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The Soft Storey Impact on the Seismic Resistance of Masonry Structures
A Linear and a Non-Linear Colloquial Narration

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Abstract: *Mundane construction in Palestine is predominantly comprised of reinforced concrete framed structures with infill stone clad walls; they are generally formed of modest quality concrete elegantly dressed with natural stones. Moreover, such walls, where austerity prevails, are built of hollow concrete blocks; they are normally constructed at the center lines of the periphery columns. Masonry walls are conceptuallyconsidered nonstructural elements and are seldom included in analysis. Moreover, functional requirements in urban areas demand that certain floor levels are reserved for parking or open reception spaces; hence soft storeys emerge as a functional requirement which leaves some floor levels essentially unbraced i.e., a soft storey. In Palestine and elsewhere in the Middle East, seismic analysis and design are increasingly imposed requirements by local governments due to the proximity of the Dead Sea fault. Infill walls add considerable stiffness and mass which together considerably impact the response. The following investigative study of the influence of stone clad walls on the general response of structures during seismic events is carried out on a repertoire of numerical models that represent the various forms a selected G+6 edifice. The study makes particular focus on structures with a soft storey.*

The seismic analysis for the linear perspective is based on the quasi-static Response Spectrum Method, while the performance-based discourse is conducted according to the first generation of Pushover Analysis, namely Capacity Spectrum Method of ATC 40 and the improved guidelines of Fema 440. The study investigates, inter alias, story displacements and story drifts hence the impact of the soft storey is quantified for the scrutinized numerical models. Nonlinear analysis is obligatory under high intensity earthquakes where inelastic response and cracking are potentially expected. The study concludes that certain forms of masonry construction of structures with a soft storey are prone to seismic induced damages.

Keywords: *Irregular Structure; Response Spectrum Analysis; Infill Walls; Base Shear; Strut Elements.*

I. Introduction

Reinforced Concrete structures form the backbone of mundane construction in Palestine. Moreover, in certain Palestinian geographic areas, it is a mandatory practice that structures incorporate stone clad façades; this is an esthetic requirement imposed in respect to traditional construction. However, in areas where the terrain is sandy and rock formations are rare, austerity demands thatbuilding façades are built of hollow concrete blocks with little or no stone cladding.Technically the term masonry construction in modern day Palestine is erroneous when exterior wall panels are comprised of cast concrete and when such walls are stone cladded,yet the term remains accurate when the exterior infill walls are of the mundane hollow concrete blocks. Infill wall panels built of concrete blocks within a reinforced concrete frame serve the function of a diagonal strut rather than a shear wall because of the lack of connection between the blocks and the adjacent columns which hampers resistance to lateral loads resulting from potential seismic events. Moreover, designing seismically resistant structures is obligatory according to local government bylaws sincePalestine lies well within an active earthquake prone zone proximal to the Dead Sea fault. Furthermore, due to the prohibitive land cost in Palestine particularly in urban areas, parking facilities are mandated to be placed either at or below ground level. This is an architectural obligation that results in a soft storey.This is defined as the level that enjoys a stiffness of less than 70% of the stiffness of the storey directlyabove. Hence renderingthe structure vertically irregular.

Structural seismic resistance is imperative for avoiding unwarranted repair cost and for protecting against loss of life. The present study exercise tackles the favored medium rise structures having the aforementioned features, namely stone clad façades with a soft storey. Exterior walls, stone clad or otherwise, possess significant abilityto resist lateral loads and hence deserve adequate consideration in structural design undertakings. In order to analytically investigate theaforementioned influence on the general response during seismic events, a repertoire of different numerical modelsis considered to representa selected G+6 edifice. The models are characterized by being simple yet reliable. For the present study, wall panels with minimum reinforcement to emulate the standard practice are considered. The topology of the selected structures and their floor height are representative of most local apartment buildings. Moreover, some wall bays are judiciously left without walls as the present architectural trend is to induce glass paneled façades. The present narrationincludes, but is not limited, to the natural frequencies,storey displacements and inter-storey drifts. This is in addition to quantifying the performance level of all building models with and without a soft storey. The plan selected for the study is a symmetric one;thus, avoiding torsion modes.

The present exercise is based on performing seismic analysis by the standard linear elastic methods, i.e., the quasi-staticResponse Spectrum Method. The 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 and when it happens. Moreover, the study provides added insight into nonlinear structural behavior without resorting to the complexities of nonlinear dynamic analysis.Since the investigated structures are regularthenumerical resultsarepresentedinthe weaker direction only.

II. Numerical Modeling of the Structures

A symmetrical medium rise reinforced concrete structure is investigated. It is comprised of a G+6 Storey levels having four bays in one direction and three bays in the orthogonal direction. The overall size of the building is 20meters by 15 meters; the grid spacing is uniform and of 5-meter width. All storeys have a height of 3.2 meters; the slab flooring system is comprised of 13-centimeter-thicksolid slabs resting on drop beams. A semirigid property is assigned to all slabs to ensure integral lateral inclusion of all beams in each floor and force all joints connected to the slab to displace equally. Figure 2 shows the numerical models created by CSI ETABS 22.4.0; they include all fundamental components that impact the response. Columns and beams are representedby two node linear frame elements. For stone clad wall facades, the concrete wall panels are modeled as thin shell elements while the concrete block infill walls are modelled as double compression-only equivalent struts. The struts are assigned moment releases at both ends. The Response Spectrum curve selected for the dynamic analysis is a smooth one with a soil profile C and a damping of 5%. The mass source is defined as the entire Dead Load addedto the Superimposed Dead Load and 25% of the applied Live Load. The applied loads are in accordance with standard vernacular building predilection. Ground supports are assumed to enjoy total fixity. Effective stiffness modifiersare set following Fema 356 guidelines. [Beams 0.35 E_cI_g; Columns 0.7 E_cI_g; Walls 0.7 E_cI_g; Slabs 0.25 E_gI_g]. Table 1 shows pertinent structural and material data. The study,linear and performance based, considers the following numerical models:

Model 1: A bare reinforced concrete frame structure

Model 2: A reinforced concrete frame with concrete walls withouta soft storey,yetthe walls are modeled as shell elements.

Model 3: Same as Model 2 yet with a soft storey included.

Model 4: A reinforced concrete frame without a soft storey.The hollow concrete block periphery walls are modeled as double compression-only equivalent diagonal struts confined by columns.

Model 5: Same as Model 4 yet with a soft storey included.

Model 6: A reinforced concrete frame with infill walls modeled as a continuous dead load on the respective periphery beams.

The thickness of the equivalent unreinforced struts is set to be the same as that of the thickness of the stone clad walls, namely 20 cm. Moreover, the strut width, is computed according to the macro-modeling approach based on Fema 356 recommendation.Themselves are based on the works of Pauley and Priestly [8]. Pertinent seismic parameters are presented in Table 1.

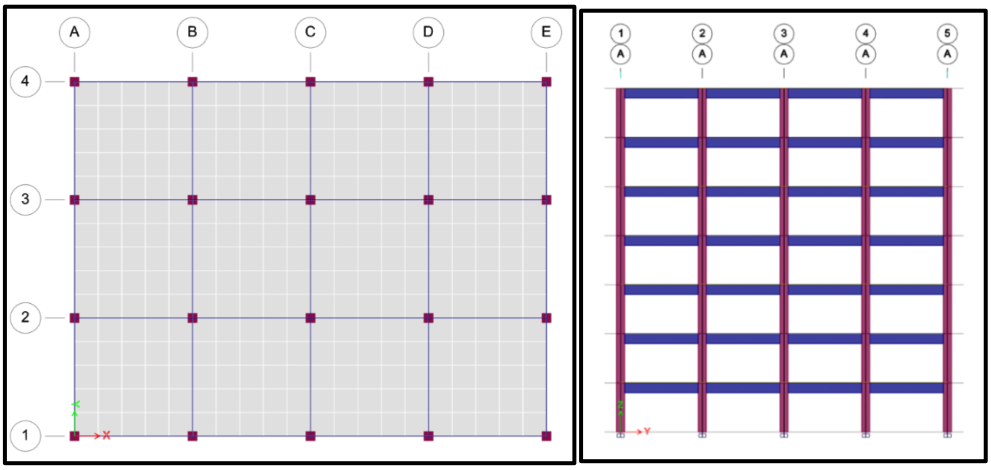

Figure 1: Plan and Elevation of the Structure

III. Relevant Seismic Analysis Parameters

- Uniform Live Load = 2.0 KN/m^2
- Uniform Superimposed Dead Load = 4 KN/m^2
- Seismic Importance Factor = 1 [ASCE 7-22, Table 1.5-2]
- Occupancy Category II [ASCE 7-22, Table 1.5-1]

- Site Class C [ASCE 7-22, Table 20.2-1; very dense sand or hard clay]
- $S_{D1} = 0.24$
- $S_{DS} = 0.84$
- Seismic Design Category D [ASCE 7-22, Table 11.6-1 and Table 11.6-2]
- Ordinary Moment Resisting Frame, R = 3.0 [ASCE 7-22, Table 12.2-1]
- Over Strength Factor, $\Omega_o = 3$ [ASCE 7-22, Table 12.2-1]
- Deflection Amplification Factor, $C_d = 2.5$ [ASCE 7-22, Table 12.2-1]
- Structural Damping: 5%
- Mass Source = DL + SDL + 0.25 LL
- Weight of infill walls = 15 KN/m

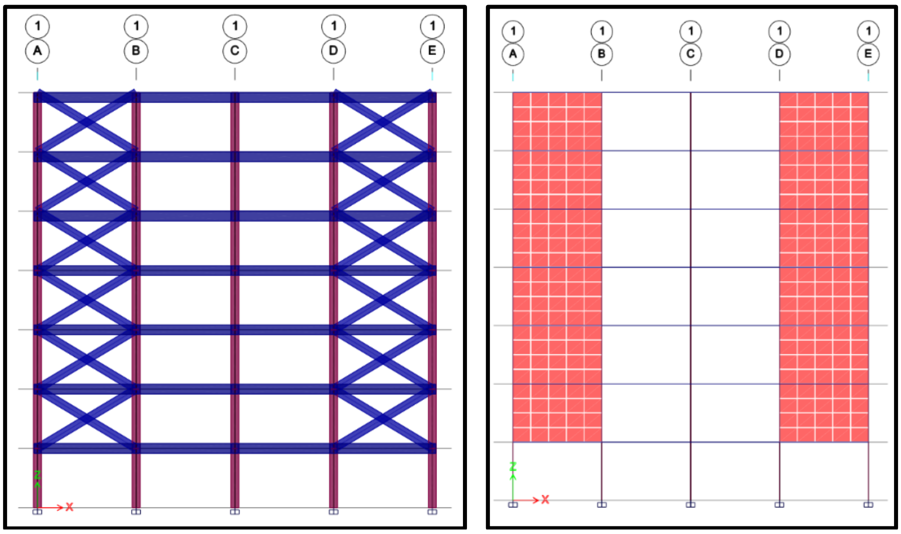

Figure 2: The Equivalent Struts Model and the Shell Model with a Soft Storey

Table 1: Structural Design Data

Storey Height	3.2 m
Beam Size	30 cm x 50 cm
Column Size	40 cm x 40 cm
Slab Thickness	13 cm
Infill Wall Thickness	20 cm
Concrete Grade M35 (f'_c)	28 MPa
Concrete Grade M20 (f'_c)	16 MPa
Reinforcement Bars (F_y)	410 MPa
S_s	1.05g
S_1	0.24g
$T_S=S_{D1}/S_{DS}$	0.29
Poisson Ratio of Concrete	0.20
Unit Weight of Concrete	25 KN/m^3
E Concrete - B35	25000 MPa
E Concrete- B20	23000 MPa
E Concrete - Masonry	12410 MPa
Masonry Concrete (f'm)	14 MPa

IV. The Strut Width

The strut width is decided in accordance with the macro-modeling approach presented by Pauley and Priestly [10]which is one fourth of the diagonal length. While the thickness of the equivalent unreinforced strut is equal to the thickness of the hollow concrete block infill wall.For the present studya 150x20 cm masonryrectangular strut section of zero reinforcement is assigned.

$W_s = \rho\ W_d$

W_d = one fourth of the diagonal length

A_r = Opening Ratio

ρ is a correction factor given as follows

$\rho = 1$ if $A_r \leq 0.05$

$\rho = 1 - 2.5\ A_r$ if $0.05 < A_r < 0.4$

$\rho = 0$ if $A_r > 0.4$

V. The Linear Analysis Procedure

For the present undertaking the Standard Response Spectrum procedure is adopted, the quasi static method includes constructing a numerical structural model which defines the spatial distribution of mass and stiffness, extracting the eigen-values and the eigen-vectors, defining proper number of modes necessary to secure at least 90% contribution of the total participating mass, defining an appropriate Response Spectrum function, selecting a suitable scaling factor, carrying out lateral analysis for each modeand finally implimenting modal superposition by appropriate means. The first twelve mode shapes are selected for the present discourse, and the CQC method is set for modal combination while a SRSS is invoked for the 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 superceded by the Equivalent Lateral Load Method. It is a mandatory step,in certain cases, for better quatifying the scaling factors in the two principal directions. Following ASCE 7-22 andparagraph 16.3.4 a 5% accidental eccentricity is called for. The results are presented forthe weaker direction only.

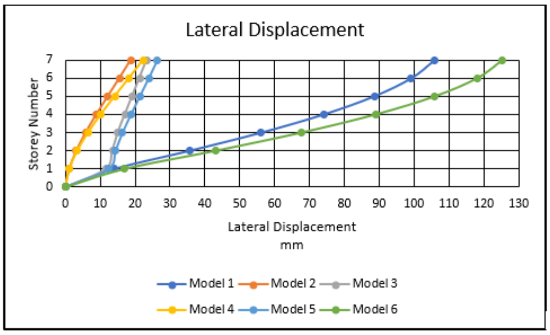

Figure 3:LateralDisplacements vs. Height

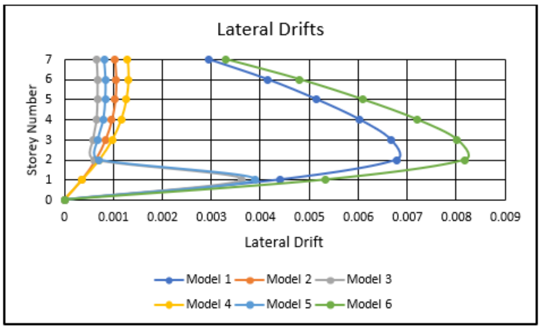

Figure 4: Lateral Drift Variation

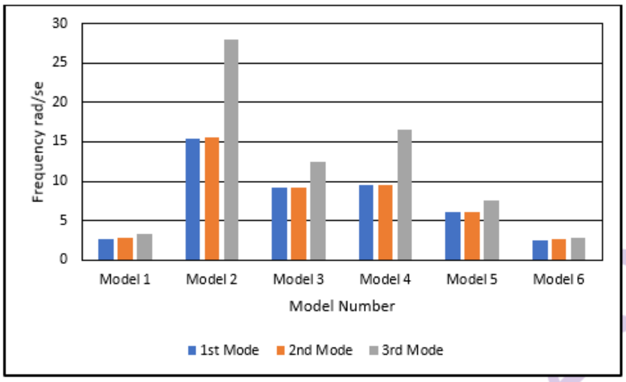

Figure5: Natural Frequencies of the Various Models (rad/s)

VI. The Nonlinear Static Pushover Analysis Procedure

Pushover Analysis is the numerical implementation of the Performance Based Design philosophy which emerged towards the end of the last century in response to the desire of insurance companies and the communities of real estate investors. It is a rather recent seismic engineering protocol; the presently prevailing exponential growth of the numerical computational power contributed tremendously to the evolvement of the procedure which is conducted in lieu of the elaborate nonlinear Time History Analysis. The objective is to have adequate control over maintenance cost post-earthquake events; in addition to designing safe structures with a short occupancy interruption. Therefore, the present study targets the prediction, beyond yield, of the structural behaviour under strong seismic events. It is a complex undertaking primarily due to the gradual change of section properties during structural deformation; this prompts continuous change in seismic forces due to the progressive change that occurs in the vibration period and damping.

The objective of the Performance Based Design [PBD] procedure is therefore to focus on life safety under strong earthquakes in order to minimize repair cost and to better manage disruptions of buildings post-earthquake events. The method is favoured due to its optimal accuracy, efficiency and for its easeof application. The present method maybe force controlled or displacement controlled. The present exercise is based on the first generation of PBD; this is a displacement-controlled procedure known as the Capacity Spectrum Method. Essentially, the method reduces a multi-degree of freedom system into a single degree of freedom system having its own effective damping ratio. The following is a summary of the procedure per ATC 40, Chapter 8.

For a well-designed structure, a nonlinear finite element model is created by adding non-linear hingesthat have well defined backbone characteristicsto all structural elements. For columns and according to ASCE 41-17, coupled axial flexural or PM_2M_3 frame plastic hinges are assigned at 10% and at 90% of all element lengths. In beams, uncoupled moment rotation frame plastic hinges are assigned at the same relative locations keeping the middle the beams elastic; i.e. plastic hinges are assigned close to the edges of all elements, where cracks are generally expected to happen. The selected locations generally have higher reinforcement ratio and thereforeare expected to be rather brittle. For the strut elements a force controlled axial hinge is assigned at mid-length while at the end points moment releases are specified. The nonlinear structural model is then pushed laterally by a monotonically increasing invariant force pattern until a maximum target displacement of a control node at the roof level is reached; P-Delta effects are properly considered. For the present undertaking the widely used monotonic lateral load pattern correspondingto a uniform acceleration in each direction is applied.In the stronger direction the second mode is applied. A target displacementat a roof monitored node is set at 5% of the building height.The master node selected is close in location to the center of gravity at the roof level. A plot of the base shear versus the roof displacement of a control point at the roof level is hence generated. The curve represents the inelastic load carrying capacity of the structure. However, prior to pushing the structure, a Pushdown non-linear static gravity load case is defined; this involves the entire dead load and the super dead load in addition to 25% of the applied Live Load. The Pushdown load case is force controlled while the lateral load case is displacement-controlled; the process continues until the target displacement is reached. This Pushover curve is then converted to anSa-Sd format or the so-called Capacity Spectrum while the demand curve expressed by the selected site response spectrum is converted to ADRS [Acceleration Displacement Response Spectrum] or the Demand Spectrum. This leads to the creation of two plots having unified coordinates.Figure 5 shows five points A, B, C, D and E on aplastic hinge backbone curve. The points define the force deflection behavior of a hinge while the three points IO, LS, and CP define the acceptance criteria for that 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 level 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 earthquake forces stronger than normally expected.

The intersection of the Pushover curve with a reduced form of the demand curve i.e., modified to accommodate the emerging equivalent damping, defines the Performance Point or the maximum inelastic displacement of the control node during a potential seismic event. This is normally implemented in accordance with the Improved Linearization procedure of Fema 440, the Improved Non-Linear Static procedure. Once the Performance Point is located the physical status of all nonlinear hinges is thus determined.Moreover, structural peer review and audit are mandatory for the indicated analysis protocol. The pushover analysis results shown are for $S_1 = 1$ and $S_S = 1.15$ with a soill profile C. The resultsprovide space for comparison among the variousinvestigated models.

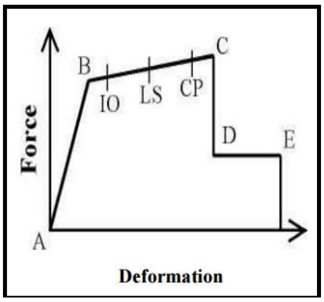

Figure 5: A Typical Hinge Load Standard Deformation Backbone Curve

Table 3: Performance Points of the Various Models

	Performance Point		Status of Hinges		
	Base Shear V KN	Roof Δ mm	IO - LS	LS - CP	>CP
Model 1	2869	263	0	0	0
Model 2	10558	32	2	0	0
Model 3	10118	76	6	0	0
Model 4	6958	51	4	0	112
Model 5	4667	75	22	0	96
Model 6	3652	340	4	0	0

VII. Conclusion

The study of the soft storey impact on the seismic resistance of masonry structure was performed on G+6 structure.The linear and the performance-based methodologies formed the basis for the investigation.The following are the conclusions:

a. The model without a soft storey and walls modelled as shells have the maximum frequency of vibration of all other models; this is followed by the model with compression struts.The lack of parity among the two magnitudes implies that the behaviour of the models is distinctly different.
b. The least magnitude of the frequency of vibration happens in Model 6 indicating that the model is the most susceptible to lateral excitations.
c. The maximum lateral displacement due to a seismic excitation within the elastic discoursehappens in Model 6 while the least displacement happens in Model 2.
d. The lateral drifts in Model 3 and in Model 5.The models with a soft storey show the same behaviour in magnitude and in trend. This implies that the strut method of modelling is appropriate.
e. Model 2 and Model 4 show similar trend of displacements and so do Model 3 and Model 5. This implies that the strut representation of the masonry walls is adequate.
f. Scrutinizing the performance points resulting from the Pushover Analysis shown in Table 3 Model 2 and Model 4behave well underearthquake load for the magnitude and the specified soil profile.[The performance points shown are for S_1 = 1.05;S_S= 0.15 and a soil profile C].
g. Finally,although hollow concrete block walls add to the general stiffness of the structure yet wall elements quickly reach the CP region and thus render the structure unsafe. They are not recommended in high seismicity zones.

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