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Suitability of Current Assessment Techniques to Retrodict the Seismic Damage of Buildings: A Case Study in Van, Turkey

Viorel Popa, Radu Văcăreanu*, Vasile Oprisoreanu, Emil Albotă and Dietlinde Köber

Technical University of Civil Engineering, Bucharest, Romania

Abstract: Two strong earthquakes hit the region of Van (Turkey) in 2012, generating major losses. A large part of the building stock was damaged causing the evacuation of the residents. A post-earthquake investigation team dispatched by the UNESCO through the IPRED platform, six month after the earthquakes, observed the damage state in some still standing buildings. Ten buildings having different structural characteristics were observed. Slight structural damage and severe non-structural damage were observed in three multi-storey apartment buildings, being evacuated at the time of the investigation. Despite the slight structural damage, two multi-storey reinforced concrete shear wall buildings were listed for demolition based on the results of the post-earthquake rapid assessment. These two buildings were recently built based on modern seismic design regulations. The design blueprints were available to the investigation team with the support of the local community of Van. Various rapid post-earthquake investigation techniques applied by the investigation team generated contradictory results. A comprehensive seismic assessment was carried out to retrodict the observed seismic damage. Various methods were applied starting from simple rapid assessment techniques to more elaborated structural analysis based on nonlinear dynamic procedure. In the latter case, strength and stiffness degrading hysteretic models were applied and the non-structural masonry walls were considered in the analytical model. This paper presents the results of these structural analyses in comparison with the observed damage on site. Conclusions regarding the suitability of the applied seismic assessment techniques to retrodict the damage level of the investigated structures are drawn. Some findings of the post-earthquake investigation team are presented as well.

Keywords: Earthquake damaged building, post-earthquake investigation, seismic assessment.

1. INTRODUCTION

Nowadays a large effort is spent worldwide for the seismic vulnerability assessment of existing buildings. The assessment process is not always fully reliable because the structural response to seismic action is subject to extended uncertainties and randomness. Such uncertainties are mainly related to the quality of building materials, quality of workmanship or the applied gravity loads. The randomness in building response due to different ground motions makes the prediction of the seismic response even more difficult.

Standards and/or guidelines [1-3] prescribe various seismic assessment methods with different levels of complexity in which the seismic action effects can be evaluated using linear static analysis, modal response spectrum analysis, non-linear static analysis or non-linear time history analysis. It is generally accepted that the use of more complex methods, if properly fed with data and applied, increases the reliability of the seismic assessment.

Using any assessment method a researcher or structural engineer can “blindly” predict the expected response for a seismic action at a given site. However the accuracy of such predictions can be hardly checked in practical situations. Seismic assessment offers basic information about the vulnerability of an existing building but can hardly accurately predict the building response under a given earthquake scenario.

There is a justified concern in the scientific community about the reliability of the extensively used of seismic assessment procedures. A suitable way to check the reliability of current seismic assessment techniques is to try to retrodict the response of damaged buildings during a past earthquake. In this respect, information regarding the layout of the building, structural details, quality of building materials, damage sustained by the building during the earthquake and the seismic action at the building site should be available. Such information can be readily collected during post-earthquake investigation missions.

A post-earthquake investigation mission in the city of Van, Turkey, was completed in 2012 by a team of academic staff and professional engineers from Turkey and Romania [4]. This investigation was carried out within the interna-

*Address correspondence to this author at the Technical University of Civil Engineering, Bucharest, Romania; Tel: +40-21-2421208;
Fax: +40-21-2429722; E-mail: radu.vacareanu@utcb.ro

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tional UNESCO IPRED Platform. The first two authors of this paper participated within this mission. The city of Van was strongly affected by two earthquakes in the fall of 2011: Van-Ercis earthquake on October 23, M_w=7, 1 and Van-Edremit earthquake on November 09, M_w=5.6 [5].

The first earthquake produced most of the damage in the town of Ercis, having a population of 77000 and located about 40 km NNW of the epicentre. 191 buildings totally or partially collapsed and more than 600 people were killed. City of Van having a population of 332000 and located about 15 km SSW of the estimated epicentre was spared by the first earthquake. Only six buildings collapsed. The second earthquake struck about 10 km SW of Van and claimed 40 lives and the collapse of 25 buildings in the city. More than 30000 buildings were heavily damaged or collapsed by these earthquakes [5].

Inadequate lateral-load resisting systems, soft and weak ground floors, torsional flexibility of the structures, in-plane flexible block-infill joist floors, strong beam - weak column failure mechanisms, pounding of adjacent buildings were observed. Severe and extended damage of masonry infills were observed in many still standing buildings with minor to moderate structural damage. In most situations, the infill walls were made using hollow clay units with horizontal holes. Such infills are rather weak as reported by Schwarz *et al.* [6]. Similar damage of the masonry infills with almost no residual deformations or cracks in the structural elements were observed by Mostafaei & Kabeyasawa [7] after the 2004 Bam earthquake in the telephone centre building.

A large part of the vulnerable building stock in Van consists of multi-storey (5-8 story) reinforced concrete frame or shear wall structures with brittle masonry infill partition walls. At the time of the mission the debris of the collapsed buildings were removed from the city. Many evacuated buildings listed for demolition (but) still standing were observed. Most of these buildings sustained severe structural and non-structural damage. For many buildings the non-structural damage of exterior masonry walls was obvious from the outside.

In some buildings, most of the damage of the masonry infills was observed in the ground floor or immediately above it. This kind of damage is rather unexpected because the infill masonry should suffer most damage at the stories where high drift values occurs. This is hardly the case of the ground floor for structures with concrete shear walls running continuously from the foundation to the top story.

To increase the reliability of the seismic assessment for a concrete structure with soft masonry infills, the structural model should incorporate adequate masonry characteristics. A proper modelling of the connections between the structural and non-structural elements is required. Past research shows that the presence of masonry infills fundamentally changes the structural seismic response [8].

A large variety of masonry infills is used worldwide for concrete structures. These infills often rely on local materials and in many cases no structural testing data is available. Collecting information regarding the strength, stiffness and deformability of such infills can be a difficult task. Infills similar with those observed in Van were investigated by Schwarz *et al.*, [6]. Additional information was published by Yuksel *et al.* [9], Ozkaynak *et al.* [10] and Papanicolau *et al.* [11]. Kaushik *et al.* [12] showed that infill walls can be modelled for in-plane response using single or multi-struts. Kardysiewski & Mosalam [13] explained that the out-of-plane behaviour of the masonry can be taken into account by modelling a strut midpoint node with an out-of-plane mass.

This paper refers at two identical RC residential buildings (A and B) with seven stories located in Van downtown inspected by the authors. The most damaged one, *i.e.* building B (Fig. **1**), was analysed in detail. Both buildings were designed and built in 2005. The structural system consisted of cast in place reinforced concrete shear walls and frames (Fig. **2**). In the transversal direction the lateral resisting structure consisted mainly of shear walls. Five shear walls were identified during site inspection. In the longitudinal direction only two shear walls were identified. In this case, the concrete frames were an important part of the lateral resisting structure.

The structural system presented vertical regularity. No sudden changes in stiffness or strength of the structural elements were noticed over the building height. The cross-sections of shear walls, columns and beams were similar at each floor.

The shear walls locations within the structure suggest that the building was likely to experience a strong torsional response. In the transversal direction, the lever arm of the forces in shear walls to resists the building torsional moment was merely 7.60m. In the longitudinal direction, the existing shear walls were collinear so no lever arm of the shear forces in the concrete walls could be considered for the torsional strength and stiffness. This particular layout of the shear walls was determined by the architectural constraints. The building had relatively large windows on each facade regularly placed in each bay and at each story. No full masonry infills were placed in the outer frames.

The concrete frames consisted of rather stiff beams with height of 60cm and widths of 25 or 30cm. Most of the columns had rectangular cross-sections of 30x60cm or 30x80cm usually aligned with the *x-x* axis of the building (longitudinal) to compensate for the smaller shear area of the concrete walls. Relatively stiff frames were obtained in this direction. Most of the beams intersection points correspond to a column or wall end. Only two transversal beams were supported at one end by two longitudinal beams. Outer frames present beam offsets of 50 to 150cm from one bay to another. This unfavourable layout of the beams was caused by architectural constraints regarding the aspects of the facades. These beams were supported on thin columns that connected the beams end. The effectiveness of such layout upon the strength and stiffness of the outer frame is questionable.

A 12 cm continuous slab was supported by the beams. Given the layout of the slab a rigid diaphragm assumption could be made. For many of the multi-storey RC buildings built in the recent years in Van, infill joist system was used for the floor structural system. This resulted in shallow beams of 30-35 cm not suitable for earthquake resisting frames.

Fig. (1). Main façade of building B.

The foundation system consisted of foundation beams connecting the shear walls and columns under the basement. The basement was used also for apartments so windows were placed on the perimeter just above the ground level. No change in the vertical structural system was noticed at this level. Given the relatively weak foundation beams in comparison with the shear walls, uplift and even plastic hinging of the foundation beams were foreseeable.

Important changes in the layout of the shear walls in contrast with the specifications in the design drawings were observed. Apparently, these changes were made without any structural engineer guidance. Shear walls in the longitudinal (*x-x*) direction were relocated during construction from one axis to another. Their web length was reduced from 6.55m to 4.55 while the width was preserved. In the transversal direction, two additional shear walls were erected increasing the shear strength in this direction and improving the torsional stiffness of the building. Shear walls had rectangular cross sections having 30cm width and were horizontally and vertically reinforced with 2φ12mm bars at 150cm. Boundary elements at each end of the cross section were vertically reinforced with 4φ16mm bars. Confinement of the boundary elements was made using φ8mm closed stirrups spaced at 150mm.

Reinforcement details for beams and columns, as given in the design drawings, generally met the requirements of both Turkish [14] and European [15] seismic codes. The columns had continuous longitudinal reinforcement from bottom to top with reinforcement ratio of around 1%. The beams had continuous reinforcement at both sides. Longitudinal reinforcement ratios ranging roughly from 0.5% to 1% could be noticed in beams.

Closed stirrups with hoops bended at 135º were used; 8mm diameter stirrups spaced at 160mm or 180mm were usually provided for beams. Spacing of the stirrups in the critical regions of the beams did not meet the requirements of EN1998-1 for high ductility class.

In some elements, curtailment of the top bars in beams at the limits of the critical regions indicated the possibility of plastic hinging in the middle of the beam.

At interior columns, anchorage length of the bottom reinforcements was not provided by bending the bar inside the beam-column joint. Instead, the straight bottom bars passed through the interior joints being cut in the critical regions of the adjacent beam. Otherwise, anchorage length of more than 40 bar diameters was used for the longitudinal rebars.

Masonry infills were built, according to the construction practice in the region, using hollow clay units. The width of

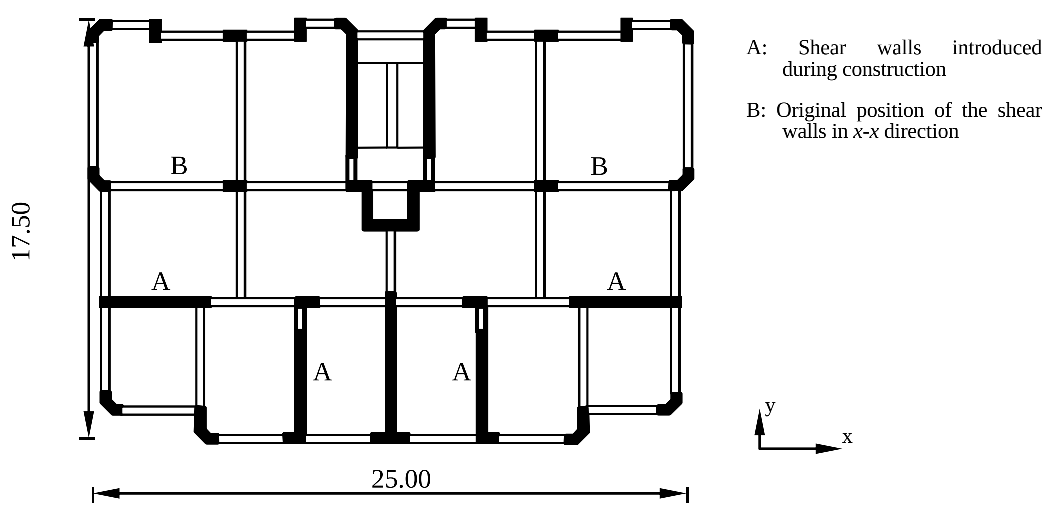

Fig. (2). General layout of the structural system.

Fig. (3). Observed building damage.

the masonry panels was 30cm for exterior walls and 15cm for interior walls. Double layer masonry panels with thermal insulation in the middle were used for exterior panels. Usually, no concrete ties were used to confine the masonry near windows or doors or to prevent out of plane failure for the large interior masonry panels. Apparently, no steel anchors were provided to connect the panels to the adjacent beams and columns.

The analysed buildings were inspected by the UNESCO IPRED mission in June 2012. At the time of the inspection both buildings were evacuated and listed for demolition. This decision was made by the public authorities based on the rapid assessment performed immediately after the earthquakes. According to the owner, the buildings sustained limited damage after the first earthquake (October 23, 2011) and was evacuated. The buildings suffered most of the existing damage during the second earthquake (November 11, 2011).

For building A, no major damage could be observed from the outside. Building B sustained more severe damage as reported in the following.

Most of the outer masonry panels and parapets below the windows at the lower stories were damaged. Most of the interior masonry panels at the first three stories sustained severe damage beyond reparability. Replacement of these panels was considered necessary for future use of the building. Full or partial collapse of some masonry panels parallel to the *x-x* direction at the ground floor was observed (Fig. **3a**). These masonry infills replaced the concrete shear walls that were relocated to the adjacent bay.

Damage to the non-structural masonry panels with no corner bonding can be noticed in Fig. (**3b**). Two perpendicular panels connected using just polyurethane foam inserted in the vertical separation joint detached from each other with a vertical crack. The panel oriented in the *x-x* direction of the building crashed at the bottom corner.

Overturning of an exterior leaf of a double layer masonry panel is represented in Fig. (**3c**). No connectors between the leaves were noticed on the site. In the same figure, the shear crack developed in the captive column between the basement windows can be noticed.

No major structural damage was noticed. Shear and flexural cracks occurred in the critical regions of the beams. Inclined shear cracks occurred in beams due to the interaction with the masonry infills as well.

Inclined shear cracks were observed in the columns and shear walls as well. The largest crack width measured in the shear walls was 0,3mm. Thinner cracks could be observed in the L shaped columns on the building perimeter.

Despite the shallow structural damage, the site inspection revealed the low quality of structural concrete. Removal of the finishing layers exposed the members' concrete surfaces (Fig. **3d**). In some members even the steel reinforcements were exposed as can be seen in the case of the coupling beam connecting two sections of the shear wall around the elevator shelf (Fig. **3e**). Ready mix concrete for building construction is available in Van area since 5-7 years ago [5]. Inspection showed that the construction quality of the concrete structure was rather poor. The low concrete quality is further illustrated in Fig. (**3f**). Improper water/cement ratio, improper combination of gravel sorts, segregation, lack of vibration, and concrete settlements under steel rebars were noticed.

Samples of the concrete extracted from the columns and shear walls revealed a very low compressive strength of 6-7MPa for B building and 12MPa for A building.

No severe settlement could be observed around the building perimeter or at the basement floor. An uplift of the building of 3cm from the sidewalk level was measured at one corner of the building.

2. STRUCTURAL ANALYSIS

2.1. Static Nonlinear Analysis

Static nonlinear analysis (pushover) was used to investigate the structural response in the nonlinear range. The main objective of the structural analysis was to determine the lateral yielding force and the displacement demand. Static nonlinear analysis was performed using Etabs [16].

A 3D computer model which accounts for the nonlinear response of the structural system was developed. Beams and columns were modelled using frame elements. Shear walls having the length of the cross-section less than 1.5m were modelled using frame elements. Shear walls having the length of the cross-section larger than 1.5m were modelled using shell elements.

Within the boundaries of the columns cross-sections the longitudinal and the transversal beams were considered rigid. If the transversal beam axis, the longitudinal beam axis and the column axis did not intersect in a single point stiff beam links were considered to model the connection. The equivalent "elastic" stiffness was taken equal to half of the gross stiffness for each structural member.

Foundation beams were supported on springs with homogenous elastic behaviour in vertical direction. The translations of the foundation in both horizontal directions were restrained. No other external restraints were applied. At each story, horizontal rigid diaphragms were considered.

Gravity loads, other than the dead weight of the structure, were applied as uniform loads on the slabs at each floor. An equivalent uniform load of 12.8 kN/m^2 resulted for the entire building.

Nonlinear M3 hinges were introduced at the end of each beam. Nonlinear P-M2-M3 hinges were introduced at the bottom of each column, directly above the foundation beams. Nonlinear P hinges were introduced at the boundary columns of shear walls. Nonlinear P hinges were used to simulate the yielding of the vertical reinforcement in the web of the shear walls, as well (Fig. **4**).

Pushover analysis was performed in both horizontal directions. Two lateral load distribution patterns were considered for each direction: mass proportional and acceleration proportional.

The contribution of the infill masonry panels upon the lateral strength and stiffness of the building is questionable. Interior masonry panels are 15 cm thick. All masonry panels were made with hollow masonry units. Multi-cell clay blocks with large rectangular holes oriented in the horizontal direction were used. The vertical interior walls of the bricks had out of plane defects which significantly reduce their strength and stiffness (Fig. **5**). Exterior masonry panels had large openings for windows and interior infill panels had door openings, usually located at one end. The formation of the compression diagonal struts was prevented as the small lateral drifts were usually accommodated by the door opening.

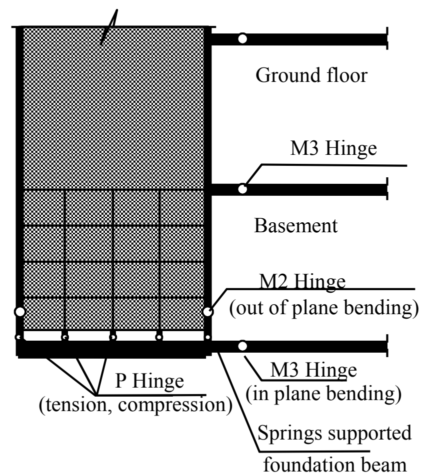

Fig. (4). Modelling assumptions.

Fig. (5). Typical masonry work in the building.

As the main objective of the structural analysis was to determine the peak lateral drift demand using the capacity spectrum method, the contribution of the masonry infills was not accounted for directly. A sensitivity study to account for the influence of the masonry infills by changing the lateral stiffness of the structure was performed using time-history linear analysis. Subsequently, non-linear time-history analysis was used to account directly for the contribution of the masonry infills.

The modal analysis of the building revealed a strong torsional response. The modal participating mass ratios are listed in Table **1**.

The second and the third vibration modes were considered to determine the seismic demand for the equivalent lateral load method in the Turkish Earthquake Standard [14]. The absolute acceleration response spectra of the horizontal components of the strong ground motion recorded at Van Markez Seismic station and the elastic design spectrum at the site according to thhe Turkish Seismic Code are compared in Fig. (**6**). The required lateral seismic coefficient in the longitudinal direction resulted:

$$C^x = \frac{A(T)}{R} = \frac{A_0\, I\, S(T)}{R} = 0.108 \qquad (1)$$

considering an effective ground acceleration coefficient A_0=0.3, a spectrum coeficient S(T)=1.42, an importance factor I=1 and a reduction factor R=4. For the transversal direction S(T)=1.80 and C^y=0.135.

Based on the simplified representations of the force-displacement capacity curves as presented in Fig. (**7**), the normalized yield strength of the structure in x and y directions resulted:

$$R_y^x = \frac{F_y^x}{W} = \frac{8700}{39000} = 0.22 \qquad (2)$$

$$R_y^y = \frac{F_y^y}{W} = \frac{12000}{39000} = 0.31 \qquad (3)$$

The calculated values of the R_y are larger than the required lateral seismic coefficient. Overstregth factors of 2.03, in the longitudinal direction, and 2.30, in the transversal direction, were determined considering the characteristic value of the steel yielding strength. If a partial safety coeficient γ_S=1.15 for the steel yileding strength was taken into account, lower-bound values of the lateral overstregth of 1.77 and 2.00 were obtained.

The Capacity Spectrum Method [1], was used to determine displacement demand for the structure. Two levels of the effective ground acceleration were considered:

- 0.25g, corresponding to the recorded earthquake of November 11, 2011, at Van Merkez Station, E-W direction
- 0.4g, corresponding to the design acceleration of the updated version of the Turkish Earthquake Standard. This value was selected for Van region after the earthquakes of 2011.

Considering a control period of the acceleration response spectrum of 0.4 sec the following coefficients resulted: C_a=C_v=0.25 for a_g=0.25g and C_a=C_v=0.4 for a_g=0.4g.

The performance points determined using capacity spectrum method are presented in Fig. (**8**). For the effective ground acceleration of 0.25g peak lateral displacements of 46mm in *x-x* direction and 36mm in *y-y* direction were determined. These values correspond to maximum lateral drifts in the longitudinal direction of approximately 0.25% at the first story. Such values of the lateral drift suggest an

Table 1. Modal participating mass ratios.

Mode	Period	Participating Mass Ratios		
		UX	UY	RZ
	(s)	%	%	%
1	0.91	21.1	0	58.3
2	0.77	57.2	0	21.7
3	0.61	0	78.1	0

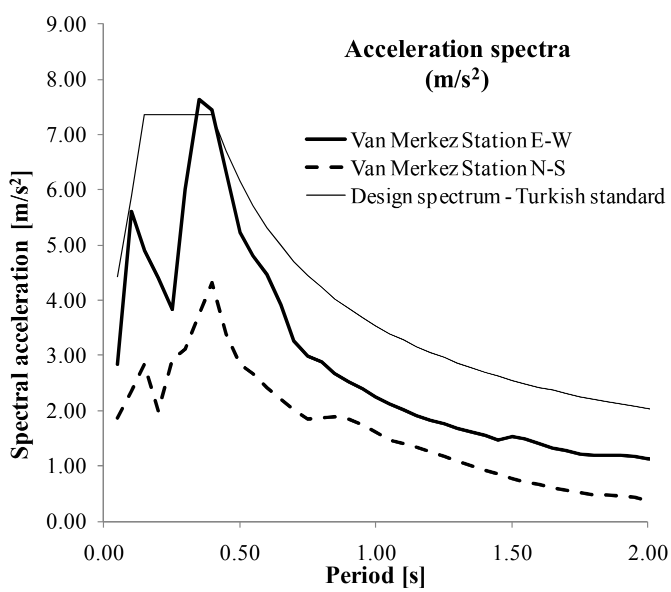

Fig. (6). Absolute acceleration response spectra.

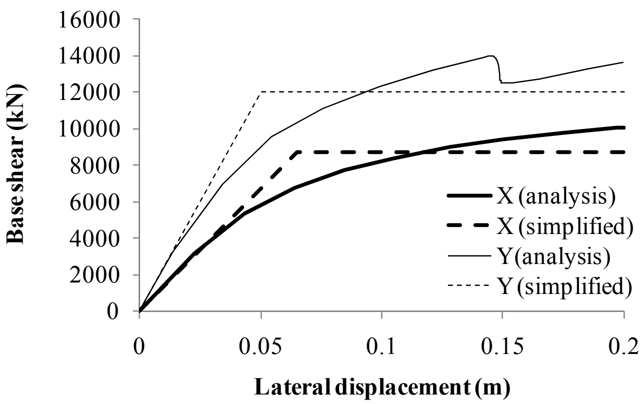

Fig. (7). Pushover results: force-displacement curve.

essentially elastic response of the structure. The same comment is valid noticing the location of the performance points in Figs. (**8b** and **8d**).

For the design acceleration of 0.4g, peak lateral displacements of 74mm and 56mm were determined for the longitudinal and transversal direction of the building. Even under the design earthquake limited structural incursion in nonlinear range can be expected.

These relatively low values of the lateral displacement demand can be explained by the short control period of the acceleration response spectrum for the building site and the relatively high lateral strength of the building. Regarding the strength of the building, a lateral overstrength factor of 2.0 corresponding to a reduction factor R=4 suggested that limited nonlinear behaviour can be expected.

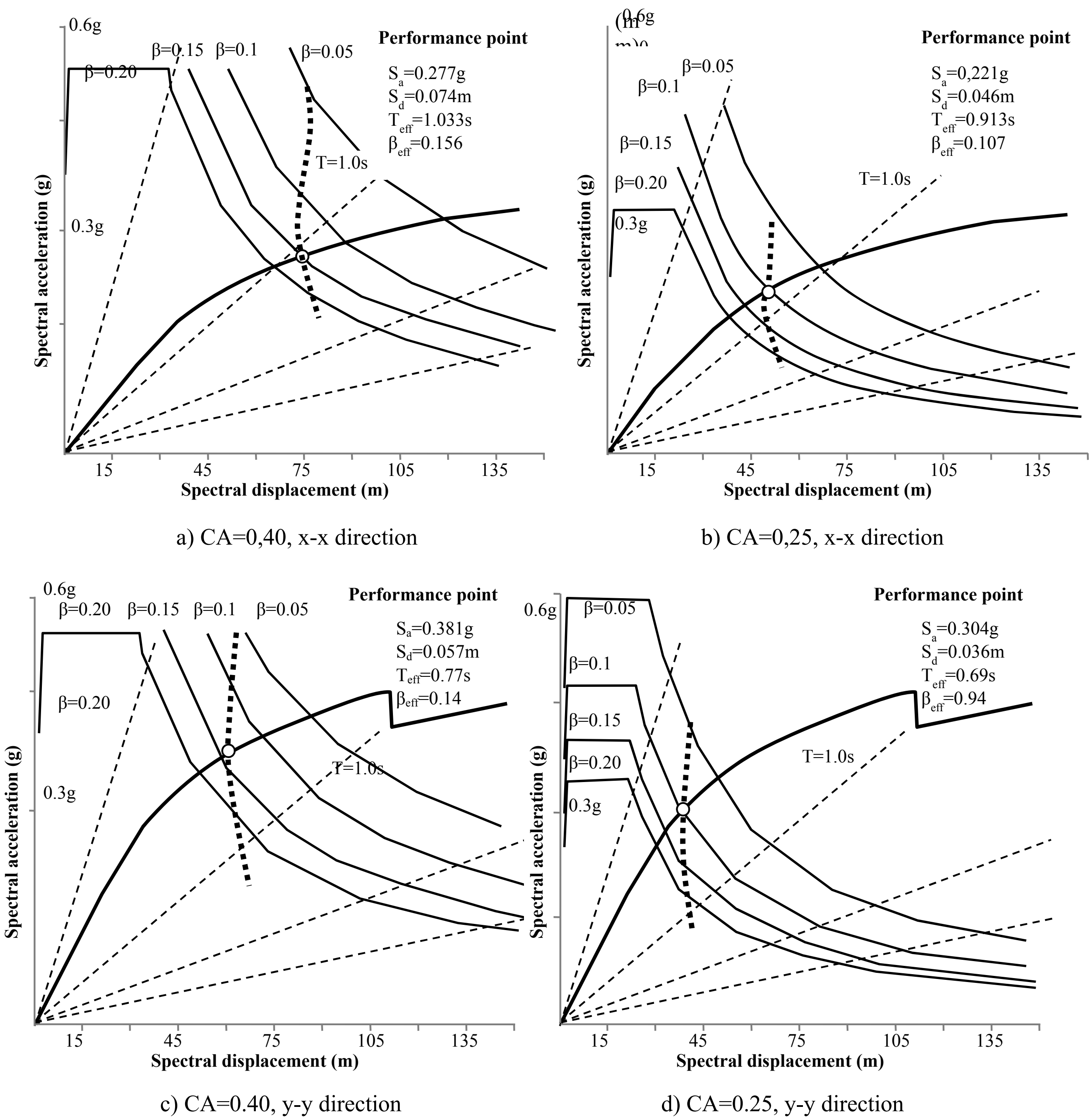

Fig. (8). Capacity spectrum method.

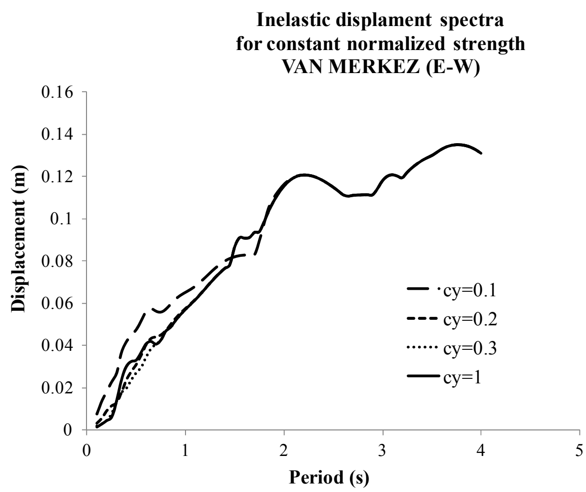

Fig. (9). Inelastic displacement response spectra.

2.2. Linear Time-History Analysis

To determine the amplification of the lateral displacement caused by the structural torsional response, linear time-history analysis was performed. The low lateral displacement demand determined using capacity spectrum method suggested that time-history analysis considering a linear structural response is suitable. Unscaled ground motions recorded at Van Merkez station on November 11, 2011, on N-S and E-W direction were simultaneously used in the analysis [17]. E-W component was considered in the longitudinal direction of the building while N-S was considered in the transversal direction. This is consistent with the actual location of the building as the longitudinal façade was roughly aligned with E-W direction. The Van Merkez seismic station is located in the Van downtown at around 1.5 km from the building site.

A damping ratio of 5% was considered in the analysis.

The inelastic displacement response spectra for E-W component of the recorded ground motion showed that for any single degree of freedom system with relatively high lateral strength the inelastic displacements are roughly equal to the displacements of the elastic system with same vibration period (Fig. **9**). This is consistent with the Newmark rule for relatively flexible systems [18]. This remark makes the results obtained using the linear time-history analysis more credible.

The time history analysis revealed that the building can experience a relatively strong torsional response. Only the inherent torsion caused by the eccentricity between the centre of mass and centre of rigidity was considered in the analysis. Accidental torsion was not accounted for. Peak displacements in the longitudinal direction at roof level varying from 58mm to 70mm on the building perimeter were obtained (Fig. **10a**). This represents a 10% amplification of the perimeter displacements in comparison with the displacement calculated in the centre of mass. This amplification increased in the following cycles but the absolute value of the displacement decreased (Fig. **10b**).

An additional time history analysis carried out using the unscaled El Centro ground motion record showed that at the time step when the peak lateral displacement at the roof level of 10.3cm was reached in the main façade in the rear façade the displacement was close to 0. That means that the maximum lateral displacement on the building perimeter is twice as large as the displacement in the centre of mass. This analysis clearly shows the torsional sensitivity of the structure. Interaction with the non-structural infill masonry walls can further increase this torsional instability.

The time history analysis confirmed the low values of the lateral displacement obtained using the capacity spectrum method. For the Van Merkez station record, maximum base shear forces of 8000 kN and 6900 kN were determined in the longitudinal and transversal direction. This proves that the structural response was essentially elastic, with limited incursions in the nonlinear range.

The lateral stiffness of the structure was difficult to be accurately assessed due to the scattered and uncertain quality of the concrete and interaction with the masonry infills. The dynamic characteristics of the building are related to the lateral stiffness and directly influence the displacement demand. In this respect, a sensitivity analysis was carried out. The variation of the peak displacement demand at the roof level was determined for different values of the vibration period of the building in the longitudinal direction. The concrete modulus of elasticity was varied from 5000 MPa to 22500 MPa and a variation of the vibration period from 1,43 s to 0,68 s was determined. The peak displacement increased with the vibration period (Fig. **11**) as revealed both by the linear time-history analysis and the displacement spectrum analysis but limited values were obtained. This shows that a change in the lateral stiffness of the structure is not likely to significantly modify the conclusions of this study. Such variation of the lateral stiffness might occur if the masonry infills are accounted for or the assumed concrete modulus of elasticity is reduced.

A similar sensitivity study was performed to determine the variation of the peak displacement demand with the stiffness of the linear springs that vertically supports the foundation beams. The results presented in Fig. (**12**) showed that the stiffness of the support springs have little influence on the calculated peak displacement demands at roof level.

2.3. Nonlinear Time-History Analysis

Nonlinear time-history analysis (NTHA) was performed using Perform 3D [19] in order to investigate the structural response in the nonlinear range. The contribution of the masonry infills to the lateral strength and stiffness of the building was considered in the analysis. The infill walls were explicitly defined in the model.

The main objective of the NTHA was to determine if the results of a detailed analysis can explain the structural damage observed during the site inspection.

In the analytical model, the beams and columns were modelled using frame elements having plastic hinges concentrated on each end of the element (lumped plasticity). A trilinear shape was used to define the nonlinear behaviour (Fig. **13**). Cyclic degradation was considered during the

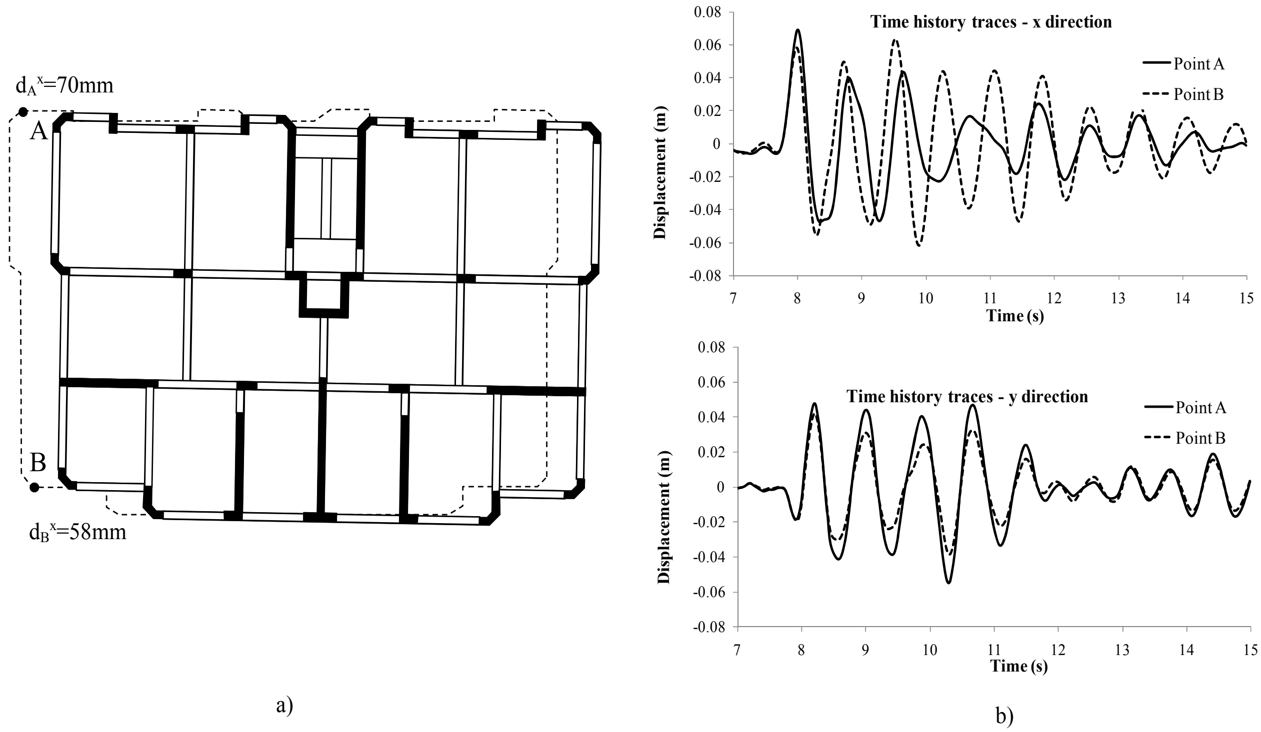

Fig. (10). Amplification of the lateral displacement caused by torsion and time history traces at the roof level.

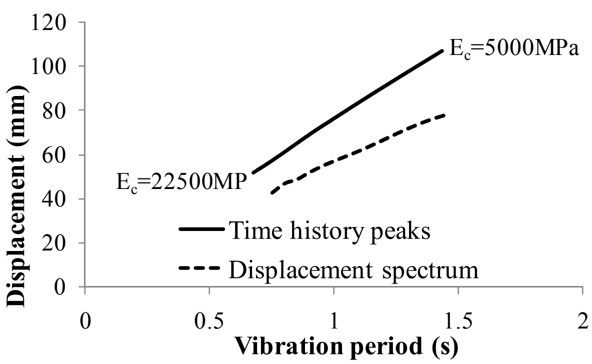

Fig. (11). The variation of the peak displacement with the vibration period of the structure.

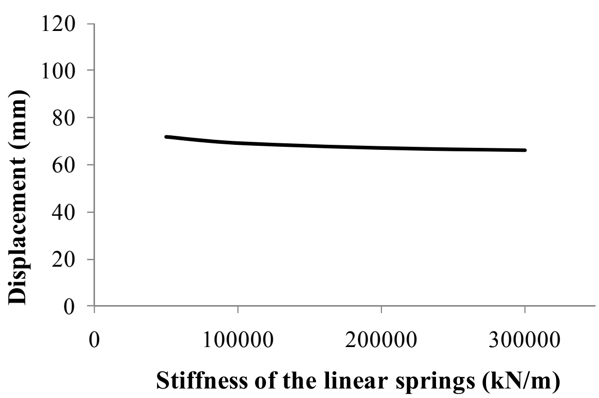

Fig. (12). The variation of the peak displacement with the stiffness of the support springs.

analysis. For the columns the effect of the axial force was taken into account and P-M_2-M_3 nonlinear hinges were used.

The concrete shear walls were defined using a fiber model. For each wall, each section was divided in several macro-fibres and for each fibre a material was assigned. The nonlinear behaviour was assigned at the material level. For the concrete, a trilinear stress-strain rule was used (Fig. **14**). The concrete tensile strength was ignored and loss of compression strength was considered. The steel behaviour was defined using a symmetrical elastic perfectly-plastic stress-strain rule. Cyclic degradation was considered both for concrete and steel.

Foundation beams were supported on nonlinear vertical springs. The tension stiffness for these elements was considered equal with zero. Translations of the foundations in both horizontal directions were restrained. Horizontal rigid diaphragms were considered at each story of the building.

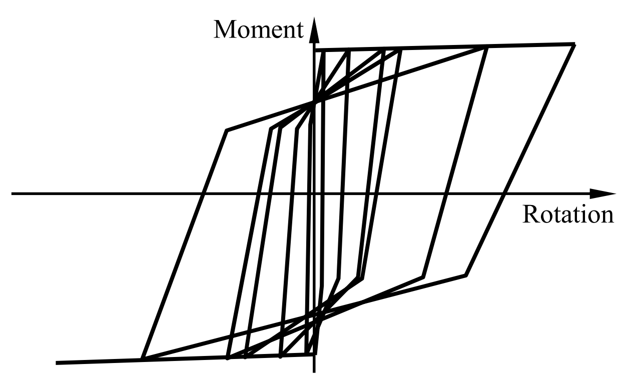

Fig. (13). Hysteretic behaviour of a nonlinear M3 hinge.

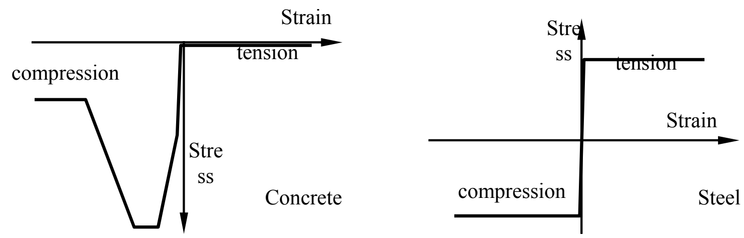

Fig. (14). Stress-strain rule for the concrete and steel.

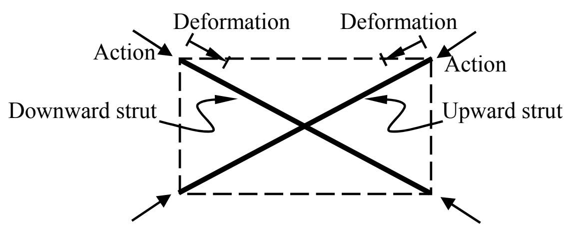

Fig. (15). Infill panel diagonal strut model (Perform 3D – Components and Elements. Version 4, august 2006).

The infill walls were modelled using an inelastic panel - diagonal strut model. This model consists of two struts, each of which resists compression force only. The actions and deformations are the compression forces and compression deformations of the struts, as shown in the Fig. (**15**).

One objective of this NTHA was to determine if the damage sustained by the masonry infills during the Van-Ercis earthquake on October 23 caused a soft storey behaviour of the building during the Van-Edremit earthquake on November 09. Concentration of the infills damage in the ground floor observed during the site inspection suggested that a soft storey response might have been occurred.

A seismic loading scenario consisting of two successive natural accelerograms was used. The first accelerogram was recorded during the October 23 earthquake at the Van Muradiye station and the second one was recorded during the November 11 earthquake at the Van Merkez Station. The Van Muradiye accelerogram was recorded at almost 40 km from the buildings site; nevertheless, the extrapolation of the strong ground motion to be used on the building site was necessary since no other strong ground motion closer to the site was available.

Since the actual information on the strong ground motion that actually occurred on the site of the building during the November 11, 2011 earthquake was unknown, several NTHA's were performed. In each analysis the Van Muradiye strong ground motion was linearly scaled while the Van Merkez ground motion was unscaled. Scaling factors of 0.4, 0.7, 1.0 and 1.3 were successively used in each analysis. Four loading scenarios resulted as presented in Table **2**.

The horizontal displacements envelopes in the longitudinal direction calculated at each story for each loading scenario are presented in Fig. (**16**); with dashed line are presented the results obtained for the Van Muradiye strong ground motion and with dotted line for Van Merkez strong ground motion.

The analysis results showed that the maximum lateral displacements after the first earthquake for each loading scenario increased almost linearly with the applied scaling factor for the first accelerogram. The shape of the maximum drift distribution over the building height remained largely the same, irrespective to the applied scaling factor, but the absolute maximum values of the drift increased. The largest drift values were calculated in the second floor.

The maximum lateral displacements and drifts after the second earthquake scenario remains largely the same in each loading scenario. It seems like the damage sustained by the building during the first earthquake were not severe enough to significantly alter the response during the second earthquake.

Maximum lateral deformations of 55 mm were obtained in the fourth loading scenario at the top of the building. The maximum lateral drift in this scenario was around 1% at the third story. This maximum value was calculated during the incidence of the first earthquake. However, this scenario seems to be very severe. In the third loading scenario, maximum displacement at the top of building of 40mm and maximum drifts of 0.75% in the third and fourth story were obtained. These results are not in line with the observations

Table 2. Seismic loading scenario.

Loading Scenario	Van Muradiye Strong Ground Motion	Van Merkez Strong Ground Motion
	Scaling Factor	
0.4 MUR followed by 1.0 VAN	0.4	1.0
0.7 MUR followed by 1.0 VAN	0.7	1.0
1.0 MUR followed by 1.0 VAN	1.0	1.0
1.3 MUR followed by 1.0 VAN	1.3	1.0

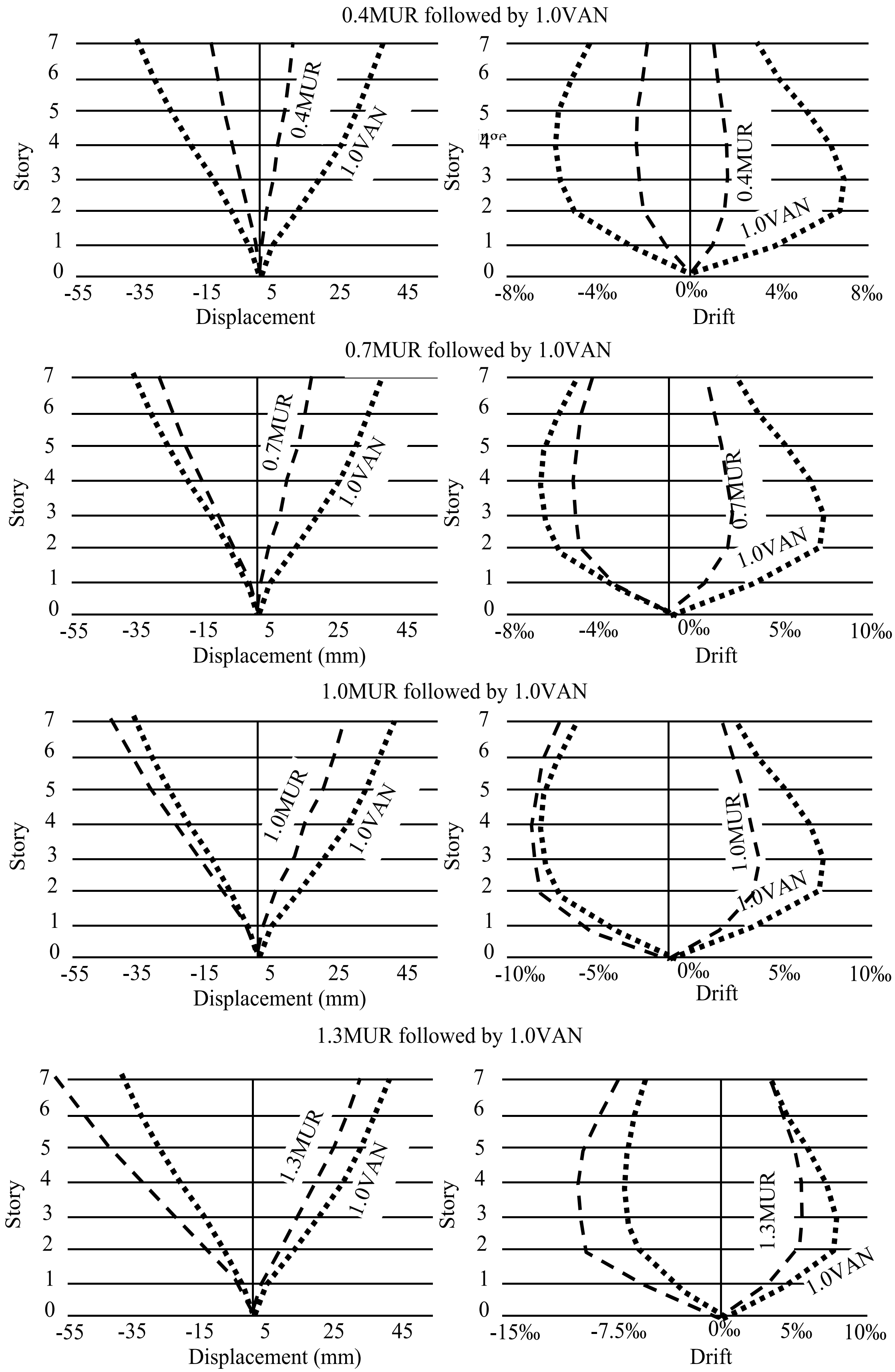

Fig. (16). Horizontal displacement envelopes.

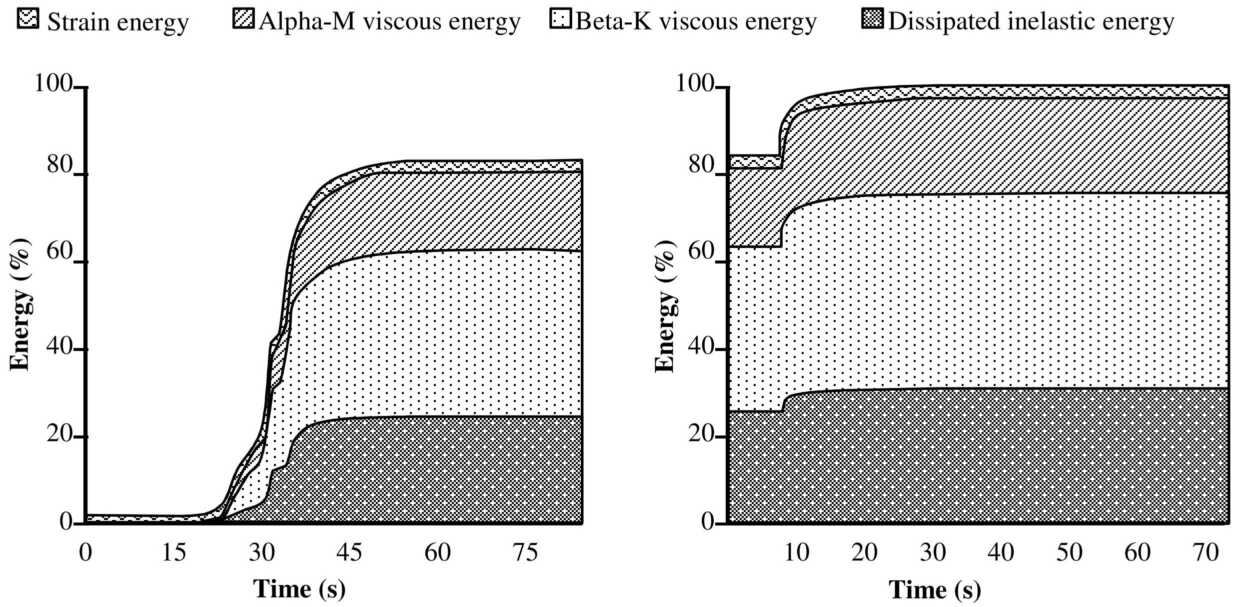

Fig. (17). Energy balance: 1.3MUR (left) followed 1.0 VAN (right).

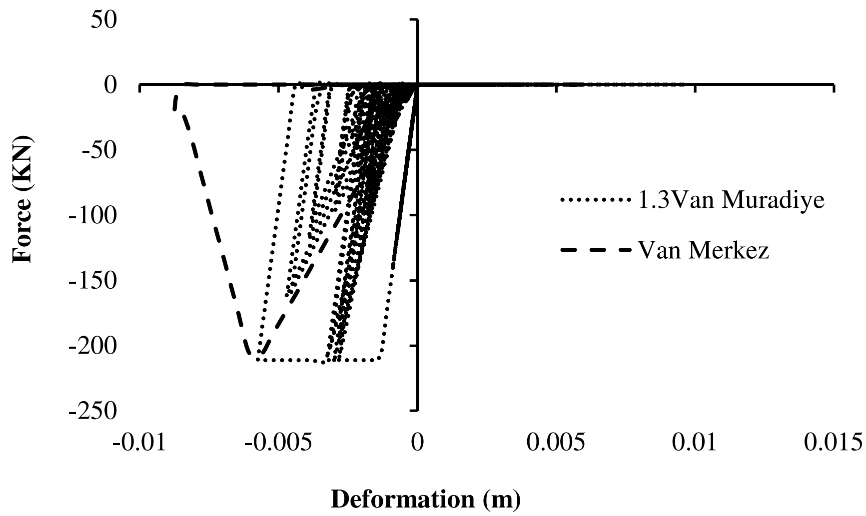

Fig. (18). Hysteresis loops for the downward diagonal strut.

made during the post-earthquake investigation on site when the most damaged infills were observed in the ground floor and the damage after the first earthquake (October, 23) was significantly less severe than that sustained by the masonry partitions after the second earthquake (November, 11).

The energy balance for the fourth loading scenario shows that 24% of the total energy during the first earthquake is dissipated energy by inelastic deformations (Fig. **17**). This ratio increases up to 27% during the second earthquake. The hysteresis loops for a downward diagonal strut shows that during the first earthquake the masonry infills sustained inelastic deformations (Fig. **18**).

Although the NTHA's were performed with advanced tools and the infill walls were very carefully modelled, the results obtained were not able to capture and explain the observed concentration of damage at the ground floor level. This shortcoming might be attributable to (i) the epistemic uncertainties (the lack of appropriate models that can reliably and accurately model the seismic interaction between the infill walls and RC frames) and (ii) the lack of actual strong ground motion recorded close or even on the site of the analysed buildings. These findings point to the stringent need of more research on understanding and modelling the seismic interaction between infills and structural elements as well as to the densification of seismic networks.

3. CONCLUSION

Severe non-structural damage and slight structural damage was observed in two similar concrete shear wall structures during a post-earthquake investigation mission attended by the authors in the city of Van, Turkey. These structures were evacuated at the time of the inspection. A very low concrete quality was observed. A low quality of the masonry works could be observed as well. During the construction the design prescriptions were not followed as some concrete shear walls were relocated without structural engineer guidance. The structural design solution meets most of the requirements of current advanced seismic design codes. The main shortcoming refers to the torsional sensitivity of the structure and the details for local ductility.

The structural analysis showed that buildings have enough strength according to the provision of the Turkish earthquake standard at the time of construction. Lateral overstrength factors of around 2.0 were determined. After the earthquake the design acceleration was increased with 25% but the lateral strength of the buildings is still enough.

The lateral displacement demand, obtained using capacity spectrum method or linear time-history analysis, is relatively low with peak lateral drifts of around 0.5%. This includes the amplification caused by the torsional response observed using linear time history analysis. Nonlinear time-history analysis, performed based on loading scenarios consisting of two successive earthquakes, revealed larger values of the displacements. A maximum 1% lateral drift was calculated based on the most severe loading scenario.

The structural analysis proved that the buildings have suitable lateral strength and deformation capacity. Retrofitting of the buildings for future use is feasible. However, none of the advanced seismic assessment methods used in this research could predict the extensive damage sustained by the masonry partitions in the ground floor. All the assessment methods converged to a similar positive conclusion regarding the seismic vulnerability of the building.

The rapid assessment of these buildings failed to converge to this rather positive conclusion. Nevertheless, the results of the rapid assessment are justified by the low quality of concrete and the noncompliance with the blueprints of the design.

Quality of the construction works represents the main reason for the damage suffered by this building during the earthquakes. The structural works presented major deficiencies opposite to the relatively good quality of the finishing's. Special attention should be paid to the quality of the retrofitting works as it strongly influences their effectiveness.

The post-earthquakes investigation techniques applied worldwide proved to be not always successful because they did not incorporate parameters regarding the local construction techniques, materials, social and economic climate and the expectations of the residents at risk. The lack of methodologies adapted to the regional constraints might lead to overwhelming social problems and misguided decisions of the engineers. Further developing of credible, reliable custom made post-earthquake investigation techniques incorporating parameters regarding the local construction techniques, materials, social and economic climate and the expectations of the residents at risk such as to alleviate the social and economic impact and to improve the decision of engineers is desirable.

CONFLICT OF INTEREST

The authors confirm that this article content has no conflict of interest.

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