European Commission

# Innovative hybrid and composite steel-concrete structural solutions for building in seismic area

(INNO-HYCO)

Research and Innovation

EUR 26932 EN

**EUROPEAN COMMISSION**
Directorate-General for Research and Innovation
Directorate D — Key Enabling Technologies
Unit D.4 — Coal and Steel

E-mail: rtd-steel-coal@ec.europa.eu
RTD-PUBLICATIONS@ec.europa.eu

Contact: RFCS Publications

European Commission
B-1049 Brussels

European Commission

# Research Fund for Coal and Steel

# Innovative hybrid and composite steel-concrete structural solutions for building in seismic area
## (INNO-HYCO)


Andrea Dall'Asta, Graziano Leoni, Alessandro Zona
**University of Camerino**
Piazza Cavour, 19F, 62032 Camerino (MC), Italy

Benno Hoffmeister, Hetty Bigelow
**Rheinisch-Westfälische Technische Hochschule (RWTH) Aachen**
Templergraben 55, 52062 Aachen, Germany

Hervé Degée, Catherine Braham, Teodora Bogdan
**University of Liège**
Place du 20 Août 7, 4000 Liège, Belgium

Walter Salvatore, Francesco Morelli
**University of Pisa**
Lungarno Pacinotti, 43, 56126 Pisa, Italy

Prokopis Tsintzos
**Shelter SA**
CHLM Larisas Sykourious 6, 41500 Larisa, Greece

Spyros A. Karamanos, George E. Varelis
**University of Thessaly**
Argonauton & Filellinon, 38221 Volos, Greece

Andrea Galazzi, Ermes Medici, Paolo Boni
**OCAM SRL**
Via Pietro Giusti, 26, 41043 Formigine (MO), Italy



Grant Agreement RFSR-CT-2010-00025
01/07/2010 – 30/06/2013


**Final report**

Directorate-General for Research and Innovation


2015 EUR 26932 EN

More information on the European Union is available on the Internet (http://europa.eu).

Cataloguing data can be found at the end of this publication.

Luxembourg: Publications Office of the European Union, 2015

ISBN 978-92-79-44080-9
doi:10.2777/85404



*Printed in Luxembourg*

PRINTED ON WHITE CHLORINE-FREE PAPER

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# 1. Final summary

WP1 Critical evaluation of existing solutions and new proposals

Task 1.1 Critical evaluation and comparison of European and extra European codes
The objective of this task is the analysis of current European and extra-European codes in order to identify the hybrid systems for which design, detailing, and construction recommendations are already available as well as the most important missing information. It was concluded that Eurocodes provide rather limited information on steel and concrete hybrid seismic resistant structural solutions and extra European codes do not provide much more indications. Thus, the development of design guidelines for steel and concrete hybrid structural systems are a necessary step.

Task 1.2 Overview of technical literature and existing solutions
The objective of this task is the analysis of the state of the art on seismic-resistant hybrid systems studied in the past using numerical and/or experimental methods in order to focus attention on the effectiveness of already proposed solutions as well as their limits and lack of knowledge on design procedures or experimental data. The overview on the technical literature on HCSW systems and relevant existing solutions evidenced that structural steel coupling beams provide a viable solution, that steel coupling beams should be preferably designed to yield in shear, and the importance of the coupling ratio in the overall structural behaviour. The overview on the technical literature on SRCW systems and relevant existing solutions evidenced studies and applications involving steel frames with concrete infill walls, steel shear walls, and composite shear walls, either designed to carry the full seismic shear or as structures designed as a dual system.

Task 1.3 Evaluation of solutions in relation to specific performance of interest in the European area
The objective of this task is the evaluation of the solutions already available in the technical literature by considering both the demand typically expected in seismic prone zones in Europe as well as the more frequently adopted constructive solutions. Different solutions are selected in this research stage based on the limits pointed out in the cases analysed. The analysis of the technical literature highlighted that HCSW systems can achieve high efficiency in resisting lateral loads and dissipating seismic energy if the coupling beams are correctly designed while SRCW systems are suitable for moderate seismic events producing limited cracks in the reinforced concrete wall, unless the wall is replaceable and the boundary system that remains undamaged.

Task 1.4. Evaluation of feasibility and construction complexity of examined solutions
The proposed innovative systems are considered in this task according to their workshop aspects, identifying solutions that enable prefabrication, and analysing relevant critical aspects and costs. Construction aspects including ease of site assembly are considered as well. The analysis of the technical literature shown that workshop aspects in HCSW and SRCW systems do not present major critical aspects and that there are no particular demands and issues concerning their construction.

WP2 Performance analysis of solutions involving innovative HCSWs

Task 2.1 Definition of a reduced set of HCSW systems
The objective of this task is the identification of the most promising structural solutions of innovative HCSW systems and their subsequent selection based on the preliminary work made in WP1. An innovative HCSW system, made by a reinforced concrete shear wall with dissipative steel links and steel side columns was defined in this task. 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.

Task 2.2. Some case studies considered paying attention to the most diffused requirements in the European building market
The objective of this task is the analysis of the most promising innovative HCSW solutions with specific attention to the most common frame configurations such as number of storeys, bay width, interstorey height, as well as to the most common values of the design vertical loads and design ground acceleration. In this task a set of case studies was defined to evaluate the effectiveness and feasibility of the proposed innovative seismic-resisting HCSW system.

Task 2.3 The dimensions of resisting elements assigned by means of preliminary design rules
In this task a first attempt to design the innovative HCSW system was made based on existing results available in current design codes for similar systems. In this way the limits of the available procedure as well as any lacking information are directly evaluated, eventually opening the way to the definition of specific design rules and recommendations. The following design parameters were

considered: number of reinforced concrete walls, thickness and length of reinforced concrete walls, length of the steel links and their connection type (rigid or pinned). It was concluded that increasing the length of the reinforced concrete wall reduces the bending moment in the links and allows decreasing their cross sections, however, the bending moment at the base of the shear wall becomes too high resulting in non-realistic reinforcement schemes. Results showed that a the preferable height-to-length ratio of the wall in order to limit the bending moment at the base is about 10, thus, a relatively slender wall that should work more in flexure than in shear. It was also proposed to use the side steel columns to reduce the length of the steel links and consequently their bending moment demands.

Task 2.4 The seismic behaviour assessed by means of incremental dynamic analyses
The objective of this task is the analysis of the expected seismic behaviour of the innovative HCSW systems by means of numerical analysis in order to evaluate the potentialities of the proposed hybrid structural system as well as the areas of possible improvement in terms of analysis models, design recommendations, and structural details. It was concluded that the proposed HCSW system has interesting potentialities (ductile behaviour with steel links yielding while limited damage occurs in the reinforced concrete wall, the interstorey drifts up to collapse are quite regular regardless of the non-simultaneous yielding in the steel links) and that the preliminary design method requires additional studies to clarify the relationships between wall over-strength and to provide criteria to support the dimensioning of the various elements.

WP3 Performance analysis of solutions involving innovative SRCWs

Task 3.1 Definition of a reduced set of SRCW systems
The objectives of this task is the identification of the most promising structural solutions of innovative SRCW systems and their subsequent selection based on the preliminary work made in WP1. A set of SRCW systems as considered in the Eurocodes was selected for the work to be developed in following tasks. Preliminary analyses were carried out to understand the influence of some parameters, such as the wall width, the concrete strength, the diameter of the stud shear connectors and their spacing, in order to select the most promising solutions.

Task 3.2. Some case studies considered paying attention to the most diffused requirements in the European building market
The objective of this task is the analysis of the most promising innovative SRCW solutions with specific attention to the most common frame configurations such as number of storeys, bay width, interstorey height, as well as to the most common values of the design vertical loads and design ground acceleration. The buildings chosen for the performance analysis of solutions involving SRCWs are the same of Task 2.2 as the gravity-resisting structure is not interested by the lateral-resisting system because pinned connections are considered between beams and columns and between beams and seismic resistant elements.

Task 3.3 The dimensions of resisting elements assigned by means of preliminary design rules.
The objective of this task is the development of a first attempt to design the innovative SRCW system based on existing results available in current design codes for similar systems. In this way the limits of the available procedure as well as any lacking information are directly evaluated, eventually opening the way to the definition of specific design rules and recommendations. It was observed that capacity design rules are not available.

Task 3.4 The seismic behaviour assessed by means of nonlinear analyses
The objective of this task is the analysis of the expected seismic behaviour of the innovative SRCW system by means of numerical analysis in order to evaluate the potentialities of the proposed hybrid structural system as well as the areas of possible improvement in terms of analysis models, design recommendations, and structural details. Results of pushover analyses shown that the designed SRCW systems are very stiff and capable of withstanding high horizontal loads, with limitation of second-order effects and fulfillment of service state criteria easily satisfied. However, it was observed that fragile failure mechanisms occur (problems at beam-to-column connections and crushing of the concrete at a diagonal band), thus, the global behaviour has insufficient ductile to comply with the behaviour factor adopted in the design. This unsatisfactory behaviour is due to the lack of a specific capacity design procedure that allows to control the formation of a proper dissipating mechanism.

WP4 Design of HCSW systems

Task 4.1 Preliminary analyses of the connection typologies
The connection typologies involved in the study developed in this task are: the connections between steel link and reinforced concrete wall; the connection between steel link splices, i.e., part embedded in the concrete wall and replaceable part where yielding and consequent energy

dissipation are developed; the connection between steel link and steel side column. The objective of the analyses of the connection typologies is to identify the most interesting solutions to be experimentally tested in WP6. Two typologies were considered for the connections between steel link and reinforced concrete wall, i.e. in one case the bending moment transferred by the link to the wall is resisted by shear studs, in the other case the moment transferred by the link to the wall is balanced by a couple of vertical reactions provided by the compressed concrete. Two typologies were considered for the splice connection between steel link splices, i.e. connection placed at a distance from the concrete wall sufficient to allow an easy bolting of the replaceable part, connection at the face of the wall with threaded bushings to allow replacement of the dissipative tract of the steel link. The connection between the steel link and the additional steel side column is made by a conventional web angle connection bolted to the column flange.

Task 4.2 Definition of a design procedure
The objective of this task is to develop a comprehensive design procedure for the proposed innovative HCSW system in order to achieve a satisfactory structural behaviour where the seismic energy dissipated in the steel links while the reinforced concrete wall remains undamaged or experience limited damage. The result obtained in WP2 from the preliminary design and subsequent seismic analysis are the base from which this task is developed. A design procedure that inherits recommendation for capacity design from other structural systems involving similar dissipative mechanisms in the links, i.e. eccentric braces in steel frames, as well as indication to reduce damages in the reinforced concrete wall, was defined.

Task 4.3 Evaluation of effectiveness of design provisions proposed
In this task the outcomes of the proposed design procedure for the studied innovative HCSW system are evaluated by means of numerical models. Such models are defined also based on the results of the experimental tests on the connection systems. Nonlinear static and dynamic analyses shown a ductile global behaviour with regular interstorey drifts. However, some limitations of the proposed design approach were highlighted, i.e. lack of an effective control of the sequence of yielding of the links on one side and the reinforcements in the wall on the other side, excessive quantities of required reinforcements in the concrete wall in some cases. This limitations pushed the development of an improved design approach.

Task 4.4 Final assessment of the design procedure
The objective of this task is to assess and refine the proposed design procedure for the innovative HCSW system, based on the critical points and limitations highlighted in the previous task. Given the issues and limits evidenced in the previous task, a new design approach based on limit analysis equilibrium having as key design parameter the coupling ratio, was developed. The HCSW systems designed with such an approach gave quite good results, as assessed through extensive incremental nonlinear static and dynamic analyses, in terms of ductile global behaviour, uniform distribution of interstorey drifts, wall effectively protected against damage as the dissipative steel links yield long before the first yielding in the wall reinforcements, hence starting dissipating seismic energy while the wall is still in its elastic range.

Task 4.5 Evaluation of a design based on the linear dynamic analysis with response spectra and structural reduction factor
The objective of this task is to determine the seismic reduction factor for the proposed innovative HCSW system based on the numerical analyses performed and to evaluate the possibility of using a simplified design method based on linear dynamic analysis as an alternative to the design method proposed in the previous task. Although a structural reduction factor was identified, the linear dynamic analysis provides a distribution of stresses quite different from the distribution of stresses from limit analysis that provided the best design outcomes. Thus, it was concluded that a design based on linear dynamic analysis with response spectra and structural reduction factor currently cannot provide satisfactory results and more studies are required.

WP5 Design of SRCW systems

Task 5.1 Preliminary analysis of the wall morphology and connection typology
In this task the study on the infill wall morphologies as well as the connection typologies constitute the basis for the design of the experimental tests to be performed in WP7. After that preliminary pushover analyses on SRCW systems designed according to Eurocode 8 highlighted critical issues, an innovative SRCW system was proposed. The morphology and connection detailing of the proposed innovative system were accordingly studied to ensure the desired structural behaviour.

Task 5.2 Definition of a design procedure
The objective of this task is to develop a comprehensive design procedure for the proposed innovative SRCW system in order to optimize its global ductility and allow replacement of the dissipative elements damaged after a major seismic event. The result obtained in WP3 from the

preliminary design and subsequent seismic analysis are the base from which this task is developed. A design procedure was developed considering the latticed statically determined scheme representing the limit behaviour of the for the proposed innovative SRCW system. It is concluded that the procedure is straightforward but attention should be given to the fact that the limit structural scheme adopted may not represent the behaviour of the system especially in the linear range for weak earthquakes.

Task 5.3 Evaluation of effectiveness of design provisions proposed
In this task the outcomes of the proposed design procedure for the studied innovative SRCW system were evaluated by means of numerical models. Such models are defined also based on the results of the experimental tests. Static nonlinear analyses demonstrated that the yielding pattern is characterized by plastic strains only at the ductile elements, according to the hierarchy principles involved in the design methodology, and show struts developed in the concrete panels. It was observed that simplified models based on the design scheme provide excellent approximation of the results of advanced nonlinear finite element models.

Task 5.4 Final assessment of the design procedure
In this task the proposed design procedure for the innovative SRCW system was refined based on the critical points and limitations highlighted in the previous task. A final assessment of the refined design procedure was performed. The numerical analyses for the designed SRCW systems carried out with refined finite element models highlighted that the proposed rules for capacity design are able to ensure the formation of the plastic mechanism involving only the lateral vertical elements while preserving the wall from crushing.

Task 5.5 Evaluation of a design based on the linear dynamic analysis with response spectra and structural reduction factor
The objective of this task is to determine the seismic reduction factor for the proposed innovative SRCW system based on the numerical analyses performed and to evaluate the possibility of using a simplified design method based on linear dynamic analysis as an alternative to the design method proposed in the previous task. The results on the determination of the reduction factors derived from the cases designed were not fully satisfactory as the computed factors are lower than those adopted in the design in the cases of non-regular systems.

WP6 Experimental based models for HCSWs

Task 6.1. Design of the experimental campaign
In this task the specimens were designed to characterize the performance of the connection of the seismic link embedded in a concrete shear wall and the efficiency of the capacity design of such a system. Additional tests were designed in order to evaluate the ultimate capacity of the link-to-wall connection.

Task 6.2. Supply of raw material and building of specimens
The supply of the material and building specimens was made by OCAM as scheduled, also including the planned additional specimens.

Task 6.3. Displacement controlled tests under cyclic path with increasing amplitude
The objective of this task is to perform the designed experimental tests in order to evaluate the cyclic behaviour of the dissipative elements as well as the behaviour of their connection systems. The experimental results shown that the clearance between bolts and holes in the link-to-column connection has a negative influence of the cyclic behaviour of the tested steel links, with pronounced pinching phenomena due to the relative displacements between the connected elements. Improvements in the link-to-column connection made for the second connection typology reduced the relative displacements between the connected elements, producing hysteresis cycles considerably fatter and with pinching phenomena reduced. Regarding the connection between the link and the wall, the monotonic tests highlighted very good performances with a basically linear global behaviour up to the maximum applicable load.

Task 6.4. Constitutive models of local behaviour based on experimental tests
The objective of this task is to develop suitable models to describe the structural behaviour of the dissipative elements and relevant connection systems to be used in the analyses required in WP4 and WP8. It was concluded that a simple frame model with un dimensional elastoplastic spring is adequate to model the behaviour of the steel link, provided that the actual value of the yield stress is identified.

WP7 Experimental based models for SRCWs

Task 7.1 Design of the experimental campaign

The objective of this task is the design of the specimens to be tested. Such specimens were defined and designed according to the indications and results of the preliminary study made in WP5.

Task 7.2 Supply of raw material and building of specimens
The objective of this task is to build the specimens designed in the previous task. The supply of the material and building specimens was made by OCAM as scheduled.

Task 7.3 Experimental tests on the connections and steel side elements
The objective of this task is to perform the designed experimental tests on the connection and side steel elements in order to evaluate their behaviour. The experimental activity on the shear connections demonstrated that their expected strength was exceeded and failure occurred in the studs (shearing of their shank) while no concrete crushing was observed. The shear connection did not withstand the prescribed number of cycles at the loading level equal to 75% of their monotonic experimental strength, as requested for shear connection in seismic resistant elements, whereas the specimens withstood the cyclic loading carried out with the maximum amplitude equal to 50% of their monotonic experimental strength. The experiments carried out on the side steel elements demonstrated their capability in withstanding important deformations with the formation of the plastic hinge under combined axial-bending actions. This condition is necessary to ensure the formation of the ultimate resisting mechanism assumed for the dissipative elements that are not actually pinned at their ends.

Task 7.4 Experimental tests on the downscaled specimen of the structural system
The objective of this task is to perform the designed experimental tests on the downscaled SRCW system in order to evaluate its behaviour. The experiments performed on the downscaled specimen demonstrated that the plastic mechanism assumed in the design procedure (formation of the diagonal strut and plasticization of the side elements) can actually develop.

Task 7.5 Development of constitutive models based on experimental tests
The objective of this task is to develop suitable models to describe the local and global structural behaviour of the developed innovative SRCW system to be used in the analyses required in WP5 and WP8. The results obtained on the downscaled specimen were used for the evaluation and calibration of mechanical model that was specifically developed for the proposed innovative SRCW system.

WP8 Case studies

Task 8.1 Definition of functional and dimensional characteristics of the case study
Two case studies were selected in this tasks, considering the most interesting functional distribution for the European market and deciding the structural properties on the basis of results from previous WPs. The functional distribution were defined with the support by OCAM and SHELTER and it involves parking at the basement, shops and storerooms at the ground floor and residences at the upper five floors. The planned two case studies have similar architectonical details and structural gravity load systems but they are different for what concerns the structural seismic resistant system, HCSW in one case and SRCW in the other case. It is assumed that buildings are located in a medium-high seismic area in Italy.

Task 8.2 Development of a complete design of the case study A (HSCW)
The development of a complete design of the case study A (HCSW) was made. The design of the gravity load system was developed coherently with loads related to the functional distribution and architectonical solutions have been designed to provide acoustic and thermal comforts. This design path, starting from preliminary analyses for the choice of the most important parameters and leading to final safety verification and design of the details, was developed by using the procedure defined in WP4 and the analysis models coming from WP6.
The design method demonstrated to be effective in leading to effective solutions and the detailed description of the procedure, including suggestions, is reported on the delivery 8.2 that provides a useful guidelines for designers. Drawings and technical documents (material amount, cost analyses, timetable, construction sequences) also provide effective solutions for the structural/architectonical details concerning the seismic resistant system and useful information about the global costs and the construction process.

Task 8.3 Development of a complete design of the case study B (SRCW)
The development of a complete design of the case study B (SRCW) was made. The design of the gravity load system and architectonical solutions are the same of the previous case. This design path, starting from preliminary analyses for the choice of the most important parameters and leading to final safety verification and design of the details, was developed by using the procedure defined in WP5 and the analysis models coming from WP7. The design method demonstrated to be effective in leading to effective solutions and the detailed description of the procedure, including

suggestions, is reported on the delivery 8.2 that provides a useful guidelines for designers. Drawings and technical documents (material amount, cost analyses, timetable, construction sequences) also provide effective solutions for the structural/architectonical details concerning the HSCW and SRCW systems and useful information about the global costs and the construction process.

Task 8.4 Final evaluation of the performance and Workshop
In this task the final evaluation of the two proposed structural systems was made by extending the discussion to steel designers and construction companies. The case studies and the outcomes of the research were discussed both in the final workshop in Turin and in a preliminary presentation in Salerno (not strictly due by the contract, supported by OCAM). The final workshop was held in a session of a conference on steel construction (CTA) in order to have a large audience (366 participants) from research centres, designer and construction companies. During the final conference and meetings the evaluation of the proposed solutions lead to a number of final conclusions concerning the structural safety, the design and construction process, the consumption of row material and the feasibility for buildings with different use, as parking, stores and apartments. The main conclusions follow.
Design. The methods proposed confirmed their feasibility in real cases and their simple application.
Structural safety. The structural safety and the damage level after an earthquake in the case studies confirmed the local details designed as in WP4/WP5. As a related result, the current provisions of EC8 seems to be not adequate to provide the capacity and the structural behaviour factor suggested.
Construction. The final computation of row materials quantities and operative costs confirmed that construction costs enjoy some general benefits from a rational use of materials with respect to steel bracings systems or RC wall systems: reinforced concrete is economically used to provide required stiffness and shear strength, steel is used for dissipative components (replaceable after seismic events). Architectural and structural details proposed in the case studies were judged as economically effective by the construction companies OCAM and SHELTER SA.
Functional and architectonic issues. Both the HSCW and SRCW systems have an high shear strength capacity and this permits to reduce the number of seismic resistant elements, making more flexible the distribution of internal space at the different floors and reducing the intersection with plant piping.

## 2. <u>Scientific and technical description of the results</u>

### *2.1 Objectives of the project*

Composite structures are widely used in building construction in seismic areas whereas hybrid solutions are less diffused and require further investigation. Contrary to commonly used composite structures, in which the deformation demands for steel and concrete components are in the same range (concrete and steel act as one member), hybrid structures allow for a more efficient design of concrete and of steel components with deformation demands tailored to the capacity of the materials. This project aims to define innovative steel reinforced concrete hybrid systems for the construction of feasible and easy repairable earthquake-proof buildings exploiting the best properties of both steel and concrete construction techniques by combining them in the most efficient way possible. Hybrid coupled shear walls (HCSW) and steel frames with reinforced concrete infill walls (SRCW) are considered. The achievement of the overall objectives is articulated in 8 WPs as hereafter described.

WP1 Critical evaluation of existing solutions and new proposals

Task 1.1 Critical evaluation and comparison of European and extra European codes
This task involves the analysis of current European and extra-European codes in order to identify the hybrid systems for which design, detailing, and construction recommendations are already available as well as the most important missing information.

Task 1.2 Overview of technical literature and existing solutions
This task involves the analysis of the state of the art on seismic-resistant hybrid systems studied in the past using numerical and/or experimental methods in order to focus attention on the effectiveness of already proposed solutions as well as their limits and lack of knowledge on design procedures or experimental data.

Task 1.3 Evaluation of solutions in relation to specific performance of interest in the European area
In this task the solutions already available in the technical literature are evaluated by considering both the demand typically expected in seismic prone zones in Europe as well as the more frequently adopted constructive solutions. Different solutions are selected in this research stage based on the limits pointed out in the cases analysed.

Task 1.4. Evaluation of feasibility and construction complexity of examined solutions
The proposed innovative systems are considered in this task according to their workshop aspects, identifying solutions that enable prefabrication, and analysing relevant critical aspects and costs. Afterwards, construction aspects including ease of site assembly will be considered.

WP2 Performance analysis of solutions involving innovative HCSWs

Task 2.1 Definition of a reduced set of HCSW systems
In this task the most promising structural solutions of innovative HCSW systems are identified and selected based on the preliminary work made in WP1.

Task 2.2. Some case studies considered paying attention to the most diffused requirements in the European building market
In this task the most promising innovative HCSW solutions are analysed with specific attention to the most common frame configurations such as number of storeys, bay width, interstorey height, as well as to the most common values of the design vertical loads and design ground acceleration.

Task 2.3 The dimensions of resisting elements assigned by means of preliminary design rules
In this task a first attempt to design the innovative HCSW system is made based on existing results available in current design codes for similar systems. In this way the limits of the available procedure as well as any lacking information are directly evaluated, eventually opening the way to the definition of specific design rules and recommendations.

Task 2.4 The seismic behaviour assessed by means of incremental dynamic analyses
In this task the expected seismic behaviour of the innovative HCSW system is analysed by means of numerical analysis in order to evaluate the potentialities of the proposed hybrid structural system as well as the areas of possible improvement in terms of analysis models, design recommendations, and structural details.

WP3 Performance analysis of solutions involving innovative SRCWs

Task 3.1 Definition of a reduced set of SRCW systems

In this task the most promising structural solutions of innovative SRCW systems are identified and selected based on the preliminary work made in WP1.

Task 3.2. Some case studies considered paying attention to the most diffused requirements in the European building market
In this task the most promising innovative SRCW solutions are analysed with specific attention to the most common frame configurations such as number of storeys, bay width, interstorey height, as well as to the most common values of the design vertical loads and design ground acceleration.

Task 3.3 The dimensions of resisting elements assigned by means of preliminary design rules
In this task a first attempt to design the innovative SRCW system is made based on existing results available in current design codes for similar systems. In this way the limits of the available procedure as well as any lacking information are directly evaluated, eventually opening the way to the definition of specific design rules and recommendations.

Task 3.4 The seismic behaviour assessed by means of nonlinear analyses
In this task the expected seismic behaviour of the innovative SRCW system is analysed by means of numerical analysis in order to evaluate the potentialities of the proposed hybrid structural system as well as the areas of possible improvement in terms of analysis models, design recommendations, and structural details.

WP4 Design of HCSW systems

Task 4.1 Preliminary analyses of the connection typologies
The connection typologies involved in the study developed in this task are: the connections between steel link and reinforced concrete wall; the connection between steel link splices, i.e., part embedded in the concrete wall and replaceable part where yielding and consequent energy dissipation are developed; the connection between steel link and steel side column. The objective of the analyses of the connection typologies is to identify the most interesting solutions to be experimentally tested in WP6.

Task 4.2 Definition of a design procedure
The objective of this task is to develop a comprehensive design procedure for the proposed innovative HCSW system in order to achieve a satisfactory structural behaviour where the seismic energy dissipated in the steel links while the reinforced concrete wall remains undamaged or experience limited damage. The result obtained in WP2 from the preliminary design and subsequent seismic analysis are the base from which this task is developed.

Task 4.3 Evaluation of effectiveness of design provisions proposed
In this task the outcomes of the proposed design procedure for the studied innovative HCSW system are evaluated by means of numerical models. Such models are defined also based on the results of the experimental tests on the connection systems.

Task 4.4 Final assessment of the design procedure
In this task the proposed design procedure for the innovative HCSW system is refined based on the critical points and limitations highlighted in the previous task. A final assessment of the refined design procedure performed.

Task 4.5 Evaluation of a design based on the linear dynamic analysis with response spectra and structural reduction factor
The objective of this task is to determine the seismic reduction factor for the proposed innovative HCSW system based on the numerical analyses performed and to evaluate the possibility of using a simplified design method based on linear dynamic analysis as an alternative to the design method proposed in the previous task.

WP5 Design of SRCW systems

Task 5.1 Preliminary analysis of the wall morphology and connection typology
In this task the study on the infill wall morphologies as well as the connection typologies constitute the basis for the design of the experimental tests to be performed in WP7.

Task 5.2 Definition of a design procedure
The objective of this task is to develop a comprehensive design procedure for the proposed innovative SRCW system in order to optimize its global ductility and allow replacement of the dissipative elements damaged after a major seismic event. The result obtained in WP3 from the preliminary design and subsequent seismic analysis are the base from which this task is developed.

Task 5.3 Evaluation of effectiveness of design provisions proposed
In this task the outcomes of the proposed design procedure for the studied innovative SRCW system are evaluated by means of numerical models. Such models are defined also based on the results of the experimental tests.

Task 5.4 Final assessment of the design procedure
In this task the proposed design procedure for the innovative SRCW system is refined based on the critical points and limitations highlighted in the previous task. A final assessment of the refined design procedure performed.

Task 5.5 Evaluation of a design based on the linear dynamic analysis with response spectra and structural reduction factor
The objective of this task is to determine the seismic reduction factor for the proposed innovative SRCW system based on the numerical analyses performed and to evaluate the possibility of using a simplified design method based on linear dynamic analysis as an alternative to the design method proposed in the previous task.

WP6 Experimental based models for HCSWs

Task 6.1 Design of the experimental campaign
The specimens to be tested are defined and designed on the basis of the preliminary study in WP4.

Task 6.2 Supply of raw material and building of specimens
The objective of this task is to build the specimens designed in the previous task.

Task 6.3 Experimental tests on the links and their connections
The objective of this task is to perform the designed experimental tests in order to evaluate the cyclic behaviour of the dissipative elements as well as the behaviour of their connection systems.

Task 6.4 Development of constitutive models based on experimental tests
The objective of this task is to develop suitable models to describe the structural behaviour of the dissipative elements and relevant connection systems to be used in the analyses in WP4 and WP8.

WP7 Experimental based models for SRCWs

Task 7.1 Design of the experimental campaign
The specimens to be tested are defined and designed on the basis of the preliminary study in WP5.

Task 7.2 Supply of raw material and building of specimens
The objective of this task is to build the specimens designed in the previous task.

Task 7.3 Experimental tests on the connections and steel side elements
The objective of this task is to perform the designed experimental tests on the connection and side steel elements in order to evaluate their behaviour.

Task 7.4 Experimental tests on the downscaled specimen of the structural system
The objective of this task is to perform the designed experimental tests on the downscaled SRCW system in order to evaluate its behaviour.

Task 7.5 Development of constitutive models based on experimental tests
The objective of this task is to develop suitable models to describe the local and global structural behaviour of the developed innovative SRCW system to be used in the analyses required in WP5 and WP8.

WP8 Case studies

Task 8.1 Definition of functional and dimensional characteristics of the case study
The task requires the definition of two case studies by considering the most diffused and interesting functional distributions while general structural characteristics and the seismic input must be selected on the basis of previous parametric analyses (WP4/WP5).

Task 8.2 Development of a complete design of the case study A (HSCW)
A complete design of a solution with the HCSW seismic resistant system has to be provided by considering architectonical issues, acoustic and thermal insulation requirements, solutions for structural details, construction time and sequences, costs. The work is oriented to a global evaluation of the solution effectiveness and to a final assessment of the design procedure.

Task 8.3 Development of a complete design of the case study B (SRCW)
The work due in this task has the same objective of the previous task 8.2 (case study complete design, final evaluation and final assessment of the design procedure) but it involves the SRCW seismic resistant system studied in WP5 and WP7.

Task 8.4 Final evaluation of the performance and Workshop
The objective of this task consists in the organization of a final workshop for the discussion of the research outcomes, with a special attention to the complete designs developed for the case studies.

## *2.2 Description of the activities and discussion*

WP1 Critical evaluation of existing solutions and new proposals

Task 1.1. Critical evaluation and comparison of European and extra European codes in order to point out lack of information and provisions.

The evaluation of the European codes showed that in the current European situation SRCW and HCWS systems are allowed by Eurocode 8 (EN 1998-1) but not extensively documented. Design principles are mainly based on similar situations considered in other sections of Eurocode 8 (concrete walls and conventional composite action). Composite structural systems are addressed in sections 7.3.1 (e), 7.3.2 and 7.10 of Eurocode 8. Three configurations are depicted in Figures 7.1 and 7.2 of Eurocode 8 and here reported in Figure 1. The information provided is rather limited and deals essentially with: definition of behaviour factor q values; design principles ("P" clauses according to Eurocodes nomenclature) to be particularized for each specific design situations; suggestions for performing the structural analysis, with reference to sections 5, 7.4 and 7.7; a limited set of detailing rules.

a) b)

**Figure 7.1: Composite structural systems. Composite walls: a) Type 1 – steel or composite moment frame with connected concrete infill panels; b) Type 2 – composite walls reinforced by connected encased vertical steel sections.**

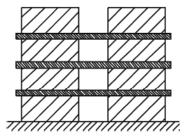


**Figure 7.2: Composite structural systems. Type 3 - composite or concrete walls coupled by steel or composite beams.**

**Figure 1.** Composite structural systems considered in Eurocode 8.

The situation is not much different in extra European codes. In Table 1 a list of the design codes analysed is given with comments underlining to what extent the structural typologies more similar to those considered in this project are taken into consideration.

**Table 1.** Overview of extra European codes.

| **CODE** | **Comments** |
|---|---|
| UBC 1997 | - Nothing detailed on HCSW or SRCW |
| AISC 341-05 | - Detailed procedure for plated shear walls<br>- prescriptions for the design of coupling beams |
| AISC 360-05 | - Stability of shear wall resisting systems |
| ACI 318 | - Design of concrete shear walls and concrete coupling beams |
| NEHRP 1997 | - Based on ACI |
| Canadian code | - Concrete shear walls and coupling beams<br>- Ducility requirements<br>- Precast concrete shear walls |
| New Zealand code | - Similar to ACI 318 |
| Mexico seismic code | - Nothing detailed on HCSW or SRCW |
| Indian building code | - Nothing detailed on HCSW or SRCW |
| UDC (China) | - General recommendations on positioning and geometric limitation of shear walls<br>- No reference to steel or composite systems |

Task 1.2. Overview of technical literature and existing solutions with focus on their effectiveness and limits and pointing out lack of knowledge about design procedures or experimental data. Innovative solutions and optimization of existing solutions will be also provided.

Existing solutions of SRCW systems documented in the technical literature include: steel frames with concrete infill walls; steel frames with steel shear walls; steel frames with composite shear walls. In steel frames with concrete infill walls two design options are possible: infill walls designed to carry the full seismic shear; structures designed as a dual system, i.e., shear walls inside or in parallel with a moment frame. The steel frame can be designed with fully or partially restrained connections with the concrete walls connected to the steel frame by classical shear stud connectors. The structural system is identified in the technical literature as particularly suitable for moderate seismicity. From a constructive point of view steel frames considered in this solution can be built as usual while the critical point is the steel-concrete interface zone. Part of the connecting elements can be prefabricated. The required confining steel in the interface zones does not lead to important congestions. An important part of the concreting job has to be carried out on site. A potential alternative and improvement can be obtained substituting concrete walls with steel walls. Steel frames with steel shear walls exhibit very good performances and the ductility level is very high and the cyclic behaviour is stable. Many lab tests validated the system and structures built according to these principles showed good performances during real strong earthquake. The frame-to-infill connection can be either welded or bolted, with welded connections identified as mechanically more efficient but more time-consuming in the execution stage. The system allows for a high level of prefabrication. Another viable alternative is the use of steel frames with composite shear walls. Composite shear walls are introduced to limit the infill slenderness. The system is mainly suitable for high seismic loads in high rise buildings and was validated experimentally. However, this composite system appears very expensive and combines drawbacks of concrete and steel infill walls.

The technical literature dealing with HCSW systems evidences that coupling beams should be preferably designed to yield in shear and the importance of the coupling ratio, i.e., portion of the system overturning moment resisted by the coupling action, although it comes out that a successful design can be reached for a wide range of coupling ratio. Different types of coupling beams are possible: steel, concrete or composite. Recently proposed alternative solutions include post-tensioned systems. Pure steel coupling beams allow for an efficient fuse-type design of the link based on the link design procedures developed for eccentrically braced steel frames. In addition structural steel coupling beams provide a viable alternative to reinforced concrete coupling beams, particularly where height restrictions do not permit the use of deep reinforced concrete or composite coupling beams, or where the required capacity and stiffness cannot be developed economically by conventional reinforced concrete coupling beams. Composite beams are more complicated to set up on site but the extra-cost is felt compensated by the increased performances. Post-tensioning solutions look useful mainly for retrofitting. Regarding the arrangement of the HCSW systems in the buildings, two structural configurations can be found: coupled shear walls located in the perimeter area of the building and coupled core walls. These structures are typically designed for moderate to high seismicity and a number of practical realisations were identified.

Task 1.3. Evaluation of solutions in relation to specific performance of interest in the European area by considering both seismic demand and the more frequently adopted constructive solutions.

SRCW systems are particularly suitable for moderate seismic events when the energy is dissipated by concrete cracking, as those cracks are easily repairable with epoxy. More severe seismic events lead to more severe cracks that could be more difficult or impossible to repair. Nevertheless the concrete wall is replaceable as long as the boundary system remains undamaged.

HCSW systems can achieve high efficiency in resisting lateral loads and dissipating seismic energy if the coupling beams are correctly designed, i.e., adequate stiffness and capable of stable hysteresis, yielding attained in the coupling beams before damage in the wall piers. Well-proportioned coupling beams above the second floor generally develop plastic hinges simultaneously, and are subjected to similar end rotations over the height of the structure. Hence, dissipation of input energy can be distributed over the height of the building. Previous researches showed that the lateral stiffness and strength of concrete shear walls can be significantly increased by coupling the shear walls using embedded steel beams.

Task 1.4. Evaluation of feasibility and construction complexity of examined solutions.

Similarly, workshop aspects in HCSW systems do not present major critical aspects. No contraindications are found to any particular aspects of the structural detail of the node beam to column or the node beam to girder that could affect the proper and functional construction of the steel structures. The connection of the steel beams to the walls does not seem difficult to prepare on the workshop location. It is therefore felt that the attention should concentrate on the issues related to the process of site assembly. The system is not complex in terms of creation and construction in a workshop. Junction nodes can be used under either partial or complete restoration of the resistance. Bolted joints facilitate the construction in the workshop if they are claimed to be appropriate, which happens when the number of bolts per quantity and type results applicable on the junction. The disadvantages of using bolted joints pairs are highlighted on the site, since they make assembly operations complex, because the system does not consent to foresee the compensation of the tolerances resulting from the coupling system. The small tolerances, however, can be moved to the pairs of beams on the shear walls. In this solution, specific issues regarding the characteristics of the steels of standard media and resilience of ordinary strength, or standard steel production, do not emerge.

Workshop aspects in SRCW systems do not present major critical aspects. The steel frames can be made with either from welded or bolted joints. In order to guarantee an efficient lateral stability, the concrete panel cladding should be appropriately linked with connecting elements which makes the transfer of forces possible. The interface between the steel structure and the cladding is definitely the critical area. Solutions that enable a complete fabrication of the system can be identified, introducing threaded rods protruding from the rear-end collisions. Welding joints can also be used in the pipeline. The steel component that is particularly stressed is represented by the bracket that supports the buffering system of the precast concrete panel. The bond interpretation and the subsequent realization do not result particularly burdensome and expensive for the workshop.

In the construction of HCSW systems, the steel coupling beams have the benefit that they can be constructed (built-up sections) in a steel workshop and placed in the work site quite easily. Since they are made of steel, there is no need for additional concrete casting for the coupling beams in the working site. Their erection and exact positioning can be handled with typical erecting machines and trained working crew. These benefits result to increased feasibility of the solution and improvement of the total construction type of such a structure. The main drawback is that the safe connection of the coupling beams with the shear walls requires an increased anchorage length which depends on the coupling ratio. The embedment length can reach up to 1 m. This means that the local width is going to be greater than the middle wall width for an increased portion of the total wall length. Finally, the insertion of the steel coupling beam in the shear wall results to the misconnection of the edge hoop reinforcement that usually has spacing in the vertical direction less than the coupling beam height. The local hoop reinforcement has to be modified in two smaller closed hoops. In addition, the longitudinal transfer bars have to be connected to the steel coupling beam by drilling and welds, which increases the fabrication cost and complicates the erection procedure.

In the construction of SRCW systems there are no particular demands concerning the steel frame design and construction. Conventional and widely used steel parts and connection elements can be used which makes the proposed solution attractive for application. The only time consuming process is the proper arrangement and connection of the shear studs through welding to the steel frame. This procedure can be realized with conventional welding equipment and it can be part of the prefabrication of the steel parts to improve the construction time effectiveness. It has been experimentally illustrated that the interface area between the steel frame and the reinforced concrete infill panel is critical and vulnerable to cracking and failure. The existence of a local reinforcement confining cage that helps the shear studs to develop all their resistance capacity and prevents early concrete cracking introduces minor difficulties from the construction point of view. Overall, the required steel reinforcing is not causing any congestion problems that would decrease the construction feasibility. The total steel reinforcement arrangement and the concrete wall casting have to be realized in the construction place. This is more time consuming compared to a steel shear wall or a prefabricated composite shear wall which are both existing alternatives. Finally, the infill concrete wall can be easily repaired or replaced after being damaged resulting to a quick and effective rehabilitation of the building.

WP2 Performance analysis of solutions involving innovative HCSWs

Task 2.1. Definition of a reduced set of systems as a consequence of a plenary discussion about RC-steel coupled systems.

The innovative HCSW system that the partners found the most promising and agreed to study is made by a reinforced concrete shear wall with steel links, as depicted in Figure 2. 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 (Figure 3).

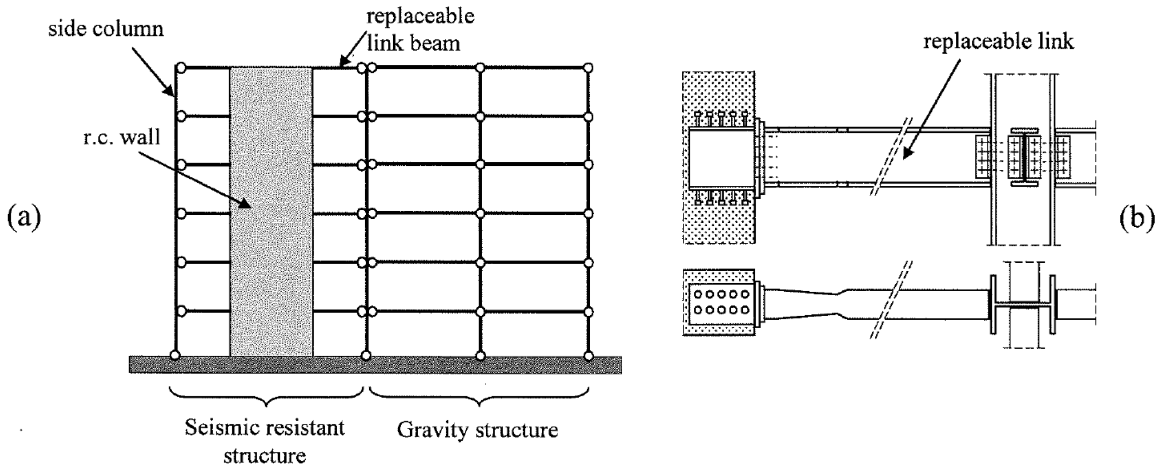



**Figure 2.** (a) Scheme of the proposed innovative system connected to a gravity-resisting steel frame with pinned beam-to-column joints, (b) example of replaceable steel link.

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.

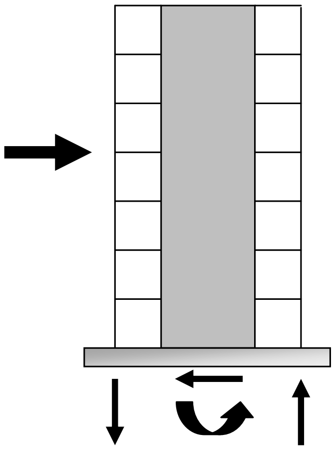


**Figure 3.** Resisting mechanism of the proposed innovative system.

Task 2.2. Some case studies will be considered paying attention to the most diffused requirements for building in the European market (number of storeys, bays width, inter-storey heights). The choices will be calibrated in order to be effective for the structural systems considered in this task.

After a preliminary overview, the following case studies were defined. Floor geometry is indicated in Figure 4 with the suggested position of the bracing systems marked in green. The assigned vertical loads are a permanent load Gk = 6.5 kN/m2 and a variable load Qk = 3.0 kN/m2, the seismic mass for each floor is 1200 kNs2/m. The gravity-resisting structure is made of pinned beam-to-column joints (non-moment-resisting frame) with storey height 3.40 m. Columns are continuous and pinned at the base, beams are IPE500. Two cases are considered: 4-floor and 8-floor frames.
The design of the innovative seismic-resisting structures is made for the most promising typology selected as described above. Initially only one selected spectrum representative of a medium-intensity event (EC8, ag = 0.25g, type 1 spectra, ground type C) in order to focus attention on design issues rather than seismic input variability. No-collapse is required for the reference spectrum (ULS). Damage limitation is required for a design input obtained by reducing the reference spectrum by 2.5 (DLS). Interstorey drift allowed equal to 0.005h.

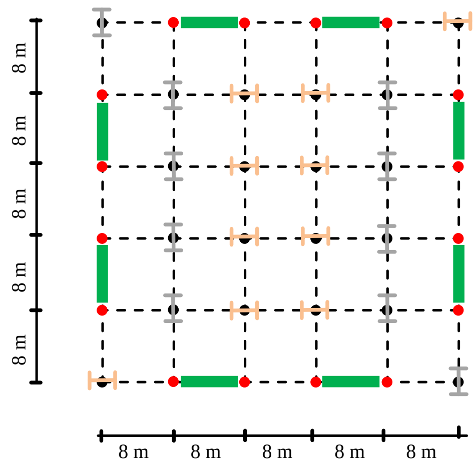



**Figure 4.** Floor geometry with positions of the HCSWs for the considered case studies.

Task 2.3. The dimensions of resisting elements will be assigned by means of preliminary, already known, design rules. When a structural reduction factor q is available or it can be estimated from structures with similar post-elastic mechanisms, the design will be based on the linear analysis and the response spectra proposed in the Eurocede 8. Otherwise, the design will be based on static nonlinear analyses.

The preliminary design of the innovative HCSW system that the partners found the most promising was based on linear static analysis with the base shear force calculated in accordance with Eurocode 8 using an approximation of the value of the fundamental period of the building. A structural behaviour factor q = 3.3 was assumed because that value is given in Eurocode 8 for composite structural systems type 3 (composite walls coupled by steel beams, EC8 paragraph 7.3.2, ductility class DCM), the configuration closest to that of the innovative solution here studied. The first preliminary study undertaken was a feasibility analysis considering the following parameters: number of reinforced concrete walls; thickness and length of reinforced concrete walls; length of the steel links and their connection type (rigid or pinned). It was found that four HCSW systems for each direction is preferable to only two HCSW systems for each direction. It was also found that increasing the length of the reinforced concrete wall reduces the bending moment in the links and allows decreasing their cross sections, however, the bending moment at the base of the shear wall becomes too high resulting in non-realistic reinforcement schemes. Thus, the preliminary design procedure was based on a first step consisting in varying the length of the wall and the cross sections of the links in order to get acceptable bending moments in the links and at the base of the shear walls. Results showed that a the preferable height-to-length ratio of the wall in order to limit the bending moment at the base is about 10, thus, a relatively slender wall that should work more in flexure than in shear. Reducing the length of the reinforced concrete walls results in long links if the entire bay span has to be adopted for the HCSW system. However, this solution did not appear as optimal, as increasing the length of the steel links results in increasing the demand in the link bending resistance. Therefore it is proposed to implement additional columns on both sides of the wall in order to reduce the length of the steel links and consequently their bending moment demands. The additional steel columns are continuous and pinned at the base; the steel links are pinned to these columns and rigidly connected to the reinforced concrete wall. The position of the additional columns and hence the length of the steel links was varied and the obtained cross sections for the steel link designed. It was checked that: bending moments in the links are lower than the plastic resistance; shear force in the links do not exceed the plastic shear resistance; bending moment at the base of the reinforced concrete wall can be resisted with a realistic reinforcement scheme.

Task 2.4. The seismic behaviour will be more precisely assessed by means of Incremental Dynamic Analyses (IDAs) in order to obtain an overview of the structural performance at different levels of seismic intensity. At this stage, synthetic accelerograms derived from the spectra proposed in Eurocode will be considered. The results will provide a preliminary information about the effectiveness of rules adopted in the preliminary design of Task 2.3 and they should highlight critical aspects related to ductility connection demands and ductility/stiffness demand at different storeys.

The seismic behaviour of the most promising HCSW systems designed in the previous task was assessed through multi-record nonlinear incremental dynamic analysis (IDA). Preliminary nonlinear static analysis under applied lateral loads (pushover analysis) were also run. The reinforced concrete shear walls were modelled using linear elastic frame elements (axial, flexural and shear deformability) with flexural elastic stiffness as obtained from the initial slope of the nonlinear moment curvature relation of their cross section with concrete confinement included according to Eurocode 2. The nonlinear moment curvature relations were also used to define the relevant idealized moment rotation curves of the nonlinear hinge located at the base of the wall. Mean values of the material properties were used for the concrete and reinforcements. The steel shear links were modelled using linear elastic frame elements (axial, flexural and shear deformability) with nonlinear flexural hinges introduced in each link at the end clamped to the shear wall (point of maximum bending moment) as well as shear hinges introduced at mid span of each steel link (shear force constant along the link). Geometric nonlinearity was included as well as a preliminary evaluation found that geometric effects result in non-negligible reductions of the load bearing capacity. The obtained pushover curves (shear base versus top displacement) are shown in Figure 5. In the 4-storey case it is observed that reinforcements in the concrete wall yield together with yielding in bending of the steel links in the first three storeys. Afterwards, further plastic dissipation is fostered by the successive yielding in bending of the links at the last storey. Then the peak strength of the reinforced concrete wall is achieved leading to the maximum sustained base shear before leading the bracing system to failure due to failure of the reinforced concrete wall. A different seismic performance is observed in the 8-storey design, where the steel links at all storeys yield in bending before any damage in the reinforced concrete wall. Afterwards reinforcements yields and shortly afterwards all links yield in shear. The concrete peak strain is attained but the bracing system is still able to exhibit global hardening. Collapse is reached when the link at the fifth storey fails in combined banding and shear. Despite these differences, pushover verifications are satisfied (capacity displacement larger than target displacement) in both case studies.

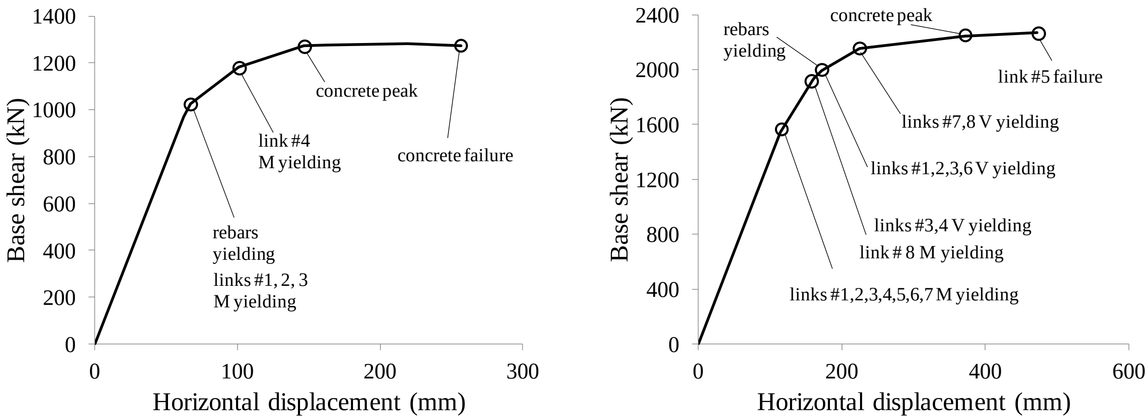



**Figure 5.** Pushover curves for the 4-storey case (left) and 8-storey case (right)

The IDAs were performed according to the "Protocol for the execution of the incremental dynamic analyses" of WP2. Seven artificial accelerograms were used as seismic input (selection and signal baseline correction in WP3), scaled from the scale factor (SF) 0.3 up to 1.0 with steps of 0.1 amplitude, then using steps of 0.2 amplitude. Results were averaged over the seven accelerograms using the running mean with zero-length window, i.e., calculating values of the engineering demand parameters (EDPs) at each level of the intensity measure (IM) and then finding the average and standard deviation of EDPs given the IM level. Results show that the targeted limitation at the damage limit state (DLS SF = 0.4, interstorey limit 0.50%) is not satisfied (by a small over quota) in the upper three floors of the 4-storey case and in the last floor of the 8-storey case. It is also observed that the distribution of the interstorey drift along the height is quite regular but for the first one (4-storey case) and first two (8-storey case) storeys influenced by the base restraint of the reinforced concrete wall. The analysis of the evolution of the plastic rotations reveals that the wall in the 4-storey case undergoes significant plastic rotations and the rotation at peak in the moment rotation curve is soon attained, leading to the premature collapse of the bracing systems, before the design level (SF = 1) is reached. On the other hand, the wall in the 8-storey case has a much better behaviour with limited damage while the plastic deformation are basically limited in the steel links of the last storeys. In this way collapse is attained for SF larger than the design level.
To sum up, the analysis of the case studies that were designed according to the adoption of existing rules in the Eurocodes has highlighted the potentialities of the proposed innovative HCSW systems, namely: it is possible to develop a ductile behaviour where plastic deformation are attained in the steel links and limited damage occurs in the reinforced concrete wall; the interstorey drifts up to collapse are quite regular regardless of the non-simultaneous activation of the plastic hinges in the steel links and/or in the reinforced concrete wall; the adopted design

approach based on well-known concepts and procedures already available in the Eurocodes give a promising starting design solution. On the other hand, the following issues have been encountered: the designed solutions, although a promising starting point, require additional studies to clarify the relationships between wall over-strength and links in order to provide additional design recommendations as integration of the Eurocodes; the slenderness of the HCSW systems needs to be better controlled in order to limit the negative effects of geometric nonlinear effects and improve the behaviour at the damage limit states; criteria to support the dimensioning of the various elements of the HCSW systems need to be established given that it is not feasible that a trial and error procedure can be efficiently adopted in this innovative structural solutions (the considered solutions were obtained from a large number of possible configurations studied in this preliminary design evaluation).

WP3 Performance analysis of solutions involving innovative SRCWs

Task 3.1. Definition of a reduced set of systems as a consequence of a plenary discussion about RC-steel coupled systems.

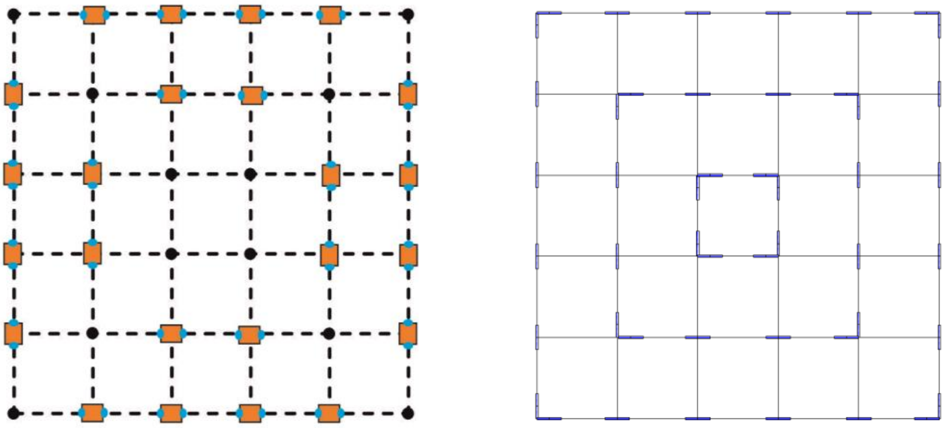
The SRCW system that the partners agreed to study in order to optimize and improve their behaviour and design procedures are steel frames with reinforced concrete infill walls, systems already considered in Eurocodes. Preliminary analyses were carried out to understand the influence of some parameters, such as the wall width, the concrete strength, the diameter of the stud shear connectors and their spacing, on the behaviour of SRCWs in order to select the most suitable solution. After the analysis and comparison of the achieved results, it was decided to use headed studs with diameters of 19 mm and a concrete strength for all walls of C40/50. For the 4-storey case study a of 20 cm is used and for the 8-storey case study a 24 cm wall thickness.

Task 3.2. Some case studies will be considered paying attention to the most diffused requirements for building in the European market (number of storeys, bays width, interstorey heights). The choices will be calibrated in order to be effective for the structural systems considered in this task.

The buildings chosen for the performance analysis of solutions involving SRCWs are the same of Task 2.2 as they are representative for typical cases. The gravity-resisting structure is not interested by the lateral-resisting system because pinned connections are considered between beams and columns and between beams and seismic resistant elements. The same vertical loads (permanent and variable) and the same spectrum representative of a medium-intensity seismic event are also considered. Based on the preliminary analyses made in Task 3.1, the arrangement of SRCW systems depicted in Figure 6 was selected for the 4-storey and 8-sorey case studies.

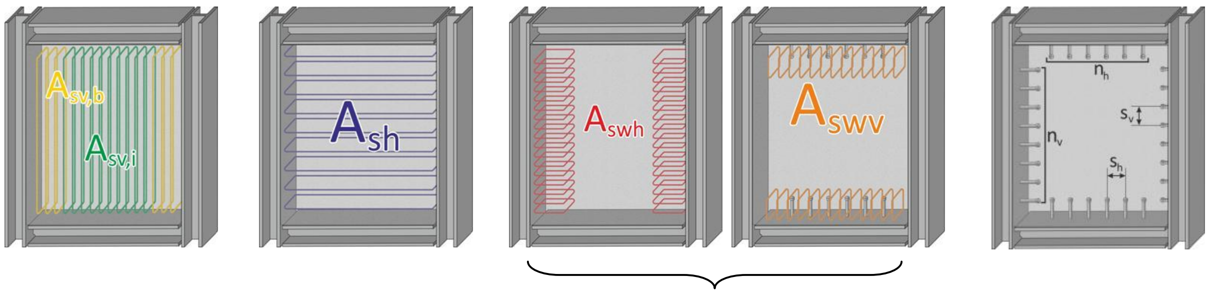
Task 3.3. The dimensions of resisting elements will be assigned by means of preliminary design rules. When a structural reduction factor q is available or it can be estimated from structures with similar post-elastic mechanisms, the design will be based on the linear analysis and the response spectra proposed in the Eurocede 8. Otherwise, the design will be based on static nonlinear analyses.

The design was carried out with a linear static analysis by assuming a suitable behaviour factor q and the response spectra already mentioned as proposed in Eurocode 8. The SRCW considered in this stage of the research are those classified as Type 1 in EN 1998-1 7.3.1 that are considered to behave essentially as reinforced concrete walls capable of dissipating energy in the vertical steel sections and in the vertical reinforcements of the walls. The storey shear forces are considered to be carried by horizontal shear in the wall and in the interface between the wall and beams. The structures investigated in WP3 were dimensioned for ductility class medium (DCM) that, in case of the SRCW systems, led to the behaviour factor q = 3.3. A design procedure is not available in Eurocode 8 and the selection of elements is based on a trial procedure in which the wall reinforcements and the shear connectors are finally determined so that all verifications are satisfied. In particular: (i) the vertical reinforcements are divided in the rebars placed in the boundary area of the wall (Asv,b), which have to resist the overturning moment, and the rebars placed outside the boundary area (Asv,i), which contribute both to bending moment and shear resistance (Figure 7a); (ii) the horizontal reinforcement of the wall (Ash) is set to hold shear forces (Figure 7b); (iii) confining hoops are set at the vertical (Aswh) and the horizontal (Aswv) boundary zones of the wall to guarantee the correct performance of shear connection (Figure 7c); (iv) the headed stud connectors are placed in both lateral sides, to hold the vertical shear VEd*, and in the top and the bottom of the wall to carry the horizontal shear VEd (Figure 7d). It has to be remarked that a capacity design procedure is not available, thus, the dissipative mechanism is not preserved against possible fragile failures.



(a) (b)

**Figure 6.** Arrangements of SRCWs for the (a) 4-storey and (b) 8-storey buildings




(a) (b) (c) (d)

**Figure 7.** Wall reinforcements: (a) vertical reinforcements; (b) horizontal reinforcements; (c) confining hoops; (d) stud connectors.

Particular attention was paid the shear connectors that have to be designed in order to prevent separation between reinforced concrete infill and boundary elements (EN 1998-1 7.10.1(2)) and to transfer vertical and horizontal shear forces between the structural steel of the boundary elements and the reinforced concrete (EN 1998-1 7.10.3(5)). The maximum horizontal forces to be carried by the shear connectors (between beams and the wall) are the shear forces which each wall has to resist.

Task 3.4. The seismic behaviour will be more precisely assessed by means of Incremental Dynamic Analyses (IDAs) in order to obtain an overview of the structural performance at different levels of seismic intensity. At this stage, synthetic accelerograms derived from the spectra proposed in Eurocode will be considered. The results should provide a preliminary information about the effectiveness of rules adopted in the preliminary design of Task 3.3 and they should highlight critical aspects related to ductility connection demands and ductility/stiffness demand at different storeys.

A preliminary pushover analysis was performed for the 4-storey building with the computer program ANSYS. The results of the nonlinear pushover analysis show that the structure is very stiff and capable of withstanding high horizontal loads. The displacements and inter-storey drifts obtained from the calculation show that all criteria to rule out second-order effects and to fulfil SLS criteria can be easily satisfied. Nevertheless, the calculation highlights critical aspects as well. Concrete cracks and crushing occur at a very early state ending the linear behaviour of the structure. Theoretically the system should behave linearly until the design load is reached and then nonlinear behaviour should occur. The boundary structure starts yielding before the design load is reached which is not critical as the design is based on plastic resistance. The shear connectors were the critical elements in the preliminary design defining minima lengths and thickness of the infill walls. The analyses demonstrate that the behaviour in shear of the connectors at the interface between the steel frame and the RC infill is uncritical but it is worth to note that possible critical behaviour in traction are not captured by the model. Another important observation is that yielding in the columns as anticipated in the proposal and necessary for the development of the dissipative devices could only be noted in the elastic infill and even in that case only in the bottom regions of each column/storey. The nonlinear concrete behaviour shows that severe damage in the concrete occurs long before any yielding in the boundary structure can be achieved.

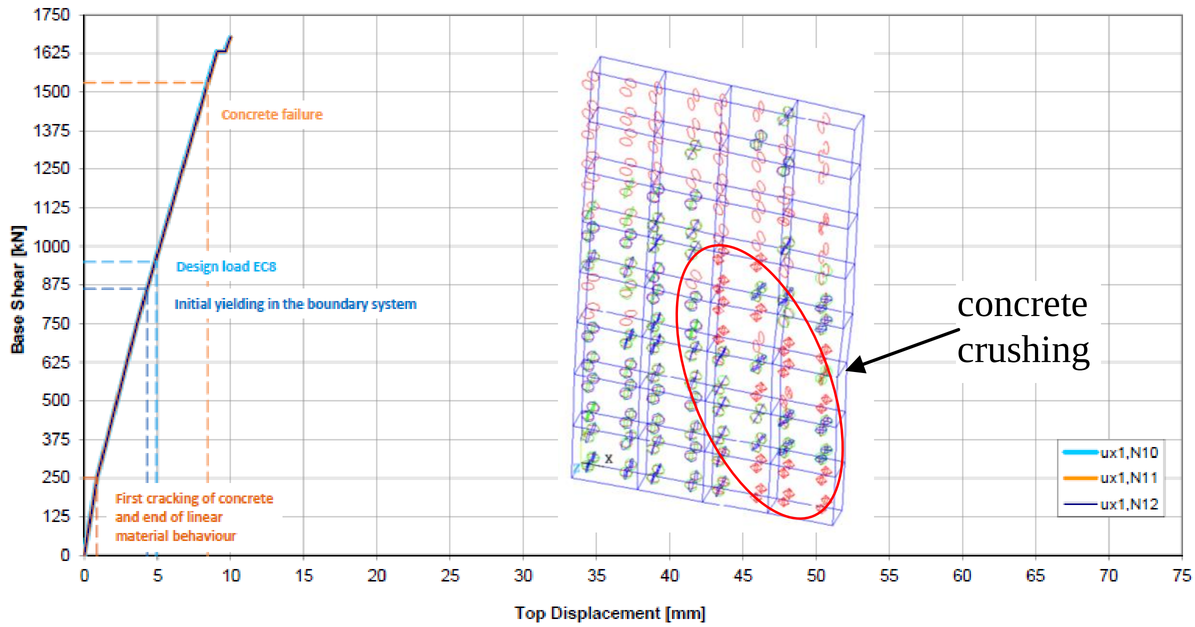



**Figure 8.** Pushover curve and failure mechanism for the 4-storey SRCW.

For what concerns the 8-storey building, a detailed numerical model was developed for the examination of the shear wall in ABAQUS FE code taking into consideration geometrical and material nonlinearities. The nonlinear special purpose "slide-plane" connector elements were implemented in the model to simulate the shear studs which connect the steel surrounding frame to the concrete infill walls. The pushover analysis demonstrate that the behaviour of the shear wall presents a deviation from the linear global behaviour at much lower loads compared to the Eurocode 8 design load. Through plastic deformation of the steel parts of the wall, as well as concrete crushing that takes place in the infill walls, the examined shear wall presents only 19.25% higher resistance attained with a fragile failure.

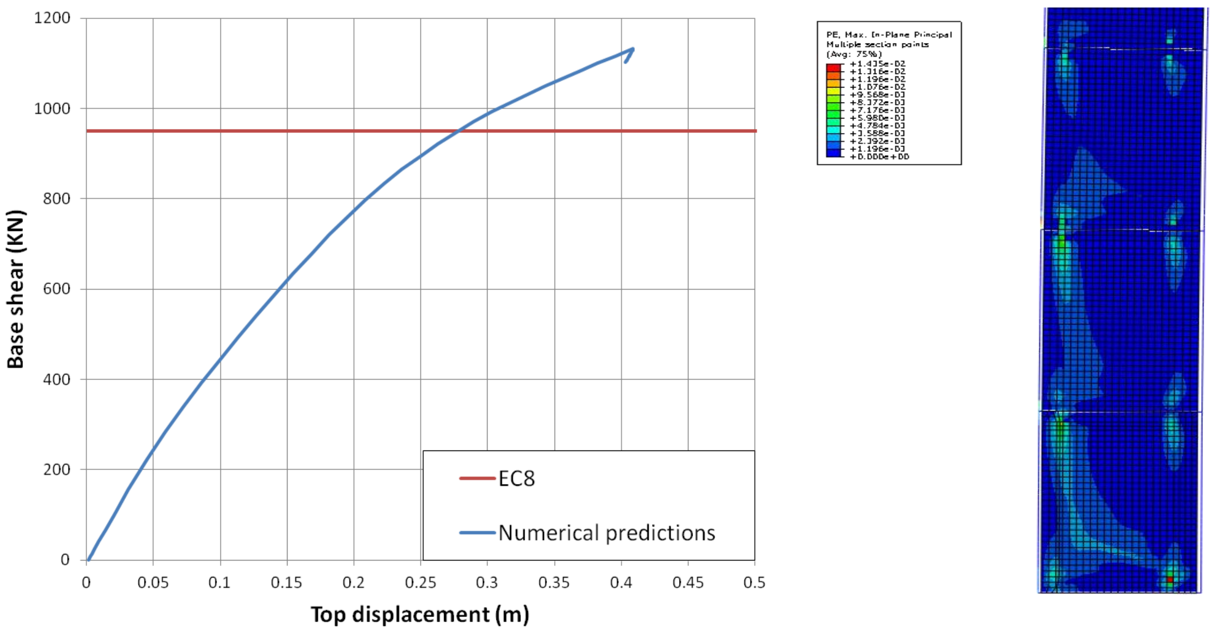



**Figure 9.** Pushover curve and plastic strain in the concrete for the 8-storey SRCW.

The failure mechanism is characterised by yielding of the steel frame concentrated mainly in the elements near the bottom of the wall and more specifically at the connections of the horizontal to the vertical parts. The resulting stresses on the concrete part of the wall are widely spread over the height of the wall up to the fifth floor and are higher at the part in compression. The distribution of the plastic deformation on the concrete infill walls follows a diagonal trend indicating clearly the expected locus of cracking located at the concrete diagonal in tension. In addition localized plastic deformations are also present near the corners of the infill walls due to the local action of the first studs of the horizontal and vertical elements.

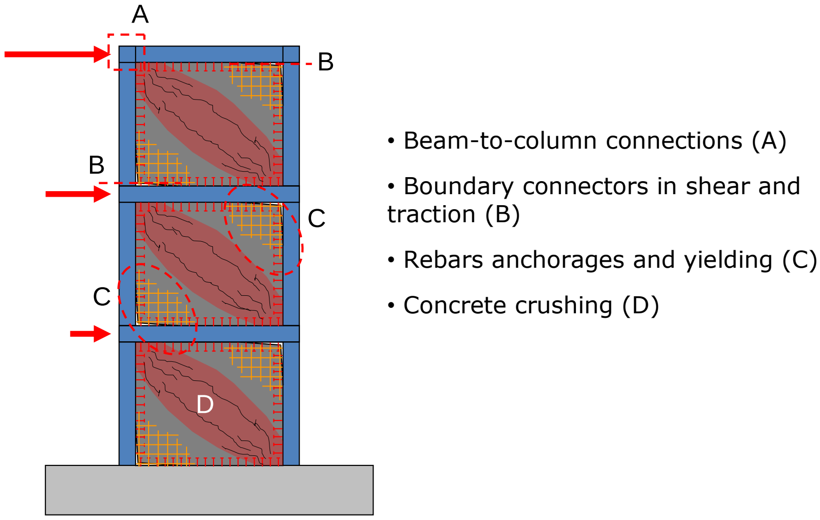



**Figure 10.** Critical aspects in the behaviour of SRCW systems.

In both 4 and 8-storey buildings, the same fragile failure mechanisms were observed. Despite the base shear force is higher than the design one, the behaviour is not acceptable as it is not sufficiently ductile to comply with the behaviour factor q = 3.3 adopted in the design. This unsatisfactory behaviour is due to the lack of a specific capacity design procedure that allows to control the formation of a proper dissipative mechanism. In particular the two designed systems suffered for problems at beam-to-column connections and for crushing of the concrete at a diagonal band. Furthermore, for the mechanism observed, some doubts arise about the behaviour in traction of the shear connectors and the anchorage of rebars that, to ensure the behaviour considered in the design procedure adopted, have to yield at their ends without a transition zone.

WP4 Design of HCSW systems

Task 4.1. Preliminary analyses of the connection typologies will be performed in relation to results of WP 2 and oriented to provide a base for the experimental campaign of WP 6.

The analysis of the connection typologies involved: (I) the connections between steel link and reinforced concrete wall; (II) the connection between steel link splices, i.e., part embedded in the concrete wall and replaceable part where yielding and consequent energy dissipation are developed; (III) the connection between steel link and steel side column.
Two typologies were considered for the embedment of the steel link in the reinforced concrete wall: in one case (typology 1) the bending moment transferred by the link to the wall is resisted by shear studs; in the other case (typology 2) the moment transferred by the link to the wall is balanced by a couple of vertical forces, as depicted in Figure 11.

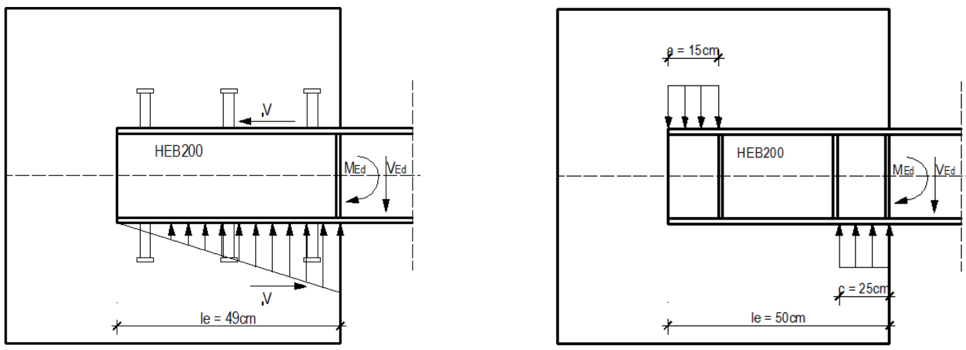



**Figure 11.** Connection typology 1 (left) and typology 2 (right)

Two solutions were considered for the link splice: the splice connection placed at a distance from the concrete wall sufficient to allow an easy bolting of the replaceable part (solution associated to typology 1); the splice connection is placed at the face of the wall and threaded bushings to allow replacement of the dissipative tract of the steel link (solution associated to typology 2), as depicted in Figure 12. The design of the connection is made to enforce the creation of a plastic hinge in the replaceable part of the steel link acting as a fuse and give adequate over-strength (capacity design) to the fixed part of the link embedded in the shear wall, the link-to-wall connection and the bolted beam splice connection between the fixed and replaceable parts of the link.

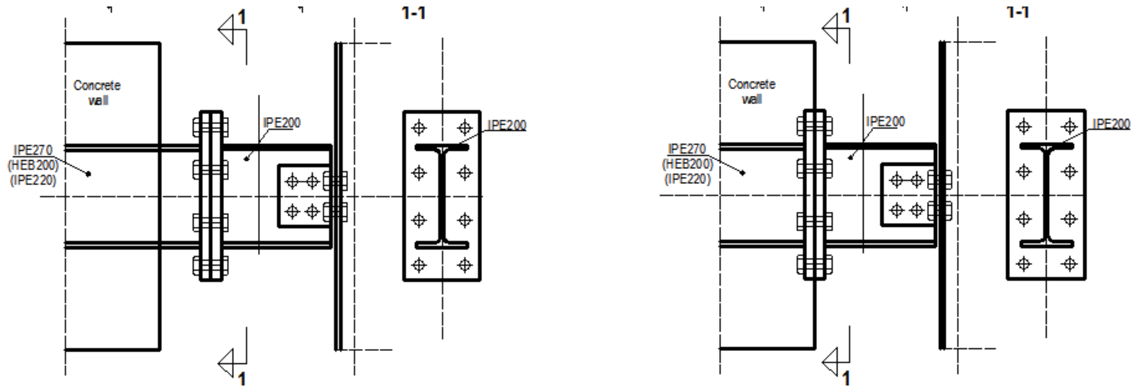



**Figure 12.** Link splice typology 1 (left) and typology 2 (right)

The connection between the steel link and the additional steel column is made by a conventional web angle connection bolted to the column flange, as depicted in Figure 13. Although in the structural analysis pinned connections are considered between links and additional columns, the joint exhibits a (limited) bending stiffness. However, when load increases, bending and shear increases until yielding is reached. The joints then rotates until a stress state characterized by a zero bending moment. The connection behaviour is the resulting behaviour of the two components of the joint: the web-profile to angle connection, mainly governed by the contact pressure between the bolts and the web of the profile; the angle to column-flange connection, governed by the behaviour of the angle itself. Therefore the hinge will appear in the component that will be the first to reach yielding. In the suggested design procedure a conservative approach is followed. A collapse mechanism corresponding to reaching a plastic hinge in the profile is assumed as target. Therefore, the maximum shear in the joint is given and it is felt as more conservative to work under the assumption of hinge at the angle-to-link connection. Of course, design must provide an appropriate rotation capacity of the connection.

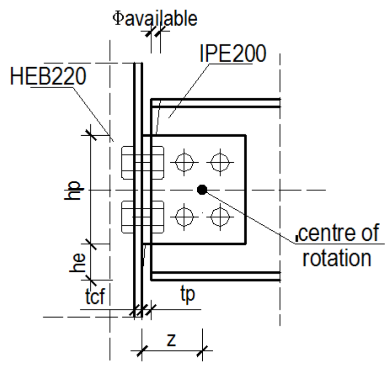



**Figure 13.** Connection between steel link and steel side column

Task 4.2. Definition of a design procedure oriented to optimize the global ductility of the system. Initially, the stiffness ratios between components will be studied in order to obtain a homogeneous yielding of dissipative zone along the building height. In a second phase the plastic demands at storeys and their compatibility with connection ductility will be analysed in order to establish the global ductility of the structural systems.

Preliminary designs based on the conventional force approach were made in order to identify possible optimal geometries during WP2. The results obtained in this first design stage directed the research towards the definition of a design approach that inherits recommendation for capacity design from other structural systems involving similar dissipative mechanisms in the links, i.e. eccentric braces in steel frames, as well as indication to reduce damages in the reinforced concrete wall. The design procedure is subdivided in the following steps: (1) assign dimensions of the reinforced concrete wall by selecting height-to-length ratio and thickness; (2) design of the steel links based on bending and shear obtained from linear analysis (e.g. spectrum analysis) with assigned uniform over-strength; design of the steel side columns using the summation of the yield shear forces of the links (amplified with $1.1\gamma_{ov}$ where $\gamma_{ov}$ is the over-strength coefficient defined in Eurocode 8) as design axial force; (3) design of the wall longitudinal reinforcements to provide an assigned over-strength compared to the bending moment obtained from linear analysis; design of the transverse reinforcements to avoid shear collapse of the wall considering the maximum shear at the base derived from the limit condition of yielded steel links and yielded wall in bending;

reinforcement detailing according to Eurocode 8 DCM rules. The use of this design approach is made through procedures that are familiar to structural engineers trained to steel seismic design according to Eurocode 8. While the application of the proposed design approach to 4-storey and 8-storey buildings did not result in steel links and relevant connections systems having specific problems (commercial IPE profiles can be used for the steel links and the obtained links are classified as short or intermediate links according to Eurocode 8), some problems were highlighted in some design cases as significant quantities of longitudinal bars were required, in some cases exceeding Eurocode upper limits.

Task 4.3. Evaluation of effectiveness of design provisions proposed in the previous task 4.2 by means of nonlinear static and dynamic analyses, considering different number of storeys and different geometries of the coupled system. The task will run in parallel with experimental campaign of WP 6. Preliminary connections models will be used and then progressively refined according to the availability of experimental results.

The seismic behaviour of 4-storey and 8-storey HCSW systems designed according to the proposed force approach was assessed through incremental nonlinear static analysis under applied lateral loads (pushover analysis) and incremental multi-record nonlinear dynamic analysis (IDA) using the software FinelG at the University of Liège. For the sake of simplicity, the evaluation of the seismic performances was based on a plane model (Figure 14) of a single HCSW connected to two continuous columns equivalent to the relevant parts of the gravity-resisting steel frame. Geometric nonlinear effects are included. The reinforced concrete shear wall was represented by frame elements using a fibre description for the behaviour of the concrete in the longitudinal direction, allowing an accurate estimation of the nonlinear behaviour in bending. At the beginning, simple elastoplastic connection models for the steel link and relevant connections were adopted: the steel shear links were modelled using nonlinear frame elements for the bending contribution as well as nonlinear shear springs introduced at mid span of each steel link to account for the shear deformability of the link and for the possible yielding in shear of the links. Nevertheless, the obtained experimental results confirmed the validity of the application of such simple elastoplastic models, that were proven effective in representing the global behaviour of the steel links.

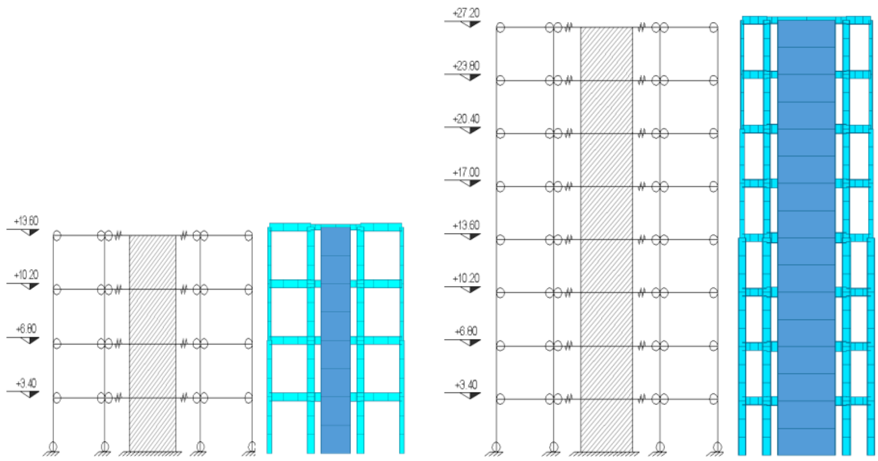



**Figure 14.** Numerical model of the HCSW systems

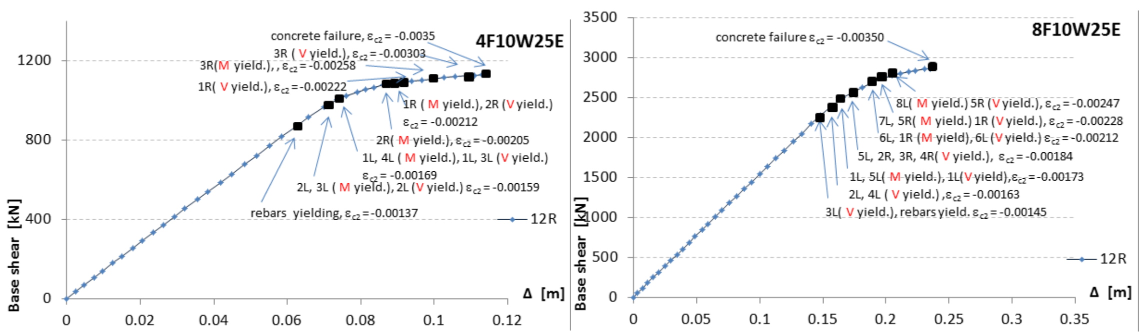



**Figure 15.** Pushover curves for HCSW systems designed with the force approach

Results from both static and dynamic analysis highlighted the potentialities of the proposed innovative HCSW systems, namely: it is actually possible to develop a ductile behaviour where plastic deformation are attained in the steel links and limited damage initially occurs in the reinforced concrete wall; the interstorey drifts up to collapse are quite regular regardless of the non-simultaneous activation of the plastic hinges in the steel links and/or in the reinforced concrete wall. On the other hand, the main limitation of the proposed design approach was identified in the lack of an effective control of the sequence of yielding of the links on one side and the reinforcements in the wall on the other side. This issue can be seen for example in the pushover curves in Figure 15 where the first yielding of reinforcing bars is attained before or at the same time of the first yielding of the dissipative steel links. Thus, damage, even if limited, is attained in the wall before the steel links start to dissipate seismic energy, and this is a non-desirable structural behaviour for the HCSW systems.

Task 4.4. A final assessment of the design procedure will be defined in this task, considering the critical points and limits evidenced in the task 4.3 and considering the more representative models elaborated in WP 6. In this task the number of situations (structural typologies and seismic inputs) will be enlarged with respect to task 4.3 in order to furnish a wider overview of cases of interest in the seismic prone areas of Europe. The seismic response will be analysed both by considering synthetic and natural accelerograms. A. large number of numerical analyses are required in this task. The incremental dynamic analyses will be carried out by UniCam while CPR will perform the post-processing, the critical evaluation and the comparison of results. The task will be developed with the support of DSD (subcontractor UniCam) for what concerns feasibility aspect.

Given the critical points and limits evidenced in the proposed force approach for HCSW design, a completely new approach was developed, this time based on limit analysis equilibrium having as key design parameter the coupling ratio, i.e. fraction of the bending moment resisted by the coupling mechanism. The resulting design method, described in details in the following, is rather simple and gave quite good results as assessed through extensive incremental nonlinear static and dynamic analyses. In fact, the analyses showed that the designed HCSW have ductile global behaviour, quite uniform distribution of interstorey drifts, and, above all, the wall is effectively protected against damage as the dissipative steel links yield long before the first yielding in the wall reinforcements (see for example the results from dynamic analysis in Figure 16), hence starting dissipating seismic energy while the wall is still in its elastic range.

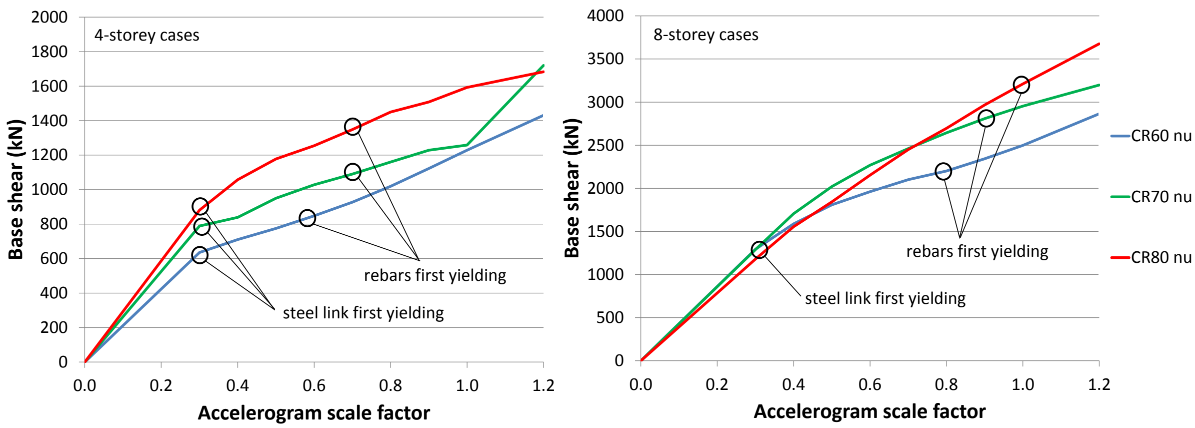



**Figure 16.** IDA curves for HCSW systems designed with the limit analysis approach

Task 4.5. Previous results will be analysed to evaluate the reliability of a design based on the linear dynamic analysis with response spectra and structural reduction factor.

The analysis of the design methods (the first one based on elastic analysis, the second one based on limit analysis) shown the superiority of the latter approach in providing HCSW systems that behave as intended in terms of sequence of yielding (capacity design). Although a structural reduction factor was identified ($q = 2.26$ for wall yielding as ultimate condition, $q = 3.74$ for system failure as ultimate condition), the linear dynamic analysis provides a distribution of stresses quite different from the distribution of stresses from limit analysis. Thus, it is advised that a design based on the linear dynamic analysis with response spectra and structural reduction factor cannot provide satisfactory results, unless ad hoc modifications of the results (e.g. application of amplification and/or reduction factors for the computed stress distribution, specific capacity rules) are developed.

WP5 Design of SRCW systems

Task 5.1. Preliminary analyses of the wall morphology and the connection typology will be performed based on the results of WP 3 and oriented to provide a bases for the experimental campaign of WP 7.

Preliminary pushover analyses performed with refined finite element models on 4 and 8-storey buildings designed by following Eurocode 8 provisions highlighted some critical issues resulting in an unsatisfactory and non-sufficiently ductile structural behaviour (Figure 8): (1) severe damage in the concrete occurs long before any yielding in the boundary structure can be achieved; (2) the boundary structure starts yielding before the design load is reached (this is not critical as the design is based on plastic resistance); (3) the distribution of the plastic deformation on the concrete infill walls follows a diagonal trend; (4) the shear connectors at the interface between the steel frame and the RC infill are uncritical even if they were the critical elements in the design; (5) localized plastic deformations are present near the corners of the infill walls; (6) concrete crushing that takes place in the infill walls determines its fragile failure.
The behaviour of SRCWs designed according to Eurocode 8 resulted to be non-satisfactory for the fragile failure of the RC infill walls due to the lack of a capacity design procedure. Furthermore, conceiving the system as a reinforced concrete shear wall is not compliant with the behaviour observed in which diagonal struts form within the RC infill walls at each storey. In addition, there are some doubts on the behaviour in traction of the shear connectors and the anchorage of rebars that, to ensure the behaviour considered in the design procedure adopted, have to yield at their ends without a transition zone.
The proposed innovative SRCW system is conceived to control the formation of diagonal struts in the infill walls (Figure 17). The energy dissipation takes place only in the vertical elements of the steel frame subjected mainly to axial forces without involving the reinforcements of the infill walls. Detailing of connection of the dissipating elements should allow their replacement and the possible use of buckling-restrained elements. The formation of the diagonal strut is ensured by joint stiffeners and bearing plates. The joint may be welded in shops allowing to speed up the erection phases. The stud connectors are not required to transfer shear forces but they are used to connect the infill and the frame together during the seismic shakings.

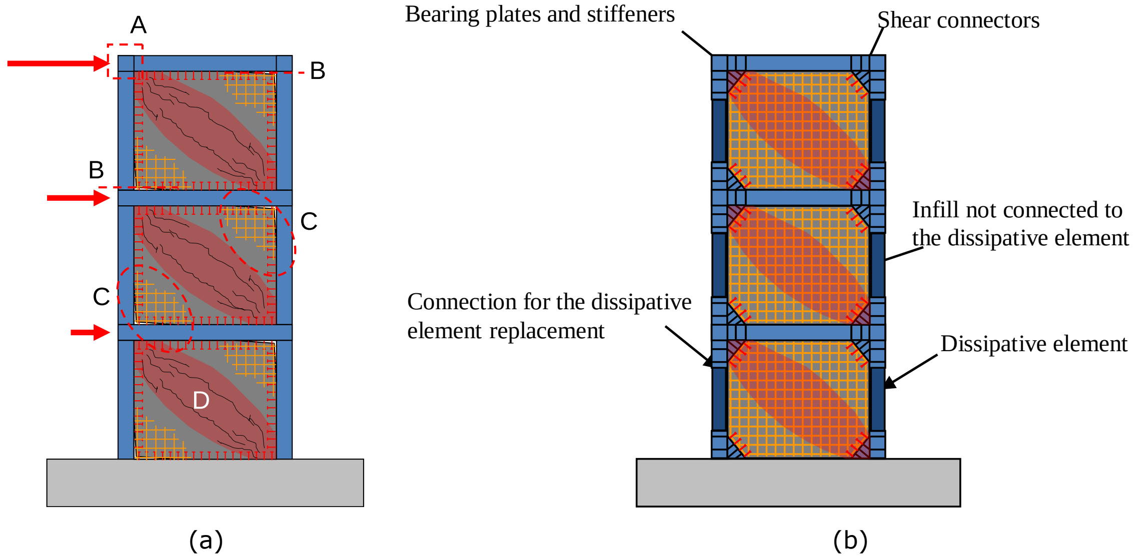



**Figure 17.** (a) Critical zones in SRCW systems; (b) innovative SRCW system

Task 5.2. Definition of a design procedure oriented to optimize the global ductility of the system. Initially, the stiffness ratios between components will be studied in order to obtain a homogeneous yielding of dissipative zone along the building height. In a second phase the plastic demands at storeys and their compatibility with connection ductility will be analysed in order to establish the global ductility of the structural systems.

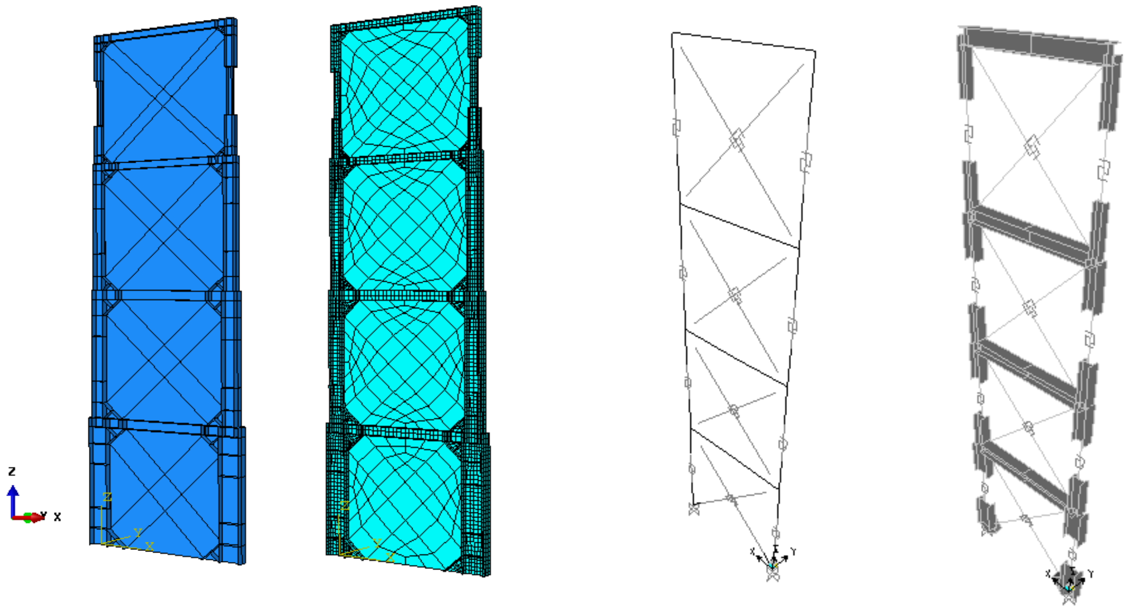
The proposed innovative SRCW is characterized by elements with specific tasks by allowing for the execution of a proper capacity design. The design procedure is force-based and is applied by considering the latticed statically determined scheme representing the limit behaviour of the SRCW depicted in Figure 17b. The design procedure is articulated according to the following 9 steps: (1) definition of the static equivalent lateral loads and calculation of the truss actions; (2) design of the cross sections of the ductile boundary elements in traction; (3) capacity design of the connection of the ductile elements and of the adjacent elements performed with the usual Eurocode 8 formula; (4) calculation of geometric over-strength factors; (5) calculation of axial forces in non-ductile elements by combining the effects of gravity loads with those of the seismic action suitably magnified; (6) capacity design of the reinforced concrete infill against concrete crushing; (7) design of the beams in traction; (8) check of the compressed edge elements and design of the wall reinforcements and the shear connection with the frame if the buckling verifications of the steel element is not satisfied; (9) calculation of the length of the dissipative element, in order to ensure the compliance between local and global ductility. Details of the design procedure are in the following. The procedure is straightforward but the limit structural scheme adopted may not represent the behaviour of the system especially in the linear range for weak earthquakes. The definition of the behaviour factor needs analyses carried out by considering sophisticated models able to capture the behaviour of all the elements with particular attention to the reinforced concrete infill and to its interface with the boundary steel frame.

Task 5.3. Evaluation of the effectiveness of design provisions proposed in the previous task 5.2 by means of nonlinear static and dynamic analyses, considering different number of storeys and different geometries of the coupled system. Numerical investigations of the interrupted shear connections in comparison to continuous connections will be carried out. The task will run in parallel with experimental campaign of WP 7. Preliminary connections models will be used then progressively refined according to the availability of experimental results.

The proposed force based design procedure was applied to 4-storey and 8-storey SRCW systems characterised by different aspect ratios of the infill walls and overall aspect ratios of the system. Four different scenarios have been also supposed for the over-strength distribution of the ductile elements in order to investigate the effects of the regularity of the systems in the development of the plastic dissipative mechanism. Finally, three different seismic intensities were considered. The structures designed were investigated by performing push over analyses with a simplified procedure based on the truss-like scheme adopted in the design.
The design procedure demonstrated to be reliable and the results obtained gave information on the better aspect ratios to achieve well-designed structures. The system was feasible for 4-storey buildings in almost all the cases analysed. In the case of 8-storey buildings, instead, the stability verification of the compressed side elements led to the need of using non-commercial profiles and to involve the wall to resist a part of the compression forces; in such cases the shear connectors welded to the vertical elements outside the fuses could not be properly designed. Other difficulties were encountered in the design of the bearing plates needed for the concrete wall to withstand the diagonal compression field. In the case of infill walls with higher aspect ratio, the system could be designed only for low and medium seismicity level whereas the high seismicity case remains critical for the need of using non-commercial profiles.

Task 5.4. A final assessment of the design procedure will be defined in this task, considering the critical points and limits evidenced in the task 5.3 and considering the more representative models elaborated in WP 7. In this task the number of situations (structural typologies and seismic inputs) will be enlarged with respect to task 5.3 in order to furnish a wider overview of cases of interest in the seismic prone areas of Europe. The seismic response will be analysed both by considering synthetic and natural accelerograms. A large number of numerical analyses are required in this task. The incremental dynamic analyses will be carried out by SHE with the support of its sub-contractor UniTh, while CPR will perform the post-processing, the critical evaluation and the comparison of results.

Refined finite element models were developed in the ABAQUS program (Figure 18a) for a reduced set of the systems previously designed selected according to feasibility considerations. The analyses were carried out to investigate the presence of the reinforced concrete infill walls that, despite they are connected only to the steel frame joint, may behave differently to simple diagonal struts since biaxial stress fields are expected to develop. Furthermore, joints are not hinged and are rather characterised by the presence of the diagonal welded plate, suitably stiffened, to control the formation of the diagonal compression field.
Static nonlinear analyses demonstrated that the yielding pattern is characterised by plastic strains only at the ductile elements, according to the hierarchy principles involved in the design methodology (Figure 19a). Figure 19b reports the minimum in-plane principal stresses in the walls to show struts developed in the concrete panels.
Simplified plane models of the systems were also developed in the computer program SAP2000 by using beam finite elements to model the frame and link elements with suitable constitutive laws to simulate the ductile elements and the compressive and tensile diagonals of the concrete infills. Geometry of the system is reproduced by using rigid constraints in order to account for the actual position of links simulating the concrete diagonals (Figure 18b).
Figure 19c shows comparisons of results obtained with the two kinds of models and with the simplified statically determined model used in the design procedure. The excellent approximations achieved validate the use of simplified models in the assessment analysis.




(a) (b)

**Figure 18.** (a) Refined finite element model; (b) simplified frame model

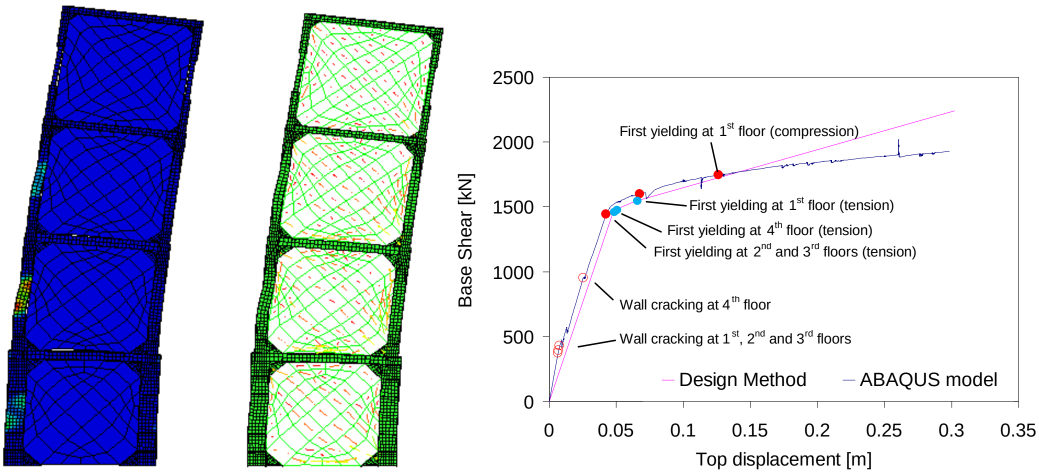



**Figure 19.** (a) Yielding pattern; (b) wall stress field; (c) comparison of results

Task 5.5. Previous results will be analyzed to evaluate the reliability of a design based on the linear dynamic analysis with response spectra and structural reduction factor.

The seismic behaviour of 4-storey and 8-storey SRCW systems designed according to the proposed force approach was assessed through incremental nonlinear static analysis under applied lateral loads (pushover analysis) and incremental multi-record nonlinear dynamic analysis (IDA) using the software SAP2000. The relevant reduction factors were evaluated in order to make comparisons with that used in the design (q = 3.3). It was shown that the same design reduction factor is obtained only for the 4-storey buildings, for structures with the higher regularity and for the modal force distribution. For the 8-storey buildings the maximum reduction factors evaluated were not greater than q = 2.7 for regular structures with minimum values q = 2.1 in the case of not regular structures.
The application of the linear dynamic analysis with response spectra pointed out that the analysis carried out for the design is capable of a good prediction of the response of the structures. Major critical points were encountered in the use of the simplified formula suggested by the code to evaluate the fundamental period of the system that resulted to be underestimated; this partially compensated the effects of using the higher reduction factor q = 3.3 in the evaluation of forces in the elements. Overall, the design procedure provides good solutions but nonlinear assessments of the designed systems are deemed to be necessary because of the drawbacks previously pointed out.

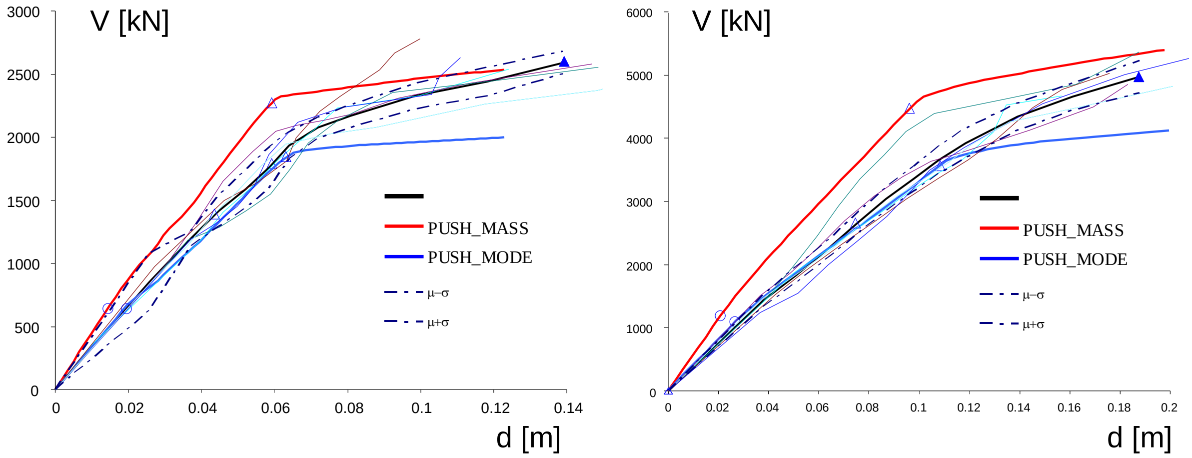



**Figure 20.** IDA curves for SRCW systems designed with the proposed procedure

WP6 Experimental based models for HCSWs

Task 6.1. Design of the experimental campaign. The definition of type of tests and morphological characteristics of specimens will be assessed by CPR that will execute the tests and ULG, involved in WP4, that will provide indications on the basis of preliminary consideration of WP4-Task 4.1. The experimental campaign aims at evaluating the performance of dissipative zones and joint connections between RC and steel elements under cyclic loads.

The specimens for the experimental campaign were designed to study and characterize the performance of the connection of a seismic link embedded in a concrete shear wall and the efficiency of the capacity design of such a system, made with the objective of developing a plastic hinge in the replaceable part of the link, acting as a fuse, with all other components of the connection that must remain undamaged. Due to the loading limits of the testing facility there was need to down-scale the link system. The objective of down-scaling was to keep a link that can be classified as intermediate with similar values of shear and moment's ratios as in the case study. The two solutions earlier described in Task 4.1 were considered: typology 1 (Figure 21) where the bending moment transferred by the link to the wall is resisted by shear studs and the link splice connection is placed at a distance from the concrete wall that is sufficient to allow an easy bolting of the replaceable part; typology 2 (Figure 22) where the moment transferred by the link to the wall is balanced by a couple of vertical forces and the link splice connection is placed at the face of the wall and threaded bushings to allow replacement of the dissipative tract of the steel link.

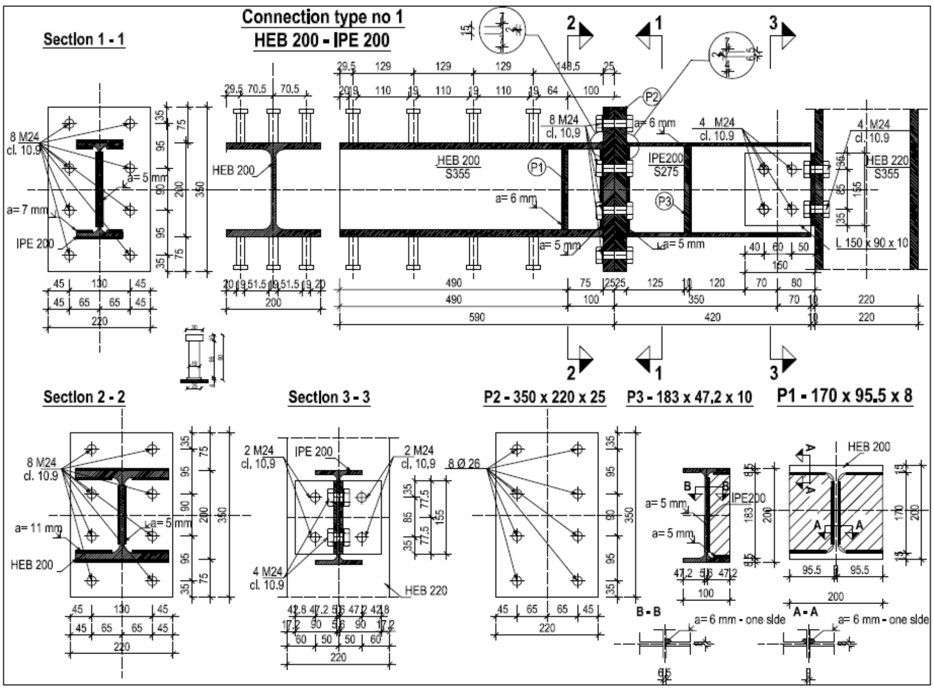



**Figure 21.** Link specimen for connection typology 1

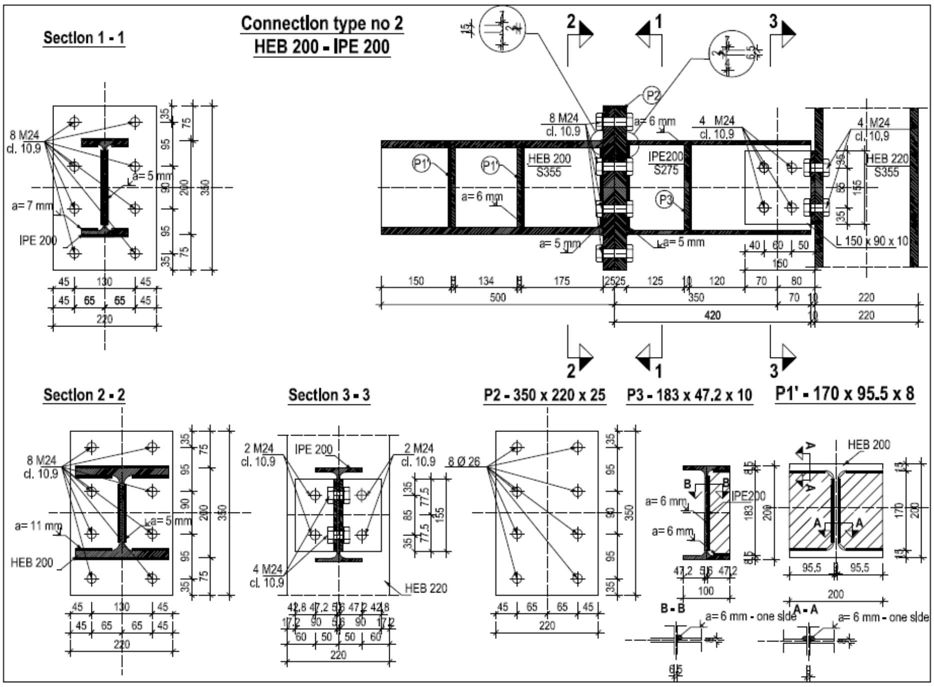



**Figure 22.** Link specimen for connection typology 2

Once that the tests are performed and the steel links damaged, a second series of tests are performed in order to evaluate the ultimate capacity of the link-to-wall connection. For this purpose the damaged IPE200 steel S275 links are substituted with HEB200 steel S355 links of the same length before running this second series of tests. The configuration of the specimens is depicted in Figure 23 while the overall testing facility is depicted in Figure 24.

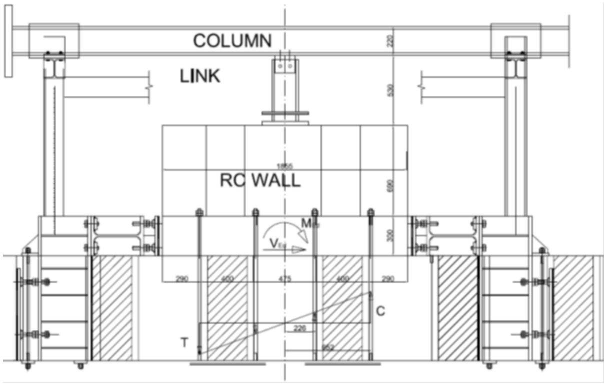



**Figure 23.** Configuration of the test specimens for the link-to-wall and link-to-column connection systems

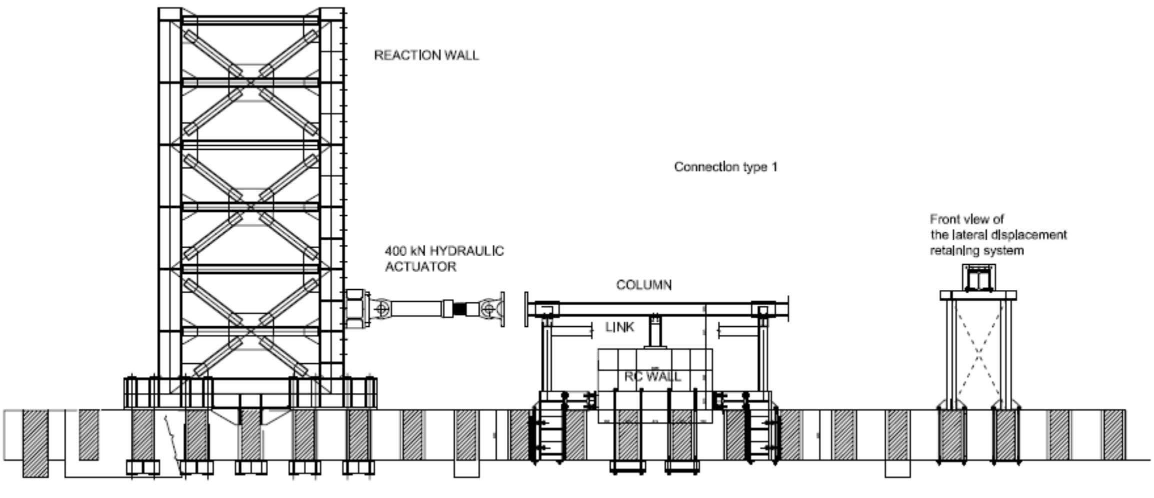



**Figure 24.** Overall configuration of the testing system

Task 6.2. Supply of raw material and building of specimens (two sets of 4 specimens).

The supply of the material and building specimens was made by OCAM as scheduled.

Task 6.3. Displacement controlled tests under cyclic path with increasing amplitude in order to evaluate low cycle fatigue limits, ductility of dissipative zones, strength and deformability of the steel-concrete connections (4+4 tests). Tests will be carried out according to the procedure described in A2.6.

For each connection typology, five test were performed, i.e., two sets of cyclic tests with increasing amplitude, two sets of cyclic tests with constant amplitude, a monotonic test up to failure in order to evaluate the ultimate resistance of the link-to-wall connection (in this last test the link is substituted with a HEB200 profile, steel grade S355. A selection of results is presented in Figure 25, Figure 26, and Figure 27.

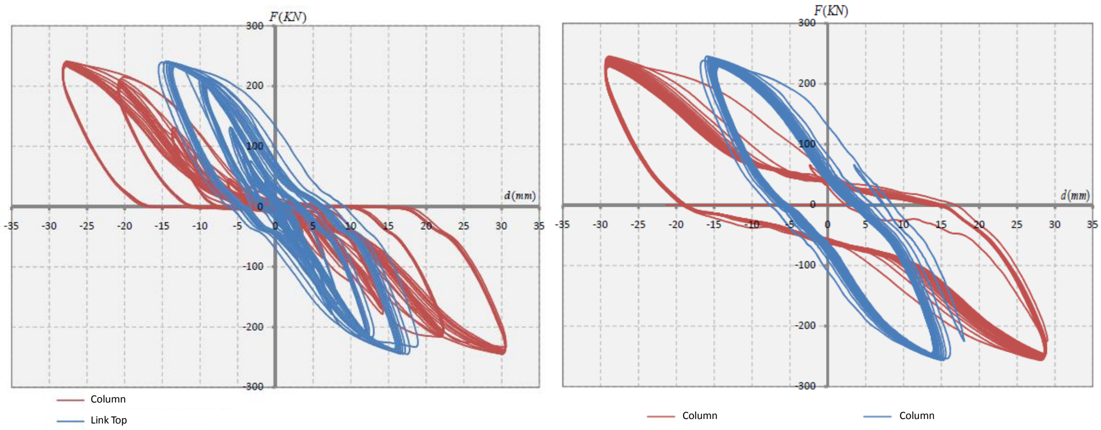



**Figure 25.** Connection typology 1: load displacement curves for link top and column in the cyclic test with increasing amplitude (left figure) and constant amplitude (right figure)

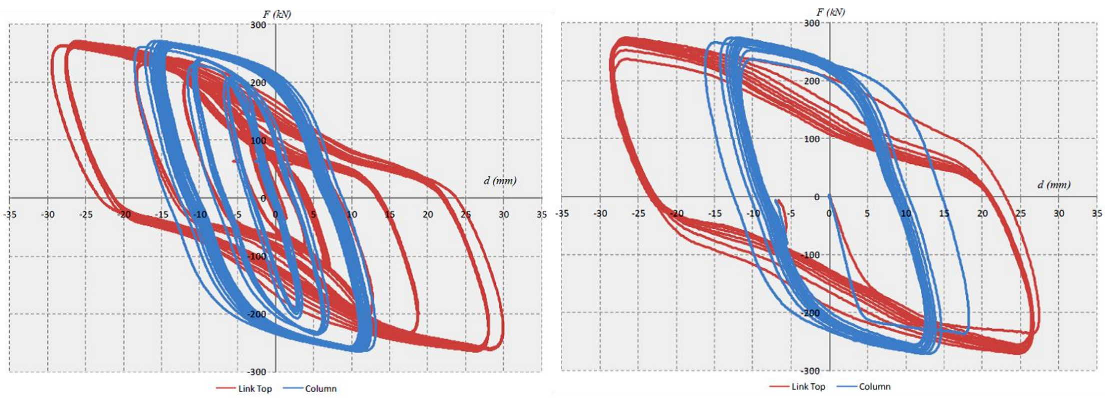



**Figure 26.** Connection typology 2: load displacement curves for link top and column in the cyclic test with increasing amplitude (left figure) and constant amplitude (right figure)

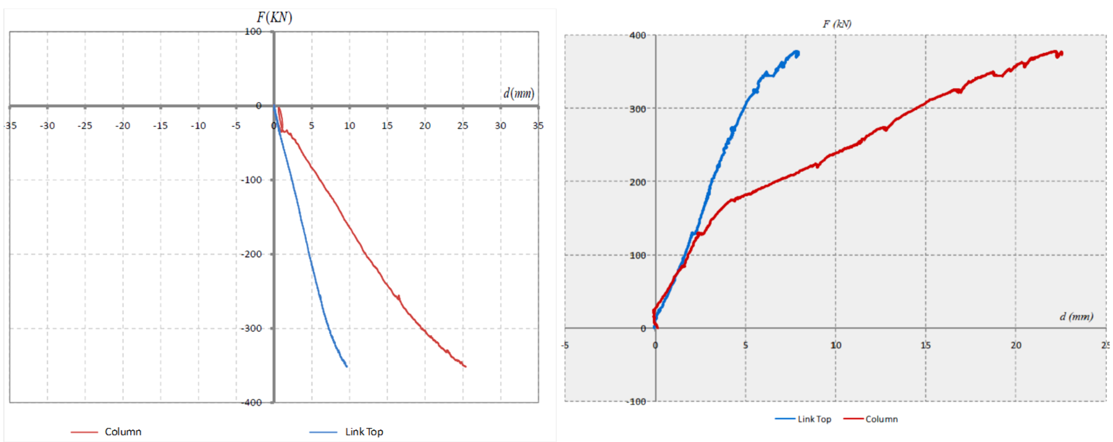



**Figure 27.** Monotonic test for connection typology 1 (left) and typology 2 (right)

From the analysis of the results, it is observed that the clearance between bolts and holes in the link-to-column connection is responsible for the relative displacements between the connected elements, and the differences increase as the maximum displacement imposed increases due to the increasing hole plasticization that consequently enlarge the relative clearance between holes and bolts. Another critical aspect that was observed during the tests is the excessive rotation of the angle profile that causes the angle leg to lose contact with the column flange, thus, amplifying the local plastic deformations around the holes. Improvements in the link-to-column connection made for typology 2 reduced the relative displacements between the connected elements, producing hysteresis cycles considerably fatter and the pinching phenomena reduced. Regarding the connection between the link and the wall, the monotonic tests highlighted a basically linear global behaviour up to the maximum applicable load (Figure 27).

Task 6.4. The experimental results will be elaborated in order to define constitutive models of local behaviour to be adopted in the structural analyses to be performed in WP4 and WP8.

A simple frame model with a simple elastoplastic model (Zona and Dall'Asta 2012) was proven adequate to model the behaviour of the steel link (see for example Figure 28), provided that the actual value of the yield stress is identified. This result gives great benefits for the nonlinear static and dynamic analysis of HCSW systems, as conventional elastoplastic models commonly available in many structural analysis programs are adequate.

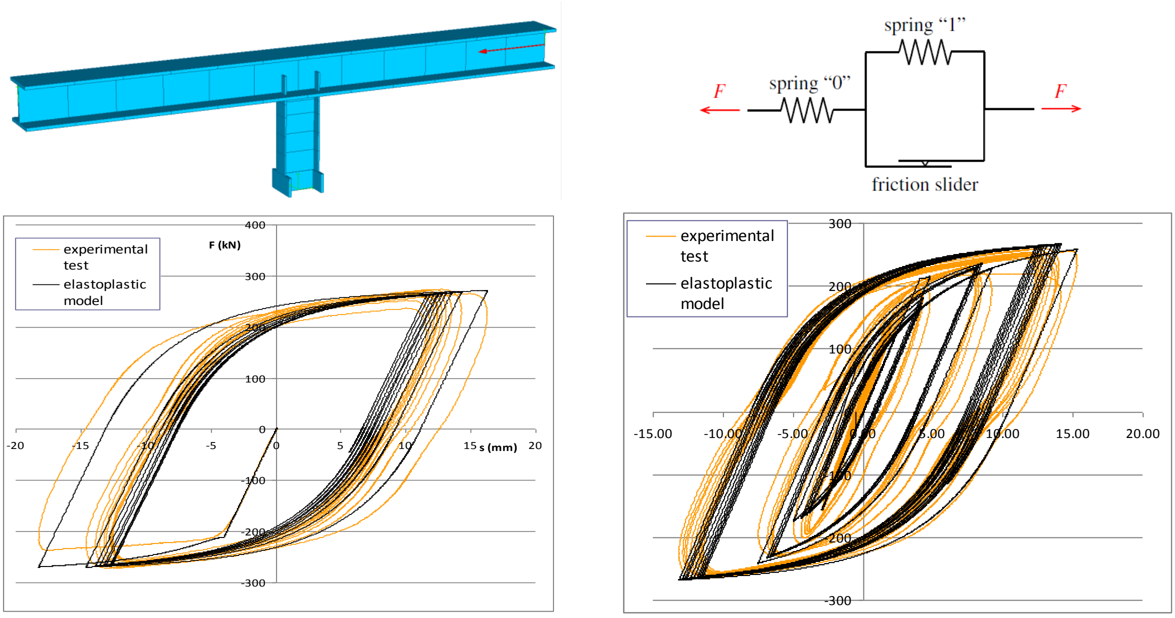


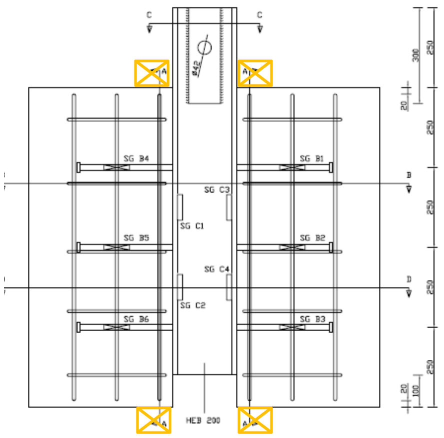

**Figure 28.** Model for the tested connection system and relevant calibration based on experimental results

WP7 Experimental based models for SRCWs

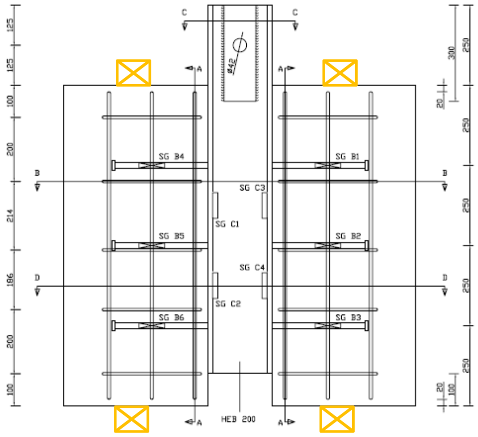
Task 7.1. Design of the experimental campaign to evaluate the performance of joint connections between RC and steel wall elements and side steel elements under cyclic loads. The details and morphological characteristics of specimens concerning the connection system will be assessed by RWTH that will execute the tests and SHE, leader of WP5, that will provide indications on the basis of preliminary consideration of WP5-Task 5.1. The details and morphological characteristics of downscaled specimen concerning the complete wall-frame system will be assessed by CPR that will execute the tests and UniCam, with the support of SHE, leader of WP5, that will provide indications on the basis of preliminary consideration of WP5-Task 5.1.

Three experimental activities are included in this WP: tests on shear connection specimens, tests on isolated ductile side elements, and tests on one-storey SRCW. Tests on shear connections are aimed at understanding the behaviour of headed studs placed at the wall edge when subjected to cyclic loading; tests on the ductile side elements are aimed at understanding the cyclic behaviour under undesired compression and secondary bending; tests on the wall are aimed at understanding the behaviour of the reinforced concrete infill and the effectiveness of the formation of the diagonal strut in the development of the dissipative mechanism.
As for the tests on shear connectors, a total number of three displacement controlled tests were initially planned. These tests were extended to a total number of six specimens each consisting of a HEB 200 profile with three headed studs welded to each flange. The increased number of tests

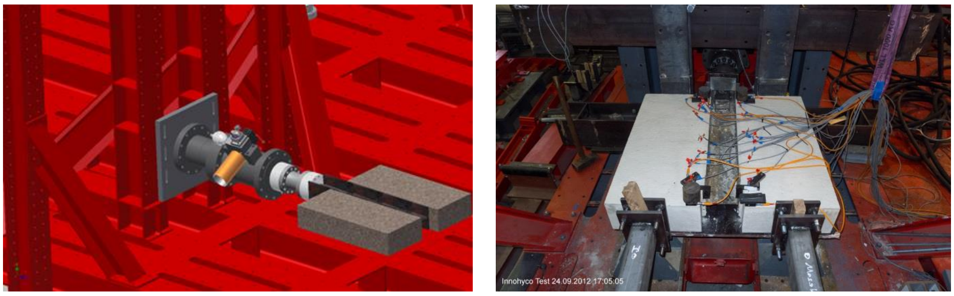
reflects the consideration of two effects to be investigated: connectors subjected to pure shear and connectors subjected to shear and traction. For this purpose, two different test configurations (Figure 29a,b) were considered and, for each configuration, one monotonic and two cyclic tests were done.




(a)




(b)



(c)

**Figure 29.** (a) stud connectors subjected to pure shear; (b) stud connectors subjected to shear and traction; (c) test rig

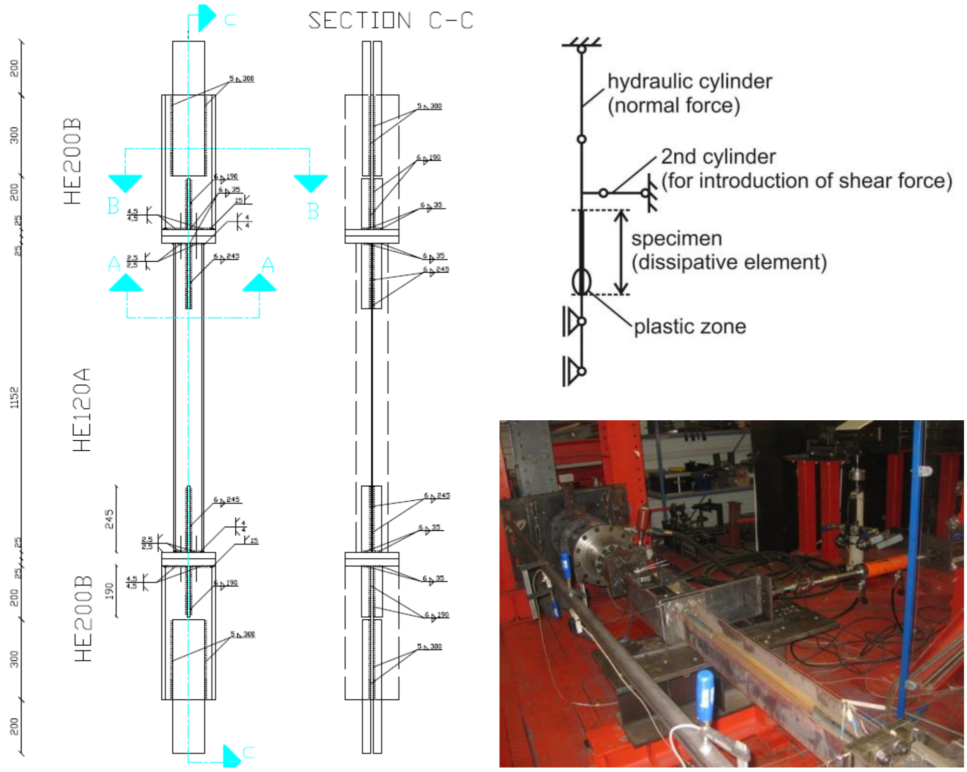



**Figure 30.** Specimen of the ductile side element and relevant testing setup

The specimen of the side steel elements consists of the same profiles as the top storey of one of the cases designed in WP5 (Figure 30). During seismic action the wall experiences horizontal loading from alternating directions. This results in alternating tensile and compressive action on the side steel element, combined with smaller horizontal and moment actions. Tests were performed by deriving actions from a numerical simulation. Cyclic axial and shear forces were applied with a couple of hydraulic jacks as depicted in Figure 30.
For what concern the downscaled SRCW system, a one-storey specimen was designed (Figure 31). The specimen was 2:3 downscaled and designed to comply with the capacity of ductile elements by applying the procedure proposed in WP5. The wall was instrumented so that displacements, and forces were measured as well as strains in the ductile side elements (Figure 32).

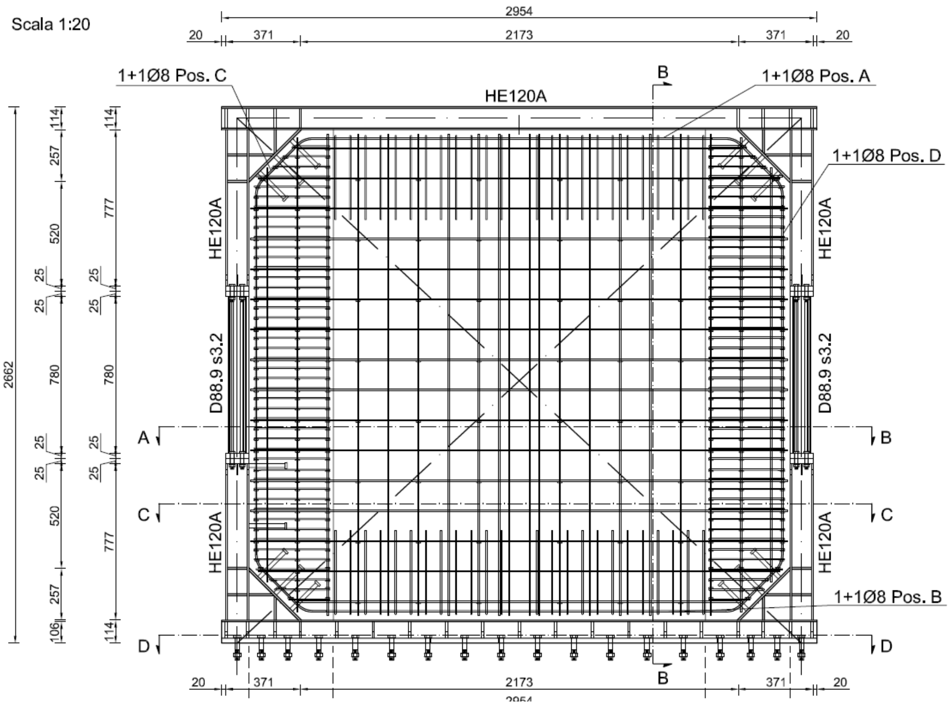



**Figure 31.** (a) SRCW specimen; (b) testing setup

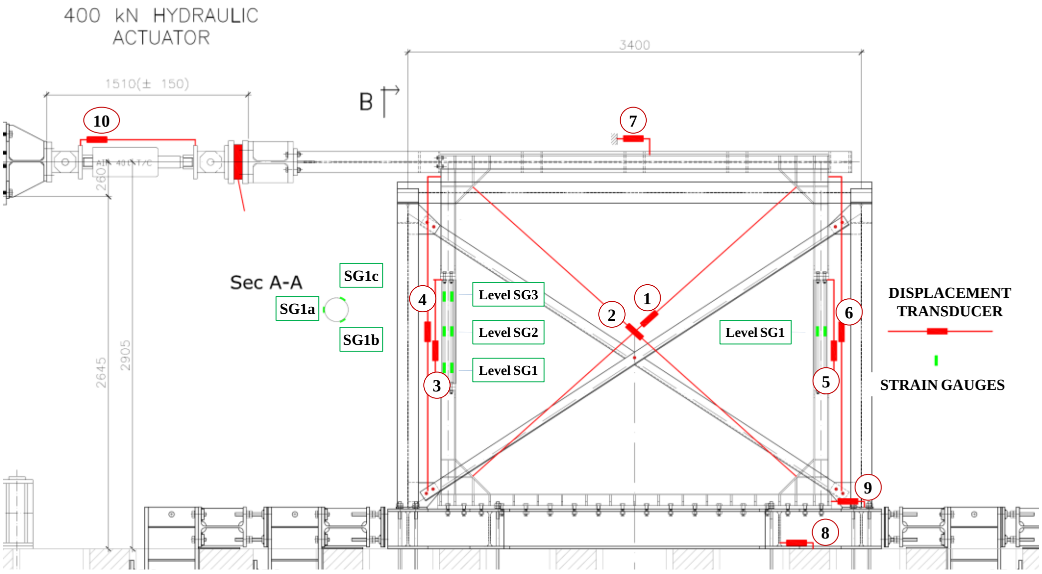



**Figure 32.** Position of the sensors

Task 7.2. Supply of raw material and building of connection specimens (3 specimens), side steel element specimens (2+2 specimens) and structural downscaled system (1 specimens), according to provisions elaborated in the previous task 7.1.

The supply of the material and building specimens was made by OCAM as scheduled.

Task 7.3. RWTH will carry out displacement controlled tests on connection (3 tests) and cyclic tests on side steel elements (2+2 tests) in order to evaluate low cycle fatigue limits, ductility of dissipative components, strength and deformability of the steel-concrete connections.

Experiments performed on the shear connection demonstrated a large discrepancy between strength obtained with the Eurocode 4 design formulas and the measured strength that resulted almost 1.5 times higher. Furthermore, shear failure occurred in the stud shank and not in the concrete crushing as was expected from the design formulas. As for the cyclic behaviour the specimen did not withstood the prescribed number of cycles at the loading level equal to 75% of the experimental strength, as requested for shear connection in seismic resistant elements. The test was thus repeated with the maximum amplitude equal to 50% of the monotonic capacity (Figure 33).

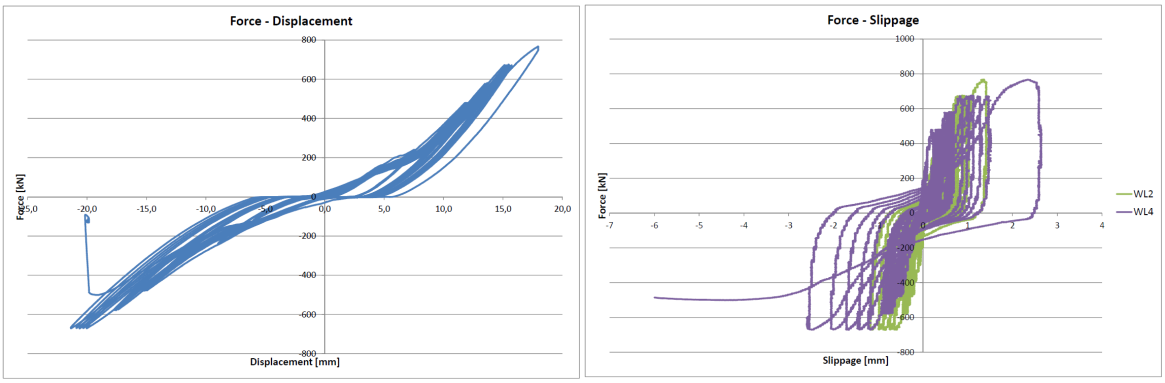



**Figure 33.** Force-displacement and force-slip curves (cyclic loading reference 50%)

Two monotonic tests were performed for the side steel elements: (a) specimen subjected to compression forces, and (b) specimen subjected to tension forces. Lateral forces were contemporarily applied with a second cylinder (Figure 30) to induce a concomitant bending moment. The maximum amplitude of the actions were derived from a theoretical model. A plastic hinge formed at the end of the fuse for the combined axial force-bending moment actions. The cyclic tests demonstrated the capability of fuse to develop a plastic hinge necessary for the formation of its assumed ultimate resisting mechanism given that it is actually not pinned at its ends.

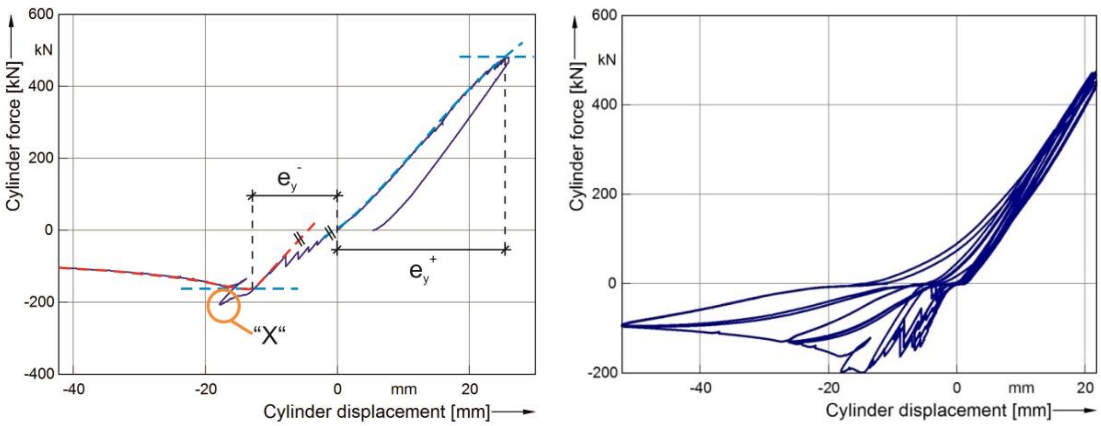



**Figure 34.** Results of monotonic test and one of the cyclic tests

Task 7.4. CPR will carry out cyclic displacement controlled tests on the downscaled specimen of the structural system subjected to constant vertical loads and increasing horizontal actions up to failure (1 tests) in order to evaluate strength deformability and ductility of the SRCW system.

The experimental activity on the downscaled specimen Figure 35 permitted to observe the real behaviour of a panel that confirmed the validity of the resisting mechanism assumed in the design. The diagonal strut formed without crushing of the concrete as demonstrated by the crack pattern observed. Strain measured at the vertical elements confirmed the yielding of the material constituting the fuses whereas other steel elements were preserved.

**Figure 35.** Testing of the downscaled specimen

Task 7.5. The experimental results will be elaborated in order to define constitutive models of local and global behaviour to be adopted in the structural analyses to be performed in WP5 and WP8. Local models will be developed by SHE, supported by its subcontractor UniTh and global models will be developed by UniCam.

Different mechanical models were developed in SAP2000 by using beam finite elements (elastic) and nonlinear links (lumped plasticity) in order to closely reproduce the experimental results (Figure 36). The actual geometry of the joints is considered by introducing rigid body geometrical constraints. Ductile elements are modelled with nonlinear links by using the built-in Wen plasticity for the uniaxial properties.

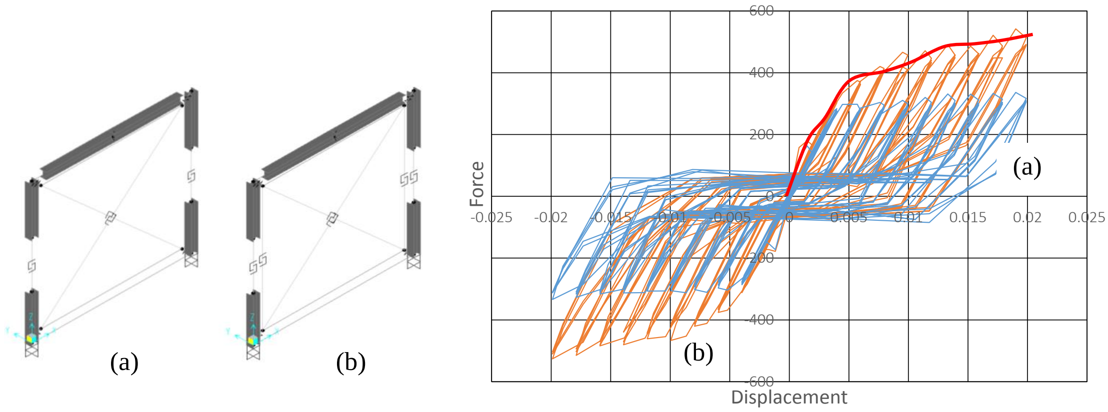



**Figure 36.** Mechanical models for the SRCW specimen

The wall is schematized with an increasing level of complexity. In the simpler model, nonlinear links are used just to capture the formation of the diagonal struts. In the more complex model, vertical nonlinear links are also considered to capture the effects of the wall that may be activated by the interaction with the steel frame at the node. For the diagonal elements, the compression force-displacement relationship is derived by the stress-strain Mander's (1988) law for unconfined concrete; a linear elastic behaviour followed by a linear softening branch is considered for tensile stresses; the Takeda's (1970) hysteretic behaviour is considered.
For the vertical wall elements, a couple of links are placed in parallel and connected at the same nodes of the model: one is used for the concrete component, for which the Mander's law for confined concrete is considered, the other is used for the reinforcements, for which the Wen model in considered. The calibration of the material parameters permits to capture the experimental results.

Task 8.1 Definition of functional and dimensional characteristics of the case study.

The functional and dimensional characteristics were chosen by considering suggestions from OCAM and SHELTER S.A. for what concerns the most diffused distribution of use at different floors while position, number and dimensions of seismic resistance structures were decided based on results of parametric analyses (WP4 and WP5). The location (Italy, Appenino mountains) was decided in order to face with a medium-high seismic input.
The considered building is set among an existing residential complex in Camerino, Italy, elevation 670 m above the sea level, and are composed by two symmetrical bodies, divided by a structural seismic gap. Each body holds parking at the basement, shops at the ground floor and residences at the upper floors; the module has a rectangular floor plan, with a longitudinal axis oriented east to west, covering six floors above ground (total height 21,45 meters above ground level), plus one underground floor for parking. The size of the rectangular module is about 25,00 x 14,15 meters, the interstorey height is 3.50m.
All lighting checks are satisfied in according to hygienic and sanitary laws; in particular for living spaces the ratio between area and area of windows is never less than 1/8. The elevator and the stairway are in the structure central core, so the core of service areas takes up the middle section of the structural grid. The flat roof is accessible from the internal stairway. Outside the space is provided by a large parking area to serve the shops and the residences and is equipped with green areas. All the spaces and paths are conceived according to Italian law about accessibility (Italian L.13/1989 and subsequent additions).

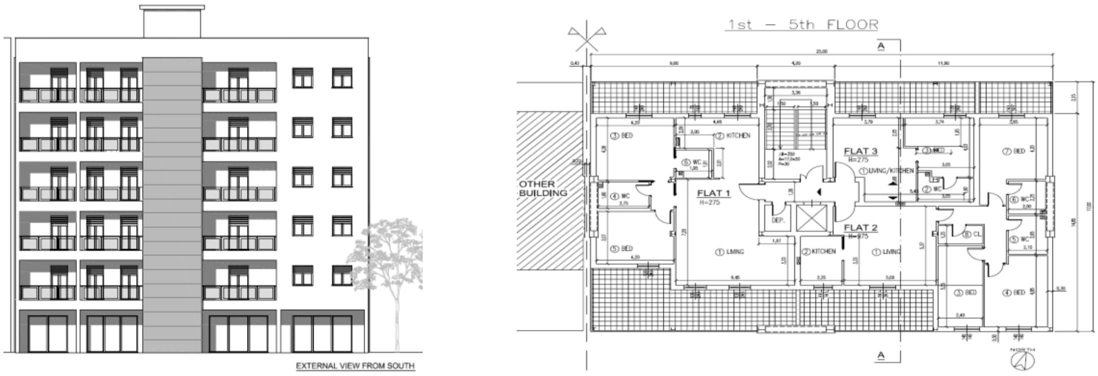



**Figure 37.** Frontal view and apartment floor

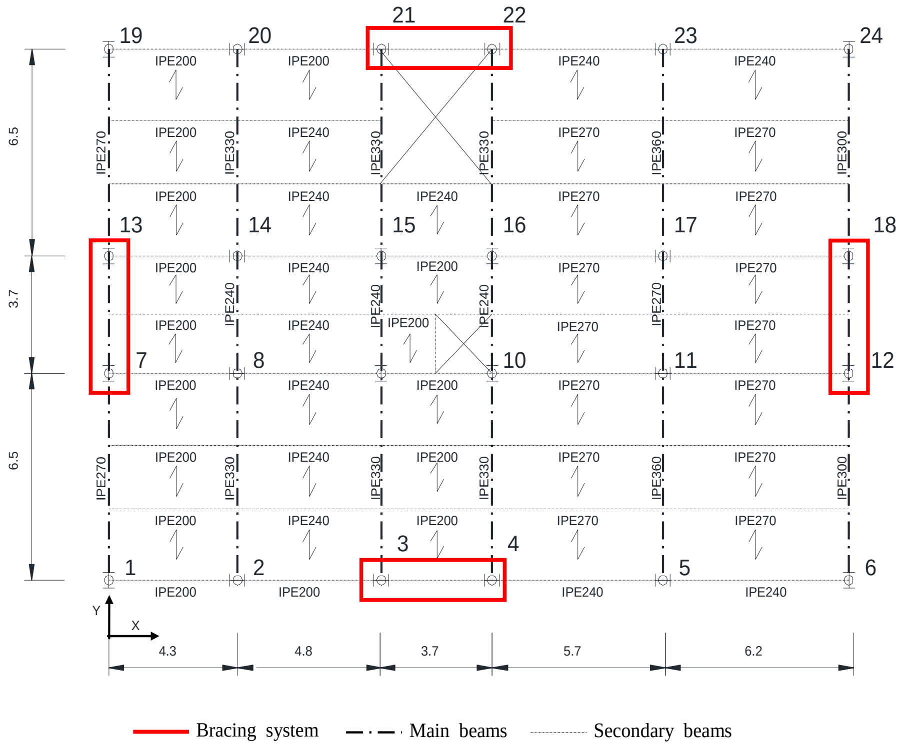



**Figure 38.** Distribution of the gravity-resistant structure and seismic-resistant systems

All external walls are low colour interrupted by stone cladding in correspondence of structural dissipative devices that are present in each external view. For floor coverings, soft grey porcelain stoneware slabs were chosen for their technical characteristics, such as durability and easy maintenance and cleaning. Aluminium profile with low-emissivity double glazes are used for the transparent surfaces and in the shops shatterproof glasses are used. The considered case study is designed as having a gravity-resistant steel frame structure (floors, beams, columns) where beam to columns joints and restraints at the base of the columns can be considered as pinned connection. The gravity-resistant frame is connected to the innovative hybrid systems considered in this research project, i.e. in case A the HCSW systems is used, in case B the SRCW system is used, as later detailed.
The analysis are conducted by considering the seismic action evaluated with a reference probability of exceedance of 10 % in 50 years (TRN = 475 years). The earthquake motion is represented by the Eurocode 8 elastic acceleration response spectrum, for importance factor I = 1, reference peak ground acceleration ag = 0.193g, spectrum type 1, ground type A.

Task 8.2 - Development of a complete design of the case study A (HSCW).

The design of the case studies concerns some aspects that are not influenced by the particular seismic resistance system, as gravity load structural system and architectonical solutions to provide acoustic and thermal comforts. All horizontal and vertical partitions are designed to ensure acoustic and thermal comforts in respect with current Italian codes (DL311/2006 and subsequent additions). The following transmittance values are reached: external curtain: U=0,27 W/m2K; roof floor plan: U=0,29 W/m2K; underground walls: U=0,80 W/m2K. External curtains and floor plans respects all the acoustic standards according to Italian DPCM 05/12/1997 and subsequent additions; in particular the following acoustic limits are considered: sound insulation of the internal partitions R'w: > 50 dB(A), standardized sound insulation of facade D2mnt: > 42 dB(A), level of impact noise L'w: < 55 dB(A), noise produced by plants: < 35 dB(A).

The structural design of this task, considering the HCSW seismic resistant system, includes:
- a preliminary design method oriented to rapidly evaluate the effectiveness of different choices of characteristic parameters (i.e. coupling ratio and wall slenderness) found in WP4 in order to define the optimal solution;
- 2D nonlinear static and dynamic analyses for the evaluation of the effectiveness of the post-elastic behaviour and for the refinement of the wall, reinforcement and link sizing;
- 3D analyses for the final assessment of the solution and the design of details.

**Table 2.** Design of the steel links

| CR | $M_{w,Rd}$ | $n_{links}$ | $l_w$ | $l_{links}$ | section | $e_S$ | $e_L$ | $f_y$ | $M_{p,link}$ | $V_{p,link}$ | $\gamma_w$ |
|---|---|---|---|---|---|---|---|---|---|---|---|
| (-) | (kNm) | (-) | (mm) | (mm) | (-) | (mm) | (mm) | (MPa) | (kNm) | (kNm) | (-) |
| 0.60 | 4050.67 | 6 | 2100 | 500 | IPE270 | 289 | 542 | 355 | 127.00 | 351.44 | 1.75 |
| 0.70 | 4050.67 | 6 | 2100 | 500 | IPE330 | 340 | 637 | 355 | 208.04 | 489.60 | 1.84 |
| 0.80 | 4050.67 | 6 | 2100 | 500 | IPE400 | 392 | 734 | 355 | 333.41 | 681.26 | 1.72 |

**Table 3.** Design of the steel side columns for CR = 0.60

| Storey | Section | $N_{Ed}$ | $M_{Ed}$ | $\sigma_{Ed}$ |
|---|---|---|---|---|
| | | (kN) | (kNm) | (MPa) |
| 6 | HE400B | 483.23 | 48.32 | 51.21 |
| 5 | HE400B | 966.46 | 96.65 | 102.59 |
| 4 | HE400B | 1449.69 | 144.97 | 154.14 |
| 3 | HE400B | 1932.92 | 193.29 | 205.86 |
| 2 | HE400B | 2416.15 | 241.61 | 257.76 |
| 1 | HE400B | 2899.38 | 289.94 | 309.84 |

This design path, starting from preliminary analyses for the choice of the most important parameters and leading to final safety verification and design of the details, is described in an extended way in the D8.1-Part2A whose contents can be used as guidelines for the design of this structural system. A summary of the design procedure is reported in the following.
The preliminary design provided the steel link and column dimensions reported in the following tables. The preliminary (simplified) design procedure was also used to explore and compare the seismic performance of different solutions in order to optimize the final structural system. The structural component dimensions were confirmed in the other refined nonlinear analyses on 2D and 3D models, including also the control of second order effect at ULS and the deformation check at

SLS. Nonlinear static and dynamic analyses are reported in Figure 39 and Figure 40 to show the progressive yielding of the steel links and the exploitation of their dissipative properties. The reported curves show that the dissipation properties are exploited with very low damage in the RC wall.

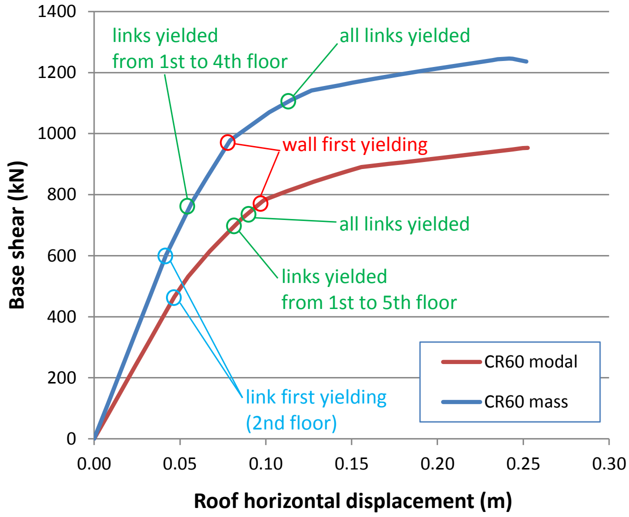



**Figure 39.** Pushover curves for the HCSW system designed with CR = 0.60 with indication of the hinge state

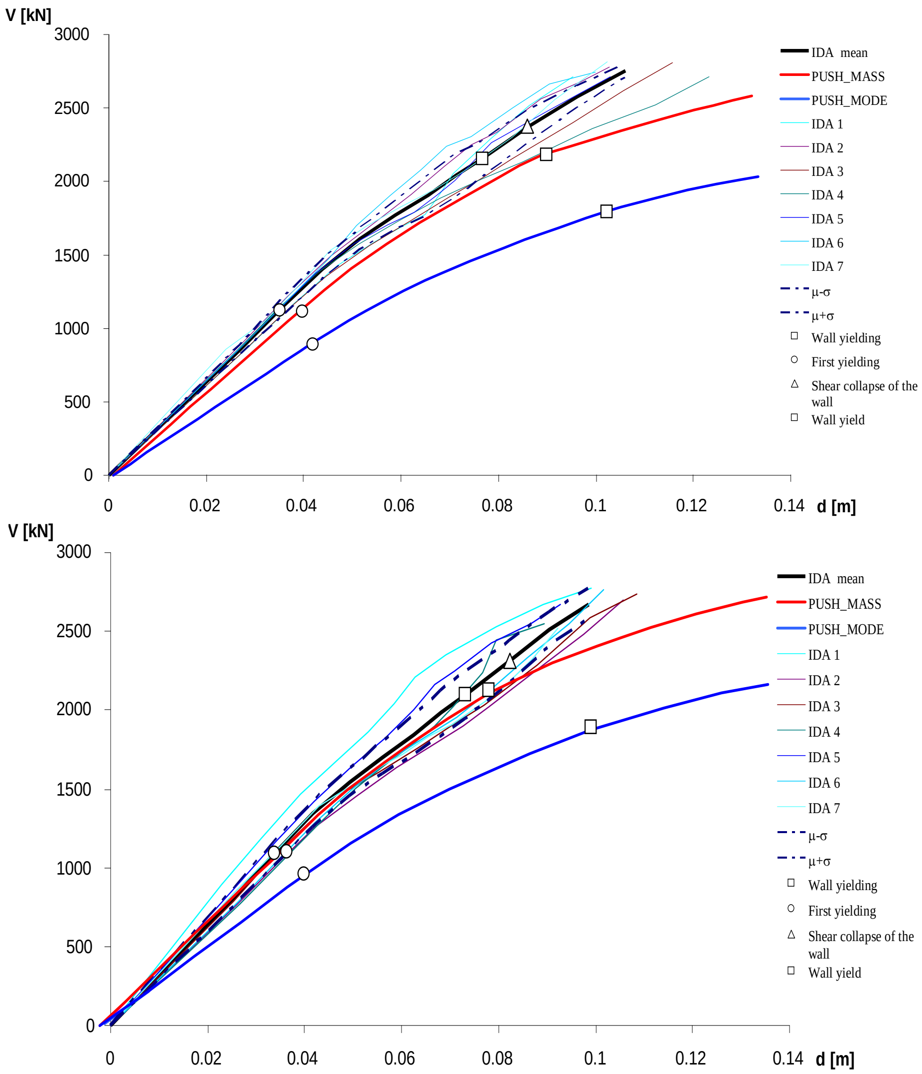



**Figure 40.** Comparison between static pushover and dynamic pushover curves in x and y direction

A selection of the most interesting details concerning the dissipative components are reported in Figure 41 where the link and its connection with the concrete wall is depicted. The complete description of details and total amount of row material is reported in D8.1-Part 3 (Drawing and Technical documents), the D8.1-Part 4A is an interactive, 3D model produced by Tekla software, containing all the detail of components and connections.

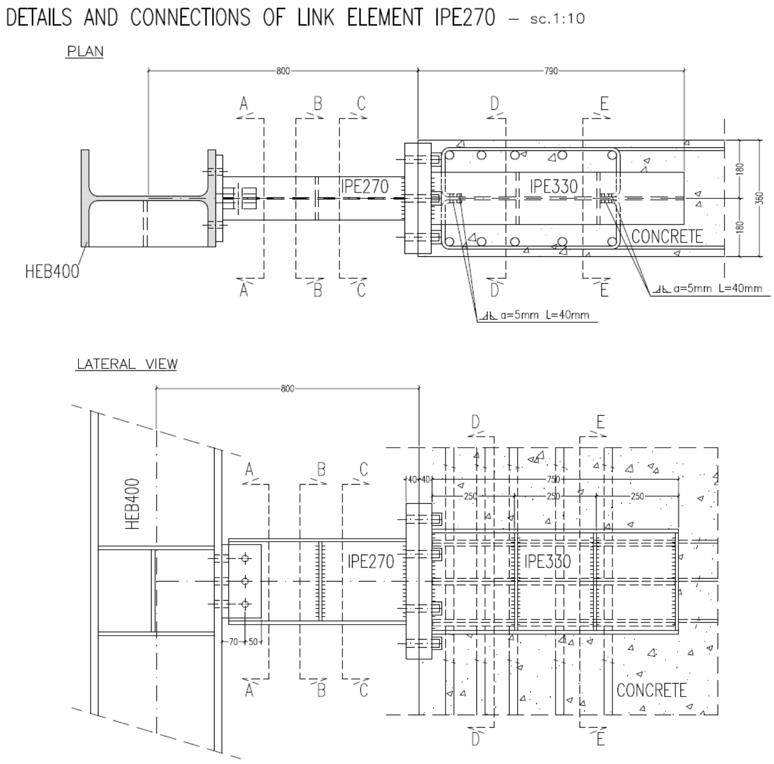



**Figure 41.** HSCW system: structural details

Task 8.3-Development of a complete design of the case study B (SRCW).

Even if the structural behaviour of case B (SRCW) is notably different from the case A (HSCW), the structural components occupy almost the same space and they are located in the same positions. Consequently, there are not a significant difference with respect to case A for what concerns gravity load systems, functional distribution and architectonical details, reported in D8.1-Part 1. The most interesting aspect discussed in this task concerns the design of the seismic-resistant structures (SRCW), consisting of RC infill walls conceived to control the formation of diagonal struts in the infill walls and dissipating energy by means of vertical steel components not connected to the concrete wall.
A design procedure permitting to define an optimal solution by starting from simplified analyses is proposed and it includes:
- a preliminary design method oriented to rapidly evaluate the effectiveness of different choices of characteristic parameters (i.e. length of the dissipative components, RC wall thickness and global aspect ratio of the bracing) found in WP5 in order to define the optimal solution;
- 3D analyses for the final assessment of the solution and the design of details.
This design path, starting from preliminary analyses for the choice of the most important parameters and leading to final safety verification and design of the details, is described in an extended way in the D8.1-Part2B whose contents can be used as guidelines for the design of this structural system. A summary of the design procedure is reported in the following.
The preliminary design procedure is subdivided in steps, each commented in D8.1-Part2B with details, suggestions and possible critical aspects as well as application to the considered case study. In particular, design consists of 9 steps, it is force-based and is applied by considering a simple static determined scheme. This procedure provides the dimensions required for all the structural components: dimensions (cross-section and length) of dissipative elements and characteristics of their connection, dimensions of beams and columns, thickness of the RC wall, reinforcements due to shear and bending, edge elements and concrete-steel connection. In the following tables the dimensions of dissipative elements and RC wall resulting from the calculation are reported.
The structural component dimensions were confirmed in the other refined nonlinear analyses on 2D and 3D models, including also the control of second order effect at ULS and the deformation check at SLS. The following figure describe the nonlinear response in x direction and y direction obtained by static and multi-record dynamic analyses. The curves are extended to the ULS seismic intensity

and they show the moderate damage of the RC wall also for strong earthquake. As usual, the dynamic pushover curve is stiffer than the curve provided by a static analysis as a consequence of higher mode contributions, even if the difference observed in this case are smaller with respect to the previous solution A (HSCW).

**Table 4.** Results of the design of the ductile elements and wall dimensions

| Storey | $N_{Ed}$ | $f_y$ | Sections |
|---|---|---|---|
| [---] | [kN] | [MPa] | [---] |
| L1 | 2017.596 | 235 | HE260A |
| L2 | 1509.817 | 235 | HE220A |
| L3 | 1032.446 | 235 | HE180A |
| L4 | 615.895 | 235 | HE140A |
| L5 | 290.574 | 235 | HE140A |
| L6 | 86.890 | 235 | HE140A |

| Storey | $N_{Ed}$ | $t_w$ | $l_b$ | $N_{Rd1}$ | $N_{Rd2}$ | $N_{Rd}$ |
|---|---|---|---|---|---|---|
| [---] | [kN] | [m] | [m] | [kN] | [kN] | [kN] |
| L1 | -1586.69 | 0.22 | 0.426 | 1593.240 | 1682.461 | 1593.240 |
| L2 | -1525.76 | 0.22 | 0.409 | 1529.013 | 1614.638 | 1529.013 |
| L3 | -1403.89 | 0.22 | 0.376 | 1405.457 | 1484.162 | 1405.457 |
| L4 | -1221.08 | 0.2 | 0.360 | 1221.526 | 1289.931 | 1221.526 |
| L5 | -588.574 | 0.16 | 0.217 | 591.574 | 624.702 | 591.574 |
| L6 | -290.825 | 0.16 | 0.107 | 292.043 | 308.397 | 292.043 |

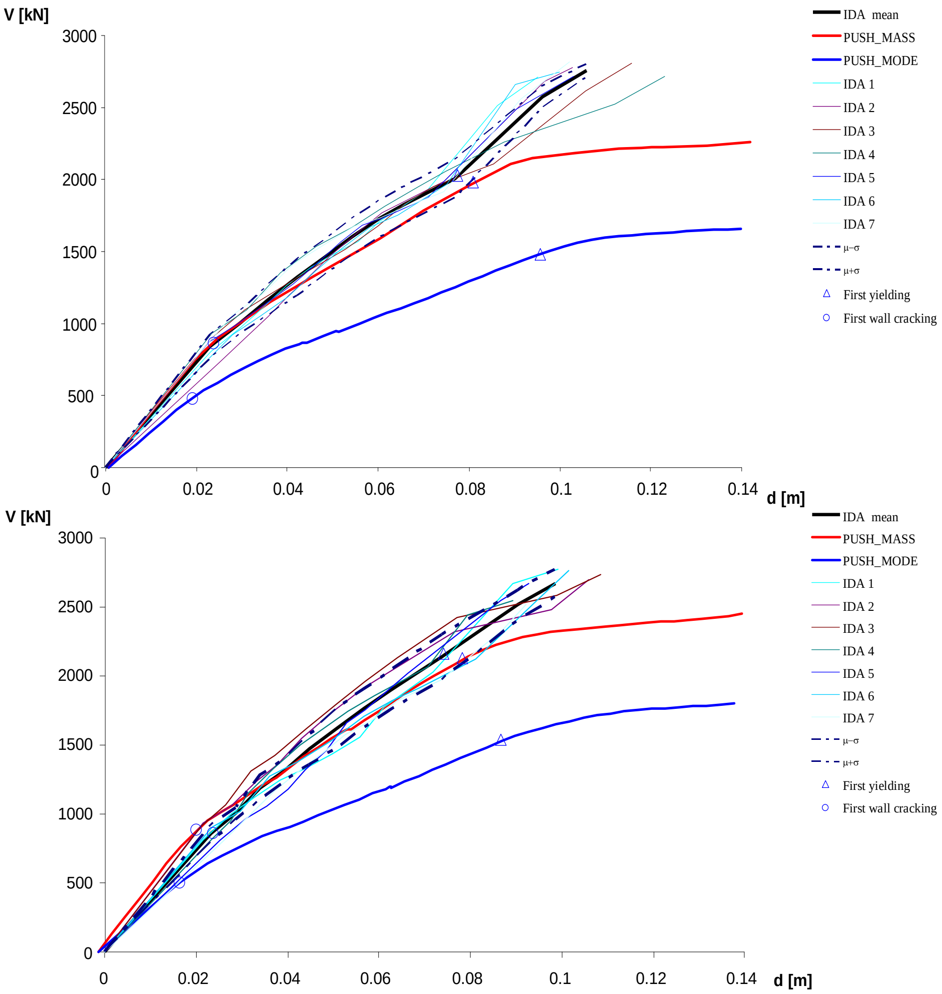



**Figure 42.** Comparison between static and dynamic pushover in x and y direction

A selection of the most interesting details concerning the dissipative components are reported in Figure 43 where the steel side dissipative element and the beam-to-column connection are depicted.

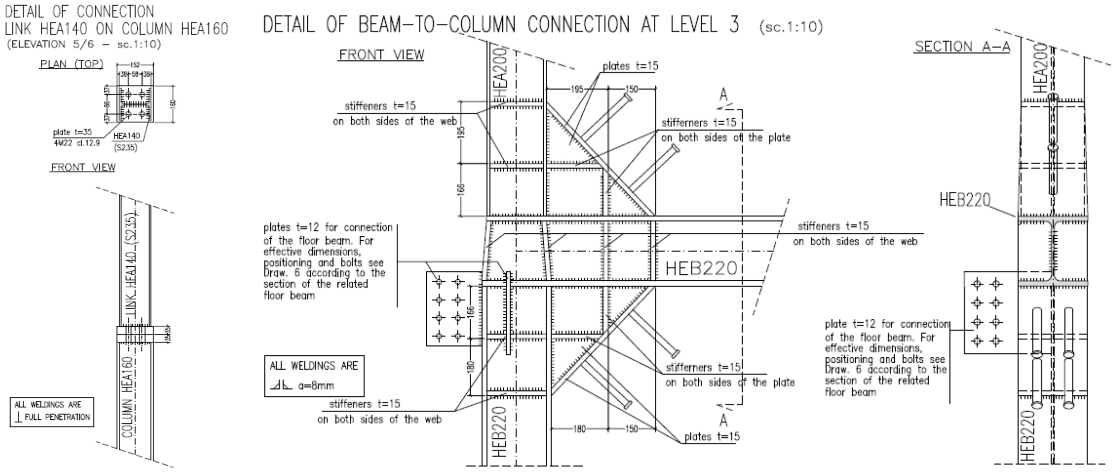



**Figure 43.** SRCW system: structural details

Task 8.4-Final evaluation of the performance and Workshop.

The case studies, obtained as outcomes of proposed design methods, were discussed both in the final workshop in Turin and in a preliminary presentation in Salerno (supported by OCAM while not strictly due by the contract). The final workshop was held in a session of a conference on steel construction (CTA2013) in order to have a large audience (366 participants) form research centres, designer and construction companies. During the final conference and meetings, the evaluation of the proposed solutions lead to a number of final conclusions concerning the structural safety, the design and construction process, the consumption of row material and the feasibility for building with different use, as parking, stores and apartments. Conclusions and potential impacts are reported in the following sections 2.4 and 2.5.

## *2.3 Conclusions task by task*

WP1 Critical evaluation of existing solutions and new proposals

Task 1.1 Critical evaluation and comparison of European and extra European codes
The conclusion of this task is that Eurocodes provide rather limited information on steel and concrete hybrid seismic resistant structural solutions and extra European codes do not provide much more indications. Thus, the development of design guidelines for steel and concrete hybrid structural systems are a necessary step.

Task 1.2 Overview of technical literature and existing solutions
The overview on the technical literature on HCSW systems and relevant existing solutions evidenced that structural steel coupling beams provide a viable solution, that steel coupling beams should be preferably designed to yield in shear, and the importance of the coupling ratio in the overall structural behaviour. The overview on the technical literature on SRCW systems and relevant existing solutions evidenced studies and applications involving steel frames with concrete infill walls, steel shear walls, and composite shear walls, either designed to carry the full seismic shear or as structures designed as a dual system.

Task 1.3 Evaluation of solutions in relation to specific performance of interest in the European area
The analysis of the technical literature highlighted that HCSW systems can achieve high efficiency in resisting lateral loads and dissipating seismic energy if the coupling beams are correctly designed while SRCW systems are suitable for moderate seismic events producing limited cracks in the reinforced concrete wall, unless the wall is replaceable and the boundary system that remains undamaged.

Task 1.4. Evaluation of feasibility and construction complexity of examined solutions
The analysis of the technical literature shown that workshop aspects in HCSW and SRCW systems do not present major critical aspects and that there are no particular demands and issues concerning their construction.

WP2 Performance analysis of solutions involving innovative HCSWs

Task 2.1 Definition of a reduced set of HCSW systems
An innovative HCSW system, made by a reinforced concrete shear wall with dissipative steel links and steel side columns was defined in this task. 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.

Task 2.2. Some case studies considered paying attention to the most diffused requirements in the European building market
In this task a set of case studies was defined to evaluate the effectiveness and feasibility of the proposed innovative seismic-resisting HCSW system.

Task 2.3 The dimensions of resisting elements assigned by means of preliminary design rules
In this task the preliminary design of the proposed innovative HCSW system was made considering the following parameters: number of reinforced concrete walls, thickness and length of reinforced concrete walls, length of the steel links and their connection type (rigid or pinned). It was concluded that increasing the length of the reinforced concrete wall reduces the bending moment in the links and allows decreasing their cross sections, however, the bending moment at the base of the shear wall becomes too high resulting in non-realistic reinforcement schemes. Results showed that a the preferable height-to-length ratio of the wall in order to limit the bending moment at the base is about 10, thus, a relatively slender wall that should work more in flexure than in shear. It was also proposed to use the side steel columns to reduce the length of the steel links and consequently their bending moment demands.

Task 2.4 The seismic behaviour assessed by means of incremental dynamic analyses
In this task it was concluded that the proposed HCSW system has interesting potentialities (ductile behaviour with steel links yielding while limited damage occurs in the reinforced concrete wall, the interstorey drifts up to collapse are quite regular regardless of the non-simultaneous yielding in the steel links) and that the preliminary design method requires additional studies to clarify the relationships between wall over-strength and to provide criteria to support the dimensioning of the various elements.

WP3 Performance analysis of solutions involving innovative SRCWs

Task 3.1 Definition of a reduced set of SRCW systems
In this task a set of SRCW systems as considered in the Eurocodes was selected for the work to be developed in subsequent tasks. Preliminary analyses were carried out to understand the influence of some parameters, such as the wall width, the concrete strength, the diameter of the stud shear connectors and their spacing, in order to select the most promising solutions.

Task 3.2. Some case studies considered paying attention to the most diffused requirements in the European building market
The buildings chosen for the performance analysis of solutions involving SRCWs are the same of Task 2.2 as the gravity-resisting structure is not interested by the lateral-resisting system because pinned connections are considered between beams and columns and between beams and seismic resistant elements.

Task 3.3 The dimensions of resisting elements assigned by means of preliminary design rules
In this task the preliminary design of SRCW systems was made according to the available Eurocode prescriptions. It was observed that capacity design rules are not available.

Task 3.4 The seismic behaviour assessed by means of nonlinear analyses
Results of pushover analyses shown that the designed SRCW systems are very stiff and capable of withstanding high horizontal loads, with limitation of second-order effects and fulfillment of service state criteria easily satisfied. However, it was observed that fragile failure mechanisms occur (problems at beam-to-column connections and crushing of the concrete at a diagonal band), thus, the global behaviour has insufficient ductile to comply with the behaviour factor adopted in the design. This unsatisfactory behaviour is due to the lack of a specific capacity design procedure that allows to control the formation of a proper dissipating mechanism.

WP4 Design of HCSW systems

Task 4.1 Preliminary analyses of the connection typologies
Two typologies were considered for the connections between steel link and reinforced concrete wall, i.e. in one case the bending moment transferred by the link to the wall is resisted by shear studs, in the other case the moment transferred by the link to the wall is balanced by a couple of vertical reactions provided by the compressed concrete. Two typologies were considered for the splice connection between steel link splices, i.e. connection placed at a distance from the concrete wall sufficient to allow an easy bolting of the replaceable part, connection at the face of the wall with threaded bushings to allow replacement of the dissipative tract of the steel link. The

connection between the steel link and the additional steel side column is made by a conventional web angle connection bolted to the column flange.

Task 4.2 Definition of a design procedure
A design procedure that inherits recommendation for capacity design from other structural systems involving similar dissipative mechanisms in the links, i.e. eccentric braces in steel frames, as well as indication to reduce damages in the reinforced concrete wall, was defined.

Task 4.3 Evaluation of effectiveness of design provisions proposed
Nonlinear static and dynamic analyses shown a ductile global behaviour with regular interstorey drifts. However, some limitations of the proposed design approach were highlighted, i.e. lack of an effective control of the sequence of yielding of the links on one side and the reinforcements in the wall on the other side, excessive quantities of required reinforcements in the concrete wall in some cases. This limitations pushed the development of an improved design approach.

Task 4.4 Final assessment of the design procedure
Given the critical points and limits evidenced in the previous task, a new design approach based on limit analysis equilibrium having as key design parameter the coupling ratio, was developed. The HCSW systems designed with such an approach gave quite good results, as assessed through extensive incremental nonlinear static and dynamic analyses, in terms of ductile global behaviour, uniform distribution of interstorey drifts, wall effectively protected against damage as the dissipative steel links yield long before the first yielding in the wall reinforcements, hence starting dissipating seismic energy while the wall is still in its elastic range.

Task 4.5 Evaluation of a design based on the linear dynamic analysis with response spectra and structural reduction factor
Although a structural reduction factor was identified, the linear dynamic analysis provides a distribution of stresses quite different from the distribution of stresses from limit analysis that provided the best design outcomes. Thus, it was concluded that a design based on linear dynamic analysis with response spectra and structural reduction factor currently cannot provide satisfactory results and more studies are required.

WP5 Design of SRCW systems

Task 5.1 Preliminary analysis of the wall morphology and connection typology
After that preliminary pushover analyses on SRCW systems designed according to Eurocode 8 highlighted critical issues, an innovative SRCW system was proposed. The morphology and connection detailing of the proposed innovative system were studied to ensure the desired structural behaviour.

Task 5.2 Definition of a design procedure
In this task a design procedure was developed considering the latticed statically determined scheme representing the limit behaviour of the for the proposed innovative SRCW system. It is concluded that the procedure is straightforward but attention should be given to the fact that the limit structural scheme adopted may not represent the behaviour of the system especially in the linear range for weak earthquakes.

Task 5.3 Evaluation of effectiveness of design provisions proposed
Static nonlinear analyses demonstrated that the yielding pattern is characterized by plastic strains only at the ductile elements, according to the hierarchy principles involved in the design methodology, and show struts developed in the concrete panels. It was observed that simplified models based on the design scheme provide excellent approximation of the results of advanced nonlinear finite element models.

Task 5.4 Final assessment of the design procedure
The numerical analyses for the designed SRCW systems carried out with refined finite element models highlighted that the proposed rules for capacity design are able to ensure the formation of the plastic mechanism involving only the lateral vertical elements while preserving the wall from crushing.

Task 5.5 Evaluation of a design based on the linear dynamic analysis with response spectra and structural reduction factor
The results on the determination of the reduction factors derived from the cases designed were not fully satisfactory as the computed factors are lower than those adopted in the design in the cases of non-regular systems.

WP6 Experimental based models for HCSWs

Task 6.1. Design of the experimental campaign
In this task the specimens were designed to characterize the performance of the connection of the seismic link embedded in a concrete shear wall and the efficiency of the capacity design of such a system. Additional tests were designed in order to evaluate the ultimate capacity of the link-to-wall connection.

Task 6.2. Supply of raw material and building of specimens
The supply of the material and building specimens was made by OCAM as scheduled, also including the planned additional specimens.

Task 6.3. Displacement controlled tests under cyclic path with increasing amplitude
The experimental results shown that the clearance between bolts and holes in the link-to-column connection has a negative influence of the cyclic behaviour of the tested steel links, with pronounced pinching phenomena due to the relative displacements between the connected elements. Improvements in the link-to-column connection made for the second connection typology reduced the relative displacements between the connected elements, producing hysteresis cycles considerably fatter and with pinching phenomena reduced. Regarding the connection between the link and the wall, the monotonic tests highlighted a basically linear global behaviour up to the maximum applicable load.

Task 6.4. Constitutive models of local behaviour based on experimental tests
In this task it was concluded that a simple frame model with un dimensional elastoplastic spring is adequate to model the behaviour of the steel link, provided that the actual value of the yield stress is identified.

WP7 Experimental based models for SRCWs

Task 7.1 Design of the experimental campaign
The specimens to be tested were defined and designed according to the indications and results of the preliminary study made in WP5.

Task 7.2 Supply of raw material and building of specimens
The supply of the material and building specimens was made by OCAM as scheduled.

Task 7.3 Experimental tests on the connections and steel side elements
The experimental activity on the shear connections demonstrated that their expected strength was exceeded and failure occurred in the studs (shearing of their shank) while no concrete crushing was observed. The shear connection did not withstand the prescribed number of cycles at the loading level equal to 75% of their monotonic experimental strength, as requested for shear connection in seismic resistant elements, whereas the specimens withstood the cyclic loading carried out with the maximum amplitude equal to 50% of their monotonic experimental strength. The experiments carried out on the side steel elements demonstrated their capability in withstanding important deformations with the formation of the plastic hinge under combined axial-bending actions. This condition is necessary to ensure the formation of the ultimate resisting mechanism assumed for the dissipative elements that are not actually pinned at their ends.

Task 7.4 Experimental tests on the downscaled specimen of the structural system
The experiments performed on the downscaled specimen demonstrated that the plastic mechanism assumed in the design procedure (formation of the diagonal strut and plasticization of the side elements) can actually develop.

Task 7.5 Development of constitutive models based on experimental tests
The results obtained on the downscaled specimen were used for the evaluation and calibration of mechanical model that was specifically developed for the proposed innovative SRCW.

WP8 Case studies

Task 8.1 Functional and dimensional characteristics of case studies
Both the proposed solutions provided a wide flexibility in the design, as demonstrated in the chosen case studies where parking, commercial spaces, storerooms and apartments are present. This is mainly due to the capacity of the two systems in providing an high shear strength with small dimensions, thanks to the high stiffness of the concrete component.

Task 8.2 Development of a complete design of the case study A (HSCW)
The proposed design method for the HCSW systems was reviewed and applied in the definitive form, potentially usable as guidelines. Results of nonlinear static pushover analysis and nonlinear

multi-record incremental dynamic analysis show that the designed system has lateral stiffness able to control the entity of the interstorey drift well below the accepted values, the interstorey drift distribution is rather uniform over the height of the building, the steel dissipative links are activated before the wall is yielded. Thus, it is concluded that behaviour of the designed HCSW system satisfy the targeted structural behaviour under the design seismic input. No special problems arose in the detail design at the intersection between structural and non-structural components. The seismic resistant elements occupy a small space in plan and they do not introduced significant restraints in the functional distribution.

Task 8.3 Development of a complete design of the case study B (SRCW)
The advantages of the innovative infill RC walls proposed confirmed their effectiveness in the design of the case studies and a the final design procedure is presented. As for the other considered seismic resistant system, the design can be developed with different level of details and the reported design procedure can be used as a guideline to approach the final solution by starting from preliminary design methods. Results of nonlinear static pushover analysis and nonlinear multi-record incremental dynamic analysis show that the designed system has lateral stiffness able to control the entity of the interstorey drift well below the accepted values, the steel dissipative ductile elements are activated after the cracking of the infill wall and these latter remain in elastic field. Thus, it is concluded that behaviour of the designed SCRW system satisfy the targeted structural behaviour under the design seismic input. No special problems arose in the detail design at the intersection between structural and non-structural components. The seismic resistant elements occupy a small space in plan and they do not introduced significant restraints in the functional distribution.

Task 8.4 Final evaluation of the performance and Workshop
Main general conclusions about outcomes of case studies and proposed solutions, discussed in the final workshop in Turin and in a preliminary presentation in Salerno (not strictly due by the contract, supported by OCAM), are reported in the following.
Design. the preliminary design methods proposed in WP4 (plastic method for HWSC) and WP5 (statically determined method for SRCW) confirmed their feasibility in real cases and their simple application requiring few computation.
Structural safety. The results obtained in term of structural safety and damage level after an earthquake confirmed previous expectations for what concerns the seismic capacity and dissipative properties, once local details are designed as in WP4/WP5. As a consequent result, the provisions of EC8 seems to be not adequate to provide the capacity and the structural behaviour factor suggested.
Construction. The final computation of row material quantities confirmed that construction costs enjoy some general benefits from a rational use of materials with respect to steel bracings systems or RC wall systems: reinforced concrete is economically used to provide required stiffness and shear strength, steel is used for dissipative components (replaceable after seismic events). Furthermore, the SRCW system also permits to reduce the headed studs. Architectural and structural details (others than the seismic details studied in the previous WPs) developed in the case studies have been judged as economically effective by the construction companies OCAM and SHELTER SA, for what concerns raw material consumption, construction process cost and maintenance.
Functional and architectonic issues. Both the HSCW and SRCW systems have an high shear strength capacity and this permits to reduce the number of seismic resistant elements, making more flexible the distribution of internal space at the different floors and reducing the intersection with plant piping.

### *2.4 Exploitation and impact of the research results*

Structural design and seismic performance
For both the systems, a preliminary design method is proposed, its application is simple and the results provide a reliable basis for a full detailed analysis and verification of the global seismic behaviour and of the detail properties, as demonstrated in the case studies. The design methods can be used as guidelines by designers and may promote the diffusion of these structural solutions. The results obtained in term of structural safety and damage level after an earthquake confirm the initial expectations for what concerns the seismic capacity and dissipative properties, once local details are designed as in WP4/WP5. Both the innovative structural systems are competitive with respect to other traditional (steel and concrete) solutions because they permit to decide in advance the damage level expected after an earthquake and they permit a fast and economic restoration of the seismic performance by means of the simple substitution of the dissipative elements.

Code provisions
The review of European and extra-European design codes pointed out lack of information on specific indications for seismic design of hybrid steel-concrete structural systems as the ones considered in this research project. In particular, only general principles and limited detailing recommendations can be found in Eurocode 8 (EN 1998-1).
Furthermore, the preliminary analyses leading to the final assessment showed that the provisions currently contained in Eurocode 8 (EN 1998-1) seems to be not adequate in order to provide the capacity and the structural behaviour factor suggested. The analyses developed during the project and the research results can provide useful information for a the revision and the extension of the relevant Eurocode 8 part.

Experimental tests and models
The experimental test program was mainly conceived to provide results specifically oriented to the design of the two innovative seismic resistant systems. In addition, some of the tests involves the behaviour of sub-components and the results can be useful also in other situations involving steel and concrete structure. This is the case of cyclic tests on headed stud connections under cyclic behaviour that may be of interest in more general situations involving steel-concrete connection under variable actions.

Construction process and costs
Benefits deriving from a rational use of materials with respect to traditional steel bracings systems or RC wall systems, make the proposed innovative hybrid systems potentially interesting for the construction market. The feasibility and the limited costs were confirmed by the judgement of the industrial partners and they are documented in details in the case studies reports.
The structural detailing studied with the support of the industrial partner is effective and does not introduce additional complexity in the construction process, if compared with structural solutions providing the same seismic performance. The small costs required for the substitution of dissipative elements after an earthquake is an additional value of the proposed innovative HCSW and SRCW systems.

Functional and architectonic issues
The proposed innovative HSCW and SRCW systems have both an high shear strength capacity and this permits to reduce the number of seismic resistant elements, making more flexible the distribution of internal space at the different floors and reducing the intersection with plant piping. The proposed structural solutions do not require complex or special details for what concerns the intersection with non-structural components, i.e. curtain walls or insulation systems.

European policies
The proposed solutions are characterized by an high level of prefabrication and potential standardization, the raw material consumption is competitive with respect to alternative solutions and these aspects match with the European policies on energy consumptions and worker safety.

## *2.5 Conferences and dissemination of the results*

The research outcomes (experimental tests, structural models, static and dynamic analysis results, design approaches and proposed methodologies, case studies, feasibility, possible interests in the construction market in seismic area) were discussed both in the final workshop in Turin and in a preliminary presentation in Salerno (not strictly due by the contract, supported by OCAM). The final workshop was held in a special session of a conference on steel construction (The Italian Steel Days CTA 2014 Turin, September 30$^{th}$ to October 2$^{nd}$) in order to have a large audience from research centres, designer and construction companies. Workshop discussions provided interesting conclusions and suggestions included in this final report.

Preliminary results were presented on a number of international and national conferences, including the 15th World Conference on Earthquake Engineering and the XV Italian Conference on Earthquake Engineering ANIDIS 2013, revealing signs of interest by attendees in stimulating discussions and request for further details on the research development.

## *2.6 Publications*

Zona A., Braham C., Degée H., Leoni G., Dall’Asta A. Innovative hybrid coupled shear walls for steel buildings in seismic areas. Proceedings of the XXIII Italian Steel Structure Conference (C.T.A.), 9-12 October 2011, Ischia, Italy, pp. 741-748, ISBN 978-88-89972-23-6.

Zona A., Leoni G., Dall’Asta A., Braham C., Bogdan T., Degée H. Behaviour and design of innovative hybrid coupled shear walls for steel buildings in seismic areas. Proceedings of the 15th World Conference on Earthquake Engineering, 24-28 September 2012, Lisbon, Portugal, paper 2737.

Bogdan T., Zona A., Leoni G., Dall’Asta A., Braham C., Degée H. Design and performance of steel-concrete hybrid coupled shear walls in seismic conditions. Proceedings of the 4th International Conference on Computational Methods in Structural Dynamics and Earthquake Engineering (COMPDYN 2013) 12-14 June 2013 Kos, Greece, Paper 1295.

Zona A., Leoni G., Dall’Asta A., Scorpecci A., Bogdan T., Degée H. Seismic behaviour and design of innovative hybrid coupled shear walls. Proceeding of the XV Italian Conference on Earthquake Engineering (ANIDIS), Padova, June 30th - July 4th, 2013, paper D6.

Leoni G., Zona A., Dall'Asta A., Bigelow H., Hoffmeister B., Varelis G., Behaviour and design of innovative steel frames with RC infill walls. Proceeding of the XV Italian Conference on Earthquake Engineering (ANIDIS), Padova, June 30th - July 4th, 2013, paper F4.

Zona A., Leoni G., Dall’Asta A., Braham C., Bogdan T., Degée H. An innovative solution for earthquake resistant hybrid steel-concrete systems with replaceable dissipative steel links. Proceedings of the Fifth International Conference on Structural Engineering, Mechanics and Computation, 2-4 September 2013, Cape Town, South Africa, pp. 341-346.

Zona A., Leoni G., Dall’Asta A., Braham C., Bogdan T., Degée H., Design procedure for earthquake-resistant innovative hybrid coupled shear walls, Proceedings of the XXIV Italian Steel Structure Conference 30 September - 2 October 2013, Torino, Italy, pp. 693-700.

Leoni G., Zona A., Dall’Asta A., Bigelow H., Hoffmeister B., Varelis G. Design procedure for innovative earthquake-resistant steel frames with RC infill walls. Proceedings of the XXIV Italian Steel Structure Conference 30 September - 2 October 2013, Torino, Italy, pp. 701-708.

Zona A., Leoni G., Degée H., Dall’Asta A. Design of innovative seismic-resistant hybrid steel-concrete shear walls. Proceedings of IABSE Workshop 'hybrid2014 'Exploring the Potential of Hybrid Structures for Sustainable Construction, 22-24 June, 2014, Fribourg, Switzerland.

Leoni G., Carbonari S., Morici M., Tassotti L., Zona A., Varelis G.E., Dall’Asta A. Design of innovative seismic-resistant hybrid systems made of steel frames and reinforced concrete infill walls. Proceedings of IABSE Workshop 'hybrid2014' Exploring the Potential of Hybrid Structures for Sustainable Construction, 22-24 June, 2014, Fribourg, Switzerland.

Zona A., Leoni G., Degée H., Dall'Asta A. Nonlinear Seismic Analysis of Innovative Hybrid-Coupled Shear Walls, Proceedings of The Twelfth International Conference on Computational Structures Technology, B.H.V. Topping and P. Iványi, (Editors), Civil-Comp Press, Stirlingshire, United Kingdom, paper 164, 2014. doi:10.4203/ccp.106.164

Leoni G., Carbonari S., Morici M., Tassotti L., Zona A., Varelis G.E., Dall'Asta A. Nonlinear Seismic Analysis of Innovative Steel Frames with Infill Walls, Proceedings of The Twelfth International Conference on Computational Structures Technology, B.H.V. Topping and P. Iványi, (Editors), Civil-Comp Press, Stirlingshire, United Kingdom, paper 166, 2014. doi:10.4203/ccp.106.166

Manfredi M., Morelli F., Salvatore W. Component-Based Model and Experimental Behavior of a Dissipative Steel Link for Hybrid Structures, Proceedings of The Twelfth International Conference on Computational Structures Technology, B.H.V. Topping and P. Iványi, (Editors), Civil-Comp Press, Stirlingshire, United Kingdom, paper 165, 2014. doi:10.4203/ccp.106.165

Morelli F., Salvatore W. Structural Performance of an Innovative Composite Shear Wall System: Experimental Results, Proceedings of The Twelfth International Conference on Computational Structures Technology, B.H.V. Topping and P. Iványi, (Editors), Civil-Comp Press, Stirlingshire, United Kingdom, paper 167, 2014. doi:10.4203/ccp.106.167

Zona A., Leoni G., Degée H., Dall’Asta A., 2014. Design of innovative seismic-resistant steel-concrete hybrid coupled shear walls. Proceedings of the 37th IABSE Symposium Engineering for Progress, Nature and People, Madrid, Spain, Paper n.469.

Leoni G., Carbonari S., Morici M., Tassotti L., Zona A., Varelis G.E., Dall’Asta A., Design procedure and analysis of innovative steel frames with reinforced concrete infill walls. 7th European

Conference on Steel and Composite Structures, Napoli, Italy, 10-12 September 2014, Abstract 645-646, full paper on memory stick.

Zona A., Leoni G., Degée H., Dall'Asta A., Design procedure and analysis of innovative hybrid coupled shear walls. 7th European Conference on Steel and Composite Structures, Napoli, Italy, 10-12 September 2014, Abstract 691-692, full paper on memory stick.

## 3. Steel and concrete hybrid coupled shear walls

### *3.1 Existing systems*

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. The coupling beams provide transfer of vertical forces between adjacent coupled walls, which creates a coupling action that resists a portion of the total overturning moment induced by the seismic action. This coupling action reduces the moments that individual walls shear withstand and therefore a more efficient structural system is obtained as compared to the walls acting uncoupled. Furthermore, the coupling action provides a means by which seismic energy is dissipated over the entire height of the wall system if the coupling beams undergo inelastic deformations.

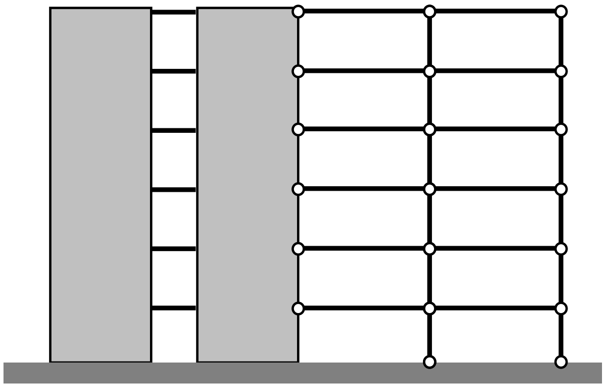


**Figure 44.** Example of conventional hybrid coupled shear wall system connected to a gravity-resisting steel frame with pinned beam-to-column joints.

Structural performance is strongly influenced by the coupling ratio CR. The CR represents the proportion of system overturning moment resisted by the coupling action. For example, CR = 50% implies that the coupling action resists half the imposed overturning moments, while the remaining half of the resistance is provided by wall moment reactions. 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 (Paulay and Priestley 1992) 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 diagonal reinforcement consists of relatively large diameter bars which must be adequately anchored and confined to avoid buckling at advanced limit states.

Structural steel coupling beams or steel-concrete composite coupling beams provide a viable alternative (El-Tawil S., et al. 2010), particularly for cases with restrictions on floor height. In contrast to conventionally reinforced concrete members, steel/composite coupling beams can be designed as flexural-yielding or shear-yielding members. The technical literature dealing with steel and concrete hybrid coupled systems evidences that coupling beams should be preferably designed to yield in shear. Pure steel coupling beams allow for an efficient fuse-type design of the link based on the link design procedures developed for eccentrically braced steel frames. The coupling beam-wall connections depend on whether the wall boundary elements include structural steel columns or are 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. During major earthquakes large seismic forces are transferred between individual wall piers through the coupling beams. Previous researches showed that shear walls with steel beams can have a satisfactory structural behaviour when subjected to seismic input (Figure 45 shows the experimental set-up of a HCSW specimen).

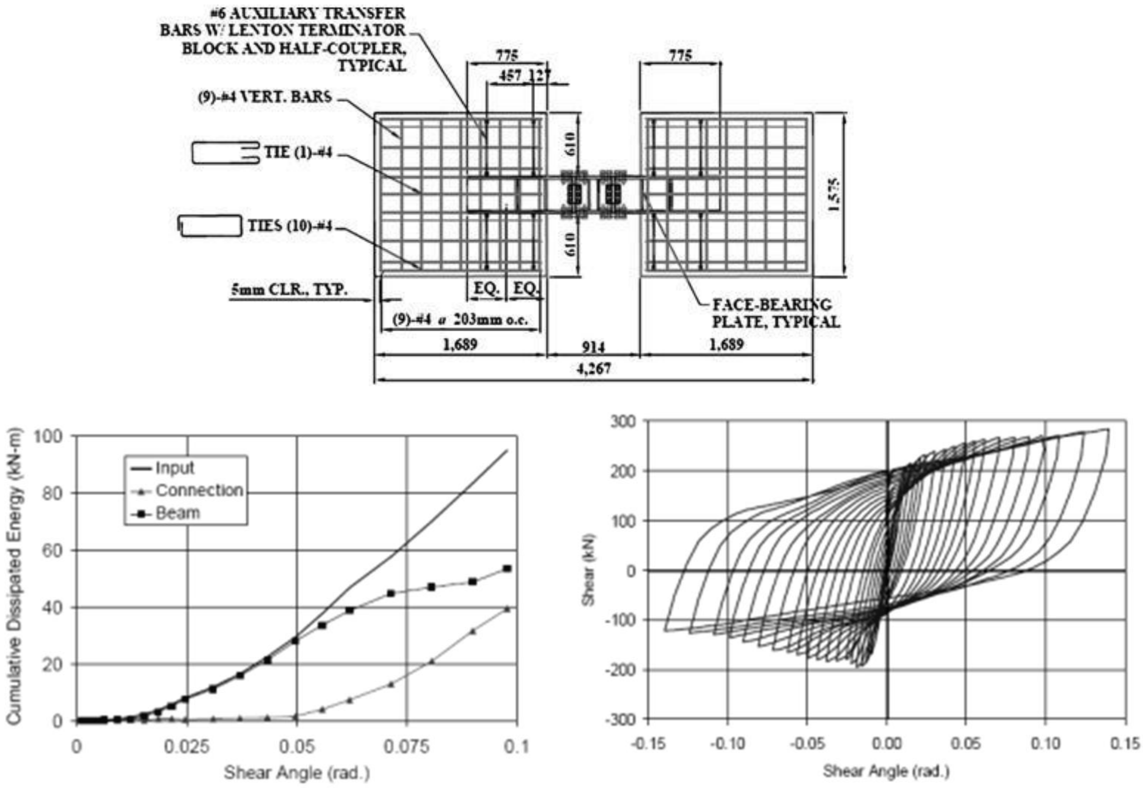



**Figure 45.** Examples of energy dissipation in a conventional HCSW system.

### *3.2 Available code recommendations and guidelins*

The evaluation of the European codes showed that in the current European situation HCWS systems are allowed by Eurocode 8 (EN 1998-1) but not extensively documented. Design principles are mainly based on similar situations considered in other sections of Eurocode 8 (concrete walls and conventional composite action). Composite structural systems are addressed in sections 7.3.1 (e), 7.3.2 and 7.10 of Eurocode 8. A configuration is depicted in Figure 7.2 of Eurocode 8 and here reported in Figure 46. The information provided is rather limited and deals essentially with: definition of behaviour factor q values; design principles ("P" clauses according to Eurocodes drafting standards) to be particularized for each specific design situations; suggestions for performing the structural analysis, with reference to sections 5, 7.4 and 7.7; a limited set of detailing rules. The situation is not much different in extra European codes, as previously reported.

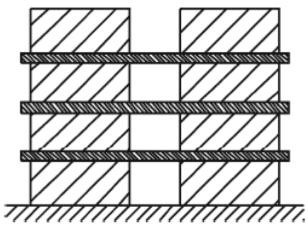


**Figure 46.** HCSW systems considered in Eurocode 8.

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 (ASCE 2009).

### *3.3 Proposed innovative HCSW system*

The innovative HCSW system that the partners found the most promising and agreed to study is made by a reinforced concrete shear wall with steel links, as depicted in Figure 47. 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 (Figure 48).

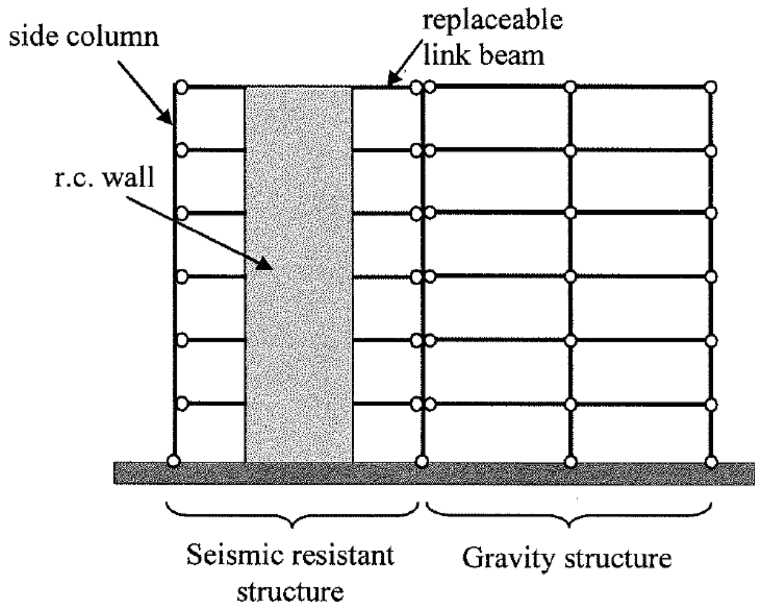



**Figure 47.** Example of innovative hybrid coupled shear wall system connected to a gravity-resisting steel frame with pinned beam-to-column joints.

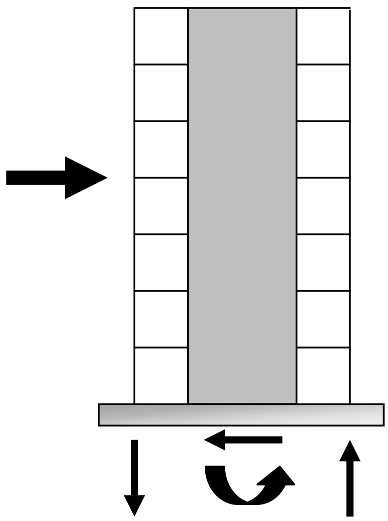


**Figure 48.** Resisting mechanism of the proposed innovative system.

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. The structure is simple to repair if the damage is actually limited to the link steel elements. To this end, it is important to use 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. As an alternative, links can include a replaceable fuse acting as a weak link where the inelastic deformations are concentrated while the other components of the system remain elastic. Clearly, the proposed hybrid system is effective as seismic resistant component if the yielding of a large number of links is obtained.

## 4. Steel frames with infill walls

### 4.1 Existing systems

Existing solutions of SRCW systems documented in the technical literature include: steel frames with concrete infill walls; steel frames with steel shear walls; steel frames with composite shear walls. In steel frames with concrete infill walls two design options are possible: infill walls are designed to carry the full seismic shear, i.e., structural scheme as depicted in the left model in Figure 49; the structures is designed as a dual system, i.e. structural scheme as depicted in the left model in the same Figure 49.

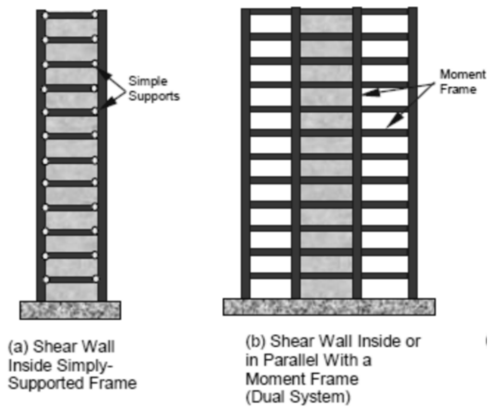



**Figure 49.** Design of steel frames with walls as (a) single system; (b) dual system.

The steel frame can be designed with fully or partially restrained connections with the concrete walls connected to the steel frame by classical shear stud connectors. The structural system is identified in the technical literature as particularly suitable for moderate seismicity but is also possible for design in high seismicity areas. From a constructive point of view steel frames considered in this solution can be built as usual while the critical point is the steel-concrete interface zone. Part of the connecting elements can be prefabricated. The required confining steel in the interface zones does not lead to important congestions. An important part of the concreting job has to be carried out on site.
A potential alternative and improvement can be obtained substituting concrete walls with steel walls. Steel frames with steel shear walls exhibit very good performances and the ductility level is very high and the cyclic behaviour is stable. Many lab tests validated the system and structures built according to these principles showed good performances during real strong earthquake. The frame-to-infill connection can be either welded or bolted, with welded connections identified as mechanically more efficient but more time-consuming in the execution stage. The system allows for a high level of prefabrication. Another viable alternative is the use of steel frames with composite shear walls (Figure 50). Composite shear walls are introduced to limit the infill slenderness. The system is mainly suitable for high seismic loads in high rise buildings and was validated experimentally. However, this composite system appears very expensive and combines drawbacks of concrete and steel infill walls.

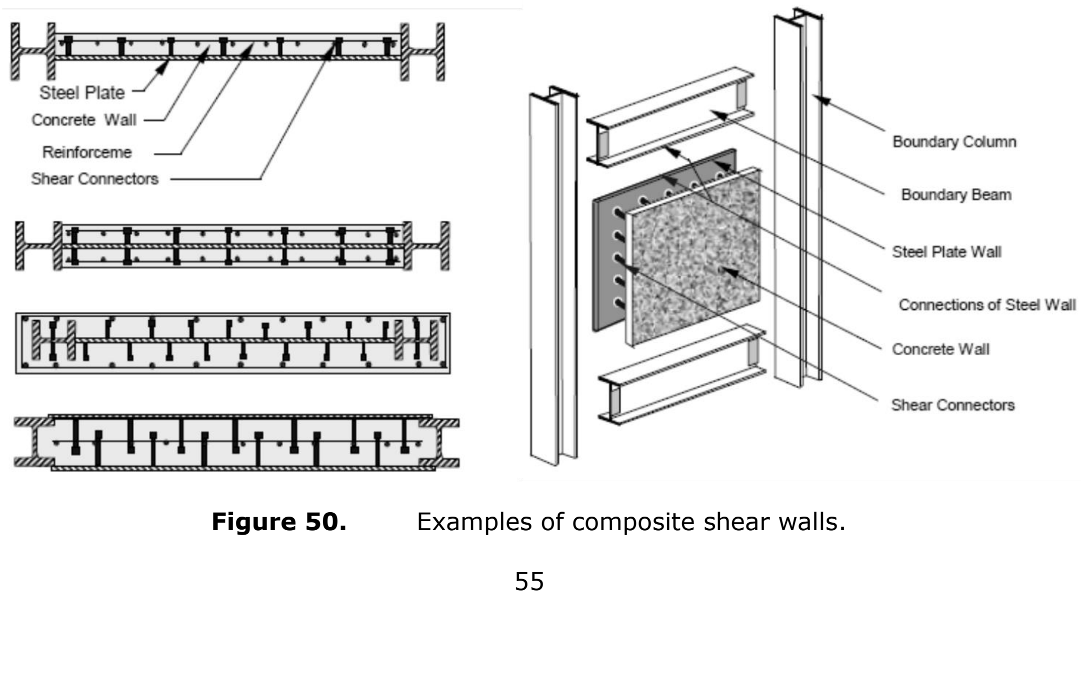



**Figure 50.** Examples of composite shear walls.

SRCW systems are particularly suitable for moderate seismic events when the energy is dissipated by concrete cracking, as those cracks are easily repairable with epoxy. More severe seismic events lead to more severe cracks that could be more difficult or impossible to repair. Nevertheless the concrete wall is replaceable as long as the boundary system remains undamaged. Figure 51 shows the experimental set-up of a two-story SRCW system and the hysteresis loop obtained for the upper specimen. In the course of the loading cycles with increasing story drift the maximum energy dissipated in each cycle increases during the first cycles and then decreases.

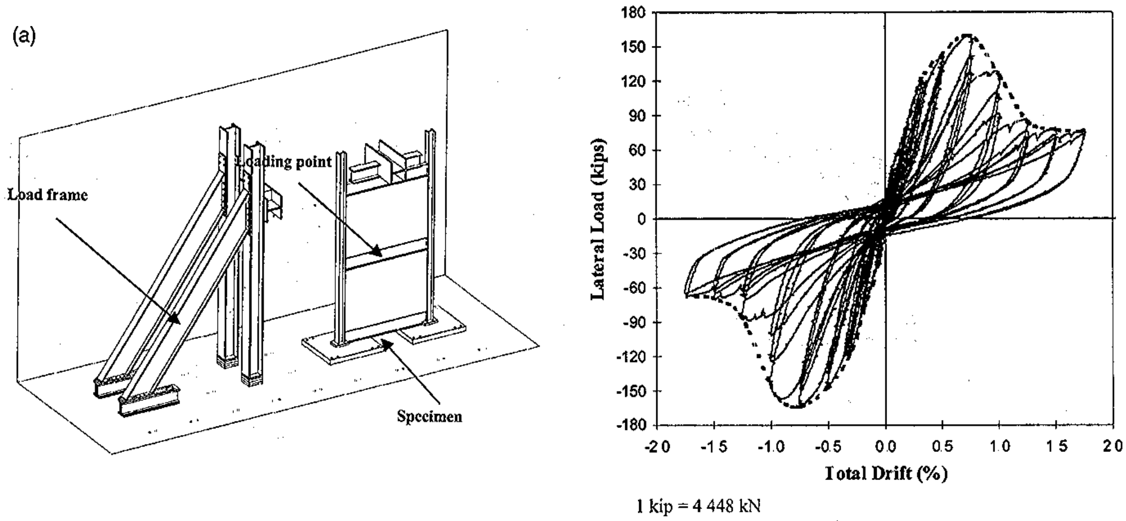



**Figure 51.** Examples of energy dissipation in a conventional SRCW system.

## *4.2 Available code recommendations and guidelines*

The evaluation of the European codes showed that in the current European situation SRCW systems are allowed by Eurocode 8 (EN 1998-1) but not extensively documented. Design principles are mainly based on similar situations considered in other sections of Eurocode 8 (concrete walls and conventional composite action). Composite structural systems are addressed in sections 7.3.1 (e), 7.3.2 and 7.10 of Eurocode 8. Three configurations are depicted in Figure 7.1 of Eurocode 8 and here reported in Figure 52. Similarly to HCSW systems, the information provided is rather limited and deals essentially with: definition of behaviour factor q values; design principles ("P" clauses according to Eurocodes drafting standards) to be particularized for each specific design situations; suggestions for performing the structural analysis, with reference to sections 5, 7.4 and 7.7; a limited set of detailing rules. The situation is not much different in extra European codes, as previously commented. Design recommendations following the criteria of Performance or Displacement Based Design (PBD and DBD) and Force Based Design (FBD) are missing.

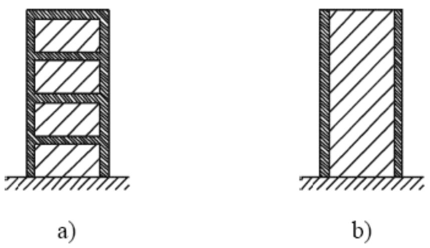



**Figure 52.** SRCW systems considered in Eurocode 8.

## *4.3 Proposed innovative system*

Analyses previously carried out on SRCW systems designed according to Eurocodes demonstrated an unsatisfactory fragile behaviour due to the severe damage occuring to concrete long before yielding of the ductile elements. The failure mechanism is generally characterised by yielding of the steel frame concentrated mainly in the elements near the bottom of the wall (more specifically at the connections of the horizontal to the vertical parts). The plastic deformation on the concrete infill walls concentrates in a diagonal path clearly indicating the distribution of cracking. In addition, localized plastic deformations are also present near the corners of the infill walls due to the local action of the first studs of the horizontal and vertical elements (Figure 53).

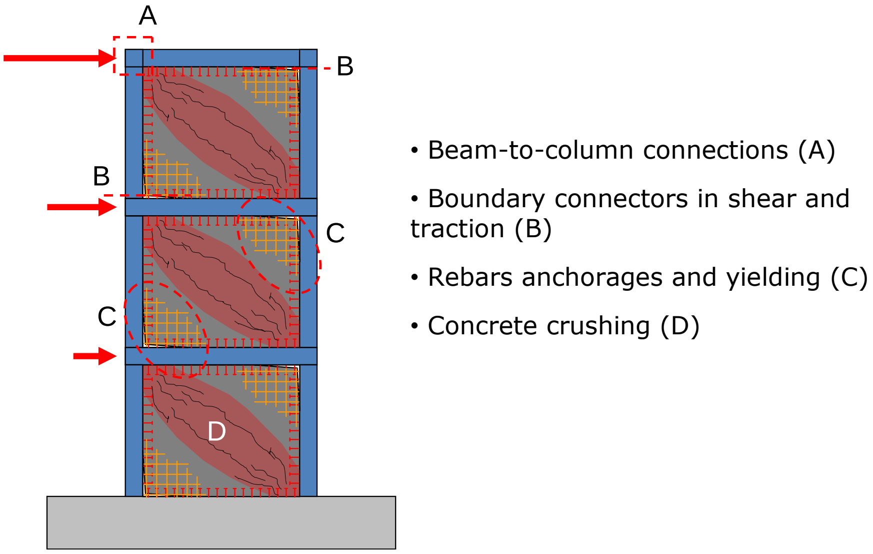



**Figure 53.** Critical aspects in the behaviour of SRCW systems.

The innovative system depicted in Figure 54 is proposed to overcome the previous critical aspects. The RC infill walls are not connected to the vertical columns where the energy dissipation is expected.

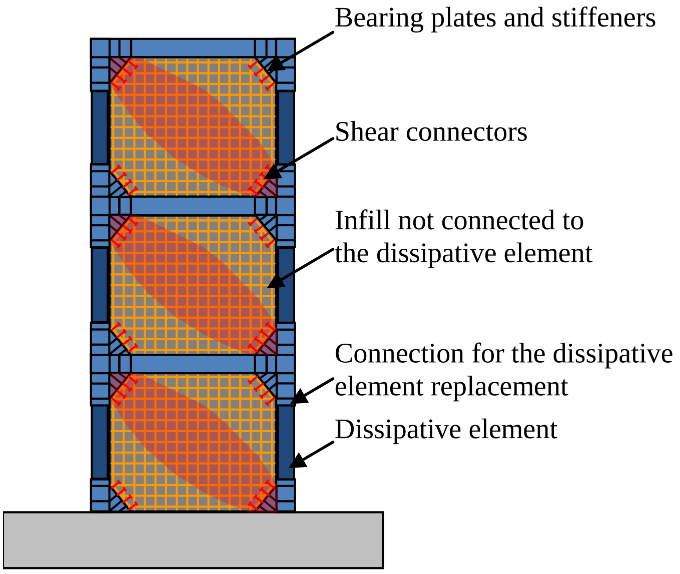



**Figure 54.** Proposed innovative SRCW system.

The system is conceived to control the formation of diagonal struts in the infill walls and behaves as a latticed brace instead of a shear wall. The energy dissipation takes place only in the vertical elements of the steel frame subjected mainly to axial forces without involving the reinforcements of the infill walls. Detailing of the connection of the dissipating elements should allow their replacement and the possible use of buckling-restrained elements. The formation of the diagonal strut is ensured by joint stiffeners and bearing plates. The joint may be welded in shops allowing speeding up the erection phases. The stud connectors are not required to transfer shear forces but they are used to connect the infill and the frame together during the seismic shakings.

## 5. Experimental tests on HCSW systems and relevant mechanical models

### *5.1 Design of the link specimens*

The specimens for the experimental campaign were designed to study and characterize the performance of the connection of a seismic link embedded in a concrete shear wall and the efficiency of the capacity design of such a system, made with the objective of developing a plastic hinge in the replaceable part of the link, acting as a fuse, with all other components of the connection that must remain undamaged. Due to the loading limits of the testing facility there was need to down-scale the link system. The objective of down-scaling was to keep a link that can be classified as intermediate with similar values of shear and moment's ratios as in the case study. Two solutions were considered, as detailed below.

In typology 1 (Figure 55, Figure 56, and Figure 57) the bending moment transferred by the link to the wall is resisted by shear studs and the link splice connection is placed at a distance from the concrete wall that is sufficient to allow an easy bolting of the replaceable part.

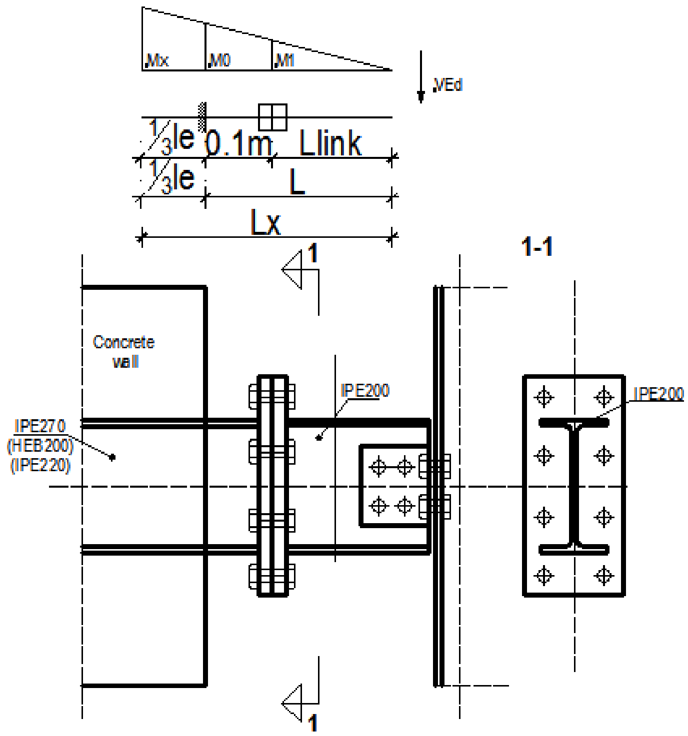



**Figure 55.** Static scheme for connection typology 1

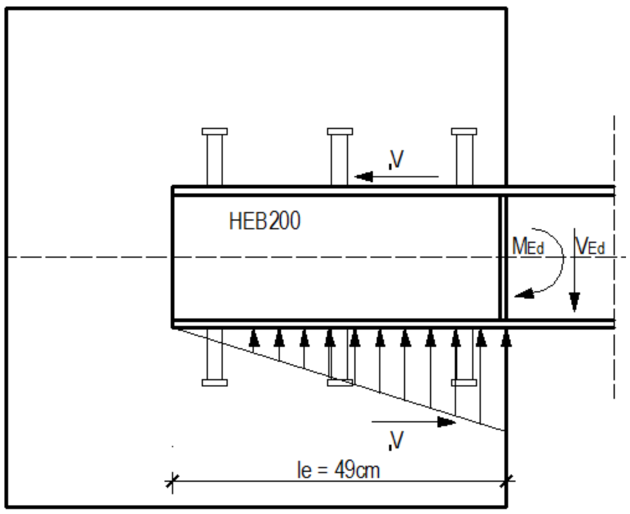



**Figure 56.** Transfer of forces to the wall for connection typology 1

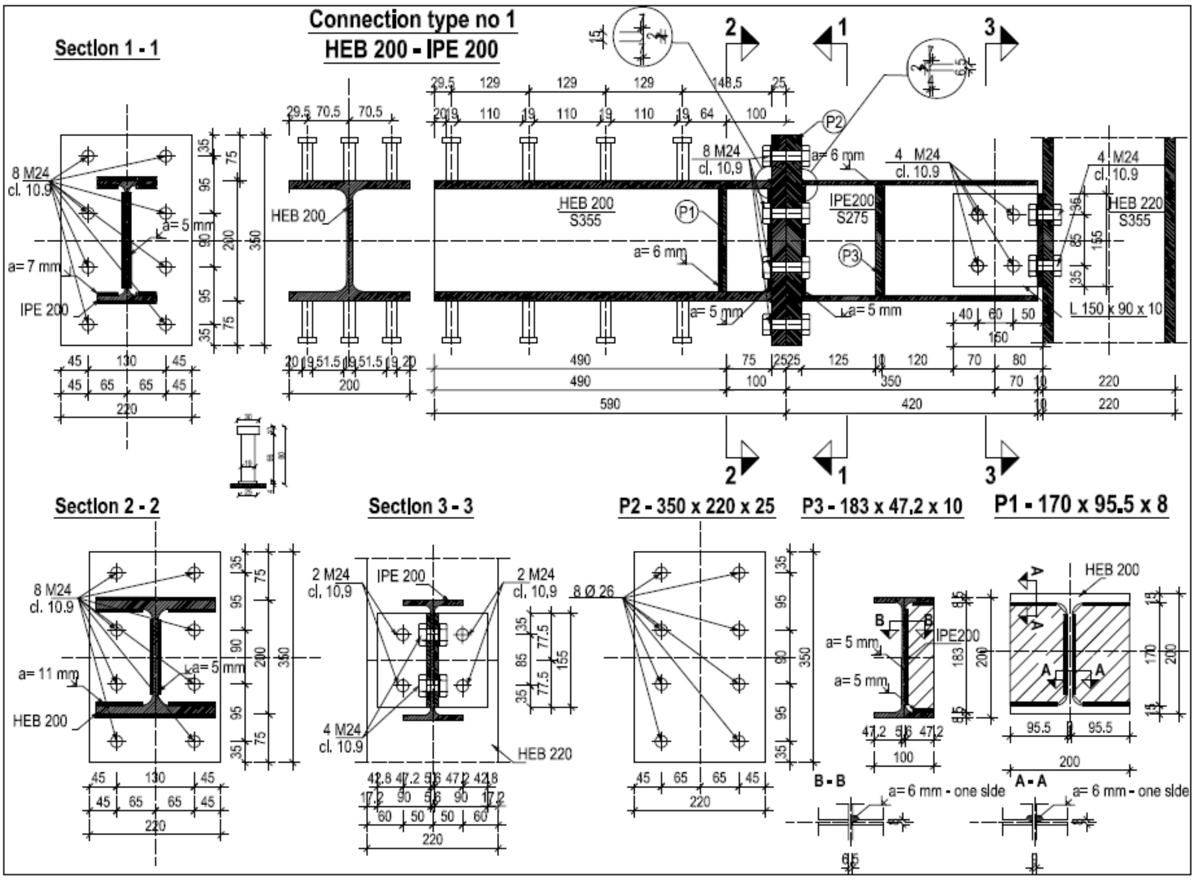


**Figure 57.** Link specimen for connection typology 1

In typology 2 (Figure 58, Figure 59, and Figure 60) the moment transferred by the link to the wall is balanced by a couple of vertical forces and the link splice connection is placed at the face of the wall and threaded bushings to allow replacement of the dissipative tract of the steel link.
Once that the tests are performed and the steel links damaged, a second series of tests are performed in order to evaluate the ultimate capacity of the link-to-wall connection. For this purpose the damaged IPE200 steel S275 links are substituted with HEB200 steel S355 links of the same length before running this second series of tests.

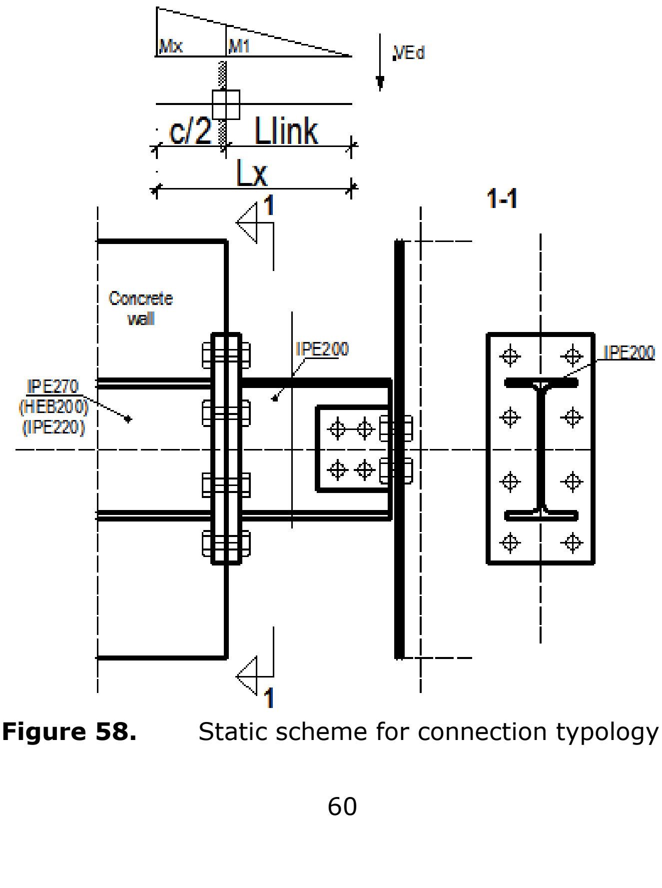



**Figure 58.** Static scheme for connection typology 2

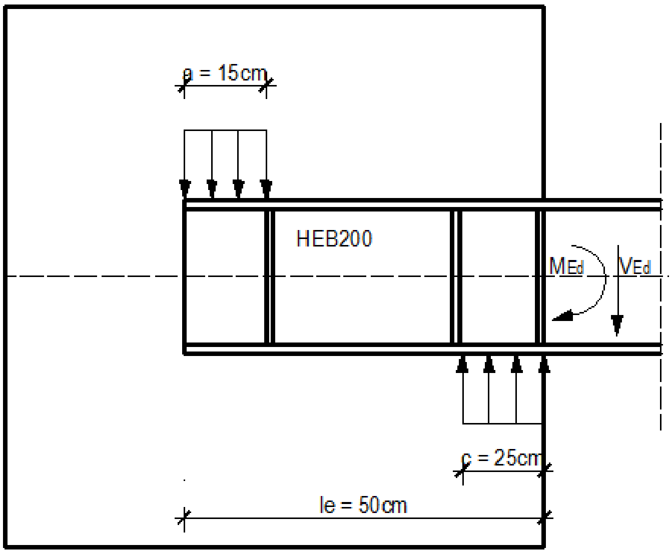



**Figure 59.** Transfer of forces to the wall for connection typology 2

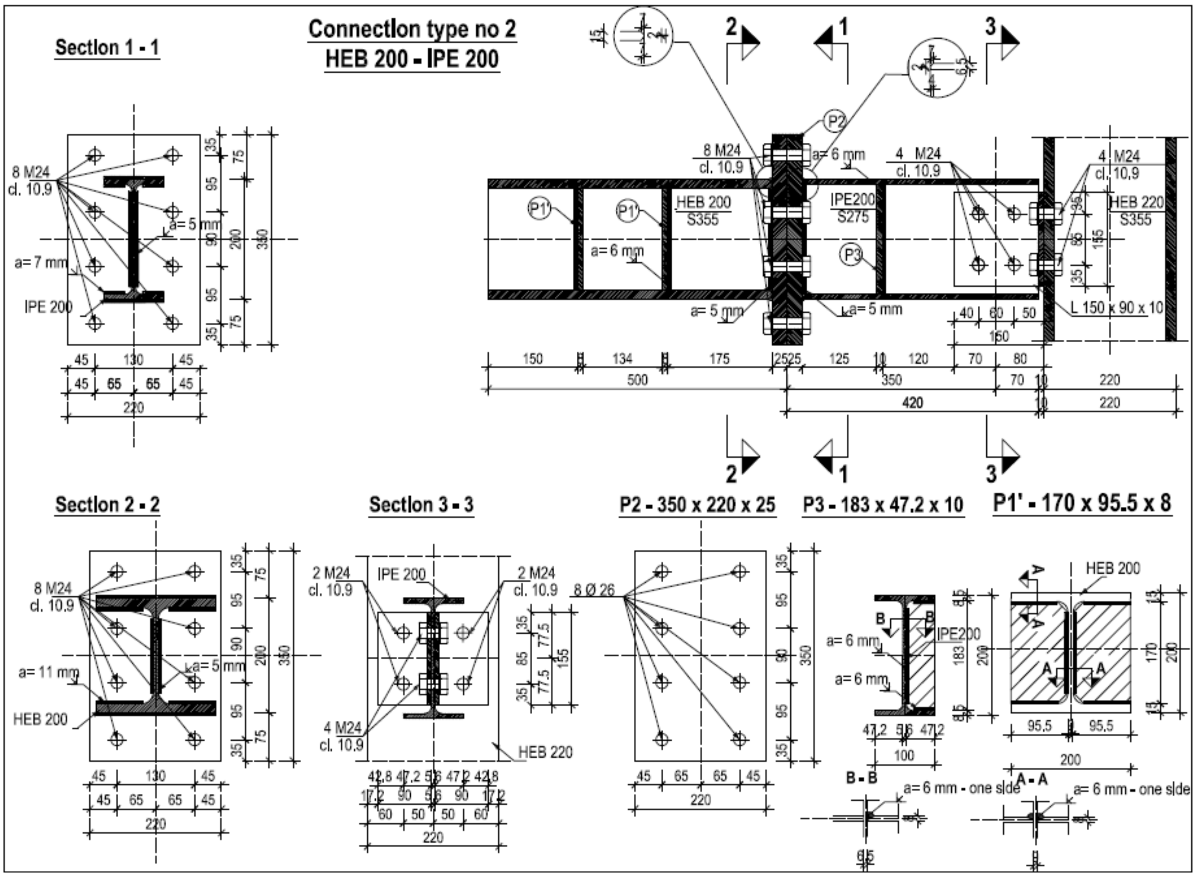



**Figure 60.** Link specimen for connection typology 2

## *5.2 Testing facility*

The global test setup, shown in Figure 61, Figure 62 and Figure 63, includes, besides the elements to be tested (reinforced concrete wall, embedded connection, steel link and steel column), several devices needed to assure the execution of the tests: a 400 kN hydraulic actuator with a 500 mm displacement capacity; a steel reaction wall; two horizontal reaction elements, in order to avoid the RC wall sliding; a column bearing frames that restrain the lateral displacements of the column ends and allows the axial sliding of the column minimizing the friction force. Each of the aforementioned devices were designed considering a safety factor equal, at least, to 1.5 with respect to the actuator capacity. The reaction wall, see Figure 63, and the hydraulic actuator are facilities of the *"Laboratorio Ufficiale per le Esperienze dei Materiali da Costruzione"* of the University of Pisa.

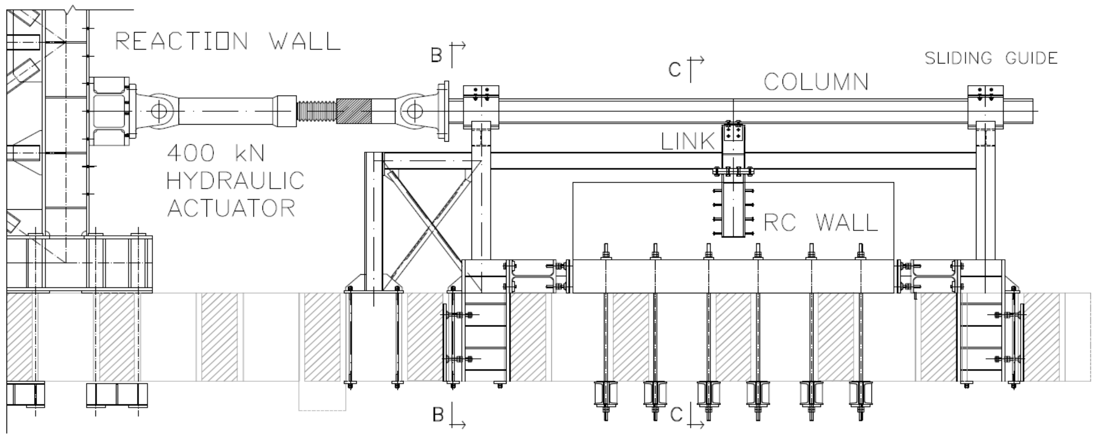



**Figure 61.** Front view of the global test setup

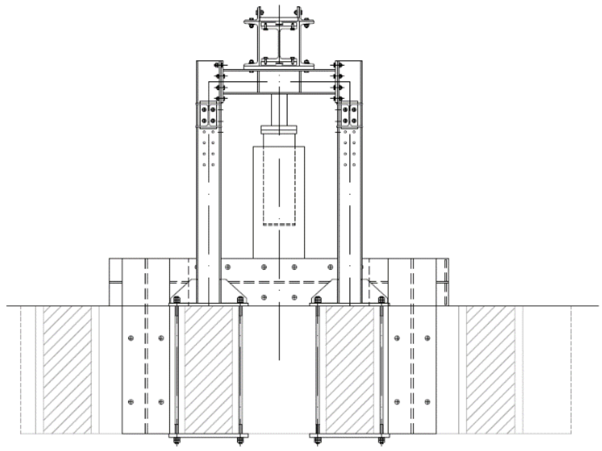


**Figure 62.** Lateral view of the column bearing frame

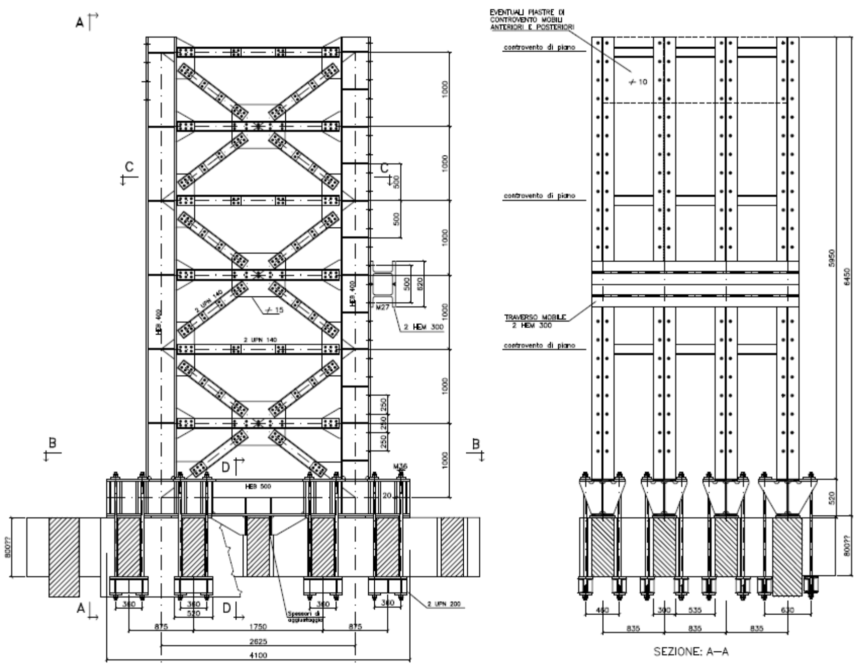



**Figure 63.** Lateral and front view of the steel reaction wall

During the test, besides the local displacement and deformation of the link, the following data are recorded (see Figure 64): column horizontal displacement (1 LVDT); actuator force (1 Load Cell); RC wall displacements horizontal (2 LVDT), vertical (2 LVDT), transverse (2 LVDT).

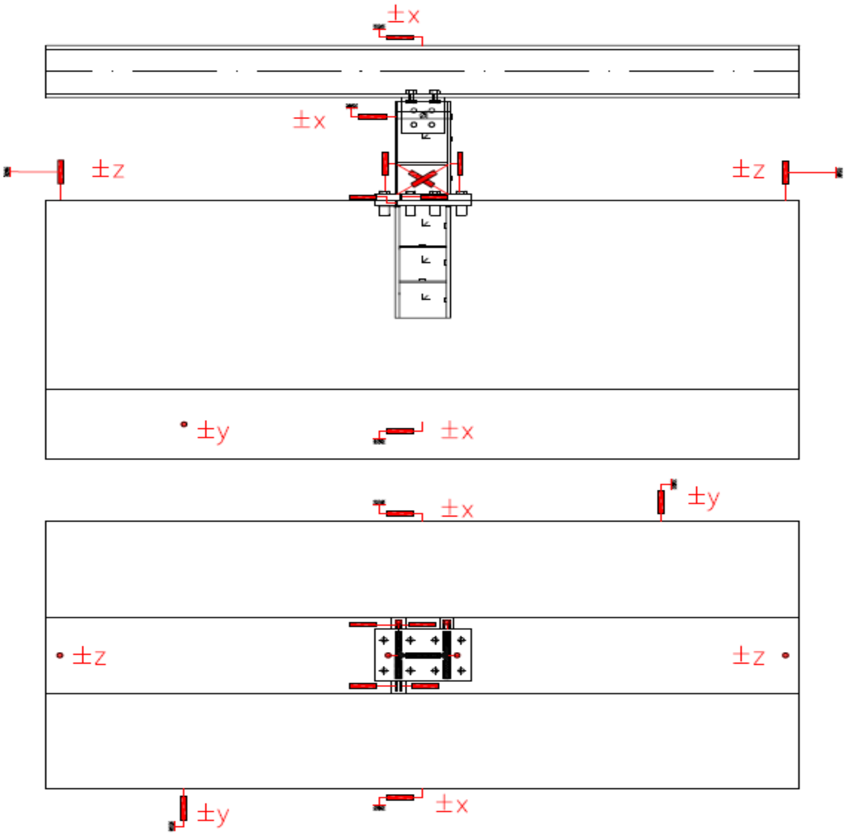



**Figure 64.** Disposition of sensors

### *5.3 Cyclic tests*

For each connection typology, five test were performed, i.e. two sets of cyclic tests with increasing amplitude, two sets of cyclic tests with constant amplitude, a monotonic test up to failure in order to evaluate the ultimate resistance of the link-to-wall connection (in this last test the link is substituted with a HEB200 profile, steel grade S355).
The results of the cyclic tests of connection typology 1 (a selection is presented in Figure 65) show the negative influence of the clearance between bolts and holes in the link-to-column connection. Such a clearance, further increased by the local yielding around the holes due to the bearing stress (Figure 66), together with the contribution from the angle profiles (Figure 67), is responsible for the differential displacements between column and links as well as for the pinching behaviour. In connection typology 2 the differential displacements were limited with an higher bolt torque and the relevant test results (a selection is presented in Figure 68) show an improved cyclic behaviour with hysteresis cycles considerably fatter and the pinching phenomena reduced.

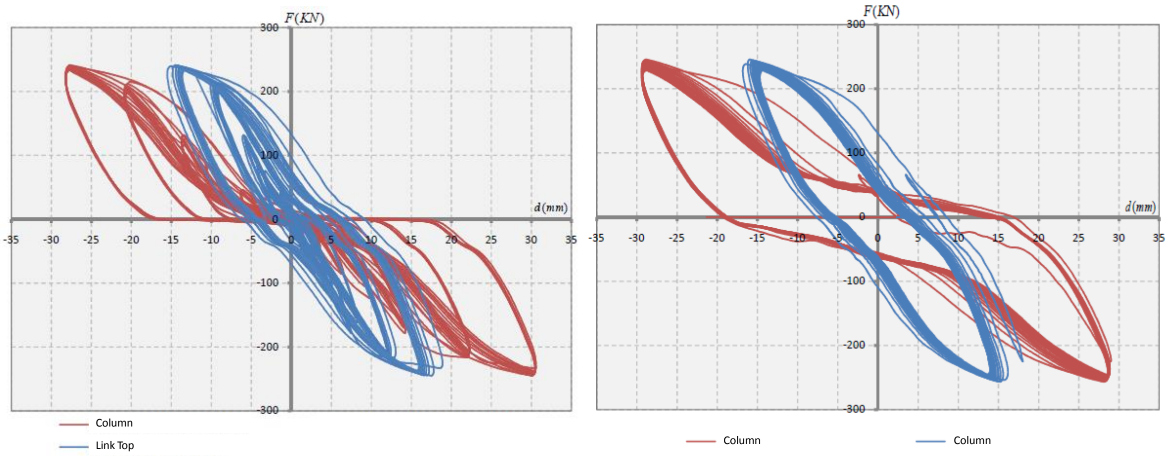



**Figure 65.** Connection typology 1: load displacement curves for link top and column in the cyclic test with increasing amplitude (left figure) and constant amplitude (right figure)

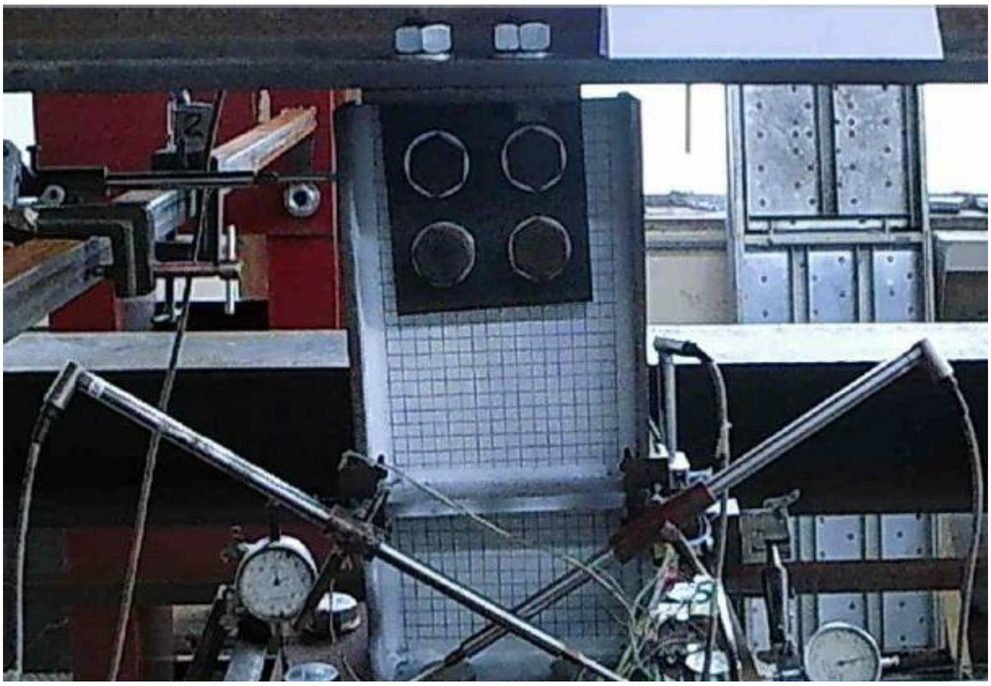


**Figure 66.** Connection typology 1: deformation of the link-to-column joint at maximum displacement

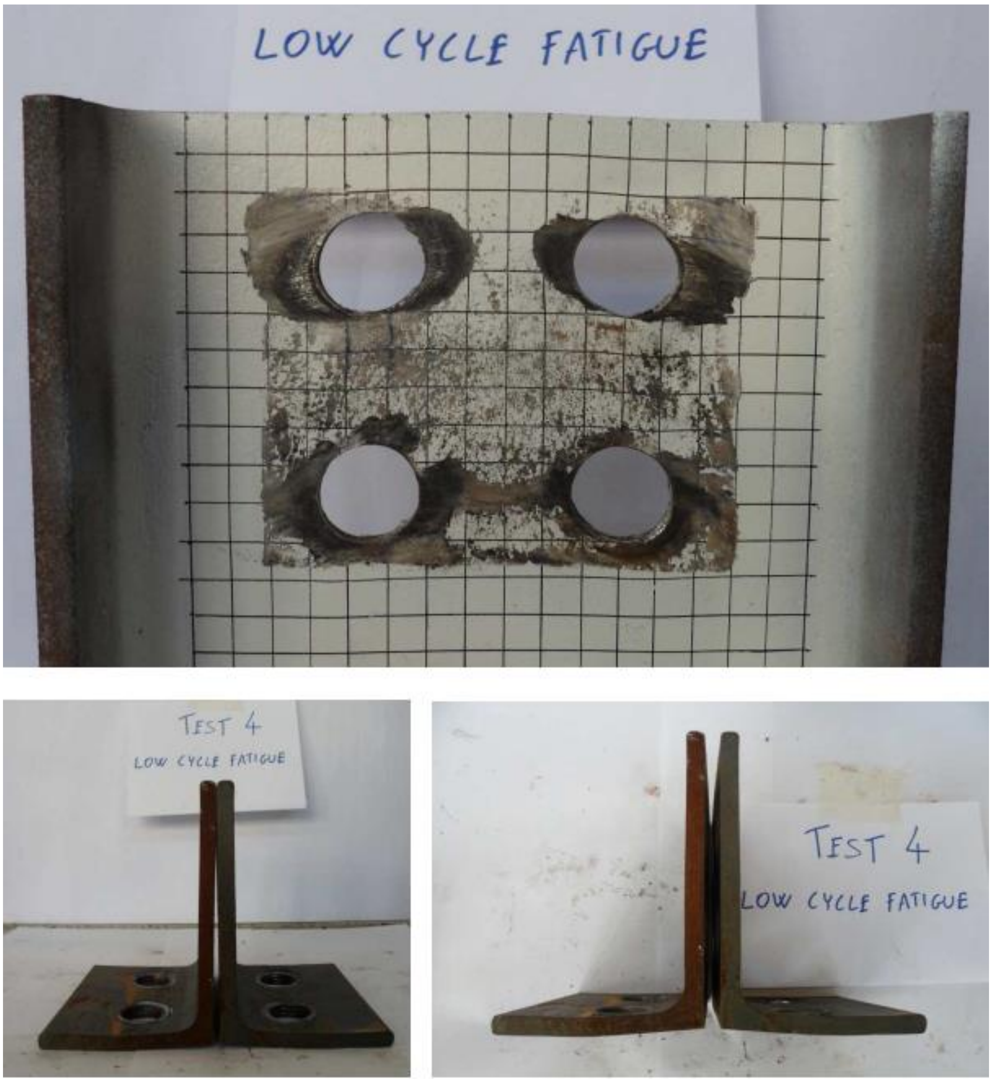



**Figure 67.** Connection typology 1: dissipative link specimen and angle profiles after the test

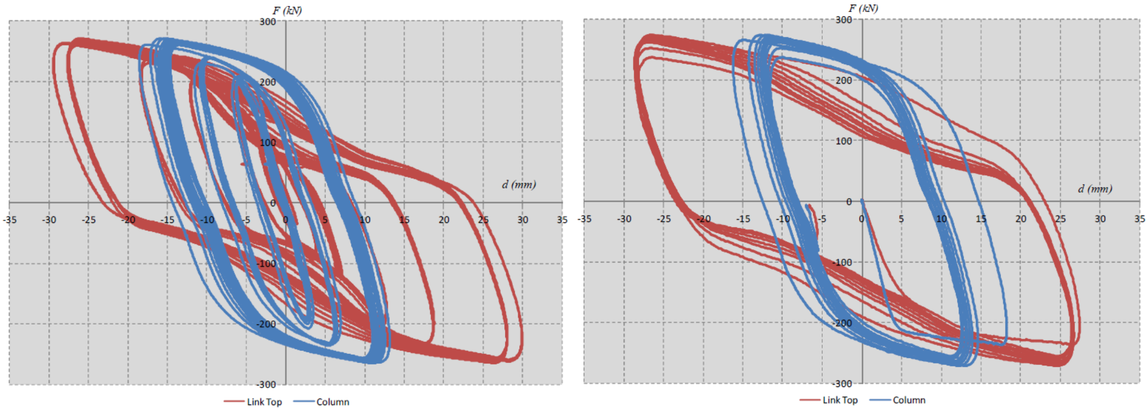



**Figure 68.** Connection typology 2: load displacement curves for link top and column in the cyclic test with increasing amplitude (left figure) and constant amplitude (right figure)

**Figure 69.** Connection typology 2: dissipative link specimen after the test

### *5.4 Monotonic tests*

Regarding the connection between the link and the wall, the monotonic tests highlighted a basically linear global behaviour of the connection up to the maximum applicable load, as shown in Figure 70 and Figure 71. As already observed for the cyclic test, the clearance and subsequent yielding in the link-to-column joint is responsible for the differential displacement measured between the two connected elements.

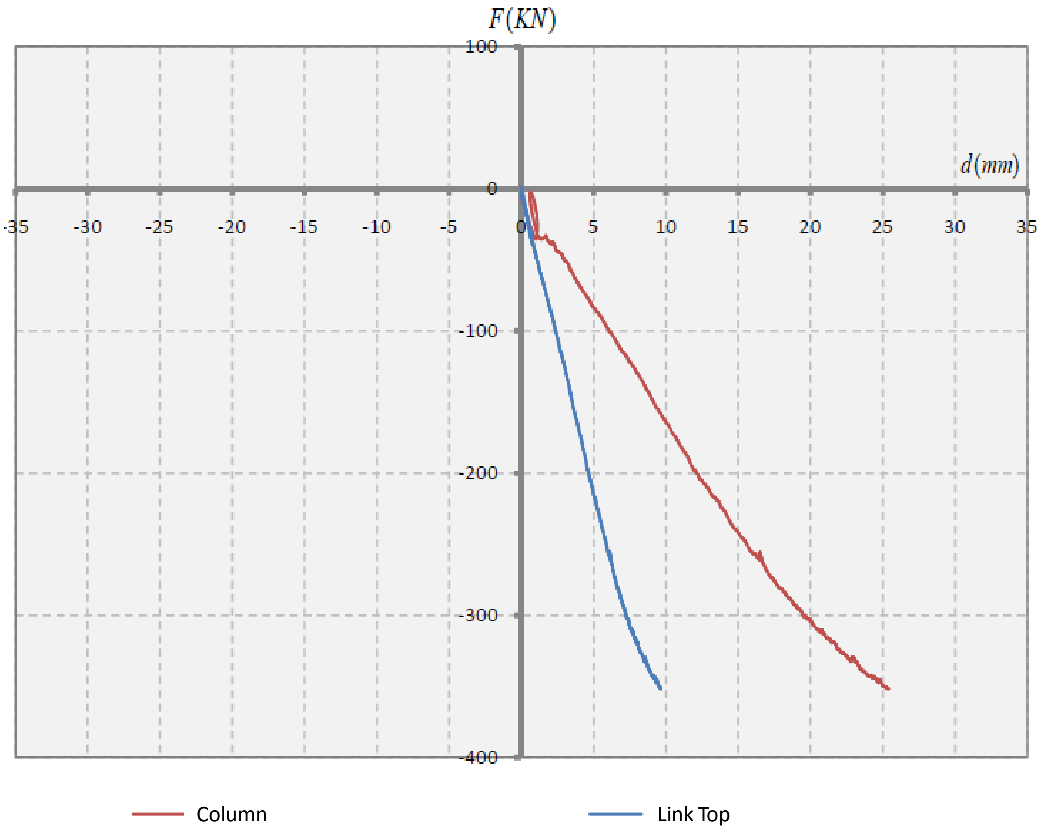



**Figure 70.** Monotonic test for connection typology 1

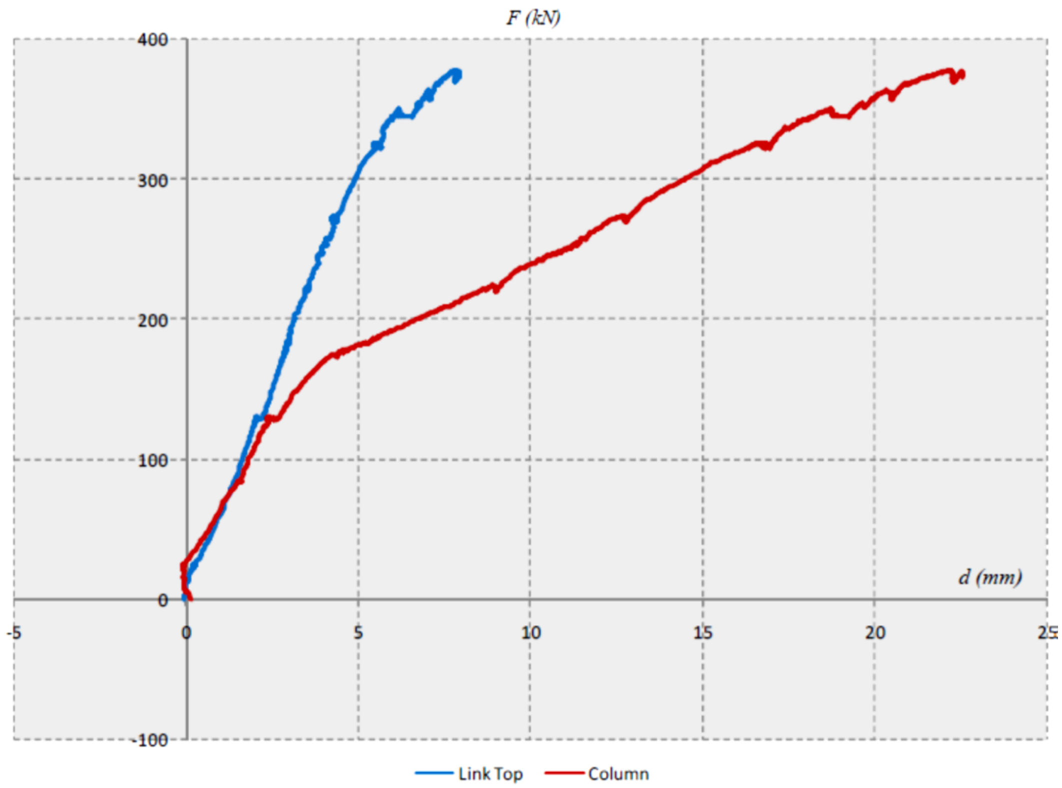



**Figure 71.** Monotonic test for connection typology 2

### *5.5 Mechanical models*

A frame model with a one-dimensional elastoplastic constitutive law (Figure 72) recently proposed (Zona and Dall'Asta 2012) was used to model the behaviour of the steel link whose nonlinear behaviour is described by a simple uniaxial constitutive model as the one shown in Figure 73. It was found that a satisfactory approximation of the experimental results can be obtained (as shown in selected results reported in Figure 74 and Figure 75) provided that the actual value of the yield stress is identified. This result gives great benefits for the nonlinear static and dynamic analysis of HCSW systems, as conventional elastoplastic models commonly available in many structural analysis programs are adequate.

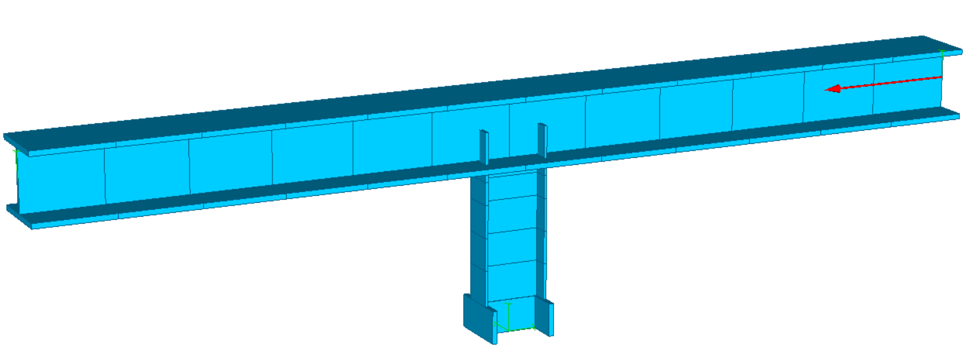


**Figure 72.** Frame model for the tested connection system

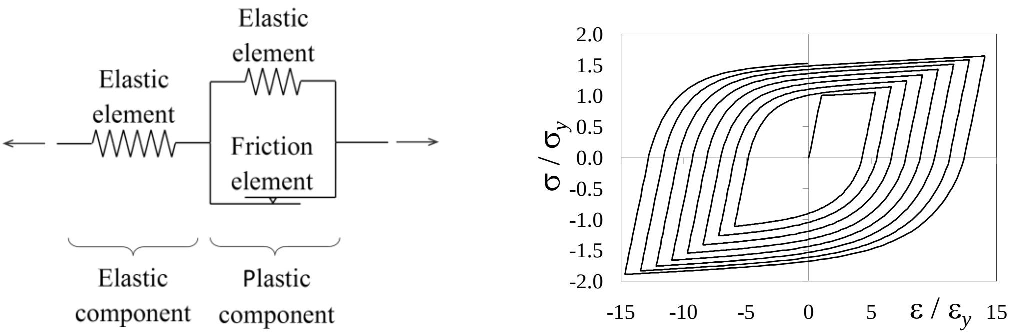



**Figure 73.** Uniaxial cyclic constitutive law

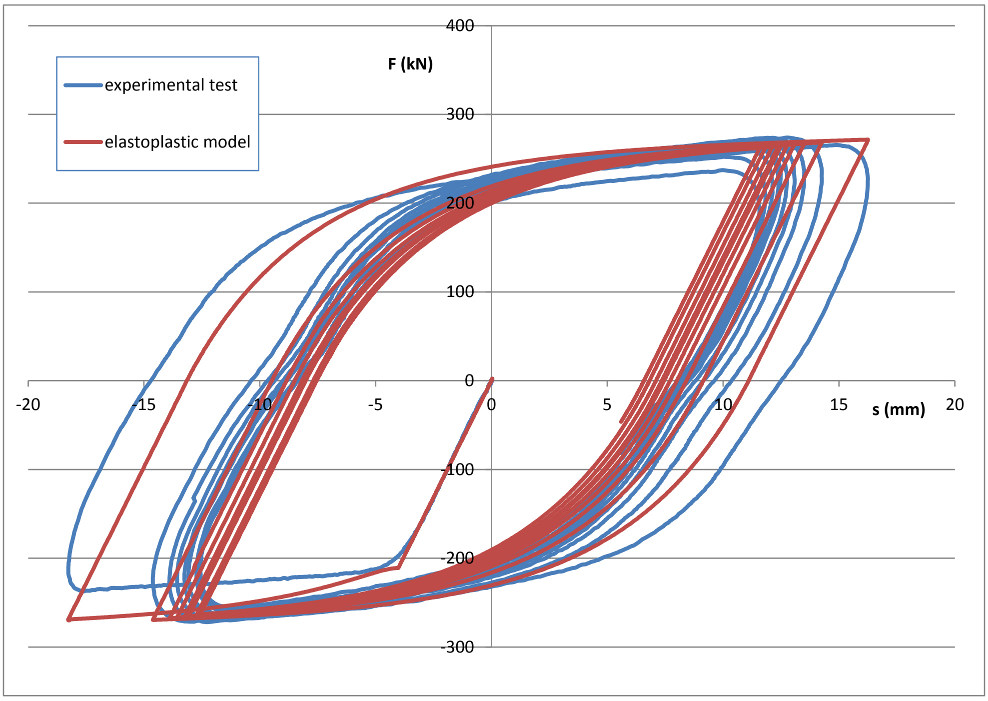



**Figure 74.** Comparison between model prediction and experimental results (constant amplitude)

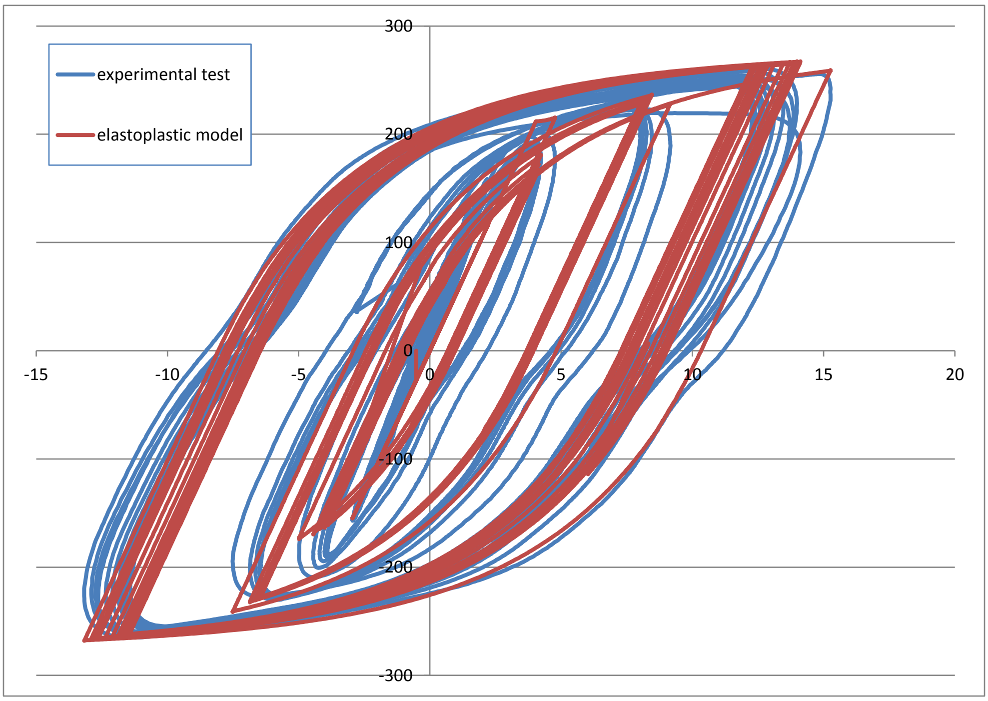


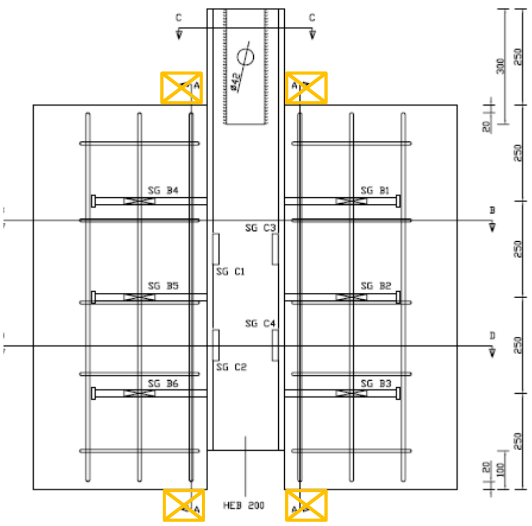

**Figure 75.** Comparison between model prediction and experimental results (increasing amplitude)

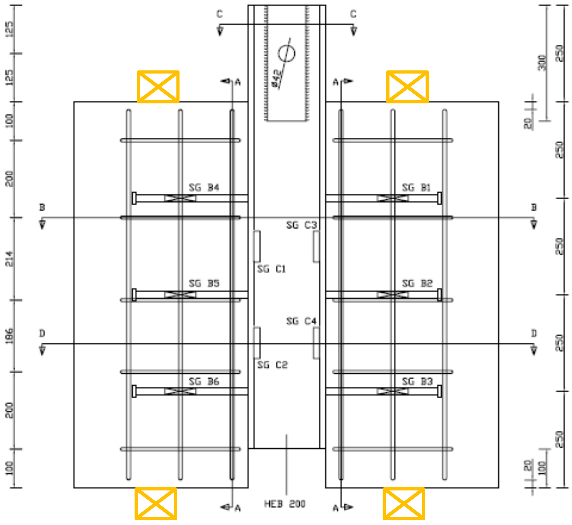
# 6. Experimental tests on SRCW systems and relevant mechanical models

## 6.1 Design of the specimens and testing facilities

Three experimental activities are included: tests on shear connection specimens, tests on isolated ductile side elements, and tests on one-storey SRCW. Tests on shear connections are aimed at understanding the behaviour of headed studs placed at the wall edge when subjected to cyclic loading; tests on the ductile side elements are aimed at understanding the cyclic behaviour under undesired compression and secondary bending; tests on the wall are aimed at understanding the behaviour of the reinforced concrete infill and the effectiveness of the formation of the diagonal strut in the development of the dissipative mechanism.

As for the tests on shear connectors, a total number of three displacement controlled tests were initially planned. These tests were extended to a total number of six specimens each consisting of a HEB 200 profile with three headed studs welded to each flange. The increased number of tests reflects the consideration of two effects to be investigated: connectors subjected to pure shear and connectors subjected to shear and traction. For this purpose, two different test configurations (Figure 76) were considered and, for each configuration, one monotonic and two cyclic tests were done.


(a)


(b)

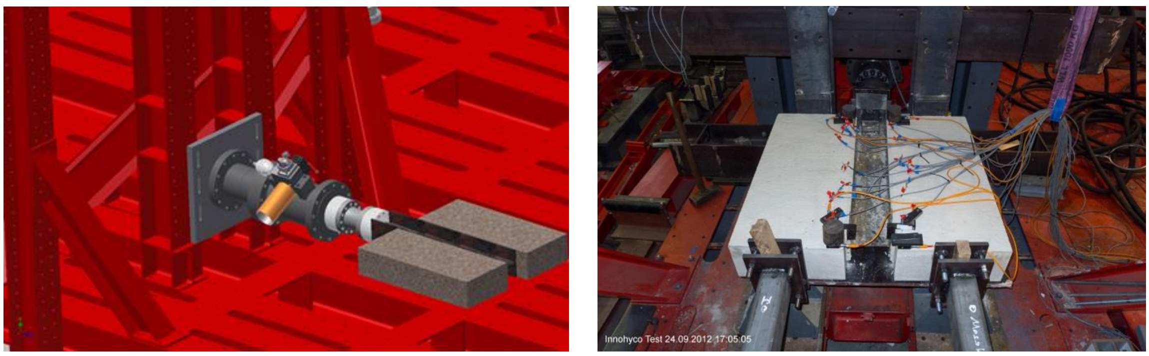

(c)

**Figure 76.** (a) stud connectors subjected to pure shear;
(b) stud connectors subjected to shear and traction; (c) test rig

The specimen of the side steel elements consists of the same profiles as the top storey of one of the cases designed in WP5 (Figure 77). During seismic action the wall experiences horizontal loading from alternating directions. This results in alternating tensile and compressive action on the side steel element, combined with smaller horizontal and moment actions. Tests were performed by deriving actions from a numerical simulation. Cyclic axial and shear forces were applied with a couple of hydraulic jacks as depicted in Figure 77.

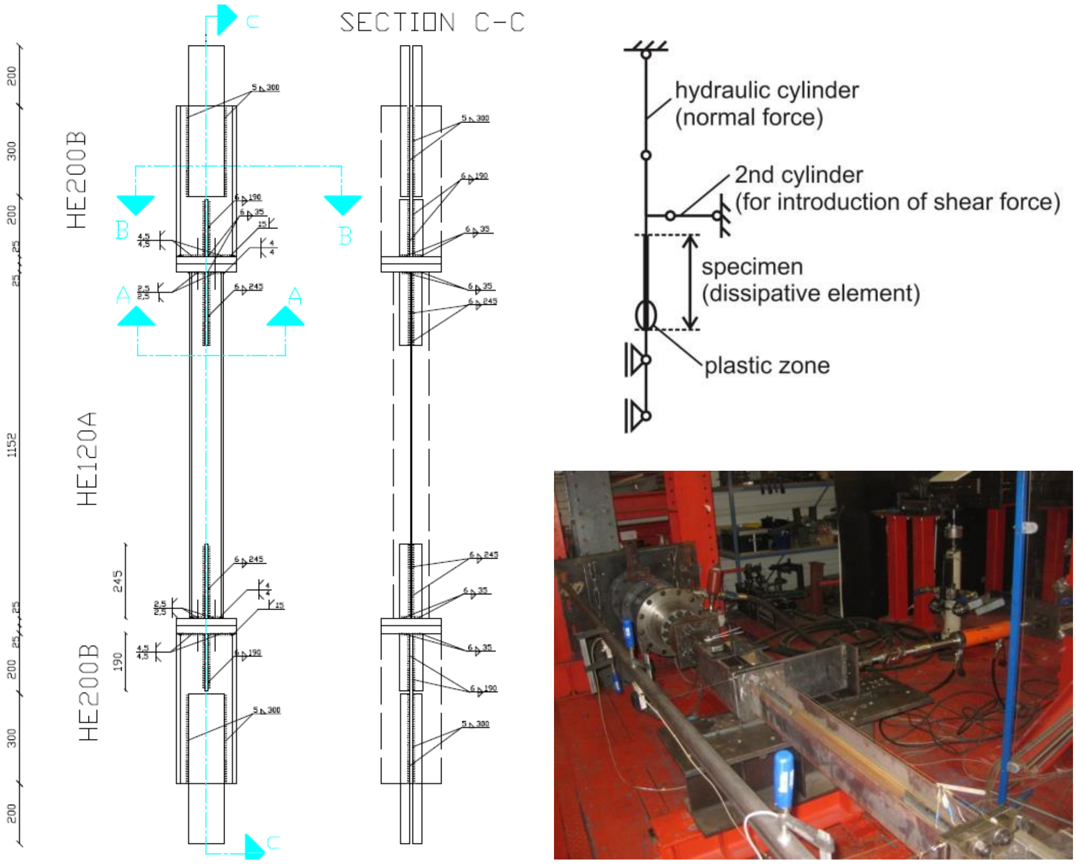



**Figure 77.** Specimen of the ductile side element and relevant testing setup

For what concern the downscaled SRCW system, the one-storey specimen shown in Figure 78 was designed by applying the procedure proposed in WP5 in a 2:3 downscale to comply with the capacity testing facility.

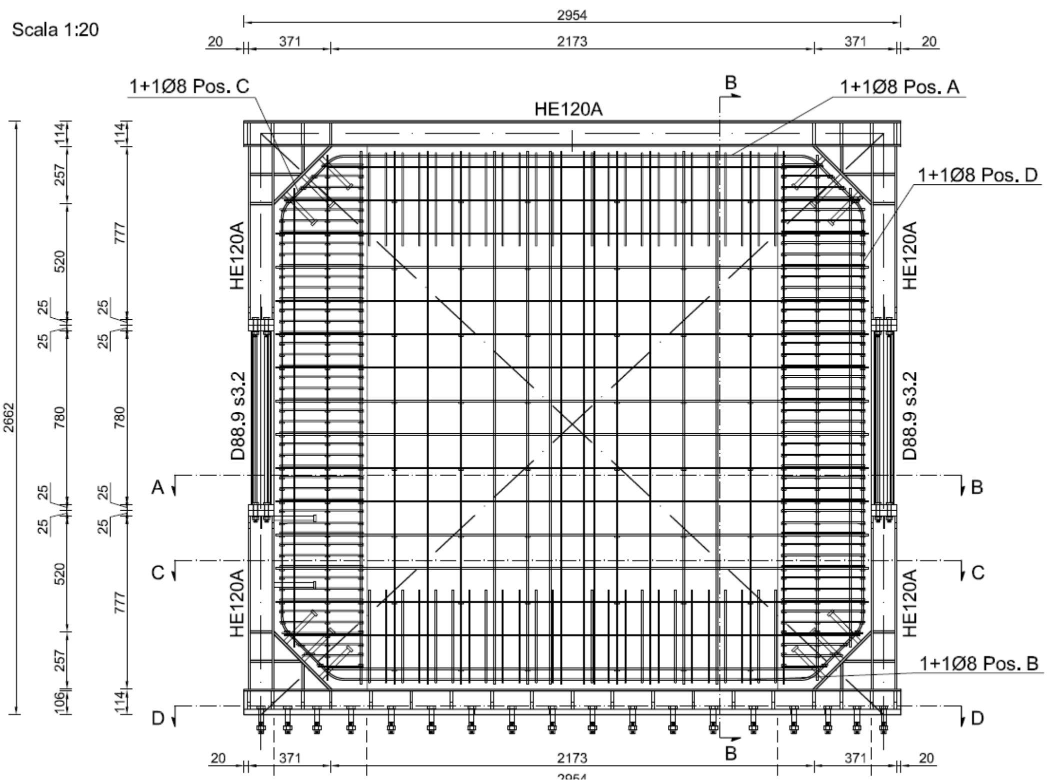



**Figure 78.** Designed SRCW specimen

The wall was instrumented so that displacements, and forces were measured as well as strains in the ductile side elements (Figure 79).

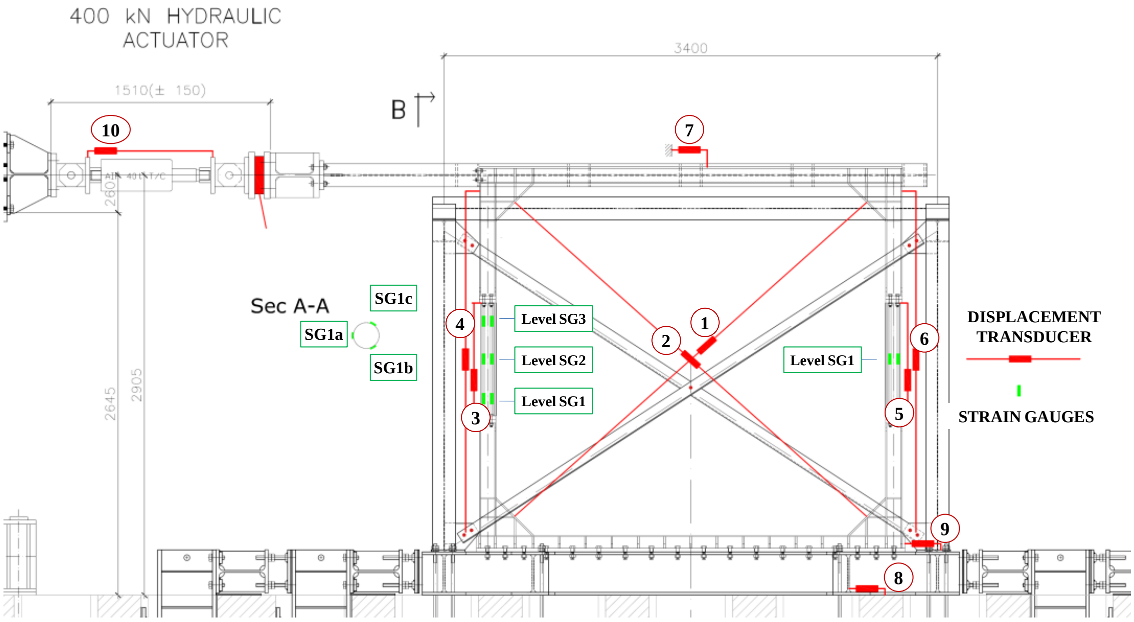



**Figure 79.** Position of the sensors in the SRCW downscaled specimen

**Figure 80.** Construction of the specimen

### 6.2 Experimental results

Experiments performed on the shear connection demonstrated a large discrepancy between strength obtained with the Eurocode 4 design formulas and the measured strength that resulted almost 1.5 times higher. Furthermore, shear failure occurred in the stud shank and not in the concrete crushing as was expected from the design formulas. As for the cyclic behaviour the specimen did not withstood the prescribed number of cycles at the loading level equal to 75% of the experimental strength, as requested for shear connection in seismic resistant elements. The test was thus repeated with the maximum amplitude equal to 50% of the monotonic capacity (Figure 81).

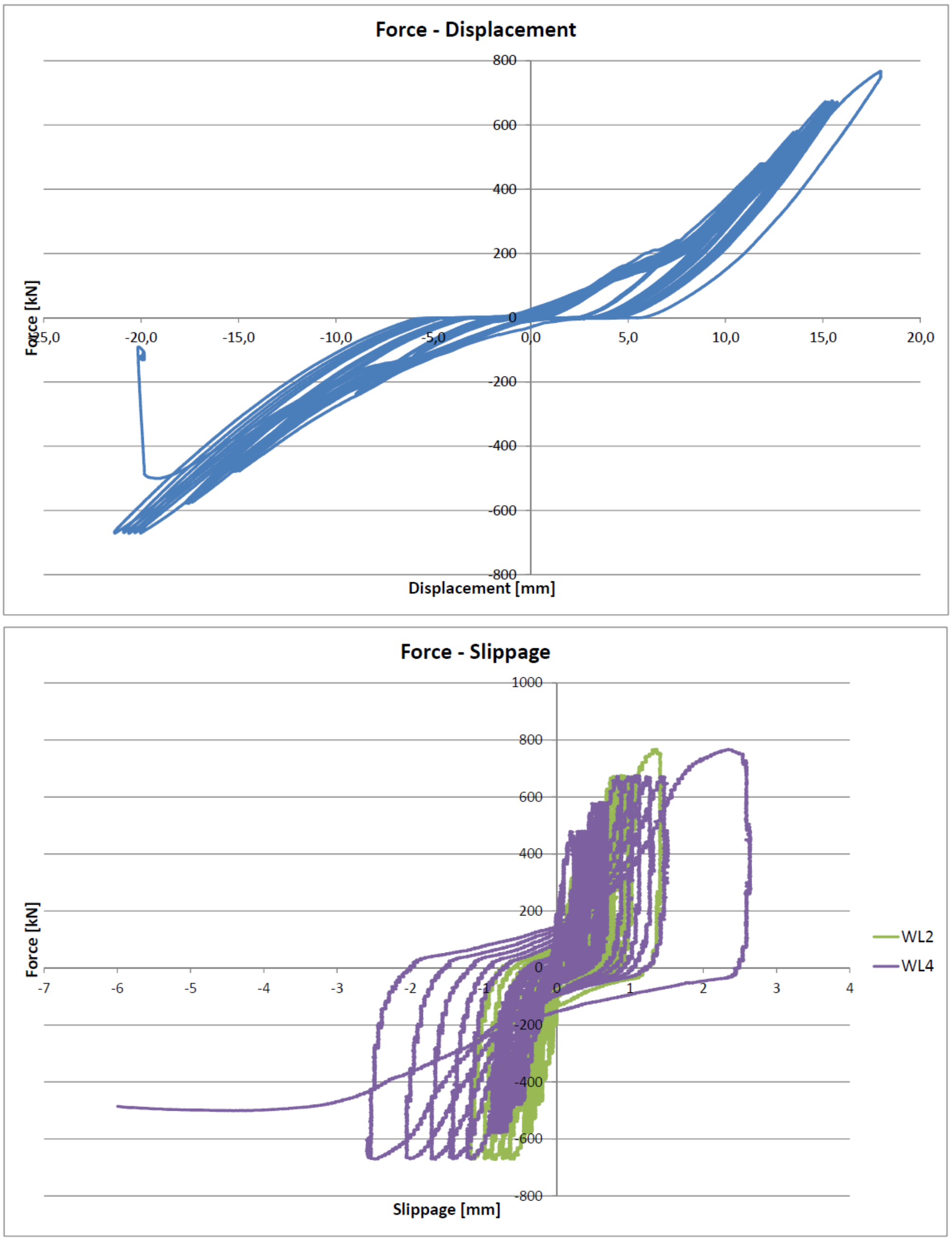



**Figure 81.** Force-displacement and force-slip curves (cyclic loading reference 50%)

Two monotonic tests were performed for the side steel elements: (a) specimen subjected to compression forces, and (b) specimen subjected to tension forces. Lateral forces were contemporarily applied with a second cylinder to induce a concomitant bending moment. The maximum amplitude of the actions were derived from a theoretical model. A plastic hinge formed at the end of the fuse for the combined axial force-bending moment actions. The cyclic tests demonstrated the capability of fuse to develop a plastic hinge necessary for the formation of its assumed ultimate resisting mechanism given that it is actually not pinned at its ends.

The experimental activity on the downscaled specimen permitted to observe the real behaviour of a panel that confirmed the validity of the resisting mechanism assumed in the design. The diagonal strut formed without crushing of the concrete as demonstrated by the crack pattern observed. Strain measured at the vertical elements confirmed the yielding of the material constituting the fuses whereas other steel elements were preserved. Figure 83 shows the measured force-displacement curve.

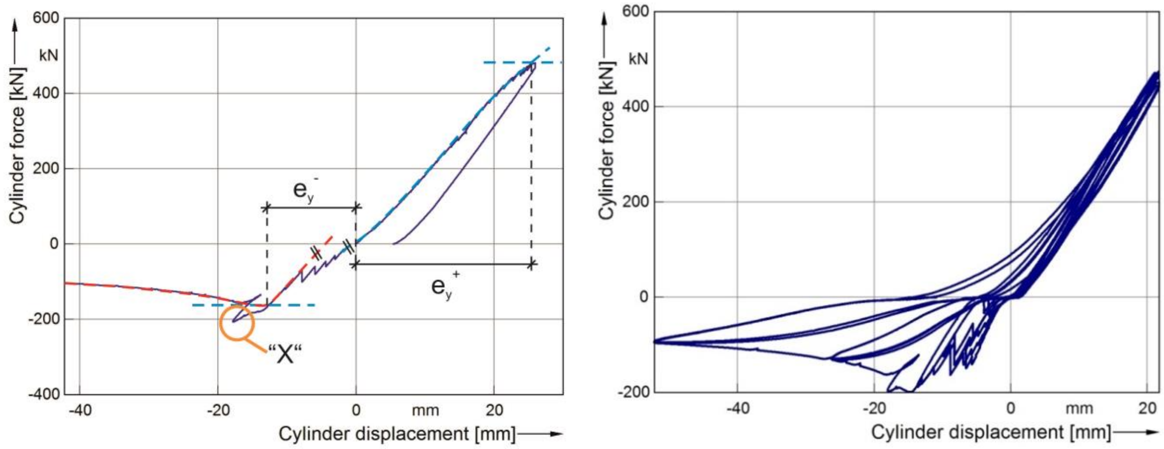



**Figure 82.** Results of monotonic test and one of the cyclic test

**Figure 83.** Testing of the downscaled SRCW specimen

## *6.3 Mechanical models*

Different mechanical models were developed in SAP2000 by using beam finite elements (elastic) and nonlinear links (lumped plasticity) as shown in Figure 84 in order to closely reproduce the experimental results. The actual geometry of the joints is considered by introducing rigid body geometrical constraints. Ductile elements are modelled with nonlinear links by using the built-in Wen plasticity for the uniaxial properties. The wall is schematized with an increasing level of complexity. In the simpler model, nonlinear links are used just to capture the formation of the diagonal struts. In the more complex model, vertical nonlinear links are also considered to capture the effects of the wall that may be activated by the interaction with the steel frame at the node. For the diagonal elements, the compression force-displacement relationship is derived by the stress-strain Mander's (1988) law for unconfined concrete; a linear elastic behaviour followed by a linear softening branch is considered for tensile stresses; the Takeda's (1970) hysteretic behaviour is considered. For the vertical wall elements, a couple of links are placed in parallel and connected at the same nodes of the model: one is used for the concrete component, for which the Mander's law for confined concrete is considered, the other is used for the reinforcements, for which the Wen model in considered. The calibration of the material parameters permits to capture the experimental results.

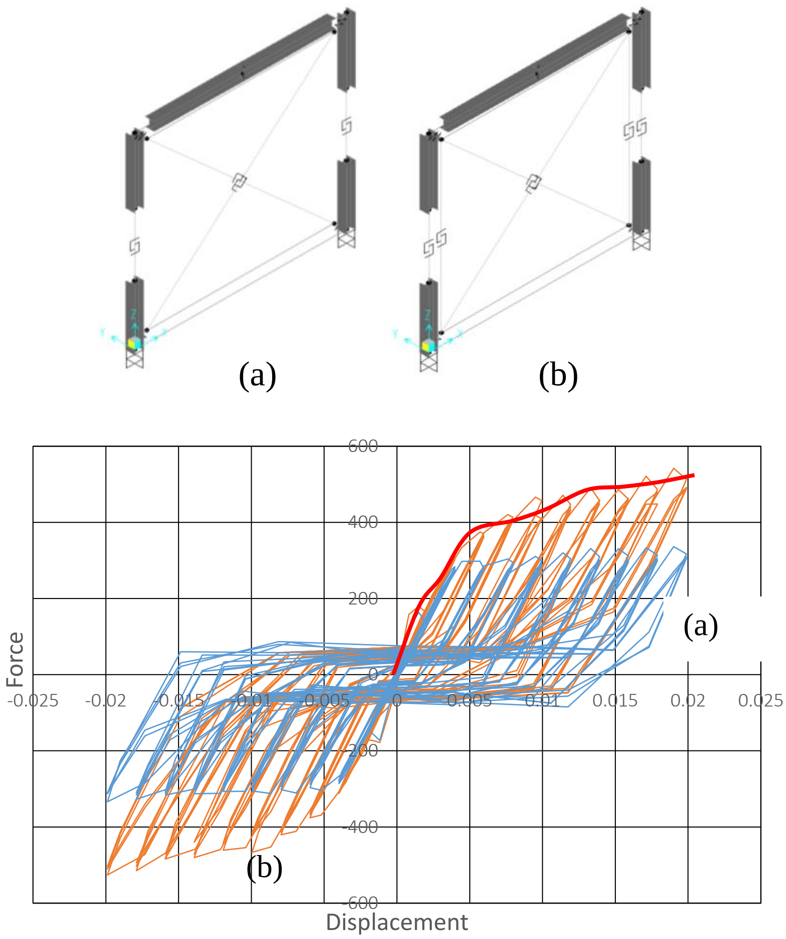



**Figure 84.** Mechanical models for the specimen

# 7. <u>Design of innovative HCSW systems</u>

The design of the proposed HCSW systems requires a preliminary dimensioning of the reinforced concrete wall, steel links and side steel columns, followed by an assessment of the seismic behaviour through nonlinear analysis. The preliminary dimensioning can be based on linear elastic analysis or on limit analysis.

In the linear elastic approach the components of the HCSW system are tentatively dimensioned and then linear elastic analysis, e.g. modal combination with reduced design spectrum, is used to evaluate the seismic demand. In this way, once that the reduction factor and the seismic input are assigned, the tentative design is compared to the seismic input and refined with successive iterations.

In the limit state approach, the components of the HCSW system are directly dimensioned (with no iterations) in order to have an adequately post-elastic behaviour where the first yielding of the wall is enforced after the steel links are already yielded and, thus, have started to dissipate seismic energy. The HCSW post-elastic behaviour and relevant capacity are assessed through nonlinear analysis. Afterwards, the comparison between system capacity and seismic demand, for example in terms of target displacement, permit to define the number of HCSW systems required for the considered building. In addition, the level of damage accepted in the concrete wall can be directly tailored based on the outcomes of nonlinear analysis.

## *7.1 Preliminary design based on linear elastic analysis*

The design procedure of the proposed innovative HCSW system based on linear elastic analysis is subdivided in the following steps, each commented with details and recommendations.

**Step 1**: assign the dimensions of the reinforced concrete wall by selecting its height-to-length ratio $h_w/l_w$ and thickness $b_w$.

The analyses performed in this research project give indications for suggested optimal value of $h_w/l_w$ = 10 while the minimum value of $b_w$ should be selected according to the required capacity and to adequately accommodate the connection between wall and steel links.

**Step 2**: design of the steel links with assigned uniform over-strength based on bending and shear obtained from linear seismic analysis (e.g. spectrum analysis);

This step is a trial-and-error iterative procedure as the results of the seismic analysis are influenced by the link and column design. Critical aspects of this step are the selection of the q factor for linear seismic analysis and the identification of the optimal link length. In addition, considerations should be made on the validity and actual need of the link regularity limits given that, as observed, the HCSW systems designed seem not prone to soft storey formation, even when the yielding of the steel links is not simultaneous, thanks to the contribution of the reinforced concrete wall. Results obtained permit to suggest a behaviour factor q = 2.3 if the wall is required to remain in the elastic cracked range or a behaviour factor q = 3.7 if damage is accepted in the wall under the design seismic input yet collapse is prevented. Given that these values of the behaviour factor q are just suggestions that require further investigations, the post-elastic behaviour of the obtained design solution must be assessed by means of nonlinear analysis. Geometric nonlinear effects are controlled with the amplification of seismic loads according to Eurocode 8 Part 1, paragraph 4.4.2.2, equation 4.28. Link properties and classification are made according to Eurocode 8 paragraph 6.8.2. The link design resistances in bending and shear are computed according to Eurocode 8 Part 1 paragraph 6.8.2(3):

$$M_{p,link} = f_y b t_f \left(d - t_f\right)$$

$$V_{p,link} = \frac{f_y}{\sqrt{3}} t_w \left(d - t_f\right)$$

where $f_y$ is the nominal yield stress and the other terms are defined in Figure 85.

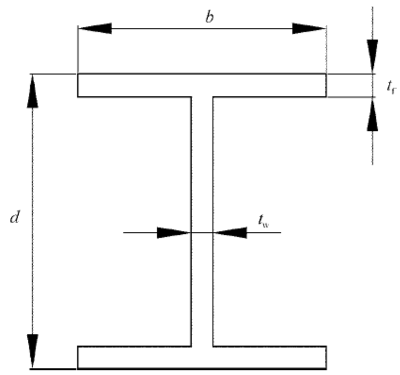



**Figure 85.** Definition of symbols for I-section links.

The link categories are determined according to Eurocode 8 Part 1 paragraph 6.8.2(9) from their length *e* as per the following scheme.

| short links | intermediate links | long links |
|---|---|---|
| $e < e_S = 0.8\dfrac{M_{p,link}}{V_{p,link}}$ | $e_S < e < e_L$ | $e > e_L = 1.5\dfrac{M_{p,link}}{V_{p,link}}$ |

Over-strength regularity is verified with the condition $\Omega_{max} / \Omega_{min} < 1.25$ inherited from the design of eccentrically braced steel frames in Eurocode 8 paragraph 6.8.3, assumed that $\Omega_{max} = \max\{\Omega_i\}$, $\Omega_{min} = \min\{\Omega_i\}$ having assumed for the *i*-th link:

| short links | intermediate and long links |
|---|---|
| $\Omega_i = 1.5\dfrac{V_{p,link,i}}{V_{Ed,i}}$ | $\Omega_i = 1.5\dfrac{M_{p,link,i}}{M_{Ed,i}}$ |

where $V_{Ed,i}$ and $M_{Ed,i}$ are the design values of the shear force and of the bending moment in *i*-th link in the seismic design situation.

**Step 3:** design of the steel side columns using the summation of the yield shear forces of the links (amplified with $1.1\gamma_{ov}$) as design axial force.

The steel side columns at the *j*-th floor are designed using as design axial force using the summation of the yield shear forces of the links:

$$N_{b,Rd,j} < N_{pl,c,j} = 1.1\gamma_{ov}\sum_{i=j}^{n} V_{p,link,i}$$

where *n* is the number of storeys, $\gamma_{ov} > 1$ is the material over-strength factor (i.e. possibility that the actual yield strength of steel is higher than the nominal yield strength) as defined in Eurocode 8, paragraph 6.1.3 and $f_{y,max} \leq 1{,}1\gamma_{ov}\, f_y$ as in 6.2(3), $N_{b,Rd,j}$ is the axial capacity of the *j*-th tract determined according to Eurocode 3.

**Step 4**: design of the wall longitudinal reinforcements to provide an assigned over-strength compared to the bending moment obtained from linear analysis.

The design of the wall longitudinal reinforcements is based on the following relation

$$M_{Rd,w} > \gamma_R M_{Ed,w}$$

where $M_{Ed,w}$ is the design value of the bending moment at the wall base in the seismic design situation, $\gamma_R \geq 1$ is an over-strength factor (optimal value to be determined according to the results of the nonlinear analyses) to limit damage in the wall, $M_{Rd,w}$ is the design resisting moment.

**Step 5**: design of the transverse reinforcements to avoid shear collapse of the wall considering the maximum shear at the base derived from the limit condition of yielded steel links and wall ultimate limit state in bending.

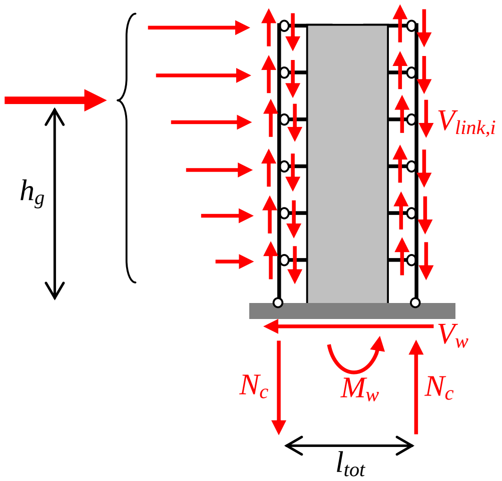



**Figure 86.** Equilibrium of forces in the HCSW system for the computation of the design base shear.

The design of the wall transverse reinforcements is made to provide a shear resistance $V_{Rd,w}$ in the wall that exceeds the maximum shear $V_{pl,w}$ in the limit condition of attainment of the ultimate bending moment in the wall and all links yielded:

$$V_{Rd,w} > V_{pl,w} = \frac{M_{Rd,w} + N_{pl,c} b_{tot}}{h_g}$$

where

$$N_{pl,c} = 1.1 \gamma_{ov} \sum_{i=1}^{n} V_{p,link,i}$$

with $l_{tot}$ and $h_g$ as depicted in Figure 86. Reinforcement can for example be detailed according to Eurocode 8 paragraph 5.4 DCM rules in order to ensure a sufficiently ductile behaviour of the wall.

### *7.2 Remarks on preliminary design based on linear elastic analysis*

Applications of this design approach on a number of case studies gave interesting results through procedures that are familiar to structural engineers trained to steel seismic design according to Eurocode 8. However, some problems were also highlighted as significant quantities of longitudinal bars were required, in some cases exceeding Eurocode upper limits. This issue is avoided in the alternative approach proposed based on limit analysis, as described in the following paragraph.

### *7.3 Preliminary design based on limit analysis*

The design procedure considers the forces and moments highlighted in Figure 87. The summation of the shear forces $V_{link,i}$ ($i$ = 1, …, $n_{links}$) with $n_{links}$ the number of links for each side column (typically equal to the number of storeys) gives the axial force $N_c$ on each side column:

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

Thus, the moment resisted by the two side columns is given by:

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

where $L_{tot}$ is the total length of the HCSW system obtained summing the length of the RC wall $l_w$ and two times the length of the steel links $l_{link}$. The ratio of the moment resisted by the two side columns by $M_c$ and the total resisted moment ($M_c$ + $M_w$) is called coupling ratio (CR):

$$CR = \frac{M_c}{M_c + M_w}$$

where the moment resisted by the RC wall is indicated with $M_w$. From the previous relation the moment resisted by the two side columns $M_c$ can be obtained from the moment resisted by the RC wall $M_w$ and the coupling ratio CR:

$$M_c = \frac{CR}{1-CR} M_w$$

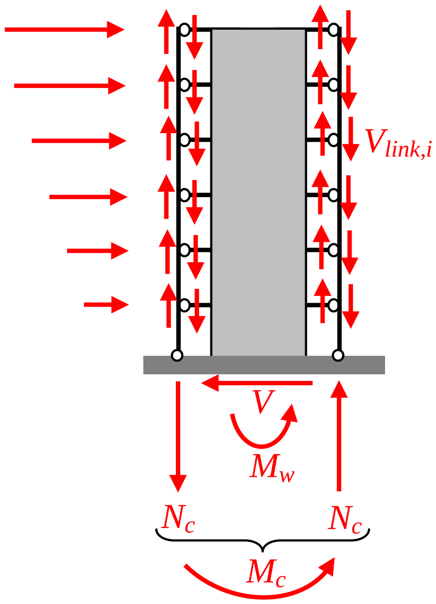



**Figure 87.** Horizontal actions and relevant resisting forces (axial, bending, and shear)

The working bending moment at the base of the RC wall is assigned using:

$$M_w = \frac{M_{w,Rd}}{\gamma_w}$$

thus reducing the wall bending capacity with a safety factor $\gamma_w > 1$ defining the RC wall capacity reserve. If the coupling ratio CR is chosen, then the axial force $N_c$ is consequently derived from:

$$N_c = \frac{M_c}{L_{tot}} = \frac{CR}{1-CR}\frac{M_w}{L_{tot}} = \frac{CR}{1-CR}\frac{M_{w,Rd}}{L_{tot}\gamma_w}$$

The distribution of the force $N_c$ among the links depends on the coupling ratio where a low CR fosters a uniform distribution of $N_c$ among the steel links while a higher CR fosters higher forces in the steel links at the bottom half of the HCSW system. Thus, the following distributions of shear force on the links can be assumed:

$$V_{link,i} = \psi_i \frac{N_c}{n_{links}} = \psi_i \frac{1}{n_{links}}\frac{CR}{1-CR}\frac{M_{w,Rd}}{L_{tot}\gamma_w}$$

where $\psi_i$ is a distribution coefficient. Two different values of $\psi_i$ are used and compared, namely uniform distribution:

$$\psi_i = 1$$

and non-uniform distribution:

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

The proposed design procedure is subdivided in the following steps, each commented with details, suggestions and possible critical aspects.

**Step 1**: definition of the wall geometry (length $l_w$, width $b_w$, and total height $H$).

Based on the performed preliminary studies, the suggested value for $H/l_w$ is 10 while $b_w$ has to be selected in order to accommodate links, give space to reinforcements, and provide adequate flexural and shear strength (to be verified a posteriori).

**Step 2**: design of the longitudinal reinforcements in order to maximize the wall flexural capacity.

Based on Eurocode 8 DCM rules, excessive reinforcements are automatically excluded and the bending moment capacity $M_{w,rd}$ of the wall can be directly determined using conventional nonlinear sectional analysis.

**Step 3**: design of the coupled resisting mechanism leading to the definition of the dissipative steel links.

The coupled resisting mechanism reviewed in the previous paragraph is designed in this step, indeed the most critical. The value $\gamma_{w,min}$ = 1.5 is suggested, in agreement with the previous applications made with the previous design approach. The length of the link $l_{link}$ and its cross section are assigned in order to have a system with stable energy dissipation, e.g. short or intermediate links. If link properties and classification are made according to Eurocode 8 paragraph 6.8.2, the link design resistances in bending and shear are computed according to Eurocode 8 Part 1 paragraph 6.8.2(3). The link categories are determined according to Eurocode 8 Part 1 paragraph 6.8.2(9) from their length *e*.

**Step 4**: design of the side steel columns.

The side steel columns are supposed to carry the summation of the shear forces in the links. Given that the columns are non-dissipative elements, they are dimensioned according to capacity design rules consistent with Eurocode 8 recommendations for similar systems, i.e. the design axial force is amplified according to the relation:

$$N_{c,Ed} = 1.1\gamma_{ov}\sum_{i=1}^{n_{links}} V_{link,i}$$

with $\gamma_{ov}$ = 1.25. In addition, the effect of the eccentricity between the axis of the column and the connection transferring the shear force between the link and the column must be considered in the design.

**Step 5**: design of the wall shear capacity.

Capacity design rules available for RC structures are adopted to allow adequate margin against non-ductile shear failure, assuming that the entire base shear is resisted by the RC wall alone (contribution of the two side columns is small and is ignored in this step). The design of the wall transverse reinforcements is made to provide a shear resistance $V_{Rd,w}$ in the wall that exceeds the maximum shear $V_{pl,w}$ in the limit condition of attainment of the ultimate bending moment in the wall and all links yielded with yielding force amplified as commented in the previous step:

$$V_{Rd,w} > V_{pl,w} = \frac{\gamma_w M_w + 1.1\gamma_{ov} M_c}{H_1}$$

where $H_1$ is the resultant height of the fundamental mode inertial force distribution or a fundamental-mode based equivalent lateral force distribution. The above equation can be rewritten as:

$$V_{Rd,w} > V_{pl,w} = \frac{1}{H_1}\left[ M_{w,Rd} + 1.1\gamma_{ov}(l_w + 2l_{link})\sum_{i=1}^{n_{links}} V_{link,i} \right]$$

and from the estimated maximum base shear at collapse, the transverse reinforcements for the reinforced concrete wall are designed.

## *7.4 Remarks on preliminary design based on limit analysis*

Applications of this design approach on a number of case studies gave very promising results through procedures that are quite simple. The optimal value of the coupling ratio was identified as CR = 0.60.

## *7.5 Design assessment of HCSW systems*

The final assessment of the designed HCSW system is made through nonlinear analysis, either static (pushover) or dynamic (multi-record incremental dynamic analysis). As previously commented, this is a necessary step to evaluate the post-elastic behaviour of the design outcomes and to tailor in the limit analysis design the accepted damage in the reinforced concrete wall under the design seismic input.

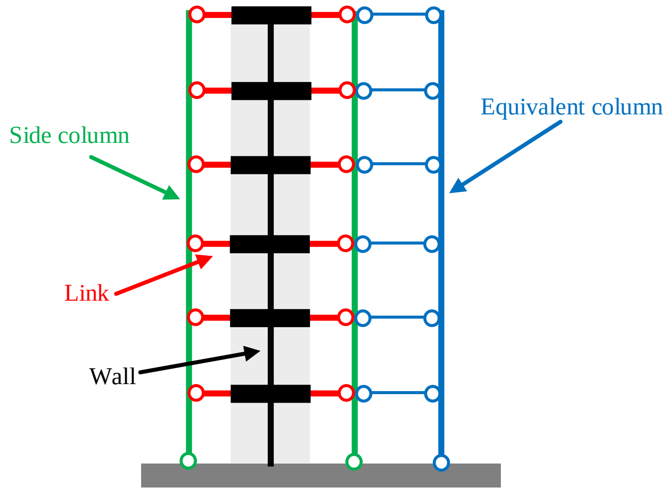



**Figure 88.** HCSW plane model used design assessment

A simple model implemented in a finite element software can be used to this end. The reinforced concrete shear wall is modelled using linear elastic frame elements (axial, flexural and shear deformability) with flexural elastic stiffness as obtained from the initial slope of the nonlinear moment curvature of its cross section, with nonlinear flexural hinges introduced at both ends of each tract of the wall between two subsequent floors. The steel shear links are modelled using linear elastic frame elements (axial, flexural and shear deformability) with nonlinear flexural hinges introduced in each link at the end clamped to the shear wall (point of maximum bending moment) as well as shear hinges introduced at mid span of each steel link (shear force constant along the link). The HCSW system is connected to a continuous steel column equivalent to the pertinent part of the gravity-resisting steel frame in a plane model or to the spatial frame in a three-dimensional model. Loads and masses are those of the HCSW system as well as loads and masses from the gravity-resisting steel frame. Geometric nonlinearity needs to be included in the analysis.

The suggested protocol for multi-record incremental dynamic analysis is the one adopted in this research project. Seven artificial accelerograms are used as seismic input, scaled from the scale factor (SF) 0.2 up to 1.2 with steps of 0.1 amplitude. Structural response results, also called engineering demand parameters (EDPs) are averaged over the seven accelerograms using the running mean with zero-length window, i.e., calculating values of the EDPs at each level of the SF and then finding the average and standard deviation of EDPs given the SF. The dispersion of the

EDPs for each SF is represented in a non-dimensional format using the ratio between the standard deviation and the mean value, ratio known as coefficient of variation (COV). This procedure works well up to the point where the first IDA curve reaches capacity. The IDA curves beyond such points are representative of a dynamical unstable behaviour and structural resurrection might occur for higher scale factors (Vamvatsikos and Cornell 2002). Structural resurrection occurs when a system is pushed all the way to global collapse at some intensity measures, only to reappear as non-collapsing at a higher intensity level, displaying high response but still standing. After the first IDA curve reaches capacity, the running mean can be computed conditional to survival, i.e., calculating the mean values based only on the results on of the analyses where the ultimate capacity is not yet attained, or unconditional to survival, i.e., calculating the mean values based on the results on of all analyses regardless the attainment of the ultimate capacity.

In this research study numerical instability in the evaluation of the nonlinear dynamic response and dynamic instability are considered equivalent, even thou numerical instability can suffer from the quality of the numerical code, the stepping of the integration, and the round-off error. Therefore, it is here assumed that such matters are taken care of as well as possible to allow for accurate enough predictions.

Given the above consideration, it is assumed that the maximum seismic capacity of the considered structure is achieved when its maximum capacity is attained for the first time in one the accelerograms considered, and possible structural resurrections for higher scale factors are neglected for the sake of safety. However, results are reported for the sake of completeness even for scale factors that are higher than the level at which the ultimate capacity is attained for the first time, with average values computed unconditional to survivals.

## 8. Design of innovative SRCW systems

The design of the proposed HCSW systems requires a preliminary dimensioning of the dissipative elements, reinforced concrete infill wall, steel beams and relevant connection and adjacent elements, followed by an assessment of the seismic behaviour through nonlinear analysis. The preliminary dimensioning can be based on limit analysis on a simple statically determinate scheme. The components of the SRCW system are directly dimensioned following capacity design rules in order to have an adequately post-elastic behaviour. The SRCW post-elastic behaviour and relevant capacity are afterwards assessed through nonlinear analysis.

### *8.1 Preliminary design procedure*

The proposed innovative SRCW is characterized by elements with specific tasks by allowing for the execution of a proper capacity design. The design procedure is force-based and is applied by considering the simple statically determined scheme representing the limit behaviour of the SRCW depicted in Figure 89. The 9 steps of the procedure are described hereafter.

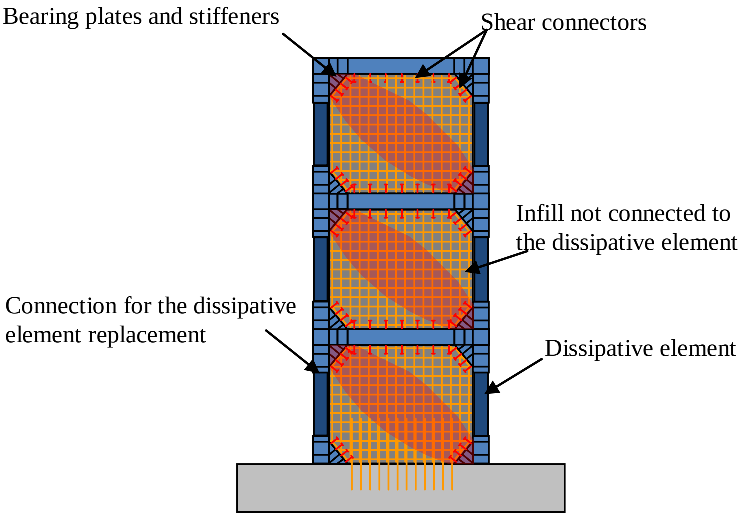



**Figure 89.** Innovative SRCW system

**Step 1**: definition of the static equivalent lateral loads (using a design spectrum accounting for a suitable behaviour factor) and calculation of the truss actions.

This is a delicate point as the proper values of the behaviour factors should reflect the real behaviour of the structure. The limit structural scheme adopted may not represent the behaviour of the system especially in the linear range for weak earthquakes.

**Step 2**: design of the cross sections of the ductile boundary elements in traction.

These elements are subjected also to compression under the reversed loadings but they are not expected to undergo plasticization under these forces. Even if in principle no specific provisions (e.g. the cross section class) are necessary, it is better to assure the elements being at least of Class 2 according to Eurocode 3. These elements may be constituted by pipes, hollow elements or hot rolled profiles. To ensure a good behaviour under compression, in order to optimise the design, HE profiles are preferred. Obviously, welded elements may also be used profitably even if this slows the shop phases and increases costs.

**Step 3**: capacity design of the connection of the ductile elements and of the adjacent elements.

The design of the connection of the ductile elements and of the adjacent elements is performed with the formula

$$R_d \geq 1.1\gamma_{ov} R_{fy}$$

where $\gamma_{ov}$ is the over-strength coefficient of the dissipative element with plastic capacity $R_{fy}$. The connection should be designed to resist the force in the linear range in order to reduce damage in the non-ductile elements and to permit the replacement of the ductile element after strong

shakings. End-plates connections should be preferred to other types with the ductile element connected to the split plate by means of full penetration welding.
Also the vertical elements to which the ductile elements are connected should be over-strengthened. This can be assured by using a different steel grade or by suitably enlarging the resisting cross section that should have transverse dimensions according to the infill wall thickness.

**Step 4**: calculation of geometric over-strength factors.

These factors are calculated as usual for steel structures by the ratio of the real plastic resistance of the ductile element and the relevant design force

$$\Omega = \min\left\{\Omega_i = \frac{N_{pl,Rd,i}}{N_{Ed,i}}\right\}$$

To guarantee yielding of the edge steel elements at the different levels, and to avoid formation of soft storeys, the maximum over-strength $\Omega_i$ should not differ from the minimum value by more than 25%. At the higher levels, this condition may be difficult to be satisfied. In such case, the yielding takes place only at a limited number of storeys and the ductile elements should be designed to guarantee the global ductility.

**Step 5**: calculation of axial forces in non-ductile elements by combining the effects of gravity loads with those of the seismic action suitably magnified.

These forces are calculated by suitably magnifying the seismic design component accounting for the material and geometric over-strength of the ductile elements with the usual formula

$$N_{Ed} = N_{Ed,G} + 1.1\gamma_{ov}\Omega N_{Ed,E}$$

If the systems are suitably arranged within the building, these may be not sensibly affected by gravitational loads.

**Step 6:** capacity design of the reinforced concrete infill against concrete crushing (wall thickness $t_w$ and width of the bearing plate $l_b$).

This step is crucial as it has to assure the good performance of the system that should not be affected by the wall failure (concrete crushing). As previously described, bearing plates are placed at the beam-to-column nodes to control the formation of the diagonal strut within the wall (Figure 90). A fan-shaped stress field is expected at the bearing plate: the effective width of the wall should be equal to the bearing plate width $l_b$ at the diagonal ends whereas the effective width is imposed by a coefficient $\alpha > 1$ at mid diagonal. It is worth to notice that in this region of the wall the concrete stress field is also characterised by transverse traction that reduces the relevant compression strength. The design formula

$$\left\{0.85\frac{f_{ck}}{\gamma_c}t_w l_b; 0.85\frac{f_{ck}}{\gamma_c}\nu\left(1-\frac{f_{ck}}{250}\right)(\alpha t_w l_b)\right\} \geq N_{Ed,G} + 1.1\gamma_{ov}\Omega N_{Ed,E}$$

is derived from Eurocode 2 (EN 1992-1-1:2004, 6.5); the second value of the strut strength takes into consideration the transverse tension ($\nu$ = 0.6 may be assumed) whereas the first value considers a simple compression field.

Width $l_b$ and coefficient $\alpha$ are the two design parameters that can be determined with a trial procedure or by imposing a tentative value for $\alpha$ (e.g. $\alpha$ = 2). The bearing plate should be then proportioned and suitably stiffened to avoid stress localization in the concrete.

The wall reinforcements should be checked to guarantee the diffusive mechanism that depends on the choice of parameter $\alpha$; for this purpose rules for partial discontinuity regions suggested by Eurocode 2 (point 6.5.2) are considered. In this case, tractions to be resisted by reinforcements is evaluated by means of the formula

$$T = \frac{1}{4}\left(1-\frac{1}{\alpha}\right)N_R$$

Two different reinforcement layouts may be adopted (Figure 91), the first one is constituted by two sets of orthogonal reinforcements whereas the second by a set of specific transverse (with respect to the strut direction) reinforcements. In the first case, vertical and horizontal reinforcements should fulfil the conditions

$$\frac{T^2}{f_{yd}^2} = {A_{sl}^v}^2 + {A_{sl}^h}^2 \qquad\qquad \frac{A_{sl}^h}{A_{sl}^v} = \frac{L}{h}$$

It is worth to note that the first reinforcement layout is simpler but maybe less stiff than the second that instead requires a third order of reinforcements that may be placed only in the case of thick walls.

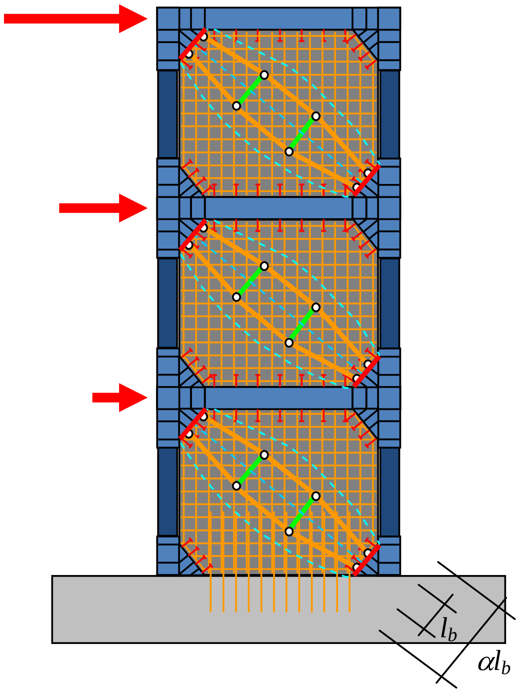
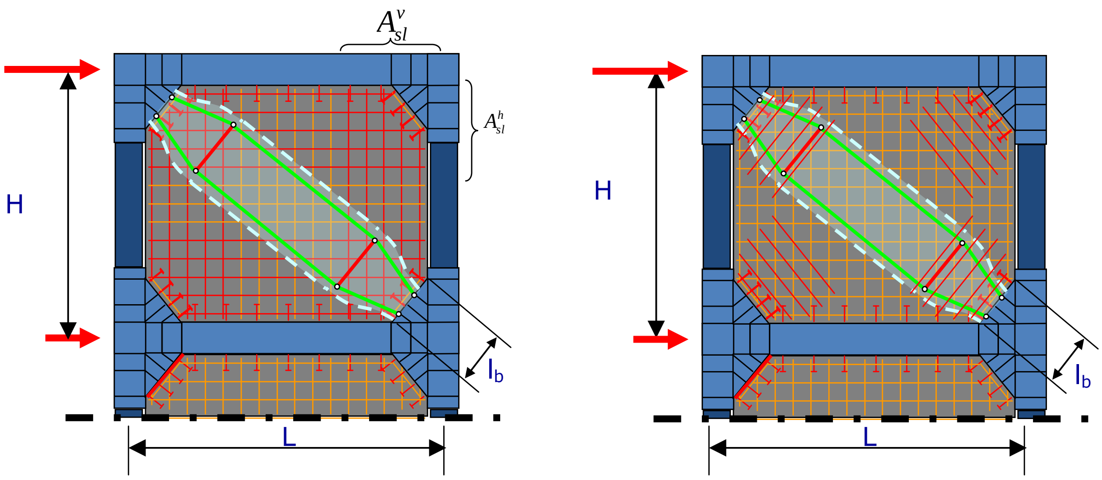



**Figure 90.** Diagonal struts within the infill walls




(a) (b)

**Figure 91.** Wall reinforcements: (a) with orthogonal rebar layout; (b) with additional stirrups

**Step 7**: design of the beams in traction.

Also these elements have to be designed to resist magnified axial forces. To improve the system feasibility, it is better that their width is compliant with the wall thickness.

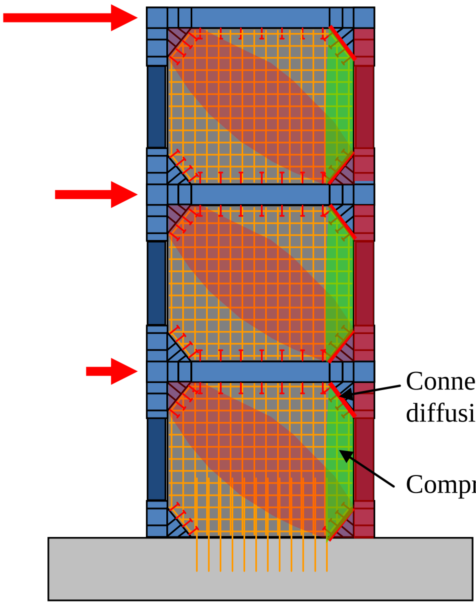



**Figure 92.** Compression fields involved to resist the axial force in the case in which lateral elements fail due to instability

**Step 8:** check and possible re-design of the compressed edge elements.

The ductile elements have to be checked for instability by using the formula

$$\frac{Af_y\chi}{\gamma_{M1}} \geq N_{Ed,G} + 1.1\gamma_{ov}\Omega N_{Ed,E}$$

The effective length of the element can be selected to be equal to the distance between the node zones enlarged due to the bearing plate. This verification is usually not critical for low rise buildings. In the case the verification is not satisfied, it is expected that the adjacent strip of the concrete wall collaborates to bear the compression force (Figure 92); in such a case, the following points (8-1, 8-2) have to be carried out.

**Step 8-1**: design of the shear connection between the wall and the frame.

The shear connection has to be designed to transmit to the adjacent RC wall the force in excess with respect to the bearing capacity of the ductile element given by

$$\upsilon_{Rd} \geq N_{Ed,G} + 1.1\gamma_{ov}\Omega N_{Ed,E} - \frac{Af_y\chi}{\gamma_{M1}}$$

The shear connection has to be placed at the vertical elements. It has to be designed by taking into account that possible splitting failure mechanisms of the wall may occur instead of the usual failures due to concrete crushing and the yielding. For this purposes, rules suggested by EN 1994-2:2005 (6.6.4 and Annex C) may be considered.

**Step 8-2**: check of the vertical strut developing in the wall.

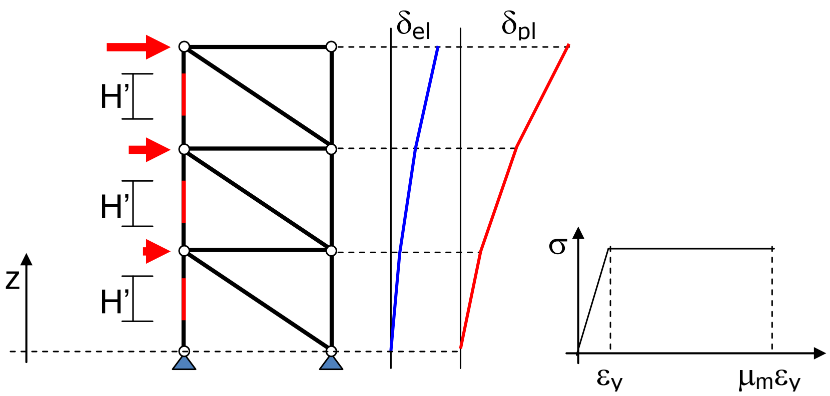
This element has to withstand the same force calculated and has to be suitably reinforced with confinement stirrups. The same detailing rules suggested for RC walls (EN 1998-1:2004 (5.4.3.4.2)) may be adopted.

**Step 9** calculation of the length of the dissipative element, in order to ensure the compliance between local and global ductility.

For this purpose, formulas derived by considering simplified mechanisms (Figure 93) can be adopted. To a first approximation, the following formula may be adopted:

$$H' = \frac{(\mu_s - 1)}{(\mu_m - 1)} \frac{\delta_{el} L}{\varepsilon_y \sum_{i=1}^{N} (z_N - z_{i-1})}$$

where $\mu_s$ and $\mu_m$ are the ductility of the structure and of the material of the dissipative element and the summation is extended only to elements that fall in the $\Omega \div 1.25\Omega$ range.




$\delta_{el}$ – Elastic displacement evaluated for the static equivalent loading inducing the first yielding

$\delta_{pl}$ – Plastic displacements evaluated by considering only the elements expected to yield

**Figure 93.** Elastic and plastic deformations of the SRCW

### *8.2 Some remarks on detailing rules*

Detailing rules suggested by EN 1998-1:2004 are not mandatory as the system is conceived as a latticed lateral resisting structure. Shear connectors are not placed at horizontal beams whereas they are placed at vertical elements only in the case in which the wall should bear a part of the compression force. Suitable transverse reinforcements have to be placed according to EN 1994-2:2005 (6.6.4 and Annex C).
The wall reinforcements should comply with EN 1992-1-1:2004 (9.6); confinement of edge regions is needed only when they are considered to resist compression force, in such cases rules suggested for RC walls (EN 1998-1:2004, 5.4.3.4.2) may be adopted.

### *8.3 Design assessment of SRCW systems*

A simplified model based on the same statically determined scheme used in the design can be adopted to evaluate static pushover curves and, thus, assess the post-elastic behaviour of the designed SRCW system. In fact, comparison between the pushover curve obtained using the simplified model and the pushover curve given by advanced finite element analysis shows good agreement. The yielding pattern, characterised by plastic strain only at the ductile elements, fully agrees with the dissipating mechanism to which the design procedure is aimed (Figure 94) with also the sequence of yielding of ductile elements well predicted in the design phase. The good agreement between results demonstrates that the design procedure proposed is suitable for the dimensioning of the system elements. The refined finite element model adopted in such a comparison is developed in the computer program ABAQUS by using shell elements. Geometry of the system is closely reproduced and the concrete infill walls are supposed to be connected only at the inclined bearing plates where stud connectors are placed. Wall reinforcements are considered by introducing two layers of reinforcements. A coarse mesh (mean size of 0.5 m) is adopted for the concrete walls whereas a more refined mesh (mean size of 0.1 m), is adopted for the steel members. A smeared cracking model with full shear retention is adopted for concrete by assuming the Mander's law in compression and a linear elastic law in tension; a linear softening branch is adopted to simulate the tension-stiffening of reinforcements after cracking. Elastoplastic-hardening models are considered for reinforcements and steel frame calibrating stiffness coefficients, yielding points and ultimate strengths with the mechanical characteristics of materials adopted in the design.

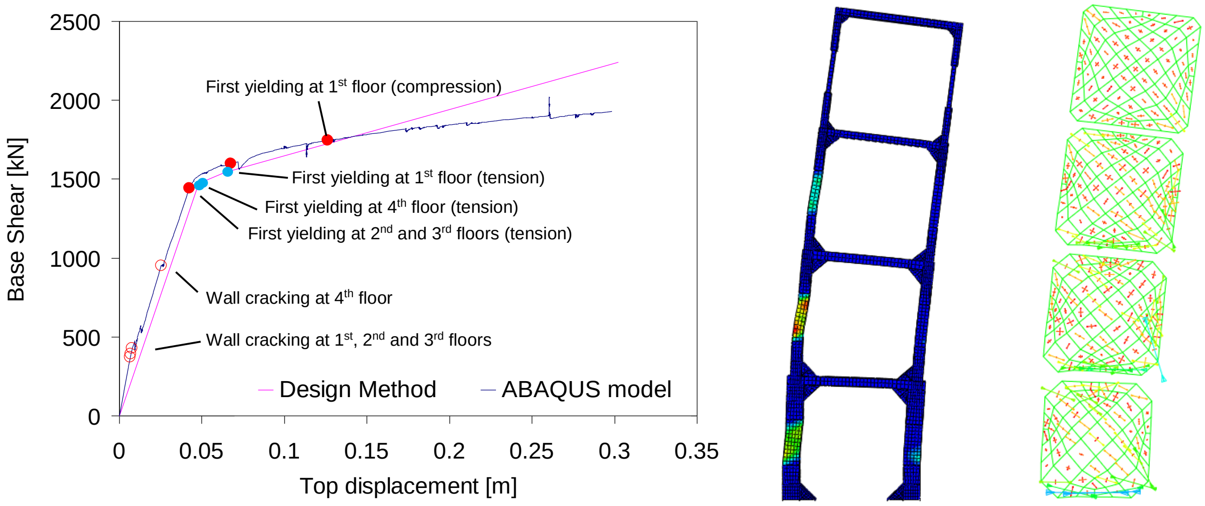


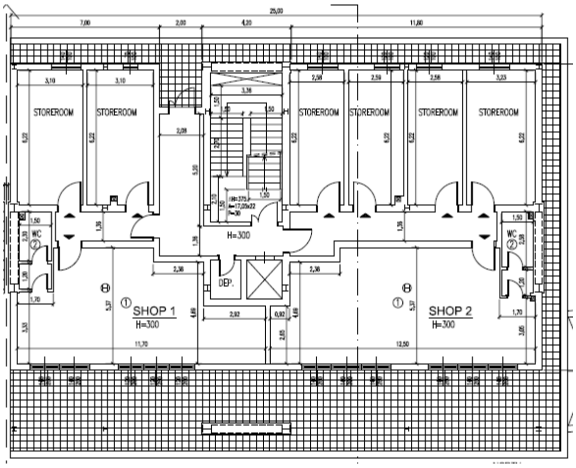

**Figure 94.** Comparison of obtained pushover curves (left) and stress fields obtained from an advanced model (right)

# 9. <u>Case studies</u>

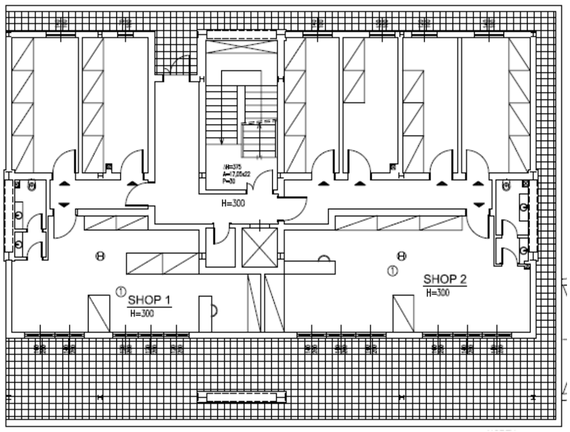
## *9.1 General description and functional requirements*

The functional and dimensional characteristics were chosen by considering suggestions from OCAM and SHELTER S.A. for what concerns the most diffused distribution of use at different floors while position, number and dimensions of seismic resistance structures was decided based on results of parametric analyses (WP4 and WP5). The location (Italy, Appenino mountains) was decided in order to face with a medium-high seismic input.
The considered building is set among an existing residential complex in Camerino, Italy, elevation 670 m above the sea level, and are composed by two symmetrical bodies, divided by a structural seismic gap. Each body holds parking at the basement, shops at the ground floor and residences at the upper floors; the module has a rectangular floor plan, with a longitudinal axis oriented east to west, covering six floors above ground (total height 21,45 meters above ground level), plus one underground floor for parking. The size of the rectangular module is about 25,00 x 14,15 meters, the interstorey height is 3.50m.
All lighting checks are satisfied in according to hygienic and sanitary laws; in particular for living spaces the ratio between area and area of windows is never less than 1/8. The elevator and the stairway are in the structure central core, so the core of service areas takes up the middle section of the structural grid. The flat roof is accessible from the internal stairway. Outside the space is provided by a large parking area to serve the shops and the residences and is equipped with green areas. All the spaces and paths are conceived according to Italian law about accessibility (Italian L.13/1989 and subsequent additions).
All external walls are low colour interrupted by stone cladding in correspondence of structural dissipative devices that are present in each external view. For floor coverings, soft grey porcelain stoneware slabs are chosen for their technical characteristics, such as durability and easy maintenance and cleaning. Aluminium profile with low-emissivity double glazes are used for the transparent surfaces and in the shops shatterproof glasses are used.
The considered case study is designed as having a gravity-resistant steel frame structure (floors, beams, columns) where beam to columns joints and restraints at the base of the columns can be considered as pinned connection. The gravity-resistant frame is connected to the innovative hybrid systems considered in this research project, i.e. in case A the HCSW systems is used, in case B the SRCW system is used, as later detailed.

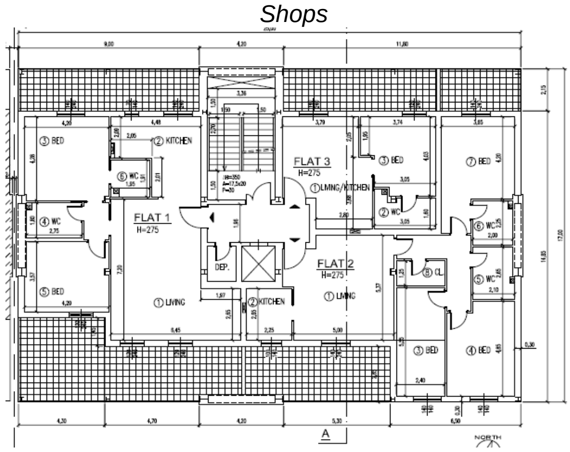

**Figure 95.** Masterplan and external views



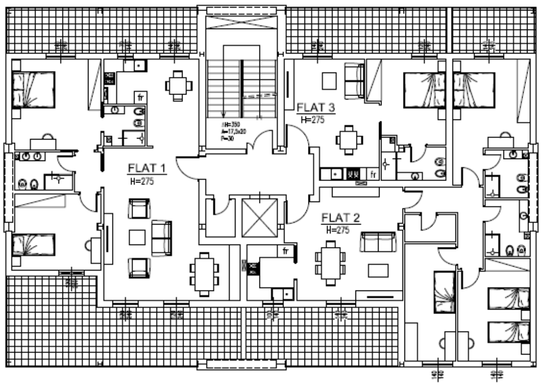

*Shops*




*Shops (furniture)*




*Apartments*




*Apartments (furniture)*

**Figure 96.** Shop and apartment floor

The analysis are conducted by considering the seismic action evaluated with a reference probability of exceedance of 10 % in 50 years ($T_{RN}$ = 475 years). The earthquake motion is represented by the Eurocode 8 elastic acceleration response spectrum, for importance factor $\gamma_I$ = 1, reference peak ground acceleration $a_g$ = 0.193$g$, spectrum type 1, ground type A.

## *9.2 Design and details of parts shared by the cases studies*

The design of the case studies concerns some aspects that are not influenced by the particular seismic resistance system, as gravity load structural system and architectonical solutions to provide acoustic and thermal comforts.

Gravity load system

The loads acting on the structure are: permanent actions of structural elements G1 (self-weight of the structural elements); non-structural members loads G2 (flooring, utilities, etc.); variable actions Q (imposed loads arising from occupancy). The combinations of actions for persistent or transient design situations is the fundamental combination employed for the ultimate limit state (ULS), according to Eurocode 1 part1. The self-weight of the structural elements are automatically computed by the calculation software based on the unit self-weight of concrete equal to 25 kN/m$^3$ and the unit self-weight of the steel elements equal to 78.5 kN/m$^3$. The permanent loads of the stairs is assumed equal to 4.75 kN/m$^2$. The external curtain is realized through two wood fibre mineralized panel, a rook wall panel including between the two layers and the external and internal surface is plastered. The unit weight of the wall is equal to 1.50 kN/m$^2$. For an interstorey height of 3.50 m, the distributed load on the beams is equal to 4.20 kN/m (the value is reduced of 0.8 times to account for the incidence of the openings). Furthermore it is hypothesized a distributed load of 0.15 kN/m to consider the self-weight of the railing. Beams and columns are designed according to Eurocode 3 prescriptions, having assumed steel grade S275 and a limitation to the vertical deflection at service limit state equal to L/250. The obtained results are reported in Figure 97.

**Table 5.** Summary of the gravity loads

| Description | Permanent loads $G_k$ [kN/m$^2$] | Live loads $Q_k$ [kN/m$^2$] |
|---|---|---|
| Ground floor | 8.75 | 4.00 ( Cat. D) |
| Floor type | 4.30 | 2.00 (Cat. A) |
| Roof floor | 3.30 | 2.00 (practicable roof)<br>1.97 (snow) |
| Stairs | 4.75 | 4.00 |
| External Curtains | 1.50 | |

**Table 6.** Gravity-resistant structure: columns

| Elevation | C1-6, C7, C12, C13, C18, C19-24 | C8-11, C14-17 |
|---|---|---|
| L1 | HE 200 B | HE 240 B |
| L2 | HE 200 B | HE 240 B |
| L3 | HE 200 A | HE 200 B |
| L4 | HE 200 A | HE 200 B |
| L5 | HE 160 A | HE 200 A |
| L6 | HE 160 A | HE 200 A |

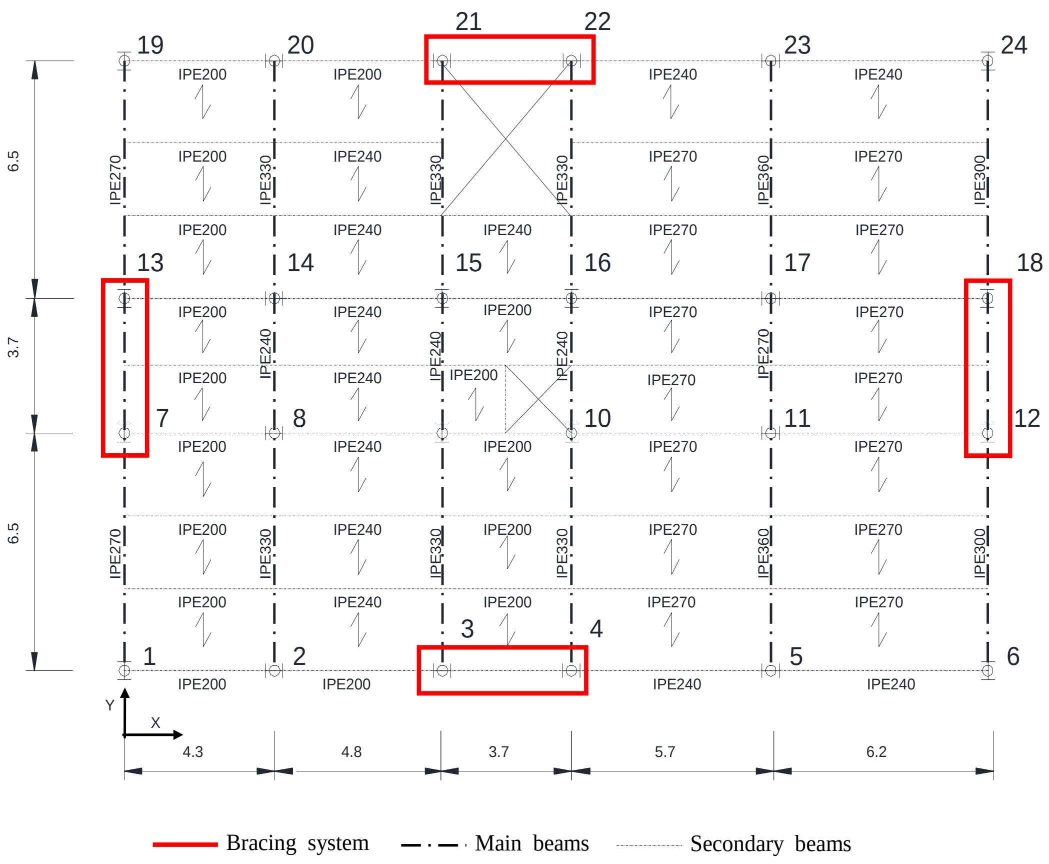
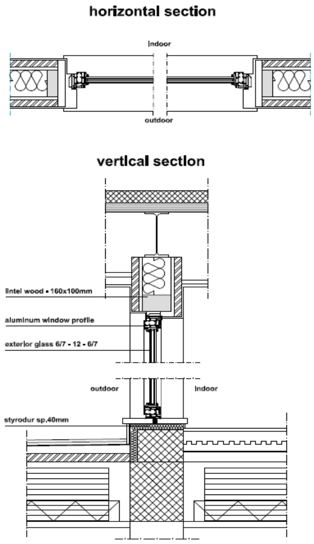
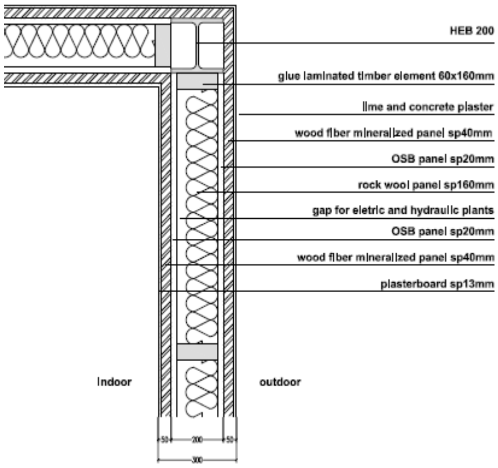


**Figure 97.** Gravity-resistant structure: beams

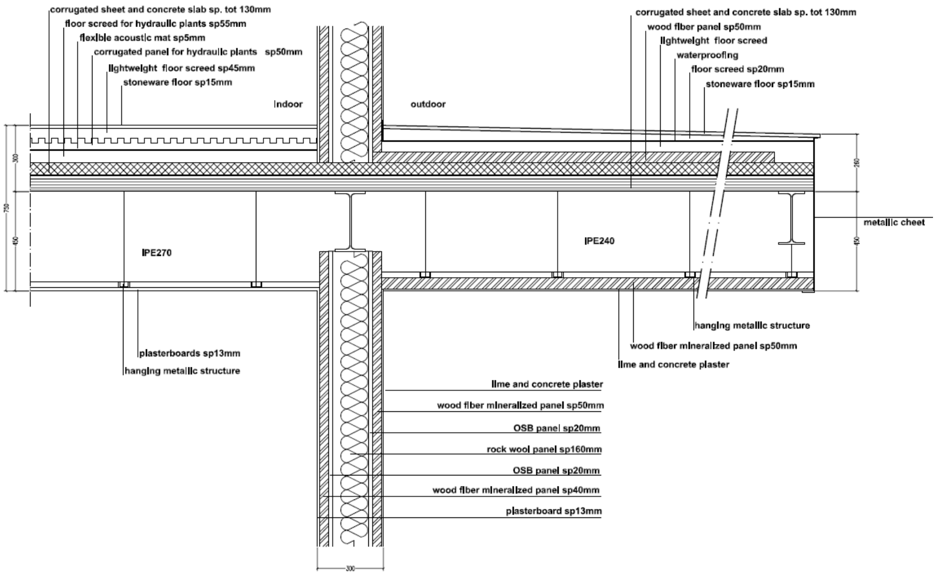
All horizontal and vertical partitions are designed to ensure acoustic and thermal comforts in respect with current Italian codes (DL311/2006 and subsequent additions). The following transmittance values are reached: external curtain: U=0,27 W/m$^2$K; roof floor plan: U=0,29 W/m$^2$K; underground walls: U=0,80 W/m$^2$K. External curtains and floor plans respects all the acoustic standards according to Italian DPCM 05/12/1997 and subsequent additions; in particular the following acoustic limits are considered: sound insulation of the internal partitions R'w: > 50 dB(A), standardized sound insulation of facade $D_{2mnt}$: > 42 dB(A), level of impact noise L'w: < 55 dB(A), noise produced by plants: < 35 dB(A).  The most interesting details are reported in the following figure.



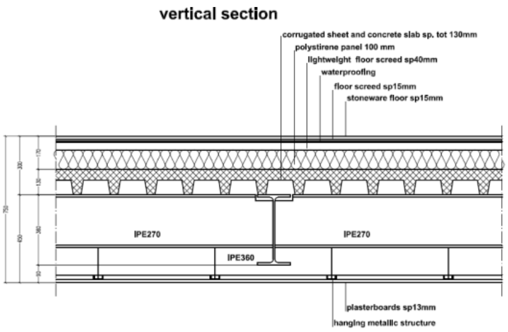

*External windows*



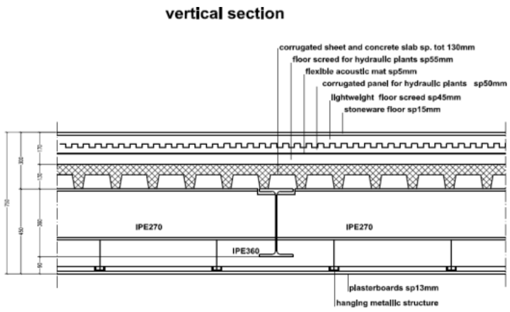

*External curtain*




*Terrace node*




*Roof floor*




*Apartment floor*

**Figure 98.** Architectural details

## *9.3 Structural design and details of the HSCW seismic resistant system*

The structural design considering the HCSW seismic resistant system, includes:
- a preliminary design method oriented to rapidly evaluate the effectiveness of different choices of characteristic parameters (i.e. coupling ratio and wall slenderness) found in WP4 in order to define the optimal solution;
- 2D nonlinear static and dynamic analyses for the evaluation of the effectiveness of the post-elastic behaviour and for the refinement of the wall, reinforcement and link sizing;
- 3D analyses for the final assessment of the solution and the design of details.

This design path, starting from preliminary analyses for the choice of the most important parameters and leading to final safety verification and design of the details, is described in an extended way in the following paragraphs whose contents can be used as guidelines for the design of this structural system.

*Preliminary analysis.*

The preliminary (simplified) design procedure is initially used to explore and compare the seismic performance of different solutions in order to optimize the final structural system. The chosen aspect ratio $H/l_w$ of the wall aspect ratio is 10 while, as suggested by the conclusions of WP4 and the wall thickness $b_w$ is selected in order to accommodate links, give space to reinforcements, and provide adequate flexural and shear strength. In the selected case study $H$ = 21.00 m, hence $l_w$ = 2.10 m and $b_w$ = 0.36 m. Concrete C30 ($f_{ck}$ = 30 MPa) and reinforcements B450C ($f_{yk}$ = 450 MPa) are assigned.

The length of the link $l_{link}$ and its cross section are assigned in order to have a system with stable energy dissipation, e.g. short or intermediate links. Initially, three values of CR are considered, i.e. 0.60, 0.70, and 0.80; they provided the link dimensions reported in the table. The side steel columns are supposed to carry the summation of the shear forces in the links. Given that the columns are non-dissipative elements, they are dimensioned according to capacity design rules consistent with Eurocode 8 recommendations for similar systems. The application of the design rules to the case study leads to the results collected in the following tables, where the bending moment due to the assumed eccentricity is computed according to the scheme of continuous column with lateral restraints, i.e. $M_{Ed} = 0.5eN_{Ed}$. It is observed that, as expected, the higher the CR, the higher the demand in the side column.

Capacity design rules available for RC structures are adopted to allow adequate margin against non-ductile shear failure, assuming that the entire base shear is resisted by the RC wall alone (contribution of the two side columns is small and is ignored in this step). The design of the wall transverse reinforcements is made to provide a shear resistance $V_{Rd,w}$ in the wall that exceeds the maximum shear $V_{pl,w}$ in the limit condition of attainment of the ultimate bending moment in the wall and all links yielded with yielding force amplified (over-strength).

Given that the designs CR = 0.70 and CR = 0.80 result in quite high quantities of shear reinforcements as well as quite large steel side columns with possible problems in the foundations, only the case with CR = 0.60 is considered of interest and its structural performances assessed.

**Table 7.** Design of the steel links

| CR | $M_{w,Rd}$ | $n_{links}$ | $l_w$ | $l_{links}$ | section | $e_S$ | $e_L$ | $f_y$ | $M_{p,link}$ | $V_{p,link}$ | $\gamma_w$ |
|---|---|---|---|---|---|---|---|---|---|---|---|
| (-) | (kNm) | (-) | (mm) | (mm) | (-) | (mm) | (mm) | (MPa) | (kNm) | (kNm) | (-) |
| 0.60 | 4050.67 | 6 | 2100 | 500 | IPE270 | 289 | 542 | 355 | 127.00 | 351.44 | 1.75 |
| 0.70 | 4050.67 | 6 | 2100 | 500 | IPE330 | 340 | 637 | 355 | 208.04 | 489.60 | 1.84 |
| 0.80 | 4050.67 | 6 | 2100 | 500 | IPE400 | 392 | 734 | 355 | 333.41 | 681.26 | 1.72 |

**Table 8.** Design of the steel side columns

| Storey | Section | $N_{Ed}$ | $M_{Ed}$ | $\sigma_{Ed}$ |
|---|---|---|---|---|
| | | (kN) | (kNm) | (MPa) |
| 6 | HE400B | 483.23 | 48.32 | 51.21 |
| 5 | HE400B | 966.46 | 96.65 | 102.59 |
| 4 | HE400B | 1449.69 | 144.97 | 154.14 |
| 3 | HE400B | 1932.92 | 193.29 | 205.86 |
| 2 | HE400B | 2416.15 | 241.61 | 257.76 |
| 1 | HE400B | 2899.38 | 289.94 | 309.84 |

*CR=0.60*

| Storey | Section | $N_{Ed}$ | $M_{Ed}$ | $\sigma_{Ed}$ |
|---|---|---|---|---|
| | | (kN) | (kNm) | (MPa) |
| 6 | HE500B | 673.20 | 67.32 | 56.47 |
| 5 | HE500B | 1346.39 | 134.64 | 113.09 |
| 4 | HE500B | 2019.59 | 201.96 | 169.85 |
| 3 | HE500B | 2692.78 | 269.28 | 226.77 |
| 2 | HE500B | 3365.98 | 336.60 | 283.84 |
| 1 | HE500B | 4039.17 | 403.92 | 341.07 |

*CR=0.70*

| Storey | Section | $N_{Ed}$ | $M_{Ed}$ | $\sigma_{Ed}$ |
|---|---|---|---|---|
| | | (kN) | (kNm) | (MPa) |
| 6 | HE700B | 936.74 | 93.67 | 63.44 |
| 5 | HE700B | 1873.48 | 187.35 | 126.97 |
| 4 | HE700B | 2810.22 | 281.02 | 190.61 |
| 3 | HE700B | 3746.95 | 374.70 | 254.34 |
| 2 | HE700B | 4683.69 | 468.37 | 318.17 |
| 1 | HE700M | 5620.43 | 562.04 | 302.95 |

*CR=0.80*

Refined analyses

The structural component dimensions were confirmed in the other refined nonlinear analyses on 2D and 3D models, including also the control of second order effect at ULS and the deformation check at SLS. Nonlinear static and dynamic analyses are reported in the following figures to show the progressive yielding of the steel links and the exploitation of their dissipative properties. It is observed that the steel links are all yielded in bending when the wall is still in the undamaged state in the modal distribution of horizontal forces while all links are yielded but the last two floors when the wall is still in the elastic range in the mass distribution of horizontal forces. In the figures concerning the static nonlinear analysis, the capacity curves computed by using mean values of the material strength are reported together with the demand spectra to evaluate the expected mean level of damage under earthquake.

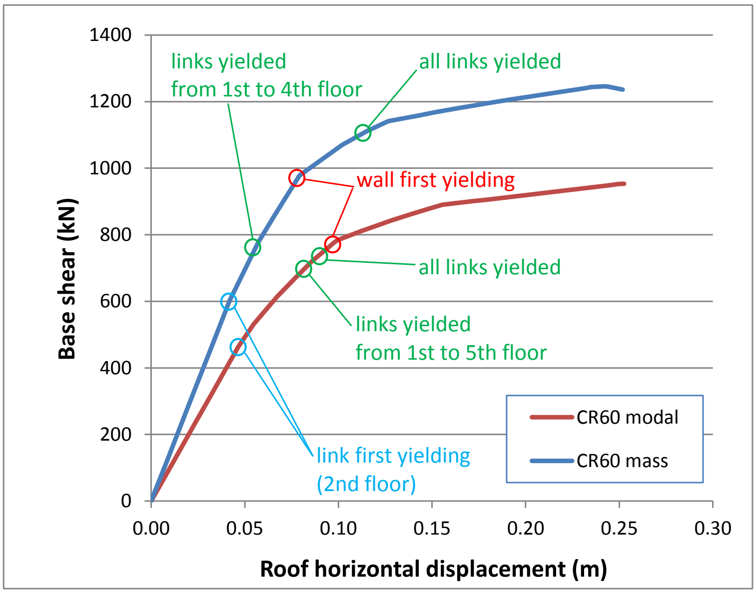



**Figure 99.** Pushover curves for the HCSW system designed with CR = 0.60 with indication of the hinge state

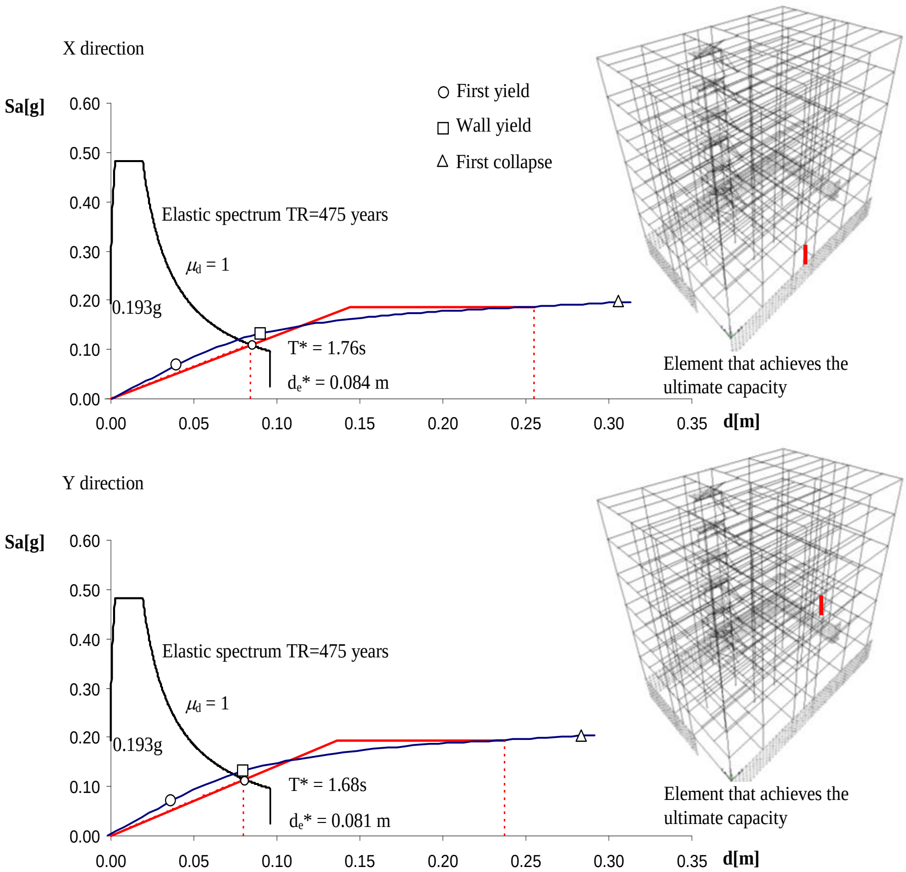



**Figure 100.** Nonlinear static pushover analyses: forces proportional to the first mode

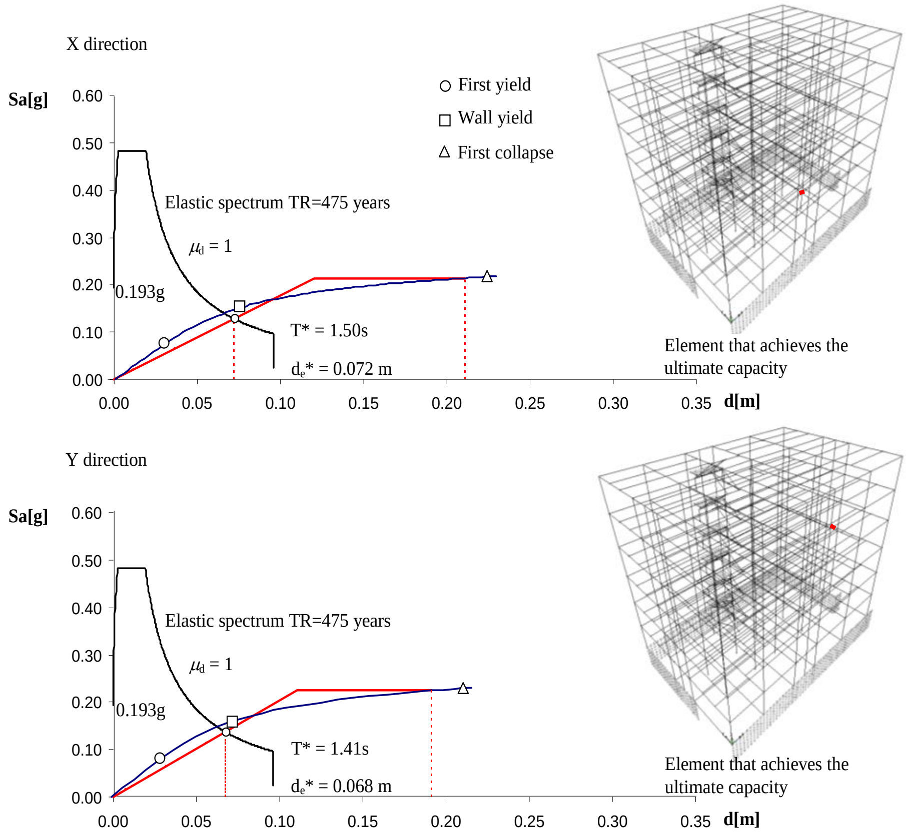



**Figure 101.** Nonlinear static pushover analyses: forces proportional to the masses

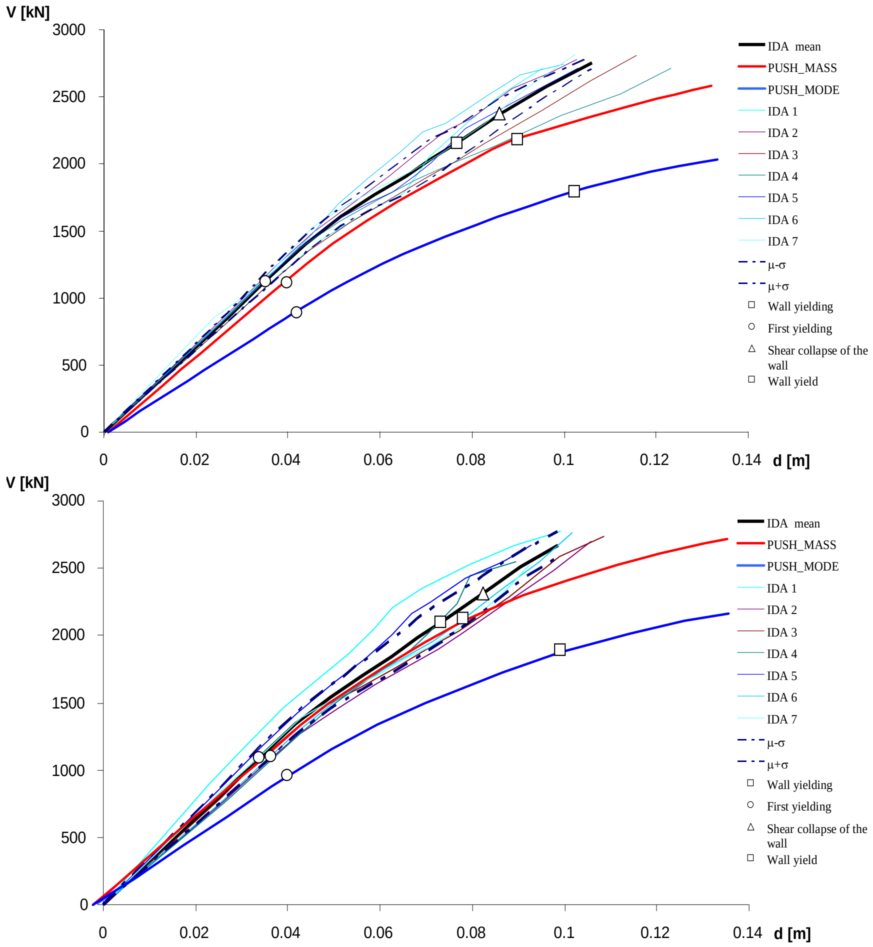
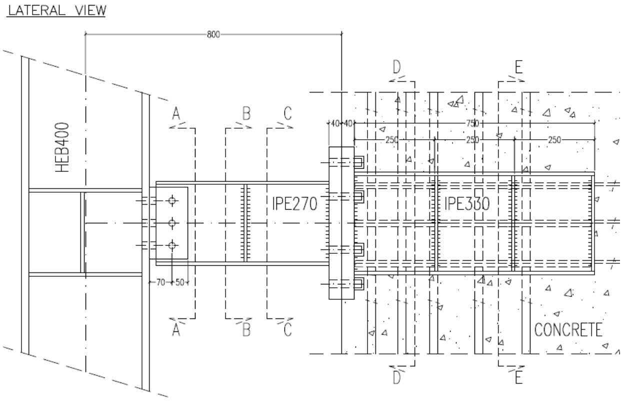
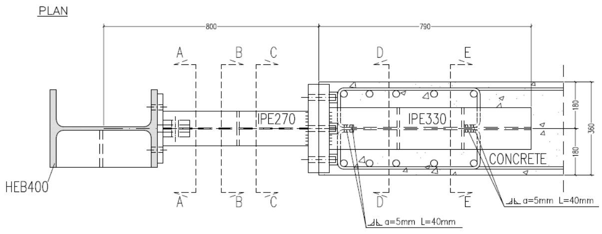



**Figure 102.** Comparison between static pushover and dynamic pushover curves

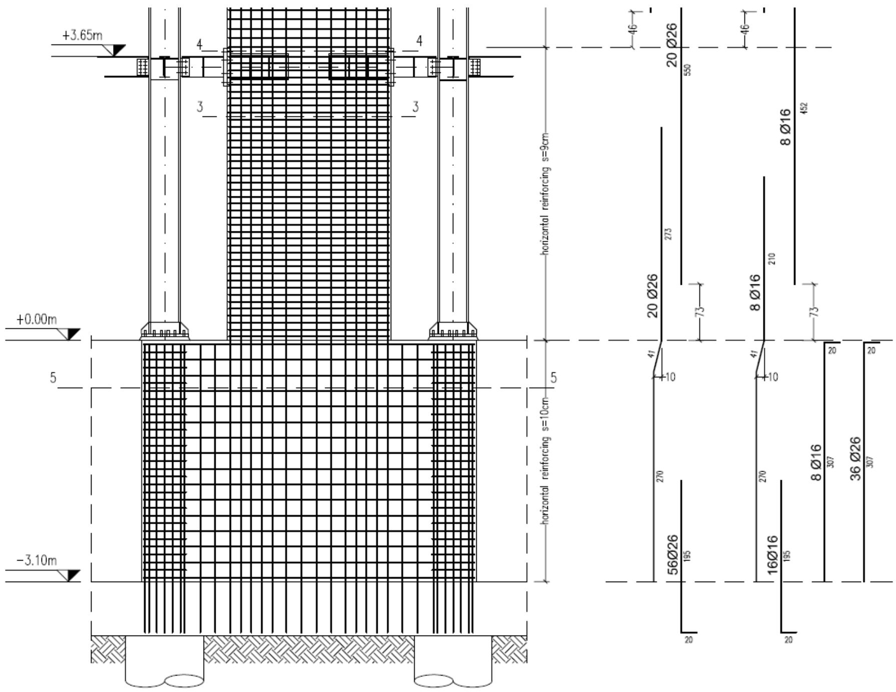
*Structural details*

A selection of the most interesting details concerning the dissipative components are reported in the following. They describe the link and its connection with the concrete wall.

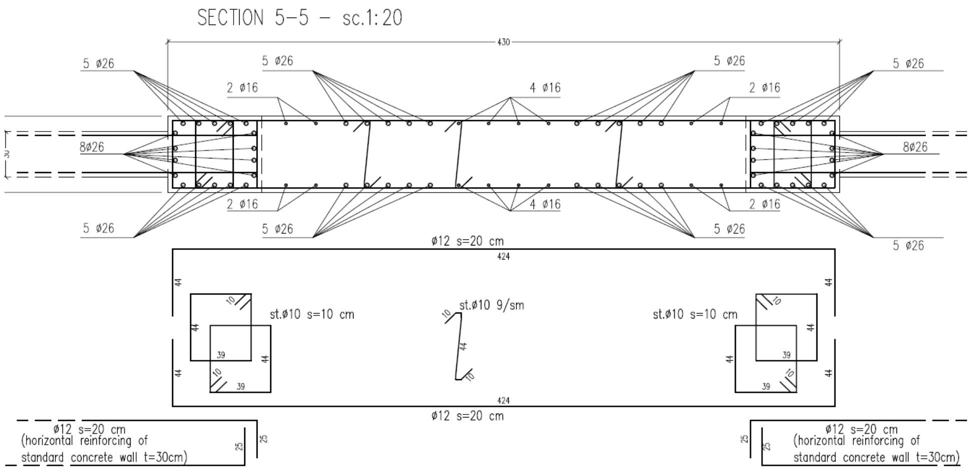



*Link: lateral view* *Link: details*




*RC Wall: basement and ground floor lateral view and reinforcements*




*RC Wall: ground floor cross-section and reinforcements*

**Figure 103.** HSCW system: structural details

## *9.4 Structural design and details of SRCW seismic resistant system*

The design procedure permitting to define an optimal solution by starting from simplified analyses includes:
- a preliminary design method oriented to rapidly evaluate the effectiveness of different choices of characteristic parameters (i.e. length of the dissipative components, RC wall thickness and global aspect ratio of the bracing) found in WP5 in order to define the optimal solution;
- 3D analyses for the final assessment of the solution and the design of details.

This design path, starting from preliminary analyses for the choice of the most important parameters and leading to final safety verification and design of the details, is described in the relevant whose contents can be used as guidelines for the design of this structural system

Preliminary analysis

The preliminary design procedure is subdivided in steps, each commented in the previous paragraphs with details, suggestions and possible critical aspects as well as application to the considered case study. In particular, design consists of 9 steps, it is force-based and is applied by considering a simple static determined scheme. This procedure provides the dimensions required for all the structural components: dimensions (cross-section and length) of dissipative elements and characteristics of their connection, dimensions of beams and columns, thickness of the RC wall, reinforcements due to shear and bending, edge elements and concrete-steel connection.

The seismic actions are evaluated by assuming a tentative behaviour factor equal to 3.3 and the fundamental period $T_1$ of the system, computed according to the EN 1998-1:2005 by posing the $C_t$ coefficient to 0.05, is equal to 0.49 s. The seismic base shear force $F_b$ is equal to 631.99 kN and the distribution of the lateral loads is summarized in the following figure, together with the simplified model of the earthquake resistant system.

The boundary element are mainly subjected to traction and the to compression under the reversed loadings should not provide plasticization. To ensure a good behaviour under compression and to optimise the design, HE profiles are chosen. Structural steel S235 ($f_y$ = 235 MPa) is assigned to the ductile elements. The sections obtained from the design are detailed in the table below. Side columns and relevant connections are designed by using the local over-strength factor $1.1\gamma_{ov}$. and by controlling the global ductility by evaluating the differences in over-strength at different levels. These differences are below 25% in the lower 4 levels and it is expected that yielding does not occur at the top two levels.

| Storey | $z_i$ | $F_i$ |
|---|---|---|
| [---] | [m] | [kN] |
| L1 | 3.50 | 32.147 |
| L2 | 7.0 | 64.295 |
| L3 | 10.50 | 96.442 |
| L4 | 14.0 | 128.590 |
| L5 | 17.50 | 157.084 |
| L6 | 21.0 | 153.431 |

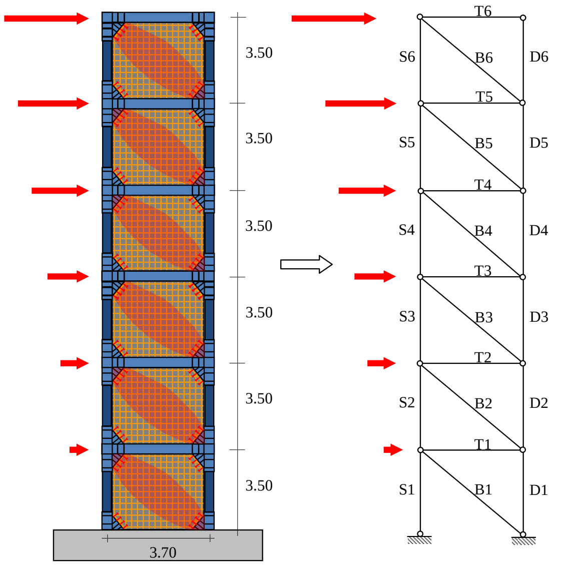



**Figure 104.** Design seismic force

The RC wall capacity design against concrete crushing is a crucial step as it assures the good performance of the system that should not be affected by the concrete failure. In a first passage the thickness $t_w$ and width of the bearing plate $l_b$ is calculated. In this case study a concrete class C30 ($f_{ck}$ = 30 MPa) and reinforcements B450C ($f_{yk}$ = 450 MPa) are assigned. Bearing plates are placed at the beam-to-columns nodes to ensure the formation of the diagonal strut within the wall. One can observe that there is a fan-shaped stress field within the concrete infill wall. The thickness has been found according to the design formula reported in the previous paragraph. The results of the RC wall design and the wall dimensions are reported in the following table.

**Table 9.** Results of the design of the ductile elements and wall dimensions

| Storey | $N_{Ed}$ | $f_y$ | Sections |
|---|---|---|---|
| [---] | [kN] | [MPa] | [---] |
| L1 | 2017.596 | 235 | HE260A |
| L2 | 1509.817 | 235 | HE220A |
| L3 | 1032.446 | 235 | HE180A |
| L4 | 615.895 | 235 | HE140A |
| L5 | 290.574 | 235 | HE140A |
| L6 | 86.890 | 235 | HE140A |

| Storey | $N_{Ed}$ | $t_w$ | $l_b$ | $N_{Rd1}$ | $N_{Rd2}$ | $N_{Rd}$ |
|---|---|---|---|---|---|---|
| [---] | [kN] | [m] | [m] | [kN] | [kN] | [kN] |
| L1 | -1586.69 | 0.22 | 0.426 | 1593.240 | 1682.461 | 1593.240 |
| L2 | -1525.76 | 0.22 | 0.409 | 1529.013 | 1614.638 | 1529.013 |
| L3 | -1403.89 | 0.22 | 0.376 | 1405.457 | 1484.162 | 1405.457 |
| L4 | -1221.08 | 0.2 | 0.360 | 1221.526 | 1289.931 | 1221.526 |
| L5 | -588.574 | 0.16 | 0.217 | 591.574 | 624.702 | 591.574 |
| L6 | -290.825 | 0.16 | 0.107 | 292.043 | 308.397 | 292.043 |

The beam elements are designed to withstand the magnified axial forces. Their width is compliant with the wall thickness. for simplicity the same section of the non-ductile elements are employed.

The shear connection to be placed at the vertical elements is finally designed by taking into account that possible splitting failure mechanisms of the wall may occur instead of the usual failures due to concrete crushing and yielding. For this purposes, rules suggested by EN 1994-2 (6.6.4 and Annex C) are considered. The rules leads to the results collected in the following table.

**Table 10.** Design of the headed studs

| Storey | $d$ | $h$ | $a$ | $n_r$ | $n_s$ | $P_{Rd,s}$ | $P_{Rd,c}$ | $P_{Rd,L}$ | $P_{Rd}$ | $v_{Rd}$ | Ver. |
|---|---|---|---|---|---|---|---|---|---|---|---|
| [---] | [mm] | [---] | [mm] | [---] | [---] | [kN] | [kN] | [kN] | [kN] | [kN] | [---] |
| L1 | 19 | 190 | 110 | 2 | 9 | 90.729 | 83.126 | 84.186 | 83.126 | 1496.3 | **OK** |
| L2 | 19 | 190 | 110 | 2 | 9 | 90.729 | 83.126 | 84.186 | 83.126 | 1496.3 | **OK** |
| L3 | 19 | 190 | 170 | 2 | 6 | 90.729 | 83.126 | 95.931 | 83.126 | 997.51 | **OK** |
| L4 | 19 | 190 | 170 | 1 | 6 | 90.729 | 83.126 | 104.56 | 83.126 | 498.75 | **OK** |

Although the headed studs are not necessary at the fifth and sixth storey to transmit the axial force at the compressed edge, one disposes equally some connectors at the bearing plate to the purpose to connect the wall with the steel frame.

Refined analyses

The structural component dimensions were confirmed in the other refined nonlinear analyses on 2D and 3D models, including also the control of second order effect at ULS and the deformation check at SLS. The following figures describe the nonlinear response in x direction and y direction obtained by static and multi-record dynamic analyses. The curves are extended up to the ULS seismic intensity and they show the moderate damage of the RC wall also for strong earthquake. As usual, the dynamic pushover curve is stiffer than the curve provided by a static analysis as a consequence of higher mode contributions, even if the difference observed in this case are smaller with respect to the previous solution A (HSCW).

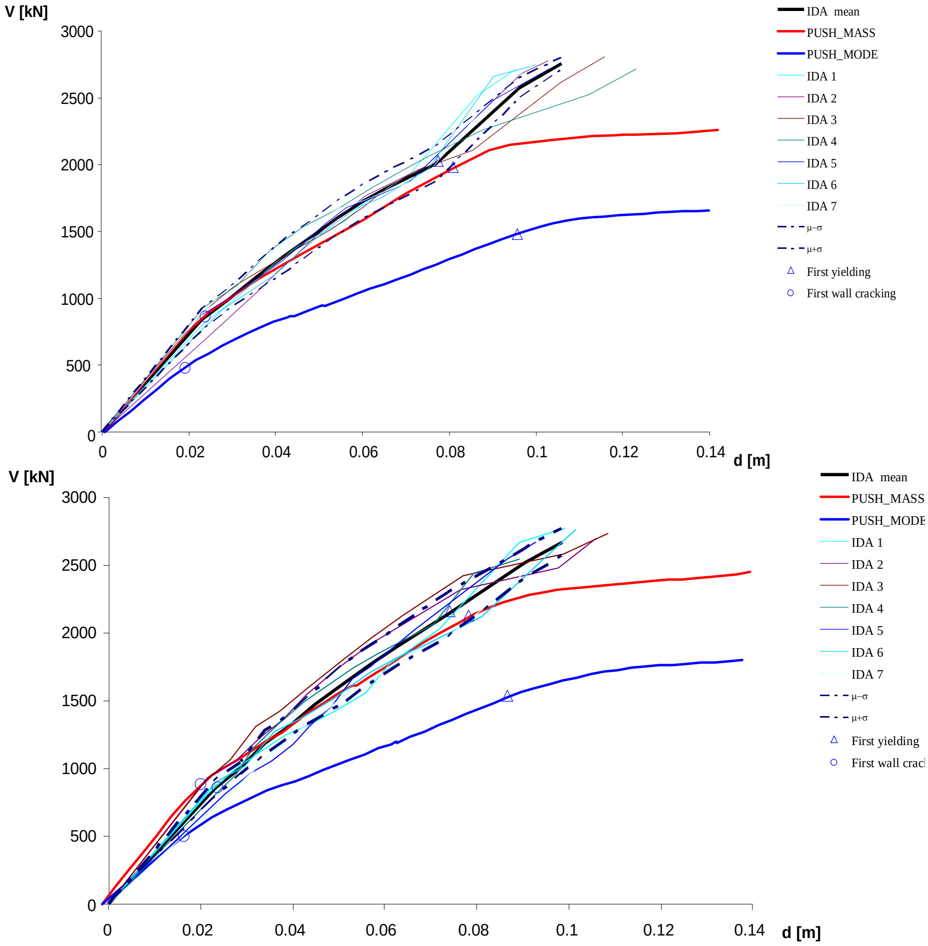



**Figure 105.** Comparison between static and dynamic pushover in x and y direction

Structural details
A selection of the most interesting details concerning the dissipative components are reported in the following figure.

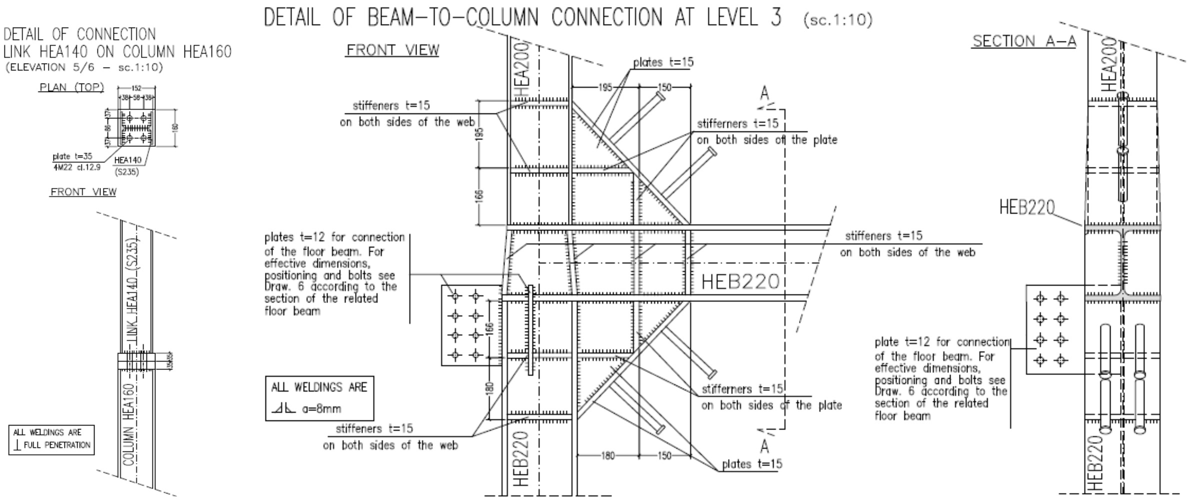



**Figure 106.** SRCW system: structural details

## *9.5 Construction process and economical evaluations*

In the following, the total weight and the cost analysis concerning the steel structure is reported for the case A (HSCW) and the case B (SRCW). The global costs of all the structural components (steel and concrete) are reported in the following tables, while the construction planning is reported in the last tables, considering both structural and non-structural parts and temporal intersection among different works. The costs concerning non-structural elements are not reported because they are not influenced by the particular structural solution and they are the same for both the systems (HCSW and SRCW). The two solutions were judged competitive by the industrial partners

**Table 11.** Total weight and cost analysis of HSCW system

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**RFSR-CT-2010-00025 - Proposal RFS-PR-09003**

**Project: INNO HYCO - INNOvative and COmposite steel-concretestructural solutions for buildings in seismic area**

| Structural types | WEIGHT (kg) | ANALYSIS OF COSTS: RAW MATERIAL | WORKING | TREATMENT | INSTALLATION | TECHNICAL COSTS | TRANSPORTS | TOTAL DIRECT COSTS | OVERHEAD | GROSS OPERATING MARGIN |
|---|---|---|---|---|---|---|---|---|---|---|
| BEAMS | 70.227,30 | 1,00 | 0,39 | 0,28 | 0,44 | 0,10 | 0,10 | | 112% | 110% |
| | | € 70.227,30 | € 27.388,65 | € 19.663,64 | € 30.900,01 | € 7.022,73 | € 7.022,73 | € 162.225,06 | € 181.692,07 | € 199.861,28 |
| PILLARS | 47.801,42 | 1,10 | 0,30 | 0,28 | 0,44 | 0,10 | 0,10 | | 112% | 110% |
| | | € 52.581,56 | € 14.340,43 | € 13.384,40 | € 21.032,62 | € 4.780,14 | € 4.780,14 | € 110.899,29 | € 124.207,21 | € 136.627,93 |
| STAIR | 6.466,29 | 1,00 | 0,60 | 0,40 | 0,44 | 0,10 | 0,10 | | 112% | 110% |
| | | € 6.466,29 | € 3.879,77 | € 2.586,52 | € 2.845,17 | € 646,63 | € 646,63 | € 17.071,01 | € 19.119,53 | € 21.031,48 |
| ANCHOR BOLTS, BRACINGS, PLATES AND BOLTS | 17.320,42 | 1,56 | 1,00 | 0,28 | 0,44 | 0,10 | 0,10 | | 112% | 110% |
| | | € 27.019,86 | € 17.320,42 | € 4.849,72 | € 7.620,98 | € 1.732,04 | € 1.732,04 | € 60.275,06 | € 67.508,07 | € 74.258,88 |
| HEADED STUDS "NELSON" TYPE | 600 | 2 | 0,28 | 0 | 10 | 0,3 | 1 | 0 | 112% | 110% |
| | | € 1.200,00 | € 168,00 | € 0,00 | € 6.000,00 | € 180,00 | € 600,00 | € 8.148,00 | € 9.125,76 | € 10.038,34 |
| RAILING | 3.500,00 | 1,20 | 1,00 | 0,80 | 1,00 | 0,60 | 0,50 | 0 | 112% | 110% |
| | | € 4.200,00 | € 3.500,00 | € 2.800,00 | € 3.500,00 | € 2.100,00 | € 1.750,00 | € 17.850,00 | € 19.992,00 | € 21.991,20 |
| TREAD | 3.900,00 | 1,5 | 0,5 | 0,5 | 0,5 | 0,1 | 0,15 | 0 | 112% | 110% |
| | | € 5.850,00 | € 1.950,00 | € 1.950,00 | € 1.950,00 | € 390,00 | € 585,00 | € 12.675,00 | € 14.196,00 | € 15.615,60 |
| CORRUGATED SHEET 8/10 | 28.300,00 | 1,05 | | | 0,5 | 0,1 | 0,15 | 0 | 112% | 110% |
| | | € 29.715,00 | € 0,00 | € 0,00 | € 14.150,00 | € 2.830,00 | € 4.245,00 | € 50.940,00 | € 57.052,80 | € 62.758,08 |

| | | | |
|---|---|---|---|
| TOTAL WEIGHT | 178.115,43 | TOTAL COSTS | € 542.182,78 |

**Final result from cost analysis of steel structures is of 3,04 €/kg.**

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**RFSR-CT-2010-00025 - Proposal RFS-PR-09003**

**Project: INNO HYCO - INNOvative and COmposite steel-concretestructural solutions for buildings in seismic area**

| | | ANALYSIS OF COSTS | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|
| Structural types | WEIGHT (kg) | RAW MATERIAL | WORKING | TREATMENT | INSTALLATION | TECHNICAL COSTS | TRANSPORTS | TOTAL DIRECT COSTS | OVERHEAD | GROSS OPERATING MARGIN |
| BEAMS | 70.362,61 | 1,00 | 0,37 | 0,28 | 0,44 | 0,10 | 0,10 | | 112% | 110% |
| | | € 70.362,61 | € 26.034,17 | € 19.701,53 | € 30.959,55 | € 7.036,26 | € 7.036,26 | € 161.130,38 | € 180.466,02 | € 198.512,62 |
| PILLARS | 31.021,10 | 1,00 | 0,50 | 0,28 | 0,44 | 0,10 | 0,10 | | 112% | 110% |
| | | € 31.021,10 | € 15.510,55 | € 8.685,91 | € 13.649,28 | € 3.102,11 | € 3.102,11 | € 75.071,06 | € 84.079,59 | € 92.487,55 |
| STAIR | 6.466,29 | 1,00 | 0,60 | 0,40 | 0,44 | 0,10 | 0,10 | | 112% | 110% |
| | | € 6.466,29 | € 3.879,77 | € 2.586,52 | € 2.845,17 | € 646,63 | € 646,63 | € 17.071,01 | € 19.119,53 | € 21.031,48 |
| ANCHOR BOLTS, BRACINGS, PLATES AND BOLTS | 15.036,89 | 1,56 | 1,00 | 0,28 | 0,44 | 0,10 | 0,10 | | 112% | 110% |
| | | € 23.457,55 | € 15.036,89 | € 4.210,33 | € 6.616,23 | € 1.503,69 | € 1.503,69 | € 52.328,38 | € 58.607,78 | € 64.468,56 |
| HEADED STUDS "NELSON" TYPE | 700 | 2 | 0,28 | 0 | 10 | 0,3 | 1 | 0 | 112% | 110% |
| | | € 1.400,00 | € 196,00 | € 0,00 | € 7.000,00 | € 210,00 | € 700,00 | € 9.506,00 | € 10.646,72 | € 11.711,39 |
| RAILING | 3.500,00 | 1,20 | 1,00 | 0,80 | 1,00 | 0,60 | 0,50 | 0 | 112% | 110% |
| | | € 4.200,00 | € 3.500,00 | € 2.800,00 | € 3.500,00 | € 2.100,00 | € 1.750,00 | € 17.850,00 | € 19.992,00 | € 21.991,20 |
| TREAD | 3.900,00 | 1,5 | 0,5 | 0,5 | 0,5 | 0,1 | 0,15 | 0 | 112% | 110% |
| | | € 5.850,00 | € 1.950,00 | € 1.950,00 | € 1.950,00 | € 390,00 | € 585,00 | € 12.675,00 | € 14.196,00 | € 15.615,60 |
| CORRUGATED SHEET 8/10 | 28.300,00 | 1,05 | | | 0,5 | 0,1 | 0,15 | 0 | 112% | 110% |
| | | € 29.715,00 | € 0,00 | € 0,00 | € 14.150,00 | € 2.830,00 | € 4.245,00 | € 50.940,00 | € 57.052,80 | € 62.758,08 |

| TOTAL WEIGHT | 159.286,89 |
|---|---|

| TOTAL COSTS | € 488.576,48 |
|---|---|

**Final result from cost analysis of steel structures is of 3,07 €/kg.**

**Table 12.** Total weight and cost analysis of SRCW system

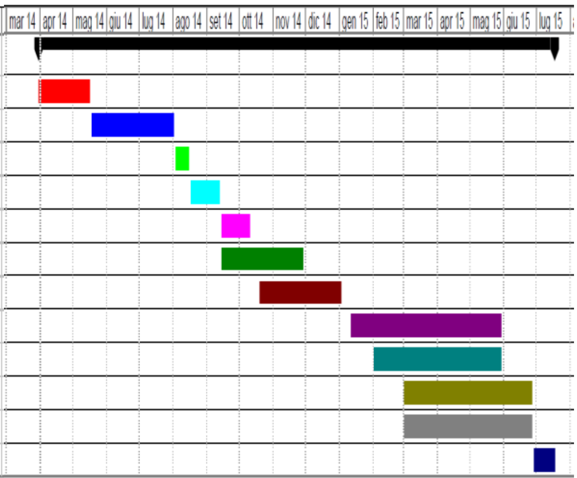
**Table 13.** Cost analysis of the HCSW and SRCW systems

**RFSR-CT-2010-00025 - Proposal RFS-PR-09003**
**Project: INNO HYCO - INNOvative and COmposite steel-concretestructural solutions for buildings in seismic area**

| | 1st CASE - Distinta D01 - SRCW System | | 2nd CASE - Distinta D01 - HCSW System | |
|---|---|---|---|---|
| Excavation | € | 35.000,00 | € | 35.000,00 |
| Foundation (piling, retaining concrete wall, wall footing) | € | 163.000,00 | € | 163.000,00 |
| Frame in elevation (in concrete) for system SRCW and HCSW | € | 76.000,00 | € | 44.000,00 |
| 1st floor | € | 38.500,00 | € | 38.500,00 |
| Concrete slab at the basement floor | € | 34.000,00 | € | 34.000,00 |
| Casting of concrete on corrugated sheets from the 2nd to the 7th floor | € | 42.000,00 | € | 42.000,00 |
| Ramps of garage | € | 22.000,00 | € | 22.000,00 |
| Two ramps of stairs (in the basement) | € | 6.500,00 | € | 6.500,00 |
| Steel structures | € | 488.576,48 | € | 542.182,78 |
| **TOTAL** | **€** | **905.576,48** | **€** | **927.182,78** |

**Table 14.** Construction work planning

| Nome attività - Activity name | duration | Start | End |
|---|---|---|---|
| **09C010(r0)[PROJECT INNO-HYCO RFSR-CT-2010-00025]** | **17 mes** | **lun 31/03/14** | **ven 17/07/15** |
| Excavations and foundations retaining walls | 35 g | lun 31/03/14 | ven 16/05/14 |
| Erection of steel structures | 55 g | lun 19/05/14 | ven 01/08/14 |
| Erection of the 1° floor and concrete casting | 10 g | lun 04/08/14 | ven 15/08/14 |
| Erection from 2° to 7° floorplates and concrete casting | 20 g | lun 18/08/14 | ven 12/09/14 |
| Roof covering | 20 g | lun 15/09/14 | ven 10/10/14 |
| External wall cladding | 55 g | lun 15/09/14 | ven 28/11/14 |
| Internal wall cladding | 55 g | lun 20/10/14 | ven 02/01/15 |
| Wiring | 100 g | lun 12/01/15 | ven 29/05/15 |
| Mechanical and thermal plants | 85 g | lun 02/02/15 | ven 29/05/15 |
| Interior flooring, coating and infixes | 85 g | lun 02/03/15 | ven 26/06/15 |
| External arrangements, drainage system and paving | 85 g | lun 02/03/15 | ven 26/06/15 |
| Any other unforeseen | 15 g | lun 29/06/15 | ven 17/07/15 |



## 10. List of figures

## 11. List of tables

## 12. List of acronyms and abbreviations

| | |
|---|---|
| **CPR** | Consorzio Pisa Ricerche (beneficiary) |
| **DBD** | Displacement based design |
| **DSD** | Dezi Steel Design Srl (subcontractor) |
| **FBD** | Force based design |
| **HCSW** | Hybrid coupled shear walls |
| **IDA** | Incremental dynamic analysis |
| **OCAM** | OCAM srl (beneficiary) |
| **PBD** | Performance based design |
| **RWTH** | Rheinisch-Westfälische Technische Hochschule Aachen (beneficiary) |
| **SHE** | Shelter S.A. (beneficiary) |
| **SRCW** | Steel frames with reinforced concrete infill walls |
| **ULG** | University of Liège (beneficiary) |
| **UniCam** | University of Camerino (coordinator) |
| **UniTh** | University of Thessaly – Research Committee (subcontractor) |

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KI-NA-26932-EN-N

The INNO-HYCO research project aims at defining innovative steel and reinforced concrete hybrid systems for construction of feasible and easy repairable buildings in seismic zones, able to fully exploit the stiffness of concrete components and the ductility, dissipation capacity, and replaceability of steel elements. Hybrid coupled shear walls (HCSW) and steel frames with reinforced concrete infill walls (SRCW) are considered.

An innovative HCSW system is obtained through the connection of a reinforced concrete wall to side steel columns by means of replaceable steel links acting as dissipative elements. A capacity design procedure based on limit analysis is developed in order to enforce a suitable dissipative mechanism while limiting the wall damage.

An innovative SRCW system is derived as an evolution of the system considered in Eurocode 8 through the adoption of solutions capable to overcome its drawbacks as observed in preliminary analyses. The infill walls are connected only at their corners to bearing plates to foster the formation of diagonal concrete struts. The dissipative elements are replaceable portions of the side columns that are not connected to the infill walls. A specific capacity design is developed to avoid the wall crushing.

Experimental investigations on the two systems are used to validate the theoretical assumptions and to derive specific nonlinear models for the assessment of the designs based on the proposed procedures. Finally, the two innovative hybrid systems are used in the design of a realistic case study consisting in a 6-storey building in a medium-high seismic area.

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ISBN 978-92-79-44080-9
doi:10.2777/85404