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The Relevance of Non-Linear Seismic Analysis for Masonry Structures within the Seismic Zones of Palestine

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Abstract: Design of seismically resistant structures in Palestine is presently an obligatory requirement according to the local government bylaws because the region lies well within an active earthquake prone zone. The Seismic Zones in Palestine are either in 1, 2A, 2B and 3; i.e. between 0.15g and 0.3g which are equivalent to about 5 - 6.9 degrees on the Richter Scale. Local structures are generally designed in accordance with the 1997 Uniform Building Code regulations with the intention of providing life safety performance under a potential design earthquake; the ubiquitous Jordanian Design Code is a replicate of the UBC97. Furthermore, structural analysis is accomplished by either the Equivalent Lateral Load Method or the Response Spectrum Method. Seldom does mundane structural engineering resort to time domain analysis. Present local design practices assume that structures continue to behave elastically which may not be the case under strong seismic events.

Contemporary building structures in Palestine are, by enlarge, comprised of reinforced concrete frames in-filled with masonry walls but the interaction of the later with the rest of the structure is seldom investigated. Masonry structures therefore refer to buildings that have exterior infill un-reinforced stone-clad walls. The present undertaking presents a detailed structural study example that includes, inter alias, a regular and an irregular plan of a common six story edifice; the same exercise is repeated for a twenty story edifice. The numerical models investigated are built with due respect to the common construction practices and the local building materials and are subjected to the expected seismic action. The discourse follows the philosophy of Performance Based Seismic Engineering procedure better known as FEMA 356 Nonlinear Pushover Analysis and using the universally acclaimed Finite Element Method software ETABS 2015. The PBSE procedure is gaining popularity worldwide because it is capable of adequately addressing existing as well as new structures. It has the capability of predicting the points of weakness within the structure, so that remedial retrofitting procedures may be applied if and when they become necessary or for testing the adequacy of new structural designs.

The focus of the following study is the relevance of nonlinear analysis to local building structural systems; or conversely a study of the adequacy of the local construction systems to comply with modern seismic design requirements and approach knowing the severity of tremors potentially expected in Palestine.

Keywords: Masonry Structures; Fema 356; Nonlinear Analysis; Performance Based Engineering

I. Introduction:

Since Palestine is considered to be located in an active seismic prone region, local governments demand design of seismic resistant structures. Normally structural design undertakings are based on either the Equivalent Lateral Load method or, at best, on the Response Spectrum Method when structures under consideration enjoy vertical or horizontal structural irregularities. On the other hand universal building codes are continuously upgraded due to perpetual advancement in engineering knowledge considering the widely available major finite element software coupled with computers of tremendous numerical power. Therefore it is argued of whether or not the state of the art non-linear methods of analysis exercises are prudent for the local structural design industry, particularly that Palestine is defined to lie within Seismic Zones 1 through 3. Furthermore, it is well understood that although building codes provide reliable prediction of the performance of the individual structural elements yet structural performance under strong seismic actions remains a desired feature.

Moreover, in Palestine the common construction practice involves the construction of reinforced concrete bare frames that have facades covered with either curtain walls or masonry in-fill walls. The present study aims at scrutinizing the practice from the view point of whether Performance Based Analysis philosophy is adequate, necessary or forms a superfluous added luxury item. This is while remembering that such popular around the world methods, form a considerable deviation from the traditional design techniques.

The objective of the present study undertaking is, in brief, to apply and to understand Performance Based Analysis advantages and limitations. It aims also at evaluating the prevailing structural systems under the action of the seismic events expected in Palestine.

II. Topology of the Investigated Structures:

For the present study two typical yet structurally regular buildings are selected. They have the same layout plan but of different heights. One is a low rise 6 story structure, G+5, while the second is a moderate rise 20 story structure, G+19. The buildings are comprised of 4 bays of 6 meters in the X-direction and 3 bays of 5 meters in the Y-direction. The standard story height is 3 meters. The low rise building is further modified to form a system of an irregular structure. The systems selected are believed to cover a wide spectrum of local building structures. Beams and columns are modeled as line elements while the slabs are modeled as two dimensional rigid shell elements. All supports are assumed fixed; the foundation system is beyond the objective of the present study. Since the buildings are masonry in character the weights of the stone clad walls are applied as a uniformly distributed dead load on the periphery beams; infill walls are thus treated as non-structural elements. In brief the buildings are essentially comprised of solitary ordinary moment resisting frames as they represent an added conservative system; shear walls are excluded from the analysis because their presence would reduce base shear and the associated stress level. The frame elements have the sectional geometry shown in Table 1.

The assigned shallow beams are in agreement with the widespread local construction practice. The slabs are of 17 cm thickness and may represent a ribbed or voided slabs. Cracked sections are properly accounted for by including the proper modification factors per UBC97 requirement 1630.1.2. The effects of cracked sections are therefore included in all elements of the numerical models [for Beams: 0.35 I_g; for Columns: 0.7 I_g; for Walls: 0.35 I_g; for Slabs: 0.25 I_g]. The following are the applied loads and the material properties:

Live Load 3 KN/m2; Super Dead Load 5 KN/m2

Concrete f'_c = 24 MPa; Steel Reinforcement Bars F_y = 400 MPa

Occupancy Category 4; Medium Soil Type 2, Importance Factor = 1

Ordinary Moment Resisting Frame R = 3.5

Seismic Zones 1 and 3 [i.e. 0.15g - 0.3g]

Beam and Column Dimensions				
Structure	Inner Beams	Periphery Beams	Level	Column Size
G+5	25x70	30x60	1 -3	50 x 50
			4 - 6	40 x 40
G+19	25x70	30x60	1 - 6	50 x 50
			7 - 20	40 x 40

Table 1 Column and Beam Geometry

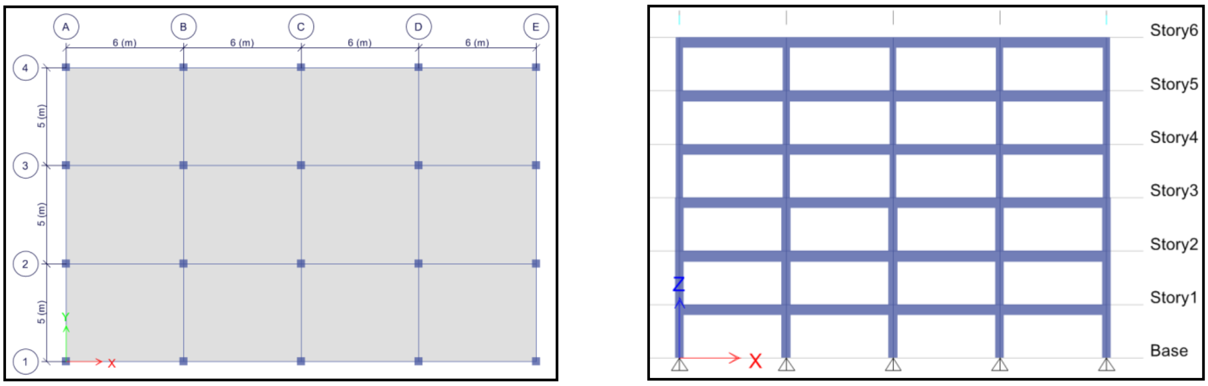


Figure 1 Plan and Elevation of the Investigated G+5 Structure

III. Pushover Methodology:

Pushover is a static non-linear method of analysis that is brought forward after the advent of the Performance Based Design philosophy. It 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 and monotonically increasing lateral loads which in fact represent, albeit approximately, the 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. Hence the real strength or weakness in the structure can be adequately identified, located and eventually addressed.

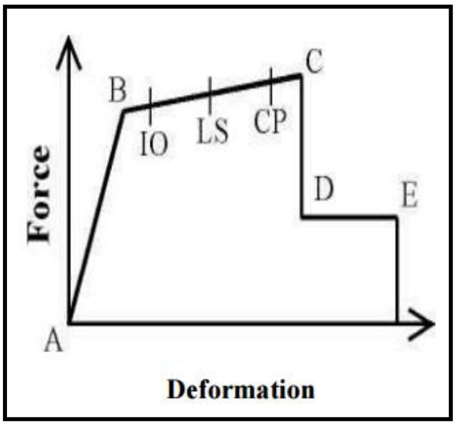


Figure 2 A Hinge Load Deformation Backbone Curve

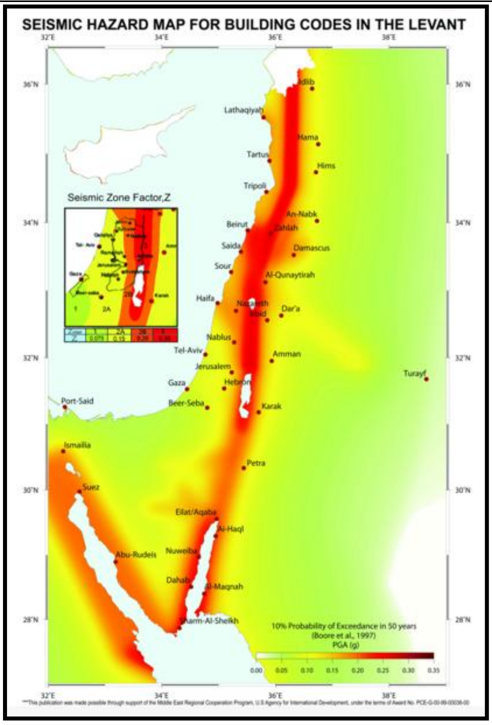
The five points shown on the hinge backbone curve shown in Figure 2 define the force deflection behavior of the hinge while the three points define the acceptance criteria for the hinge. Point A signifies the unloaded structure; point B shows the first yield of the element; point C shows the place where 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 where the maximum deformation takes place and no longer is the element able to 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 has to be determined. This is the point where the capacity curve intersects the demand curve. It is this component of the study that forms the main objective of the present exercise. If the point of intersection happens to be near the elastic range, the structure is judged satisfactory. If the intersection point leaves modest reserve of strength of the capacity then the structure is judged as poor and therefore would behave poorly under a strong seismic action. For the prediction of the Performance Point it is customary to subject the structure to earthquakes stronger than expected.

IV. The Pushover Procedure:

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 2015 recommends an uncoupled moment hinge; i.e. M3 hinges, for beams and an uncoupled axial force and biaxial bending moment hinge; i.e. PMM hinges, for columns. A control node is identified at the roof of the structure and close to its center of rigidity, the displacement of which versus the base shear form 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 start up 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. In the present study it is set at about double the magnitude of the Response Spectrum based elastic analysis displacement. For the G+5 building the target displacement is set at 30 cm while for the G+19 building the target displacement is set to 60 cm. Two lateral load cases are then applied to the structure; one in the x- direction, PUSHX and the other in the y-direction, PUSHY. These are in the form of accelerations representing the forces that would be experienced during a seismic action. The lateral forces are increased until some members yield. The capacity of the structure is represented by a plot of roof displacement versus base shear, the so called pushover curve, while the demand is the estimate of the maximum predicted response during ground excitation.



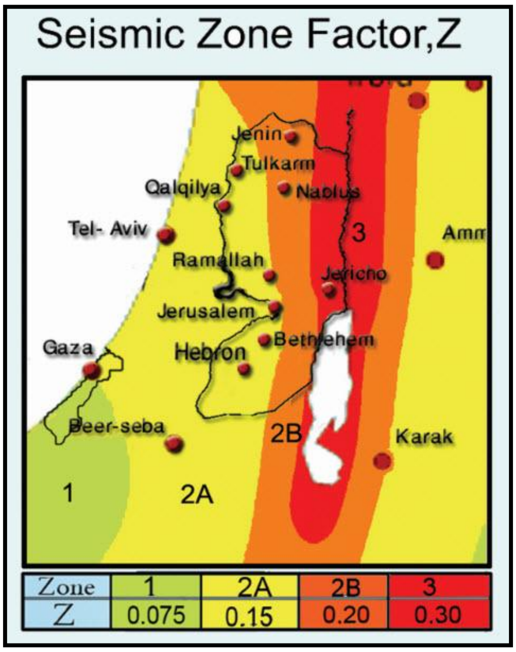


Figure 4 -- Seismic Hazard Map of Palestine

V. The Numerical Models:

Three dimensional numerical models are prepared and tested for equilibrium and compatibility using ETABS 2015 which is a state of the art, general purpose, world acclaimed Finite Element Structural Analysis program. One model is a regular G+5 building, the second is a G+5 irregular building and the third model is a G+19 regular structure. The standard static loads are applied and seismic analysis is performed based on the requirements of the prevalent code, UBC 97, and on the Response Spectrum method. Modal analysis revealed that 5 modes are generally adequate to secure the 90% mass participation necessary for adequate seismic analysis. Proper scaling factors are introduced in order to equate the base shear values in both the Response Spectrum output and the Equivalent Lateral Load method. Once an effective structural system is available with due regard to serviceability and other relevant engineering objectives the stage is therefore ready for the Performance Based Analysis procedure. In all models the mass source is defined as the sum of the dead load and the superimposed load in addition to 30% of the applied live load.

VI. Analysis Results:

Structural performance is essentially established once the overall capacity of the structure is quantified and equated to the earthquake demand. The structural capacity spectrum is determined by the Pushover Procedure as a form of non linear analysis; this is generally understood to be the capacity beyond the elastic range. The structural demand is presented as a plot of the estimated maximum displacement of a selected control point at the roof of the structure during the ground motion converted to an Acceleration-Displacement Response Spectra format. The Performance Point is the intersection of the two curves. Hinge formation sequence and hinge behavior scrutiny together with a comparison against the hinge backbone follows.

For the G+5 regular structure in Seismic Zone 2, shown in Figure 1, no distinct Performance Point could be detected. This applies to both directions and within the two prevailing seismic zones. Moreover it is observed that the seismic demand generally falls well within the elastic range of the structural behavior. Therefore it is concluded that the structure continues to satisfy the acceptance criteria for Immediate Occupancy and Life Safety of FEMA 440. The performance detected proves that the structure is safe according to the established criteria; this is manifested in Figure 6.

For the G+5 irregular structure the general results do not change appreciably for a lower level earthquake. However, for higher intensity earthquakes the post yield behavior become better manifested. This is illustrated in Figure 5; it is an accepted procedure to subject the structure to a higher intensity earthquake when performing pushover analysis. It should be realized however that over and above, the Performance Point remains generally dependent on the relevant dynamic properties of the structure under consideration. The locations of Collapse Prevention (CP) are shown in Figure 7.

For the G+19 structure. A bare frame with no shear walls, yet regular in plan and in elevation; it has the same constituents as the G+5 structure shown in Figure 1. Fema hinges are defined in all beams and columns. Figure 8 shows that the demand and the capacity spectrum curves intersect at about the extent point of capacity. Thus it can be concluded that PBEA is adequate and prudent approach for such level of medium rise local structures.

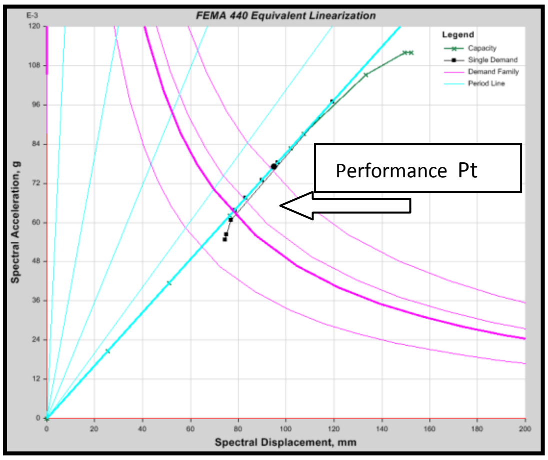


Figure 5

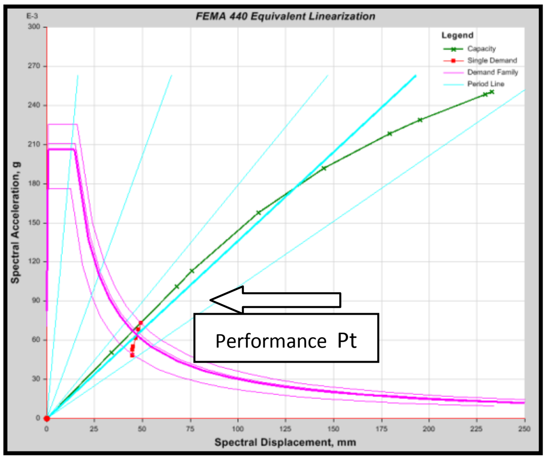


Figure 6

Figure 5: Performance Pt. for an Irregular G+5(a)
Figure 6: Performance Pt. for a Regular G+5(b)

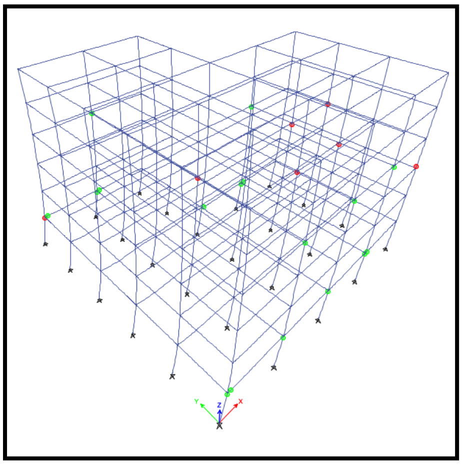


Figure 7

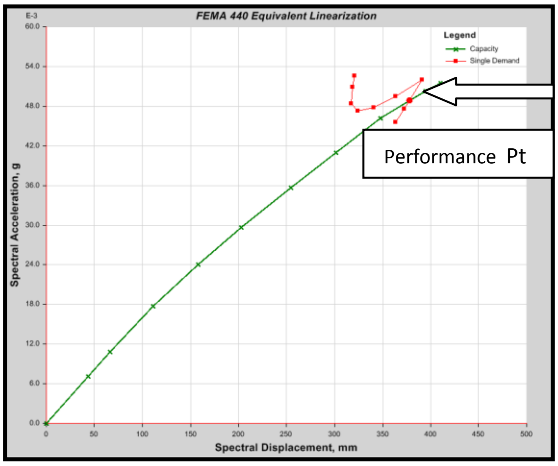


Figure 8

Figure 7: G+5 Irregular Building Locations where Sites of CP are Identified
Figure 8: Fema 440 Equivalent Linearization Curve for the G+19 Structure

VII. Conclusions and Recommendations

Performance based engineering using pushover analysis method is increasing in popularity for investigating post yield behavior of structures. The technique forms the contemporary approach to modern earthquake resistant structural design. However it is numerically demanding, rigorous and sophisticated. It requires extended computer running times which depend on the size and complexity of the model; hence comprehending the benefits and limitations is desired for judicious study planning based on comprehensive understanding of the process involved. Furthermore, since the Performance Based Method is philosophical in nature and does not follow code prescriptive techniques, peer review or structural design audit is always as prudent as it is recommended. The topologies selected for the present study are meant to form a proper representation of local structures, albeit on the conservative side. The choice of the Response Spectrum Method of analysis is based on UBC97 and follows similar logic but certainly not mandatory. The study shows that for modest rise edifices in seismically low zones there is limited need for nonlinear analysis; however for moderate or high rise structures PBE is useful and effective particularly when the structure under investigation enjoys a clear structural irregularity. The discourse presented bare frame structures with infill walls modeled as a distributed load over all periphery beams; no shear walls were present; an argument that amplifies conservatism. Moreover, although the present undertaking is limited to the seismic zones of Palestine yet the cited examples capitulated on local construction practices. The final verdict depends naturally on the particular dynamic properties of the structure under investigation in addition to the intensity of the ground motion. It should further be emphasized that it is desirable to subject the structure to earthquakes of a more severe intensity in order to better extract the post yield behavior. Furthermore, blanket statements and conclusions are rather difficult to formulate due to the excessive number of fundamental parameters involved.

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