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Model quality assessment for RC frames with URM infill walls subjected to horizontal cyclic loading

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ABSTRACT

Any engineering judgement of the functionality, safety, reliability and robustness of structures requires forecasting models, which in general have (often unknown) limits of their applicability and a somehow uncertain quality concerning their evidence and predicted quantities. In particular, this is true in cases of dynamic action reaching extreme intensity levels to cause non-linear response. The paper focusses on RC frame with unreinforced masonry (URM) infills. Since the turn of the millennium, a number of distinct experimental studies led to a number of numerical macromodels with different complexity able to represent the behavior of the investigated primary and secondary structural systems addressing the in-plane (IP) response of the RC frame with URM infill walls. Concentrated plasticity models like lumped plastic hinges and fiber-based modeling approaches are often used to simulate the behavior of the RC frame elements. Besides, the concept of equivalent diagonal struts is usually utilized to represent the effect of full URM infill walls. The nonlinear cyclic behavior of the numerical model is described by means of a strength envelope curve linked with a distinct hysteresis law. Several well-documented experimental tests are chosen for the evaluation of the simulation results obtained by utilizing a set of different macromodels. They represent the proposed modeling techniques of RC frame elements and URM infill walls as well as different hysteretic models. By comparing the experimental and numerical results the model quality has been assessed with respect to deformation and stiffness properties as well as application of error measures. A tested four-story RC frame structure with URM infill walls, considering the IP response only, will be used to re-assess the failure mechanism of the structure through the use of the proposed simplified numerical model. Finally, a simple and reliable model can be recommended to be further used.

Keywords: Model quality, RC frames, URM infill walls, macromodelling.

INTRODUCTION

Past and recent earthquakes always indicated that the performance of reinforced concrete (RC) frame structures is strongly associated with the presence of unreinforced masonry (URM) infill walls (e.g. Bingöl 2003, L'Aquila 2009, Chile 2010, etc.). Additionally, various damage observations have shown that the quality and the material of the infill walls have a strong impact on the interaction with the structural frame (primary system) and can result in different damage grades independent of the story class [1, 2]. Thus, the masonry infill walls with and without opening typically enhances the strength, stiffness and energy dissipation capacity of the RC frame structural system [3]. Hence, it may lead to unexpected distribution of horizontal forces and induce local damage at columns and beams when the structure exposed to seismic loads. Therefore, the primary (frame) and secondary (infill) structural system have to be considered in part of the assessment of RC frames with URM infill walls. Various techniques are available to simulate the response of RC frames with infill walls, ranging from micromodels to simplified macromodels [4]. Since micromodels are computationally time-consuming, the adoption of simplified macromodel are desirable.

In this paper a set of different macromodels for the primary and secondary structural system are utilized to predict the response of documented experimental tests. Afterwards, the achieved results for the different model sets are compared with the experimental results to assess its model quality with respect to deformation and stiffness properties as well as application of error measures.

MODELING ASPECTS FOR PRIMARY AND SECONDARY STRUCTURAL ELEMENTS

Reinforced concrete frame elements

Up to date, numerous modeling strategies have been proposed to simulate the structural behavior of reinforced concrete (RC) frame structures with unreinforced masonry (URM) walls. However, the complex nonlinear behavior of RC structures still lead to large discrepancies among these models. This is especially the case, when the frame is subjected to random load reversals causing inelastic response up to the near collapse or even collapse of the structural system. As a result of that, there is still a need for more reliable and practical numerical tools. The level of sophistication in each model corresponds to the level or degree of discretization. To this extent, [5] roughly divide modeling strategies into three categories in accordance with the increasing level of refinement and complexity:

a) Lumped (global) models: Lumped models were firstly developed by centralizing all inelasticity of the elements into critical regions (such as column ends, beam-column joints as well as locations near mid-span) as it was widely observed in part of experimental studies and field observations. Additionally, any kind of hysteretic law can be assigned to the lumped plasticity model to consider effectively the nonlinear cyclic behavior as well as stiffness deterioration due to cracking. In the definition of rules for loading, unloading and reloading, a large number of choices are possible, to include or to neglect phenomena such as the effect of shear, interaction between moment and axial load, biaxial bending and slippage of rebars [6].

b) Fiber models: These models are a subcategory of the global models. Inelastic behavior is either limited to the member end (lumped plasticity models), or distributed along the member (spread plasticity models). The spread of inelastic deformations into the member accounts for a better description of the element inelastic behavior. The first distributed model was introduced by [7]. The first element with distributed nonlinearity was formulated with displacement method. Similar to displacement-based elements, reinforced concrete element can be subdivided into longitudinal fibers represented by uniaxial constitutive relationships of concrete and reinforcing steel.

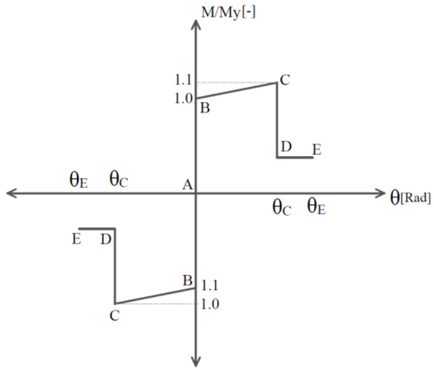
c) Finite element models: FE models represent a detailed micromodelling solution in which material constitutive relationships are assigned to each element. It is on one hand obvious that this latter approach represents the most detailed way to represent the elements; on the other hand it is computationally demanding, and it asks for the definition of numerous input parameters that, in turn, need to be calibrated.

The lumped plastic hinges and fiber-based modelling technique are the two most common approaches usually adopted in nonlinear analysis of frame structures and therefore applied in this study. In the first considered model, concentrated rigid plastic hinges are assigned at beam and column end sections. For beams localized moment plastic hinges are used (Figure 1a). Whereas, to account for the axial-force/biaxial-moment interactions, axial-force/biaxial-moment plastic hinges are assigned to the columns (Figure 1b). The model developed by [8] is used to define the monotonic moment-rotation curve. The model requires five parameters: elastic stiffness (K_e), yield strength (M_y), the maximum to yielding moment percentage (M_c/M_y), post yielding rotation (Θ_{cap}) and post-capping rotation (Θ_{pc}). All the parameters are determinable using the predictive equations developed by [9].

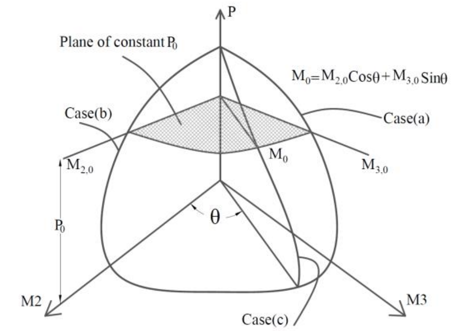
Unreinforced masonry infill walls

In case of nonlinear cyclic loading, the use of equivalent diagonal struts as a macromodel to represent the secondary structural (URM infill walls) elements requires a convenient hysteretic model which simulates the cyclic behavior of the URM infill panel. Due to the fact that the cyclic response of the masonry infill walls is not easy to be captured, hence, up to date several hysteretic models have been proposed with different complex rules [10].

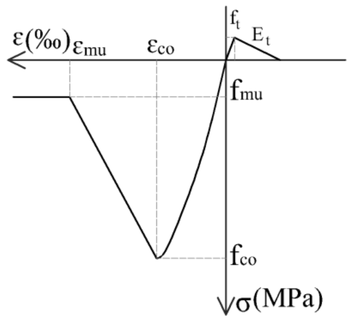
Klingner and Bertero, 1978 [11] proposed one of the first cyclic law of URM infill walls. In this model, the unloading stiffness is parallel to the elastic loading one; and the reloading stiffness starts from the axes origin at zero with a slop depends on the previous achieved maximum displacement. The model was calibrated to reproduce the URM behavior in case of reinforcement connector between the RC frame and the masonry wall. One of the main limitations of the model is its inability to consider the damage accumulation due to the URM panel shrinkage effect. To overcome the problem, *Doudoumis and Mitsopoulou, 1986* [12], presented a model in which the tension and compression part of the cyclic law is inactive till reaching a certain deformation limit. *Panagiotakos and Fardis, 1996* [13] calibrated a cyclic force - displacement law for the diagonal strut, in which the strength envelop is a multilinear curve to reproduce the cracking as well as the ultimate and residual strength of the infill panel. The model initial stiffness depends on the panel geometry and the masonry wall shear modulus. *Madan et al., 1997* [14] developed a hysteretic law composed by the sum of a Bouc-Wen hysteresis model with a strength envelope according to the work presented by [15]. The resulting cyclic law has the capability to reproduce the strength and stiffness degradation as well as the pinching effect. *Crisafulli and Carr, 2007* [4] proposed a more detailed hysteretic model based on [16]. The compression behavior is reproduced by different hysteresis rules. Further, the bed joint sliding is considered through the use of a frictional spring with a distinct hysteretic relationship.



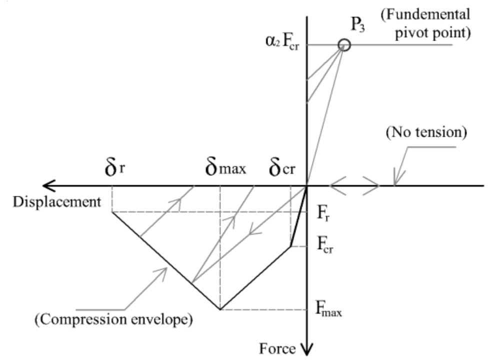
a) Moment plastic hinge model for beams



b) Axial-force/biaxial-moment interactions for columns



c) Concrete hysteresis model (Con)



d) Pivot hysteresis model (PHM)

Figure 1. Applied models and hysteresis schemes for (a, b) primary and (c, d) secondary elements

The identification of the model parameters is quite difficult, specifically in case when the experimental material properties are not available. To enhance the model proposed by [11], *Cavaleri et al., 2014* [17] suggested accounting for the pinching effect during the reloading by a zero value of the URM infill wall strength before reloading. *Liberatore, 2001* [18] presented a simple cyclic law, in which the stiffness unloading degradation branch is controlled through the parameter α (ranges between 0 and 1). In case α=1 residual displacement is expected and unloading stiffness branch is parallel to the initial elastic one. The reloading stiffness degradation is accounted for by means of the parameter β.

For the model quality assessment two hysteretic models are considered, which are able to capture a variety of URM material laws with rather few parameters, relative low computational cost and are available in SAP2000 [27] material library: *(i) Concrete constitutive law (Con); (ii) Pivot Hysteresis Model (PHM)*. Both models were firstly used as a preventative model of the URM walls by [4] and [17], respectively.

The first model is intended for unreinforced concrete and similar materials (e.g. masonry), whereas a non-zero force-deformation curve should always be defined for compression. The force-deformation curve for tension may be ignored, or it may be non-zero provided that the maximum force value is of smaller magnitude than that for the compression side. For this model is specified a single parameter E (between 0 and 1.0) to describe the energy degradation. A value of E = 0 is equivalent to a clean gap when unloading from compression and dissipates the least amount of energy. A value of E = 1.0 dissipates the most energy and could be caused by rubble filling the gap when unloading from compression.

The second *PHM* model was first proposed in [17]. Tension and compression behavior are independently defined. Since the URM infill wall is modeled as a single strut element, just the parameters linked to the compression part of the *PHM* model have to be identified [19].

Table 1. Pivot hysteresis model envelope parameters.

	F_{cr} / δ_{cr}	F_{max} / δ_{max}	F_r / δ_r	$\alpha 2$
Case study 1	25 kN / 0,22 [mm]	36 kN / 3,8 [mm]	18 kN / 20 [mm]	0.022
Case study 2: right panel	423 kN / 0,23 [mm]	507 kN / 3,8 [mm]	74 kN / 16 [mm]	0.2
left panel	522 kN / 0,8 [mm]	680 kN / 5,8 [mm]	136 kN / 25 [mm]	0.2

NUMERICAL SIMULATION AND MODEL ASSESSMENT

Case study 1: Simple one story RC frame with and w/o URM infill wall

The assessment of the proposed models (see Table 2) with respect to the adaption of the in-plane (IP) behavior is done by comparing the analytical results with the experimental results of the cyclic test performed by [20]. The experimental specimen was a simple one story RC frame with and w/o URM infill wall (Figure 2a). To simulate the effect of vertical forces, each column was loaded with 50 kN on the top. Material properties of concrete compressive strength and modulus of elasticity are 28 MPa and 30000 MPa, respectively; yielding strength of reinforcement steel bars is 390 MPa. The vertical compressive strength and modulus of elasticity of the URM infill wall are 1.74 MPa and 2837 MPa, respectively, the cracking shear strength f_{vm} is 0.31 MPa and the URM thickness is 52 mm.

For the aforementioned bare frame experiment a 2D finite element model is created using SAP2000 [27], in which RC frame elements are represented as single beam and columns element. Non-linearity's are lumped at elements end sections by using: *(i) moment plastic hinges; (ii) fiber hinges* for both beams and columns, available in SAP2000. The uniaxial concrete material model according to [21] is used for both unconfined and confined concrete fibers. In addition, linear decay of the tensile strength after the maximum one is considered. Concerning reinforcing steel, the uniaxial material model developed by [22] is adopted for stress-strain relation of steel fibers. The envelop curve parameters for the URM infill panel are given in Table 1. These are used to identify the envelope curve of the hysteretic models for URM walls considered in this study. Namely, *(i) Concrete material model (Con,* cf. Figure 1c), and *(ii) Pivot hysteresis model (PHM,* cf. Figure 1d*).*

Figure 2b shows the comparison of the hysteretic curves between the experimental results and the numerical predictions of the RC bare frame using the lumped plastic hinge concept (*LPH*). The hysteretic curves obtained by the numerical analyses are in satisfactory matching with the experiments in term of loading, reloading and the pinching effect, while the unloading branch is less precisely predicted. The level of underestimation of the maximum force V_{max} value obtained from the numerical model is 8.3%. The experimental specimen is further modelled utilizing the lumped fiber hinge approach (*LFH*). Figure 2e illustrates the experimental/numerical cyclic curves comparison applying the *LFH* approach. A good agreement between the two is obtained. The unloading phase is captured with better accuracy than the former model, whereas the maximum strength of the numerical model underestimates the experimental strength by 5.1%.

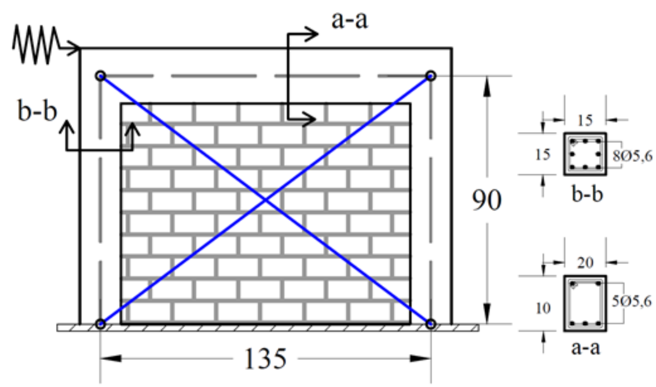
The experimental specimen with URM infill walls is numerically reproduced by means of the lumped plasticity model in combination with the equivalent diagonal strut. The nonlinearity of the diagonal strut element is considered through the use of the two above described models: "*Con*" and "*PHM*". Figure 2c depicts the comparison of the cyclic force-displacement curves between the experimental results and the numerical predictions of the infilled RC frame using the model *LPH-Con*. In general, good matching between experimental and numerical curves is attained. The percentage error in the peak strength is about 11.4%. However, relevant discrepancies can be observed in the unloading branches. It points out a drawback of the "*Con*" model, which does not allow for a smooth unloading response. Figure 2d illustrates the experimental/numerical cyclic curves comparison utilizing the *PHM* model. The numerical model is able to adequately reproduce the experimental response, specifically in the unloading phases. The percentage difference in the maximum strength of URM infilled frame is about 11.3%.

Table 2 summarizes the pro- and cons as well as error in prediction of the maximum strength for the different model combinations. The comparison shows, that the *PHM* model gives better results for the secondary elements, whereas both models for the primary system succeeds in representing the observed experimental behavior with acceptable precision. The observation of the hysteretic curve shapes highlights slight priority of the *LFH* model in term of predicting the loading, unloading, reloading and the pinching effect.

Table 2. Quantitative model assessment.

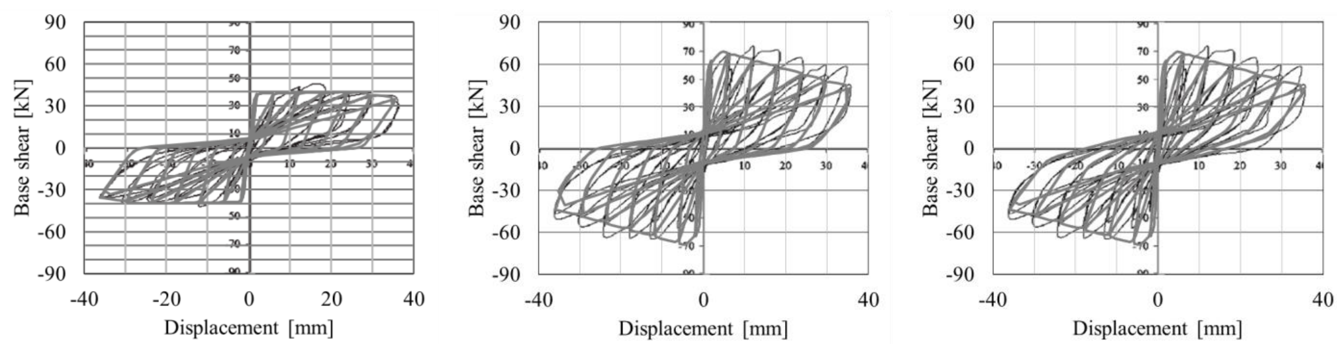
Model	**Error [%]** $V_{max\_exp.} / V_{max\_num.}$	**Loading**	**Reloading**	**Unloading**	**Pinching**
LPH	8.3	√	√	o	√
LFH	5.1	√	√	√	√
LPH-Con	11.4	o	o	x	√
LFH-Con	4.2	√	√	x	o
LPH-PHM	11.3	√	√	o	√
LFH-PHM	4.1	√	√	√	√

Deviation: √ - small O - moderate X - large

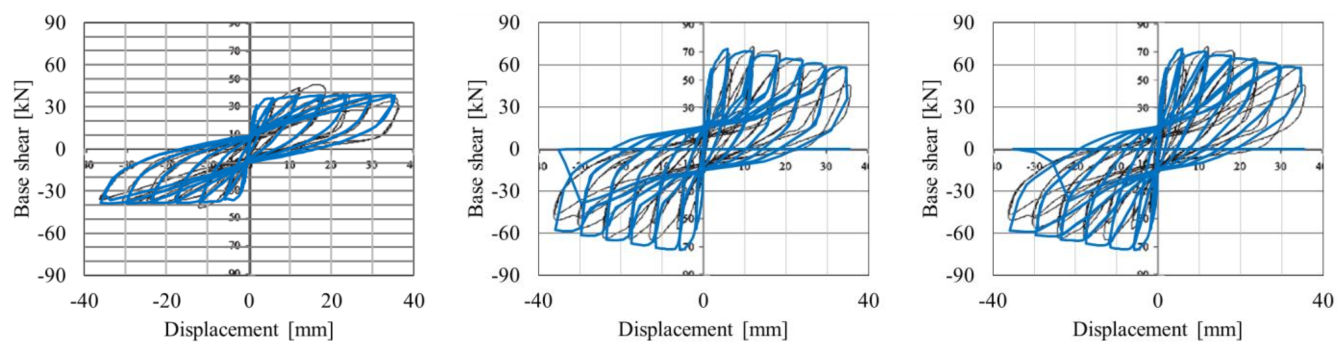


a) RC frame according to [20]

Bare Frame **Frame with URM infill wall**



b) Model: LPH *c) Model: LPH-Con* *d) Model: LPH-PHM*



e) Model: LFH *f) Model: LFH-Con* *g) Model: LFH-PHM*

Figure 2. Comparison of the hysteretic experimental (black line) and numerical (LPH … grey line, LFH … blue line) results for the different model combinations [Note: Comparison mainly done graphically via digitalization of published results.]

Case study 2: Multistory RC frame with and w/o URM infill walls

In order to qualitatively assess the accuracy and abilities of the selected models, additional experimental tests carried out at the ELSA reaction-wall laboratory within the framework of the ICONS TMR-Network research programme [23] are selected and numerically reproduced. The four-story frame is composed of two bays of 5.0m length and one bay of 2.5m length. The inter-story height is 2.7m, the slab thickness is 0.15m with a width of 4.0m. All the longitudinal beams have the same geometry in all stories: 0.25×0.50m², 0.20×0.50m² for the transversal ones. The columns have equal geometric characteristics at all stories: 0.20×0.40m² and 0.20×0.30m², a part from the second "stocky" column with section dimensions of 0.25×0.60m² at the 1st and 2nd story and at the 3rd and 4th story are 0.25×0.50m². The resulting masses, which were taken into account in the pseudo-dynamic tests, were taken as: 47.25t, 37.486t, and 83t for the bottom, second, third, and top stories, respectively. Three acceleration time-histories of increasing return periods of 475, 975, and 2000 years corresponding to 0.22g, 0.28g and 0.38g were used for the pseudo dynamic test. Further information about the case study frame as well as the tests conducted in ELSA can be found in [23].

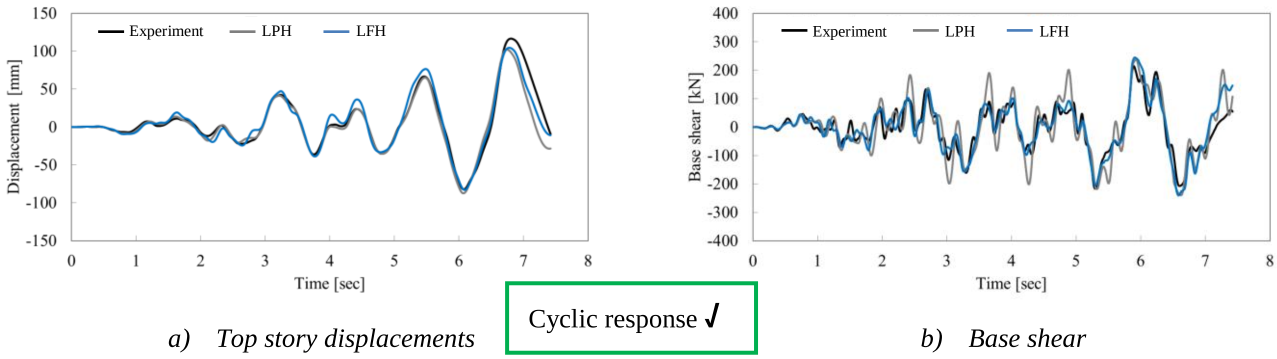


Figure 3. Comparison of the experimental results and numerical prediction of the RC bare frame for models LFH & LPH.

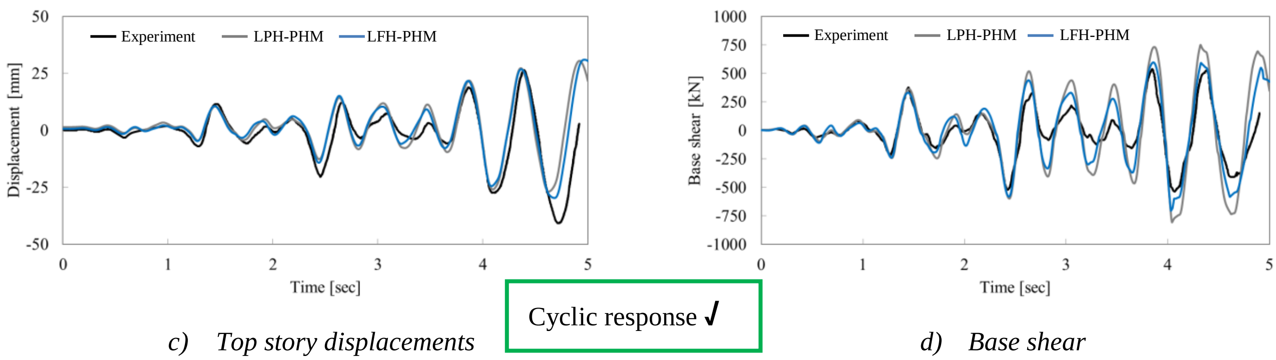


Figure 4. Experimental vs. numerical results of the RC frame with infill walls for model: LFH-PHM & LPH-PHM.

[Note: Difference in time length (in Figure 3 and 4) due to stopped experimental studies and due to observed heavy damage in case of the 2000 years return period time history.]

The experimental/numerical time histories in term of top displacement and base shear are shown in Figure 3 and 4. It can be stated, that all utilized macromodels show a good agreement between the experimental results and the numerical predictions. It is worth mentioning that for both models, bare frame and the infilled one, the numerical results indicate less displacement demands in the last cycle corresponding to the imminent structure collapse.

For a further evaluation of the applied models (*"LPH-PHM"* & *"LFH-PHM"*), the matching between the computed numerical and measured experimental time history displacement at the top of the structure and the base shear are determined by the normalized correlation coefficient ($Corr_{norm}$) according to the following equation (2):

$$Corr_{norm} = \frac{\sum_{i=1}^{N} D_{exp,i}\ D_{num,i}}{\sqrt{\sum_{i=1}^{N} {D_{exp,i}}^2 \quad \sum_{i=1}^{N} {D_{num,i}}^2}} \tag{2}$$

The calculated correlation coefficient is giving values between ~74% and ~87%. Thus, it can be concluded, that both numerical models are able to represent the experimental results realistically (cf. Figure 5). The correlation slightly decreases in case of increasing damage, whereas the LFH model shows always a better behavior. Nevertheless, it has to be noted that the modelling effort in case of LFH model is much larger than in case of LPH model. The correct definition of fibers as well as separate material constitutive laws for concrete and reinforcing steel need an advanced knowledge level.

In addition to the quantitative model assessment by the correlation coefficient, the numerical results for the model *LPH-PHM* are evaluated while comparing the predicted damage grades and zones with the observed ones (cf. Figure 6). For practical realization, it is necessary to introduce a scheme/methodology to transfer observed damage into damage levels, which are comparable with the dynamic analysis results due to the formation of plastic hinges. SAP2000 determines the building performance based on the regulation of FEMA 356 [24] by classifying the damage (post-yield behavior) into five levels: exceedance of the yield point (B – plastic deformation starts), Immediate Occupancy (IO), Life Safety (LS), Collapse Prevention (CP), and Collapse (C) based on deformation limitation criteria.

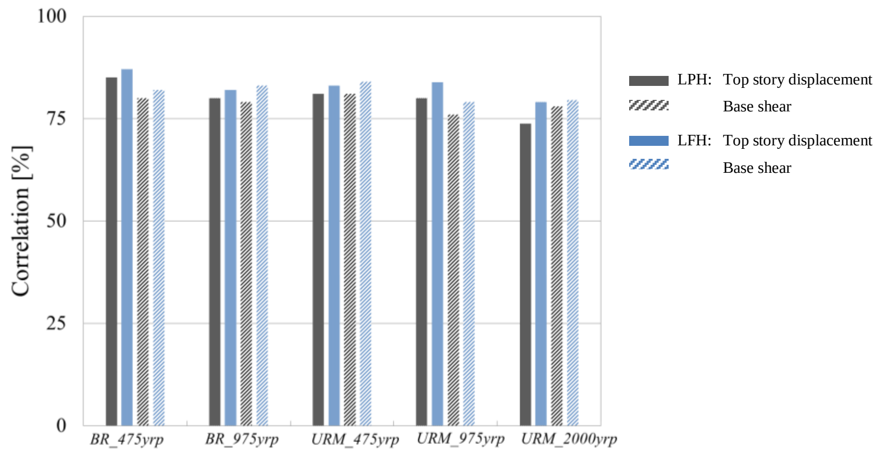


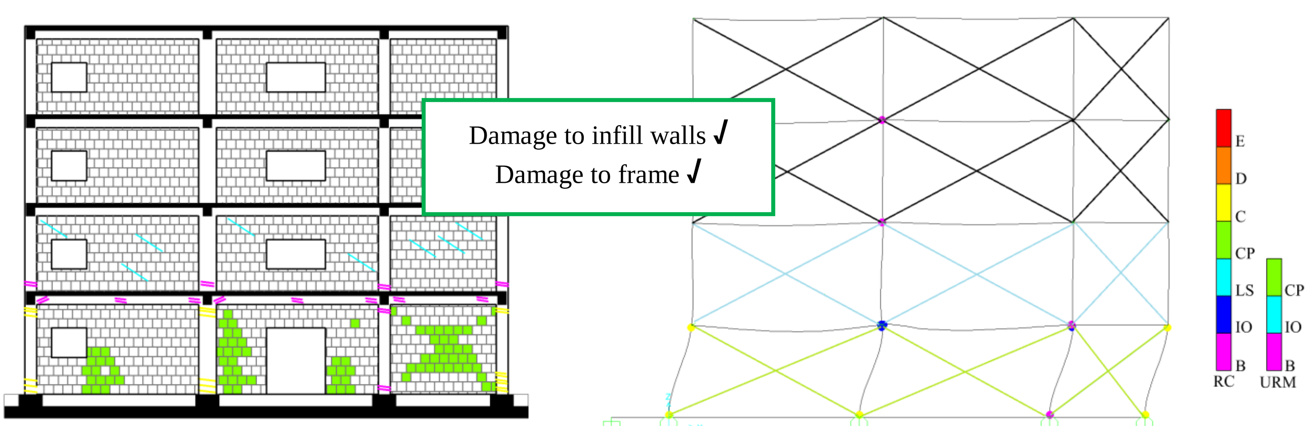
Figure 5. Correlation between experimental and numerical displacement time histories of models LPH & LFH for the bare frame (BR) and the models LPH-PHM & LFH-PHM for the frame with infill walls (URM) for different intensity levels.

Damage observations are typically described by phenomenological description (e.g. according to EMS-98 [25]) with respect to the number of observed damage states (few, many, most). Thus, for the consideration of the prediction accuracy of damage states – a systematization of the observed damages has to be introduced, not at least providing an additional criterion for the model quality assessment. A preliminary, but still refined interpretation scheme has been proposed by *Schwarz et al.* [26], where respective material strains are assigned to local damage grades. For further justification and confirmation of this approach, additional experimental tests are transformed and compared with the numerical predicted performance.

In the case of the bare frame structure and at the end of the second time history (975 year return period), the numerical results indicate a collapse prevention damage state at the third story columns leading to a soft story mechanism, which are more or less comparable with the observed damage according to [23]. Figure 6a shows the systemized and transformed observed damage at the end of the pseudo-dynamic tests for the infilled frame. The RC frame columns and URM infill walls in the first story were heavily damaged, i.e. collapse prevention to near collapse damage state (CP - C), whereas the infill walls in the 2nd, 3rd and 4th story suffered slight damage, i.e. minor cracks. It has to be noted that the case of infilled frame did not manage to complete the test under the 2000 years return period record, which was interrupted as the frame approached imminent collapse. The corresponding numerical predicted results are shown in Figure 6b: heavy damage of the URM infill walls and columns at the first story; immediate occupancy limit state (IO) reached at the 2nd story; intact URM infill walls at 3rd and 4th story. In both cases (i.e., the bare and infilled frames), comparable numerical and experimental results are attained.

CONCLUSIONS

The present study assesses the model quality of state of the art models for primary and secondary structural elements; applied in the widely used standard software SAP2000. Therefore, well documented experimental tests are utilized to compare the experimental results with the numerical predictions based on phenomenological criteria as well as correlation values.



a) Systematization and transformation of damage *b) Numerically predicted damage*

Figure 6. Comparison of systemized experimental results with numerical damage prediction for model LPH-PHM

The results show, that the fiber hinge model for the primary (frame) elements in combination with the pivot hysteresis model for the secondary (infill wall) elements leads to realistic agreements in case of a single frame as well as multistory structure. Nevertheless, even the lumped plasticity hinge model in combination with the pivot hysteresis model is able to provide quite reliable results, indicating the usefulness of other assessment criteria like the prognosis quality of damage state predictions. Due to the limited number of similar comparative studies, the proposed methodology requires further investigations.

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