USING MACRO ELEMENTS TO PREDICT NEAR-COLLAPSE PERFORMANCE OF TWO TYPICAL RC BUILDING STRUCTURAL SYSTEMS WITH LIGHTLY REINFORCED WALLS AND SLENDER PRECAST COLUMNS

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ABSTRACT:

The emerging performance based procedures call for more complex models able to monitor near-collapse behavior. Are macro models, defined as an assemblage of inelastic springs controlled by force-deformation relations, able to describe such behavior? The ability of the multiple-vertical-line-element model to predict seismic response of lightly reinforced RC structural walls as well as the performance of the lumped plasticity beam-column model in the case of slender columns in RC precast industrial buildings were studied. Presented examples have demonstrated that macro elements could be used in predicting global near-collapse performance, if adequate experimental background was provided. While near-collapse flexural behavior of RC structural walls was adequately modeled in advance to the experiments, additional research of shear behavior as well as the shear behavior of coupling beams has been needed. Semi-empirical models were used to calibrate the lumped plasticity element empoyed to describe the post-critical behavior of slender precast columns. It was concluded that these models, which were developed for much lower shear-span ratios, cannot be used for such slender columns without additional considerations and modifications. An appropriately modified lumped plasticity model incorporating in-cycle and repeated-cycle strength deterioration was chosen for the future use in the performance-based design and seismic risk studies.

KEYWORDS: Macro models, Near-collapse behavior, RC structural walls, Slender columns, Shear behavior

1. INTRODUCTION

Macro elements are defined here as elements consisting of an assemblage of several springs monitored by force-displacement rather than stress-strain relationships. In the past such elements have served the engineering community well to evaluate life safe performance. With the emerging performance based design procedures and related complex seismic risk studies engineers need information about the near-collapse response of structures in the case of larger, less probable ground shaking demands. Question arises to what extent, and if at all, relatively crude macro elements could provide such information. This problem is particularly serious in the case of the non-standard structural geometry and/or reinforcement details. Related experimental and analytical studies of two RC structural elements (lightly reinforced thin structural walls and precast columns with high shear-span ratio) are presented in this paper. There has been no intention to compare the efficiency of the macro elements with the performance of standard FE micro elements. The authors believe that in general such discussions are not constructive. They would just like to comment about the efficiency of macro models in some specific cases of highly complicated near-collapse seismic response.

The paper addresses two typical macro models – the multiple vertical line element model (MVLEM) for structural walls and the well-known and frequently-used beam-column element with concentrated plastic hinges. In the past MVLEM was successfully verified by several shake-table tests of RC structural walls (i.e. Fischinger & Isakovic, 2004). This paper addresses a shake-table test results obtained for a particular, thin, lightly reinforced 5-storey H-shaped structural wall with openings, which was tested up to the collapse. A lumped plasticity model was used in the analysis of slender cantilever column having shear-span ratio 12.5. A number of different procedures to define the backbone curve and hysteretic rules for the moment-rotation relationship were investigated. Special attention was given to the modeling of the post-critical response.

2. SEISMIC RESPONSE OF RC STRUCTURAL WALLS MODELLED BY MULTIPLE-VERTICAL-LINE-ELEMENT (MVLEM)

2.1. Multiple-vertical-line-element model

MVLEM was originally developed by Japanese researchers (Kabeyasawa et al, 1983). It consists of several parallel vertical springs modeling flexural and axial behavior and additional horizontal spring to address shear behavior. The element has been several times modified by the researchers at ULJ and a 3D version (Figure 1) has been recently incorporated into the OpenSees (Kante, 2005). Hysteretic rules for vertical springs are illustrated in Figure 2a. The stress-strain relationship for confined concrete was considered to determine the behavior of the springs in compression. Hysteretic rules for horizontal springs are illustrated in Figure 2b.

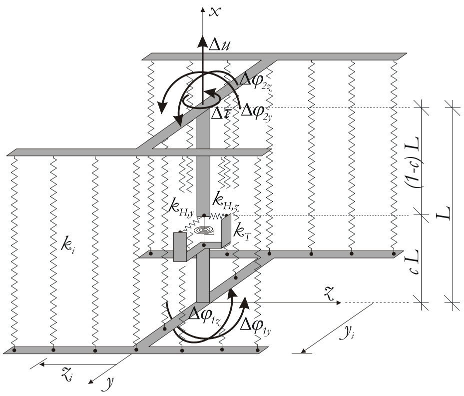
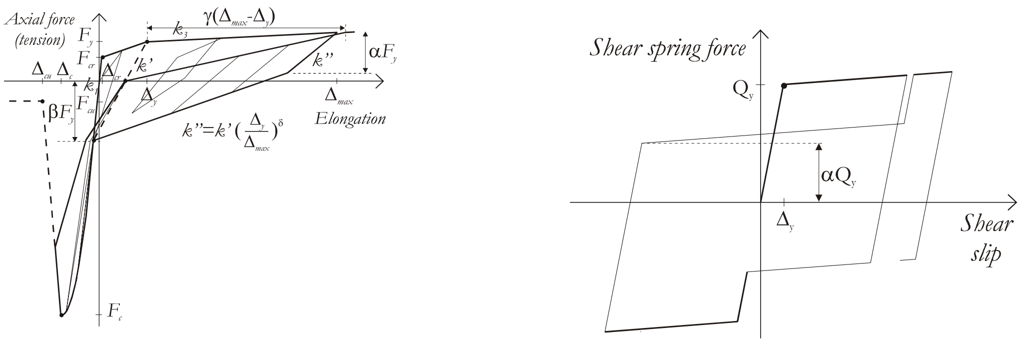


Figure 1 3D multiple-vertical-line-element model (MVLEM-3D)



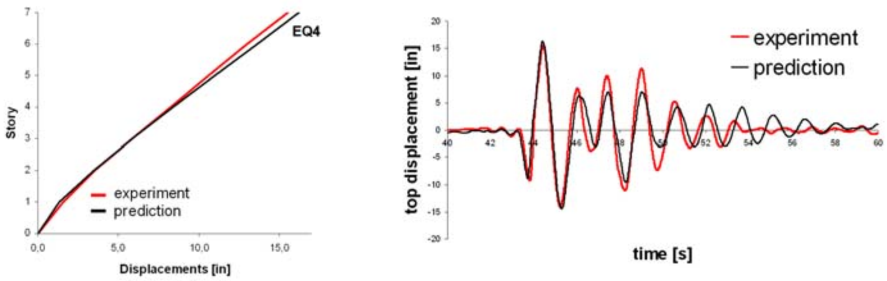
(a) Vertical springs (b) Horizontal springs

Figure 2 Hysteretic rules controlling the springs in the MVLEM

2.2. Previous applications of MVLEM

The 2D version of the MVLEM developed at ULJ has been successfully used in several benchmark studies (i.e. CAMUS3; Combescure & Chaudat, 2002). The researchers at ULJ got the NEES prize (EERI, 2006) for the best prediction of the seismic response of the full-scale 7-story building slice with rectangular RC structural wall (Figure 3) tested on the shaking table at UC San Diego in the frame of the NEES project (Panagiotou et al, 2006). Predicted deformation parameters matched almost perfectly with the experimental results. The results (maximum displacements over the height of the wall and the response history of the top displacement) for the last test (with the shake table acceleration about 1g) are compared in Figures 4a and 4b, respectively. Not only the global parameters matched well, also the predicted tension deformation of the reinforcement at the boundary area (0.0315) was close to the experimental value (0.0263). It has been fully realized that such, almost perfect, mach could not be achieved without some luck. Macro elements depend on a number of empirically based parameters, the choice of which is rather arbitrary. Nevertheless, in this project, as well as in several other projects, MVLEM proved the ability to monitor predominantly flexural response of structural walls well into inelastic range.

Figure 3 The set-up of the NEES 7-story building slice at UC San Diego



(a) displacement profile

(b) response history of the top displacement

Figure 4 Comparison of the predicted response and experimental results for the NEES building

2.3. Shake table response of the H-shaped coupled wall – experiment and results

The 1:3 model of a 5-story wall consisted of two coupled T-shaped piers (Figures 5 and 6). The piers were reinforced by very light (minimum reinforcement) according to the Slovenian building practice (Figure 6). Some free edges of the wall piers were confined and some were not. Very simple and weak diagonal reinforcement consisting of 2 crossed bars (Figure 7) was used in the coupling beam. Heavy additional mass was added due to the reduced scale and to account for the mass in the adjacent fields in realistic structures. This required relatively thick slab.

The shake table test was performed at LNEC in Lisbon, Portugal in the frame of the ECOLEADER project, coordinated by ULJ. The Tolmezzo accelerogram, recorded during the Friuly 1976 earthquake was used in 2 directions in a series of tests with increasing intensity. In the last of the series of the tests (6th run) the table acceleration in the direction of the web wall with openings was larger than required (1.02g instead of 0.75 g) and the acceleration in the direction of the flange walls was 0.52g.

Therefore the failure occurred in the direction of the web (Figure 8). Typical shear failure of the wall piers was observed. The flange walls were not damaged much. Some damage was observed at the unconfined edges and due to punching caused by the web wall. To a surprise the supposedly weak coupling beams were practically undamaged.

Figure 5 Coupled structural wall, designed according to the Slovenian building practice

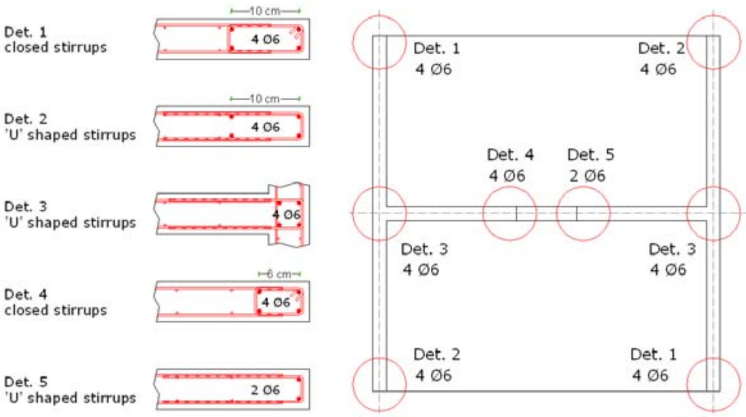


Figure 6 Reinforcement details for the piers of the coupled structural wall

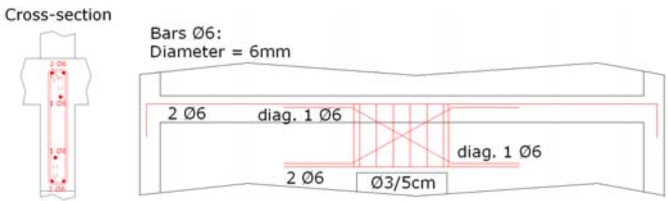


Figure 7 Reinforcement in the coupling beams

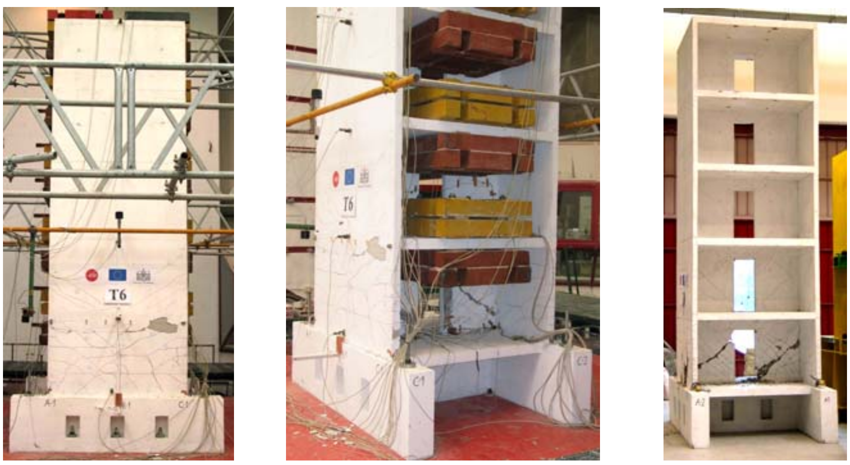


Figure 8 Damage in the coupled wall after the last test

2.4. Shake table response of the H-shaped coupled wall – analytical modeling and comparison with the experimental results

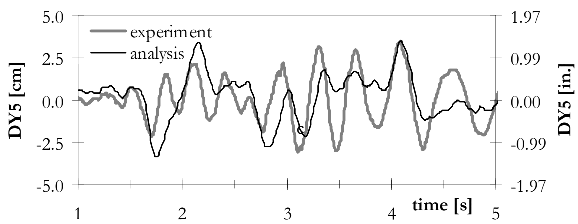
3D – MVLEM model was used for the piers. The properties of the confined concrete were used where appropriate. Beam element with concentrated shear springs at the ends was used for coupling beams. The shear strength was calculated as the upper bound determined by the flexural mechanism for the coupling beam with rectangular cross section (without contribution of the slab reinforcement).

The calculated top displacement response history in the 6th (last) run will be compared with the experimental results in the next paragraphs. MVLEM again proved to be successful in predicting predominantly flexural response of the flange walls. However, the prediction of the response of the web wall, where shear failure had occurred, was not good (Figure 9a).

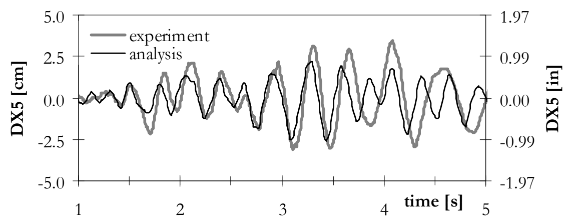
The predominant reason for the unsuccessful prediction of the behavior in the direction of the coupled wall was the underestimated strength of the coupling beams. Consequently, the individual wall piers were heavily loaded and they failed in shear, which was neither predicted nor modeled. Both factors (better estimation of the strength of coupling beams and inelastic shear behavior of the wall piers) were included into the improved model.

(1) The full 3D model, taking into account the interaction of the slab and coupling beam was analyzed by ABAQUS program upgraded by ANATECH modulus for reinforced concrete. Practically elastic shear behavior of the coupling beam, observed in the test, was confirmed. Taking into account elastic shear behavior of coupling beams, the analytical results were improved (Figure 9b).

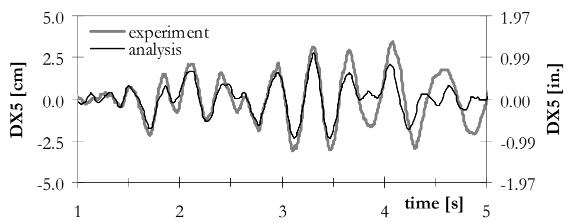
(2) In addition to the improvement (1), inelastic shear behavior of the wall piers was modeled by the modified compression field theory (Vecchio and Collins, 1986) using the Response-2000 computer program. Values valid for monotonic loading were used in the presented study. The correlation of the analytical and test results for the global response quantities was quite good (Figure 9c) with the exception of the last part of the last run, when the piers of the coupled wall actually collapsed in shear. The web also punched through the flange walls and this was not modeled.



(a) Initial model



(b) Coupling beam/slab interaction considered



(c) Coupling beam/slab interaction and inelastic shear behavior of wall piers considered

Figure 9 Top displacement response history in the direction of the flange wall with openings

3. SEISMIC RESPONSE OF SLENDER RC COLUMNS MODELED BY LUMPED PLASTICITY BEAM-COLUMN MODEL

3.1. Experimental results

Seismic response of RC columns with shear-span ratio 12.5 was studied (Kramar, 2008) within the frame of a seismic risk study for typical European RC precast industrial buildings (Figure 10). In the frame of the project "Seismic behavior of precast concrete structures with respect to Eurocode 8" (Toniolo, 2007) a series of PSD tests and cyclic test until failure were performed at ELSA, Ispra, Italy.

While the detailed description of the tests and the results are given in Fischinger et al (2008), as well as in the companion paper by Kramar et al (2008), only the most important experimental results are given in this paper. The deformability and the deformation capacity of the columns were large. The yield drift was 2.8% (much more than the values reported for columns with smaller shear-spans). In the final cyclic test, the columns exhibited quite stable response up to a large drift close to 7 %. Buckling of the longitudinal reinforcement bars then led to subsequent tension failure of the bars in the first column. The strength of the structure dropped considerably, but it was stabilized by the other five columns. A 20 % drop in maximum strength was observed at about 8 % drift, following considerable in-cycle strength degradation and the flexural failure of several columns. At this drift, the capacity of the experimental facilities was exhausted, and the test was stopped. The curvature distribution at the base of the external column at 7 % drift (i.e. before the rebar failure) showed that plasticity was concentrated mainly in the short length (about half of the cross-section dimension of the column) above the base.

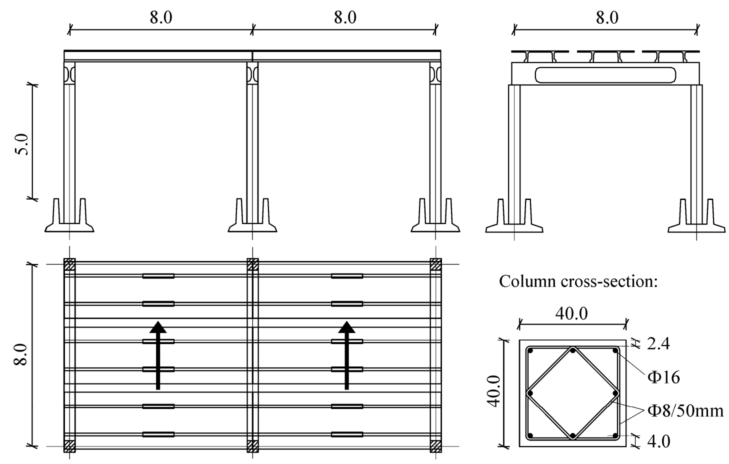


Figure 10 Precast industrial building tested at ELSA, Ispra, Italy

3.2. Analytical modelling and comparison with the experimental results

The beam-column model with lumped plasticity has been used for decades as well as a number of extensive data bases for RC columns exist (PEER, 2007; Panagiotakos and Biskinis, 2001). Nevertheless, the existing analytical models had troubles to describe the observed behavior.

(a) The long-known semi-theoretical approach based on the idealized moment-curvature relationship and empirically determined equivalent plastic hinge length was relatively successful (Fischinger et al, 2008) if very short plastic hinge length (equal to one half of the column cross-section dimension) was used. However, this method has not been able to identify the post-critical branch of the hysteretic behavior.

(b) The empirical expressions proposed by Fardis and co-workers (i.e. Fardis and Biskinis, 2003) worked fine for the yield drift. The prediction for the ultimate drift was too large. This value has depended considerably on the shear-span ratio. The shear-span ratios of the columns in the data base prepared by Fardis do not exceed 6.5

and the extrapolation to columns having shear-span ratio 12.5 is questionable. In addition the expressions proposed by Fardis and al. do not give the value for the capping drift and post-capping stiffness.

The best results were obtained using Ibarra hysteretic model (Ibarra et al, 2005) that accounts for history-dependent strength and stiffness deterioration. The behavior is first described by a monotonic backbone curve. Pre-capping and post-capping cyclic strength deterioration, based on the energy dissipation criterion, is then considered (Figure 11). Haselton (2006) has calibrated Ibarra hysteretic model for a large number of column tests. If Haselton expressions, except for the yield drift (which was determined analytically taking into account empirical corrections for pull-out and shear-slip), were used, the match of the analytical and experimental results was very good (Figure 12). The yield drift in Haselton expressions is independent of the height of the column, which is true for the relatively short columns. For these columns the sum of the flexural drift and the drift due to the shear and pull-out is practically constant. However, for the columns having shear-span 12.5 flexural component clearly dominates.

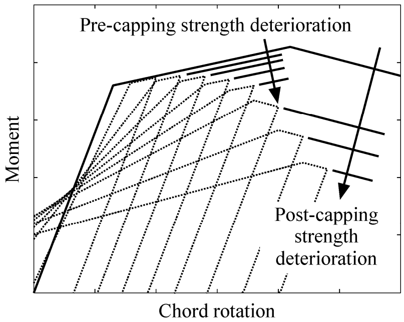


Figure 11 Strength deterioration in the Ibarra's model

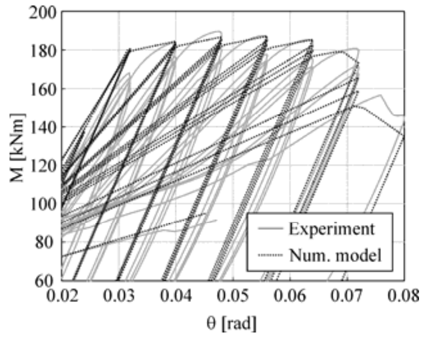


Figure 12 Numerical versus experimental results

4. CONCLUSIONS

If adequate experimental background was provided, macro elements could be used in predicting global near-collapse performance, as demonstrated by the presented examples. However, the volume of experimental data needed to support up-to-date performance based procedures (regardless if macro or micro elements are used) is huge and it will take a tremendous effort and long time to obtain the required information.

Multiple-vertical-line-element was able to predict predominantly flexural behavior of thin lightly reinforced walls having limited confinement at the free edges. However, modeling of the coupling beam-slab interaction and the inelastic shear behavior of the wall piers in coupled walls needed experimental calibration. The contribution of the slab to the shear capacity of the coupling beam was much greater than expected. Further research of these topics is needed. Although the modified compression field theory provided acceptable parameters for the analysis of the inelastic shear behavior of the wall piers in this particular study, it still has to be generalized for the complex axial-flexural-shear cyclic interaction.

Semi-empirical models were used to calibrate the lumped plasticity element employed to describe the post-critical behavior of slender precast columns. It was concluded that these models, which were developed for much lower shear span ratios, cannot be used for such slender columns without appropriate additional considerations and modifications. An appropriately modified lumped plasticity model incorporating in-cycle and repeated-cycle strength deterioration was chosen for future use in performance-based design and seismic risk studies.

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