Modeling Inelastic Shear Response of RC Walls

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SUMMARY:
The large inelastic shear magnification has been seldom considered in the design practice. Consequently the shear resistance of existing buildings might be insufficient and inelastic shear behaviour should be considered in the (risk) analyses. A model to account for inelastic shear behaviour and inelastic shear-flexural interaction has been proposed. It is based on the multiple-vertical-line-element macro model. Additional shear spring, accounting for aggregate interlock, dowel action and horizontal reinforcement resistance is linked to each of the vertical springs. The characteristics of each component depend on the deformations at the crack (in particular the width of the crack) within the individual strip. The model successfully simulated the response of a 5-story coupled wall tested on the shaking table under bi-axial excitation. In the last run both piers suffered a complete shear failure in the first story. The shear resisting mechanisms within the cracks were adequately modelled up to the collapse.

Keywords: inelastic shear, inelastic shear flexural interaction, multiple-vertical-line-element, RC coupled wall

1. INTRODUCTION

Reinforced concrete structural walls have been efficiently used in seismic regions all over the world. A typical structural system used for apartment buildings (in Europe and Chile for example) is shown in Figure 1.1.

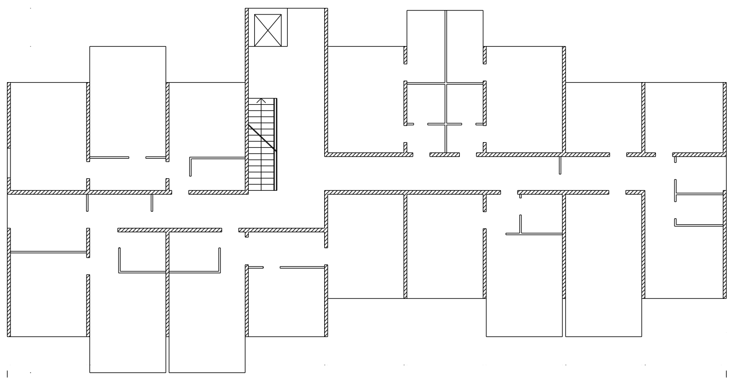

Figure 1.1 Typical multi-story apartment building with structural walls

The system is characterized by high wall-to-floor area ratio and by quite thin as well as relatively lightly reinforced/confined walls. After the recent Chile earthquake (Boroschek and Bonelli, 2012) attention has been driven to the possible compression failure of thin edges in such walls. But it should be realized that this damage occurred primarily because the system was forced beyond its practical limits, which is about 10-15 stories.

When this limit was observed in the past, the system behaved quite well (Fajfar et al., 1981; Wallace and Moehle, 1993). There were only few failures reported and they were first of all tension-shear failures (Wood 1991). An example of such failure is shown in Figure 1.2 taken after the 1979 Montenegro earthquake. Please note that the typical construction practice in seventies was to use very weak horizontal reinforcement.

Figure 1.2 A pier in a coupled wall damaged during the 1979 Montenegro earthquake

Even some recently built structures suffered similar problems during the L'Aquila earthquake (Figure 1.3).

Figure 1.3 Shear failure of the wall in an apartment building during the L'Aquila earthquake

It should be noted again that even in this quite new buildings the horizontal reinforcement has been very weak. The problem of the insufficient shear resistance is therefore not limited only to the walls in the old existing buildings. Although the large shear magnification during inelastic response was pointed out by Blakely (1975) long time ago, even today many designers are not fully aware of this phenomenon and only few codes, like in New Zealand or Eurocode 8 (CEN, 2004), consider this magnification explicitly. Eurocode 8 requires that the shear forces obtained by the equivalent elastic analysis V_{Ed}' are multiplied by the shear magnification factor ε, in order to obtain the design shear forces V_{Ed}:

$$V_{Ed} = \varepsilon \cdot V_{Ed}' \tag{1.1}$$

For ductility class high (DCH) walls, the shear magnification factor is determined by the expression Eqn. 1.2 that was originally proposed by Keintzel (1990)

$$\varepsilon = q \cdot \sqrt{\left(\frac{\gamma_{Rd}}{q} \cdot \frac{M_{Rd}}{M_{Ed}}\right)^2 + 0.1 \cdot \left(\frac{S_e(T_C)}{S_e(T_1)}\right)^2} \begin{cases} \leq q \\ \geq 1.5 \end{cases} \tag{1.2}$$

where:
q is the behavior (seismic force reduction) factor used in the design;
M_{Ed} is the design bending moment at the base of the wall;
M_{Rd} is the design flexural resistance at the base of the wall;
γ_{Rd} is the factor on design value of resistance accounting for various sources of overstrength;
T_1 is the fundamental period of vibration of the building in the direction of shear forces;
T_{C} is the upper limit period of the constant spectral acceleration region of the spectrum;
$S_{\mathrm{e}}(T)$ is the ordinate of the elastic response spectrum.

Recent studies by Rejec et al. (2011) confirmed that due to the higher modes effects and flexural overstrength inelastic shear forces in structural walls can be several times higher compared to equivalent elastic code forces. Since this amplification has been frequently ignored, doubts exist about the adequate shear resistance of structural walls even for new buildings. Yet most (risk) studies nowadays assume that the behavior of the walls will be flexural.

Numerical models for inelastic flexural response of structural walls are indeed quite well developed and efficient (i.e. Fischinger et al., 2008). However very few models are able to simulate inelastic shear behavior and in particular inelastic shear-flexural interaction (Kabeyasawa et al., 1997; Chen and Kabeyasawa, 2000; Orakcal et al., 2007; Kim et al., 2011 and Fischinger et al., 2012). A robust and efficient model has still to be developed.

In this paper the authors have proposed an extended and modified model based on the multiple-vertical-line-element macro model (MVLEM), which was repeatedly proved as very efficient in simulating predominantly flexural response. However, in the original versions, MVLEM does not consider shear-flexural interaction and hysteretic models for shear are frequently purely empirical and very crude. Therefore the authors have embraced the concept (previously used by Orakcal et al., 2007 for example) in which the multiple shear springs are linked to the vertical springs of the MVLEM (see Section 3 for detailed description).

The newly developed model (Section 3) has been integrated into OpenSees (McKenna and Fenves, 2007) and used to simulate the shake-table experiment on a large 1:3 model of a coupled shear wall (Section 2). In this experiment the shear failure of both wall piers occurred. The model was able to simulate the response up to the final failure (Section 4).

2. SHAKE-TABLE TEST OF A 5-STORY COUPLED SHEAR WALL

A 1:3 model of a 5-storey H-shaped coupled wall (Figure 2.1) was organized by Slovenian researchers at LNEC in Portugal (Fischinger et al., 2006). Those characteristics of the specimen, which had a crucial influence on the shear behavior of the piers, are summarized below:

a) The cross-section of the specimen consisted of two T-shaped walls/piers coupled with short beams and relatively thick inter-story slabs.
b) The specimen was constructed according to the Slovenian practice in nineties, which did not fully take into account the shear magnification effects. Therefore a relatively low (minimum) amount of horizontal reinforcement (A_s/A_c=0.25%) was used. Moreover, due to very small diameter of steel wires, steel with a low ultimate deformation $\varepsilon_{s,u}$ = 1.5% was used for the web reinforcement.

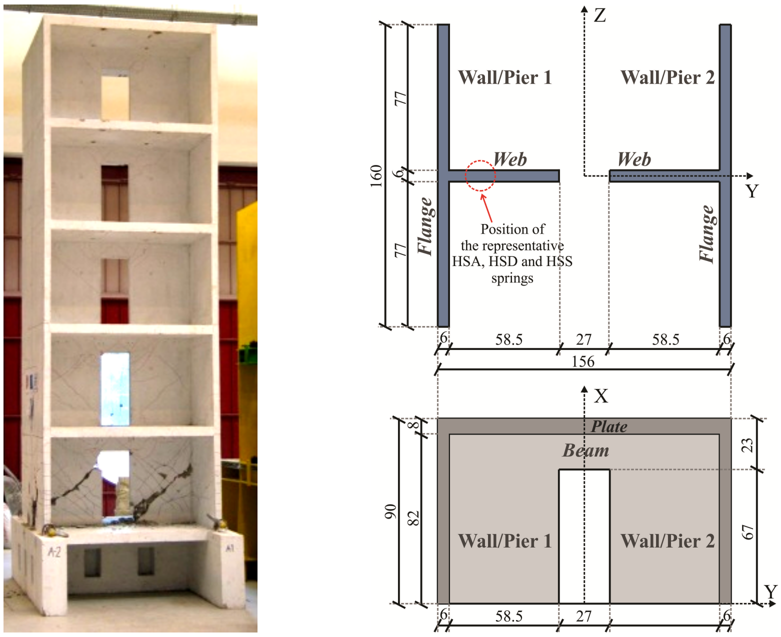

Figure 2.1 The wall designed according to the Slovenian practice and tested on the shaking table at LNEC in Lisbon (Fischinger et al., 2006)

In the last (6th) experimental run (maximum table acceleration in the direction of the flanges was $a_{g,max,z}$ = 0.52 g and in the acceleration in the direction of the web was $a_{g,max,y}$ = 1.02 g) the complete shear failure of the webs in both piers was observed. It was demonstrated that the coupling beams were much stronger than anticipated (primarily due to the overstrength provided by relatively thick slabs) and therefore the capacity design was not efficient.

A preliminary analysis of the inelastic shear response using compression field theory (Vecchio and Collins, 1986) was made in 2005 (Kante, 2005; Fischinger et al., 2006). However several questions of how to apply the theory for the cyclic response have remained opened. The need for the efficient modeling of the inelastic shear-flexural interaction became particularly evident.

3. NUMERICAL MODELLING OF THE INELASTIC SHEAR RESPONSE AND INELASTIC SHEAR-FLEXURAL INTERACTION

3.1. Past developments

For several decades the research group in Ljubljana has successfully used the multiple-vertical-line-element macro model (MVLEM) in the analysis of the RC structural walls. Several modifications and improvements were made. In 2005 a 3-D version (Figure 3.1) was incorporated into the OpenSees program (Kante, 2005). Therefore it was decided to upgrade this model with the features needed for the inelastic shear response analysis.

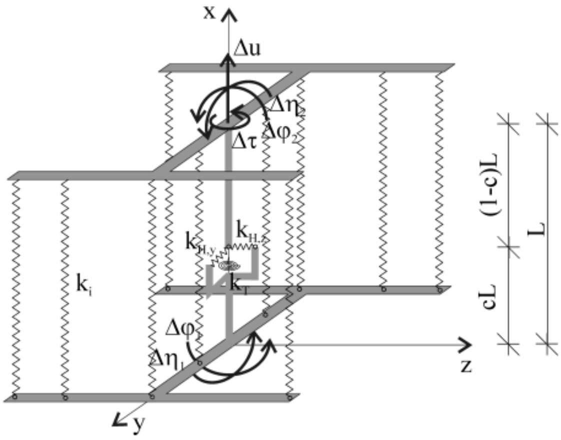

Figure 3.1 3D multiple-vertical-line-element (MVLEM)

An early modification of the classic MVLEM in order to consider the relation between the longitudinal deformations and the shear behavior of RC members was made by Colotti (1993). Petrangeli et al. (1999) proposed that the full *N-M-V* interaction can be covered by modeling the shear mechanism at each concrete fiber (strip) of the cross section. The shear mechanism corresponding to a selected fiber is dependent on the stress-strain state in the same fiber. As it is explained in the continuation, such approach was adopted in the formulation of the new model developed at the University of Ljubljana. The most complete modifications of the multiple-vertical-line-element model including the *N-M-V* interaction were made by Chen and Kabeyasawa (2000), the Wallace research group (Orakcal et al., 2006) and Kim et al. (2011). However, the authors felt that simpler model explicitly accounting for shear transfer in the cracks was still needed.

3.2. The proposed macro model

Shear springs were added into each vertical spring of the MVLEM (Figure 3.2).

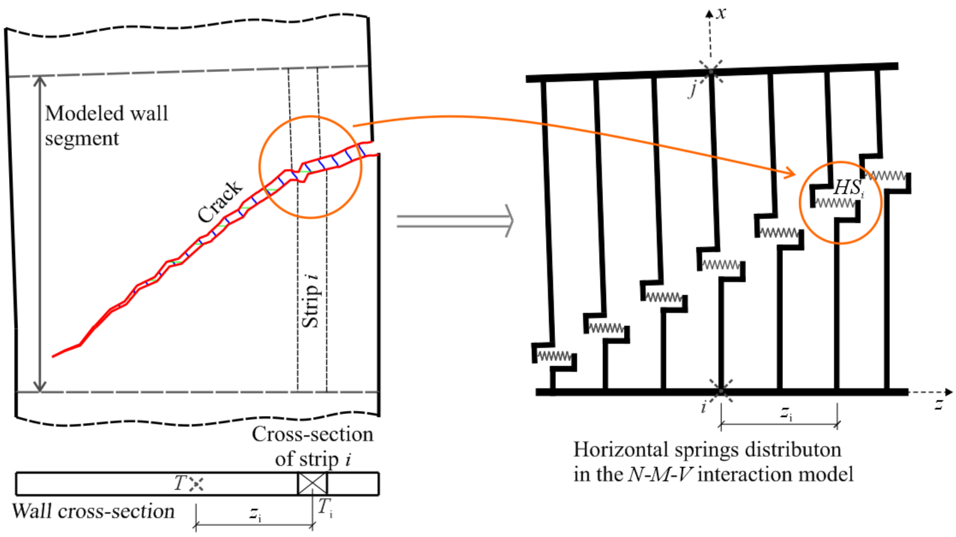

Figure 3.2 Scheme of the upgraded MVLEM having multiple horizontal springs

These springs are first of all intended to model the shear transfer mechanism across the cracks in the case of the diagonal tension failure (Figure 3.3).

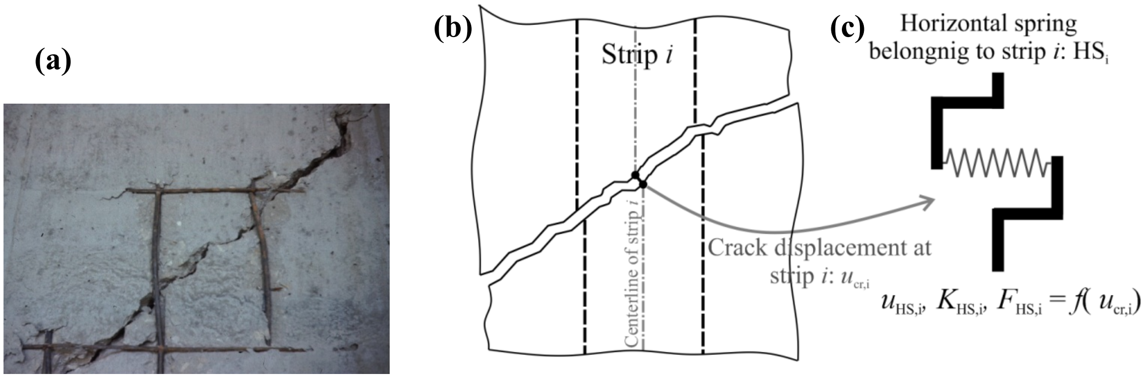

Figure 3.3 Modeling of shear transfer mechanisms according to the proposed model: Each horizontal spring (c) represents the shear transfer mechanisms across cracks (a) for the assigned strip (b)

The procedure (Rejec, 2011) for evaluation of the current stiffness and force in horizontal springs consists of two steps: (i) the method to determine the current crack displacement according to the current strain state, which is represented by the displacement of vertical and horizontal springs; (ii) constitutive rules that define the relation between the crack displacement and current force and stiffness of the horizontal springs.

The shear behavior and resistance modeled by the horizontal springs depend on the mechanisms that transfer shear force over the cracks (Figure 3.4). The mechanisms consist of (a) dowel effect of vertical bars, (b) axial resistance of horizontal/shear bars and (c) interlock of aggregate particles in the crack. The capacity of the latter is highly dependent on the cracks' width.

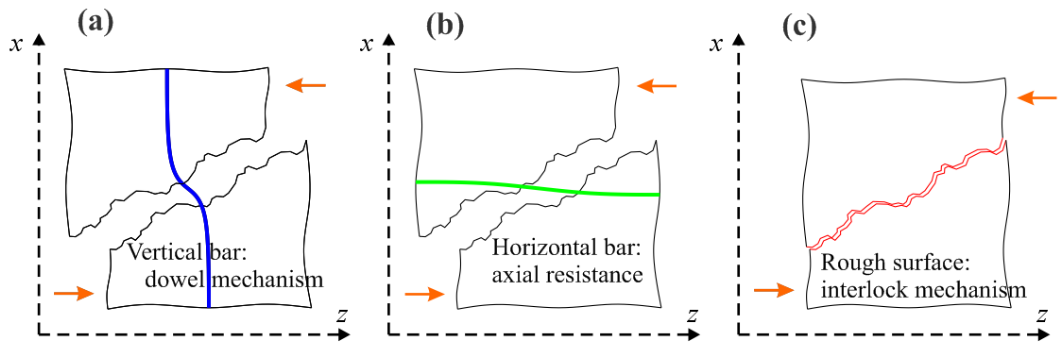

Figure 3.4 Mechanisms of shear force transfer over the cracks: (a) dowel effect of vertical reinforcement; (b) axial resistance of horizontal/shear reinforcement and (c) interlock of aggregate in the crack

Therefore each spring is composed of 3 components (Figure 3.5): HSA to account for aggregate interlock, HSD to account for the dowel action and HSS to account for the axial resistance of the shear reinforcement. The current characteristics of each component depend on the deformations/displacements at the crack within the individual strip. The displacements are linked to the current displacements of the nodes of the element.

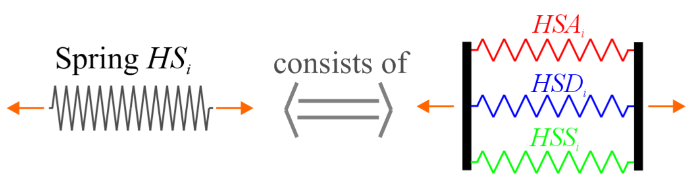

Figure 3.5 Each horizontal spring is composed of 3 components to account for aggregate interlock (HSA), dowel (HSD) and shear/horizontal reinforcement (HSS) mechanisms

The constitutive relations for the individual springs are based on the semi-empirical relations found in the literature (a detailed description is given in Rejec, 2011). Aggregate interlock is modeled by the Lai-Vecchio model (Vecchio and Lai, 2004), dowel action by the expressions proposed by Dulacska (1972) and Vintzeleou and Tassios (1987). The force-displacement relation for HSS springs has been based on the bar-slip model proposed by Elwood and Moehle (2003).

4. INELASTIC SHEAR RESPONSE OF A COUPLED SHEAR WALL

Experiment described in Section 2 was used to verify the proposed model. The modified accelerograms based on the Tolmezzo 1976 earthquake records with increasing intensity were used in the experiment. In the 5th run the table acceleration in the direction of the web was 0.73g and the acceleration in the direction of the flanges was 0.42g. In the 6th run the accelerations increased to 1.02g and 0.52g, respectively.

4.1 Moderate inelastic response and cracking in the 5th run

Only moderate inelastic response and cracking was observed during the 5th run (Figure 4.1). Global response was modeled very well (Figure 4.2). Shear springs successfully indicated deterioration of the aggregate interlock (HSA springs) in one direction. Dowel mechanism (HSD springs) and shear reinforcement mechanism (HSS springs) were subsequently activated (Figure 4.3).

Figure 4.1 Crack pattern in the web of the first story after the 5th run

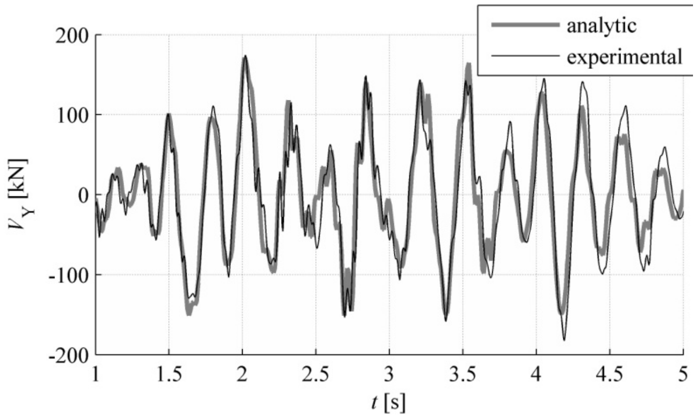

Figure 4.2 Base shear response in the direction of the web during the 5th run

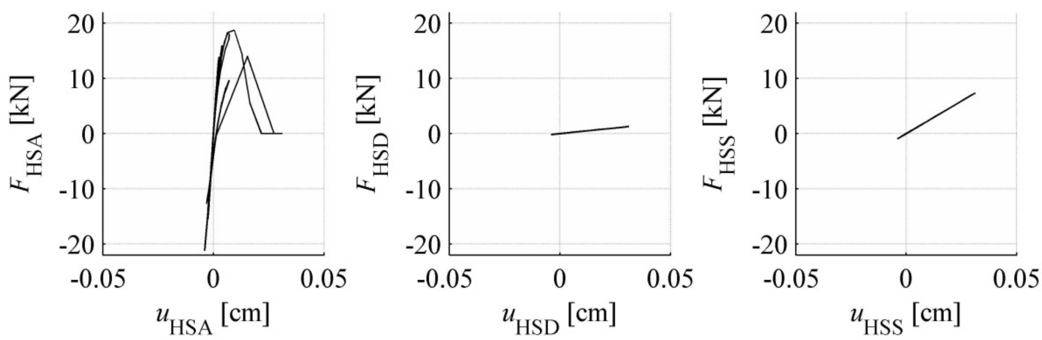

Figure 4.3 Response of the shear springs accounting for different shear mechanisms in the 5th run

4.2 Shear failure of the web in both piers – 6th run

The total shear failure of the web in both piers in the first story is shown in Figure 4.4. The aggregate interlock mechanism (HSA springs), which had considerably deteriorated in the previous run, was completely destroyed (Figure 4.6) and the HSS spring indicated the rupture of the very brittle horizontal reinforcement, which was actually used in the test specimen for the web reinforcement. Dowel mechanism (HSD springs) has been first fully activated and then failed completely. Total shear failure was indicated shortly after the 3rd second of the response (Figure 4.5). The residual resistance of the wall observed in the test (and not numerically verified) is attributed to the frame action of flanges and slabs (which was not included in the model). The simultaneous and same failure pattern of the webs in both piers has been attributed to the bi-axial loading. Results of the analysis identified that at the time of the failure the webs in both piers were in net tension, which explains the same inclination of the crack in both piers.

Figure 4.4 Shear failure in the first story

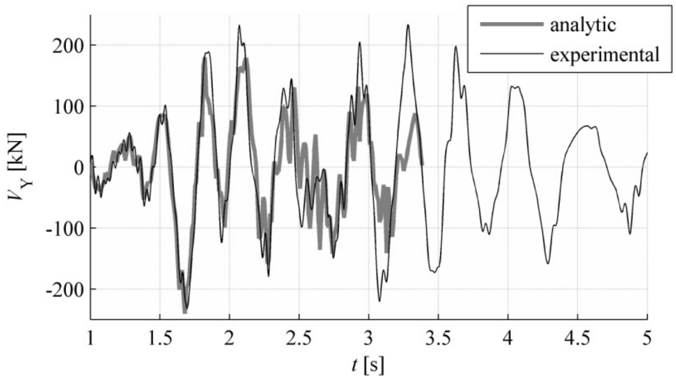

Figure 4.5 Base shear response in the direction of the web in the 6th run

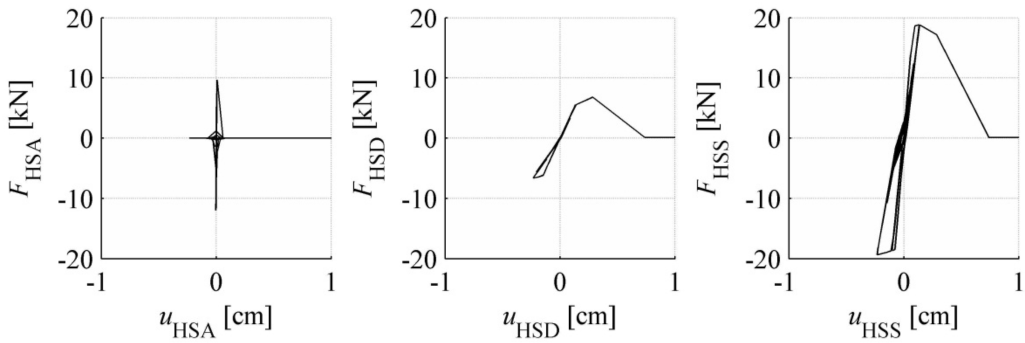

Figure 4.6 Response of the shear springs accounting for different shear mechanisms in the 6th run

5. CONCLUSIONS

The inelastic shear magnification, which is due to overstrength and the higher modes effect, can be very large. Since it has been seldom considered in the design practice, the shear resistance of existing buildings might be insufficient and inelastic shear behaviour should be considered in the (risk) analyses.

A model to account for inelastic shear behaviour and inelastic shear-flexural interaction has been proposed. It is based on the multiple-vertical-line-element macro model. Additional shear spring, accounting for aggregate interlock, dowel action and horizontal reinforcement resistance is linked to each of the vertical springs. The current characteristics of each component depend on the deformations at the crack (in particular the width of the crack) within the individual strip.

The model successfully simulated the response of a 5-story coupled wall tested on the shaking table under bi-axial excitation. In the last run both piers suffered a complete shear failure in the first story. The shear resisting mechanisms within the cracks were adequately modelled up to the collapse of both piers.

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