Toward Worldwide Guidelines for the Development of Analytical Vulnerability Functions and Fragility Curves at Regional Level

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SUMMARY
The employment of capacity curves for the assessment of seismic performance of buildings has found prominence in various procedures, such as, N2 Method and Capacity Spectrum Method. This paper examines the various modelling techniques to identify their strengths in relation to the accuracy of seismic capacity curves prediction and their applicability for seismic performance evaluation. The case of Reinforced Concrete structural system is considered with the inclusion in the analyses of infill panels, in order to determine the limits and differences of these modelling techniques. The present study is conducted within the framework of the research project "Global Vulnerability Estimation Methods" funded by the Global Earthquake Model (GEM) foundation. The purpose of the project is to develop guidelines for the derivation of robust analytical seismic vulnerability functions and fragility curves for applicability worldwide.

Keywords: Capacity curve, nonlinear static analysis, seismic vulnerability assessment

1. INTRODUCTION

The N2 Method (Fajfar 2002) and the Seismic Capacity method (Freeman 1998), which have been recommended in Eurocode 8 (CEN 2004) and ATC-40 (ATC 1996), respectively, are considered as sufficiently simple and reliable methods to conduct seismic performance assessment, considering large sets of buildings, for instance at urban level, with the aim of deriving vulnerability functions and fragility curves. The two methods are based on the representation of the seismic behaviour of a building by a capacity curve, which can be obtained by using nonlinear approaches, such as, Static Pushover Analysis.

Applications of pushover analysis approach are increasingly common in practice due to its relative simplicity in estimating the response of inelastic structures (Albanesi et al. 2002). This approach, first introduced in ATC-40 (ATC 1996), in the last decade has been fully taken up in standards and guidance materials, first in the US, Japan and EU, (FEMA 356 (FEMA 2000), the Building Standard Law of Japan (MOC 2000), Eurocode 8 (CEN 2004)), then in other recent national Codes worldwide, from the Italian Seismic Code OPCM 3274/03 (OPCM 2003), to the Turkish Seismic Code (MPWS 2006), to the Guidelines of the New Zealand Society for Earthquake Engineering (NZSEE 2006, Davidson 2010), to the Chinese seismic design code GB50011-2001 (National Standard of PRC 2005), to the Colombian Seismic Code, (NSR 2010)…etc.

This paper examines the various modelling techniques to identify their strengths in relation to the accuracy of seismic capacity curves prediction and their applicability for seismic performance evaluation. The case of Reinforced Concrete (RC) structural system is considered with the inclusion in the analyses of infill panels, in order to determine the limits and differences of these modelling techniques.

2. ANALYTICAL MODELLING

The purpose of the GEM project is to develop guidelines for the derivation of robust analytical seismic vulnerability functions and fragility curves. Hence, it is important to propose a simple and effective tool, involving relatively modest calculations effort and computing time. Accordingly, the performance of pushover analysis will depend upon the sophistication of modelling structures, the adopted materials behaviour, and the simplified assumptions that are made to reduce the calculation efforts. On the other end the methodology proposed needs to be sufficiently flexible to accommodate the variety of code requirements and construction detailing, past and present, which can be found worldwide.

2.1. Modelling Reinforced Concrete Members

Fiber-based structural modelling was adopted to model the reinforced concrete members using the finite elements software package SeismoStruct (SeismoSoft 2011). This type of modelling allows characterizing in higher detail, the nonlinearity distribution in RC elements by modelling separately the different behaviour of the materials constituting the rc cross-section (Fig. 1) and, hence, to capture more accurately response effects on such elements.

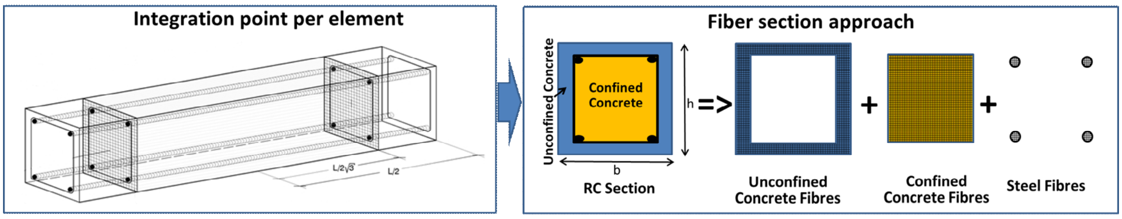


Figure 1. Idealisation into fibers of reinforced concrete members

2.2. Modelling of the Masonry Infill Panels

In the literature, many models of infilled reinforced concrete frames were proposed in an attempt to provide a better understanding of infill panels' behaviours and define potential infills failure mechanisms. The Diagonal Strut model (Fig. 2a) is the most adopted in many documents and guidelines, such as, FEMA (2000), NZSEE (2006). This model was first introduced by Polyakov (1956), and reviewed by Holmes (1963) and Stafford-Smith (1967). Later, a number of authors (Paulay and Priestly (1992), Mehrabi et al. (1996), Biondi et al. (2000)) established a wide range of supplementary equations to estimate masonry strength and stiffness.

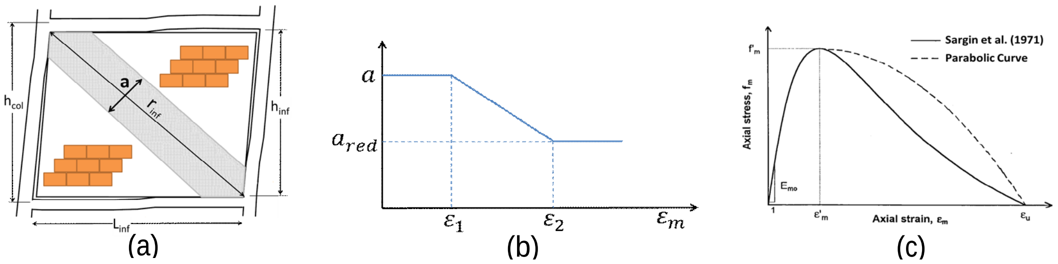


Figure 2. (a) Equivalent diagonal strut representation of an infill panel, (b)Variation of the equivalent strut width as function of the axial strain, (c) Envelope curve in compression (Crisafulli 1997)

2.2.1. Equivalent Strut Width

The width of equivalent strut "*a*" is the most investigated parameter that can be used to assess the stiffness and strength of an infill panel. The different formulae proposed by several researchers (Holmes 1963, Klingner and Bertero 1976, Liauw and Kwan 1984, Paulay and Priestley 1992) lead, given the same geometry, to differences up to 35%. According to the recommendation given by

FEMA-356 (FEMA 2000) and by several other provisions and guidelines, the equivalent strut width can be calculated using the formula based on the early work of Mainstone and Weeks (1970) and Mainstone (1971):

$$a = 0.175\,(\lambda_I h_{col})^{-0.4} r_{inf}\,, \quad \lambda_I = \left[\frac{E_m t_{inf} \sin 2\theta}{4 E_c I_{col} h_{inf}}\right]^{\frac{1}{4}} \tag{2.1}$$

When the elastic limit of the infill panel is exceeded due to the cracking, the contact length between the frame and the infill decreases as the lateral and consequently the axial displacement increases, affecting thus the area of equivalent strut. To take into account this fact the width of the equivalent strut must be reduced. In SeismoStruct, it is assumed that the strut area varies linearly as function of the axial strain as shown in Fig. 2(b). This variation takes place between two strains: strut area reduction strain (ε_1) and residual strut area strain (ε_2).

2.2.2. Envelope Curve in Compression

It is widely observed that failure of infill panel occurs at small lateral displacement before the frame reaches its strength. However, the system frame-infill panel is able to resist increasing lateral loads, hence, restrains the cracked infill panel. This effect leads to smoother decrease of the resistance of the infill panel. According to Crisafulli (1997), the descending branch of the strength envelope can be described by a parabolic curve as it is shown in Fig. 2(c). Crisafulli (1997) also assumed that the expression of strain-stress proposed by Sargin et al. (1971) originally for concrete can approximately represent the envelope curve for masonry.

2.3. Performance Damage Limit States

2.3.1. Crushing of Concrete

Within the context of a fibre-based modelling approach for the reinforced concrete frames, the different performance checks are carried out for each integration section of the selected member. Material strains do usually constitute the best parameter for identification of the performance state of a given structure. Priestley et al. (1996) provided a simple relationship to determine the ultimate concrete compressive strain

$$\varepsilon_{cu} = 0.004 + \frac{1.4 \rho_s f_{yh} \varepsilon_{su}}{f_{cc}} \tag{2.2}$$

Table 2.1. Calculation of shear strength carried by concrete (V_c) according to guidelines and seismic codes

For members subject to axial compression, as well as bending and shear $V_c = 0.166\left(1 + 0.073\frac{N_u}{A_g}\right)\sqrt{f_c'}\,bd$ For members subject to axial tension, as well as bending and shear $V_c = 0.166\left(1 + 0.29\frac{N_u}{A_g}\right)\sqrt{f_c'}\,bd$ For members subject to bending and shear only $V_c = 0.166\sqrt{f_c'}\,bd$	Recommended in UBC-1997, ACI 318-05
$V_c = 3.5\lambda\left(k + \frac{N_u}{2000A_g}\right)\sqrt{f_c'}\,bd$	Recommended in ATC-40, FEMA-273 and FEMA-306
$V_c = 0.182\left(1 + 0.07\frac{N_u}{A_g}\right)\sqrt{f_c'}\,bd$	Turkish Standards Institute TS500 (2000)

2.3.2. Shear-Strength Capacity

It clearly understood that for a reliable assessment of reinforced concrete buildings, the possibility of shear failure in members should be taken into account, especially for building designed without considering horizontal actions, or building with low concrete strength. Different formulae have been proposed in the literature for the calculation of the shear strength. The total shear strength of each reinforced concrete member is calculated as the sum of the shear capacity of the concrete (V_c) and the shear capacity of the reinforcement (V_s). For the calculation of V_c, Table 2.1 shows some of the existing formulae that have been recommended in guidelines and seismic codes. V_s is calculated in accordance with equation (2.2):

$$V_s = \frac{A_s f_s d}{S} \tag{2.2}$$

2.4. Modelling Validation

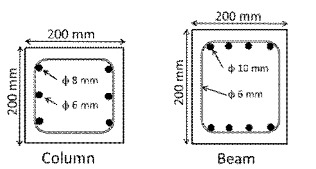
To validate the modelling assumptions presented in the previous sub-sections, simulations of the testing programme reported by Colangelo (2005) were undertaken. For this study, the selected benchmarks are two single-storey (column height=1425mm), single bay (bay length=2500mm), half-size-scale specimens: reinforced concrete without masonry infill (V10), reinforced concrete with masonry infill (V11). These two specimens were intended to represent the ground floor of a four-storey masonry infilled concrete building, and are representative of older structures designed using Italian reinforced concrete non-seismic code provisions. Table 2.2 shows the strut model parameters used for the simulation of infill panel in specimen V11.

Table 2.2. Model parameters used in strut simulation of infill panel in specimen V11

Strut Model	Strut width [mm]	Elastic Modolus of Masonry E_m [N/mm2]	Diagonal comprenssive strength f_m [N/mm2]
Double	160	3188	1.9

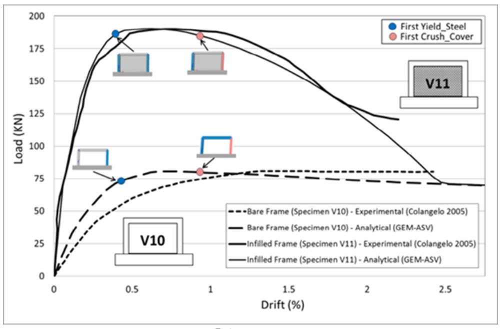
The experimental capacity curves presented in this paper (Fig. 4) resulted from pseudo-dynamic test without failure. For the structural elements, it is stated that the experimental results did not evidence any significant damage, such as, shear failure. In fact, flexural cracking was the mostly observed, especially, in the columns. For the infills, the observed damage was mostly local failure, especially around the top corners. The two specimens were characterized by a high concrete compressive strength (see Fig. 4a), which leads to assume to be sufficient to prevent shear failures.

Specimen	Concrete Properties	
	fc [MPa]	Ec [MPa]
V10	49	35.3
V11	39.6	33.1



Specimen	Steel Strength					
	fy [MPa]			fu [MPa]		
	ϕ = 6 mm	ϕ = 8 mm	ϕ = 10 mm	ϕ = 6 mm	ϕ = 8 mm	ϕ = 10 mm
V10	345	385	364	437	482	462
V11	345	385	364	437	482	462

(a)



(b)

Figure 4. (a) Average concrete material properties and steel strengths, (b) Comparison of load-displacement curves obtained from analytical models with those obtained from experimental test for RC Bare Frame (V10) and masonry-infilled RC Frame (V11).

Fig. 4b shows a comparison of load-displacement curves obtained from analytical models with those obtained from experimental test for RC Bare Frame (V10) and masonry-infilled RC Frame (V11). It is clearly seen that a very good representation of the experimental responses was obtained from the analytical models.

In an attempt to obtain these results, several modelling parameters were investigated which allowed to identify the significance of each modelling parameters of infill panel in deriving the capacity curve. It is worth to mention that a number of researchers have attempted to calibrate some of the parameters for modelling infill panels and suggest default values (Blandon-Uribe 2005, Smyrou et al. 2006).

2.4.1. Reduced Strut Width

According to Al-Chaar (2002), a reduction factor for existing infill panel damage can takes values of

0.7 and 0.4 for moderate and severe damage, respectively. Form Fig. 5(a), it is observed that this parameter has a significant influence on the peak load, leading to differences of up to 38% from the minimum value. However, this factor does not seem to have a significant effect on the ultimate drift at failure of the infill panel.

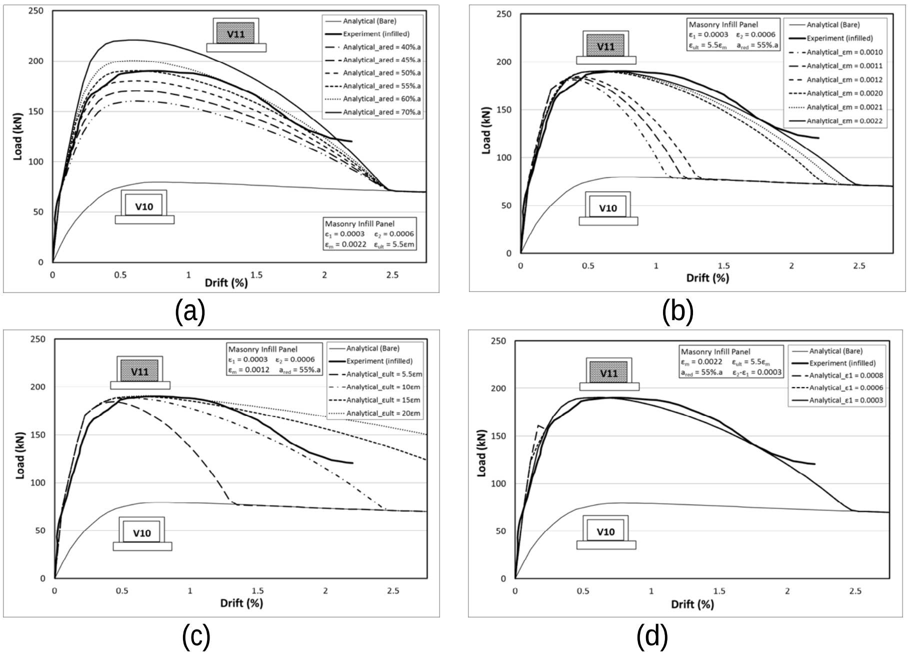


Figure 5. Effect of different diagonal strut parameters on the simulation of the capacity curves for the masonry-infilled RC Frame (V11): (a) Effect of the reduction strut width parameter, (b) Effect of the strain at maximum stress, (c) effect of the ultimate strain, (d) effect of strut area reduction strain and residual strut area strain.

2.4.2. Strain at Maximum Stress ε_m

This factor, which should be calibrated through the consideration of experimental data, may vary from 0.001 to 0.005 (SeismoStruct 2011). Fig. 5(b) shows that this parameter does not seem to have an effect on the peak load capacity, while it significantly influences the post-peak branch of the capacity curve, hence, influencing the uncertainty in evaluation of post peak performance points.

2.4.3. Ultimate Strain ε_{ult}

As suggested by Crisafulli (1997), this parameter was modelled with a parabola, so that the decrease of the compressive strength becomes smoother and the analyses more stable. However, it is widely observed from experiment that a complete collapse may occur just after appearance of cracking. Accordingly, several ratios $\varepsilon_{ult}/\varepsilon_m$ were considered as shown in Fig. 5(c). The effect of ε_{ult} on capacity curves is similar to ε_m.

2.4.4. Strut Area Reduction Strain (ε_1) and Residual Strut Area Strain (ε_2)

For these two parameters, which are related to the strut area reduction there is no enough experimental supporting evidence. It is mentioned that ε_1 may be in the range of 0.0003 to 0.0008 whilst for ε_2 in between 0.0006 and 0.016 (SeismoStruct 2011). According to what is observed from Fig. 5(d), these two parameters do not seem to have any effect on the capacity curves, besides a small peak assumed to be due to numerical error.

3. PARAMETRIC ANALYSIS

The aparametric analysis is conducted on a typical 4-storey RC building located in a high-seismicity region of Turkey. The building is designed according to the 1975 Turkish seismic code (MPWS 1975). Material properties are assumed to be 16 MPa for the concrete compressive strength and 220 MPa for the steel yield strength. The building is 16 m by 12 m in plan (4 bays and 5 frames), and 11.2 m in elevation. Typical floor-to floor height is 2.8 m. More detailed description of the structure can be found in literature (Inel and Ozmen 2006).

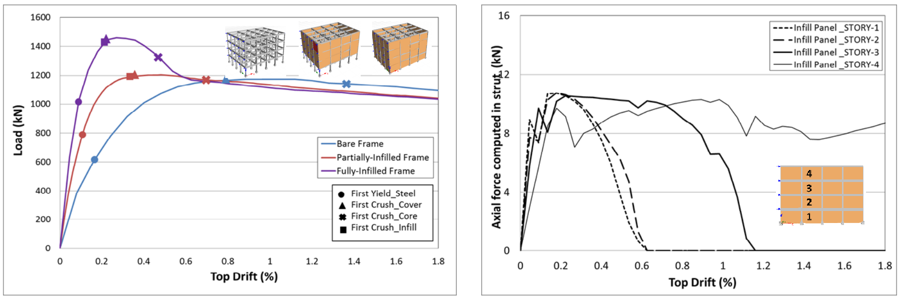


Figure 6. (a) Capacity curves for bare, partially infilled and fully infilled building, (b) behaviour of infill panel models at different floors.

3.1. Bare, Partially Infilled, and Fully Infilled Frames

The capacity curves obtained from the analyses, for the bare, partially infilled and fully infilled frames are shown in Fig. 6(a). The envelope curve model used is $\varepsilon_{ult} = 5.5\varepsilon_m$ ($\varepsilon_m = 0.0012$). For the three cases, the result did not evidence any shear failure. The behavior was dominated by flexure, even for the case of infilled building where, usually, the shear failure occurs at the ground floor.

On comparing the behaviour of three frames, it is clearly seen that the stiffness increased with the presence of infills. However, at small value of displacement (value of top drift 0.22%) a first crush of infill was observed for the fully infilled building. This first crush occurred for the infill panels located at ground floor and then second floor, as shown in Fig. 6(b).

On the other hand, the presence of infill panels have caused the occurrence of first crush of concrete members at earlier stage for fully infilled at top drift 0.45%, and then for partially infilled building at top drift 0.68%. However, for bare frame the first crush of concrete member occurred at top drift 1.36%. Hence, this observation shows how much the uncertainty can be significant in evaluation of seismic performance if infills are not considered in modelling.

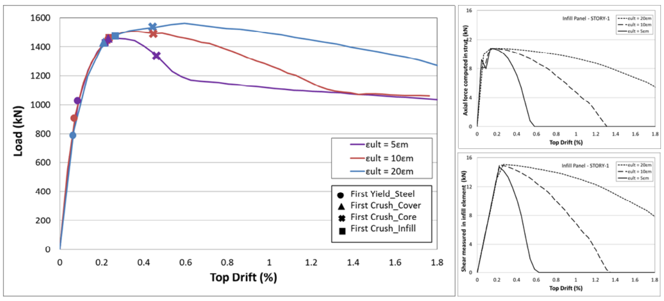


Figure 7 Influence of the shape of decrease of the compressive strength in infill panels on the capacity curves of fully infilled building.

Fig. 7 shows the influence of the $\varepsilon_{ult}/\varepsilon_m$ ratios on the capacity curves. It is clearly seen that in term of global performance of building, there is nearly no significant difference for different values of ratio. For the infill panel, the first crush occurred almost at the same value of top draft (0.22%). Similarly, the first crush on concrete members occurred almost at the same value of top drift (0.45%).

3.2. Influence of Masonry Strength

For Mediterranean RC buildings, the thickness of masonry infill may ranges between 120 mm to 200 mm, and the compressive strength may ranges between 1.0 MPa (weak) to 1.5 MPa (strong). In order to determine their effect on the behaviour of the building, three values of thickness (130 mm, 160 mm, and 190 mm) and three value of compressive strength were considered (1.0 MPa, 1.25 MPa, and 1.5

MPa). Fig. 8(a) shows the influence of thickness on the capacity curves. In term of damage limits, there is nearly negligible difference for the three cases. The first crush of infill panel occurred almost at the same value of top drift (0.22%), and similarly for concrete members, the observed first crush for the three cases was nearly at the same value of top drift (0.45%). The only difference observed was a small increase in peak load capacity.

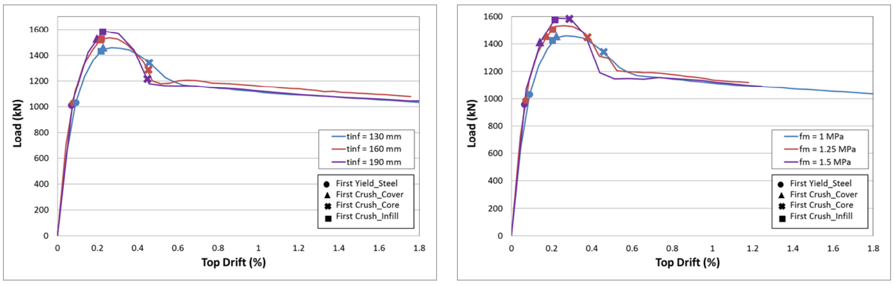


Figure 8. Influence of masonry strength on capacity curves: (a) influence of thickness of infill panel, (b) influence of compressive strength.

Fig. 8(b) shows results of capacity curves obtained for different values of compressive strength. Here also, results show a small increase in peak load, however, a clear difference (0.15% of top drift) can be observed concerning the first crush of concrete members between the three cases. The earliest first crush of concrete member occurred for building with higher compressive strength of infill panels (0.30% of top drift). For building with weak compressive strength, the first crush of concrete members occurred for top drift of (0.45%). However, for the three models of compressive strength nearly negligible difference was observed for first crush of infill panels.

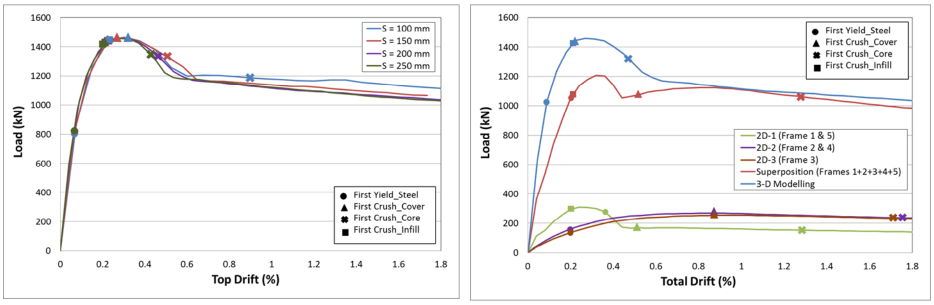


Figure 9. (a) Influence of the transvers reinforcement on the capacity curves. (b) Comparison of capacity curves obtained by 3-D model with those obtained by superposition of 2-D models

3.2. Influence of Transverse Reinforcement Spacing

For transverse reinforcement spacing in the potential plastic hinge regions, three layouts were considered: 100 mm, 150 mm, 200 mm, and 250 mm representing the ranges in typical construction, irrespective of code requirement. Fig. 9(a) shows the effect of transverse reinforcement on the behaviour of masonry infilled reinforced concrete building. The result did not evidence any shear failure in any of the analysed cases. Even in the case of a 250 mm transverse steel spacing, the behavior is dominated by flexure. It is worth to mention that shear failure of reinforced concrete still remains difficult to predict and not fully understood despite much experimental research and analysis. Taking into account the fact that the building is characterized by low concrete compressive strength (16MPa), no clear difference was observed for the capacity curves. In addition, nearly negligible variation of resistance against crush of concrete was observed between the cases s=150mm, 200mm, and 250mm. However, an increase in strength against crush of concrete was observed for s=100mm.

3.2. Superposition of 2-D models

A number of researchers have attempted to evaluate the capacity curve of buildings using 2-D models to reduce the calculation efforts. However, this simplified assumption may highly increase the uncertainty and, hence, there will be a question on the reliability of obtained results especially for infilled building. Fig. 9(b) shows a comparison of capacity curves obtained by 3-D with those obtained by superposition of 2-D models. It is clearly seen that there is a remarkable difference between the two procedures. By using 2-D models, the displacement corresponding to first crush of concrete member seems to be overestimated. The first crush is estimated to be at top drift of 1.28%. However, for 3-D model the first crush of concrete is estimated to be at top drift of 0.45%. In addition to that, the peak loading capacity is underestimated by using 2-D models.

4. BASIC PARAMETERS REQUIREMENT FOR RC BUILDINGS

As stated earlier, the performance of any used methodology for seismic assessment will certainly depend upon the sophistication of modelling structures and materials. Tables presented in this section (Tables 4.1, 4.2, 4.3, and 4.4) are results of investigation on the basic parameters requirement and their sensitivity for an accurate development of seismic capacity curve for masonry-infilled RC buildings. For modelling requirement, parameters are divided into three classes of relevance (Table 4.1): Essential, if no meaningful result can be obtained without it; Qualifying, if relevant to discriminate behaviour; and Desirable, for results refinement.

Table 4.1. Modelling requirement of parameters for RC buildings

Basic Elements		Modelling Requirement			Source of Information
		Essential	Qualifying	Desirable	
Structural and Non-Structural Elements	Frame Elements	X			design documentation, on site observation, literature reference, code reference
	infills		X		
	Diaphragm Elements	X			
	Roof	X	X		
	Claddings		X	X	
Modifications	Retrofitting		X	X	on site observation, literature reference
	Damage		X	X	

Tables 4.2, 4.3, and 4.4 show the sensitivity of results to configuration and dimensions, mechanical properties, and geometry characteristic parameters. The sensitivity is classified into three levels: High, Medium, and Low. In the literature, a number of researchers attempted to suggest default values for these parameters for future users. Accordingly, the same tables present also three type or category of data values of parameters that can be found in literature or programs: Typical, Average, and Range.

Table 4.2. Sensitivity of configuration and dimension parameters for RC buildings

Basic Parameters		Sensitivity Analysis			Data Values			Source of Information
		High	Medium	Low	Typical	Average	Range	
Building Configuration	Number of Stories	X					X	design documentation, on site observation, literature reference
	Number of Lines in X-Direction (Number of Bays)		X				X	
	Number of Lines in Y-Direction (Number of Frames)		X				X	
Building Dimensions	Story Heights	X			X			
	Spacing in X-Direction (Bay Lengths)		X			X		
	Spacing in Y-Direction (Frame)		X			X		

Table 4.3. Sensitivity of mechanical property parameters for RC buildings

Basic Parameters		Sensitivity Analysis			Data Values			Source of Information
		High	Medium	Low	Typical	Average	Range	
Reinforcing Bar	- Modulus of elasticity	X			X			design documentation, literature reference, code reference
	- Yield stress	X			X			
	- Ultimate stress	X			X			
Masonry Infills	- Compression strength	X					X	
	- Modulus of elasticity	X			X		X	
	- Poisson coefficient			X	X			
	- Shear strength		X				X	
	- Specific weight		X		X			
Concrete	- Compression strength	X					X	
	- Modulus of elasticity	X			X			
	- Poisson coefficient			X	X			
	- Shear strength		X				X	
	- Tensile strength	X					X	
	- Specific weight		X		X			

Table 4.4. Sensitivity of geometry characteristic parameters for RC buildings

Basic Parameters	Sensitivity Analysis			Data Values			Source of Information
	High	Medium	Low	Typical	Average	Range	
Cross-sections:							design documentation, literature reference, code reference
- Dimensions		X			X		
- Transversal Reinforcement		X		X			
- Longitudinal Reinforcement		X				X	
Plastic hinge properties of elements	X						

5. CONCLUSIONS

This paper examined the various modelling techniques to identify their strengths in relation to the accuracy of seismic capacity curves prediction and their applicability for seismic performance evaluation. The case of reinforced concrete structural system is considered with the inclusion in the analyses of infill panels, in order to determine the limits and differences of these modelling techniques. The outcome showed that masonry-infill panel can play a predominant role on the accuracy of seismic performance prediction. An investigation was conducted for an old Turkish RC building with low compressive strength of concrete. Hence, more investigation should be conducted for building with high compressive strength in aim to analyse the sensitivity of infill-frame system on seismic capacity curves prediction. This study highlighted also the difficulties that might be encountered to predict shear failure which is still not fully understood despite much experimental research and analysis.

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