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Simplified Seismic Assessment of Confined Masonry Buildings Based on Displacement

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ABSTRACT

The experiences from past Earthquakes show the seismic vulnerability of the confined masonry structures (CMS) against earthquake. Based on the results of experimental analysis, damage in these structures depends on lateral displacement which is exerted to the walls. In this paper based on analytical and numerical models a simplified nonlinear displacement-based approach is presented for seismic assessment of CMS. The methodology is based on the concept of ESDOF and displacement demand compared with displacement capacity at the characteristic period of vibration according to performance level. Displacement demand is identified by using nonlinear displacement spectrum corresponding to a specified limit state. This approach is based on macro model and nonlinear incremental dynamic analysis (IDA) of 3 dimension prototype structure taking into account uncertainty of mechanical properties that finally results into a precise method in its simplicity for seismic assessment of CMS. For validation of this approach the case study is done and it results in the form of an analytical fragility curve which is compared with the precise method.

Keywords: Confined masonry; Displacement Based Assessment; Fragility curves; vulnerability Assessment; Performance Based; Analytical Model; OpenSees; DIANA.

1. Introduction

Considerably, confined masonry structures (CMS) exist or are built as private or public buildings in the world. This kind of structures consist of masonry walls (clay brick or concrete block) accompanied with vertical and horizontal tie elements(steel or concrete) in the four sides of the wall. Simple access to materials, low cost materials and simple technology in construction are some of the reasons for the construction of CMS. The masonry walls play a major role in bearing the vertical and lateral load and tie elements provide ductility for walls against seismic load. The experiences from past earthquakes show that CMS could be vulnerable against earthquake. In addition to the poor detailing of construction, the lack of proper modeling and analysis of CMS against seismic load in determining capacity and demand is the reason for this vulnerability.

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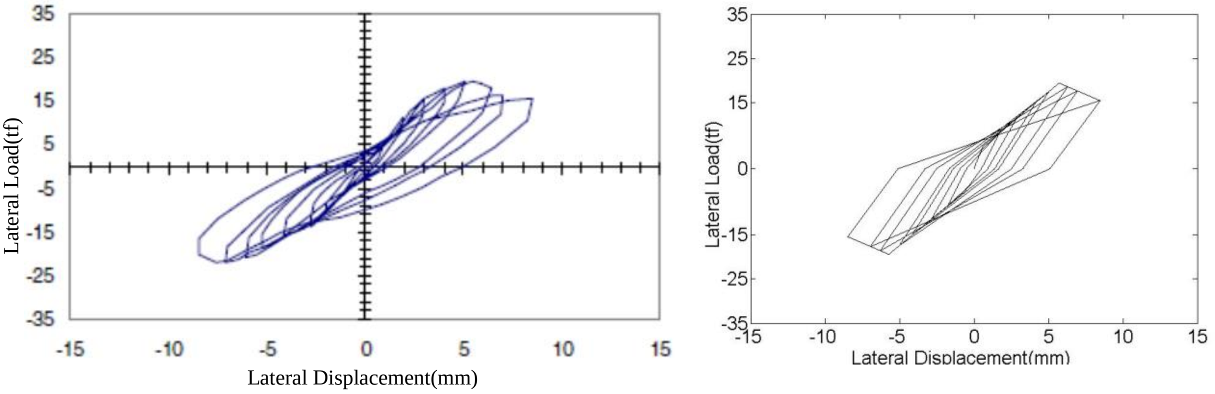
This kind of structure is more often constructed on advice and experimental work because of difficulty and complexity of modeling the confined masonry walls. Based on previous investigations by some researchers it is possible to model the CMS in numerical method (Riahi et al.,2009; Flores and Alcocer,1996;Teran-Gilmore and et al.,2009; Moroni and et al.,1994; Tomazevic and Klemenc,1998;Ranjbaran and et al.,2012), but especially when there is considerable number of CMS in an area with the purpose of risk analysis, it could take a long time and is much an effort for the modeling of CMS. On the other hand by using the concept of designing structures to achieve a specified performance limit state defined by drift limits and using the idea of ESDOF(equivalent single degree of freedom), it is possible to assess the structure against earthquake based on displacement in the simplified method (Priestley and et al.,2007;Ahamad and et al.,2010).
Based on previous investigations, damage of CMS depends on lateral displacement which is exerted to the walls (Ruiz-Garcia and Negrete,2009;Ranjbaran and Hosseini,2014).It shows that it is possible to use the concept of performance and limit states for assessment of this kind of structures against earthquake.
The purpose of this research is to present a simplified method for assessment of confined masonry structures against earthquake based on displacement. This method is based on calculating demand and capacity displacement ratio (DCR) corresponding to the specified drift limit (performance limit) in the ESDOF which is equivalent to the actual building. If DCR is greater than one the building is vulnerable against earthquake otherwise it is invulnerable. The proposed method is achieved by modeling of the 3D prototype structure with the usual plan and variety in mechanical and geometrical properties in OpenSees software (OpenSees,2006;OpenSees,2009). The analyses are based on nonlinear incremental dynamic analysis (IDA) and macro modeling of prototype structure. The author proposed analytical and macro models to analyze CMS based on numerical modeling of confined masonry walls by DIANA software (DIANA,2005;Lourenco,1996) which validated the experimental models(Fig.1)(Ranjbaran and Hosseini,2014).In this method each of the bearing elements of CMS (i.e. masonry wall with tie elements) is modeled by linear element as macro model with similar geometrical properties in masonry wall and nonlinear behavior of confined masonry wall (after crack behavior) is modeled by shear hinge at mid span of macro model with characteristic behavior based on proposed analytical model. The cyclic behavior of macro model is captured by the proposed analytical models in the form of backbone curve and thin Takeda-type hysteretic behavior while stiffness of unloading decreases with progression of displacement (Ranjbaran and Hosseini,2014).

2. Methodology of Displacement-Based Seismic Assessment

Displacement-based seismic assessment of a structure is based on a concept of substitute structure named ESDOF which was proposed by Shibata and Sozen (Shibata and Sozen, 1976).In this approach the actual nonlinear behavior of a building is idealize with an equivalent single degree of freedom (ESDOF) linear system with a bilinear force-displacement response (Fig. 2). In the figure2, H_T is the total height, h_i and$\boldsymbol{\Delta}_i$ are the i-th floor height and lateral displacement for a given displaced shape respectively and mi is floor mass. Me and He represent the equivalent mass and height of the ESDOF system.



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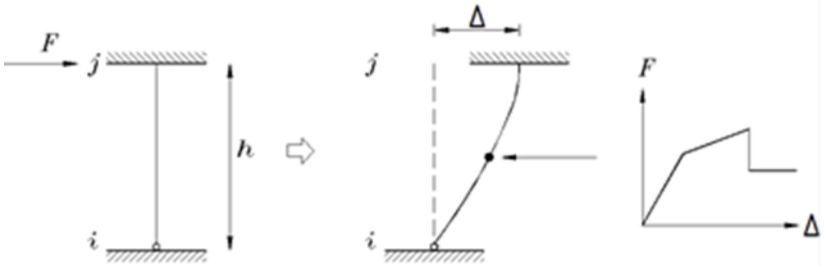

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Fig.1 Comparison between experimental and numerical models related to the masonry wall with confinement: a) Experimental model by Marinilli (Marinilli and Castilla 2004), b) the Numerical model(Ranjbaran and Hosseini,2014), c) The proposed macro model for confined masonry wall.

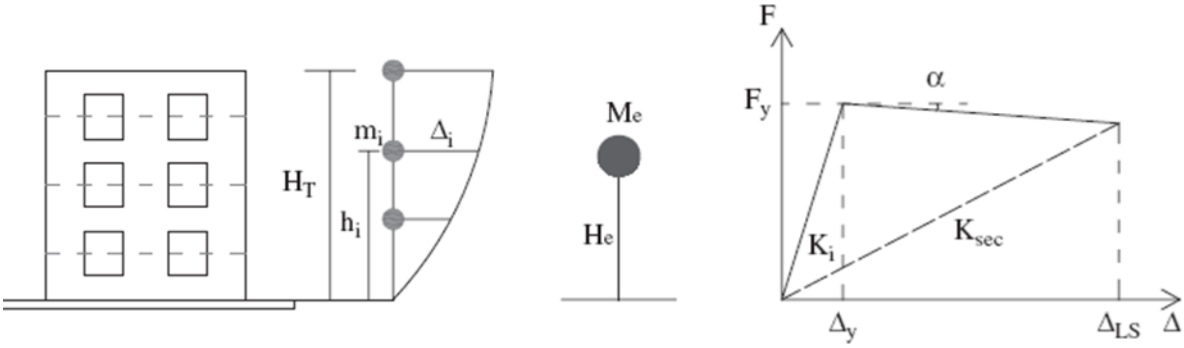


Fig.2 Single degree of freedom idealization of building.

$\boldsymbol{\Delta}_y$ and $\boldsymbol{\Delta}_{LS}$ are equivalent yield and limit state displacement (corresponding to drift limit in actual building) of ESDOF that represents the displacement capacity of the actual building at the center of seismic force according to a specified deformed shape considered for the actual building, Ki and Ksec are the initial and secant stiffness and Fy is the yielding force of the ESDOF,$\boldsymbol{\alpha}$ is the ratio of the post stiffness to the initial stiffness of the ESDOF system which represents the reduction in stiffness and strength of the actual structure due to cyclic response with increasing drift demand. For any limit state the ESDOF system vibrates linearly at secant period, the secant stiffness and equivalent mass of the actual building with viscous damping represent the equivalent damping of the actual building at the specified limit state. Briefly the ESDOF system represents the characteristics of the actual building in terms of its equivalent displacement and the actual energy dissipation at the seismic demand (Fig. 3).In Figure 3, $\boldsymbol{\theta}$ represents drift ratio limit in actual building and ESDOF system.

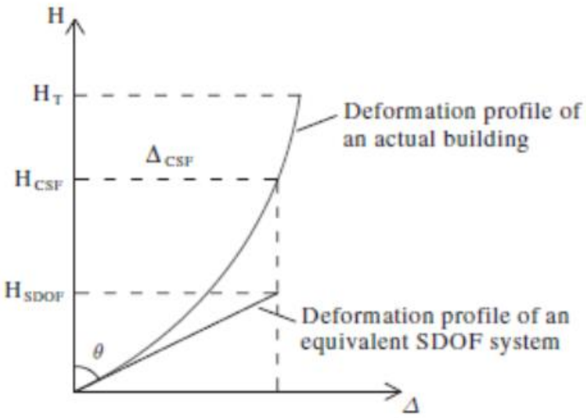


Fig.3 Displacement of actual building and its representation by ESDOF system.

As said previously, limit state displacement depends on the specified deformed shape considered for the actual building(Priestley and et al.,2007).In multi-storey confined masonry buildings with rigid diaphragm in the ceilings, earthquake damage is usually concentrated in the first floor of the building (soft story mechanism in the first floor)(Fig.4) (Ranjbaran and et al.,2012; Alcocer and et al.,1996;Design code,2011).Therefore this failure mechanism is considered to calculate limit state displacement($\boldsymbol{\Delta}_{LS}$), also $\boldsymbol{\Delta}_y$ corresponds to the elastic limit displacement of the building(linear shape of displacement).With the assumption of rigid diaphragm in the ceilings and regularity of plan, $\boldsymbol{\theta}$ could be considered drift ratio limit of the masonry wall.
As said previously, equivalent yield and limit displacement could be formulated in Eqs. (1)–(3), H_e is height of ESDOF system (H_{SDOF})(Priestley and et al.,2007; Ahamad and et al.,2010, Lang,2002):

$$\Delta_y = \theta_y H_e \tag{1}$$

$$\Delta_{LS} = \theta_y H_e + (\theta_{LS} - \theta_y) H_1 \tag{2}$$

$$H_e = \frac{\sum h_i m_i \varphi_i}{\sum m_i \varphi_i} \tag{3}$$

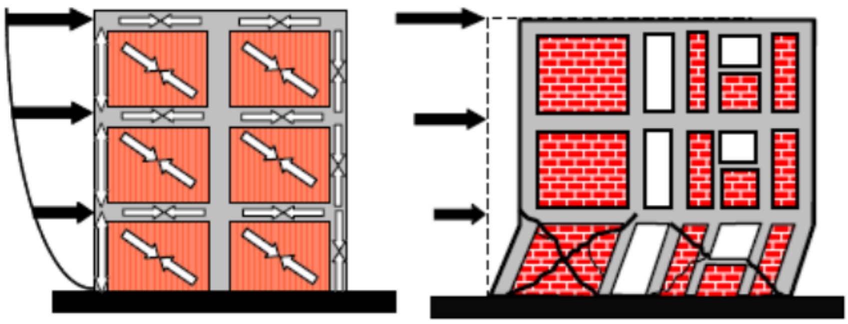


Fig.4 Soft storey mechanism for multi-storey confined masonry building.

In the Eqs.(1,2&3) H_1 is the height of first floor, $\boldsymbol{\theta}_y$ and $\boldsymbol{\theta}_{LS}$ are drift ratio corresponding to yielding and specified drift ratio of the confined masonry wall, φ_i is the first mode displacement at the i-th floor level normalized such that the first mode displacement at the top story φ_n=1.Based on previous investigation of author fragility curves were represented for confined masonry walls based on drift ratio for two limit states LS1& LS2 corresponding to elastic and maximum limit strength respectively(Fig.5) (Ranjbaran and Hosseini,2014). The drift ratio of $\boldsymbol{\theta}_y$ and $\boldsymbol{\theta}_{LS}$ corresponding to LS1&LS2 is calibrated in section 5. Capacity displacements corresponding to elastic limit and maximum strength displacement of ESDOF are calculated By Eqs.(1&2) respectively. Consequently other capacity displacements between the two mentioned capacity displacements corresponding to specified drift ratio limit could be determined.

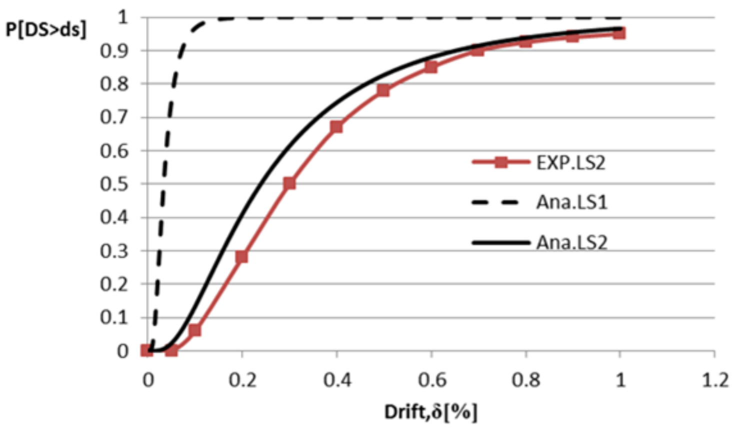


Fig.5 The drift-based fragility curves corresponding to the maximum strength (LS2) and the elastic limit strength (LS1) for CMWs.

In order to calculate demand displacement in ESDOF, the equivalent period of ESDOF corresponding to specified limit state needs to be specified and the inelastic displacement spectra defined. Indeed ESDOF vibrates with secant stiffness (K_{sec}) or equivalent period (T_{LS}) in specified limit state. This parameter could be determined by Eqs. (4&5) (Priestley and et al.,2007;Ahamad and et al.,2010;Chopra and Goel,2001)

$$T_{LS} = T_y \sqrt{\frac{\mu_{LS}}{1 + \alpha\mu_{LS} - \alpha}} \qquad (4)$$

$$\mu_{LS} = \Delta_{LS}/\Delta_y \qquad (5)$$

μ_{LS} is the ductility of the ESDOF at a specified limit state and Ty is the yield period of vibration. In this research α and Ty is calibrated from the nonlinear dynamic time history analysis of the prototype structure.
By specifying fundamental vibration periods at different limit states (T_{LS}) by considering the energy dissipation of the system provided by the actual nonlinear behavior of the buildings, it is possible to define demand displacement from nonlinear displacement spectra. The energy dissipation of the system is considered by lowering the 5% damped or linear displacement spectra using an appropriate reduction factor as proposed by EC8 (CEN,1994). This factor is defined by Eq.(6):

$$\eta = \sqrt{7/(2 + \xi eq)} \qquad (6)$$

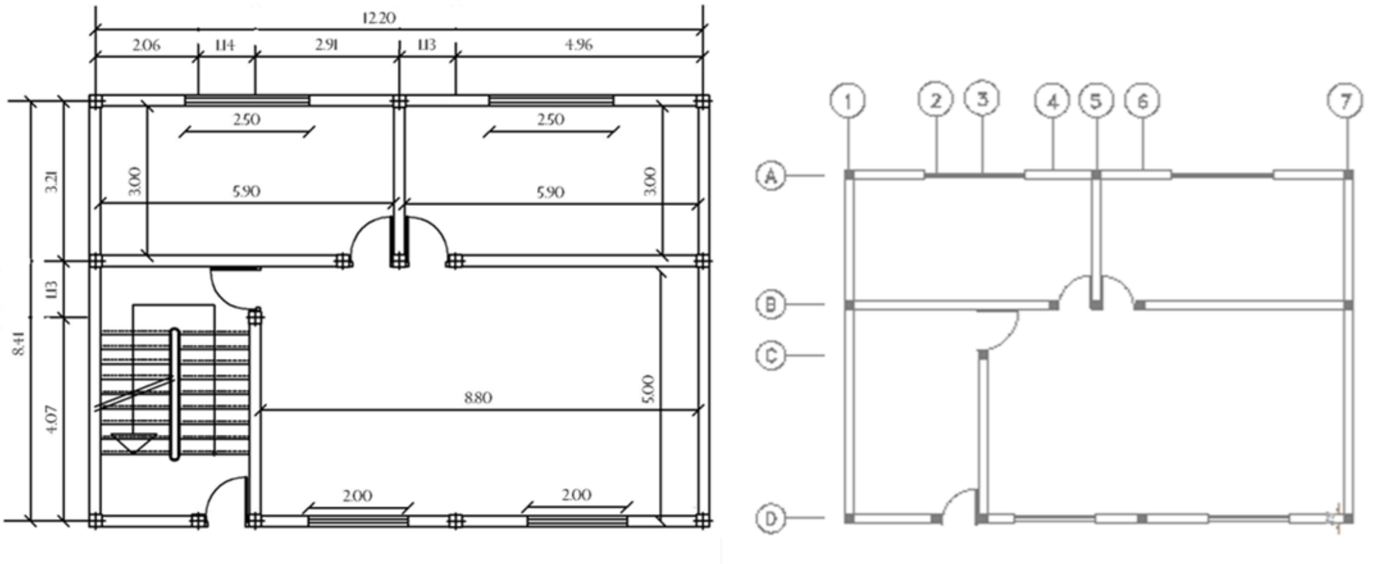
In Eq.(6) η is the reduction factor to reduce elastic displacement spectra and ξeq is the equivalent viscous damping (in percentage) of the system at a given limit state. In this paper the equivalent damping is the sum of the elastic and hysteretic damping (Eq.7)(Priestley and et al.,2007; Chopra and Goel,2001; Dwairiand and et al.,2007):

$$\xi_{eq} = \xi_{el} + \xi_{hyst} \qquad (7)$$

In Eq.(7) ξ_{el} is considered to be 5% (Flores and Alcocer,1996; Ranjbaran and Hosseini, 2014; Tomazevich and Klemenk,1997)and ξ_{hyst} is determined by the total energy absorbed in the hysteretic behavior of substitute structure during response to specific accelerogram (Priestley and et al.,2007;Dwairi and et al.,2007;Chopra and Goel,1999).In this paper ξ_{hyst} is calibrated by the nonlinear dynamic time history analysis of the prototype structure.

3. Prototype structure

The prototype structure is considered as 1, 2 &3-stories of CMS with 3m height for each storey made of clay bricks and having rigid diaphragm in ceilings. The ties were assumed to be of concrete type based on the Iranian Standard No. 2800. The plan of the building is represented in Figure 6.The analysis were performed by OpenSees software (Ranjbaran and Hosseini,2014;OpenSees,2006). Modeling of the prototype structure is carried out using the proposed analytical and macro model developed by author (Ranjbaran and et al.,2012; Ranjbaran and Hosseini,2014) and the nonlinear IDA approach is applied for deriving the results (Fig.7).
The tensile strength of the masonry unit(f_t) is a very important parameter which affects the features of CMW such as the ductility, strength and mechanical properties (Flores and Alcocer,1996; Ranjbaran and et al.,2012), So this parameter was considered as random variables and varies from 0.04 to 0.25 (MPa) (E_m=444 -2778, G_m=178-1111), corresponding to cement-sand mortar with ratio 1:12 and 1:6 respectively.

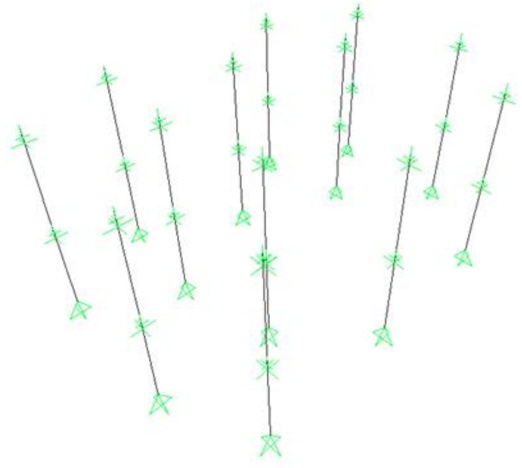


a) b)

Fig.6 Prototype structure: a) With Dimension, b) With Center Line

The thickness of the walls is assumed to be 22cm, and horizontal and vertical ties, which are assigned in the analytical model, are considered in the form of reinforced concrete with dimensions of 20*20cm.The ties were assumed according to the recommendations of Iranian Standard No. 2800 and in both directions of the building, the density of the walls is 5 percent. The reinforcement of the ties consists of 4 steel bars of 10mm diameter with the yielding strength of 300Mpa and the compression strength of concrete was also assumed to be 15Mpa.As an example the properties of elements in the first story of 2 stories CMS with tensile strength of unit masonry equal to 0.25 MPa are represented in Table 1.

Ensembles of 7 earthquake ground motions in the form of two-component records (longitudinal and transverse components) were extracted from the PEER Strong Motion Database (Table 2). The selected accelerograms have PGA values between 0.3 to 0.4g and their significant duration is at least 10 seconds, and they have been recorded on firm soil site (site classification 'B'[USGS]) with a general main period less than 0.3sec due to threatening conditions for typical masonry constructions because of frequency content and the area having high relative risk (Ranjbaran and Hosseini,2014).



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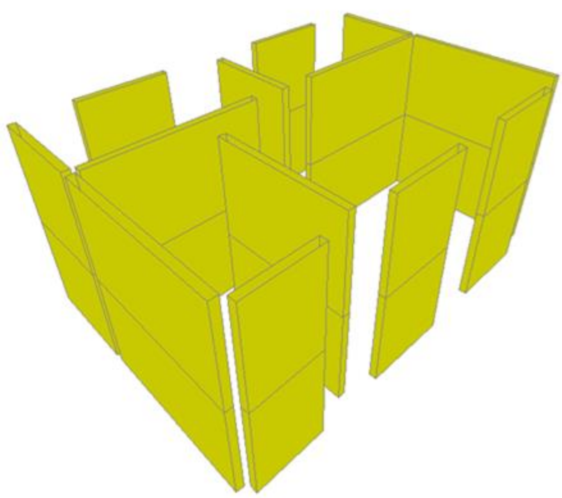

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Fig.7 The3-Dimensional model of the prototype structure: a) Linear Element, b) Extrude View

Table1. The properties of confined masonry walls

	K[KN/mm]	Qp[KN]	Qu[KN]	Qr[KN]	Du[mm]	Dy[mm]	D=(Du-Dy)/Dy	L[m]
B,1-4	197.68	160.1	210.68	118.47	1.73	0.81	1.13	5
D,1-2	50.04	69.8	91.8	43.6	11.68	1.39	7.38	2.1
A,1-5 A,5-7	158.54	68.6	70.7	39.8	1.82	0.43	3.21	5
D,3-6,D,6-7	126.34	65.7	67.7	39.3	1.75	0.52	2.37	4.4
B,6-7	197.68	180.1	236.92	152.01	0.91	0.91	1.00	5
1,5,7 A-B	94.81	94.8	124.74	61.8	6.71	1.00	5.71	3
1,B-D	197.68	160.1	210.68	118.47	1.73	0.81	1.13	5
3,C-D	151.66	135.0	177.62	97.5	3.38	0.89	2.80	4.1
7,B-D	197.68	147.1	193.56	102.01	2.24	0.74	2.01	5

K: Initial stiffness
$\mathbf{Q_u}$: Maximum resistance
$\mathbf{Q_p}$: Elastic limit resistance
$\mathbf{Q_r}$: Residual resistance
D: Ductility
$\mathbf{D_u}$: Ultimate Displacement
$\mathbf{D_y}$: Yield Displacement
L:Wall length

Table 2. Selected earthquake records

Earthquake	Distance(Km)	Station
SAN FERNANDO	24.2	24278
VICTORIA, MEXICO	34.8	6604
WHITTIER NARROWS	22.5	14403
LOMA PRIETA	13	58065
NORTHRIDGE	29	90021
NORTHRIDGE	9.2	24087
CHI-CHI	33.01	TCU047

4. Calibration of the required parameters for Displacement-Based method

As mentioned in section 2 for determination of displacement demand in Displacement-Based seismic assessment approach, it is required to specify T_y, $\boldsymbol{\alpha}$ and ζ_{hyst}, yield period of vibration, ratio of the post stiffness to the initial stiffness and hysteretic damping of ESDOF respectively. For this purpose the assumed prototype structure is analyzed in the form of nonlinear IDA with records in table 2.Although the prototype structure fails beyond PGA of 0.65g in the most of the cases (Ranjbaran and Hosseini,2014), the 7 couple records is scaled from PGA=0.1 to PGA=1g with step of 0.1 and applied in two directions of the model and finally the illogical results are eliminated from the data base.

4.1.Determination of T_y and $\boldsymbol{\alpha}$

The yield period of vibration of each building could be estimated from a simplified Eq. (8):

$$T_y = aH^b \tag{8}$$

Where H_T is total height of actual building and a and b represent coefficients defined for different typologies of buildings by seismic assessment codes or preferably obtained using nonlinear dynamic time history analysis. In this paper these coefficients are calibrated by using the second method.

After analyzing the prototype structure by considering all uncertainty and scaled accelerograms, the base shear force and lateral displacement are obtained. The data are converted to the equivalent properties in terms of lateral force and displacement to represent the building response as an ESDOF system. Equivalent displacement and corresponding equivalent lateral force is obtained by Eqs. (9) to (11) (Priestley and et al.,2007; Ahamad and et al.,2010):

$$\Delta_{\mathrm{eq}}=\frac{\sum_{\mathrm{i=1}}^{\mathrm{n}}\mathrm{M_i}\Delta_{\mathrm{i}}^{2}}{\sum_{\mathrm{i=1}}^{\mathrm{n}}\mathrm{M_i}\Delta_{\mathrm{i}}} \tag{9}$$

$$\mathrm{VB_{eq}}=\frac{\mathrm{VB}}{\mathrm{M_{eq}}} \tag{10}$$

$$M_{eq}=\sum_{i=1}^{n}\frac{M_i\Delta_i}{\Delta_{eq}} \tag{11}$$

Where M_i is the i-th floor mass, VB is maximum base shear force and Δ_i is maximum displacement demand of the i-th floor of the prototype structure for a given accelerogram, also VB_{eq} and M_{eq} is the equivalent lateral force and mass of the ESDOF system respectively. For a given prototype structure with considered properties, Vb_{eq} and Δ_{eq} are obtained for all the accelerograms by increasing PGA, then the equivalent capacity curve corresponding to ESDOF system is derived(Fig.8).Only the pre-yield and yielding points of this curve are used for the computation of yield vibration period by using Eq.(12):

$$\mathrm{T_y}=2\pi\sqrt{\Delta_{\mathrm{eq}}/\mathrm{VB_{eq}}} \tag{12}$$

This procedure is repeated for the prototype structure with all of the mechanical properties separated from total height of structure. Finally ensembles of data could be plotted in the form of Ty (yield period vibration) versus H (height of CMS)(Fig.9).By using nonlinear regression the coefficients of a and b are equal to 0.06 and 0.75 respectively. The coefficients of a and b were obtained as 0.05 and 0.75 respectively for unconfined masonry structures by Ahmad and et al. (2010). There is a slight over estimation of Ty which could be due to assumption of low value of unit masonry tensile strength in its range (0.04-0.25MPa).
In Figure9 it is shown that the dispersion of the T_y is low for buildings with lower heights and high for buildings with higher heights because of the increase in degrees of freedom and possible higher mode of participation.
Tomazevich (1999) proposed the range of tensile strength of the masonry unit $0.03f_m \leq f_t \leq 0.09f_m$ and the range of modules of elasticity $200\mathrm{f_m} \leq E_\mathrm{m} \leq 2000\mathrm{f_m}$($f_m$ is the compressive strength of unit masonry) that results in illogical high values of yield vibration period corresponding to low values of modules of elasticity and vice versa, so in this paper it seems the range of $1000\mathrm{f_m} \leq E_\mathrm{m} \leq 1500\mathrm{f_m}$ is more logical for relatively good details of construction of masonry structures.
Due to cyclic degradation the reduction of stiffness with increasing demand is considered in constitutive behavior of CMS (Ranjbaran and Hosseini,2014), so the factor $\boldsymbol{\alpha}$ is obtained from the identification of the slope of the post yield branch of the equivalent capacity curve corresponding to ESDOF system (Fig8). The slope computation is carried out for all the cases of CMS in each direction separately and then considered together to estimate its mean value. A mean value of zero is observed for all of the cases. It should be mentioned that this value was estimated -0.05 for unconfined masonry buildings (Ahamad and et al.,2010).

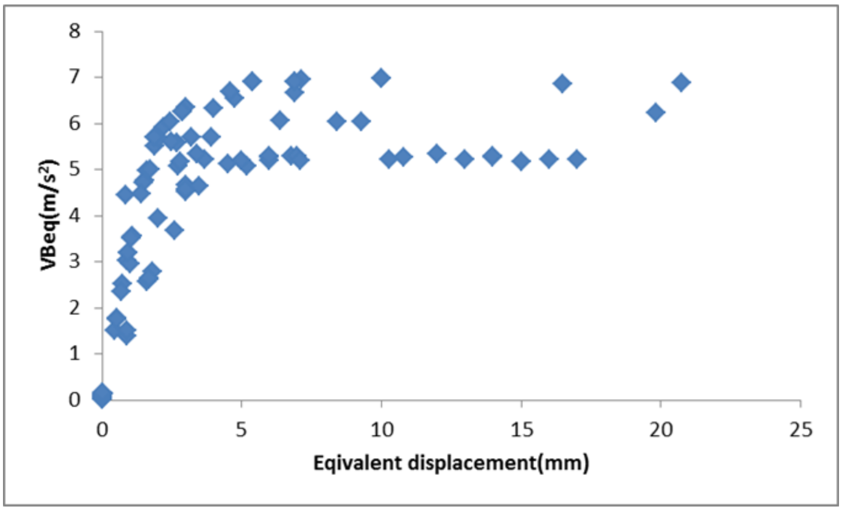


Fig.8 Equivalent capacity curve for two story building in two direction (ft=0.25MPa).

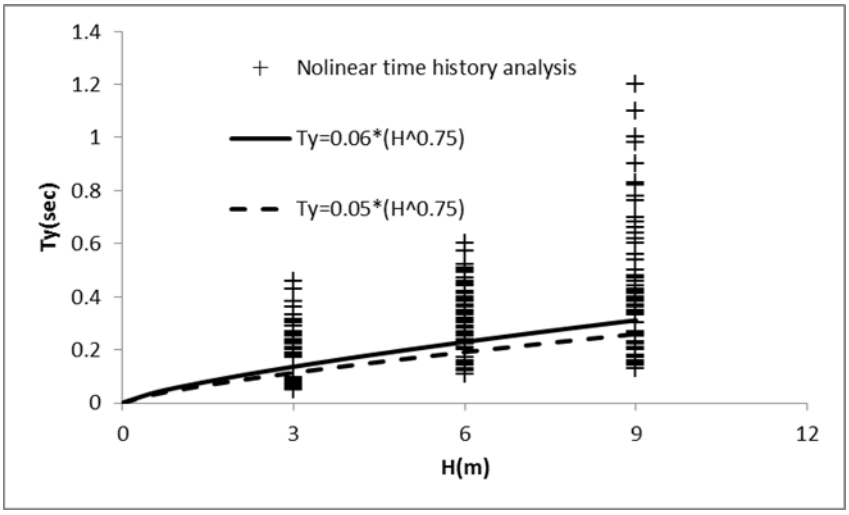


Fig.9 Yield vibration period of prototype structure

4.2. Determination of ζ_{hyst}

The most common method for defining equivalent viscous damping which resulted from hysteretic damping is to equate the energy dissipated in a vibration cycle of the inelastic system and of the equivalent linear system (Dwairi and et al.,2007; Chopra and Goel 1999).Based on this concept, it can be shown that the equivalent viscous damping ratio is:

$$\xi_{hyst} = \frac{1}{4\pi}\frac{E_D}{E_S} \tag{13}$$

E_D is considered as the energy dissipated in the inelastic system given by the area enclosed by the hysteresis loop results from nonlinear dynamic analysis and E_S is the strain energy of the linear system with stiffness k_{sec}(Fig. 10).

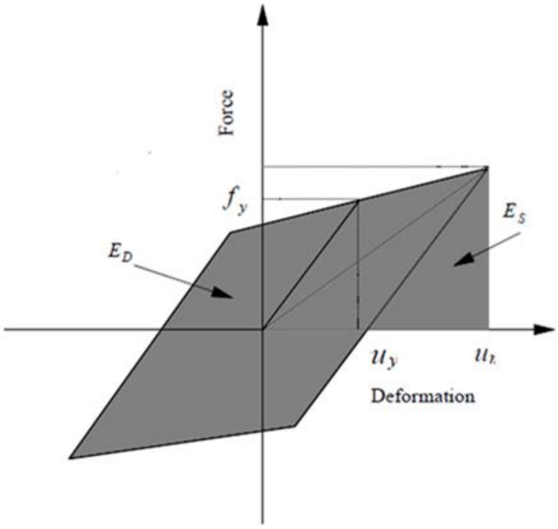


Fig.10 Equivalent viscous damping due to hysteretic energy dissipation.

It is possible to measure the energy dissipation in the prototype structure in terms of ductility (Eq.5) by using the idea of ESDOF system. For this purpose the prototype structures with various properties and height are idealized with ESDOF which is characterized with Me,He and Ke and also hysteretic behavior is assigned to shear hinge at mid span of He(Fig.11), He is defined by Eq.(3) and Me is defined by Eq. (14) (Lang ,2002).

$$\mathrm{M_e} = \sum \mathrm{m_i}\,\varphi_\mathrm{i} \tag{14}$$

m_i is the mass of the i-th floor of the prototype structure and φ_i is the displacement amplitude of the i-th floor of the fundamental mode shape normalized to have unit value at the roof.

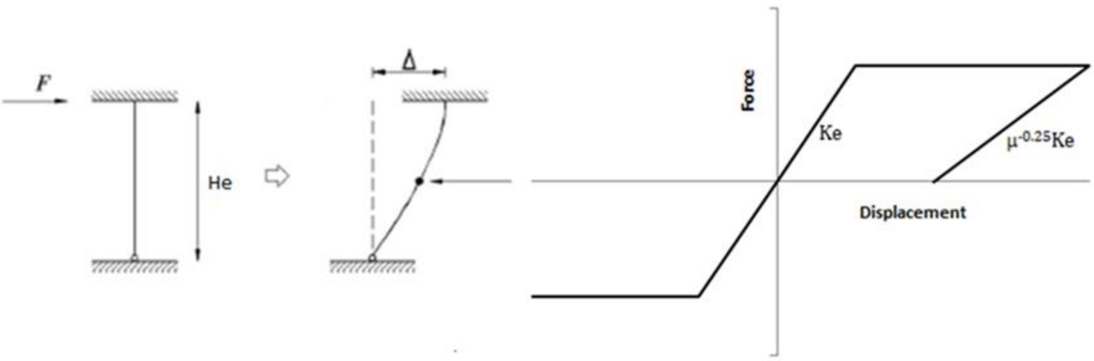


Fig.11 ESDOF system for determination of equivalent viscous damping.

The capacity curves of the prototype structures are obtained from push over analysis with lateral forces in proportion to product of masses and fundamental mode shape and it is transformed to the force-displacement relationship of the ESDOF system. For example Figure12 shows the capacity curve of the 2 storey building with tensile strength of unit masonry equal to 0.25 MPa in the X direction of the building. The equivalent base shear Ve and the equivalent displacement Ue of the ESDOF system are determined by the corresponding values of the prototype structure divided by the modal participation factor as Eqs. (15) to (17) (Jeong and Elnashai,2007):

$$V_e = \frac{V}{\Gamma} \tag{15}$$

$$U_e = \frac{U}{\Gamma} \tag{16}$$

$$\Gamma = \frac{\sum \mathrm{m_i}\, \varphi_\mathrm{i}}{\sum \mathrm{m_i}\, \varphi_\mathrm{i}^2} \tag{17}$$

The transformed capacity curve of ESDOF is defined in the form of perfect elastic-plastic (***α***=0), then for assigning the cyclic behavior of ESDOF system for nonlinear dynamic analysis, the degrading stiffness model thin Takeda-type was employed by using parameter (***β***=0.25) to determine the degraded unloading stiffness based on ductility (Fig.11)(Ranjbaran and Hosseini,2014). The geometrical properties of ESDOF system is defined by Eq.(18):

$$\mathrm{K_e} = \frac{3\mathrm{EI_e}}{\mathrm{H_e^3}} \tag{18}$$

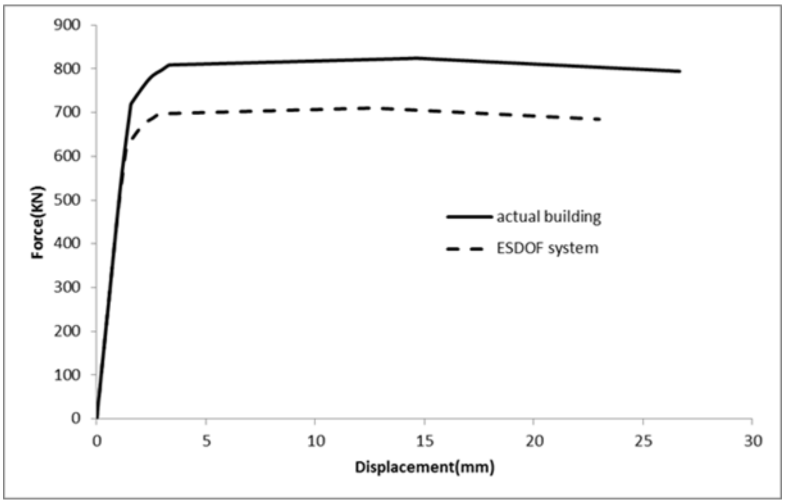


Fig.12 Capacity curve of 2 storey building and its ESDOF system.

By using nonlinear dynamic analysis of ESDOF system through the application of an assumed accelerogram, the response of base shear versus displacement is achieved (Fig.13) and then by nonlinear IDA of the ESDOF system and Eqs.(5)&(13), it is possible to gather the ensembles of data that represent ζ_{hyst} versus μ.

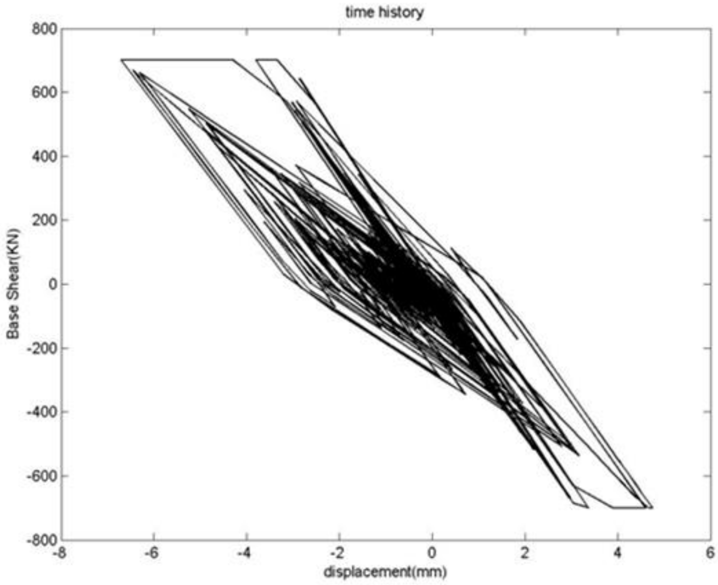


Fig.13 Time history response of base shear versus displacement.

According to Eq.(7) and by nonlinear regression between data in the form of Eq.(19) which is recommended by researchers (Priestley and et al.,2007), the parameter of C is equal to 0.49(Fig.14).

$$\xi_{eq}=0.05+C(\frac{\mu-1}{\mu\pi}) \quad (19)$$

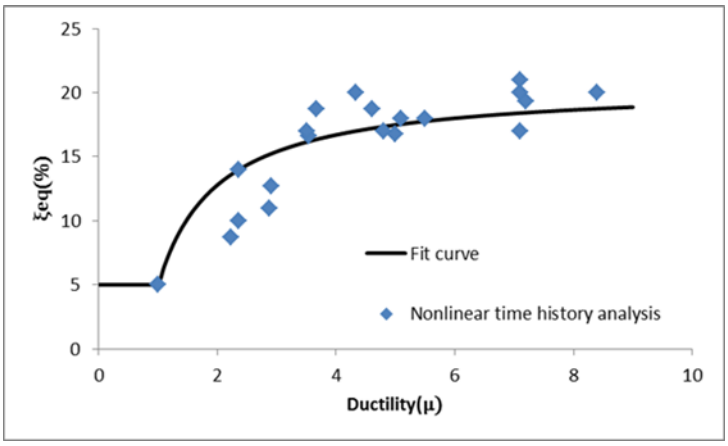


Fig.14 Equivalent viscous damping in terms of ductility.

In figure 15 the equivalent viscous damping of confined masonry building is compared with other types of structures (Priestley and et al.,2007).The hysteretic damping of CMS is near to concrete wall and is much more than the unconfined masonry structures.

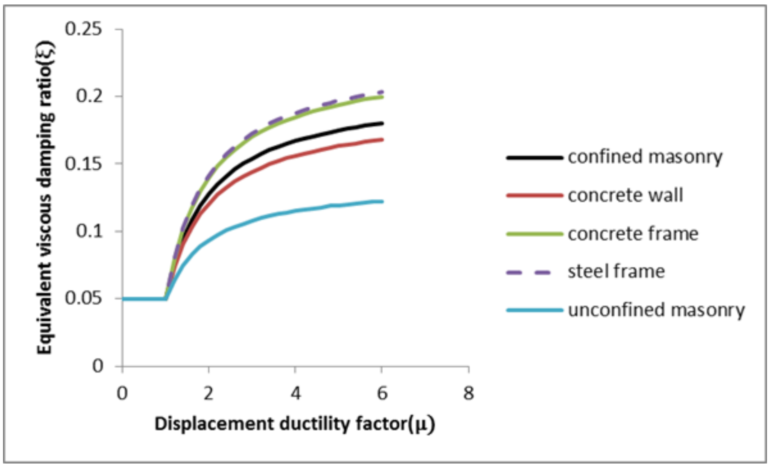


Fig.15 Hysteretic damping of various kinds of structures.

4.3. Relationship between response of ESDOF and actual building

The uncertainty due to simplification of ESDOF approach is quantified by comparing maximum displacement of roof and ESDOF of the prototype structure resulting from nonlinear IDA analysis (Fig 16).The good correlation exists between the results(R^2=0.9).Based on this correlation the following relation is suggested:

$$\delta(\text{actual}) = 0.76 \times \delta(\text{ESDOF}) \tag{20}$$

The above equation is used for identification of demand displacement in the proposed approach in this paper.

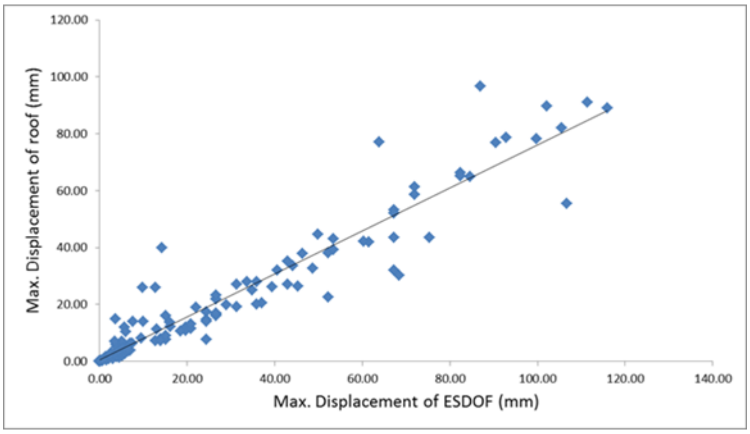


Fig.16 Correlation between response of actual building and ESDOF system.

5. Case study

In this section in order to clarify the proposed method, the 2 storey confined masonry structure placed in regions with high seismicity (with design acceleration of 0.35g) and located in firm site, is assumed as a numerical example. The limit states are considered as displacement corresponding to yield ($\boldsymbol{\Delta}_y$) and maximum strength ($\boldsymbol{\Delta}_u$).The required parameters are as follows:

φ_1 =0.65, $\varphi_2 = 1$, h_1=3m, h_2=6m, $m_{1,2}$=69029Nm/s^2, Me =113.89KNm/s^2, H_e=4.8m,

Γ= 1.16

For simplification φ_i could be considered proportional to the ratio of height of the storey.
As it was mentioned in section 2, $\boldsymbol{\theta}_y$ and $\boldsymbol{\theta}_u$ are calibrated based on previous investigation of author (Ranjbaran and Hosseini,2014). According to previous studies the mean value of $\boldsymbol{\Delta}_y$ and $\boldsymbol{\Delta}_u$ of the 2 storey prototype structure with various parameters are 4 and 24.3mm respectively, so the value of $\boldsymbol{\Delta}_y$ for ESDOF system is 3.45 and consequently $\boldsymbol{\theta}_y$ is equal to 0.072%. According to Fig5, this value of $\boldsymbol{\theta}_y$ corresponds to 90% probability of failure which also corresponds to the value of maximum strength drift ratio of 0.66% simultaneously. These values result in maximum strength displacement of ESDOF system which is equal to 21.1mm or 24.5 mm for actual building. So it seems the allocation of values of $\boldsymbol{\theta}_y$ and $\boldsymbol{\theta}_u$ is equal to 7.2e-4 and 6.6e-3 is logical.
The calculated values corresponding to the limit states are as follows:

$\boldsymbol{\theta}$y= 7.2e-4, $\boldsymbol{\theta}$u =6.6e-3,y=3.45mm,$\boldsymbol{\Delta}$u=21.1mm, μ_{LS} =6.11,Ty=0.23sec,T_{LS}=0.57sec,

ξ_{eq}=0.18,η=0.6

The properties of the assumed accelerograms are presented as Table3:

Table3. The properties of assumed earthquakes

	Direction	r(km)	Mw	PGA(g)	δmax(mm)	Tc(sec)
Sanfernando	x	24.9	6.6	0.32	100.87	3.25
	y	24.9	6.6	0.26	100.87	3.25
victoria	x	34.8	6.4	0.62	45.54	2.75
	y	34.8	6.4	0.58	45.54	2.75
whittier	x	22.5	6	0.29	28.04	1.75
	y	22.5	6	0.39	28.04	1.75
lomaprieta	x	13	6.9	0.51	385.52	4
	y	13	6.9	0.32	385.52	4
northridge	x	29	6.7	0.4	109.04	3.5
	y	29	6.7	0.36	109.04	3.5
northridge	x	9.2	6.7	0.34	343.72	3.5
	y	9.2	6.7	0.3	343.72	3.5
chi-chi	x	33.01	7.6	0.41	760.94	5.75
	y	33.01	7.6	0.3	760.94	5.75

The general form of elastic displacement response spectra could be considered as Figure17 (Priestley and et al.,2007):

$$\delta_{max} = C_s \times \frac{10^{(M_w - 3.2)}}{r} \text{mm} \quad (21)$$

$$T_c = 1 + 2.5(M_w - 5.7)\text{sec} \quad (22)$$

In the Eq's.(21)&(22) r(km) and M_w are the closest distance to fault rupture and magnitude of earthquake respectively. Also C_s depends on the site effect as follows:

Rock: Cs=0.7

Firm ground: Cs=1

Intermediate soil: Cs=1.4

Very soft soil: Cs=1.8

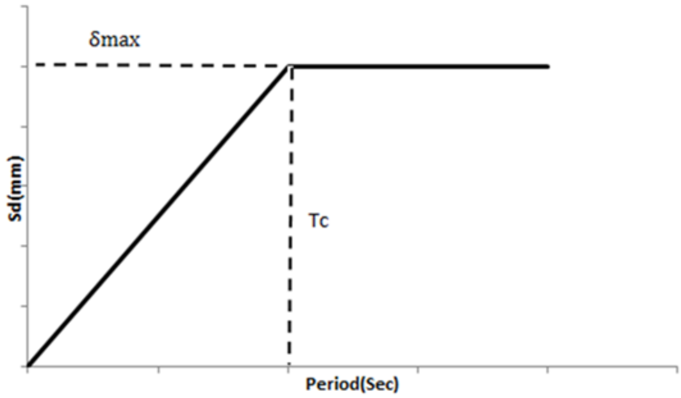


Fig.17 General form of elastic displacement spectra.

In this paper due to assumption of firm ground in the exact method for seismic vulnerability assessment of CMS and comparison of the results with simplified method, the ground of the building is assumed firm ground (Ranjbaran and Hosseini,2014).
For example the values of δ_{max} and T_c for Chi-Chi and Northridge are equal to 760.94mm, 5.75 sec and 109.04mm, 3.5 sec respectively. The inelastic displacement response spectra corresponding to maximum strength limit state assuming PGA equal to 0.25g is determined with multiplying δ_{max} by η and scaling of PGA:

Chi-Chi: $\frac{0.25}{0.3} \times 0.6 \times 760.94 = 379.21mm$

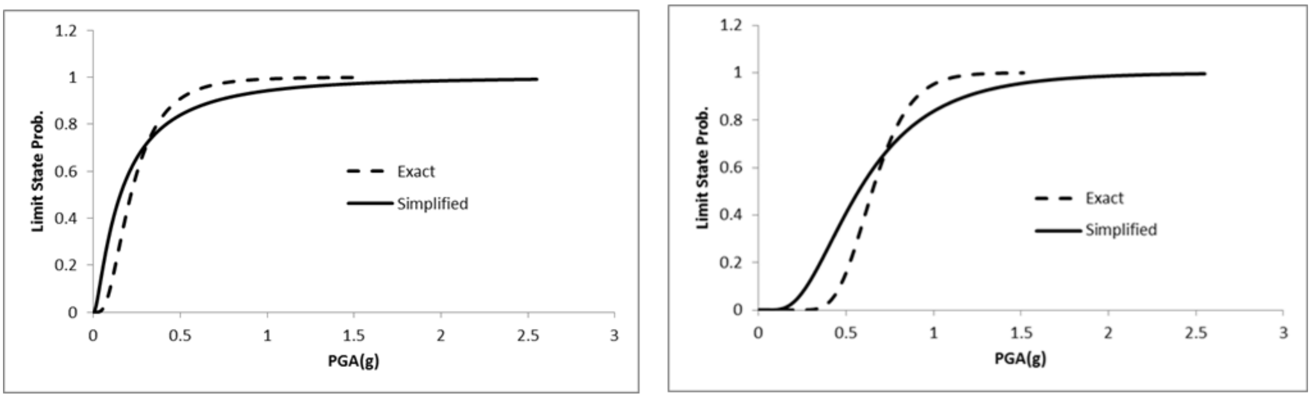
Northridge: $\frac{0.25}{0.4} \times 0.6 \times 109.04 = 40.78mm$

Therefore Demand displacement corresponding to T=0.57 sec and calibration factor of 0.76 for Chi-Chi and Northridge are equal to28.56 and 5.04 mm respectively.

Chi-Chi: $28.56 > 21.1 \rightarrow Vulnerable$

Northridge: $5.04 < 21.1 \rightarrow Not\ Vulnerable$

For comparison of the results between proposed simplified and exact methods which were carried out previously by author (Ranjbaran and Hosseini,2014), the fragility curves by two methods are compared with each other corresponding to elastic (LS1) and maximum strength (LS2) limit states (Fig.18).
It should be mentioned that in the exact method the prototype structure is considered for walls with 22 and 35 cm of thickness but in simplified method only 22 cm of wall thickness is considered. It is believed that by considering wall thickness of 35 cm in the proposed method the yield vibration period (Ty) decreases so that the fitness of the two methods could be more precise.



a) b)

Fig.18 The fragility curves of CMS by simplified and exact method: a) LS1 limit state, b) LS2 limit state.

6. CONCLUSIONS

A simplified method is presented for seismic vulnerability assessment of confined masonry structures based on displacement. For this purpose the ratio of demand and capacity displacement (DCR) of equivalent single degree of freedom (ESDOF) system, which corresponds to actual building, are compared. If DCR is more than one the building is vulnerable and if less it is invulnerable. The demand displacement is determined based on nonlinear displacement spectra corresponding to specified limit state and the capacity displacement is determined based on concept of single degree of freedom system and capacity of drift corresponding to specified limit state. The proposed method is precise in its simplicity.

This method is especially useful for hazard analysis in the earthquake prone areas with considerable number of confined masonry buildings. For validation of the proposed method the results are compared with exact method and the satisfied results are achieved.

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