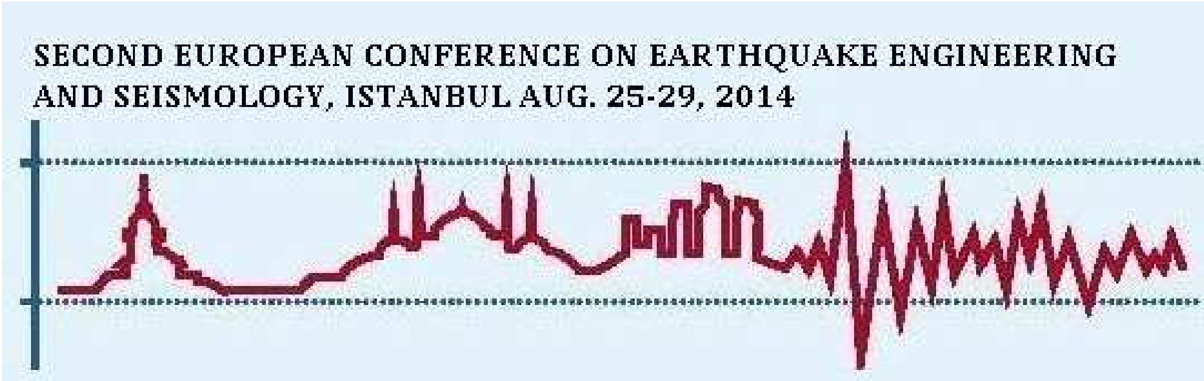

ESTIMATION OF MAXIMUM INELASTIC DISPLACEMENT DEMAND FOR DOMINANT RESIDENTIAL BUILDINGS IN JORDAN UNDER EARTHQUAKE EXCITATION

Hanan AL-NIMRY[1] and Galal AL-MEKHLAFI[2]

ABSTRACT

The residential building stock in Jordan post 1990 is dominated by a local type of construction wherein weak RC frames are infilled with multi-layered walls utilizing limestone masonry backed with plain concrete (stone-concrete walls). On the other hand, bearing walls of limestone masonry backed with plain concrete were used for the exterior walls of a very large number of residential buildings in Jordan prior 1990. This study is concerned with the estimation of maximum lateral displacements for the dominant residential buildings in Jordan that comprise stone-concrete walls under earthquake excitations. Eighteen buildings representing the two structural systems (infilled RC frames and bearing wall construction) were examined. In addition to the structural system, the investigated parameters included the building height, plan area, and vertical stiffness irregularities. Using SAP2000N, three-dimensional models were built for each of the representative buildings. Nonlinear static analysis was used to arrive at their capacity curves. Four approximate techniques were implemented to estimate the maximum inelastic displacement demand of these buildings under earthquake excitation: nonlinear dynamic analysis of an equivalent single-degree-of-freedom (SDOF) system, constant ductility procedure, capacity spectrum method and displacement coefficient method. Accordingly, upper and lower bound displacement values were obtained. Analysis results confirmed that the maximum lateral displacements of the investigated buildings do not exceed 1.2% of the total building height. This signifies the major contribution of the stiff exterior stone-concrete walls in limiting the lateral drift of stone-concrete buildings. The maximum displacement demand of mid-rise frames was found to be 100-108% of the demand on bearing wall systems in zones of low seismicity and 78-119% in zones of moderate seismicity. In low-rise infilled frames, the maximum displacement demand was less (0.82-0.90 times) than that of the bearing wall system. Furthermore, results showed that in medium-rise buildings subjected to low seismicity levels, the soft story increases the maximum displacement demand up to 2.7 times that of regular buildings.

INTRODUCTION

Dominant residential building typologies in Jordan and its vicinity use thin limestone masonry courses backed with plain concrete (stone-concrete) to construct the exterior walls. Two building typologies are usually associated with the use of stone-concrete walls. The first building typology, dominating residential construction before the nineties, uses the exterior stone-concrete walls as bearing walls conforming to provisions of the Jordanian National Building Code for Loads and Forces (JCLF, 1985). In this typology, the exterior walls consist of stone masonry courses back-filled with plain concrete (see cross-sectional layout in Fig.1(a)) and confined with reinforcing columns at their ends and tie

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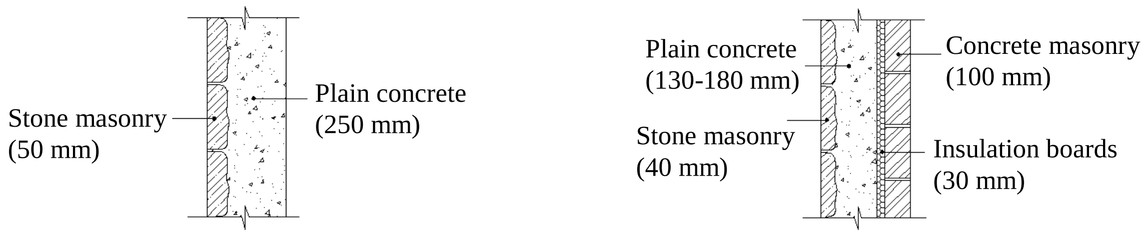
beams at the different story levels. Both the confining columns and backing concrete, within a single story, are cast in several horizontal layers with a time gap that could reach several days. The second building typology, which represents the current construction practice in Jordan, consists of gravity load-designed RC frames bounding stone-concrete panels. The infill panels in this type of construction comprise a thin layer of stone masonry back-filled with plain concrete and separated from a second layer of concrete masonry by 30 mm thick polystyrene insulating boards as shown in Fig.1(b). To avoid toppling of the thin stone masonry units, the back filling concrete used in either type of construction is typically cast in several horizontal layers over the story height. Local building specifications prohibit concurrence of the horizontal bed joints between the stone masonry courses and the resulting construction joints between the different plain concrete layers.



(a) Bearing wall

(b) Infill wall

Figure 1. Cross-sectional details of stone-concrete walls

Seismic response of residential RC buildings in Jordan incorporating either type of the stone masonry walls described earlier is not well-defined. Residential buildings constructed before the enforcement of the local seismic code (Jordanian Code for Earthquake-Resistant Buildings, 2005) were designed to resist gravity loads only. This study aims at evaluating the seismic demand of residential stone-concrete buildings in Jordan. Maximum inelastic displacement demand is evaluated analytically using the general finite element software, SAP2000 Nonlinear (Computers and Structures Inc., 2009). Four techniques are used to compute and compare the displacement demand: nonlinear dynamic analysis of an equivalent single-degree-of-freedom (SDOF) system, the constant ductility procedure, the capacity spectrum method and the displacement coefficient method.

METHODOLOGY

Eighteen buildings representing the two dominant residential RC building typologies (infilled RC frames and bearing wall construction) in Jordan are selected, designed and analyzed to estimate their maximum inelastic displacement demand. The investigated parameters include the structural system, building height and area in addition to the presence of a soft story at the ground floor level. The soft story is the most common stiffness irregularity encountered in local residential buildings. Several approximate methods have been developed to estimate maximum inelastic displacements of structures rather than using nonlinear dynamic analysis. These approximate methods use nonlinear static analysis as the first step. Four common methods are utilized in this study for the estimation of maximum inelastic demand of the investigated buildings. Nonlinear time history analysis of an equivalent single-degree-of-freedom system (ATC-40, 1996), the constant ductility method (Chopra and Geol, 1999), the capacity spectrum method (ATC-40, 1996) and the displacement coefficient method (FEMA-356, 2000) are implemented to estimate the maximum inelastic displacement demand.

REPRESENTATIVE BUILDINGS

The investigated buildings (as summarized in Table 1) included twelve buildings, representative of infilled RC frames constructed post 1990, categorized as follows:

- Three low-rise regular buildings with typical floor areas of 186, 320 and 500 m^2.

- Three mid-rise regular buildings with typical floor areas of 186, 320 and 500 m^2.
- Three low-rise irregular buildings with typical floor areas of 186, 320 and 500 m^2.
- Three mid-rise irregular buildings with typical floor areas of 186, 320 and 500 m^2.

In addition, three low-rise and three mid-rise regular buildings with typical floor areas of 186, 320 and 500 m^2 were considered to investigate residential buildings constructed prior 1990, i.e. buildings with exterior stone-concrete walls utilized as gravity load bearing walls.

Following stipulations of the local design codes, the selected buildings were designed to resist gravity loads only. Typical material properties used in residential RC buildings in Jordan within the time periods of interest were adopted for the model concrete and steel reinforcement of the investigated buildings. Possible variation in material properties were not taken into account at the modeling stage. Accordingly, normal weight concrete with an average 28-day compressive strength, f_c' of 21 MPa and 25 MPa was considered for the bearing wall and infilled frame construction, respectively. For the back filling concrete, an average 28-day compressive strength of 14 MPa was used in all buildings. Mild steel reinforcing bars (f_y = 280 MPa) were used in bearing wall construction whereas high tensile steel reinforcing bars were used for main reinforcement in the more recent frame buildings. Mild steel reinforcement (f_{yv} = 280 MPa) was assumed for the transverse reinforcement.

Table 1. Study parameters and designation of representative buildings

Building Typology	Vertical Regularity	Area (m^2)	Building Designation[1]	
			Low-Rise	Mid-Rise
Infilled RC Frames (New: post 1990)	Regular	186	N-A1-L-R	N-A1-M-R
		320	N-A2-L-R	N-A2-M-R
		500	N-A3-L-R	N-A3-M-R
	Irregular (soft story)	186	N-A1-L-I	N-A1-M-I
		320	N-A2-L-I	N-A2-M-I
		500	N-A3-L-I	N-A3-M-I
Bearing Walls (Old: prior 1990)	Regular	186	O-A1-L-R	O-A1-M-R
		320	O-A2-L-R	O-A2-M-R
		500	O-A3-L-R	O-A3-M-R

[1] N: new construction (post 1990); O: old construction (prior 1990); A1, A2 and A3: plan areas of 186, 320 and 500 m^2, respectively; L: low-rise; M: mid-rise; R: regular; I: irregular (with a soft story at ground floor level).

STRUCTURAL MODELING

Frame Elements: Using SAP2000 v14 (Computers and Structures Inc., 2009) three dimensional structural models were built for each of the representative buildings. Elastic beam elements with a T-section were used to model the joist construction in the different floor and roof slabs. Beam and column elements were modeled as elastic frame elements with lumped plasticity at the two member ends. Plastic hinges were assigned at member ends at a distance equal to half the plastic hinge length from the face of the joint. Stiffness properties of the frame members (beams and columns) were based on the approximate values presented in ACT-40 (1996) for the effective initial stiffness. Hence, the effective moments of inertia for beams and columns were set as $0.5I_g$ and $0.70I_g$, respectively where I_g is the gross moment of inertia.

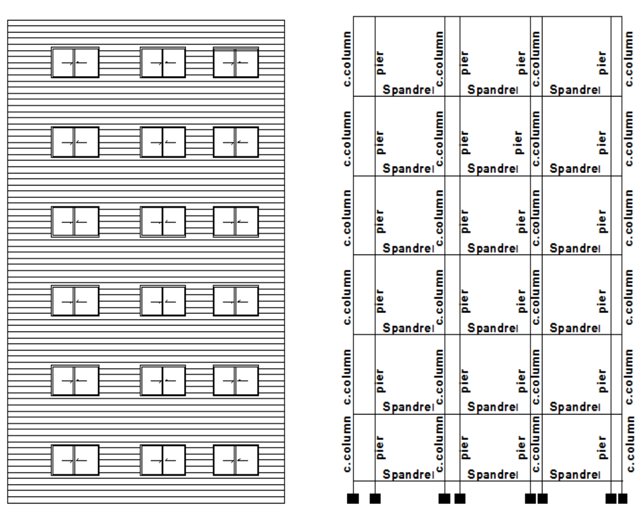
To arrive at the moment-curvature relations for the beam and column elements, the stress-strain model proposed by Mander et al. (1988) for confined and unconfined concrete is adapted. The complete stress-strain curve with parabolic strain hardening was used to model the stress-strain relationship for the main and secondary reinforcement. Roof and floor slabs were assumed to act as rigid diaphragms, distributing lateral loads to all vertical resisting elements in accordance to their stiffnesses. Fixity was assumed at the base of the ground floor columns and the effect of soil-structure interaction was disregarded.

Infill walls: Local infill walls are usually constructed using 100 mm or 200 mm thick hollow concrete masonry units (blocks). These masonry units are typically produced with a unit compressive strength of about 3.0 MPa whereas the local mortar mix proportions are 1: 0.5: 4 (cement: lime: aggregates) by volume and hence the mortar is classified as type (ii) mortar according to the mix proportions given by BS 5628-2 (2000). Accordingly, the characteristic compressive strength of the local masonry walls, f_k, is computed as 3.0 MPa. The modulus of elasticity for this type of concrete masonry is computed using $E = 1000\, f_k$ MPa, which is recommended by Eurocode 6 (1996) resulting in a value of E = 3000 MPa. Cracking strength of the infill panel, f_{tp}, is approximately taken as 0.25 MPa.

Assuming perfect contact between the masonry infill panel and the bounding beams and columns and neglecting out-of-plane failure, all infill walls are modeled using the equivalent strut model proposed by Fajfar et al. (2001) which considers two damage mechanisms for the infill: concrete crushing and bed joint sliding while neglecting the infill wall damage caused by deformation of the bounding frame. The effect of openings on the infill wall stiffness is considered by multiplying the effective width of the equivalent diagonal compression strut by the infill wall stiffness reduction factor (λ) proposed by Asteris (2003) based on the ratio of the opening area to the infill wall area.

Exterior Bearing Walls in Stone-Concrete Buildings Prior 1990: Several methods have been developed over the past years to model bearing walls. From complicated finite element micro-models to limit analysis approaches, a broad range of numerical methods are available in the literature. Among these methods are the equivalent frame models (Gilmore et al. (2009), Kappos et al. (2002), Salonikios et al. (2003), Pasticier et al. (2007), Belmouden and Lestuzzi (2007)). The equivalent frame method can be used to perform nonlinear analyses for bearing walls. Limited data is needed to model the material properties since isotropic and homogenous material idealization is made. The nonlinear behavior of the bearing walls is modeled through the use of nonlinear plastic hinges whose force-displacement properties are generally defined from experimental test results. Bearing walls are usually divided into vertical elements (piers) and horizontal elements (spandrels) between the openings of windows and doors. Piers and spandrels are assumed to be rigidly connected.

As mentioned previously, the exterior bearing walls in residential stone-concrete buildings in Jordan built before the 1990s consist of two layers: an outer layer of stone masonry and an inner layer of back-filling concrete. In this study, the contribution of the stone masonry units is neglected in the analytical model; the bearing walls are considered to be made up of plain concrete. However, self weight of the stone masonry layer is incorporated in the model. Fig.2 displays the equivalent frame model of a typical bearing wall in building O-A1-M-R.



(a) Elevation (b) Equivalent frame

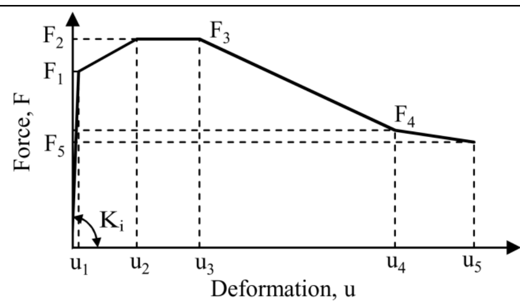
Figure 2. Equivalent frame modeling of a typical exterior bearing wall in building O-A1-M-R.

Exterior Infill Walls in Residential Buildings Constructed Post 1990: Al-Nimry (2010) investigated the behavior of stone-concrete infilled frames under earthquake loading using quasi-static experimentation of one-third scale infilled frame specimens (single-story, single-bay) with an aspect ratio of 0.8. Results from the quasi-static tests were used to develop and calibrate a simple macromodel for the infilled frames using SAP2000N (Computers and Structures Inc., 2009). The beam and column elements were modeled as frame elements with plastic hinges located at the end sections of column elements only to conform to experimental results wherein the beams displayed elastic behavior nearly up to failure. The effect of the infill panel on the lateral resistance of the bounding frame was modeled using a nonlinear link element. The nonlinear link element was assigned a multi-linear plastic property with nonlinear behavior for the axial direction only. Table 2 summarizes the characteristic points of the multi-linear force-deformation relation of the link element for the full scale single-story, single-bay RC frame as reported by Al-Nimry (2010). Stone-concrete infill walls in buildings constructed post 1990 were modeled using this force-deformation relation.

Nonlinear Frame Hinge Properties: The post-yield element behavior in one or more degrees of freedom is represented using plastic hinges. Properties of default hinges are provided in SAP2000N (Computers and Structures Inc., 2009) based on FEMA-356 criteria (FEMA-356, 2000). The distance over which the plastic curvature or plastic strain takes place defines the hinge length. Numerous plastic hinge lengths have been proposed in the literature. The plastic hinge length proposed by Park and Paulay (1975) and recommended by ATC-40 (1996) of half the section depth in the direction of analysis is adopted. Multiple plastic hinges can be used to achieve full nonlinear behavior over the whole length of an element. In this study, plastic hinges are concentrated at both ends of the element. Hinges in beam and column elements are assigned at half the plastic hinge length from the joint face. Plastic hinges in beams are considered to be of type M3 (bending moment) whereas plastic hinges in the columns are considered as (P-M_3) and (P-M_2) hinges. The moment-curvature relations for beam elements are defined automatically in SAP2000N (Computers and Structures Inc., 2009). On the other hand, the nonlinear hinge properties of column elements are defined by the user. Interaction diagrams for the column elements are obtained using SAP2000N (Computers and Structures Inc., 2009).

Table 2. Force-deformation characteristic points of the link element for the full scale single-story, single-bay RC frame as reported by Al-Nimry (2010).

Point	Axial deformation (mm)	Axial force (kN)
(u_1, F_1)	1	375
(u_2, F_2)	9	444
(u_3, F_3)	17	444
(u_4, F_4)	44	250
(u_5, F_5)	55	225



Nonlinear Spandrel and Pier Hinge Properties: Bearing walls in older residential buildings in Jordan, which are constructed using stone masonry back-filled with plain concrete, are confined with reinforcing columns that are placed at distances less than 4 meters around the building perimeter. These columns are typically reinforced with 6Φ12 (longitudinal reinforcement) and Φ6 or Φ8 ties at a spacing of 200 mm on centers. These bearing walls are modeled using the equivalent frame method wherein shear and flexural hinges are used simultaneously.

In the local construction of bearing walls, the back-filling concrete is cast in layers: seven layers may be cast within the typical floor height. Construction joints are formed between the consecutive layers and these construction joints form potential surfaces for sliding shear failure. Under earthquake excitations shear failure may take place due to diagonal tension failure or sliding shear failure. Failure will take place at critical locations with the least strength (either flexure or shear). In other words, if the moment straining action reaches the ultimate flexural strength before the appearance of shear cracks, the failure will be driven by a flexure mechanism. In this study, moment-rotation hinges are located at both ends of the piers, spandrels and confining columns. In addition, shear-displacement hinges are assigned to the mid-span of the piers and confining columns. Shear hinges are assigned to both ends of spandrels as well (see Fig.3). Properties of the above-mentioned hinges in piers, confining columns and spandrels are defined by the user. Table 3 summarizes the hinge properties assigned to the various wall elements (piers, confining columns and spandrels).

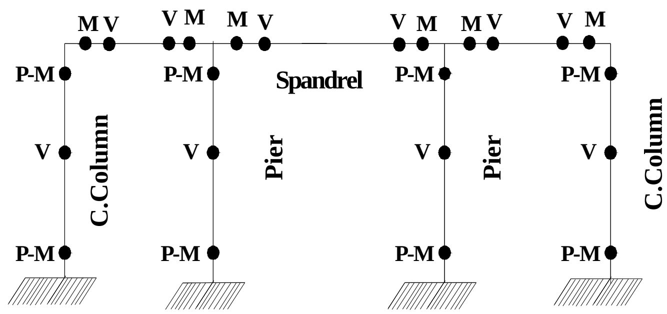


Figure 3. Locations of plastic hinges in piers, confining columns and spandrels

Table 3. Hinge properties of typical exterior bearing walls

Element	Flexural hinges	Shear hinges
Spandrel	M_3	V_2 and V_3 (diagonal shear)
Pier	P-M	V_2 and V_3 (diagonal or sliding shear whichever is smaller)
Confining column	P-M	V_2 and V_3 (sliding shear)

To arrive at the sliding shear strength for piers and confining columns; the Mohr-Coulomb failure equation is used:

$$V_{uf} = A(\sigma f + C) \tag{1}$$

where V_{uf} is the ultimate sliding shear force; A is the potential sliding shear surface area (cross-sectional area of the element); σ is the normal stress; f is a friction coefficient; and C is the cohesive strength of plain concrete.

As mentioned earlier, the back filling concrete is cast in layers to avoid toppling of the thin stone masonry units. The fresh concrete is cast against the hardened concrete surface. The latter surface is not intentionally roughened. Therefore; a friction coefficient value of 0.6 is assumed based on ACI-318 (ACI-318, 2011) while a cohesive strength value of 0.25 MPa is assumed based on Paulay and Priestley (1992) suggestions.

Sliding shear strength depends on the level of normal stress associated with gravity loads and is thus calculated at each floor level. To define the sliding shear hinges, V_{uf} is calculated using Eq. (1) and by assuming that yielding and ultimate strengths are equal (elastic-perfectly plastic behavior) then V_{yf} is also known. On the other hand, the yield displacement can be calculated using laws governing elastic behavior as shown in Eq.(2) and Eq.(3):

$$\gamma_y = \frac{V_{yf}}{A\,G} \tag{2}$$

and,

$$G = \frac{E}{2(1+\nu)} \tag{3}$$

where V_{yf} is the yield sliding shear strength; γ_y is yield sliding shear strain; G is modulus of fragility or shear modulus; E is modulus of elasticity of back-filling concrete; and ν is Poisson's ratio.

The ultimate sliding shear displacement can be determined using the ductility ratio for this type of failure. Based on the literature, the ultimate shear sliding displacement is set equal to twice the yield sliding shear displacement (Madan et al., 1997). Force-displacement relationships for diagonal shear strength of spandrels and piers are calculated based on plain concrete section properties using Response-2000 (Collins and Bentz, 1998). Moment-curvature relations for piers, confining columns and spandrels of the representative buildings are also calculated using Response-2000 (Collins and Bentz, 1998) based on the sectional properties of the elements.

PUSHOVER ANALYSIS

The conventional nonlinear static (pushover) analysis is used to construct the capacity curves of the representative buildings. The simultaneous effects of gravity and lateral loads are typically included in the nonlinear analysis of structures. ATC-40 (1996) suggests the use of dead loads (DL) and likely live loads (LL) in the analysis of gravity load effects. Furthermore, it is suggested to carry out a number of analyses to cover the range of possible gravity load levels acting on the structure during an earthquake, and then decide on the most critical value. Mahdi and Darehshiri (2009) developed capacity curves for reinforced concrete buildings subjected to three gravity load cases: 1.1DL+1.1LL, 0.9DL and DL+0.2LL. Analysis results confirmed that, within the range of investigated loading, capacity curves exhibited minor variations. Furthermore, taking into account that dead loads constitute the most significant proportion of gravity loads in residential buildings and that the variance in live loads is insignificant; pushover analysis was carried out using 25% of the live loads set by the design code (JCLF, 1985) in addition to dead loads.

Nonlinear static analysis was performed, using SAP2000 v14 (Computers and Structures Inc., 2009) for the two principal directions (x and y) of the representative buildings to arrive at their capacity curves. In conventional pushover analysis, gravity loads are applied first using force control. A predefined pattern of horizontal forces is then applied to the structural model using displacement control wherein lateral forces are increased monotonically while preserving the ratio between the lateral forces applied at the different story levels. Lateral forces were applied in the form of an inverted triangle that matches the fundamental mode shape of the building in compliance with the provisions of the equivalent static force method in the Jordanian Code for Earthquake-Resistant Buildings (JCERB, 2005). One of the roof nodes is chosen as the control node to monitor the lateral displacement according to ATC-40 (1996). In this study, the choice of the control node is considered to be irrelevant as the floor slabs were assumed to act as rigid diaphragms. The target displacement was set equal to 3% of the building height.

Earthquake characteristics (frequency content and duration) are not taken into account. Out of plane failure of the stone-concrete walls is neglected. Buildings under consideration are assumed to be constructed on soil type S_B (Rock) with a shear wave velocity ranging between 760 and 1500 m/sec.

CAPACITY CURVES

Nonlinear static analysis was performed for the two principal directions (x and y) of the representative buildings to arrive at their capacity curves. Differences in pushover curves obtained for the x and y directions were observed. This is associated with the fact that columns and walls in gravity load-designed RC frames are located and oriented with no intention to provide for uniform stiffness in the two orthogonal directions. The resulting capacity curves of the investigated buildings were reduced by 25% to compensate for the different uncertainties in the fundamental information of the real response of the buildings under consideration and uncertainties concerned with the modeling assumptions and analytical procedures. The pushover curve with lower resistance capacity was selected and idealized by a bilinear (elastic-plastic) curve in accordance with ATC-40 (1996) considerations for the displacement coefficient method. An example is shown in Fig. 4.

Once the idealized bilinear capacity curves are obtained, the maximum inelastic displacement for each of the model building is estimated using the four methods mentioned earlier. Estimation of maximum inelastic displacement demand will be based on the assumption that direction of the earthquake is parallel to the weak axis of the building.

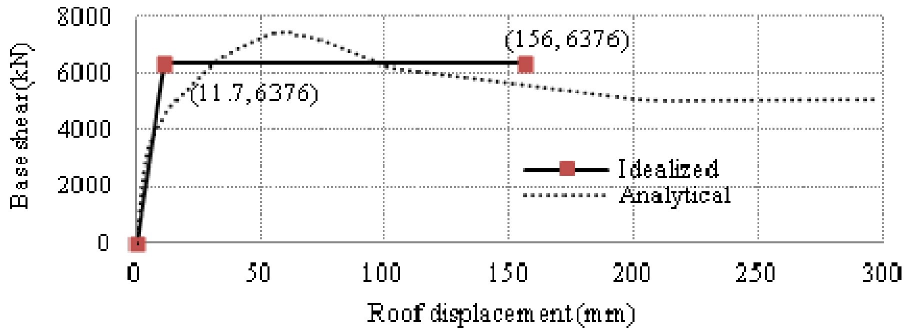


Figure 4. Bilinear idealization of the capacity curve of N-A3-L-R building

ESTIMATION OF MAXIMUM INELASTIC DISPLACEMENT DEMAND OF DOMINANT RESIDENTIAL BUILDINGS UNDER EARTHQUAKE EXCITATIONS

Nonlinear Time History Analysis of an Equivalent SDOF: This method provides the maximum global inelastic displacement demand of a multi-degree-of-freedom structure using an equivalent single-degree-of-freedom system with appropriately modeled hysteretic characteristics. Three earthquake records (Kocaeli 1999, Duzce 1999 and Kobe 1995), with two horizontal components each, were used to estimate the maximum inelastic displacements of the model buildings. Displacement demand is estimated assuming different levels of seismicity in accordance with the 4 seismic zones given in the local code which correspond to zones 1, 2A, 2B and 3 of the 1997 Uniform Building Code (UBC 1997).

Constant Ductility Method: The graphical procedure of the constant ductility method is used to calculate the displacement demand of inelastic SDOF systems due to seismic actions. Fig.5 displays a sample of the constant ductility curves applied for building N-A3-L-R in seismic zones 1, 2A, 2B and 3. S_d and S_a coordinates of the performance point are shown on the curves.

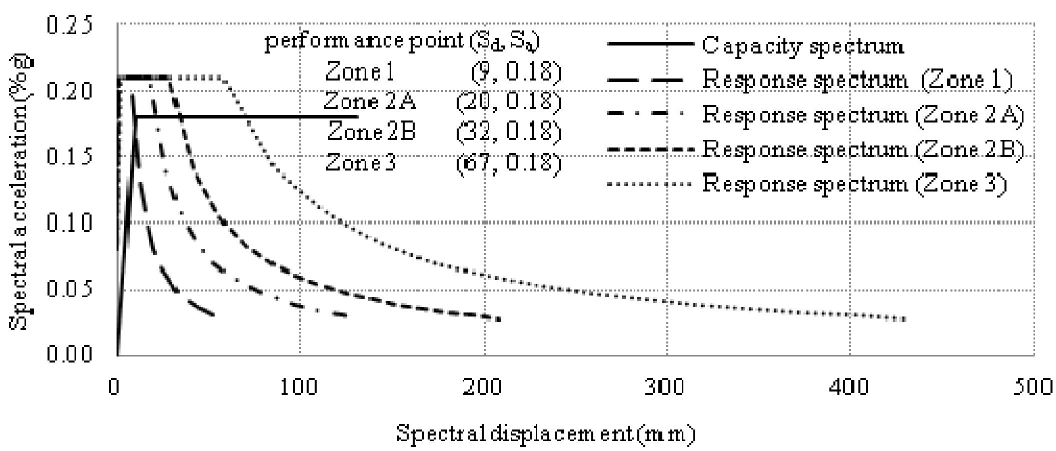


Figure 5. Constant ductility curves for the N-A3-L-R building

The global maximum inelastic displacement of each of the model buildings is estimated by multiplying the displacement demand of the inelastic SDOF calculated from the constant ductility curves (i.e. the S_d value indicated on the curves) with the factor of fundamental modal participation at the roof level. The fundamental modal participation factor is taken as 1.2 for regular buildings and 1.0 for buildings with soft story irregularity.

Capacity Spectrum Method (ATC-40 Procedure A): To estimate maximum inelastic displacements of the representative buildings, procedure A of the capacity spectrum method is used to calculate the displacement demand of inelastic SDOF systems due to seismic actions. Fig.6 displays the capacity spectra of building N-A3-L-R and the relevant response spectra that represent the seismic demand in the four seismic zones. The intersections between the different capacity spectra and response spectra represent the maximum displacements of SDOF systems representative of the model building. The S_d and S_a coordinates of the above-mentioned intersection points are indicated on the figure.

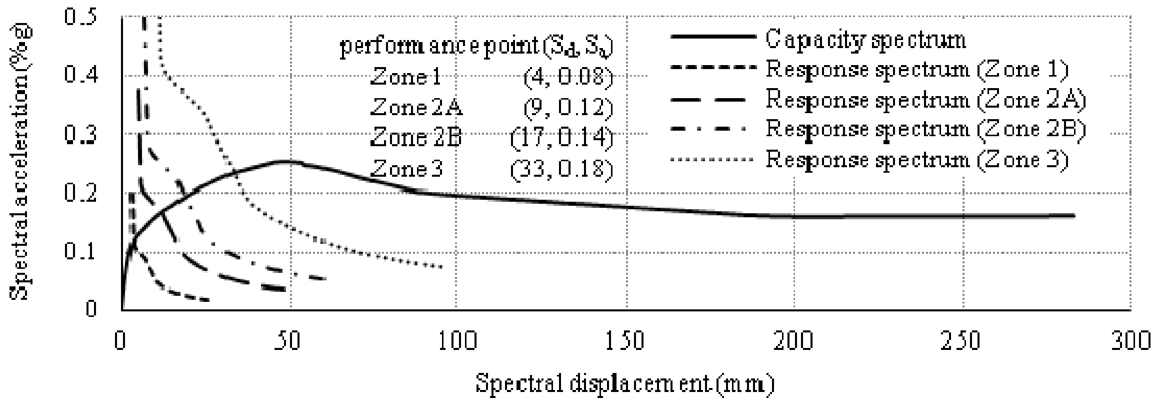


Figure 6. Application of the capacity spectrum method for the N-A3-L-R building

Using the capacity spectrum method, global maximum inelastic displacements are estimated by multiplying displacement demand of inelastic SDOF defined by the S_d coordinate of the performance point with the factor of fundamental modal participation at the roof level ($PF_1\Phi_{roof,1}$ = 1.2 for regular buildings and 1.0 for the case with a soft story). Displacement values are reported for each building in the four seismic zones. The soil at the building site is assumed to be of type S_B.

Displacement Coefficient Method (FEMA-356): Maximum inelastic displacements for the 18 representative buildings are estimated using the displacement coefficient method described in FEMA-356 (2000) which is an approximate numerical method.

RESULTS

The maximum inelastic displacement values obtained for the 18 model buildings, using the four different computation techniques, show a wide range of variability. To arrive at an informed estimation of the maximum inelastic displacement of similar buildings, the maximum inelastic displacements are reported as a percentage of the total height of the model buildings. Furthermore, displacement values for the three area categories (A1, A2 and A3) are grouped. In other words, the buildings are classified into six general categories instead of eighteen. Minimum (lower bound) and maximum (upper bound) percent values obtained for the six building categories are summarized in Table 4 and Table 5, respectively. The average percent values for each of the six building categories are also summarized in Table 6.

Table 4. Lower bound of maximum inelastic displacements of stone-concrete buildings

Building Typology	Building Height and Regularity	Maximum Inelastic Displacement Demand (%H_t)			
		Zone 1	Zone 2A	Zone 2B	Zone 3
Infilled Frames	Low-Rise, Regular	0.02-0.05	0.05-0.11	0.08-0.21	0.11-0.33
	Mid-Rise, Regular	0.03-0.05	0.05-0.08	0.07-0.13	0.09-0.16
	Low-Rise, Irregular	0.11-0.13	0.15-0.19	0.26-0.34	0.31-0.40
	Mid-Rise, Irregular	0.08-0.09	0.12-0.13	0.20-0.23	0.24-0.27
Bearing Wall Construction	Low-Rise, Regular	0.01-0.03	0.03-0.05	0.04-0.08	0.06-0.12
	Mid-Rise, Irregular	0.03-0.04	0.06-0.061	0.09-0.10	0.11-0.12

Table 5. Upper bound of maximum inelastic displacements of stone-concrete buildings

Building Typology	Building Height and Regularity	Maximum Inelastic Displacement Demand (%H_t)			
		Zone 1	Zone 2A	Zone 2B	Zone 3
Infilled Frames	Low-Rise, Regular	0.06-0.12	0.17-0.34	0.36-0.51	0.80-0.98
	Mid-Rise, Regular	0.09-0.15	0.19-0.26	0.31-0.40	0.67-0.70
	Low-Rise, Irregular	0.23-0.29	0.42-0.78	0.59-0.68	1.08-1.20
	Mid-Rise, Irregular	0.13-0.16	0.23-0.26	0.30-0.35	0.49-0.53
Bearing Wall Construction	Low-Rise, Regular	0.10-0.14	0.17-0.31	0.30-0.44	0.58-0.74
	Mid-Rise, Irregular	0.15-0.17	0.28-0.32	0.41-0.44	0.70-0.74

Table 6. Average values of maximum inelastic displacements of stone-concrete buildings

Building Typology	Building Height and Regularity	Maximum Inelastic Displacement Demand (%H_t)			
		Zone 1	Zone 2A	Zone 2B	Zone 3
Infilled Frames	Low-Rise, Regular	0.07-0.10	0.13-0.18	0.21-0.27	0.39-0.43
	Mid-Rise, Regular	0.04-0.09	0.09-0.19	0.18-0.31	0.41-0.58
	Low-Rise, Irregular	0.12-0.13	0.19-0.22	0.27-0.31	0.41-0.45
	Mid-Rise, Irregular	0.17-0.21	0.30-0.43	0.45-0.49	0.73-0.79
Bearing Wall Construction	Low-Rise, Regular	0.09-0.10	0.17-0.19	0.27-0.28	0.49-0.51
	Mid-Rise, Regular	0.06-0.08	0.10-0.16	0.17-0.46	0.36-0.47

EFFECT OF SOFT STORY ON THE LATERAL RESISTANCE AND MAXIMUM DISPLACEMENT DEMAND OF RESIDENTIAL RC INFILLED FRAMES

The existence of a soft story is a common weak link of the existing building stock in Jordan due to the absence of infill walls in the ground floor which is mainly used as a parking area. Presence of a soft story in the ground floor of low-rise residential buildings was found to reduce the lateral resistance, compared to regular buildings by 51% to 55% as indicated in Table 7. A lower effect was detected on the resistance of medium-rise buildings: Lateral resistance was only reduced by 16% to 40% of that of companion regular buildings.

On the other hand, effect of the soft story on the lateral displacement depends mainly on the seismicity level and building height. The displacements presented in Table 8 are average of the displacement values obtained using the four analysis methods and summarized in Table 6. In areas of low seismicity (seismic zones 1 and 2A), the maximum displacement of low-rise buildings comprising a soft story was found to be 1.31-1.44 times that of companion regular buildings. In seismic zones (2B and 3) characterizing a level of moderate seismicity, the maximum displacement in low-rise buildings with the vertical stiffness irregularity amounts to 1.05-1.21 times that of regular buildings of the same area and height.

Table 7. Lateral resistance of regular and irregular infilled frames

Building Regularity	Yield base shear (kN)					
	Low-Rise			Mid-Rise		
	A1	A2	A3	A1	A2	A3
Irregular (Soft story)	1179	2203	2946	1552	3155	5639
Regular	2406	4854	6376	2611	4617	6723
(Irregular/ Regular)	0.49	0.45	0.46	0.60	0.68	0.84

Table 8 shows that in medium-rise buildings subjected to low seismicity levels, the soft story increases the maximum displacement demand to 2.64-2.71 times that of regular buildings. Regular buildings (6 stories) in zones of moderate seismicity showed displacement demands 0.53-0.66 times those of the companion irregular buildings comprising the soft story. The main observation regarding the effect of the soft story on displacement demand of residential buildings is that higher lateral displacements (compared to regular buildings) are observed. The difference in lateral displacements is reduced as the level of seismicity increases.

Table 8. Maximum displacement demand of regular and irregular infilled frames

Building Regularity	Maximum displacement (%H_t)							
	Low-Rise				Mid-Rise			
	Low seismicity		Moderate seismicity		Low seismicity		Moderate seismicity	
	1	2A	2B	3	1	2A	2B	3
Irregular (Soft story)	0.13	0.21	0.29	0.43	0.19	0.37	0.47	0.76
Regular	0.09	0.16	0.24	0.41	0.07	0.14	0.25	0.50
(Soft/Regular)	1.44	1.31	1.21	1.05	2.71	2.64	1.88	1.52

EFFECT OF STRUCTURAL SYSTEM ON THE LATERAL RESISTANCE AND MAXIMUM DISPLACEMENT DEMAND OF RESIDENTIAL BUILDINGS

Pushover analysis results showed that lateral resistance, in terms of yield base shear values, of infilled RC frames is lower than the resistance of companion buildings with the same area and height but with stone-concrete walls acting as bearing walls. Table 9 shows that low-rise and medium-rise RC infilled frames have a lateral resistance capacity of about 70% (68-74%) and 80% (78-91%), respectively, of the capacity of similar buildings (same area and height) that could have been constructed prior 1990.

On the other hand, maximum displacement demand of low-rise infilled RC frames was found to be about 90% of the demand on bearing wall systems in zones of low seismicity and 82-86% in zones of moderate seismicity as shown in Table 10. In mid-rise infilled RC frames, the displacement demand was 100%-108% of the demand on bearing wall systems in zones of low seismicity and 78%-119% in zones of moderate seismicity. The displacements presented in Table 10 are based on the average displacement values.

Table 9. Lateral resistance of infilled frame and bearing wall systems

Building Typology	Yield base shear (kN)					
	Low-Rise			Mid-Rise		
	A1	A2	A3	A1	A2	A3
Infilled RC Frames	2406	4854	6376	2611	4617	6723
Bearing Walls	3539	6690	8577	3342	5059	8445
(New/ Old)	0.68	0.73	0.74	0.78	0.91	0.80

Table 10. Maximum displacement demand of infilled frame and bearing wall systems

Building Typology	Maximum displacement (%H_t)							
	Low -Rise				Mid-Rise			
	Low-seismicity		Moderate-seismicity		Low-seismicity		Moderate-seismicity	
	1	2A	2B	3	1	2A	2B	3
Infilled RC Frames	0.09	0.16	0.24	0.41	0.07	0.14	0.25	0.50
Bearing Walls	0.10	0.18	0.28	0.50	0.07	0.13	0.32	0.42
(New/Old)	0.90	0.89	0.86	0.82	1	1.08	0.78	1.19

CONCLUSIONS

The following points summarize the main findings of the study:

- Maximum inelastic displacement values obtained for the eighteen model buildings, using the four computation techniques showed a wide range of variability.
- Lower bound values of the maximum displacements were obtained from the displacement coefficient method (FEMA-356). Whereas the upper bound values were obtained from the nonlinear dynamic analysis of an equivalent single-degree-of-freedom (SDOF) system.

- In this study, the maximum lateral displacement under earthquake excitations of all the investigated buildings does not exceed 1.2% of the total building height. This signifies the major contribution of the stiff exterior stone-concrete walls in limiting the lateral drift of the buildings subject of the study.
- Results clearly show that there is not direct relation between the lateral displacement demand of a building and its plan area; since the lateral displacement is mainly dependent on the building mass and stiffness, rather than its area, among other parameters.
- The investigated regular buildings located in seismic zone 1 remained in the elastic range when exposed to earthquake forces stipulated in the local seismic code.
- Presence of a soft story in the ground floor of low-rise residential buildings was found to reduce the lateral resistance, compared to regular buildings, by 51% to 55%. Presence of a soft story in the ground floor of mid-rise residential buildings was found to reduce the lateral resistance, compared to regular buildings, by 16% to 40%.
- In areas of low seismicity (seismic zones 1 and 2A), the maximum displacement of low-rise buildings comprising a soft story was found to be 1.31-1.44 times that of companion regular buildings. In seismic zones (2B and 3) characterizing a level of moderate seismicity, maximum displacements ranged between 1.05-1.21 times that of regular buildings of the same area and height.
- In mid-rise buildings subjected to low seismicity levels, the soft story increased the maximum displacement demand to 2.64-2.71 times that of regular buildings.
- Regular buildings in areas of moderate seismicity showed displacement demands 0.53 to 0.66 times those of the companion irregular buildings comprising the soft story.
- Low-rise and mid-rise infilled RC frames have a lateral resistance capacity of about 70% (68-74 %) and 80% (78-91%), respectively, of the capacity of similar buildings (same area and height) that could have been constructed prior 1990.
- Maximum displacement demand of mid-rise infilled RC frames was found to be 100-108% of the demand on bearing wall systems in zones of low seismicity and 78-119% in zones of moderate seismicity. In low-rise RC infilled frames, the maximum displacement demand was lower (0.82-0.90 times) than that of bearing wall systems.

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