

"The Seismic Susceptibility of RC Structures with Stone Clad Façades" A Post-Yield Perspective

Samir H. Helou Ph. D., P.E.
Civil Engineering Department
An-Najah National University
Nablus, Palestine

Abstract - Reinforced Concrete buildings with stone clad façades form the backbone of mundane construction in Palestine loosely labeled masonry construction. Moreover, since seismic analysis is an increasingly imposed requirement by local governments and due to the proximity to the Dead Sea fault, the country therefore lies in an active earthquake prone region. The subject of adequate scrutiny of the present construction practices demands proper attention. Local vernacular masonry construction practice generally involves façades primary comprised of a five- centimeter thick stone cladding and about 15 centimeters of unreinforced or minimally reinforced concrete backing; the so-called infill walls. Concrete grade for infill wall panels is normally of modest compressive strength. This is contrary to the general notion that masonry infill walls are comprised of concrete blocks. In the structural design undertakings, the infill walls are considered nonstructural elements, seldom included in the structural design practice although the simplified bare frame structural analysis and design remains inconclusive; the underlying reason for the omission is to avoid a somewhat complicated engineering. Towards this objective, the present study targets the susceptibility of masonry structures to seismic events. The selected structural topology for the present scrutiny is representative of the various forms frequently adopted by local architects for multi-family dwelling. The edifice selected is a representation of the popular medium to low rise structures; it has a uniform floor plan with an X-shape. It is a G+4 structure with stone clad façades having window openings judiciously distributed around the periphery walls. The study scrutinizes the behavior of bare frame structures and structures with stone clad walls under different study scenarios. The investigation covers also the impact of the soft story in the ground level on the selected structure. Moreover, the study scrutinizes the adequacy of modeling such stone clad walls by a compression only, single or double equivalent strips. Linear structural analysis and static nonlinear analysis, which is becoming increasingly significant, i.e. pushover analysis form the primary focus. This presents a comprehensive discourse that includes post yield behavior. The results compared are, inter alia, the standard dynamic parameters such as the time-period, the story drift, the lateral displacement and the base shear magnitudes as well as the Performance Point. The seismic analysis is conducted based on the standard Response Spectrum Method; the nonlinear analysis is conducted according to the Capacity Spectrum Method of ATC 40 and Fema 440 guidelines. The present study elaborates on the adequacy of the common construction trends; evaluate the potential seismic susceptibility and offers prudent design recommendations. Inner partition walls are beyond the scope of the present study.

Keywords— *Irregular Structure; Response Spectrum Analysis; Stone Clad Wall Panels; Base Shear; Strut Elements, Performance Point.*

I. INTRODUCTION

It is a mandatory practice in the state of Palestine, like elsewhere in the Middle East, that building structures incorporate stone clad façades; this is an esthetic requirement imposed by local governments in respect to traditional construction. Therefore, it is a ubiquitous architectural and a general construction practice. Furthermore, designing a seismically resistant structure is obligatory according to local government bylaws because Palestine lies well within an active earthquake prone zone proximal to the Dead Sea fault. Seismic resistance is imperative for avoiding unwarranted repair cost and for protecting against loss of life. The present study exercise tackles the favored low to medium rise structures having the aforementioned features, namely stone clad façades; yet it focuses on their vulnerability during seismic events. Exterior walls possess a significant role in resisting lateral loads and deserve adequate consideration in any structural design undertakings. In order to analytically investigate the influence of stone clad walls on the general response of structures during seismic events, a repertoire of different modeling techniques is followed to model the selected G+4 edifice. They are a bare frame, a frame with stone clad walls modeled as an added mass, masonry walls modeled as a distributed load on the periphery beams and as actual minimally reinforced walls with window openings. The models are characterized by being simple yet reliable. The study further considers the impact of the Soft Storey on such structures. Exterior walls are customarily unreinforced or minimally reinforced, better known as infill walls; they are normally comprised of modest quality concrete. For the present study, reinforced wall panels with minimum reinforcements and unreinforced are considered which emulate standard practice. The topology of the selected structure and the floor height is representative of most local apartment buildings. The window-opening ratio in all exterior walls is set at 25%. The analysis results include, but are not limited, to the fundamental periods, the natural frequencies and the Storey displacements. Furthermore, the study scrutinizes the validity of modeling stone clad walls by a single or double compression-only equivalent struts. The plan selected is a symmetric one in order to avoid the effect of potential torsion modes.

The present narration is based on seismic analysis by the standard linear elastic methods, i.e. the Response Spectrum Method. The performed non-linear Pushover Analysis of ATC 40 (1996) investigates the trend of progressive failure that may occur and identifies potential weak zones likely to reach critical states during seismic events. Pushover analysis

provides information on the collapse mechanism if when it happens. Soil structure interaction is beyond the scope of the present investigation. Due to strict planar symmetry, the Non-Linear Pushover analysis is performed in one direction only. The exercise is conducted using the Finite Element Method based software ETABS 2019.

NUMERICAL MODELING OF THE STRUCTURES:

A symmetrical yet medium rise structure is investigated. The structure is comprised of a G+4 floor levels having five bays in both directions with a uniform grid of 5-meter spacing. The height of each Storey is 3.2 meters while the flooring system is comprised of 13-centimeter thick slabs resting on drop beams. Figure 2 shows a 3D view of the numerical model created by ETABS 2019. Columns and beams are modeled using two node linear frame elements. When equivalent Struts are to represent stone clad wall panels they are modeled with full moment releases at both ends. A single strut as well as double struts are used separately in order to determine the better emulation of stone clad infill walls. Window opening area amounts to 25% of each wall panel. A rigid diaphragm action acts on all slabs to ensure integral lateral inclusion of all beams of each floor and forces all joints connected to the slabs to displace equally. The Response Spectrum curve selected is a smooth one with soil profile C and a damping of 5%. The loads and the architectural modules are in accordance with standard vernacular building predilection. Effective stiffness is set following Fema 356 guidelines for. [Beams $0.5 E_c I_g$; Columns $0.7 E_c I_g$; Walls $0.7 E_c I_g$; Slabs $0.25 E_g I_g$]. Table 1 shows pertinent structural and material data. The structure selected is a vertically regular one having stone clad infill walls. Window openings are assigned symmetrically around the building. A total fixity is assigned for all ground supports. The following study, linear and performance based, focuses on the following seven different numerical models. They are:

- 1) A Bare Frame (Model 1)
- 2) A Frame with Stone Clad walls modeled as a continuous load on relevant beams (Model 2)
- 3) A Frame with Stone Clad walls modeled as an added mass on relevant beams (Model 3)
- 4) A Frame having actual Stone Clad infill walls with openings (Model 4)
- 5) A Frame with infill walls with a soft Ground Storey (Model 5)
- 6) A Frame with Stone Clad walls modeled as a single compression-only equivalent diagonal struts. (Model 6)
- 7) A Frame with no struts in the Ground Storey thus emulating a Soft Storey. Single struts for the rest of building. (Model 7)

Models 6 and 7 are replicated to include double compression-only struts; they are Model 8 and Model 9. The thickness of the equivalent unreinforced struts is the same as that of the thickness of the stone clad wall. However, the strut width, according to the macro-modeling approach based on the suggestions presented by Pauley and Priestly.

$$W_s = \rho W_d$$

Where W_d is equal to one fourth of the diagonal length and ρ is a correction factor given as follows:

$$\begin{aligned} \rho &= 1 & \text{if } A_r \leq 0.05 \\ \rho &= 1 - 2.5 A_r & \text{if } 0.05 < A_r < 0.4 \\ \rho &= 0 & \text{if } A_r > 0.4 \end{aligned}$$

Where A_r = Opening Ratio

The present conditions produce a strut width of 56 centimeters. The mass source is defined as the entire Dead Load plus the Superimposed Dead Load in addition to 30% of the applied Live Load. The defined Pushdown load case is a nonlinear Static analysis is a force controlled one while the defined Pushx load case is displacement controlled monitored by the displacement of a roof node located near the center of gravity of the building. The lateral load is selected as proportional to the first mode of vibration. This is recommended for medium rise buildings. For the present study Infill Stone clad wall panels are defined of concrete having $f'_c = 16$ MPa. The following are pertinent data:

- Live Load = 2 KN/m²
- Superimposed Dead Load = 4 KN/m²
- Importance Factor = 1 [ASCE Table 1.5-2]
- Risk Category II; Soil Profile C; Seismic Design category D
- Ordinary Moment Resisting Frame, $R = 3.0$
- Over Strength Factor, $\Omega_o = 3$
- Deflection Amplification Factor, $C_d = 2.5$
- Structural Damping: 5%
- $A_r = 0.3$

Storey Height	3.2 m
Beam Size	30 cm x 40 cm
Column Size	40 cm x 40 cm
Slab Thickness	13 cm
Infill Wall Thickness	15 cm
Concrete Grade M35 (f'_c)	28 MPa
Concrete Grade M20 (f'_c)	16 MPa
Reinforcement Bars (F_y)	410 MPa
S_s	0.60
S_l	0.14
Response Reduction Factor	3.0
Modulus of Elasticity of Concrete [KN/m ²]	30×10^6 MPa
Poisson Ratio of Concrete	0.20

Table 1: Model Description, Material and Earthquake Data

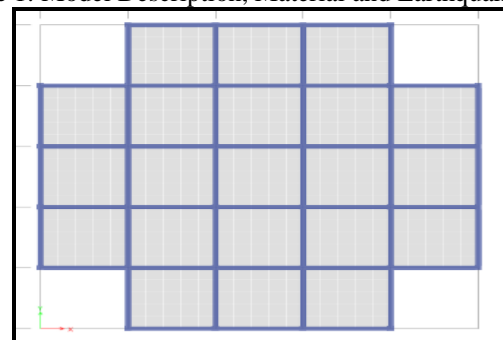


Figure 1: Plan of the Investigated Structure

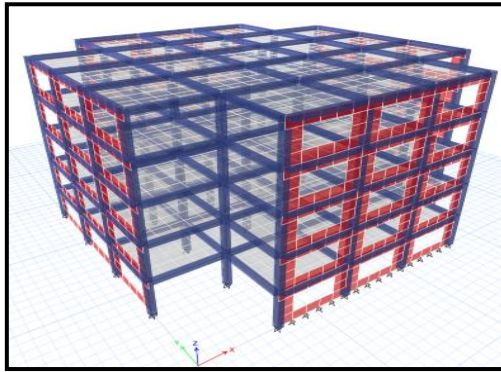


Figure 2: A 3D of the Investigated Structure

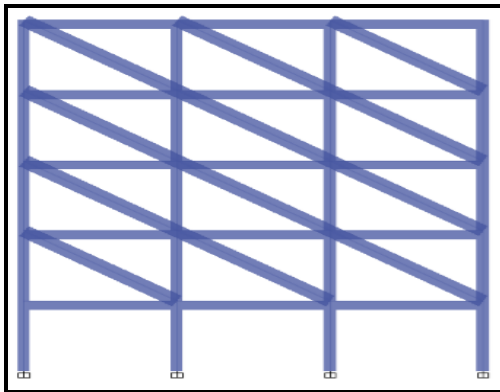


Figure 3: Single Diagonal Struts on one Elevation of the Structure with a Soft Storey

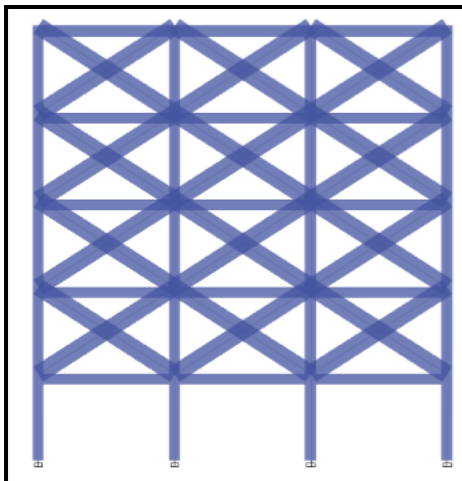


Figure 4: Double Diagonal Struts on one Elevation of the Structure with a Soft Storey

THE LINEAR ANALYSIS PROCEDURE

The Standard Response Spectrum procedure includes extracting the proper number of mode shapes, setting a mass source, defining an appropriate Response Spectrum function, selecting a suitable scaling factor and finally carrying out modal superposition. Twelve mode shapes are selected for the present discourse, ample to capture 90% contribution of the total mass, and the CQC method is set for modal combination while a SRSS is set for directional combination. The seismic mass source is defined as 25% of the applied live

loads added to the entire self-weight of the structure together with the superimposed dead load. The analysis is initially performed by the Equivalent Lateral Load Method. This is mandatory for better quantifying the scaling factors in the two principal directions. Following ASCE 16.3.4 a 5% accidental eccentricity is called for.

RESULTS OF THE LINEAR ANALYSIS:

Table 2 and Figure 5 exhibit the Fundamental Period of all models. It is clear that no difference exists in the fundamental period between Model 1 and Model 2; Eigenvalues are independent of loads. Model 3, presents a slight difference due to the definition of the mass source. In Model 4 however, the infill wall contribution to stiffness is clear; it changed considerably in Model 5 when Soft Storey action comes into play. Furthermore, scrutinizing the Strut Models it is clear that for the fundamental period of the 2-Strut Models come closer to the values of Models 4 and 5. Figure 6 illustrates that the base shear in the 2-Strut Model is closer to the values manifested in the actual infill wall models, Model 4 and Model 5. In other words, the inclusion of infill wall panels caused a significant increase in stiffness.

Moreover, The lateral displacement due to Seismic Force resulting from the Response Spectrum Analysis shows very close values in Model 4 and in Model 8. The same observation is manifested between Model 5 and Model 9. This is although the lateral displacements in Models 6 and 7 are within the same order of magnitude. Similar conclusions are drawn from Figure 9 vis a vis the Lateral Storey Drift. Model 8 and Model 9 are therefore used for the subsequent discourse. Model 4 through Model 9 present reduced lateral deflection than Model 1, Model 2 and Model 3 due to their substantially higher Storey stiffness; Figure 8.

Table 2: The Fundamental Periods and Base Shear for the Study Models

	T (Sec)	BS (KN)
Model 1	2.08	1080
Model 2	2.089	1093
Model 3	2.174	1215
Model 4	0.316	4000
Model 5	0.772	1623
Model 6	0.485	2466
Model 7	0.842	1415
Model 8	0.363	3370
Model 9	0.799	1524

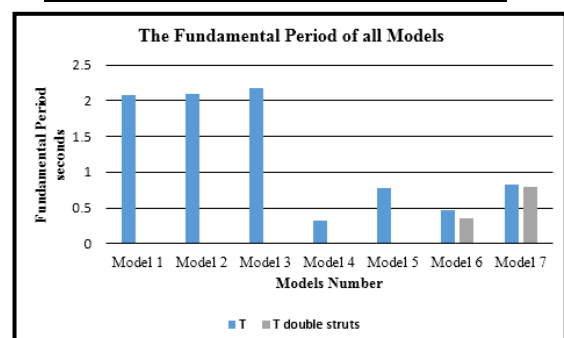


Figure 5: Variation of Fundamental Period

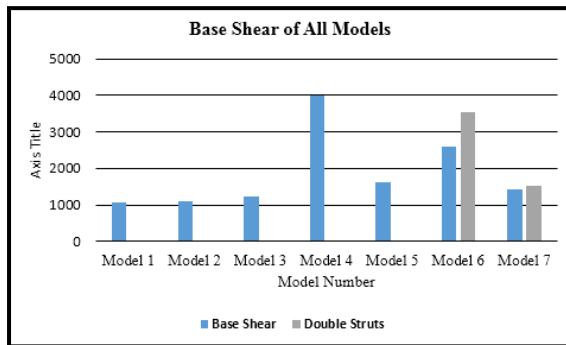


Figure 6: Variation of Base Shear

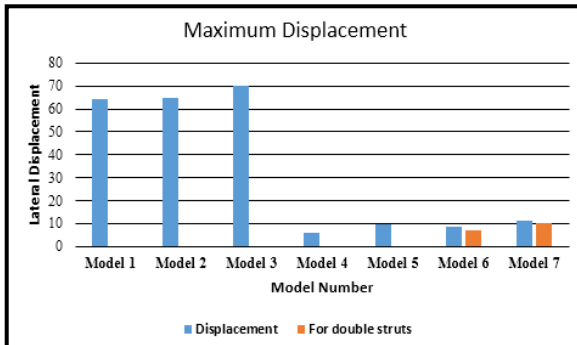


Figure 7: Maximum Displacements

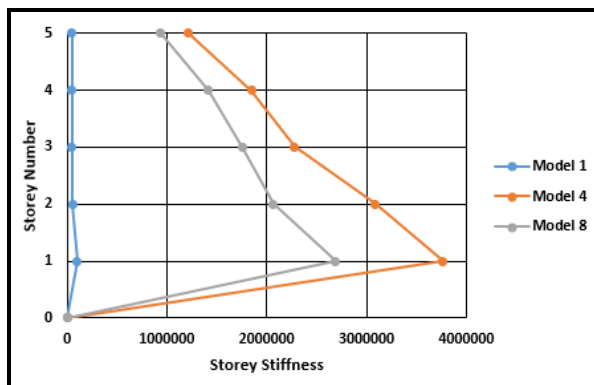


Figure 8: Storey Stiffness

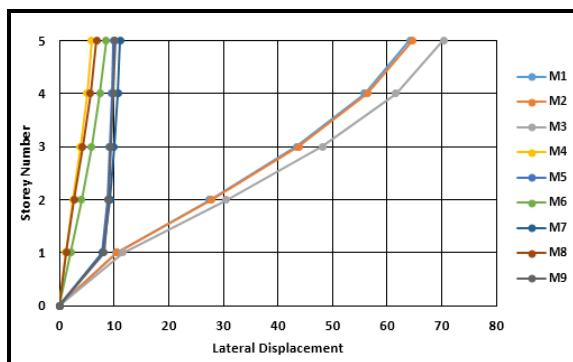


Figure 9: Lateral Displacement

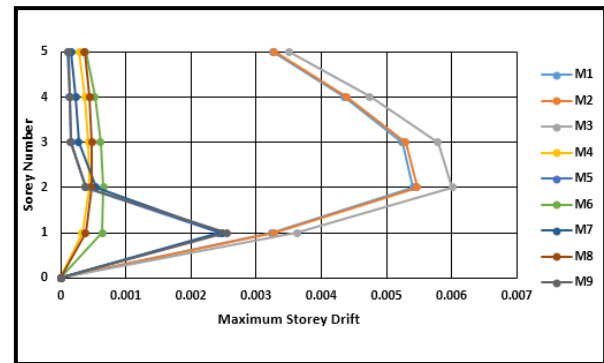


Figure 10: Maximum Storey Drift

THE NONLINEAR STATIC PUSHOVER ANALYSIS PROCEDURE:

The objective of the present study is to predict, beyond yield, the structural behavior under strong seismic events. This is a complex undertaking primarily due to the gradual change of section properties during deformation; this prompts continuous change in seismic forces due to the progressive change that occurs in the structural period and its damping. Pushover Analysis is the numerical implementation of Performance Based Design philosophy. Here static analysis is used to predict nonlinear behavior of structures. The method maybe force controlled or displacement controlled. The following discourse is a displacement-controlled procedure known as the Capacity Spectrum Method.

For a well-designed structure, non-linear hinges are added to all structural elements. For columns and from ASCE 41-17, coupled axial flexural or PM_2M_3 frame plastic hinges are assigned at 5% and at 95% of the element length. In beams, M_3 frame plastic hinges are assigned at the same relative locations; i.e. where cracks are generally expected to happen. For the struts, axial hinges are assigned at mid length of every strut. For the outer stone clad walls fiber hinges are assigned, PM_3 . Separate investigation is performed for unreinforced walls as well as walls with minimum reinforcement. The structure is then pushed laterally by a monotonically increasing invariant force pattern until a maximum target displacement of a node at the roof level is reached. P-Delta effects are properly considered. The target displacement is decided at 4% of the building height. The master node selected is close in location to the center of gravity at the roof. A plot of base shear versus roof displacement is hence generated. The curve represents the inelastic behavior of the structure or the inelastic load carrying capacity of the structure.

Prior to pushing the structure, a Pushdown non-linear static load case is defined; it involves the entire Dead Load in addition to 30% of the applied Live Load. The Lateral Load pattern applied is proportional to the first mode of vibration as recommended for medium rise buildings (OGUZ 2005). Pushdown load case is force controlled while the latter is displacement-controlled; the process continues until the target displacement is reached. The Pushover curve and the ground motion Response Spectrum representing the seismic demand are then converted to an ADRS format [Acceleration-Displacement Response Spectrum]. The intersection of both

curves defines the Performance Point or the maximum inelastic displacement.

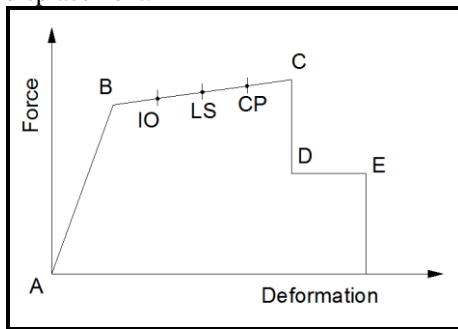


Figure 11: A Typical Hinge Load Deformation Backbone Curve

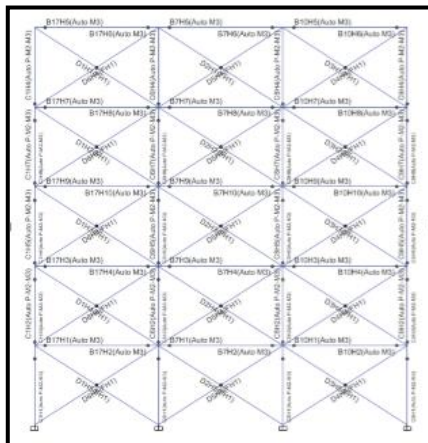


Figure 12: Location of Non-Linear Hinges in all Models

A nonlinear hinge represents the localized force-displacement relation of a member through the elastic and inelastic phases under seismic action. The five points A, B, C, D and E shown on the hinge backbone curve, Figure 10, 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. When the Performance Point is near the elastic range, the structure is considered satisfactory. Conversely, when the point of intersection leaves a modest reserve of capacity then the structure is pronounced weak and 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.

RESULTS AND CONCLUSION:

The purpose of the present investigation was to predict the behavior of structures having the so-called masonry infill walls within the elastic range as well as at post yield. The Software Etabs 2019.0.0 was used to construct nine different

numerical models for a G+4 building; the plan shape is representative of local architectural forms. In the process, buildings with a soft ground Storey were included in the discourse. The feasibility of modeling such stone clad wall panels by compression only single or by double equivalent struts was scrutinized. Linear analysis was conducted via the Response Spectrum Method while the Nonlinear Pushover Analysis procedure was followed to predict the behavior at post yield. Moreover, since the customary practice is to subject the structure to a stronger earthquake than the structure is designed for, the following results are for an earthquake with $S_1 = 1.08$ and $S_5 = 0.23$. These are the highest values recorded in Palestine. The following are the conclusions:

The Palestinian practice of masonry structures is not technically masonry except for the interior partition walls. The so-called masonry structures are essentially comprised of unreinforced concrete infill wall panels clad with stones. In-fill masonry walls are usually comprised of concrete blocks, hollow or otherwise, built in courses using mortar. In both instances, infill walls are constructed after concluding the skeleton works.

- 1) Marking the Performance Point is one pivotal result of Pushover Analysis. The following are the Performance Points for the three different bare frame models.

Model 1: (4220, 255)

Model 2: (4548, 271)

Model 3: (4769, 288)

The values above indicate that for the three models the Performance Points are close. Modeling walls as continuous load or as added masses does not appreciably influence behavior prediction.

- 2) While equivalent struts are indicative for the linear analysis. They are slender and show collapsed hinges early in the pushing phase of the nonlinear Pushover Analysis.
- 3) Figures 12 shows the FEMA 440 EL curves for the cases when the building is with reinforced infill walls while Figure 13 shows the same yet with unreinforced walls. It is clear that for the unreinforced masonry walls no Performance Point is identified for the earthquake level considered. This implies that the unreinforced masonry infill walls make the structure more susceptible to earthquake excitations. This is due to the absence of ductility. Unreinforced walls add substantially to the stiffness and strength, shown in Figure 8 yet ductility is compromised thus promoting brittle failure.
- 4) The models with full infill reinforced concrete stone clad panel walls enjoy the lowest collapse risk. Bare frames are the most vulnerable. The maximum sustained shear force leads to this conclusion. Models with struts prove ineffective, as struts are generally slender.
- 5) For the case of a soft Storey, the Performance Point for the design level earthquake and unreinforced panels is (6172, 40) while it is (8333, 56) with

reinforced infill panels. This exhibits the added advantage of reinforced panels.

- 6) The difference in behavior between reinforced infill wall panels and unreinforced one is exhibited in the Figure 14. loss of overall structural ductility.
- 7) Comparing structures with unreinforced masonry wall panels with partially in filled structures i.e. with a soft story leads to the conclusion that the later structure is safer during strong seismic events. This underlines that added stiffness with reduced ductility is detrimental. This is exhibited in Figure 14 and Figure 14. This feature is obscure within the elastic range.
- 8) Numerical modeling as well as scaled test modeling of masonry structures with stone clad infill wall panels is an open field for further investigation.

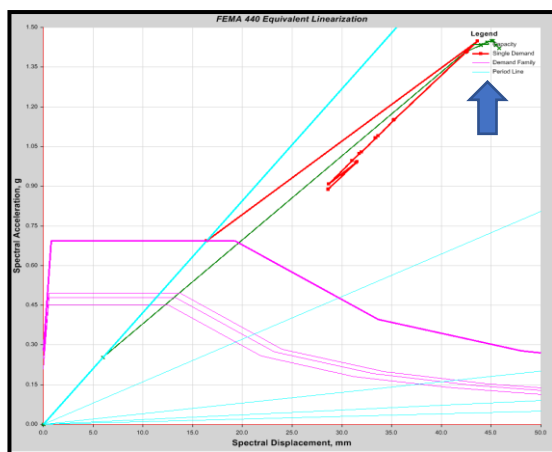


Figure 12: Fema 440 EL for a Structure with Reinforced Stone Clad Panels

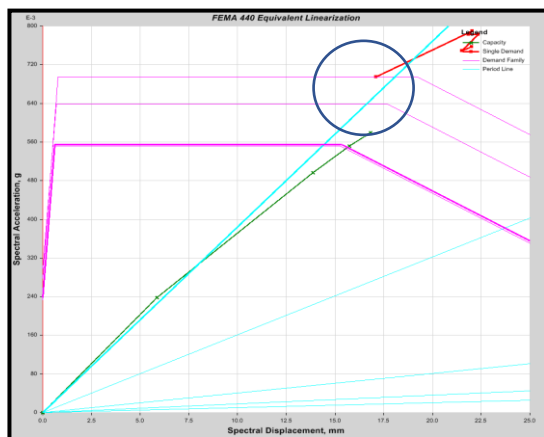


Figure 13: Fema 440 EL for a Structure with Unreinforced Stone Clad Panels

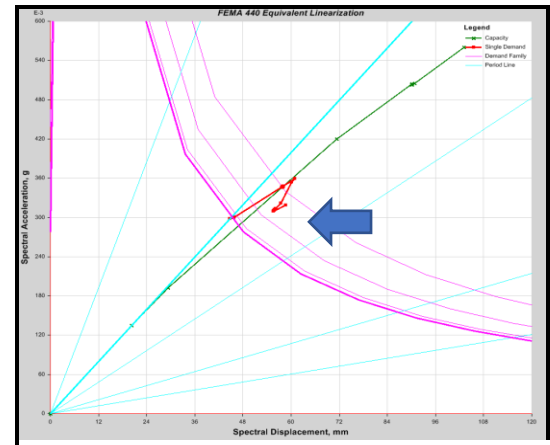


Figure 14: Fema 440 EL for a Structure with Partial Unreinforced Stone Clad Panels

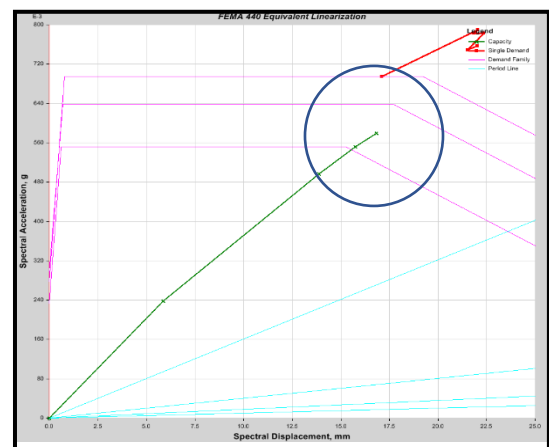


Figure 15: Fema 440 EL for a Structure with Full Unreinforced Stone Clad Panels

REFERENCES

- [1] ASCE 7-12 American Society of Civil Engineers, Minimum Design Load for Buildings and Other Structures.
- [2] Helou S. H. "An Investigative Seismic study of Structures Built on a Sloping Grade Versus Structures Built with or Without Setbacks, A Colloquial Narration" Second International Conference on Civil Engineering- Palestine, Nov 2019
- [3] Irfanullah M Irshad A Vishwanath B P Seismic Evaluation of Rc Framed Buildings with Influence of masonry infill panel", International Journal of research and technology EISSN: 2319-1163
- [4] Lakshmi G P and Santhi M H, "Seismic Evaluation of Residential Building with Masonry Wall using ETABS", International Journal of Engineering Sciences & Research Technology
- [5] Magdy Genidy et al "Seismic Response Evaluation of Moment Resisting Frame Multistory Buildings with Soft Story" International Journal of Civil and Structural Engineering Research, vol 3, Issue 1, April 2015
- [6] Minimum Design Loads for Buildings and Other Structures, ASCE/SEI 7-10, American Society of Civil Engineers, Reston, VA, 2010.
- [7] Rajurkar A. F. and Masram N. K. Seismic Analysis of RC Building with and without Infill Wall", International Research Journal of Engineering and Technology, Vol. 03, Issue 05, May 2016.
- [8] S. H. Helou and Ibrahim Muhammad, "Equivalent Lateral Load Method vs. Response Spectrum Analysis; Which Way is Forward"; Asian Journal of Engineering and Technology (ISSN: 2321-2462) Volume 02 – Issue 05, October 2014.
- [9] Suresh R. and Narasimhulu K., "Seismic Analysis of Medium Rise Open Ground Storey Framed Building by Response Spectrum Analysis Method", International Journal for Scientific Research & Development Vol 5, Issue 10, 2017