16th World Conference on Earthquake, 16WCEE 2017

Santiago Chile, January 9th to 13th 2017

Paper N° 4917

Registration Code: S-K1464721179

NUMERICAL SIMULATION OF RC FRAME STRUCTURES WITH INFILL WALLS UNDER CONSIDERATION OF OUT-OF- PLANE BEHAVIOR

M. Hisham AL Hanoun[(1)], L. Abrahamczyk[(2)], J. Schwarz[(3)]

[(1)] *PhD Student, GRK 1462, Bauhaus-University Weimar, Germany, Eng.hisham2007@gmail.com*
[(2)] *Researcher & CEO, GRK 1462, Bauhaus-University Weimar, Germany, lars.abrahamczyk@uni-weimar.de*
[(3)] *Head, Earthquake Damage Analysis Center, Bauhaus-University Weimar, Germany, schwarz@uni-weimar.de*

Abstract

In case of an earthquake a typical unreinforced masonry (URM) infill wall is subjected to a three dimensional acceleration field and undergoes simultaneous in-plane and out-of-plane loading. Depending on the direction of seismic action, the observed damage mechanisms on URM may be classified as in-plane (IP) and out-of-plane (OoP). In-plane damage is caused mainly by inter-story drift. The typical in-plane damage mechanisms can be classified as cracking due to separation from the structural frame, cracking due to horizontal bed joint sliding, cracking due to tension across the diagonals of the panel, and cracking due to crushing of panel corners.

Out-of-plane over turning collapse is the most dominated mechanism in perpendicular wall direction. The overturning effects are increased by the damage due to the horizontal in-plane components of seismic action (shear cracking) and the inter-story drift in out-of-plane direction, which causes the separation from the upper beam.

After L'Aquila earthquake in 2009, a conducted damage survey showed that the greatest damage is located on the lower stories (up to the second story in taller buildings). On the contrary, collapse due to out-of-plane mechanisms is expected on the upper stories of buildings (due to higher expected accelerations). Therefore, the collapse due to out-of-plane mechanisms has to be ascribed to the early presence of heavy in-plane induced cracking. In fact, in-plane actions can cause disconnection of the infill panels from structural elements, reducing their seismic capacity.

Since the mid-1950s, a number of distinct approaches in the field of analysis of in-filled frames lead to different analytical models. The equivalent strut model is the most common one. A main disadvantage of the strut models is the disability to represent the out-of-plane response of an infill masonry panel. A new model was been proposed, which consists of diagonal beam-column members utilizing fiber element cross sections. It is suitable for nonlinear time history analysis. The model considers both the in-plane and out-of-plane response of the infill, as well as the interaction between IP and OoP capacities. Since the building stock is often mostly composed of reinforced concrete frame buildings with URM, collapse simulation of buildings under seismic loads becomes an important issue. In this paper, the failure mechanisms of the URM will be numerically simulated by using the proposed state of the art strut model. A set of reinforced concrete frame with URM walls with different number of stories will be investigated to examine typical damage pattern and to quantify the damage for RC frames representing different story classes. Finally the achieved results will be used to come up with a proposal for the description of damage to primary and secondary structural elements in multistory and high-rise buildings. Results can be implemented into the EMS-98 or its up-date to an International Macro seismic Scale (IMS).

Keywords: In-plane; Out-of-plane; Damage mechanism, masonry infill walls.

1. Introduction

From a survey on damaged and collapsed reinforced concrete (RC) buildings in recent earthquakes, a large number of buildings that suffered severe damage or collapse had their poor performance associated with the influence of URM infill walls. The observed damage mechanisms of URM infill walls, may be classified as in-plane (IP) and out-of-plane (OoP).

For low to mid-rise URM in-filled RC frames, ground story infill walls are expected to be damaged first since they are subjected to highest IP demands. However, under the effect of bidirectional loading, where the two components of a ground motion are equally significant, infill walls of the upper stories may fail under the combination of IP and OoP effects. The magnitude of IP demands reduces at the upper stories, while that of OoP forces increases due to the increase of accelerations, subjecting the upper story infill walls to failure under the effects of IP and OoP interaction. Fig. 1 shows lower story infill wall failures of buildings during 1999 Düzci, Turkey earthquake [11].

For the assessment of in-filled RC frame structures, the nonlinear behavior induced by earthquake demands should be taken into account [3, 6]. Different techniques are available in the literature for the simulation of the response of in-filled frames, from refined micro-models to simplified macro-models [5]. For the non-linear analysis of complex structures, when subjected to earthquakes, in many cases it is not suitable to adopt refined models. Thus, for the response simulation of in-filled frame structures, considering URM walls and its interaction with the surrounding frame elements, the adoption of simplified models is unavoidable. In the present paper a simplified macro-model which accounts for the in-plane and out-of-plane behavior of URM walls, developed by [12] and implemented in the computer software OpenSees [14] is used. This numerical model takes also into account an element removal algorithm which allows removing elements during an earthquake response simulation.

In purpose to demonstrate the effect of considering the OoP failure mode, three different building were considered in this study, four, seven and ten story. The building was subjected to bidirectional non-linear dynamic analysis and the results are compared for the three different models.

2. Simplified URM wall macro-model elaboration

Macro-modeling strategy for infill masonry frame implements a single global structural member, composed most often of equivalent diagonal struts instead of masonry panel. *Stafford et al., 1961* [24] modeled infill panel using an equivalent pin-joint strut. Based on experimental data from a large series of tests on masonry infill frames, *Crisafulli et al., 2007* [5] developed a macro model based on a multi-strut formulation. A 4-node panel element which is connected to the frame at beam-column joints was implemented to take into account compressive and shear behavior of masonry separately. A main disadvantage of the strut models is the disability to represent the out-of-plane response of an infill masonry panel.

a) … b) … c) …

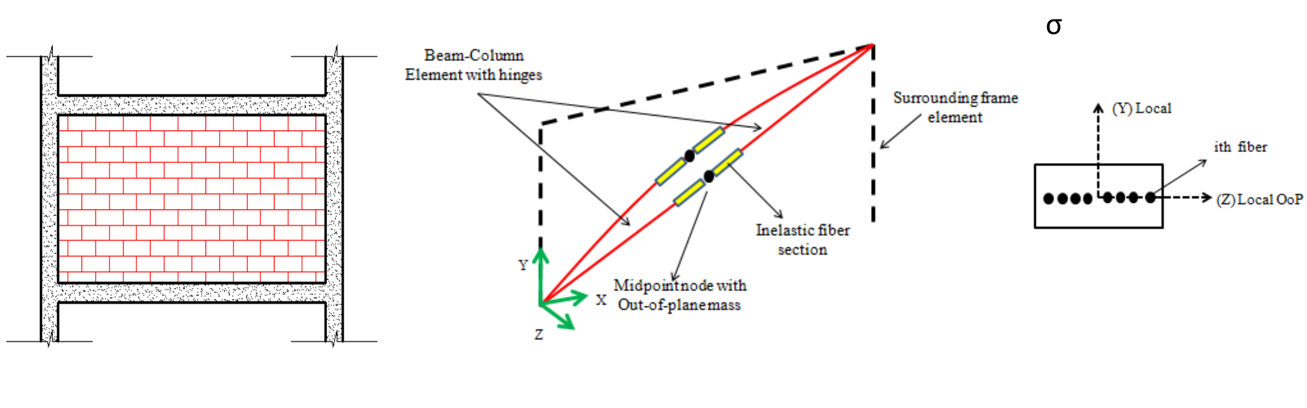
Fig. 1 – Different levels of damage of different height reinforced concrete frame buildings with URM infill walls [11]

Rodrigues et al., 2010 [25] have developed an equivalent bi-diagonal compression strut model to evaluate the behavior of masonry infill walls which were subjected to cyclic loads. As damage on panel in one direction affects its behavior in the other direction, the interaction between frame and masonry infill in two directions was considered in the proposed model. Utilizing the element removal algorithm which was principally developed by [12], this model recently improved to consider in plane and out of plane behavior.

In this study a simplified macro model is used, which was initially developed in [12]. The analytical model considers the IP and OoP behavior and the interaction between them in both directions. In the model, each infill wall is represented by a single diagonal strut, composed of two force-based beam-column elements represented as "beamWithHinges" in OpenSees [14] and connected at a midpoint node with an assigned lumped mass in the OoP direction, Figure 2(b). The cross-section of the beam-column element is modeled by strategically locating nonlinear fibers along a line in the OoP direction, Figure 2(c). In this way, the beam-column element acts as truss and flexural elements in the IP and OoP directions, respectively. The OoP mass and stiffness are calculated such that the model has the same natural frequency as the infill wall.

3. URM failure through element removal algorithm

In order to explicitly account for the failure of URM infill walls during an earthquake excitation under combined IP and OoP effects, the analytical infill wall model described above is implemented for use in a previously developed progressive collapse algorithm [14]. In that regard, the IP displacement is the relative horizontal displacement between the top and bottom nodes of the diagonal in Figure 2(b). On the other hand, the OoP displacement is that of the midpoint node where the lumped OoP mass is attached with respect to the chord connecting the top and bottom nodes. The interaction between the IP and OoP drift can be defined to follow an elliptical interaction, as can be observed in Figure 3, and the limits can be selected based on [7, 8]. In the literature only a small number of biaxial experimental tests of URM infill walls can be found and further investigations should be performed to quantify this interaction, and to define the displacement interaction law limits. When the pair of IP and OoP displacements from the analysis reaches or exceeds the envelope curve, e.g. Figure 3, the two beam-column elements and the midpoint node, representing the infill wall, are removed to directly represent the failure of the URM infill wall. The algorithm for the removal of an infill wall is shown in Figure 3.



(a) (b) (c)

Fig. 2 – Macro-model for unreinforced masonry infill walls

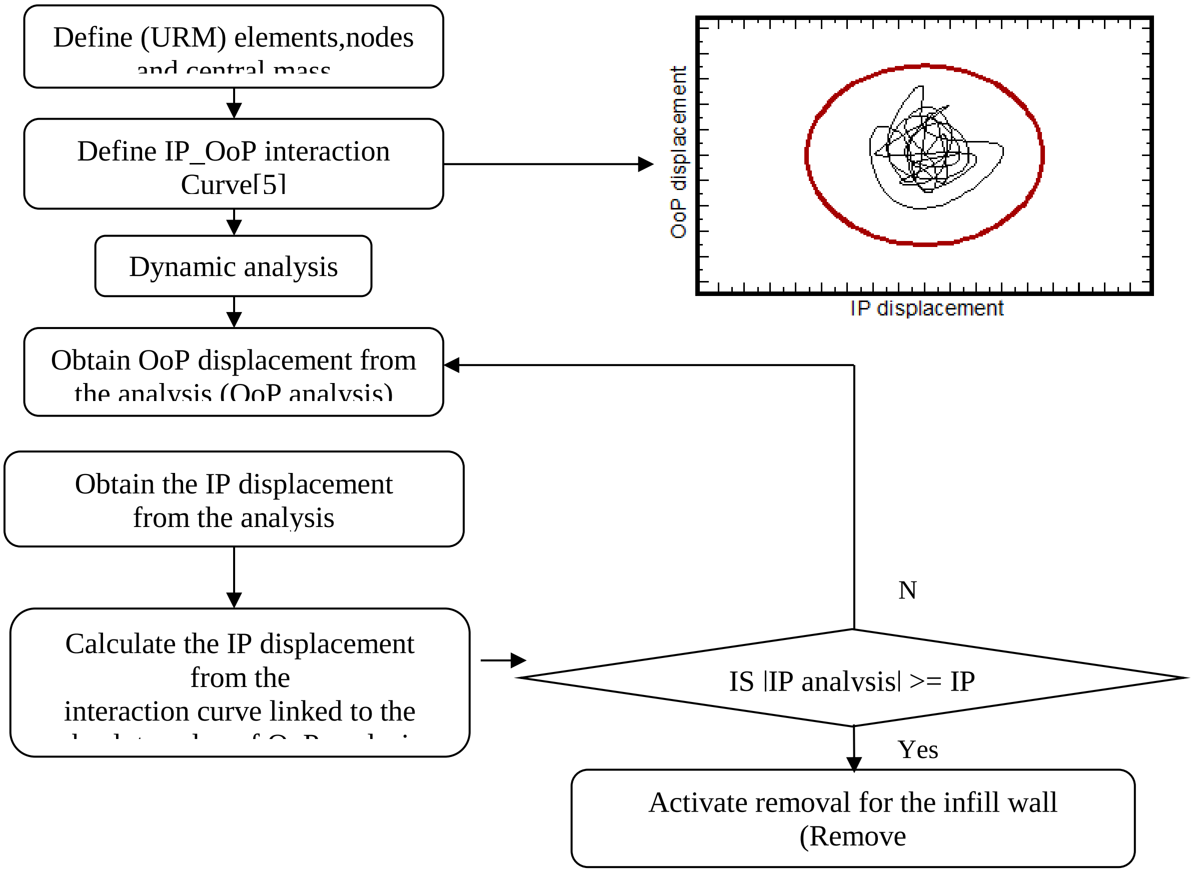


Fig. 3 – Algorithm for the URM infill wall removal as implemented in [14]

4. The effect of URM out-of-plane response in the structural behavior of RC buildings

4.1 General Description

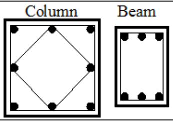
The influence of the URM infill walls, with the consideration of the out-of-plane behavior in the structural response of RC multi-story buildings subjected to seismic action was studied taking as an example three different height buildings, four, seven and ten story. The building has in-plan dimension of 15x10m² arranged in 5x5 m² modules, with 3.5m storey height, two configurations of the building: (a) bare frame without infill walls and (b) URM infill walls placed in the bays along the longitudinal building perimet as shown in Figure 4. The preliminary design was carried out in accordance with the rules of Eurocode 2 and 8, assuming typical loads (additional dead load 2.0 kN/m², to represent floor finishing and partitions, and live load 2.0 kN/m²), and high seismicity.

4.2 Numerical modelling strategies

The computer program OpenSees [14] is used, adopting a forced based beam column element with distributed plasticity model. Three different models for each structure have been created, namely, bare, IP and IP_OoP model, accordingly nine numerical models were created. The consideration of non-linear material behavior in the prediction of the RC structures response requires accurate modeling of the uniaxial material stress-strain cyclic response, as illustrated in the next paragraph.

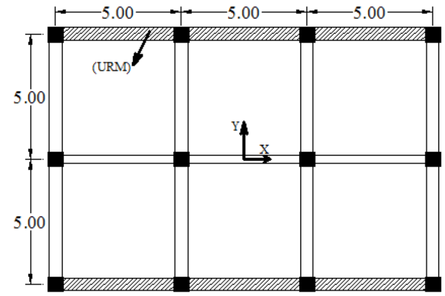
4.2.1. Concrete model

The monotonic concrete model, shown in Figure 5a which follows the constitutive law proposed by [15] was used. The cyclic rules included in the model for the confined concrete were proposed by [17]. The effect of transverse reinforcement considered according to the rule proposed by [16], wherby transverse reinforcement introduce a constant confinment pressure throughout the whole stress strain range.



Section	No.Story	Column	Beam	Longitudinal reinforcement		Transverse stirups	
	4	450*450	250*300	6T18	6T12	T8/100mm	T6/150mm
	7	500*500	250*450	8T20	6T14	T8/100mm	T6/150mm
	10	600*600	250*450	8T25	6T14	T8/100mm	T6/150mm

a) Geometric details and reinforcement



b) Typical ground plan

Fig. 4 – Structural details of the considered case study building

4.2.2. Reinforcement steel model

The relatively simplified uniaxial model proposed by [18] shown in Figure 5b, linked with the hysteretic rules proposed by [19], is used for the representation of steel reinforcement in these analyses.

4.2.3. Unreinforced masonry infill wall model

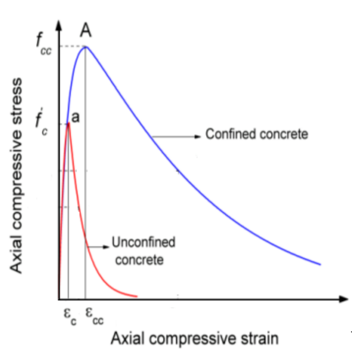
For the infill wall case, 150 mm thick walls are employed with modulus of elasticity 3.9 GPa, compressive strength 7.0 MPa, and shear strength 1.56 MPa. The IP and OoP drift limits adopted for the IP_OoP model were set in order to follow an elliptical interaction between them [5], with a maximum IP drift of 1.5% and OoP drift of 3%. These values were selected based on [11].

4.3. Natural frequencies of the numerical models

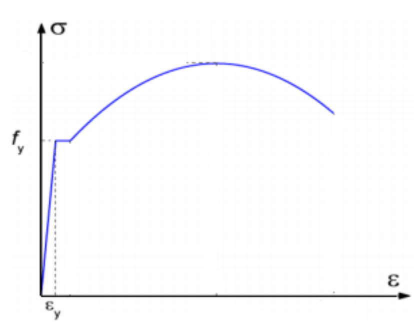
In purpose to evaluate the effect of the URM infill walls in the structural response, the natural periods for the numerical models were calculated and included in Table 2. It was observed that the natural periods are about (1.5 – 2) higher for the numerical models with URM infill walls in the x-direction and this were confirmed in [24].

Table 1 – Material mechanical parameters for the numerical model

Material	**Modulus of Elasticity Ec (GPa)**	**Compressive Strength fc (MPa)**	**Tensile Strength ft (MPa)**	**Strain at Peak strength εc (%)**	**Strain Hardening Parameter r (%)**	**Transition curve Initial shape (R_0)**	**Transition Curve shape R_1**	**R_2**
Concrete	23.5	27.5	1	0.2	------	------	------	
Steel	200	450	450	------	1	18	0.925	0.15
Masonry	3.9	7	0.2	0.12	------	------	------	



a)



b)

Fig. 5- Monotonic stress strain relation of (a) Concrete (b) Steel

Table 2 – Natural periods for the studied numerical models.

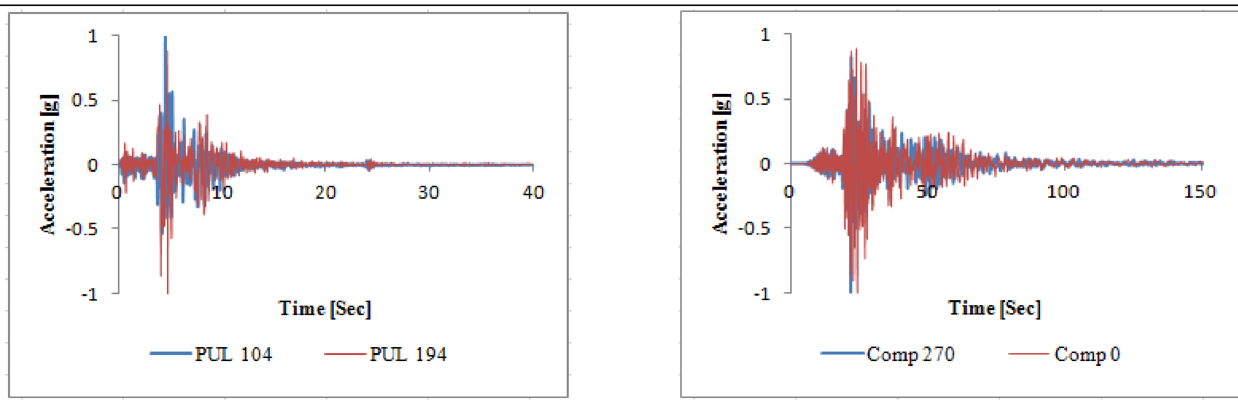
No. of Stories		**4**		**7**		**10**	
mode		**Y**	**X**	**Y**	**X**	**Y**	**X**
Periods [Sec]	Model 1 (Bare)	0.78	0.76	1.46	1.4	1.8	2.1
	Model 2 (IP)	0.82	0.39	1.54	0.68	1.82	1
	Model 3 (IP_OoP)	0.82	0.39	1.54	0.68	1.82	1

5. Dynamic analysis

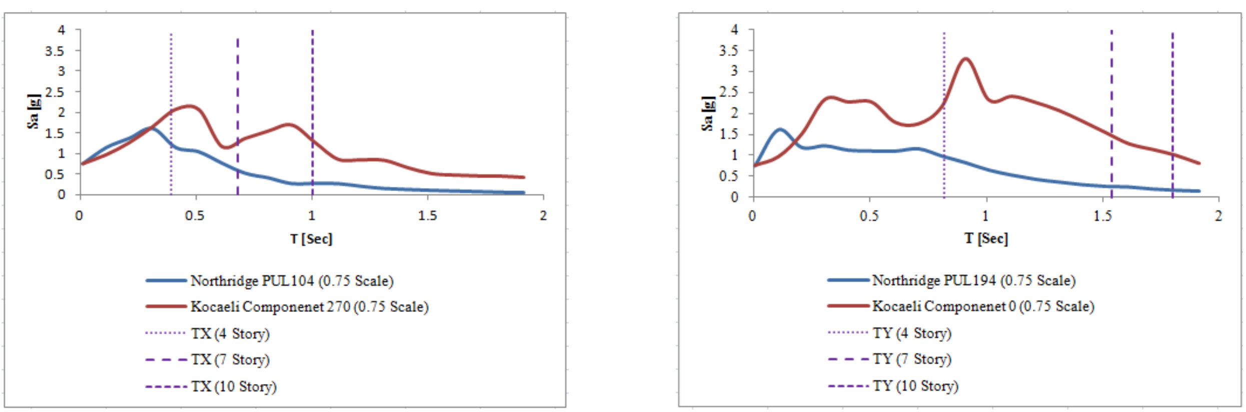
5.1. Seismic action

For studying dynamic response, as shown in Fig.6, two horizontal components of the ground motion recorded in Pacoima Dam Station during 1994 Northridge earthquake are used for the analyses of the three models of each building. The ground motion is scaled to three different level of intensity namely (0.75g, 1.5g and 1.75g). Time history analysis with the time step length of 0.02sec is used. Rayleigh's damping with the damping coefficient 2% of the critical is used. First two modes of vibration are used for calculating mass and stiffness matrix multipliers.

Northridge EQ | Kocaeli EQ



a) Time history

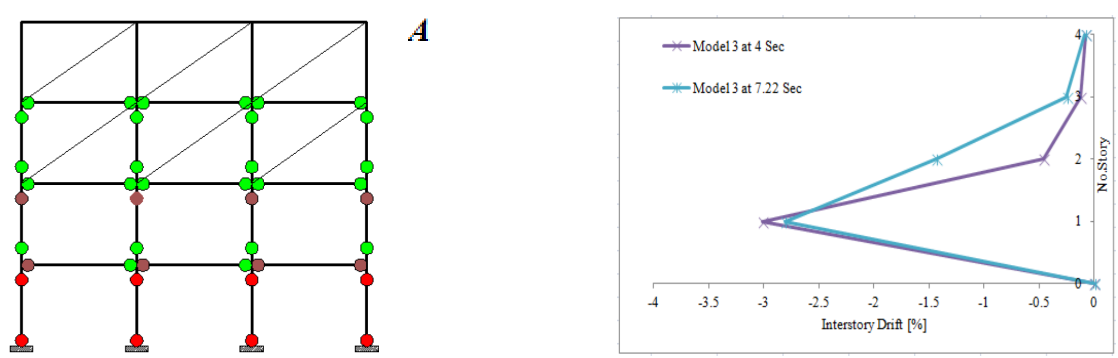


b) Normalized response spectrum

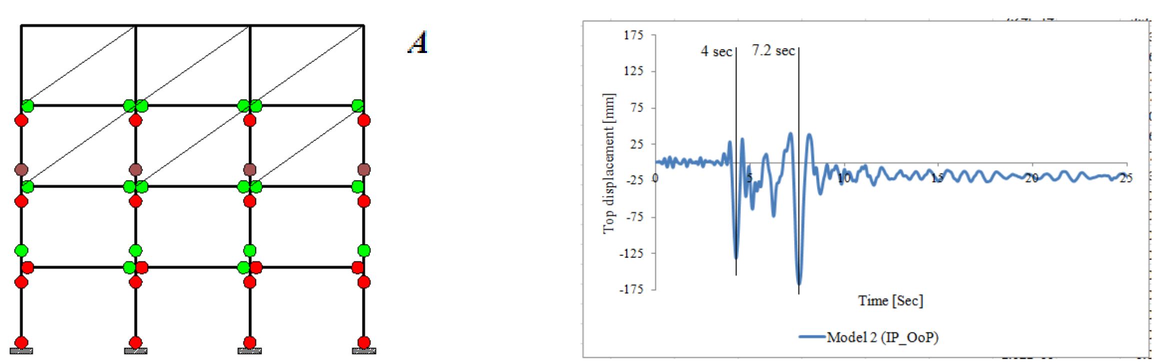
Fig. 6 – Considered seismic action: Northridge and Kocaeli EQ

Table 3 – Definition and Description of Local Damage Grades according to [11]

Element	Damage Description	Material Strains	Local Damage Grade [LDG]
Structural (primary Elements)	Maximum tension strain of concrete	$\varepsilon \geq +0.0001$	LDG_p 1
	Yield strain of reinforcement steel	$0.002 \leq \varepsilon_s \leq 0.005$	LDG_p 2
	Spalling of concrete cover	$\varepsilon_{uc} \geq -0.001$	LDG_p 3
	Moderate damage of longitudinal steel bars	$0.005 \leq \varepsilon_s \leq 0.01$	
	Strength degradation of core concrete	$-0.002 \leq \varepsilon_{cc} \leq -0.004$	LDG_p 4
	Substantial damage of longitudinal steel bars (steel failure)	$0.01 \leq \varepsilon_s \leq 0.02$	LDG_p 5a
	Ultimate strain of confined core concrete (transverse rebar failure)	$\varepsilon_{cc} \geq -0.004$	LDG_p 5b
Non-Structural (Secondary) Elements	Crack. diagonal compression strain of masonry	$\varepsilon_m = -0.0012$	LDGs 1
	Strength degradation of masonry	$-0.0012 \leq \varepsilon_m \leq -0.0024$	LDGs 2
	Ultimate strain of diagonal compression	$\varepsilon_m \geq -0.0024$	LDGs 3a
	Collapse due to IP_OoP interaction	See Fig.3	LDGs 3b



a) at step (4 Sec)



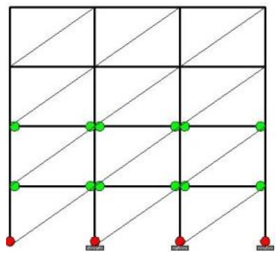
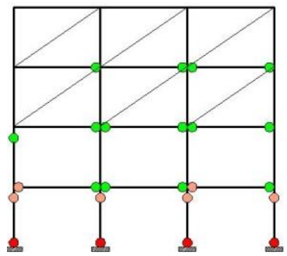
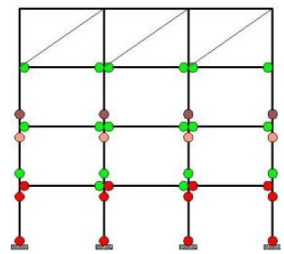
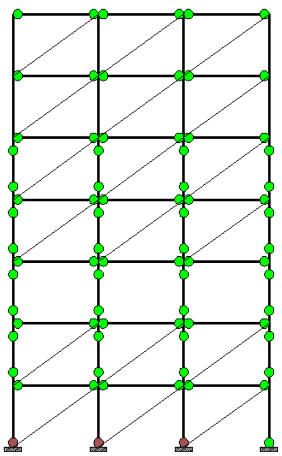
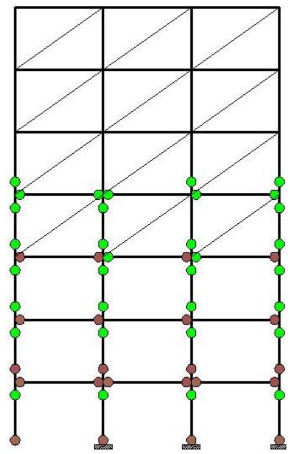
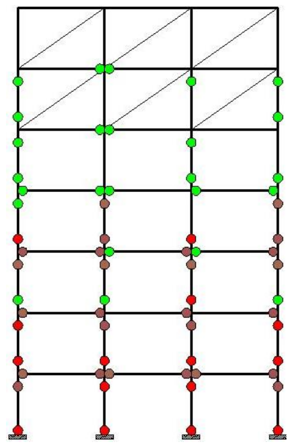
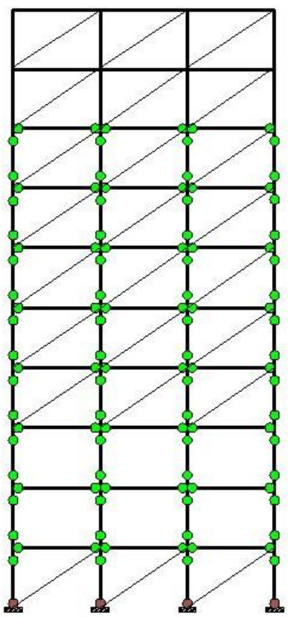
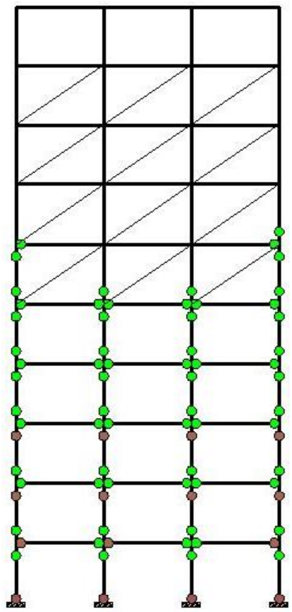
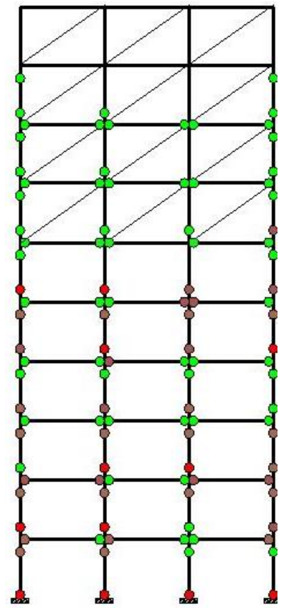
b) at step (7.22) Sec

Fig. 7 – Damage prognosis, inter story drift and displacement response at point A of four story numerical Model 3 (IP_OoP) in the longitudinal direction.

5.2. Damage description

Models are analyzed by performing nonlinear dynamic analysis. For comparative study, damage limit states are defined and are related with the observed material strains as stated in [6]. In Table 3, description for different damage states and the related material strains are included.

Table 4 – Damage degree of four, seven and ten story model 3 IP_OoP

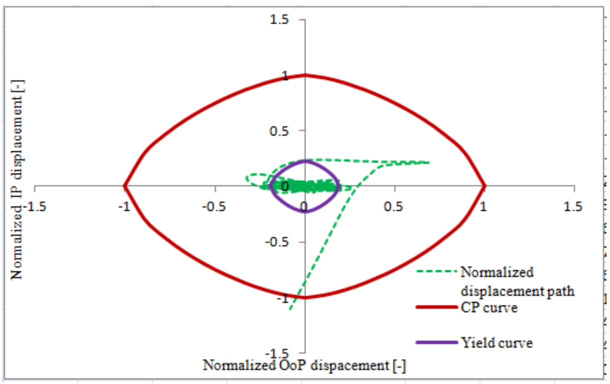
Northridge EQ		Kocaeli EQ	
0.4g	0.75g	0.4g	0.75g
No damage			
No damage			
No damage			
● LDG$_p$ 1 ● LDG$_p$ 3 ● LDG$_p$ 4		◩ No Collapse ☐ Collapse	

5.3. Damage prognosis

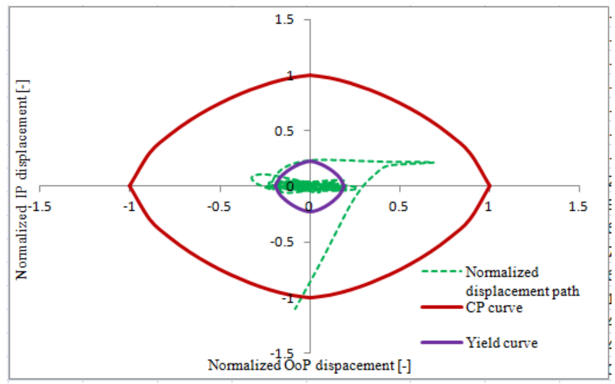
Damage degree for model 3 is marked and shown in Table 4. Four story models suffered substantial to severe damage at the first story columns bases and light to moderate damage at the top of the first and second story columns in low excitation namely (0.75 g) and no collapse in URM observed. Increasing the excitation level lead to more damage in columns and beams specifically in the first and second story and the URM wall were collapsed due to IP_OoP interaction. In the case of seven and ten story model collapse for some URM walls was observed starting from low excitation level.

Table 5 – IP_OoP normalized displacement and interaction curves development

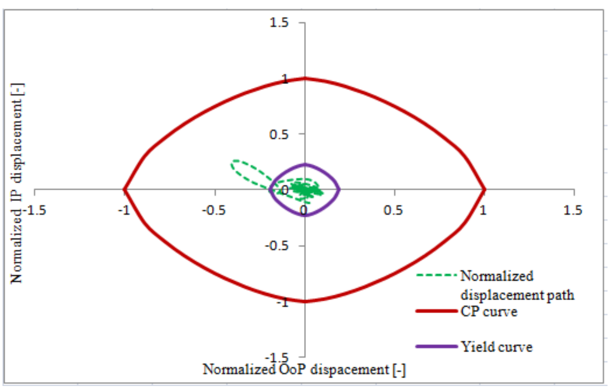
At Step 4 Sec (first peak displacement)



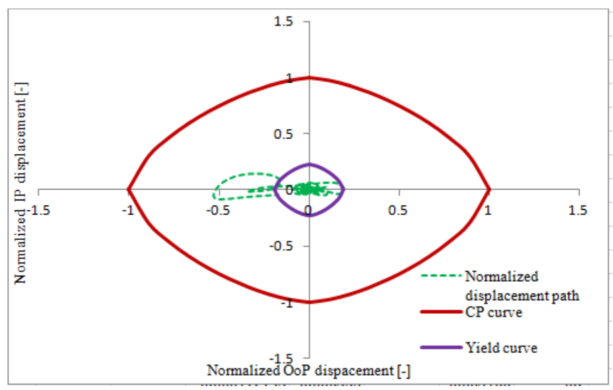
1st Floor



2nd Floor

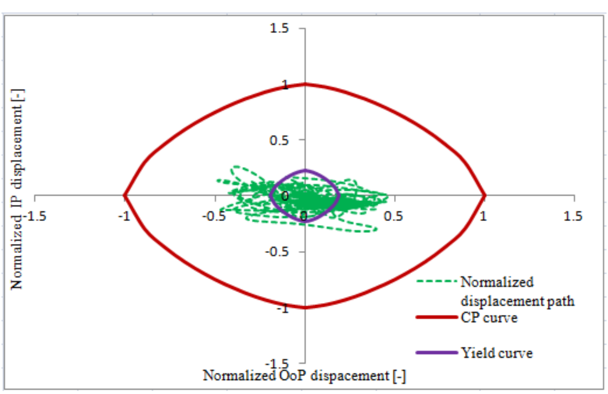


3rd Floor

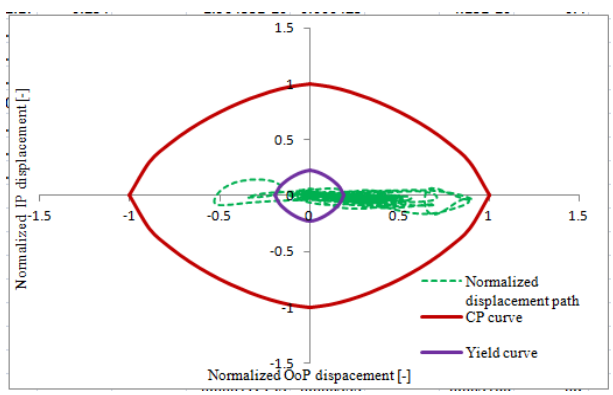


4th Floor

At Step 7.22 Sec (peak displacement)



3rd Floor



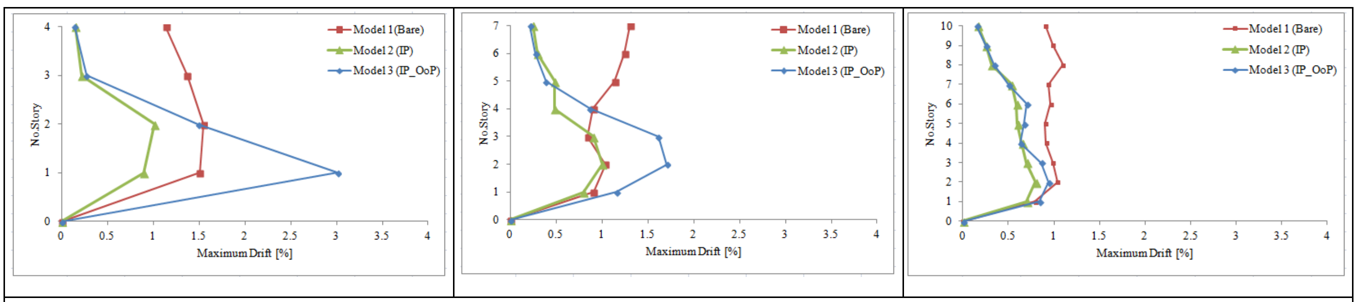
4th Floor

5.4. Interaction curves

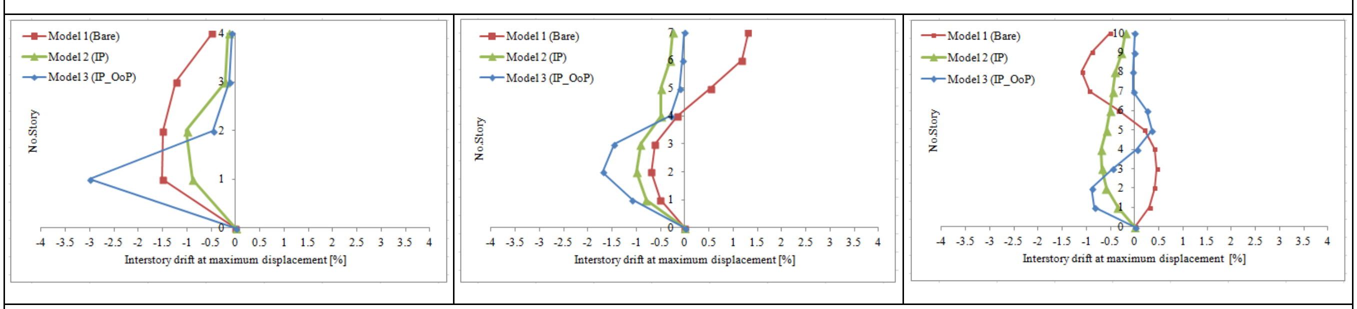
Table 5, shows the deformation path of in-plane and out-of-plane displacement, normalized by their respective unidirectional CP values, and plotted on the same plane. These results are obtained from the time history analysis of ground motion scaled to 1.75g related to the case of four story building. The yield and the CP limit state boundaries are also included. Since the deformation path crosses the CP boundary in the first and second story, then the URM walls were collapsed, as illustrated in Table 5, while the deformation path in the third and fourth story doesn't cross the CP boundary, which indicate no collapse occurred in the URM infill walls.

5.5. Interstory Drift Response

In this study maximum interstory drift response of model 1, 2 and 3 is also compared for the ground motion scaled with 1.75. The observed maximum interstory drifts are drawn in Table 6. Four story infille showed considerably more drift magnitudes at the first story as compared with bare. Because the URM walls were collapsed and removed from mdel 3 and the quantity of damage were the same for both URM infilled model and bare frame. The second story drift is the same for both models, third and fourth story drift were less for model 3 since the URM wall were not collapsed. Seven story frames indicates bigger drift at the second and third story for model 2, and the drift at the fourth story for the same reasons observed in the four story models. However, relatively more magnitude is observed in the above stories of model 1. Because of the absence of the URM infill walls. Maximum interstory drifts for ten story bare model shown in Table 6 are relatively more, since the quantity of damage for colomns and beams is bigger, However, larger drift observed starting from the seventh story, because URM walls in model 3 are not collapsed and the impose omre stiffness in the upper stories.



a) Maximum Interstory Drift



b) Interstory Drift at time of maximum displacement

Fig. 8 – Interstory drift.

5.5. Effect of considering IP and OoP interaction

Since no experimental data to discribe the mutual behavior between the IP-OoP of the URM infill walls and the rusulting possible collapse of URM infill, in this study the observed damage of RC building with infill from previous earthquakes was used as a basic refrence to compare the simulated damage in multi story building whith the actual observed damage in the past event. In the simulation process a simplified URM infill wall macro model was used, The analytical model consider the IP_OoP behavior and the interaction between them in both directions and the possible collapse of the URM infill through element removal algorithm[12]. Three RC buildings with the different geometric namely in height and same mechanical properties were modeled numerically and subjected to dynamic time-history analysis and three different cases was considered for each model: bare model; model with URM infill walls considering only IP behavior and with URM infill wall considering IP and OoP behavior. The simulation results show that consideration of IP_OoP interaction and infill wall removal capabilities are essential for simulating the possible soft story formation in upper stories, it was also observed that considering The IP and OoP interacion leads to the collapse of most URM infill walls in seven and ten story building at the heighst scale of the ground motion. The results of this study stress the neccicity of considering the IP and OoP interaction and the resultant possible collapse of URM infill walls during the design process of the new multi story RC structures.

6. Conclusion

To determine the typical damage patterns of URM infill walls in multi story RC building frames, three different height building, namely four, seven and ten story were anlyised by using nonlinear dynamic analysis and explicitly considering out of plane collapse through element removal algorithm. Northridge record was used and scaled into three different level of intensity, in the case of four story building and under low excitation no collapse was observed for URM wall, and for (1.5, 1.75) scale the first tow story URM walls collapsed and this was observed in several previous event as stated in[6]. For seven and ten story building, collapse of the URM walls start from the low excitation level, for both buildings collapse observed in the first story due to large drift and upper story due to high acceleration. The result of damage prognosis for different limit states were linked to the Local Damage Grade used in EMS-98. It should be emphasized that, This study will be extended to consider more time history records with different frequency contents as a part of larger study in the field of quality evaluation of URM infill walls numerical macro models due to cyclic loading.

7. Acknowledgements

This research is supported by the German Research Foundation(DFG) via Research Training Group “Evaluation of Coupled Numerical and Experimental Partial Models in Structural Engineering (GRK 1462)”, which is gratefully acknowledged by the authors.

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