Doctoral School on Engineering Sciences
Università Politecnica delle Marche

Extended summary

Seismic Design of Hybrid Coupled Shear Walls (HCSW)

Curriculum: Ingegneria delle Strutture e delle Infrastrutture

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Date: 30-January-2014

Abstract. In the last decades of the 20th Century hybrid steel and concrete systems have been the subject of numerous studies by the international scientific community. The possibility of coupling, in distinct resistant elements, materials with different and complementary properties has captivated the attention of a lot of designers.

All the studies have highlighted the enormous potential of such systems, not yet widely found in the most famous international codes.

This work, starting from the study of the state of the art for these systems, which incorporates structural configurations also very different from each other, want to present an innovative system, defined by the acronym HCSW (Hybrid Coupled Shear Walls).

In the examined structural case, the hybrid behaviour is expressed on two levels: the system deputy to face the gravitational loads is constituted by a series of plane steel frames, while the seismic action is entirely entrusted to a hybrid bracing system, constituted by a reinforced concrete wall and steel links. The choice to use the term "*link*" to define the steel beams connecting the reinforced concrete wall and the adjacent frame system, is related to the study of eccentrically braced steel systems, similar to the light of many points of contact that will be highlighted from time to time in the discussion.

The design of the link is at the heart of the entire work and is carried out by two approaches, in the first one, defined "*elastic-oriented*", the designer controls what happens in the elastic range; in the second one defined "*plastic oriented*" starting from a boundary condition the project takes place in the plastic range.

The linear and non-linear analysis, static and dynamic, will be carried out on a simple case study, which allows an immediate interpretation of the results.

Keywords. Coupled Shear Walls, Displacement-Based Design method for Hybrid Structures, Force-Based Design method for Hybrid Structures, Hybrid structures, Steel Links.

1 Problem statement and objectives

This work deals with the seismic design of a particular type of hybrid steel-concrete system identified by the acronym HCSW (Hybrid Coupled Shear Wall).

In the first instance it is necessary to emphasize the difference between "*composite system*" and "*hybrid systems*": in composite system different materials are found to coexist within the same structural element, in hybrid systems, however, structural elements, with different tasks in the against external actions which a building is subjected, are made of different materials, without however that these are found to coexist within the same element.

In seismic design hybrid systems can offer great benefits, unlike composites ones, in which problems of stress transfer at the interface attest a significant degree of uncertainty.

In the examined structural case, the hybrid behaviour is expressed on two levels: the system deputy to face the gravitational loads is constituted by a series of plane steel frames, while the seismic action is entirely entrusted to a hybrid bracing system, constituted by a reinforced concrete wall and steel links.

The design of seismic bracing system is the main focus of the entire work. The study of the peculiar characteristics of the bracing system, in which the reinforced concrete wall must provide adequate stiffness and strength, while the link is entrusted the entire dissipative task, is an essential starting point for the validity of all the proposed results. The discussion focuses on the application of design methodologies, innovative or extensively present in the codes, for the constituent elements of the system.

Coupled shear wall systems obtained by connecting reinforced concrete shear walls by means of beams placed at the floor levels constitute efficient seismic resistant systems characterised by good lateral stiffness and dissipation capacity. Coupling beams must be proportioned to avoid over coupling, i.e., a system that acts as a single pierced wall, and under coupling, i.e., a system that performs as a number of isolated walls.

Extensive past research has led to well established seismic design guidelines for reinforced concrete coupling beams, typically deep beams with diagonal reinforcements, in order to satisfy the stiffness, strength, and energy dissipation demands. The coupling beam-wall connections depend on exclusively made of reinforced concrete elements. In the former case, the connection is similar to beam-column connections in steel structures. In the latter case the connection is achieved by embedding the coupling beam inside the wall piers and interfacing it with the wall boundary element. In the past decade, various experimental programs were undertaken to address the lack of information on the interaction between steel coupling beams and reinforced concrete shear walls. However, coupled shear wall systems suffer from being difficult to be repaired after strong earthquakes. Design recommendations following the criteria of Performance or Displacement Based Design (PBD and DBD) and Force Based Design (FBD) are still missing or at their early stage of development.

2 Research planning and activities

2.1 Presentation of an innovative hybrid system

The first step of the research project concerns the study of the state of the art of hybrid systems. This phase consists of two levels: the overall design of such systems and the detail of the individual elements of a bracing system that withstands seismic actions (reinforced concrete walls, steel link, connections,…). This phase involves case studies, designs of buildings actually realized, laboratory prototypes.

The first fundamental result derived from the preliminary investigation stage concerns the absolute innovative character of the hybrid system proposed. While in the systems studied in the past, the steel links act just as connections between two or more concrete walls that provide stiffness and strength to the system, in the innovative hybrid system (HCSW) steel links are entrusted in the entire dissipative task, while the RC wall provides stiffness necessary to allow high links' excursions in the plastic range.

The research phase on the state of the art has specifically analyzed the guidelines in European and extra-European context for such systems. The result is that, not only the innovative systems are obviously not present, but also the information provided on similar systems are mainly descriptive and never prescriptive or performance-oriented, not actually offering any support to the designers.

The example of the innovative hybrid system proposed in this work is the reinforced concrete shear wall with steel links presented in Figure 1. The reinforced concrete wall carries almost all the horizontal shear force while the overturning moments are partially resisted by an axial compression-tension couple developed by the two side steel columns rather than by the individual flexural action of the wall alone. The reinforced concrete wall should remain in the elastic field (or should undergo limited damages) and the steel links connected to the wall should be the only (or main) dissipative elements. The connections between steel beams (links) and the side steel columns are simple: a pinned connection ensures the transmission of shear force only while the side columns are subject to compression/traction with reduced bending moments. Even if a capacity design is required, columns are expected to have a relatively small cross section. The negative effects of the reinforced concrete wall on the foundations would also be reduced. The structure is simple to repair if the damage is actually limited to the link steel elements. To this end, it would be important to develop a suitable connection between the steel links and the concrete wall that would ensure the easy replacement of the damaged links and, at same time, the preservation of the wall. Clearly, the proposed hybrid system is effective as seismic resistant component if the yielding of a large number of links is obtained.

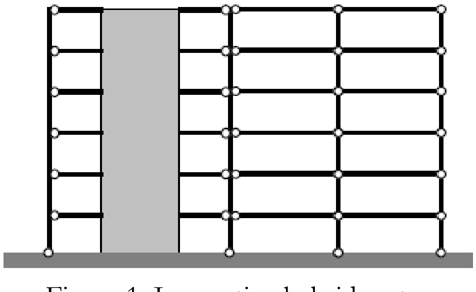

Figure 1: Innovative hybrid system

2.2 Seismic design method for steel links through an "elastic-oriented" procedure

As just underlined, HCSW is a dual system in which steel frames are designed to withstand gravitational loads, while the bracing system constituted by the wall and the steel links is designed to withstand the seismic action.

The first design procedure for HCSW systems presented in this work can be defined "*Force-Based*". At this point it is necessary to make some clarifications on what "*Force-Based method*" means. If we trace the historical-scientific line that led to the definition of a so-called "*Force-Based Design procedure*", attention was mainly focused on the definition of "R", or "q" according to the European codes, i.e. on the reduction factor of forces. It is in fact the introduction of "R" which marked an innovative approach in the field of seismic design; through the reduction factor it is possible to consider the post-elastic behaviour of the structure, while continuing to carry out the project in the elastic range, with all simplifications inherent in this choice. It is precisely on the design in the elastic range that develops the connection between the traditional "*Force-Based Design*" method and the design method for HCSW systems we are going to present. Although in the proposed procedure you are not going to use an R coefficient imposed by the codes, however, the whole procedure is in the elastic range with a final validation in the plastic range.

The geometrical sizing of the bracing system is the central matter of this work.

The heart of the matter is the relationship of stiffness among an hypothetical bracing system constituted only by a reinforced concrete wall and the hybrid bracing system in which the links go to add to the wall, as shown in Figure 2-3:

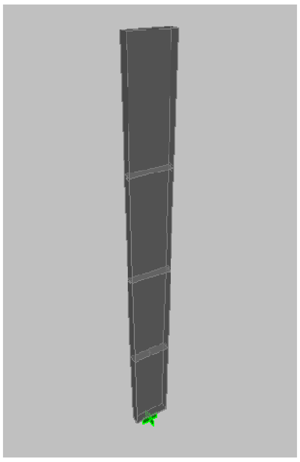

Figure 2: RCSW

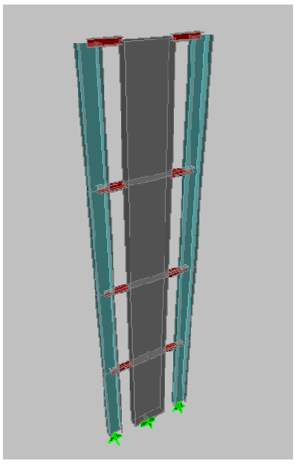

Figure 3: HCSW

The sizing of the bracing system is developed in elastic range: RC wall and links are characterized by an elastic behaviour. The study assumes as limit condition the complete yield of the links that has to happen before the wall yield (yield of the wall reinforcements).

In first phase arbitrary dimensions, width, thickness, height, must be assigned to the wall considering the relationship among the wall width "B" and total height of the building "H"; for instance, once an arbitrary thickness to the RC wall is assigned, the whole system can be studied considering the ratio H/B varying from 10 to 15. Such values are justified by constructive issues guaranteed by practice and by considerations about the relationship between resistance and stiffness: the wall, in fact, has to be sufficiently flexible to avoid links' yield and in the same time sufficiently strong to maintain its performance in elastic range. To this point a structural scheme is obtained in which the RC wall is geometrically defined and its stiffness can be found.

The aim of the procedure is to detail the links that constitute the bracing system with the RC wall. The sizing method is a geometric one, in which first a reference section is chosen (HE, IPE, …) and then it proceeds with the detailing of the constituent elements (web and flanges). In this preliminary step it is not required to obtain links' dimensions that can characterize them as commercial profiles, but the study operates in the welded profiles range in order to choose the best performing sections.

The conclusive arbitrary parameter examined in this phase is the relationship of stiffness among the bracing system constituted only by the RC wall (RCSW = Reinforced Concrete Shear Wall) and the bracing system constituted by wall and links (HCSW = Hybrid Coupled Shear Wall). These two systems are represented in Figure 2 and 3. The stiffness ratio of the RCSW to the HCSW is indicated as "R".

How much stiffer has to be the HCSW system?

In first phase reference values of the stiffness ratio R varying from 2 to 3 are assumed, that is the HCSW bracing system is 2 or 3 times more stiff than the RCSW one. To this point it results defined the first fundamental condition of the link detailing procedure: the HCSW system has to be R-times stiffer than the RCSW one.

In the second step other two conditions are imposed:

1. when links show elastic behaviour the deformed shape must be linear (therefore the interstorey drift has to be constant);
2. the links are owed to yield at the same time, i.e. for the same level of plastic rotation required (contemporary entry of the links in plastic range).

2.3 Seismic design method for steel links through a "plastic oriented" procedure

The fundamental factor around which the development of the Displacement-Based Design Procedure for HCSW revolves is the coupling ratio "CR" between links and wall. The coupling ratio, as already mentioned in Chapter 1 during the presentation of the state of the art for hybrid systems, is without doubt one of the most characteristic properties for this type of structure.

The CR definition is derived from hybrid systems similar to HCSW, but not exactly equal; the highly innovative nature of these systems, in fact, made it possible to trace in the literature studies on similar systems but never identical.

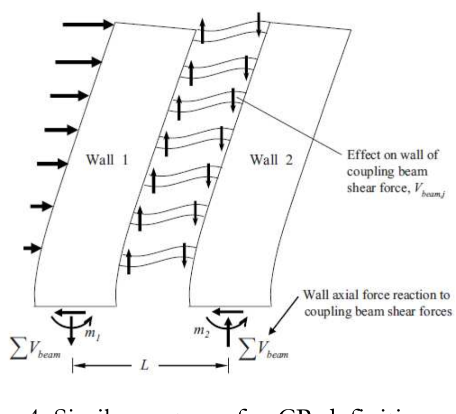

Figure 4: Similar systems for CR definition

Research over the past half century on coupled wall systems has shown that their structural performance is strongly influenced by the amount of coupling provided by the system. Although the majority of studies have focused on reinforced concrete coupled wall systems (Fig. 4), the system behaviour and mechanics are the same for all coupled wall structures including hybrid systems.

In analytical terms, the degree of coupling "CR" can be expressed in the following way:

$$CR = \frac{M_C}{M_C + M_W}$$

where:

M_C is the total resisting moment;

M_W is the reinforced concrete "working moment".

The coupling ratio CR in substance quantifies the contribution of the "system link" to face the bending moment at the base of the bracing system (called "overturning moment"), for a system of horizontal seismic forces. By convention, the calculation of CR is made at the base of the wall when the system forms a mechanism. In this idealized case, the coupling beams are assumed to maintain their plastic shear capacity as the wall piers yield.

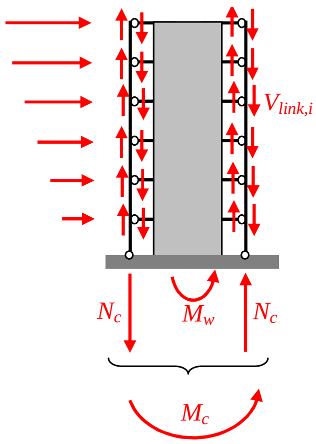

Figure 5: Wall's and links' contributes to face the total bending moment

The starting point of the whole procedure is the definition of the wall, both in terms of geometrical and mechanical properties.

The properties of the wall are invariant during the analysis. The "plastic-oriented" method, in fact, is finalized to links' sizing once geometric characteristics of the span in which the bracing system is inserted and the mechanical characteristics of the wall are known. The geometric properties of the wall are defined once the ratio H/B is known, where H is the overall height of the building, and B is the width of the wall. The thickness "t" of the wall is defined in an arbitrary manner.

With regard to the definition of strength and ductility properties, the reinforcements of the wall have been designed according to the statement in UNI EN 1998-1:2005, with respect to the critical areas, the boundary element and trying to maximize the flexural and shear capacity.

To this end, the geometrical ratio of reinforcement in critical areas has been maximized and set equal to 4% of the area of the section:

$$\rho = 0.04A_c$$

where:
ρ geometric ratio of reinforcement;
Ac area of the concrete section.

Once the geometrical and mechanical characteristics of the wall are known, the contribution MW is known.
With reference to Figure 5 it is possible to explain the axial load at the base of the columns NC and then the shear stress for each link:

$$N_C = \sum_{i=1}^{n_{links}} V_{link,i}$$

where:
N_c is the axial load in the columns;
$V_{link,i}$ is the shear force in the i-th link.

At this point, the contribution provided by the "link system" to face the total bending moment is given by the following relationship:

$$M_C = L_{tot} N_C = \left(l_w + 2l_{link}\right) \sum_{i=1}^{n_{links}} V_{link,i}$$

where:
M_c is the overturning moment part faced by the “link system”;
L_{tot} is the span width;
l_w is the wall width;
l_{link} is the link’s length;
$V_{link,i}$ is the shear force in the i-th link.

The following expressions are consequences of the previous formulas:

$$N_C = \frac{M_C}{L_{tot}} = \frac{CR}{1-CR}\frac{M_W}{L_{tot}} = \frac{CR}{1-CR}\frac{M_{W,Rd}}{L_{tot}\gamma_W}$$

$$V_{link,i} = \psi_i \frac{N_C}{n_{links}} = \psi_i \frac{1}{n_{links}}\frac{CR}{1-CR}\frac{M_{W,Rd}}{L_{tot}\gamma_W}$$

where ψ is the vertical shear link distribution.

In order to estimate the shear stress in each link, two distribution are proposed, as shown in Figure 6:

1. Uniform distribution: the axial stress at the base of the columns is distributed along the height of the building assigning to each link the same rate of the total shear. The distribution coefficient assumes unit value;

2. Non uniform distribution: the axial force at the base of the columns is not equally distributed to each floor, but the shear rate to be assigned to the links is defined as follows:

$$\psi_i = \begin{cases} 2CR \leq 1.6 & z_i \leq \dfrac{H}{2} \\ 2(1 - CR) > 0.4 & z_i > \dfrac{H}{2} \end{cases}$$

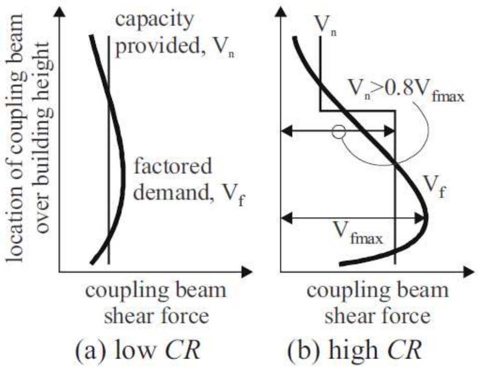

Figure 6: Vertical distribution of coupling beam shear

What is the meaning of the two selected shear distributions?

The uniform load distribution ($\psi i = 1$) leads to the same link's profile in height. In fact, the procedure just explained gives the same link section to all floors. In analogy to what is seen in the pre-sizing elastic procedure, in which solutions with COV(J) = minimum were judged the best ones, this choice aims at a uniform mechanical configuration of the resistant elements.

This option can be justified by constructive reasons: it removes the burden of designing different link-wall and link-column connections on the various floors.

The non-uniform distribution, however, tends to correct "defects" of the original structure in terms of stiffness distribution of the resisting elements.

When CR assumes low values (<< 0.5) the wall is the predominant resistant system and it typically faces the seismic action showing a "cantilever behaviour": the request of plasticity (in terms of displacement and therefore strain) is maximum on the upper floors; high CR values (>>0.5), however, correspond to a typical "frame behaviour": the link system prevails on the wall and the request of plasticity is maximum on the lower floors.

With a in height non-uniform distribution the link sizing proceeds according to the needs dictated by the CR: this alternative, in fact, tends to return larger sections of the links on the upper floors for structures with values of CR low, on the lower floors for higher values of CR.

3 Analysis and discussion of main results

3.1 Presentation of the case-study

The designing of the links in the elastic and plastic range and all the validation procedures are performed starting from a case study of reference, briefly presented below:

All the reference structural models on which analyses have been conducted show evident characteristics of simplicity and regularity; this is a fundamental condition to develop numerous attempts necessary to validate an empirical-experimental procedure. The selected structure here presented is quite simple, constituted by four elevations of constant height equal to 3.4 m, for a total height of 13.6 m. The square plan configuration is gave by 5 spans of 8 m in both directions.

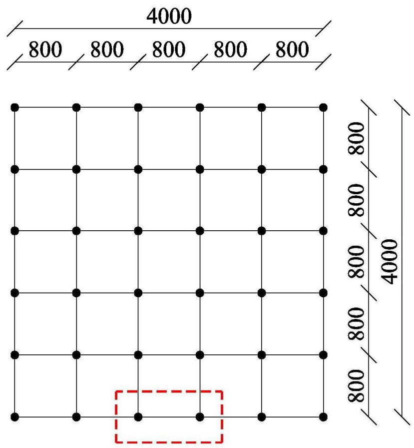

Figure 7: Plan view

In a first attempt the bracing system formed by the reinforced concrete wall and the steel links is symmetrically inserted inside the central span on each side. The total mass is 1200 kNs^2/m at each storey (residential type building).

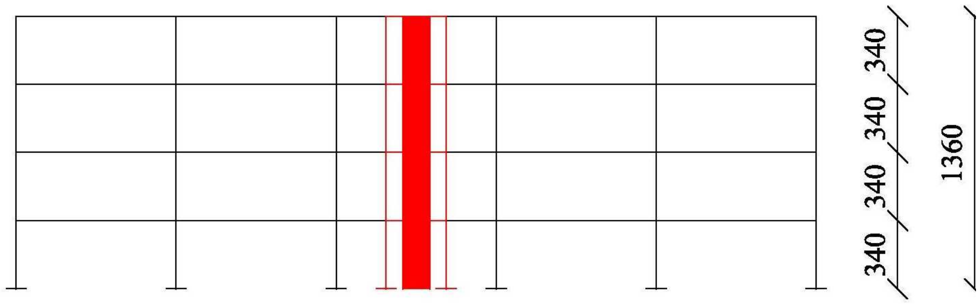

Figure 8: Section view

3.2 Result of the "*elastic-oriented*" procedure

The pre-sizing method is carried out starting from different sections of the links on the first attempt and assuming R=3, once fixed the dimensions of the wall (H/B=10; H/B=12).

In order to consider as acceptable those solutions that at the 3rd and last iteration may return as result a realistic section of the link at 2nd, 3rd and 4th floor, the profiles of the first attempt that returned to the last iteration the best sections are:

- HE140B (Hwall / Bwall =10)
- HE160A (Hwall / Bwall =10)
- HE160B (Hwall / Bwall =10)
- HE180A (Hwall / Bwall =10)
- HE180B (Hwall / Bwall =10)
- HE200B (Hwall / Bwall =10)
- HE260B (Hwall / Bwall =10)

The study of the pre-sizing method has highlighted that the results obtained depend on the first attempt section with which the iterative procedure starts: starting from different structural configurations the procedure always comes to the same solution in terms of overall stiffness of the system, but then this global stiffness is differently distributed to the various floors according to the initial stiffness conferred. Indeed, as shown in the tables above, the pre-sizing method returns different valid solutions (i.e. solutions with average ratio h/b or b/h < 1.5), each linked to the corresponding first attempt links' section.

It is therefore necessary at this stage to introduce additional criteria to select the solutions obtained, thus freeing the acceptability of the solution from the geometric properties of the sections (control of the h/b ratio). Observing that since up to this point the analysis are carried out only in the elastic range, the first hypothesis is to consider preferable those sections that have a minimum coefficient of variation of the module of inertia (J) to the various floors. This criterion is justified by construction reasons (simplification of the study of the link-wall connection at each floor) and by hypothetical predictions about the post-elastic behaviour: in fact, assuming the same seismic demand, the sections with constant values of J reach the final capacity in an almost uniform and simultaneous way, thus maximizing the ductile resources of the structure.

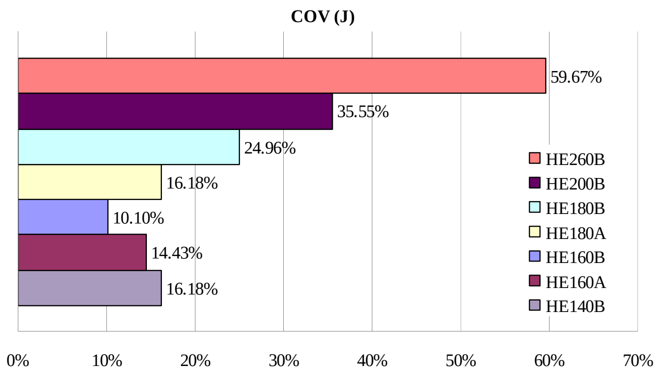

Figure 9: COV(J) values for each solution

Based on this criterion, the best solution in the linear range is the one resulting from a first attempt section HE160B that returns a solution characterized by the minimum COV(J)=10.10%.

The validation of the procedure in the plastic range is carried out by subjecting selected models in the elastic phase to static nonlinear analysis (pushover) and to nonlinear dynamic analysis. The analysis in the plastic range requires the definition of non-linear characteristics for materials and sections.

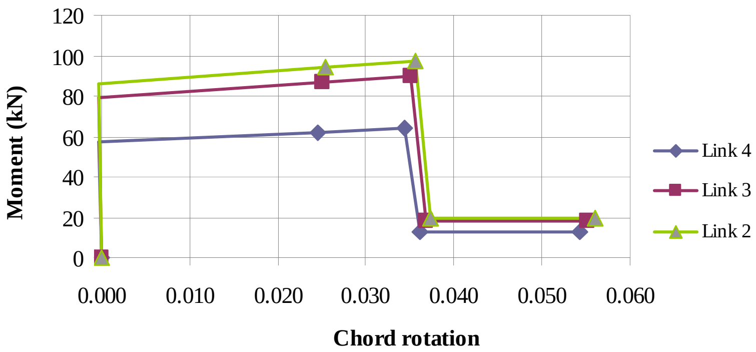

Figure 10: Plastic hinge definition for the non linear characterization of the links

In order to evaluate the results obtained from the validation in the plastic range the following parameters of evaluation are introduced:

- **Index of "goodness" of the solution (I)**: when the seismic demand is defined, the best solution is the one closest to the achievement of performance points (PP) in the ADRS (**A**cceleration-**D**isplacement **R**esponse **S**pectrum)

$$I = \frac{d_{max}}{\sqrt{k}/k} = \frac{d_{max}}{\sqrt{k}} k = d_{max}\sqrt{k}$$

- **Performance factor of the link (ηlink)**:the performance factor "ηlink" controls the request of plasticity to the links at the various floors, providing an indication of the level of plasticity in the structure when the first link has reached the performance level:

$$\eta_{link} = \sum \frac{(\theta_i/\theta_{SD,i})}{n}$$

- **Performance factor of the wall (ηwall)**: the performance factor of the wall is a parameter that allows you to monitor the application of plasticity on the wall as links yield. In fact, starting from the basic assumption of the pre-sizing procedure in the elastic phase, which provides that links go into the plastic range before wall, it is necessary to consider the state of deformation of the wall to the attainment of a predetermined limit state:

$$\eta_{wall} = \frac{\theta_{step}}{\theta_3}$$

- **Equivalent ductility (Rμ)**: on the equivalent bilinear curve obtained from the pushover analysis it is possible to define the ductility of the system "Rμ" evaluated as the ratio between the maximum displacement to the achievement of the performance level and the displacement corresponding to the first link enter in plastic range.

$$R_{\mu} = \frac{d_{SD}}{d_{yield}}$$

- **Reduction factor of forces (R)**: the evaluation of the reduction factor of forces "R" is conducted on the basis of the indications in the technical literature:

$$R = R_S R_{\mu} R_R$$

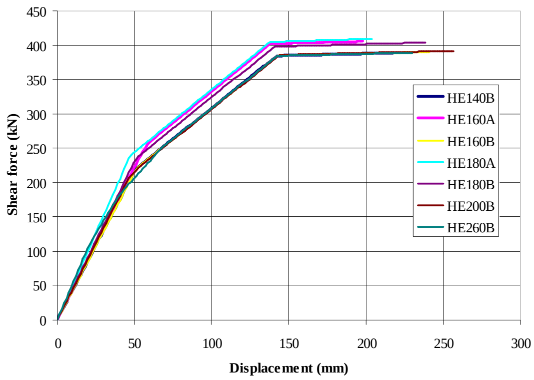

Figure 11: Pushover curves performed

From the pushover analysis carried out the following results are observed:

a) the links are increasingly engaging in the plastic range before the wall, confirming the basic assumption of the pre-sizing method in the elastic phase;

b) in all the models examined, when the first link reaches the SD performance level, the wall is in the plastic range (ηwall > 1), except for the model characterized by a section of the first attempt HE180A in which the wall remains in the elastic range (ηwall = 0.96);

c) the model that shows the maximum value of the "I" index is the model HE200B;

d) the model HE200B has also the highest performance factor of the links;

e) the HE200B model is the one with the maximum excursion in the plastic range of the wall;

f) all models have a link with efficiency greater than 80 %, except for the HE260B model in which the performance factor is 78% (assuming as a minimum an efficiency of 80 % to confirm the basic assumption of the sizing method which provided for the plasticization of contemporary links, this model is ruled out for the final assessments);

g) the model that maximizes the index of the goodness and performance of the link shows high values of the force reduction factor (R), in fact, the value of this coefficient is the second highest among the cases examined.

3.3 Result of the "plastic-oriented procedure"

The "plastic-oriented" procedure shown in the previous paragraph is applied to CR values between 0:30 and 0.80. For each CR value two solutions in terms of link's design are obtained: the first derives from a uniform distribution of the shear resistance at the various levels, the second one derived from a non-uniform distribution.

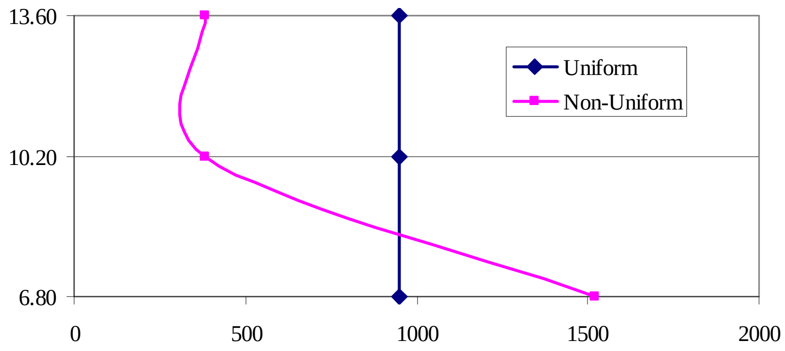

Figure 12: Uniform and Non-uniform distribution for CR = 0.80

All the solutions obtained by varying the CR and the type of distribution, are subjected to non-linear static analysis. For the characterization of the non-linear link and wall please refer to the previously shown in Chapter 2. Even for the solutions obtained by this "reverse procedure", the sections of the links are defined "intermediate" or "long", so these elements are marked with a flexural plastic hinge bending moment-chord rotation

In Figure 13 all "Shear force – Displacement" curves obtained from pushover analysis are shown. With similar colors the curves obtained by keeping fixed the value of the coupling ratio (CR) and by varying the distribution of shear resistance in height (U: uniform distribution; NU: non-uniform distribution) are represented.

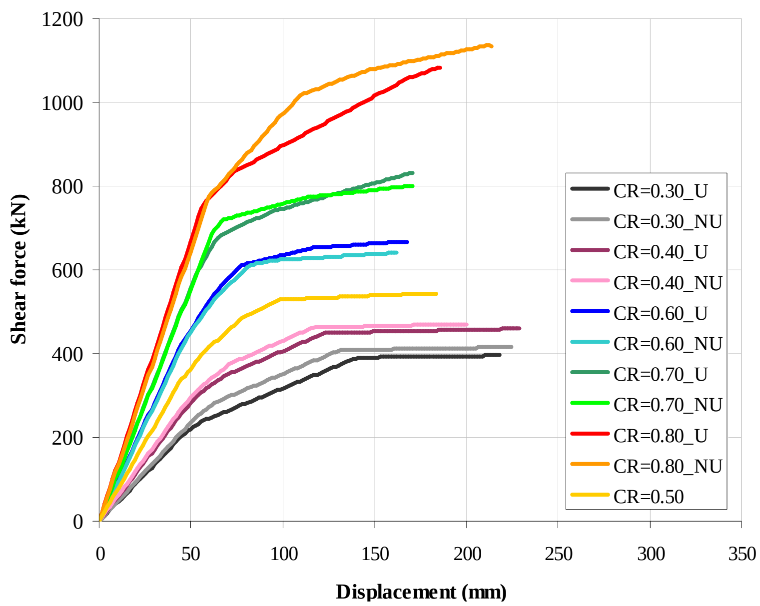

Figure 13: Pushover curves

From the graph, which shows all the pushover curves obtained, is also visible the contribution of the non-uniform distribution in terms of the overall performance of the system.

4 Conclusions

A pre-sizing method for the steel links, which constitute the dissipative elements of HCSW systems, is presented through a “Force-Based” approach. As widely discussed "*force approach*" is not a synonymous for a performance method, widely presented in European and extra-European codes, which allows to develop the project in the elastic range through the use of a R factor, but rather an analytical approach that controls what happens in the elastic range, making predictions on the plastic behaviour of the elements. The method, in fact, could properly be called "*elastic oriented*".

The procedure of preliminary design is based on two basic assumptions: the links, entrusted with the task to dissipate energy, should yield before the RC wall and the deformed shape in the elastic range should be linear. The method is carried out in an iterative procedure, which leads to a solution dependent on the initial stiffness supplied to the system (that is dependent on the section of the first attempt with which the procedure starts) and criteria of choice of solutions obtained are therefore necessary. Moving from purely geometrical criteria (height/width control of the welded profiles) to other based on performance and constructive rules (COV(J) = minimum), we arrive to a selection of solutions considered valid. ù

The structural configurations resulting from these models are subjected to static and dynamic non-linear analysis.

The results of the non linear static analysis (pushover) have subverted the hypotheses developed in the light of the results of the preliminary design in the elastic phase, in fact, it is found that the best solution is the one that shows the maximum values of I and it is characterized by a low coefficient of variation of J, but is not the solution with a coefficient of variation of J minimum.

The justification for this result is that once the link yield the deformed shape of the system is no longer linear but it is cantilever as the deformation of the RC wall becomes predominant. Then the request of plasticity is not constant at various levels, in fact, the best performance is obtained with those profiles that show values of J decreasing with height (lower values on the top floor).

The analysis in the post-elastic range shows that the best solution is the solution generated by the link HE200B first attempt.

With reference to the analysis in the elastic range is noted that the solution generated by a first attempt link HE200B restores sections with decreasing values of J from level 2 to level 4. Values of the modulus of inertia lower on the top floor translate into a greater deformation capacity (expressed as chord rotation). Hence the coherence with the results obtained in the plastic range: once all links yield the deformed system is no longer linear, but the cantilever deformed shape given by the reinforced concrete wall becomes prevalent (when links yield the RC wall is still in its elastic phase).

The plasticity demand, shown as displacement request, will therefore be highest at the top, so solutions in which there are more deformable profiles to the higher storeys show better performance in the plastic range.

Finally, through an incremental dynamic analysis, has been possible to establish a consistency between the static and dynamic behaviour of such kind of systems. The incremental dynamic analysis has allowed to investigate a lot of aspects remained hidden during the pushover analysis, such as the importance of the higher modes of vibration and the dynamic behaviour of the plastic hinges.

Through a non-linear incremental dynamic analysis of the is however possible to estimate a mean value of the strength reduction factor R, which takes values very similar to those obtained with a pushover analysis.

The "Displacement – Based" procedure a reverse approach. This procedure, more properly defined "Plastic Oriented", starting from a boundary condition formed by the reinforced concrete wall at the yield point, designs the links by varying the coupling ratio CR, and therefore their contribution in withstanding seismic action, and shear distribution in height.

Keeping the wall fixed for the entire procedure, as the coupling ratio increases, you get larger and larger sections of the links and then increasingly more resistant systems, especially in light of the results derived from a pushover analysis. A higher performance configurations causes an increase of material (steel) used for dissipative elements. It has been so natural to wonder if increasing the links' section is always convenient. With the aid of the parameters of evaluation of the response in the plastic range already presented in the Chapter 2, important considerations are found. Although it is inevitable that as the size of the link increases the system becomes globally more resistant, however, starting from values of CR = 0.40, the increase in performance is less than proportional to the increase of the material used. For structures with a behaviour similar to those of the case study in question, coupling ratio values lower than CR = 0.50 are preferable.

It 's interesting at this point to bring a final remark on the convergence of these two methods. By controlling the coupling ratio of the solutions selected in Chapter 2, it is found that this value shows small oscillations, standing firmly on values equal to 0.40. CR 0.40 is the coupling ratio optimum value obtained with a reverse procedure proposed in Chapter 3. Opposite approaches, one "Elastic-Oriented", the other one "Plastic-Oriented", developed independently, leading to the same optimal CR values.

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