Displacements were measured at two points: one at the top of the wall and the other at the mid-height. These figures show that, in most of the cases, the displacement at the mid height of the wall is more than half the top wall displacement. This indicates that there is a change in slope in at the mid-height of the wall. This may be due to the presence of opening at the mid height which is initiating failure.

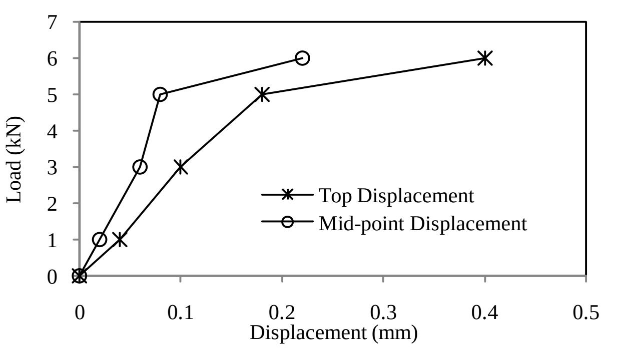

Figure 2. Load-displacement relation for Wall Panel W-S1

The experimental results presented above are tabulated in Table II. This table also compare the experimental results with the ASCE/SEI 41-06 recommendations. This is to be noted that the ASCE/SEI 41-06 recommendations are for the solid walls whereas the experimental results presented here are for both solid walls and walls with openings.

TABLE II. COMPARISON OF THE EXPERIMENTAL RESULTS AND ASCE/SEI 41-06 RECOMMENDATIONS

Specimen ID	ASCE/SEI 41-06 (Solid)		Experimental		
	K (kN/m) × 10^4	Q_{CE} (kN)	K_e(kN/m) × 10^4	Q_y (kN)	Δ_y (mm)
W-S1	7.34	2.53	2.78	5.0	0.18
W-S2	52.80	4.46	4.29	6.0	0.14
W-S3	3.67	1.27	2.00	4.0	0.20
W-S4	26.40	2.23	2.50	4.0	0.16
W-01	13.10	2.53	1.20	6.0	0.50
W-02	6.33	2.53	0.70	3.5	0.50
W-03	21.20	4.50	1.20	6.0	0.50
W-04	44.20	4.46	2.12	5.5	0.26
W-05	6.16	1.27	0.52	2.5	0.48
W-06	3.12	1.27	0.37	2.5	0.68
W-07	10.80	2.25	0.57	3.0	0.56
W-08	22.10	2.23	0.77	2.0	0.26
W-09	9.95	5.64	0.55	3.5	0.64
W-10	5.03	4.06	0.56	2.0	0.36
W-11	18.40	4.50	0.64	5.0	0.78
W-12	31.30	4.46	1.39	5.0	0.36

The experimental results show that introduction of opening in an URM wall reduces the wall stiffness significantly with a marginal reduction of strength. Fig. 3 presents typical crack pattern of the test specimen.

Figure 3. Typical crack pattern of the test specimen (W-10)

IV. *Pushover Analysis (ASCE/SEI 41-06)*

Pushover analyses were carried out for all the models as per the procedure outlined in the manuals ASCE/SEI 41-06. Fig. 4 present load-deformation responses of a typical wall as obtained from pushover analysis. The experimental results for corresponding walls are also shown for comparison. The responses for other fifteen walls are also found to be identical.

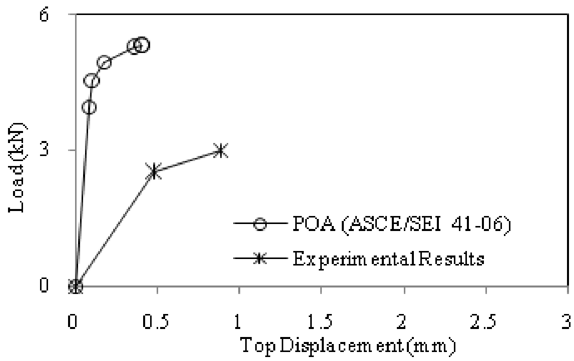

Figure 4. Comparison of experimental and pushover analysis results (W5)

This figure shows that the initial stiffness of the wall estimated by the nonlinear static analysis as per ASCE/SEI 41-06 is quite high. This leads to a lesser displacement response by the wall models. Also, the shear strength estimated by the nonlinear static analysis as per ASCE/SEI 41-06 is found to be more than the experimental results.

To predict the lateral load-deformation response of URM wall better through pushover analysis some modification over the pushover analysis procedure outlined in ASCE/SEI 41-06 is proposed. There are multiple modifications proposed with regard to structural modelling and hinge modelling of URM wall. The proposed modification is listed as follows:

i) When a two dimensional wall is divided into segments of piers and spandrels and modelled with one-dimensional line elements, the stiffness of the actual wall may get altered. Therefore to model a wall with one dimensional line elements requires suitable material properties that will keep the total elastic stiffness of the wall unaltered. To ensure this Young's modulus of the material is needs to be suitably modified to match the elastic modal properties of the two-dimensional wall segment. All other material constants should be kept similar to that of brick masonry.

ii) The piers and the spandrels should be modelled with cracked section modulus instead of gross section modulus. Cracked moment of inertia of URM wall is found to be 40% of the gross moment of inertia of the same section.

iii) The expected shear strength of URM wall can be divided in to two parts: first part is the strength coming from mortar-brick joint and the second part is due to the presence of axial force on the wall. However, ASCE/SEI 41-06 considers on the second part to calculate expected shear strength of the wall as shown in the following equation:

$$Q_{CE} = 0.9\alpha P_D \left(\frac{L}{h_{eff}} \right)$$

Here, Q_{CE} is the expected shear strength of the unreinforced masonry wall. α is a dimensionless coefficient (generally taken as 0.5), P_D is the axial force acting on the wall, L is the length and h_{eff} is the effective height of the wall. In contrary to this the experimental results show that there is a contribution of the mortar brick joint to the shear strength of a URM wall even when there is no axial force presents. To take this in to account the following relation is established by careful observation of the experimental results.

$$Q_{CE} = 0.9\left[\tau l_{eff}\, t + \alpha P_D \left(\frac{L}{h_{eff}} \right) \right]$$

Here, τ is the shear stress capacity of the unreinforced masonry wall generally taken as 1.75 MPa. l_{eff} is effective length of the wall (total length of the wall minus the length of the opening), t is the thickness of the wall. Also, under lateral load the axial stress in a wall may not be uniform over its cross section. Therefore, it is not proper to depend on the axial force too much for assessing the shear strength of a wall segment in a URM wall building. A value of $\alpha = 0.2$ is arrived using trial and error method to fit the experimental results presented here.

Pushover analyses carried out on all the wall models considering the above modifications. The resulting pushover curves were plotted with the experimental results and a typical plot is presented in Fig. 5.

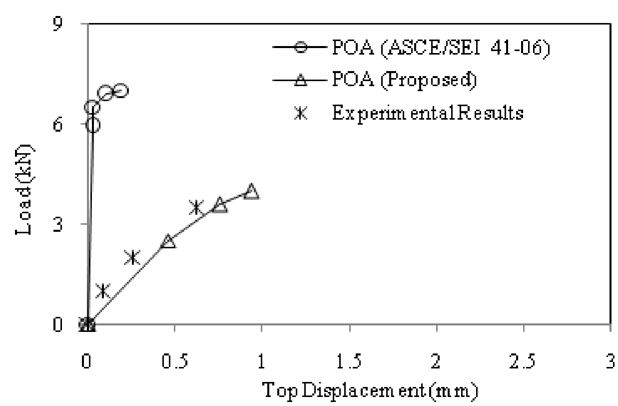

Figure 5. Comparison of capacity curves for Wall Panel W8

The figure presented here show that the results of pushover analysis with proposed modifications closely match the experimental results. It is to be noted that the proposed method slightly underestimates the base shear capacity (with a variation up to 10%), which is conservative.

"Pushover curve error index" (E_{PC}) is introduced as a measure of the discrepancy between the pushover analyses and experimental results in terms of base shear versus roof displacement relation. It is numerically simple and very efficient to define the difference between the ordinates of a pushover curve and the base shear versus roof displacement response obtained from the experimental results for the same wall panels. This is based on a similar concept due to standard error of displacement profile (Menjiver, 2004). A value of pushover curve error index approaching to zero implies high accuracy in the pushover analysis results (proximity to the experimental results). Table III presents the pushover curve error index for different frames for various load patterns used in pushover analysis. The table shows that proposed profile predicts results with more accuracy compared to the ASCE/SEI 41-06.

TABLE III. PUSHOVER CURVE ERROR INDEX

Wall ID	ASCE/SEI 41-06	Proposed
W1	0.86	0.14
W2	0.84	0.07
W3	0.86	0.10
W4	5.53	0.34
W5	1.11	0.12
W6	0.53	0.30
W7	1.29	0.12
W8	2.50	0.03
W9	1.13	0.07
W10	0.68	0.30
W11	2.34	0.12
W12	0.95	0.14
Mean Error	1.55	0.15

V. Concluding Remarks

Based on the work presented in this paper following point-wise conclusions can be drawn:

i) Modelling walls with plate element performs well in linear analysis but it is difficult to model nonlinear element properties with the plate modelling. Hence the URM building has to be modelled with equivalent frame (line) element for the non-linear analysis. The wall portion in between two openings should be considered as pier and the portion above and below the opening should be considered as spandrel. Width of pier is the clear distance between adjacent openings and depth of the pier is the thickness of wall. Similarly depth of spandrel should be the depth of wall segment available above or below opening and thickness is same as wall thickness.

ii) The total stiffness of the URM building is going to be altered (reduced) due to the frame modelling as the connectivity gets reduced in the frame model. To account for this reduction in stiffness Young's modulus of the material needs to be suitably modified in frame model to match the elastic modal properties of the URM building. All other material constants can be kept similar to that of brick masonry.

iii) The piers and the spandrels should be modelled with cracked section modulus instead of gross section modulus. Cracked moment of inertia of URM wall is found to be 40% of the gross moment of inertia of the same section.

iv) Experimental results show that the pushover analysis procedure given in ASCE/SEI 41-06 for URM wall panels is un-conservative for strength and stiffness estimation.

v) The expected shear strength of URM wall can be divided in to two parts: first part is the strength coming from mortar-brick joint and the second part is due to the presence of axial force on the wall. However, ASCE/SEI 41-06 considers on the second part to calculate expected shear strength of the wall as shown in the following equation:

$$Q_{CE} = 0.9\alpha P_D\left(\frac{L}{h_{eff}}\right)$$

Here, Q_{CE} is the expected shear strength of the unreinforced masonry wall. α is a dimensionless coefficient (generally taken as 0.5), P_D is the axial force acting on the wall, L is the length and h_{eff} is the effective height of the wall. In contrary to this the experimental results show that there is a contribution of the mortar brick joint to the shear strength of a URM wall even when there is no axial force presents. To take this in to account the following relation is established by careful observation of the experimental results.

$$Q_{CE} = 0.9\left[\tau l_{eff}\, t + \alpha P_D\left(\frac{L}{h_{eff}}\right)\right]$$

Here, τ is the shear stress capacity of the unreinforced masonry wall generally taken as 1.75 MPa. l_{eff} is effective length of the wall (total length of the wall minus the length of the opening), t is the thickness of the wall. Also, under lateral load the axial stress in a wall may not be uniform over its cross section. Therefore, it is not proper to depend on the axial force too much for assessing the shear strength of a wall segment in a URM wall building. A value of $\alpha = 0.2$ is arrived using trial and error method to fit the experimental results presented here.

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