G. Manfredi,
M. Dolce
editors

This book presents the main results of the largest research program on earthquake engineering ever held in Italy. The ReLUIS project funded by the Department of the National Civil Protection granted 15.000.000 € and involved more than 600 researchers all over the country between 2005-2008. The ten research tasks range from the seismic risk of existing structures to new design paradigms, include geotechnical earthquake engineering issues and innovative approaches to seismic risk reduction as earthquake early warning systems as well as emergency management. Therefore, the papers herein contained are likely to express the state of the art of earthquake engineering in Italy which as acknowledged by the new Italian seismic code.

The pictures on the front cover are all taken from effects of the recent April 2009 L'Aquila earthquake, each photo recalls the core topic of a research task of the ReLUIS project. Descending from the top left corner:

- The masonry structure of the local government building in L'Aquila;
- A soft storey in a reinforced concrete building in Pianola (AQ);
- A bridge collapsed in Paganica (AQ);
- The displacement spectra of the unrotated ground-motion recorded during the mainshock in L'Aquila;
- Damages to some steel structures in Bazzano (AQ);
- Geotechnical effects in Sinizzo (AQ);
- A detail of the isolating systems for the post-earthquake housing project (C.A.S.E.);
- Strengthening of a school building in L'Aquila by means of fiber-reinforced plastics (FRP);
- The structural monitoring of the post-earthquake headquarter of the Civil Protection in L'Aquila and site of the 2009 G8 event;
- Tents for post-earthquake homeless and typical building stock in L'Aquila.

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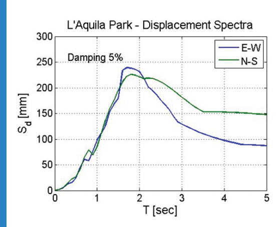
The state of Earthquake Engineering Research in Italy:
the ReLUIS-DPC 2005-2008 Project





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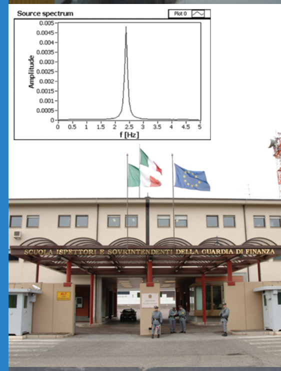

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PREFACE

The ReLUIS (www.reluis.it) consortium is the network of the Earthquake Engineering University Labs (Rete dei Laboratori Universitari di Ingegneria Sismica) and is a center of competency for the National Department of Civil Protection providing technical and scientific support also during emergencies as happened in the recent April 6 2009 L'Aquila earthquake.

ReLUIS enhances synergy among large test infrastructures in Italy ensuring a competitive high quality research offer at international level. Moreover, ReLUIS coordinates earthquake engineering research in Italy carrying out R&D for companies, supporting building codes development, and providing tools and education for practitioners and governmental agencies.

This book presents the main results of the largest research program on earthquake engineering ever held in Italy. The ReLUIS project funded by the Department of the National Civil Protection granted 15.000.000 € and involved more than 600 researchers all over the country between 2005-2008.

The ten research tasks range from the seismic risk of existing structures to new design paradigms, include geotechnical earthquake engineering issues and innovative approaches to seismic risk reduction as earthquake early warning systems as well as emergency management. Therefore, the papers herein contained are likely to express the state of the art of earthquake engineering in Italy which as acknowledged by the new Italian seismic code.

The emerging research needs after the L'Aquila earthquake are also presented, they are the basis of the next ReLUIS project which is about to start.

The successful ReLUIS activity, representing a landmark for earthquake engineering in the world, is mainly due to the continuous relationship with the Civil Protection and its staff. To this regard it is to mention the long view of Guido Bertolaso and Vincenzo Spaziante who aimed at the born of ReLUIS and followed its development along with the members of the Coordinating Board.

A final acknowledgment is given to all who contributed to the ReLUIS research assets and in particular Luigi Di Sarno, Marco Di Ludovico and Iunio Iervolino who supported the coordination of the project with competency and dedication.

Napoli, July 28 2009.

Mauro Dolce and Gaetano Manfredi

THE RELUIS CONSORTIUM

The Network of University Laboratories in Earthquake Engineering (in Italian, ***Re**te dei **L**aboratori **U**niversitari di **I**ngegneria **S**ismica*, ie. ReLUIS) is a consortium comprising a number of leading universities, located in strategic positions throughout the Italian Peninsula. The consortium was established in 2003 and it is aimed at managing all the research work carried out in the laboratories of the affiliated universities. In so doing, it provides technical, managerial and financial support to the university centres of ReLUIS and promotes the advances and innovative materials and technologies in the field of earthquake engineering.
The consortium ReLUIS has many similarities with other earthquake engineering networks established in recent years word-wide, such as the George E. Brown Jr. Network for Earthquake Engineering Simulation (NEES) in the USA and the Asian-Pacific Network of Center of Earthquake Engineering Research (ANCER). Its target is to provide a robust and dynamic platform for the exchange of the know-how and technical background between universities and research centres, on one hand, and governmental institutions, public and private research establishments and stakeholders, on the other hand, to minimize the seismic risk of the built environment.
The consortium is headquartered at the University of Naples, Federico II, Department of Structural Engineering (DSE). Its structure includes a board of directors and a chairman; the present chair is professor Gaetano Manfredi, head of the DSE; past chairs were professor Edoardo Cosenza, Dean of the Faculty of Engineering of the University of Naples, Federico II and professor Mauro Dolce, Director of the Department of Civil Protection, Seismic Risk.
The founding and partner Italian universities of the ReLUIS consortium are as follows (Figure 1):

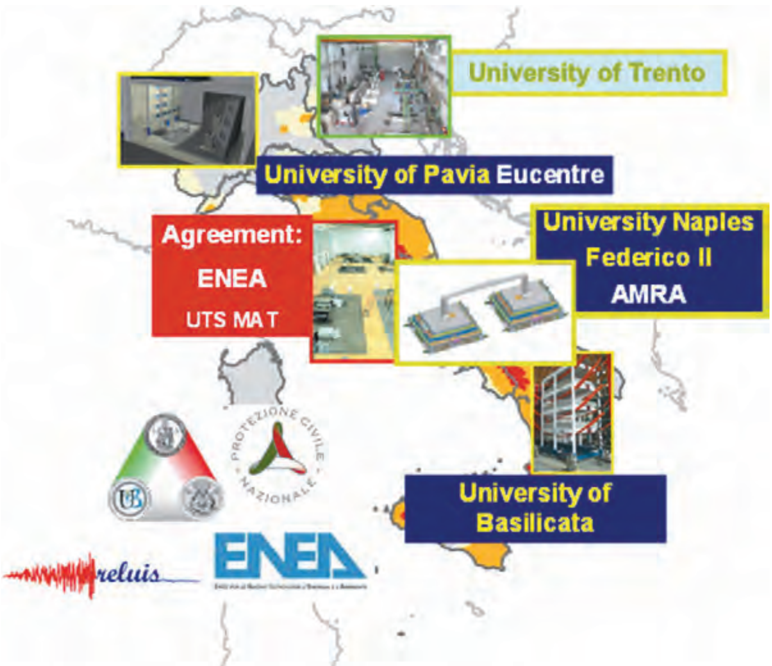

Figure 1. Founding and partner universities of the ReLUIS Consortium.

- *University of Basilicata at Potenza*, Department of Structures, Geotechnics and Geology applied to the Engineering;
- *University of Naples*, Federico II, Department of Structural Engineering;
- *University of Pavia*, Department of Structural Mechanics;
- *University of Trento*, Department of Structural Mechanics.

A formal agreement for collaborative research was also signed with the research centre ENEA UTS Materials and New Technologies, based at Casaccia, Rome.
The partner universities and ENEA possess large experimental facilities, as also shown pictorially in Figure 1, to perform full-scale static, pseudo-static, dynamic and earthquake response analyses. The ReLUIS consortium tends to optimize the features of such facilities, which possess different and complementary technical properties. The technical features of the laboratory large facilities are as follows:

- *University of Basilicata at Potenza*: The DeSGG Lab (Laboratory of the Department of Structures, Geotechnics and Geology applied to the Engineering) features a number of electro-hydraulic testing machines; their maximum capacity ranges between 1000kN (elastic testing) and 3000kN (compression tests). A large (6mx10m) reaction wall (Figure 2) is connected to a 16m long strong floor and may be employed to perform static and pseudo-dynamic testing. Hydraulic jacks with a load capacity ranging between 10kN and 640kN are used for tests on components and structures, e.g. large scale isolation devices. The laboratory hosts two shaking tables: the smaller table is used for scaled specimens (up to 10kN), the larger is used to simulate high magnitude earthquakes for specimens (up to 100kN).

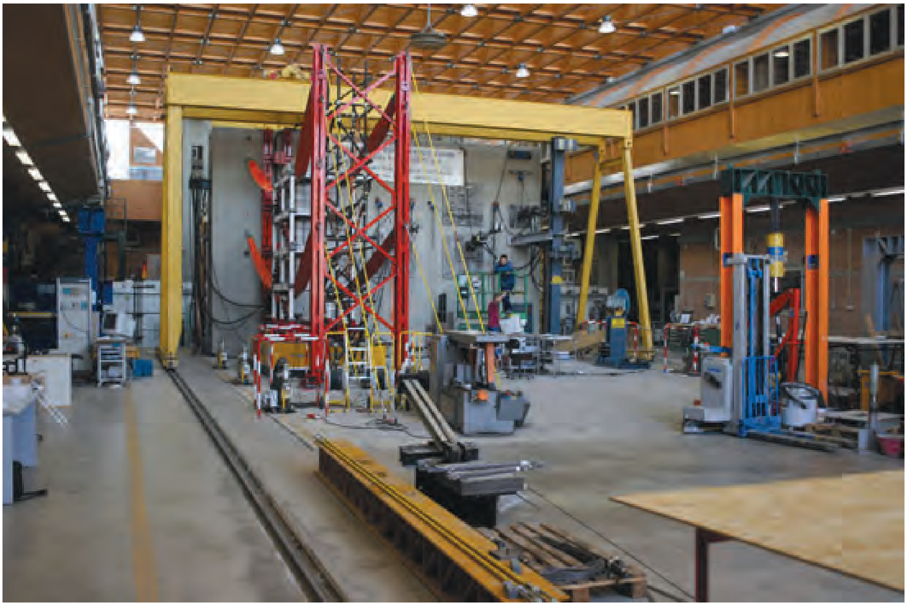

Figure 2. Prospective view of facilities featured by the DeSGG Lab, Potenza.

- *University of Naples*: The DSE Lab (Laboratory of the Department of Structural Engineering) has a number of testing machines to perform static, cyclic, pseudo-dynamic, dynamic and earthquake tests on members, joints, sub-assemblages and full-scale structures. Tests on 4.1m high columns with maximum axial loads of 3000kN may be carried out with and without lateral forces. The shaking table systems (Figure 3) consist of two two-single degree-of-freedom 3.0mx3.0m tables. The maximum payload is 20tons per table; the total load of each table is 63 tons as shown in the breakdown of the components in Figure 3. The maximum acceleration is 1.0g with a specimen of 20 tons; the peak velocity is 1m/sec and the maximum allowable displacement is ± 250mm per axis. The tables can be used to perform dynamic tests in the range 0 to 50 Hz and to simulate asynchronous seismic motion on large infrastructures.

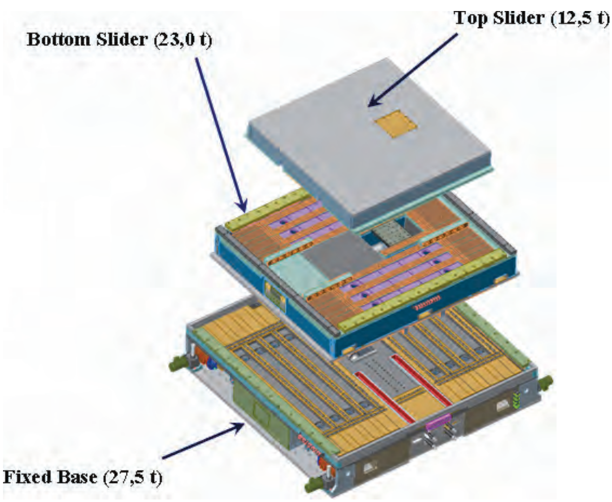

Figure 3. Aerial view (*up*) and basic components (*down*) of the asynchronous shaking tables of the DSE Lab, Naples.

- *University of Pavia*: The TREES Lab (Laboratory for Training and Research in Earthquake Engineering and Seismology), headquartered at EUCENTRE, features three major experimental facilities (Figure 4), i.e. the strong floor-reaction wall system for full scale pseudo-static and pseudo-dynamic tests; the high performance uniaxial shaking table and the biaxial bearing test system. The North reaction wall is 14.40m long, 12.0m high and 2.40m thick, the west wall is 9.0m long, 12.0m high and 2.40m thick. The test floor is a reinforced concrete box girder 14.40 m long, 12.0 m wide. The thickness of the top test floor slab is 2.40.m. The bearing testing system is one of the few large testing machine in the world and may be used to perform biaxial tests on full-scale bearings and isolation devices subjected to either static or dynamic loads. It has a table platform size of 1.6mx4.4m a total tale mass of 18.0 tons; the horizontal and vertical strokes are ±400mm and ±60mm, respectively. The maximum acceleration is ±1.8g, the maximum velocity is 1400mm/sec (horizontally) and 250 mm/sec (vertically) the operating frequency is 0.20Hz.

Figure 4. Reaction wall (*left*) and bearing tester system (*right*) of the TREES Lab, Eucentre, Pavia.

- *University of Trento:* The MST Lab (Laboratory of the Materials and Structural Testing) features a bi-directional reaction wall, consisting of a 9.5 m tall pre-stressed concrete wall, and the 42 m long reaction strong floor. The wall and the strong floor are both endowed of regular pattern holes for a fast and effective connection of structures and loading devices (Figure 5). The wall and the floor are designed to resist forces of several MN, which are necessary to deform and seriously damage the full-scale test models of structures. They are equipped of an high-pressure hydraulic distribution system. Two 10 tons bridge-cranes permit the movement and positioning of test structures. Static and cyclic tests may be carried out on large full-scale components, sub-assemblages and structures. Furthermore, the facility may be utilised to perform pseudo-dynamic tests on full scale building structures. Oleodynamic (1000kN) and mechanical (100kN) universal testing machines, compression test rig, dynamic jacks and reaction frames to resist loads up to 2000kN are also utilized in the MST Lab.

Figure 5. Aerial view (*left*) and large reaction wall (*right*) of the MST Lab, Trento.

- *ENEA*: The CRC Lab (Casaccia Research Centre Laboratory) headquartered at ENEA, Rome, features two six-degree-of-freedom shaking tables (Figure 6) that may be utilised for tri-axial testing of structures with maximum 10 tons masses and a peak acceleration of 3g, applied at the gravity centre located at 1m above the ground. The table dimensions are 2m x 2m (small table) and 4mx4m (large table), respectively. The small table can be employed to perform tests in the frequency range 0 to 100Hz, with a peak acceleration of 5g, maximum velocity of 1m/sec and maximum lateral displacement of 300mm. The large table can be used for dynamic tests with frequencies in the range 0 to 50Hz, with peak acceleration of 3g, maximum velocity of 0.5 m/sec and maximum lateral displacement of 250mm.

Figure 6. Aerial view of the shaking tables at CRC Lab, Rome.

The ReLUIS consortium is also extremely active in continuing education, outreach and provides a sound and interactive support to researchers, students, designers and practitioners in the interdisciplinary field of earthquake engineering. Databases of experimental and

numerical tests, advanced software and technical papers are shared freely on the official website of the consortium (www.reluis.it). Additionally, handbooks for seismic analysis and design, guidelines and recommendations are also published periodically with the financial support of the consortium.
The members of the consortium are high-profile earthquake engineering researchers that participate to challenging national and international advanced projects, to seismic risk analysis and mitigation, to code drafting and to post-earthquake emergency management, as per example in the recent 6^{th} April L'Aquila (Italy) earthquake.
The ReLUIS consortium applies for fund raising both locally and nationwide. It was awarded in 2005 by the Italian Department of Civil Protection for a three-year project research, as further illustrated in the next section.

THE 2005-2008 RELUIS-DCP RESEARCH PROJECT

The ReLUIS consortium was awarded in 2005 by the Department of Civil Protection (DCP) with a three-year 15million euros (5 million euros per annum) research project; it was proposed than more than 50% of the total budget was allocated to perform experimental tests on components, sub-assemblages and structures subjected to either static or dynamic monotonic and/or cyclic loading. The project has involved a large of universities and research centres nationwide, as shown pictorially in Figure 7. In particular, the total number of universities involved in the project is 40, with 127 research groups (research unit) and more than 800 researchers. The majority of the researchers were PhD students and young research assistants. Special emphasis was placed on the experimental investigations, with inter-and intra-research unit cooperation to achieve effective sharing of data and results.
The 2005-2008 ReLUIS-DCP research project encompasses two thrust areas:

- *Risk Mitigation*: it is aimed at targeting any type of structural system and configuration (e.g. ordinary buildings, critical facilities, monumental buildings, lifelines, etc.) and it addresses any party involved and/or interested in earthquake risk management (e.g. researchers, designers, policy makers, etc.);
- *Engineering tools for risk prevention:* it is aimed at providing updated seismic provisions (highly efficient but not redundant, easy for use in the routine seismic design and/or assessment of structures), detailed guidelines, comprehensive handbook chiefly based on sound experimental test results and several numerical applications, continuing education and professional training, forecasting and monitoring tools for pre-earthquake scenarios (e.g risk maps, scenarios, early warning procedures), post-earthquake intervention techniques (e.g. for surveying, decision making, scheduling and optimizing rescue operations and planning of provisional interventions).

The above thrust areas include 10 research topics that can be grouped as follows:

<u>Risk mitigation</u>

1. *Evaluation and reduction of the vulnerability of existing masonry buildings* (PIs: professors S. Lagomarsino and G. Magenes);
2. *Evaluation and reduction of the vulnerability of the existing RC buildings* (PIs: professors E. Cosenza and G. Monti);
3. *Evaluation and reduction of the vulnerability of the existing bridges* (PIs: professors G. Mancini and P.E. Pinto);

4. *Development of displacement-based design approach for the design and evaluation of the vulnerability* (PIs: professors G.M. Calvi and M.J.N. Priestley);
5. *Development of innovative approaches for the design of steel and composite steel and concrete structures* (PIs: professors F.M. Mazzolani and R. Zandonini);
6. *Innovative methods for the design of geotechnical structures and to evaluate the stability of slopes* (PIs: professors A. Burghignoli, M. Jamiolkowski, G. Ricceri and C. Viggiani);

<u>Technological innovation in seismic engineering</u>

7. *Technologies for the seismic isolation and the control of the structures and infrastructures* (PIs: professors M. Dolce and G. Serino);
8. *Innovative materials for the reduction of the vulnerability in the existing structures* (PIs: professors G. Manfredi and L. Ascione).

Figure 7. University and research centres participating at the 2005-2008 ReLUIS-DPC research project.

<u>Engineering tools for risk prevention</u>

9. *Monitoring and early warning of structures and infra-structures* (PI: professor P. Gasparini);
10. *Definition and development of databases for the evaluation of the risk and post-event scenarios* (PI: professor D. Liberatore)

During the 2005-2008 ReLUIS-DCP research project, the PIs and all the members of the Research Units provided continuing technical support to the DPC:

- To apply the Seismic Code Document 3274 and 3431 and for the vulnerability assessment and mitigation;
- To draft the new Seismic Italian Code (Testo Unico delle Costruzioni) issued in 2008, along with the Commentary;
- To develop and revise the CNR DT200 Guidelines and other Recommendations;
- To develop and disseminate the Guidelines for the assessment and retrofit of cultural, historical and monumental heritage, as in Seismic Code Document 3274;
- To promote the activities of the working group UNI EC8.

A number of workshops (kick-off, mid-term and final) were sponsored by the ReLUIS Consortium during the 2005-2008 Project:

RELuis Workshops

- 15th November 2005, Naples: Kick-off Workshop of the 2005-2008 ReLUIS-DCP Project;
- 22nd-23rd November 2006, Udine. The applied research in Italy. Results of the 1st Year of the 2005-2008 ReLUIS-DCP Project;
- 17th-18th January 2008, Florence. The applied research in Italy. Results of the 2nd Year of the 2005-2008 ReLUIS-DCP Project;
- 1st-3rd April 2009, Naples: Final Workshop of the 2005-2008 ReLUIS-DCP Project.

RELuis Sponsored Workshops

- 2nd February 2006, Naples: *The use of accelerograms in the nonlinear seismic analysis of structures*;
- 12th-13th June 2006, Capri: *The definition of seismic input for the damage scenarios and for the design*;
- 12th-13th Febrary, 2007: *Innovative Materials and Approaches for the Design in Seismic Areas and for the Mitigation of the Vulnerability of the Structures*;
- 27th May 2008, Campobasso: *Operational Modal Analysis of the structures: dynamic identification in the operational conditions*;
- 29th-30th May, 2008, Rome: *Evaluation and reduction of the seismic vulnerability of RC existing buildings*;
- 4th-5th December, 2008, Naples: *Technologies for the seismic isolation and the control of structures and infrastructures*.

Technical reports were issued yearly and were reviewed by a panel of international experts. These experts include: Professor M.N. Fardis (University of Patras, Greece), Professor R. Leon (GeorgiaTech, USA), Dr. A. Pecker (Geodynamique & Structures, France), Professor R. Spence (University of Cambridge, UK) and Professor M. Tomazevic (National Building and Civil Engineering Institute, Slovenia).

The deliverables of the 2005-2008 ReLUIS-DCP research project include guidelines and recommendations for the next generation seismic codes of practice, detailed design handbooks, advanced software for designers and practitioners. For example, the following software was implemented and disseminated as part of the deliverables of the project:

- **REXEL (release 2.31)**: it provides the results of the search of the combinations of accelerograms which are compatible with the acceleration response spectra compliant with the new Italian seismic code (NCT 08) and Eurocode 8. The accelerograms are selected for given source characteristics, e.g. magnitude and epicentral distance.

- **BIAXIAL:** it provides the ultimate bending moments, the moment-curvature and the Mx-My interaction domain for a generic RC cross-sections subjected to either uni-axial or bi-axial bending. Different types of confinement may be accommodated in the evaluation of the flexural properties of the RC cross-sections.
- **VC (RC Vulnerability)** and **VM (Masonry Vulnerability):** the software estimates, employing a simplified method, the seismic vulnerability and the seismic risk of single RC (VC) and masonry (VM) building structures.

Additionally, databases of the numerous experimental tests carried out in different laboratories during the thee years, were compiled and are available on the ReLUIS official website (www.reluis.it).
Finally, the ReLUIS consortium was extremely active in promoting and disseminating the effects of the seismic risk in the social and technical communities. In particular, exhibitions focusing on earthquake engineering were supported nationwide. Among them, the exhibition named “Italian Earthquakes” (Terremoti d’Italia) was very popular and successfully; its target was the dissemination the basic principles of the seismic risk among students. The exhibition was hosted by several cities from North to the South of Italy (Figure 8):

- *Foligno*: 27th September-18th November 2007
- *Ancona*: 5th December 2007-11th January 2008
- *Gibellina*: 31st January-15th March 2008
- *Rome*: 28th March-27th April 2008
- *Messina*: 27th December 2008-8th February 2009
- *Naples*: 6th March-4th April 2009
- *Codroipo*: 21st April-2nd June 2009

Figure 8. Brochures of the exhibition “Italian Earthquakes”.

A professional journal (Progettazione Sismica, Seismic Design) dealing with the main earthquake engineering topics was also founded and launched during the 2005-2008 ReLUIS-DPC Project; the Editor is Professor Gian Michele Calvi (EUCENTRE and University of

Pavia) and it is published by IUSSPRESS, Pavia. Progettazione Sismica (Figure 9) is aimed at providing to the academic community and professionals concrete design experiences and sound applications of seismic codes to improve the knowledge and expertise in Earthquake Engineering.

Figure 9. The new professional journal "Progettazione Sismica" (Seismic Design) sponsored by ReLUIS Consortium.

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EVALUATION AND REDUCTION OF THE VULNERABILITY OF MASONRY BUILDINGS

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1 INTRODUCTION

Line 1 of the ReLUIS 2005-2008 Framework Project consisted of a coordinated research on the evaluation and reduction of seismic vulnerability of existing masonry buildings. The research, which involved nineteen different research units, has developed along five main topics: the assessment and strengthening of structural units within building aggregates, the methods for assessment of mixed masonry-reinforced concrete structures, the strategies and techniques for strengthening of masonry buildings, considering both horizontal (floors, roofs, vaults) and vertical (walls) structural elements, the methodologies for modelling the seismic response of masonry structural systems. Within each topic several subtasks were considered, involving surveying, modelling, in-situ and laboratory testing. Within the project also some large scale static and dynamic testing was carried out, involving shake table tests on building models and structural components. In this paper the developments and outcomes of the project are synthesized.
Masonry buildings, whether ordinary or monumental, still constitute one of the most vulnerable classes of structures. Despite the numerous studies that were carried out in recent decades, still almost countless open problems exist regarding assessment methods, strengthening strategies and techniques, availability of adequate design-assessment-strengthening standards and codes of practice. The revision of OPCM 3274 (2003) published as OPCM 3431 (2005) contained new guidelines for the seismic safety assessment of existing masonry buildings which are quite advanced and up to date with the most recent developments in the field. Nevertheless, several of the methodologies that were proposed therein need further verification, especially when considering the variety of geometric and technological configurations that can be found in the existing and historical structures.
An important issue that has been introduced in OPCM 3431, possibly for the first time in a normative document, is the approach to the assessment of buildings in aggregates, consisting of a few principles and suggestions for the survey, identification of the structural unit, structural analysis. In this case the guidelines for the methods of analysis and assessment are anyways quite summary, and deeper studies are needed for the validation/correction of the proposed methodologies, taking into consideration the possible interaction of adjacent buildings.
Very few research results and a lack of established criteria are reported regarding the seismic assessment of mixed structural systems (such as masonry and reinforced concrete), that in different forms are very common in the existing building stock: confined masonry buildings, as made of masonry walls framed with RC elements (slightly reinforced beams, both horizontal and vertical), buildings with peripheral masonry walls and internal RC beam-

column systems, masonry buildings stiffened with RC walls or cores (especially corresponding to staircases, elevators or service areas), masonry buildings subjected to raising-up by mean of RC frames. For most of these situations there is the need to define reliable methods of analysis and assessment, especially regarding the applicability of linear methods.
Finally, regarding the seismic strengthening criteria and techniques, it is evident how still nowadays there are several important issues on which there is neither a consolidated scientific knowledge neither a general consensus of opinion. The recent experience of the Umbria-Marche earthquake has contributed to bring into question some strengthening techniques that were proposed after the Friuli and Irpinia earthquakes and had had widespread applications throughout the national territory. Particularly relevant are the following issues:

- role of horizontal floor or roof diaphragms (rigid/flexible), of the connections between walls and floors and effects of the interventions that modify diaphragms and connections;
- effectiveness of ring beams added to the existing structure and of their connections with floors and walls, depending on the structural context in which they are inserted;
- necessity and effectiveness of interventions aiming to improve the strength and deformation capacity of the masonry material;
- strengthening strategies to be adopted in building aggregates.

2 BACKGROUND AND MOTIVATION

Four principal themes had been identified on which the research project would focus: building aggregates, mixed buildings in masonry and reinforced concrete, criteria and techniques for strengthening of masonry buildings, seismic analysis and assessment methods. Obviously these aforementioned themes are strictly interconnected and the distinction was made purely with the scope of creating a clearer conceptual framework for the various research activities.

2.1 Building aggregates

The analysis of the conditions of the context in which the building is inserted is a fundamental issue for the assessment and reduction of the seismic vulnerability of masonry buildings. There exist indeed several factors conditioning their performance, depending on the interference between the single structural unit and those adjacent to it. Particularly important are the types of damage that may occur for buildings included in aggregates, as well as the criteria and techniques of intervention. The current project recognised the fact that clear and reliable methodologies and analytical tools for the analysis of seismic vulnerability of masonry building aggregates are not available and seismic codes provide scarce or insufficient information. On one hand, the issue of the seismic assessment of local mechanisms can be similar as for isolated building, as regards the fundamental modelling approaches to the problem: definition of the seismic input, evaluation of the demand (in terms of forces/acceleration or displacements), evaluation of the capacity. Nevertheless, specific mechanisms can take place within aggregates which are induced by the interconnection or contact by adjacent structural subsystems As regards the issue of the assessment of the so called "global" response, within an aggregate, the distinction of structurally "independent" units is problematic if not impossible, and any structural analysis that aims at evaluating the "global" response should in principle either model the whole block or model the structural unit with suitable boundary conditions that take into account the effect of the adjacent ones. Such an approach is clearly problematic even resorting to the most updated analytical tools, and beyond the possibility of average practicing engineers. Any method of structural analysis which considers the structural unit as independent is therefore clearly conventional. The possibility of using conventional simplified analyses should be foreseen. Suitable simplified criteria should be validated and made available to professionals, such as evaluation based on nonlinear storey-mechanism models and/or analysis of suitable structural subsystems.

2.2 Mixed masonry and reinforced concrete buildings

From the beginning of the 19^{th} Century, the advancement of reinforced concrete technology has been spreading in Italy, with a substantial transformation of the theory and methods of construction, both in terms of architectonic expression as well as construction practice. Unlike other countries, the Italian scenario, especially between the two world wars, is characterised by the specific use of reinforced concrete technology combined with load-bearing masonry frame. Certain specific technical characteristics are seen in Italy: in areas with high seismic hazard, the building norms, issued with the Regi Decreti after 1909 (i.e. after the Messina earthquake), suggest the adoption of mixed masonry-r.c. solutions, with different configurations. During the reconstruction after the second world war, within a decade, in the 1950s, the urban landscape was remodelled with the presence of r.c. buildings , where the use of load-bearing masonry progressively gave way to the use of lightweight infills. An evolution, characterised by a higher heterogeneity in the building solutions was noticed: from structures in r.c., in which the masonry is inserted as an infill, to mixed structures where horizontal and vertical r.c. elements are in contact with masonry, to structures in which the vertical elements are simultaneously composed of masonry walls, frames and r.c. walls, isolated or in contact with the masonry, to structures with floors composed of different technologies (low floors in masonry and upper floors in r.c.), to masonry structures with bracing r.c. elements. The variety of mixed structures is rendered even more complex because of the interventions on existing buildings, especially due to extensions in height and plan.

Despite the spread of this typology in our country, the scientific research on mixed structures is extremely limited and the research terrain is still largely unexplored. This is attributable first of all to the already discussed variability in the typology and to the fact that this construction typology is not completely well known and standardised. Code prescriptions and, in particular, sufficiently robust analysis and assessment methodologies and criteria to be used in practice are scarce or absent. Recent codes, such as EC8 or Annex 2 of the OPCM 3274 provide very scarce operative indications to the designer, mostly limited to general principles.

The international scientific literature on the seismic behaviour of structural masonry with isolated r.c. columns and masonry coupled with r.c. walls is extremely poor: in particular there is almost a total absence of experimental information with regard to the latter, especially for structural configurations which are irregular in plan.

Even mixed structures composed of floors with different technologies have received very low attention from the scientific community, since this structural typology presents in most cases several drawbacks for newly designed structures. Nevertheless the high number of existing buildings to which storeys have been added in different phases calls for research focus on this typology, in which the irregularity in elevation, introduced with extension in height, along with the increase in the masses can induce a higher seismic vulnerability.

A larger number of studies, including experimental, is available for mixed structures composed of masonry with r.c. members in contact, mainly with reference to the “confined” masonry technique, in which the function of the confining elements is to provide dissipative capacity and ductility. However the majority of literature studies is devoted to framed r.c. structures with masonry infills, in which the masonry is introduced primarily with the function of closing spaces and therefore non-structural, while the primary structure is in r.c.

In the past few years, a notable advancement of models for the non linear analysis of masonry structures and r.c. frames with masonry infills was observed. Several different nonlinear macro-models, which permit seismic nonlinear analysis of entire buildings with satisfactory results, have been developed. These macro-models with appropriate adaptations and improvements can be perceived as a starting point for the development of models for nonlinear analysis of mixed masonry-r.c. structures, mainly with the aim of investigating their seismic global behaviour, taking into account the different deformation capacities of the structural elements.

Even for mixed structures it seems possible to follow a logical process starting from a diagnostic approach based on several investigations (anamnesis, objective examination and experimental investigations) to identify the intrinsically weak structural situations and therefore, if necessary, to formulate the therapy aimed at removing the causes of damage, through appropriate strengthening and retrofit interventions.

The literature on strengthening interventions considers the masonry structure separately from the r.c. structure. Besides classical manuals and textbooks, the main contributions are derived from post-earthquake experiences, after the events that influenced the building construction scenario in post-war Italy. These contributions are specifically significant due to their strong relation with the national construction scene, a very relevant aspect from the point of view of research. Works related to criteria and techniques of seismic strengthening and retrofit specific to mixed structures are missing. Despite this, it is important to consider the aforementioned contributions for a better understanding of the masonry-r.c. interaction, in order to develop appropriate intervention techniques.

2.3 Seismic strengthening criteria and techniques for masonry buildings

The seismic events of the past years, in particular Umbria-Marche 1997, constituted an evaluation basis for the seismic retrofit criteria and techniques developed after the earthquakes of Friuli and Irpinia, and contributed to criticise some of these criteria and techniques. The failure of some techniques is most probably due to the inappropriate application of structural behaviour schemes, valid for new construction, to existing structures. Such schemes, when applied to existing structures, were neither technically feasible nor efficient or, in some cases, they could even be counterproductive or irrelevant from the structural point of view.

An example of this situation is represented by the interventions on floor and roof diaphragms: an element largely characterising the criteria of intervention in the last 20 years has been the stiffening of floor and roof diaphragms, sometimes obtained by completely replacing the existing structures.

While in a new building the construction of a well connected rigid floor (typically through r.c. tie beams) does not present particular problems during the execution and it would offer a number of structural advantages (basically a higher redundancy and robustness of the building, smaller number of out-of-plane failures, simpler modelling), in existing masonry buildings, an intervention aiming at introducing stiff diaphragms could produce negative effects, due to the following reasons:

- if the building is irregular in plan, in-plane stiffening of the diaphragm could lead to an unfavourable redistribution of the seismic forces between the walls and an amplification of the torsion effects, with a higher deformation demand on some perimeter walls;
- the diaphragm stiffening can lead to a higher demand on the wall to floor connections; the connection can then become a critical element: as an example, the inefficiency of

partially embedded tie beams in double-leaf stone masonry walls is recalled, as observed in the Umbria-Marche event;
- a significant increase in the floor masses, subsequent to the introduction of r.c. slabs, results in an increment of the seismic demand, which could lead to the failure of a low-quality masonry.

Typically, the coupling of elements with very different stiffness could result in concentration of damage in the vicinity of the connection, during seismic response. Therefore, recently, intervention methods, aimed at providing additional strength to the diaphragms without excessively increasing their stiffness, have been proposed. The emphasis now is hence on the need for diaphragms which activate the in-plane resistance of walls, avoiding first mode mechanisms, without requiring "infinite" stiffness. Limited, but significant, dynamic experiments have shown that masonry buildings sufficiently regular in plan, well consolidated and braced with tie rods, in which flexible wooden diaphragms have been retained, can have a seismic resistance completely comparable to buildings with diaphragms stiffened with r.c. slabs.
As mentioned before, the issue of connections is crucial in the floor strengthening interventions. Often floors and roofs are constituted of wooden elements, both in existing buildings (not necessarily historical) and in new constructions. Especially in Italy, all seismic events (even moderate intensities) have shown how vulnerable a building can be if even some simple requisites and construction details are not guaranteed. Problems related to the roofs, which are often not adequately connected to the masonry or which are thrusting either from the beginning or as a consequence of malfunctioning, or which have elements and internal connections that do not guarantee a ductile response (brittle failures are typical in wooden constructions, due to material properties), are frequently reported.
The response of the connections between timber elements and between floors/roofs and the wall structures has always been fundamental. Poorly designed or executed construction details often result in local or global failures as well, under a seismic event even of moderate intensity.
The problem of vaults, often associated to situations in which interventions are necessary within principles of conservation, should be mentioned, given the role of horizontal elements in seismic response. On the one hand, it seems necessary to reorganise the existing literature in order to allow a clearer view of the role and behaviour of vaults under seismic action, which is a fundamental requisite for appropriately designing any type of intervention. On the other hand, it is necessary to consider and experimentally validate new retrofit techniques using, other than the well known FRP (*Fiber Reinforced Polymer*), a new generation of composite materials. The latter are reinforced with high resistance steel wires and known as SRP (*Steel Reinforced Polymer*) and SRG (*Steel Reinforced Grout*). In addition to the traditional advantages typical of composite materials (ease in application, low invasiveness, reduced intervention time, etc.), the presence of steel as the principal reinforcing element enhances the performance and permits structural retrofit interventions comparable to those with FRP. Moreover, with respect to FRP, the cost is further contained and the retrofitted element can show an increased in ductility which is much higher than what could be obtained with glass or carbon fibres. In addition, the retrofit using SRG overcomes the problems of fire resistance typical of reinforcing materials with polymer matrices.

2.4 Analysis and assessment methods

The problem of seismic analysis and assessment of masonry buildings was considered in depth by the scientific community from the 1970s and in particular, in Italy, after the Friuli earthquake. The peculiarity of the problem becomes clear if one considers that, even in the 1970s, assessment procedures for masonry buildings based on static nonlinear analyses were introduced in the codes, even before this happened for other recurrent structural typologies. This highlights the incapability of classical methods of elastic analysis of describing adequately the seismic response of masonry buildings.

The appreciable development of nonlinear analysis methods for masonry, that occurred mainly in Italy, permitted the use of methods for the seismic global response of a building, dominated by the in-plane strength of the walls, at a limited computational cost. Such methods are based on the use of appropriate macro-elements, with typical dimensions comparable to those of openings or inter-storey height, and therefore are usually defined as "simplified". The validity of macro-elements models has been substantially demonstrated for sufficiently regular geometrical configurations. Nonetheless there are few issues and limits of validity in their application to complex configurations, which are very common in existing buildings.

An extensive comparison between different approaches for the nonlinear modelling of masonry walls subjected to in-plane forces, carried out within researches financed in the late 1990s by GNDT within the "Catania Project", had highlighted how the results of nonlinear analyses are sensitive to the modelling assumptions on the response of masonry spandrels. A complete lack of experimental references on the seismic response of unreinforced masonry spandrels, essential for the validation of analysis models based on macro-elements or more refined approaches, is noticed.

Another relevant issue is the role of diaphragm flexibility both for the global and the out-of-plane (local) response of the building (filtering effect of the structure on the seismic input imposed on the walls). Some foreign codes provide operative indications for modelling the seismic response of masonry buildings with flexible diaphragms, as well as some experimental and numerical literature works which however address only a few specific structural typologies and consider the elastic response. A reorganisation work and additional numerical and experimental studies are necessary to develop applicative methodologies for the structural analysis and assessment of buildings with flexible diaphragms.

As mentioned before, elastic analysis has a number of difficulties and limitations in the description of seismic response. For example, the distribution of internal forces in a masonry building at the ultimate limit is not governed by the stiffness of the elements in the elastic regime but, instead, by equilibrium and compatibility relationships between forces and the strength criteria. This would reduce the utility of multi-modal elastic response spectrum analysis, which provides non-equilibrated results, and favours the use of static nonlinear analysis methods, since nonlinear dynamic analysis is still not commonly used in practice. However, since practicing engineers still use linear analysis, it is urgent and necessary to clearly define the modalities and limits of use of linear analysis, with specific reference to masonry. In this case indeed a simple translation of concepts applicable to other typologies (e.g. r.c. structures) can lead to inadequate code-based solutions or concepts difficult to apply.

3 RESEARCH STRUCTURE

The research programme aimed, on one hand, to collect and put into implementation a rather large amount of research that has been carried out in Italy in recent years without systematic publication and interpretation (such as damage/technological surveys in historical centres, in-situ and laboratory testing on masonry materials…), on the other hand to carry out new experimental and theoretical research to investigate many of the open problems that have been discussed above. All of the activities had the main objective of producing results and documents that can be used as a support to the engineering practice, either in the form of guidelines, recommendations, databases or pre-normative documents.

The Research Units and the corresponding scientific responsibles participating into the project are the following:

1. UNIPV (University of Pavia), Responsible: Guido Magenes
2. UNITN (University of Trento), Responsible: Maurizio Piazza
3. UNINA (University of Naples, DAPS), Responsible: Nicola Augenti
4. UNIBAS (University of Basilicata, Potenza), Responsible: Domenico Liberatore
5. UNIGE (University of Genoa), Responsible: Sergio Lagomarsino
6. UNIPG (University of Perugia), Responsible: Antonio Borri
7. UNIPD (University of Padua), Responsible: Claudio Modena
8. POLIMI (Polytechnic of Milan), Responsible: Luigia Binda
9. ROMA3 (University of Rome 3), Responsible: Gianmarco De Felice
10. UNIFI (University of Florence), Responsible: Andrea Vignoli
11. UNIBS (University of Brescia), Responsible: Ezio Giuriani
12. POLIMI-b (Polytechnic of Milan), Responsible: Maria Adelaide Parisi
13. UNIPI (University of Pisa), Responsible: Mauro Sassu
14. UNICT (University of Catania), Responsible: Ivo Caliò
15. ROMA1 (University of Rome “La Sapienza”), Responsible: Luis Decanini
16. UNISR (University of Syracuse), Responsible: Caterina Carocci
17. UNIRC (University of Reggio Calabria), Responsible: Vittorio Ceradini
18. IUAV (Architecture University Institute of Venice), Responsible: Francesco Doglioni
19. UNINA-b (University of Naples, Architecture), Responsible: Claudia Casapulla

The project is articulated in five main tasks, each one divided in sub-tasks, as shown in Table 1 . Each task has a General Task Coordinator (GTC) and individual sub-task coordinators.

Table 1. Tasks and coordinators.

Task	*Coordinator*	*Research Units*
1 – Building aggregates – General Task Coordinator: Borri		
1.1 Typological classification and damage mechanisms	Carocci	5,6,7,8,16,17
1.2 Analysis of local mechanisms	De Felice	1,4,5,9,15,16,17,19
1.3 Global seismic analysis of built units	Borri	1,5,6
1.4 Criteria and techniques for strengthening interventions	Lagomarsino	1,5,6,7,8,9,16,17,18
2 – Mixed masonry and reinforced concrete buildings – GTC: De Felice		
2.1 Typological analysis	Ceradini	3,5,9,13,14,15,16,17
2.2 Experimentation and diagnostics	Sassu	3,5,6,10,13,14,15,16,17
2.3 Modelling and assessment criteria	Decanini	1,3,5,9,13,14,15
3 – Seismic strengthening of masonry buildings		
3a – Floors, roofs, vaults – GTC: Piazza		
3a.1 Role of floor, roofs and tie-beams	Modena	1,2,5,6,7,11,16,17,18
3a.2 Role and behaviour of vaults	Borri	5,6,16,17,19
3a.3 Experimental testing	Giuriani	2,6,11,12
3a.4 Strengthening criteria and techniques for floors, roofs, vaults	Piazza	2,6,7,11,12
3b – Masonry vertical structures – GTC: Modena		
3b.1 Regional classification	Binda	3,5,6,7,8,16,17,18
3b.2 Collection of existing data on mechanical properties	Augenti	3,6,7,8,10
3b.3 Diagnostics and in-situ testing	Vignoli	1,4,5,6,7,8,10,13
3b.4 Strengthening criteria and strengthening techniques for masonry	Modena	5,6,7,8,16,17,18
4 – Methods of seismic analysis – GTC: Magenes and Lagomarsino		
4.1 Criteria for the definition of the Knowledge Level	Lagomarsino	2,5,6,7,8,16,17,18
4.2 Static and dynamic testing on buildings	Magenes	1,5,7,10,17
4.3 Seismic behaviour of URM masonry spandrels/beams	Augenti	1,3,5,10
4.4 Validations and comparisons of different modelling techniques	Caliò	1,5,7,9,10,14

4 MAIN RESULTS

4.1 Task 1: Building aggregates

4.1.1 Subtask 1.1: Typological classification and damage mechanisms

The aim of the sub-task was to develop a methodology for the design of seismic up-grade interventions on building units within complex aggregates in historical centres. The designers run into difficulties because in these cases the analysis of the building units should take into consideration the behaviour of the whole aggregate (or at least that of the adjacent building units) but it is difficult and burdensome to perform surveys and investigations out of the building unit of intervention.

The proposed methodology considers the analysis of the construction modes and the phases of formation and transformation of the aggregate, as a starting point for the foreshadowing of possible seismic collapse mechanisms and the consequent safety checks. Starting from this knowledge and the interpretation of the actual behaviour, strengthening interventions may be defined, taking into consideration both safety and conservation of the cultural heritage.

In the first phase of the research, the large amount of data collected in the previous years by the Research Units were collected, critically revised and placed at disposal. In particular, POLIMI placed in a server of the Milan Polytechnic (http://www.stru.polimi.it:8180/) a data-base on the vulnerability of historic centres (Binda et al., 2009f); it contains the data of four historic centres of Umbria Region, hit by the 1997 earthquake and surveyed through the form set up by Milan Polytechnic, containing typological characteristics of the isolated and aggregated buildings and the vulnerability evaluation through Vulnus and C-Sisma. In the next future, all collected data will be saved referred also to other historic centres of Umbria, Liguria, Abruzzi and Lombardy Regions.

Some new applications have been developed in the three years of the project, in order to validate the proposed methodology. In particular UNISR studied the historical centre of Catania (Carocci and Indelicato, 2008), with a detailed analysis on the complex aggregate of the ex-convent of Saint Giuliano, in Catania (Neri and Marino, 2009); Figure 1 shows an example of a seismic scenario in a building aggregate.

The aim of the methodology is not to give a support to vulnerability evaluation at urban scale but to design strengthening intervention (limited to a seismic up-grade) on single building units. To this end, besides some papers (Carocci and Tocci, 2007; Carocci, 2008), a specific report was drawn, which will be useful for the designer as a Guidelines document, explanatory of the few instructions contained in the Italian Technical Code on the problem of historical building aggregates (Carocci et al., 2009).

Figure 1. Seismic scenario of a building aggregate in Catania.

4.1.2 Subtask 1.2: Analysis of local mechanisms

The problem of a correct evaluation of the vulnerability to local collapse mechanisms is of fundamental importance in the vulnerability evaluation of building units in complex aggregates in historical centres. The presence of a thick and redundant connection among the building units (and sometimes also between building aggregates) prevents in most cases from the occurrence of strength collapses due to in-plane mechanisms (see sub-task 1.3), but the construction irregularities (e.g.: walls not well clamped in the last building units, that obstructed the empty spaces) or the geometric irregularities (e.g.: different height between adjacent buildings) give rise to the conditions for occurrence of local collapse mechanisms, usually related to the out-of-plane behaviour. In sub-task 1.1 a methodology is proposed for the singling out of the possible mechanisms; the problem is to have at disposal proper tools for the evaluation of the seismic action which activates the mechanism or determines the collapse, with the aim of a preventive strengthening intervention, if necessary.

To this end, the limit analysis is the most effective tool; it leaves aside the masonry compressive strength (assuming the structure as made of rigid blocks, able to rotate and slide one with respect to the other) and considers only equilibrium conditions, related to the ratio between geometry (thickness and slenderness of the walls) and mass distribution.

The state of art before the Reluis project, at application level, was limited to the solution of simple configurations, such as a single overturning wall, without considering the connection with orthogonal walls or the possibility of articulated mechanisms with inclined fracture lines. Under these hypotheses the vulnerability is much higher than the actual one; thus, it is necessary to have at disposal new procedures able to take into account the interlocking between blocks and the friction contribution.

To this end, UNINA-b developed a simplified procedure of analysis to assess the seismic actions which activate local in-plane and out-of-plane damage mechanisms, with particular attention to the friction resistances involved (Casapulla and D'Ayala, 2006; Casapulla, 2008). The Coulomb friction model of dry rigid block masonry with non-standard behaviour (non-associative frictional sliding) is herein adopted and a global macro block analysis is used instead of a detailed discrete element analysis. The advantages of the transfer from the *micro* to *macro* scale of the model of rigid blocks are both to keep memory of the discrete model and to strongly reduce computational efforts as it allows hand or simple computer-based analysis. This procedure is carried out by means of the classical kinematic approach of limit-state analysis, with the aim of evaluating lower and upper thresholds in closed form for the seismic actions which can activate the local damage mechanisms under study, in relation to the meaningful geometrical and mechanical parameters.

In practice, the simplified procedure implies the following steps: 1) evaluation of the maximum frictional resistances along the crack lines of the mechanism under study; 2) evaluation of the maximum kinematic load factor and the geometry of the correspondent damage mechanism, based on the hypothesis of complete activation of frictional resistances along the cracks; 3) evaluation of the minimum kinematic load factor correspondent to the same damage mechanism, but based on the hypothesis of nil activation of frictional resistances along the same cracks. According to this, the "exact" values of the ultimate load factors (computed by using micro block model) are bounded from above by the assumption of full development of the friction force on every contact surface and from below by the total absence of friction, when using macro element analysis.

The procedure has been applied to three classes of mechanisms, which involve frictional shear resistances and torsion-shear interactions. Solutions in closed form of upper and lower bounds for the real load factors of the most recurrent damage mechanisms were obtained, according to the kinematic approach of limit-state analysis. The methodology was applied to a real case

(Casapulla et al., 2008) and was validated by some experimental data, in particular considering some tests of the wide campaign on reduced scale specimens of dry stone regular masonry (Figure 2), performed by UNIPV (Restrepo-Vélez and Magenes, 2009).

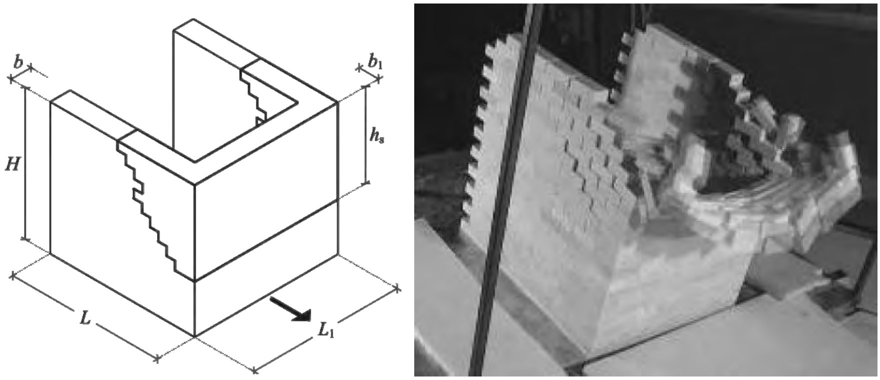

Figure 2. Evaluation of the maximum kinematic load factor and simulation of experimental tests.

Another critical aspect of the use of limit analysis for the out-of-plane response of masonry structures is the hypothesis of rigid block, because in real masonry wall it is not always expected that it behaves as monolithic and the compressive strength is not unlimited. Interesting results were obtained by ROMA3, which investigated the dependence of seismic capacity on the geometry and the arrangement of the stones within the wall section (de Felice, 2009). To this end, a set of about thirty rough stone masonry sections, with the same overall geometry (6 meters height and 50 cm. thickness), but different arrangement of stones, is considered. The out-of-plane seismic capacity is then estimated by means of pushover analysis with triangular horizontal increasing forces, as well as dynamic analyses with impulsive loading.

The masonry sections are chosen form a database set-up by POLIMI, within sub-task 1.1, and from the data collected by the UNIPG. The database collects a wide number of masonry sections surveyed from Italian historic buildings, together with the characteristics of stones and mortar. The sections considered for the analyses are chosen such as to be representative of several Italian regions. They are first reproduced by CAD and then, through an automatic generation program set-up for the purpose, the corresponding mesh is generated. A distinct element code (UDEC), is used for the analyses, where the stones are modelled as rigid bodies and the mortar joints as interfaces with cohesion and Coulomb friction, while the geometry of the stones is reproduced from the survey of the wall, as explained before. The horizontal static acceleration that produces collapse resulted on average equal to 75% of that of a rigid block (c.o.v.=0.16).

The dynamic analyses are carried out by imposing a constant acceleration a acting for a prescribed time interval Δt and then recording the motion of the structure. Three acceleration levels of 0.15g, 0.25g and 0.35g are considered, for increasing time interval, until failure takes place. The results provide the collapse mechanism of the wall, the displacement capacity, defined as the maximum top displacement the wall may sustain before collapse, the impulsive loading capacity, i.e. the maximum impulse defined as the product $a\Delta t$, the wall may sustain. The results show that, in the range examined, the impulsive capacity is almost independent form the acceleration level; the displacement capacity lies between 50% and 98% of that of a rigid bock, depending on the dimension and arrangement of the stones. The collapse mechanism is, in several cases, the same as the one registered under increasing static forces; only in some cases, the mechanism is slightly different form the previous one, since only a smaller part of the wall overturns. Some correlation exists between the displacement capacity obtained with dynamic analyses and strength capacity reached with pushover analysis. The complete pushover curves are show in Figure 3, in comparison with that of a rigid block; it is evident that in the case of a limited compressive strength, the pushover curve by non-linear kinematic analysis can be obtained by moving the hinge (centre of rotation) inside the base section. Only in few cases, characterized by a bad connection between external leaves, the wall has very low displacement capacity, due to disaggregation instead of rocking.

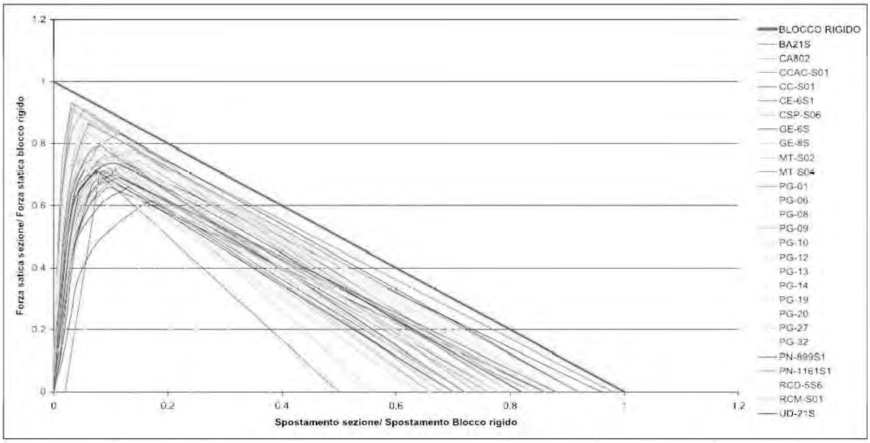

Figure 3. Pushover curves of out-of-plane response of different walls.

The above mentioned dynamic analyses are not representative of the seismic action, characterized by a sequence of impulses in the two directions. It is well known that, in the case of a seismic action, the maximum acceleration that produces the complete overturning may be much more higher than the one that induces the beginning of rocking. The response is strongly dependent on the shape of the accelerogram (maximum displacement, maximum velocity, number of peaks, duration, frequency content, etc.). The problem of identifying proper measures of the seismic input for the safety checks was faced by different UR, by performing non linear dynamic analyses.

ROMA1 worked on dynamic modelling, with particular attention to the kinetic energy dissipation and the selection of significant recorded accelerograms, to be used in non-linear dynamic analyses (Decanini and Sorrentino, 2006). By using seventy different accelerograms, a lot of dynamic analysis were developed with the aim of evaluating the correlation of the response with different measures of the seismic intensity: PGA, PGV, PGV/PGA, Arias intensity, EPA - effective peak acceleration, Housner intensity, energetic measures. In particular a good correlation with the overturning was observed with intensity measures of the input motion based on the area of the pseudovelocity spectrum (Decanini et al., 2006).

ROMA1 has also defined an analytical model for the dynamic analyses of a vertical spanning strip wall, with a special emphasis on the role of energy damping, the forecast of the height of the intermediate hinge, and the amplitude dependency of the period of vibration. The dissipation of kinetic energy was studied from the theoretical point of view, considering two different configurations: a wall leaning against transversal walls (Decanini et al., 2007) and a wall constrained at the top (Sorrentino et al., 2008a).

UNIBAS carried out a lot of numerical analyses, considering many different earthquakes, aimed at investigating the response in one-sided and two-sided rocking (Liberatore et al., 2007; Liberatore and Santarsiero, 2009). For the latter, the influence has been analysed of the energy dissipation during the impact with the base, introducing different values of the restitution coefficient. The results of the numerical analyses were represented by overturning diagrams, which confirm the size effect which affects the behaviour of rocking blocks. For some earthquakes, an unexpected strong influence is found of the direction of the seismic motion. Synthetic domains have been determined, both for one-sided and two-sided rocking, as function of peak ground acceleration and peak spectral displacement, which permit to determine the vulnerability of the blocks on the basis of their geometric characteristics. Figure 4 shows that in most cases overturning occurs only if the wall's thickness is lower than four times the maximum displacement imposed at the base. This limit (obtained by the parametric analysis as a characteristic value at 95% probability) is coherent with the one proposed in the Italian Code, which is 40% of the limit displacement (which in the case of a single block is half of the thickness).

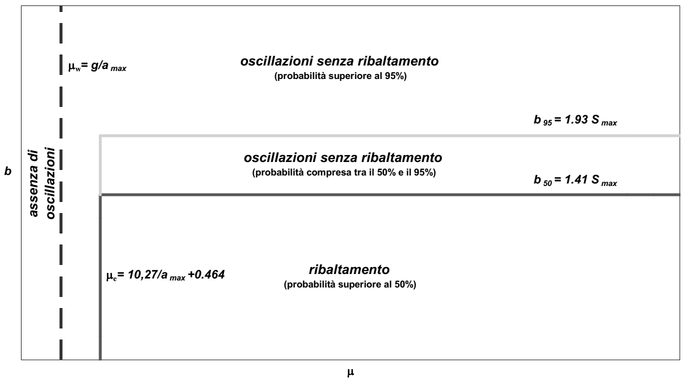

Figure 4. Overturning domains as a function of the slenderness and the wall thickness.

UNINA-b studied the rocking response of a rigid free-standing block under the action of ground motion considering, in alternative to the more classic probabilistic approach, artificial accelerograms characterised by such a combination of acceleration pulses as to involve the most disadvantageous conditions for the block (Casapulla et al., 2007; Casapulla and Maione, 2008). Then, a stabilised phase of the resonance response has been identified, characterised by a periodic motion and the definition in closed form of the thresholds for the maximum rotation angle of the block. The results in terms of spectral parameters should be reasonably reduced by excluding progressively all the features which do not occur in a great number of natural accelerograms. This phase, not developed yet, will include a useful comparison with the results from the elastic linear single-degree-of-freedom oscillator under the same resonance conditions. On these bases, new important guidelines should be pointed out for the construction of reliable response spectra for masonry structures and for improving the current seismic codes in this field.

Within the ambit of sub-task 1.2, some laboratory investigations were made, which result very useful to partially fill a gap in the literature on this topic. The experimental activity has considered both free and forced vibrations, and has been completed by numerical analyses.

As for free vibrations tests, ROMA1 has considered two configurations (two-sided rocking and one-sided rocking), two materials for the units (tuff and clay brick), several height-to-thickness ratios, several lateral contact depths between façade and transverse walls (Sorrentino et al., 2008b; Decanini et al. 2008). Roughly 500 displacement time histories have been recorded, which have been used to assess the so-called coefficient of restitution. The latter is approximately some 5% lower than the analytical one, if two-sided rocking is considered. As for one-sided rocking, the tests are even more valuable because there are no others reported in the literature. Measured values are around 40-50% of the analytical one valid for two-sided rocking. Energy damping is approximately constant with test repetition. The same behaviour has been observed as for displacement capacity.

As for forced vibration tests, performed by ROMA3 and ROMA1 (Al Shawa et al., 2009a and 2009b), resorting to Research Line 1 common funding, about seventy tests are carried out, for three different structural configurations: the wall connected to the transversal walls through the mortar cohesion (Figure 5), the simply-standing wall without connections, the wall strengthened with steel bars. One of the specimens was made of rough-hewn limestone units and the others of dimensioned tuff units. The tests allow to experimentally evaluate the so called restitution coefficient, the discrepancy between the effective dynamic response and the one provided by the rigid block rocking model, the influence of the mortar joints and the structure defects, the changes in the response during motion, the energy dissipation related to the opening of connections, the damage caused by the impacts, the dissipation energy mechanisms which take place.

The shaking-table tests show that out-of-plane loaded walls have a significant seismic capacity, even in the absence of connections to the orthogonal walls. When some connection is ensured by the mortar tensile strength, the wall tested, having height 3400 mm and thickness 250 mm, is able to resist to natural inputs such as Bagnoli or Calitri scaled to a PGA of 0.27g. In the case of the simple standing wall, with no connections to the orthogonal ones, the experimental results show the capability of the wall to stand under the following records scaled to the values of PGA listed below: Bagnoli (PGA 0.19), Calitri (PGA 0.18), Sturno (PGA 0.23), Nocera Umbra (PGA 0.30).

These results are very interesting for the engineering practise as they prove the behaviour factor for these kind of mechanisms may be between 2.5 and 4 (in the Instructions to the Italian Technical Code q is assumed equal to 2). However, it is worth to note these values are related to an isolated overturning wall; if the wall is leaning against transversal walls, the behaviour factor increases to, at least, 3.6 (indeed, the free vibration tests performed by ROMA1 showed a significant increase of the restitution coefficient in the case of one-sided rocking).

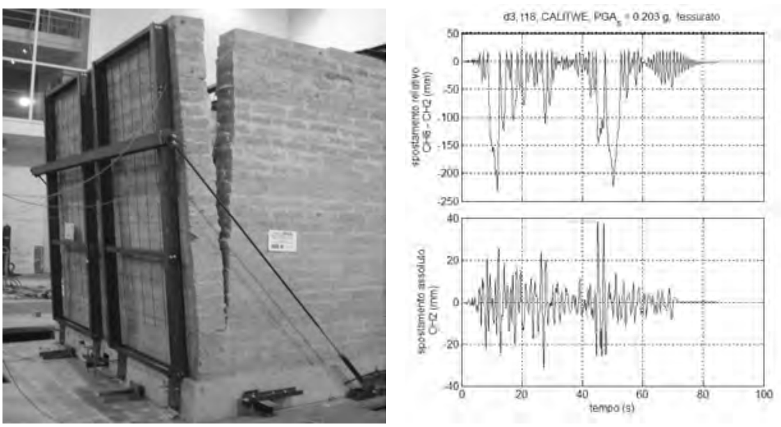

Figure 5. Shaking table tests on out-of-plane behaviour of a wall leaning against transversal walls.

Even if some extended experimental evidences would be necessary to draw some general conclusion, it appears from the tests that, in low seismicity area (classified as 3 or 4 according to Italian rules), the walls, when well done such as not to separate into two faces, are able to resist to the design seismic action. This statement changes profoundly the actual design rules, which neglect the structural contribution of the out-of-plane loaded walls, and require to connect the wall such as to make a box resisting structure. This solution is clearly suitable for increasing the seismic resistance, but seems not to be necessary for existing masonry building in low seismicity areas.

The experimental results also show the great efficacy of traditional strengthening techniques, consisting in chaining the out-of-plane loaded wall to the orthogonal ones by means of steel bars with end keys. In the case under study, the wall strengthened with chains, despite the specimen was damaged with a vertical fracture in the central span and completely detached from the orthogonal walls, displays a very high seismic strength up to PGA values of 0.6g.

Finally, another important topic of this sub-task is the definition of demand spectra for the seismic check of local mechanisms involving portions of the building located at the higher levels. Usually, in ancient existing buildings in historical centres overturning does not involve the whole wall, being it connected at least with some floor structure or tie-rod or clamping with transversal walls, but interest the upper part. The procedure for the verification of such mechanisms may be the same, but it is necessary to take into account the amplification of the input motion with respect to that at the base of the building, due to the filtering effect of the main structure. UNIPV and UNIGE worked on this topic.

UNIPV modelled the wall element subjected to out-of-plane loads as a secondary s.d.o.f. system, supported by a primary s.d.o.f. system (the building) filtering the seismic input. The primary system may develop a nonlinear response. A wide nonlinear dynamic simulation campaign was carried out, in which attention was focused on the role of the nonlinear response of the primary system on the filtering effect on the secondary system. A semi-empirical formulation of the spectral acceleration response to be utilized for the seismic assessment of a local mechanism in which the primary system (the building) responds globally in the inelastic range, has been proposed. The formulation is currently valid for systems dominated by the fundamental mode of vibration (Menon and Magenes, 2008).

UNIGE worked on the same topic, analyzing at the beginning the problem of local collapse mechanisms in bell tower cells; a mechanical based method for the definition of seismic demand spectra at higher levels than the ground has been defined in order to take into account the filtering effect of the structure. The demand spectra can be used both for linear (amplification of the PGA at higher levels) and non-linear kinematic analyses; spectra were defined both for 5% damping (for the application with the method proposed in the Italian Code) and for higher values of damping, for the use of capacity spectrum method with overdamped spectra (1.2-UR05-1).

4.1.3 Subtask 1.3: Global seismic analysis of structural units

Even if the main problem in the case of buildings in complex aggregates is that of local mechanisms (see sub-task 1.2), it is always necessary to verify the shear strength on masonry piers. The relevant dimensions and irregularities of aggregates in historical centres make in most cases almost impossible the use of 3D equivalent frame models for the seismic analysis, as a model of the single building unit of analysis is not meaningful, because it is not able to consider the interactions with the adjacent buildings.

In order to quantify these effects, by means of detailed 3D pushover analyses, UNIPV performed parametric analyses to evaluate the mutual interaction of adjacent buildings within an aggregate (Senaldi et al., 2009). The effect of the position of a single structural unit within

the aggregate (row conglomerations) on its seismic response has been studied, with the hypothesis of flexible diaphragms (e.g. wooden). The results show the differences that can be observed between the response of single structural units, “extracted” from the aggregate and separately analyzed, and the response of the same unit when the dynamic behaviour of the entire aggregate is modelled.
UNIGE performed similar 3D pushover analyses on complex aggregates. The achieved results have been finalized to the drawing up of a guideline document (in cooperation with UNISR and already cited in 1.1.1) which, in addition to proposing simplified method of analysis, contains a methodological procedure for an accurate knowledge of the aggregate (Carocci et al., 2009). The simplified mechanical model evaluates the peak ground acceleration which produces the collapse of the structural unit examined (based on the hypothesis that the global collapse occurs in correspondence of the simultaneous attainment of the maximum strength on the vertical masonry panels located on the same floor). The introduction of specific corrective coefficients allows to characterize the response of the building in order to take into account of the behaviour modifications related to its insertion in the context of an aggregate. As an example, corrective coefficients are introduced to take into account of the torque effects induced in the unit examined by the irregularity of the aggregate plan (calibrated as a function of the position of the single unit into the aggregate). It is important highlighting that the seismic verification procedure, illustrated in the Guidelines, includes also the verification of the local mechanism in addition to that of the global response.

4.1.4 Subtask 1.4: Criteria and techniques for strengthening interventions

The research on strengthening intervention techniques for existing masonry buildings was carried out in sub-task 3b.4 (Modena, 2009). However, some typical situations may be encountered in building’s aggregates and are discussed in the above mentioned Guidelines (Carocci et al., 2009). For instance, in aggregates it is very important to take into consideration the effect on the masonry quality due to rearrangements, following transformations in the past (e.g.: closure of previous openings). Another important subject is that usually a horizontal bracing is not needed (in-plane stiffening of floors), except from the case of particular irregularities in the structural system (e.g.: very weak wall, with large openings or small thickness) or in proximity of the aggregate’s corners, where a light stiffening may be useful for the prevention from some local mechanisms.
For the design of seismic up-grade interventions, the correct approach should follow some typical problems in the proper sequence: 1) structural nodes, i.e. connections (wall/wall; wall/roof; wall/floor; wall/vault); 2) masonry quality; 3) bracing (sharing of seismic actions among structural elements).

4.2 Task 2: Mixed masonry and reinforced concrete buildings

4.2.1 Typological analysis

The first step to understand what methods and strategies should be followed to assess and increase the seismic safety of mixed masonry buildings is to know what variety of typologies can be encountered on the Italian territory. To this end, archival researches and on-site surveys were carried out within the project by UNIRC, UNIPI and UNICT. An in depth bibliographical survey was also carried out by ROMA1.
UNIRC carried out an in depth archival research in five Italian regions of higher seismicity, namely the Sicilian side and the Calabrian side of the “Strait” area, Irpinia (Campania), Marsica (Abruzzi) and Friuli. After the first archival researches it was realized that a first distinction between residential and specialized (e.g. schools, hospitals..) buildings was necessary, since only the residential buildings tend to present repetitive typologies in terms of

building techniques and layout, whereas specialized buildings tend to have a higher variation and really seem to be in first instance much more problematic. Criteria for a typological classification of the residential buildings are proposed in Ceradini et al. (2007) and Ceradini (2009). The proposed classification concerns buildings in which r.c. elements are cast in contact with masonry to guarantee a mutual collaboration as a composite structural system (i.e. confined masonry).

The work of UNICT has focused more on a sample of buildings constructed after the 1908 Messina earthquake (confined masonry), from which a representative configuration was defined as a case study for modelling methods (Caliò et al., 2008).

UNIPI has analyzed a sample of more than 200 school buildings in the Tuscany region, of which about 1/3 are mixed masonry and reinforced concrete buildings, singling out (Mariani et al., 2007) the changes of the construction techniques with time. For many of the mixed buildings serious modelling problems were envisaged. Furthermore, materials degradation was found to be a significant issue especially for the schools built in the 1960s.

4.2.2 Experimentation and diagnostics

Within this task experimental activities were carried out considering in-situ testing techniques to provide information on concrete and masonry properties. The in situ test were carried out on existing buildings, of which some were of the mixed type, but the techniques per se are also applicable to standard masonry or reinforced concrete structures.

UNIPI developed (Sassu 2008) a combined in-situ compression and shear testing method on masonry samples extracted from masonry parapets and non destructive penetrometer tests to characterize masonry material properties. Such techniques are further described in the results of task 3b.

UNIFI (Vignoli et al. 2008) has carried out in situ tests to correlate concrete core tests, SonReb tests, and chemical tests to evaluate the mechanical properties of concrete in relation with the degradation induced by chlorides and carbon dioxide.

4.2.3 Modelling and assessment criteria

The researches on modelling and assessment criteria were oriented mainly to two typologies of mixed structures: the masonry concrete *dual systems*, where masonry walls and r.c. elements (frames or walls) act in parallel, coupled by roof or floor diaphragms, without significant local interactions, and the *confined masonry* typologies.

Among the first typologies (Figure 6), a few meaningful building configurations were analyzed by different research units using different modelling techniques. Some works have also dealt with examples where an r.c. story has been added on top of an existing masonry building. Such technique is sometimes used to increase the volume of existing buildings adding an extra story.

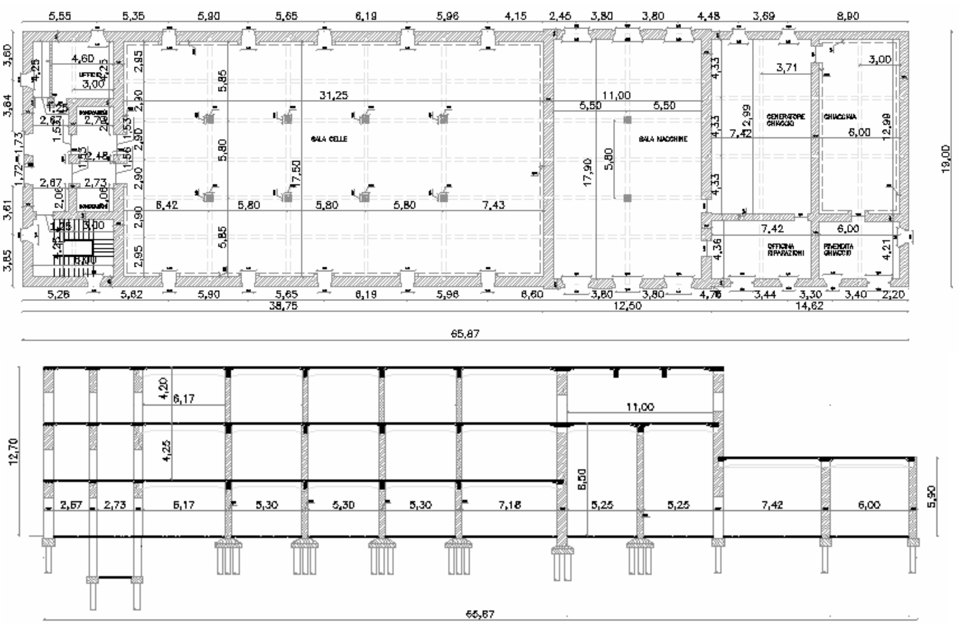

Figure 6. Mixed masonry-r.c. building: plan and section of the Ex-Slaughterhouse in Rome.

A state-of-art has been prepared by ROMA1 (Liberatore et al., 2007), where 160 national and international publications covering code regulations, post-earthquake performance assessment, typologies such as dual systems and confined masonry, were examined. Together with the surveys carried out in task 2.1, such a preliminary work has confirmed the great variety of typologies and the difficulty in singling them out, especially as regards their seismic performance. In post-earthquake surveys, dual systems are rarely recognised, and therefore the information on their performance remains very scarce. More information is available regarding the behaviour of confined masonry systems, especially from Latin American countries.

As for modelling, among the nonlinear modelling techniques that have been adopted in the project, two main approaches can be envisaged: macro-element or equivalent frame approaches and finite element approaches. The first approaches were followed by UNIPV and UNIGE (Fugazza and Magenes, 2006, Cattari et al., 2008) in the analysis of dual systems. The finite element approaches were used by ROMA1 (Decanini et al., 2007, Liberatore et al., 2008) and ROMA3 (de Felice and Malena, 2009). While ROMA3 has used the constitutive models available in the commercially available code ADINA, ROMA3 has developed a two-parameter continuum damage model for masonry, while fibre frame elements were used for r.c. members. Examples of the structural configurations that were analyzed by the research groups are shown in Figure 7 and rather detailed description of the modelling hypotheses are give in the above mentioned references; Figure 8 shows a comparison of results from different numerical models.

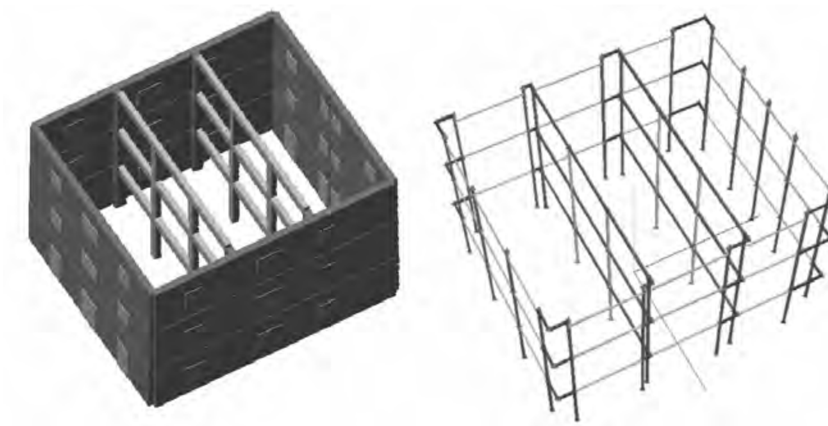

Figure 7. Mixed masonry-r.c. building: 3D models of a building in Capri by Tremuri and SAM software.

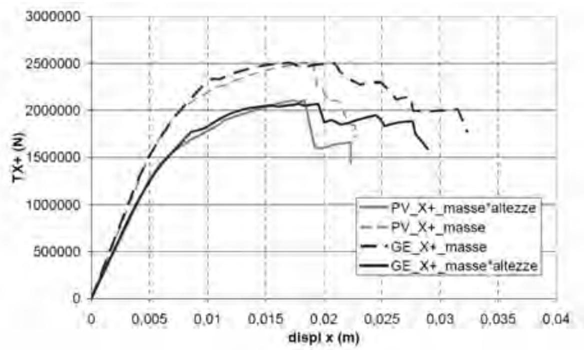

Figure 8. Comparison between Tremuri and SAM results on the study case of Capri.

UNINA (Augenti and Parisi, 2009a) has undertaken preliminary analytical studies on dual systems which are irregular in plan, which for the time being are limited to the linear behaviour, but present however some interesting considerations regarding the non distribution of seismic shear ad torque on the different structural components.

Attempts to model the confined masonry typologies were made by UNICT and UNIGE. The first modelling strategy (Caliò et al., 2008) is based on a suitable assemblage of macro-elements which are in turn made by assemblages of nonlinear springs. The second modelling strategy follows a more refined nonlinear finite element formulation (Calderini et al. 2008), which however for the time being has been applied to the simulation of small structural components (single panels) with the aim of comparing the results with existing code or simplified formulations for the prediction of strength of confined masonry walls.

A synthesis report of the activities carried out in the present task is given by Decanini and Liberatore (2009). The main findings can be summarized as follows.

Given the standpoint of the research in the field, almost the entirety of the non linear methods of analysis used were limited to static (pushover) analysis. The great part of the case studies that were considered were rather regular in elevation, so that no significant contribution of higher modes should be expected.

It has been confirmed that for dual systems nonlinear analyses provide quite different results from linear analyses, as a consequence of the different stiffness, strength, deformation capacity of masonry and r.c. members. When only r.c. frame members are present, and floor diaphragms are stiff, the frame contribution to the lateral resistance is generally much lower than the contribution of masonry, and its neglection is on the safe side. Care should be taken when flexible diaphragms are present, since only limited transfer of horizontal forces may be possible from the frame to the walls, and the frames should be able to withstand the seismic actions that derive from the vertical loads carried by the frames themselves.
When both r.c. walls and URM masonry are present and rigid diaphragm action can be assumed, both systems give an important contribution to the seismic resistance and the neglection of either system would produce misleading and unrealistic results. In such cases however the difference that can be found in the distribution of seismic shear comparing elastic linear and non linear analysis can be huge.
No definitive conclusions can be drawn yet on systems with different materials/structural systems used at different storeys, regarding the use of linear vs. non linear methods of analysis, since very limited nonlinear dynamic analysis have been carried out within the project.

4.3 Task 3a: Seismic strengthening of masonry buildings – Floors, roofs, vaults

4.3.1 Floor diaphragms

A wide experimental campaign has been carried out by UNITN on different kinds of timber floors (Baldessari et al., 2008; Tomasi et al., 2009) aiming to characterize their in-plane strength and stiffness (Figure 9): (a) existing simple layer of wood planks on the timber beams; (b) second layer of wood planks crossly arranged to the existing one and fixed by means of steel studs; (c) diagonal bracing of the existing wood planks by means of light steel plates or FRP laminate; (d) three layers of plywood panels glued on the existing wood planks; (e) stud connected reinforced concrete slab. Experiments in real size (5x4m) and small size (2x1) specimens were tested, showing the importance of the size of the specimen on experimental results, in terms of stiffness and strength. A special test setup had to be designed to carry out in-plane monotonic and cyclic deformation tests of the floor diaphragms.
Within the same scope, tests on dowel type connectors have been carried out at UNITN to validate the yield model reported in European standards (Johansen model). The effectiveness of inclined dowels has been confirmed and a new approach for calculation of inclined dowels has been proposed, which can be also used for on-site applications where new timber planks are connected to the existing timber beams (Crosatti et al., 2009).

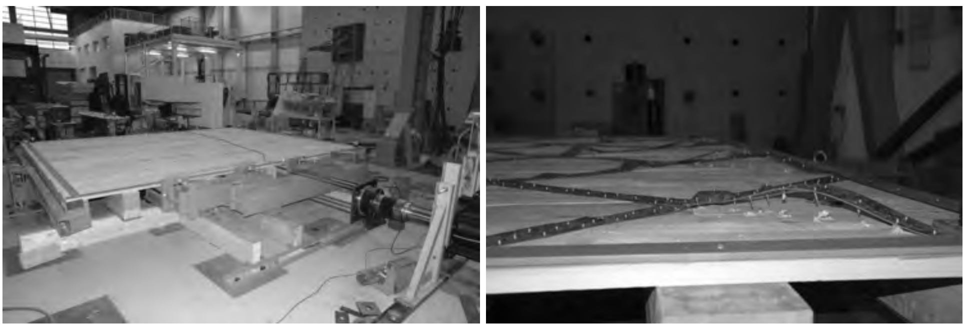

Figure 9. Setup for in-plane testing of full scale floor diaphragms (left) and view of a broken floor specimen after the tests (right) (Baldessari et al., 2009).

Tests on floor diaphragms were also carried out by UNIPD (Modena et al., 2009) on timber floor diaphragms reinforced with different techniques. Monotonic tests have been carried out so far, accompanied by numerical simulations, and the prosecution of the experimental campaign with cyclic tests is foreseen.

UNIBS has also carried out in-plane testing on floor diaphragms stiffened by means of either thin reinforced concrete slabs or nailed plywood panels, and an experimental study on the cyclic behaviour of the connections between the diaphragm components and between the diaphragm and the peripheral walls. The results have been used to validate the simplified analytical models, developed for the proportioning of the main structural components. The main problem of the connection of the roof diaphragms to the perimeter walls has been also addressed (Figure 10). As a conclusion of the research project, a final document, including all the main results and describing the design criteria of floor diaphragms, has been edited (Giuriani and Marini, 2008 and 2009).

4.3.2 New technologies for ring beams

A ductile design approach for steel reinforced timber beams have been considered by UNITN. Steel reinforced timber beam can be conveniently used as tie-beam at the roof level in historical masonry buildings, as alternative to the traditional concrete curb. An innovative steel-to-timber joint have been proposed, where a steel reinforced timber element is connected to a steel stub by means of an end plate and glued–in steel rods. The results of experimental analysis on such components showed the high potential of such joints in terms of local ductility (Tomasi et al., 2008). Furthermore, the joint model, based on the component method, provides a satisfactory approximation of the test results, in particular for the ultimate load capacity, and failures modes. Moreover the structural advantages derived from coupling laminations either of different species (two-species glued-laminated timber beam) or of different strength classes from a single species (combined glued-laminated timber beam), have been studied through theoretical and experimental analysis.

UNIPG has experimented the behaviour of reinforced masonry ring beams where the reinforcement is made by composite materials (e.g. FRP). Several specimens have been tested under cyclic flexure and under long duration loading. The results (Borri et al. 2009) show that such type of reinforced masonry can be conveniently used to build ring beams for the connection between roof and vertical walls, with an advantage in lower weight and better material compatibility with respect to the traditional r.c. ring beams.

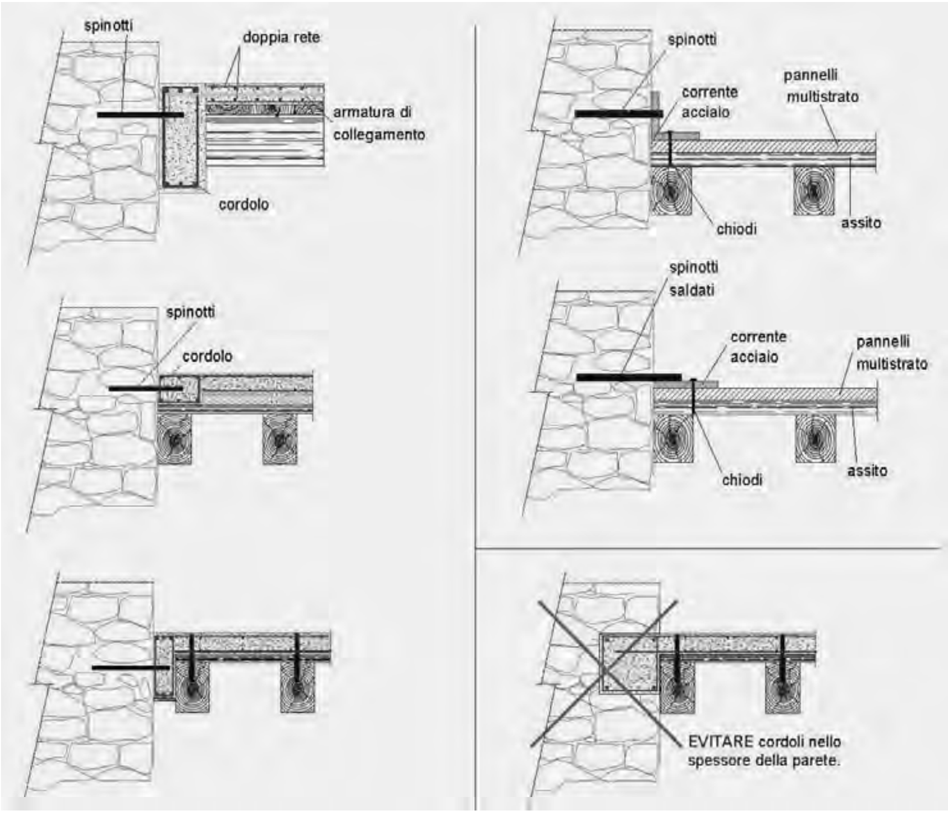

Figure 10. Different solutions for the ring beam and the connection to the timber floor.

4.3.3 Roof structures

As to the global seismic behaviour of roof structures, the elements that contribute to vulnerability had been identified and selected by POLIMI-b in collaboration with UNITN as vulnerability indicators in the assessment procedure that had been outlined (Chesi et al., 2008; Parisi et al., 2008). The work has mainly focused on the effect of conceptual design on the seismic response. Modal and response spectrum analyses have been extended to a number of typologies, each analyzed for different combinations of design parameters values. In particular, the effect on the seismic response of three-dimensionality and of how it is realized in these structures has been investigated. Reference values and criteria for the relevant vulnerability indicator have been developed.

4.3.4 Carpentry joints

The study on carpentry joints was carried out mainly with reference to joints of traditional timber roof structures. Experimental and theoretical work, carried out by UNITN and POLIMI-b in cooperation also with the University of Minho (Guimarães, Portogallo), on carpentry joints and on their reinforcing techniques allowed to characterize the semi-rigid behaviour for this kind of joints and to evaluate the energy dissipation of traditional timber roof during seismic events, in addition to providing results for the drafting design guidelines for the improvement of traditional timber joints (Piazza et al. 2006; Parisi and Piazza, 2007 and 2008; Tomasi et al., 2007; Parisi and Cordié, 2009).

4.3.5 Vaulted structures

The research units of UNIPG, UNIGE and UNINA have contributed to the drafting of a technical report for professionals regarding the role and analysis of structural vaults and the diagnostics of their possible pathologies (Bussi, 2009; Cangi, 2009; Paradiso and Tempesta, 2009). The final product of the research consists of the assemblage of the different contributions and it is a starting point for a design handbook.
UNIGE has worked to establish a correlation between the stiffness in an elastic field of certain typologies of vaults and the axial and shear stiffness of an equivalent orthotropic panel. From the results obtained from numerical simulations (Figure 11), analytical formulas were found in order to provide an useful tool which can be immediately utilized for project design, in particular in those cases in which modelling approaches, like as the equivalent frame type (as proposed also by the Italian and international codes), are adopted for the seismic verification of existing masonry buildings (Cattari et al., 2008).
UNIBS has delved into the behaviour of barrel vault subjected to horizontal actions, through experimental and analytical models (Marini et al, 2009). The essential role of the vault tie system under transversal rocking motion has been demonstrated, and a simplified analysis method based on limit analysis has been developed for possible use in design.

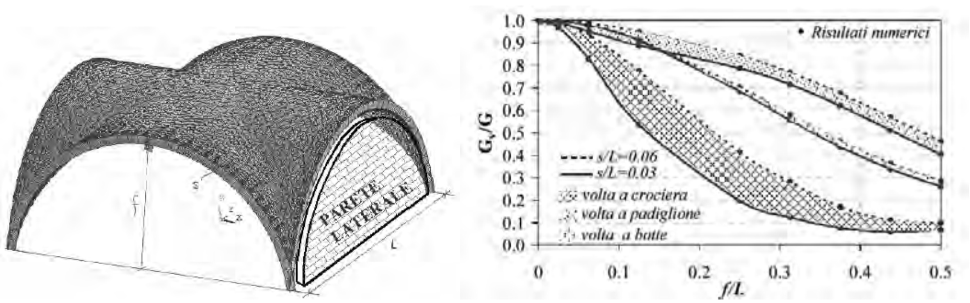

Figure 11. Finite element model of a cross vault. Equivalent in-plane stiffness of different types of vaults as a function of ratio between rise and span of the vault.

4.4 Task 3b: Seismic strengthening of masonry buildings – Masonry vertical structures

4.4.1 Subtask 3b.1: Regional classification

Focusing the attention on existing masonry building (built at least fifty years ago), the proper identification of the masonry type represents a fundamental issue. It is well-known that making an accurate classification of masonry is very difficult and, indeed, non theoretically rigorous, since the historical masonry is too much complex and variegated in relation to: single constituent components (mortar joints and blocks), shape and size, methods of laying, quality of mortar, in plane and cross section masonry pattern. In addition to these issues, an objective difficulty in characterizing the masonry mechanical parameters by performing experimental tests in situ has to be stressed; this difficulty may be related to both the invasiveness of these tests (if employed in widespread and statistically significant way) and their cost (not always bearable in the case of interventions on private or minor structures). For these latter reasons, the Italian Code, which has been recently revised, proposed a classification of some masonry types (Instructions to the Italian Technical Code, Annex C8A.2, February 2009), to which are associated range of reference values of the main

mechanical parameters, which have to be considered as representative of the 68% of each class (± one standard deviation).
Main objective of this sub-task has been to define a survey procedure for masonry able to lead to a proper assignment of the masonry type. Regarding this issue, the possibility to provide a much more detailed masonry classification, by proposing significant types at regional level, has been examined. However, at the end, it not appeared necessary, although the strength mechanical parameters (issue which is deepen in the following sub-tasks) at national level showed some differences (noticed for macro-areas).
POLIMI, UNIPG and IUAV are the main research units which worked in this sub-task. To these latter, UNIPD, which collaborated with POLIMI in applying the survey form on some case study, and UNIGE, which worked within the Line 10 together with the research unit CNR-ITC of L'Aquila on a similar topic (of which the results have been compared and integrated), have to be added. In particular, a procedure has been proposed which, starting from a visual survey of a set of parameters aimed to describe the masonry external face, automatically leads to the assignment of the masonry type according to the classification proposed in the Instructions to the Italian Technical Code.
A document "Form for the evaluation of the masonry quality" (Binda et al. 2009a) was defined including contributions of the research units of Perugia (A. Borri), Venice (F. Doglioni) and a modification of the form realised by Milan Polytechnic (L. Binda), proposed at the beginning of the project as a starting point for the evaluation of the masonry quality.
The survey methodology proposed by POLIMI starts from the visual inspection of masonry texture in façade and in cross section and is based on the Non-Destructive experimental investigation: sonic pulse velocity tests are used to detect the masonry density distribution, single flat jack tests are used to detect the local compressive state of stress (see subtask 3b.2, Binda et al. 2009b) and double flat jack tests are used to detect the stress-strain behaviour due to compression, elastic parameters and the first crack stress (see subtask 3b.2, Binda et al. 2009c). At the end of the tests, after a small disassembling of a small masonry portion, it is possible to survey the real cross section of the masonry wall and to sample materials for their laboratory characterisation.
An important deliverable of sub-task 3b.1 is the data-base on masonry cross sections (Binda et al., 2009g), that was updated also with data coming from UNIPG.
UNIPG proposed a procedure for the assessment of a Masonry Quality Index (IQM) which is based on the identification of the masonry buildings typical features evaluated with respect to the "rules of art" (Borri and De Maria 2009a), as reported in ancient and modern handbooks; from the visual inspection of masonry texture in façade and in cross section, a numerical evaluation is given to different parameters and a quality index can be obtained (Figure 12). This methodology represents a development of that arranged and proposed by the Umbria Region for the seismic mitigation interventions. Among the innovations introduced, it is worth highlighting that the evaluation of the IQM is performed by defining the masonry quality as a function of the load type examined: 1) vertical loads; 2) horizontal in-plane loads; 3) horizontal ou-of-plane loads. Moreover, a correlation between the masonry quality index and the values of the mechanical parameters proposed in the Annex C8A.2 of the Instructions to the Italian Technical Code has been proposed; in particular a relationship aimed to provide values of mechanical parameters "coherent" to those proposed in Annex C8A.2 as a function of the IQM has been obtained. Finally, as final derivable, several illustrative forms (Borri and De Maria 2009b) and two documents of guideline for the evaluation of the IQM (Borri and De Maria 2009c) and for the correlation with the tables contained in Annex C8A.2 (Borri and De Maria 2009d) have been provided.

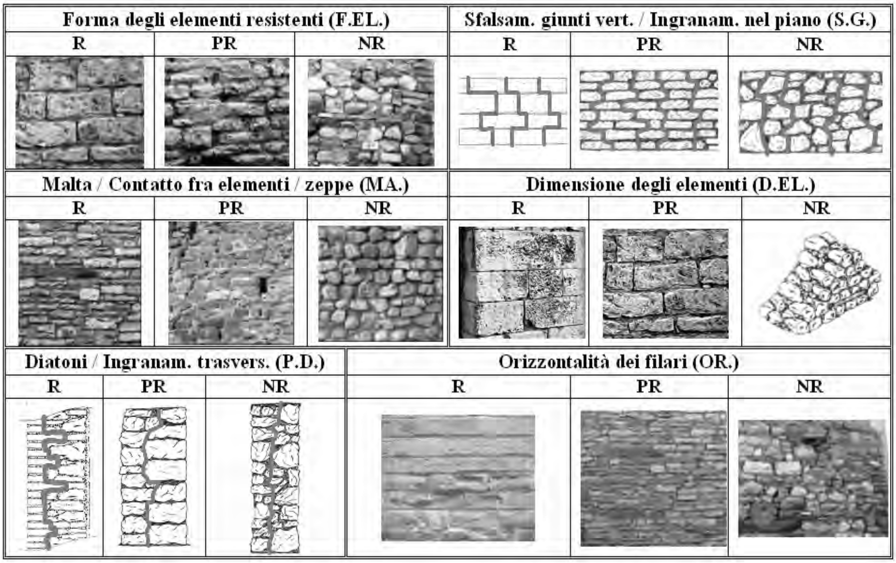

Figure 12. Masonry Quality Index (IQM): visual inspection of masonry texture and cross section.

IUAV focused its attention to finding a quick evaluation procedure of the masonry quality and of the related vulnerability against seismic actions, depending on the efficiency degree and constructional quality of the masonry itself. Analyzing some sample buildings and specimen set up by Pavia's RU (Doglioni et al. 2009a), it was possible to observe that masonry structures show conditions and characteristics very heterogeneous, which variation affect in a very substantial way the seismic response of the whole building: masonries with effective adhesive and cohesive bonds and higher mechanical characteristics, in particular of the mortar, usually discretized in large blocks, undergoing the action of the main damage mechanisms connected to the geometrical configuration of the building; on the contrary, when these characteristics decrease, sub-mechanisms of smaller extent are activated, attaining to a diffused crumbling in the case of inadequate mechanical and adhesive-cohesive properties, that can lead to the collapse without the activation of the typical damage mechanisms due to geometrical and constructional characteristics of the building.

Starting from the concept of masonry efficiency connected to the maintenance or the loss of initial constructive performances, different reference levels have been selected for the vulnerability against seismic actions, not only by the different aptitude to the damaging, but also by the variation of the damaging pattern (by large mechanisms or by diffused crumbling) which, by the way, may be associated to significantly different intervention approaches.

Therefore, the *Masonry Quality Index* can be expressed as value or degree of seismic vulnerability function of *Direct Measurements* (as the evaluation of the LMT – Line of Minimum Tracing; Doglioni et al. 2009b; Doglioni et al. 2009c), of *Observation Groups*, as the cracking pattern analysis, and of *Non-Destructive Tests*, as sonic tests, subdivided in five ranging classes (Doglioni et al. 2009d): lower masonry, medium-low, medium masonry, medium-high, higher masonry. In this way the index aims to define the tendency of a building, or its single parts, in the actual state, to undergoing a structural damage in consequence of defined actions, both vertical and horizontal. First of all, vulnerability is in fact a damage multiplier, but also a factor driving the damage to assume some shapes and not others, conditioning in this way both the quantity and the quality, or the collapse mode. So the masonry vulnerability is qualified and quantified, and through it the building state and the structural requirements are described, that are essential for an adequate and careful choice of seismic improvement interventions. Many different ways and levels of knowledge and visual characterization of masonry panels have been studied and tested, and the parameters that must be chosen as reference for the masonry quality evaluation have been defined, as well as the correspondent critical levels of behaviour related to specific masonry face typologies.

4.4.2 Subtask 3b.2: Collection of existing data on mechanical properties

The research activity of this sub-task has been mainly carried out by UNINA, with the contribution of some other research unit (UNIFI, UNIPD) which provided data of experimental tests performed on different masonry types.

Scope of sub-task was to create a tool able to collect and manage results of experimental tests carried out for evaluating mechanical parameters of different types of masonry.

To this end, UNINA designed and implemented of a fully original electronic database (Augenti 2008, Augenti and Romano 2007); in such a tool, results of experimental tests were organically collected, based on the parameters already contained in previously edited storage files. Information regarded either characteristics of masonry panels, or the ones of their constituting units and mortars. To facilitate homogeneous input and consultation of data for each entry to be inserted into the database, “curtain menus” were arranged to detect recurrent terms. Results of more than one hundred experimental papers for a total of about 400 files were entered into the database and several operation tests were conducted. The produced electronic tool allows an optimal management of all the collected data via multiple keywords. It is also possible to visualize original papers from which the results were extracted. Another prerogative of the database is to be placed in Internet and to interact with other databases.

Finally, some comparative analyses between values of the database and those listed in the new Italian code were carried out only for tuff and clay brick masonries, in order to revise the code (Augenti and Romano 2008, Augenti and Parisi 2009b).

Moreover, POLIMI worked on the correlation between the data achieved through sonic pulse velocity tests and the through flat jack tests. The collected parameters were compared with the values reported in the national standard code (Binda et al., 2009d). UNIFI provided an extensive application of in situ tests in two case studies (Vignoli et al., 2008a).

4.4.3 Subtask 3b.3: Diagnostics and in-situ testing

Main goal of this sub-task has been the improvement and the development of diagnostic techniques to perform in situ or in laboratory for the masonry mechanical characterisation.

As final product of the research, technical schedules containing the execution and results processing procedure of each experimental test examined have been arranged (Vignoli and Modena, 2009). Main aim to this product is to provide a standard procedure of these experimental tests to be adopted as reliable reference for engineering practice.

All the schedules complied with a standard format proposed by UNIFI. To this sub-task contributed several research units, involved in different testing procedures.

UNIFI worked on the following in-situ tests (Vignoli et al. 2008b): compression test, compression-shear test, diagonal test and DRMS (Drilling Resistance Measurement System) test for mortar, that was properly calibrated (Del Monte and Vignoli 2008).

Even UNIBAS worked on another kind of the penetrometric test for mortar, aiming at calibrating the test procedure and classifying the masonry types. A wide test campaign on 6 monuments of Basilicata has been carried out; in particular 25 penetrometric tests on mortar has been performed, each consisting of a set of max 10 perforations. For each type of mortar investigated, the resistance to penetration has been determined, together with the mean shear strength.

UNIPI worked on two different in situ testing procedures with the aim of combining low costs, simplicity of execution and reliability. The first one, named Pile-Model, is not much invasive and combines mortar penetrometric measures with PNT-G and laboratory compressive tests on blocks, together with the survey of the masonry texture (Gucci et al. 2006). The penetrometer PNT-G is a hand drill which measures the energy consumed making a hole deep 5 mm with a drill bit of diameter equal to 4 mm. The perforation energy of PNT-G is related to the compressive strength of mortar. Tests on mortar joints can be easily performed in situ; the values obtained not depend on the strength exercised by the technical worker; the instrument provide the best results in case of weak mortar. A technical report with technical details for a procedure to use penetrometer PNT-G has been prepared (Gucci et al. 2008).
The second experimental technique, named Opposite-Panels, consists of in-situ racking tests (vertical compression and shear) on twin panels, obtained by a single spandrel masonry beam (width 2,20 – 3,00 m) or two adjacent spandrel masonry beams (width 1,10 – 1,50 m); horizontal actions were applied to a closed system, with the aim of avoiding any action in the rest of the building. The test may be led until failure. The masonry panels were strengthened after failure and the test was repeated, in order to evaluate the effectiveness of the strengthening technique (Figure 13).
The combined use of the above mentioned techniques (Pile-Model and Opposite-Panels) allows the experimental evaluation of stiffness and strength parameters of masonry panels in existing buildings. A technical report with technical details for a procedure to manage shear-compression in situ test for opposite masonry panels has been prepared (Sassu 2008).

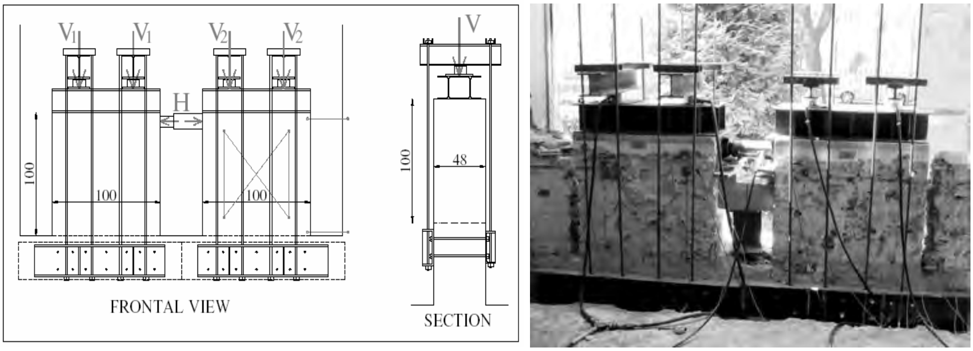

Figure 13. Opposite-Panels in-situ test: set-up and view of a sample tested.

UNICT worked on the development of draft specifications for a new in-situ shear-sliding test by means of flat jacks (Caliò 2009a), aimed at the evaluation of the mechanical parameters that characterize the shear behaviour of masonry (Figure 14). Some applications of the proposed test have been performed (Caliò 2009b). Furthermore experimental and numerical validation of the test has been considered (Caliò 2009c) .

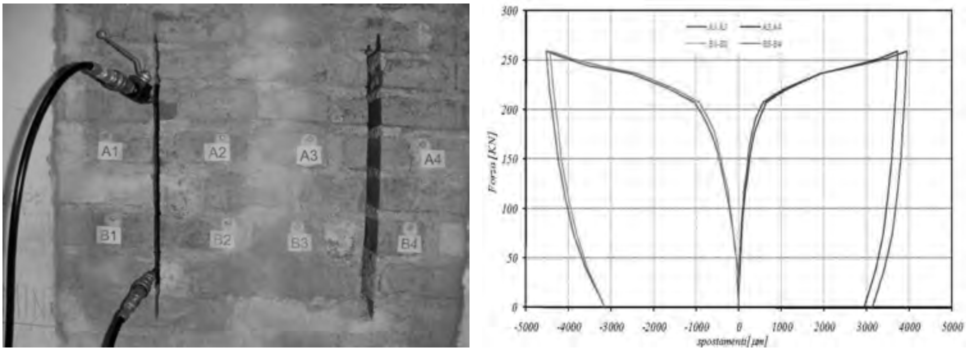

Figure 14. In-situ shear-sliding test by means of flat jacks.

POLIMI, in cooperation with UNIPD, studied the advantages and limits of the flat jack test (Binda et al., 2009b; Binda et al., 2009c), both for the measurement of the compressive stress and for the evaluation of the mechanical properties (in particular the Elastic modulus). The strain diffusion over the area subjected to compression was investigated, finding a proper method to evaluate the Elastic modulus and the transversal dilatation coefficient. Experimental tests for the calibration of single and double flat-jack test were carried out, in order to define the testing methodology. Several aspects were studied, particularly: the preparation and execution of tests, the acquisition and elaboration of data and the interpretation of results (Figure 15). The report will be directly available by the designer.
POLIMI studied also the possibility of a survey of the wall section by a small demolition of the external leaf of masonry (Figure 16).

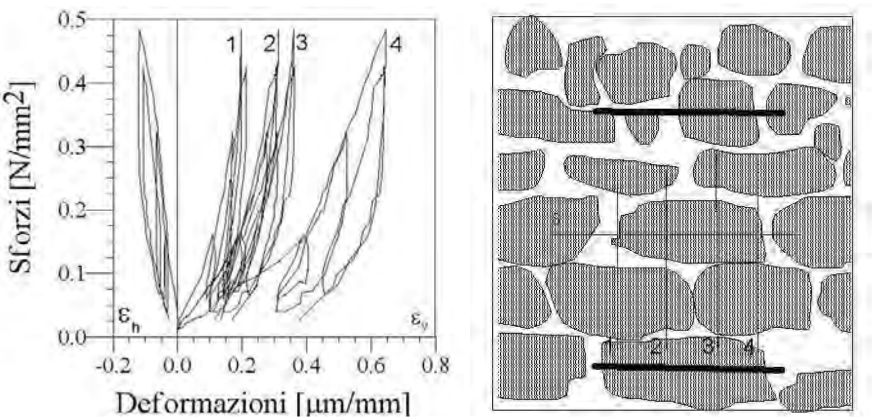

Figure 15. Application of double flat jack test on irregular masonry.

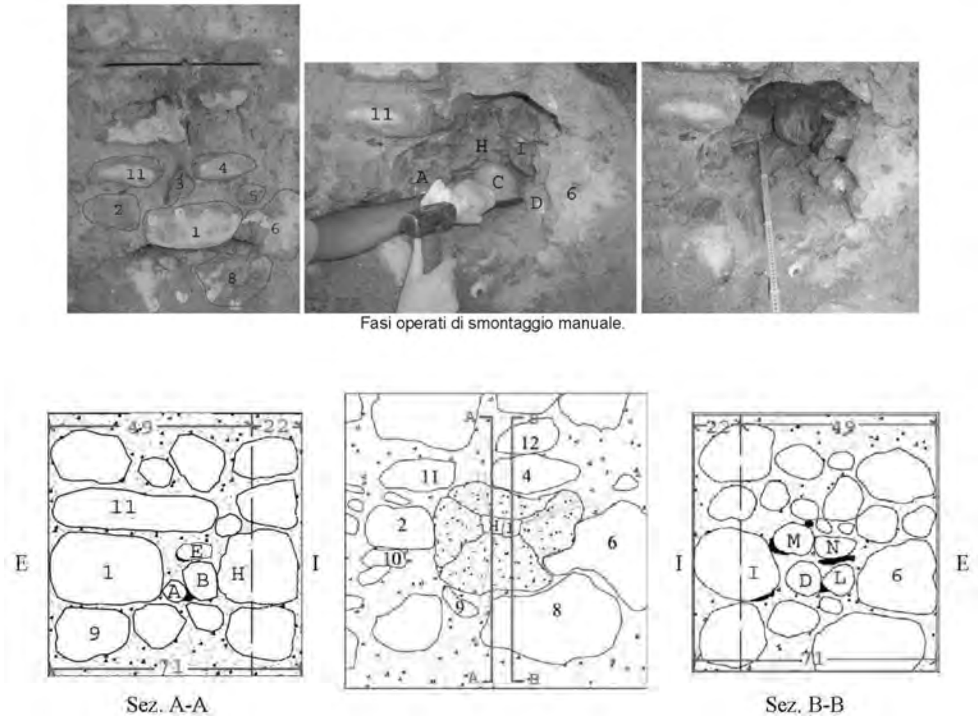

Figure 16. Survey of the wall section by a small demolition of the external masonry leaf.

UNIGE worked on the interpretation of diagonal compression tests, considering different masonry types. From the experimental point of view, the final results are two technical specifications, for the laboratory (Brignola et al. 2009a) and the in situ tests (Brignola et al. 2009b), aimed at illustrating the test setup and defining the relationship to use for a proper interpretation of the test results. In particular, the interpretation relations proposed by the well known specifications by RILEM and ASTM turned out to be wrong, as the consider a reference mean stress state at the centre of the panel, where failure occurs, which is not correct (Figure 17). The right interpretation was validated bay non linear numerical finite element analyses (Brignola et al.2009c).

Moreover, the possibility of using the diagonal compression test to obtain mean values of the cohesion and the friction coefficient of mortar joints of a given masonry has been investigated (Calderini et al. 2009). Traditionally the diagonal compression test is used to evaluate the tensile strength of masonry (diagonal cracking failure mode), to be adopted in the resistance criterion proposed by Turnšek and Čačovič (1971). However, in case of regular or quasi-periodic masonry, the adoption of resistance criterion based on the mechanical properties and dimensions of blocks and mortar joints (Mann and Müller 1980), such as cohesion, friction coefficient and interlocking, seems much more suitable. Although different experimental tests, like as the triplet test, are usually adopted to determine these "local" parameters of the mortar joints, their in situ application on actual masonry, where the mortar joints are not regularly arranged, poses many problems. On the other side, the results obtained by the triplet test would not be representative of the entire set of joints of the masonry. For these reasons, the possibility of using the diagonal compression test to obtain a "mean" evaluation of the cohesion and the friction coefficient of mortar joints of a given masonry seems very attractive. The proposed procedure is based on the combined use of diagonal compression tests performed by applying different values of a lateral compression on two opposite sides of the panel.

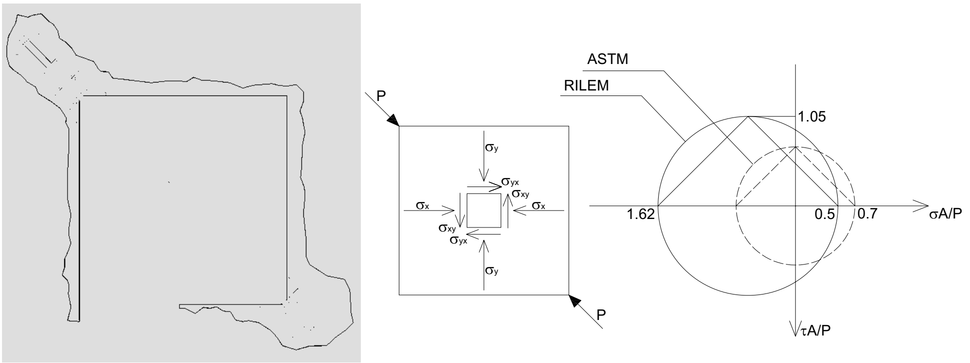

Figure 17. In-situ diagonal compression test.

4.4.4 Subtask 3b.4: Strengthening criteria and strengthening techniques for masonry

In this sub-task, several research units studied different strengthening techniques for masonry. For each technique examined a technical schedule has been arranged according to a standard format: 1) definition of the strengthening technique; 2) application; 3) materials employed; 4) practical execution; 5) criteria to verify its effectiveness.

The strengthening techniques which have been examined are synthetically described in the following.

On the basis of experimental campaigns performed, UNIPD produced in collaboration with POLIMI schedules on the use of mortar injections (Modena et al., 2009a) and of repointing (both plain and reinforced) (Modena et al., 2009b).

IUAV studied the possibility of strengthening masonry structures by means of soaking techniques (Doglioni et al. 2009e), in order to improve the cohesion between blocks in poor masonry; moreover, some specific aspects of connections among external walls and floors were studied referring to typical solutions adopted in Venetian civil buildings.

The idea is that the seismic behaviour of a masonry structure cannot be improved only by means of interventions contrasting the out-of plane mechanisms, as a diffused intervention aimed to increase the cohesive characteristics of masonry is necessary if the quality is beyond a threshold that is estimated critical. Experimental tests were performed considering soaking strengthening technique, first on cylindrical specimen and then on a specific case-study. Many different mix have been prepared and tested, composed by a range of specific materials, and once the more suitable for the use has been selected, the test has been conduced on some masonry panels in the case-study at Villa Tomitano in Feltre (BL). By means of sonic tests it was possible to verify the degree of efficiency and the attainment of a good level of adhesion at the mortar-stone interface in the strengthened panels.

UNIPG proposed and tested an innovative technique (named "Reticolatus") for the strengthening of irregular masonry (Borri et al. 2008). This reinforcement technique consists of a continuous mesh made of tiny steel cords embedded perfectly in the mortar joints after a first repointing, and anchored to the wall by means of galvanized steel eyebolts driven into the facing. A second repointing covers the cords and the heads of the eyebolts completely. This leads to genuine reinforced fair-face masonry in which, as already confirmed by the first experiments, the compression, shear and flexural strength are increased, effective transverse connection between the facings of the masonry due to the presence of the eyebolts and also the capacity to withstand tensile stresses. The activity is non-invasive and reversible, and is aimed at integrating the masonry rather than transforming it. It is also compatible with preservation of the material.

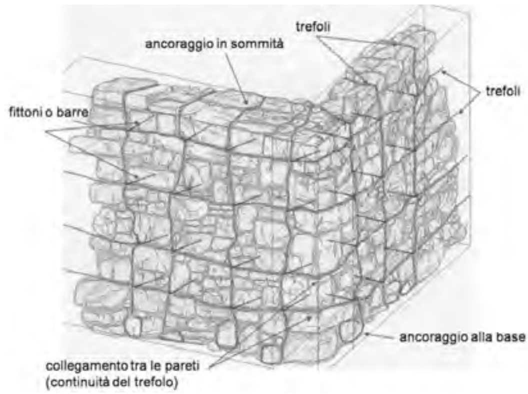

Figure 18. "Reticolatus", an innovative technique for repointing of irregular masonry.

To investigate the efficiency of this technique a series of experimental tests on two masonry panels are still in progress. The obtained results will be presented at the 12th National Conference on Seismic Engineering (ANIDIS) to be held in Bologna (Borri et al. 2009) and at the third National Conference on Mechanics of Masonry structures strengthened with FRP-Materials (Mu.Ri.Co) to be held in Venice. The proposed technique (of which a technical schedule containing all the execution specifications has been arranged by Borri et al. 2008) seems usable for strengthening interventions both on single masonry panels and globally on the overall structure (that is as a strengthening technique to improve the overall response of the building). A possible use of this technique is the case of boundary masonry walls, in particular surrounding walls of a city.

UNIGE worked on the strengthening intervention consisting in the transversal insertion of artificial elements, named "diatoni", with the aim of improving the connection between external leaves in two or three-leaf masonries. This intervention increases the compressive strength, as limits the instability of each external leaf, which appears as a swelling; it improves also the seismic out-of-plane behaviour, as after the intervention the wall may be considered as a rigid block. The effectiveness of the intervention was proved by numerical analyses and by experimental test on uniaxial compression. Specifications are given for the application of the technique in real masonry buildings.

Finally, UNIPD made an experimental program estimating the static and dynamic behaviour of multi-leaf stone masonry walls (Valluzzi et al. 2009). The walls were tested statically and dynamically in original and repaired/strengthened configuration, using or not strengthening by means of different techniques (grout injections, transversal ties, and a combination of both). Subsequently an experimental program of reinforcement by repointing was realised.

The experimental program on dynamic behaviour of multi-leaf stone masonry walls started with the construction of 8 specimens (two on initial conditions; two strengthened using transversal ties; two strengthened using injection; two using a combination of previous techniques). A test system was designed and realised to simulate the dynamic behaviour of last store of masonry building with wooden roof. To evaluate the influence on dynamic behaviour of injections, two small-scale building models were built: first model was realized and tested on original condition whilst the second one was strengthened before the experiment. Intentionally the collapse was avoided during the test on first model with the aim to test again this building after repairing using the same previous technique.

4.5 Task 4: Methods of seismic analysis

4.5.1 Criteria for the definition of the knowledge level

For the design of strengthening intervention of existing buildings, modern seismic codes gives importance to investigations and in particular to in-situ testing for the evaluation of material parameters. Depending on the type and number of tests that are used, different *Knowledge Levels* are defined and corresponding values of the *Confidence Factor* must be used in the safety checks.

The case of existing masonry buildings is particularly difficult, for two main factors: 1) very few in-situ testing procedures are available for a reliable evaluation of shear and compressive strength parameters; 2) as they are invasive and costly, they cannot be applied diffusely and it is impossible to get statistics of the parameter. The in-situ diagonal compressive test is necessary to reach the highest knowledge level, as it is the simplest test that gives the diagonal tensile strength of masonry, necessary for the shear criteria of masonry piers. For a medium knowledge level double flat jack tests must be done, even if they are not able to give the compressive strength. Other non-destructive test may be used (sonic tests), but are suggested by codes only as supplementary.

Within sub-task 3b.3 a lot of research was done, in order to: a) verify the applicability of the well known in-situ tests for masonry; b) define standard procedures for the test's execution and the interpretation of the results; c) proposed new testing procedures.

This sub-task was aimed to the definition of methodology for the design of the correct set of in-situ investigations. Many experimental tests were made, with the aim of establishing correlations between different testing procedures.

POLIMI and UNIPD focused the attention on double flat-jack tests, which appears to be the most useful testing procedure because it gives directly some mechanical parameter (elastic modulus, an estimate of compressive strength – initial cracking) with a rather low impact on the building. Some interesting results were obtained for the optimization of testing procedure (Binda et al., 2009c).

A complete set of non-destructive tests were performed by POLIMI on two building models: 1) the two story masonry building of the TREMA project, at ENEA-Casaccia (Binda et al., 2006); 2) the stone masonry models built by UNIPV, in the Eucentre laboratory, and some masonry specimens used for the material characterization (Binda et al., 2009e). Very interesting results were obtained by sonic pulse velocity tests, carried out in direct transmission on the two leaves masonry panels, before and after some mechanical characterization tests made by UNIPV.

4.5.2 Static and dynamic testing on masonry buildings

The tests on masonry structural systems (buildings) foreseen in the project were meant to provide a reference for numerical modelling activities to be carried within the project. However, due to the difficulties in planning large and complex experimental activities, great part of the experimental tests were carried in the last period of the triennium and several still have to be completed.

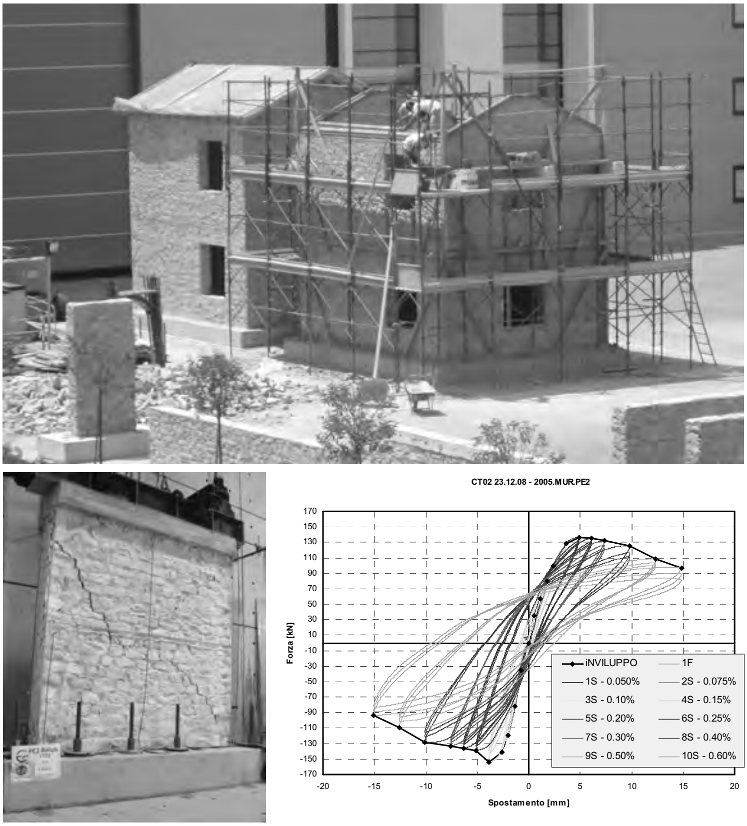

Figure 19. Construction of full scale stone masonry buildings (top) to be tested dynamically in Pavia and (bottom) shear-compression test on a double-leaf stone masonry wall (Galasco et al. 2009).

UNIPV has carried out numerical and experimental activities in support of the design, execution and interpretation of the shake table tests on three unreinforced stone masonry building prototypes to be tested at EUCENTRE. The project foresees the testing of three full scale buildings according to these configurations:

1. unstrengthened building, with wooden flexible floor/roof diaphragms,
2. strengthened building, with moderately stiffened diaphragms (second layer of wood planks crossly arranged to the existing one and fixed by means of steel studs)
3. strengthened building, with highly stiffened diaphragm

In the second and third configuration the floor-to-wall connections are strengthened trying to avoid local failures, taking into account also the quality of masonry (double leaf with no through connections). One of the objectives is to obtain sufficiently differentiated responses as the stiffness of the diaphragms changes, with a single cell, two-storey configuration. The different torsional response is obtained by means of an asymmetric plan (opening) configuration. The preliminary nonlinear dynamic analyses have confirmed that a large difference should be obtained between the flexible floor and the stiff floor torsional response. Two building specimens have been built (Figure 19) and the first shake table tests on specimen 1 should take place in early May 2009. Specimen 2 is characterized by a reinforced masonry ring beam at the top, and by a steel ring beam at the intermediate floor level. The

choice of the original form of the roof and floor diaphragms and of the stiffening techniques was made in collaboration with the research units involved in task 3, especially with UNITN.
The experimental activity carried out in support of the program is a comprehensive campaign of mechanical characterization of the stone masonry which constitutes the building prototypes (Galasco et al., 2009a and 2009b). Masonry specimens have been tested under cyclic compression, diagonal compression, and in-plane cyclic shear-compression tests were carried out on large masonry walls. In the latter tests, two levels of axial load and two slenderness ratios have been considered in order to obtain different failure modes (rocking/flexural and shear). Useful information for this masonry typology have been collected from the tests: strength parameters, displacement capacity and hysteretic energy dissipation properties. Attention has been devoted also to the damage evolution during the test, by recording position and width of cracks for increasing values of in-plane drift. A correlation can be established also between the mechanical parameters derived from simpler tests (simple compression, diagonal compression) and the behaviour of the larger wall panels. The results are made available to research groups interested in the numerical prediction of the shaking table tests via a website (www.eucentre.it/provemurature/), where also complete information regarding the building prototype can be obtained.
UNIPD has carried out an experimental campaign involving three shaking table tests on two 2/3 scaled building models (Mazzon et al., 2009). As in the programme carried out at Pavia, the goal of the tests is to provide reliable experimental information on the behaviour of stone masonry buildings subjected to different strengthening techniques (Figure 20). The first test was carried out on a model in original conditions, without strengthening; the second experiment was performed on a model strengthened before the test (SM), with the aim to evaluate the effectiveness of lime grout injections; the third test was carried out on the first model, after both local and extensive repairing interventions (grout injection and joint repointing, RM). The masonry is three-leaf (two external leaves with rubble core), and the horizontal diaphragms are traditional timber floors. All building models are provided with steel ties in two orthogonal directions to prevent out-of-plane failure of walls.
The experiment has shown the effectiveness, for the masonry typology considered, of lime grout injection both for preventive strengthening (to increase the seismic resistance of the building) and for post-damage repairing (to restore the mechanical properties of the structure). However, specimen SM could withstand, as expected, a higher PGA than specimen RM. The dynamic testing has also shown that lime grout injection does not significantly alter the stiffness properties of the buildings.
Further developments are under way regarding the energy response of the three different models, in addition to further more detailed analysis of the dynamic response.

Figure 20. Construction, detail and global view of a model stone masonry building tested on shaking table (Mazzon et al., 2009).

4.5.3 Masonry spandrel beams

The research on the role and behaviour of masonry spandrel beams in the seismic response of URM buildings has been undertaken by several research units using theoretical/numerical modelling or experimental testing.

Experimental programmes were started by UNINA and UNIPV, which have devised two different experimental methodologies for the cyclic in-plane testing of spandrels.

UNINA (Augenti and Romano, 2008, Augenti and Parisi, 2009c) has focused on regular tuff brick masonry, carrying out a first series of direct shear tests on masonry specimens (Figure 21), which have allowed to obtain new constitutive σ–ε and τ–γ, both in probabilistic form (characteristic values, median values and maximum likelihood values) and in analytical form, including $\tau(\sigma, \gamma)$ surfaces. The study has been carried out considering different masonry bonds.

Tests on single storey portal-type masonry specimens have been designed and planned for the evaluation of the spandrels contribution to in-plane response of perforated masonry walls (Augenti and Parisi 2009d). The test system and the first specimens have been built and testing is currently ongoing; the first results are very interesting and show the occurrence of both the typical damage mechanisms of spandrel beams: at first the flexural behaviour occurs, with tensile cracks at two opposite corners; then, diagonal shear cracking follows (Figure 21).

UNIPV has focused on irregular double-leaf stone masonry, using the same materials that have been used for the experiments carried out under subtask 4.2. The general objective of the work is to develop a new test setup for quasi-static cyclic tests on masonry spandrel elements and to perform tests to validate numerical modelling, in which the static boundary conditions of the spandrels are fully known and the essential static and kinematic parameters can be controlled. A new experimental setup for cyclic testing of full scale masonry spandrel beams has been designed, for which numerical simulations of the non linear behaviour of spandrels have been necessary, in order to define all the details of the testing apparatus (Graziotti et al., 2009). The setup allows to introduce also controlled horizontal precompression in the spandrel, such as due to the presence of horizontal steel ties. Two specimens of masonry spandrels have been built with the same materials and technique used for the building prototypes that will be tested on the shake table, and the tests are scheduled to be carried out within the Spring of 2009 .

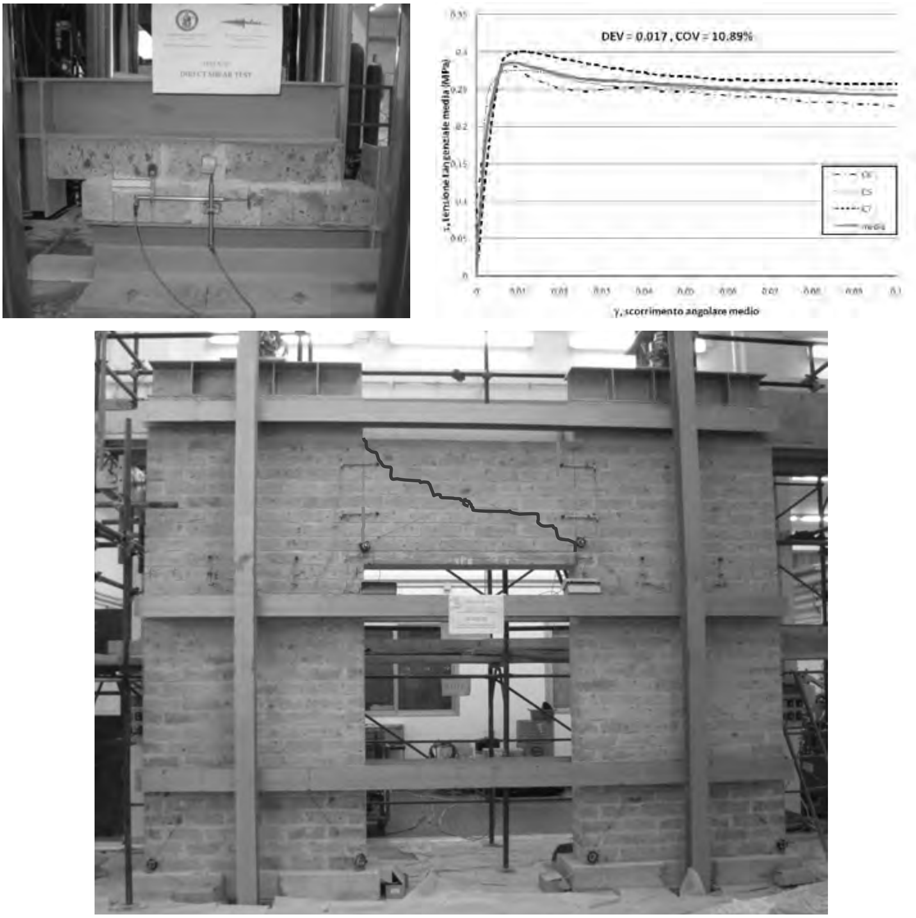

Figure 21. Direct shear test on mortar joints in tuff masonry (Augenti and Parisi, 2009c). Tests on single storey masonry portal for the evaluation of the spandrels.

Preliminary numerical studies on the structural behaviour of masonry spandrel in the response of masonry walls subjected to in-plane seismic loads have been carried out by UNIFI (Betti et al., 2008). The research aimed to verify the formulae of shear and flexural strength of masonry spandrels which are given in the recent Italian Standards. Seismic pushover analyses have been carried out using finite element models of unreinforced walls and strengthened walls introducing reinforced concrete (RC) beams at the floor levels. Results obtained for the unreinforced and the strengthened walls are compared with equations for shear and flexural strength provided in the code. The comparison between the finite element model results and the code formulae show that the latter tend to give in some cases higher strength predictions

especially for flexural strength, in particular for the case in which the axial force has not been determined by the structural analysis. On the other hand, the code formulae would underestimate the spandrel strength when no axial force is present.
As a further analytical and numerical contribution on the subject, UNIGE formulated a resistance criterion finalized to interpretation of the flexural behaviour of URM spandrel beams (Cattari and Lagomarsino, 2008). The formulation proposed is founded on the assumption that the response as "equivalent strut" of spandrel may also occur by virtue of the interlocking phenomena which can be originated at the interface between its end-sections and the contiguous masonry: as a consequence, it can define an "equivalent" tensile strength f_{tu}, which properly characterizes the spandrel element, not the masonry material. A set of parametrical non linear analyses, using the non linear constitutive law proposed by Calderini and Lagomarsino (2008), has been performed in order to validate the proposal (Figure 22). Finally, some applications on complex masonry walls have been presented in order to assess the effects on the global response, which derive from varying the hypothesis adopted for the spandrel.
The numerical researches confirm therefore that code formulae for spandrel strength could be improved, although still the results given by the numerical models need to be compared with the experimental results which will be available in the next future. It is worth to mention under this regard that recently some interesting experimental tests have been carried out on clay brick masonry spandrel elements at the University of Trieste (Gattesco et al., 2008) without the financial support of Reluis.

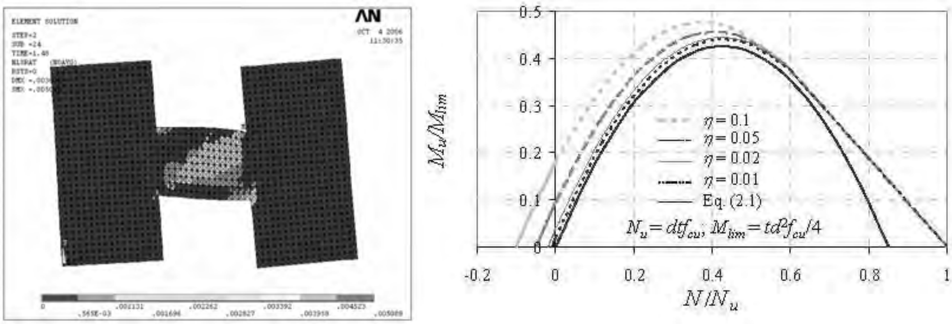

Figure 22. Resistance criterion for the flexural behaviour of URM spandrel beams.

4.5.4 Validation and comparison of different modelling methods

Main goals of this sub-task consisted of:

- the development of suitable models for the evaluation of the global seismic response of masonry buildings; the verification and improvement (if necessary) of the methods of analysis proposed by seismic codes
- the comparison among the results achievable by adopting different modelling approaches.

The research units which were mainly involved are: UNIPV, UNIBAS, UNIGE, UNIPD, UNIFI, UNICT, UNIPI, ROMA3.
Particularly relevant to the aims of this sub-task has been the analysis of selected "reference" structural configurations. Regarding this, the main activity has been addressed to the blind prediction analyses of the TREMA- Tre.Re.M prototype building experimental response.

The TREMA project (Technologies for the Reduction of seismic Effects on Architectural Manufactures) was a project partially funded by the Italian Ministry for Research. The project has been coordinated by ENEA. Within the project an experimental investigation on shaking table of a 2:3 scaled masonry building models was carried out. Two identical test structures were built, two storey high, with irregular tuff masonry, limestone mortar and wooden floors. This type of construction is very common in Central and South Italy Apennines zones. A sequence of increasing intensity of natural input records in the X, Y and Z directions have been used in the dynamic tests. In the first model a system of seismic isolators has been inserted in order to experiment the effectiveness of isolation for URM buildings. Subsequently the isolators were removed and the same structure, which had not been damaged significantly, was upgraded with the CAM system (a system of 3D stainless steel ties) and tested up to near collapse. The second model was tested without any upgrading up to near collapse. The shake table tests were completed on December 7, 2007. The results of the experimental tests have been useful to validate the static and dynamic models for the prediction of the seismic behaviour of masonry building, fixed at the base, isolated, and with or without upgrading systems. Thus TREMA project provided useful results non only for the 1st Research Line of Reluis “Estimation and Reduction of Masonry Building Vulnerability” (aimed to evaluate the seismic response of existing masonry buildings with irregular stone layout and flexible floors) but also for the 7th Research line of Reluis “Technologies for Seismic Isolation and Control of Civil Structures and Infrastructures” (aimed to investigate the seismic behaviour of seismically isolated masonry buildings).
With reference to the blind prediction analyses carried out, several modelling approaches and analysis types have been adopted. In particular nonlinear dynamic analyses have been performed by the following research units: UNIPV by adopting equivalent frame models (by means of the software Tremuri and SAM II choosing to model masonry panels as macro-element models); UNIGE by adopting both the finite element method (ANSYS rel.8) and the equivalent frame models (by using the software Tremuri choosing to model masonry panels as nonlinear beams); UNICT by adopting macro-element models (software 3DMACRO); ROMA3, UNIFI and UNIPI by adopting finite element models (respectively UNIPI used the software Straus 7 and UNIFI the ANSYS one). Non linear static analyses have been performed by the following research units: UNIPV with macro-element models (by adopting the software SAM II); UNIGE with the same models adopted for the nonlinear dynamic analyses; UNIPD with finite element models (DIANA software); UNICT with both finite element models (LUSAS and ADINA software) and the macro-element one adopted for the nonlinear dynamic analyses; UNIPI by adopting the same model adopted for the non linear dynamic analyses. Local mechanisms have been performed by: UNIPV, UNIGE, UNIBAS. Moreover UNIPD performed analysis via vulnerability index (VULNUS) and ROMA3 nonlinear Distinct Element analyses.
All the results obtained related to the blind prediction analyses have been collected in a final report (Caliò, 2009). It is worth noting that the global failure mode occurred during the test has been ascribable to the occurrence of out-of-plane mechanisms, as a consequence the predictions provided by the models which have assumed that the ultimate capacity is mainly related to the activation of in-plane failure modes cannot be validated.
After the end of the experimental program, from some research units, the extensive modelling carried out during the blind prediction have been followed by a comparison with experimental results. Thus, some further elaborations or updating of the results have been obtained on the basis of the actual signals recorded during the tests (which have been quite different from those used in the blind tests). UNIGE provided an updating of the evaluation of the collapse multipliers in order to verify the reliability of the procedure (Curti and Resemini 2009).

UNIFI (Vignoli et al. 2007) analysed the building case study with respect to the approaches of pushover analysis and Incremental Dynamic Analysis (IDA) in order to compare the classical analysis methods for seismic assessment of masonry buildings with simplified approaches. To this aim two different numerical instruments has been adopted by UNIFI: a) a commercial finite element code (ANSYS); b) a macro-element approach (under Matlab).
In addition to the activities related to the numerical analyses performed for the experimental campaign of TREMA Project, the following main other activities have been carried out.
UNIPV in collaboration with UNIGE worked on defining a procedure for pushover analysis particularly suitable in case of masonry buildings with flexible floors. In particular a new displacement-based algorithm for the adaptive pushover analysis of masonry walls and buildings was developed (Galasco et al. 2007): the load pattern, in this case, is directly derived, step-by-step, by the current deformed shape evaluated during the pushover analysis.
UNIGE provided a critical review of resistance criteria proposed in the literature for verifying the response of masonry piers (Calderini at al. 2009). The research has been carried out both by performing parametric finite element analyses both in linear and non linear filed (by adopting the non linear constitutive law formulated by Calderini and Lagomarsino 2008) and by analysing available experimental data. In discussions regarding the examined models, one of the crucial points was to choose between the suitable alternative failure criteria proposed by Turnšek and Čačovič (1971) and Mann and Müller (1980). In fact the first appears to be more coherent for describing the response of masonry characterized by an isotropic behaviour; to the contrary, the second seems more coherent for interpreting the behaviour of regular masonry with mortar joints which have limited resistance in comparison to blocks. Measurements for estimating the level of anisotropy of the masonry can be traced to the following factors: the typology of masonry texture (masonry classified as regular, nearly periodic, or chaotic); the ratios between mechanical properties of resistance and deformation, which characterizes the mortar joints and the blocks.
UNICT addressed its research activity to the development of a new computer code for the simulation of the seismic response of masonry and mixed masonry and reinforced concrete buildings by means of a macro-element approach. In particular UNICT worked on the following issues: the formulation and applications of a three-dimensional macro-element able to describe the in-plane and the out-of-plane behaviour of masonry panels (Caliò et al. 2008); the formulation and application of a new three-dimensional macro-element for the simulation of the seismic response of masonry elements with curved geometry (Caliò et al. 2009).
ROMA3 (Amorosi et al. 2007) formulated and implemented in a FE code (Opensees) a macroscopic model for in-plane analysis of periodic masonry. This model has been used for simulating the experimental tests up to collapse, conducted by the R.U. of Pavia and Reggio Calabria, on dry block masonry specimens on a tilting table, providing encouraging results.

5 DISCUSSION

The subject of seismic vulnerability of masonry buildings is undoubtedly vast; many problems are still open, especially at the operational and application level, notwithstanding the amount of research carried out internationally and particularly in Italy, where a remarkable expertise exists in the field of built heritage. The programme of Reluis Line 1 had been designed with quite ambitious objectives, with the intent of tackling a wide spectrum of topics, rather than focusing on deepening few selected significant issues. This had been dictated by the fact that the new national seismic code, still in evolution, on the one hand had introduced in the professional practice new methods of analysis and of performance

assessment that in some cases were still needing a systematic validation, on the other hand had avoided to give provisions or guidelines because of a lack of knowledge.
To pursue the objectives, many research groups were involved (Line 1 is, within the Reluis programme, the one with the highest number of research units involved). This choice was somehow bold, but the aim was to collect and join as many expertises as possible, from the Engineering and from the Architecture fields, ranging from the more theoretical aspects of structural modelling to the more applied aspects of in-situ testing, surveying and technological aspects of strengthening methods. Obviously, experimental research was privileged since it is an indispensable basis for the development of knowledge and for any model or for the validation of any strengthening technique.
The project was thus articulated in tasks and subtasks, each with a coordinator; however, in each subtask several research units were operating. This choice has produced appreciable results and many deliverables; the expertise and the views of the different research groups could be put in comparison and such interaction was a basis for the development of the activities. However, the coordination was more difficult, such as was the synthesis work. Not always the final deliverables are a product of a general consensus as would be ideally desired, but this is unavoidable, especially in the fields in which the knowledge is less consolidated.
Undoubtedly, a critical aspect of the project has been the delay with which several important experimental activities could be started, in particular some of the large scale tests. This fact is somehow understandable, when it is considered that this first triennial Reluis framework programme had been activated after a long period in which the Italian research in seismic engineering had been addressed to other topics, in general with limited financial investments in experimental activities. Some experimental campaigns were completed just before the end of the triennium and others, quite meaningful, have just started and are being completed. The final reports give a first description of the results and a series of preliminary interpretation, but the results need further elaborations and interpretations both from the research groups that have directly carried out the experiments and from the entire scientific community.
In summary, however, it is believed that the most of the objectives of the project have been reached. Objectives that were not reached within the triennium are here briefly described.

- A small progress was achieved in task 4.1 (Criteria for the definition of knowledge levels), which had as a fundamental prerequisite the updated collection of available experimental data including the results of the experimental researches of the other tasks, and their statistical analysis. The conclusion of the experimental activities and the processing of the data will allow to carry out the necessary probabilistic studies for a better definition of the uncertainties as a function of the quality and quantity of in-situ collected data.
- The activities carried out on methods and models for structural analysis were numerous and of high quality, both as regards the mixed masonry-r.c. buildings and the traditional masonry buildings; however many of the problems that were open at the beginning are still open, since even the interesting results that have been achieved do not allow yet to draw final conclusions. As an example, among these open problems is the well known issue of the choice between a linear multimodal response spectrum analysis and a nonlinear static (pushover) analysis, in the case of an irregular masonry building.

In view of a new triennial framework project on masonry buildings, in first instance a reconsideration of the tasks and subtasks should be made. It is believed that, although partial overlapping could not be avoided among the different subtasks, especially in a such a complex and articulated project, the structure of the project has been in general effective. Possibly not fully effective was the choice to treat the strengthening issues in three separate tasks: 1) building aggregates; 3a) floors, vaults and roof; 3b) masonry vertical structures. In the future the strengthening strategies and techniques could be collected in a single task, in which the general strategies are studied and the different specific problems are investigated, such as those related to: a) connections and structural joints, b) strengthening and stiffening of floor and roof diaphragms and of vaulted structures; c) strengthening of vertical structures. Such topics could be dealt with reference to the different building typologies: a) isolated masonry building b) isolated mixed masonry-r.c. building c) structural unit or subsystem within an aggregate.

Similarly, the modelling issues were subdivided with reference to the construction typology (buildings in aggregate, mixed buildings, isolated masonry buildings). In the case of buildings in aggregate, most of the work has dealt with out-of-plane local mechanisms; some of the issues related to in-plane response of walls have made use of the models studied under task 4. Also in task 2, dedicated to mixed masonry-r.c. structures a sub task dedicated to modelling was present, which in great part was drawing from the models studied in task 4. In this last task the issues related to local out-of-plane mechanisms was not treated since they were studied in detail in task 1. In the future, the definition of a single task dedicated to modelling issues, articulated in suitable subtopics, could be more appropriate.

All in all, in the past triennium a distinction of tasks based on the building typology was thought to be fundamental, to better understand the peculiarities of each typology and to keep the project closer to what the “real objects” are. However, now that much work has been carried out and many results have been achieved through typological studies and surveys, a clearer picture is available, and a more rational framework can be thought, in which some issues can be studied more rigorously and systematically.

6 VISION AND DEVELOPMENTS

On the basis of what discussed above, a possible structure for a future coordinated project could be as follows:

1) *In-situ and laboratory testing*: development of in-situ testing or surveying techniques and collection of data; laboratory testing on materials, structural components, subsystems and systems (building prototype), including shake table testing.
2) *Structural modelling and performance assessment methods*: local response/collapse mechanisms; global response of buildings or subsystems; analysis of built units or subsystems interacting with other structures.
3) *Strategies and techniques for seismic strengthening*: detection of the sources of vulnerability, definition of the strategies, load paths and distribution of actions; structural connections; strengthening and consolidation of masonry; strengthening/stiffening of floors, roofs, vaults.

However, given the way in which the results of the past triennium have been presented above, in the following the vision for future developments will be discussed making reference to the same framework. Some of the themes will consist of the completion or development of themes of the past triennium, others will consist of new topics.

Building aggregates in historical centres

Significant improvements were achieved in the project regarding the capability of analysing the out-of plane collapse mechanisms, but reliable procedures for the evaluation and seismic verification are still needed.

Some dynamic testing on two-sided and one-sided rocking were made, both in free vibration and under seismic actions (shake table); however, more configurations should be considered,

in order to get comprehensive experimental results, useful for validation and calibration of theoretical models. In particular, the influence of different types of connection with transversal walls should be investigated as well as the behaviour of three leaves masonry walls, with a poor internal connection and the possibility of cyclic degradation and crushing at the corners.
The interpretation of these experimental tests will be the base for the development of theoretical models. It is well known that the out-of-plane response to earthquake excitation is very sensitive to many parameters (imperfections, direction of the motion in one-sided-rocking, sometimes a reduction of the displacement demand is observed increasing the input motion, etc.), but the experimental results of this triennium showed that in the case of actual masonry walls, far from the hypothesis of rigid block, dissipation and deformability tend to regularize the behaviour. More research should be made on singling out intensity measures of the earthquake excitation which are significant for the prevision of overturning; the correlation of these parameters with those characterizing the seismic action in codes (acceleration and displacement response spectra) is necessary, in order to define procedures which can be implemented in the engineering practise.
An important topic studied in this project was the definition of demand spectra at different levels of the building, because local mechanisms usually occur at higher levels were the motion is amplified by the filtering effect of the structure. However, no experimental validation of the proposed theoretical models is available. Some information will be available from the forthcoming shaking table tests in Pavia, but specifically aimed dynamic testing should be very useful.
More research is needed also on the interaction between adjoining building units in complex aggregates, in order to define how it is possible to analyse the single building taking into account the interactions and its position in the aggregate.
Finally, the survey and classification of building aggregates in historical centre made in this project can be considered exhaustive; however, some applications of the procedure proposed for the vulnerability analysis and the design of upgrade interventions should be necessary, considering some case studies.

Mixed masonry and reinforced concrete structures

The work carried out during the triennium has allowed to gain knowledge on some types of mixed structures, but the overall picture is far from being complete, especially regarding the dual systems that have been created after interventions on existing buildings or have been initially built as dual systems (e.g. masonry buildings with elevator/stair r.c. core) .
Besides the additional survey work and data collection to complete a possible reference catalogue of mixed typologies over the national territory, experimental work is needed to create a reference for theoretical modelling and numerical simulations. To this end, experimental testing on mixed structural systems is lacking and mostly needed. In this first triennium no extensive testing activities had been foreseen on the topic. Besides the pursuance of a continuous improvement of the computational tools and models (especially for the confined masonry typology and for the simulation of local interactions), numerical analyses must also be extended to further building configurations to obtain more references to better understand the potential sources of vulnerability and to guide the definition of the procedures that should be suggested at the code level to the designer, especially as regards the modelling and safety check criteria. Such better understanding is believed to be a priority for the development of suitable strengthening strategies and techniques.

Floors, roofs and vaults

The testing setups that have been developed during the triennium and that allowed to obtain fundamental and new results on selected typologies of structural components, subsystems, joints and strengthening techniques can and should be exploited for further testing on different typologies. This is especially relevant for floor diaphragms (and strengthening/ stiffening techniques and detailing), carpentry joints, diaphragm-to-wall connections. Also, engineering models still should be developed for several of the tested systems basing on the data already obtained. However, what really seems to be necessary as a new development are the criteria and methods to decide if and to what extent the diaphragms need to/can be stiffened, in relation with the properties of the vertical structures. Such progress can be achieved with integrated and coordinate analytical/numerical modelling and testing (possibly dynamic) on structural systems, and the completion of the ongoing shake table experiments will be an excellent startpoint.
In the field of vaulted structures, it is felt that dynamic experimentation and modelling is still an open exploration field, especially as regards the effectiveness of strengthening techniques, whether of the traditional type or more innovative. The current knowledge, as testified by the deliverables of the triennium, allows to understand why and with what possible means a vaulted structure can/should be strengthened, but still the confidence in the analysis of the seismic behaviour of vaulted structures before and after the strengthening intervention is spoiled by the scarcity of reference dynamic experimental tests.

Material properties of masonry

The activity carried out in the triennium was useful to investigate the capability of different in-situ tests to evaluate the mechanical parameters of masonry and to improve the knowledge of the constructive features (e.g.: connection between masonry leaves). Some new testing procedures were proposed (different types of penetrometers for mortar, vertical flat jack for shear characterization of masonry). Much more research is needed in this field, it is fundamental to have at disposal reliable techniques with a low impact on the building, which can be applied extensively in a building, due to the inhomogeneity and variability of masonry.
A systematic collection of in-situ experimental data on masonry properties should be advisable, in order to validate and improve the reference table which is present in the current Instructions to the National Technical Building Code (Norme Tecniche per le Costruzioni, NTC, 2008).

Strengthening of masonry buildings

Another important result of the project is the classification of strengthening techniques, with specific technical annexes defining their application, the materials employed and the criteria to verify their effectiveness. Some additional research is needed in particular for the experimental validation of masonry strengthening, considering different masonry types.
The Instructions to the NTC propose a table of coefficients for the estimation of the improvement of material properties, depending on the different techniques (mortar injection, transversal connection, etc.). Further laboratory and in-situ tests are necessary to validate these coefficient and to develop simple theoretical models to evaluate the increase of material properties depending on the characteristics of masonry.
Another topic which should be further investigated, expanding the work started by the University of Padua, is multi-leaf masonry. In many cases masonry walls are made by external leaves which are very different from the internal core; sometimes a new leaf is added in a subsequent phase to a pre-existing wall. The equivalent material properties, necessary for

modelling and seismic verifications, cannot be defined on average, and simple but rigorous homogenization procedures should be developed in order to obtain the parameters from those of the single leaves.

Criteria for the definition of the knowledge levels

The activity carried out in the triennium has been useful to develop and improve in-situ testing techniques and their use and interpretation. Still, much work needs to be carried out regarding the definition of the following elements that should guide the decisions in the design process regarding the level of knowledge to be acquired for the safety assessment: uncertainties regarding the data that are collected from survey and in-situ testing, uncertainties due to the model/methods of structural analysis that are used in the safety assessment, uncertainties in the definition of the seismic input. All these sources of uncertainty must be considered for a consistent seismic assessment and for an optimal allocation of resources.
The research carried out in the triennium has been useful to make a clearer picture regarding the material properties and their dispersion, although still the experimental database is far from being sufficient and its improvement is still a goal to be pursued. To evaluate if a rational definition of confidence factors is possible, a more complete reliability study must be carried out with analytical and numerical methods, similarly to what is being carried out for other structural typologies.

Spandrel elements

The knowledge of behaviour and role of masonry spandrels is making a significant improvement with the experiments that were designed and started in the triennium and that are still ongoing. The natural development of these activities is the extension of the tests (which are still quite limited in number) to a larger combination of masonry typologies, geometric/mechanical parameters, strengthening techniques, and improvement/updating of theoretical/numerical models at all levels, including code formulae.

Modelling and methods of analysis

Modelling of local mechanisms has been already discussed in the section dedicated to building aggregates in historical centres. Some issues regarding global modelling and methods of analysis and assessment have been already briefly discussed in chapter 5 of this paper. To further expand on these issues, the need for reliable but practical analysis tools and criteria that can be used to assess irregular masonry buildings and/or with flexible diaphragms is strongly felt. First of all, there is a need of including non linear behaviour of flexible floors in the 3D equivalent frame model, considering that experimental tests give us the drift capacity of these elements. Moreover, the increasing amount (yet not sufficient) of experimental data regarding deformation capacity of masonry structural components can be a startpoint for the introduction of deformation-based assessment criteria, which can be applied both to the case of linear analysis and nonlinear analysis. The use of linear analysis in a strength-based safety check is unsatisfactory for a series of reasons which have mainly to do with the impossibility of providing a unique behaviour factor that is valid for all types of buildings. Being nonlinear dynamic analysis still not an option for the practitioners, improvements of the nonlinear static procedures is also needed to handle the cases in which single-mode response is deemed to provide inaccurate results.
Finally the improvement and refinement of computational tools must be pursued since in many cases the available tools are still insufficient to capture the complexity of the behaviour

of existing masonry buildings. The interaction of theoretical and experimental research is, as always, a key element for the progress in this field.

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ASSESSMENT AND REDUCTION OF THE VULNERABILITY OF EXISTING REINFORCED CONCRETE BUILDINGS

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1 INTRODUCTION

Research Line 2 focuses on the assessment of the seismic performance of existing reinforced concrete buildings, covering a wide spectrum of problems, each one treated within a single Task. These aspects span from those related to the preliminary knowledge phase, to the use of nonlinear assessment methods, while placing emphasis on peculiar modeling problems, such as those related to the presence of stairs, infills, beam-column joints, and biaxial behavior of the elements. The Research Line is also devoted to the study of mixed-type (masonry/RC) buildings and of prefabricated industrial buildings.
The following Task list collects and organizes the entire scientific activity:

1. MND: Non-Destructive Methods for the Knowledge of Existing Structures
2. FC: Calibration of Confidence Factors
3. IRREG: Assessment of the Nonlinear Behavior of Buildings, with Emphasis on Irregular Ones
4. MIX: Assessment and Strengthening of Mixed-type (Masonry/RC) Buildings
5. TAMP: Influence of Infills on Structural Response
6. SCALE: Behavior and Strengthening of Stairs
7. NODI: Behavior and Strengthening of Beam-Column Joints
8. BIAX: Behavior and Strengthening of Columns under Combined Axial Load and Biaxial Bending and Shear
9. PREFAB: Behavior and Strengthening of Prefabricated Industrial Structures

For the sake of clarity of exposition, given the large variety of different subjects, the structure of the paper, in each of its sections, follows the Task organization above.

1.1 MND: Non-Destructive Methods for the Knowledge of Existing Structures

Task MND specifically focuses on the knowledge of the constituent material properties of Reinforced Concrete (RC) existing structures. Particularly, Task MND is devoted to the estimation of the in-situ concrete strength by using destructive and non destructive methods. Data on material properties from several in-situ and laboratory investigations were collected and analysed with the major objective of defining reliable as well as not very expensive procedures and criteria for the estimation of the in-situ concrete strength. Further, methods for the treatment of the uncertainty that characterizes experimental data obtained through in-situ and laboratory investigations were analysed and some theoretical simulations of the influence of material properties on the seismic capacity of existing buildings were carried out.

1.2 FC: Calibration of Confidence Factors

A fundamental phase in the assessment of existing reinforced concrete buildings and in their strengthening design is the knowledge process that one has to follow to acquire the necessary information. This is based on the collection of different kinds of information regarding: a) the structural system configuration, b) the materials strength, c) the reinforcing steel details, and d) the conditions of the structural elements.

The Italian Code (OPCM 3431, 03-05-05, Annex 2) as well as the most advanced International Codes (FEMA 356, EC8 Part 3) specifies data collection procedures about the configuration of the structural system, as well as material strength and condition of the structural elements comprising the building, and ensuing Confidence Factors (CF) to apply to the mean values materials properties, based on the quantity and quality of the information gathered (the so called Knowledge Level). In the current approach, the CFs are given through tables.

Aim of the Task has been: a) evaluation of CF effects on the assessment of buildings seismic performances; b) development of a procedure for the evaluation of concrete and steel strength, to be reliably used in assessing members capacity; c) new definition of CF, evaluated by a closed-form equation as a function of number, kind and reliability of each testing method employed, and of the reliability of prior information.

1.3 IRREG: Assessment of the Nonlinear Behavior of Buildings, with Emphasis on Irregular Ones

Task IRREG deals with problems related to the definition of plan and elevation irregularity and the effects of irregularity on the structural behavior and its prediction though different methods of analysis provided by the current design codes.

More specifically, the main focuses of this tasks are:

a. Study of the definition and effects of plan irregularity on the response of RC buildings up to the Ultimate and Collapse limit states;
b. Comparison and calibration of different linear and nonlinear methods for reinforced concrete structural members, with emphasis on pushover and nonlinear dynamic analyses;
c. Comparison between research-oriented and professional-oriented structural analysis software, in order to identify analytical tools that satisfy both modeling precision and computational speed.

1.4 MIX: Assessment and Strengthening of Mixed-type (Masonry/RC) Buildings

The work has been developed on the evaluation of the seismic response of mixed-type buildings behaving as parallel systems, with regard to both local (interaction between masonry and RC elements) and global features, by performing a series of non linear numerical analyses.

The research activity has been focused on:

a. the classification of the main geometrical characteristics of such kind of buildings and the study of their response – behaving as parallel systems – subjected to horizontal forces;
b. the problems concerning the modelling of mixed-type buildings and on the distribution of the seismic action between masonry and reinforced concrete elements by performing a series of numerical analyses obtaining the capacity curves of individuals resistance elements and the building as a whole.

1.5 TAMP: Influence of Infills on Structural Response

Several theoretical and numerical analyses, and, above all, the damage distributions on buildings that have suffered an earthquake show that masonry infills can modify substantially the expected seismic response of framed structures although special devices connecting the infill panels with the surrounding meshes of frame are not applied.

In spite of that, most seismic codes (the more recent too) give some provisions in order that the resisting elements of frame bear the unfavourable effects of a non-uniform infill distribution in plan or/and in elevation, but they do not suggest any procedure to quantify these effects or the favourable lateral stiffness and resistance contributions that infills give when they are uniformly located.

This gap occurs since the influence of the masonry infills on the seismic response of framed buildings is a still open research topic, where univocal and general results do have not been achieved. The present study refers to this subject with the following main objectives:

- mechanical characterization of the masonry infill kinds that are commonly utilized in the Italian country by means of experimental tests on their components (resisting elements and mortar) and masonry samples; - experimental investigation on infilled meshes of RC frames, with the aim of calibrating a pin-jointed equivalent diagonal strut model.

A companion study has also been devoted to verifying the influence of the infills on the seismic response of RC framed structures; for this purpose, shaking table tests on a 1:2 scaled 3D building and numerical nonlinear analyses of multi-storey frames have been carried out.

1.6 SCALE: Behavior and Strengthening of Stairs

The main objectives of this Task are the following: Identification of the main stairs typologies used in the past construction practices; Numerical investigation of the influence of the stair substructure on the structural seismic response. In particular, both global and local seismic performance have to be investigated with reference to frame and stairs members connections; Construction of building sub-assemblages, including a stair substructure, for experimental tests execution, specifically targeted at understanding their seismic performance.

1.7 NODI: Behavior and Strengthening of Beam-Column Joints

This Task aims at investigating the experimental behaviour of RC structural members, particularly beam-column joints without or with strengthening, thus providing a contribution to a more reliable evaluation of the seismic vulnerability of RC existing buildings. In particular of great interest is the understanding and the validation of capacity models relevant to the joint panel zone in beam-column sub-assemblages reported in the literature and in seismic codes. Further, there is a need of knowledge in the field of strengthening and retrofit systems that can be used taking into account the actual geometry of joints: e.g. presence of slab and other framing elements that could prevent an effective arrangement of the retrofitting system. To this purposes, wide bibliographic research on the experimental investigations on beam-column joints and on different repairing/strengthening techniques as well as experimental researches on different joint specimens have been carried out.

1.8 BIAX: Behavior and Strengthening of Columns under Combined Axial Load and Biaxial Bending and Shear

Modern approach to safety assessment of existing reinforced concrete structures and design of strengthening interventions, in particular those aimed at increasing ductility of columns, are based on enhanced and complex methods for structural analysis (seismic demand), but also on the availability of data concerning performances of members at failure (seismic capacity).

On the other hand, common constructions are not necessarily affected by regular shapes and/or regular distribution of seismic resistant substructures, so that seismic actions result in complex deformation paths on columns and in general on compressed resisting members.
This is the reason why Task BIAX research activity has been devoted to provide an insight on the response of r.c. members subjected to biaxial bending and axial load. In particular, some aspects have been analysed in detail. In compliance with the overall objectives of the research programme as a whole, Task BIAX duties were the definition of a set of reliable and well documented data and procedures concerning: (a) rotation capacity of r.c. members subjected to generalised bending and axial forces; (b) development of simplified methods of analysis for general r.c. cross sections for design safety checks; (c) development of refined methods for assessment of generalized moment-curvature relationships of cross sections; (d) extension of results to r.c. members reinforced with FRP materials.

1.9 PREFAB: Behavior and Strengthening of Prefabricated Industrial Structures

The assessment and reduction of seismic vulnerability of a widespread category of precast structures typically used for industrial buildings is a topic of high importance. The production of these structures starts since from the years ‘50s of last century with elements and construction solutions which had a relevant evolution through the subsequent times. It is a social important interest to know the state of this wide building heritage with respect to its seismic vulnerability so to address, following rational criteria, possible interventions of upgrading of inadequate structures.

2 BACKGROUND AND MOTIVATION

The essential motivation for each Task stems from recognizing some gaps in the code, related to certain procedural and methodological aspects in the seismic assessment of existing buildings.
Specifically, for Task MND, it is noted that, in the case of non-destructive testing, a lack of clearness exists about the relative importance of such tests with respect to destructive ones for the evaluation of material properties.
The data acquisition modality has immediate consequences on the calibration of Confidence Factors, treated in Task FC, which may assume different values from those given in the code, in case one accepted to include results from non-destructive tests in addition to – or even in substitution of – destructive ones.
Moving to the level of analysis methods for seismic safety evaluation, the need of a deeper insight into the usual assessment techniques is recognized, with particular emphasis to their predictive capacity when dealing with irregular buildings, dealt with in Task IRREG. It would be expedient to identify, for example, a synthetic parameter capable of quantifying the level of irregularity and, possibly, an associated applicability threshold that helped selecting the most appropriate assessment method, be either of simplified nature, such as pushover analyses, or more refined, such as nonlinear dynamic analyses.
For mixed-type (masonry/RC) buildings, the lack of code provisions, which could guide the designer towards the assessment of the compound behavior in a unitary manner – also accounting for interface actions between different constructive typologies – , is absolutely striking and appropriate methods and provisions should be identified in Task MIX.
A different remark is needed for the influence of infills on the structural response, where the motivation for the research carried out by Task TAMP stems from the absence of code provisions to account for the meaningful interactions that develop between infills and

structure, with significant effects, both, at the global level (behavior factors), and a the local level (collapse mechanisms induced by the presence of localized forces).
The following three Tasks SCALE, NODI and BIAX, related to stairs, beam-column joints, and biaxial behavior of columns, respectively, deal with three aspects where the need of providing the designers with operational tools is imperative, especially for as regards the assessment of the capacity of such elements. In the first case, the motivation is to obtain a deeper insight about the influence of stiffening elements – the stairs – on the structural response. Generally, when modeling the resisting system, these elements are either neglected or modeled with unacceptable simplifications. In the second case, that relative to beam-column joints, it is necessary to develop more accurate capacity models, accounting for the joint panel behavior, but also for the presence of secondary phenomena significantly modifying the resisting mechanism, such as concentrated forces ensuing from hook-bent bars, or bond-slip in rebars. In the third case, that of biaxial behavior of columns, the motivation for research stems from the awareness that the capacity equations currently available in the code are calibrated on the mono-axial behavior, besides, without interaction with shear.
Finally, for the prefabricated structures studied in Task PREFAB, the intention is to provide the normative framework with more complete indications than those currently available, with the objective of bridging the current information gap through the proposition of specific guidelines for the seismic assessment and strengthening of such buildings.

The above considerations are expanded in the following sections.

2.1 MND: Non-Destructive Methods for the Knowledge of Existing Structures

Modern seismic codes require that a knowledge level (KL) is defined (e.g. 3 KLs in EC8 part 3: limited, normal and full knowledge) in order to choose the admissible type of analysis and the appropriate confidence factor values in the evaluation. Among the factors determining the KL, there are the mechanical properties of the structural materials. In RC structures, the compressive strength of concrete has a crucial role on the seismic performance and is usually difficult and expensive to estimate. Reliable procedures to take into account the factors influencing the estimation of in-situ concrete strength, particularly in case of poor quality concrete, are not currently available. According to various codes (e.g. in Europe EC8-3, in Italy NTC 2008) estimation of the in-situ strength has to be mainly based on cores drilled from the structure. However, non-destructive tests (NDTs) can effectively supplement coring thus permitting more economical and representative evaluation of the concrete properties throughout the whole structure under examination. The critical step is to establish reliable relationships between NDT results and concrete strength. The approach suggested in most codes (e.g. in EC8-3) is to correlate the results of in-situ NDTs carried out at selected locations with the strength of corresponding cores. Thus, NDTs can strongly reduce the total amount of coring needed to evaluate the concrete strength in an entire structure.

2.2 FC: Calibration of Confidence Factors

Data collected for the assessment of a building are obtained from available drawings, specifications, and other documents for the existing construction, and must be supplemented and verified by on-site investigations, including destructive and nondestructive examination and testing of building materials and components.
As a function of the completeness of as-built information on buildings (Knowledge Level) the Italian Code specifies different analysis methods and Confidence Factors (CF) to be applied to the mean values of materials strengths.

Difference in the knowledge procedure about the single structural parameters and the actual possibility of propagation to the structure as a whole of information gathered on single members unlikely can be accounted for by a single CF to be applied to mean materials strength values.

Material strength is characterized by, both, an intrinsic spatial variability and an epistemic uncertainty, caused by either workmanship (for instance not compliance with the original project, execution of structural elements in different times with different materials strength), or reliability of testing methods, or degradation of material properties with time, or a combination of the former. On the other hand, amount and detailing of reinforcement, defective detailing, etc., neglecting the intrinsic uncertainties, are characterized by epistemic uncertainties only, mainly due to lack of the original project and/or not compliance with it; collected data on one structural element are certain but do not allow to eliminate uncertainties about other elements.

Objectives of recent studies (Franchin et al. 2008, Jalayer et al. 2007) have been the evaluation of the effect of CF on the assessment of the structural reliability and new proposals for calibration of a CF.

2.3 IRREG: Assessment of the Nonlinear Behavior of Buildings, with Emphasis on Irregular Ones

Many of the existing RC structures were built without accounting for seismic actions, thus much attention has been paid in recent years to the development of reliable methods of analysis and assessment. Linear methods seem inappropriate in most cases; many current seismic codes and guidelines include provisions for nonlinear analysis (Eurocode 8, 2003a, EuroCode 8, 2003b, FEMA 356, 2000, ATC-40, 1996), which seems to be the natural choice for existing structures subjected to moderate and strong design earthquakes. This is obviously a big issue in Italy, a seismically active country where many buildings were erected in the ‘60s, ‘70s and ‘80s usually accounting for only gravitational actions. Furthermore, the new seismic zonation classifies areas previously considered non-seismic as seismic, thus new assessment are needed even on recently built structures.

Following the publication of the most recent Italian Seismic Codes, the ReLUIS program of the Italian Department of Civil Protection intends to validate and improve the new code, to propose alternate procedures when deemed necessary, and to provide practical examples to practicing engineers. These activities are particularly important for new methodologies, such as nonlinear methods of analysis. Focus of these studies is not only the application of the nonlinear methods of analysis, but also the use of the results of the nonlinear analyses to assess the seismic vulnerability of structure.

2.4 MIX: Assessment and Strengthening of Mixed-type (Masonry/RC) Buildings

From the early 20th-century the combined RC-masonry buildings widely spread in European, Mediterranean and Southern America countries. Despite the diffusion of this combined building typology, the international guidelines have not followed building evolutions; nowadays, international guidelines are not exhaustive to deal with specific problems of this building typology, such as: horizontal loads repartition, connections between different technology elements and over strength factor.

The Argentinean guideline (NAA-80) points out the fundamental role performed by slab, on the base of the own relative stiffness, for sharing seismic action between vertical different technology resistant elements. During the years, the Italian guidelines have provided discordant indications.

The Italian guideline (D.M. 1996) suggested to assign the total seismic action to masonry walls in the case of new buildings, while for existing buildings, the combined RC-masonry buildings should be considered as structural elements typology that prevalently supports horizontal loads, generally masonry walls.
Regarding masonry buildings, the Italian guideline (O.P.C.M. 3431) allows to employ different technology elements to support gravity loads, only if the seismic action is fully supported by elements with the same technology. In the need to consider the collaboration of masonry walls and different technology systems to sustain the seismic action, a non-linear analysis should be carried out according to O.P.C.M. 3431. The latest Italian code (D.M. 2008) confirms the instructions provided by O.P.C.M. 3431 by which the real structural system should be considered with particular attention to, both, stiffness and strength of the slabs, and the connections effectiveness between the structural elements.

2.5 TAMP: Influence of Infills on Structural Response

The very numerous papers that concern the behaviour of infilled frames are quite uniformly distributed within the last forty years (Figure 1a). This subject has kept topical mainly because of the following reasons: - different materials that can be utilized for the infill panels; - difficulty in modelling the frame-infill interactions; - high number of parameters governing the lateral response of an infilled mesh of frame.

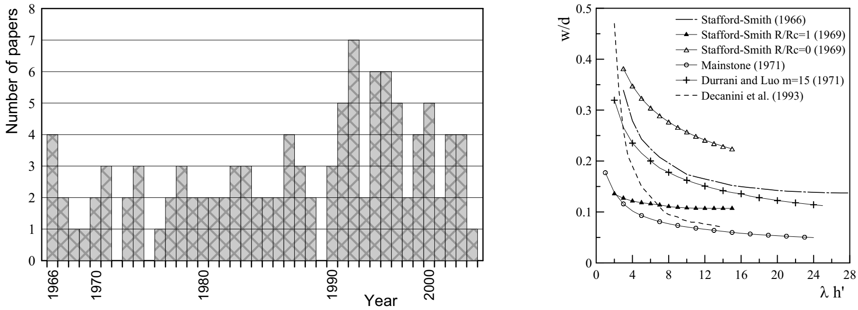


Figure 1. a) Main published papers ; b) Ratios w/d proposed by different authors.

It follows that the models that have been proposed by different researchers are strongly related to the kind of masonry infills that were examined and to the experimental tests validating the models themselves. As regards this, Figure 1b shows how the section of the equivalent diagonal strut by different authors is differently related to the same synthetic parameter λh', which depends on the geometrical and mechanical properties of the two sub-systems (frame and infill). In the figure, *w* denotes the height of the section and *d* is the diagonal length of the infilled mesh (Refs. 3-7). Further discordant results can be found considering models including the post-elastic hysteretic behaviour.
Therefore, the main motivation of the present study lies in the non-availability of an univocal approach, able to define an appropriate infill model depending on the properties of the masonry utilized.

2.6 SCALE: Behavior and Strengthening of Stairs

In general the presence of a stair creates a discontinuity in a regular reinforced concrete skeleton frame made of beams and columns; in fact, from the geometrical point of view, a

stair is composed by inclined elements (beams and slabs) and by short (squat) columns. These elements contribute to increase the stiffness of the stair due to the elastic behaviour of inclined elements and of squat columns. For these reasons the elements that constitutes the stair are often characterized by a high seismic demand: the squat columns are subjected to high shear force that can lead to a premature brittle failure; the inclined beams, differently from the horizontal beam, are defined by high variation in axial forces that can modify the resistance and deformability of all these elements.
Although this is well known, no studies have been conducted by researchers to evidence the role of stairs on the seismic capacity of existing RC buildings; the identification of the weakest elements of the structure and the failure type considering the presence of the stairs are of particular interest. In this way, the knowledge of structural solutions and design practice of stairs is an important step in order to define their real geometric definitions and to understand their seismic performances.

2.7 NODI: Behavior and Strengthening of Beam-Column Joints

Observation of the damage caused by strong earthquakes on RC buildings designed to resist only to gravity loads showed that the main mechanisms that characterize structural collapses are, beyond the yielding of primary elements such as column and beams, slippage of longitudinal bars in columns and beams and joint failures (e.g. Braga et al. 2001).
Based on these observations and on the results of extensive experimental campaigns, some provisions were inserted in the Italian technical regulations imposing performance criteria for the design of new RC structures placed in seismic zones. The capacity design approach provided by current Italian and European codes (NTC 2008, CEN 2004) aims at preventing brittle failure mechanisms in beam, column and joint members as well as at ensuring a weak beam-strong column global collapse, being more favourable in terms of overall ductility. For existing RC structures, designed without anti-seismic criteria, there is the problem of a reliable assessment of their seismic resistance also in order to identify the more appropriate strengthening intervention systems. Improving knowledge on capacity models, particularly as for typical Italian building structures, is the principal thrust for the research activity of task 7 (NODI) in the framework of DPC-ReLUIS 2005-2008 Project.
The workflow in terms of literature review, experimental testing and numerical analysis performed by the RUs involved in the task is, then, finalized to the analysis and validation of the provisions of Italian and European codes in order to improve them and make them more adherent to the reality of the Italian existing RC building stock.

2.8 BIAX: Behavior and Strengthening of Columns under Combined Axial Load and Biaxial Bending and Shear

Modeling of reinforced concrete members is really a traditional topic of structural engineering, but some aspects need further development when seismic assessment of existing constructions is concerned. In fact, well-established results for modern concrete structures do not cover a large population of members built with obsolete materials and structural details like smooth bars. This is actually a relevant issue, since bond between steel bars and the surrounding concrete is poor and anchoring mechanical devices can play a relevant role in the development of plastic deformation and therefore of the drift capacity.
A number of models characterized by different models of complexity can be found in the National and International technical literature (fib, 2003; Panagiotakos and Fardis, 2001; Park and Paulay, 1975) and provide an estimation of the rotation capacity at yielding and at failure of columns member. However, they generally are able to well represent response of r.c. members where deformed bars are used. Based on such a background, the research on

columns subjected to biaxial bending and axial force has been conceived to cover the lack of knowledge at the time of proposal. In fact, advances in seismic Codes and increasing need of data for design purposes can be addressed among the primary motivations of Task BIAX. On the other hand, since tools for the estimation of strength and deformation of bare cross sections were not so consolidated, a specific focus on columns strengthened with FRP materials is certainly of applicative interest. This circumstances confirm the rational basis of the research and above all the actual usefulness of its results.

2.9 PREFAB: Behavior and Strengthening of Prefabricated Industrial Structures

Precast structures passed through the check of weak and strong earthquakes and have been submitted to a wide specific experimental and theoretical investigation performed in the main international research centres. From these experiences some key aspects turned out to be determinant for the good seismic behaviour of precast structures. These key aspects are listed below:

- *dry friction supports,* not suitable to avoid the loss of bearing;
- *diaphragm action*, important to avoid joint distortions;
- *lateral supports,* necessary to avoid the overturning of beams;
- *2nd order effects,* to be considered to avoid early collapses;

A positive condition of the existing buildings of concern is the possible presence of a bridge-crane which required a structural design with relevant horizontal forces and a proportioning of the columns which could be adequate also for seismic action even in the presumption of low ductility.

The regulation in force for the design of structures in seismic zones at the time of construction is obviously a conditioning aspect which affects the seismic capacity of existing buildings. Actions and rules for design have been taken from that regulation which may result inadequate on the base of the today knowledge. The problem concerns the seismic zoning on one hand and the design criteria on the other.

3 RESEARCH STRUCTURE

As inferred from the previous sections, Research Line 2 is organized in 9 different Tasks, and benefits from the contributions of 16 different Research Units (RUs). The organization of the Research Line is presented in the following Figure 2, where the contribution from each RU is clearly identified within each Task. Each Task is identified by an acronym, explained in the following paragraph. Each RU is also identified by an acronym, explained in the subsequent Table 1, where the local leaders are also highlighted.

The organization below allowed to facilitate the exchange of information and documents among the different levels of: Line coordinators, Task leaders and RU leaders. Also note the interactions with the other Research Lines.

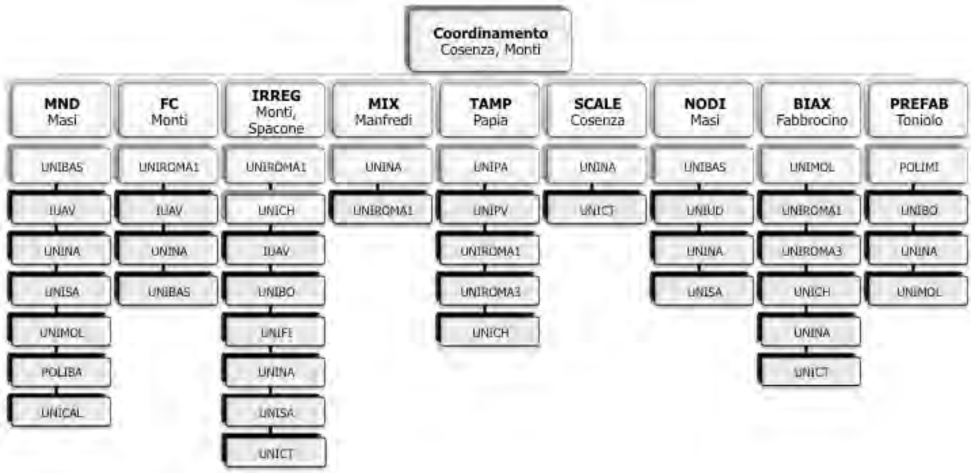


Figure 2. Organisation of Research Line 2.

Table 1. Research Units involved in Research Line 2.

	Research Unit	Acronym	RU leader(s)	Involvement in Task(s)
1	Napoli Federico II	UNINA	Cosenza, Manfredi, Ramasco	MND, FC, IRREG, MIX, SCALE, NODI, BIAX, PREFAB
2	Pavia	UNIPV	Pinho	TAMP
3	Basilicata	UNIBAS	Masi	MND, FC, NODI
4	Roma La Sapienza	UNIROMA1	Monti, Decanini	FC, IRREG, MIX, TAMP, BIAX
5	Milano	POLIMI	Toniolo	PREFAB
6	Udine	UNIUD	Russo	NODI
7	Venezia	IUAV	Foraboschi	MND, FC, IRREG
8	Bologna	UNIBO	Benedetti	IRREG, PREFAB
9	Firenze	UNIFI	De Stefano	IRREG
10	Chieti	UNICH	Spacone	IRREG, TAMP, BIAX
11	Molise	UNIMOL	Fabbrocino	MND, BIAX, PREFAB
12	Salerno	UNISA	Faella	IRREG, NODI
13	Bari	POLIBA	Mezzina	MND
14	Catania	UNICT	Ghersi	IRREG, SCALE, BIAX
15	Palermo	UNIPA	Papia	TAMP
16	Calabria	UNICAL	Spadea	MND
17	Roma Tre	UNIROMA3	Nuti	TAMP, BIAX

In the following sections, the objectives pursued in each Task are described.

3.1 MND: Non-Destructive Methods for the Knowledge of Existing Structures

Research has been mainly focused on the evaluation of the role of the main factors affecting the estimation of the in-situ concrete strength through destructive and non-destructive tests, on the determination of the design concrete strength, on the evaluation of the possible damage on core specimens due to drilling and, finally, on the load bearing capacity of structural members subjected to drilling before and after restoration interventions. Other activities were mainly devoted to analyze the correlations between the various methods and the possible spatial variability of concrete properties throughout the surveyed members. Regarding data collection, a large amount of experimental data from destructive and non destructive in-situ investigations on real strucures were collected and analysed, also through treatment of uncertain variables with different mathematical nature. In addition to the above topics, the seismic behavior assessment of buildings with structure composed of unidirectional RC frames was carried out by means of non-destructive in situ tests, with the objective of estimating their horizontal load-carrying and dissipative capacity. Finally, another important objective was the evaluation of the dispersion of experimental results from non-destructive measurements based on a critical review of data reported in the literature.

3.2 FC: Calibration of Confidence Factors

This Task had two objectives. The first one was to propose a methodology for the calibration of the CF for materials strength, taking into account the uncertainties characterizing existing building and the effects on the reliability of the assessed structural performance. A methodology was also sought for the evaluation of material strength by destructive and non destructive in situ testing methods taking into account the relevant reliability. The procedures were meant to be based on the application of the Bayesian method. The proposed methodology and the equation developed for FC have been validated on several simulated cases and on tests made on several buildings. The second objective was to develop a probabilistic methodology for seismic assessment of existing buildings taking into account explicitly the uncertainties in the material properties and the structural detailing parameters and implementing the available test and inspection results. This methodology may be used for determination of confidence factors.

3.3 IRREG: Assessment of the Nonlinear Behavior of Buildings, with Emphasis on Irregular Ones

The main objectives are:

- Validation of available modeling alternatives for RC buildings, mainly lumped-plasticity and distributed plasticity models, both in commercial and research software.
- Validation of current methods of analysis for the seismic assessment of existing RC buildings, with emphasis on nonlinear methods and their applicability to plan-irregular buildings

The above validations were carried out through the analysis of several buildings selected by the different research units. Three buildings (shown in Figure 3) were selected as common tested structures: one is a doubly symmetric rectangular building, one is an L-shaped building, and the third is a rectangular building with an internal court. These buildings are representative of the structural buildings commonly found in Italy. Several commercial and research programs were used for the nonlinear analyses, including SAP2000, OpenSees, Midas, etc.

Figure 3. Tested buildings.

The final objective of this task is the compilation of a document that contains: an introduction to nonlinear modeling of RC buildings and to the nonlinear methods of analysis of the European seismic codes: a description of the main sources of nonlinearities in existing RC buildings: the application of different modeling techniques to the seismic vulnerability assessment of the three building mentioned above. The document is intended to be a primer for practicing engineers who want to use nonlinear methods of analysis.

3.4 MIX: Assessment and Strengthening of Mixed-type (Masonry/RC) Buildings

With reference to the first goal, technical literature and international guidelines have been studied in order to define the classification of the main geometrical characteristics of such kind of buildings and the study of the response of mixed-type buildings – behaving as parallel systems – subjected to horizontal forces.
With reference to the second goal, a series of numerical analyses based on different and progressively refined modelling assumptions have been performed in order to investigate he seismic action distribution between different technology elements, changing the size and then the stiffness of the RC elements, but retaining the geometry of the building and comparing the seismic behaviour of the mixed-type building with the original masonry one. Pushover analysis have been performed by using a lumped model to evidence critical zones and possible failures.

3.5 TAMP: Influence of Infills on Structural Response

A first experimental investigation was devoted to determining the mechanical properties of three typical kinds of resisting elements, commonly used for infill masonry, and of the mortar utilized for their assembly. Then, several infill samples were subjected to compressive tests by assuming orthogonal or parallel loading directions with respect to the mortar layers. Further results were obtained under diagonal compressive loading, to determine shear modulus and resistance. At the end of this phase, the experimental values of elastic moduli and resistances were compared with the values that Italian M.D. ’87 provides by linking the mechanical properties of masonry elements to those of their components.
A second phase of the experimental research was devoted to acquiring the response of square infilled meshes of RC frames subjected to a cyclically varying lateral forces. Two 1:2 scaled samples were tested for each of the three kinds of infill that had been mechanically characterized previously. The results of these tests have made it possible to calibrate the hysteretic model of pin-jointed diagonal strut proposed in Cavaleri *et al.*, 2005.
A further experimental investigation was carried out by means of shaking table tests on a 1:2 scaled 3D infilled RC frame, reproducing an actual non-infilled building, previously subjected to pseudo-dynamic tests at the ELSA-JRC-Ispra. These tests had the following objectives: - to quantify the lateral stiffness and resistance contributions that infills can provide; - to verify the influence of the infills on the crack distribution and the collapse mechanism. The same objectives were pursued by nonlinear numerical analyses on multi-storey RC frames subjected to natural seismic accelerograms. These analyses also showed the negative effects of non-uniform infill distribution along the height.
Another experimental campaign on infilled r.c. frames (1: ½ scale), on materials (concrete, steel, blocks and mortar), and on subassemblages (small panels) has been performed. These tests had the objective of calibrating equivalent strut models, through comparison of experimental results on bare and infilled frames, in order to evaluate the infill contribution as well as its uniaxial force-displacement relationship. The constitutive models for masonry infills have been also calibrated in order to predict the cyclic response of infilled frames.

3.6 SCALE: Behavior and Strengthening of Stairs

With reference to the first goal, several available manuals and books at the time of construction have been studied in order to define the typology classification and the corresponding evolution of this classification during the years with the increasing knowledge on the use of the materials and of computational machines. An analysis of the codes used from 1909 to the 1980ies has been conducted in details with a critical judgement based on the actual knowledge. Examples of stairs designed for only gravitational loads have been studied with reference to different typologies.

With reference to the second point a series of numerical analyses based on different and progressively refined modelling assumptions and criteria has been performed in order to investigate the principal failure modes. A critical study has been conducted on the different shear strength formulations present in literature (Biskinis et al.2004; Sezen et al. 2004; Zhu et al., 2007), in order to simulate potential shear failure in squat columns, which can be easily found in most buildings. The pushover analysis by using a lumped model has been performed to evidence critical zones and possible failures.

With reference to the third point, a test set-up has been defined in order to investigate the experimental behaviour of a building sub-assemblages, including a stair substructure.

3.7 NODI: Behavior and Strengthening of Beam-Column Joints

A wide experimental campaign on beam-column joints representative of typical members present in Italian existing buildings was planned, designed and carried out. In particular, the research activities were devoted to outline the influence of some parameters on the mechanical behaviour and the failure mechanism of the joints, such as axial force, amount of reinforcing steel and earthquake design level. Furthermore, the research focused on the code expressions for the evaluation of the ultimate rotation of RC elements in order to highlight possible discrepancies between the theoretical and experimental results. Another research objective was the analytical modelling of beam-column joints by using DIANA software to analyze the main parameters affecting their seismic performance and, specifically, the analytical modelling of experimental tests conducted on external joints. Other experimental tests on beam-column joints relevant to existing buildings were performed as well, following an experimental program complementary to the above-mentioned one, that is, in this case specimens reinforced with smooth bars were tested and, in some cases, after the first test, joints were retrofitted to evaluate the effectiveness of some retrofit systems. Finally, supplemental activities followed two main branches: on one hand, a series of either reinforced or unreinforced base joints were tested thus evaluating the performance of several strengthening systems, and, on the other hand, a wide database of tests on beam-column joints was built and analyzed.

3.8 BIAX: Behavior and Strengthening of Columns under Combined Axial Load and Biaxial Bending and Shear

The activities have been planned to cover four main objectives: (1) review of technical literature with specific reference to available experimental data; (2) development of refined and simplified models for bare and FRP reinforced members; (3) experimental activity on columns subjected to cyclic actions; (4) drafting of a technical report summarizing the main applicative aspects of the work.

3.9 PREFAB: Behavior and Strengthening of Prefabricated Industrial Structures

The study envisages a preliminary classification of the industrial prefabricated building typologies existing in Italy, from which the most frequent characteristics of element-to-element connection types will emerge. This first cataloguing phase is then followed by a purely experimental phase in which some connections, identified as more vulnerable (e.g., friction connections), are subjected to a series of cyclic tests to simulate seismic conditions. Results and information obtained from the experimental tests will serve as a basis to develop practical models for assessing the capacity of such connection zones and to orient towards the definition of criteria and techniques for strengthening interventions.

Figure 4. Poster for the National Conference of Research Line 2.

4 MAIN RESULTS

The activities performed – and the results obtained – within the Research Line have been the object of a National Conference held in Roma on 29-30 May 2008, with more than 200 participants from 23 different Italian Universities, from the professional and industrial community, where 82 papers were presented, equally divided between methodological and applicative, of which 67 were produced by researchers belonging to this Research Line. Figure 4 shows the conference poster.
The papers presented at the conference are divided among the nine Tasks as follows:

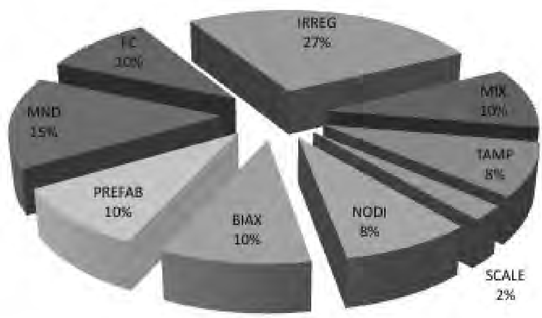


Figure 5. Papers per Task presented at the RL2 Conference.

The main results obtained by each Task are summarized in the following sections.

4.1 MND: Non-Destructive Methods for the Knowledge of Existing Structures

The results obtained during the Project are mainly made up by the execution and analysis of experimental investigations either on in situ real structures or on laboratory specimens, by the implementation of some procedures to estimate the in situ concrete strength and by the uncertainty treatment of the structural characteristics of existing strucures.
A wide experimental program was carried out, comprising more than 20 RC beam and column members, several hundreds of non destructive tests (NDTs) and more than 50 destructive tests (cores). Analysis of results has shown a large scatter of the core concrete strength both in a single member and among members extracted by the same story of a building. Lower scatters have been detected for the NDT results with the exception of the surface ultrasonic velocity (see Figure 6). As a result of these findings, the role of some factors influencing the in situ concrete properties has been carefully evaluated, and some criteria to suitably select locations for sampling have been provided. A procedure for the evaluation of the concrete strength based on the Sonreb method, using both core and NDT measurements, has been set up and widely validated, clearly showing its higher prediction capacity when compared to the relationships currently available in the technical literature. It requires that the relationship between the in situ concrete strength and the NDT measurements is experimentally derived for the specific concrete under test.
As for the possible damage on core specimens due to drilling, the results have shown that the strength reduction suffered by cores can be significantly influenced by the original strength value of the in situ concrete. Consequently, adopting a constant coefficient to take into account drilling damage, as suggested in the technical literature, can determine incorrect

results. On the contrary, it appears suitable adopting coefficient values, obtained during the research, which are inversely proportional to the core strength as provided by the compression test. Finally, some important results regarding the effect of core drilling on the structural members, performing tests before and after a possible restoration. Further, some factors influencing the relationship between the “local” strength provided by core specimens and the in situ strength of the structural member as a whole, have been highlighted.

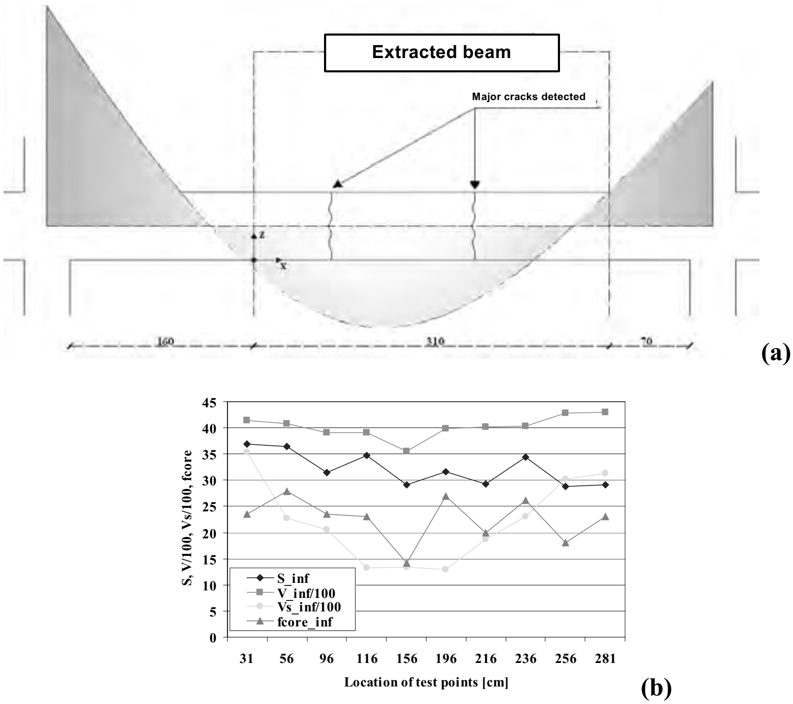


Figure 6. Role of past applied loads on in-situ measurements: (a) qualitative bending moment due to vertical loads (a), and (b) test results along the lower part of the extracted beam (rebound number S, direct velocity V, surface velocity V_s, core strength f_{core}).

Regarding the variability of concrete mechanical properties throughout single structural members and among different sampling locations, investigations based on a wide series of experimental data gathered from surveys carried out on structures assessed for seismic vulnerability were carried out.

Main results obtained are briefly outlined below:

- no general trends have been recognized regarding the spatial variability of the key mechanical properties of concrete throughout column members as a possible result of the combination of the effects of the load pattern and the casting process;
- although carried out on members already cracked and damaged, the results of sonic tests are affected by scatters smaller than those deriving by the compression tests on concrete samples; further, the ratio between ultrasonic velocity derived by indirect and direct measures are normally distributed around the average value of 0.75;

- rebound tests have confirmed the substantial impossibility of recognizing general trends in the spatial variability of the mechanical properties of concrete and led to values affected by scatters quite similar to the destructive ones.

Regarding the treatment of uncertainties in determining the characteristics of materials and more generally of existing building parameters. A fuzzy-logic based approach for uncertainty treatment has been set up and a computer code for its implementation has been developed.

In addition, the prediction capability of some formulations provided by the current technical literature was verified based on experimental investigations through non-destructive and destructive tests on existing structures.

Based on a a critical review of available literature, a database was prepared that collects literature data on non-destructive tests on concrete specimens for concrete grade assessment. Dispersion of experimental results has been estimated and the influence of uncertainties coming from the concrete grade estimation on seismic capacity of RC existing buildings has been investigated.

Some results obtained during the project have been reported in papers published on journals and in proceedings of Conferences (e.g. Masi and Vona, 2008; Marano et al., 2008; Olivito et al., 2008).

4.2 FC: Calibration of Confidence Factors

A first aim of the task has been the evaluation of the confidence level on structural safety of existing buildings given by seismic structural assessment carried out according to the indications of the Italian OPCM 3431, 03-05-05, Annex 2.

Uncertainties in reliability structural analysis are due to material properties, structural details and condition of the structural elements. The prior distribution of the considered uncertainties takes into account their mechanical effects. The proposed probabilistic models are subsequently updated by in-situ information.

A parameter describing the structural performance is defined as the demand/capacity ratio and its probability distribution is assessed by a Monte Carlo simulation. Each realization corresponds to an application of the capacity spectrum method and needs the execution of a structural linear static analysis. The Bayesian up-dating of the structural reliability is carried out by a Markov Chain Monte Carlo algorithm. The structural performance prior probability distribution function is evaluated in two different cases: 1) taking into account the uncertainties about material properties and structural details; 2) updating the structural assessment based on in-situ tests and inspections.

The updating process consists of two different levels: in the first, destructive tests and relating errors are taken into account; in the second, non-destructive tests and relating errors are taken into account. Structural seismic performance has been evaluated in three cases: a) by using the mean material strength values (CF=1); b) by using the mean material strength values scaled by the CF for a Normal knowledge (CF=1.2); c) by using the mean material strength values scaled by the CF for a Limited knowledge (CF=1.35).

For each value of the CF the demand/capacity ratio has been evaluated and positioned in the structural performance distribution, for the three knowledge levels, with reference to the structure at hand. The CF value considered as the exact one has been defined as the value corresponding to the demand/capacity ratio value with 5% exceedance probability. In this way, the CF for material strength is evaluated on the basis of the probabilistic evaluation of structural performance taking into account all the intrinsic and epistemic uncertainties, including uncertainties on the testing methods.

A simplified method is proposed too, based on a limited number of Monte Carlo simulations, which is able to approximate the probability distribution of the structural parameter. This can

be a basis for the development of simple procedure to use in for evaluation of structural safety.
A second objective was to develop a procedure for evaluation of material strength and calibration of CF based on the application of Bayesian method, to take into account the number and the reliability of the in-situ tests carried out.
The Bayesian method allows to employ destructive and non-destructive testing results to update a prior probability distribution function. Destructive and non-destructive testing results are separately employed, taking into account individual testing reliability (reliability due to testing errors and errors in regression curve that provides the material resistance as a function of the testing parameter). More than one test method can be employed performing consecutive up-dating of the probability distribution function.
The statistics reliability of the mean value is improved by applying the confidence interval for the mean; a 95% lower confidence level is considered, which represents the value for the structural assessment.
In order to facilitate its evaluation, a simplified procedure is defined. The material strength value for structural assessment can be obtained scaling with an appropriate Confidence Factor a weighted mean of the sampling mean vales obtained by, both, different testing methods and prior information:

$$\mu_D = \frac{\mu}{FC} \cong \breve{m}'''_{\mathrm{inf},\breve{m}_f} \tag{1}$$

where:

$$\mu = \left[\frac{\mu'_f + n_{DM} \cdot \bar{x}_{DM} + n_{NDM} \cdot \bar{x}_{NDM}}{1 + n_{DM} + n_{NDM}} \right] \tag{2}$$

where $\bar{x}_{DM}$ and $\bar{x}_{NDM}$ are the sampling mean of the destructive and non destructive tests, respectively; n_{DM} and n_{NDM} are the corresponding sampling dimension.
Generally, if M_i is the *i*-th testing method adopted, the material strength for the assessment is:

$$\mu = \left[\frac{\mu'_f + \sum_i n_{M_i} \cdot \bar{x}_{M_i}}{1 + \sum_i n_{M_i}} \right] \tag{3}$$

where $\bar{x}_{Mi}$ is the sampling mean of the i-th testing method and n_{Mi} its dimension.
The CF can be expressed in an explicit form as a function of the Bayesian coefficient of variation V_μ for the median value of the material strength:

$$FC = a + c \cdot V_\mu^\omega \tag{4}$$

The parameter V_μ, which estimates the reliability of the available information, is defined as:

$$V_\mu = \frac{\sigma_\mu}{\mu_\mu} = \frac{\sqrt{\sum_i \frac{n_{Mi}}{s_{s,Mi}^2 + s_{t,Mi}^2}}}{\sum_i \frac{n_{Mi} \cdot \bar{x}_{Mi}}{s_{s,Mi}^2 + s_{t,Mi}^2}} \tag{5}$$

where $s_{s,Mi}^2$ and $s_{t,Mi}^2$ are the sampling variance and the variance of the regression curve of the *i*-th testing method, respectively.
The Eq. (4) has been calibrated for concrete strength by applying the least squares method.
A Monte Carlo method has been used to simulate sampling with destructive and non destructive testing and the resulting equation for the CF is:

$$FC = 0.9 + \sqrt{V_{\mu}} \tag{6}$$

The Eq. (6) is effective if samples have been extracted from homogeneous zones of the structure. If in the structure potential non homogeneous zones are identified, the t-Student test can be executed on the mean values extracted from the two zones. If the t-student test identify non homogeneous zones these must be separately evaluated considering two different median value for concrete strength with two CF.
A method has also been investigated for the evaluation of reliability of the correlation function for the assessment of material strength by in situ tests.

4.3 IRREG: Assessment of the Nonlinear Behavior of Buildings, with Emphasis on Irregular Ones

The main results of the project are as follows:

- Regarding the validation of the modified pushover procedure proposed by Fajfar (2002), it was found that it is conservative for plan-regular framed structures with respect to NTHA, while it is unreliable for shear wall structures. For irregular frames, multi-modal procedures have to be used in order to improve the accuracy of static non linear analyses. Furthermore, predictions drawn from non linear static analyses were often found un-conservative in terms of interstorey drifts or chord rotations;
- A new pushover method that explicitly takes into account the torsional behaviour of asymmetric-plan buildings was defined. The effectiveness of the proposed procedure was evaluated by comparing the seismic demand of selected case studies with that obtained through both nonlinear dynamic analyses and other pushover methods;
- The nonlinear analyses on the regular and irregular buildings have shown the importance of damping in nonlinear dynamic analysis. More specifically, as the hysteretic model improves, the damping should be decreased. Furthermore, viscous damping is hard to assign and little or no indications are given in the published literature to guide the user. A value of 2-3% damping for the elastic modes appear reasonable, but no final indications were found;
- In NTHA, for both natural and generated accelerograms, the application of the seismic input in the principal directions of the structure may underestimate the demand the structural demand varies considerably as the seismic input direction changes, more so for natural accelerograms. However, as for the number of input ground motions to use in nonlinear dynamic analyses, enhancements to the EC8 requirements were proposed;
- For irregular buildings, pushovers applied in different directions indicated demands and capacities that depend on the direction considered. Partial results on the L-shaped building are shown in Figure 7.
- Shear collapse in existing buildings not designed according to the capacity design rules, is often dominant. If only the flexural capacity is modeled, most frames are verified both at the Ultimate and at the Near Collapse Limit State. However, additional checks show that the shear capacity has already been reached on several elements prior to reaching the design accelerations. This indicates the need to use

models that consider shear failure too (this is very rare in the available software). Furthermore, early shear failures point to possible retrofitting actions, such as shear strengthening of beams and columns;

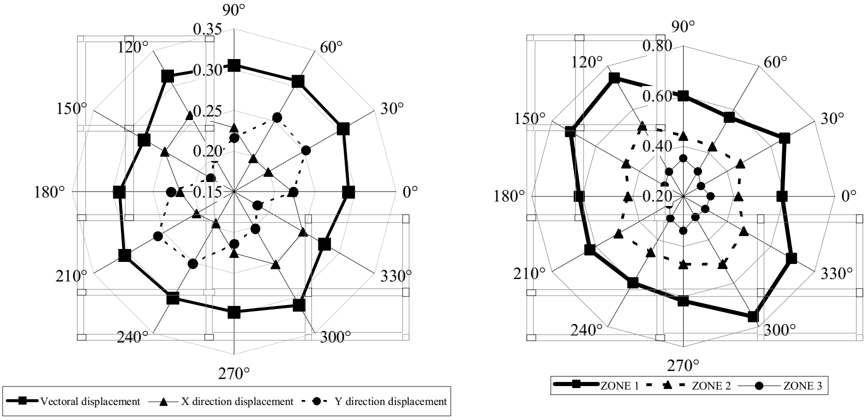


Figure 7. Pushover analyses on L-building: Influence of seismic input angle on the demand: displacements (left) and rotation ratios (right).

- Synthetic expressions were developed that express the plan-regularity of a building with respect to stiffness and strength distributions. Also, simple corrective coefficients were developed to compute the structural demand increase as the structural eccentricity increases. In alternative, a simplified procedure was developed that can help optimize the structural design by producing a plan-regular building;
- Severe convergence problems were encountered in commercial codes that use lumped plasticity models. These problems were related to the software limitations rather than the modelling selection. As expected, fibre section models provide a better, more physically-based prediction of the section response. It is however important to extend the model to include shear failure, in order to avoid post-processing of the results;
- Different Limit State definitions for performance assessment were considered, such as interstorey drift, plastic rotation and chord rotation. It was shown that different measures lead to different, sometimes very different, capacity predictions. The limits usually assumed for interstorey drift result in larger chord and plastic rotation than limits proposed by the European and American Codes. Furthermore, the procedure provided by Eurocode 8 to compute the chord rotation capacity yields predictions that are very different from those obtained analytically, that is integrating the member curvature throughout the plastic hinge length;
- A simplified method for deriving fragility curves and evaluating the probability of failure was proposed. The method is based on an incremental application of the so called N2-Method with natural response spectra, whose irregularity covers the record-to-record variability of the structural response without the need for performing non-linear dynamic analysis.

4.4 MIX: Assessment and Strengthening of Mixed-type (Masonry/RC) Buildings

The main activities of the first goal have involved both the literature review of mixed-type buildings and the modelling aspects related to the RC substructure of such structures. A report was produced (Decanini et al., 2006) including the main results obtained in the Task.

The work involved the study of 160 papers, as indicated in Table 2. Confined masonry is listed separately to underline the difference in the amount of studies between those constructions and structures in which elements of different technologies (masonry/reinforced concrete) are not bonded. The latter case includes constructions in which masonry and reinforced concrete frames are present at the same level and construction in which reinforced concrete frames are placed in the upper floors only.

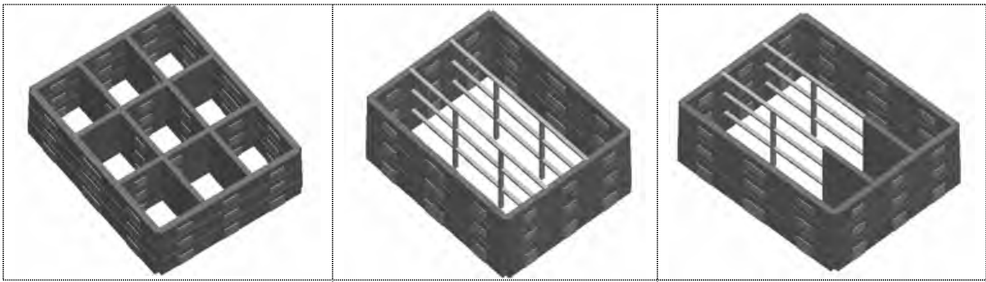
Table 2. Literature review on mixed-type buildings.

Codes	National 7	International 15	Total 22
Damages after earthquake[1]	National 16	International 63	Total 79
Mixed-type building[2]	Experimental 7	Analytical studies 1	Total 8
Confined masonry[3]	Experimental 27	Analytical studies[4] 24	Total 51

[1] Reports and other papers enclosing field observations after earthquakes.
[2] Building in which elements of reinforced concrete and masonry are not bonded.
[3] Building in which bearing masonry walls are confined by reinforced concrete elements.
[4] Analytical studies, including also proposal for design and for the modelling of confined masonry structures.

The main conclusions inferred by the study can be summarized as follows. In many codes the confined masonry structures are taken into consideration, even though usually just empirical dimensional rules are given, while methods for assessment and design are lacking and, when present, they differ substantially from a code to another. Concerning the mixed structures (unconfined masonry), only a small number of codes gives some indications, which are usually about the behaviour factor and/or the distribution of horizontal forces among different structural elements. A similar trend was found also when gathering papers on field observations, experimental tests and analytical studies: a lot of data were found concerning confined masonry structures while just a few were gained about other kinds of mixed structures. In general it is possible to state that confined masonry can withstand to seismic actions, given that materials are of good quality and good constructions rules are followed. On the contrary, some deficiency, such as small amount of transverse steel in columns, high thickness of mortar joint, lack of RC element near the openings, can generate the bed behaviour of this kind of structures. Concerning the mixed structures (other than confined masonry), the main problems which arise in the assessment and design are related to the distribution in plan of the horizontal actions, to the determination of the behaviour factor and to the study of the connections between masonry and reinforced concrete elements. In literature, indications about these points are few and further studies are needed.

With reference to the second goal, the main problems concerning the non-linear analyses on RC-masonry mixed buildings have been highlighted. Particular attention has been devoted to the seismic action distribution between different technology elements. The results of the analysis carried out on 3D RC-masonry mixed building (see Figure 8) with external walls and internal frames, underline the fundamental masonry role to withstand horizontal action, while a significant translation increase could be offered by introducing RC frames. This research has shown the growing capacity offered by mixed building to bear the seismic action by increasing the RC elements stiffness. The increasing RC elements stiffness is significant for the seismic action distribution both in linear and non-linear field. From the analyses it has emerged that, by increasing the stiffness of internal RC frames, the maximum sustainable seismic action of mixed building increases, while the rate of seismic action supported by masonry walls decreases (Nardone et al., 2008). The important slab role in sharing the seismic action between the vertical resistant elements and the importance of slab to perform this function in order to avoid undesirable behaviour of the building has been underlined.



model A	model B	model C

Figure 8. Analyzed models (Nardone et al., 2008).

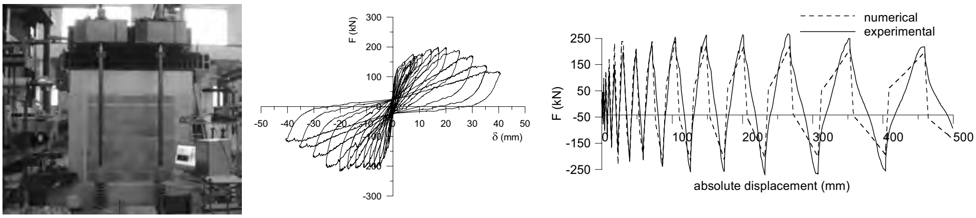
In addition to that, concerning the first model, the analyses highlighted that the presence of the RC frame is generally detrimental for the masonry wall. In fact, the resisting total base shear of the mixed system is smaller than that of the masonry wall alone when the contribution of the frame wall is smaller than about 15-20% of the total base shear. Moreover, when the top connection is non effective, hinges develop at the top and the bottom of RC columns. Therefore, it seems appropriate to design the RC elements considering seismic action deriving from the pertinent vertical loads. While, if the connection between masonry and frames are effective, it would be expedient to assign the whole of the horizontal loads to the masonry (Decanini et al., 2008).

In general, the analyses performed on the 3D model highlighted the difficulties in modelling the masonry and the connections between different elements (beam-masonry, floor-masonry) and indicated the strong influence on the global structural behaviour of the connections effectiveness. The parametric analyses showed the importance of the masonry tensile strength among other mechanical parameters on the global response and highlighted that the RC elements remain elastic and give a negligible contribution to the overall performance. Finally, the comparison between the mixed structures results and those obtained for the masonry structures confirmed how the practice of replacing masonry walls with RC frames, in the interior of old masonry buildings, can have negative consequences on the vulnerability of the building themselves.

4.5 TAMP: Influence of Infills on Structural Response

The resisting elements that were chosen to construct commonly used masonry infills were: calcarenite ashlars, hollow clay brick and hollow lightweight concrete blocks. The results of the compressive tests on the related masonry samples lead to the following main conclusions: *i)* calcarenite masonry exhibits middle compressive and shear strengths, and ductile behaviour up to the collapse for each of the three considered loading directions; it behaves like an orthotropic material in which the ratio between the two elastic moduli depends on the mortar properties; *ii)* clay brick masonry is about twice resisting in compression, but ductile behaviour was observed only under diagonal loading, when the shear behaviour of the mortar

joints is involved; *iii)* lightweight concrete masonry has very low resistance capacities along all the loading directions; the results obtained from this kind of masonry were rather scattered, because vertical mortar joints are not provided for, due to the vertical profile of the resisting elements. The experimental values of elastic moduli and resistances showed that the values deduced by the M.D. '87 provisions are not always reliable. In particular, the shear strength values are strongly underestimated by this code for all masonry tested.
The cyclic tests on the infilled meshes of RC frames have made it possible to verify that the cross-section of an equivalent strut can be determined by using the procedure and parameters shown in Amato *et al.*, 2008, where a not negligible role is exerted by the transverse strain ratio in the diagonal direction and the compression level transmitted to the columns after the masonry infills have been constructed. The lateral resistance of the infill can be deduced from the masonry shear strength; this can be translated into a compressive strength to be conferred to the equivalent strut, by means of a suitable criterion taking into account mainly the disconnection arising between frame and infill. Finally, these tests showed that the calibration of the adopted model leads to sufficiently approximate results (Figure 9).

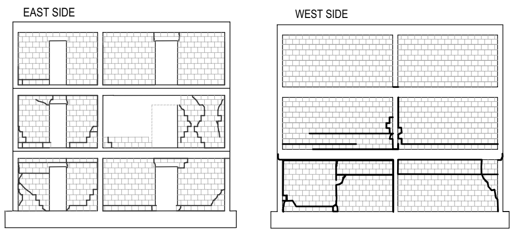


a) b) c)

Figure 9. Cyclic tests on infilled RC frames:
a) test set-up; b) experimental results; d) validation of model.

The shaking table tests on the 3D scaled building (Figure 10) have been carried out by assuming a natural input accelerogram (Herceg-Novi, Montenegro 1979), scaled to three levels of PGA. These tests showed that: - the fundamental period of vibration of the structure is influenced by the presence of the infills (PGA = 0.04 g); - the crack distribution depends on the location of the infills with opening (PGA = 0.3 g); - the infilled structure bears a PGA value (0.54 g) that proves to be about three times the value that had been deduced by pseudo-dynamic tests on the corresponding non-infilled structure.

a)



b)

Figure 10. Results of shaking table test (PGA = 0.3 g): a) 3D building; b) crack distributions.

With regard to the numerical investigation, two series of four-storey and twelve-storey RC frames have been considered. Frames of the first series were designed to bear only gravitational loads; the ones of the second series were designed according to the EC8

provisions. The results lead to the following remarks: - the presence of infills implies greater quantity of input energy for the structure; nevertheless, the infills can dissipate a lot of this energy so that the total balance is favourable to the resisting elements of frame with respect to the case of bare structure; - in the frames of the first series the infills can exert a decisive role towards a seismic event; - in the frame of the second series a non-uniform distribution of infill in elevation would be considered in defining the structural regularity.
Finally, the studies regarding the calibration of a three-strut model arrived at the conclusion that this approach is useful for the comprehension of the local behaviour of columns in case of degradation due to infill-induced shear effects. For what concerns response in terms of displacement, single strut models seem to give satisfactory results. Strut parameters can be directly obtained from the mechanical characteristics of infill components, i.e., mortar and blocks. Moreover, a design criterion for a protecting dissipative bracing systems has been developed. The proposed approach allows to define the required minimum dimensions giving the desired interstory drift.
The results of this Task, synthetically presented here, are discussed in detail in the final Report of Task TAMP. The following remarks can be made, concerning their use and possible improvement of provisions: - the proposed model of equivalent diagonal strut could be utilized for analyses of existing structures designed to bear only gravitational loads; - the expressions proposed by M.D. '87 would be revised, also including values to be assigned to the transverse strain ratios; - infills would be considered for evaluation of the period of vibration of the structure and definition of structural regularity.

4.6 SCALE: Behavior and Strengthening of Stairs

According to the literature, the existing stairs can be classified into two main categories depending on the static behaviour of the stair steps: *(i)* stairs with steps performing as cantilever beam, stair type A (see Figure 11a), *(ii)* stairs with simply supported steps stair, type B (see Figure 11b). Generally the stair type B are used worldwide, in Europe and USA, while the stair type A, with inclined beams, are much more adopted in Europe (Tecnica y Pratica del Hormigon Armado, 1989, Reynolds and Steedman, 2002, Guerrin and Lavaur, 1971). According to the USA's manuals a great scatter in stair structural solutions can be found (Berry, 1999).
According to the manual design criteria (Marrullier, 1910; Rosci, 1939; Santarella, 1953, 1957; Pagano, 1963; Migliacci, 1977) stair type A could be designed considering only gravity loads, any seismic actions could not be taken into consideration. The permanent and live loads on the steps generate on the beam element (bs1-bs2-bs3) a torsional moment T and a distributed load producing on the beam shear force V and bending moment M. Each flight step is designed modelling it as a cantilever beam subjected to a distributed load.
As it is explained in several dated manuals (Santarella, 1953, 1957; Pagano, 1963), the design bending moment into the beam (bs1-bs2-bs3) is evaluated on the basis of different static schemes corresponding to different constraints at the extreme ends of the beam. In particular, two extreme constraint conditions are suggested: full constraint and simply supported. The torsional moment is considered of relevant importance, it leads to add transversal reinforcement (stirrups) along the length of the beam (bs1-bs2-bs3). The adopted values of the torsional moment depend on the hypothesis upon the flexural stiffness of the inter-storey slab: flexible and rigid diaphragm.
About stair type B, manuals indicate two limit structural schemes: *(i)* an horizontal beam full constraint at the end, *(ii)* an horizontal beam simply supported at the extreme ends. Normally bending moment and shear are the internal forces taken into consideration. In the manuals of the construction time the only severe prescription is regarding the design of the steel bars: a

reinforcing bar should not bend to form angles that favour pull-out of the concrete cover (Pagano, 1963).

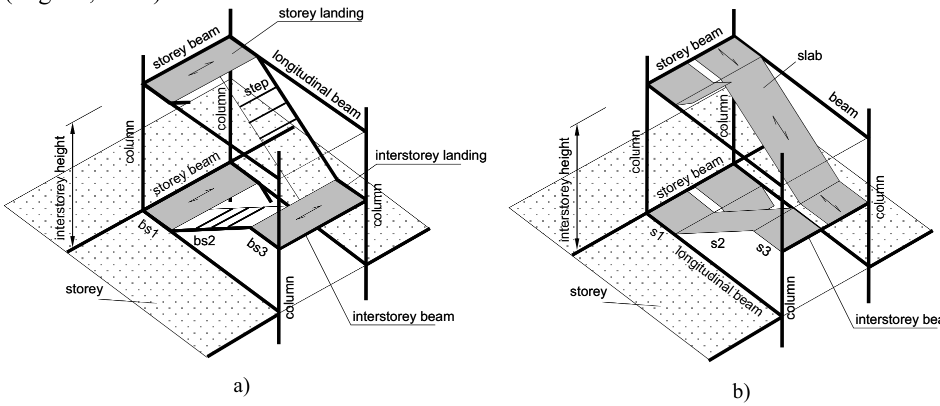
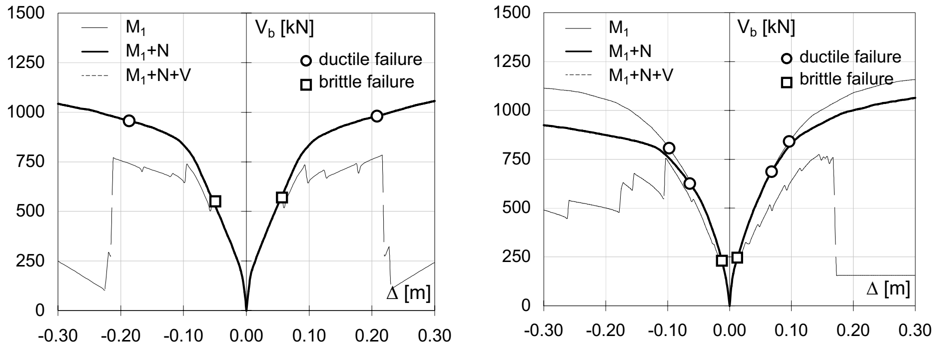


Figure 11. Stair typologies: (a) stairs type A with steps as cantilever beams, (b) stairs type B with simply supported steps.

The Italian stair design practice during the period 1954-1980 has been analyzed in order to identify the most common typologies and the effective adopted design criteria. As already remarked, according to the dated technical manuals and codes stairs could be designed for only gravity load. The predominant stair type in the studied building sample is type A; the flight steps are cantilever elements constraint to inclined beams having one point of discontinuity in the 53% of the cases, two points of discontinuity in the 37% of the cases and is directly connected to the column without any discontinuity in the 5% of the cases. The stair type B is present in the sample with incidence of 3%. The design practice of the most common type of stairs, composed by flight steps constrained into a beam, is herein studied. The static design scheme has a great scatter; beams were designed considering a maximum moment $M^{+} = qL^{2}/\alpha$ in the midspan with α=12 (30% cases) or α=8 (30% cases), while the minimum moment at the extremes of the beam is obtained with α=12 in most cases (76% cases).

Regarding the influence of stairs on the seismic capacity, this preliminary study on the structural typologies of the building sample has evidenced the following problems related to the presence of stairs: distribution of seismic forces (not considered in the design), different modelling design of stair structure, material strengths, element detailing.
The structural typology of stairs generally introduces discontinuities into the typical regular reinforced concrete skeleton, composed by beams and columns; in fact, the sub-structure "stair" is an assemblage of inclined elements as slabs or beams. All these elements contribute to increase the stiffness of the stair due to the elastic behaviour of inclined elements and of squat columns. For these reasons the elements that constitutes the stair are often characterized by a high seismic demand: the squat columns are subjected to high shear demand that can lead to a premature brittle failure; the inclined beams, differently from the horizontal beam, are defined by high variation in axial forces that can modify the resistance and deformability of all these elements.
All these aspects are discussed with a series of analysis on a RC building representative of the studied sample; non linear static analyses (static push over analysis) finalized to the evaluation of the role of the stairs, of their elements and modelling is performed.



Longitudinal direction **transversal direction**

Figure 12. Results of the push-over analysis in terms of base shear versus roof displacement (Cosenza et al., 2008).

The building without any stair is defined as reference. Two models have been considered to study stair type A with inclined beams and stair type B having reinforced concrete slab. For each structure, different modelling have been adopted to evidence the influence on the global response of: biaxial bending modelling in the beams of the substructure "stair"; bending moment-axial force (M-N) interaction into the inclined elements (beam and slabs); bending moment-shear (M-V) interaction into the inclined elements and columns.

In general, the presence of stair brings to an increase of lateral strength and to a reduction in displacement capacity with respect to the building without stair (Cosenza et al., 2007a). On the contrary, the results have confirmed the need to utilize biaxial bending modelling and to account for the interaction of the different internal forces (Cosenza et al., 2008) as: bending moment-axial force interaction that characterizes the inclined elements, and the bending moment-shear interaction that governs the behaviour of squat columns. Shear failure becomes predominant in the squat columns and in the reinforced concrete slabs and precedes the conventional ductile failure (see Figure 12).

4.7 NODI: Behavior and Strengthening of Beam-Column Joints

Main results obtained in this Task are made up by the execution and analysis of the experimental program on beam-column joints without strengthening, as well as by the execution and analysis of some tests on strengthened and unstrengthened beam-column joints and on base joints. Numerical-experimental comparisons have been performed, as well, regarding some highly representative test campaigns available in the literature and relevant to some of the tests carried out. A careful literature review was carried out relevant to: (i) experimental programs carried out by other researchers, (ii) joint capacity models, and (iii) code provisions on beam-column joints.

A wide experimental program was carried out on full scale external beam-column joints relevant to typical existing RC buildings having different Earthquake Resistant Design (ERD) level. Quasi-static tests have been performed with 3 loading cycles for each drift value gradually increased from 0.25% up to the total failure of the joint. Results showed yielding force equal to about 20 kN for the NE joints (gravity loads only designed), and about 40 kN for both Z2 joints (designed for seismic zone 2, medium seismicity) and Z4 joints (designed for seismic zone 4, very low seismicity), as a consequence of the minimum requirements on reinforcement amount prescribed by the Italian code.

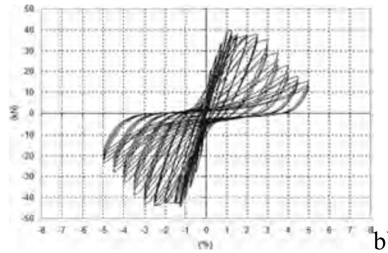
a)   b)

Figure 13. Test results on a Z2 joint (design for medium seismic zone, low axial load): a) damage pattern at drift=7%, and b) force-drift relationship.

Observed failure mechanism in all the joints showed a wide and heavy cracking in the beam, due to the small amount of the longitudinal reinforcing bars as the beams were not loaded by the floor slabs in the considered building model. Joint failure was generally caused by the tensile failure of the reinforcing bars in the beam. A different as well interesting behaviour was displayed by some tests on Z2 joints (Masi and Santarsiero, 2008), where a wide cracking also in the joint panel and a softening mechanical behaviour (Figure 13) were observed, due to the reduced amount of the applied axial force (ν=0.15). Drift value (coincident with chord rotation value) at failure in the NE joints is about 3.0%, while in more ductile Z2 and Z4 joints, values equal to about 4.5% have been detected.

As for the contribution of joint panel own deformations to the total drift of the sub-assemblage, experimental results showed that it is rather low, being always lower than 10% even in the heavily damaged specimens. Further, interesting results have been found by comparing the strength of the joint panel provided by the tests and that one obtained applying the European (CEN, 2004) and the Italian Code (NTC, 2008) expressions to evaluate the capacity of beam-column joints. Results show a good estimation ability of code expressions that have been able to predict which of the specimen was subjected to diagonal cracking of the joint panel. Regarding the ductile capacity, the difference between NE and seismic specimens was lower. In all the tests with high axial load a failure mechanism with extensive cracking of the beam and evident deterioration of the concrete at the beam-column interface has been noted.

A parallel experimental program was devoted to the test (under imposed increasing cyclic displacements applied to the beam end) of 4 external reduced scale beam-column joints provided with smooth bars, later retrofitted after a first series of tests. For 2 of them only the anchorage of the beam bars had been restored by welding threaded bars to the ends of the longitudinal beam bars and bolting them to steel plates placed on the column external surface. For the other 2 joints, besides restoring the anchorage, also vertical and horizontal carbon fiber fabrics had been applied on the column, below and above the joint panel. Moreover, other 2 real scale beam-column joints were built and tested under cyclic loads, either with or without retrofitting, to evaluate the increase of joint performance. As for the first 4 joints, the restoration of the anchorage of the beam bars proved to be very efficient, since it provided increases in strength up to 300% with respect to not retrofitted joints. The additional carbon fiber reinforcements did not provide noticeable increases in strength, because the cracking of the joint, which would have required the carbon fiber contribution, did not occur. Regarding the other 2 joints, it was observed that the not-retrofitted one attained low strength values due to the lack of steel reinforcement for negative moment in the beam. As regards the joint retrofitted before testing, the carbon fiber fabrics applied on the beam significantly increased both strength and displacement ductility.

Additional research activities were mainly focused on experimental tests on full scale RC columns (base joints not-strengthened and strengthened) tested under constant axial load and monotonic or cyclic flexure. The strengthening systems consisted in confinement by partially wrapping unidirectional carbon or glass layers around the element at the base. On some specimens, in addition to confinement, steel angles (in some cases, anchored at the foundation) were placed in correspondence of the member corners. The specimens were designed according to the Italian codes in effect during the ’60s and ‘70s with the aim of reproducing typical dimensions, rebar amount and details common at that time. Main results achieved during the Project are as follows: (i) regardless of the axial load value, the FRP confinement produces significant increases in terms of ductility, especially if a GFRP (glass) jacket is used; (ii) the arrangement of the longitudinal steel angles unconnected to the foundation leads to higher ductility levels than those measured for members strengthened by only FRP systems; this ductility gain is lower for columns tested under $\nu=0.40$, even if in these cases the unconnected angles also provide an improvement of the flexural strength; (iii) when the longitudinal angles are anchored to the foundation the flexural strength of the RC columns significantly increases, but a reduction of the available ductility is observed.
A numerical-type activity focused on the analysis of the performance of RC beam-column joints through numerical simulations by using the F.E. software DIANA, validated by means of experimental test campaigns available in literature (e.g. by the Shiohara working group) as well as some of the tests performed. In particular, many non linear analyses have been performed on typical existing external beam-column joints as they can be found in real buildings built in the past. The definition of typical deficiencies found in real beam-column joints and the analysis of the main parameters governing the structural behaviour of such joints allowed to highlight some effective strengthening solutions, especially for external joints. Further, such work allowed to validate some theoretical models able to predict the behaviour of both external and internal joints, as well as to validate the expressions proposed by some codes to predict the failure of the joint panel.
Some results obtained during the project have been reported in papers published on journals and in proceedings of Conferences (e.g. Masi et al., 2008).

4.8 BIAX: Behavior and Strengthening of Columns under Combined Axial Load and Biaxial Bending and Shear

The presentation of the results follows the research outline that has been above discussed. In particular, methodological and numerical contributions are presented separately from the main experimental findings. This choice is actually related to the nature of the results and their impact on applicative aspects of structural seismic design, namely codes and design and assessment practice.
In particular, the detailed review of technical literature that has carried out during the early stages of the research pointed out the relevance of the type of reinforcement used for construction of existing buildings. In fact, it has been demonstrated that experimental response of r.c. members can be affected by type of reinforcement, smooth or deformed, and by bond interaction between steel and concrete especially in post-yielding phase. As a consequence, a concrete effort has been devoted to perform a comparative analysis of inelastic performances of members depending on type of reinforcement; this means that both numerical and experimental activities have been calibrated in order to cover at local -strength and ductility of cross sections, bond under static and cyclic loads of rebars- and at global – stiffness and rotation capacity of members-.
Results of both theoretical and experimental activities have been published in the context of National and International conferences and meetings. Specific attention has been paid also to

continuous education of young engineers and technical updating for practitioners. Diffusion of software packages (free download of executables and tutorials) and research results has been supplied on Reluis website.

The first set of results refer to the development of methods for numerical analysis of cross sections subjected to generalised bending and axial force. Different approaches have been proposed by the different teams involved in the Task, comparative analyses have been carried out and multiple applicative perspectives have been covered.
A specific software has been developed and made available at the project website (www.reluis.it) for download. It is based on a Fortran language procedure and takes advantage of a user-friendly Visual Basic interface and multiple language platforms. The computational engine of the software has been also used for the development of an extended parametric analysis aimed at the evaluation of the influence of the biaxial actions not only on the ultimate strength but especially on the cross section ultimate curvature. In particular, such influence was studied on square RC cross-section characterized by different values of axial load and geometrical percentage of reinforcement. The study has been developed in order to define simplified analytical formulation to easily predict the ultimate curvature reduction in the case of biaxial bending and axial load with respect to the case of uniaxial bending. Figure 14 gives a view of the program interface (top right), an example of typical results in terms of generalised relationship between the reduction of ultimate curvature compared with the reference uniaxial value ($\eta=\phi_{u,biax}/\phi_{u,uni}$), normalised axial load ν and the inclination of the stress plane angle β (top and bottom left) and finally a comparison between simplified and refined results (bottom right).

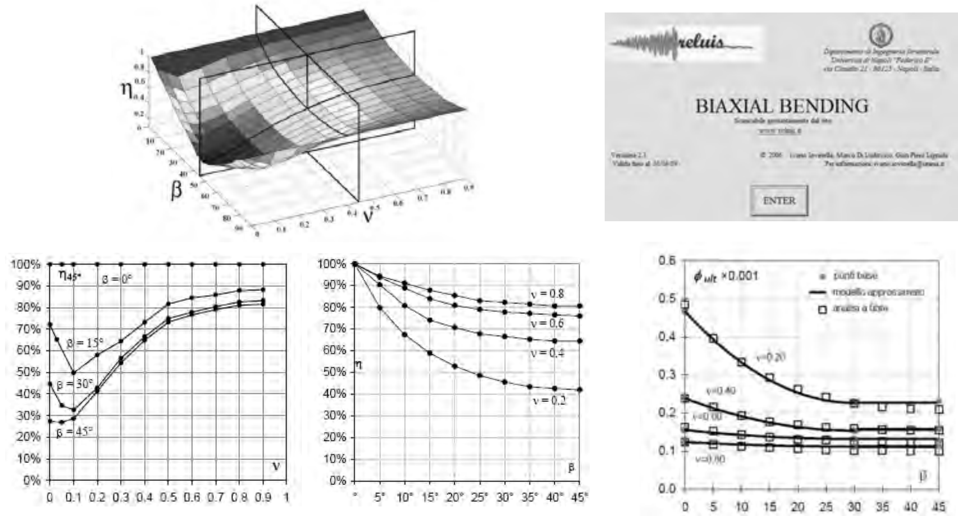
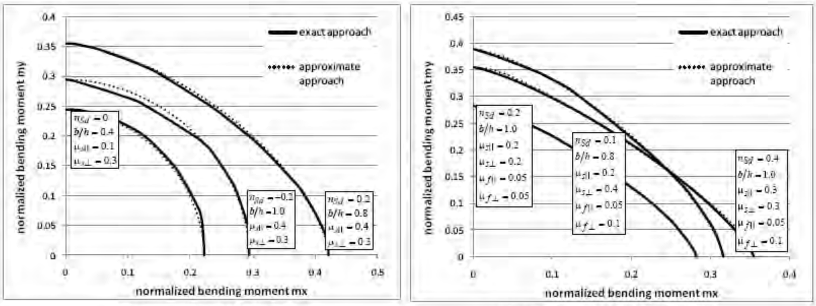


Figure 14. Effect of biaxial bending on cross section local deformation (Di Ludovico *et al.* 2008a, 2008b).

An alternative method has been proposed and implemented. It arrives at defining closed-form equations for performing assessment of existing RC columns with two-way steel reinforcement, under combined biaxial bending and axial load. Starting from the load contour method, an efficient procedure for estimating the strength/deformation section capacity has been developed. In addition, simple closed-form equations for computing section uniaxial resisting moments and ultimate/yielding curvature has been defined.



Ustrengthened sections FRP-strengthened sections

Figure 15. Comparison between exact (fibre method) and approximate approaches for bare (left) and FRP reinforced r.c. members (right) (Monti and Alessandri, 2008).

The method has been also extended to cross sections reinforced with FRP, resulting in a very effective guide for fast implementation of results in more general software packages and direct application by structural engineers using easy to manage electronic sheets.

Figure 15 shows an example of results that can be obtained according to this design tool, both for bare and FRP-strengthened members and its ability to give reliable results with a reduced computational effort.

Another relevant achievement is related to the response of the whole member under axial force and biaxial bending. In particular, the attention has been focused on the development of the actual stiffness of the member depending on the stress level and influence of biaxial bending. A fibre model of the member has been implemented in order to provide recommendations for characterization of equivalent stiffness to be used in elastic analysis. The study carried on has been limited, at the moment, to the definition of the problems which must be taken into account while passing from a cross section to the whole member, Bosco *et al.* 2008. In this context, it is worth noting the contribution of UNICH that carried out nonlinear analyses of reference regular and irregular reinforced concrete buildings and on problems related to the selection of input ground motion, Canducci *et al.*, 2008.

In compliance with the main issues derived from the theoretical analysis of r.c. members and cross sections, experimental activity has been carried out at different scales.

At local scale, a number of bond tests on smooth rebars were devoted to the definition of a constitutive stress-slip law to be used in numerical simulations.

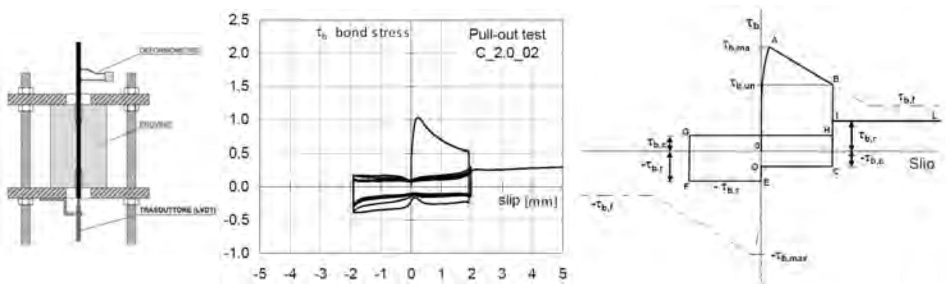


Figure 16. Fibre model of the cross section (left), force-displacement curve for a column (center), normalised equivalent stiffness (right) (Verderame *et al.*, 2008a, 2008b).

Figure 16 reports sample data concerning specimens (left), typical cyclic experimental data (center) and finally the idealized constitutive law calibrated against the test (right).
At large scale, eight experimental tests on r.c. square or rectangular full scale columns under constant axial load and uniaxial bending were performed. Monotonic or cyclic action were applied on specimens. Details about the experimental program are reported in Table 3 and Figure 17. Each test was performed under displacement control.

Table 3. Summary of experimental tests on r.c. columns.

	UNIAXIAL TESTS - Cross Section BxH (cmxcm)			
	30x30	**30x30**	**50x30**	**30x50**
Longitudinal Reinforcement	8Φ12	8Φ12	12Φ12	12Φ12
Steel Rebars type	Plain	Deformed	Plain	Deformed
Normalized Axial Load, ν	0.2	0.2	0.1	0.1
Type of action	Monotonic	Cyclic	Cyclic	Cyclic
Number of tests	2	2	2	2

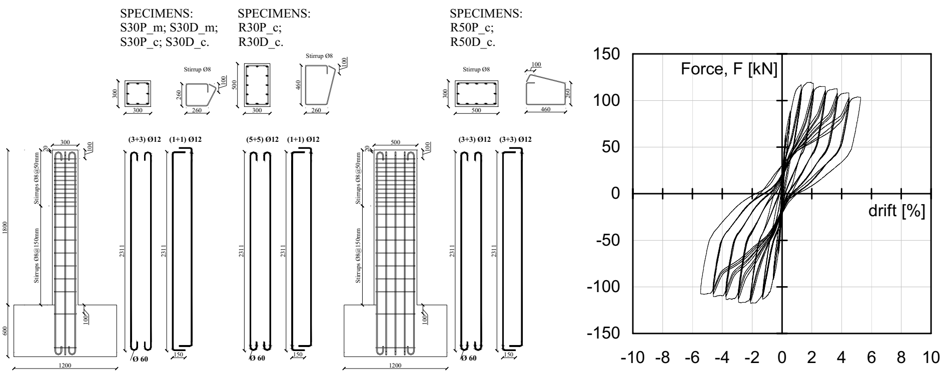


Figure 17. Experimental tests on columns.

Primary experimental outcomes clearly indicate that the contribution of the base rotation on the global deformation mechanism is noticeably different in case of columns reinforced by plain or deformed rebars, however, the overall member global deformation and energy dissipation capacity is not strongly affected by the bond performances of the internal rebars. The global deformation capacity in the case of plain rebars, is mainly due to a localized source of deformability at the column foundation interface (fixed end rotation), while a more diffused crack pattern along the column end was observed in columns reinforced by using deformed steel rebars. The significant influence of P-Δ effect on the global behavior of specimens has also clearly emerged by the experimental tests; if such effect is neglected, the ultimate rotation recorded on columns reinforced by deformed steel rebars is clearly less than that observed in columns reinforced by using plain rebars. However, due to P-Δ effect the ultimate rotations related to the two different columns typologies is very close. Such result can be explained by considering two main aspects: the higher strength of members reinforced with deformed rebars and the higher slope of the softening branch of the shear-drift curves, if P-Δ effect is considered. A calibration of a numerical model able to take account of specific aspects related to bond of smooth rebars and anchoring end details has been also developed (Verderame *et al.*, 2008c).

4.9 PREFAB: Behavior and Strengthening of Prefabricated Industrial Structures

The first activity consisted of a wide survey of the existing buildings produced from the '50s up to today and of the rendering in a reasoned way of the investigation results. This activity enjoyed the support of the National association of prefabrication industries Assobeton which involved ten member companies to provide the design documentation of a number of constructions built in different times. So about 150 project documentations have been collected covering some decades of production. Since information about far times were lacking, this survey has been integrated with the historical memory of some experts, exploiting their knowledge together with the old bibliography of specific journals.
From the examination of the project documentations a synthetic description for any building has been recorded in a standard format, summarising its features in a specific form of easy reference. This work led to the printing of the booklet "Precast structures: list of projects of existing buildings" which provides a good evidence of some decades of precast production.

Figure 18. FrontPage cover of two produced catalogues on precast concrete buildings.

A complete catalogue of the different types of precast structures has been drafted and published in a specific booklet (Figure 18). Every type has a short description with sketches and indications about the years of production, the regions of destination and the relative diffusion with respect to the global precast production. Some notes are added with indication of possible behaviour deficiencies for seismic destinations.
Being a key point of precast construction, special attention has been addressed to the seismic behaviour of connections. Some tests have been performed to quantify their seismic capacity following a standard approach.
Five principal categories of connections have been considered:
- *floor-to-floor* connections between adjacent floor or roof elements;
- *floor-to beam* connections between floor or roof elements and the supporting beam;
- *beam-to-column* connections between the beam and the column;
- *column-to-foundation* connections which provide the base support to the columns;
- *cladding-to-structure* connections for the support of the wall panels.

Two level of tests have been performed:

- *particular tests:* referred to the qualification of single connectors inserted between two overdimensioned blocks and subjected to the principal action expected in the structural system (Figure 19, left);
- *local tests:* referred to the qualification of the connection included between two significant portions of the elements, representing the structural arrangement and subjected to the relevant components of the action (Figure 19, right);

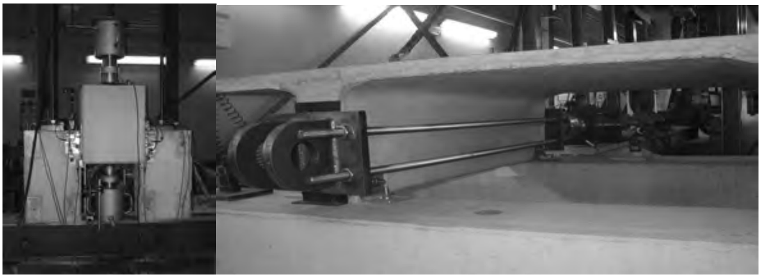


Figure 19. Details of testing of structural connection.

Previously a standard protocol for testing has been drafted. It defines the six parameters:
- *strength*: maximum value of the force which can be transferred between the parts;
- *ductility*: ultimate plastic deformation compared to the yielding limit;
- *dissipation*: specific energy dissipated through the load cycles;
- *deformation:* ultimate deformation at failure limit;
- *decay*: strength loss through the load cycles compared to force level;
- *damage*: residual deformation at unloading compared to the maximum displacement;

to be measured both by:
- *push over tests* following a monotonic increase of displacement:
- *cyclic tests* following an alternate history of displacements (Figure 20).

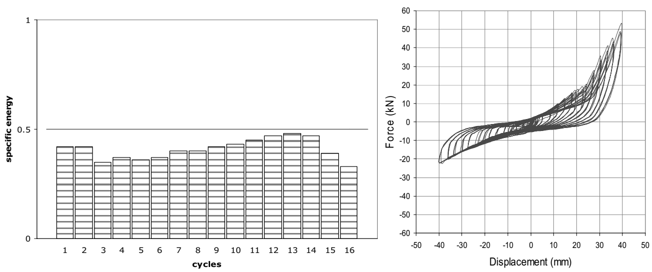


Figure 20. Specific energy and cyclic behaviour of the connection.

A number of tests on roof-to-beam connections (8 push-over and 11 cyclic) have been performed. A complete report is available with the results of the experimentation. Three types of steel connectors have been examined (Felicetti et al., 2008).
In parallel, a testing frame has been set up for push over and cyclic tests on beam-to-column connections. The common type with a couple of bars protruding from the column and passing through the holes of the beams is examined both in longitudinal and transverse direction.
The experimental test results in terms of force displacement allowed to characterise the connection roof element to beam, as global ductility, ultimate strength and dissipative

capacity. In Figure 21, force-displacement curves are illustrated (monotonic test on rigid blocks) comparing the traditional existing connection with a modified solution of the connection. While a brittle behaviour is expected for the traditional existing connection (concrete crash of the beam edges), a ductile behaviour is then obtained if the steel plate in the connection is opportunely reduced. Almost similar conclusions can be obtained for quasi-static cyclic tests as reported in the research reports and papers.

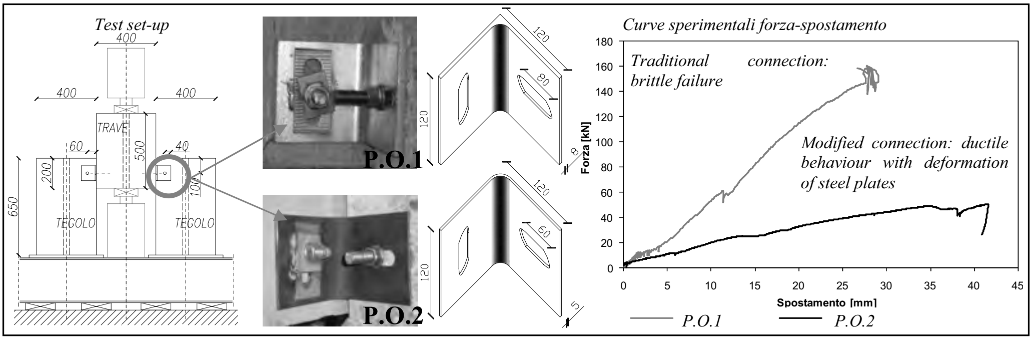


Figure 21. Push-over test: comparison of the two different solution (traditional vs. slightly modified).

Some other tests have been performed on dry bearing in order to measure the friction factor of neoprene pads over the concrete surface and their deformation parameters. Tilting tests and on inclined plane have been made with different levels of normal loads (Magliulo et al., 2008).
The comparison between tests results and friction coefficient values provided by PCI Handbook (1999), CNR 10018 (1999), and UNI-EN 1337:3 (2005), is shown in Figures 22 and 23. In Figure 22 the neoprene compressive stress (σ) is reported on the horizontal axis, while the shear one (τ) on vertical axis; in Figure 23 on this axis the friction coefficient is presented. It is evident that PCI Handbook and CNR 10018 curves well approximate the experimental data linear regression curve; this does not happen in the case of UNI-EN 1337:3 curve. However CNR 10018 provides a bit larger friction strength with respect to the experimental results, while PCI Handbook provides larger friction strength only for compressive stress lower than 3 N/mm^2. Furthermore, the tests results confirm the light increment of friction strength, which corresponds to a light decrement of the friction coefficient, as the compressive stress increases.
On the base of experimental results, the following relationships for neoprene-concrete friction coefficient are proposed:

$$\mu = 0.49 \quad \text{if } \sigma_v \le 0.15 N/mm^2$$

$$\mu = 0.1 + \frac{\beta}{\sigma_v} \quad \text{if } 0.15 < \sigma_v \le 5 N/mm^2$$

where σ_v is the compressive stress and $\beta = 0.055$; σ_v=5 N/mm^2 is the neoprene maximum compressive strength according to CNR 10018. These formulations along with the linear regression curve of tests mean results are plotted in Figure 24: the two curves are almost coincident.

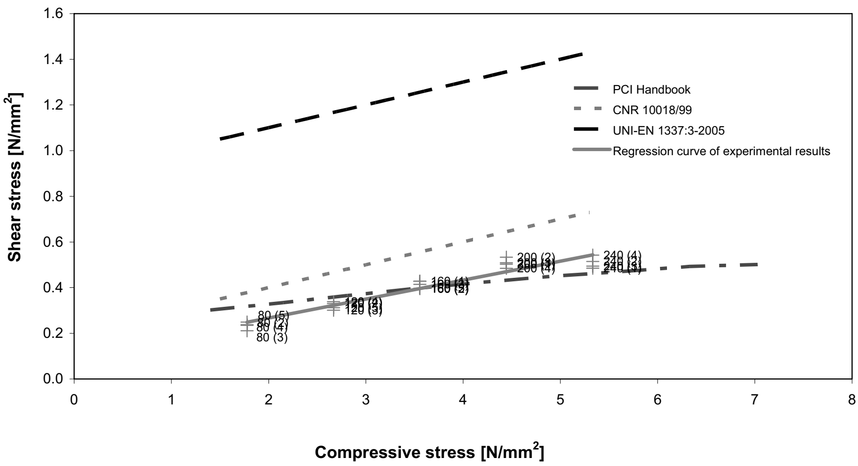


Figure 22. Comparison between compressive-shear stress curves provided by PCI Handbook, CNR 10018 and UNI-EN 1337:3 and tests regression curve.

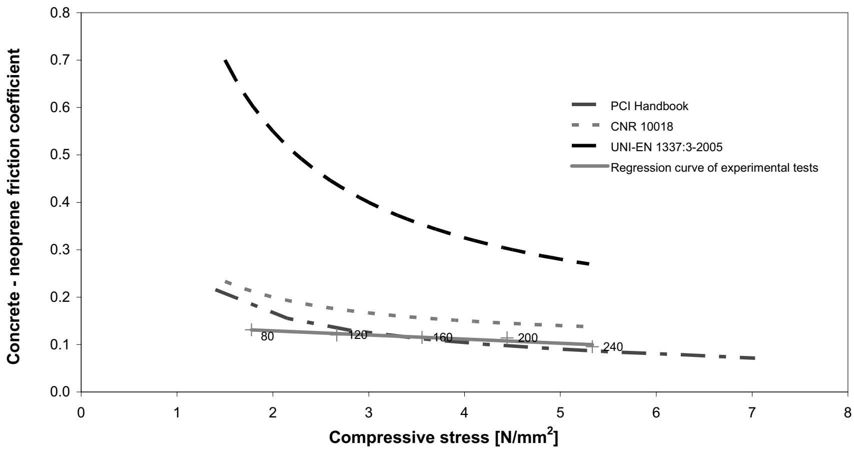


Figure 23. Comparison between compressive stress – friction coefficient curves provided by PCI Handbook, CNR 10018 and UNI-EN 1337:3 and tests regression curve.

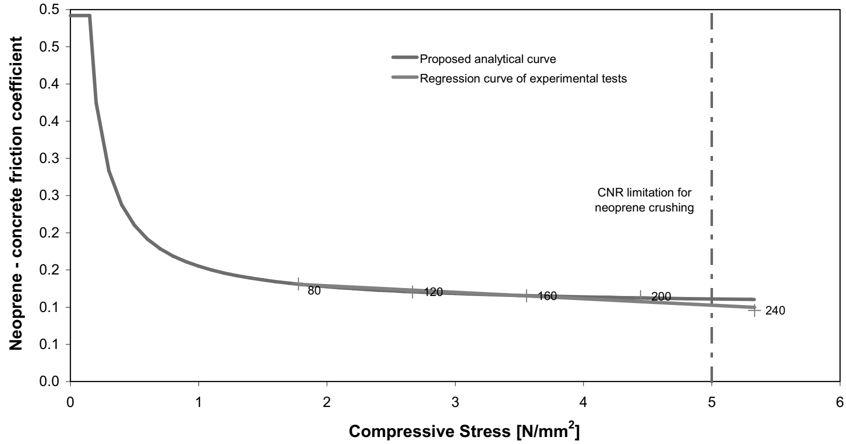


Figure 24. Proposed compressive stress - concrete–neoprene friction coefficient relationship.

5 DISCUSSION

5.1 MND: Non-Destructive Methods for the Knowledge of Existing Structures

The research activities scheduled within the Task have been to a great extent carried out without significant delays or anticipations, and the main objectives have been achieved.

5.2 FC: Calibration of Confidence Factors

The results obtained agree with those expected, concerning the development of a Bayesian procedure for material strength evaluation and the calibration of confidence factors.

5.3 IRREG: Assessment of the Nonlinear Behavior of Buildings, with Emphasis on Irregular Ones

The results obtained by the Task are in line with the objectives originally established. Several papers by the different research units have been published or are in press in international journals or conferences. The only problems encountered by some of the units originated from convergence issues in nonlinear codes. This is a well-known problem, but some software failed to converge on a regular basis, thus delaying advances in the research. However, overall, the task followed the schedule of work originally outlined.

5.4 MIX: Assessment and Strengthening of Mixed-type (Masonry/RC) Buildings

The Task has pursued the proposed objectives. In particular, aspects of modelling the behaviour of mix-type buildings through non linear analysis have been investigated. These analyses have highlighted the different steps in which the resistant elements withstand the seismic action.

The analyses performed allowed the identification of the main parameters affecting the structural behaviour of mixed building and gave indications on feasible modelling and verification criteria.

5.5 TAMP: Influence of Infills on Structural Response

The objectives that have been pursued by this research are consistent with the expected ones. Nevertheless, it must be observed that the experimental calibration of the parameters defining the cyclic behaviour of the proposed equivalent diagonal strut model is affected by the following main limitations: - it has been made considering only square meshes of infilled RC frames; - the possible presence of an infill with opening has not been considered.
These limitation, due to not sufficient time and resources, did not allow the model to be validated by its use to reproduce the experimentally detected response of the 3D infilled building subjected to the shaking table tests.

5.6 SCALE: Behavior and Strengthening of Stairs

The Task has pursued the proposed objectives. A detailed investigation of the main stairs typologies and of the most used design procedures have been performed; in particular, a report including the main results was developed (Cosenza et al., 2007b). Numerical investigations have been performed in order to understand the seismic behaviour, and the possible failure mechanisms of buildings having the most common stair typologies. The results have confirmed the need to utilize biaxial bending modelling and to account for the interaction of the different internal forces as: bending moment-axial force interaction that characterizes the inclined elements, and the bending moment-shear interaction that governs the behaviour of squat columns. Shear failure becomes predominant in the squat columns and in the reinforced concrete slabs and precedes the conventional ductile failure. An experimental set-up has been designed on the basis of some simulations performed by using different modelling: dimensions of a single span frame with inclined beam, loads and resisting-wall have been defined.

5.7 NODI: Behavior and Strengthening of Beam-Column Joints

The research activities scheduled within the Task have been to a great extent carried out without significant delays or anticipations, and the main objectives have been achieved.

5.8 BIAX: Behavior and Strengthening of Columns under Combined Axial Load and Biaxial Bending and Shear

Summary of results reported in the previous sections leads to recognize that the development of the work basically complies with the initial schedule. In particular, as numerical analyses and software deliverables are considered, a good agreement with the program can be addressed.
In fact, different approaches and numerical procedures have been set and made available to technical community. They cover at different levels the need of tools for checks required by modern codes in terms of strength and local deformation. This applies both to design and assessment of existing un-strengthened concrete structures and to seismic upgrading using FRP materials. Interaction between groups involved in the study of irregular structures and of use of FRP for seismic upgrading of structures is another positive aspect of the work.
When experimental program are concerned, it is worth noting that a relevant contribution to the knowledge of bond mechanisms for smooth bars has been given and an approach to the comparative analysis of performances in terms of rotation capacity of full scale r.c. members with smooth and deformed bars has been accomplished. The work is not actually exhaustive, since trial biaxial tests have been designed depending on the findings of the theoretical work, but not completed. This results in the need to extend and validate the results obtained in the

context of the present task and give a direct contribution to the development of specific design recommendations for members under combined axial load and biaxial bending.

5.9 PREFAB: Behavior and Strengthening of Prefabricated Industrial Structures

The research activities scheduled within the Task have been to a great extent carried out without significant delays or anticipations, and the main objectives have been achieved.

6 VISIONS AND DEVELOPMENTS

6.1 MND: Non-Destructive Methods for the Knowledge of Existing Structures

A large amount of RC buildings, both private and public, now placed in seismic zones, were originally designed taking into account only gravity loads and without explicitly provide ductile detailing. An extraordinary rehabilitation program needs to be implemented on such buildings, where an accurate evaluation of the available seismic capacity is important to set up cost-effective interventions. Investigations have a crucial role to adequately know the structure to be evaluated. For this reason, there is an increasing need to set up and put at disposal of technicians and other involved stakeholders sufficiently reliable as well as not very expensive methods to estimate in-situ material properties. Number of tests required to suitably apply these methods have to be as low as possible, thus making the total required budget sustainable to building owners and thus further encouraging their use. To this purpose, the results obtained in this Task confirm that a smart combination of NDTs and direct tests (such as core extraction) gives effective solutions from both the economical and technical point of view.

Future research work should be devoted to the following:

- Provide methods more and more capable of achieving effective results in terms of prediction capability of concrete properties taking into account both intrinsic randomness and epistemic uncertainty.
- NDTs currently available on concrete do not provoke damage on structural members but on some other building components (e.g. partitions, infills, plaster, etc.) thus determining remarkable repair costs: new methods are necessary to make them really not very expensive.
- As for reinforcement, taking into account the heavy damage caused by the extraction of steel bars, non destructive methods to estimate its mechanical properties need to be set up.

6.2 FC: Calibration of Confidence Factors

The work performed by the Task has focused on problems related to the definition of a reliable assessment of structural seismic performance.

A probabilistic model for structural performance has been developed and a method for calibration of Confidence Factors has been proposed.

Furthermore, a procedure for evaluation of material strength has been developed based on the application of the Bayesian method, taking into account, both, amount and reliability of the in-situ tests performed.

In addition to that, with reference to the prior probability distribution of structural detailing parameters, a preliminary list of possible structural defects was prepared, in which, for each defect, a set of possible values and their relative weights are envisaged.

To proceed further along this path, the following developments are needed:

- A complete procedure should be developed defining all phases of the knowledge process of an existing building. The procedure shall give a CF calibrated on the basis of the distribution of the assessment results conditional on the acquired knowledge.
- Further studies are needed regarding the evaluation of material strength values from in-situ non-destructive tests, whose reliability depends on the reliability of the regression curves used to transform the test parameter into a material strength value.
- A questionnaire, meant to be addressed to professional engineers as a survey, has been prepared. The results of such survey shall be useful in creating a thorough database of structural defects and their probability distributions.

6.3 IRREG: Assessment of the Nonlinear Behavior of Buildings, with Emphasis on Irregular Ones

Several directions for future work have emerged from the project, most of them related to seismic code enhancements:

- Further studies for assessing the applicability of nonlinear pushover procedures to plan irregular buildings;
- Further studies on different engineering demand parameters, such as interstory drift, chord rotation, plastic hinge rotation, as means for assessing the structural response;
- The need for the definition of the damping to be used in NTHA, depending on the model level of refinement. Using 5% damping results in an un-conservative assessment;
- A clear definition of spectrum compatible accelerograms (according to EC8) to be used in NTHA;
- The need to establish a framework for Performance Based Earthquake Engineering, based on a fully probabilistic approach;
- Assessment of the current modeling capabilities for shear failure prediction, with the possible development of simplified approaches that can provide an engineering estimate without adding complexity to the convergence procedure;
- Possible guidelines on how to consider other sources of nonlinearities/failures, such as bond-slip, beam-column failure, etc.

6.4 MIX: Assessment and Strengthening of Mixed-type (Masonry/RC) Buildings

Considering the obtained results, the development of research aimed at evaluating the seismic behavior of mixed-type buildings can be envisaged as follows:

- Establish through non-linear analyses the influence of RC-masonry connections on the global behavior of the building. Particular attention should be paid at intersections between beams and perimeter masonry walls.
- Evaluation of q-factors. This objective should be pursued through the implementation of (static and dynamic) non-linear analyses.
- Evaluation of the seismic response (numerical and not only) of other structural configurations of mixed-type buildings not included in this study.

6.5 TAMP: Influence of Infills on Structural Response

This research gave useful results concerning the effects of masonry infills on the lateral response of RC frames, supported by experimental validations.

Possible developments could be aimed to pursue the following further main objectives:

- to revise the available empirical expressions linking the masonry properties to those of its components, by carrying out further tests on masonry samples made of other kinds of resisting elements;
- to generalize the proposed model of equivalent diagonal strut, by evaluating experimentally the role of factors that have not been considered here: infilled mesh geometry, different compression levels on the frame columns after the masonry infills have been constructed, presence and size of an opening in the infilled mesh of frame.

6.6 SCALE: Behavior and Strengthening of Stairs

Considering the obtained results the future research developing would be oriented to the evaluation of the influence of the stair on the seismic response analysing the following aspects:

- Experimental assessment of the strength capacity and deformability of squat columns. In this way, the behaviour of squat columns would be better understood considering the large number of models found in literature that are not completely exhaustive for a reliable assessment of the seismic behaviour of reinforced concrete structures.
- Experimental evaluation of the seismic response of 2-D and 3-D frames with inclined elements and squat columns. The comparison of different test results would allow to evidence the influence of the introduced specific elements such as inclined beams and squat columns.
- Numerical and experimental evaluation of the influence of the stair in buildings with different location of infill walls. The numerical analysis would be performed by using static and dynamic tools. This studies would allow the evaluation of the interaction between stairs and infills that are commonly modelled in different manner.

6.7 NODI: Behavior and Strengthening of Beam-Column Joints

The behaviour of beam-column joints can strongly affect the seismic global behaviour of RC building structures. Some mechanisms (e.g. concrete cracking, slippage of longitudinal reinforcing bars, etc.) can be responsible of additional deformability, on one hand, while, on the other hand, can alter the capacity design assumptions on the framing structural members (beams and columns) and on the joint member itself. For this reason, research activities needs to be increasingly devoted to develop accurate capacity models of beam-column joints to reliably evaluate the performances of RC building structures.
Research carried out in this Task already provided important results relevant to the role of the key behavioural parameters of RC beam-column joints, thus giving useful suggestions on the reliability of current code expressions and on possible improvements. However, many other issues needs to be further studied regarding both as-built and strengthened joints (already damaged or not damaged), skilfully combining purposely designed experimental investigations, review of experimental campaigns reported in the literature, and accurate numerical simulations.
Among others, some future research developments can be recognized as follows:

- design and execution of extensive experimental programs on joint specimens having different characteristics (e.g. internal or external, bi- or tri-dimensional, shape, beam type, etc.) well targeted on the types representative of the Italian and European built environment;
- experimental and numerical validation of different strengthening techniques based on the comparison of the relative performance and application limits, particularly dealing with tri-dimensional joints and joints with embedded beams;

- codification of test protocols to make possible and promote experimental result exchange.

6.8 BIAX: Behavior and Strengthening of Columns under Combined Axial Load and Biaxial Bending and Shear

This Task research activity dealt with a specific, but relevant aspect of seismic design and assessment of r.c. constructions. An effective and fruitful approach to the development of tools for the theoretical estimation of strength and curvature ductility of members has been carried out. Flexural mechanisms are clearly identified both at local and global scale and an encouraging capacity of simulation is demonstrated by both static and cyclic proposed models of members with smooth bars. This means that numerical sensitivity analyses can give a positive contribution to an optimized design of an experimental program able to confirm numerical forecasts, show possible points of weakness of the theories and/or give an insight on specific aspects of the behavior under severe cyclic loads. The large variety of existing constructions and the diffusion of smooth bars in many very urbanized areas exposed to seismic risk point out the need to continue the investigation on such type of members and even develop customized strengthening techniques using advanced materials.

Despite such positive feedbacks of the research, it is worth noting that further work is strongly recommended to assess:

- the behavior of short columns, and
- the flexure-shear interaction in presence of smooth reinforcement.

In fact, the observed deformation mechanisms can produce effects on the strength and ductility of members subjected to complex forces, like those generated on columns of irregular constructions.

6.9 PREFAB: Behavior and Strengthening of Prefabricated Industrial Structures

The experimental campaign and the numerical investigations carried out have highlighted that the most vulnerable buildings are those with disarticulated diaphragm behaviour. However, as emphasised in Palermo et al. (2008), an accurate study on the modelling of connections needs to be done in order to correctly predict the overall response of precast concrete buildings.

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In the following sections, all the publications produced by Research Line 2 during the Project are listed, for a total of 180.

Since 2005, starting year of the Project (where only November and December are considered), until the end of 2008, the number of publications has developed as shown in Figure 25.

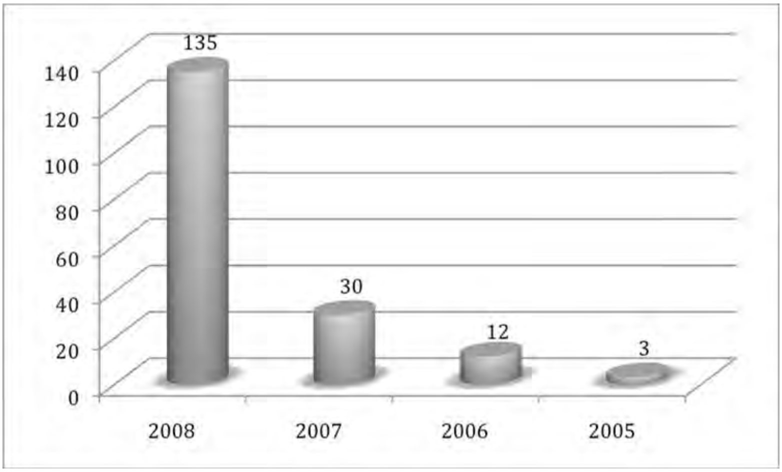

Figure 25. Number of publications produced by RL2 from 2005 to 2008.

The publications are distributed among the 9 Tasks as shown in the following figure.

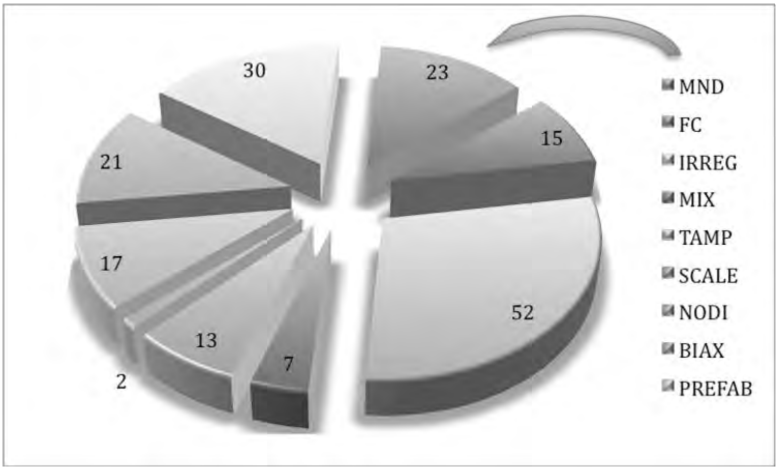

Figure 26. Number of publications produced by RL2, distributed among the nine Tasks.

The following figure shows how publications are distributed among Journal Papers, Reports, Papers presented at the ReLuis2Rm08 Conference, Papers presented at the 14th World Conference on Earthquake Engineering, Papers presented at ANIDIS Conferences, Papers presented at the EWICS Conference, and Papers presented at other Conferences.

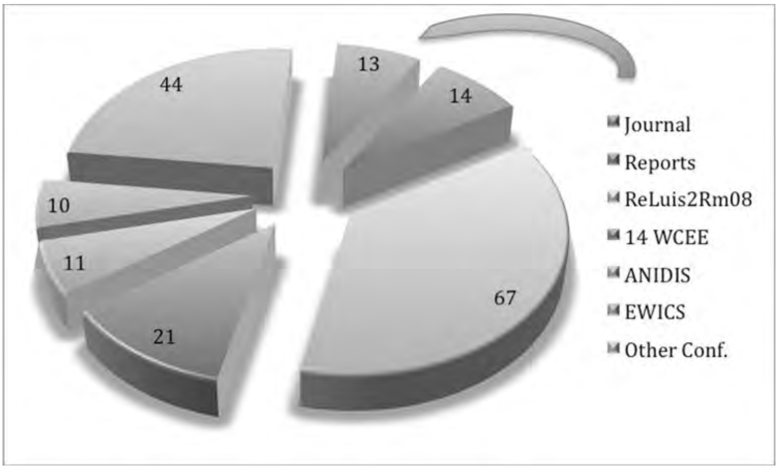

Figure 27. Publications produced by RL2, distributed according to type.

The following two figures show, respectively, the relative percentage of publications in Italian or in English, and the relative percentage of publications produced within a single RU, more RUs and with collaborations abroad.

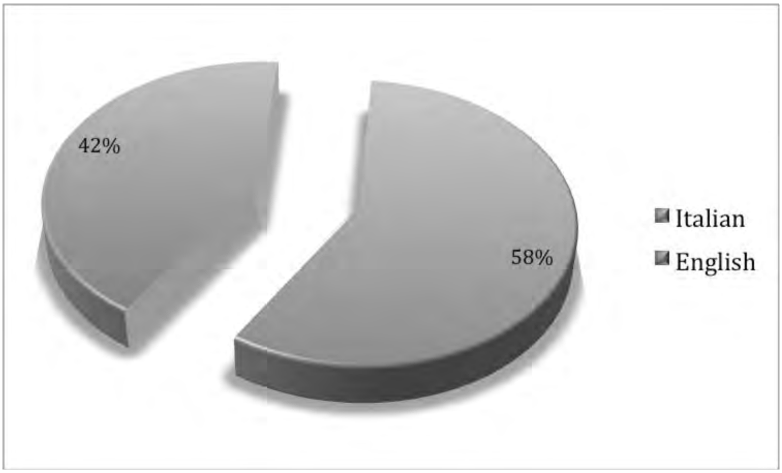

Figure 28. Publications produced by RL2, distributed according to language.

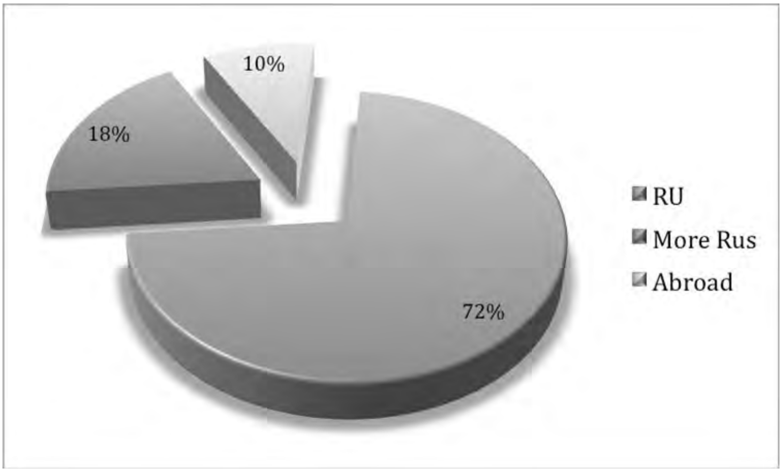

Figure 29. Publications produced by RL2, distributed according to collaborations.

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SEISMIC ASSESSMENT AND RETROFIT OF EXISTING BRIDGES

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1 INTRODUCTION

The perception of the risk associated to the seismic vulnerability of the transportation infrastructure, and in particular to that of bridge structures, on the part of both the relevant authorities and the profession is a quite recent acquisition in Italy. This is possibly due to the fact that in the last two major events that have struck the Country in the second half of the 20th century (Friuli 1976 and Irpinia 1980) the transportation infrastructure has not suffered significant distress. In particular, in Friuli the construction of highways was just at the beginning. In the Apennine crossing of the A16 highway the bridges did undergo some damage, mainly due to the inadequacy of the bearing devices, but this was promptly remedied by the owner through the systematic adoption of the then innovative technique of seismic isolation.

On the other hand, it can be observed that this delay in the appreciation of the risk is not exclusive to Italy. For example, it is enough to mention that it took twelve years after the spectacular failures of quite modern bridges (Figure 1, left) during the San Fernando (1971) earthquake, for the Federal Highway Administration (FHWA) to publish a first document titled "Retrofitting guidelines for Highway Bridges" (FHWA-ATC, 1983). Still, in 1989, despite of the large retrofit program set up (later proved to be fully inadequate), the Loma Prieta earthquake exposed substantial deficiencies in bridges in California (Figure 1, right).

Figure 1. Damage to bridges during the San Fernando, 1971 (left) and Loma Prieta 1989 (right) events.

The situation as briefly outlined above is sufficient to understand that the state of the art on seismic assessment and retrofit of bridges still needs to be advanced in several areas. The research undertaken within this Line of the DPC-Reluis Project aimed at providing a contribution in this direction. The areas considered to be of prioritary interest were assessment methods, retrofit criteria and techniques, abutments and foundations, with the final goal of producing a comprehensive document with guidelines and example applications. This result, which has been achieved, represents the first European document on the topic and could be envisaged to form the basis for a future addition to the Eurocodes system.

2 BACKGROUND AND MOTIVATION

Starting from the year 1992 on funding from the FHWA, a vast research program has been undertaken in the US to clarify several aspects related to the seismic assessment and retrofit of bridges.
The first product of the above research appeared in 1995 in the form of the “Seismic Retrofit Manual for Highway Bridges” (FHWA, 1995): its further development has led to the “Seismic Retrofitting Manual for Highway Structures: Part 1 Bridges” and the “Seismic Retrofitting Manual for Highway Structures: Part 2 Retaining structures, Slopes, Tunnels, Culverts and Roadways” (FHWA- MCEER, 2005).
In Europe the Eurocodes system includes a normative document for the seismic design of new bridges, which is at least partially based on the recent concepts of performance-based design: Eurocode 8 Part 2 (CEN, 2005a). This document, however, is not matched by a companion one covering existing bridges, differently with the situation of buildings, for which such a document is available in the form of Eurocode 8 Part 3 (CEN, 2005b).
In the year 2003 a firm change of direction towards the harmonisation with Eurocode 8 has occurred in the Italian normative framework for seismic design. In that occasion the priority was given to the drafting of documents for the design of new structures, both buildings and bridges. A document for existing structures was also introduced, but again limited to buildings. These documents served later as the basis for the production of the seismic chapter of the current Eurocodes-aligned national design code produced by the “Ministero delle Infrastrutture” (DM2008).
The need for a document dealing explicitly with the problem of assessing and retrofitting bridges in seismic areas dates back actually to 2003, when the update of the seismic design code was accompanied by the obligation of assessing, within the time limit of five years, all the strategic structures and infrastructures in the Country. Adhering to this obligation and with reference to bridges, with funding from the Civil Protection Department, the National Agency for Roads and Highways (ANAS) has launched a program for the assessment of all its bridge structures. Further, the theme is of pressing interest due to the widespread activity currently ongoing on the Italian highway network to increase its traffic capacity.

Within the above context the DPC-Reluis research project, with its Line 3 is intended to respond to the outlined needs, and in particular to that of producing a document to be used as a proposal of a future normative text on the seismic assessment and retrofit of existing bridges.

3 RESEARCH STRUCTURE

Research Line 3 had seven participants: Politecnico di Torino (coord. Prof. G. Mancini), Università di Pavia (coord. Prof. A. Pavese), Università di Genova (coord. Prof. L. Gambarotta), Università “G. D’Annunzio” di Chieti-Pescara (coord. Prof. E. Spacone), Università di Roma "La Sapienza" (coord. Prof. P.E. Pinto), Università di Roma Tre (coord. Prof. R. Giannini) and Università di Cosenza (coord. Prof. A. Vulcano).

As it has already been stated, the main objective of the program was to progress the state of knowledge in the area of the seismic assessment and retrofit of existing bridges to a degree adequate to support the drafting of a pre-normative document.

In this direction the activity of the program was articulated into five main tasks:

1. Identification of bridge typologies:
   Bridge typologies characterising the Italian road- and railway systems were identified through contacts with the main national and regional administrations, as well as with major contractors. In particular, existing contacts were exploited and new ones were established with ANAS, Autostrade, Italferr, RFI, Ferrovie della Calabria in order to acquire detailed documentation on a number of representative structures.
2. Assessment methods:
   Existing methods developed for the assessment of buildings were extended to the deal with bridge structures, and methods specifically devised for bridges were further developed. The goal was to fine-tune several methods of increasing level of accuracy and required effort to be used according to the importance and size/regularity of the bridge. These include displacement-based linear and non linear static methods, as well as simplified behaviour-factor-based methods. Special attention was also devoted to modelling for non-linear analysis.
3. Retrofit criteria:
   This task focussed on the specific aspects of the application of traditional and innovative retrofit techniques to structural elements typical of bridges. The program of activity included the execution of an experimental campaign aimed at establishing the effectiveness of alternative retrofit techniques. This task also included the development of seismic isolation solutions to be applied in the retrofit of bridges that were not initially designed to be isolated, as well as bridge-specific seismic isolation design criteria.
4. Assessment of abutments, earth-retaining structures and foundations:
   Abutments and foundations are often weak elements in existing bridges. The goal of this task was to advance the state of the art in the seismic analysis and assessment of these components, an area still characterised by the widespread use of mostly empirical or conventional approaches.
5. Model applications to bridges of different typologies:
   It was planned that under this task at least one bridge for each of the main typologies identified under task 1 was to be subjected to a detailed assessment and retrofit design, and then documented in an application manual to complement the pre-normative document.
   The guidelines and the companion manual represent the main outcome of the project.

The five broad tasks outlined above were split into a number of sub-tasks according to the table below. The table also shows the progress of activity over the whole project duration of three years. The progress achieved during the second year is briefly summarised in the following.

Table 1. Subdivision of the research activity into tasks and sub-tasks.

Task	Sub-Task	Year		
		1	2	3
1	Identification of bridge typologies			
2	Development of analysis, modelling issues			
	Definition of testing protocols			
	Development of assessment methods			
3	Retrofit criteria			
	Definition of test program			
	Execution of experimental tests			
	Interpretation of results			
4	Abutments and foundations			
5	Numerical application to case-studies			
*	Guidelines: first draft			
	Guidelines: second draft			
	Guidelines: final draft			
	Application manual			

4 MAIN RESULTS

The main results of the research activity are summarised in the following according to the research structure described in the previous section.

4.1 Task 1: Identification of bridge typologies

The main bridge typologies on several seismic-prone portions of the Italian railway and road/highway networks have been identified during the first year of activity. In summary, the data collected from various sources (mainly national/regional administrations) pertain to the Torino-Bardonecchia-Frejus (TBF) and the Parma-La Spezia (PLS) highways (Politecnico di Torino), the Firenze-Bologna (A1FiBo) portion of the A1 Milano-Napoli and the Apennine portion of the A16 highways (Università di Roma “La Sapienza” and Università di Roma Tre), the Adriatic portions of the A14 Bologna-Canosa highway and of the SS16 Adriatica state-road (Università di Chieti-Pescara), the Roma-Viterbo (RMVT) and Roma-Sulmona (RMSu) railways (Università di Roma Tre), the regional railway and roadway networks of Calabria (Università di Cosenza).

Structural typologies characterizing the TBF and the PLS highways are quite different. The first highway, built in between 1983 and 1992, includes rather uniform typologies: a) about 300.000 m^2 of precast segmental box girder bridges with pier heights up to 90m and span lengths between 40 and 100m, b) about 200.000 m^2 of girder bridges in concrete and in composite steel-concrete with pier heights between 5 and 30m and span lengths between 20 and 80m. Representative bridges are: the Borgone viaduct (20+26×40+20 m), the *Ramat* viaduct (50+9×100+50 m and tall piers), *Bardonecchia* bridge (7×42 m) and the *Millaures* bridge (6×80 m, composite steel-concrete). The second highway, built between 1965 and 1975 shows a greater typological variability, which can be reduced, however, to a few homogeneous sets. Representative bridges are: the *Borgotaro* viaduct (slab bridge with several interconnections), the *Narboreto* bridge (4×30 m), the *Rio Verde* viaduct

(2×65+6×95+76 m and very tall piers, h=150 m) and the *Roccaprebalza* South viaduct (13×45 m and tall piers), *Rio Barcalesa* (7×43m).

As it regards the infrastructure in the Abruzzi region typologies and conditions were monitored along A14 and SS16. Data sheets, consisting of 7 sections that provide location, type, category, geometrical and environmental characteristics, condition, and photographic description, were compiled to catalogue all 52 bridges of the latter road. Bridges were classified according to structural type, material and geometry. The bridge conditions, the piers, and the cracking and vegetative maps were considered.

Bridge structures on the A1FiBo and A16 were scrutinised, and a selected number of bridges either representative of the most frequent typologies or significant for their outstanding design was identified. A further screening of the set including these structures and those identified by the on the TBF and PLS has led to the definition of the final case-studies for the detailed applications and the development and calibration of assessment methods.

In the roadway and railway system of Calabria the most common typologies are single-stem or frame piers with full, single-, or multi-cellular hollow-core cross-section; simply supported decks, made up of reinforced or prestressed concrete girders and a cast-in-place RC deck slab. Two study cases were selected for the study of non-conventional protection/retrofit techniques. The first one is the Follone viaduct on the A3 Salerno-Reggio Calabria highway, where the spans have been connected with by means of longitudinal devices, while the second case is the Val di Leto viaduct, on a provincial road, which was recently retrofitted using oleodynamic devices.

Finally, the data collected during the survey of the two railway lines RMVT and RMSu, have allowed selection of four masonry arch bridges, two per line, to be used as case-studies for the calibration of analysis methods for masonry bridges.

4.2 Task 2: Assessment methods

In summary, Task 1 has shown that the relatively many important bridges crossing wide valleys in the mountain tracts of the Central and Southern Apennine (A1FiBo, PLS, A3) represent a negligible percentage of the total bridge stock, made up essentially of bridges with simply-supported decks (prestressed or reinforced concrete girders plus slab) with single stem of frame piers. For this reason a special effort has been devoted to devising a simplified non-linear method suitable for the analysis of bridges with simply-supported decks (Università di Roma La Sapienza). As it regards statically indeterminate bridges (continuous decks) a distinction was made between those with special configuration, for which inelastic dynamic analysis is in most cases the method of choice, and simpler bridges, for which two methods have been thoroughly explored: the Modal Pushover Analysis (MPA) method (Università di Roma La Sapienza) and the Secant Mode Superposition (SMS) method (Università di Pavia). Finally research has also focussed on two more issues, namely the always debated problem of directional combination rules (Università di Chieti-Pescara) and the non linear modelling of seismic protection devices (Università di Cosenza).

4.2.1 Simplified non linear method for bridges with simply-supported decks

For these bridges it is possible to set up an *ad hoc* assessment procedure which represents a convenient trade-off between simplicity and accuracy. The reference model is that of a vertical cantilever with a continuous distribution of mass, on top of which rest the pier cap and the deck masses. As long as the pier height is not such as to make higher mode contributions significant, in the transversal direction each pier represents a single-degree of freedom oscillator (see Figure 2a). In the longitudinal direction the entire bridge can also be represented as a SDOF system if seismic restrainers are provided that minimise the relative movements of adjacent decks on top of the pier caps (see Figure 2d). In this case the system has mass equal to the sum of the tributary masses of the piers and resisting force sum of the resisting forces of the piers (assuming that maximum displacements are permitted by the abutments joints).

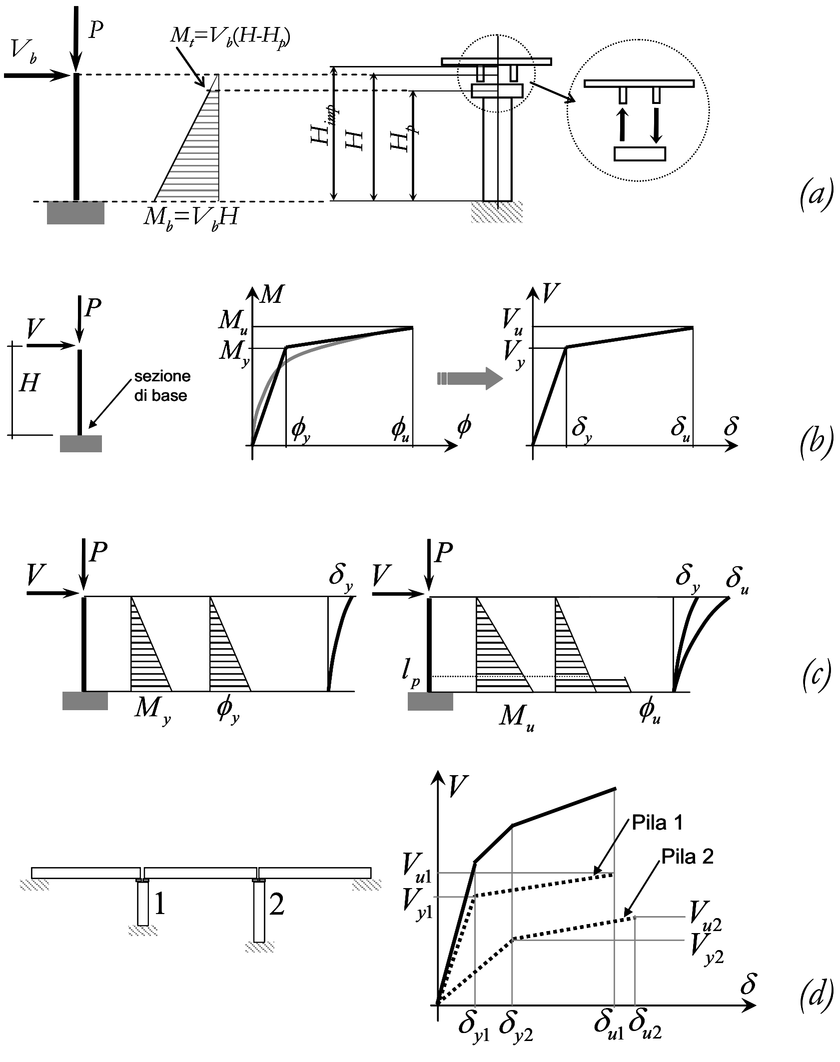


Figure 2. Simplified non linear method for the assessment of bridges with simply-supported decks.

The method consists of a simplified non linear static analysis in which the force-displacement laws are constructed based on the results of moment-curvature analysis of the pier bases (see

Figure 2b). The following equations give the tributary mass, Eq.(1), the effective height in the transversal direction, Eq.(2), the yield and ultimate displacement (see Figure 2b and c), Eq.s (3) and (4), the period, Eq.(5), and the corresponding demand displacement, Eq.(6), as for single-mode conventional pushover analysis. The effective height equals the pier's height in the longitudinal direction. Displacement capacity follows from ultimate displacement with appropriate safety factors.

$$m = 0.3m_{pila} + m_{pulv} + m_{imp} \tag{1}$$

$$H \cong \frac{\left(m_{pulv} + 0.3m_{pila}\right)H_p + m_{imp}H_{imp}}{m} \tag{2}$$

$$\delta_y = \frac{1}{\nu}\phi_y H^2 / 3 \tag{3}$$

$$\delta_u = \delta_y + \left(\phi_u - \phi_y\right)\cdot l_p\left(H - l_p / 2\right) \tag{4}$$

$$T = 2\pi\sqrt{m_i / k} = 2\pi\sqrt{m_i\delta_y / V_y} \tag{5}$$

$$\begin{array}{ll} \delta_{\max} = S_{De}(T) & T \ge T_C \ \text{o} \ q^* \le 1 \\ \delta_{\max} = \dfrac{S_{De}(T)}{q^*}\left[1 + \left(q^* - 1\right)\dfrac{T_C}{T}\right] & T < T_C \end{array} \tag{6}$$

The guidelines and application manual give a detailed description of the method, of the safety verifications of bearings, piers and foundations, and a complete worked-out example.

4.2.2 Verification of applicability of MPA method to bridge structures

The MPA method by Chopra and Goel (2002) has been devised for the analysis of tall buildings. Its applicability as an alternative to inelastic dynamic analysis and adaptive pushover methods for the assessment of bridge structures has been investigated through its application to the Rio Torto viaduct (see Figures 3 and 4), one of the case studies selected for the project. The structure, built at the end of 50's, is characterized by thirteen-span twin decks realized with two girders and top slab. The twelve supports consists of a pair of framed piers, one under each deck. Each pier is a multi-storey reinforced concrete frame with variable height, realized with two circular columns of diameter D =120÷160 cm.

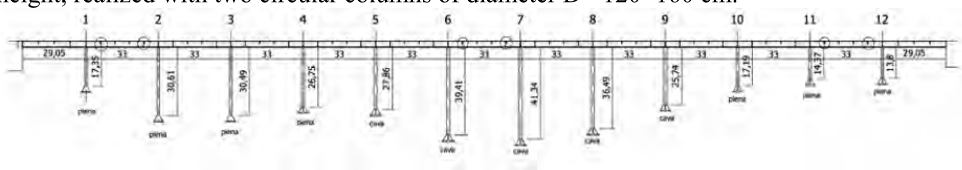


Figure 3. Longitudinal profile of the Rio Torto viaduct (A1FiBo).

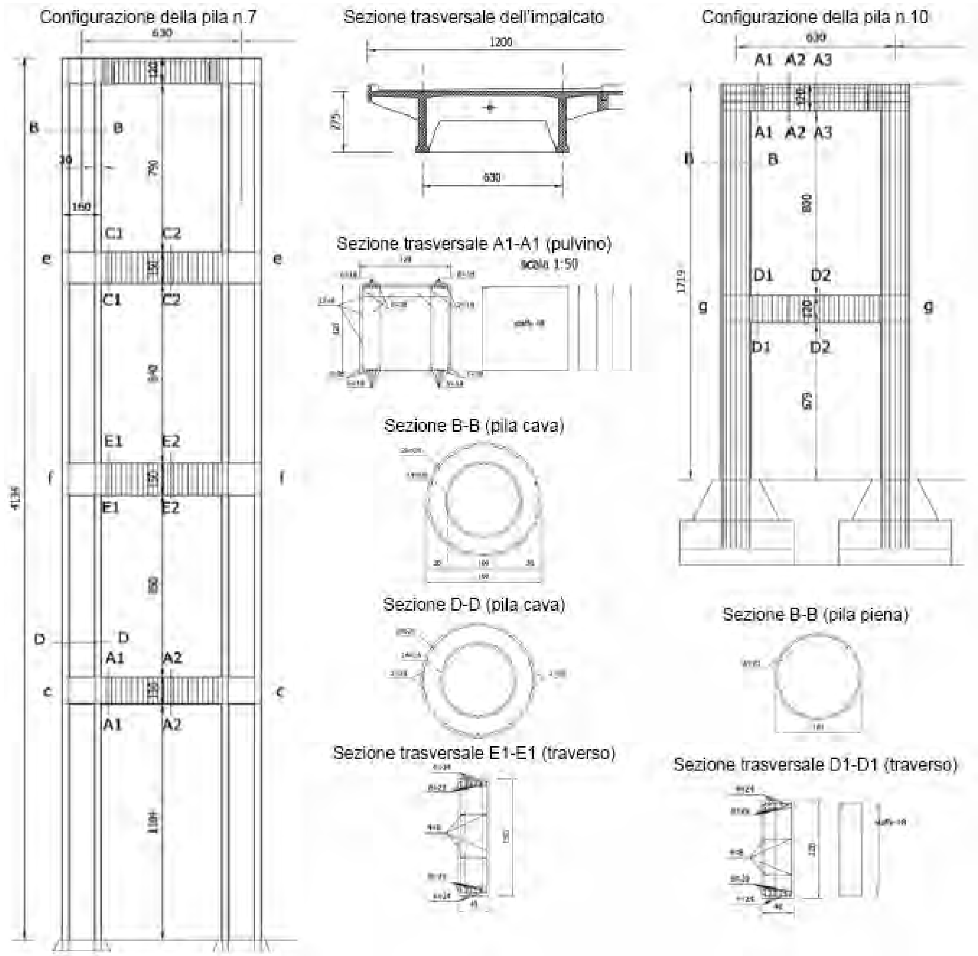


Figure 4. Two piers of the Rio Torto viaduct (A1FiBo).

The results from inelastic dynamic analysis have been taken as benchmark for the purpose. The response has been compared for several intensity levels (to assess the influence on accuracy of the level of non-linearity in the response) and in terms of different response quantities, both local and global (section curvatures and element displacements).

The comparison provided the following indications:

- The location of maximum modal displacement is the best choice as the reference degree of freedom (DOF) for estimating the demand on the structure. Each significant mode is therefore characterized by its own reference DOF.
- The variation of the lateral load distribution, from one based on the modal (elastic) displacement shape to another based on the (plastic) displacement shape at failure, does not affect appreciably the results.
- The best estimate of the displacements by the MPA (i.e. the one derived taking the optimal reference DOFs) matches reasonably well that from TH. It is worth noting

that a comparable amount of approximation on the response of the structure is obtained both in the elastic and in the plastic response regimes. This observation, together with the previous one, indicates that the main approximation of the method, i.e. being based on the initial elastic modal vector, may not represent a major limitation.

- Differences between the nodal displacements estimated by the MPA with respect to those by the TH are found to be in the order of 15%, independently of the intensity level of the ground motion. Analogous results are observed also for the curvatures at members end-sections, resulting in almost coincident patterns of plastic hinge location and predictions of members failures.

For the considered case, the application of the MPA method has shown to lead to fully acceptable results. Such a favourable conclusion still awaits substantiation from a larger number of applications. These results have led to the introduction of the method among the allowed methods in the draft guidelines.

4.2.3 SMS method

The Secant Mode Superposition method consists essentially of an iterative multi-modal response spectrum analysis on a structural model with secant stiffness properties and equivalent viscous damping. The procedure can be summarised in the following steps:

- Step 0: A starting displacement profile and stiffness distribution are assumed;
- Step 1: The stiffness matrix of the equivalent linear structure is assembled;
- Step 2: Modal analysis is carried out;
- Step 3: Displacement in each vibration mode are obtained either from an over-damped elastic or from an inelastic displacement spectrum;
- Step 4: Modal contributions are combined to yield displacement profile and moment distribution (different combinations rules were examined);
- Step 5: Two response indices are computed, that evaluate convergence on displacement profiles and force distributions, while checking that the structural capacity is not violated.
- Step 6: A final response index is obtained as an average of the first two and checking the convergence of the proposed iterative procedure.

The method has been thoroughly tested on the six "typological" bridges, with regular and irregular configurations, and different number of spans and span length. Verification of the method is versus non-linear time-history analysis in terms of maximum deck displacement, and maximum pier shear forces has been carried out.

4.3 Task 3: Retrofit measures

The experimental part of the research activity of Line 3 has been carried out at the University of Pavia and of Roma Tre. The two experimental campaigns have focussed with different goals on the testing of piers. The tests performed in Pavia were aimed at ascertain the effectiveness of FRP retrofit measures in order to confine hollow-core piers with insufficient lap-splices, while those performed in Roma Tre were aimed, using large-scale specimens, at the characterisation of the response of frame piers built in the '60s.

Finally, as it regards masonry bridges, a comprehensive survey of the existing retrofit techniques for this type of structures has been carried out by the University of Genova.

4.3.1 Experimental activity on FRP strengthening for insufficient lap-splice

Four 1:2 scaled bridge piers were designed with an insufficient overlapping length of the longitudinal bars across the critical zone that should lead to an early loss of the lateral strength due to bar slippage. The built specimens (see Figure 5 left and middle) have the following characteristics:

- Hollow-core rectangular cross-section (see Figure 5, right) with external dimensions 800×1500mm and wall thickness of 150mm;
- Pier height of 6 m (aspect ratio equals 4);
- Longitudinal reinforcement: 80ϕ10 (ρ_L = 1.05%) with an overlapping length equal to 20 diameters (200mm) at the base of the pier;
- Transversal reinforcement: stirrups ϕ6/150mm (ρ_V = 0.38%);
- Axial load equal to 1000kN (ν = 4.3%) or 2000kN (ν = 8.6%);
- Concrete Rck400;
- Steel FeB44K.

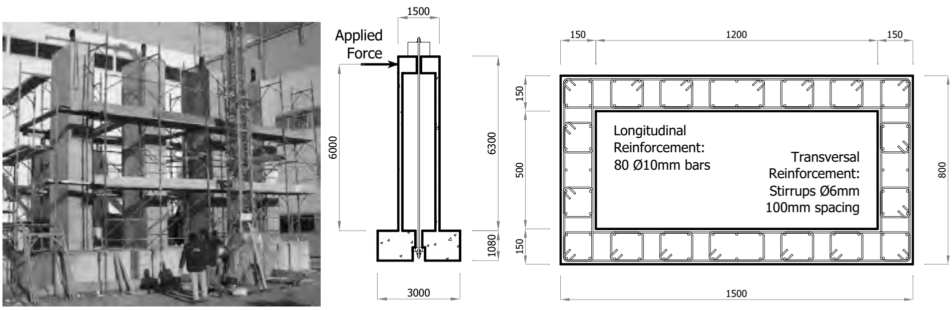


Figure 5. Pier section and specimens built at the University of Pavia.

The retrofit intervention aimed to restore the tensile stress path from the pier section to the foundation, avoiding at the same time any plasticization of the overlapping region. The new stress path created using longitudinal FRP strips applied to the overlapping region is expected to cause the plastic hinge shift upwards where the longitudinal steel is well anchored allowing for an efficient energy dissipation.

During the design phase different possible solutions have been considered concerning the retrofit materials (carbon, aramid or glass FRP), the retrofit geometry (width and length of the region to be retrofitted), the techniques to be used for the anchoring of the FRP strips to the foundation. This was possible employing a numerical FE model developed to predict/reproduce the tests results.

Table 2. Considered retrofit materials

Material	Description	f_u [MPa]	E [MPa]	ε_{ult} [%]	Layer thickness [mm]
SRP 3x2	High Density	1167	77773	1.50	1.1938
SRP 12x	High Density	948	64811	1.46	1.1938
CFRP	High Modulus	3000	390000	0.77	0.165
GFRP	Alkali Resistant	1700	65000	2.62	0.23
AFRP	High Modulus	2800	105000	2.67	0.214

Regarding the materials, the final choice was to use carbon FRP (C-FRP): the analyses indicated that this material is the only one able to sustain the acting tension forces. Too many

FRP layers would have been needed to carry the same force using glass or aramid fibres, affecting the effectiveness of the retrofit intervention.
For what concern the geometry of the retrofit intervention, the final solution was to apply longitudinally two C-FRP layers on the four sides of the specimen. As far as the exploitation of the material strength is concerned this choice appears to be questionable since the fibres applied to the pier sides parallel to the imposed motion will not have the same stress as those on the other two sides, but the adopted solution seemed to be the only possibility to assure the maximum stress diffusion across the pier section. It is worth mentioning that even though anchoring 1500kN force to the foundation of the scaled specimen would have been probably feasible using a steel collar fixed to the foundation with some high-strength steel bars taking advantage of the deep foundation of the specimen, moving back to real structures the anchoring to the foundation of the tensile force induced in the FRP by a seismic excitation would have been much more difficult, if not unfeasible. Spreading the tensile force on the four sides of the pier, the anchoring is clearly easier. Between different possibilities initially considered to anchor such force, final choice was to use an anchoring system realised with FRP too. The idea was to employ aramid connectors, normally used to transfer shear stresses. If this solution will be found to be effective, as it seems from its design, multiple advantaged will arise both on the economic and technologic sides.

Due to external constraints only two piers have been tested within the duration of the project, those without the FRP retrofit in the lap-splice region. The tests confirmed that, as expected, lap-splice with an overlapping length equal to 20 times the diameter of the spliced bars is insufficient to assure the anchoring of the bars (see Figure 6a). The tests also underlined that the effectiveness of the lap-splice decreases while the axial load increase: that is because of the higher stresses and damages (such as partial concrete spalling) in the overlapping region.

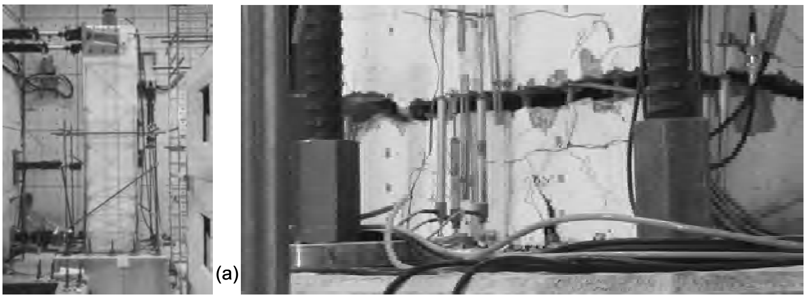

(a) (b)

Figure 6. Test Set-up (a) and open crack at the pier base during the test with 2000kN axial load (b).

Figure 7 shows the base shear-top displacement diagrams derived from the performed tests: here the lateral load carrying capacity drops quite quickly because of the bars sliding. The red curves are the result of the numerical model.

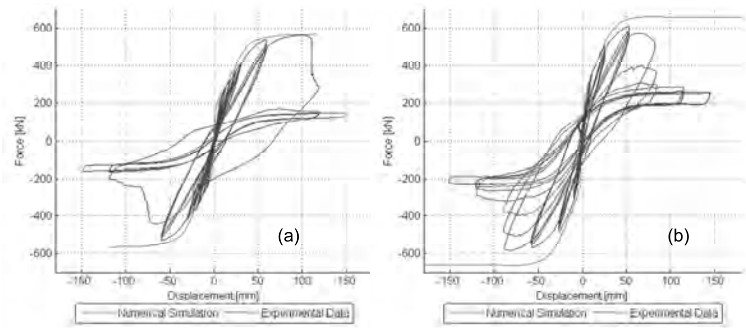


Figure 7. Base shear-top displacement diagrams of the two "as-built" pier (a) N = 1000kN (b) N = 2000kN.

4.3.2 Finite element modeling calibrated to the experimental tests results

Given the large number of seismically under-designed bridges, that need to be assessed and potentially retrofitted due to insufficient lap-splicing, the development of an efficient analytical model to simulate the response of FRP-retrofitted elements was deemed critical. A quite simple though effective finite element model was developed using Seismostruct (Seismosoft, 2006). Figure 8 shows the adopted numeric model. The longitudinal FRP layers have been represented like an element itself. Rigid links has been used to place the FRP at the right distance from the longitudinal axis of the retrofitted member in order for them to be able to give the right contribution to the flexural strength of the pier. Furthermore, each FRP element has pinned connection at both ends in order to be subjected to pure axial load. On the other hand, the FRP wrapping can be modelled in approximation without adding elements to the model, since its main effect is the increased concrete confinement that can be represented by the confinement factor already present in adopted concrete stress-strain representation (Mander, 1988; Martinez-Ruenda and Elnashai, 1997). To tests the effectiveness of the adopted finite element model, the behaviour of 1:4 scaled square hollow section piers from previous experimental campaigns (Calvi et al., 2005 and Pavese et al., 2004) has been reproduced through push-over analysis.

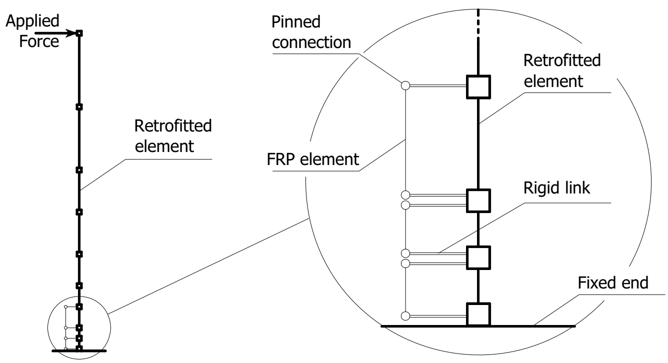


Figure 8. FRP retrofitted pier model.

4.3.3 Experimental activity on large-scale specimens of frame piers

Large-scale tests on framed piers have been undertaken at the Università of Roma Tre. This typology, characteristic of many old viaducts of the Italian highway system, has been chosen for its high seismic vulnerability. Among the representative bridges, a framed pier from the “Rio Torto” viaduct has been chosen (see Figures 3, 4 and 9).

Figure 9. The viaduct “Rio Torto”.

For the experimental program three mock-up’s of pier 12 without retrofit have been realized and tested, with the goal of characterizing its cyclic response and relative collapse mechanism (see Figure 10). Subsequently, one or more reinforcing systems were meant to be applied to the tested piers, for repeated testing to check the efficiency and the reliability of the proposed reinforcing solutions.

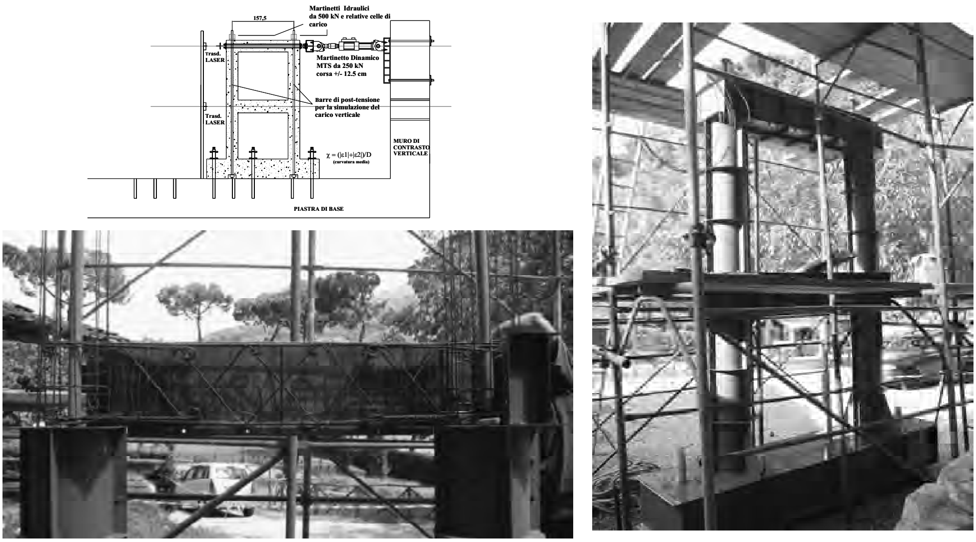
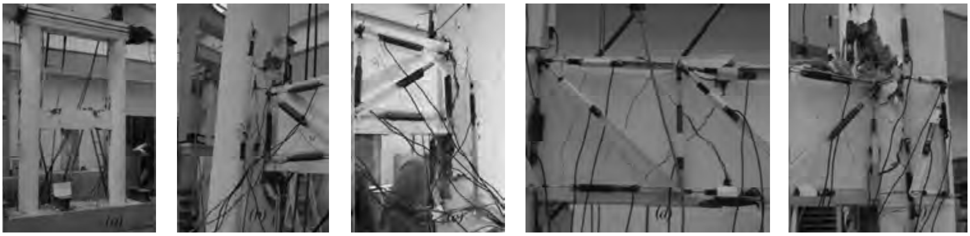


Figure 10. The viaduct “Rio Torto”.

A ductile flexural failure was predicted with formation of plastic hinges for this pier while, on the contrary, all three specimens failed in shear, either in the transverse beam or the joints, suggesting that the formula employed for the evaluation of the shear strength tends to overestimate the ultimate shear.

Since the first test has shown a premature shear failure of the transverse beam, (see Figure 11a), a suitable grid of displacement transducers has been placed on this beam, in order to measure the cracks amplitude, in the other two specimens. In the second two mock-up’s a different failure mechanism occurred on the same transverse beam. In particular, during the second test both beam-column joints collapsed (see Figure 11b-c), while in the third one, only the left end of the beam failed in shear (Figure 11d), with a simultaneous failure of the right beam-column joint (Figure 11e). This outcome was a nice experimental verification of the effect of material fluctuation on determining which amongst similarly resistant failure mechanisms actually occurs in reality.

The differences between the failure mechanisms of the three piers, however, have a little influence on the global behaviour, as shown in Figure 12, which compares the global force-displacement cycles of the three specimens.



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

Figure 11. Failure mechanisms of the transverse beam in the three tests.

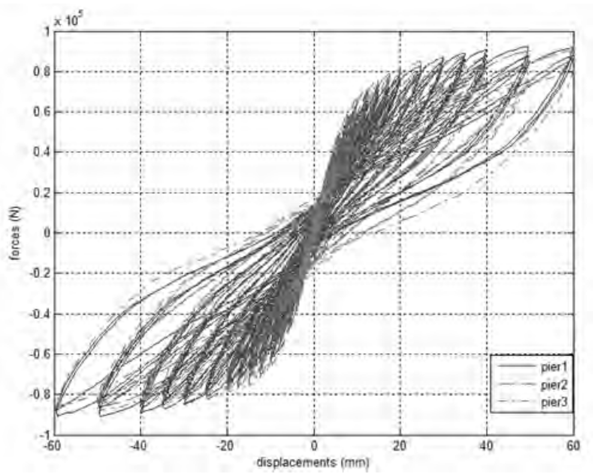


Figure 12. Experimental force-displacement cycles.

The experimental results have been compared with the results of a numerical model, which was set up using the non-linear code “OpenSEES”. Shear failure has been introduced using a shear force-deformation relationship with a tri-linear backbone and an appropriate degradation law, included in a fiber non-linear element, using the section aggregator command. The yield-penetration at the base of the column effect is particularly relevant due to the presence of plain steel bars. This phenomenon, if neglected, can induce an overestimation of the structural stiffness. This effect has been taken into account using a zero-length element placed at the column base with a properly modify stress-strain low of the steel bars. Finally, the buckling phenomenon has been taken into account using a corrected constitutive law of steel. The FE model used was able to reproduce accurately both the global as well as the local behaviour, as shown in Figure 13.

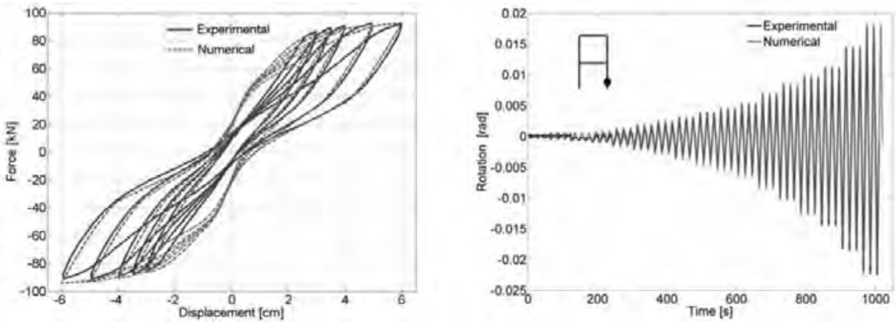


Figure 13. Left: Comparison between theoretical and numerical force-displacement curves; right: base column rotations.

4.4 Task 4: Abutments and foundations

The activity under this task has been carried out at University of Rome La Sapienza, and has dealt with two distinct problems: a) the development of an efficient non linear method for the analysis of diaphragm-type abutments, free standing and retrofitted with tie-backs; b) the review of the literature on soil-foundation-structure interaction with the goal of providing detailed indications for practitioners to be included into the assessment guidelines.

4.4.1 A simplified non linear dynamic model for the analysis of abutments

A simplified model for the dynamic analysis of diaphragm walls retaining dry cohesion-less soils with horizontal back-slope subjected to seismic excitation has been developed (Franchin *et al*, 2007a). The model is based on the well-known one-dimensional Winkler approximation and on the non-linear shear-beam model for the ground layers on both sides of the wall (see Figure 14). The model can include anchor-ties and can account for non-linearity in all of its elements (retained soil, anchors and wall). According to preliminary numerical applications, which include validation of the proposed model results versus those of a refined plane-strain nonlinear finite-element analysis carried out with a commercial code, the model appears to yield quite accurate predictions of static and dynamic bending moment distributions and permanent wall displacements.

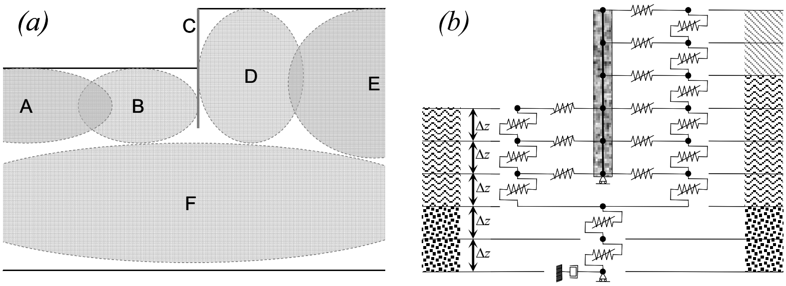


Figure 14. The developed model for diaphragm type.

Next the developed model has been applied for the analysis of the response of a diaphragm abutment prior and after upgrading intervention with change of the support conditions and insertion of tie-backs (Franchin *et al* 2007b). The analysed structure is represented in Figure 15.

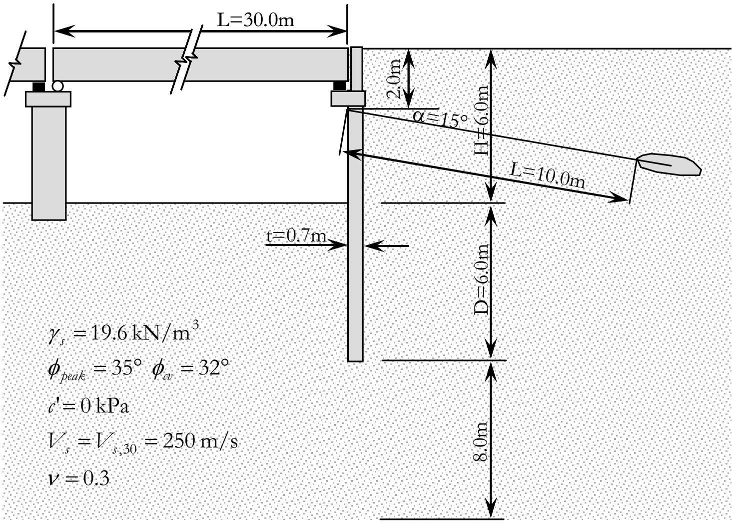


Figure 15. Diaphragm abutment retrofitted with anchor ties.

The application of the model has shown its versatility in assessing the system response in its existing state and in progressive states of upgrading, in terms of both forces (Figure 16 left) and dynamic displacements (Figure 16 right). To the extent that it has been validated at present, the model represents a very efficient tool for realistic design and assessment purposes.

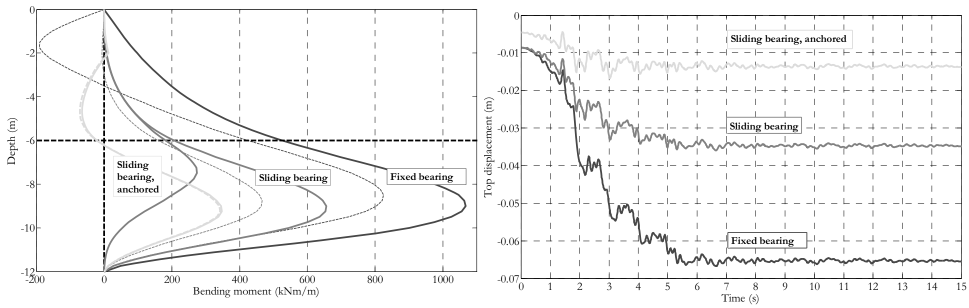


Figure 16. Results of the abutment analysis: left, moment diagrams; right, top displacement time-histories.

4.4.2 Critical review and recommendations on methods for the analysis of soil-foundation-structure interaction

A comprehensive review of the literature on the treatment of the response of deep foundations has been carried out. This has led to identifying the available methods and their pros/cons. After the survey a selection has been made of those procedures considered suitable for practical application and some numerical applications have been carried out to assess the relevance of the phenomenon (input motion modification by kinematic interaction and foundation flexibility), in terms of the response of the superstructure.

One example is the bridge structure shown in Figure 17. It is a simply-supported prestressed concrete deck of span length 30.0m typical of the Italian highway construction practice of the ’50s-‘70s with piers consisting of a single-stem with hollow-core circular section. The dimensions are in the figure. The foundation consists of a mat on 5 piles of 1.5m diameter. Pile length is 20m. The bridge has 6 spans and a pier of height 20m has been considered. Soil can be classified based on the available information as type D. The structure has been modelled as shown in Figure 17e, i.e. as a three-degree of freedom system (including horizontal and rocking component of the base).

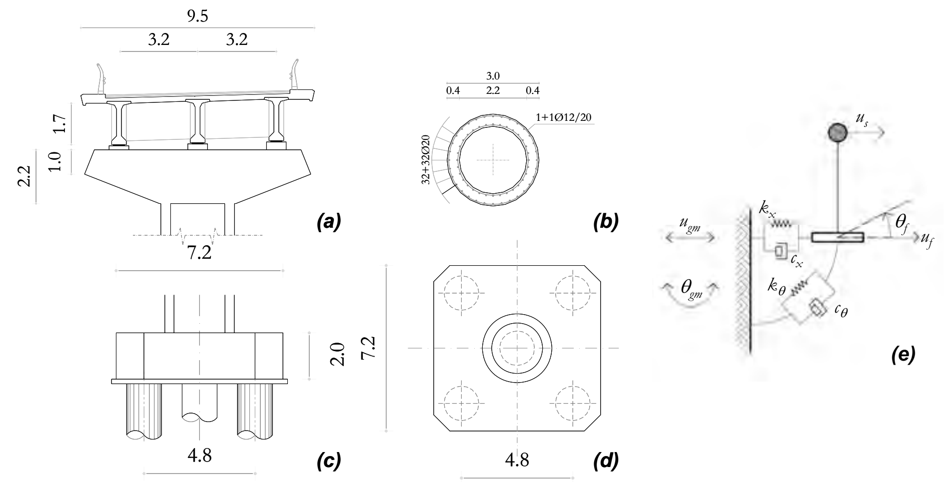


Figure 17. Simply-supported deck on single-stem hollow-core pier founded on piles.

The analysis has been carried out in the frequency domain using the substructuring approach (). The steps of the analysis include:

- Evaluation of the modification of the surface free-field motion (supplied as an acceleration response spectrum) due to the kinematic interaction between soil and pile group. This step provides the power spectrum of input displacement at the pier base.
- Evaluation of the complex frequency-dependent impedance of the soil-foundation system, consisting of the stiffness and damping functions (of the frequency) to be assigned at the pier base. This impedance includes the evaluation of the frequency dependent “dynamic” group effect, i.e. the modification of the impedance obtained as a simple summation of the individual pile impedances to account for the interaction of the wave-fields produced by each pile.
- Evaluation of the response in the frequency domain. This has been done both with a purpose-made code and with a commercial finite element software that implements frequency-domain analysis (Sap2000). The resulting power-spectrum of the displacement components can be integrated to yield the root-mean-square (RMS) or standard deviation of response, from which maxima to be used in verification are readily obtained by multiplication for the peak factors.

Figure 18 shows the real (stiffness, top) and imaginary (damping, bottom) parts of the complex impedance at the base of the pier, for the translation (left) and rocking (right) displacement components. These are reported for two different values of the shear wave velocity Vs, both compatible with the soil type D. The figure also reports the stiffness/damping obtained by simple summation of the single pile contributions. Comparing the latter with those of the group allows to appreciate the frequency-dependent effect of the pile-to-pile interaction. This effect reduces, by more than 50% in this case, the total stiffness.

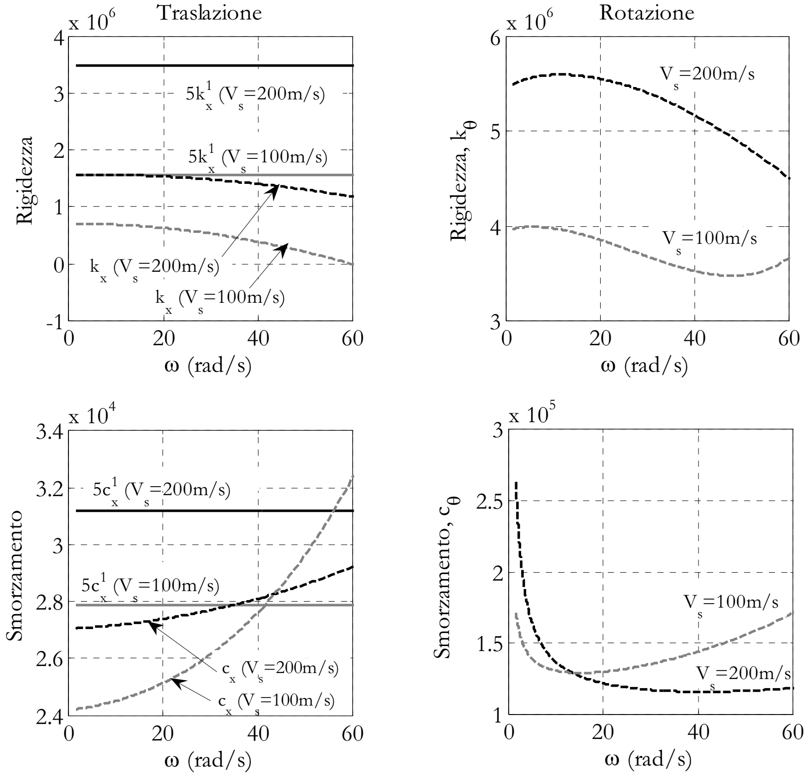
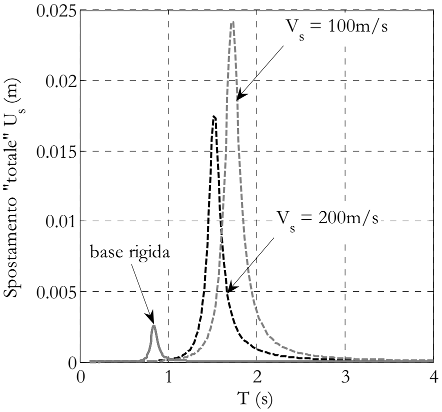


Figure 18. Complex impedance at the pier base: stiffness (top), camping (bottom), translation (left) and rotation (right).

Finally, Figure 19 shows the power spectral densities of the response in terms, on the left, of total displacement (relative to input motion, i.e. sum of the foundation translation, the structure deflection and the translation due to rigid foundation rotation), and on the right of the structural deflection only. Results are reported for the two Vs values and, for reference, for the fixed-base response. As it can be seen, as expected, the fundamental period of the system elongates considerably due to the introduction of the foundation flexibility: it starts at T=0.83 s in the fixed base case, and reaches about 1.5s and 1.75s for Vs=200m/s and 100m/s, respectively. This increases the total displacements. The drifts, however, are considerably reduced as shown in Figure 19b.
The above application, as well as the others carried out, allowed to introduce in the guidelines quantitative indications on the need for inclusion of SSI into the modelling.



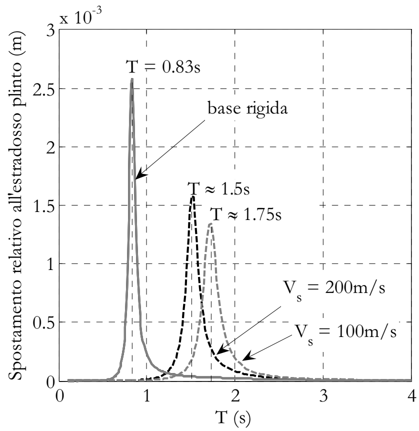


Figure 19. Results of SSI analysis on a bridge pier: power spectral densities of the response in terms of total displacement (left) and structural deflection (right).

4.5 Task 5: Numerical application to case-studies

All research units have contributed in producing a vast amount of case-studies that have been of considerable usefulness in checking consistency and practicality of the indications that now form the guidelines for assessment. In this section only a limited overview of the applications is given to illustrate the work done. A more detailed description can be found in the final report for the Line 3. Table 3 reports all the analysed bridges.

Table 3. List of case-studies analysed according to the assessment guidelines.

Unit	Case-study	Description	Analysis
Torino	Narbareto (PLS)	4 simply supp. spans, circular hollow-core piers	Elastic RS analysis + q-factor
Torino	Rio Barcalesa (PLS)	7 simply supp. spans, polygonal bi-cell. piers	Elastic RS analysis + q-factor
Torino	Borgotaro (PLS)	Hollow-core slab deck, highly irregular plan, Figure 20	Elastic RS analysis + q-factor
Torino	Rio Verde (PLS)	9 spans, steel deck with hollow-core RC piers	Elastic RS analysis + q-factor
Torino	Ramat (TBF)	5 spans, box-section, steel pier	Elastic RS analysis
Chieti	Vasto Marina (SS16)	15 spans, frame piers	Elastic RS analysis + q-factor
Chieti	Della Valle (A25)	10 spans, box-section, single-stem hollow-core piers	Elastic RS analysis + q-factor
Roma Tre	Rio Torto (A1FiBo)	13 spans, inelastic time-history analysis	Inelastic time-history analysis
La Sapienza	Rio Torto (A1FiBo)	13 spans	Modal pushover analysis
La Sapienza	Standard viaduct (E45)	5 spans, simply-supported, and continuous after section widening	Simplified non-linear method, pushover, linear dynamic
Cosenza	Follone (A3)	4 simply supp. spans, retrofitted with “link system”	Inelastic time-history analysis
Cosenza	Val di Leto	5 simply supp. spans, retrofitted with oledynamic devices	Inelastic time-history analysis

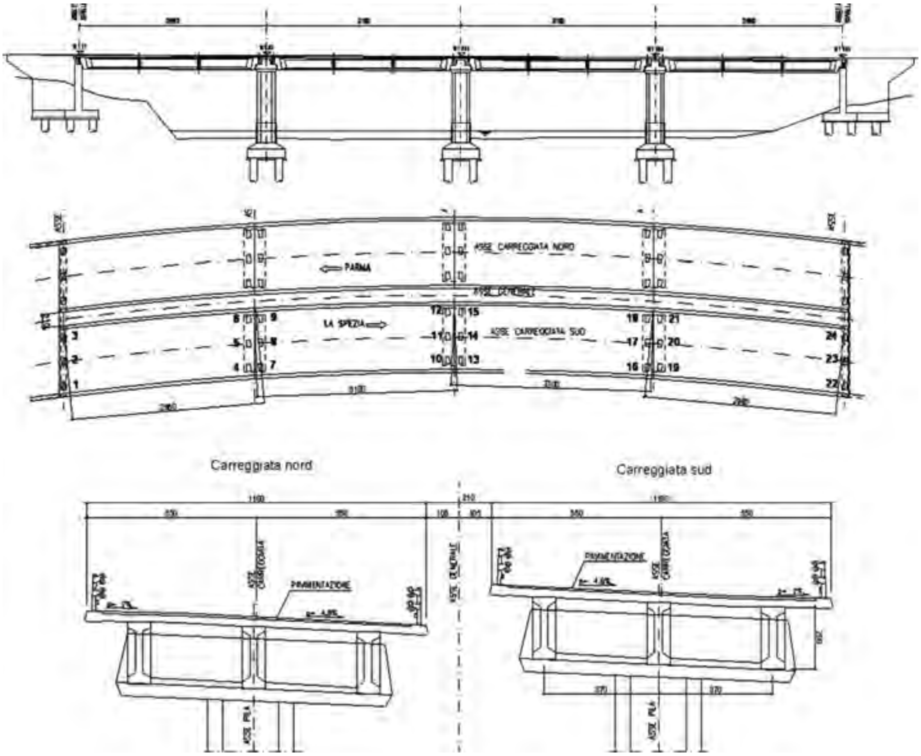


Figure 20. The Narbareto viaduct.

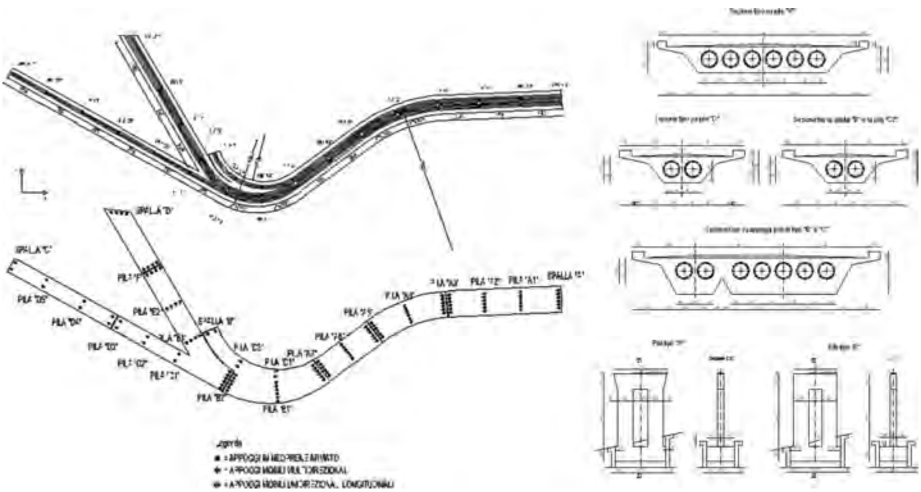


Figure 21. The Borgotaro viaduct.

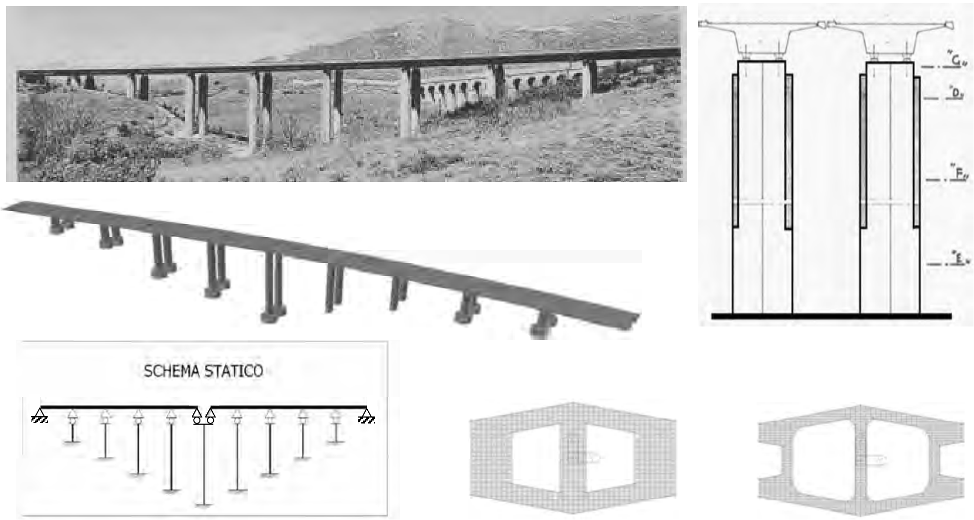


Figure 22. The Della Valle viaduct.

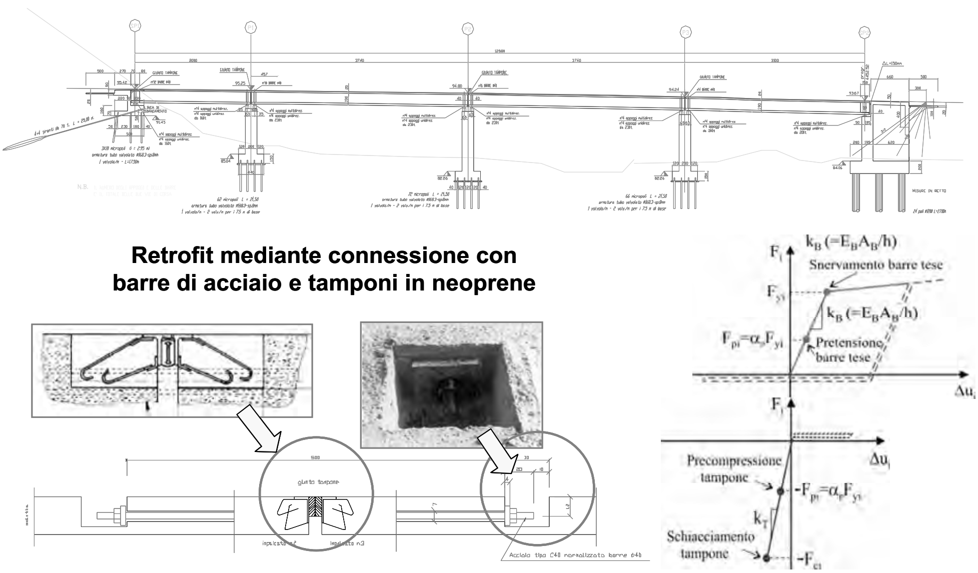


Figure 23. The Follone viaduct.

4.6 Guidelines and Application manual

The activity only briefly summarised in the previous sections has represented a necessary support for undertaking the task of writing what was the final product expected from Research Line 3: a proposal for a guidance document on the seismic assessment of existing bridges, and a companion set of example applications. The task, carried out by University of Rome La Sapienza, has gone through several rounds of scrutiny by all the units. In its final version it represents the first European document on the topic and could be envisaged to form the basis for a future addition to the Eurocodes system. Indeed, the document is fully in line with Eurocodes and reflects to some extent the experience on the seismic assessment of existing structures gained with the use of Eurocode 8 Part 3 on buildings. It is also in line with the relevant chapters of the DM2008, related to seismic design of bridges, and incorporates its most recent developments on the definition of seismic action.

The document produced consists of four chapters and two appendices:

- Chapter 1: gives an introduction to the problem of seismic assessment of existing bridges;
- Chapter 2: contains the guidelines;
- Chapter 3: is an overview of the most common retrofit measures and criteria employed, without entering into the specifics of their design, making reference for this purpose to specialised texts on the topic;
- Chapter 4: contains the numerical examples that illustrate the application of the methods presented in the guidelines. There are four applications covering:
    - assessment, by means of the simplified non linear method, of a typical simply-supported bridge with single-stem cantilever piers in its present state;
    - assessment, by means of linear and pushover analyses, of the previous bridge in two different configurations, with a new continuous, wider, composite steel-concrete deck, with and without seismic isolation;
    - assessment of the Rio Torto viaduct by means of inelastic time-history analysis.
- Appendix A: presents the fundamentals of the response to multiple-support excitations and reviews a number of methods that can be employed to analyse bridge structures for this effect;
- Appendix B: presents the fundamentals of the soil-foundation-structure interaction phenomenon and reviews a number of methods that can be employed to analyse bridge structures for this effect.

The main body of the manual is represented by chapters 2 and 4, as well as by the appendices. In the following the most significant or problematic aspects are briefly reviewed and commented.

4.6.1 Chapter 2, guidelines: definition of the seismic action

The seismic action is defined, in line with DM2008, by means of an elastic acceleration or displacement response spectrum characterized by an average return period specified as a function the limit state of interest.

The return period T_R is obtained from the probability of exceedance P_{VR} over the *reference* life V_R. The latter is given in DM2008 as the product of two factors, the *nominal* life V_N and the “use factor” C_U. The minima for P_{VR} for each limit state are given in DM2008.

In the tentative applications of the guidelines it was raised the problem of the value to be attributed to V_N and C_U, especially with reference to the first one. The uncertainty may arise in the choice between 50 and 100 years for V_N, when considering bridges over highways. The DM2008 indicates 50 years for bridges of ordinary dimensions, typology and importance, and 100 years for bridges of large dimensions and “strategic” importance. One would then be probably directed towards 100 years, in consideration of the importance of the bridge (it is on a highway). The next choice is that of C_U which leads unambiguously to 2.0, since highways are roads of type A according to the Italian classification of roads (i.e. considering, again, the functional importance of the road on which the bridge is located). The above choices would imply a reference life of 200 years and, for the life-safety limit state, a T_R of about 2000 years. It is observed that this conclusion would not to be in line with the safety criteria contained in Eurocode 8 Part 2 (Bridges) which indicates for highway bridges an importance factor γ_I=1.3 to be applied to the action with T_R = 475 years. This multiplication leads in most of Italy to an action with a return period of about a 1000 years. This latter in turn is consistent with a reference life of about a 100 years, which is also the design life specified in the Eurocodes for other actions (e.g. corrosion).

An official response to the mentioned problem, whose relevance needs not to be underlined, cannot but come from the competent authorities, which are in charge of choosing the safety levels.

Within the framework of the definition of the reference life one aspect that deserves particular consideration in the case of existing bridges is the possibility of defining the concept of “residual” reference life. Though it is admitted that in our Country it seldom occurs that the decision to demolish a bridge can be taken several years in advance, it may happen that, due to planned substantial modification of the traffic capacity of the link, it will be economically more convenient at a future date to replace the bridge. In this case, if seismic upgrade must be undertaken, the concept of residual reference life may be invoked to assign to V_R a more realistic reduced value. This possibility is not currently included in the guidelines, though it is regarded as being in line with the possibility allowed for exisiting structures to derogate from standard safety levels dictated for new structures.

4.6.2 Chapter 2, guidelines: methods of analysis

With respect to classification of methods in static and dynamic, linear and non linear, now common to all modern seismic design codes and giving rise to the usual four alternatives, the guidelines restrict somewhat the field of applicability of linear analysis. This is not unexpected. For new well-designed structures the role of analysis is a relatively minor one, due to the many constraints (arising mainly from global and local capacity design) that guide the design. On the other hand, when assessing an existing structure, the accuracy in the analysis may have a major economic impact on the retrofit, possibly avoiding it altogether.

The guidelines admit linear analysis of two types only: modal analysis with unreduced elastic spectrum and verifications in terms of deformation/forces (subject to stringent conditions on the response regularity), and modal analysis with a spectrum reduced by a limited value of the behaviour factor of q=1.5.

The main methods put forward by the guidelines are non linear static and dynamic analyses. As already anticipated in § 4.2.1, a simplified non linear static method is proposed for the very frequent case of bridges with simply supported decks. For continuous irregular bridges the use of more recent pushover variants (adaptive and/or multi-mode) is introduced as an alternative to full-fledged inelastic time-history analysis. The allowance for more than single-mode invariant pushover represents a small step forward with respect to Eurocode 8 Part 2, which builds upon the results of recent wide-ranging studies on the performance of such methods in the analysis of bridges [see for ex. (Casarotti 2005), (Kappos *et al*, 2005), (Isakovic and Fischinger, 2005), (Lupoi *et al*, 2007), (***fib***, 2007), as well as the draft document “Inelastic methods for seismic deisgn and assessment of bridges” by Task Group 11 of the European Association of Earthquake Engineering].

4.6.3 Chapter 2, guidelines: safety verifications

The guidelines introduce a format for bi-directional verification for both deformations and forces. In particular the format reads:

$$\sqrt{\left(\frac{D_x}{C_x}\right)^2 + \left(\frac{D_y}{C_y}\right)^2} \leq 1 \tag{7}$$

where D_x and D_y denote the demand quantities along the two orthogonal axes x and y, with C_x and C_y denoting the corresponding capacities. This format becomes, in terms of chord rotations and shear forces:

$$\sqrt{\left(\theta_x/\theta_{u,x}\right)^2 + \left(\theta_y/\theta_{u,y}\right)^2} \leq 1 \tag{8}$$

$$\sqrt{\left(V_x/V_{u,x}\right)^2 + \left(V_y/V_{u,y}\right)^2} \leq 1 \tag{9}$$

In the above equations the demand terms are understood as the combined effect of both orthogonal components of the seismic action. For example, with reference to chord rotation, for the case of multi-modal non linear static analysis one has:

$$\theta_x = \theta_{xG} \pm \sqrt{\sum_{i=1}^{N}\left[\left(\theta_{xE_X,i} - \theta_G\right)^2 + \left(\theta_{xE_Y,i} - \theta_G\right)^2\right]} \tag{10}$$

where the directional combination is of the SRSS type and the summation is over the modes.

4.6.4 Appendices

The matter covered in these two appendices, i.e. the response of bridge structures to different motions at the piers' bases and the effect of the soil-foundation system deformability in modifying the input motion as well as the response of the structure, has been always mentioned in codes without, however, neither precise quantitative indications on the instances in which these phenomena have to be accounted for, nor of physically sensible yet practically applicable methods to do it. The reason for this resides clearly in the insufficient advancement on basic research. In drafting the guidelines, however, it was considered appropriate to provide a presentation of selected state-of-the-art approaches which are susceptible of practical application.

For what concerns the effect of multiple-support excitation, the guidelines indicate that the phenomenon should be accounted for whenever soil conditions along the bridge belong to different soil categories. The guidelines also present:

- A stochastic model of the motion at the supports (Der Kiureghian, 1996) that can be used either to generate samples of correlated motions to be used in time-history response analysis or in linear random vibration analysis;
- The multiple response spectrum method (Der Kiureghian and Neuenhofer, 1992), which provides a solution for the random vibrations problem of a system subjected to multiple inputs based on the use of the corresponding input displacement response spectra;

- A simplified proposal for time-history analysis employing independent motions at the supports representative of the local soil conditions, which can be applied using currently available commercial finite element codes (Monti and Pinto, 1998);

For what concerns soil-foundation-structure interaction the guidelines give a classification of the approaches and present with some detail the *substructuring* method, in its application to pile (Novak 1974, Makris and Gazetas, 1991 and 1992) and caisson foundations (Gerolymos and Gazetas, 2006a,b). In this method the structure and the soil-foundation system are separated and studied accordingly. The study of the soil-foundation system consists of the solution of so-called kinematic interaction and inertial interaction problems, leading to the modified input motion for the structure and to (complex) impedance to be put at the structure base, respectively. Then the structure is analysed, with a flexible support condition, under the previously determined modified motion. All the formulas necessary to perform this procedure are present in the Appendix.

5 DISCUSSION

The main objective of the project, which was the production of the draft guidelines and their application manual, has been met. In this respect the Research Line was successful, since the product has been delivered and its quality is believed to be high.

Though it wasn't explicitly included into the remit for the Line, it must be noted that the research group initially intended to cover in the guidelines both structural concrete and masonry bridges. In spite of the research carried out, however, this more ambitious goal could not be achieved.

Research on this front was essentially under the responsibility of the unit of Genova. This unit has produced during the three years of the project a considerable amount of high-quality research that has been regularly documented in the annual as well as the final reports, and it is also available in research reports from the unit uploaded on the project website. Quoting from the final report the issues dealt with by the unit cover the following: "i) statistical characterization of the Italian bridge population; ii) mechanical models for solid clay brickwork, needed for detailed and simplified structural models; iii) in field testing of masonry bridges, aiming at the identification of the main mechanical properties of the materials and of the bridge as a whole; iv) laboratory testing of brickwork prisms; v) reduced scale testing aiming at identifying the load carrying capacity and the collapse mechanisms of shallow and deep arches taking into account the fundamental collaboration of the so called "non structural elements"; vi) reduced scale testing aiming at identifying the dynamic properties of shallow and deep arches taking into account the collaboration of the so called "non structural elements"; vii) Limit Analysis procedures for the analysis of masonry bridges taking into account the contribution of all the bridge elements; viii) retrofitting techniques for the bridge and its components." As it may be seen all of the investigated topics are of clear scientific interest, though not specifically relevant to seismic assessment of bridges. This is simply the unavoidable consequence of the international lack of fundamental knowledge on the seismic behaviour of masonry bridges.

6 VISIONS AND DEVELOPMENTS

The research carried out within Research Line 3 has included the state-of-the-art into a document usable for assessing the protection level of bridges against a number of limit-states. There are certainly several areas where improvement is possible and desirable, and in particular these are:

- The non-linear static analysis for bridges of complex geometry;
- The ultimate strength and deformation capacity of structural members such as those encountered in bridge structures (e.g. polygonal multi-cell. hollow-core cross-sections)
- The generation of ground motions for multiple-support excitation. While generated motions are being progressively replaced with recorded ones for the analysis of buildings, their use appears unavoidable for the analysis of bridges whenever different motions must be considered at the supports. Currently available procedures are in need of considerable improvement.
- The vast literature on SSI needs to be acquired and digested by structural engineers to become a practical tool. This a crucial aspect in view of the displacement-based framework of the guidelines and the corresponding need for more accurate evaluations of deformations.

The guidelines do not cover the seismic isolation technique. The reason for this choice is that the design of seismic isolation does not vary between new and existing bridges. Seismic isolation, however, will certainly see much further diffusion in the coming years, for new as well as existing bridges, while isolation device technology continues to evolve rapidly with the ensuing need of developing appropriate analysis and design techniques. In this respect this can be regarded as an ongoing research topic.

To the extent that solutions to the problem of assessing the protection of a bridge against its ultimate state can be considered to be sufficiently mature, the next important passage is that of being able of estimating structural and monetary damage as a continuous discrete function of seismic intensity. Achievement of this goal would allow for the estimation of expected loss related to any given bridge. Looking now at the problem of bridge protection from an higher perspective, the attention should be directed at the bridges as components of road links forming a transportation infrastructure. The seismic performance of the single bridge would then be put in relation with the performance of all other bridges to be able to estimate the overall decrease in functionality of the whole infrastructure. In this respect the very challenging problem of determining the loss in traffic capacity of a damaged bridge represents an essential element.

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DEVELOPMENT OF A MODEL CODE FOR DIRECT DISPLACEMENT BASED SEISMIC DESIGN

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1 INTRODUCTION

Research Line IV of the RELUIS (Rete di Laboratori Univeristari Ingegneria Sismica) project was responsible for the development of displacement-based design guidelines, considering structural typologies commonly utilised in Italy. The main aim and product of the research line has been the formation of a model code for displacement-based seismic design. The background of this research holds an interesting place in the history of seismic design and recent developments provide several strong motives for the further development of displacement-based seismic design methods.

2 BACKGROUND AND MOTIVATION

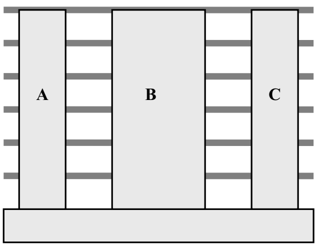
Seismic design in current codes is based on force (and hence acceleration) rather than displacement, essentially as a consequence of the historical developments of an understanding of structural dynamics and, more specifically, of the response of structures to seismic actions and the progressive modifications and improvement of seismic codes worldwide.

In the first decades of the last century, after several major earthquakes, such as Messina (Italy, 1908), Kanto (Japan, 1925), Napier (New Zealand, 1932), and Long Beach (USA, 1933) the first design codes started being developed. These codes were essentially prescribing specific detailing and construction rules and in cases the application of some lateral inertia forces. Typically, and possibly in analogy with some kind of wind design, a value of about 10% of the building weight applied as a vertically distributed lateral force was required, regardless of building period.

This initial force-based approach has been essentially retained with the progressive increasing of understanding of the significance of structural dynamic characteristics, that lead to period-dependent design lateral force levels in most seismic design codes, and even when it became clear that many structures had survived earthquakes capable of inducing inertia forces many times larger than those corresponding to their structural strength, if a linear response was assumed.

This apparent inconsistency was explained after the first simple inelastic time-history analyses had been performed, and the concept of ductility introduced, to reconcile the anomaly of survival with inadequate strength. In the seventy's, relationships between ductility and force-reduction factor were developed, introducing the well known concepts of "equal displacement", "equal energy" and "equal force" approximations, that appeared to be appropriate to estimate the "real" structural response as a function of linear response and period of vibration of the structures.

Since then, ductility has been considered the fundamental parameter to estimate appropriate "force reduction factors", to be used to determine the design lateral force levels. Much research effort was therefore directed to determining the available ductility capacity of different structural systems, performing extensive experimental and analytical studies to determine their safe displacement capacity. It is now clear that this approach is implicitly assuming displacement capacity, and not force capacity, as the basis for design. However, the design process is still carried out in terms of required strength in essentially all codes of practice around the world and displacement capacity, if directly checked at all, is only a final product of the design procedure.
This brief summary of the history of seismic design indicates that initially design was purely based on strength, or force considerations. However, as demonstrated by Priestley (1993, 2003) there are several conceptual drawbacks associated with the use of force-based methods in seismic design. With reference to the design of a RC building possessing three walls A, B and C, illustrated in Figure 1, a brief review of some of the problems with force-based design can be made.



Country	Force Reduction Factor (q) for RC Walls
Europe	4.4
New Zealand	7.5
U.S. West Coast	5.0
Japan	1.8-3.3

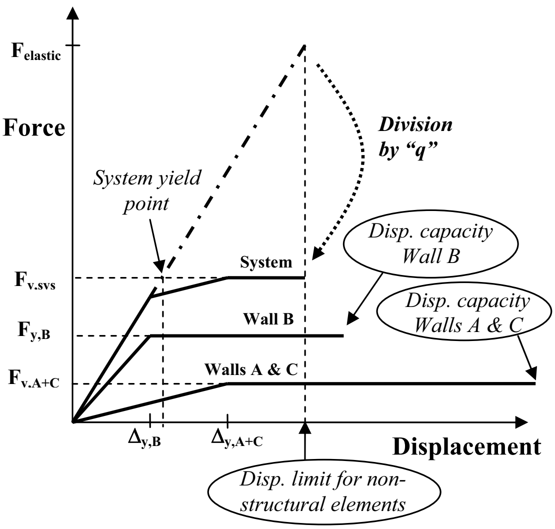


Figure 1. Response of a RC structure possessing walls of different length – used to highlight issues associated with force-based design.

The main issues associated with current force-based design methods, identified and discussed by Priestley (2003) in detail, can be shown to involve:

- *The use of ductility capacity dependent force-reduction factors*. In order to obtain design force levels, code methods divide elastic forces by a force-reduction factor which is set in proportion to the ductility capacity of the structure. However, the actual ductility demand for a structural system will typically be smaller than the ductility capacity of the structure. The right side of Figure 1 presents the force-displacement behaviour for the three walls (A, B and C) and the total system as it is displaced to a deformation limit required to control damage to non-structural elements. It is clear that in controlling non-structural deformations the system ductility is less than the ductility capacity of the long stiff Wall B and considerably less than the shorter flexible Walls A & C. Furthermore, as Walls A & C have considerably larger yield displacements than Wall B, it is evident that the ductility capacity could not be developed in all the walls simultaneously. This point suggests that the use of force-reduction factors that are based on ductility capacity is inappropriate.

- *The force distributions predicted through the use of the elastic stiffness for analysis*. As mentioned above, the force-based approach makes a prediction of the elastic forces of the structure and uniformly reduces these by a behaviour factor to obtain inelastic design forces. However, because elements within the structure do not all yield at the same level of deformation, the elastic force distribution can be very different to the inelastic force distribution. This can be seen in Figure 1 by considering the elastic shear proportions that have developed when Wall B first yields. It is clear that the shear in Walls A & C is only one-quarter that for Wall B in the elastic state. However, at the design displacement of the system, it is clear that the proportion of shear in Walls A & C has now doubled to be 50% of that in Wall B. This point demonstrates that elastic analyses using the initial stiffness are inappropriate for predictions of inelastic force distributions.
- *The difficulty in defining the system ductility for mixed structural systems*. Current codes typically require that the behaviour factor for mixed structural systems be set equal to the lower of the two systems. However, this does not consider how the ductility demands and forces will develop for the combined structural system. For example, suppose that Wall B shown in Figure 1 only had a ductility capacity of three, such that at the design displacement, its ductility capacity had been reached. The system ductility at this displacement is obtained by dividing the design displacement by the yield displacement of the system, whereby the yield displacement of the system would typically be obtained through a bi-linear representation of the system response. Interestingly, it is evident that the system ductility demand is actually lower than the ductility capacity of the critical element. The amount by which it is lower depends on the mixed system considered and larger differences could be expected for different mixed systems, such as frame-wall structures. This point demonstrates that force reduction factors cannot be easily set for mixed structural systems.
- *The inter-dependency of strength and stiffness for certain structural types* (such as RC structures). It has been shown by Priestley (1998), Priestley and Kowalsky (1998) and Paulay (2002) that the yield curvature of RC sections is principally a function of the section geometry and yield strain of longitudinal reinforcement. Consequently, given that the cracked section stiffness is best defined using the secant stiffness to first yield as $EI=M_n/\phi_y$, (where E is the section modulus, I is the second moment of inertia, M_n is the section flexural strength and ϕ_y is the yield curvature) it is clear that the stiffness of a RC element will depend on the strength it is assigned. In other words, the initial stiffness of Wall B in Figure 1 for example, could be doubled by simply doubling the amount of longitudinal reinforcement in the section. As such, the stiffness is not purely a function of the section geometry and therefore in order to know the cracked elastic period of vibration of an RC structure, the flexural strength is required. As the force-based design procedure relies on the period of vibration in order to determine the required strength, this point shows that the design procedure cannot be easily implemented for RC structures.
- *The relationships used to relate elastic displacement to inelastic displacement response*. The force-based design approach estimates the inelastic displacement response based on the elastic displacement response. The relationship illustrated for the wall system within Figure 1 uses the equal-displacement rule, whereby the inelastic displacement is assumed equal to the elastic displacement. This is the

approximation adopted in Eurocode 8. In the United States, however, a different relationship is used whereby the inelastic displacement is typically approximated as being less than the elastic displacement, whereas in Japan the opposite occurs and an equal energy approach is used such that the inelastic displacement is estimated to be larger than the elastic displacement. In reality, the relationship between elastic and inelastic displacements should depend on the hysteretic properties of the structure being considered. Consider the behaviour of two moment-resisting frames with the same strength, stiffness and mass, different only in that one is formed with steel elements and the other with RC elements. As the initial period of the structures is the same, the equal-displacement rule would predict the same displacement for the two systems. However, it is well known that steel frames dissipate considerably more hysteretic energy than concrete frame systems and as such, one would expect that the inelastic displacement of the steel frame would be less than that of the concrete frame. This point shows that relationships between elastic and inelastic displacement should consider the hysteretic properties of the structure.

As these problems with force-based design have emerged and the importance of displacement has come to be better appreciated, attempts to modify and improve existing force-based approaches have been made. In the last decade, however, several researchers started pointing out the inconsistency associated with the use of force for design, proposing displacement-based approaches for earthquake engineering evaluation and design, with the aim of providing improved reliability in the engineering process by more directly relating computed response and expected structural performance.

A rather complete literature review of the subject is reported in *fib* Bulletin 25 (Calvi, 2003), where most displacement-based approaches proposed in the literature, are summarized, critically reviewed and compared, to favour code implementation and practical use of rational and reliable methods.

Comparison of different approaches, however, does not imply the production of a scale of merit, but rather an assessment of the relative ease or difficulty with which the design methods can be applied and any apparent limitations the methods may have. To compare the required strength obtained by each method and the performance of the methods in terms of predicted displacement or drift values for each case study and those obtained through time-history analysis may allow a future refinement of most proposals and a possible focusing towards the most efficient ones.

This objective has to be pursued discussing performance and hazard levels, fundamental principles to estimate structural response and displacement demand, proposed displacement – based design approaches, specific features of the description of seismic input, simple methods to assess the real displacement capacity of members and actual performance of different design approaches when applied to a number of different case study structures.

Recognising these considerations and in particular the observation that among the displacement-based approaches proposed in the literature, the direct displacement-based design (DDBD) method developed by Priestley, Calvi and Kowalsky (2007) amongst others, seems to be very promising and efficient, the goals of RELUIS research line IV were to:

- develop DDBD for different structural typologies commonly found in Italy;
- verify its applicability and effectiveness considering both simplified structural schemes as well as real case studies;
- verify its reliability running time history analyses of the designed structures and checking the structural response with the design expectations.

- verify its effectiveness applying force-based design to the same case studies and comparing the performance of the two methods.
- improve the method in those cases where the performed analyses have highlighted an unsatisfactory performance and re-run the analyses.
- obtain a set of displacement-based design guidelines enriched by the collection of the developed case studies to be used as reference and example.

In order to achieve these goals, a number of research units were needed and a clear research plan was required.

3 RESEARCH STRUCTURE

In order to achieve the numerous research goals, a number of Universities across Italy were contacted to participate in the project. Considering the background and expertise of the researchers involved, each University was designated as a research unit and was assigned a structural typology for which the DDBD methodology was to be developed, as shown in Table 1.

Table 1. Research units for the project.

Research Unit	Research Topic	Research Institute
1	Principles, General Aspects, Actions	University of Pavia
2	RC Frames Structures	University of Bologna
3	RC Wall and Frame-Wall Structures	University of Ferrara
4	Pre-cast Reinforced Concrete Structures	University of Bergamo
5	Masonry Structures	University of Genova
6	Steel Structures	University of Napoli
7	Composite Steel-Concrete Structures	University of Benevento
8	Timber Structures	University of Trento
9	Bridge Structures	Politecnico di Milano
10	Isolated Structures	University of Basilicata
11	Foundation Structures	Politecnico di Milano
12	Retaining Structures	University of Perugia

Together with the allocation of structural typologies, a research programme, illustrated in Table 2, was set for the research units to follow. The time table of activities presented in Table 2 identified a series of activities (common to all the Units) divided by year. In the first year the main activities were the definition of the general and specific aspects of DDBD, selection of case study structures, and force-based design. In the second year the aim was to undertake displacement-based design and time history analyses of the case study structures selected in year 1, highlighting and discussing any problematic aspects. In the third and final year the main activities envisaged were to improve the method, re-run case studies comparing design predictions with non-linear time-history response, and finally define a set of guidelines for DDBD, producing a model code and commentary as well as a set of case-study examples.

Table 2. Research programme.

	1° ANNO												2° ANNO												3° ANNO											
Definition of general aspects	■	■	■	■	■	■																														
Definition of specific aspects				■	■	■	■	■	■	■	■	■																								
Selection of case studies							■	■	■	■	■	■																								
Design using traditional methods										■	■	■	■	■	■																					
Design using displacement-based methods													■	■	■	■	■	■																		
Non-linear time-history analyses																■	■	■	■	■	■															
Identification and discussion of issues																						■	■	■	■	■	■									
Specific improvements to the displacement based method																									■	■	■	■	■	■						
Re-design and verification																												■	■	■	■	■	■			
Development of specific guidelines																												■	■	■	■	■	■	■	■	■
Development of a model code and general commentary																															■	■	■	■	■	■

It was, however, recognised from the start that the existing guidelines for DDBD of certain structural typologies were much further developed than for others. As such, it was not possible to apply the same schedule for all the units as a large range of structural typologies, from RC to timber structures, from large buildings to deep foundations and retaining walls (as shown in Table 1), were covered. Consequently, the programme served as a guide but it was clear that the development of DDBD would, in some cases, be a matter of refining and researching details of existing methods, while for some structural typologies it is the first time that a displacement-based method has been developed and therefore only more general developments could be expected.

4 MAIN RESULTS

Many useful results have been obtained by the research units. The type of results vary significantly owing to the range of activities that were conducted for the different structural types. Literature reviews demonstrated that a significant amount of work had already been done to develop DBD methodologies for RC structures, whereas, in contrast, very little work has been published in relation to the displacement based design of foundation and retaining structures. As such, there were research units that applied existing DDBD methodologies to case study structures, performed verification analyses, and refined the DDBD approach. Others had to start by considering suitable deformation limits and possible means of undertaking DDBD for the structural typology being considered. Overall, as demonstrated by the results presented in this section, the research has been very successful as many findings have enabled the formation of the first model code for displacement-based seismic design.

4.1 Principles, actions and general aspects of Direct DBD

The main product of RELUIS Research Line IV has been the formation of a model code for Displacement-Based Seismic Design (Calvi and Sullivan 2009). This achievement was made considerably easier because of the release, during the course of the research, of a book on Displacement-Based Seismic Design by Priestley et al. (2007). This publication provided the various research units a clear understanding of the DDBD approach, and highlighted the state-of-the-art for the various structural typologies. In order to form the model code, however, there were still a number of considerations that had to be made in order to provide a relatively complete set of design recommendations.

The DDBD method aims to establish the level of strength that a structure requires in order that the displacements are within pre-defined limits during the design seismic event. To this extent, the exact intensity of the design event and the limits of displacement are inputs to the design procedure that can be selected by the designer (in consultation with the client or building owner) so that a suitable performance-based design solution can be achieved. Another aspect of the procedure is that rather than use a design acceleration spectrum, the approach utilises a design displacement spectrum. In the model code, the magnitude of design displacement spectrum has not been set as this will depend on the seismicity of the zone in which the structure is located. However, the form of the design displacement spectrum has been set, through consultation with members of the Research Unit 6 of the Project S5 (Cauzzi et al. 2009), to be that shown in Figure 2.

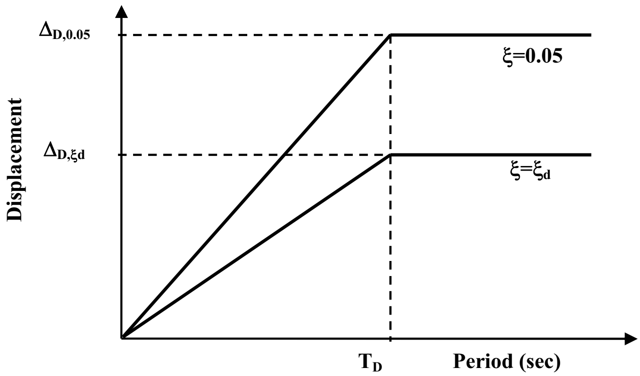


Figure 2. Form of the design displacement spectrum specified in the model code.

The shape of the displacement spectrum indicated in Figure 2 is very simplistic in that it only requires definition of a spectral displacement corner period, T_D, together with the magnitude of the displacement, $\Delta_{D,05}$, at the corner period (note that suitable values for such parameters were one of the objectives of Project S5; refer to report by Cauzzi et al. 2009). One initial concern with such a spectrum was that corresponding acceleration demands in the short period range are considerably higher than normal acceleration spectra would specify. To overcome this, the model code sets a limit on the design base shear, as indicated in Eq. (1), that corresponds to the plateau of the equivalent acceleration spectrum.

$$V_b = K_e \Delta_d + C \frac{P\Delta_d}{H_e} \leq 2.5 R_\xi .PGA.m_e + C \frac{P\Delta_d}{H_e} \tag{1}$$

Where V_b is the design base shear, K_e is the effective stiffness, Δ_d is the design displacement, C is a constant used to control P-delta effects, P is the total expected load and H_e is the effective height of the equivalent SDOF system. The limit to the base shear is set using R_ξ, the spectral reduction factor associated with the expected energy dissipation, the peak ground acceleration (PGA) expected at the site and m_e, the effective mass of the equivalent SDOF system. As such, the approach assumes that the elastic spectral acceleration plateau can be taken as 2.5 times the PGA. While such an approach may need refinement in the future to account for different soil types, this new proposal is clearly a simple but effective way of obtaining reasonable design base shear values for short period structures whilst maintaining the simplified form of the design displacement spectrum.

Another innovative inclusion within the model code relates to the required strength of a structure that possesses a design displacement greater than the corner displacement Δ_D. In the model code, the required effective period, T_e, is obtained using Eq.(2).

$$T_e = \frac{\Delta_d}{\Delta_{D,\xi}} \cdot T_D \tag{2}$$

Where T_D is the spectral displacement corner period (refer Figure 2) and $\Delta_{D,\xi}$ is the spectral displacement demand at this period for the anticipated level of equivalent viscous damping. Typically, designers will find that Eq.(2) yields effective period values less than T_D, since the design displacement of the system will typically be less than $\Delta_{D,\xi}$. However, in the event that Δ_d is greater than $\Delta_{D,\xi}$, Eq.(2) essentially linearly extrapolates the displacement spectrum ignoring the fact that the displacement demands plateau and do not develop the design displacement level. Considering the first mode response only, the designer might argue that because the spectral displacements plateau the structure should not require lateral strength to control the deformation demands to acceptable values. However, if the lateral strength is set to very low values such that the structural periods are very long, it is likely that higher mode deformations would become much more significant and performance limits states could be exceeded. As such, to ensure some mitigation of higher mode effects in a simplified manner, Eq.(2) is included in the model code. In addition however, in recognition of the fact that the 1st mode displacement demands are limited at long periods, a maximum value for the required effective stiffness, $K_{e,max}$, has been set through Eq.(3).

$$K_{e,\max} = 4\pi^2 m_e / T_e^2 \frac{\Delta_{D,el}}{\Delta_d} \tag{3}$$

Where $\Delta_{D,el}$ is the corner spectral displacement demand for the elastic damping level, and the other symbols have been defined above. It is evident that Eq.(3) reduces the required base shear by the ratio of the maximum expected spectral displacement demand to the design displacement. While this reduction is not supported by rigorous theory and it is expected that improved procedures could be developed as part of future research, it does provide a simplified means of considering the spectral displacement plateau in design.

In addition to the design spectral shape, the design intensity for different performance objectives has been indicated by specifying the probability of exceedence of design ground motions as a function of the importance class of the structure at three different performance levels, as indicated in Table 3.

Table 3. Specified probability of exceedence for different structural categories and performance levels.

	Earthquake Design Intensity		
Importance Class	**Level 1**	**Level 2**	**Level 3**
I	Not Required	50% in 50 years	10% in 50 years
II	50% in 50 years	10% in 50 years	2% in 50 years
III	20% in 50 years	4% in 50 years	1% in 50 years
IV	10% in 50 years	2% in 50 years	1% in 50 years

The three different performance levels, level 1, level 2 and level 3, correspond to a serviceability limit state, a damage control limit state and a collapse prevention limit state respectively. In order to achieve the desired performance, deformation limits have also been set for each limit state. Tables 4, 5 and 6 present the drift and strain limits set in order to control the performance of structural and non-structural elements. The research group has taken the initiative to set a number of strain limits for the first time, as guidance on strain limits for certain material types (e.g. soils and structural steel) could not be found in the literature. This fact perhaps emphasises the lack of importance that has traditionally been placed on the role of deformations for design. While effort has been made by Research Line IV to ensure that the limits indicated in these tables will provide the intended structural performance, it is expected that as the relationships between damage and deformation become better quantified (particularly for non-structural elements) the values indicated will be updated and the model code revised.

Table 4. Drift limits specified for different performance levels.

Drift Limit	Level 1	Level 2	Level 3
Buildings* with brittle non-structural elements	0.005	0.025	No limit
Buildings* with ductile non-structural elements	0.0075	0.025	No limit
Buildings* with non-structural elements detailed to sustain building displacements	0.010	0.025	No limit
Framed timber walls	0.01	0.02	0.03
RC Bridge Piers**	θ_y	0.03	0.04
Piers of Base Isolated Bridges	$2/3{*}\theta_y$	$2/3{*}\theta_y$	θ_y

Base isolated buildings also fall into this category. Optional – see code commentary.*

Table 5. Strain limits specified for structural elements for different performance levels.

Material	Level 1	Level 2	Level 3
Concrete comp. strain	0.004	Eq.(2.2) < 0.02	1.5Eq.(2.2)
Re-bar tension strain	0.015	$0.06\varepsilon_{su} < 0.05$	$0.09\varepsilon_{su} < 0.08$
Structural Steel Strain - Class 1 Sections, Flexural Plastic Hinges	0.010	0.025	0.04
Structural Steel strain - Class 2* & 3 Sections, Flexural Plastic Hinges	ε_y	ε_y	ε_y
Steel Brace Deformation Limits (see Eq. 2.3)	$\chi_{br}\varepsilon_y$	$0.25\mu_f\varepsilon_y$	$0.5\mu_f\varepsilon_y$
Reinforced Masonry comp. strain	0.003	Eq.(2.2) < 0.01	1.5Eq.(2.2)
Uneinforced Masonry comp. strain	0.003	0.004	0.004
Timber tension strain	$0.75\varepsilon_y$	$0.75\varepsilon_y$	$0.75\varepsilon_y$
Base Isolated Structures	*As per section 2.4.3 of code*		

Table 6. Strain Limits for Soils at Different Design Intensity Levels.

System	Level 1	Level 2	Level 3
Soil strain, γ	γ such that $G/G_{max} \geq$ 0.80	γ such that $G/G_{max} \geq$ 0.20	γ such that $G/G_{max} \geq$ 0.20
Retaining Walls Distant from Structures	γ such that $G/G_{max} \geq$ 0.40	γ such that $G/G_{max} \geq$ 0.20	γ such that $G/G_{max} \geq$ 0.20

In addition to maximum drift and strain limits, the model code has also proposed residual drift limits as shown in Table 7. However, except for the case of retaining structures (see Section 4.10) the residual drift limits are not obligatory in the code owing to the current difficulty that exists in proving compliance for normal structural solutions. The magnitude of the residual drifts were set considering limits typically set on the height over displacement ratios for other load types such as wind.

Table 7. Proposed Residual Drift Limits.

Drift Limit	Level 1	Level 2	Level 3
Building Structures	0.002	0.004	No limit
Bridge Structures	0.002	0.004	No limit
Retaining Walls distant from Structures	0.005	0.010	No limit

In the seismic design of a structure there are many complex phenomena that should be considered. However, in order to ensure a practical set of guidelines, efforts have been made to maintain simplified procedures whenever possible. For example, in order to control P-delta effects, special analyses are not specified in the code and instead the design base shear includes a single p-delta component which can be seen on the right side of Eq.(1). This term is in line with the recommendations of Priestley et al. (2007) and accounts for different hysteretic types through a p-delta constant C. Values of C of 1.0 and 0.5 are specified for steel and concrete structures respectively, thereby enabling designers to quickly account for p-delta effects in calculating the required design strength.

Another example is the simple procedure that has been provided to account for the twist that can develop for structures that are irregular in plan. Twist of a structure tends to increase the deformations on one side of a structure, and if not accounted for peak drifts would exceed the design drifts in this region. To account for such behaviour in a simplified manner, the design displacement of a system is multiplied by a torsion factor, ω_t, as shown in Eq.(4).

$$\Delta_d = \omega_t \sum_{i=1}^{n}\left(m_i \Delta_i^2\right) / \sum_{i=1}^{n}\left(m_i \Delta_i\right) \tag{4}$$

The basic reduction provided by ω_t should take account of the additional deformations that occur at the perimeter of the structure relative to the centre, due to torsion. As such, the model code states that ω_t is the ratio of maximum storey drift at the centre of mass to the maximum storey drift at the perimeter of the structure. Research by Beyer et al.(2008) indicated that the displacements of the perimeter of a plan-irregular building are typically not greater than 10% the centre of mass displacements if the strength eccentricity (see 7.1.3) is zero. Effectively, the findings of Beyer et al. (2008) indicate that the inelastic twist of a structure can be controlled by designing for zero strength eccentricity. As such, the model code states that the drift components required to define ω_t can be obtained from elastic analyses if the structure is provided with zero strength eccentricity. Alternatively, the model code suggests (in the

Commentary) that torsion be allowed for through advanced analyses or following the recommendations provided by Priestley et al. (2007).
By monitoring the work done by researchers external to the RELUIS project, it has been possible to incorporate several new research findings that are useful for displacement-based design. For example, a set of simplified yield curvature expressions are provided in Annex 1 of the model code and these include new expressions for the yield curvature of T and C-shaped RC walls developed by Smyrou et al. (2008) and Beyer et al. (2008) respectively. Another example is the set of recommendations provided to consider floor diaphragm flexibility and consider diaphragm design forces that are based on the findings and recommendations of Rivera (2008).
The model code has not only addressed the need for a coherent set of displacement-based guidelines for structures, but has also recognised that the success of a seismic design solution depends equally on an adequate capacity design method. As such, in the final chapter of the model code a number of alternatives for capacity design have been specified. The main uncertainty for capacity design is the effect that higher modes have on member forces. For example, Priestley and Amaris (2002) demonstrated that higher mode forces in RC wall structures are not effectively limited by the development of a plastic mechanism, as current code modal response spectrum methods assume. The general capacity design approach that is permitted for all structural types in the model code is to undertake non-linear time-history analyses of the structure, with strengths of plastic hinge regions set in line with the DBD findings. However, given the high level of expertise required to undertake non-linear time-history analyses, two alternative procedures have also been included. The first is a modified modal superposition approach in which first mode forces are set considering the actual design strengths provided and higher mode force components are estimated by undertaking modal analyses using structural models characterised with the effective stiffness of the structure at peak displacement response. While this approach has not been verified for all structural types, Priestley et al. (2007) argue that it provides the most consistent results of the modal methods available. The second, more simplified alternative to non-linear time-history analyses is to use an approximate method that is provided for RC frame structures, RC wall structures and RC frame-wall structures. An approximate but rather involved capacity design method is also given for isolated structures in an Annex of the model code. The approximate methods include empirical relations obtained by reviewing the results of a large number of non-linear time-history analyses of such structures. While the capacity design procedures provided in the model code do provide practitioners with a useful means of providing a good level of protection against undesirable inelastic mechanisms, it is considered that refined capacity design procedures, applicable to a wider range of structural typologies, could be developed as part of future research.

4.2 Reinforced concrete structures

As seen in Table 1, a number of different research units were formed to study RC structures with work done on reinforced concrete frame, wall, frame-wall and pre-cast structures. A relatively large amount of literature already existed for the displacement-based design of RC structures, and therefore much of the work done in this field was to test and refine the existing methodologies considering structural configurations common in Italy.
The research on RC frame structures reviewed the effectiveness of existing DBD procedures by undertaking a number of case study structures. One observation was that taller RC frame structures may often have a design displacement greater than the peak spectral displacement demand. This observation helped prompt the introduction of the general requirements on K_{emax} discussed in the previous section. The refined recommendations for RC frame structures

finally included in the model code were verified through consideration of a suite of case-study structures of 6, 9 and 12 storeys in height. One particular aspect that was studied was the recommendation in the model code to neglect the gravity loads in setting the flexural strength of beams. For comparison purpose the seismic design of the frames with nine and twelve storeys was undertaken with and without distributed gravity loads considered. The application of the design procedure was shown to be simple and useful. The actual behaviour of designed frames was investigated by performing pushover and non-linear dynamic analyses of the case study structures. A group of spectrum-compatible earthquake records characterized by increasing spectral displacement up to a period equal to 4 second were used. The envelope of displacement obtained from non-linear dynamic analyses was seen to be affected by the criteria adopted for distributing reinforcements in the columns. It was shown that distributed gravity loads on beams do not significantly influence the deformations recorded in the non-linear dynamic analyses, but their consideration in seismic design did considerably modify the forces that develop. The use of increased material strength and the non consideration of distributed gravity loads in the seismic design of beams helped limit the base shears to obtain a force-displacement response similar to that assumed in the design procedure. Furthermore, little differences were found between top displacement estimates from analyses and design. These results (refer Benedetti et al. 2008) indicate that use of the displacement-based design procedure enabled good control of the inelastic response, enabling performance-based design solutions to be realised.

Work on case-study RC wall and RC frame-wall structures provided support for the aforementioned capacity design recommendations included in the final chapter of the model code. Furthermore, various case studies reported by Rizzato et al. (2009), demonstrated that displacements could be better controlled using the displacement-based approach in comparison to traditional force-based design methods.

The research unit examining pre-cast RC structures also made some interesting observations about the relative performance of force-based and displacement-based design methods. Acknowledging that force-based design methods should not use the displacement ductility capacity to set a force-reduction factors (refer Section 2) the traditional FBD approach was modified by setting reduction-factors in proportion to the expected ductility demand. This improved FBD approach was then used to design a series of SDOF pre-cast column systems with the design objective of limiting storey drifts to 2.5%. The same SDOF systems were then designed using the DDBD approach, and non-linear time-history analyses were then undertaken using spectrum-compatible accelerograms to evaluate the performance of the different design solutions. Figure 3 compares the average of the maximum displacements for the force-based and displacement-based design solutions as predicted through non-linear time-history analyses.

The comparison made in Figure 3 is particularly interesting because it shows that while the DDBD solution has been fairly conservative, the level of protection against unwanted damage is fairly constant. This is in stark contrast to the FBD solutions in which it is seen that the potential damage appears to vary greatly as a function of the pier diameter selected. The variation in the FBD results has been attributed to the poor selection of cracked concrete section properties which codes recommend be set as a fraction of the gross section stiffness, without consideration of the strength assigned to the section. As such, this study by the pre-cast structures group illustrated how even when the ductility demand is used instead of the ductility capacity to set force-reduction factors, the design solutions can still be unsatisfactory and DDBD performance is much more consistent thereby enabling the provision of uniform risk design solutions.

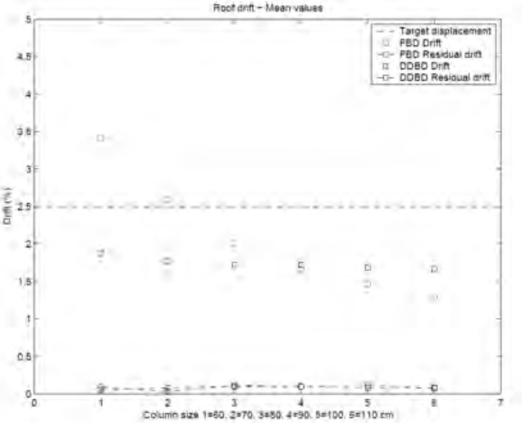


Figure 3. Comparison between target and recorded drift for pre-cast concrete columns of varying diameter, designed using both a force-based and displacement-based approach for a target drift of 2.5% and subject to spectrum-compatible non-linear time-history analyses.

Another useful set of guidelines developed for pre-cast concrete structures was to consider different base connection details in the seismic design of pre-cast structures. Four different connection types were studied and by considering how connection details influenced the hysteretic response, equivalent viscous damping expressions were calibrated for each type. As a result of this work, the model code includes the Eq.(5) for the equivalent viscous damping of pre-cast connections that are formed with grouted sleeves.

$$\xi_{eq} = 0.05 + 0.35\left(1 - \frac{1}{\mu^{0.226}}\right)\left(1 + \frac{1}{(T_{eff} + 0.719)^{0.681}}\right) \tag{5}$$

Where μ is the displacement ductility demand and T_{eff} is the required effective period. Refer to the work by Belleri and Riva (2008) for more detailed information on the influence of pre-cast column base connections within a DBD context.

4.3 Steel structures

The research into the displacement-based design of steel structures has focused on three types of steel structures: (i) steel moment resisting frame structures, (ii) steel frames incorporating buckling-restrained braces, and (iii) concentrically braced steel frames that utilise inverted V-bracing. Investigations into all three systems proceeded by forming trial methodologies (which required estimates of equivalent viscous damping and displaced shape) and evaluation of the performance of the design methods through advanced non-linear dynamic analyses. The new model code includes design expressions for the various steel systems. However, arguably the most significant findings have been obtained for the concentrically braced frame systems. The main feature of inverted V-braces is that the beam flexural stiffness must be explicitly taken into account in the design method, because it strongly affects the system response. This poses an additional difficulty for the analytical derivation of the displaced shape of V-braces. In fact, in addition to the cross section area and moment of inertia of braces and columns, the beam cross section second moment of area is also required. This aspect, which is specific of V-braces, is additional to other issues that characterize braced structures in general, such as (i) the small redundancy, which makes the system prone to weak storey mechanisms, (ii) the strong redistribution of column axial strain from the elastic through the inelastic range of response.

As a consequence of the previous issues, it emerged from both the theoretical and numerical studies that controlling the inelastic displaced shape of braced structures requires a careful selection of elastic member properties (refer Della Corte 2006). In other words, controlling the inelastic displaced shape requires controlling also the elastic vibration mode, because a strict relationship exists between the initial response in the elastic field and the subsequent plastic behaviour of the structure.

Equations have been proposed for calculating both the elastic and inelastic vibration modes. The proposed approach in the model code is based on the kinematics of braced structures, where the flexural and shear deformability of each storey are summed. The strict relationship between the response at buckling of the braces (which signals the end of the elastic system response) and the fully inelastic displacement response emerges clearly form the Equations.

In particular, in the proposed method, for a given distribution of brace and column slenderness, the required cross-section area of braces and columns, and the flexural stiffness of beams, can be calculated once the desired collapse mechanism has been fixed.

Numerical studies have been carried out in order to verify the efficiency of the proposed design Equations. All the numerical tests suggest that the procedure works properly and the response of structures designed according to the DDBD methodology is better than the response of similar structures designed according to standard and codified force-based design procedures. Figure 4 shows the results of a verification of a 10-storey concentrically braced frame structure designed using a simplified version of the developed DDBD method.

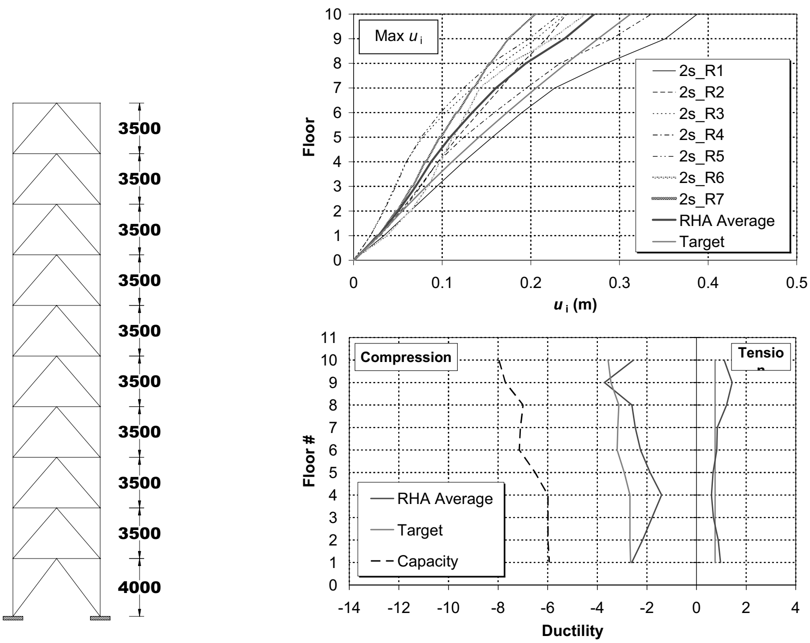


Figure 4. Results of case study verification analyses of a 10-storey concentrically-braced steel frame designed using DBD (i) elevation of case study structure, (ii) maximum displacements compared to design profile, and (iii) average ductility demands compared to target values.

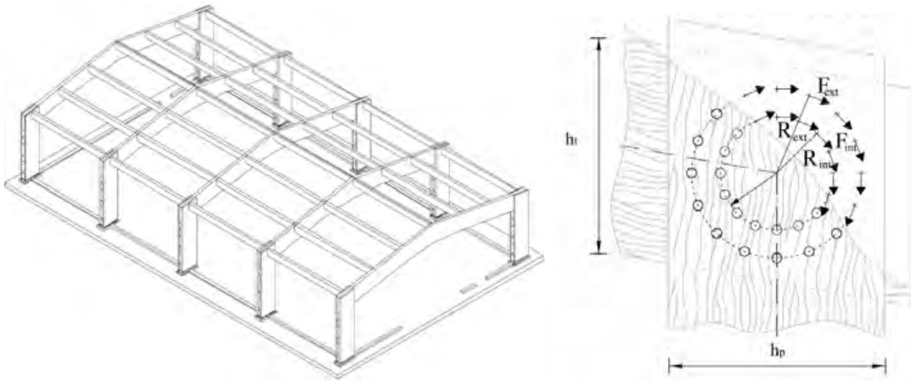
It is worth emphasizing that the numerical studies have also shown that some degree of empirical judgment must be placed on the selection of appropriate distributions for brace and column slenderness, which is an input choice in the design process. In particular, it was found that inappropriate distributions of brace slenderness may lead to design solutions that are only theoretical, because of the impossibility to find commercial shapes having the properties coming out of the design process. As such, some trial and error could be required before reaching a satisfying and feasible design solution. For more detailed information on the proposed DBD procedure for CBF systems refer to Della Corte and Mazzolani (2008).
It was noted for some case study structures that dynamic effects could be significant and the variability in the response may be sensitive to the model of elastic viscous damping selected for the non-linear time-history analyses. This was identified as an area requiring further research. In addition, the equivalent viscous damping expression utilised for the design of CBF systems was taken as the Takeda model. In reality the hysteretic response of braced frames can be quite different from the Takeda model and separate research by () has recently developed an expression for the equivalent viscous damping of CBF systems. Future research into the DDBD of steel structures should look to develop guidelines for eccentrically braced frame systems, as well as dual systems possessing steel moment-resisting frames and bracing systems in parallel.

4.4 Unreinforced masonry structures

Given the relatively limited work that has been done previously into the displacement-based design and assessment of masonry structures, a large amount of work is required to form of a comprehensive set of displacement-based design and assessment guidelines for masonry structures. In this project a DBD methodology was outlined for masonry structures, but in order to test this a number of modelling developments were required. As such, a significant amount of work focussed on the mechanical response of masonry piers and on the dynamic non-linear response of simple masonry structures. It was found that modern earthquakes have shown that masonry buildings often collapse due to the formation of soft-storey mechanisms, which suggests the structures can be considered as SDOF systems. This should enable simplifications in the design or eventual displacement-based assessment procedures.
In terms of modelling developments, work has been done to consider the difference between the peak load and the residual strength on the overall behaviour of a masonry structure. A constitutive model was developed to enable detailed analyses and to estimate the hysteretic damping of the structures, that maintains the simplicity of an elastic-perfectly-plastic model, but that takes account of the energy absorbed by the cohesive forces acting within the walls. The research unit has proposed the use of two different equivalent viscous damping expressions for masonry structures, one for masonry systems that develop a shear sliding mechanism and the other for masonry systems that develop a diagonal cracking mechanism. The expressions were obtained considering the results of a limited number of experimental studies and further research is required to fully develop a set of guidelines for the displacement-based design and assessment of masonry structures.

4.5 Timber structures

For the development of DBD guidelines for timber structures, the focus was on warehouses or commercial building structures formed glue-laminated timber portal frames as these are commonly built in Italy. In addition, the work focussed on the response of dowelled connections, as these were considered to represent the type of connection most extensively used in glulam construction technology in Italy.



(i) (ii)

Figure 5. Illustration of the timber portal case study structures of Zonta et al. (2008) (i) perspective view of portal structure, and (ii) annular bolted moment resisting connection arrangement.

In the model code strain limits have been set for timber to ensure that the timber does not yield. Inelastic deformations in timber structures should occur in ductile connections. For timber framed wall structures this can occur in the nailed connections and drift limits were set in line with the observations made by Folz and Filiatrualt (2001) and Filiatrualt and Folz (2002). It is noted that the experimental work undertaken on timber framed structures in support of the DDBD recommendations is fairly limited and future work in this area may lead to changes in the model code recommendations. For the timber portal structures it was recognised that it would be advantageous if the target displacement Δ_d of a portal could be estimated a priori, and the relation between equivalent damping ratio ξ_d and ductility μ, without prior knowledge of the final section dimensions or connection strengths. The formulation of target displacement Δ_d has been first theoretically stated and later refined, based on a calibration carried out using a Monte Carlo simulation. A total of one thousand different case studies were generated, assuming geometrical dimensions, joint configuration, and mechanical properties of materials as random variables. A statistical procedure was developed to select the geometrical dimensions of the members simulating the logical path commonly followed by the designer. In a similar way, a distribution of the mechanical properties were considered in order to reproduce the actual variability of the materials (glulam and steel) available in the market. For each sample, the actual ultimate displacement of the structure was calculated using non linear static pushover analysis. This analysis resulted in the following practical expression for the target displacement for the damage control limit state:

$$\Delta_d = c_1 \cdot \delta_{u,k} \cdot \frac{\theta \cdot \gamma \cdot \beta}{1 + 1/\theta \cdot \gamma \cdot \beta} \cdot \left(1 - \frac{q_{GRAV}}{q_{LIM}}\right) + c_3 \cdot H \cdot (\theta + 1) \cdot \gamma \tag{6}$$

where: $\delta_{u,k}$ it is the characteristic ultimate sliding capacity of the dowel; θ is the aspect ratio of the portal, γ=L/h is the rate between the span of the portal L and the height of the members cross-section h; β= is the rate between h and the external radius of the moment resisting joint; $q_{GRAV/qLIM}$ is the rate between design and ultimate gravitational loads; H is the height of the portal; c_1 =1.15 and c_3 =0.00555 are constants resulting from the calibration. As for the equivalent viscous damping ξ_d, the classical expression typically used was selected:

$$\xi_d = \xi_0 + \frac{a}{\pi}\left(1 - \frac{1}{\mu^{0.5}}\right) \tag{7}$$

and values were set for parameter a and ξ_0 based on the experimental outcomes of tests on full-scale fastened glulam joints, carried out according to UNI EN 12512. This load protocol includes a sequence of reversed cycles of increasing amplitude and is the standard used in Europe to characterize the response of fastened joints. The general assumption is that, for fastened connections, the hysteretic dissipation is mostly due to the steel dowels that embed in the wood during the load action. Because this mechanism implies a reduction of energy dissipation after each cycle, the total amount of energy dissipated is strictly dependent on the load protocol. Taking account of this issue, three different types of damping-to-ductility curves were defined. The first curve is that obtained under the same load protocol used in the experimental tests, while the other two represent the upper- and lower-bond curves, theoretically expected in the case of monotonically increasing cyclic load and constant amplitude cyclic load, respectively. In order to validate the design procedure Zonta et al. (2008) applied the Direct DBD method to three representative case studies. Using pushover non-linear analyses, it was demonstrated that the expression provides lower bound values of the displacement capacity that are close to those obtained through back-analysis using a much more refined model. On the other hand, the simulations also illustrated that the design base shear is very sensitive to the dissipation capacity of the joints. This is an area for future research required into the capacity design of timber structures. The comparison with the results of the current Eurocode 8 procedure shows that the DBD method can potentially overcome some of the limitations of the traditional Force Based Design methods.

4.6 Bridges

Guidelines for the DDBD of bridge structures had already put forward in a number of publications including that of Priestley, Calvi and Kowalsky (2007). As such, the main activities of the research unit studying bridge structures has been to consider whether the existing guidelines could be extended to irregular bridge configurations as well as bridge systems with tall piers. In order to achieve this a large number of case studies of both regular and irregular RC bridges were undertaken in this research project so as to develop the existing displacement-based design guidelines. In nearly all case studies examined for irregular bridges, it was observed that the DBD procedure performed rather well, whereas the FBD approach had difficulty in predicting the displacement profile and moments for the deck. An example case study bridge arrangement is shown in Figure 6 and results of non-linear time-history analyses for the bridge for two different pier heights is shown in Figure 7.

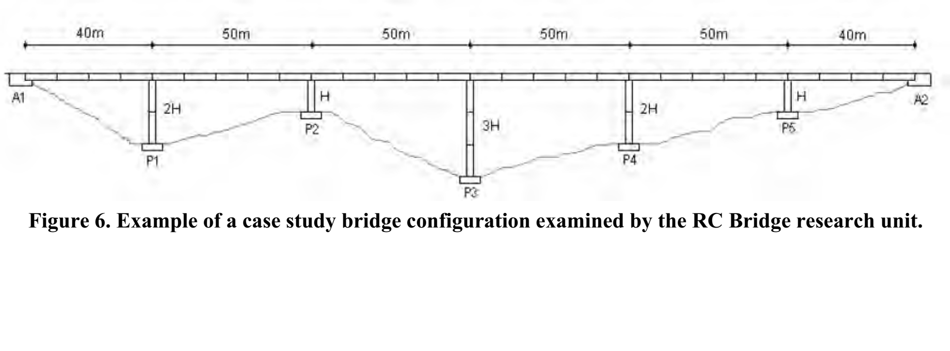


Figure 6. Example of a case study bridge configuration examined by the RC Bridge research unit.

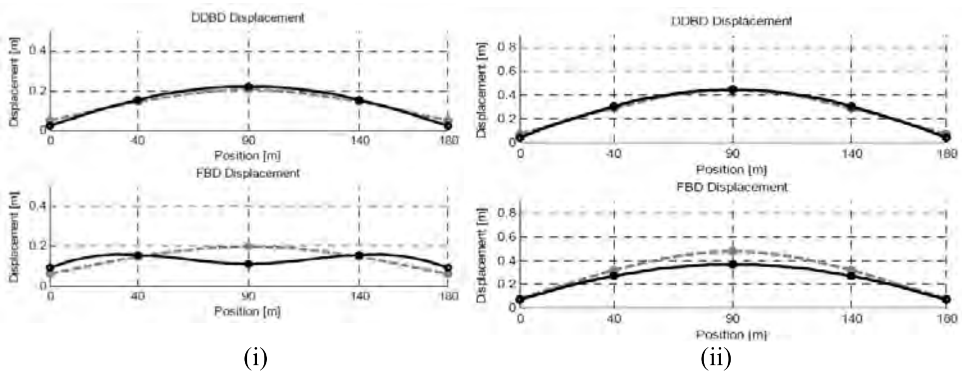


Figure 7. Comparison of DDBD (upper) and Force-Base Design (lower) predictions with non-linear time-history results for the transverse response of the case study bridge presented in Figure 6 with (i) H = 7.5m and (ii) H = 15m.

The relative performance of the methods for the short and tall pier scenarios presented in Figure 7 is worth considering carefully. It is seen that for the same proportions of a structure the force-based design approach may underestimate or overestimate deformations, and therefore damage. This highlights what is considered one of the main shortcomings with force-based design; difficulty in providing uniform risk solutions. Note that the DDBD procedure performs excellently for both cases.

To further improve the DDBD approach and capacity design recommendations, the following design refinements and recommendations have been proposed through the course of the work:

- *Distribution of shear force for design*. In general, for bridges with regular span and moderate pier heights, axial load distributions at pier bases are almost the same. For ease of construction, piers also have same cross-sections and reinforcement content. Thus moment capacity of all the piers is the same. Consequently, the shear force is distributed in proportion to inverse of the pier height or to ratio of displacement ductility demand and pier height, if the pier is yielded or not. For bridges with significant irregularities in span and pier heights (as the ones studied here) the assumption of equal axial load for all the piers is not valid and hence moment capacities are different at different pier bases. In these cases we suggest to represent the moment capacities as a percentage of moment capacity of the critical pier.
- *Modelling approach for tall piers*. Piers are commonly modelled as a single element with a percentage of mass lumped at top: this is valid for moderate height piers. For tall massive piers, lumping a percentage of mass at the top was found to underestimate the base shear significantly. Thus, either a distributed mass model or discretized pier elements are required where masses can be lumped at the intermediate nodes. Since the distributed mass model is computationally expensive, a lumped mass model is recommended. A modified equation is proposed that uses basic principles of statics and considers the inertia forces as acting at intermediate nodes to calculate an equivalent secant stiffness of the piers. The secant stiffness is obtained from the equation.

$$E_c I_{sec} \Delta_{top} = \left(V_B - \sum_{i=1}^{N-1} F_i \right) \frac{H^3}{3} + \sum_{i=1}^{N-1} \frac{F_i h_i^2}{2} \left(H - \frac{h_i}{3} \right) \quad (8)$$

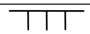
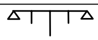
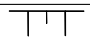
where, V_B and H are the base shear and overall height of the pier under consideration respectively, F_i's and h_i's are the inertia force and height of its application from base of the pier respectively. E_c is the modulus of the elasticity for concrete, I_{sec} is the secant moment of inertia of the pier section at base and Δ_{top} is the design displacement at pier top.

- For capacity design, a new modal superposition principle to consider higher mode effects at the potential flexural plastic hinge location for elastic and/or limited ductile pier could be used. In particular, effective modal superposition with a 5% damped acceleration spectrum is used to estimate the abutment shear and deck transverse moment, whereas effective modal superposition with the design displacement spectrum scaled down by an appropriate system damping factor is used to calculate the flexural moment demand at the potential plastic hinge locations (when higher modes are equally important as first inelastic mode).

Seven bridges were studied with different configurations to consider the effectiveness of these recommendations. All the bridges were designed according to refined DDBD methodology, compared with original DDBD and FBD procedures and validated by means of non-linear time history analyses. The results indicated that the above recommendations lead to better estimates of the inelastic displacement demand on the structures and lead to reinforcement quantities which are always more efficient than those obtained from FBD.

For the model code, an effort was made to simplify the DDBD procedure for the transverse direction. Design in the transverse direction might typically be considered difficult because the displaced shape is often uncertain and an iterative procedure is therefore required in order to establish an appropriate displaced shape. To simplify the procedure in the model code, Table 8 has been included together with Eq.(9) to Eq.(11).

Table 8. Displaced shape profiles provided for design of bridges in the transverse direction.

Pier Configuration		Abutment Type	Displacement Shape
Uniform		Pinned	Eq. 9
Uniform		Free	Eq. 10
Valley		Pinned	Eq. 9
Valley Ridge		Free	Eq. 11

$$\delta_i = \frac{16}{5L^4}\left(x^4 - 2Lx^3 + L^3x\right) \tag{9}$$

$$\frac{\delta_i}{\delta_c} = 1.0 \tag{10}$$

$$\delta_i = 1.5 - \frac{8}{5L^4}\left(x^4 - 2Lx^3 + L^3x\right) \tag{11}$$

In addition to the displaced shape, some account of higher mode effects has been incorporated through the specification of a higher mode deformation factor. This factor effectively reduces the design displacement to account for the additional deformations that will be caused by higher modes and which are not directly controlled through the displacement-based design procedure. A general procedure for estimating the higher mode deformation factor is provided. However, it is expected that future research will better quantify the magnitude of higher mode deformations and that the factor will then be updated.

4.7 Composite structures

Only limited developments were made during the course of the research for composite structures. Work was done to consider the best modelling possibilities for composite systems and various case study structures were investigated. However, further research is required to clearly identify deformation limits for different performance levels in composite structures, to identify likely displacement profiles at peak response, and to develop equivalent viscous damping expressions that consider the likely hysteretic characteristics of composite systems.

4.8 Isolated structures and structures with added damping

Two Direct Displacement-Based Design (DDBD) procedures for buildings and bridges with seismic Isolation Systems (IS's) have been developed, implemented and tested within the project. The proposed design procedures have been realized considering different idealized force-displacement cyclic behaviours, which may be used to describe the response of a wide variety of IS's, including: Lead-Rubber Bearings (LRB), High-Damping Rubber Bearings (HDRB), Friction Pendulum Bearings (FPB) and combinations of Flat Sliding Bearings (FSB) with different auxiliary devices (e.g. SMA-based re-centring devices, Viscous Dampers (VD), Steel energy dissipating components, etc.).

The design philosophy of the proposed procedures is based on the general requirement that the full serviceability of the structure should be maintained after the design seismic event, in order to avoid any repair or interruption of activities. The key aspect of the proposed design procedures is the target displacement profile of the structure. It is specified by assigning a predetermined displacement pattern and a target displacement amplitude to the structure.

For Base Isolated (BI)-buildings, the displacement pattern is derived from an approximate expression of the first modal shape of the BI-building. The target displacement amplitude is then assigned, by selecting suitable values of the maximum IS displacement and maximum storey drift, which comply with a full serviceability performance level of the building.

For bridges with IS, a rigid translation of the deck in the transverse direction is assumed, for both continuous and simply supported deck bridges. The target displacement amplitude of the deck, in both the longitudinal and transverse direction, is assigned to comply with a full serviceability performance level of the bridge, expressed through a limit value of the maximum displacements of IS's, piers and joints.

Several design examples and validation studies through Nonlinear Time-History Analyses (NTHA) have been performed to verify the accuracy of the proposed design procedures. A set of seven natural and artificial ground acceleration-time histories, compatible (on average) with a DBD-adapted version of the displacement response spectrum provided by the Eurocode 8 for soil type C, has been used in the numerical simulation analyses.

Several configurations of framed buildings, differing in (i) number of storeys (3, 5 and 7), (ii) IS types (HDRB, LRB, FPB with or w/o VD and FSB+SMA with or w/o VD) and (iii) target IS displacements and maximum storey drifts, have been examined within comprehensive parametric analyses. The results presented in Table 9 (Cardone et al. 2008) and in particular the ratio of the maximum storey drift to the design storey drift ($\theta_{SAP}/\theta_{Exp}$), illustrate that the method controls the displacement response excellently for a wide range of isolation systems.

Table 9. Results of verification analyses (from Cardone et al. 2008) of 3, 5 and 7-storey buildings designed with different base isolation systems (IS).

Conf.	IS	T_{fb} (sec)	T_{IS} (sec)	ξ_{IS} (%)	$D_{d\text{-}Exp}$ (mm)	θ_{Exp} (%)	$D_{d\text{-}SAP}$ (mm)	θ_{SAP} (%)	$F_{d\text{-}Exp}$ (kN)	$F_{d\text{-}SAP}$ (kN)	$D_{d\text{-}SAP}$/ $D_{d\text{-}Exp}$	θ_{SAP}/ θ_{Exp}	$F_{d\text{-}SAP}$/ $F_{d\text{-}Exp}$
3 Piani	HDRB	0.37	1.32	21.6	150	0.25	137	0.242	1358	1477	**0.91**	**0.97**	**1.09**
	LRB						133	0.215	1358	1262	**0.89**	**0.86**	**0.93**
	FPS		3.19	23.6	350	0.10	353	0.091	543	544	**1.01**	**0.91**	**1.00**
	SMA						356	0.091	543	545	**1.02**	**0.91**	**1.00**
5 Piani	HDRB	0.57	2.10	19.4	250	0.25	246	0.281	1342	1549	**0.98**	**1.12**	**1.15**
	LRB						274	0.267	1342	1436	**1.10**	**1.07**	**1.07**
	FPS		2.85	18.3	350	0.20	344	0.188	1020	1008	**0.98**	**0.94**	**0.99**
	SMA						332	0.185	1020	995	**0.95**	**0.93**	**0.98**
7 Piani	HDRB	0.78	2.92	19.3	350	0.25	343	0.276	1295	1469	**0.98**	**1.10**	**1.13**
	LRB						378	0.266	1295	1366	**1.08**	**1.06**	**1.05**
	FPS						349	0.250	1295	1289	**1.00**	**1.00**	**1.00**
	SMA						322	0.242	1295	1242	**0.92**	**0.97**	**0.96**

As further verification, the proposed design procedure has been applied to a case study, given by a 4-storey, 2-bay, three-dimensional RC frame designed for gravity loads only. The comparison between DDBD predictions and NTHA results confirmed the accuracy of the design procedure in the attainment of the target displacement profile of the BI-building, with percent differences lower than 10-15% in terms of IS displacements and lower than 15-20% in terms of maximum interstorey drifts, regardless the IS type used. The NTHA results, however, also pointed out that, in order to get an accurate estimation of the stresses in the structural members, the lateral force distributions to be used in the Linear Static Analysis (LSA) of the building must reflect the actual mechanical behavior of the IS. A distribution of lateral forces proportional to the storey masses (like that suggested in the standard formulation of the DDBD method) may be suitable for (quasi-)elastic isolation systems (e.g. LDRB, HDRB and LRB with low degree of non linearity) while do not for strongly nonlinear IS's (e.g. FPB, FSB+SMA and LRB with high degree of non linearity).

An extensive parametric investigation of NTHA has been carried out with the aim of defining accurate distributions of equivalent static forces, specific for each IS, that can be used to ensure successful capacity design. A new approach for the evaluation of accurate lateral force distributions for the LSA of BI-Buildings has then been proposed. The proposed enhanced lateral force distributions are proportional to the displacement profile of the structure, which is expressed as a linear combination of the first three approximate modal shapes of the BI-building, with the IS modelled through its effective stiffness at the design displacement. For this reason, the acronym 3-MM (3-Mode Method) has been used to identify the proposed method. The combination coefficients of the modal shapes are expressed as a function of a number of IS parameters, accounting for its actual nonlinear mechanical behaviour and isolation ratio of the BI-building. The guidelines for this method are included within Annex 4 of the new model code.

Further studies, based on NTHA of SDOF models of BI-buildings, have been conducted to assess the accuracy of the Jacobsen's equivalent damping approach (Jacobsen, 1960), used in the DDBD (along with the secant stiffness method) for the linearization of the structure. The results of these studies essentially confirmed the suitability of the adopted linearization approach in the evaluation of the maximum displacements of buildings equipped with different IS types.

The DDBD procedure for bridges designed with Isolation Systems (IS) has been verified through extensive NTHA, carried out on different configurations of multi-span continuous and simply supported deck bridges, differing in the layout of pier heights, IS types and associated mechanical parameters. Further NTHA have been conducted on a case study given by a 10-span simply supported deck bridge of the A16 (Napoli-Canosa) Italian Highway. The comparison between DDBD predictions and NTHA results effectively confirmed the reliability of the proposed design procedure in the attainment of the target displacement profile of the bridge, which is characterised by a uniform translation of the deck in the transverse direction. The attainment of the target displacement profile implies the achievement of the performance objectives of the design, which can be summarised as follows: (i) elastic behaviour of the piers, (ii) maximum IS displacements lower than their ultimate displacement capacity and (iii) adequate margin with respect to the gap closure of the joints. Obviously, a suitable compromise between the optimal IS configuration provided by the design procedure and a number of economic/practical needs must be pursued. Basically, this means to limit the number of different types of IS devices to one or two, thus accepting some errors in the attainment of the target displacement profile of the bridge.

Overall, significant developments have been made for the seismic design of isolation systems in this work and the model code incorporates many of the new findings. Future work in this area could aim to simplify some of the procedures and consider the development of design displacement profiles for different forms of building structure.

4.9 Foundation structures

The main, and ambitious, goal for research into the DDBD of foundation structures in this project has been to define a suitable procedure for seismic design of foundations in the framework of Direct Displacement Based Design (DDBD), accounting for non-linear soil-foundation interaction. The recent development of performance-based approaches for seismic design has raised awareness of the potentially relevant role of nonlinear dynamic soil-structure interaction (DSSI) effects on the response of the superstructure but that on the other hand there is currently a lack of suitable methods and experimental results in the literature required to support any design approach based on nonlinear DSSI concepts.

The lack of existing consolidated approaches has lead the research into the DDBD of foundation structures to focus, in the first half of the project, on the recovery of experimental data and development and calibration of numerical, albeit simplified, methods. The second half of the project was devoted to devise a suitable procedure to account for nonlinear DSSI in the framework of DDBD. Due to the innovative character of this research, priority was given to the problems related to the development of the procedure itself, rather than on the complexity of the structural types to be investigated. Therefore the research was mostly limited to shallow foundations of bridge piers, although, with suitable modifications, it could be extended to other structural types and deep foundations.

A synthesis of our research activities and findings related to foundation structures is as follows.

(a) Recovery of existing experimental data on the seismic behaviour of shallow foundations, making essentially reference to the results from the large-scale cyclic tests carried out at the JRC in Ispra during the European TRISEE project (1996-98), and, some years later, at the PWRI in Tsukuba, Japan, in the framework of the research cooperation agreement between the Public Works Research Institute and the Politecnico di Milano. Mostly important to shed light on the role of nonlinear DSSI were the shaking table tests carried out at PWRI within the same agreement.

(b) Calibration, based on the previous experimental results, of nonlinear constitutive models at different level of complexity, but with the common idea to represent the soil-foundation system by a single macro-element. This approach allowed fast and reliable parametric analyses to be undertaken, without making use of more sophisticated but time-consuming and difficult-to-calibrate finite elements approaches. The research activity on the numerical simulation of the PWRI shaking table tests has lead to the publication of Paolucci, Shirato and Yilmaz (2008).

(c) Based on the parametric analyses carried out within activity (b), several graphs and simplified formulas were proposed, suitable to capture the foundation stiffness decay and the corresponding increase of damping of the soil-foundation system, as a function of rocking and displacement of the foundation itself. Curves such as those shown in Figure 8 below (from Paolucci et al. 2009) are included in Annex 3 of the model code to aid designers consider foundations within the DDBD procedure.

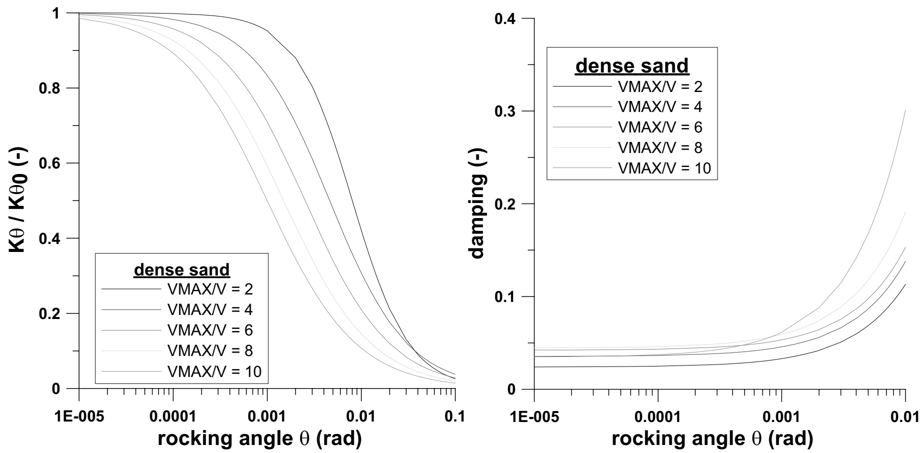


Figure 8. Variation of rotational stiffness (left) and equivalent viscous damping (right) for a shallow foundation on dense sand (from Paolucci et al. 2009).

(d) Development of a numerical code in Matlab to study the dynamic interaction between a single dof nonlinear oscillator with a foundation modelled by a nonlinear macro-element. The interesting feature of this software is to model the nonlinear DSSI in a simplified albeit rationale and physically sound way, with the purpose to analyze in which conditions the nonlinear DSSI effects may play a role in the seismic response of the super-structure.

(e) Finally, a procedure to account for non-linear DSSI in the DDBD was devised, in cooperation with the bridges research unit, making special reference to bridge piers on shallow foundation, with the production of different examples of application. A synthesis is presented in publication Paolucci et al.(2008).

Clearly, there has been a considerable amount of valuable work done into the seismic design of foundation structures and a number of useful publications have been produced. However, there is still a large amount of research required in this field as, for example, viscous damping for deep foundations is still uncertain and foundations on cohesive soils have not yet been studied. This should be the attention of future research projects into DDBD.

4.10 Retaining structures

Very little work had been done into the displacement-based design of retaining structures prior to this research project. The first challenge for the research unit examining retaining structures was therefore to consider how a DDBD procedure could be formulated for such systems. The work has focussed on the development of a DDBD approach for cantilever diaphragm wall structures embedded in coarse-grained soils, with and without anchors. Through collaboration with research unit 1, it was recognised that the soil-structure system could be represented as an equivalent SDOF system by considering layers of soil that would displace during a seismic event, as illustrated in Figure 9.

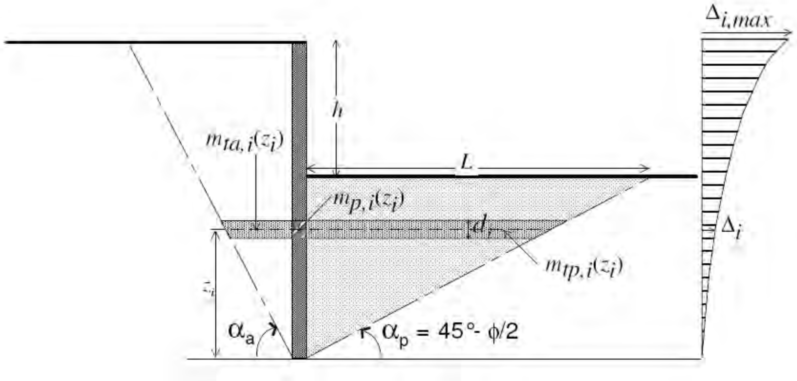


Figure 9.Discetisation approach used in the equivalent SDOF representation of retaining structures.

Some consideration has been given to the deformation limits to be used as part of a DBD approach. In collaboration with research unit 11, it was agreed that soil strains should be limited in relation to the effective reduction in the soil modulus (see Table 6). However, it was also considered that the major performance criteria for retaining structures that do not form part of the foundations of buildings, is the residual deformation that remains in the walls following an earthquake event. Limits on residual deformations for retaining structures have been specified in the model code as shown in Table 7. Note that there is not a strong scientific background to these residual deformation limits and future research could prompt changes. Clearly, the peak displacement is likely to be larger than the residual displacement. However, as little is known about the likely relation of peak to residual deformations of retaining structures, it has been proposed in the model code that the residual deformation be assumed equal to the residual deformation. By assuming a displacement profile for the wall, the equivalent SFOF characteristics of the MDOF soil-structure system could then be determined and a standard DDBD approach followed (see Cecconi et al. 2006).

Another significant proposal for the retaining walls DBD procedure has been to propose an equivalent viscous damping value (ξ_e) for the system in accordance with Eq.(12).

$$\xi_e = \frac{\xi_{ta}\sum_{i=1}^{n} m_{ta,i} + \xi_{tp}\sum_{i=1}^{n} m_{tp,i} + \xi_p\sum_{i=1}^{n} m_{p,i}}{\sum_{i=1}^{n}(m_{ta,i} + m_{tp,i} + m_{p,i})} \tag{11}$$

Within this evaluation the soil damping (active, ξ_{ta}, and passive, ξ_{tp}) components and the wall damping component, ξ_p, have been factored by the relative masses. It has been proposed in the model code that the damping components of the soil can be related to the soil shear strain in line with the work of Vucetic and Dobry (1991). While the model code has included these recommendations to enable designers to familiarise themselves with the DDBD approach, it is considered that additional verification analyses may be required to fully verify the equivalent viscous damping of the complex systems.

Different analysis procedures available in the current engineering practice were critically reviewed by the retaining structures research unit to consider how DDBD solutions compare with more traditional methods. Comparison of the design base shear required by the DBD calculation and more traditional pseudo-static design approaches (e.g.: Mononobe-Okabe method) suggested that the former approach is likely to be more realistic than the current pseudo-static analysis, despite its immediateness and simplicity. Numerical analyses were performed using the commercial finite-element-method (FEM) code PLAXIS.v.8.5. The comparison between the results of the method and those obtained from dynamic numerical analyses has shown fair agreement. However, it was noted that an adequate choice of the soil masses participating in the seismic event - i.e. the geometry of the active/passive soil wedges - and of the equivalent damping are fundamental in the calibration of the design procedure. To this aim, the availability of well-documented case histories is strongly needed and should constitute one of the main objectives of future research. Clearly, there is also a large amount of additional research that could be undertaken in order to consider other forms of retaining structures and other considerations such as sloping backfills, water table effects, and cohesive soils.

5 DISCUSSION

Overall this project has been very successful in achieving a set of ambitious objectives. The main product has been the formation of a model code that may be considered a world first. This was only possible because of excellent leadership and active collaboration between internal research units. In addition to the model code, many high quality publications and developments to the DDBD procedure have been made. However, not only has this research been successful on a technical front, but it has also been a very effective means of increasing awareness of the capability of displacement-based design methods throughout Italy. With greater numbers aware of the DDBD approach it is expected that further developments to the guidelines and state-of-the-art will occur more rapidly.

In terms of management considerations, it was noted that collaboration between universities was assisted by having a fairly limited number of research units. However, collaboration tended to occur between research units that were located geographically close to each other. This could be considered for future research proposals in which collaboration might be desired.

It should be noted that progress in some cases was significant, as seen in previous section. However, in other cases only limited progress was achieved during the three year research project. Two examples would be the cases of composite structures and masonry structures. As there was very little existing literature on the DDBD of such structures it appears that the research line goals of developing a set of guidelines for such systems were possibly too ambitious. However, there is also a management challenge in such situations, to encourage research units to seek assistance in overcoming some of the significant technical challenges they may encounter. Clearly, because of the limited progress that was made, DDBD of composite and masonry structures will need to be the subject of future research projects. There are of course a number of other research areas that also need development. In

particular, while an excellent start has been made, further research into foundation structures and retaining structures would be very beneficial.
A final point for discussion is the difficulty that researchers face when trying to compare DBD and FBD procedures. The code-based FBD approaches require a number of decisions to be made in relation either to the structural modelling or choice of design parameters, such as the structural behaviour or reduction factor. Without clear guidance as to the exact modelling and design approach, two different designers may obtain very different solutions using the same code procedures. As such, the comparison between DBD and FBD procedures can usually only be indicative.

6 VISIONS AND DEVELOPMENTS

The work undertaken by research line IV of the RELUIS project has seen the development of a world first for earthquake engineering; a model code for displacement-based seismic design. The vision is that this document will be considered and refined as research in the field continues in years to come, and that it will be eventually developed into a formal code for displacement-based seismic design.
The code is the main product of a three year research project involving 11 different universities throughout Italy. The work undertaken by the different research units has clearly demonstrated some of the benefits and advantages of displacement-based seismic design over current code force-based design methods. Some of the findings and recommendations made in the work are preliminary and future research will endeavour to validate or refine the current provisions in the model code. However, the production of the model code and the set of supporting case study examples will mean that in the future the recommendations for DDBD can be more easily developed, refined and extended to an even greater number of structural typologies.
Another area that was identified as a possible area for research at the start of the RELUIS project was the displacement-based assessment of structures. Limited progress was made towards this goal principally because of the large number of uncertainties that needed to be overcome to firstly enable displacement-based design guidelines to be formed. However, considering the seismic risk associated with the existing building stock in Italy, it is clear that the possibility of providing practitioners with more effective and more reliable tools for seismic assessment is very attractive. As such, in addition to refining the design guidelines, the next major goal for research in this field may be the formation of model code for the displacement-based seismic assessment of structures.

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DEVELOPMENT OF INNOVATIVE SEISMIC DESIGN CRITERIA FOR STEEL AND STEEL-CONCRETE COMPOSITE STRUCTURES

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1 INTRODUCTION

1.1 General

Several objectives were planned at the starting of the ReLUIS Project, Research Line 5, based on the general aim to improve the new Italian seismic design code (OPCM 3274) which was proposed at that time. Through the three years of research activity, a strong discussion took place among the researchers, practitioners and legislators, finally leading to a revolution in the Italian building code. This has slightly changed the perspective of this research, but the final results can be considered basically consistent with the initial premises.

The research has been developed by 12 Research Units (RU), each of which was composed by people belonging to 12 Universities and engaged on a different topic, as described below:

1) RU 1: University of Naples “Federico II” – Faculty of Engineering; research coordinator: Federico M. Mazzolani; Topic: Design criteria and response assessment of buckling restrained braces.
2) RU 2: University of Naples “Federico II” – Faculty of Architecture; research coordinator: Raffaele Landolfo; Topic: Rotation capacity and classification of steel members.
3) RU 3: University of Chieti-Pescara; research coordinator: Gianfranco De Matteis; Topic: Design criteria and response assessment of metal shear panels.
4) RU 4: Second University of Naples; research coordinator: Alberto Mandara; Topic: Design criteria of steel moment resisting frames and methods of nonlinear analysis.
5) RU 5: University of Salerno; research coordinator: Vincenzo Piluso; Topic: Design criteria and response assessment of composite steel-concrete bridge piers.
6) RU 6: University of Pisa; research coordinator: Walter Salvatore; Topic: Design criteria and response assessment of steel bridges.
7) RU 7: Polytechnic of Milan; research coordinator: Carlo A. Castiglioni; Topic: Procedures to assess low-cycle fatigue of steel members and connections.
8) RU 8: University of Trento; research coordinator: Riccardo Zandonini and Oreste Bursi; Topic: Design-by-testing for buildings and bridges
9) RU 9: University of Sannio; research coordinator: Maria Rosaria Pecce; Topic: Rotation capacity and classification of composite elements.
10) RU 10: University of Molise; research coordinator: Giovanni Fabbrocino; Topic: Methods for structural analysis of composite steel-concrete structures.
11) RU 11: University of Ancona; research coordinator: Luigi Dezi; Topic: Composite frames with dissipative braces.

12) RU 12: University of Trieste; research coordinator: Claudio Amadio; Topic: Degrading hysteresis models and fragility curves of frames with composite semi-continuous joints.

In the following, the activity developed for each of the 12 topics is presented separately. This is just a very short summary of the whole research activity. A detailed description of the activity done by each RU will be given in a separate book, which is currently under preparation.

1.2 Design criteria and response assessment of buckling restrained braces (RU 1, University of Naples Federico II – Engineering Faculty – coordinator: F.M. Mazzolani)

The development of advanced steel bracing systems has been the main task of the research activity developed by RU 1. In particular, the research has been focused on the study of an innovative type of "all-steel" buckling restrained brace (BRB). The most common type is the so-called "unbonded" BRB, where the restraining unit is a tube filled with mortar or concrete and an unbonding material is placed at the contact surface between steel and concrete. "All-steel" BRBs may be seen to have some advantages over "unbonded" braces. In fact, "all-steel" BRBs can be designed to be detachable, so that they can be inspected after each seismic event and, if necessary, the yielded steel core can be replaced by a new one. Besides, a detachable BRB allows maintenance during the life-time. To do this, two or more restraining tubes are connected each other by bolted steel connections (Tsai *et al.*, 2004). Moreover, an 'all-steel' BRB is lighter than an 'unbonded' one; this implies a technical and economical advantage during the fabrication and the erection. These considerations led to study "all-steel" detachable BRBs. An experimental program has been planned, which consisted of three full-scale tests on a real reinforced concrete (RC) building equipped with the specially designed BRBs. One fundamental characteristic of the proposed novel type of BRBs, which will be better described in the following, is that they can be hidden in the inner space between the two panels of masonry walls commonly used as claddings of RC buildings.

1.3 Rotation capacity and classification of steel members (RU 2, University of Naples Federico II – Architecture Faculty – coordinator: R. Landolfo)

The work of Research Unit 2 has the purpose to furnish a contribution to the topic of the behaviour under seismic actions of Steel Moment Resisting Frames and in particular on the Member behaviour. Starting from the assumption that in modern design practice it is generally accepted that steel is an excellent material for seismic-resistant structures because of its strength, ductility and capability to withstand substantial inelastic deformations, an experimental campaign on steel beams has been made. The principal scope of the work has been the revision of the classification criteria of steel members actually adopted by seismic codes and the introduction of a new criterion which takes into account the principal factors that influence the structural response.

1.4 Design criteria and response assessment of metal shear panels (RU 3, University of Chieti-Pescara – coordinator: G. De Matteis)

The activity carried out by the Research Unit n.3 focuses on the experimental and numerical assessment of the performance of metal shear panels (MSPs) for seismic protection of steel and RC structures in order to achieve a better knowledge of their behaviour and to implement useful design criteria.

1.5 Design criteria of steel moment resisting frames and methods of nonlinear analysis (RU 4, Second University of Naples – coordinator A. Mandara)

The building codes generally use strength as the main design criterion and they consider the lateral force procedure at the base of the earthquake resistant design. The displacement control usually plays a secondary role, and the deformation demand is generally checked at the end of the design process for the serviceability limit state. However, recent earthquakes have shown that structures may suffer irreparable or too costly to repair damages. Furthermore, inelastic behaviour, indicating damage, is observed even during smaller earthquakes. As a consequence, the modern seismic design requires the application of performance-based concepts, with multi-level objectives pursued. From this perspective, the fundamental design parameters are the displacements and the ductility demands, and the most suitable approach in modern seismic design is to ensure that these parameters will not be exceeded under the design-level of the seismic action. The defined level of damage during a specified earthquake ground motion should be ensured by performance-based criteria. These criteria should be selected such that at the specified levels of ground motion and with defined levels of reliability, the structure will not be damaged beyond certain limiting states.

1.6 Design criteria and response assessment of steel bridges (RU 5, University of Pisa – coordinator: W. Salvatore)

The UR5 target was the evaluation of the problems related to seismic protection of steel bridges and steel-concrete composite decks, paying attention to new criteria and rules introduced by Ordinanza 3431/2005.

The research aimed to analyze the relevant typologies of steel and steel-concrete decks selected for the realization of road and railway bridges, identifying the critical details and the actually available methodologies for safety and serviceability assessment.

Afterwards, research program expected to perform numerical analysis of some case studies, representative of the most diffused road and railway steel bridges, in order to verify the applicability of such methodologies, paying particular attention to the design rules indicated by Ordinanza 3431/2005. Main target of this step was the development of models characterized by a high level of reliability in the representation of bridge case study dynamic behaviors in order to evaluate properly the structural seismic responses, following Italian OPCM 3431/2005 rules.

Final step of research program was the development of some guidelines on modeling technique and seismic analysis of steel and composite steel-concrete bridges.

1.7 Seismic response and design rules of steel-concrete composite bridges (RU 6, University of Salerno – coordinator V. Piluso)

Modern international seismic codes suggest to design bridge structures according to the capacity design approach. Italian code is in agreement with this approach. In particular, under severe earthquakes the seismic input energy has to be absorbed by the piers, which are recognised as dissipative zones, while all the other parts of the bridge have to remain in elastic range. Therefore, in terms of seismic behaviour, a bridge can be considered made of a steel-concrete composite structure provided that the bridge piers are made of a composite solution, while constructional technologies adopted for the bridge deck are not relevant. For this reason, the main topic of the planned research activity is represented by the monotonic and cyclic behaviour of steel-concrete composite columns. In particular, Concrete Filled steel Tubular (CFT) columns rather than Concrete Encased steel Composite (CEC) ones are examined, since they are more suitable as bridge piers from both the structural and the technological point of view. Therefore, due to the interaction between the two materials, within the research activity the attention has been devoted to all the parameters influencing the behaviour of composite columns under monotonic or cyclic loading conditions. In fact, the response of a composite member subjected to axial force and variable bending moment is affected by the confinement of concrete, the biaxial stress state in the steel tube, the local buckling effects for the steel profile and the constitutive laws of materials. The final purpose of the research activity of Unit N.6 is the development of theoretical models able to predict the response of composite beam-columns, i.e. composite bridge piers. In particular, the models are devoted to the determination of the moment-curvature relationship and force-displacement response of such members. These models have been compared with the results of the experimental tests led on CFT members at the *Materials and Structures Laboratory* of the *Department of Civil Engineering* of *Salerno University*. These models are useful to carry out non linear analyses on steel-concrete composite bridge piers aiming at the development of design rules for such structures which, currently, are not covered by seismic code provisions.

1.8 Procedures to assess low-cycle fatigue of steel members and connections (RU 7, Polytechnic of Milan – coordinator: C. A. Castiglioni)

The objectives of the research carried out by this Unit was to provide reliable and effective procedures to estimate the low-cycle fatigue behavior of steel and steel-concrete composite members and joints.

1.9 Design-by-testing for buildings and bridges (RU 8, University of Trento – coordinators: R. Zandonini and O. Bursi)

The activity of the Research Unit 8 is focused on the evaluation of the feasibility and effectiveness of the use of composite joints in moment-resisting frame structures subjected to earthquakes and fire in sequence; and on the development of performance-based design tools and design rules for new steel-concrete composite road bridges and for the retrofit of existing bridges.

1.10 Rotation capacity and classification of composite elements (RU 9, University of Sannio – coordinator: M. R. Pecce)

The general topics of the research dealt with the post-elastic deformability of composite beams and columns.
In particular three specific topics have been focused: 1) the definition of the transfer mechanism (bond) at the concrete – steel interface in composite columns type partially encased, 2) the study the base connections of partially encased columns, 3) the evaluation of the rotational capacity of composite beams under hogging bending moment.
For each subject it has been done a state of art about the most significant aspects and recent knowledge. Then experimental tests have been projected and realized for topics 1) and 3), furthermore available experimental results from previous research have been analysed in order to develop a finite element model; in some cases the results give direction for review the provisions of the Italian code.

1.11 Methods for structural analysis of composite steel-concrete structures (RU 10, University of Molise – coordinator: G. Fabbrocino)

The activity of the University of Molise (UNIMOL) research unit has been related to the analysis of the structural behaviour of steel-concrete composite structures, with reference to the concrete slab and steel beam behaviour and their interaction. The following are specific topics dealt with:

- Review of the state of the art of the experimental response of shear connections;

- Numerical modelling of inelastic behaviour of composite beams and shear connection systems;
- Validation of numerical modelling with reference to existing experimental data, including data from experimental research from others Operative Units.
- Development of guidelines for the seismic analysis of composite steel-concrete structures.

1.12 Composite frames with dissipative braces (RU 11, University of Ancona – coordinator: L. Dezi)

A large dissipation capacity may be achieved in steel and composite moment frames, but the energy dissipation implies damage to the structure and repairing such damage after an earthquake is typically expensive. Additionally, although this design criterion satisfactorily prevents the structure from collapsing in the case of high intensity earthquakes it often leads to very significant lateral deformability which brings on excessive inter-storey drift under low intensity earthquakes or wind loads. Conventional concentrically braced frames, on the other hand, ensure high-lateral stiffness for drift control and the earthquake-induced energy is dissipated only by the diagonal bracing members. The energy dissipation capacity of concentric braces is however strongly reduced by brace buckling. In order to improve the dissipation capacity of concentric braces dissipation devices may be used to produce innovative highly dissipative braces. Dissipative braces may be coupled with a moment resistant frame. In this case, under the philosophy of performance-based design, the dissipative brace system may be designed to prevent damage to the structure by seismic loads or at least limit them. In this research project attention was focused on Buckling Restrained Braces and dissipative braces based on High Damping Rubber (HDR). In this research project attention was focused on two kinds of dissipative braces: Buckling Restrained Braces (BRBs) and dissipative braces based on High Damping Rubber (HDR). In particular the aim of the research unit was to investigate the main modelling problems concerning the use of dissipating braces and to developed appropriate design methods.

1.13 Degrading hysteresis models and fragility curves of frames with composite semi-continuous joints (RU 12, University of Trieste – coordinator: C. Amadio)

The main activities planned for the research unit (RU) 12 are:
- The development of a cyclic component model for steel-concrete composite joints.
- The evaluation of fragility curves for steel-concrete composite frames.

2 BACKGROUND AND MOTIVATION

2.1 Design criteria and response assessment of buckling restrained braces (RU 1, University of Naples Federico II – Engineering Faculty – coordinator: F.M. Mazzolani)

In the last years, steel dissipative bracing systems have been widely and successfully used as complementary structural elements, and sometimes also as substitutive elements of other lateral load resisting systems under seismic actions (Brown *et al.* 2001, Della Corte *et al.* 2005). Among them buckling-restrained braces (BRBs) are one of the most versatile and effective solutions. Indeed, differently from conventional braces, BRBs do not exhibit appreciable difference between tensile and compression force capacity and no degradation of brace response under cyclic loading. Since lateral and local buckling behaviour modes are restrained, large inelastic capacities are attainable. Nowadays, many theoretical studies and experimental tests of retrofitting systems are available in the technical literature (Watanabe *et al*. 1988, Black *et al.* 2002, Tsai *et al.* 2005, Wada & Nakashima 2004, Xie 2006). Laboratory experiences are valuable for studying the intervention techniques, but they present serious limitations, due to the difficulty to correctly reproduce actual boundary conditions and to take into account the scale effects in reduced scale models, as well as to introduce the actual structure defects (e.g. constructional tolerances, bad execution, reinforcing bars corrosion and/or concrete degradation). Therefore, the present study has an additional value because it consists of both the analysis of real existing buildings and the comparison of different technologies for seismic upgrading.

2.2 Rotation capacity and seismic classes for steel members (RU 2, University of Naples Federico II – Architecture Faculty – coordinator: R. Landolfo)

Dissipative systems are required to withstand inelastic deformations without significant decrease of resistance. With this regard, the evaluation of available and required ductility plays a key role. The attainment of a global ductile behaviour of the structure is possible only if members have adequate local ductility which can be measured in terms of plastic rotation ϑ_p. The rotation capacity can be measured in different ways (A. Kemp, 1985; B. Kato 1989; U. Kuhlmann, 1989). Starting from the generalized moment-rotation curve obtained through the three point bending test, the rotation capacity can be defined as $R = \frac{\vartheta_{\max}}{\vartheta_y} - 1$. The evaluation of the R parameter can be obtained through experimental tests, measuring and calculating the rotations ϑ_{max} and ϑ_y, or with the following methods (Piluso, 1992): (i) *theoretical methods,* which are based on the theoretical evaluation of the moment-curvature relationship (M–χ); (ii) s*emi-empirical methods,* which consider local buckling phenomena through relationships determined by means of experimental tests; (iii) e*mpirical methods,* based on statistic analyses of a large number of experimental or numerical data.
In the seismic design of MRFs the subdivision of members into ductility classes is necessary. At the same time, the subdivision of the members into overstrength classes is necessary in order to apply capacity design criteria. The twice classification of steel members in ductility and overstrength classes should be the most appropriate approach for the seismic applications. The work of RU 2 is finalised to provide a contribution on the discussion about this topic, through a critical analysis of the classification criteria currently proposed by some European codes and the evaluation of their reliability starting from available experimental results.

2.3 Design criteria and response assessment of metal shear panels (RU 3, University of Chieti-Pescara – coordinator: G. De Matteis)

The use of MSPs is developing very quickly as a suitable way for the improvement of the seismic response of both steel and reinforced concrete buildings. Two typologies of this seismic resisting device are usually detected: one based on the use of stiffening thin metal plates and the other exploiting the dissipative performance of stiffened dissipative panels. The first significant study on stiffening thin metal panels were developed at the University of Alberta (Canada) during the eighties (Thorburm et al. 1983; Timler and Kulak, 1983). These represented the basis for the set up of simplified models, such as the “strip model” which is indicated for design purpose by the ANSI/AISC-341 (2005). Research carried out by Tromposh et al. (1987), Rezai (1999), Lubel et al. (2000), Sabouri-Ghomi and Roberts (1991) and Astaneh-Asl (2001), must be mentioned. As far as the dissipative panels are concerned, many studies have been undertaken in the last two decades. They were addressed to the investigation of two types of devices. The first, mainly developed in the Japanese area

(Nakashima et al., 1994 and 1995; Nakashima 1995; Nakagawa et al.,1996; Katayama et al., 2000), is based on the use of low yield steel, whilst the latter envisage proper weakening techniques of the base plate, such as holes (Roberts et al, 1992; Bruneau et al, 2005) or slits (Hitaka and Matzui, 2003). While for new constructions a significant numbers of buildings equipped with shear metal panels can be cited starting from the 80s, it has to be stated that applications for existing RC structures is still inadequate. In fact, the first numerical application of Mo and Perng in 2000 on portal RC frames with light gauge trapezoidal sheeting remained isolated for many years. It has been followed only in 2006 by Wang, who compared the efficiency of MSPs with some other techniques (RC jackets, RC shear walls and steel braces) for retrofitting a non ductile RC hospital building. On the other hand, as far as the numerical activities are concerned, only in 2007 Kono et al. were able to both propose and test a revised version of the Mo and Perng's system based on the use of corrugated thin steel plates. Nevertheless, these limited applications have been never occurred in Europe.

2.4 Design criteria of steel moment resisting frames and methods of nonlinear analysis (RU 4, Second University of Naples – coordinator A. Mandara)

The aim of the conventional design approach is to satisfy two requirements: a) under ordinary actions the structure must be stiff in order to minimize structural and non-structural damage; b) under severe earthquakes the structure must be safe from collapse even if some structural damage may be tolerable. The serviceability limit state (SLS) and the ultimate limit state (ULS) are checked with a linear elastic analysis. Undesirable brittle failure mechanisms are usually avoided with local ductility requirements and capacity design rules. Three different approaches are generally used to apply the capacity design rule: 1) amplification of the acting moment in the seismic design situation (ECCS 1988; New Italian Code 2008); 2) check of hierarchy criterion after analysis (EC8 2003; UBC 1997 and AISC 1997; Lee 1996); 3) Plastic design (Mazzolani-Piluso 1997) based on the application of the kinematic theorem of plastic collapse. So, the steel moment resisting frames (SMRFs) are expected to be able to sustain large plastic deformations in bending and shear. However structural damage and collapses during recent earthquakes have evidenced some critical aspects in the seismic behaviour of steel structures even when designed according to the current design codes.

2.5 Seismic response and design rules of steel-concrete composite bridges (RU 5, University of Salerno – coordinator V. Piluso)

The issues concerning the behaviour of steel-concrete composite members are nowadays worldwide investigated. In fact, during last years the use of composite structures is widely increased due to their characteristics of stiffness, resistance and ductility. A lot of researchers have focused their attention on issues concerning this topic, such as the interaction between steel and concrete (Mander *et al.* 1988, Shams and Saadeghvaziri 1999, Susantha, *et al.* 2002), the axial load carrying capacity (Shams and Saadeghvaziri 1997), the response under cyclic flexural loads (Gourley et al. 2001), in some cases providing models for the prediction of the behaviour at collapse. Nevertheless, only in few cases the proposed models exhibit a defined theoretical genesis, while in most cases they are obtained by means of numerical calibrations requiring regression analyses of available experimental results. Conversely, the aim of the present research is the development of analytical procedures based on well recognised theoretical formulations describing all the effects influencing the ultimate behaviour of composite members. The developed models are strictly related to the theoretical interpretation of the phenomena affecting the behaviour of CFT bridge piers, so that they assume a general relevance.

2.6 Design criteria and response assessment of steel bridges (RU 6, University of Pisa – coordinator: W. Salvatore)

In recent years, steel and composite steel-concrete solutions became more and more used in the field of bridge structures. Many factors underlie this change: the use of new high yield thermo-mechanical steels; the use of high thick plates and improved fatigue behavior; new methods of structural analysis; the reduction of maintenance costs (new treatments for steel and new products for the painting); the increase in transport capacity and assembly; the improvement of welding technologies. But, from the seismic point of view, small attention has been paid to steel and composite steel-concrete solutions. In the design of new bridges generally superstructures are expected to remain elastic also during earthquakes. Substructures and bearings can enter into plastic domain. It is also possible to include ductile elements capable to dissipate seismic energy or isolation systems, generally located near to the piers or to the end cross-frames. From the analysis of damages induced by past earthquakes it was clear that the major damages are typically present in bearings and substructures. Nevertheless, once triggered, those "primary" damages lead to the occurrence of "secondary" damages on the superstructure. It is clear that seismic design and testing of bridges demands a proper evaluation of dynamic characteristics of the structure. This assessment is usually performed by numerical analysis of finite element models, for which in current regulations the modeling rules and the values of mechanical parameters such as mass, stiffness and damping properties are rather uncertain. In case of existing structures, it is possible to evaluate the seismic behavior through the use of experimental dynamical tests and updating procedures of mechanical model ("model updating"), capable of providing an accurate estimate of resistant scheme and current values of mechanical parameters. In the design of new structures, instead at present it is clearly necessary to develop a technical knowledge specifically aimed at identifying the best modeling techniques for the analysis of bridge's typical structures.

2.7 Procedures to assess low-cycle fatigue of steel members and connections (RU 7, Polytechnic of Milan – coordinator: C. A. Castiglioni)

The loading history to be applied in the simulation of the response of structures subjected to earthquakes by means of laboratory tests is a fundamental topic, because it affects the evaluation of the ductility capacity. Indeed, it has long been recognized that the application of repeated fully-reversed cyclic loading in the inelastic range causes the structure response to degrade, both in stiffness and strength. The phenomenon is known as low-cycle fatigue behaviour. Recent test results show that the assessment of the ductility capacity is sensitive to the loading protocol. In principle, the loading protocol should be defined on the basis of a probabilistic analysis of the seismic demand to the specific structural member or connection under analysis. In practice, conventional loading history are often applied (ECCS, ATC 24), but this may produce significant overestimation of the available ductility in some cases.

2.8 Design-by-testing for buildings and bridges (University of Trento – coordinators: R. Zandonini and O. Bursi)

Current structural codes deal with seismic safety and fire safety separately and the sequence of seismic and fire loadings are not taken into account. But, the risk of loss of life increases if a fire occurs within the building after an earthquake. In the Kobe Earthquake (1995) many people died due to the collapse of buildings exposed to the fire that followed the earthquake. It is obvious therefore that fire after earthquake is a design scenario that should be properly addressed in any performance-based design. Therefore, one objective of RU 8 was to develop a design procedure of composite joints under earthquake and post-earthquake fire. Besides, a second objective was to study the performance of steel-concrete composite bridges. Bridge engineering has always been a branch of civil engineering that has attracted great interest, as it deals with structures of fundamental importance for the economic framework of a region. In Italy, the new building code (NTC 2008) introduced, by means of the OPCM (2005), a new approach in the seismic design of structures as well as a new seismic classification of the national territory. This entailed that portions of the country that previously were not classified as seismic sensitive suddenly changed their nature becoming low-seismicity zones. This implied that existing strategic structures, such as bridges, had to be checked against seismic actions. With regard to the design of new bridges the OPCM (2005) imposes that the deck remains elastic and that the seismic energy should be dissipated by the substructures or particular seismic isolation devices. Therefore, if the bridge is not seismically designed, the deck might experience low-cycle fatigue in hot-spot details that shall be identified. As a result, one existing steel-concrete composite box-girder viaduct that has to be retrofitted was analysed, both with regards to the static and the seismic behaviour.

2.9 Rotation capacity and classification of composite elements (RU 8, University of Sannio – coordinator: M. R. Pecce)

Composite steel-concrete structures have been found to be cost-effective especially for multi-storey buildings (Broderick & Elnashai, 1996). Composite systems for buildings may include steel moment resisting frames with composite beams and columns or braced frames with composite columns (Cosenza & Zandonini, 1997). Under severe earthquake loading concrete encasement cracks and reduces the flexural stiffness of composite beam-columns but the steel core acts as a back-up system in providing the shear strength (Elnashai & Elghazouli, 1993; EC8, 2004). The assessment of structural response of composite steel and concrete columns is thus of paramount importance, especially in seismic design (Fabbrocino *et al*., 2001; Thermou *et al*., 2004, Bursi et al, 2004). The inelastic response of composite columns is significantly affected by the beam-column joint (Shanmugam & Lakshmi, 2001) and column base connections. Code formulations of rotational capacity of composite beams and columns are still lacking. About the beam column joint, a relevant role could be assumed by steel-concrete bond. The bond mechanism is complex and depends on many parameters (Virdi & Dowling, 1980; Hunaiti, 1994). Experimental tests are available only for concrete filled columns under monotonic loads (Hunaiti, 1994; Johansson & Gylltoft, 2002; Mouli & Khelafi, 2007). Therefore a research program for partially encased columns was projected. The ductility of base column connections depends on the type of column and connection system. It is noteworthy that analytical and experimental research focusing on the effects of the base connection is lacking (Spacone and El-Tawil, 2004). Few results are available and their applicability within the capacity-design framework should be further investigated (Hajjar, 2002; Mazzolani, 2003). The composite action may affect the failure modes endangering the inelastic performance (Di Sarno and Elnashai, 2002). Experimental tests (Di Sarno et al., 2006) have shown that an innovative base-column joint is the socket type system. However a few experimental data and numerical models (Pertold et al. 2000) are available. As regards the deformation capacity of composite beams the behaviour under hogging bending moment is particularly interesting, since it is the most probable stress condition at the column joint due to vertical load effect and is an unfavourable working condition of a composite beam. The problem solution could be very complex since the rotational capacity of composite beams depends on many parameters: the concrete slab and its reinforcing rebars, the type of steel profile, the beam-column connection and the shear connectors.

2.10 Methods for structural analysis of composite steel-concrete structures (RU 9, University of Molise – coordinator: G. Fabbrocino)

Evaluation of composite members at the ultimate limit state (Eurocode 4, CNR 10016) requires specific procedures, able to take account of the actual redistribution capacity of structures, which however depends on complex phenomena of interaction between concrete slab and steel profile (Cosenza and Zandonini 1997). These aspects are very important for structures located in seismic zones (Fabbrocino *et al.* 2000). One fundamental issue with the analysis of composite beams is the determination of the "effective width". A number of numerical studies are available and some codes provide detailed rules based on these studies. However, recent parametric analyses indicated that simpler expressions could be adopted. These outcomes are reflected by the Italian seismic design code (OPCM 3431), but further analyses need to be carried out for continuous beams. Different degrees of continuity may be achieved between beams and columns and, possibly, between adjacent beams. Furthermore, the degree of continuity can vary significantly in relation to the performance of joints which can be designed to be continuous or semicontinuous. The transfer of large shear forces dictates that suitable mechanical connectors must be used. New types of connectors have been continuously developing since the early stages of composite construction. The behaviour and modes of failure of each type of connector highly depend on the local interaction with the concrete. Due to the inherent difficulty of deducing the resistance of shear connectors from beam tests, most of the usable data come from the push-out tests, which do not entirely reproduce the more favourable connector's condition in a composite beam. A further problem arises from the lack of consistency among the geometrical and detailing features of push-out specimens designed differently in different research studies. This situation demands for a careful selection of "comparable" data, in the background studies to code specifications.

2.11 Composite frames with dissipative braces (RU 10, University of Ancona – coordinator: L. Dezi)

The concept of BRBs was firstly developed in Japan with the aim of improving traditional concentric braces (Uang and Nakashima 2004). In Italy these devices were mainly developed to retrofit existing RC structures, such as the buckling restrained axial dampers (BRAD) manufactured by "Fip Industriale". These devices usually have a limited length and consequently are placed in series with over-strength braces (Antonucci *et al.* 2007). The internal core is usually made of steel but different materials, like aluminum, may be used in order to obtain less stiff devices to adopt in more flexible structures.

Whit regard to HDR-based devices, HDR is extensively adopted in bearings for the seismic isolation or the vibration control (Grant et al. 2005), however may be conveniently used even for innovative dissipative braces, to use in new or existing structure (Fuller et al. 2000, Bartera and Giacchetti 2004). These devices generally provide lower energy dissipation with respect to elastic-plastic dampers but may be preferable because they withstand a large number of cycles without permanent deformation and dissipate energy even for very small displacements.

Finally, regarding design methods for dissipative braces, procedures suggested by current codes for conventional braces (PrEN 1998-1, NTC2008) are force-based procedures and thus have some limitations. Firstly, since the natural period of the structure is unknown at the beginning of the design procedure, it is based on empirical formulas giving an approximate value of the natural period of the structure. Alternatively, complicated iterative procedures have to be adopted in order to estimate the correct structural period. Moreover, in the FBD procedures, structure displacements are only the final output of the design process to be calculated once all the member cross sections have been fixed. Consequently structural and non structural damages, usually related to story drifts cannot be controlled. These limits may be overcome by using modern Displacement-Based-Design (DBD) procedures (Priestley et al. 2007), where displacements are used as the main input to the design process and the structural period is a result of this process.

2.12 Degrading hysteresis models and fragility curves of frames with composite semi-continuous joints (RU 11, University of Trieste – coordinator: C. Amadio)

The Performance Based Seismic Design (PBSD) is the modern approach in seismic design, based on the fulfilment of specific design performance objectives. These are damage levels that are expected to be not exceeded when a structure is subjected to an earthquake ground motion of specified intensity. If the Load and Resistance Factor Design approach is followed, specific partial safety factors and seismic design spectra are used. They are calibrated so as to guarantee a fixed level of structural reliability for each limit state. Such design approach, even though recognizes uncertainties in structural behaviour, doesn't allow the designers to control the actual probability to exceed a specific performance in the case of an event, the earthquake ground motion, which is uncertain. The procedure that enables to evaluate the seismic performance of a structure accurately is the probabilistic approach. The main sources of uncertainties must be statistically defined and the capacity of the structure must be evaluated performing nonlinear dynamic analyses using advanced structural models. For these reasons it is also important to use appropriate models to simulate the cyclic joint behaviour of partially-restrained steel-concrete composite frames to evaluate the fragility and performance curves of frames designed with actual codes.

3 RESEARCH STRUCTURE

3.1 Design criteria and response assessment of buckling restrained braces (RU 1, University of Naples Federico II – Engineering Faculty – coordinator: F.M. Mazzolani)

The research activity has been addressed to both evaluate the benefits of ductile steel braces on global response of RC frames with non-ductile details and, on the other hand, to assess proper design criteria of the studied devices so that to improve their mechanical performance.
During the first year of the research program, the RU1 advanced the knowledge about BRB devices thanks to the reworking of the results of two experimental tests within the ILVA-IDEM project (Della Corte & Mazzolani 2006, Mazzolani 2006). During the second and third year of the research program, the RU1 designed and tested an innovative "all-steel" BRBs. However, differently from the past experiences, the BRB system under examination was designed such that to be hidden in the hole between the backing and the facing of common masonry infill panels. Then, this device has been mounted onto a real two-story RC building that was tested up to formation of a clear collapse mechanism.

3.2 Rotation capacity and classification of steel members (RU 2, University of Naples Federico II – Architecture Faculty – coordinator: R. Landolfo)

The first part of the work has regarded the state of art concerning steel Moment Resisting Frames (MRF) in seismic areas. Subsequently the attention has been focused on the classification criteria of steel members and steel sections adopted by seismic design codes. The study has clearly indicated the necessity of a unified classification criterion for both ductility and overstrength. With this purpose an experimental campaign has been carried out in the Laboratory of Civil Engineer at the University of Salerno. The main scope of the experimental campaign has been the study of the behaviour of steel members under monotonic and cyclic loads. The adopted scheme is the cantilever beam which reproduces the behaviour of a beam in a frame subject to seismic actions (Figure 1).

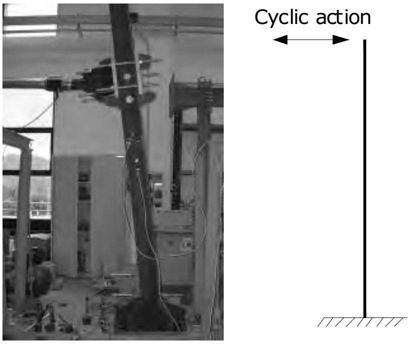


Figure 1. Adopted scheme for tested specimens.

The principal purpose has been the investigation of the parameters influencing rotation capacity and overstrength, such as material strength, ratios of slenderness between flange and web of plates in the section, moment gradient and cyclic loads. Three different typologies of cross-section have been investigated: 1) I sections, 2) Rectangular hollow sections, 3) Square hollow sections. The experimental activity has been constituted by two parts: 1) the identification of the material mechanical properties through the tensile tests on specimens taken from flange and web of the beams; 2) tests on members performed through an equipment that applies predetermined strengths or displacements checking their values through a control device during monotonic and cyclic tests.
Subsequently finite element analyses on I, RHS and SHS members, to simulate the experimental tests, have been performed. The proposed numerical models are finalized to reproduce the observed monotonic and cyclic behaviour of tested members. The program ABAQUS (Version 6.7-1) has been used to perform the numerical analyses.

3.3 Design criteria and response assessment of metal shear panels (RU 3, University of Chieti-Pescara – coordinator: G. De Matteis)

In the framework of the research activity briefly described in Section 2, the Research Unit n.3 has defined three main tasks to be developed in the RELUIS project, aiming at analysing the behaviour of three different typologies of shear panels (slender, compact and stiffening) made of metallic materials.
The first task: Theoretical and Numerical Studies on Slender Shear Panels
Theoretical and numerical (both simplified and refined) studies on slender steel shear panels are dealt with in order to set-up effective design criteria. Parametric numerical analyses and simplified theoretical models have been performed aiming at assessing the influence of the panels geometry (aspect ratio and thickness) on their behaviour.
The second task: Experimental and Numerical Studies on Pure Aluminium Shear Panels This activity concerns the experimental-numerical evaluation of aluminium shear panels performance in order to give efficient low-yielding devices as alternative to plates made of low-yield steel which is not produced in the European market. Both Full bay type and Bracing

type panels have been tested under cyclic loading. Some of the experimental tests have been simulated by refined FEM .

The third task: Numerical-Experimental study on metal (steel and pure aluminium) shear panels for seismic retrofitting of existing RC buildings

This activity has the task of detecting he main properties of such panels aiming at foreseeing their practical use.

The full activities related to the three tasks are broadly described in De Matteis *et al.* (2007) and Mazzolani *et al.* (2007).

3.4 Design criteria of steel moment resisting frames and methods of nonlinear analysis (Second University of Naples – coordinator A. Mandara)

At first, the specific performance criteria (stiffness, strength, ductility and low-cycle fatigue) and corresponding performance parameters for steel moment-frame buildings (interstory drift ratio, roof drift ratio, residual permanent drift ratio, plastic hinge rotation, Miner damage index) were investigated. Then a comparative evaluation of design procedures considering both building code provisions (DM 96 - CNR 10011, Eurocode 8, OPCM 3274, New Italian Code 2008) both innovative methods based on the limit analysis or on the second order plastic analysis was carried out. At this aim, a multi-objective performance evaluation of steel moment-frame buildings was realised with a combination of the performance criteria both for structural members and for non-structural components at the different limit states. The objective is to select design procedures that balance stiffness, strength, and ductility in order to have the desired seismic performance at each intensity level of the earthquake ground motion. Analytical models of various complexities are evaluated using nonlinear static pushover analysis and nonlinear time history analysis. Both higher modes contribution, and variation of inertial force as an effect of spectral amplification were investigated. A simplified seismic demand estimation procedure in which the spectral characteristics of the ground motion are related to the inelastic deformation capacity for the structure is developed. In particular, as an alternative to incremental response history analysis, an incremental non-iterative nonlinear static procedure based on the inelastic and capacity spectra method was used for the displacement-based seismic assessment. Both adaptive and non-adaptive pushover procedures were considered in the analyses. These procedures were finally compared with the nonlinear static procedures proposed in FEMA 356, ASCE/SEI 41-04, ATC40, FEMA440-ATC-55. Their effectiveness was estimated by comparison with incremental time-history response.

3.5 Seismic response and design rules of steel-concrete composite bridges (RU 5, University of Salerno – coordinator V. Piluso)

The research has been developed according to three phases.

In the first phase, an adequate knowledge of the worldwide research activity concerning concrete filled steel tubes has been carried out. The attention has been mainly focused on the confinement of concrete, the biaxial stress state in the steel tube, the local buckling effects for the steel profile, the constitutive laws of materials.

In the second phase, starting from the theoretical formulations of the issues mentioned above, analytical models aimed at the evaluation of the moment-curvature relationship and force-displacement behaviour of CFT members have been developed.

Finally, in the third phase an experimental program has been carried out on steel Square Hollow Section filled with concrete, whose results have been compared with numerical ones.

Interaction between steel profile and concrete core

The interaction between the concrete core and the steel profile in CFT columns is due to the fact that the lateral expansion of concrete is restrained by the steel tube. Therefore, the stress state acting in the plane of the section is responsible of several effects: the stress state of the concrete benefits of the confinement effect and the bi-axial stress state of the steel modifies the yield stress in the axial direction (Elremaily and Azizinamini, 2002). The behaviour of confined concrete has been described by means of several analytical formulations, the most accredited probably being the one proposed by Mander *et al.* (1988). As a consequence of the bi-axial stress state acting on the steel profile, the steel yield stress in tension and in compression are different and an asymmetrical stress-strain relationship has to be considered within the framework of fiber models. In addition, due to the presence of the concrete core, the inward deformations of the steel tube are prevented, so that only the outward local buckling of plates can occur. As a consequence, ductile behaviour is observed even after the occurrence of local buckling (Shams and Saadeghvaziri, 1999). From the analytical point of view, a useful approach to account for local buckling of compressed plates of SHS columns is constituted by the effective width approach.

Numerical models

Aiming to predict the ultimate behaviour of CFT columns, numerical procedures have been developed to evaluate the moment-curvature relationship and the force-displacement curve of the member (Mastrandrea and Piluso 2007, Mastrandrea *et al.* 2008). The moment-curvature relationship is obtained using fiber models. Starting from the knowledge of the moment-curvature relationship, a procedure able to provide the force-displacement curve of a CFT member has also been implemented. The reference scheme is represented by a cantilever beam-column (which also corresponds to the behaviour of the half of a simply supported member subjected to a three-point bending test). However, the implemented procedure has a general relevance. The force-displacement curve is obtained by means of an iterative step-by-step procedure, under displacement control.

Experimental program

During the third phase of the research project, UR 6 spent efforts in experimental tests on Concrete Filled Tubular (CFT) members (Iannone *et al.* 2009). The experimental program is aimed at the evaluation of the ultimate behaviour of such members. Square Hollow Section CFT beam-columns have been investigated, both under monotonic and cyclic loading conditions. The specimens have been located horizontally and fixed to the end hinges by means of bolted connections, to assure the continuity of the member. Three point bending tests under cyclic loading conditions have been performed. No axial load was applied to the specimens, because the axial load acting on bridge piers is not significant, being a small part of the squash load. Even if a simply supported scheme is adopted to test the beam-column, the results can be easily extended to a cantilever beam-column scheme (which is relevant for bridge piers) by considering an half member. One of the two end hinges presented slotted hole, in order to allow free sliding of the specimen in the longitudinal direction. Several measuring devices have been adopted to record displacements and deformations. The measure of the displacement at force location has been carried out by means of the LVDT of the hydraulic actuator and also by means of external displacement transducers. In addition, six couples of strain-gauges have been applied on the vertical lateral plate elements of the steel box, in two control sections, one on the right side and one on the left side of the rigid stub. Each couple of strain gauges has been located in order to allow the measure of the curvature.

Each specimen has exhibited a satisfactory ductility. Hysteresis cycles are large and stable, without sudden loss of strength. Many cycles are needed to achieve 50% loss of strength. In all the cases, bulges formed at both top and bottom flanges of steel tubes, close to the rigid stub, where bending moment assumes its maximum value. This means that CFT columns exhibit high global ductility and are able to absorb a great amount of seismic input energy.

3.6 Design criteria and response assessment of steel bridges (RU 6, University of Pisa – coordinator: W. Salvatore)

In the first year, UR5 program was constituted by the study of the most relevant typologies of street and railway bridges with regard to the seismic behavior of deck structures and supports. In this step, research target was the analysis of typical seismic damages of bridge most diffused solutions, critical details and modern seismic protection techniques.
In the second year, research target was the identification of case studies representing the most diffused typologies of composite steel-concrete street and railway bridges. Actual numerical analysis techniques for the evaluation of seismic forces had also to be analyzed in order to develop finite element models of case studies.
In the third year, numerical dynamic response of finite element models had to be analyzed in order to study the abilities of actual modeling techniques of street and railway composite bridges in describing the real dynamic behavior of case studies. The abilities of finite element model updating processes based on experimental data had also to be studied. The target was the development of reliable finite element models in the evaluation of dynamic behavior of case studies, in order to obtain a correct analysis of seismic forces acting on structures. Finally some guidelines on modeling technique and seismic analysis of steel and composite steel-concrete bridges had to be defined.

3.7 Procedures to assess low-cycle fatigue of steel members and connections (RU 7, Polytechnic of Milan – coordinator: C. A. Castiglioni)

An innovative testing procedure was developed which intends to capture the hybrid nature of loading imposed to critical beam-to-column connections when subjected to vertical and action load effects. The testing procedure is composed by a sequence of reversed cycles (repeated when in the post-elastic range) in which each cycle has an initial force-controlled part and a final displacement-controlled part. Figure 2 depicts typical positive and negative succeeding cycles. This innovative testing procedure has been used to test beam-to-upright specimens representing sub-assemblages of steel storage racks. The studied specimens consist of typical beam-to-upright connection of steel storage racking systems. Connection of the beam to the column is achieved through beam end connectors (welded onto the beam), which are slotted and bolted to the upright element, Figure 2. The loading history was applied according to the ECCS cyclic procedure and the innovative loading procedure in order to obtain direct comparisons.

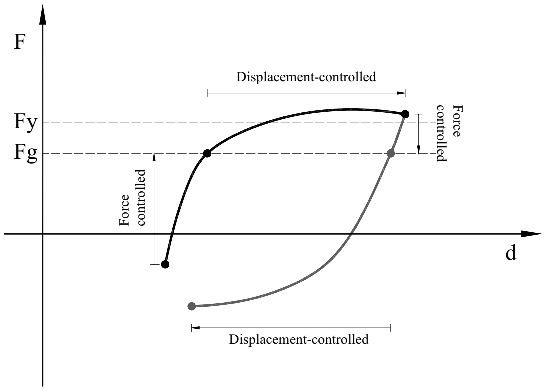


Figure 2. Typical positive and subsequent negative cycles (positive force induces negative bending moments at critical beam-to-column connection).

3.8 Design-by-testing for buildings and bridges (RU 8, University of Trento – coordinators: R. Zandonini and O. Bursi)

The research program has been aimed at developing the following activities:

- Analysis of the state of the art of both steel-concrete composite structures and steel-concrete composite bridges;
- Definition of the analysis and design methodologies for beam-to-column joints under seismic and fire load condition and for composite bridges in seismic prone area;
- Design of the composite joints to be tested under monotonic and cyclic loading;
- Assembly of the designed interior beam-to-column specimens;
- Execution of the experimental tests;
- Development of 3D numerical models of the joints in order to evaluate stress and strain distribution, thermal analysis on the different components, yielding in critical zone, ductility and rotation capacity.
- Design of reference frames under Fire;
- Numerical simulations on two-dimensional (2D) frames in order to study different fire scenarios acting in the reference buildings and to evaluate the performance of different elements;
- Design of beam-to-column joints subjected to fire load;
- Carrying out of fire tests on full-scale substructures representing interior both pre-damaged and undamaged bolted beam-to-column joints;
- Identification of critical details in composite bridges by means of the development of 3D FE models in which the structures are subjected to strong earthquakes;
- Validation of the 3D FE models by means of output-only ambient vibration tests;
- Tests on lattice girders in order to study the behaviour of reinforcing bars in compression during the construction stage of concrete decks;
- External Post-Tensioning Retrofit and High- and Low-Cycle Fatigue Study of Connections and Joints of Steel-Concrete Box-Girder Bridges;
- Development of optimised solutions for steel details of the end diaphragms in steel-concrete composite bridges;
- Reliability analysis by using the validated FE model of the steel-concrete composite bridge, taken as case study.

3.9 Rotation capacity and classification of composite elements (RU 9, University of Sannio – coordinator: M. R. Pecce)

The transfer mechanisms (bond) between steel and concrete in partially encased columns

The state of art highlighted the parameters influencing the bond behaviour at the steel-concrete interface: type of composite section (partially or fully encased, concrete filled), mode of load application (on concrete, on steel or on both materials), type of stress (compression, tension, bending, cyclic loads), concrete properties (type, age, hardening conditions and temperature). Experimental results are lacking for partially encased columns and the Italian code gives low limits to the bond at the web interface and neglect the contribution of flanges. An experimental campaign of bond tests has been projected and carried out for partially encased columns; specific set-up and specimens have been realized with instrumentation to measure the bond-slip law and the measure of local strain. Sixteen monotonic tests have been executed, but the first five have been excluded from the analysis since problems occurred during the tests. Three cyclic tests were carried out, too.

Model of connection at base column

A finite element model has been developed for a partially encased column already tested during another research program (Di Sarno et., 2006); the connection system at the base is an innovative one type socket. Three-dimensional finite elements were used for column and foundation block; the constitutive relationship was assumed elastic-plastic for steel, bilinear for concrete in tension and multilinear in compression. The topic of the activity was to assess the model for this innovative base connection in order to work out a parametric analysis.

Rotational capacity of composite beams under hogging bending moment

The classification of composite beams for local buckling depends hardly on the extension of the compressed part of the web, especially if it is longer than 50% of the entire web; furthermore other parameters influence the rotational capacity: percentage of slab reinforcement, connection degree and connectors distribution, semi rigid joint to column.

A state of art has been elaborated (Loh et al., 2004; Kemp et al, 1995; Fabbrocino et al., 2001) about experimental results of composite beams under hogging moment.

Experimental tests have been projected and executed on 4 composite beams under hogging moment. The length of the beams is 4 m, that could be significant of a continuous beam 6-8m long in a frame and the loading pattern of three point bending test was used. The profile is an IPE360 and the steel is S275; for the rebars the steel is Feb 44k. Two different distributions of the connectors (uniform and not uniform) and two types of RC slabs have been realized; the slab is solid or with profiled sheeting, the thickness is always 13cm and two widths are considered (Figures 3 and 4). The tests were conducted in displacement control and many global and local measurements (deflection, strains, slip between slab and profile) were done.

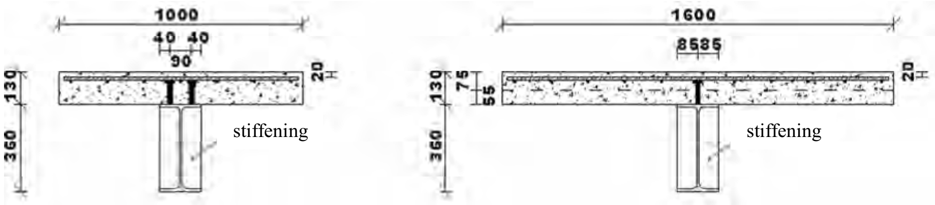


Figure 3. Cross sections of the beams tested.

Figure 4. The beams before concrete casting.

3.10 Methods for structural analysis of composite steel-concrete structures (RU 10, University of Molise – coordinator: G. Fabbrocino)

A total of n.6 tests have been conducted under displacement load, using stud connectors Φ 16, Φ 19 and Φ 22. Two specimens were build for each stud diameter, the first one having the slab subjected to tensile stresses, the second having unloaded slabs.
In the first case, the shear connector acted on the slab via an eccentric load. Consequently a rotation effect of the slab was showed, which was related to the restraint of the reaction system on the slab. Such a rotation cannot be avoided by the flexural stiffness of the steel bar connections to the test frame.
The unloaded condition of the slabs was obtained by means of rigid steel plates at the top of the slabs. In such a way the slabs directly acted onto the test frame.
When geometrical eccentricity of test setup become significant, a rigid rotation between concrete slabs and steel profile is displayed. Therefore the connectors resulted under tension and consequently a lower load carrying capacity of the specimen resulted. A reliable estimation of the stresses in the connectors is a very difficult task; to this end, however, specific experimental tests were conducted by Aribert (1988), characterised by different levels of accuracy.

3.11 Composite frames with dissipative braces (RU 11, University of Ancona – coordinator: L. Dezi)

In the first year of the research project, attention was focused on numerical simulation of HDR-based dissipative devices. In particular the following activities were carried out:

- an analytical model of HDR was developed on the basis of available experimental results referring to several tests carried out on single devices;
- results of experimental tests carried out on a real scale steel-concrete composite frame equipped with a dissipating bracing system based on HDR devices were analyzed and numerical simulations of the tests were performed by using the proposed model;
- a parametric analysis on S-DoF systems equipped with HDR-based devices was conducted in order to highlight the main non linear phenomena occurring in this kind of structure.

In the second year of the research program the activities concerning HDR-based devices were continued and attention was also placed on the use of BRB devices. The following activities were carried out:

- regarding the HDR devices, simplified models were proposed for design applications. These were based on a parametric study on S-DoF systems equipped with HDR devices;
- regarding the BRB devices, numerical models available in the scientific literature and able to reproduce their dynamic behaviour, are pointed out and simulations were

carried out to model the results of experimental tests performed on a kind of BRB devices (BRAD devices) provided by “Fip industriale”.

Finally, in the last years, design methods were developed for dissipative braced structures. In particular the following activities were carried out

- A DBD procedure was defined for frames with non-moment resisting frames and braced by means of BRBs, which represent the most critical structural typology concerning possible plastic deformation localization at story level;
- A similar DBD procedure was applied for frame with HDR-based dissipative braces;

finally, in both the situations and in the case of regular buildings, the DBD procedures were applied to equivalent continuous models in order to obtain simple and useful analytical expressions of the normalized stiffness distributions, to be used in the preliminary design.

3.12 Degrading hysteresis models and fragility curves of frames with composite semi-continuous joints (RU 12, University of Trieste – coordinator: C. Amadio)

The RU12, involved in the research project, will develop an advanced cyclic model for beam-to-column composite joints. This model is also used to investigate the seismic performance of composite frames coupled also with dissipative bracings. The seismic reliability of a partially-restrained steel-concrete composite frame, designed in accordance with the provisions provided by current codes of practice, will be evaluated.

4 MAIN RESULTS

4.1 Design criteria and response assessment of buckling restrained braces (RU 1, University of Naples Federico II – Engineering Faculty – coordinator: F.M. Mazzolani)

The main results of RU 1 are (i) the development of a novel type of BRB, specifically conceived to be used for seismic protection of existing RC structures, (ii) the experimental validation of the developed device, by means of full-scale tests on a real RC building equipped with the novel BRBs, (iii) the validation of numerical models of the BRBs by comparison with the experimental results. The RC structure equipped with BRBs was previously tested in its original conditions (Della Corte *et al.* 2006). After these tests, the structure was repaired and the above mentioned new BRB system has been fabricated and erected. Figure 5a shows a picture of the structure equipped with the BRBs, while Figure 5b shows the brace inserted within the masonry claddings during construction. Three tests were carried out on three slightly different prototypes, each of which was conceived to improve the response of the one previously tested. Figure 6 shows the last tested prototype.

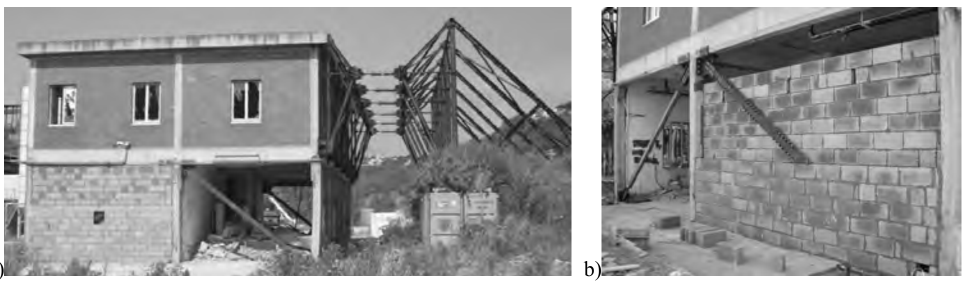
a) b)


Figure 5. The tested RC building equipped with the novel BRBs.

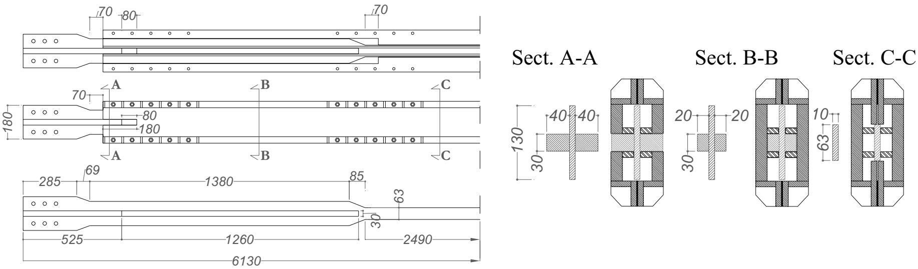
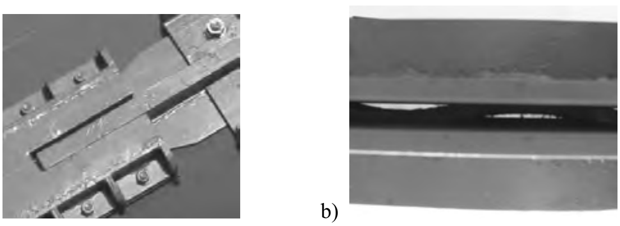


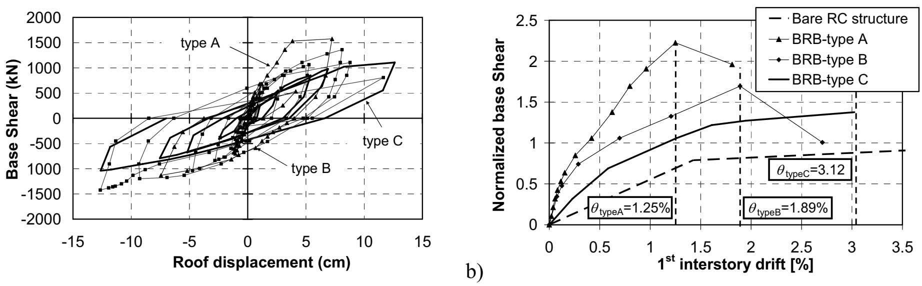
Figure 6. Geometry and local details of BRB Type C.

The overall response of the structure equipped with all the tested BRB prototypes was almost satisfactory, with significant improvements of stiffness, strength and energy dissipation capacity. However, some undesired failure modes occurred during the first test (D'Aniello *et al.* 2009). The second BRB solved those problems, but the best performance was reached by the last device (BRB type C). Figure 7a illustrates the end portion of the tested BRB at the peak compressive deformation, while Figure 7b shows the controlled local buckling of the core. Figure 8a shows the cyclic response of the structure submitted to three subsequent tests corresponding to the three prototypes of BRBs. Figure 8b shows the comparison in terms of positive envelopes of the cyclic response curves. The theoretical response of the bare RC structure is also illustrated in Figure 8b, showing the marked improvement of stiffness, strength and energy dissipation capacity of the structure equipped with the BRBs.



a) b)

Figure 7. Response of the BRB type C at peak compressive deformation.



a) b)

Figure 8. Comparison of response curves.

4.2 *Rotation capacity and classification of steel members (RU 2, University of Naples Federico II – Architecture Faculty – coordinator: R. Landolfo)*

The obtained results confirm the necessity of a classification criterion which takes into account the characteristics of the whole member and not only of cross-section and takes into account simultaneously rotation capacity and overstrength parameters. To evaluate the influence of cyclic loads on rotation capacity and overstrength factors, comparisons between energy dissipated by monotonic and cyclic tests have been performed. Subsequently the objective has been to propose a new expression of the overstrength factor "s" for "I" profiles. For the validation of the obtained expressions of the overstrength factor the percentage errors EP[%] have been plotted for the considered tests. These errors have been calculated by the introduction of the factor MSE (Mean Square Error) to evaluate the differences between the experimental and numerical values. Another objective of the current research was to propose new expressions of the overstrength factor "s" for members with different cross-sections as RHS and SHS. With the same procedures, new expressions of the ductility factor R for I, RHS, SHS profiles are provided (Figure 9).

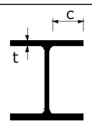
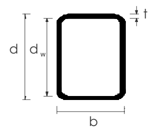
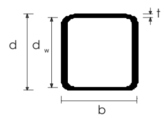
	$\frac{1}{s} = 1,323 + 0,827\lambda_f^2 + 0,03\lambda_w^2 - 0,239\frac{b_f}{L^*}$ $R = 16,8 + 0,877\lambda_f^2 - 0,962\lambda_w^2 + 30,77\frac{b_f}{L^*}$
	$\frac{1}{s} = 2,7 + 0,62\lambda_f^2 + 0,0206\lambda_w^2 - 2,11\frac{b_f}{L^*}$ $R = 9,97 + 0,0015\lambda_f^2 - 0,00136\lambda_w^2 - 0,01032\frac{b_f}{L^*}$
	$\frac{1}{s} = 0,711 + 0,09\lambda_f^2 + 0,318\lambda_w^2 - 0,189\frac{b_f}{L^*}$ $R = -38 + 30,82\lambda_f^2 + 45,4\lambda_w^2 - 0,34\frac{b_f}{L^*}$

Figure 9. Proposed expressions for R and s.

4.3 Design criteria and response assessment of metal shear panels (RU 3, University of Chieti-Pescara – coordinator: G. De Matteis)

First task: Theoretical and Numerical Studies on Slender Shear Panels

Refined FEM analyses put into evidence that when the plate aspect ratio ranges between 0.8 and 2.5 (compact panels), the full development of a stable tension field mechanism occurs. Simplified theoretical and numerical studies have been able to interpret such a behaviour. Contrary, steel plates with aspect ratios lower than 0.8 have a more flexible behaviour. In this case, two improving solutions have been introduced, namely the insertion of either an intermediate beam in the surrounding frame or two coupled fishplates on the panel surface, allowing to achieve the same global behaviour of compact panels (Formisano *et al.* 2007a).

Second task: Experimental and Numerical Studies on Pure Aluminium Shear Panels

Experimental results showed that shear panels realised with AW 5154A aluminium alloy can be considered as stiffening devices, whereas the ones made of AW 1050A aluminium alloy, can be classified as dissipative devices. Concerning the full bay typology, the obtained results showed that tested panels have high energy dissipation capability for medium and large shear deformation levels, while some slipping phenomena were observed for small levels. Aiming at investigating the dissipative behaviour in a larger deformation range, a different panel geometry with reduced side length has been tested. The new panel configuration has been conceived to be employed within a steel bracing system. For the tested bracing type shear

panels, stiffeners have been arranged according to square grids with local slenderness ratios b/t equal to 100, 50, 33 and 25. The obtained experimental results clearly showed that all tested configurations provided a good hysteretic performance (Brando, 2007, De Matteis et al., 2007b, De Matteis et al., 2007c, De Matteis et al., 2007d). The numerical simulation results, undertaken by means of FEM models, were able to interpret the behaviour (Fig. 3) (De Matteis et al. 2007e, De Matteis et al. 2007f, Brando et al. 2007, De Matteis et al. 2007h).
Third task: Numerical-Experimental study on metal (steel and pure aluminium) shear panels for seismic retrofitting of existing RC buildings
A simplified design methodology has been checked in relation to the results of experimental tests. Panels have been used to retrofit one of the module of the existing building tested in Bagnoli (ILVA-IDEM research project). The materials used for panel are DX56D steel and pure aluminium. As an example, the results of the numerical study are here reported with reference to steel panels only. Sample comparisons of results are illustrated in Figure 10. Finally, two experimental pushover tests on the retrofitted structure have been carried out. The test results, have confirmed the effectiveness of the proposed design methodology for both steel and aluminium shear panels (Figures 11 a, b, c) (Formisano, 2007, Formisano et al., 2007b, De Matteis et al, 2007g). Besides, a careful experimental study on the boundary connections between the plate and the surrounding steel frame has been carried out (Formisano et al, 2007c and Formisano et al, 2007d).

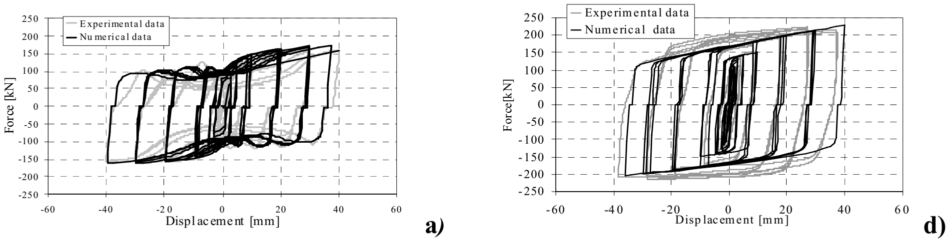


Figure 10. Comparison between numerical and experimental results: a) b/t = 100; d) b/t = 25.

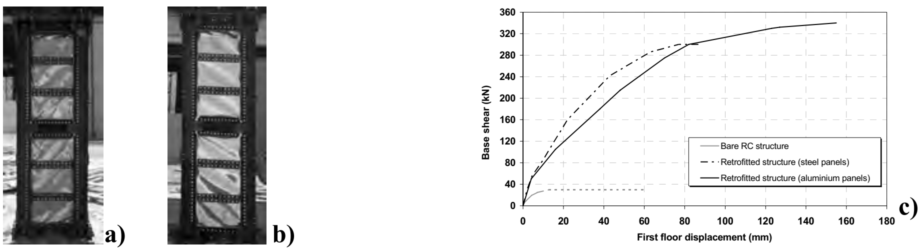


Figure 11. Tested steel (a) and aluminium (b) shear panels and comparison among test results (c).

4.4 Design criteria of steel moment resisting frames and methods of nonlinear analysis (RU 4, Second University of Naples – coordinator A. Mandara)

In order to verify the accuracy of non-linear static procedures a series of pushover analyses have been compared with the predictions of inelastic dynamic analysis, employing a set of 10 input ground motions generated to be consistent to 5%-damped EC8 elastic spectrum for soil class A. Six pushover analyses have been considered: 1) Uniform Distribution (UD); 2) First Mode Distribution (FMD); 3) Equivalent First Mode Distribution (EFMD); 4) SRSS Distribution; 5) Force-based adaptive pushover (FAP); 6) Displacement-based adaptive pushover (DAP). Some results are shown in Figure 12. The results obtained show that the

DAP analysis systematically understimates the interstorey drifts if compared to dynamic analysis, while the FMD distribution seems to be more effective. All the nonlinear static procedures understimate the interstorey drifts of the 7-storey frame designed with the Italian Code. This result derives from the high strength required to the roof columns by the capacity design criteria.
The assessment of current nonlinear static procedures (NSPs) has also been carried out by comparison against dynamic analysis. At this aim, six ground motions are selected to be consistent to EC8 8 elastic spectra for different soil classes. Sample results are shown in Figure 13. FEMA 356 and FEMA 440 give very similar performance points. This result derives from the value of the coefficient C_3 that is approximately equal to 1.0 in both cases. The procedures based on the High Damping Elastic Demand Response Spectra (HDERS) such as ATC 40 and FEMA 440 (ATC 55) give higher PGA when compared to the other NSPs and to NRHA. Furthermore, the results are more sensitive to the lateral force distribution. This result derives from the use of HDERS that give poor estimation of Inelastic Demand Response Spectra (IDRS). On the contrary, the superior physical basis of Inelastic Demand Response Spectra (CSM-N2) proved to give more accurate estimation of the seismic performance.

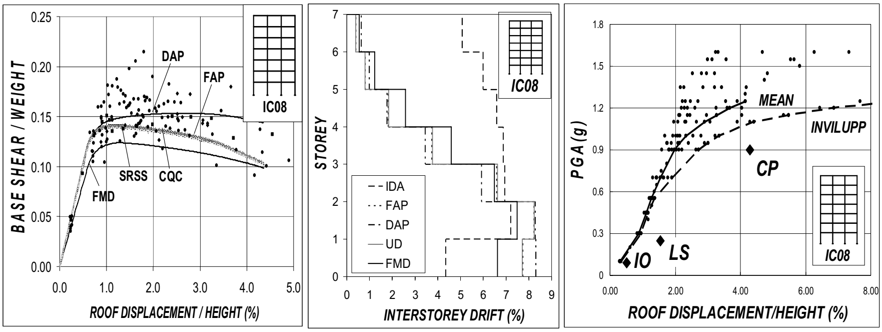


Figure 12. Base shear versus roof displacement – Pushover and Incremental Dynamic Analysis.

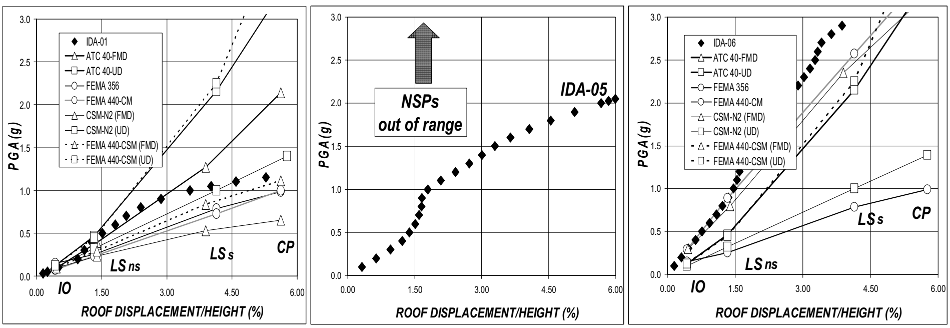


Figure 13. Peak ground acceleration versus roof displacement – NSP and IDA (PD-SLS design).

4.5 *Seismic response and design rules of steel-concrete composite bridges (RU 5, University of Salerno – coordinator: V. Piluso)*

The main results of the research project are constituted by numerical models devoted to the evaluation of the moment-curvature relationship and of the force-displacement behaviour of

CFT bridge piers starting from the knowledge of their geometrical properties, the materials constitutive laws and the boundary conditions.
In addition, the results provided by the experimental program constitute a useful data-base concerning the monotonic and cyclic response of steel-concrete composite bridge piers.
The developed models allow the evaluation of the ultimate behaviour of composite bridge piers providing the designer with the possibility of evaluating the behaviour factor of composite bridge structures. In fact, the seismic response of beam bridges is mainly dependent on the cyclic response of piers whose base sections constitute the dissipative zones. This matter can be a further development constituting an useful improvement of seismic codes with reference to composite bridge structures.

4.6 Design criteria and response assessment of steel bridges (RU 6, University of Pisa – coordinator: W. Salvatore)

Three case studies representing the most diffused typologies of composite steel-concrete bridges were identified. The first case study was a composite double girder bridge; the second was a railway viaduct consisting of composite parallel girder decks (viaduct Elvo); the third case study was constituted by a double box composite steel-concrete railway bridge (viaduct Sesia). Resistant schemes and seismic critical details of structures were identified.
Analysis of the first case study pointed out the differences of numerical dynamic and seismic responses obtained with different modeling approaches. In particular, modeling techniques had a big influence on the evaluation of seismic forces acting on critical details.
Study on viaduct Elvo highlighted the big capacities of experimental dynamic analysis that, coupled with updating procedures, permits to obtain reliable numerical models for the dynamic analysis and for the evaluation of seismic forces acting on critical details.
An extended study on numerical analysis techniques of viaduct Sesia was realized, developing finite element models of deck only and of deck-pile-foundations set. Analytical models of foundation dynamic behavior were also analyzed and used. Dynamic responses of numerical models were compared with experimental data. Deck only models were not able to describe properly the transverse dynamic behavior. The use of well-known geotechnical analytical models permitted to evaluate properly the dynamic behavior of pile foundations. It was so possible to develop finite element models of deck-pile-foundations set that showed to be more effective. The evaluation of seismic forces acting on critical details was strongly dependent on the adopted modeling technique.
Results of the study could be used as a start point for the development of recommendations for numerical structural analysis of bridges, as they focused the importance of representing piers and foundations dynamic behavior into finite element models. They also highlighted the utility of experimental analysis coupled with updating processes.

4.7 Procedures to assess low-cycle fatigue of steel members and connections (RU 7, Polytechnic of Milan – coordinator: C. A. Castiglioni)

Application of the innovative cyclic testing procedure led to a displacement history that could not be predicted a-priori as a consequence of the force-controlled part of the cycles. Loading histories applied on different tests (with different values of Fg) are consequently different as depicted in Figure 14. In test F130Fy25-1 the specimen was subjected to elastic cycles, after which it endured two cycles of +/- 2 and 4 times dy (dy - yield displacement) and failure occurred in the first positive 6 dy displacement cycle. In test F130Fy75-1 the specimen failed in the first positive cycle with 4 dy amplitude. Either of the two test results (as all the remaining tests conducted with the innovative cyclic testing procedure) are fundamentally different from those obtained through the application of the ECCS recommended testing procedure. Amongst others, the following differences could be identified:

- Imposed displacement history is unsymmetrical with the innovative testing procedure.
- Imposed forces are shifted in the positive (hogging) direction for the innovative testing procedure.
- The innovative testing procedure captures the detrimental effects of the gravitational loads which result in the decrease of available ductility leading to premature collapse.
- Failure of the connection is explicitly addressed by the innovative testing procedure since failure occurs when the connection is no longer able to withstand vertical loads.

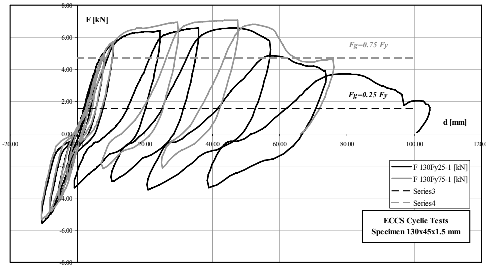


Figure 14. Innovative cyclic testing procedure results for tests F130Fy25-1 and F130Fy75-1.

4.8 Design-by-testing for buildings and bridges (RU 8, University of Trento – coordinators: R. Zandonini and O. Bursi)

A reference composite steel-concrete building was considered, made up of moment resisting frames in the longitudinal direction and braced frames in the transverse direction. Two different slab systems were analyzed: i) a steel-concrete slab with structural profiled ribbed steel sheeting; ii) a concrete slab composed of electro-welded lattice girders. The frames were designed to have a dissipative structural behavior according to EC 8 (UNI EN 1998-1, 2005). Numerical simulations on 2D frames were first performed by means of the SAFIR program (Franssen J.-M. 2000), in order to study different fire scenarios and to evaluate the performance of different elements under fire load. The joint design aimed at ensuring overstrength with respect to the beam. The proposed bolted solution derives from a parent welded design (Bursi et. al 2008) and was conceived to guarantee easiness of assembly and to avoid problems related to welding on site. Beam-to-column joint design was developed in agreement with EC 3 and EC 8 (UNI EN 1993-1-8, 2005, UNI EN 1998-1, 2005). Stiffness and strength of complex components, like top and bottom plates or concrete slab in compression, were defined by means of refined Finite Element (FE) models. The results show that, in order to activate the transfer mechanisms proposed in the Eurocode 8 (UNI EN 1998-1, 2005) it is necessary to increase the level of friction between the concrete slab and the composite column. Moreover, beam-to-column joints subjected to fire load were analyzed. Three different time-temperature curves were implemented into ABAQUS 6.4.1 (Hibbitt et al., 2000) and a 3D FE model of the joint was studied. On this basis, it was possible to characterize the reduction of the moment capacity as a function of the time of fire exposure.

The experimental programme consisted of 4 tests under cyclic loading of full-scale substructures representing interior full-strength bolted beam-to-column joints. Experimental tests were carried out at the Laboratory for Materials and Structures of the University of Trento. Joint specimens were subjected to cyclic loading up to collapse. The observed response clearly indicated that all specimens exhibit a good performance in terms of resistance, stiffness and energy dissipation. The fire tests have showed that: i) there was no noticeable difference in the fire performance between pre-damaged and undamaged specimens both with precast and steel sheeting slabs; ii) precast slabs performed better also in fire tests than the corresponding specimens with steel sheeting; iii) all specimens inherited favourable seismic properties by performing in a ductile manner also under fire loading.

Moreover, the activity of RU 8 was also focused on the analysis of a steel-concrete composite bridge: the Montevideo viaduct. The identification of its critical details was performed by means of 3D FE models subjected to strong earthquakes. The analysis showed that the bearing zone and the end-diaphragm were the most stressed parts. An optimisation of the bolted end-diaphragm connections was then proposed, with a welded-bolted solution that guarantees structural and assembly effectiveness. Since the bridge is affected by an advanced corrosion process, output-only dynamic identification tests were carried out. The modal extraction performed by applying the Frequency Domain Decomposition Technique denoted that the bridge tends to be more flexible than the FE model predictions. Moreover, in order to study the behaviour of reinforcing bars in compression during the construction stage of concrete decks, tests on lattice girders of high capacity self-supporting slab were carried out at the Structural Testing Laboratory of the University of Trento. Experimental results were utilized as reference data for the optimization of numerical models based on the FE method. As a result, an excellent numerical estimation of both deformability and critical buckling load of simply supported trussed beams was obtained, whilst the accuracy of the numerical prediction for trussed plates with cantilever ends was of lower quality, entailing further experimental investigation. The validation of the FE model of the bridge, by means of a model updating procedure, allowed the development of a reliability analysis. The computation of the β reliability index on the updated model, with respect to the bending moment at midspan, showed that the bridge still exhibits a satisfactory margin of safety, though the structural behaviour is highly influenced by an advanced corrosion of the steel box, that led to a retrofit design. In this respect, one retrofit solution with parabolic and one with rectilinear cables were investigated. The selected solution consists of a group of straight cables lying on the bottom of the steel box. This positioning allows a gradual increase of the post-tensioning force from the end to the midspan, guarantees lighter and less complex restraint structures and allows the standardization of the single parts.

4.9 Rotation capacity and classification of composite elements (RU 9, University of Sannio – coordinator: M. R. Pecce)

The transfer mechanisms (bond) between steel and concrete in partially encased columns

The experimental tests on partially encased columns evidenced that this type of element bases the bond mechanism especially on adhesion and friction; therefore the bond strength is the stress value of the post-peak branch of bond stress-slip law, but not the peak value that does not occur in all the tests or after which a steep branch takes place.

The analysis of the experimental results point out the code limitations are not quite safe. The bond-slip relation of Figure 15 show the code threshold of 0,2MPa is overcome for about all the specimens except the ones with oil at the interface, but the safety margin appears too small for a design value that needs of a characteristic meaning and a safety factor. A heavy degradation of strength and stiffness resulted from the cyclic tests.

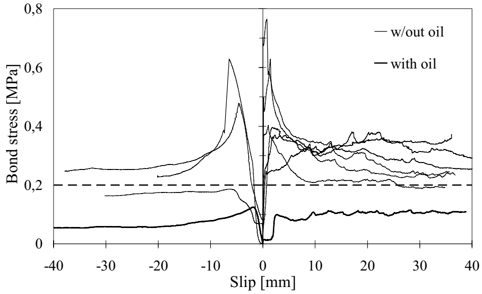


Figure 15. Experimental bond-slip curves.

Model of connection at base column

The finite element model shows a good agreement with the experimental test; in Figure 16 the comparison with the experimental test on a column with the socket type connection at the base is reported in terms of force-drift curve. Furthermore the model allows analysing the stress distribution in the foundation block; the extension of the transfer zone in the block increases enhancing the post-elastic displacement.

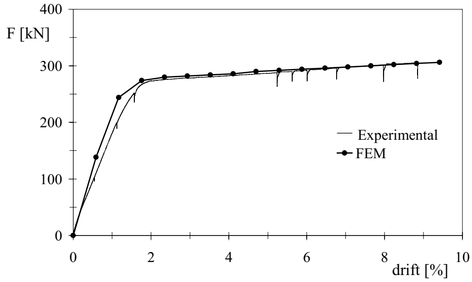


Figure 16. Comparison between numerical model and experimental results.

Rotational capacity of composite beams under hogging bending moment

The experimental tests gave results disagreeing from the predictions of the project. The beams were designed with the steel profile of class 1 or 2 to have great plastic rotations, but local buckling occurred limiting the plastic rotation of all the 4 beams. In Figure 17a local buckling is shown for one beam and in Figure 17b the force-displacement curve is depicted for all the specimens (beam 1 – non uniform distribution of studs, beam 2 – uniform distribution of studs, beam 3 – slab with profiled sheeting b=100cm, beam 4 - slab with profiled sheeting, b=160cm).

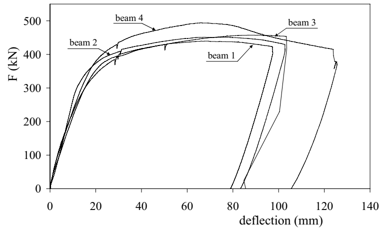
a)  b) 

Figure 17. Local buckling of one beam (a) and force-displacement response the 4 beams.

4.10 Methods for structural analysis of composite steel-concrete structures (RU 10, University of Molise – coordinator: G. Fabbrocino)

One first result of the activity carried out for the project consist of an extended state of the art focused primarily on modelling and response of composite flexural members. The attention has been in a first phase focused on basic experimental and theoretical works on the subject in order to perform a systematic review of all available data on response of connections, beams and columns. But, the main result is represented by a total of n.6 tests, which have been conducted in cooperation with the University of Sannio. Three diameters of stud connectors were examined Φ 16, Φ 19 and Φ 22. Two specimens were built for each stud diameter, the first one having the slab subjected to tensile stresses, the second having unloaded slabs. In the first case, the shear connector acted on the slab via an eccentric load. Consequently, a rotation effect of the slab was showed, which was related to the restraint of the reaction system on the slab. The unloaded condition of the slabs was obtained by means of rigid steel plates at the top of the slabs. In such a way the slabs directly acted onto the test frame.

4.11 Composite frames with dissipative braces (RU 11, University of Ancona – coordinator: L. Dezi)

It was found that the rubber behavior is quite complex because it is strongly nonlinear and both stiffness and damping properties vary with the amplitude of strain and depend on the strain rate. Furthermore, the presence of filler added to the natural rubber, makes the response of the HDR strain history-dependent and causes a transient behavior in which stiffness and damping change remarkably. The phenomenon, usually known as “Mullins effect” or “scragging”, is a consequence of the damage of the microstructure, that occurs during the process. The initial properties of the material may be however recovered (Dall’Asta and Ragni, 2006, Dall’Asta *et al.* 2006a, Dall’Asta and Ragni, 2008a). In the first year of the research project, an analytical model able to describe all these phenomena was proposed. This model was used to simulate experimental tests (Dall’Asta et al. 2006b, Dall’Asta et al. 2008b, Ragni et al 2009). The agreement between experimental and numerical data confirmed the ability of the constitutive model to predict the dynamic response. Despite its efficiency, the proposed method is quite complex and cannot be used in current practice. For this reason, in the second year , simplified linear models were defined. In particular two linear models were developed, one equivalent to the stable behavior and the other one equivalent to the transient response. Results of linear and nonlinear time history analyses carried out on SDoF systems showed that the linear model referring to the stationary behavior shows a tendency to overestimate the displacements and underestimate the forces, whereas the linear model referring to the transient behavior has the opposite tendency (Dall’Asta and Ragni 2007, Dall’Asta and Ragni, 2008b). Therefore, it may be concluded that using linear models calibrated on stationary cycles (PrEN 1998-1, NTC 2008) gives good results in terms of maximum displacements. But linear models based on the transient response should be used to evaluate extreme force values.

Regarding BRBs, experimental tests carried out on the devices provided by “Fip Industriale” showed that the device behavior is not symmetric. The devices showed large dissipation capacity, but some fatigue problems were observed. With reference to numerical simulations, classical elasto-plastic models were used to simulate the device response.

In the last year of the research project design methods were developed for dissipative structures with BRBs and HDR-based braces. The procedures were applied to several study cases and non linear time histories were carried out (Ragni et al. 2008, Dall’Asta et al.2008a). The main results obtained for dissipative structure with BRBs are reported below:

- a design story drift varying between 1.5% and 2.5% must be assumed in the design procedure to avoid obtaining excessively flexible structures;
- the proposed design procedure avoids localization of plastic deformations;
- since the design method is based on the elastic behavior of the structure, the maximum ductility values at different story levels are not constant and an adequate safety factor must be assumed;
- story drifts at the damage limit state must also be checked;
- in the case of regular floor masses, results obtained with MDOF systems were very close to those obtained with continuous equivalent systems.

Regarding dissipative structures with HDR-based braces, the following main results were obtained:

- the proposed design procedure leads to a regular structure behaviour with story displacements and device forces close to the design values;
- HDR-based devices are less dissipative than BRBs, and consequently larger columns and braces areas are obtained for the bracing systems;
- the frames braced by means of HDR-based braces offer the important advantage of being not sensitive to weak-story mechanisms and thus being a more robust solution;
- moreover, story drifts at ULS and DLS are much smaller than the story drifts obtained with BRBs and no permanent deformations remain after the seismic event;
- in the case of regular floor masses, results obtained with MDOF systems were very close to those obtained with continuous equivalent systems.

4.12 Degrading hysteresis models and fragility curves of frames with composite semi-continuous joints (RU 12, University of Trieste – coordinator: C. Amadio)

The most relevant results achieved in the research project can be summarized as followed:

- A critical analysis of previous beam-to-column joint numerical models was carried out. Some limits/shortcomings in the modeling have been pointed out.
- New cyclic mechanical laws for the main resistant components of the joints (the web panel, the T-stub, the concrete slab and its interaction with the column) were developed.
- An advanced mechanical model for the rubber of dissipative visco-elastic devices was implemented into the ABAQUS and ADAPTIC codes and numerical analyses of semi-continuous steel frames coupled with dissipative bracings were carried out.
- Appropriate design criteria for frames with dissipative bracings were defined.
- An automatic procedure to calculate the fragility curves of a semi-continuous steel-concrete composite frame was defined.
- The strength probability distributions of both the materials and the whole joints, as well as of the composite beams, were determined.
- A full probabilistic analysis was carried out and fragility and performance curves for two different composite frames were determined.

Joint modelling

As far as the joint model is concerned, a complete cyclic modelling procedure for generic beam-to-column connections in steel-concrete composite frames has been developed using the component method (Figure 18).

Moreover, the strength and stiffness degradation, that affect the behaviour of the main resistant components by increasing the number of cycles, have been accurately modelled using specific rules, based on the outcomes of experimental data.

In particular, four different models have been employed to schematize the hysteretic behaviour of the main components:

- A tri-linear hysteretic model with kinematic hardening was used for the beam web and flange in compression and in tension, for bolts in shear and for longitudinal bars in tension;
- a specific hysteretic model was used for T-stub elements (column flange, end flange and angles in bending and bolts in tension);
- a hysteretic "Pivot" model was used for concrete struts in compression, for transversal rebars in tension and for beam-to-slab connection in shear (Figure 19). The classical "Pivot" model has been modified to take into account the degradation in strength.

the hysteretic model developed by Kim & Engelhardt (2002) was used for column web panel in shear.

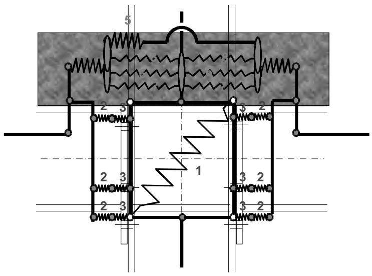


Figure 18. Model M2 for the composite joint.

The accuracy of the developed macro-model has been validated by means of many comparisons against experimental outcomes of joint tests (Caramelli *et al.* 1999, Liew *et al.* 2004) and based on the experimental response of a composite frame tested at Ispra.

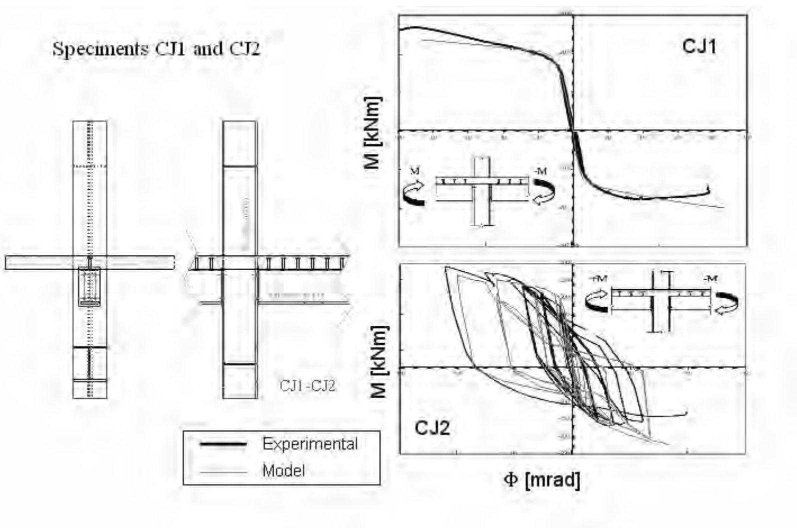


Figure 19. Numerical and experimental comparisons for Liew et al. tests (2004).

Frame analysis
Three types of steel frames have been analysed: bare frame, frame coupled with concentric steel bracings and frame coupled with visco-elastic dissipative braces. Moreover, design criteria have been developed for steel frames coupled with visco-elastic dissipative bracings. A number of nonlinear dynamics analyses was carried out under artificial spectrum-compatible earthquake ground motions.

Probabilistic analysis
Probabilistic analyses were carried out to investigate the seismic reliability of PR composite frames, using the advanced numerical models developed for the composite joints. Statistical distributions for the strength of all the composite frame components were defined and the probability distribution curves for the joints and composite beams' strength have been determined, by employing an optimized Monte Carlo method. The analysed structures corresponds to the frame tested at the European Laboratory for Structural Assessment (ELSA) of the Joint Research Center (JRC) of Ispra and to a second frame that is different only by the number of storeys (four floors). The structures have been analysed under two different sets of recorded earthquake ground motions. Samples of the fragility curves in terms of spectral acceleration and the collapse probability curve of the first frame are shown in Figures 20 and 21.

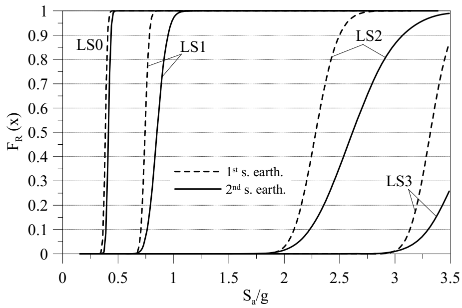


Figure 20. Fragility curves (ISDA) in terms of Sa.

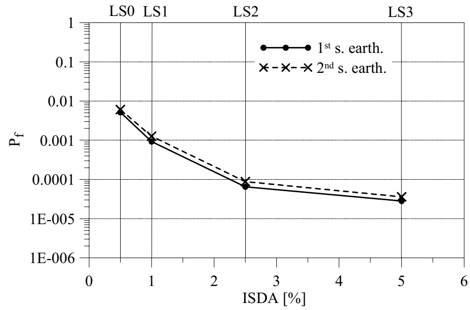


Figure 21. Performance curves of the first frame.

5 DISCUSSION

5.1 Design criteria and response assessment of buckling restrained braces (RU 1, University of Naples Federico II – Engineering Faculty – coordinator: F.M. Mazzolani)

Both experimental tests and numerical analyses of full-scale BRBs have been carried out within the three years of the research project. The tested BRBs are a novel type, which is all-steel. Though it has specifically been conceived for the seismic upgrading of existing reinforced concrete buildings, it can obviously generally used in many situations where a dissipative device with large energy absorption capacity is needed. The experimental testes and the numerical analysis permitted to assess the validity of the design rules. Further testing could be required in the laboratory, where different cyclic loading histories could be applied, looking at a more extensive analysis of the low-cycle fatigue behavior of the novel device.

5.2 *Rotation capacity and classification of steel members (RU 2, University of Naples Federico II – Architecture Faculty – coordinator: R. Landolfo)*

The results of 12 tests performed on steel members, with different cross sections, subjected to non-uniform bending have been presented. Both monotonic and cyclic tests have been performed (Brescia, 2009). Tests have confirmed the role of the local slenderness parameters of member section. After the occurrence of local buckling, a strength degradation phenomenon is exhibited, whose amount is related to flange and web slenderness. For all the specimens, the evaluation of energy absorbed during the monotonic test (E_1) and of the energy dissipated during the cyclic test (E_2) has been performed (Table 1).

Table 1. Comparison between adsorbed and dissipated energies.

	E1 (kNmm)	E2 (kNmm)	E1/E2
HEA 160	25676	298265	0.09
HEB 240	136030	563776	0.24
150x100x5	17223	95053	0.18
160x80x4	5709	35675	0.16
200x200x10	83948	455212	0.18
160x160x6.3	20682	95752	0.22
	average value (%)		17.82

As visible, the main result of the comparison between the energy absorbed under monotonic conditions and the energy dissipated during cyclic conditions is that the average value of the energy absorbed during monotonic test is about 18% of the corresponding energy dissipated under cyclic actions. Therefore, the assumption that collapse occurs when the cumulated plastic ductility in one direction, positive or negative, is greater than the one available under monotonic loading conditions, is a safe side assumption. In Figure 22, the dissipated energy of tested members under cyclic actions is depicted as a function of the number of cycles.

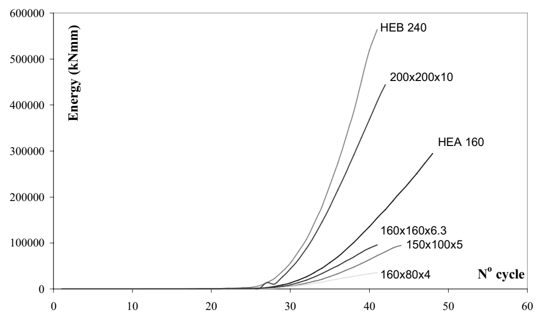


Figure 22. Energy dissipation versus number of cycles.

5.3 *Design criteria and response assessment of metal shear panels (RU 3, University of Chieti-Pescara – coordinator: G. De Matteis)*

The targets of the first task have been completely achieved, since practical information on the use of slender MSPs have been provided. Concerning the second task, the target results have been achieved and have been also used for the design purpose. In the third task the field of use of examined devices has been specified and the knowledge of their main properties has been done.

5.4 Design criteria of steel moment resisting frames and methods of nonlinear analysis (RU 4, Second University of Naples – coordinator A. Mandara)

The activities and the results are in good agreement with the tasks previewed for the local unit and with the time planning initially predisposed for the optimal organization of the research project. The information generated through nonlinear analysis will form the basis for a displacement-based design procedure of steel moment resisting frame structures.

5.5 Seismic response and design rules of steel-concrete composite bridges (RU 5, University of Salerno – coordinator: V. Piluso)

The purposes of the research activity are nowadays substantially achieved. In fact, the main objective was constituted by the prevision of the ultimate behaviour of steel-concrete composite bridge piers, which has been investigated by means of both numerical and experimental procedures.

Nevertheless, the discussed results have been obtained with reference to square and rectangular hollow steel profiles filled with concrete, while in the initial intentions also the case of circular hollow sections was foreseen. This further goal has not been achieved, because the originally planned testing activity was probably too much ambitious and, in addition, some troubles in the early phases of the research project occurred due to the need of calibration of the new testing devices, being some of them used for the first time within this project.

5.6 Design criteria and response assessment of steel bridges (RU 6, University of Pisa – coordinator: W. Salvatore)

All the activities carried out are fully included in the framework of the research program RELUIS for the Line 5.

In particular during the research execution, it was clear the need of a wide investigation on composite bridge modeling techniques, because of the differences between dynamic and seismic numerical results obtained from different structural analysis schemes.

UR5 then realized the optimization of design rules focusing its attention on the development of numerical models capable to represent properly the dynamic and seismic behavior of case studies, also adopting as reference the experimental dynamic data coupled with updating procedures. Finally guide lines for bridge dynamic and seismic analysis using finite element approach were defined.

5.7 Procedures to assess low-cycle fatigue of steel members and connections (RU 7, Polytechnic of Milan – coordinator: C. A. Castiglioni)

All the activities carried out are fully included in the framework of the research program RELUIS for the Line 5. The innovative testing procedure developed within the research could be usefully implemented to assess the effect of large gravity loading.

5.8 Design-by-testing for buildings and bridges (RU 8, University of Trento – coordinators: R. Zandonini and O. Bursi)

The actual work performed in the research project is in accordance with the plan of the activities reported in the project proposal. Only the substructuring tests on the critical steel details of the bridge were not performed, owing to the complexity of the testing technique, whose basic issues have been analysed and clarified. However, with respect to steel-concrete composite bridges we tested lattice girders of high capacity self-supporting slab in order to study the behaviour of reinforcing bars in compression during the construction stage of concrete decks.

5.9 Rotation capacity and classification of composite elements (RU 9, University of Sannio – coordinator: M. R. Pecce)

The transfer mechanisms (bond) between steel and concrete in partially encased columns
For this subject the experimental tests gave satisfactory results and the prefixed aim was reached; the bond mechanism of partially encased columns was defined and the bond strength was evaluated.
In conclusion the experimental results suggest to reduce the code limitations and to introduce provisions about the interface conditions at the casting to guarantee bond. In case of seismic constructions the bond contribution would be neglected and adequate connection devices have to be used for enduring cyclic loads, as suggested by Eurocode 8 (2004)

Model of connection at base column
The activities about this topic following the program till the development of the model for the partially encased column with the innovative socket connection at the base. However many problems were encountered to reach a good agreement with the experimental results and the parametric analysis has been only started.

Rotational capacity of composite beams under hogging bending moment
About this subject the research program was followed but the experimental results were different from the prediction, therefore the topic was not completely reached; the analysis of the experimental results was complex and information about rotational capacity were lack since it was necessary to study local buckling and steel concrete connection.
Anyway the experimental results and numerical analysis give interesting direction to review the classification of steel section in codes, especially taking into account the difference between nominal and actual values of the steel yielding stress in order to avoid a premature local buckling.

5.10 Methods for structural analysis of composite steel-concrete structures (RU 10, University of Molise – coordinator: G. Fabbrocino)

The fixed research objectives were almost completely reached. A complete state of the art was elaborated and numerical models of composite beams under hogging bending moments implemented. Difficulties were encounters in capturing complex distortional buckling modes observed during the tests described by RU 9 and further studies should be carried out and addressed to the inclusion of those modes of failure.

5.11 Composite frames with dissipative braces (RU 11, University of Ancona – coordinator: L. Dezi)

During the three years of the research project, almost all the initial objectives have been achieved, even if many topics are still open, above all those regarding the development and the application of the DBD procedures to dissipative braces.

5.12 Degrading hysteresis models and fragility curves of frames with composite semi-continuous joints (RU 12, University of Trieste – coordinator: C. Amadio)

The research was conducted substantially in compliance with the timing and methods specified in the project proposal. The applicative parts of the results achieved require further research.

6 VISIONS AND DEVELOPMENTS

6.1 Design criteria and response assessment of buckling restrained braces (RU 1, University of Naples Federico II – Engineering Faculty – coordinator: F.M. Mazzolani)

In the current research an innovative "all-steel" BRB has been studied. This work showed that the performance of the "all-steel" BRB prototype depends on some local details. The experimental activity highlighted which are the parts and the geometrical proportions to be modified in order to improve the "robustness" of the devices. The excellent experimental performance of the last prototype confirmed the effectiveness of the chosen technological and geometrical adjustments. However, further experimental investigation should be required to evaluate the cumulative ductility capacity provided by these "all-steel" devices. Indeed, starting from these tested prototypes, a set of specimens should be experimentally investigated in laboratory tests. These specimens may be useful source of information to correctly design the inner clearance giving an understanding on how the clearance width can influence the number of higher-modes in the yielding core.

6.2 Rotation capacity and classification of steel members (RU 2, University of Naples Federico II – Architecture Faculty – coordinator: R. Landolfo)

The problem of evaluating rotation capacity of steel beams and beam-columns can be analytically faced with reference to monotonic loading conditions, providing the limit values of the width-to-thickness ratios of plate elements constituting the member section which assure given values of plastic deformation capacity.

Notwithstanding, the study of the ultimate behaviour of steel members subjected to non-uniform bending, with or without axial forces, is not exhaustive, because members are cyclically loaded during seismic motions.

The knowledge of the monotonic behaviour can be sufficient to estimate the collapse condition, provided that the criterion of the maximum plastic deformation can be accepted. According to this criterion, collapse is attained when the plastic rotation demand is greater than the plastic rotation supply evaluated under monotonic loading conditions. This assumption represents a good approximation provided that members are subjected to a deformation history characterized by only one cycle with a big plastic engagement and many cycles having small plastic amplitude. In the opposite case of deformation histories characterized by many significant plastic excursions, two needs arise. On one hand, a sufficiently accurate modelling of the cyclic behaviour is needed and, on the other hand, the occurrence of failure should be characterized account for all the plastic excursions.

It is universally recognized that the most complete interpretation of collapse under cyclic actions can be obtained by means of the low cycle fatigue approach. However, the application of low-cycle fatigue approach is cumbersome from the computational point of view and, in addition, it would require the knowledge of fatigue curves for all the possible structural details, especially when the cyclic response is also affected by the beam-to-column connection typology.

Therefore, the need of simple yet accurate rules to account for the influence of the cyclic loading history on the plastic rotation supply of members still constitute a gap of knowledge. The experimental study of the plastic rotation supply of members is commonly carried out with reference to simple schemes such as the cantilever scheme or the three point bending scheme, because the results obtained for such schemes can be easily extended to the behaviour of steel members in actual frames considering an equivalent cantilever which is identified as the member part between the point of zero moment and the section where the formation of the plastic hinge is expected.

6.3 Design criteria and response assessment of metal shear panels (RU 3, University of Chieti-Pescara – coordinator: G. De Matteis)

The results obtained in the framing of the activity related to the development of theoretical and numerical (both simplified and refined) studies on slender steel shear panels, have been duly completes. However they will be used, as will be specified later on for the development of the third task. As far as the object of the activity on Pure Aluminium Shear Panel is concerned, the obtained results are the basis for the implementation of sophisticated non linear dynamic analysis on steel and RC buildings equipped with this type of system. These will be carried out in order to evaluate the effects on the primary structural elements in terms of displacement and internal force under a maximum credible earthquake. On this purpose, design criteria will be developed also on the basis of some instability curves that have been already provided in the framing of the numerical activity which has been broadly described before. The final goal of the above proposed dynamic analysis will be the extraction of synthetic parameters, such has the q-factor, which may be proposed for integration of existing codes. Moreover, some further experimental and numerical analyses will be carried out with the aim of proposing new devices based on the aluminium shear panels technology. The information acquired from the research activity related to the development of numerical-experimental study on metal shear panels for seismic retrofitting of existing RC buildings will be used for performing the next study phases, which will consist on the execution of time history analysis on RC structures equipped with MSPs in order to evaluate their real effective dissipation capacity under earthquakes and to improvement the knowledge about the prymari structure-panels interaction. In addition, on the basis of the results achieved from the first tasks, further studies will be carried out on MSPs in order to provide useful recommendations on their geometrical dimensions.

6.4 Design criteria of steel moment resisting frames and methods of nonlinear analysis (RU 4, Second University of Naples – coordinator A. Mandara)

The future goal of the research project is the development of a displacement-based seismic design method for steel moment resisting frame structures based on the inelastic deformation capacity of the structure and the displacement spectrum of the earthquake ground motion. The objective is to propose a design procedure that balance stiffness, strength, and ductility in order to have the desired seismic performance at each intensity level of the earthquake ground motion. Such objectives cannot be pursued through the comparison between local capacity and seismic demand. It is, rather necessary a multi-level and multi-objective design procedure based on the estimation of the global behaviour of the structure in terms of lateral displacement. The procedure is based on the control of the structural performance by the target roof displacement defined from the displacement design response spectrum. This performance parameter is then transformed in structural and non-structural displacement demand. In particular, the interstorey drift ratio (non-structural damage) and local ductility (structural damage) give the final design requirements for the steel members.

6.5 Seismic response and design rules of steel-concrete composite bridges (RU 5, University of Salerno – coordinator: V. Piluso)

Within the topic of steel-concrete composite bridge piers, the natural development of the research is constituted by the execution of experimental tests dealing with circular hollow sections. In this way, the most used structural shapes, with reference to CFT members, will be completely investigated.

In addition, since the experimental activity has been focused on the case of members subjected to bending moment only, the case of CFT members subjected to a combination of axial force and bending could be investigated. This matter can be significant in some cases.

6.6 Design criteria and response assessment of steel bridges (RU 6, University of Pisa – coordinator: W. Salvatore)

UR5 results highlighted the influence of representing in the same numerical model the dynamic behavior of decks, piers and foundations on the evaluation of dynamic and seismic response of bridges.

Next developments will examine the representation into numerical models of abutments dynamic behavior together with the dynamic characterization of soils, foundations, materials (i.e. ballast), bearings and isolation systems. Main target of new research will be the improvement of guide lines up to the definition of new proposal for Italian design code.

6.7 Procedures to assess low-cycle fatigue of steel members and connections (RU 7, Polytechnic of Milan – coordinator: C. A. Castiglioni)

The development of reliable cyclic testing procedures is a permanent hot topic in the field of seismic engineering, because the assessment of the deformation capacity of structures is always strongly based and dependent on the experimental verification. Recent numerical and experimental investigations have clearly shown that the measured deformation capacity of some structural components may be sensitive to the type of loading history. On the other hand, the definition of reliable cyclic loading histories

6.8 Design-by-testing for buildings and bridges (RU 8, University of Trento – coordinators: R. Zandonini and O. Bursi)

The studies regarding both bolted and welded joints in steel buildings allowed the best joint performance to be obtained by using concrete-filled-tubular columns with rigid joints (Bursi et al., 2008). In fact, plastic hinges could form in steel-concrete composite beams. Nonetheless, due to the over-design approach used for seismic and fire load situations, a substantial improvement of this structural solution could be obtained by means of the employment of High Strength Steel (HSS). In fact, HSS can increase the performance of steel-concrete composite structures and reduce both weight and construction costs. Currently at the industry level, there is the tendency to promote the use of high mechanical performance steels, but unfortunately, many limitations still affect the use of such products in design practice, owing to the limit imposed by stability and fatigue analysis. Nonetheless, the recent Eurocode 3 Part 1-12, which deals with HSS allows the use of this steel also for buildings.

Steel-concrete composite structures represent a design option that is also being increasingly adopted: i) in road networks; ii) in areas prone to high-intensity seismic events. The success of this design solution is due to the advantages that composite elements offer in terms of stiffness, resistance and ductility. However, there are topics that require more in depth investigations. For instance, the shear connection system between the steel beam and the concrete slab represents a fundamental aspect which influences sensitively the efficiency of

the composite element, expressed in terms of strength and stiffness. In detail, recent studies were carried out on different shear connection typologies. Among others, Perfobond rib connectors showed a greater static and fatigue strength in relationship to headed stud connectors (Marecek et al., 2005). The study of the combination of traditional (studs) or innovative (Perfobond) shear connectors with high performance concrete would assume particular interest for bridge structures located in seismic prone zones.
Finally and with respect the existent bridges, the use of high performance materials and different retrofitting techniques could be employed in order to achieve design aims in terms of strength for high and low-cycle fatigue, of ductility for seismic loading and of durability.

6.9 Rotation capacity and classification of composite elements (RU 9, University of Sannio – coordinator: M. R. Pecce)

The identification of rotational capacity is already an open and topic problem for composite members. Formulations are not available for evaluating the plastic rotation of beams under hogging bending moment and column connection at the base, depending on the connection system. Further experimental tests and numerical modelling need together with numerical models able to do parametric analysis for many cases.
In particular the tests on the beams under hogging moment underlined the importance of various parameters in determining the classification of the steel section to avoid premature local buckling. A model for picking the interaction of various phenomena has to be arisen as slab cracking and beam buckling, but additional tests have to carry out in case considering the effect of beam-column joint, in order to focus a design formulation that is necessary for non linear analysis of composite structures.
Analogously, the improvement of the model for the base connection is an important tool which could be extended to other types of column (concrete filled) and connections. Parametric analysis by the model could allow defining the role of many mechanical and geometrical properties. Finally design provisions and formulation could be introduced in code to design the innovative socket connection.

6.10 Methods for structural analysis of composite steel-concrete structures (RU 10, University of Molise – coordinator: G. Fabbrocino)

The visions and developments foreseen by RU 10 are very close to those already described by RU 9. Indeed, a strong cooperation was established between the two RUs, with a common activity relevant to the testing of beams and the attempt to develop numerical models for the evaluation of the beam rotation capacity under varying conditions and parameters.

6.11 Composite frames with dissipative braces (RU 11, University of Ancona – coordinator: L. Dezi)

The use of dissipative braces in new steel and steel-concrete composite structures is certainly very topical. Regarding the activity carried out by this research unit, future developments may concern whether the study of new devices, especially new and simpler configurations of HDR-based devices or whether the evaluation of the robustness of different structural solutions, by means of appropriate sensitivity analyses. Finally affective advantages of dual systems consisting of full restrained or partially restrained moment resisting frame coupled with dissipative braces should be carefully evaluated.

6.12 Degrading hysteresis models and fragility curves of frames with composite semi-continuous joints (RU 12, University of Trieste – coordinator: C. Amadio)

Subsequent developments of the research will involve the following topics:

- Development and characterization of cyclic models for the components of the joints usually employed in professional practice.
- Considering the good seismic behaviour of the coupled system semi-continuous frame-visco-elastic dissipative bracings, it would be important to extend the analysis to the use of linear and nonlinear fluid-viscous dissipative systems. The use of these last dissipative devices, characterized by high compactness and durability, would overtake the drawbacks of rubber devices.
- Definition of a simplified probabilistic method, able to provide results comparable with those obtained with the complete approach used in this research. At first attempt, the procedure proposed by Cornell and Jalayer, derived from FEMA 350 and based on a probabilistic characterization of demand and capacity could be used.

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INNOVATIVE PROCEDURES FOR DESIGN OF RETAINING STRUCTURES AND EVALUATION OF SLOPE STABILITY

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1 INTRODUCTION

The analysis of the seismic behaviour of slopes and the study of soil-structure interaction (SSI) under earthquake condition are very relevant topics both from the theoretical and the practical points of view.

While in the past very simple pseudostatic methods based on conventional seismic coefficients were normally used, performance based design is actually the most suitable approach to analyse foundations, retaining structures, tunnels, caverns, slopes, embankments, earth dams, and so on.

According to this approach, analyses of different level of complexity can be performed depending on the relevance of the problem, the availability of geotechnical and seismological data, the type of performance to be verified. In general it is possible to distinguish among three main categories:

1. *pseudostatic methods and empirical correlations*: these methods, widely diffused in the applications, require the definition of appropriate seismic coefficients “equivalent” to the actual seismic action; it is obvious that different seismic coefficients can be used for different structures and, also, for the same type of structure depending on the required performance;
2. *simplified dynamic analyses*: the main feature of such methods is that the study of SSI is generally uncoupled; different analysis methods attain to this category (*e.g.* displacements methods; subgrade reaction methods); the seismic input should be a set of acceleration time histories;
3. *fully coupled numerical analyses*: also in this case the seismic input should be a set of acceleration time histories.

The researches performed in the framework of Theme 6 referred to this scheme and were mainly devoted to assess different procedures of analysis and to calibrate them against benchmarks from field data and model tests results as much as advanced theoretical studies. Researches were devote to the following:

- **Sub-theme 6.1**: **Deep Excavations and Underground Tunnels in Urban Areas** (coordinated by Stefano Aversa);
- **Sub-Theme 6.2**: **Tunnels and Caverns in Rock** (coordinated by Giovanni Barla)
- **Sub-Theme 6.3**: **Slope Stability** (coordinated by Sebastiano Rampello)
- **Sub-Theme 6.4**: **Deep Foundations** (coordinated by Armando L. Simonelli)

The motivations and the background, the results, the possible developments of the four sub-themes are synthesized in the following pages.

SUB-THEME 6.1: DEEP EXCAVATIONS AND UNDERGROUND TUNNELS

1 INTRODUCTION

The performance of flexible retaining structures and tunnels subjected to seismic actions can be evaluated with several methods at increasing levels of complexity from pseudo-static methods, or simplified dynamic methods, to fully coupled effective stress numerical analyses under dynamic loading.

In the pseudo-static approach, the soil and the structure, be it a flexible retaining wall or a tunnel, are assumed to be subjected to an acceleration which is taken to be constant in space and time and is expressed through a seismic coefficient, K, generally a fraction of the peak acceleration expected at the site. The main conceptual drawback of the method is the selection of a representative value of the seismic coefficient such that the pseudo-static actions produce equivalent effects on the structure as those induced by the seismic actions, i.e. actions that are variable in time and space and transient in nature. In principle an appropriate value of the seismic coefficient should depend on the design ground motion parameters, such as peak ground acceleration, frequency content, and duration, but also on the soil and structure characteristics.

For flexible earth retaining structures, simplified dynamic methods include both displacement-based methods derived from the formulation originally proposed by Newmark (1965) for dams, and subgrade reaction methods. In the latter class of methods, soil-structure interaction is tackled using a decoupled approach in which the ground response is evaluated first in free field conditions and then computed displacements are applied to a set of springs and dashpots restraining the structure. These methods are frequently adopted also for the seismic analysis of tunnel linings, in which the load increments to be applied to the tunnel lining follow from a free-field seismic response analysis. The main problem with the adoption of displacement-based methods for flexible retaining structures is the definition of compatible failure mechanisms, while in subgrade reaction methods there is a problem with the selection of appropriate material parameters for springs and dashpots. Moreover, both for displacement based methods and subgrade reaction methods, representative design input acceleration time histories must be selected.

In principle, numerical methods allow the most comprehensive analysis of the response of flexible retaining structures and tunnels to seismic loading. However, like any other numerical analysis, reliable dynamic soil-structure interaction analyses require a number of simplifications and approximations of the problem, including the definition of a representative soil profile, the selection of representative mechanical properties for each layer, assumptions on ground water conditions and initial state of stress, structural geometry and boundary conditions, and the selection of representative design input acceleration time histories. Among all the idealisations, a major role is played by the constitutive model adopted for the soil, which should be able to reproduce the main features of its mechanical behaviour under cyclic loading, such as irreversibility of deformations, incremental non-linearity, hysteretic dissipation of energy, and memory of previous stress history. This can only be achieved adopting advanced constitutive models developed within the framework of bounding surface plasticity, kinematic hardening plasticity or hypoplasticity, generally not included in the libraries of commercial codes and requiring input parameters not routinely measured in field or laboratory tests. In the context of this research project, simple and advanced numerical analyses were carried out finalised both to a better understanding of the soil-structure response under seismic loading and to supply benchmarks were closed form solutions were not available or to develop simple procedures for design practice.

2 BACKGROUND AND MOTIVATION

Public demand for services to improve the quality of life in the urban environment is constantly growing; almost all greater cities in Italy face the necessity of modifying the existing balance between private and public transport. In the last ten years, due to the lack of finalised action, the quality of life has deteriorated further and further: air pollution has dramatically increased, with negative effects on public health and the cost of national health programmes, and travel times in rush hours became unacceptably long. The objective of many administrations was that of rendering the many functions of city life accessible through the creation of a sustainable and extended net of public transport, such that the use of private transport becomes an option rather than a necessity. These policies in the management of mobility have lead to a significant pressure for the construction of new underground train lines and car parks. This implies the necessity to carry out open excavations and bored tunnels in the urban environment, often in difficult ground, such as soft soils and weak rocks, and below the water table, and almost always in close vicinity of existing buildings and structures. Many important underground project are being carried out in Italy, like those in Milano, Roma and Napoli.

The ability to predict with confidence excavation and tunnelling induced displacement is a crucial aspect of the design because ground movements transmit to adjacent structures as settlements, rotations and distortions of their foundations, which can, in turn, induce damage affecting visual appearance and aesthetics, serviceability or function, and, in the most severe cases, stability of the structures (Burland and Wroth, 1974; Burland et al. 1977; Boscardin and Cording, 1989).

The availability of powerful computer codes, the formulation of constitutive models able to describe the key features of soil behaviour under different stress-paths as much as the development of experimental techniques for laboratory and in-situ tests, have made possible reliable predictions of the performance of such geotechnical structures both during construction and the subsequent life cycle. Monitoring of these structures improved the reliability of traditional design methods and permitted the development of empirical methods able to forecast displacements of the retaining structures and settlements of the ground due to the excavations. Finally the introduction of the methods of verification against limit states with partial safety factors, in agreement with EN1997-1, allowed for evaluating the safety of structures for which it is not easy to define a global safety factor.

All these progress almost exclusively have concerned the response under static conditions. In the case of seismic actions the study of the soil-structure interaction is still a complex problem to analyse. The analysis can be performed by means of pseudostatic methods based on limit equilibrium only in very simple conditions (i.e. cantilever sheet pile walls or single propped sheet pile walls), using conventional seismic coefficients defined with reference to other structures. Also the recent EN 1998 -5 faced in marginal way the design of the underground structures. These considerations have lead to a significant pressure for the study of the complex seismic interaction between ground and underground structures, with the main purpose to develop reliable and simple design procedures.

3 RESEARCH STRUCTURE

The research consisted in the following steps:
1 *Literature review:* survey of the scientific literature on the seismic behaviour of flexible retaining structures and tunnel linings.
2 *Physical modelling*: Centrifuge tests on reduced scale models in order to study the behaviour of shallow tunnels and flexible retaining walls under seismic actions performed at the Schofield Centre of the Cambridge University Engineering Department (CUED).
3 *Advanced numerical modelling*: Finite Element formulation of equilibrium and dynamic coupled flow for a two-phase medium. Identification of constitutive models capable of reproducing the main features of soil behaviour under cyclic and dynamic actions. Implementation of the selected models in existing Finite Element codes.
4 *Simplified dynamic and dynamic analyses*: the results obtained by means of centrifuge tests and advanced numerical modelling and centrifuge tests were compared with those from simplified dynamic and dynamic analyses.
5 *Pseudo-static analyses*: the principal aim of the comparison between the results of pseudo-static approach and the dynamic analyses is the definition of criteria and procedures to select rationally an appropriate value of the seismic coefficient to be used in pseudo-static methods.

4 MAIN RESULTS

4.1 Centrifuge testing

Thirteen centrifuge tests were carried out in order to define an experimental database for calibrating simplified methods and advanced numerical analyses of the behaviour of shallow tunnels and flexible retaining walls under seismic actions.
The Cambridge centrifuge permitted to run dynamic tests up to a maximum acceleration of 80g. The seismic input consists of trains of approximately sinusoidal waves of different frequency (Madabhushi et al. 1998); during one test the same model is subjected to several trains of waves of different frequency, amplitude and duration. Models were prepared using a standard fine dry silica sand, namely Leighton Buzzard Sand.
Nine tests were carried out on models of pairs of retaining walls, six of which cantilevered and three of which propped against each other, while four tests were performed on tunnel models.
The results of the tests on tunnels are reported in Lanzano (2009). The major findings of the experimental program were:

1. The maximum amplification occurs at the reference alignment, while along the free-field and the tunnel alignment the amplification was less significant, showing, in this last case, a filter effect of the tunnel.
2. The LVDTs measurements at two symmetrical points on the model surface show that densification occurred in the sand model during the tests. The measurements on the loose sand models gave rise to settlements about twice larger than those on dense sand models.
3. The time histories of bending moment and hoop force measured in the tunnel lining show the accumulation of stresses in the lining during the earthquake, probably caused by soil densification.

The models of flexible retaining walls were instrumented to measure deformation, axial load in the props and bending moments in the walls, acceleration at different locations in the model or at its boundaries, acceleration of the walls, horizontal displacements of the walls. A layout of the monitoring devices adopted for a test on a pair of propped wall is shown in Figure 1.

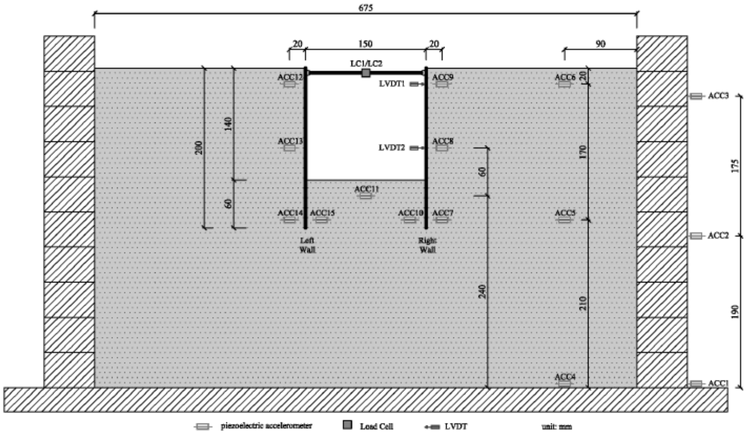

Figure 1. Test CW1: transducers layout.

Main results of the experimental programme on flexible retaining walls can be synthesized in the followings points:

1. Maximum accelerations recorded in the proximity to the retaining walls are slightly larger than those measured in free-field conditions;
2. Amplifications in terms of Arias intensity are larger than those expressed in terms of maximum accelerations, especially for earthquakes whose nominal frequency is close to the fundamental frequency of the ground.
3. During the dynamic phases cantilever retaining walls accumulate permanent displacements toward the excavation; an earthquake produces an increase of displacements only if it has an acceleration higher than the previous ones.
4. Bending moments permanently increase during earthquakes; they increase only if the wall has not experienced any other earthquake of higher acceleration.

4.2 Advanced numerical modelling

The main goal of the numerical modeling activities was twofold: i) obtaining a better understanding of the seismic behaviour of the soil-retaining structure system; ii) to use the results as a benchmark to develop simplified analysis methods in current seismic design.

For the modeling of cyclic/dynamic behaviour of fine-grained soils, the research has been focused on the following three constitutive models:

1. the Bounding Surface plasticity with radial mapping model of Tamagnini & D'Elia (1999);
2. the Bounding Surface plasticity with multiple, kinematic hardening loading surfaces of Rouainia & Wood (2000);
3. the Hypoplastic model of Masin (2005).

For coarse grained soils the research have been focused on the following constitutive models:

1. The BSKH elastoplastic model of Papadimitriou & Bouckovalas (2002);
2. The HP model of von Wolffersdorff (1997).

The model of Papadimitriou & Bouckovalas (2002) was used for FE simulations of the results of centrifuge tests through the code ABAQUS® standard. The calibration of the model for the

Leighton Buzzard sand has been carried out based on the experimental data obtained by Visone (2009).
The results of the simulation indicate that there is a clear amplification effect of the horizontal accelerations, moving from the model base to the ground surface. The amplification effect is significant in a medium-high frequency range. The evolution with time of the bending moments on the tunnel lining in four points placed along the diagonals at 45° from the horizontal and vertical axes show that the magnitude of the bending moments in all points tends to increase almost monotonically with time. This is due to the progressive accumulation of plastic strains in the soil surrounding the tunnel lining during the seismic stage, in agreement with the observed behaviour.

4.3 Simplified dynamic and dynamic analyses

4.3.1 Tunnels

Simplified dynamic analyses were carried out to develop a simple criterion for evaluating the internal force increments induced by an earthquake in the trasversal plane of a tunnel lining, according to the objectives of the research project. The proposed methodology allowed for evaluating the influence of the kinematic interaction on the basis of the results of a set of dynamic analyses, performed for several subsoil conditions, tunnel geometries and characteristics of the seismic event. Three subsoil profiles were considered, which correspond to deposits of typical soft clay, medium-dense sand and gravel. The dynamic analyses were performed by means of Plaxis® 8.2, by modeling the soil as an equivalent linear viscous-elastic medium. Interesting results were obtained in terms of bending moments in the tunnel lining.

FE element analyses were also carried out in the longitudinal direction modelling the tunnel as a series of elastic beams supported by springs, both in the longitudinal and the transversal plane. The stiffness of the springs were evaluated using the formulas presented by St. John & Zahrah (1984).
Unlike closed-form solutions, this method accounts for the effects of important factors such as the change in mechanical and geometric characteristics of the tunnel and in stiffness of the springs (along tunnel axis) as well as the presence of constraints at the ends of the structure.
The numerical analyses have been carried out considering the same tunnel and three idealized ground conditions.
In conclusion, the FE method gave rise to internal forces very similar to the analytical methods in terms of internal forces, with the exception of the following situations:

(a) when the tunnel traverses distinct geological media with sharp contrast in stiffness;
(b) when the length of the tunnel is comparable with the wave length of the seismic input;
(c) when the ends of the tunnel are connected to a station end wall or a rigid, massive structure such as a ventilation building, which represent a constraint for the ends of the tunnel.

4.3.2 Flexible retaining walls

A number of numerical analyses were carried out to study the seismic behaviour of flexible retaining walls (cantilever and propped) embedded in a coarse-grained soil with different computer codes (Plaxis®; Flac®; ANSYS®) and different types of constitutive models: the Mohr-Coulomb elastic-perfectly plastic model and the isotropic hardening model HSM in Plaxis; the elastic-perfectly plastic model with Drucker-Prager yield surface in ANSYS. The viscous damping of the soil was modelled via the Rayleigh approach. Flac analyses were performed using an elastic-plastic model with purely hysteretic damping. Boundaries were modelled to account for radiation damping.. Acceleration time histories were selected from a database of Italian seismic events (Scassera et al. 2006).

The influence of the mesh size was firstly assessed at the preliminary stage of this study. Emphasis was then placed on the effect of soil damping ratio. Results from these simulations showed that bending moments and prop forces (for the propped walls) during and after the earthquake are significantly greater than the static ones, as a consequence of the increase in contact stress at the soil-wall interface induced by the seismic excitation (Aversa et al. 2007). This result was confirmed by experimental date from centrifuge tests (Aversa et al. 2008).

A limited number of analyses were carried out with the FD method by assuming the walls to be elastic-perfectly plastic. Propagation of the two seismic records considered in this analysis (Tolmezzo and Assisi) causes instantaneous attainments of the available shear strength in soil zones interacting with the walls, resulting in a progressive increase in bending moments and axial load in the prop (Figure 2).

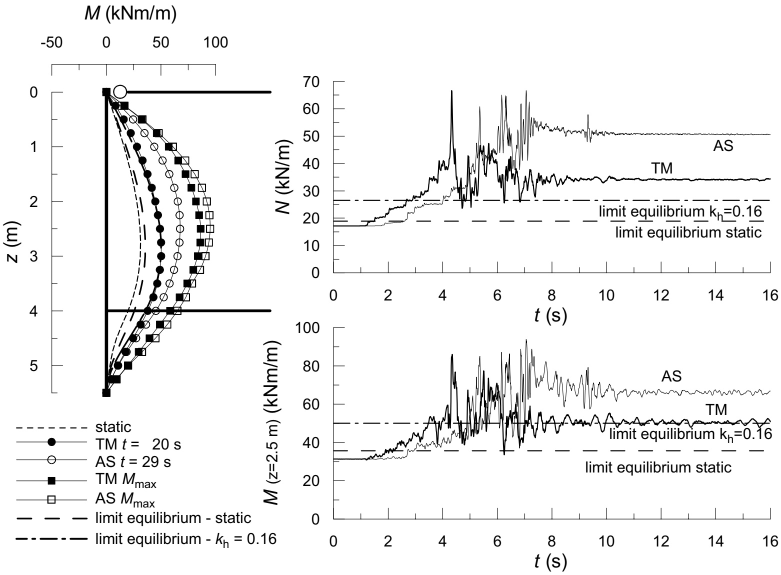

Figure 2. Bending moment and axial load in the prop.

The results showed that, while the maximum bending moment cannot exceed the yield moment, the computed wall displacements are about 30 % larger than those evaluated assuming that the walls are linearly elastic.

The main results of this study are described in detail in: Callisto & Soccodato (2007, 2009), Callisto et al. (2007, 2008).

4.4 Pseudostatic procedure analysis: revisited approaches and innovative criteria

In pseudo-static analyses of tunnels internal forces are commonly calculated from the free field shear strain profile, i.e. neglecting the kinematic interaction between the tunnel lining and the ground. This lead to the development of new expression capable to account at a very simple level the effect related to kinematic interaction phenaomena. .

For flexible retaining structures, pseudo-static solutions have been used in the past as possible design procedures for gravity retaining walls (Richards & Elms 1979, Whitman 1990). Conversely, less attention has been paid for the seismic design of embedded retaining walls. In these structures, a significant part of the resisting forces is provided by the passive reaction of the soil located below the bottom of the excavation, and this resistance is significantly affected by inertial effects. Therefore, during an earthquake a continuous interaction occurs between actions and resistances. Upon instantaneous attainment of the available strength, cantilever and single-propped walls undergo rigid-body movements in a way which is qualitatively similar to the behaviour of gravity retaining walls.
An attempt was made to define a rational approach for seismic design of cantilevered embedded retaining walls (Callisto & Soccodato, 2007; Callisto et al., 2007, 2008). Only the case of dry coarse grained soils was considered. For a given retaining wall, a critical horizontal seismic coefficient k_c can be evaluated by performing iteratively a limit-equilibrium analysis, and finding the value of the seismic coefficient for which soil strength is fully mobilised. Permanent displacements can then be assumed to result from a Newmark-type integration of the relative motion, and increase as k_c decreases. For a given soil and a given excavation height H, the value of k_c depends essentially on the embedded length d.
Therefore, the embedded length should be chosen on the basis of the maximum displacement allowed for the severe seismic event.

5 DISCUSSION

The objective of understanding the seismic soil-structure interaction for both circular tunnels and cantilever and single propped retaining walls was pursued by means of both centrifuge testing and numerical simulations of different complexities. The investigation has been performed with main reference to unsaturated coarse grained soils. For this type of soil the comparison among results of different computer codes and physical models has shown common features in terms of co-seismic and post-seismic bending moments and other internal forces, displacements of the structure and of the ground level.
Different computer codes have been tested. Particular attention has been devoted to the effects of the mesh size, to the boundary conditions, to the way of considering damping. The role of different constitutive models has been investigated too. Sophisticated constitutive models able to reproduce behaviour of both coarse grained and fine grained soils, based on Boundary Surface Plasticity or on Hypoplasticity, have been implemented in FE code, to extend the centrifuge results to more complex situations and ground conditions. At the same time, more common constitutive models, available in the libraries of commercial computer codes, have been tested.
In addition simple pseudostatic methods to design these types of structures have been developed. Some of the obtained results have been already taken into account in the recent Italian building code (DM 14/01/2008). For the case of cantilever and single propped sheet pile walls a sort of capacity design procedure has been proposed.
To sum up, it may be stated that most of the research objectives were attained with reference to the considered structures and that some of the factors affecting the seismic soil-structure interaction under plane strain conditions were recognised, their influence being isolated and evaluated.

6 VISIONS AND DEVELOPMENTS

The present research activity was mainly addressed to the study of the seismic behaviour of tunnels and cantilever and single propped sheet pile walls in homogeneous dry coarse grained soils. Centrifuge model results provided a very good base for calibrating design procedures of different complexities. Some of the results can be directly applicable in the design; some others require an increase of knowledge both in terms of experimental analyses and in terms of numerical simulation. In particular, the case of saturated fine grained soils requires more simulations; the behaviour other types of underground structures, very common in urban areas (i.e., multi-propped retaining wall; parallel tunnels; ecc.), should be more deeply investigated. Similarly the very relevant case of seismic interaction of underground structures in highly inhomogeneous soils or in correspondence of sharp changes in structures stiffness (i.e., the connection between tunnels and deep propped excavation for underground railway stations, ecc.) is still an unsolved problem. These problems could be tackled with the same type of approach used in the last three years: physical models and highly sophisticated computer codes to define a target in order to calibrate methods of analyses of different complexity, including simple pseudostastic approaches for some of the cases. The results of this research might provide guidance for the design of new underground structures, but would also assist in the seismic evaluation of the many existing tunnels and excavations present in urban areas, originally designed without taking into account seismic actions or simulating seismic loads in very simple and conventional way.

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SUB-THEME 6.2: TUNNELS AND CAVERNS IN ROCK

1 INTRODUCTION

Underground structures are critical elements when considering transportation systems. They are also critical elements in the more general framework of utility systems (e.g. hydraulic tunnels and hydroelectric caverns, tunnels for transportation of fluids such as petroleum and natural gas, etc.). The importance of these underground structures makes the problem of understanding their behaviour during a seismic event extremely relevant.
The behaviour of underground structures in seismic conditions is significantly different from that of surface structures. While these are substantially designed to sustain the inertial forces due to ground acceleration, the loads acting on underground structures are linked to the state of stress and strain in the ground surrounding them. Thus, the need arises to develop design analysis methods in order to assess the effects on underground structures in seismic conditions.
It is understood that these effects cannot be neglected at the design stage, in particular in nations like Italy where wide areas are characterised by different levels of seismicity. In addition, one may note that there are no rules or general guidelines available, in Italy or in other Countries, for the design of underground structures in seismic conditions. The present research on tunnels and underground caverns in rock in seismic conditions stems from this important need.

2 BACKGROUND AND MOTIVATION

Underground structures are generally thought to be safer than surface structures, given that the ground in which they are built provides protection against earthquake damage. However, these structures cannot be considered to be intrinsically safer than surface structures, as well demonstrated by recent seismic events: in 1995, Kobe (Japan); in 1999, Chi-Chi (Taiwan); in 1999, Kocaeli (Turkey); and in 2004, Niigata (Japan); when a number of tunnels and underground cavities underwent significant damages.
The response of underground structures to a seismic event is governed by the surrounding ground behaviour and not by the inertial characteristics of the structure itself, as the response to such an event is substantially dependent on induced displacements. Thus the design of underground structures in seismic conditions need to consider ground deformations and soil-structure interaction, which imply that the kinematics effects are predominant with respect to the inertial effects.
The presence of the structure is to modify the "free-field" deformations due to the interaction of the ground and the support. In such conditions one is to adopt either closed-form solutions or numerical solutions. The closed-form solutions may be used under simplified geometrical and loading conditions and consider separately the stresses due to the distortion of the tunnel cross section and the out of plane bending and axial deformations. The numerical solutions comprise different design analysis methods, which may comply with a different degree of approximation in relation to the current design stage and the level of understanding of the geological and geotechnical conditions to be considered.
The development of design analysis methods requires a careful understanding of the effects of past earthquakes which have damaged underground structures, so as to be able to underline the most significant factors related to the state of damage. The effects of ground motion is

analysed in the following, without considering other types of instability (e.g. earthquakes driven landslides, liquefaction, etc.). In fact, problems of underground structures crossing landslides or active fault zones, although causing relevant damages, are restricted to these zones. Therefore, it is expected that these zones can be localised at the design stage so as to minimise such effects by a careful selection of the tunnel alignment.

3 RESEARCH STRUCTURE

The main purpose of the present research has been to develop design analysis methods for the study of the interaction problems between the lining and the rock mass during severe earthquakes. In view of the present practice in the design of tunnels and underground structures (Barla, 2005), both simplified and advanced numerical methods have been developed. With the intent to compute the state of stress in tunnel linings in seismic conditions, at the design stage, most of the attention has been posed on the development and validation of simplified methods.

A significant problem which has been considered in detail during this research has been the simulation of ground motion, in view of the absence of appropriate measurements of ground acceleration at depth. Considering that a tunnel extends along its centreline, it has been felt important to be able to consider different motions in different sections of the tunnel/cavern. These aspects have been accounted for by using a number of solutions such as proposed by Hisada (Hisada, 1995; Hisada and Bielak, 2003), which allow one to compute the ground motion in a stratified half-space near fault zones.

The complexity of the problem being considered requires in most cases the adoption of advanced numerical methods (mainly: Finite Element Methods - FEM, Finite Difference Methods - FDM, Distinct Element Methods - DEM). This necessity is strictly linked to the three-dimensional conditions characterizing the problem, the complex rock mass behaviour, where in cases the adoption of continuum/equivalent continuum models may become questionable; the complex geometrical and geological characteristics of some applications; etc.. These advanced models need be developed and appropriate rules for its use are to be made available, also in view of the computations to be carried out in order to validate the simplified methods. With the above background in mind, the research structure can be defined in terms of the main topics which have been addressed during the work carried out so far:

1. State of the art of underground structures in seismic conditions.
2. State of the art of design analysis methods.
3. Database, damage scenarios, and development of fragility curves.
4. Definition of seismic input.
5. Design analyses of tunnels and caverns, by analytical and numerical methods, in 2D and 3D conditions.
6. Design analyses of well-documented case studies. Use of advanced codes (GeoELSE) and of commercial codes (FLAC and FLAC3D, Examine3D, Midas-GTS, etc).
7. Development of Early Warning innovative methods for underground monitoring.
8. Development of Guidelines for design analysis of underground structures in static and dynamic conditions.

The above topics have been developed by the three research units forming the Sub-Unit 6.2 (Politecnico di Torino, Department of Structural and Geotechnical Engineering - DISTR; Politecnico di Milano, Department of Structural Engineering - DIS; Eucentre, Pavia - Eucentre). DISTR and DIS mainly worked on the development of design methods for tunnels and underground cavities and on the validation of the adopted procedures, including the analysis of relevant case studies. The main task of Eucentre has been to define the seismic input for design analysis. DISTR has also been providing the documentation material for selected case studies and, moreover, has been working to develop innovative early warning systems for underground monitoring. Due to the limited space available, only the following points are briefly addressed in this review: Database on tunnel performance during earthquakes; Seismic vulnerability assessment and fragility curves; 3. Simplified solution for seismic design of tunnels; Advanced numerical method for seismic design of tunnels; Overview of case studies and applications.

4 MAIN RESULTS

4.1 Database on tunnel performance during earthquakes

A review of the seismic damages suffered by underground structures has been carried out by updating the database currently available (Dowding and Rozen, 1978; Owen and Scholl, 1981; Sharma and Judd, 1991; Power et al., 1998; Wang et al., 2001; etc.). This review shows that most tunnels which underwent damages were located in the vicinity of causative faults. The characteristics of ground motion in these cases can be significantly different from that of the far-field. There is no well defined distance over which a site may be classified as in near-field or far-field. A useful criterion to define the near-field zone is related to the comparison of the source dimension with the source to site distance.

From the bibliographic review of earthquakes-induced damages in underground structures, one can notice a wide variability of data, in terms of geometrical information, rock mass conditions, earthquakes characteristics, strong motion parameters, type of damage, type of support, etc. Therefore, in order to define unambiguously a damage criterion, in this work (see Corigliano, 2007a for details) the damage state has been defined according to Huang et al. (1999), combined with some considerations proposed by ALA (2001) and RTRI (2001). The damage classification includes qualitative and quantitative damage descriptions. In particular, this classification considers four levels of damage (none, slight, moderate and severe damage) grouped into three categories A, B and C, in which the group A includes none and slight damage. Each category comprises the functionality states defined by RTRI (2001). The database collected has been used to assess the effects of the following main factors on tunnel response: overburden depth, predominant rock type, type of internal support, earthquake magnitude, epicentral distance, peak ground acceleration. Figure 3 shows as example a typical plot for the influence of the epicentral distance and surface Peak Ground Acceleration (PGA) on tunnel performance, in terms of damage level: none, slight, moderate, heavy.

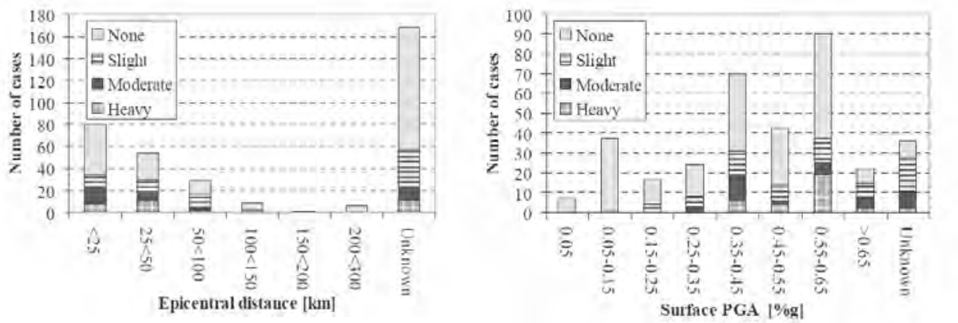


Figure 3. Effects of epicentral distance (left) an Peak Ground Acceleration (PGA) on damage (right).

4.2 Seismic vulnerability assessment and fragility curves

A useful tool for assessing the seismic vulnerability of underground structures is represented by the fragility curves. The seismic vulnerability of a structure can be defined as its susceptibility to be damaged by a ground shaking of a given intensity (Crowley et al., 2006). A fragility curve represents the relation between the probability of achieving a specified level of damage for a prescribed level of seismic hazard. Relatively few families of fragility curves have been derived for underground structures (ATC, 1985; ASCE, 1985; ALA, 2001; FEMA and NIBS, 2004). In general, the Peak Ground Acceleration (PGA) is taken as a measure of ground shaking intensity.

As already shown in Figure 3, PGA shows a poor correlation with the damage potential of ground motion. As a consequence, the intensity of ground motion shaking for assessing seismic vulnerability of underground structures has been quantified in terms of PGV at the free surface. Since a direct measurement of PGV at the location where the tunnel is damaged is generally not available, the values of this ground motion parameter can be back-calculated through an attenuation relationship purposely developed to predict PGV in near-fault conditions. Among the different attenuation relationships proposed, that given by Bray & Rodriguez-Marek (2004), which accounts also for site conditions, has been used.

Fragility curves for deep tunnels based on the empirical approach have been developed. The fragility curves are modelled as log-normal statistical distribution functions which give the probability of reaching or exceeding different states of damage for a given level of ground motion intensity. Each fragility curve is characterized by a median value of ground motion (x_{50}) with an associated log-normal standard deviation σ. The functional form of the probability distribution is illustrated in Figure 4 below which gives the Cumulative Distribution Function (CDF) versus the Peak Ground Velocity (PGV).

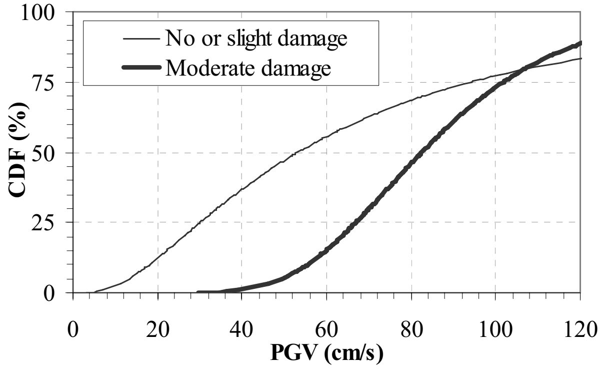


Figure 4. Fragility curves corresponding to states of “*no damage*“ or “*slight damage*” and “*moderate damage*”.

4.3 Simplified approach for seismic design of tunnels

4.3.1 Analysis of transversal response (ovaling deformations)

A simplified approach for the seismic design of tunnels subjected to strong ground shaking such as occurring at sites located in the vicinity of a causative fault has been formulated. The transversal response (ovaling deformation) has been considered by taking a lined circular

tunnel in plane strain conditions. The earthquake loading has been modelled as a uniform, quasi-static strain field simulating a pure shear deformation (Figure 5).
The relations for displacements, bending moment, thrust and shear forces have been derived following the same approach as used by Einstein and Schwartz (1979), in which however the assumption that the induced internal forces are caused by excavation has been removed and replaced with an imposed, external quasi-static loading distribution. The solution has been obtained for two contact conditions at the structure-rock interface, full-slip and no-slip (Corigliano et al., 2007 b, c; Barla et al., 2008; Corigliano et al., 2009):

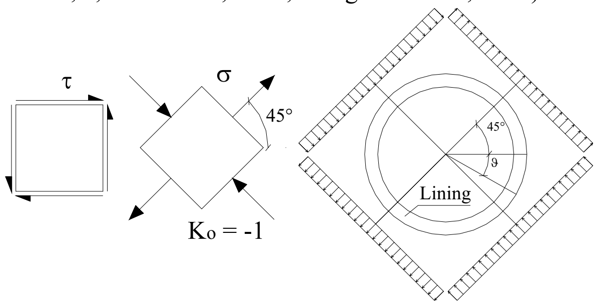


Figure 5. State of stress corresponding to a uniform, pure shear deformation.

4.3.2 Analysis of longitudinal response (axial and bending deformations)

Also considered has been the tunnel response along the longitudinal direction (which involves axial and bending deformations). A FEM model has been purposely developed by subdividing the tunnel into a finite number of beam elements with lumped mass, connected to the surrounding ground by a series of frequency-dependent springs and dashpots in parallel (i.e. a Kelvin-Voigt model, Figure 6) representing the effects of ground deformability and energy dissipation.
Wave scattering or kinematic interaction is not accounted for and thus this model can be ascribed to the class of simplified dynamic methods to analyse underground structures. The seismic excitation is input at the external nodes of the Kelvin-Voigt model through appropriate three-component free-field displacement and velocity time-histories. The synthetic records are generated using the Hisada and Bielak (2003) approach which allows to properly model not only the spatial variability and phase shift of ground motion but also the typical features of near fault ground motion (i.e. directivity and flying step effects).

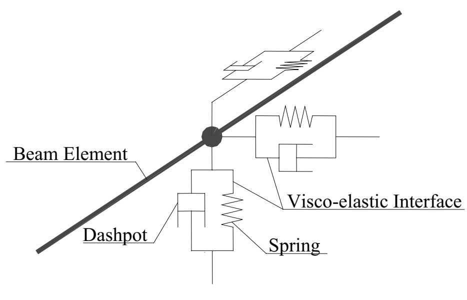


Figure 6. Spatial beam element model connected by means of Kelvin-Voigt elements to the surrounding ground.

4.4 Advanced numerical methods for seismic design of tunnels

Consideration has been given to the use of advanced numerical methods to perform a complete analysis of the tunnel problem in seismic conditions, which includes the simultaneous consideration of the seismic source, the propagation path, the geological/geotechnical site conditions, and soil-structure interaction. The Domain Reduction Method (Bielak et al., 2003), a powerful sub-structuring approach, has been applied to reduce the computational efforts required by such a large scale numerical problem.
The Domain Reduction Method (DRM) implies the subdivision of the original problem into two simpler problems each one solved in two independent steps (Figure 7). The first problem accounts for earthquake source and propagation path and it is solved with a model that includes both the source and a background structure (*external domain*) from which the structure has been removed and replaced by the same material as the surrounding soil. The second problem simulates with the desired accuracy only a region (*internal domain*) of the domain of interest which includes the structure and the surrounding soil, but not the causative fault.
The input for the solution of the reduced problem is a set of effective nodal forces calculated from solving the auxiliary problem and applied in a narrow strip of elements (see the dark boundary in Figure 7). These forces are equivalent to (and replace the) original seismic forces computed in the first step. A detailed description of the implementation of the DRM for applications with the Spectral Elements Method can be found in Faccioli et al. (2005) and Stupazzini et al. (2006).
The main feature of this approach is the possibility of coupling solutions obtained by different methods in two different domains. In this study the auxiliary problem (*external domain*) has been solved using a 3D model by the semi-analytical method developed by Hisada and Bielak (2003). The reduced problem (*internal domain*) has been instead solved by means of a 2D numerical analysis using the Spectral Element Method (SEM).

DRM STEP I (3D Free-field analysis) **DRM STEP II** (2D SEM analysis)

Auxiliary problem **Reduced problem**

a) **b)**

Figure 7. Domain Reduction Method (DRM) for advanced numerical analysis of the tunnel problem.

The above numerical method has been used by taking as example the Serro Montefalco tunnel (railway switch line Caserta - Foggia), in the northern sector of the Southern Apennines, one of the most active seismic regions in Italy. The rock masses include clay-shales, marl and marly limestone and clay and marl intercalated with limestone (Barla et al., 1986). The results obtained from the advanced numerical analyses performed have been compared with those derived with the developed closed-form solution (the maximum shear strain obtained is γff max = $1.39 \cdot 10-4$).
Figure 8 shows the comparison for thrust force and bending moment, also see Corigliano et al. (2007 b, c). The agreement between the two solutions is satisfactory. In particular, the

thrust force computed using the numerical analysis exceeds that calculated by the simplified method by approximately 30%. It is important to remark that the bending moment and the thrust force illustrated in

Figure represent the dynamic increment of the internal forces. The total moment and thrust force are obtained by adding the dynamic increments to the static values as shown in Figure 9.

4.5 Case Studies

In addition to the Serro Montefalco tunnel, briefly mentioned above, two case studies have been analysed (see Barla et al., 2008 a,b). The main interest of these studies has been to validate the approach proposed for design (from the use of simplified analysis methods to more advanced numerical methods) by performing back analyses of tunnels which underwent severe damages during a real earthquake. For this reason the decision was taken to analyse the *Pavoncelli tunnel*, i.e. the base tunnel of the Acquedotto Pugliese which, during the Irpinia earthquake of November 23,1980 suffered severe damages to the tunnel lining (Figure 10). Also, due to the interest in underground caverns, a comprehensive numerical study of the *Venaus Powerhouse*, a major component of the Pont Ventox - Susa hydroelectric scheme (Figure 11) has been carried out. With the geological and geotechnical data available (Barla, 2005; Barla et al., 2008), the analyses have been performed in both static and dynamic conditions by using the FEM method and the Midas-GTS code. Also analyses have been carried out with the Geo-ELSE code and DRM.

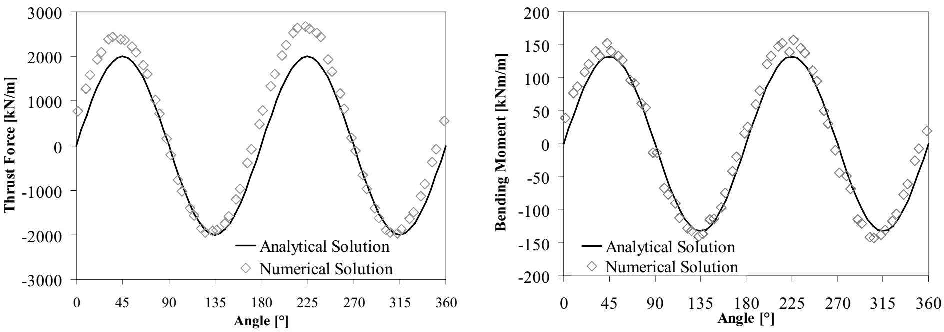
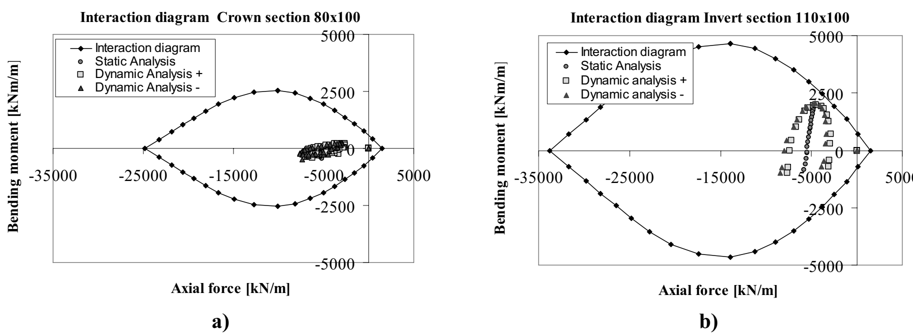
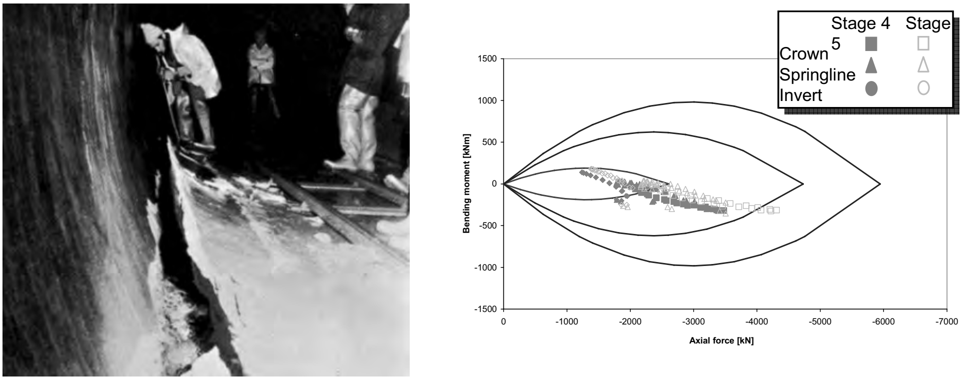


Figure 8. Results of advanced analysis and closed-form solution of the Serro Montefalco tunnel a) thrust force; b) bending moment.



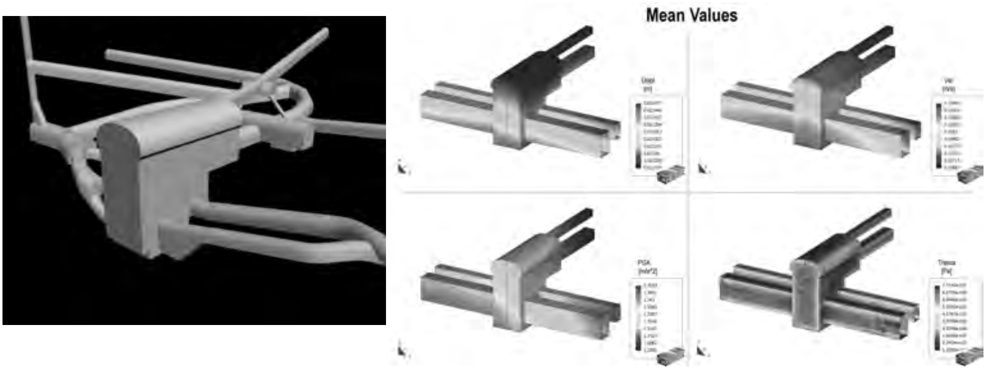
a) b)

Figure 9. Interaction diagram for the cross-section.
a) crown and b) invert , with the dynamic stress increment added.



(a) (b)

Figure 10. (a) FEM model, lining detail. (b) Photograph of typical damages occurred during the 23rd November 1980 Irpino-Lucano earthquake.



(a) (b)

Figure 11. (a) Perspective view of the cavern complex. (b) A summary of results in terms of mean values of peak acceleration, velocity and displacement, as well as of the peak maximum shear stress.

5 DISCUSSION

The research programme briefly outlined in this overview has addressed a number of problems and issues dealing with deep tunnels and caverns in seismic conditions, with the main objective to provide appropriate computational methods for design analysis. A comprehensive review of the state of the art on the response of underground structures in seismic conditions has been provided, including an overview of the earthquake induced damages and methods which are presently available in the technical literature. On this basis, the effects of past earthquakes on underground structures have been identified so as to underline the most significant factors responsible for different types of damage: overburden depth, predominant rock type, type of internal support, earthquake magnitude, epicentral distance, and peak ground acceleration.

A closed-form solution for the analysis of the transversal response of the tunnel lining due to ovaling deformations has been developed. The earthquake loading is modelled as a uniform, quasi-static strain field simulating a pure shear deformation. A key parameter for computing stress is the maximum shear strain in free-field conditions. A model based on the Finite Element Method has also been developed to analyse the tunnel response along the longitudinal direction, which involves axial and bending deformations. The tunnel is subdivided into a finite number of beam elements with lumped mass, connected to the surrounding ground by a series of frequency-dependent springs and dashpots in parallel representing the effects of ground deformability and energy dissipation. Advanced numerical methods have been used for a complete analysis of the underground structure in seismic conditions, which includes the simultaneous consideration of the seismic source, the propagation path, the geological/geotechnical site conditions, and soil-structure interaction. The Domain Reduction Method (DRM) which implies the subdivision of the original problem into two simpler problems, each one solved in two independent steps, has been adopted. The first problem accounts for the earthquake source and propagation path and it includes both the source and a background structure (*external domain*) from which the structure has been removed and replaced by the surrounding soil. The second problem simulates the region (*internal domain*) of interest which includes the underground structure and the surrounding soil, but not the causative fault.

Case studies of tunnels have been considered with the main objectives to validate the closed-form solution by means of advanced numerical methods and to perform back analyses of tunnels which underwent severe damages during a real earthquake. Also, the case of an underground cavern, which is characterised for the high level of geological and geotechnical data available, has been taken to assess the most appropriate methods to be used for design of complex structures in seismic conditions.

With the above developments completed, a design process has been defined in line with the Design Norms for Construction (NTC 2008) recently promulgated in Italy (i.e. Preliminary Design, Intermediate Design, Final Design). As underlined also in a paper recently published (Corigliano, Lai and Barla, 2009), the following design procedure can be proposed:

Preliminary Design: a seismic vulnerability assessment of the underground structure can be performed by using Fragility Curves, which provide the designer with a tool for deciding whether a given underground structure need be studied in subsequent design stages with more refined design analyses.

Intermediate Design: a simplified approach is adopted which takes into account the interaction of the underground structure with the surrounding ground and at the same time considers the near-fault ground motion. This implies the use of validated closed form solutions, under the assumption of simplified constitutive laws for the rock mass and the support (continuum, elastic model), with the loading condition represented by the maximum shear strain in free-field conditions.

Final Design: more advanced computational methods are to be used for the design of tunnels, based on numerical methods and non linear constitutive laws for the rock mass and the support (continuum, elastic and elastoplastic models), such as the Finite Element and Finite Difference Methods, with attention being paid to both the transversal and longitudinal responses. Three dimensional complete numerical analyses need be considered for the case of complex structures such as Underground Caverns, depending on the level of complexities and the site conditions to be taken into account.

6 VISION AND DEVELOPMENTS

The research programme on underground structures in seismic conditions has analysed so far the vulnerability of tunnels and caverns, with the rock mass represented as a continuum/equivalent continuum. It is proposed to develop methods for the analysis of underground structures in rock masses represented as a discontinuum. The transition from continuum to discontinuum analysis is very important if intended for design analysis of underground excavations in seismic conditions. In general, this is the case of underground structures excavated in good quality rock mass conditions near to the ground surface or at depth, where the rock mass is better simulated as a discontinuum rather than a continuum/ equivalent continuum.

Analytical and numerical methods for design analysis of discontinuous media need be considered. Given that the methods most frequently used, in two-dimensional (2D) and thee-dimensional (3D) conditions, are for a continuum/equivalent continuum, the bibliographic studies will be concerned with discontinuum. On the one end, consideration will be given to DDA ((Discontinuous Deformation Analysis) and DEM (Discrete Element Method) methods in seismic conditions. On the other end, the attention will be posed on the available analytical solutions, which will be used for validation purposes.

In general, the presence of the discontinuities in a continuous medium is the cause of seismic waves attenuation and delay of the arrival time, due to the onset of both reflected and refracted waves. Also to be accounted for is the presence of waves generated along each discontinuity (Stoneley's waves) which need be assessed in terms of its influence on the overall response of the discontinuous medium. It is important to underline that the study of wave propagation in a jointed medium is totally different from the study of wave propagation in a continuous medium. In fact, a joint comprises two mating surfaces which may undergo shear displacements so that the boundary conditions to be considered imply both the continuity of the joint surface stresses and the discontinuity of displacements. The problem, which is indeed rather complex in two dimensional (2D) conditions, becomes even more difficult if three dimensional (3D) conditions are considered.

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SUB-THEME 6.3: SLOPE STABILITY

1 INTRODUCTION

One of the main sources of seismic vulnerability in Italy is represented by the instability of slopes; therefore, this is a subject of great significance, particularly in view of the growing attention that has been recently dedicated to the reduction of seismic hazard.
The response of a slope under seismic loading is determined by the temporal and spatial distribution of the seismic forces in the soil mass, which in turn depend on the characteristics of the seismic input and on the mechanical properties of the soil.
A number of different techniques exist to address this problem, each implying some level of approximation. Experience of the use of advanced numerical analysis is still somewhat limited, and it seems difficult to generalise the results of such complex analyses. An advisable approach would be that of carrying out the analysis of the same problem using a number of approaches characterised by different levels of complexity, in order to assess the reliability and robustness of the different procedures. In the present research project, different research groups were given the task of pursuing the study of the seismic behaviour of slopes using a number of different approaches, and investigating the possibility of using the results of the more advanced analysis as a guidance for a sound and reliable use of the simplest and most common analysis method, that still form the backbone of professional practice.

2 BACKGROUND AND MOTIVATION

The simplest and most common analysis method is the pseudo-static approach in which the earthquake inertia forces are taken to be equivalent to constant static forces. Basically, the ingredients of this method are the amount of the constant forces that simulate the seismic behaviour, and the safety coefficients (or partial factors) that can be deemed adequate for the stability checks to be satisfied. As this approach neglect completely the transient and cyclic nature of the seismic forces, it must be considered as a purely conventional tool, and these input quantities do not have a physical meaning per se, but need to be calibrated against the results of more realistic approaches.
A better representation of the seismic behaviour of a slope can be obtained by the displacement method, originally developed by Newmark (1965) for the seismic analysis of earth dams. This method removes the assumption of temporal invariance of the seismic forces, and evaluates the permanent displacements of the sliding mass, regarded as a rigid body, produced by the acceleration in excess of that sufficient for the complete mobilisation of the soil strength along a sliding surface. Different adaptations of the displacement method do exist, in which the decay of the soil strength parameter or the development of the excess pore water pressures during the earthquake are included in the analysis. An important modification of the method consists in considering explicitly the deformability of the soil, through a decoupled analysis, in which a preliminary analysis is carried out to assess the dynamic response of the slope, and then the resulting acceleration time histories are used to compute the permanent displacements with a rigid block sliding analysis. Although this method inconsistently assumes that there are no permanent displacements in the seismic response analysis, and then uses the analysis results to calculate a seismically induced permanent displacement, it was found to provide a reasonable estimate of seismic displacements for many cases.

More advanced numerical analyses of the seismic behaviour of a slope can indeed be performed, using a coupled approach in which a full description of the dynamic behaviour of the soil is explicitly introduced. While these analysis is conceptually more satisfactory if applied to a well documented case-study, they result somewhat cumbersome in helping the development of simplified procedure, which is specifically the aim of the present research project, which requires the analyses of a large number of simplified slope schemes, subjected to a variety of seismic loading. Therefore, it was felt that the bulk of the results needed for the calibration of the pseudo-static method were to be obtained using the displacement method, more suitable for parametric studies. In applying the displacement method however, several different levels of sophistication were inserted, including two-dimensional amplification effects and the consideration of the deformability o the soil mass. A number of advanced, fully coupled numerical analyses were performed as well, to further corroborate the results obtained.

3 RESEARCH STRUCTURE

The following steps summarise the organisation of the research project

1) obtain a large and reliable database of input seismic recording that may be used in the parametric study; this was accomplished by re-examining all the available Italian acceleration records and organising the results in a new database for access via the internet;

2) perform the integration of the Italian accelerograms, using a rigid-block displacement approach, for the development of empirical relationships between the displacements and a number of motion parameters;

3) evaluate the relationships between the displacements computed with the displacement approach and the seismic coefficients to use in a pseudo-static approach, still considering an infinitely rigid sliding mass, and linking the pseudo-static seismic coefficient to the requested seismic performances.

4) study the seismic response and evaluate the permanent displacements of idealised slopes to a large number of seismic inputs, in one-dimensional conditions, taking into account in a parametric way the soil deformability and considering the coupling of the propagation through the soil mass and the sliding of the soil mass;

5) carry out a number of two-dimensional analyses of idealised slopes, subjected to a selected number of seismic inputs, accounting for the soil deformability and evaluate the permanent displacement through a decoupled approach;

6) from the above steps (4) and (5) find the corrections to apply to the seismic coefficients found at point (3) to account for soil deformability;

7) perform a limited number of advanced numerical analyses to corroborate the results obtained with the previous approach.

4 MAIN RESULTS

4.1 Database of Italian strong-motion accelerograms

A database of Italian strong motion accelerograms was developed as well as a site and source databank. The database consists in 247 Italian three-component corrected accelerograms obtained from 101 recording sites and related to 89 earthquakes that occurred in the period 1972-2002 (Scasserra et al., 2008d).

Available acceleration time histories were uniformly processed by the same team of seismologists, supervised by Dr. Silva, that is responsible for the PEER (Pacific Earthquake Engineering Center) data processing.

Site conditions at the 101 monitoring stations were identified through both geologic characteristics and values of shear wave velocities.

Velocity measurements are nowadays provided for 51 of the 101 sites. For the remaining sites, the equivalent shear wave velocity in the upper 30 m (V_{S30}), as defined by Eurocode 8, (EN 1998-5) was estimated using a hybrid approach, based on correlations with surface geology (Scasserra et al., 2008c).

A web site was created to disseminate the strong motion database and the source and site databank (Scasserra et al., 2008a). It is called SISMA, that stands for Site of Italian Strong Motion Accelerograms (http://sisma.dsg.uniroma1.it); it can be anticipated that the SISMA dataset will be archived also at the PEER web site. The principal objective of SISMA website is to provide high quality Italian strong motion records with consistent and reliable evaluation of associated seismic parameters.

The structure of SISMA allows records to be located in several ways, depending upon the user interest.

4.2 Empirical relationships for assessing earthquake-induced displacements

Empirical relationships were developed for evaluating permanent co-seismic slope displacements using records from Italian seismic events. Database SISMA was used to this purpose to obtain a homogeneous set of accelerograms with consistent and reliable estimates of the associated seismic parameters. According to the values of shear wave velocity in the subsoil underlying the monitoring station, acceleration time histories on rock or rock-like soils (class A - $V_{S,30}$ > 800 m/s) were distinguished from those on stiff soils (class B - $V_{S,30}$ = 360 – 800 m/s) and medium to soft soils (classes C, D and E - $V_{S,30}$ < 360 m/s). A total number of 196 free-field horizontal acceleration time histories were selected, these being related to 46 Italian earthquakes with magnitudes in the range 4 to 6.3.

For each accelerogram, the Newmark-type displacements were computed considering values of the critical acceleration a_y = 0.002, 0.005, 0.01, 0.02, 0.05, 0.1, 0.2 g. For a given critical seismic coefficient $k_y = a_y/g$, two values of displacement were determined considering each side of the acceleration time history. The maximum of the two obtained values were assumed as the displacement related to the signal.

The proposed relationships allow permanent slope displacement associated to 50th and 90th percentile to be evaluated as a function of: (1) critical acceleration ratio (a_y/a_{max}); (2) ratio of squared peak ground velocity to peak ground acceleration of input motion and critical acceleration ratio (PGV^2/PGA, a_y/a_{max}); (3) Arias intensity and critical acceleration ratio (I_A, a_y/a_{max}); (4) destructiveness potential factor and critical acceleration ratio (P_D, a_y/a_{max}); (5) mean period and critical acceleration ratio (T_m, a_y/a_{max}) (Madiai, 2009).

The obtained relationships can be used for a preliminary estimate of permanent displacements induced by seismic shaking.

4.3 Evaluation of slope displacements for multi-block sliding mechanisms

In common practice, earthquake-induced displacements are simply evaluated for translational or rotational sliding mechanisms. When the sliding surface is of general shape, permanent displacements can be evaluated either best fitting the sliding surface with circular or logarithmic spiral surfaces or describing the mixed roto-translational mechanism via a multi-block model; the latter provides a better representation of actual sliding mechanism thus yielding more reliable predictions of slope behaviour under seismic conditions.

In the work by Bandini et al. (2008), the transformation rules for slope geometry were selected to meet the following criteria: (1) each block keeps in contact with adjacent blocks and with the slip surface without separation or overlapping; (2) the total mass of the system is constant. If slope displacement is small with respect to the length of the slope, the effect of mass-transfer is negligible and it can be assumed that each block slides as a rigid body along its portion of slip surface without any mass-change. For large displacements the substantial change in the geometry of the slope and the consequent change in soil mass distribution cannot be neglected. In fact, geometrical rearrangement leads to a more stable configuration and is often the main factor yielding the slide eventually to stop.

In the proposed approach, pore pressure build-up in cohesive soils and change in slope geometry were accounted for in the analysis; the Global Limit Equilibrium Method of slices, as proposed by Fredlund and Khran (1977), was used for evaluating the critical seismic coefficient k_y.

The proposed GLE-based multi-block model was applied to parametric studies of several ideal schemes of slope and to studies of landslides reactivated by the Irpinia-earthquake of 1980 ($M = 6.9$).

4.4 Pseudo-static approach and evaluation of equivalent seismic coefficient

Displacement-based sliding block methods are an useful and handle tool for evaluating the response of slopes and earth structures to earthquake loading, having indeed the advantage of providing a quantitative assessment of earthquake-induced displacements using simple analytical procedures. However, the pseudostatic approach is still the most diffused method adopted in common practice. In this method the seismic coefficient designates the horizontal force to be used in the stability analysis, its selection being thus crucial. Starting from Seed (1979), a number of procedures have been proposed in the past in which the pseudostatic approach is calibrated to a particular level of slope performance, which is represented by the earthquake-induced displacement.

Following a similar approach, a procedure has been recently developed by Rampello et al. (2008), in which the horizontal seismic coefficient k and the corresponding safety factor F_S are evaluated using an equivalence with the results of a parametric application of the displacement method, as proposed originally by Newmark (1965). In the procedure, the seismic coefficient is expressed as a function of the maximum acceleration of the slide mass (k_{max}), the ratio of slope resistance to peak demand k_y/k_{max} and the limit displacement d_y considered as tolerable for the slope. Slope stability is satisfied for values of the safety factor $F_S \geq 1.0$. The procedure can be thought of as being related to earthquake magnitudes $M = 4$–6.5, typical of Italian seismic events.

The permanent displacements induced by an acceleration time history can be expressed as a function of the ratio k_y/k_{max}.

The equivalent seismic coefficient can then be defined as a fraction η of the maximum acceleration a_{max} of the slide mass:

$$k = \eta \cdot k_{max} = \eta \cdot \frac{a_{max}}{g} \tag{1}$$

where η decreases as the displacement tolerable for the slope increases. In principle, the application of the displacement method should be performed using the equivalent accelerograms acting in the sliding mass, as obtained by one or bi-dimensional seismic response analyses (Seed and Martin, 1966; Chopra, 1966).

In the proposed approach a rigid soil behaviour was assumed for the slide mass, this implying that, until full mobilisation of shear strength, the acceleration distribution is uniform through the soil mass. Under this assumption, amplification effects are simply taken into account using amplification factors for subsoil profile S_S and ground surface topography S_T, as specified by technical recommendations or building codes (e.g.: EN 1998-5, D.M. 14.01.2008); in eq. (1) it is then $a_{max} = S_S \cdot S_T \cdot a_g$, with a_g being the maximum acceleration at the rigid outcrop.

Permanent displacement were evaluated through a Newmark-type integration of the portion of the accelerograms in excess of k_y. A total of 214 accelerograms were used belonging to 47 events recorded by 58 stations. They were divided in three groups according to the subsoil underlying the monitoring sites, as defined by Eurocode 8 and the Italian building code (EN 1998-5; D.M. 14.01.2008): rock or rock-like soil with shear wave velocity $V_S \geq 800$ m/s (A); dense granular and stiff cohesive soil with V_S = 360–800 m/s (B); medium to loose granular and medium stiff to soft cohesive soil with $V_S < 360$ m/s (C, D; E).

For each group of accelerograms, peak accelerations were scaled to values of a_{max} = 0.05, 0.15, 0.25 and 0.35 g, limiting the scale factors in the range 0.5 – 2. Earthquake-induced displacements were computed integrating twice the equation of relative motion for translational sliding, using critical acceleration values equal to 10 to 80 % of the maximum acceleration (k_y/k_{max} = 0.1 – 0.8). Permanent displacements computed for each subsoil class and for each level of acceleration were plotted as a function of the ratio k_y/k_{max} in a semi-logarithmic scale. An example is shown in Figure 12 for subsoil class B. Computed results were best-fitted using exponential relationships written in the form:

$$d = B \cdot e^{A \frac{k_y}{k_{max}}} \tag{2}$$

Assuming a log-normal distribution around the mean value, the 90th-percentile upper-bound displacements were obtained, their relationships being characterised by the same parameter A of the mean curves and by a value of $B_1 > B$.

At a constant critical acceleration ratio, permanent displacements induced by a given accelerogram, linearly depends on a_{max}. Then it is possible to account for eventual amplification in ground motion, as produced by seismic response, multiplying the coefficient B_1 by the amplification factors S_S and S_T, thus obtaining $B_2 = S_S \cdot S_T \cdot B_1$.

For a given threshold displacement d_y, the corresponding values of η can then be obtained by inverting eq. (2) with $d = d_y$ and $B = B_2$:

$$\eta = \frac{k_y}{k_{max}} = \frac{\ln\left(d_y / B_2\right)}{A} \tag{3}$$

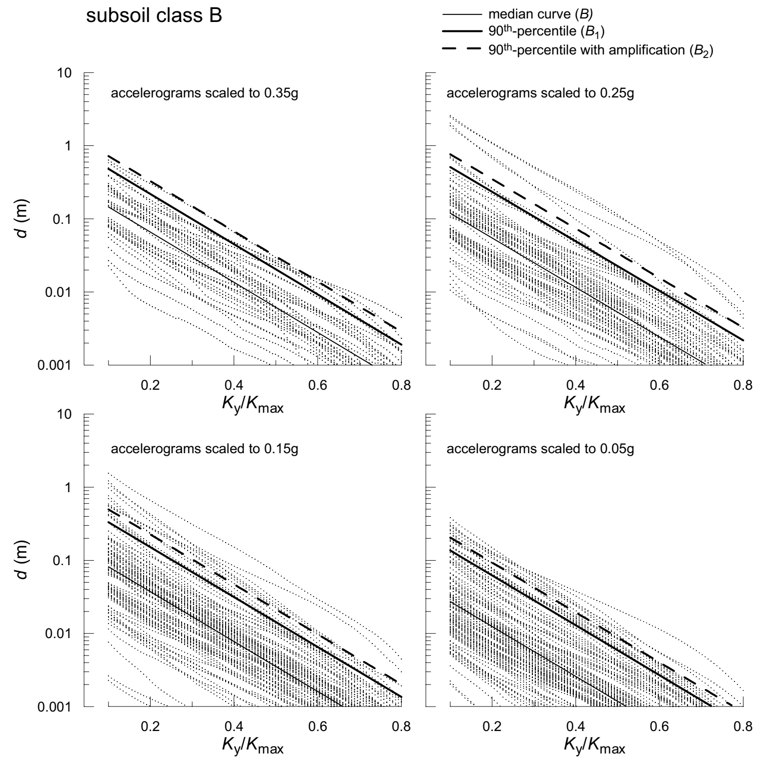

Figure 12. Permanent displacements computed using acceleration time histories monitored on class B subsoil (Rampello et al. 2008).

Values of η are given by Rampello and Callisto (2008) for threshold displacements of 15, 20 and 30 cm, corresponding to levels of damage from moderate to negligible (Idriss, 1985).

Ausilio et al. (2007b) developed a statistical model using a selection of accelerograms from database SISMA (Scasserra et al., 2008a) to compute the Newmark-type displacements. A significant reduction in the scatter of data set was observed once the displacements d were normalised by the product $a_{max} \cdot T_m \cdot D_{5\text{-}95}$ and plotted against the ration k_y/k_{max}. Considering the 90th-percentile upper-bound relationship for non-dimensional displacements, an expression for coefficient η′ was obtained by Ausilio et al. (2007b), defined as the ratio a_y/a_g (= k_y/k_g):

$$\eta' = \frac{a_y}{a_g} = \frac{1}{3.41} \cdot \left[-1.349 - \log\left(\frac{d_y}{a_g \cdot E\left[T_m \cdot D_{5-95}\right]} \right) \right] + 0.237 \tag{5}$$

in which the apex was added to distinguish this definition by that given in eq. (3), and $E[T_m \cdot D_{5\text{-}95}]$ is the expected value of statistic distribution of the product $T_m \cdot D_{5\text{-}95}$ considered as a random variable.

In this case also, ground motion amplification due to site effects can be accounted for using the amplification factors for subsoil profile S_S and ground surface topography S_T, as specified by technical recommendations or building codes (e.g.: EN 1998-5, D.M. 14.01.2008):

$$k = \eta' \cdot k_{\max} = \eta' \cdot \frac{a_{\max}}{g} = \eta' \cdot S_S S_T \frac{a_g}{g} \tag{6}$$

4.5 Influence of the deformability of the soil mass

In the procedures discussed above a rigid soil behaviour was assumed for the slide mass, this implying uniform spatial distribution of acceleration within the slope, with $a(t) = a_g(t)$. More realistic description of seismic performance of slopes can be obtained taking into account the deformable behaviour of the slide mass during ground motion. In this case, slope behaviour can be studied using the de-coupled approach, in which a preliminary analysis is carried out to assess the dynamic response of the slope, and then the resulting acceleration time histories are used to compute the permanent displacements with a rigid-block sliding analysis (Makdisi and Seed, 1978). The seismic response analyses can be carried out in one-dimensional (1D) or two-dimensional (2D) conditions, and the non linear soil behaviour can often be described through the equivalent linear approximation, that is known to yield a reasonable estimate of soil response at moderate levels of seismic intensity.

When a seismic response analysis is performed, the maximum acceleration $a_{\max}$ in eq. (1) should be intended as the maximum equivalent acceleration $a_{(eq)max}$.

A parametric study was carried out by Ausilio et al. (2007a) in which 1D seismic response analyses were carried for soil columns of height $H = 5 – 30$ m, these representing the depths of the sliding surface, and bedrock depths of 5 to 60 m. The 1D equivalent linear visco-elastic analyses were carried selecting 124 Italian accelerograms from database SISMA (Scasserra et al., 2008a) and assuming 21 different subsoil profiles, representing subsoil classes as identified by Eurocode 8 (EN 1998-5) and by Italian Building Code (D.M. 14.01.2008).

The Authors recognised that the ratio $\alpha = a_{eq(max)}/a_s$, plotted in Figure 13 against T_s/T_m, could be interpolated by a single relationship irrespective of the subsoil profile:

$$\alpha = \frac{a_{(eq)\max}}{a_s} = \frac{a_{(eq)\max}}{a_g \cdot S_{NL}} = 0.4199 \cdot \left(\frac{T_s}{T_m}\right)^{-0.815} \tag{7}$$

The ratio between the wavelengths characteristics of the seismic event and the thickness of soil column decreases with the ratio T_s/T_m; as a consequence, the inertial forces reduce substantially yielding to equivalent accelerations lower than those computed at the ground surface. Then, values of $a_{(eq)max}/a_s$ lower than unity are due to vertical incoherence of ground motion, implicitly accounted for into $a_{(eq)max}$.

Using the results of the analyses mentioned above, Ausilio et al. (2007b) included the influence of soil deformability in the estimate of the equivalent seismic coefficient. In this case, the maximum acceleration $a_{\max}$ in eq. (1) is properly intended as the maximum equivalent acceleration $a_{(eq)max}$ of the slide mass, expressed in the form: $a_{(eq)max} = \alpha \cdot a_s = \alpha \cdot S_{NL} \cdot a_g$. To maintain a simplified and conservative character of the procedure, the Authors assumed a constant value of $\alpha = 0.74$ computed for $T_s/T_m = 0.5$. Under these assumption, starting from eq. (5) they proposed a reduction factor η'' that include ground motion amplification produced by response analysis:

$$\eta'' = \frac{a_y}{a_g} = \frac{0.74 \cdot S_{NL}}{3.41} \cdot \left[-1.349 - \log\left(\frac{d_y}{0.74 \cdot S_{NL} \cdot a_g \cdot E\left[T_m \cdot D_{5-95}\right]}\right)\right] + 0.237 \tag{8}$$

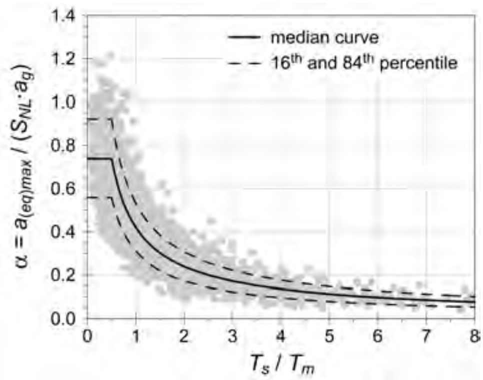

Figure 13. Ratio of $a_{(eq)max}/a_s$ versus the ratio between the fundamental period of slope and the mean period of input motion (Ausilio et al. 2007a).

In this case, the equivalent seismic coefficient can be written in the form:

$$k = \eta'' \cdot k_{max} = \eta'' \cdot \frac{a_{max}}{g} = \eta'' \cdot S_T \frac{a_g}{g} \tag{9}$$

Two-dimensional ground response analyses were also carried out to evaluate the influence of soil deformability on the seismic response of slopes and earth dams. The analyses were performed assuming the soil to behave as a linear elastic material, and assuming rotational sliding mechanisms.

The dynamic linear-elastic seismic response analyses were carried out investigating the influence of some key factors, such as the slope height and inclination, the soil stiffness E' and the frequency content of the seismic input.

Time histories of the equivalent seismic coefficient $k_{hS}(t) = a_{eq}(t)/g$, related to specified soil volumes, were evaluated as mentioned above for two-dimensional conditions.

In general, the maximum values of the equivalent seismic coefficient k_{hS} computed for given soil volumes were seen to be substantially lower than the ratio PGA/g irrespective of slope geometry and of depth and length of the sliding surfaces, at least for values of E' typical of soil deposits.

Influence of spatial incoherence of ground motion was also studied for two earth dams through coupled, effective stress dynamic analyses. For Camastra earth dam seven accelerograms, representative of possible seismic scenarios associated to different return periods T_R, were selected from the European Strong Ground Motion Database (Bilotta et al., 2008). For El Infiernillo earth dam, analyses were carried out using accelerograms from three earthquakes actually occurred at the dam site. The occurrence of asynchronous motion was detected plotting the contours of total stresses and instantaneous accelerations (Sica et al., 2008; Sica and Pagano, 2009).

The ratio $a_{hS,equiv}(t)/PGA$ computed in the two dams for all the analysed surfaces and seismic events decreases with increasing size of slide mass, as quantified by the maximum depth of the surface and by its length. Also, computed values of k_{hS} increase with peak input acceleration, but with a decreasing rate. This occurrence can be attributed to plastic soil behaviour within the dam embankment for high intensity earthquakes.

Sliding mechanisms parallel to the ground surface were also studied using the decoupled approach, for several idealised clayey slopes and a number of different seismic inputs. In the analyses, performed with the finite element method using the code QUAKE, an equivalent linear visco-elastic soil model was used in which the laws for decay of the secant shear modulus and for the increase of the damping ratio were taken from Vucetic and Dobry (1991) and a plasticity index I_P = 25%.

The ground surface of the analysed slopes had a constant inclination α = 10° and 20° with a vertical distance from a horizontal bedrock varying from 15 to about 120 m. The computed displacements were related to the soil stiffness, to the size and location of the sliding mass within the slope, and to the properties of the seismic input, to support the prediction of earthquake-induced displacements for this category of sliding mechanisms (Rampello et al., 2009).

It was shown that for soft soil deposits the use of equivalent accelerograms globally results in lower displacements than those computed using the input accelerograms. For stiff soils, larger displacements than the ones computed using the input accelerogram can be obtained only for shallow and short sliding mechanisms, that is for small volumes of the sliding mass.

4.6 Advanced numerical analyses

Coupled effective stress FE analyses were carried out to study the seismic behaviour of an ideal slope, in a clayey soil deposit, with a height H = 20 m. The two-surface elasto-plastic constitutive model MSS, that is capable of describing the main features of soil behaviour under cyclic loading, was used in the analyses. Two sets of analyses were carried out, the first referring to a soft clay deposit, while the second to a stiff and overconsolidated clay deposit. A parametric study was performed for evaluating the influence of the overconsolidation ratio, the bedrock depth, the peak acceleration of input ground motion and the geometry of the mesh.

The main advantages (Amorosi et al., 2009b) of coupled effective stress FE dynamic analyses of a slope are represented by the possibility of: *i*) describing with accuracy shear stiffness decay with shear strain and the related hysteretic damping; *ii*) limiting the amount of the fictitious viscous damping usually added in numerical analyses (1-2 %); *iii*) predicting plastic strain accumulation and excess pore water pressure build-up during earthquake loading; *iv*) describing the dissipation of the excess pore water pressures after the end of the seismic action; (*v*) describing the evolution of strain and displacement fields during and after the earthquake.

5 DISCUSSION

The objective of developing a simplified procedure for a reliable evaluation of the seismic performance of a slope was pursued by means of a number of tools, each referring essentially to a parametric evaluation of the permanent displacements of simplified slope schemes. Firstly, a database of Italian strong motion database was developed together with a site and source databank, to provide high quality Italian strong motion records with consistent and reliable associated seismic parameters.

Using this database, a number of empirical relationships were developed for a preliminary estimate of earthquake-induced slope displacements, and a code was developed for evaluating slope displacements induced by seismic shaking along sliding surfaces of general shape. In this code, the mixed roto-translational mechanism is described via a multi-block model and account is taken for pore water pressure build-up and mass transfer occurring during motion. The code was checked by back-analysing the landslides reactivated by Irpinia earthquake of 1980.

Then, a parametric Newark integration of the whole database was performed with the specific intent to evaluate, for a given seismic performance, the seismic coefficient to use in a pseudo-static calculation. This study was timely, as it was possible to include some results in a new version of the Italian building code (D.M. 14.01.2008).

The influence of soil deformability on slope response to earthquake loading was studied through one and two-dimensional equivalent linear visco-elastic analyses and with a limited number of two-dimensional fully coupled stress dynamic analyses. The analyses considered simple idealised slopes and a few real earth dams.

The soil deformability brings about two somewhat contrasting effects, namely the amplification of the base motion and the non uniform spatial distribution of the inertia forces. The latter effect may lead to a substantial reduction of these forces, and can prevail when the wave length of seismic motion become comparable or smaller than the size of the sliding mass, and this may happen in large deformable volumes of soil, and for high-frequency seismic signals. Conversely, ground motion amplification prevails for small and stiff sliding masses excited by low-frequency seismic signals, and near the top of natural slopes or the crest of earth embankments, because of multiple reflection of the seismic waves.

To sum up, it may be stated that most of the research objectives were attained and that the main factors affecting the slope response to earthquake loading were recognised, their influence being isolated and evaluated.

6 VISIONS AND DEVELOPMENTS

The present research activity was mainly addressed to the study of the seismic behaviour of natural slopes. Ideal schemes of slopes were studied and typical geometries were considered for the sliding mechanisms and the depth of the bedrock, while the stiffness and strength properties were typical of natural soils. A number of well documented case histories were also back-analysed. The main characters of the seismic behaviour of the slopes were highlighted, and simplified procedures for the study of their seismic stability were proposed, including a relationship between the pseudo-static seismic coefficients and the required seismic performance, quantified by the final permanent displacements.

This research activity finds a natural extension in the study of the seismic behaviour of artificial earth structure, and specifically of earth dams. These structure are typically made of compacted soil, characterised by a large stiffness, and may produce significant amplification of the inertial forces, because of the multiple reflection and focalisation of seismic waves near the crest. The research should distinguish between different types of earth dams: homogeneous, zoned with vertical impervious nucleus, and zoned with inclined impervious nucleus. The influence of the bedrock depth needs also be investigated, as in many cases this depth is not well documented and a degree of subjectivity does exist on the choice of the boundary to which the seismic motion should be applied.

Similarly to the case of the natural slopes, this study may be carried out using methods of analyses of increasing complexity, including the simple pseudo-static analysis. However, the singularity and importance of the earth dams calls for a larger role of the advanced numerical analyses. A number of centrifuge tests might b also devised, in which the above different categories could be reproduced to provide the experimental evidence for the calibration of the analyses. The results of this research might provide guidance for the design of new earth dams, but would also assist in the seismic evaluation of the many existing earth dams that were designed without accounting for earthquake loading.

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SUB-THEME 6.4: DEEP FOUNDATIONS

1 INTRODUCTION

This research deals with the behaviour of deep foundations under earthquake excitations, with particular reference to the kinematic interaction phenomenon between the soil and the pile invested by the seismic waves.
During strong earthquakes foundation piles tend to significantly modify soil deformations, since they oppose to the seismic motion of the subsoil. Further because of the interplay between soil and piles the motion at the base of the superstructure can significantly deviate from the free-field motion, and the piles be are subjected to additional bending, axial and shearing stresses. The bending moments, usually referred to as “kinematic” ones, may be very important even in the absence of the superstructure.
In the past the kinematic interaction between soil and piles has been studied by many researchers. In spite of the big effort on such a topic, the complex problem still need to be investigated and well consolidated in the research field. As a matter of fact kinematic interaction has been rarely accounted for in practical design. Modern seismic codes have, however, acknowledged the importance of kinematic interaction and demand piles to be designed also accounting for soil deformations arising from the passage of seismic waves. Eurocode 8 (EN 1998-5), for example, suggests that kinematic effects should be taken into account when all the following conditions simultaneously exist: 1) seismicity of the area is moderate or high (specifying that moderate or high-seismicity areas are characterized by a peak ground acceleration $a_g \cdot S > 0.1g$, where a_g is the design ground acceleration on type A subsoil and S is the soil factor); 2) subsoil type is D or worse, characterized by sharply different shear moduli between consecutive layers; 3) the importance of the superstructure is of III or IV class (e.g. schools, hospitals, fire stations, power plants, etc). The recent Italian building code (D.M. 14.1.2008) provides quite similar indications concerning the kinematic bending moments in piles.
In this context the present ReLUIS research line 6.4 has been planned, involving five Italian research groups and collaborations with other national and international experts and foreign seismic research structures. The main final goal of research Line 6.4 was to find out innovative elements for a new regulation on seismic design of piled foundations with emphasis on kinematic interaction. The evidence collected from previous studies and the results of the present research, even far from providing a definitive answer to the question, have allowed the clear individuation of the factors controlling the phenomenon at hand, the evaluation of pile response for reference soil-pile configurations, some important indications for pile seismic design, and mainly what has still to be done for this issue.

2 BACKGROUND AND MOTIVATION

2.1 Background on analytical aspects

The kinematic interaction phenomenon has been studied by many researchers by means of different approaches and analytical tools. The methods for the analysis of kinematic soil-pile interaction may be classified into three groups: <u>continuum approaches</u> (FEM, BEM), <u>Winkler methods</u> (BDWF), <u>simplified formulations</u>.

In continuum approaches soil, pile and superstructure are modelled as a whole. The soil-pile geometry typically is modelled in 3-D and discretized by F.E.M. or B.E.M. techniques (Kaynia 1996, Kimura e Zhang, 2000; Zhang et al. 2000; Zhang e Kimura 2002; Wu e Finn, 1997; Bentley & El Naggar, 2000; Finn e Fujita, 2002). Soil is usually modelled by elastoplastic models. A few works in literature adopt models of the so-called Advanced Plasticity to properly reproduce soil response under cyclic loading conditions, and some work describes the possibility to simulate gap at the pile-soil interface under strong motion events (Maheshwari et al., 2004). Among numerical approaches, the finite element method (Wu and Finn, 1997; Cai et al., 2000; Kimura and Zhang, 2000; Maheshwari et al., 2004) provides a powerful and versatile technique, since some important effects such as soil nonlinearity and heterogeneity may be directly accounted for. Nevertheless, this method is generally very expensive from a computational viewpoint, since it requires suitable boundaries conditions being introduced to simulate the radiation damping effect. Worth mentioning is the simplified continuum approach developed by Wu & Finn (1997) in which vertically propagating shear waves are modelled disregarding seismic induced deformations in the vertical direction and along the normal to the direction of shaking (quasi 3-D approach).
A more attractive approach is represented by the boundary element technique (Kaynia and Kausel, 1982; Mamoon and Banerjee, 1990; Cairo and Dente, 2007). It only needs the discretization of the interfaces and permits the condition of wave propagation towards infinity to be automatically satisfied. The ReLUIS research group from the University of Calabria developed a code called SASP (Seismic Analysis of Single Piles) based on BEM solution for the analysis of single piles and pile groups subjected to vertical loading, under static and dynamic conditions. The method makes use of the closed-form stiffness matrices derived by Kausel & Roësset (1981) to simulate the response of layered soils. The analysis is performed in the frequency domain under the assumption of soil linearity.
The methods based on the Winkler foundation model assume that the pile is modelled as a linearly elastic beam, with length L and diameter d, discretized into segments connected to the surrounding soil by springs and dashpots, which provide the interaction forces in the lateral direction (Figure 14). Springs represent soil stiffness and dashpots soil damping due to radiation and hysteretic energy dissipation. As a first approximation, the spring stiffness k may be considered to be frequency-independent and expressed as a multiple of the local soil Young's modulus Es (Kavvadas and Gazetas, 1993). The dashpot coefficient c represents both material and radiation damping. The latter one may be computed using the analogy with one-dimensional wave propagation in an elastic prismatic rod of semi-infinite extent (Gazetas and Dobry, 1984).

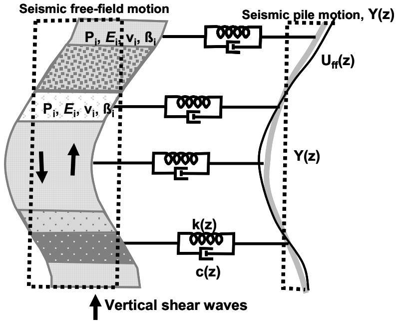

Figure 14. Beam on Dynamic Winkler Foundation model (BDWF).

Some results obtained using the Winkler approach (BDWF), as proposed by Mylonakis et al. (1997), proved to be quite accurate, computationally time saving and suitable for performing extensive parametric analyses (to investigate the role of both the variables of the system and the earthquake motion characteristics). They further allow nonlinear behaviour of the soil to be easily incorporated if solution is envisaged in the time domain (Boulanger et al., 1999; El Naggar et al., 2005; Maheshwari and Watanabe, 2006; Cairo et al., 2008).
Simplified formulations are closed-form expressions (Dobry and O'Rourke, 1983; Nikolaou and Gazetas, 1997; Mylonakis, 2001; Nikolaou et al., 2001) available in literature for approximately computing the maximum steady-state bending moment at the interface between two layers. These approaches have been derived by modelling the pile as a beam on a Winkler foundation and are based on the following simplified assumptions: each soil layer is homogeneous, isotropic and linearly elastic; the soil is subjected to a uniform static shear stress field; the pile behaves as a linear-elastic semi-infinite beam; the embedded length of the pile in each layer is greater than the so-called "active length".

2.2 Background on experimental activity

Experimental activities on pile models have been conducted in the past, mainly by shaking table apparatus (1*g*), with the aim to investigate seismic behaviour of soil-pile systems. Most of the experimental tests focused on liquefaction problems (Towhata et al., 1999; Dungca et al, 2006; Yao et al, 2004). Significant tests have also been carried out for understanding soil-pile-superstructure interaction (SSPSI) in cohesive soils and in non-liquefiable sand.
In particular Meymand (1998) carried out a series of 1*g* tests on model piles in soft clay, with the main goals of providing insight into a variety of SSPSI topics, and generating a data set with which to calibrate an advanced SSPSI analysis tool being developed at U.C. Berkeley. The tests on single piles were seen to respond with components of inertial and kinematic interaction, though the inertial components produced upper bound bending moments. These results suggest that developing pile demands from consideration of inertial loading only may be acceptable for cases where site stiffness contrasts or ground failure (lateral spreading) do not exert significant soil loads or deformations on the piles.
Recently Tokimatsu and Suzuki (2009) performed a huge experimental study utilising the big shaking table at E-Defense, Japan. The tests were conducted on soil-pile-structure models with dry and saturated sands for examining and quantifying the effects of inertial and kinematic forces. These main findings can be outlined: *i)* if the natural period of the structure is less than that of the ground, the kinematic force tends to be in phase with the inertial force, increasing the stress in piles; *ii)* if the natural period of the structure is greater than that of the ground, the kinematic and inertial forces tend to be out of phase, restraining the pile stress from increasing; *iii)* the maximum pile stress may be estimated by applying both the inertial and kinematic forces on the pile at the same time, if the natural period of the structure is less than that of the ground; it may be estimated as the square root of the sum of the squares of the two moments estimated by applying the inertial and kinematic forces on the pile separately, if the natural period of the structure is greater than that of the ground.

3 RESEARCH STRUCTURE

The general objective of the project was to reach a level of knowledge suitable for identifying elements and suggestions to be introduced in the national normative on the seismic behaviour of deep foundations with special regard to kinematical interaction. At this aim the following partial objectives were envisaged:

(a) elaboration of the state of art on the kinematical interaction;
(b) evaluation of the acceptability of the hypothesis of foundation fixed to the base;
(c) selection of the criteria for identifying the cases in which it is necessary to take into account the kinematical interaction for the calculation of the stresses induced on piles;
(d) definition of computational procedures of the stresses due to the kinematical interaction, characterized by different degrees of complexity;
(e) evaluation of the feasibility of an experimental testing activity on physical models, with the aim of validating the results acquired through the numerical research.

The following five research Units (RU) were involved: University of Sannio (alias SANNIO), University of Basilicata (alias UNIBAS); University of Calabria (alias UNICAL); University of Catania (alias UNICT) and Second University of Napoli (alias SUN). Further, other RU from the University Parthenope (I), the University of Bristol (UK) and the University of Patras (G) effectively collaborated to the research.

The above objectives have been pursued through the following research activities:

(a) the layout of state of the art report;
(b) the identification of different computational procedures, with different degree of complexity of both the physical-mathematical model and the computational algorithm;
(c) the identification of a wide range of sample cases, characterised by different characteristic parameters of the exciting waveform, stratigraphical features and geometry of the foundation system (from the single pile to the group of n-piles, with a wide variability of the main geometrical features);
(d) the execution of the numerical analyses of the sample cases, by means of the different computational procedures previously selected;
(e) the interpretation and synthesis of the results, with the identification of typical behaviours, for the discrete range of values of the significant model parameters;
(f) a first experimentation on a physical model, by means of a shaking table apparatus.
(g) the identification of elements to be introduced in the seismic normative.

The following three documents have been produced, for illustrating the whole research activities:

A. Report on the activity and results of each Research Unit;
B. Publication of all the studied cases, which have been effective for the identification of typical behaviours;
C. Elements for a technical normative on the project of deep foundations in seismic areas.

4 MAIN RESULTS

During the first year the research Units involved in Line 6.4 have carried out a wide study on the knowledge available on this specific topic. Literature on kinematic interaction was found to be often fragmentary and sometimes conflicting and incoherent. The second year of this project has been mainly dedicated to the development of "reference" numerical models for the dynamic analysis of kinematic interaction effects for piled foundations. Analyses have been carried out by models with increasing degree of complexity: P-y curves, BDWF (*beam on dynamic Winkler foundation*) models (both linear and non-linear), 3D models based on the finite element method and the boundary element method. Non-linear soil response has been simulated at a very simple level by imposing G=G(γ) and D=D((γ) material curves (G = shear modulus, D = soil damping ratio, γ= shear strain) or at a more sophisticated level by means of models capable to consider hysteretic soil behaviour.

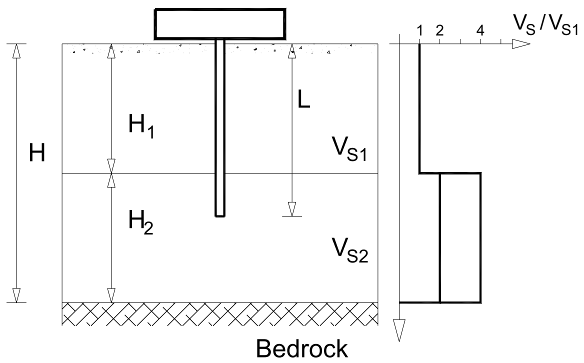

Figure 15. Foundation scheme adopted as basis by the five research Units.

Several particular soil-pile configurations have been studied by all the RU ('general cases'). In addition each RU has independently developed the study of particular cases. Most of the research has been dedicated to the response of piled foundations (single piles as well as pile groups) embedded in an ideal subsoil consisting of two layers separated by an interface at depth H_1 and underlain by a rigid base at depth H (see Figure 15). While UNISANNIO, for example, has performed a parametric study to assess the influence of the depth H of the bedrock, SUN has investigated the role of the interface depth H_1 on the kinematic response of both single piles and pile groups. At the same time the effect of non linear soil response has been investigated by UNICT , by means of a pseudostatic approach based on the P-y curve method, and by UNISANNIO and UNICAL by performing linear equivalent analyses. In addition to this a computer code capable to account for the non-linear soil response has been developed by UNICAL (Cairo et al. 2008). This is based on a dynamic Winkler model (BNWF = beam on a Non linear Winkler Foundation) and represent an enhancement of the original version developed by Conte and Dente in 1988. One out of the key features of this code is the use of P-y curves based on the classical relationship by Ramberg-Osgood.
Summarising, elastic analyses have been performed for single piles and for pile groups; the bending moments have been investigated both at the interface between soil layers and at the pile head; the results by the different analysis tools have been compared, both considering the linear and non linear soil behaviour. The detailed illustration of the results is published in the Final Scientific Reports supplied by the Research Units; here only a list of the numerous case studied by means of elastic analysis is given in Table 1.
In the third year an experimental research has been carried out by means of the shaking table at BLADE (Bristol Laboratory for Advanced Dynamics Engineering) of the University of Bristol, thanks to the support of the Universities of Sannio and Calabria (Italy), and the collaboration with the same University of Bristol (UK) and the University of Patras (Greece). Laboratory tests were carried out on a pile model, for three configurations of subsoil (made of different granular materials and even rubber) pluviated inside a special container (*shear stack*, Figure 16). The shear stack was installed on the earthquake simulator, and subjected to three different Italian earthquake. Five different pile boundary conditions were tested (pile head free and fixed against rotation, with or without a superstructure). The repeatability of the test was checked, and the effects of input motion frequency content, soil stiffness contrast, and interaction between inertial and kinematic effects were successfully investigated.

Table 1. Linear elastic analyses.

RU	Method of analysis - computer code	H (m)	H1 (m)	L (m)	V_{S1} (m/s)	V_{S2} / V_{S1}	D(%)	No. of seismic input motions*	Foun-dation type
SANNIO	BDWF-SPIAB	30-(21)[1]	5-10-15-19	20	100	2-3-4	2 10	17	Single pile
		30-(21)[2]	5-15	20	150	2-2.67-4	10	17	Single pile
	3D Continuum dyn. analysis VERSAT P3D	30	5-10-15-19	20	100	4	10	3	Single pile
	3D Continuum dyn. analysis ABAQUS	30	15	20	100	4	2	19	Single pile
UNICT	Pseudostatic P-y transfer curve method	30	15	30	$E_P/E_1$5 000	0.58-3		3	Single pile Pile in a group
	3D Continuum dyn. analysis SAP	30 24	15 8	20 12	100 80	2-3-4 2-4	2-10 10	1 3	Single pile
UNIBAS	BDWF	30	5-10-15-17-19	20	50-100	2-4	10	1	Single pile
		30	15	20	100	2	10	16	Single pile
SUN	3D Continuum dyn. anal. VERSAT P3D	30	5-10-12-15-17-19 10	20 5-9-10-20	50-100 100	2-4 4	10	3	Single pile and pile groups
		30	5	6-10-20	100	4	10	4	Single pile
UNICAL	3D Continuum dyn. anal. SASP	30	15	20	100	2-3-4	10	3	Single pile
	BDWF	30	5-10-15-19	20	100 (150)[3]	2-3-4	10	17	Single pile

[1]For H=21 m the analyses have been performed only for H_1=15 m and D=10%
[2]For H=21 m the analyses have been performed only for H_1=15 m
[3]For V_{S1}=150m/s, V_{S2}/V_{S1} ratios were 2-2.67 and 4
* For the list of the utilised accelerograms, see the document B produced by the L.R. 6.4

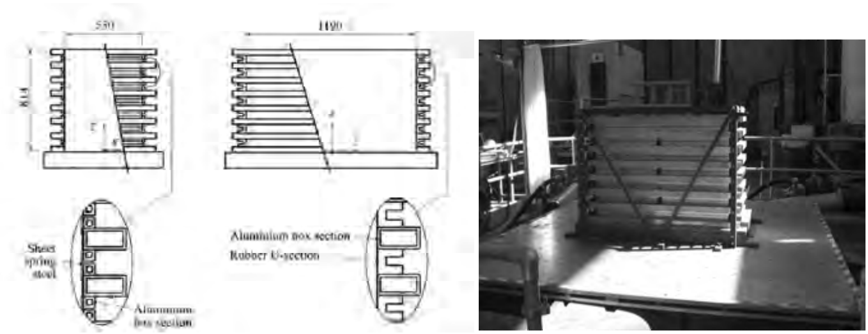

Figure 16. Shear Stack at BLADE (Bristol, UK).

5 DISCUSSION

The main objective of the research project was the evaluation of the kinematic interaction effects on pile foundations, with the final aim to find out suggestions for recent Codes. In order to achieve the goals, analytical approaches of different complexity levels have been applied. The wide set of Italian strong motion accelerograms analysed and collected in the database SISMA have been utilised as input motion.

First extensive analyses have been performed by the BDWF (Beam on Dynamic Winkler Foundations) methods, which are the most common tools utilised in the research field too, for investigating the linear response of the soil-pile systems. The same approach has been utilised for investigating the effect of soil non linearity on kinematic interaction effects. An important upgrading has been achieved in the application of BDWF methods, defining the proper range of soil parameter (stiffness and damping) values for analysing the linear response at low strain levels, or the non linear (or the simpler equivalent linear) response of soils at higher strain levels. The results obtained by different BDWF tools utilised by the involved research Units are in good agreement, and appear to provide good indications for a practical, even if conservative, pile design.

The extensive analyses confirmed the significant role of the following variables on the amount of the induced pile bending moments:

- stiffness contrast between two consecutive soil layers;
- depth of the interface between layers with stiffness contrast, with respect to the active length of the pile;
- boundary conditions at the head of the piles;
- whole subsoil stiffness (subsoil classification according to recent seismic Codes);

Their role has been also enhanced by the results of the experimental tests performed by the shaking table apparatus at BLADE of the University of Bristol.

The main conclusion that can be drawn by these applications is that kinematic interaction effects can be significant in particular conditions, but not dramatic as it could appear on the basis of the oversimplified formulas proposed in the past to evaluate the maximum bending moments. In fact, the amount of the effects does not depend only on the soil-pile configuration (i.e. the variables listed before); it has been demonstrated that the effects of the interaction strongly depend on the coupling between the input motion frequency content and the subsoil main frequency; hence this new variable has to be accounted for in analysing the induced pile bending moments. From the previous considerations, it can be derived that the kinematic effects can not be properly determined on the basis of the pile-soil system and the maximum expected input motion acceleration, unless it is accepted that conservative criteria would be adopted in the design. As regards soil non linearity effect, it appeared to be both beneficial or detrimental for pile bending response: the influence of non linearity on the variation of the relative stiffness between consecutive soil layers plays a fundamental role. Observations and suggestions on these aspects have been summarised in “Product C” document produced by ReLUIS L.R. 6.4 at the end of the Project; an interesting proposal for Code design procedure has been outlined in Cairo et al. (2009).

Important indications relative to the asynchronous effects induced at the pile head by kinematic interaction and superstructure inertial actions have been provided by both numerical analyses and the laboratory tests performed at the BLADE on a single pile model with a simple oscillator resting on it.

Several applications of more advanced numerical approaches (based on FEM and BEM techniques) confirm the results obtained by BDWF methods, if the simple assumptions there adopted are still saved (linear or non linear behaviour of the soil, linear behaviour of the pile).

In conclusion, it may be stated that most of the research objectives have been attained, and that the performed studies also represent a valid reference for addressing the research in the next future.

6 VISIONS AND DEVELOPMENTS

The research activity that has been performed to date has been mainly addressed to the study of the seismic behaviour of piles, according to the most consolidated tools for studying soil-structure interaction under earthquake excitations. Then the obtained results have been substantially validated by several more advanced numerical analyses and by some experimental tests on model piles. The research has certainly provided good results, since they allow to state some important conclusions relative to the main factors influencing pile bending response, in the hypothesis usually assumed for the soil-pile system.

First attempts have been made to investigate more complex soil-pile configurations, including soil plasticity and pile yielding. In the next future, the role of soil plasticity, and its beneficial or detrimental effects on pile bending moments have to be accurately studied, through more refined numerical analyses implementing advanced soil constitutive models. At the same time it would be interesting to investigate the response of the soil-pile system after the yielding of the structural element, and its effects on the superstructure behaviour, in order to consider the eventual removal of the pile elastic response required in most recent seismic Codes.

The behaviour at the pile head also need to be more deeply investigated, in order to confirm or not the delay between the maximum kinematic and inertial induced effects; this aspect has to be confirmed also through further specifically planned experimental tests. The results on this question could have relevant effects on pile design, since actually all the Code impose to simply sum the maximum effects, probably producing over-conservative design.

In conclusion, the three-year research activity has provided significant results, with effective suggestions for the Codes, and encourages further and more addressed activities both in the theoretical and in the experimental research field.

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TECHNOLOGIES FOR THE SEISMIC ISOLATION AND CONTROL OF STRUCTURES AND INFRASTRUCTURES

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1 INTRODUCTION

The technologies for the control of the seismic structural vibrations have been greatly developed in the last decades, with several applications all over the world. Their effectiveness is proved by laboratory and field tests as well as by their excellent performances during recent earthquakes, especially in Japan. It is nowadays acknowledged that they allow structures to achieve higher safety levels than conventional structures, even when designed by using modern seismic criteria. Their diffusion, if favoured by reliable and optimised norms, would permit to achieve a sensible reduction of the seismic risk (Soong and Spencer, 2002; Kelly, 2004; Whittaker and Constantinou, 2004).
In Italy the development and the practical exploitation of the advantages of these technologies has been slowed down by the lack of a specific norm. The enforcement of the Ordinance 3274 (PCdM, 2003), including two chapters on the seismic isolation of buildings and bridges, represented a turning point for their practical use, removing many of the difficulties (from both the bureaucratic and technical points of view) usually encountered by the designers wishing to adopt these technologies in new or existing constructions. However, at the time when the ReLUIS Executive Project started (November 2005), there were still several problems which needed further studies, to make applications more and more reliable and easy. Some of them have been dealt with and solved by the Research Units of Project Line No. 7, who also gave their contribution toward the issuing of the relevant parts of the New Technical Regulations for Constructions (Min. Infr., 2008) and Commentary (Min. Infr., 2009), so that today many new strategic buildings in Italy are provided with seismic isolation and/or supplemental energy dissipation systems.

2 BACKGROUND AND MOTIVATION

The various aspects of the wide subject of passive and semi-active seismic control necessarily requires a synthetic description of the starting point for each of them.
To simplify the problem a classification is needed referring to:

- Control technique: Seismic isolation, Energy dissipation, Tuned masses, Semi-active control;
- Type of devices: elastomeric isolators, sliding isolators, viscous/visco-elastic/hysteretical/recentring SMA/magnetorheological and wire-rope devices;
- Type of structure: ordinary r.c. or steel buildings, common masonry or monumental buildings (churches, palaces), girder bridges, arch bridges, other structures (light structures, pre-fabricated structures, etc.);

- Type of design: design of a new structure, retrofit of an existing structure;
- Type of action: earthquakes with ordinary characteristics, near-fault earthquakes.

At the starting time of the ReLUIS Executive Project, the most widely studied problems referred to the most common situations (seismic isolation or energy dissipation of r.c. buildings or girder bridges of medium dimensions), but the potentialities of the control systems were such that always more often higher security levels were supposed to be guaranteed through their applications, particularly in the most demanding projects.
More specifically, the development of codes for the application of control systems was initially mainly limited to the seismic isolation of buildings: USA, Japan, China, EU, Armenia and other countries, including Italy, issued seismic norms for the design and verification of buildings with seismic isolation (Martelli and Forni, 2003; ICC 2000; CEN-1998-1-1, 2003; CEN-1998-1-2, 2004: PCdM, 2005). However, in different works carried out both in Italy and abroad (Dolce and Santasiero, 2004a-b), it was verified that, in spite of several analogies, the different codes might lead to different results, and condition the application of seismic isolation in the different countries. It was clear that wide safety margins still exist, differently used in the various codes, which could be further optimized, also in the Italian norms. Aspects of particular relevance were the hypotheses and procedures for the of application of simplified models and linear and/or static analyses of structures equipped with passive control systems, with linear and nonlinear behaviour (shape of the equivalent load, level and effects of damping, *q*-factor for the verification of the structural elements belonging to different typologies). Their influence on the response of the structural system and, on the other hand, the conditioning imposed on the control system's design by the morphology of the structure (building or bridge), thus needed to be investigated. Also, it was well known the harmful influence of structural irregularities in plan or elevation on the seismic response of a conventional structure, subjected to large inelastic deformations, while how seismic isolation is able to attenuate such an influence was less investigated.
The norms concerning the seismic isolation of bridges are less frequent and cannot be simply derived as an extension of the criteria and prescriptions valid for buildings, due to the differences in the structural system, the number of devices and the environmental conditions. Ordinance 3431 (PCdM, 2005) included a chapter completely dedicated to the seismic isolation of bridges, and a significant number of numerical investigations on simplified models of girder bridges where already at that time available, but still few were the through analyses on actual more complex real structures, allowing a complete evaluation of the role and engagement of each installed device. There was also a lack of experimental activities on large scale structural models.
Furthermore, norms regarding isolation often did not take into account the way to experimentally verify the capacities of the devices. And this notwithstanding the critical role of the devices. Still today, EC8 refers to a CEN code in course of development (CEN-TC340, 2004). While Ordinance 3431 (PCdM, 2005) defined how to verify the different devices from a theoretical and experimental point of view (qualification and acceptance), the procedures certainly needed a revaluation, on the base of the results of experimental investigations and accurate comparisons. Regarding the most common isolation technology for buildings, i.e. the one based on elastomeric isolators, some aspects such as behaviour under tensile stress, instability, sensibility to temperature, needed to be further investigated and solved for the correct inclusion in the norms.
The application of energy dissipation systems was been dealt in a quite different way: although most recent codes explicitly allow their use (PCdM 2003; PCdM 2005; CEN-TC340, 2004), mainly guidelines and applicative manuals had been developed. An exception is given by FEMA guidelines (FEMA 273 and 274, 1997) and prestandard (FEMA–ASCE

356, 2000), which tried to indicate simple and easily applicable design procedures. However, the different behaviours that the currently available devices (viscous, visco-elastic, hysteretical, non-linear recentring) are able to provide, require different design methods (values and distributions of stiffness and strength) and analysis procedures (elastic, non-linear static, non-linear dynamic). A real simplification of these methods was considered fundamental for the formulation of a reliable and effective code favouring the adoption and diffusion of the energy dissipation strategy.

The tuned mass systems, which are undoubtedly effective from a theoretical point of view, had interesting potentialities to be applied where the mass of the system was already existing as, for example, in the case of water tanks placed at the uppermost floor of a building. The efficacy of the system when its characteristics are not perfectly stable in time was a critical aspect for a correct application. The possibility of a semi-active control could significantly attenuate the sensibility to variations of such characteristics. It is also interesting the possibility of an improvement of the isolated buildings' behaviour, if they are equipped with tuned masses, in case of anomalous actions. This was proposed in the past, but never actually verified by an *ad hoc* experimentation.

The need to determine the optimal performance of the various devices and the wide availability of electrical measurement and control systems have induced researchers to single out control strategies and technologies consisting in purely mechanical devices, called passive, interacting with electronic measurement and feedback systems, with the aim of improving the performance of the control systems. From a theoretical point of view, the control devices characterized by an absolute freedom to actuate forces (active control systems), are able to provide a wide range of performances. With respect to these systems, in the last years there is a strong tendency towards devices which can operate as passive (i.e. reactive devices) but are equipped with electrical measurement and low energy actuation components, where the actuation is applied to change the mechanical characteristics (i.e. semi-active control devices) rather than to the development of real active forces.

One of the main characteristics of the semi-active devices is their intrinsic possibility to operate in a purely mechanical way (passive), so allowing the device to be effective in common "operational conditions" under dynamic excitations with an high probability of occurrence (roadway or railway traffic, wind actions), and to activate additional capabilities under exceptional actions (seismic excitation of high intensity). Besides, the equipment of the device with electronic measurement and low energy actuation systems allows the device to provide additional performances connected to the development of proper control algorithms and on-line identification algorithms. Indeed, the wide literature on control algorithms for active devices found application also in the semi-active control technology, through suitable revisions. However, the wide technical and scientific literature (mainly theoretical) had not yet produced recognized methodologies for the design of semi-active devices for the protection of the existing structures and in the construction of new structures. Therefore, particularly felt was the need to attain general formulations for the design of devices and for the evaluation of the seismic response of structures equipped with semi-active devices.

3 RESEARCH STRUCTURE

3.1 General objectives of Research Project Line No. 7

The general objectives of the ReLUIS Executive Research Project Line No.7 has been the advancement of knowledge on some specific aspects of design and operation of passive and semi-active systems for the control of seismic structural vibrations, as well as the

improvement and simplification of the design tools (regulations, guidelines, analysis and experimental verification methods), in order to make easier, more reliable and, wherever possible, economically convenient their application.
The concerned problems were relevant to the four main control techniques (seismic isolation, passive energy dissipation, tuned mass damping, semi-active control) as applied to conventional (R/C and steel buildings, bridges with R/C piers) or special structures (R/C precast buildings, monumental masonry buildings such as churches and palaces, light-weight constructions). Almost all of the currently used technologies have been considered, both the well established ones (rubber and sliding isolators, viscous, visco-elastic and hysteretic energy dissipating devices) and the most recently proposed (shape memory alloys, magneto-rheological, wire rope devices). The design problems dealt with were relevant to both new and existing constructions. For these latter, particular attention has been devoted to monumental buildings, deck bridges and the application of energy dissipating devices to R/C buildings. A specific topic dealt with has been the response of structures with seismic isolation or energy dissipation protection systems to near-fault earthquakes, with the aim of studying suitable provisions in the design of the devices or of the structure, as a second line of defence to guarantee adequate safety margin with respect to the complete collapse of the structural system.
With reference to seismic isolation, specific topics which have been considered are:

- elastomeric isolators (buckling, tension, thermal effects, experimental and theoretical verifications), low cost isolation, combination with sliders, application potential for monumental buildings,
- modes and methods of application to monumental buildings,
- simplified analysis methods (applicability and procedures), behaviour factor for linear analyses,
- deck-bridges with seismic isolation (3D effects, devices, design methods) [activity coordinated with ReLUIS Project Line No.3]
- attenuation of irregular configuration effects in buildings and bridges, even related to the technology choice,
- optimal structural configuration for the use of different technologies,
- evaluation of the cost/benefit ratio when using the isolation technologies under study,
- effects of anomalous seismic actions and of their vertical component.

With reference to energy dissipation, specific topics which have been considered are:

- design criteria and simplified analysis methods (linear with behaviour factor, non linear – application potential and procedures),
- procedures for the experimental validation of the different device categories,
- application potential and procedures for pre-fabricated structures,
- effects of anomalous seismic actions and of their vertical component.

With reference to tuned mass damping, specific topics which have been considered are:

- application potential and procedures for variable mass systems, both in passive and semi-active control,
- design criteria and simplified analysis methods (behaviour factor for linear methods, non linear procedures),
- design and analysis methods of isolation – tuned mass combined systems.

With reference to semi-active control, specific topics which have been dealt with are:

- reliability in the long run of these technologies, functioning in passive control, functioning under service and seismic conditions, self-diagnosis and structural identification,
- behaviour analysis and evaluation of the effectiveness compared to passive control systems,
- design criteria and methods,
- experimental evaluation procedures of different device categories.

3.2 Organizational structure of Research Project Line No. 7

During the first two years of the three years duration project, the activities of ReLUIS Project Line No. 7 have been carried out by the following 12 Research Units (3 internal and 9 external to RELUIS):

	Institution	Responsible	Title of the research of the Unit
		ReLUIS	
R1	UNIBAS - University of Basilicata	F. Ponzo (formerly M. Dolce)	Passive control of buildings and bridges: experimental and numerical studies for the validation and improvement of the structural design, analysis and verification methods and of the device testing procedures
R2	UNINA_SE - University of Naples Federico II	G. Serino	Design methods of buildings and bridges with viscous devices and of light structures
R3	UNINA_DL - University of Naples Federico II	A. De Luca	Seismic isolation of historical-monumental buildings
		EXTERNAL	
E1	UNIPG - University of Perugia	M. Mezzi (formerly A. Parducci)	Design and architectural aspects in seismic isolation applications
E2	UNICAL - University of Calabria	A. Vulcano	Design of buildings with energy dissipatine braces and seismic isolation and near-fault effects
E3	POLITO - Technical University of Torino	A. De Stefano	Tuned mass systems and semi-active control for the reduction of the structural seismic response
E4	UNIUD - University of Udine	S. Sorace	Design and simplified analysis methods and experimental qualification procedures for seismic isolation and energy dissipation systems including viscous fluid devices
E5	UNISA - University of Salerno	B. Palazzo	Experimental validation of the combined base-isolation – mass damper control system
E6	UNICAM - University of Camerino	A. Dall'Asta	Dynamic response control of existing R/C frames using high damping rubber devices and buckling restrained braces
E7	UNIBO - University of Bologna	M. Savoia	Design methods and reliability of buildings with seismic energy dissipation

E8	UNIPARTH - University of Naples Parthenope	A. Occhiuzzi	Seismic vibration control through semi-active dampers
E9	UNIVAQ - University of L'Aquila	V. Gattulli	Integrated systems of control and self-diagnosis in semi-active seismic dampers

During the 3rd year, a further Research Unit joined the Research Project Line, namely:

	EXTERNAL		
E10	POLIBA - Technical University of Bari	D. Foti	Aluminium energy dissipation devices for passive protection of buildings

In order to better organize the work considering the different tasks and objectives of the research, starting from the second half of the 1st year, Project Line No. 7 has been organized in five Subgroups, and two coordinators have been nominated for each Subgroup, as indicated in the following table:

Subgroup	Title	Participating Research Units	Coordinators
L7_SG1	Isolation of buildings and bridges	R1_UNIBAS, R2_UNINA_SE, R3_UNINA_DL, E1_UNIPG, E2_UNICAL, E4_UNIUD, E5_UNISA	D. Cardone, G. Serino
L7_SG2	Energy dissipation	R1_UNIBAS, R2_UNINA_SE, E1_UNIPG, E2_UNICAL, E4_UNIUD, E6_UNICAM, E7_UNIBO	M. Savoia, F. Ponzo
L7_SG3	Modelling and testing of devices	R1_UNIBAS, R2_UNINA_SE, R3_UNINA_DL, E4_UNIUD, E6_UNICAM	A. De Luca, S. Sorace
L7_SG4	TMD/TLD passive systems	E3_POLITO, E5_UNISA	E. Matta, L. Petti
L7_SG5	Semiactive systems	E8_UNIPARTH, E9_UNIVAQ	V. Gattulli, A. Occhiuzzi

The organization in Subgroups allowed to significantly improve the coordination among Research Units on specific topics and to more efficaciously develop joint experimental activities.

3.3 Meetings

During the three years of the Research Project, besides numerous informal meetings among the various researchers belonging to the different Research Units, the following plenary and subgroup meetings were held, their agendas and minutes being available in the area reserved to Project Line No. 7 in the ReLUIS website (http://www.reluis.it/):

Plenary meetings

- kick-off meeting of 20th January 2006 (DPC, Rome)
- end of first semester plenary meeting of 3rd/4th April 2006 (DPC, Rome)

- end of second semester plenary meeting of 29th September 2006 (DPC, Rome)
- end of third semester plenary meeting of 12th July 2007 (DPC, Rome)
- end of fourth semester plenary meeting of 10th January 2008 (DPC, Rome)
- end of fifth semester plenary meeting of 21st July 2008 (DPC, Rome)
- final seminar of 4th-5th December 2008 (PICO, Naples)

<u>*Subgroup meetings*</u>

- meeting on energy dissipation/JETPACS of 30th July 2007 (UNIBAS, Potenza)
- meeting for JETPACS tests on of 14th November 2007 (UNIBAS, Potenza)
- L7_SG2 meeting of 9th January 2008 (DPC, Rome)
- L7_SG4 meeting of 3rd April 2008 (UNISA, Salerno)
- L7_SG2 meeting of 10th June 2008 (UNIBO, Bologna)

4 MAIN RESULTS

4.1 General: configuration and morphology in innovative seismic protection systems

In conventional earthquake resistant design, seismic safety is achieved through the capability of structures to dissipate energy taking into account their deformation and ductility capabilities. These aspects have always involved the choice of structural configurations and architectural morphologies satisfying specific rules, like limited global dimensions, compactness, symmetry and regularity. However, when innovative protection systems like those based on base isolation or energy dissipation are used, new frontiers open in the correlation among architectural morphology, structural configuration and seismic behaviour of buildings (Mezzi and Parducci, 2006; Mezzi, 2006a; Mezzi, 2007b-c; Mezzi, 2008a). For example, the principle of *deformation* (Figure 1) is connected to the capacity of the construction to sustain large local relative displacements enabling in this way the proper operation of dissipative devices inserted in the structural grid. Also, the principle of *movement* is very important when dealing with base or top isolated constructions, which require global movement affecting the entire structure, or suspended floors, where local movement is provided involving only one or more building sections (Figure 2): this contrasts with one of the three Vitruvius' principles ("*firmitas*") which requires a rigid connection of the structure to a firm soil, although in these cases energy dissipation is always required to limit lateral response.

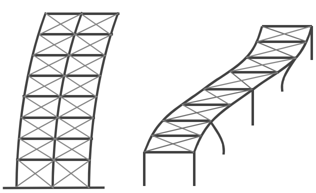


Figure 1. Deformation.

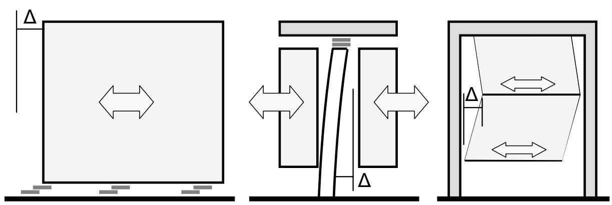


Figure 2. Movement.

The principle of *discontinuity* is strongly related to that of movement (Figure 3): external continuity with the ground and internal continuity among members are no more required, if the rigid motion of of the global construction or its components is allowed.

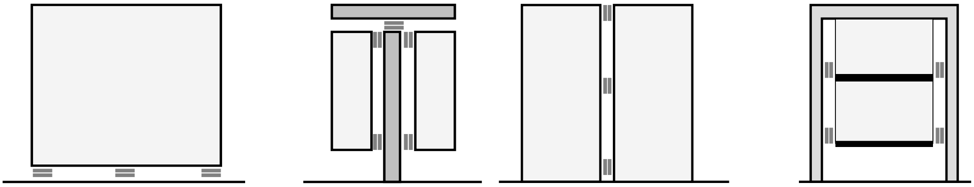


Figure 3. Discontinuity.

Furthermore, the presence of isolation and/or energy dissipation devices poses the problem of their *visibility*: sometimes protection systems are hidden and the building appears as an ordinary one, but in other cases the devices are evidenced (Figure 4), becoming an characteristic sign of architectural expression.

Figure 4. Visibility: (a) United Nation University, Tokyo; (b) Union House, Auckland.

4.2 General: retrofit of existing soft-first-story structures ("pilotis" configurations)

A widespread structural configuration of existing framed r.c. buildings in Italy is represented by "pilotis" buildings characterized by a soft first story. This configuration allows for a good use and distribution of the space at the ground floor, but it is well known that it is dangerous from the seismic point of view. In these cases, different retrofitting strategies (insertion of dissipating braces at the first story only with or without confinement of r.c. at the ends of basement columns, base isolation) can be proposed, which are certainly superior in terms of efficacy and costs compared to a traditional earthquake resistant design approach (Parducci *et al.*, 2005; Mezzi and Parducci, 2005; Mezzi, 2006b). Case studies of residential buildings located in Sotuthern Italy have been throughout analyzed in this sense (Figure 5).

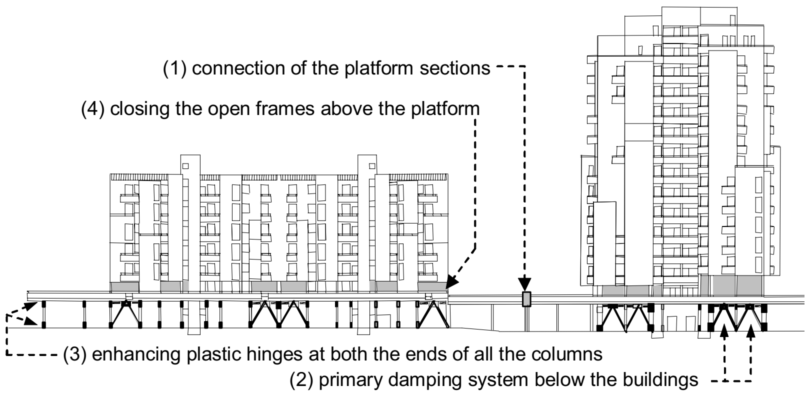


Figure 5. The compound dissipated system ("synergic" strategy) for the analysed buildings.

4.3 General: economic competitiveness of base-isolation and energy dissipation systems

The additional costs associated to the insertion of an advanced seismic protection system (base isolation and/or energy dissipation) has always represented a strong resistance against the use of these innovative technologies. A simple direct cost comparison determines a wrong evaluation, because it does not take into account of the differences in terms of performance levels characterizing the conventional and enhanced solutions.

In order to overcome the problem, comparative analyses of the seismic response, in terms of typical demand parameters (story drift and floor acceleration), of conventional fixed-base and enhanced with base-isolation / energy dissipation buildings (see Figure 6), have been carried out (Mezzi and Comodini, 2007; Mezzi, 2008c). The simulations allow to define the damage level associated to the response parameters, accounting for the uncertainties concerning: the building characteristics, the earthquake input, the damage intensity, the consequences of damage on the occupants, on the costs and on the downtime. Fragility curves of the main response parameters indicate a by far better performance (non-exceedance probability of the same performance level) of the base-isolated solution, and anyhow important improvements with the insertion of energy dissipations braces. The overall structure performances (safeguard of human life, construction efficiency and repairing costs) have also been estimated (Mezzi, 2008d; Mezzi and Comodini, 2008a). As a direct outcome of the above results, a correct methodology for the study of the economical implications of the adoption of special earthquake protection devices has been defined, allowing the development of new professional skills for the economic evaluations of the different seismic protection solutions today available on the market (Botta and Mezzi, 2008b).

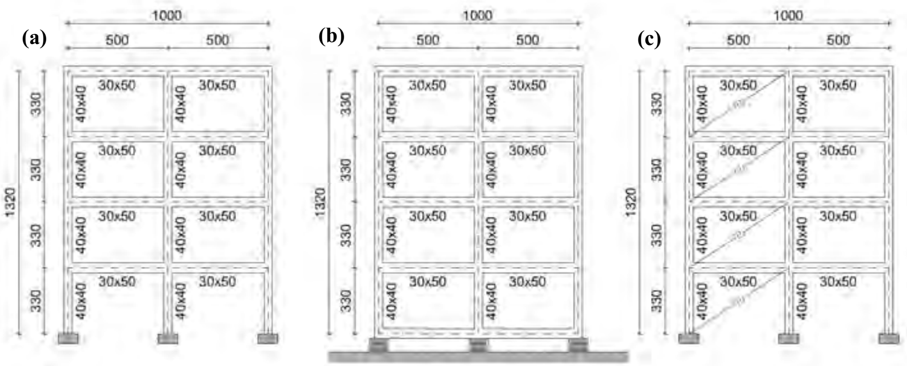


Figure 6. Analysed frame structures: (a) fixed-base, (b) base-isolated, (c) with dissipating braces.

4.4 L7_SG1: force distributions for static analysis of base isolated buildings

In most of the seismic codes, the use of the equivalent static analysis is strongly limited by the difficulty of defining a reasonable conservative distribution of inertia forces along the height of the building, particularly when the isolation system is non-linear and/or damping is significant. Two different studies aimed at evaluating more accurate distributions of equivalent static forces have been carried out. In a first one (Cardone *et al.*, 2007a), the force distributions have been evaluated based on the experimental results of shaking table and pseudodynamic tests carried out on large scale structural models. The isolation systems (ISs) considered in the experimental tests include: (i) Low-Damping Rubber Bearings (LDRBs), (ii) combinations of Flat Sliding Bearings (FSBs) and rubber-based viscoelastic devices, (iii)

combinations of FSBs and steel-based elasto-plastic devices and (iv) combinations of FSBs and shape memory alloys (SMA)-based nonlinear elastic devices. The results of the study pointed out that the parameter which mainly affects the shape of the force distribution is the ratio T_2/T_1, between the period T_2 of the BI-structure associated to the post-elastic stiffness of the IS and the period T_1 associated to the elastic stiffness of the IS. The ratio T_2/T_1 depends on the inherent IS characteristics only, while it does not change with the design displacement of the IS. For a given T_2/T_1 ratio, an important role is played by the ratio T_{is}/T_f between the period T_{is} of the BI-structure associated to the effective stiffness of the IS at the design displacement and the period T_f of the fixed-base structure. For low T_2/T_1 ratios (of the order of 1-1.5, based on the experimental results examined in this study) an excellent accuracy can be reached by assuming a uniform distribution of storey acceleration: in this case, the influence of the T_{is}/T_f ratio appears to be negligible, in the range examined in this study (from about 1 to 4.28). For medium T_2/T_1 ratios (of the order of 1.5-3, based on the experimental results) and isolation ratios T_{is}/T_f approximately equal to or greater than T_2/T_1, reference can be made to an inverted triangular distribution of storey accelerations. For high T_2/T_1 ratios (greater than 3-4, based on the experimental results), enhanced force distributions (e.g. parabolic or bilinear distributions), specialized to each type of IS and dependent on its characteristics should be used.
In the second study (Cardone *et al.*, 2007b), a large number (approx. 78000) Non Linear Time History Analyses (NTHAs) have been performed on six different base-isolated shear-type building models, characterized by a different number of floors (3, 5 and 8, respectively) and fixed-base periods. Three different ISs, with different combinations of their mechanical parameters, have been considered: (i) visco-elastic, (ii) elasto-plastic with strain-hardening, (iii) recentering system, able to describe the mechanical behaviour of practically all currently used ISs. The results point out that the simplified static force distributions considered by the Italian code often do not correspond to the numerically expected maximum seismic story forces, and new force distribution formulas have been proposed. More specifically, in buildings with visco-elastic isolators characterized by large damping ratios (20-30%), a uniform force distribution tends to underestimate actual seismic forces at the upper stories and to overestimate at lower stories (Figure 7), particularly for low T_{is}/T_f (=3÷6), with errors up to 25%. Similarly, in order to extend the simple static analysis procedure also in case of adoption of significantly non-linear Iss (Figure 8), currently not allowed by the code, a new force distribution formula which takes into account of the contribution of the second mode, has been proposed.

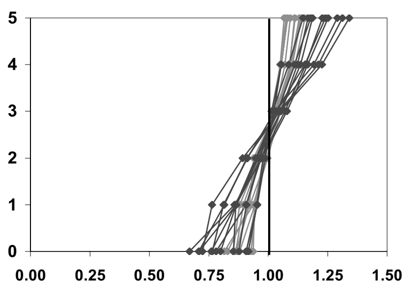


Figure 7. Static forces for 5-stories building model with viscoelastic IS (different examined cases).

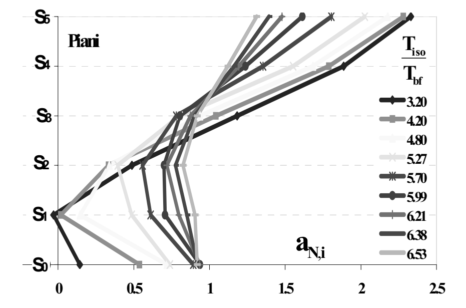


Figure 8. Static forces for 5-stories building model with elastoplastic IS (different T_{iso}/T_f ratios).

4.5 L7_SG1: effects of "near-fault" earthquakes on NL response of base isolated frames

The insertion of an isolation system at the base of a building structure allows to reduce the horizontal seismic loads through a decoupling of the structure motion from that of the soil; moreover, the superstructure behaves like a fixed- or isolated-base structure along the vertical direction, depending on the value, respectively very high or very low, of the ratio α_{K0}(=K_{V0}/PGA_{H0}) between the vertical stiffness of the isolator and the horizontal one. Also, the considerable horizontal deformability of a base-isolated structure may amplify the structural response under strong near-fault ground motions, which are characterized by long-period horizontal pulses and displacements so large that an oversizing of the isolation system could be required. More specifically, the frequency content of the motion transmitted by the isolators to the superstructure can become critical when the pulse intensity is such that the superstructure undergoes plastic deformations (Sorace and Terenzi, 2006b). Moreover, the structural response can be amplified due to the long duration of the pulse. Strong near-fault ground motions are often characterized by high values of the acceleration ratio α_{PGA}(=PGA_V/PGA_H). High values of α_{PGA} can notably modify the axial force in reinforced concrete (r.c.) columns, producing undesirable phenomena in these elements: e.g., buckling of the longitudinal bars, brittle failure in compression, bond deterioration or failure under tension. In addition, plastic hinges are expected along the span of the girders due to the vertical acceleration, especially at the upper storeys, where the effects of the gravity loads are generally prevailing over those of the horizontal loads and an amplification of the vertical motion is expected also depending on the vertical stiffness of the isolators.

The nonlinear seismic response of five-storey (Mazza and Vulcano, 2006a-c; Mazza and Vulcano, 2007a; Mazza and Vulcano, 2008f), three- and twelve-story (Mazza and Vulcano, 2006b; Mazza and Vulcano, 2007b) r.c. base-isolated framed structures, designed considering (besides the gravity loads) the horizontal seismic loads acting alone or in combination with the vertical seismic loads, has been studied under the horizontal and vertical components of near-fault ground motions. The results of the numerical investigation emphasize that for the base-isolated structures designed considering only the horizontal seismic loads, the ductility demand of the girders, especially for increasing values of the nominal stiffness ratio α_{K0} of the isolator, are significantly larger when the vertical component of the ground motion is considered acting together with an analogous horizontal component. The effects on the ductility demand of the girders have been rather evident at the end and mid-span sections, especially at the top floor (Figure 9). As regards the columns, the addition of the vertical motion induced variation of the axial force, producing in many cases even tension or a compressive force larger than the balanced-failure force, especially for increasing values of α_{K0}. Consequently, the superstructure should be designed accounting also for the vertical ground motion, especially with regard to the girders at the upper storeys when assuming a rather high value of α_{K0}, for which the superstructure behaves like a fixed-base structure in the vertical direction (Figure 10). Alternatively, for a rather low value of α_{K0}, the superstructure behaves as to be isolated also in the vertical direction, but the effects of the rocking motion, due to both the overturning moment produced by the horizontal seismic loads and the vertical deformability of the isolators, should be carefully examined.

It has also been shown (Mazza and Vulcano, 2007c) that a supplemental viscous damping at the base is favourable to controlling the isolator displacement. On the other hand, the supplemental damping at the isolation level does not guarantee in all the cases a better performance of the superstructure and it can be unfavourable, depending on the frequency content of a near-fault earthquake. However, some use of supplemental viscous damping can be considered as a suitable part of a base isolation system under a strong near-fault ground motion, provided that a suitable ξ_D value (e.g., ξ_D=0.2÷0.3) is adopted.

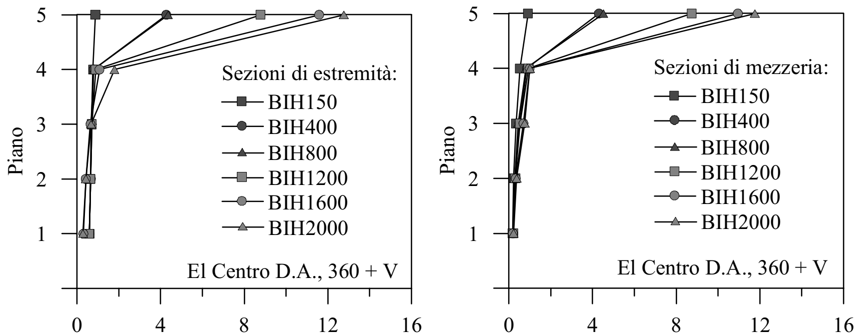


Figure 9. Girders' ductility demand (BIHnn=frame designed only for horizontal seismic load and α_{K0}=nn).

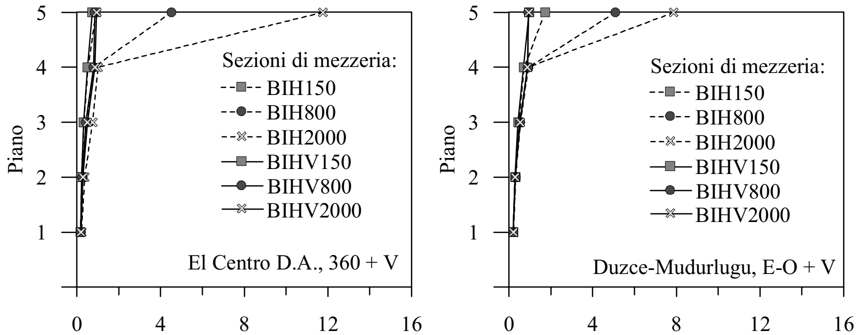


Figure 10. Girders' ductility demand (BIHVnn=frame designed for horizontal and vertical seismic load).

4.6 L7_SG1: base isolation in the seismic retrofit of masonry church buildings

In historical particularly vulnerable buildings, like basilica masonry churches, seismic rehabilitation should have a minimum impact on the construction, preserving the original structure as far as possible: base isolation balances the opposite requirements of structural safety and architectural preservation. An improved optimization process has been applied on some study cases (Brandonisio *et al.*, 2007a; Cuomo *et al.*, 2006a-b; Mele *et al.*, 2007b): the structure is first analysed through FEM, and its seismic response is evaluated through modal and time history analyses; a dimensioning procedure and list of checks on the devices is then performed, finally confirming the benefits deriving from the introduction of the base isolation system (Cuomo *et al.*, 2008a). The analyses have also shown that the presence/absence of a rigid floor diaphragm connecting the isolators does not imply variations in terms of accelerations, displacement and forces in the upper part of the construction, but only different values of the normal stress on the isolators for vertical load, due to the different deformability of the superstructure.

Further studies (Brandonisio *et al.*, 2007b-c) performed on ten churches with a basilican plan (Figure 11) allowed to estimate the seismic safety of this type of structure through the examination of the global and single macro-elements geometrical characteristics, allowing to identify the most vulnerable macro-element classes, i.e. the triumphal arches and the elements of the broad aisles, which show high seismic demand and low capacity. Simple formulas and geometrical analysis tools (Figure 12) for predicting the horizontal capacity of masonry portals (Giordano *et al.*, 2006b; Giordano *et al.*, 2007a) and arches (Giordano *et al.*, 2006a; Giordano *et al.*, 2007b), derived in the framework of limit analysis, have also been proposed and compared with FEM analysis on study cases taken from real churches, showing that the simplified approximate approach can lead to more conservative results.

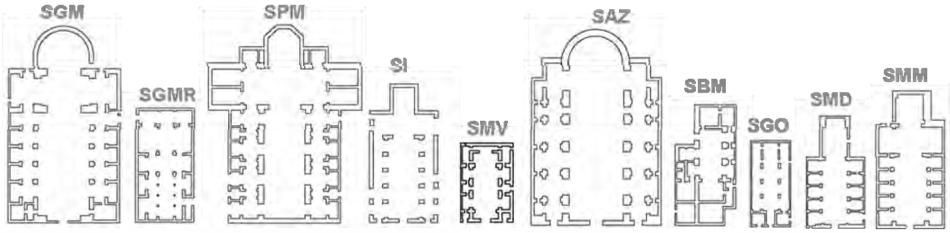


Figure 11. Plan of the ten analysed basilican churches.

4.7 L7_SG1: design procedures for seismic isolation of bridges

Two different procedures have been set up for the design of the IS's: the design force approach and the design displacement approach. Both are based on the Capacity Spectrum Method, as defined in (ATC, 1996). The design force approach is particularly useful for the retrofit of existing bridges, while the design displacement approach is better suited for new bridges. In Dolce *et al.* (2007) different isolation systems have been considered, made of steel-PTFE sliding bearings, to support the weight of the deck, and of auxiliary devices, based on different technologies and materials (i.e. rubber, steel and shape memory alloys, etc.), to provide re-centring and/or additional energy dissipating capability. A numerical simplified pier-deck model has been set up, where the pier is modelled as an elastic cantilever beam and the mass of the deck is connected to the pier through suitable non linear elements, simulating the behaviour of the isolation system. The extensive numerical investigation carried out allowed to: (i) assess the reliability of different design approaches, (ii) compare the performances of different types of isolation systems, (iii) evaluate the sensitivity of the structural response to friction variability due to bearing pressure, air temperature and state of lubrication and (iv) identify the response variations caused by changes in the ground motion, bridge and isolation characteristics.

A further methodology for determining the optimal values to be assigned to the damping and stiffness parameters of protection devices, to be inserted between the pier and the deck in an isolated bridge, has been defined (Spizzuoco *et al.*, 2005) . Within this methodology, simple design spectra are proposed, which allow to determine the optimal damping parameter as a function of the stiffness parameter (Figure 12).

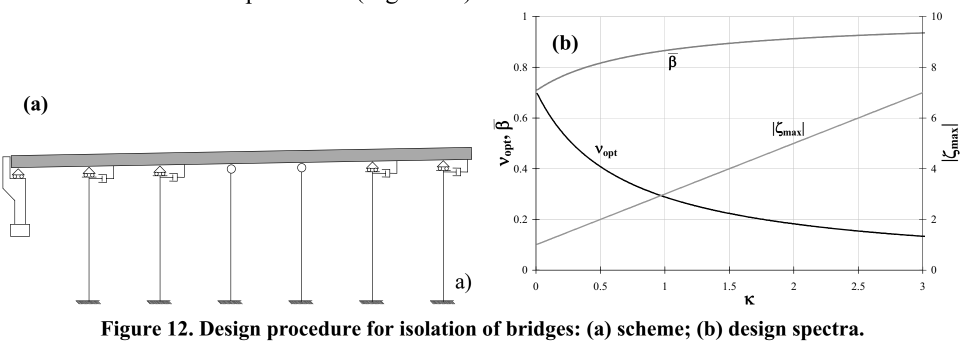


Figure 12. Design procedure for isolation of bridges: (a) scheme; (b) design spectra.

4.8 L7_SG1: design procedures and applications to relevant isolated structures

An innovative base isolation system named BISD (Base Isolation/Supplemental Damping) composed of stainless steel-Teflon bearings, operating as sliders, and recentering silicon fluid viscous pressurized spring dampers, has been extensively studied both analytically and experimentally by the research Unit E4_UNIUD (Sorace *et al.*, 2006b; Sorace *et al.*, 2008a). The proposed system has the advantages, compared to the more commonly used based on high damping steel laminated rubber bearings, of being able to: a) provide larger equivalent viscous damping and thus small relative displacements at the isolation interface; b) more easily control torsional effects in plan irregular buildings. A detailed specific design procedure has been developed for the system, which was applied for the first time to a demonstrative r.c. strategic building (Figure 13), namely the new site of the no-profit association "Fratellanza Popolare" in Grassina, near Florence (Sorace and Terenzi, 2005; Sorace and Terenzi, 2008c-d-e): the design anayses confirmed the high performance and cost competitiveness of the proposed system, compared to other base isolation systems and traditional seismic design strategies. A second application (Figure 14) of the same system was hypothesized with reference to the ground floor (main exhibition hall hosting two important marble statues) in the central wing of the historical medieval castle of Prampero located in Manzano, near Udine, strongly damaged by the 1976 Friuli earthquake (Sorace and Terenzi, 2007a; Sorace and Terenzi, 2008b): details of the design analyses have proven the effectiveness and cost efficacy of this solution for the protection of art objects in museum halls, compared to base isolation of single statues and artistic objects.

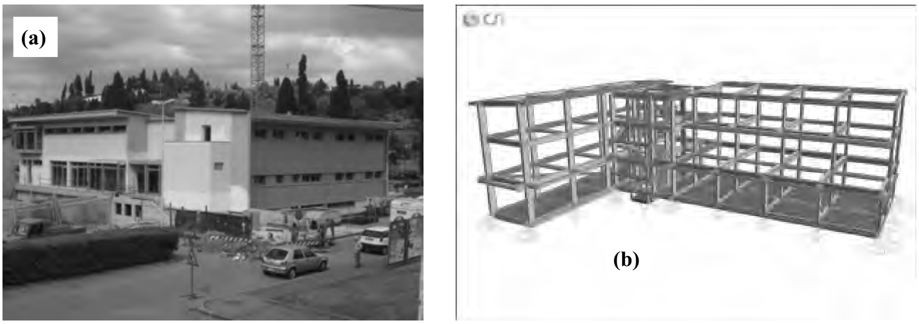


Figure 13. Demonstrative application of BISD: (a) isolated building; (b) FEM model of the structure.

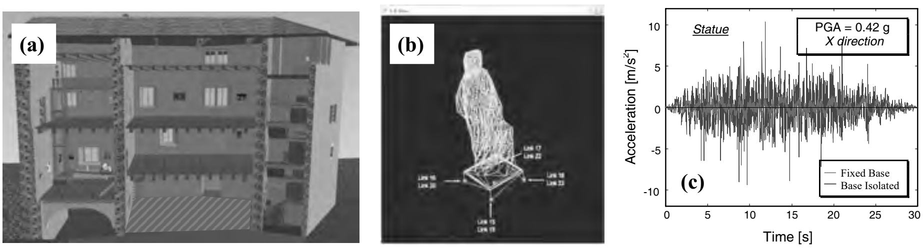


Figure 14. Floor isolation project: (a) central wing of castle; (b) FEM model of statue; (c) analysis results.

Another significant application of seismic isolation developed within Reaserch Project Line No. 7 is the seismic retrofit of the "Santuario della Madonna delle Lacrime" located in Syracuse (Figure 15), made substituting the bearings supporting its impressive dome (diameter 71.40 m at bearing axes, height 74,30 m from upper temple floor) with sliding seismic isolators equipped with elasto-plastic dissipators (Serino *et al.*, 2007a). The insertion of the protection devices between the 22 main columns of the upper temple structure and the truncated conical dome weighting more than 220 MN, required its raising and then lowering with 114 hydraulic jacks operating simultaneously. An advanced monitoring system, able to continuously detect relative thermal displacement at the isolation interface, wind speed and direction on top of the dome, together with accelerations at different points of the struture and bearings' relative displacement during earthquake events exceeding a defined threshold, has also been installed (Serino *et al.*, 2007b) and is starting providing first important data. The structure has been recently included among those of the Italian network of buildings and bridges permanently monitored by the Italian Department of Civil Protection (DPC) within the Seismic Observatory of Strutures (OSS).

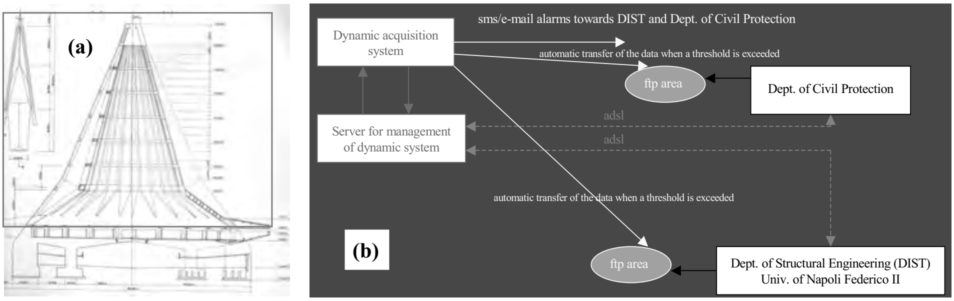


Figure 15. Syracuse Sanctuary: (a) cross section of isolated dome; (b) scheme of monitoring system.

4.9 L7_SG2: innovative use of stone in seismic resistant architecture

An advanced structural use of stone has been proposed, referring also to possible real applications, allowing for the comeback of this traditional construction material in seismic areas (Mezzi, 2007a; Mezzi and Mariani, 2008). Basic structural elements have been investigated through numerical simulations, modelling the elements with solution of continuity among the blocks and a prestressing action given by dead load or unbounded cables (Figure 16). The effect of inserting dissipating devices at the interfaces has also been analyzed. The enhancing effects of these technologies have been demonstrated, with an increasing of the overall seismic performance of the structure.

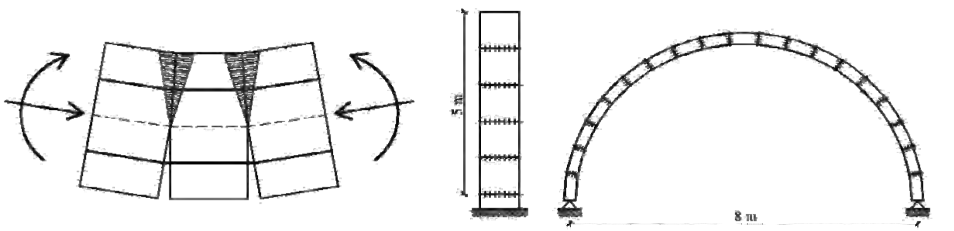


Figure 16. Connection of stone blocks through prestressing or dead weight and examined basic schemes.

4.10 L7_SG2: design procedures for energy dissipating bracing systems

In recent years, introduction of energy dissipating braces in a structural frame has been proved as an effective technique for reducing seismic effects. However, although the large number of numerical and experimental studies on many different dissipating device types and the blooming of real applications, there has not been a parallel development of related codes and guidelines, as for other passive control technique like seismic isolation. For this reason, within Project Research Line No. 7, a significant effort has been made toward the development of design procedures for energy dissipating braces. The proposed procedures represent a development of previous efforts known from the literature, in which all the input energy was supposed to be dissipated in the bracing system, i.e. the behaviour of the frame was assumed elastic.

Supplemental damping devices are usually classified in two categories: displacement-dependent (e.g. friction and metallic-yielding dampers) and velocity dependent (viscoelastic and viscous dampers). For each of the two categories, design procedures for proportioning damping braces have been defined, aimed to the attainment of a designated performance level in a given framed structure (Vulcano and Mazza, 2007b; Mazza and Vulcano, 2008g; Dall'Asta *et al.*, 2008c). In both cases, the design method follows the Direct Displacement-Based Design with a proportional stiffness criterion, and can be applied to new buildings or in the retrofitting of existing ones. More specifically, in the case of energy dissipating displacement-dependent bracing, the strength, stiffness and ductility of the braces are assumed as unknowns and the proposed iterative procedure consists in the following steps (Ponzo *et al.*, 2007a-c):

1. Definition of characteristics of unbraced frame, using push-over analysis and SDOF reduction.
2. Evaluation of the characteristics of equivalent brace, for a pre-assigned ductility.
3. Distribution of such characteristics at the stories, assuming a constant stiffness ratio between the structure and the story brace.
4. Safety verification with the N2 method and evaluation of the *q*-factor.

Similarly, in the case of visco-elastic bracing (Ponzo et al., 2007b), the stiffness, loss factor and maximum strain amplitude of the damping devices are assumed as unknown quantities and the following steps are indicated:

1. Analysis of unbraced frame to evaluate its story stiffness characteristics.
2. Design of the story bracing stiffness, assuming constant stiffness ratio between the structure and the story brace.
3. Iterative design of the dissipating device, once temperature, strain amplitude and period are fixed.
4. Evaluation of the capacity curve of the braced structure abd SDOF reduction.
5. Evaluation of the equivalent damping of the braced structure and search for the performance point.
6. Verification of the strain amplitude of the visco-elastic material.

With specific reference to High Damping Rubber (HDR) devices, whose behaviour is quite complex as both stiffness and damping depend on strain amplitude and strain rate, equivalent linear models for a fixed displacement amplitude and frequency can be used in the design procedure, within an acceptable approximation level, assuming the original unbraced frame to remain elastic (Dall'Asta *et al.*, 2006b) or to sustain limited damage due to its hysteretic behaviour (Dall'Asta *et al.*, 2008a).

The proposed procedures have been applied and verified on several real existing buildings (e.g. Figure 17).

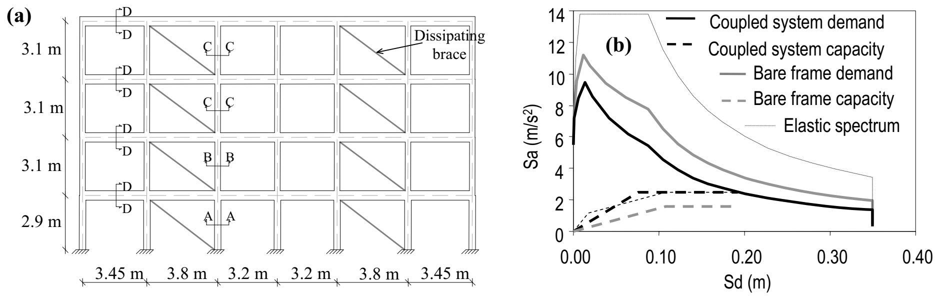


Figure 17. Design example of dampers in plane frame: (a) scheme; (b) capacity vs. demand curves.

For the specific case of frame structures incorporating damping braces with pressurized silicone fluid viscous devices, whose behaviour and device-brace-beam connection can be represented through a sophisticated analytical and FEM model (Figure 18), a detailed procedure based on an energy balance within a nonlinear dynamic analysis approach has been developed for the selection of the damping coefficient of the viscous devices (Sorace and Terenzi, 2008a). The procedure has been applied to the retrofit of a pre-normative steel school building built in Florence in the late Sixties (Sorace and Terenzi, 2007b; Sorace and Terenzi, 2008f).

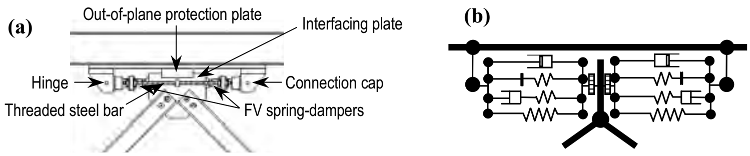


Figure 18. Pressurized FV spring-damper-brace-beam connection: (a) drawing; (b) FEM model.

A further case-study has been specifically studied, concerning a two-storey precast reinforced concrete structure of a shopping mall, with rectangular plan and maximum height of about 10.50 m (Ceccoli *et al.*, 2006; Silvestri *et al.*, 2006b; Trombetti *et al.*, 2006b; Ceccoli *et al.*, 2007;. Trombetti *et al.*, 2007a; Silvestri *et al.*, 2007b; Ceccoli *et al.*, 2008; Trombetti *et al.*, 2008c; Silvestri *et al.*, 2008). The columns and the beams are made with precast reinforced concrete elements. The main structure is not a moment-resisting frames, with columns acting like cantilevers elements as far as the horizontal seismic actions are concerned. In order to successfully satisfy all structural verifications (strength and deformability requirements), earthquake-resistant bracing systems are required. Four different structural solutions have been studied: precast structure without any bracing (NAKED solution); precast structure with rigid steel inter-storey bracings (ISB solution); precast structure with dampers connecting each floor to the ground (FPD solution); precast structure with inter-storey dampers (ISD solution). The results of the numerical time-history simulations showed that, adopting traditional steel bracing systems, the deformability requirements are satisfied, but large forces both in the bracing system and on the foundations are present. On the contrary, by inserting viscous dampers (following either a Fixed-Point or an Inter-Storey criterion) the drifts of the structure are limited and forces in the bracing system are maintained below acceptable levels (Figure 19).

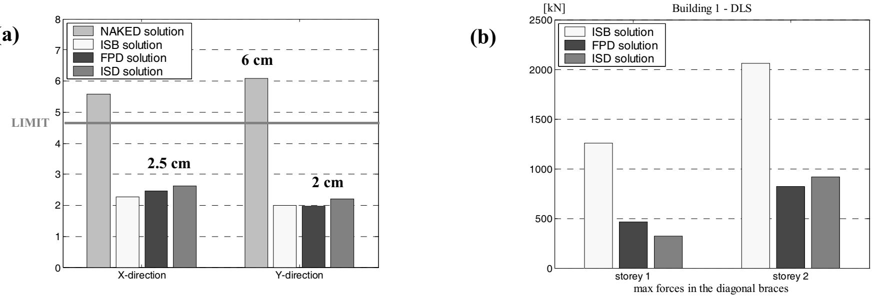


Figure 19. Two-story analysed precast r.c. structure: (a) interstory drifts; (b) max forces in the braces.

4.11 L7_SG2: computational models and design of the "damped cable system"

The "damped cable system" represents an innovative seismic protection solution for new or existing framed building. It consists in couples of pre-tensioned cables symmetrically arranged on the facades of the building, connected at the base to a pressurized silicone fluid viscous damper anchored to the foundation and at the other end to one of the top floors, with sliding contacts at the intermediate floors (Figure 20). A detailed computational model of the system has been developed, which includes as significant non-linear elements the sliding contacts between the pre-tensioned cables and the floors modelled as elastic connections between the cable-floor interfacing joints and the centre of curvature of the cable deviators plus rigid-body constraints imposing a displacement trajectory equal to the deviator shape to the interfacing joints (Sorace and Terenzi, 2006a), and the non-linear viscous fluid devices represented through the model already shown in Figure 18. Different geometrical layouts of the cables have been considered and compared, and application of the system has been proposed for the seismic retrofit of a wing of a r.c. hospital building in Latisana, near Udine (Sorace *et al.*, 2007), as well as for the same pre-normative steel school building in Florence considered for a dissipative bracing-based retrofit solution, mentioned in previous paragraph.

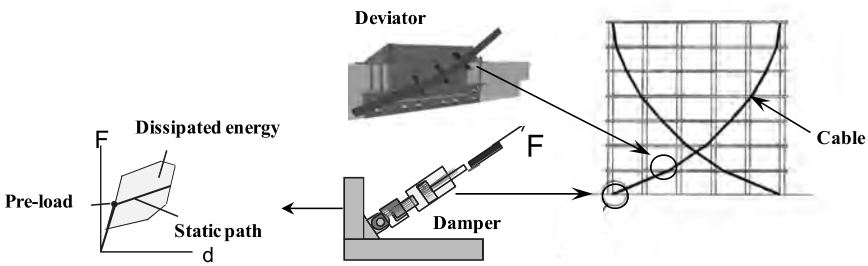


Figure 20. Damped cable system and components.

4.12 L7_SG2: use of dampers between adjacent buildings against seismic pounding

Earthquake-induced pounding between adjacent buildings with an insufficient seismic joint is a well known problem. It caused severe damage during recent strong earthquakes which occurred in the world, and many r.c. adjacent buildings in Italy completed during the economic boom of the Sixties and Seventies are particularly seismically vulnerable also because of the lack of a sufficiently large separation joint. Traditionally, proposed retrofit interventions to solve the problem are based on: a) creation of new separation joints along the height of the building or enlargement of existing joints; b) rigid connection of the potentially pounding adjacent buildings, transforming them in a single structural earthquake-resistant body; c) insertion of stiffening elements, like bracings, within each building, particularly the most flexible ones, so as to reduce lateral displacements under earthquakes and avoid hammering contact. The above techniques are however particularly costly and difficult to put into practice, so that the high level of vulnerability connected to the pounding phenomenon practically remains unchanged.

Sorace and Terenzi (2007c) developed a detailed computational model based on the "penalty method" (according to which a "penalizing" term is introduced in the mechanical energy expression of the dynamic system, representing an additional spring element at the contact point allowing partial penetration after contact) for the dynamic non-linear analysis of pounding between adjacent building representing real structures. The model has been applied to a case study of a two-story and a five-story r.c. adjacent buildings built in the Eighties in Italy, having a separation joint of 2 cm only (Figure 21): a mitigation solution based on the installation of a limited number of fluid-viscous dampers across the existing inadequate separation joint is proposed and its efficacy in terms of feasibility, low invasivity and reduced costs, particularly when dampers are inserted at the level of the top floor of the lower building only, has been demonstrated. In another significant case study (Sorace *et al.*, 2008b), represented by two adjacent six-story buildings located in Pordenone and separated by a joint of again two cm only, the insertion of 11 fluid-viscous dampers all located at the separation joint in positions external to the building (on the facades or on the terrace of the top story), and thus very easy to install, is not only able to definitively solve the hammering problem, but also to significantly reduce the global earthquake response of the buildings, due to the additional damping provided by the devices.

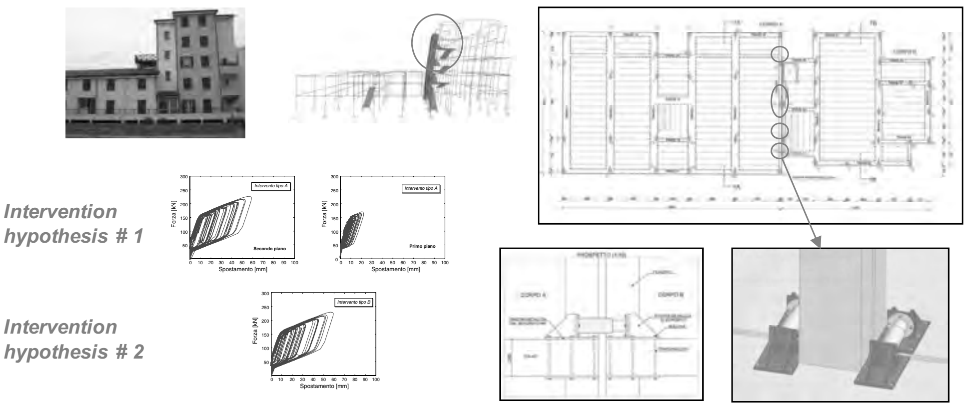


Figure 21. Insertion of viscous dampers at the separation joint between 2- and 5-story adjacent buildings.

The above results are consistent with other independent analytical and numerical studies carried out by other researchers (Silvestri *et al.*, 2006a; Silvestri *et al.*, 2007a-c) to investigate the effectiveness, for the mitigation of seismic effects, of placing viscous dampers between adjacent structures (specifically flexible frames and lateral resisting elements like shear walls or braced frames). Encouraged by the results obtained for a basic system composed by two adjacent SDOF systems linked by a single damper, the authors carried out a wide series of numerical simulations with reference to two six-story adjacent structures under different seismic inputs showing that placing dampers between them (Figure 22a) is capable of reducing structural response significantly more than as it could occur by placing the same dampers in the typical interstory locations (Figure 22b). This peculiar damper arrangement, if damper characteristics are chosen appropriately, may be traced back to the indirect implementation of the so-called MPD (mass proportional damping) system, extensively studied in previous research works by the same authors and also indicated as FPD (Fixed Point Damper) in Figure 18 above.

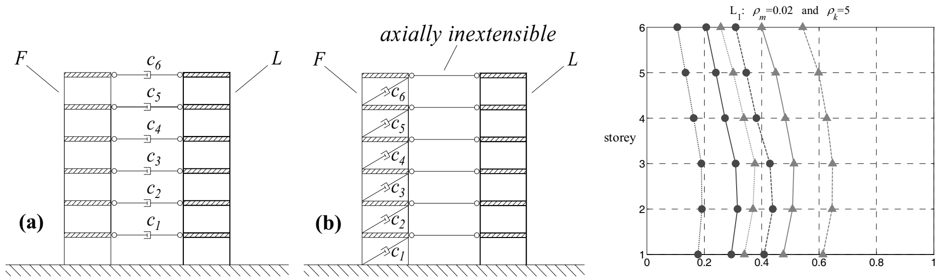


Figure 22. Analysed systems and typical response compared to no-damper case: (a) blue and (b) red lines.

4.13 L7_SG2: Joint Experimental Testing on Passive and semiActive Control Systems

Within Research Line No. 7 of the ReLUIS 2005-2008 Executive Project, an extensive experimental campaign named JETPACS (*Joint Experimental Testing on Passive and semiActive Control Systems*) involving several Line No. 7 Research Units (namely: UNIBAS, UNINA_SE, UNICAL, POLITO, UNIUD, UNIPARTH, UNIVAQ, POLIBA) has been carried out at the Structural Laboratory of the University of Basilicata (Dolce *et al.*, 2008). Under the JETPACS program, different energy dissipating bracing systems have been experimentally analysed, with the following objectives: a) to increase user awareness on some specific aspects of currently available techniques; b) to carry out a performance assessment of the different techniques; c) to simplify and suggest a design standardization procedure.

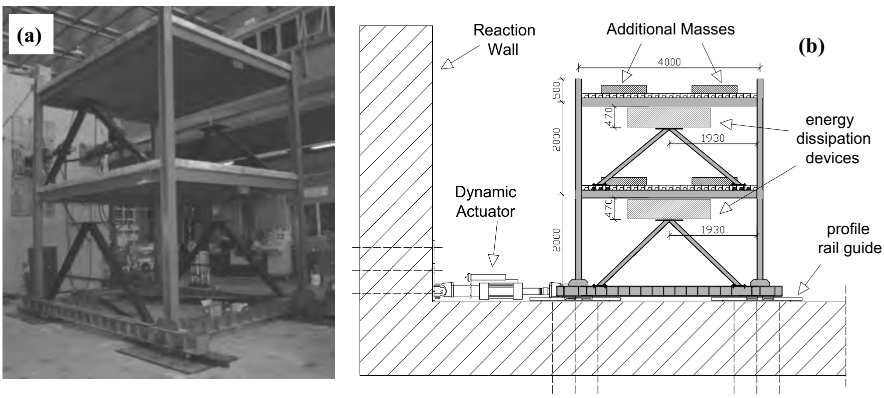


Figure 23. JETPACS structural model: (a) 3-D view; (b) longitudinal section [in the test direction].

Shaking table tests have been carried out on a 2:3 scaled structural model derived from a prototype building, namely a 2-storey, 1-bay, three-dimensional steel frame with a 4 m span in the test direction, an inter-storey height of 2 m and a 100 mm thick steel-concrete slab connected to the primary beams at each floor (Figure 23). Additional masses have been placed on each slab to account for non-structural dead loads and an adequate amount of live loading (30%). During the tests, seven groups of different type of passive or semi-active energy dissipating devices provided by each of the partners' Research Unit (see Table 1) have been inserted on top of the two stiff V-inverted steel braces placed at each storey, using unified bolted connection plates able to accommodate the different types of dampers. For the passive type of devices (No. 1, 2, 5, 6 and 7) four dampers or couple of dampers, two at each storey, have been installed, while for the semi-active devices (No. 3 and 4) only two dampers in the two braces at the first storey level have been inserted.

Table 1. Energy dissipation devices inserted in the JETPACS frame for the shaking table tests.

No.	Device Type	Manufacturer	Research Unit
1	Visco-elastic	Jarret SL, France	UNIUD
2	Fluid viscous	FIP Industriale, Italy	UNINA_SE
3	Magnetorheological	Maurer&Söhne, Germany	UNIPARTH
4	Magnetorheological	LORD Corp., USA	UNIVAQ
5	Steel hysteretic	TIS spa, Italy	UNIBAS/UNICAL
6	Aluminium hysteretic	TEC srl, Italy	POLIBA
7	Visco-SMA	TIS spa, Italy	UNIBAS

In order to obtain natural frequencies, masses (due to some uncertainties in their direct evaluation), modal shapes and damping values of the bare frame, structural dynamic identification tests on the frame without dampers have been first (October-November 2007) carried out considering a number of different excitation sources: ambient noise, instrumented hammer impact and sine sweep ground motion induced by operating a nearby isolator testing machine. Response of the structural model equipped with the different types of dampers has then (August-November 2008) been obtained subjecting it to a set of seven natural records obtained from the ReLUIS web site (www.reluis.it) characterized by a mean spectrum compatible with the EC8 response spectrum for soil type B, Seismic Zone 1. The acceleration values have been properly scaled to ensure spectrum compatibility, while to provide consistency with the scale of the model, time has been scaled according to the square root of the scale of the model ($\sqrt{2/3}$). Figure 24 provides the 5% elastic response spectra of the considered records. The shaking table tests on the frame equipped with the different dampers have been performed applying the three records whose spectra are represented in Figure 23b, gradually increasing them according to the following scheme: 10%, 25%, 50%, 75%, 100%, (125% and 150% when possible). For the intermediate seismic intensities (50%, 75%, 100%), the complete set of seven accelerograms whose spectra are represented in Figure 23a has been applied. In any case, to avoid yielding in the columns and guarantee repeatability, during the tests inter-story drift at both levels has been limited to a maximum of 0.7%. JETPACS also allowed to study and define an hybrid real-time substructuring technique (Londoňo *et al.*, 2008), according to which the response of the complete frame could be predicted actually testing the energy dissipation devices only and considering for the rest of the structure a numerical model, with significant cost savings when large number of shaking table tests are to be performed.

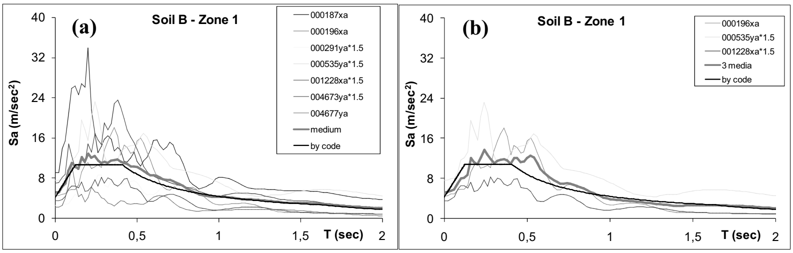


Figure 24. Elastic response spectra of the: (a) seven and (b) three natural records adopted in the tests.

A complete comparative analysis of the response of the frame with the different passive and semi-active devices is reported in (Ponzo *et al.*, 2008c). However, a comparison of the response in terms of max inter-storey drift and device force under the three basic earthquake records at 25%, 50%, 75% and 100% PGA is shown in Figure 25. The graphs indicate that all the protection systems considered – although in a more or less efficacious way due to the different design objectives adopted – are able to reduce the seismic response of the JETPACS frame, compared to naked (i.e. without dampers) case. Forces in the TEC-POLIBA aluminium hysteretic devices were not measured because their connections to the V-inverted braces were not designed to house the load cells. Further details regarding the response of some of each specific energy dissipation system can be found in Sorace *et al.*, 2008b (No.1: JARRET-UNIUD), Ponzo *et al.*, 2008a-d (No.5: TIS-UNIBAS/UNICAL), Diaferio *et al.*, 2008a-c-d (No.6: TEC-POLIBA), Ponzo *et al.*, 2008b (No.7: TIS-UNIBAS). Also, in the reference list at the end of the paper the complete series of JETPACS reports giving all the details of the completed experimental campaign is provided.

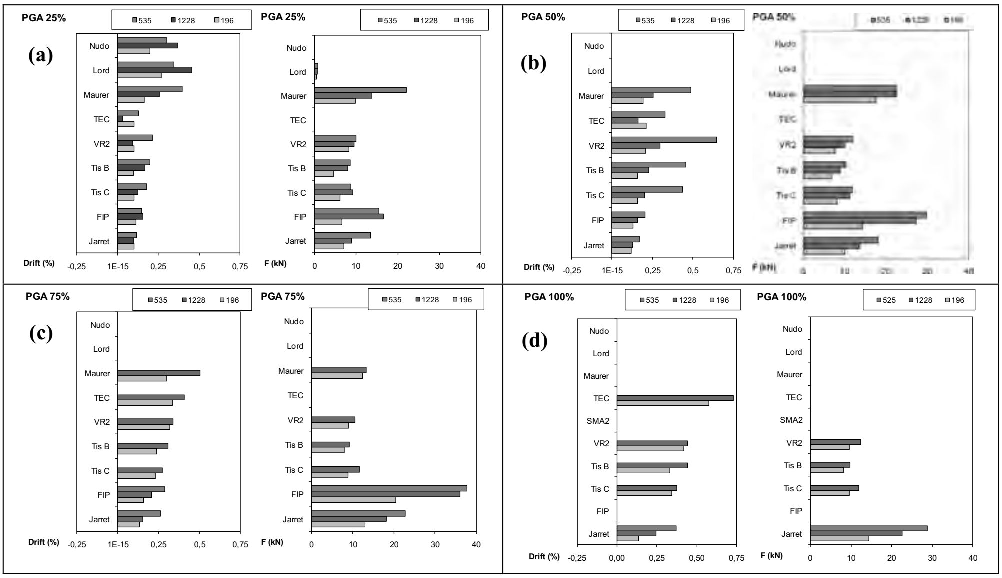


Figure 25. Max inter-storey drift and device force at: (a) 25%, (b) 50%, (c) 75% and (d) 100% PGA.

4.14 L7_SG2: experimental tests on 3-D frame equipped with HDR and BRB dampers

An experimental campaign on a real scale single storey mock-up of a steel-concrete composite space frame alternatively equipped with dissipating brace including high damping rubber (HDR) and with buckling restrained braces (BRB) has been performed by the UNICAM Research Unit. For the HDR devices, first a non-linear visco-elastic damage model has been proposed and experimentally validated, able to describe rubber behaviour under cyclic loads, taking into account its non-linear rate dependence, as well as the Mullins effect consisting in a rapid decrease of stiffness of virgin rubber in the first load cycles and its dependence on the strain amplitude (Dall'Asta *et al.*, 2005a; Dall'Asta and Ragni, 2006). Subsequently, HDR dissipation devices were inserted in the composite frame (Figure 26) and free vibration tests, displacement controlled cyclic tests at low frequencies and dynamic force-controlled cyclic tests at high frequencies were performed to demonstrate the efficiency of this type of dampers in improving the stiffness and dissipation capacity of the frame, making it possible to reduce structural and non-structural damage under earthquakes (Dall'Asta *et al.*, 2005b; Dall'Asta *et al.*, 2006a; Dezi *et al.*, 2007). The validity of the previously developed non-linear visco-elastic damage model has then been confirmed considering the frame as a linear SDOF system connected in parallel to the non-linear HDR dampers and comparing the experimental with the analytical response (Dall'Asta *et al.*, 2008b), and thereafter a complete study of the response of such a system under harmonic forces, impulsive excitation and seismic inputs of different intensities has been completed (Dall'Asta and Ragni, 2008a), showing the important role of the Mulling effect in changing the dynamic properties of the system, particularly under seismic loads. Finally, a procedure to derive equivalent linear models for the rubber to be used in seismic design has been defined, considering both transient and steady-state behaviour (Ragni and Dall'Asta, 2007; Dall'Asta and Ragni, 2008b).

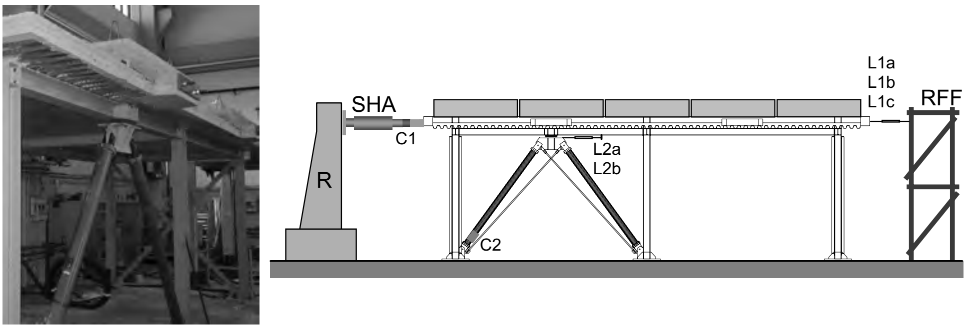


Figure 26. Steel-concrete composite space frame equipped with HDR dissipating braces.

Experimental tests were carried out on the same real scale mock-up equipped with buckling restrained braced (BRB) (Figure 27), designed in order to obtain the same protection level provided by the previously tested HDR-based devices. Since the one story mock up had a very small mass and stiffness, the core of the BRB devices was made out of aluminium. Performed experiments consisted of quasi-static displacement controlled tests. The results of these tests were compared with the results obtained from the numerical calculations, the Bouc-Wen model being used to simulate the BRB behaviour. It was shown that BRBs possess large dissipation capacity, however some cyclic fatigue problems needing further investigation were evidenced for the aluminium devices during the tests at large displacements (Dall'Asta *et al.*, 2008d).

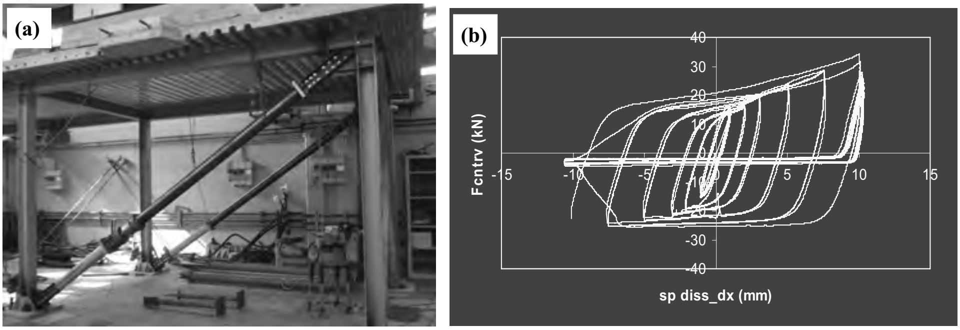


Figure 27. Steel-concrete composite space frame equipped with BRB (a) and typical hysteretic cycle (b).

4.15 L7_SG3: FEM analyses and experimental tests on laminated rubber isolators

Most recent tendencies toward the use of seismic isolation also for high-rise and existing monumental buildings call for the adoption of long isolated design periods up to 4 s and more. As a consequence, mean vertical pressure larger than 6-8 MPa and lateral shear deformations in the rubber up to 200% and above are becoming common requirements for the isolators.

An extensive FEM analysis campaign has been carried out within Research Project Line No. 7 to understand the effects of increasing vertical pressure and shear deformation on laminated rubber bearings characterized by different values of primary shape factor S_1 (Cuomo *et al.*, 2007a-2008b). A set of five HDLRBs characterized by S_1 = 6, 12, 18, 24, 30 has been subjected to vertical load only, horizontal load only and both vertical and horizontal loads varying mean vertical pressure (p_m = 3, 6, 9, 12, 15 MPa) and horizontal deformation (γ_d = 100, 200, 300%). As shown in Figures 28 and 29, for low values of S_1 significant stress variations may occur at the rubber-steel interface, particularly for larger values of p_m and γ_d, and the FEM solution differs from the simplified pressure solution valid for large S_1 values.

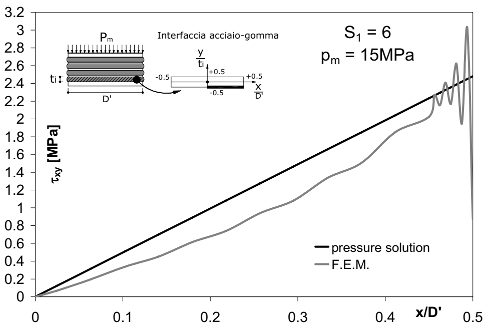


Figure 28. Comparison between pressure and FEM solution (vertical load only).

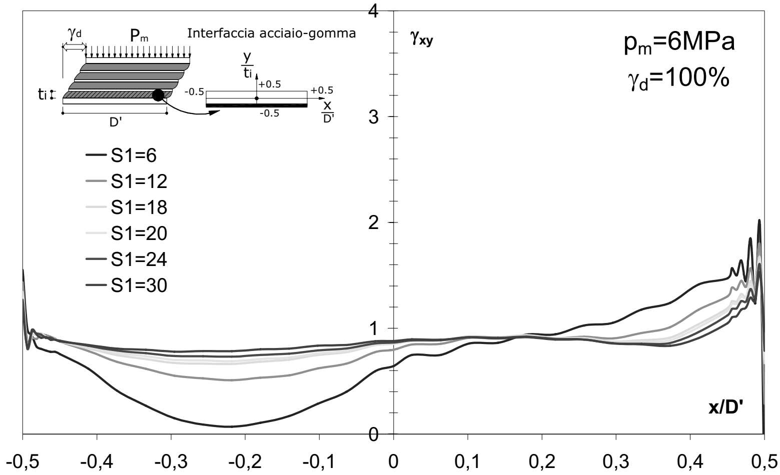


Figure 29. Effect of S_1 on shear strain distribution (vertical + horizontal load).

To experimentally characterize laminated rubber isolators under current and extreme working conditions, two large dedicated testing machines have been definitively completed with the partial contribution of Research Project Line No. 7: the first one (Figure 30) is located at the Structural Engineering Laboratory of the University of Basilicata in Potenza and is able to apply a vertical load up to 8000 kN (compression) - 1500 kN (tension) and a horizontal load up to 1000 kN with a peak-to-peak displacement of 1000 mm; the second one (Figure 31), located at the ARS laboratory in Frignano (CE) of the Campania Region Competence Center BENECON, can be used with one or two identical horizontal actuators simultaneously (Brandonisio *et al.*, 2008e-f).

Figure 30. Isolator testing machine at the University of Basilicata in Potenza.

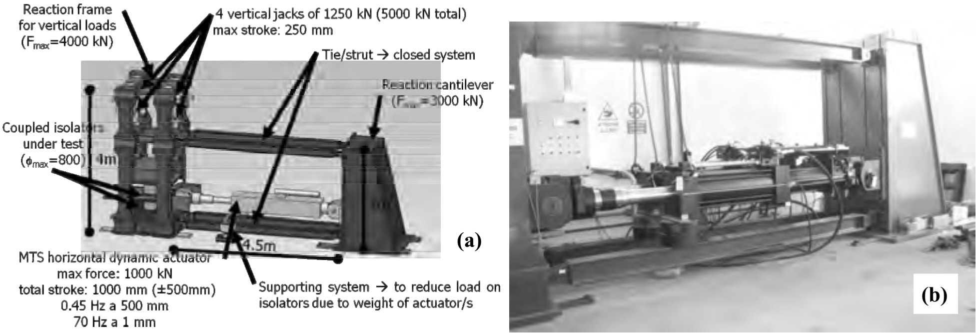


Figure 31. Isolator testing machine of BENECON: (a) characteristics; (b) with two horizontal actuators.

4.16 L7_SG3: buckling and roll-out of non-bolted elastomeric isolators

For a non-bolted laminated rubber isolator under a vertical load is possible to define two limit displacements: the one that corresponds to buckling instability and the other which produces roll-out. Marsico and Kelly (2008) showed that the buckling displacement is lower than the roll-out one under high axial stress on the isolator (Figure 32), and viceversa, and have then examined how these phenomena influence the overall safety level when isolators of large diameters are to be substituted by several small diameter ones having the same overall horizontal deformability and a certainly lower cost, with the further advantage of reducing the total number of different size of isolators to be manufactured for a defined construction.

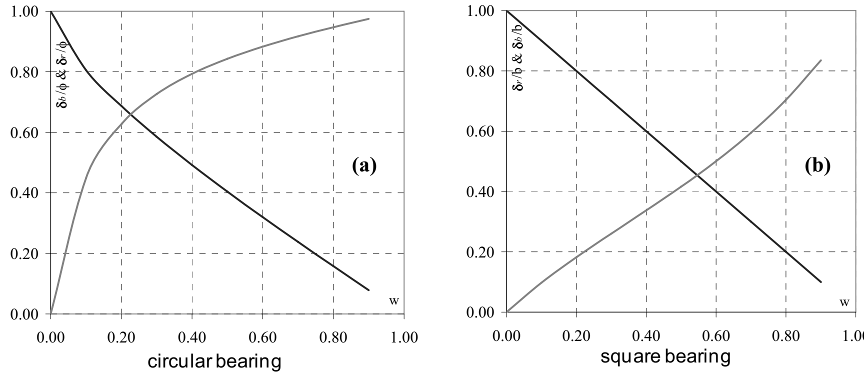


Figure 32. Adimensioned buckling (–) and roll-out (–) displacement: (a) circular and (b) square bearing.

4.17 L7_SG3: optimization and testing of aluminium-steel hysteretic dampers

A new passive metallic yielding device has been proposed, represented by an aluminium shear panel where energy dissipation is concentrated and two lateral steel hollowed plates having a stiffening function (Diaferio *et al.*, 2008a-b-c-d-e-f). The final dimensions of the device to be used in the JETPACS frame (Figure 33a) have been determined through an optimization process carried out with FEM analysis using the ABAQUS code, imposing the maximization of the elastic strain energy up to the required yielding force. For two different variants of the same device (with and without further lateral steel stiffening elements), cyclic loading tests at increasing displacements have been performed using a specifically designed test set-up (Figure 33b). It has been observed that dissipated energy is significant already in the first loading cycles and that the device with additional lateral stiffening, preventing out-of-plane instability, provides a better overall behaviour.

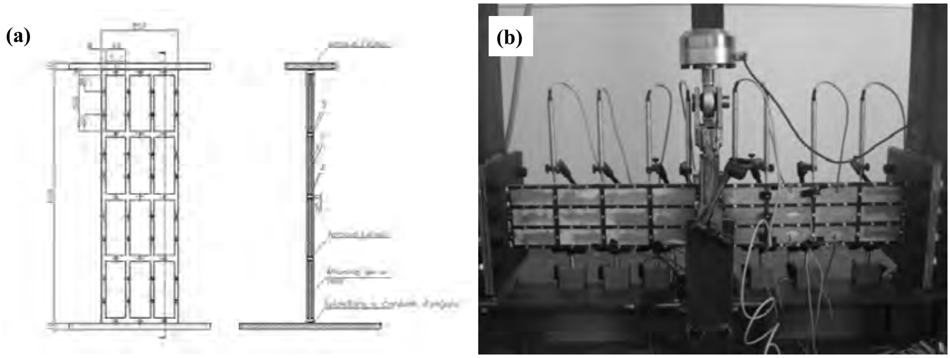


Figure 33. Aluminium-steel hysteretic damper: (a) dimensions for the JETPACS frame; (b) test set-up.

4.18 L7_SG4: tuned mass dampers for seismic response reduction

The evaluation of the potentialities offered by Tuned Mass Dampers (TMDs) and Tuned Liquid Dampers (TLDs) for seismic response reduction as well as their actual implementability in real structures has been the main scope of this activity. After a state-of-the-art and a state-of-the-practice of Tuned Mass Dampers (TMD) and Tuned Liquid Dampers (TLD), the development of analytical and numerical tools for studying TMDs' and TLDs' effectiveness and robustness as passive structural control systems for the seismic protection of buildings has been carried out. Stemming from the knowledge collected through the foregoing literature review, the idea of a new type of mass-uncertain TMD has been proposed and the corresponding analytical and numerical models developed, thus providing with the basic tools for more generally investigating into the effectiveness and robustness of TMDs for seismic response mitigation (De Stefano and Matta, 2006).

While looking for new motivations which could possibly increase the seismic benefit of TMDs, the activity stems from the idea of turning into TMDs those non-structural masses, such as roof gardens, tanks or technical installations, which are normally placed on top of buildings, in order to obtain a structural control system having low cost and minimal impact. Since such installations are susceptible of varying their mass (e.g. due to changes in the soil water content in the case of a roof-garden TMD), and therefore potentially their frequency and damping, assessing and enhancing robustness against mistuning is identified as a crucial issue for these new family of 'mass-uncertain TMDs' (Matta, 2007).

Hence, the activity can be summarized as follows: i) an innovative robust design has been formulated in the framework of a worst-case H_∞ approach as the necessary substitute for the classical nominal design, automatically providing the optimal selection of multiple TMDs' parameters for controlling single degree of freedom systems; the developed procedure has allowed on the one hand the robust analysis of systems with parametric uncertainties (both in the main structure and in the oscillators) and on the other the comparison between two alternative paradigms of single and multiple TMDs, namely the classic translational absorber and a new version of pendulum TMD, this latter configuration being preferred owing to the advantage of keeping its natural period unchanged under mass variations (Matta and De Stefano, 2008a; Matta and De Stefano, 2009a); ii) such new version has been conceived as a rolling pendulum device to minimize the non-linearity inherent in rolling systems undergoing large oscillations; to this purpose, analytical and numerical methods have been derived to divine the optimal iso-periodic rolling shape, both in 2 and 3 dimensions, resulting in the proposal of the 2d elliptical optimal profile and the 3d ellipsoidical optimal surface as the distinctive ingredients of the new device (Matta, 2008a-b); iii) numerical simulations under different excitation conditions have been conducted to evaluate effectiveness and assess the applicability range of the new device; the superior robustness of the pendulum system against uncertainty in the oscillating mass has been demonstrated with respect to the translational TMD through a parametric study; the results have shown that a translational TMD can still provide acceptable robustness only if its mass uncertainty is not too large and its mass ratio not too small; in all other cases, the pendulum system should be adopted (Matta and De Stefano, 2009b); iv) the feasibility of adoption of TMD for seismic response reduction has been finally demonstrated with reference to a building structure recently completed in a seismic site in Central Italy, designed to host a roof-garden atop for architectural purposes (Matta and De Stefano, 2008b-c-d).

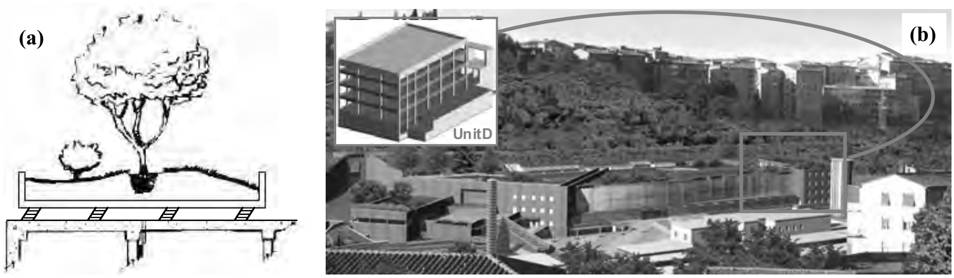
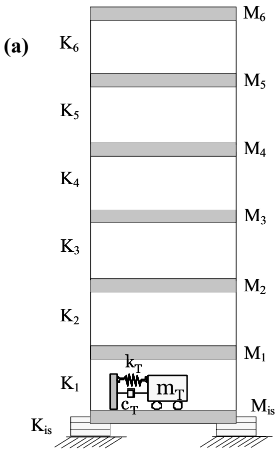


Figure 34. (a) The concept of roof-garden TMD; (b) the application to a building in Siena.

4.19 L7_SG4: base isolation and tuned mass damper combined strategy

Base isolation has been widely viewed as an effective strategy to protect structures subjected to seismic excitations. However, in the case of seismic inputs with high energy content at low frequencies, i.e. a near-fault event or a seismic wave propagating itself through alluvial soil, isolation bearings may undergo large deformations. By increasing the isolation layer damping, base displacements can be reduced, but high damping in the isolation layer unfavourably affects the behaviour of the superstructure due to spill-over effects. By observing that seismic response of a well-isolated systems is dominated by the first-modal contribution and that a Tuned Mass Damper (TMD) is able to reduce the fundamental vibration mode, a new idea is to combine both properties into a single system (Figure 35a). Considering the base isolated (BI) structure as a single-degree-of-freedom equipped with a TMD, and subsequently by applying the Laplace transform to the motion equations, it is possible to prove that the TMD works as a closed loop control on the isolation layer. The objective of the proposed combined system is to control the system response by only reducing the fundamental modal contribution which is dominant in such systems. This positive behaviour is due to the appropriate combination of three fundamental properties of the original systems: the reduction in ground motion transmission to the superstructure, the vibration mode modification due to the BI and the first vibration mode reduction by means of the TMD at this frequency. However, the inertia of TMDs does not allow devices to be immediately effective in the application of the control actions. Therefore, in the case of "near fault" seismic events, characterized by a sort of input energy impulse during the first few seconds of the earthquake, tuned mass damping shows itself to be an ineffective system in

controlling seismic response peak values. Otherwise, high effectiveness in reducing both base level peak displacement and RMS is observed for “far fault” seismic events.
The research activity has been focused on the effectiveness of the BI&TMD control strategy, and, in particular, the seismic response of a three-dimensional base isolated benchmark structure equipped with a TMD system has been investigated with the aim of evaluating the effect of the mass damper parameters on the seismic response of the isolated system, both in the case of linear and non-linear isolators (Palazzo *et al.*, 2006a-b; De Iuliis *et al.*, 2008). In terms of robustness, the obtained results show that a TMD system when applied to the base level of an isolated structure is much less sensitive to mistuning effects compared to the classical application of a TMD at the roof level of a fixed-base structure (Palazzo *et al.*, 2007b-c). Moreover, optimal design strategies to reduce the edge’s displacement in plan-wise asymmetric system by using mass damping strategy have been investigated, also toward the optimal design location for a Single Tuned Mass Damper (STMD) on varying the mechanical characteristics of the main system (Petti and De Iuliis, 2007a-b). Furthermore, a case study for a practical application in a strategic building (Villa D’Angri Hospital located in Potenza) has been proposed and its effectiveness has been numerically tested with regard to unfavorable seismic excitations (Palazzo *et al.*, 2006c; Palazzo *et al.*, 2007a). Finally, the efficacy and robustness of the combined BI+TMD strategy has been investigated through an experimental campaign conducted on small scale models on a Quanser shaking table (Figure 35b): first the main parameters (periods and damping ratios) have been identified, and then the experimental response to recorded seismic events on fixed base, base isolated and combined base isolated with tuned mass models has been analyzed, showing the overall superior performance of the proposed system (Petti *et al.*, 2008).



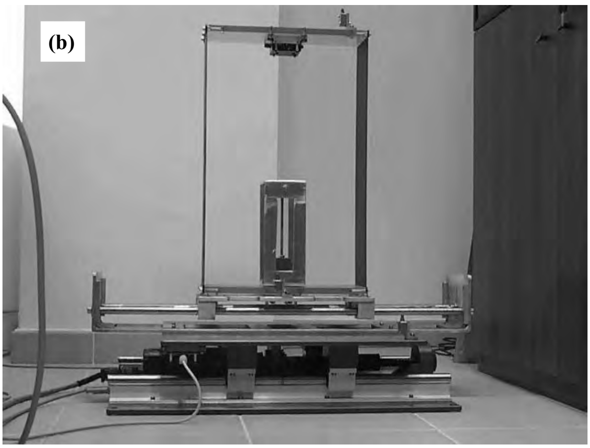


Figure 35. Combined BI+TMD: (a) basic scheme; (b) small scale experimental model.

4.20 L7_SG5: semi-active seismic control with magnetorheological dampers

The evaluation of the possibilities offered by semi-active control for seismic response reduction as well as their actual implementation in real structures was the main scope of L7_SG5. Although the first application of a semi-active control system on a real-scale buildings dates back to the 1993 as shown by Kobori *et al.* (1993), and a second remarkable example was described by Patten *et al.* in 1999, the number of civil engineering structures protected by these systems is relatively low, at the moment. These first applications adopted viscous dissipaters able to modify their mechanical parameters through the operations of added hydraulic circuits featuring one or more servo-valves. Therefore, these systems shared the common difficulty of dealing with electronic, mechanical and hydraulic parts, so that the time needed to adjust the parameters of the dampers was comparable to the structural natural period and, in turn, too long to achieve optimal performances. For this reason, a new generation of semi-active dampers based on the properties of magnetorheological (MR) fluids have attracted the attention of many researchers. A MR dampers may vary its mechanical properties in the time interval needed to modify the magnetic field in which the fluid is immersed: this time interval may be as short as few milliseconds. Due to this attractive behaviour, MR dampers seem to be the most effective way to introduce a semi-active control system in civil structures. However, the remarkable potentialities of semi-active control systems for seismic protection of civil structures could be largely underemployed, or even invalidated, due to the long time intervals in which they are forced in rest conditions, waiting for of an earthquake event. A further objective of the research was therefore the overcoming of such problem, designing the control devices against both recurrent dynamic actions (traffic, rotating machines, wind), and simultaneously for exceptional seismic actions.

MR dampers were usually modelled in numerical codes according to a phenomenological scheme originally proposed by Spencer *et al.* in 1997. This model is based on the Bouc-Wen evolutionary scheme and requires 10 parameters to describe the behaviour of a MR damper. The number of parameters increases to 14 when the fluctuation of the magnetic field inside the dampers is taken into account. Such a complex model is hard to include in more general numerical codes where both the dampers and the hosting structure should be described. Therefore, a simpler, 4-parameter model (7-parameter including the effects of a fluctuating magnetic field) has been developed and validated , based on the results of a past experimental campaign on a prototype MR damper (Occhiuzzi *et al.*, 2006). A further model, called the Weber model, was also studied and validated (Occhiuzzi and Caterino, 2008a; Occhiuzzi *et al.*, 2008) in connection with the experimental campaign carried out to completely characterize the two 30 kN MR dampers adopted in the JETPACS frame (Figure 36).

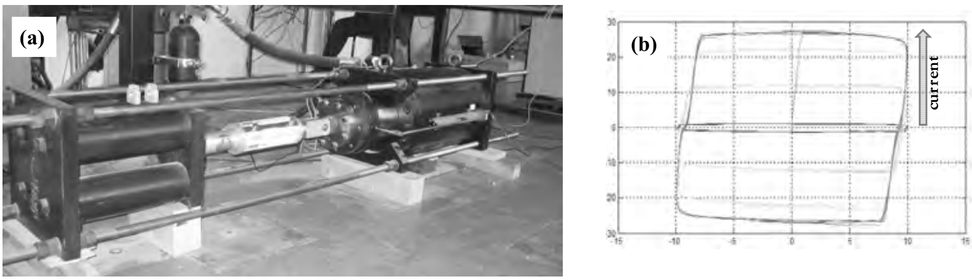


Figure 36. UNIPARTH MR dampers: (a) testing apparatus; (b) typical cyclic behaviour.

Once a given number of a selected kind of semi-active damper have been chosen to protect a structure, the possibility of having time-varying dynamic properties has to be exploited by an operation logic, i.e., a control algorithm. In the few real-scale applications and in the majority of laboratory implementations, control algorithms based on or derived from the classical theory of optimal control have been utilized. This kind of algorithms rely on a robust and consolidated theory, but they need a real time knowledge of the whole system state, i.e., the displacements and the velocities of all of the degree of freedom of the controlled structure have to be measured and fed to the control algorithm at any time. In the case of structural systems, real-time measurement of displacements and velocities yields a number of different

problems that can be too hard to solve. Therefore, control algorithms which require a limited number of real-time measurements are preferable for semi-active MR dampers. A numerical investigation carried out on a subset of 4 main control algorithm proposed in the literature applied to a structural model with semi-active bracings has clearly shown that the reduction of seismic structural response does not depend on the chosen control algorithm itself, once it has been adapted to semi-active control and its parameters properly optimized (Occhiuzzi, 2007a-b), similarly to what happens to the different available methods for the seismic design of passive dampers in a structure, which usually lead to very similar values of main modal damping ratios (Occhiuzzi and Caterino, 2008b).
With reference to the JETPACS project laboratory prototype, a discrete model has been purposely formulated in the reduced space of a few significant dynamic variables, and consistently updated to match the modal properties experimentally identified. The potential occurrence of significant eccentricity in the storey mass distribution, breaking the global structural symmetry, has also been considered. The adjustment of the damper characteristic was governed by a clipped-optimal control law, which entails the instantaneous comparison of the actual force with the active force evaluated through a non-collocated acceleration feedback designed according to the LQG method (Figure 37). Since the dampers are positioned purposely to deliver two eccentric and independent forces on the first-storey floor, the set-up allows the mitigation of the 3-D motion arising when mono-directional ground motion is imposed on the non-symmetric structure. Numerical investigations on the model controlled response to different natural accelerograms showed the effectiveness of the control strategy, analyzed through synthetic performance indexes (Carneiro *et al.*, 2007; Gattulli *et al.*, 2008b).

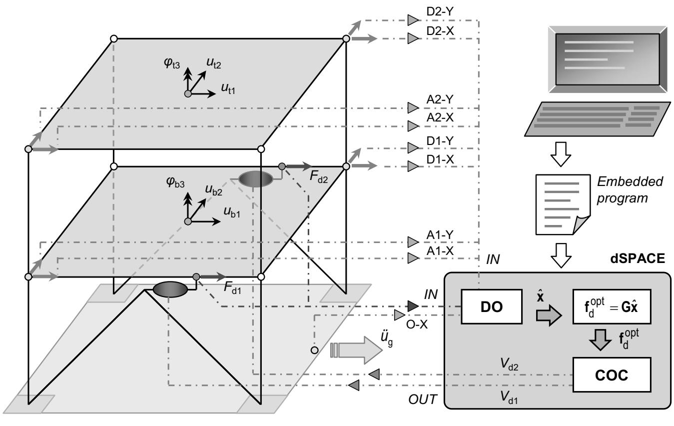


Figure 37. Non-collocated acceleration feedback control configuration for the JETPACS frame.

5 DISCUSSION

The technologies for the seismic isolation and control of structures and infrastructures is undoubtedly a vast subject, and still numerous issues remain to be completely solved from both a theoretical and a practical/implementation point of view. This is particularly true when reference is made to the new techniques making use of hybrid and semi-active control strategies, whose potentials and limitations have been evidenced in the present research. The overall initial program of ReLUIS Research Project Line No. 7 was certainly quite ambitious, however it can be said that most of its objectives have been finally reached. Particular attention has been devoted in the course of the project to the development of design procedures and practical implementation issues, today strongly requested by the engineering professionals interested in the application of the new technologies. For this reason, numerous have been the practical case studies and applications defined up to the execution details in the course of the project, also because among the expected final products there were a series of design manuals with applications examples for each of the protection technologies dealt with during the research. However, because of some of the reasons indicated below, this objective has not been completely reached, and only an advanced draft of a unified comprehensive design manual is now available at the end of the project.

On the other hands, it can be said that the initial objective of developing proposals for the improvement of the national seismic regulations regarding seismic isolation and passive energy dissipation systems and devices has been completely reached. The New Technical Regulations for Constructions (Min. Infr., 2008) and Commentary (Min. Infr., 2009) include paragraphs (§ 7.10: Constructions and bridges with isolation and/or dissipation; § 11.9: Antiseismic devices) specifically dedicated to the most consolidated technologies which have been the subject of ReLUIS Line No. 7 research. They represent an evolution of the Ordinances (PCdM 2003; PCdM 2005) issued after the collapse of a school in S. Giuliano di Puglia on 31st October 2002, which however included mainly specific rules on seismic isolation only, separately for buildings and bridges, and were partially in force when ReLUIS Executive Project started (November 2005). Undoubtedly, this development has been strongly influenced by the results of Line No. 7 research.

An activity, only partially included in the original program of ReLUIS Line No. 7 project, that has been particularly successful, also beyond the initial expectations, was the JETPACS (*Joint Experimental Testing on Passive and semiActive Control Systems*) experimental campaign. More than half (8 out of 13) of the Research Units of Line No. 7 project were directly involved, with a significant exchange of information and experiences among them. JETPACS allowed to compare design criteria for passive and semi-active energy dissipation systems and devices, to define common testing procedures, and finally to understand the advantages and drawbacks of each of the earthquake protection systems considered. Although some significant delays in the execution of the experimental tests occurred, caused among other things by some administrative complications and a not more deferrable significant upgrading of the hydraulic pumping system of the Structural Engineering Laboratory at the University of Basilicata in Potenza, JETPACS certainly represents one of the most fruitful and best accomplished joint experimental program with the ReLUIS 2005-2008 Executive Project, and an example to be followed in a possible future coordinated project. Because of the above mentioned delays, some of the experimental activities were completed just before the end of the project, and therefore some of the reports and papers fully describing the work done and the results obtained (all listed at the end of the paper) are still being completed in these months.

Another important achievement worth to be mentioned is the two-days final workshop of ReLUIS Line No. 7 project organized in Naples at PICO (Palace of Innovation and Knowledge) on 4th-5th December 2008. The workshop, whose proceedings are being put to press by a major publisher and include all the contributions listed at the end of this paper, saw the participation also of many researchers and professionals which did not belong to the Line No. 7 Research Units. It allowed to spread the information about the main results achieved to a vast public and gave the opportunity of a throughout discussion on many of the topics dealt with during the project.

Regarding the management and coordination issues, it is worth to mention that around the end of the first year of the project, the first Coordinator and Author of this paper was appointed as Head of the “Office for the evaluation, prevention and mitigation of seismic risk, activities and post-emergency operations" of the Italian Department of Civil Protection. Although he continued to be present at all the meetings and give important advices to the member of his original Research Unit and all the other participants, the much more important commitments he had inevitably prevented him to a full participation in the activities of Line No. 7 project. This caused Line No. 7 to be the only ReLUIS project (with the exception of Line No. 9 and 10 of much smaller size) ending to have a single Coordinator. However, the organization of the research in five subgroups each having two further Sub-coordinators, decided during the first year of the project, allowed to successfully overcome the difficulties regarding the management and coordination issues, and specifically the preparation of the intermediate and final documents required by the sponsoring agency.

6 VISIONS AND DEVELOPMENTS

On the basis of what has been presented and discussed above, a possible list of topics to be dealt with in a possible future ReLUIS Executive Project can be the following:

1. *Development and verification of low-cost seismic isolation systems*: the issue of the New Technical Regulations for Constructions (Min. Infr., 2008) and Commentary (Min. Infr., 2009) has definitively lead to a further significant increase of interest from the professional universe toward the seismic isolation technique, whose diffusion is however still limited to specific cases, also because of the relatively high cost of the isolation devices; the use of recycled rubber and transverse reinforcement made of composite material instead of steel could bring to a significant decrease of both cost and weight of the isolators, and also to the possibility to manufacture elements of large dimensions, to be easily cut in pieces according to the need; however, it is necessary to verify from the experimental and practical point of views that this type of devices have mechanical characteristics and long time durability able to satisfy code requirements.
2. *Study and development of integrated structural health monitoring and semi-active/hybrid control systems:* the availability today on the market of low cost integrated sensors (like MEMS), the rapid advancements of wireless technologies and the continuous decrease of cost of the conditioning, acquisition, actuation and processing hardware is making continuous structural health monitoring systems more and more easily available to a vast public; considering the fact that a semi-active dynamic control system always requires this kind of electronics to operate, it is easy to understand that they could be installed in a new or existing construction with only a small additional cost; also, the advantages may became very significant when semi-active control is used also to attenuate the effects of also dynamic excitations (like wind, automobile/train or pedestrian traffic, industrial machineries, other environmental and human-borne vibrations) more frequently occurring with respect to even minor earthquakes, because this may require a more continuous operation and thus continuous checking of the devices and system and thus avoid misfunctioning in the event of a dangerous seismic event.
3. *Drafting of a comprehensive design manual for structures equipped with passive, hybrid and semi-active earthquake protections devices and systems*: as it was said in the previous chapter, this activity has been started in the course of the project which has been just completed, with numerous case studies and application examples defined up to the

execution details. However, due to lack of time and resources, it has not been possible to complete this task, which however could be completed with a small additional effort.

4. *Support activities for seismic code development*: the New Technical Regulations for Constructions (Min. Infr., 2008) and Commentary (Min. Infr., 2009) represent, for the first time in Italy, a single and consistent normative body; however, because of their novelty and recent issuing, they still require a full verification in the field, to eliminate the inevitable inaccuracies and internal contradictions still present, also in view of the future definitive adoption of the Eurocodes; the activity indicated at point 3 above (drafting of comprehensive design manual) could certainly give a significant contribution in this direction.

More general topics, which go beyond the specific field of passive, hybrid and semi-active control, to be tackled in a possible future ReLUIS Executive Project, are:

5. *Studies on structures included in the Seismic Observatory of Structures (OSS)*: more than 100 are the structures (mainly public buildings and bridges) continuously monitored for seismic events by the OSS of the Italian Department of Civil Protection, with detailed information available relative to the location and dimensions of the structural elements, the mechanical characteristics of materials and developed FEM models at different stages of detail; in the last few years, a significant amount of data regarding their behaviour under more or less intense earthquakes has been collected, used mainly for detecting structural damage and post-emergency operations; however, the recorded available data contain important information regarding the actual behaviour of constructions in the event of an earthquake, never examined in detail; a possible idea could be to launch an call open to academic research groups for bottom-up proposal of studies on specific earthquake engineering topics based on the use and processing of the data of the OSS, with procedures similar for example to those adopted in the US by the California Strong Motion Instrumentation Program (CSMIP).
6. *Specific support activities for the Office III "for the evaluation, prevention and mitigation of seismic risk, activities and post-emergency operations" of the Italian Department of Civil Protection*: like development of numerical models of the buildings and bridges already included and of the new constructions to be included in the OSS, definition of procedures for the managing of the events and post-earthquake operations, organization of data-bases, etc.; the development of these activities would certainly bring to a better mutual knowledge (level of expertise and potentials) between Office III of DPC and the Italian academic research teams operating in the prevention of the seismic risk, with inevitable significant advantages not only for them but for the entire community in general and the efficacy of future joint projects.
7. *Experimental researches making use of the equipment acquired through ReLUIS funding during the 2005-2008 Executive Project*: in order to appraise the significant value of the equipment acquired, priority should be given to the future experimental researches which will make use of them.

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INNOVATIVE MATERIALS FOR THE VULNERABILITY MITIGATION OF EXISTING STRUCTURES

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1 INTRODUCTION

The use of fiber reinforced polymer (FRP) materials for the strengthening of masonry and concrete structures, represents a valid alternative to traditional techniques. Indeed, many advantages are provided by using FRPs: lightweight, good mechanical properties, corrosion-resistant, etc.

In Italy, the use of FRP materials for reducing the seismic vulnerability of existing structures has been allowed for the first time through O.P.C.M. 3274 and more recently by the D.M. 14.01.2008, that refer to the Italian National Research Council Design Guidelines (CNR-DT 200/2004) for the external strengthening of existing structures with FRP materials.

These guidelines provide, within the framework of the Italian regulations, a document for the design and construction of externally bonded FRP systems for the strengthening of existing structures. In particular, several issues concerning the seismic rehabilitation of Reinforced Concrete (RC) and masonry buildings have already been dealt but a further investigation is still required.

Within this context, the main aim of this research line has been the experimental validation of design indications provided by the CNR-DT 200/2004 guidelines.

The main topics investigated in this research task can be summarized as follows:

- the mechanical behaviour of FRP materials;
- the cyclic behaviour of RC elements strengthened by means of FRP;
- the mechanical and chemical anchorage devices for FRP systems;
- the ductility increasing of RC columns confined with FRP;
- the RC joint strengthening with FRP;
- the masonry strengthening with FRP;
- the historical structure strengthening with FRP;
- the quality control and monitoring of FRP applications;
- the innovative fibers (steel fabrics, natural fibers, FRP grids, etc.) and matrices (organic and inorganic);
- the mechanical behaviour of concrete structures reinforced with fiber reinforced polymer bars;
- innovative strengthening techniques (near-surface mounted (NSM) technique, FRP prestressed systems).

2 BACKGROUND AND MOTIVATION

The research activity has been performed through experimental tests and theoretical studies mainly devoted to the development of simple methods of analysis and design rules in order to improve the indications provided by CNR-DT200-2004.
FRP are ideal products for structural retrofitting and seismic upgrading. Nonetheless the small knowledge on the durability of the system is one of the main drawbacks to the use of FRP reinforcement in Civil Engineering. In particular, structural adhesives usually represent the weakest point of the reinforced system and their mechanical behaviour and durability performance need to be investigated. The first problem in using composite materials for structural reinforcement is the determination of their mechanical properties. The bond between FRP and concrete is a very important issue because the debonding is a very brittle failure mechanism and must be avoided.
According to performance-based design or seismic evaluation of RC buildings, it is crucial to provide a correct evaluation of the strength and ductility capacity of the RC columns and beams as well as of beam-column joint. Experimental and direct observation of damages occurred during recent earthquakes strongly highlighted this need.
The effectiveness of FRP systems for seismic vulnerability mitigation of masonry structures is still in debate, despite it has moved a huge interest, becoming the outstanding system in the market for this type of applications.
Indeed CNR DT 200/2004 has been the first guideline to provide design criteria for the FRP seismic strengthening of masonry buildings. However, the retrofit design of masonry structures is still not a completely solved problem. This is due to the fact that the masonry structure is load dependent and thus the FRP could be placed in an inactive area of the resistant mechanism. Furthermore, masonry can activate a large number of local mechanisms which interact with global behaviour of masonry buildings. The non linear seismic assessment of FRP reinforced masonry structures is included also in the D.M. 2008 rule. The non linear analysis requires the knowledge of the constitutive law of the masonry material both in the unreinforced or strengthened situations.
In the recent years, the scientific research has been focused on the safeguard of historical buildings. Accordingly, CNR DT 200/2004 has been published in order to provide design criteria for the use of FRP systems for strengthening existing structures and to avoid their incorrect application. The Guidelines deal with different types of FRP applications to masonry and reinforced concrete structures and take into account the important phases of quality control and monitoring that should follow a strengthening application. Several aspects affect the effectiveness of FRP systems such as the surface preparation and FRP installation. Moreover, once FRP strengthening intervention has been carried out, monitoring by non-destructive or semi-destructive tests should be performed to ensure the quality and effectiveness of the strengthening system. It is worth noting that due to the increased number of composite material applications and in order to get a better understanding of the interaction force between FRP materials and the masonry substrate, experimental tests are needed. In this research line, semi-destructive and non-destructive techniques have been also investigated for the quality control and monitoring of FRP applications to masonry structures, according to CNR DT 200/2004 Guidelines.

3 RESEARCH STRUCTURE

In order to guarantee an optimal organization of the research, the Research Units have been grouped into the following ten Tasks, each one with a specific topic:

- Task 8.1: the mechanical characterization of FRP systems at fixed environmental conditions under cyclic actions;
- Task 8.2: the delamination under cyclic actions and design of anchorage mechanical devices for FRP systems;
- Task 8.3: the confinement of RC and masonry columns subject to combined flexure;
- Task 8.4: the strengthening in flexure and in shear of RC structural elements with FRP fabrics and near surface mounted (NSM) rods;
- Task 8.5: the beam-column and beam-foundation joint reinforcement with FRP;
- Task 8.6: the design criteria for the seismic retrofit of RC and RC-masonry composite structures with FRP;
- Task 8.7: the design criteria for the seismic retrofit of masonry structures with FRP;
- Task 8.8: the strengthening of masonry structural elements with FRP systems;
- Task 8.9: the strengthening of masonry vaulted elements with FRP systems;
- Task 8.10: the quality control and monitoring of FRP applications to existing masonry and RC structures.

4 MAIN RESULTS

4.1 Task 8.1: The mechanical characterization of FRP systems at fixed environmental conditions under cyclic actions

The aim of the sub-task was to mechanically characterize FRP systems. The study has been focused on durability and mechanical behaviour of structural adhesives and FRPs, the mechanical characterization of FRP bars and strips, the values of the safety factors proposed in CNR DT 200/2004 and the effects of elevated temperatures and freeze-thaw cycling on FRP.

Durability and mechanical behaviour of structural adhesives and FRPs

Several tests to determine the mechanical properties of composite materials and structural adhesives have been performed. Conforming to the ASTM requirements, the glass transition temperature (ASTM D3418), porosimetry and the coefficient of thermal expansion (ASTM D360) were determined. Adhesives were also tested under tensile (ASTM D360), compressive (ASTM D695) and flexural loading (ASTM D790). Adhesive shear strength was determined by punch tool tests (ASTM D732). Finally adhesive cylinder specimens were tested under pure torsion load.

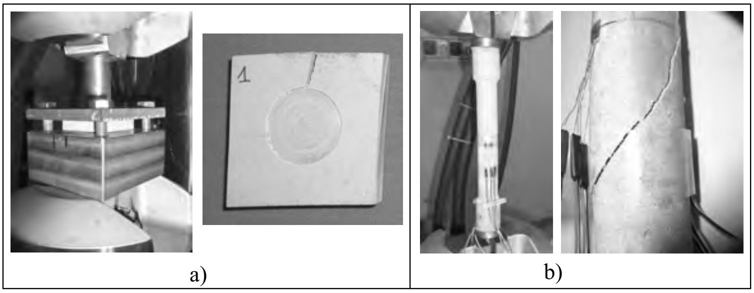
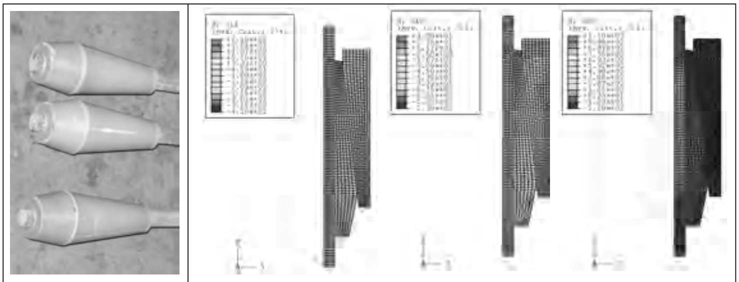


Figure 1. a) Execution of the punch-tool test and specimen after collapse, b) execution of the torsion test and specimen after collapse.

Adhesive dumb-bell specimens were prepared for tensile testing and then artificially aged in an environmental chamber in order to analyze possible detrimental effects on the adhesive mechanical properties. Exposition to deicing salts, freeze-thaw cycles and moisture may in fact deteriorate the mechanical properties with consequences on the durability performances of strengthened structures. Tensile tests were performed conforming to the requirements of ASTM D360. In all the conditioning treatments, significant losses in adhesive stiffness and tensile strength were measured. The stiffness and tensile strength reductions after exposure to salt spray fog solution may be approximated by straight parallel lines as described in the Arrhenius life-temperature relationship. Fatigue tests on adhesive dumb-bell specimens were finally performed to attain the fatigue failure curves for the adhesive joint.

Mechanical characterization of FRP bars and strips

Tensile and relaxation tests were performed on FRP bars with particular attention to the gripping system. Then, experimental tests and numerical simulations were performed to develop simple, economical and effective systems for the characterization of composite materials and adhesives. In particular, an anchor system for tension testing of unidirectional fiber reinforced plastic (FRP) bars of large diameter was developed. In the system suggested each end of the bar is embedded in a conical polymeric head that fits a conical hole inside the anchoring device. In the anchor system, the anchor body shape came from experiences for testing steel ropes and prestressing steel tendons and the shape of the resin head from test investigation. Numerical analyses were also performed to investigate the effects of anchor parameters such as cone slope angle, thickness of resin head and friction coefficient between the anchor body and the resin head. Pull-out and beam tests were also executed.



a) b)

Figure 2. a) Anchor system for large diameter GFRP bars, b) numerical analysis.

Experimental studies and numerical analyses were developed to define practical tests for the characterization of FRPs and adhesives mechanical properties. The main aim of this action was to provide the “Composites Kit Test - COKIT”; a practical tool for professionals and engineers operating in the field of FRPs applications and dealing with FRP materials for structural retrofitting and rehabilitation. The technical document “Istruzioni per la caratterizzazione ed il controllo di accettazione di materiali fibrorinforzati per il rinforzo strutturale – COKIT ” was thus published and could represent an annex of the CNR DT 200/2004 Recommendations.

Refinement of the safety factors proposed in Design Recommendations

The environmental conversion factors provided in the guidelines of the Italian National Research Council (DT200) were analyzed on the basis of the results of artificially aged adhesive specimens tested under tension. Exposition to deicing salts, freeze-thaw cycles and moisture leads to the deterioration of the mechanical properties of composite materials and in particular structural adhesives. On the basis of the experimental results, the safety factors suggested in the CNR DT 200/2004 recommendations may be considered as appropriate, but in aggressive environments the use of a slightly lower conversion factor seems to be more suitable.

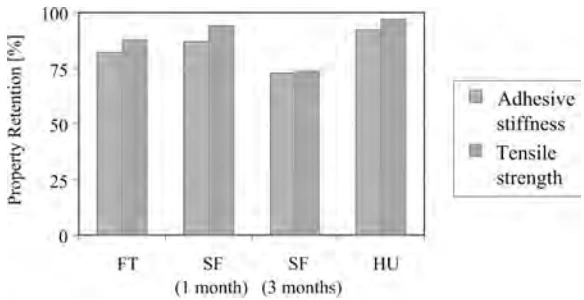


Figure 3. Stiffness and tensile strength retention for the structural adhesive subject to freeze-thaw cyclesof five hours each between –18° and +4°C for a total duration of about 2 months (FT), to salt spray fog for one month or three months (SF) and to one month humidity (HU).

Tests were performed to refine the safety factor of FRP-steel systems: the fatigue behaviour of steel structures retrofitted by using FRP materials was investigated, S–N curves were defined and the fatigue resistance of the steel-CFRP bond was compared to the one of welded detail categories described in the Eurocode 3.

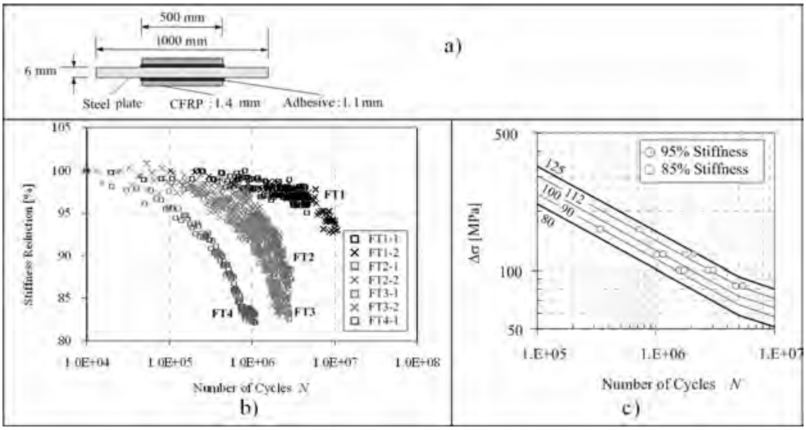


Figure 4. a) Steel-CFRP specimen b) Reduction in stiffness of retrofitted specimens during fatigue tests; c) S-N curve and comparison between the fatigue resistance of the steel-CFRP bond for a stiffness reduction of 5% (blue circles) and of 15% (red squares) and that of EC3 welded detail categories.

After performing pull-pull delamination tests on FRP-concrete specimens, cylinders were obtained from each concrete prism. Based on Eurocode 2 compressive and splitting tests were carried out to determine the conditioning effects on concrete degradation. As a consequence of the environmental conditioning, concrete characteristic strength is assumed to increase by 16% for salt spray fog conditioned specimens and to decrease by 3% for specimens subject to freeze-thaw cycles.

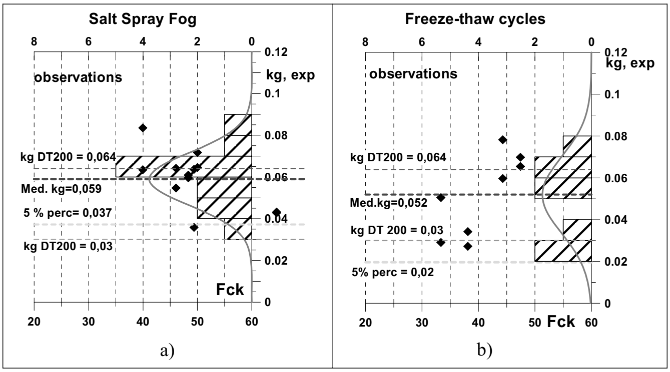
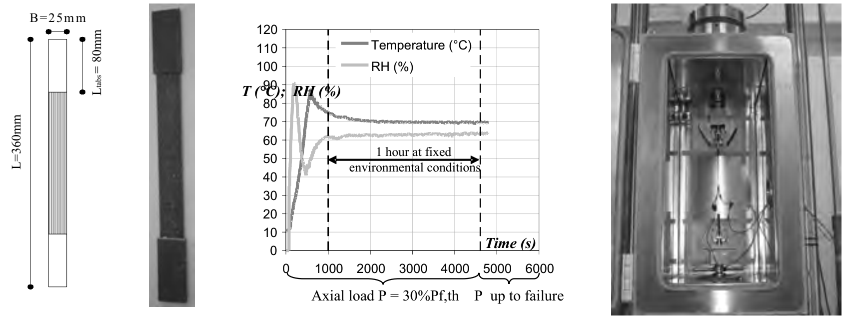


Figure 5. Statistical distribution of the coefficient kG for specimens subject to a) salt spray fog and b) freeze-thaw cycles.

Effects of elevated temperatures and freeze-thaw cycling on FRP laminates behavior

The performances at elevated temperatures and/or at freeze-thaw cycling exposure of structural members strengthened by using externally bonded FRP laminates are mainly related to two aspects: the bond behaviour between FRP and the member substrate; the mechanical properties of laminates themselves. The latter aspect has been very limited experimentally investigated; only few tests have been performed to evaluate the residual tension strength of FRP coupons after exposure to elevated temperatures or freeze-thaw cycling. Thus, experimental tension tests on carbon FRP (CFRP) laminates both under controlled temperature and relative humidity conditions or after freeze-thaw cycles exposure have been carried out. In particular, due to reduced capacity that commercially available resins have to transfer loads over fibres around glass transition temperature, T_g, two new systems based on epoxy resin have been formulated and characterized by dynamic mechanical analysis (DMA). The main goal of the new formulated systems was to increase T_g, the elastic modulus in the rubber region of the resin and to improve their performances under freeze-thaw cycles. Two different approaches were investigated. First a new epoxy system (namely neat epoxy) was formulated and cured at 60°C after an hour at room temperature. Secondly, in order to improve the mechanical properties of epoxy matrix by curing at room temperature, a nanocomposite system was obtained by direct dispersion of preformed nanodimensioned silica particles to the neat epoxy resin.



(a) (b) (c)

Figure 6. Specimen geometry; Temperature and relative humidity exposure profiles; test setup.

The experimental results point out that the developed formulations of epoxy resins provide a significant increase of ultimate strength and strain of CFRP coupons both at room and elevated temperatures with respect to commercial systems, without significant change of the elastic modulus. Negligible influence of a low number of freeze-thaw cycles was observed on the mechanical properties of coupons independently of matrices. Experimental outcomes strongly confirmed that the use of matrices characterized by higher values of T_g and elastic modulus in the rubber region with respect to those traditionally available on the market, could allow to overcome one of the main limit of FRP laminates related to their poor performances under elevated temperatures.

4.2 Task 8.2: The delamination under cyclic actions and design of anchorage mechanical devices for FRP systems

Different experimental set-ups can be found in the scientific literature dealing with FRP-concrete bond tests and it has been observed that different test methodologies may give different values of the debonding force. This task research intended to define a standard FRP-concrete bond test to be used to evaluate the maximum transmissible force by an FRP anchorage, to be included in the new version of the Italian code for design of strengthening interventions with FRP.

Experimental round robin test on FRP concrete bonding

An extensive experimental campaign on FRP-concrete debonding has been carried out by five different Italian Laboratories (University of Bologna, University of Naples Federico II, University of Sannio, Polytechnic of Milan and University of Calabria). The tests were devoted to the definition of a standard test procedure for the bond strength evaluation. According to the Round Robin procedure, 50 concrete prisms (same batch) strengthened with CFRP plates and sheets have been prepared by the same operator and subject to bonds test in five different Laboratories. The sets of homogeneous specimens have then been subject to bond test by five laboratories of the University partners using different test set-ups (Figure 7).

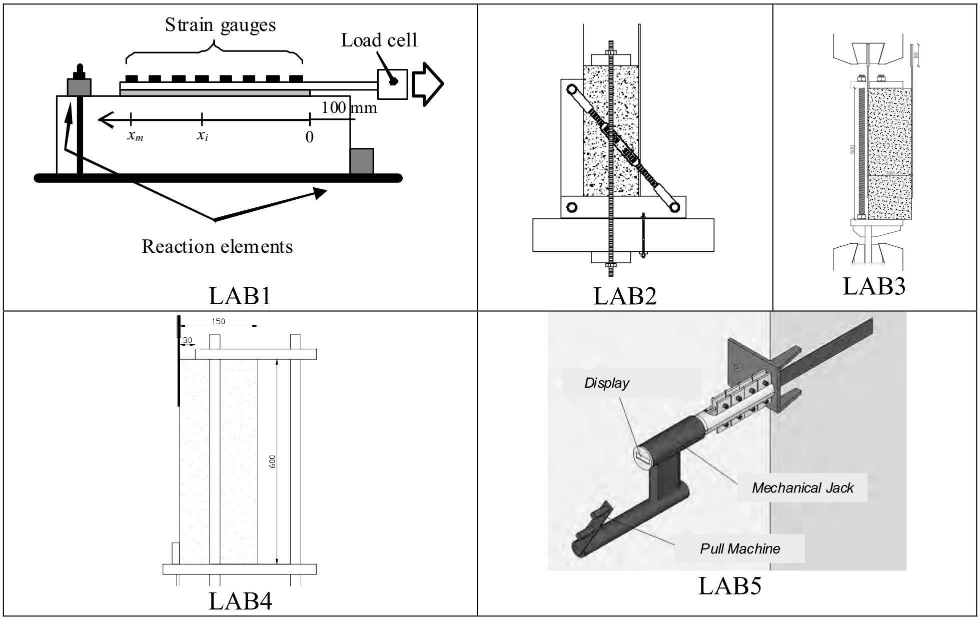


Figure 7. Experimental set-ups adopted by the five different Laboratories.

Twelve specimens (6 strengthened with sheets, 6 strengthened with plates), with two different bonded lengths (100 mm and 400 mm), have been tested by each laboratory, repeating three times the same type of test. As for the test set-ups (Figure 7), all the Laboratories adopted a single shear push-pull test. All the tests have been performed under displacement control of the FRP free end. In order to evaluate the variability of the results when different set-ups are adopted, the coefficient of variation (COV) for each set of homogeneous experimental tests has been calculated. The scatter of the results is in general small (COV about 10%), lower than that of the tension strength of the concrete, usually equal to 20-30%. For the plates, the scatter of the results is similar for the different Labs, whilst for the sheets the dispersion is usually higher. The results obtained by Lab 3 are very stable in both cases and close to the mean values. This study allowed to define a set of rules for the standardization of bond tests to be used to evaluate the maximum transmissible force by an FRP – concrete anchorage.

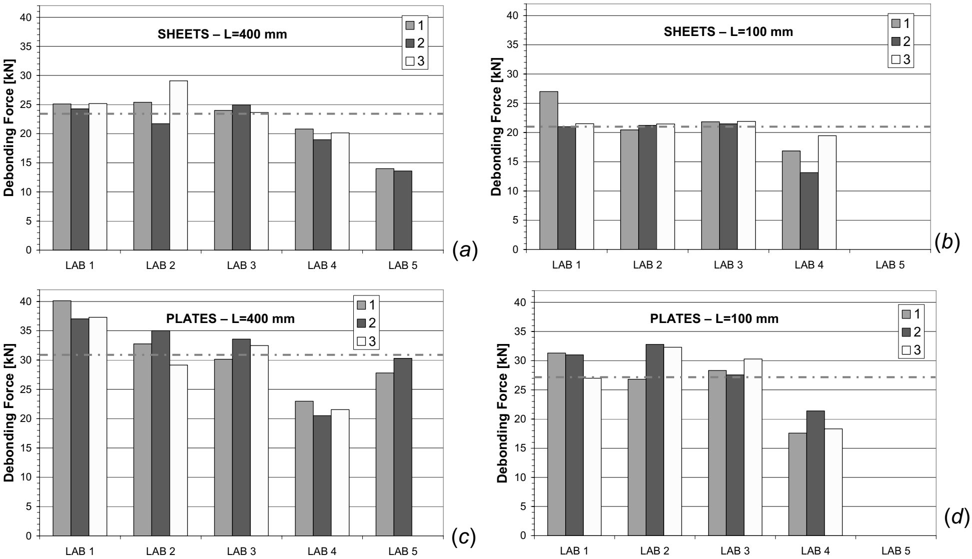


Figure 8. Debonding force for sheets with anchorage length (a) L=400 mm, (b) L=100 mm, and for plates with anchorage length (c) L=400 mm, (d) L=100 mm.

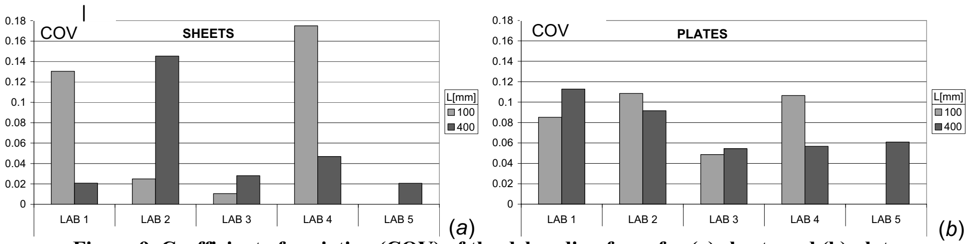


Figure 9. Coefficient of variation (COV) of the debonding force for (a) sheets and (b) plates.

Cyclic tests of FRP-concrete debonding under cyclic loadings

Many strengthened structures are subjected to fatigue loads (i.e. roads and railways bridges) or to shorter but more intense cyclic actions as seism: in this cases, the FRP-concrete interface is subject to cyclic stress regimes which can lead to premature debonding of the FRP laminate from the concrete substrate and cause the FRP failure in most cases, unless appropriate local measures are taken to prevent it.

In order to develop a more economical design for FRP-strengthened structures, research line investigated the debonding phenomenon of the FRP reinforcements (both plates and sheets) from the concrete substrate under cyclic actions. In particular, tests on little prismatic specimens have been performed by applying FRP plates and sheets on concrete prisms and testing them under both monotonic and cyclic actions without inversion of sign.

The experimental results of Single Shear Test (SST) performed on CFRP reinforcement applied on little prismatic concrete specimens and characterized by high bond length values (400mm) showed that:

- the influence of load-unload cycles up to 70% of $P_{max,M}$ was negligible for CFRP sheets and plates;

- a low number of load-unload cycles (40) up to 90% of $P_{max,M}$ reduced the debonding load of about 10% in the case of CFRP plates but did not affect particularly the bonding behaviour of CFRP sheets;
- by increasing the number of load-unload cycles (up to 300) between 70% and 90% of $P_{max,M}$, the debonding load of concrete specimens reinforced with CFRP sheets decreased by a percentage factor equal to 10%;
- the transfer of shear stresses at the FRP-to-concrete interface due to the CFRP reinforcement bond length larger than effective one, allowed to mitigate noticeably the effect of cyclic actions imposed up to 90% of $P_{max,M}$;
- a degradation of interface behaviour has been recorded after the onset of debonding, with reduction of maximum shear stress;
- the effects of cyclic actions were more significant on plates rather than sheets and its influence increased with number of cycles;

Moreover similar SST tests performed on CFRP reinforcements characterized by lower bond length values (50-250mm) allowed to observe that:

- design relationships provided by Teng et Al. and by main international codes for evaluating the effective bond length values are conservative for both sheet and plate reinforcements;
- referring to plates the effective bond length values, experimentally evaluated by means of the monotonic tests, were noticeably lower than predicted;
- even if reinforcement bond length values were significantly low, the cyclic tests outcomes confirmed that the influence of load-unload cycles up to 70% of $P_{max,M}$ was negligible for CFRP sheets and plates due to elastic behaviour characterizing FRP-concrete interface up to such load level;
- Also a further low number of load-unload cycles (10) up to 90% of $P_{max,M}$ did not affect particularly the bonding capacity of CFRP reinforcement due to the transfer of shear stresses at the FRP-to-concrete interface. Such transfer was more significant on plates rather than sheets: nevertheless, bond lengths particularly conservative for plates allowed to better mitigate cyclic action effects;
- in order to better predict design bond length values, using two different relationships for sheets and plates, respectively, could be worthwhile.

4.3 Task 8.3: The confinement of RC and masonry columns subject to combined flexure

The main goal of this research task has been to validate the design equations provided by CNR DT200 for the confinement of RC and masonry members. In particular, experimental tests have been carried out on both real scale and scaled columns wrapped by using traditional FRP (CFRP and GFRP) or an innovative typology of FRP system made of basalt material fiber.

Confinement of real scale RC columns subject to axial load

An experimental campaign has been carried out on full scale reinforced concrete (RC) columns concentrically loaded and confined by means of FRP (Glass FRP and Basalt FRP). Five series of tests were planned, for each series a reference unconfined column was tested and used as benchmark. The test matrix has been designed to assess the confinement effectiveness: applying the same reinforcement ratio and checking the effect of the shape, the side aspect ratio and the area aspect ratio; applying different FRP reinforcement ratios and checking the confinement sensitivity to the number of plies.

TYPE S-1 base lowG [5 plies] basalt [8 plies]	Height: 3.05 m 61 cm 61 cm	TYPE R-1 base lowG [5 plies] basalt [8 plies]	Height: 3.05 m 51 cm 74 cm
TYPE HR base lowG [5 plies] highG [8 plies]	Height: 3.05 m 51 cm 74 cm	TYPE WALL base lowG [5 plies] highG [8 plies]	Height: 3.05 m 36 cm 104 cm
TYPE R-0.5 base highG [5 plies] basalt [8 plies]	Height: 3.05 m 36 cm 51 cm		

Figure 10. Test Matrix.

Tested specimens represent real scale building columns designed according to dated codes for gravity loads only. The design concrete strength is 23.1 MPa to simulate concrete mixes used in past decades. Concrete cylinder specimens per each casting have been prepared in order to characterize the concrete with standard procedures. The used steel is characterized by a yield strength of 414 MPa and a modulus of elasticity of 200 GPa. Particular care has been devoted to construction details, namely: hooks, longitudinal and transverse steel reinforcement ratios and concrete cover specifications. Special care has been taken to avoid local failure at the top and the bottom ends of the columns placing steel ties with reduced spacing. Internal steel (bars and ties) reinforcement ratio is the same for each group of specimens, designed per minimum code requirements. The minimum specimen dimensions are 360 x 510 mm^2. The load has been applied concentrically under a displacement control rate. The load has been conducted in five cycles in increments of one fifth of the expected capacity for each specimen. Each loading-unloading cycle has been repeated once. Strain gages, potentiometers and LVDTs are used for strain and displacement data acquisition. In particular, strain gages have been applied on column surface and internal bars whereas LVDTs have been placed in order to obtain vertical and horizontal column displacement. Strain data acquisition have been obtained by strain gages applied on FRP sheets too.

(a) Column S-1-5GA (b) Column R-1-8H

Figure 11. Columns strengthened by means of glass (a) and basalt (b) FRP.

The data have been elaborated in order to investigate on volumetric strain and poisson ratio as a function of load level. The main results of the experimental campaign can be summarized as follows: a significant increasing in the axial displacement and a little increase in the ultimate load of the FRP strengthened columns compared to their benchmark.

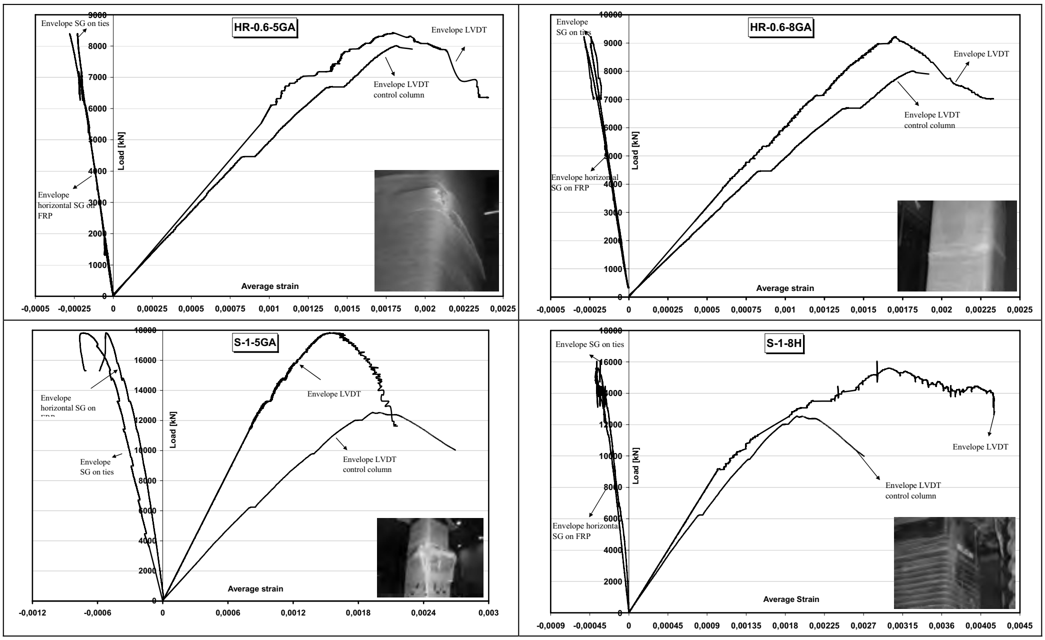


Figure 12. Load versus vertical and horizontal strains.

The hollow columns have shown a failure mode characterized by bulging, followed in such case by the rupture of the fibers. A more evident failure mode has been shown by the remaining columns for which the fiber rupture has always accompanied the concrete spalling.

(a) Hollow column	(b) Solid column

Figure 13. Typical failure modes.

Confinement of RC cylindrical specimens strengthened by means of basalt fibers and inorganic matrix

The effectiveness of such system as a confinement technique has been analyzed by means of an experimental campaign on concrete cylindrical specimens. The effectiveness of the proposed technique is assessed by comparing different confinement schemes: 1) uniaxial Glass Fibre Reinforced Polymer laminates; 2) alkali-resistant fibreglass grid bonded with a cement based mortar; 3) bidirectional basalt laminates pre-impregnated with epoxy resin or latex and then bonded with a cement based mortar; 4) cement based mortar jacket. The main objectives of the experimental program were: a) to investigate on the effectiveness of confinement based on basalt fibres pre-impregnated in epoxy resin or latex and then bonded with a cement based mortar (BRM); and b) to compare the performance (in terms of peak strength and ultimate axial strain gains) of different confinement techniques using advanced materials with respect to GFRP laminates jacketing.
The investigation was carried out on 23 concrete cylindrical specimens with a diameter of D = 150 mm and a height of H = 300 mm.

1st layer of mortar application (4mm)	Pre-impregnation of laminates	Basalt laminates installation	2nd layer of mortar application (4mm)

Figure 14. BRM wrapping installation procedure.

Experimental outcomes showed that:

- BRM confining system could provide a substantial gain both in compressive strength and ductility of concrete members inducing a failure mode less brittle than that achieved in the GFRP wrapped members;
- lower performance were observed by concrete confinement provided by a primed glass fiber grid bonded with cement based mortar with respect to BRM and almost no influence was generated by the jacketing with mortar only.
- maximum ultimate axial strain increases were provided by GFRP laminates wrapping.

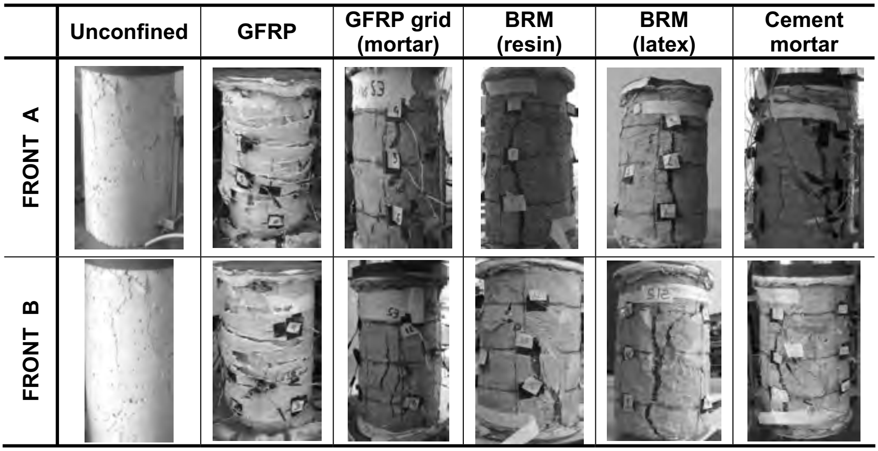


Figure 15. Failure modes.

Confinement of rectangular masonry columns subject to axial load

An experimental campaign dealing with 18 square cross-section both listed faced tuff and clay brick masonry scaled columns subjected to uniaxial compression load. In particular, three different confinement schemes were experimentally analyzed in order to evaluate and compare the effectiveness of the proposed strengthening techniques: 1) uniaxial glass FRP laminates (GFRP) wrapping; 2) uniaxial carbon FRP (CFRP) laminates wrapping; and 3) uniaxial basalt FRP (BFRP) laminates wrapping. In particular 9 tests, were performed on square tuff masonry (external tuff blocks and inner core filled with tuff chips and mortar) scaled columns (mass density equal to about 1530 kg/m^3): side average dimension equal to 220mm; and average height of about 500 mm corresponding to 8 courses of tuff bricks (height-width ratio equal to 2.27). Masonry was made by scaled yellow Neapolitan tuff bricks (50x50x100mm) and a pozzolan (local volcanic ash) based mortar (thickness of 12mm). Further 9 tests were performed on square clay brick masonry scaled columns (mass density equal to about 1700 kg/m^3): side average dimension equal to 260 mm, and average height of about 560 mm corresponding to 8 courses of clay bricks (height-width ratio of 2.20). Masonry was made by clay bricks (55x115x255 mm) and a pozzolan (local volcanic ash) based mortar (thickness of 13 mm).

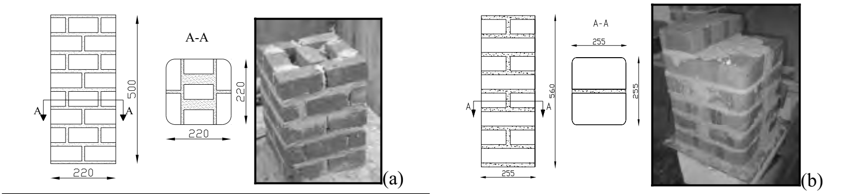


Figure 16. Specimen details (dimensions in mm): (a) tuff masonry; (b) clay brick masonry.

Masonry columns were tested through monotonically applied axial compressive loading under displacements control mode with a rate of 0.005 mm/s.

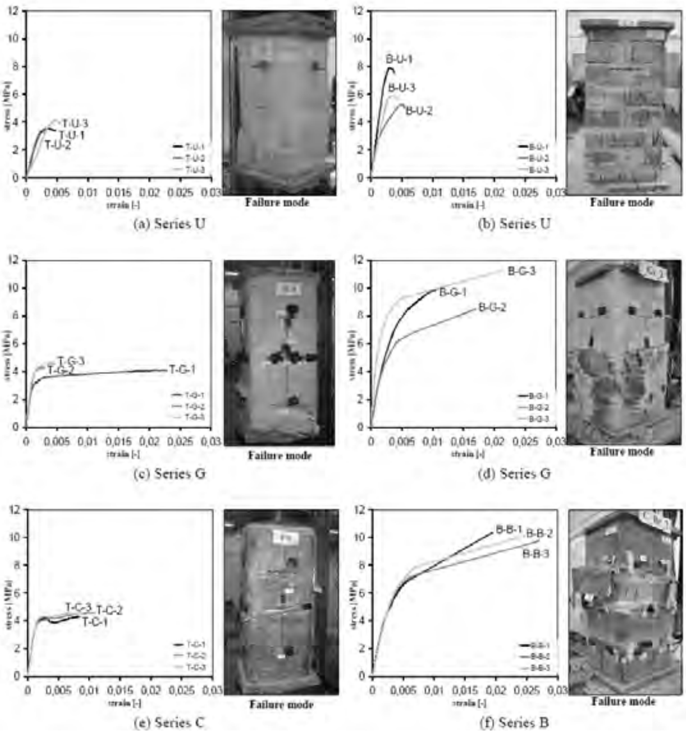


Figure 17. Stress-axial strain relationships and specimens' failure mode: (a),(c) and (e) tuff masonry; (b),(d) and (f) clay brick masonry.

The experimental outcomes showed that:

- GFRP and CFRP jackets led to similar of compressive strength gains on tuff masonry columns under axial loads.
- GFRP and BFRP confining system led to similar compressive strength gains of brick masonry columns under axial loads. BFRP wrapping was more effective in terms of global ductility increase (i.e. ultimate strain gain equal to 413% and 259% for BFRP and GFRP wrapping, respectively) even if the mechanical external reinforcement ratio of FRP laminates was lower then GFRP ones; such result could be explained by the higher values of ratios $\varepsilon_{fl}/\varepsilon_{fu}$ recorded on BFRP laminates.
- The use of high values of laminates unit height may significant reduce the effectiveness of FRP wrapping systems since it could be detrimental to the quality of confinement execution.
- The presence of voids and protrusions on masonry members reduces the ultimate transverse strain on FRP reinforcement with respect to that typically achieved on concrete members.

Another experimental campaign has been carried out in order to show the behavior of columns built with clay or with calcareous blocks, commonly found in southern Italy, especially in historical buildings. Rectangular masonry columns were tested for a total of 33 specimens; uniaxial compression tests were conducted on columns taking into account the influence of several variables: different strengthening schemes (internal and/or external confinement), curvature radius of the corners, amount of fiber-reinforced polymer (FRP) reinforcement, cross-section aspect ratio and material of masonry blocks. Materials characterization was preliminarily carried out including a mechanical test on plain masonry. For all cases the experimental results evidenced a significant increase in load carrying capacity and ductility after FRP strengthening, which identified the columns as ductile elements despite the brittle nature of the unconfined masonry. Differences in mechanical behavior, due to the geometry of the columns, to the nature of different materials, to different strengthening schemes, and to the amount of reinforcement, have been taken into account. The calibration of design equations recently developed by Italian National Research Council, CNR was conducted to compare analytical prediction and experimental results.

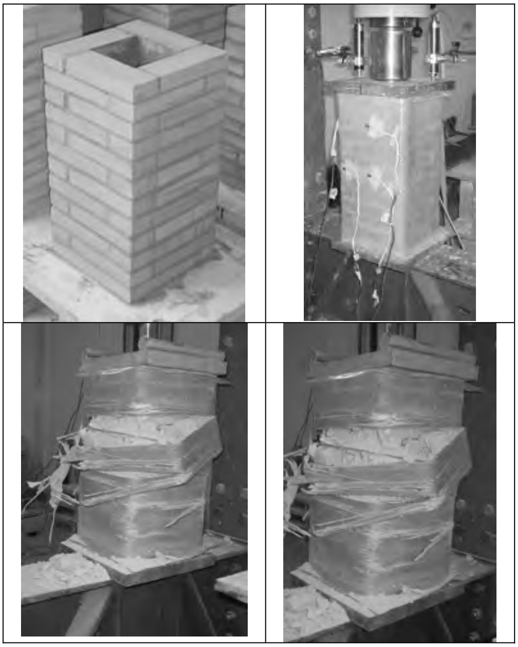


Figure 18. Limestone and clay brick masonry specimens.

The results obtained from the experimental campaign confirmed that innovative strengthening techniques, using FRP sheets and bars, are effective when confinement of masonry compressed elements is needed. Two types of masonry were investigated: the first made with clay bricks, the second made with limestone blocks. Even if the properties of the constituent materials were different, in both cases a significant increase was measured in terms of peak load and ultimate axial deformation. Two construction schemes were considered: full core and hollow-core columns; the last type reproduces the patterns often found in historical buildings. External and internal FRP confinement were tested, separately and combined. The proposed techniques are strongly recommended when a seismic retrofit is needed, since the external confinement introduces a plastic behavior of the compressed masonry which indicates a large capacity in storing elastic energy which is taken by the fibers placed in the transverse direction. The presence of internal bars used as an internal confinement system is recommended in addition to external FRP layers if ductility constitutes a main issue, since in columns strengthened only with bars the ultimate load was increased but brittle behavior of unconfined masonry remained. Columns with hollow core also showed a significant increase of mechanical properties when confinement was applied, especially in the cases of GFRP external sheets combined with internal bars.

Confinement of circular masonry columns subject to axial load

An extended experimental investigation has been performed in order to show the mechanical behavior of circular masonry columns built with calcareous blocks that may be commonly found in Italy and all over Europe in historical buildings. Different stacking schemes were used to build the columns, aiming to simulate the most common situations in existing masonry structures. Carbon FRP sheets were applied as external reinforcement; different amounts and different schemes of confining reinforcement were studied. The experimental program included a new reinforcement technique made by using injected FRP bars through the columns cross section. The structural behavior of masonry columns damaged under different levels of load and strengthened by using FRP reinforcements has been also investigated.

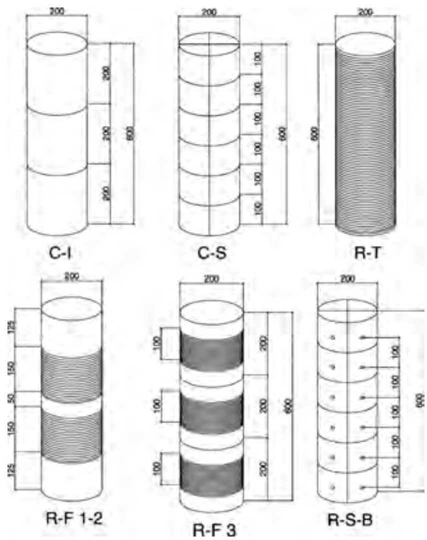


Figure 19. Specimens geometry and dimensions (in mm).

Important remarks follow:
• High increase in ultimate strength and strain were evident after strengthening;
• Complete FRP jacketing was much more effective than discontinuous wraps;
• Displacement capacity resulted increased in all cases; strengthened columns tested showed an extended postpeak plastic branch in the load versus displacement curves;
• Columns confined with three 100 mm wide sheets showed higher mechanical properties with respect to the same columns confined with two 150 mm wide sheets;
• Damage caused by overloads applied in the precracking stage before strengthening did not reduce the mechanical properties of FRP-confined columns;

• Presence of internal FRP rebars acted as an effective confining system for cross sections composed by four blocks;
• Application of design equations by Italian CNR furnished conservative results for complete FRP wrapping, whereas predic-tion of strength for masonry confined with CFRP strips showed a reduced scatter with respect to experimental results.

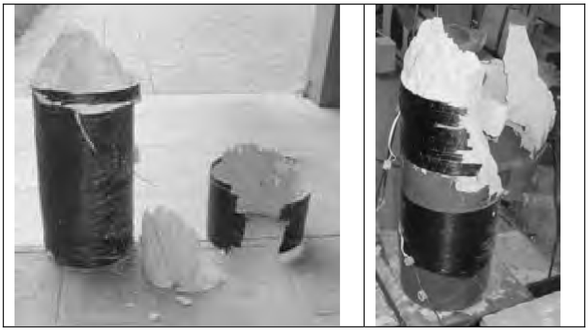


Figure 20. Specimens after failure.

4.4 Task 8.4: The strengthening in flexure and in shear of RC structural elements with FRP fabrics and near surface mounted (NSM) rods

A new technique for the shear and flexural strengthening of RC structural elements has been investigated in this research task. In particular, the use of Near Surface Mounted rods (NSM) for structural upgrading has been deeply analysed by both analytical and experimental investigations.
Further, the effectiveness of FRP laminates, traditionally used to strengthen RC or masonry members, has been investigated with reference to full scale prestressed concrete (PC) girders.

NSM bars shear contribution: a calculation procedure

A calculation procedure suitable for practitioners has been developed by simplifying a more sophisticated predictive model recently developed (Bianco 2008; Bianco *et al.* 2009a-b). That procedure briefly consists of: a) evaluating the *average structural system* composed of the average-available-bond-length NSM strip confined to the corresponding concrete prism whose transversal dimensions are limited by the spacing between adjacent strips and the beam cross section width (Figure 21); b) determining the comprehensive *constitutive law* of the average system above (Figure 22); c) determining the *maximum effective capacity* that the average system can attain during the loading process of the strengthened RC beam by imposing a *kinematic mechanism* and d) determining the NSM *shear strength contribution* by summing the contribution provided by each strip. The constitutive law (Figure 22) and in turn the equations to determine the maximum effective capacity assume different features depending on the main phenomenon characterizing the ultimate behaviour of the average structural system of the specific case at hand. Hereinafter, for the sake of brevity, the main features of that computational procedure are shown only for the case of shallow concrete fracture ($u = 4$) and a resulting resisting bond length whose value is equal to the effective bond length (Figure 23). Further details can be found elsewhere (Bianco 2008).
The predictions obtained by that calculation procedure were also appraised on the basis of experimental results (*e.g.* Dias *et al.* 2007).

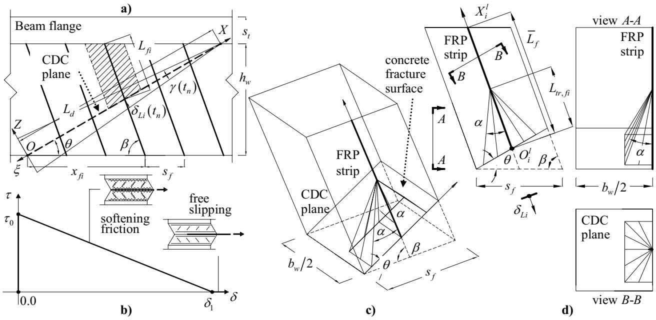


Figure 21. Main features of the calculation procedure: a) average-length NSM strip and concrete prism of influence, b) adopted local bond stress slip relationship, c) NSM strip confined to the corresponding concrete prism of influence and semi-pyramidal fracture surface, d) sections of the concrete prism.

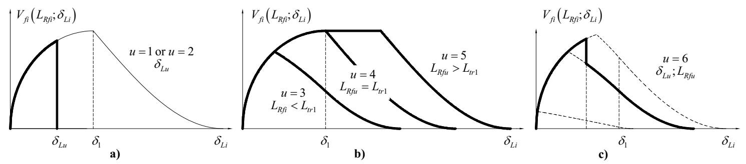


Figure 22. Possible comprehensive constitutive law of an NSM CFRP strip confined within a prism of concrete: (a) concrete that reaches the free extremity ($u=1$) or strip tensile rupture ($u=2$), (b) superficial and/or absent concrete fracture and ultimate resisting bond length smaller ($u=3$), equal ($u=4$) or larger ($u=5$) than the effective bond length and (c) deep concrete fracture ($u=6$).

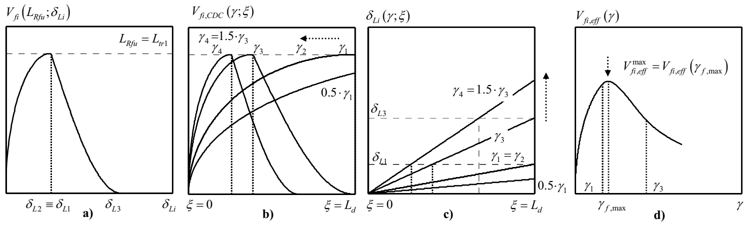


Figure 23. Maximum effective capacity along the CDC for the case $u=4$: a) comprehensive constitutive law ; b) capacity $V_{fi,CDC}(\gamma;\xi)$ and c) imposed end slip $\delta_{Li,CDC}(\gamma;\xi)$ distribution along the CDC for different values of the CDC opening angle γ and d) effective capacity as function of the angle γ .

The maximum effective capacity for the case of shallow concrete fracture and a resulting resisting bond length whose value is equal ($u=4$) to the effective bond length can be evaluated by:

$$V_{fi,eff}^{max}=\frac{1}{L_d}\cdot\left\{A_1\cdot C_1^{sf}\cdot L_d^2\cdot\gamma_{f,max}+\frac{A_2\cdot C_2^{sf}}{2\cdot A_3\cdot\gamma_{f,max}}\cdot\left[\arcsin\left(1-A_3\cdot\gamma_{f,max}\cdot L_d\right)+\left(1-A_3\cdot\gamma_{f,max}\cdot L_d\right)\cdot\sqrt{1-\left(1-A_3\cdot\gamma_{f,max}\cdot L_d\right)^2}-\frac{\pi}{2}\right]\right\} \quad (1)$$

where:

$$A_1=-\frac{L_p\cdot J_3\cdot\lambda^3\cdot\sin(\theta+\beta)}{4\cdot\tau_0\cdot J_1}\ ;\ A_2=L_p\cdot J_3\cdot\lambda\ ;\ A_3=\frac{\lambda^2\cdot\sin(\theta+\beta)}{2\cdot\tau_0\cdot J_1}\ ;\ C_1^{sf}=\delta_1-\frac{\tau_0\cdot J_1}{\lambda^2};C_2^{sf}=-\frac{\tau_0\cdot J_1}{\lambda^2} \quad (2)$$

$$\gamma_{f,\max} = \gamma_1 = \frac{2 \cdot \delta_1}{L_d \cdot \sin(\theta + \beta)} \tag{3}$$

Actual V_f and design value V_{fd} of the NSM shear strength contribution can be obtained as follows:

$$V_{fd} = \frac{1}{\gamma_{Rd}} \cdot V_f = \frac{1}{\gamma_{Rd}} \cdot \left(2 \cdot N^l_{f,\text{int}} \cdot V^{\max}_{fi,eff} \cdot \sin\beta\right) \tag{4}$$

where γ_{Rd} is the partial safety factor divisor of the capacity that can be assumed equal to 1.1-1.2 according to the indeterminateness of the input parameters.

Bond between NSM bars and surrounding concrete: experimental and analytical investigation

Pull-out test were carried out to investigate both the qualitative and quantitative influence of some of the involved parameters on the bond performance (De Lorenzis and Galati 2006, Galati and De Lorenzis 2006). Those parameters encompass: ratio between depth and width of the slit, kind of epoxy-based adhesive used as binding agent, distance of the NSM bar from the edge of the concrete prism, distance between adjacent bars and employment of external FRP strips used to confine the joint. Tests were carried out by means of a tangential-pull device to apply the load, LVDT transducers to measure the slip at both the loaded and unloaded extremity and strain-gauges throughout the adhered length of the bar to measure the deformations along the joint. The measured quantities were processed to obtain the local bond stress-slip relationship for the different values of the test parameters. Cyclic tests were also carried out subjecting the joint at a limited number of cycles whose maximum load was assumed equal to different percentages of the peak static load. The cyclic tests were useful to evaluate the joint residual strength such as the one following a seismic action.
An analytical investigation has followed the experimental program above (Rizzo and De Lorenzis 2007-2009b). In fact, the local bond stress-slip relationship obtained in the pull-out tests has been modelled by suitable analytical functions whose unknowns were calibrated for the different values of the test parameters. The local bond stress-slip relationship obtained by the cyclic tests was also modelled by analytical functions. Then, the numerical solution of the governing differential equation has allowed the peak pull-out load be determined as function of the available bond length.
The pull-out tests were also simulated by a FE model, both in the Linear and Non Linear range. The Linear FE model was adopted to evaluate the bond-induced stresses on a plane transversal to the bar, evaluating the maximum stresses for different values of the geometrical and mechanical parameters of the joint and estimating so, local tangential stress inducing the first-cracking in both resin and concrete. After that, a Non Linear model was developed by modelling: a) the several materials according to the fracture mechanics and b) concrete/adhesive and adhesive/FRP interfaces by employing interface elements.

Experimental and analytical investigation on the shear strengthening contribution provided by NSM FRP bars on RC beams

Four points bending tests were carried out on RC beams strengthened in shear by NSM FRP bars (De Lorenzis and Rizzo 2006, Rizzo and De Lorenzis 2006-2009a). Those beams were designed in such a way that the theoretical failure mode, for both the strengthened and un-

strengthened beams was due to shear-tension. Parameters investigated were: spacing, type and inclination of the NSM bars and the shear-span-to-depth ratio. Some beams strengthened by NSM strips were also tested in order to asses the relative effectiveness of the two techniques. The system of FRPs was extensively equipped to measure the deformations in the bars crossing the CDC. Tests have highlighted the possibility of a global failure modes consisting in the detachment of the strengthened cover from the underlying beam core. Such mechanisms had not been pointed out by previous investigations.
Two models were developed to predict the NSM shear strength contribution: a) one more simplified and b) a more sophisticated one. The former was based on the Mörsch truss and the employment of a perfectly plastic local bond stress-slip relationship. The latter takes into account a more realistic local bond stress-slip relationship and the interaction between existing steel stirrups and NSM bars. The different local bond stress-slip relationships obtained in the former phase of the investigation were employed to carry out some comparison. From those comparisons it was possible to point out the great importance of the fracture energy as opposed to the shape of the local bond stress-slip relationship. This phase of the investigation has led to the development of useful formulae for the evaluation of the NSM FRP shear strengthening contribution to a RC beam.

Experimental investigation on full-scale prestressed concrete beams strengthened by means of CFRP

Every year, several prestressed concrete (PC) bridge girders are accidentally damaged by over-height vehicles or construction equipment impact. Although complete replacement is sometimes deemed necessary, repair and rehabilitation can be far more economical, especially when the time and the social cost of the method are drastically reduced.
The numerous advantages provided by the use of FRP laminates are leading in a sharp increase on their use for bridge construction strengthening. Experimental investigations were conducted in order to validate such strengthening technique on PC damaged members and accurately assess the upper limit of damage amount beyond which FRP laminates are no longer adoptable as repair solution.
Starting from such purposes, an experimental campaign was conducted on five full-scale (13.0m long, 1.05m high) PC double T-beams with a reinforced concrete slab, designed according to ANAS (Italian Transportation Institute) standard specifications. One beam was used as control, and the other four were intentionally damaged in order to simulate a vehicle impact by removing the concrete cover and by cutting a different percentage of tendons (17% on two specimens and 33% on the remaining two). The repair, by using externally bonded carbon FRP (CFRP) laminates, aimed at restoring the ultimate flexural capacity of the member, taking particular attention to the laminates anchoring system. In particular, one test was performed on the control beam (referenced as S1), two tests were carried out on intentionally pre-damaged, to simulate an over-height vehicle collision, beams (named S2 and S3, respectively) and the remaining two on pre-damaged specimens upgraded by using two and three plies of CFRP laminates anchored by using U-wraps (named S4 and S5, respectively).
In Figure 24 and Figure 25 the test setup and experimental load deflection curves are reported.

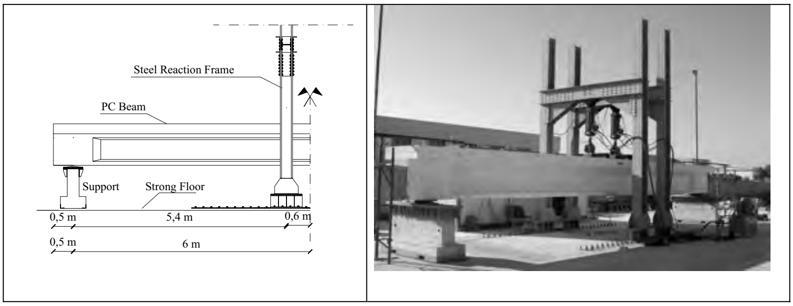


Figure 24. Test setup.

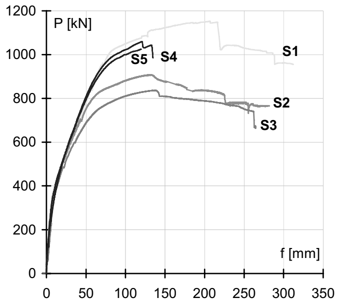


Figure 25. Experimental load deflection curves.

The experimental study has shown that: 1) a loss of strands equal to 17% and 33% caused a flexural capacity decrease equal to 20% and 26%, respectively; 2) to restore the ultimate flexural capacity of the undamaged PC specimen by using CFRP laminates it is necessary to prevent fibers debonding; 3) U-wraps (width w_f= 100mm spaced at p_f=150mm) were able to significantly delay debonding but if damaged existing concrete is patched by cementitious mortar, a perfect bond has to be guaranteed during the cross section restoration to prevent localized debonding of longitudinal reinforcement and thus fully exploit the potential effective FRP strain increase; 4) CFRP laminates increased both stiffness and flexural moment capacity of PC damaged beams (maximum moment recover equal to about 12% and 20% for specimens with 17% and 33% of strands loss, respectively; 5) the strengthening intervention led to weak failure mode with a global ductility loss.

The experimental outcomes qualify the application of FRP technique, already adopted in several cases of impacted PC bridges, as an effective tool to restore the flexural capacity of PC girders; however the calibration of theoretical expressions for the computation of the design FRP strain level considering the benefits provided by anchoring systems is strictly necessary.

4.5 Task 8.5: The beam-column and beam-foundation joint reinforcement with FRP

According to performance-based design or seismic evaluation of RC buildings, it is crucial to provide a reliable evaluation of the strength and ductility capacity of the beam column joints:

experimental and direct observation of damages occurred during recent earthquakes highlighted this.

This task focused on some aspects: namely, cracking of the joint panel, longitudinal reinforcement bars slipping are deformability sources and they could alter the capacity and interaction of beam and column members and the joint itself. F.E.M. modeling was adopted as an assessment tool. The finite element code TNO DIANA 9.1 was adopted to simulate and to analyze numerically some real beam column joint sub assemblages, characterized by nonlinear mechanical properties and geometrical detailing, smooth bars, structural deficiencies, as commonly found in existing buildings. Such deficiencies were analyzed by means of parametric analyses to evaluate the possibility to apply external strengthening on members characterized by poor concrete quality, low transverse reinforcement ratios, inadequate confinement due to lacking stirrups (especially in external joints), low bond performance of smooth and ribbed longitudinal reinforcement in columns and beams. Numerical analyses evidenced typical failure modes, crack patterns, influence of mechanical and geometrical properties on the behavior of joints. Some numerical/experimental comparisons were made based on significative tests available in scientific literature (for instance performed by Prof. Shiohara working group) or tested, during the RELUIS Project, by UNIBAS R.U., allowing the numerical F.E.M. model to be validated. The behavior of the joints controls the global seismic behavior of an entire structure and a building in particular, so that its assessment is a crucial task in the strengthening design. Based on such analyses, the validity of analytical models for unstrengthened joints, available in scientific literature, was checked. Such check was supported by local information provided by the detailed, refined, F.E.M. analyses. This numerical tool was a primary tool to understand the damage evolution and to assess the reliability of the main assumptions, equations and procedures according to the “Quadruple flexural resistance in reinforced concrete beam-column joints” (Shiohara, 2001) proposed by Prof. Shiohara working group. To provide a direct, practical tool, oriented to the profession more than a nonlinear refined F.E.M. analysis, it was evaluated the opportunity to extend such consolidated model to the case of externally bonded FRP strengthening of beam column joints.

This model is based on the solution of a system of equilibrium equations referred to the four rigid bodies in which the joint can be ideally divided, sometimes neglecting compatibility. This model is able to account for different failure modes and for the contribution of externally bonded FRP strengthening. The main assumptions can be recalled:

- Diagonal cracks in the joint form an angle of about 45°
- Normal concrete stresses acting on the main cracks can be reduced to an equivalent force
- Longitudinal reinforcement provides only axial forces, so that any dowel action is neglected
- There is a global symmetry both on the horizontal and vertical plane

The joint shear can be evaluated according to the sketch in Figure 26.

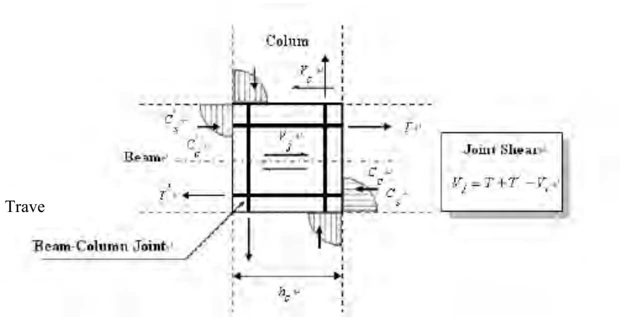
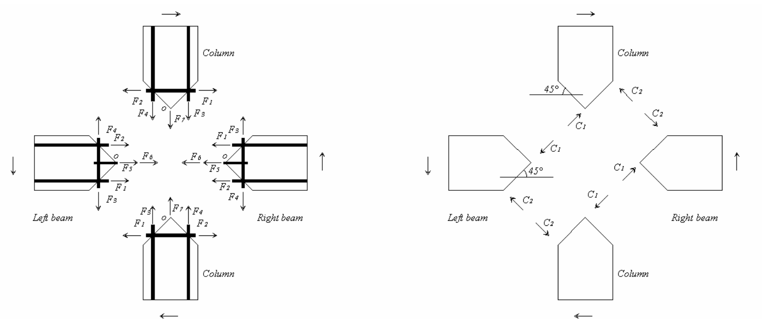


Figure 26. Details of the forces due to the elements converging in the joint panel.

$$V_f = T + C_s^{'} + C_c^{'} - V_c \tag{1}$$

$$V_f = T + T^{'} - V_c \tag{2}$$

where T and $T^{'}$ are the tractions in the longitudinal bars at the joint section, respectively; $C_c^{'}$ is the compression force in concrete, while $C_s^{'}$ is the compression force in the bars. V_j is the joint shear; and V_c is the column shear. In Figure 27 the joint division is shown: there are four rigid and interacting bodies. Each body can be associated to three equilibrium equations. The symmetry of the joints allows the system to be reduced to six equations. Moreover, Figure 3 shows the relation between the column shear, V_c , and joint shear, V_b, based on the equation $V_b = \mu \cdot V_c$, where $\mu = l_c / l_b$.



(a) Forces in reinforcement and FRP (b) Forces in concrete

Figure 27. Internal forces.

The independent equilibrium equations are five. Horizontal and vertical forces equilibrium and moment equilibrium based on point o (the middle of the joint panel) can be expressed as follows:

$$\sum F_x = 0, \quad -F_1 - F_2 - F_3 + C_1 \sin\theta + C_2 \sin\theta - N_b - F_6 = 0 \tag{3}$$

$$\sum F_y = 0, \quad F_3 - F_4 + C_1 \sin\theta - C_2 \sin\theta + \mu V_c = 0 \tag{4}$$

$$\sum M_o = 0, \quad \frac{l_b}{2}\mu V_c + \frac{1}{2} j_b (F_1 - F_2) + \frac{1}{2} j_c (F_3 - F_4) - \frac{l_j}{2} C_2 = 0 \tag{5}$$

Where N_b is the horizontal force in the beam; F_1, F_2 are tractions in the longitudinal bars of the beams at the joint section; F_3, F_4 are tractions in the longitudinal bars of the columns at the joint section; F_5 is the traction in the stirrups spread along the height of the joint and reduced to an equivalent force; F_6 is the traction in the FRP reinforcement in the x direction and reduced to an equivalent force; C_1, C_2 are compressions as shown in Figure 2; j_b, j_c are the internal lever arms in the beam and column respectively; θ is the inclination angle of the main cracks and assumed equal to 45°; V_c is the column shear; l_c is the column length; and l_b is the beam length.

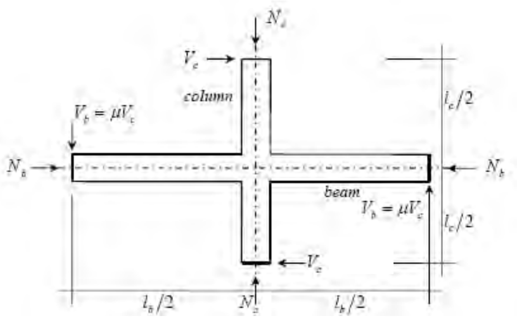


Figure 28. The joint system and the external forces.

Horizontal and vertical equilibriums of forces follow:

$$\sum F_x = 0, \qquad F_1 - F_2 + C_1 \sin\theta - C_2 \sin\theta + V_c = 0 \tag{6}$$

$$\sum F_y = 0, \qquad -F_3 - F_4 + C_1 \sin\theta + C_2 \sin\theta - N_c - F_7 = 0 \tag{7}$$

Where N_c is the vertical force in the column; F_7 is the traction in the FRP reinforcement in the y direction and reduced to an equivalent force.

To evaluate the column shear capacity of an unstrengthened joint, the F_1, F_2 e F_5 forces can be assumed equal to the yielding forces of the corresponding steel reinforcement while F_6 e F_7 are equal to zero. The five unknowns are V_c, F_3, F_4, C_1 and C_2 and can be evaluated solving the system (3)-(7).

To evaluate the column shear capacity of a strengthened joint, the F_6 and F_7 forces can be equal to the strength capacity of the external FRP reinforcement both in the x and y direction, respectively. To account for both the tensile failure of the strengthening or for a possible FRP debonding, the FRP capacity is given by the minimum between tensile strength and debonding force evaluated, for instance, according to CNR DT200.

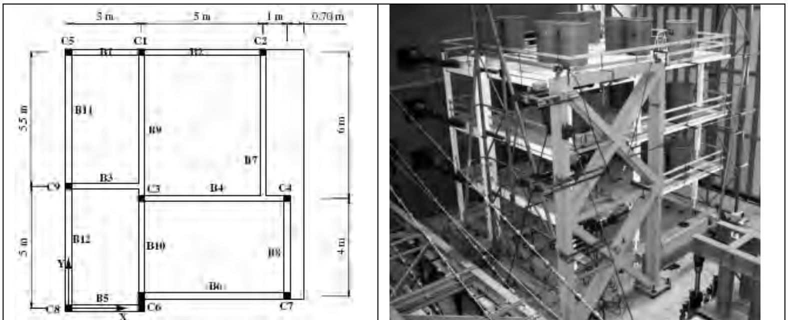
4.6 Task 8.6: The design criteria for the seismic retrofit of RC and RC-masonry composite structures with FRP

The applications of Carbon FRP (CFRP) and Glass FRP (GFRP) materials have grown during last years; at present, seismic applications have become comparable if not more frequent than those related to lacks due to gravity loads. The Italian guidelines for FRP interventions (CNR-DT 200, 2004), deal with the use of composite materials to seismically upgrade under-

designed RC and masonry structures. From seismic standpoint, FRP strengthening is regarded as a selective intervention technique. Based on the main deficiencies of the existing structure, the driving principles of the intervention are based on two main strategies: 1) preventing potential brittle failure mechanisms (i.e. shear failure, lap splice failure, buckling of longitudinal reinforcement in compression) and “soft story” collapse mechanism, and 2) increasing global deformation capacity of the structure, either by enhancing the ductility of plastic hinges without their relocalization or establishing a correct hierarchy of strength by relocalizing the plastic hinges according to capacity design criteria. The retrofit strategy is obtained by combining the above principles; the definition of the retrofit scheme depends on the gap between actual and target performance of the specific structure, costs, functional characteristics and importance of the structure.
Experimental studies, aimed at validating the effectiveness of FRP to achieve the above goals, are reported in the following.

Seismic strengthening of an under-designed RC structure with FRP

The outlined seismic strengthening strategy effectiveness was experimentally investigated within the European research project SPEAR (Seismic PErformance Assessment and Rehabilitation). The structure under examination was designed and built with the aim of creating a structural prototype featuring all the main problems normally affecting existing structures: plan irregularity, dimensions of structural elements and reinforcement designed by considering only gravity loads, smooth reinforcement bars, poor local detailing, insufficient confinement in the structural elements and weak beam column joints (see Figure 29). The structure was subjected to pseudo-dynamic tests, both in its original configuration and retrofitted by using GFRP.



(a) (b)

Figure 29. **(a) Plan view and (b) 3D view of the SPEAR structure.**

The structure in its original configuration was subjected to experimental tests with maximum peak ground acceleration (PGA) of 0.20 g. Since both theoretical and experimental results showed that the ‘as-built’ structure was unable to withstand a larger seismic action, a retrofit intervention by using FRP laminates was designed. Once the design of the GFRP retrofit was provided, the structure was subjected to a new series of two tests with the same input accelerogram selected for the ‘as built’ specimen but scaled to a PGA value of 0.20g and 0.30g, respectively. The design of the rehabilitation was based on deficiencies underlined by both the test on the ‘as-built’ structure and the theoretical results provided by the post-test assessment. They indicate that a retrofit intervention was necessary in order to increase the structural seismic capacity; in particular, the theoretical results showed that the target design PGA level of 0.30g could have been sustained by the structure if its displacement capacity was increased by a factor of 48%. In order to pursue this objective, the retrofit design strategy focused on two main aspects. First it was decided to increase the global deformation capacity of the structure and thus its dissipating global performance; such objective was pursued by confining column ends with two plies of GFRP laminates. Moreover, the second design key aspect was to allow the structure to fully exploit the increased deformation capacity by avoiding brittle collapse modes. To achieve this goal corner beam column joint panels were strengthened by using two plies of quadri-axial GFRP laminates as well as a wall-type column for its entire length with two plies of the same quadri-axial GFRP laminates used for the above joints (see Figure 30).

Figure 30. Column confinement and shear strength of corner joints (a); shear strength of wall-type column and retrofitted structure overview (b).

The assessment of structural global performance, before and after the strengthening intervention, was performed by nonlinear static pushover analysis in longitudinal direction (positive and negative X-direction, PX and NX, respectively) and in transverse direction (positive and negative Y-direction, PY and NY).

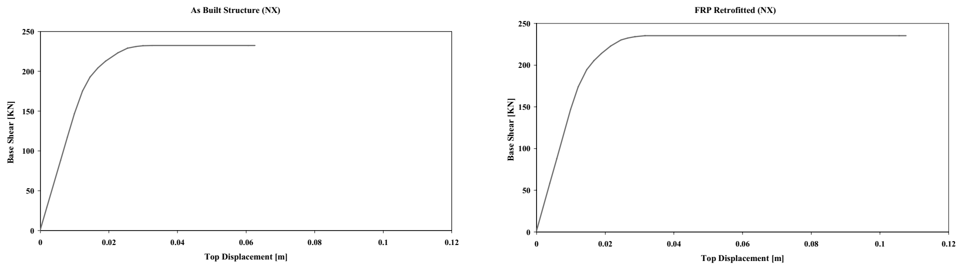


Figure 31. Base shear – top displacement curves for 'as-built' and FRP retrofitted structure.

In Figure 31, the theoretical base shear-top displacement curves for the 'as built' and FRP retrofitted structure are depicted with reference to direction NX (where the maximum capacity-demand gap was recorded for the 'as-built structure at the significant damage limit state LSSD). Figure 31, clearly shows that the FRP retrofit is able to greatly increase the global deformation capacity of the structure, slightly affecting its strength. The comparison between the seismic structural capacity and both elastic and inelastic demand is reported in Figure 32 (for direction NX) by using the Capacity Spectrum Approach (CSA) (Fajfar, 2000). Figure 32 clearly shows that column confinement provides the structure with significantly enhanced ductility, allowing it to achieve the theoretical inelastic demand by only modifying the plastic branch of the capacity curve.

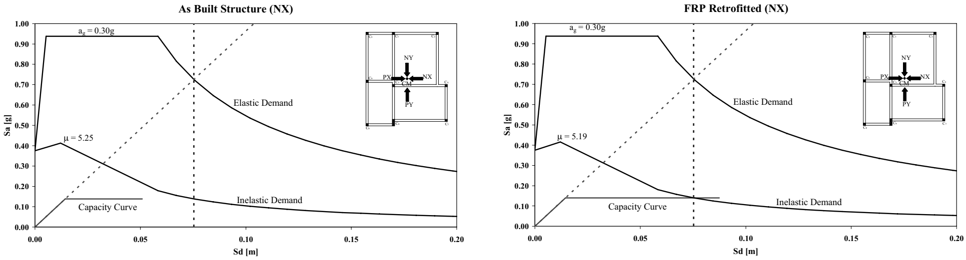


Figure 32. Theoretical seismic performance comparison at 0.3g PGA between ‘as-built’ and FRP retrofitted structure.

After that columns and joints were wrapped with GFRP, the retrofitted structure was able to withstand the higher (0.30g PGA) level of excitation without exhibiting significant damage. After tests, FRP was removed and it was shown that the RC core was neither cracked nor damaged. The comparison between the experimental results provided by the structure in the ‘as built’ and GFRP retrofitted configurations highlighted the effectiveness of the FRP technique in improving global performance of under-designed RC structures in terms of ductility and energy dissipation capacity.

Seismic rehabilitation of PC bridges by using FRP and SRP materials

An innovative strengthening technique based on the combined use of Fiber Reinforced Polymer (FRPs) and Steel Reinforced Polymer (SRPs) has been investigated with reference to a PC bridge, named “Torrente Casale” which is part of the *Salerno-Reggio Calabria* motorway. The bridge, built in the seventies, has been recently enlarged (2001) in order to satisfy the new traffic demands. Due to the recent issuing of a new seismic Italian code, it was decided to assess the bridge capacity, both for gravity and seismic loads, in relation with the new design provisions.

The bridge existing documentation has been investigated and both destructive and non-destructive tests have been performed in order to determine concrete and steel reinforcement mechanical properties. Once the bridge geometry and the material mechanical properties were determined, a theoretical analysis was performed showing that the bridge piers (circular cross-section, diameter D=1m, and total height H=6m) were not adequate to sustain the seismic actions. Thus use of SRPs spikes as columns’ flexural reinforcement combined with CFRP laminates wrapping of columns ends has been investigated to increase both member strength and ductility; the structural upgrade was completed by increasing the shear capacity of column cap through CFRP U-wraps (Figure 33). The effectiveness of such technique with respect to a traditional one, based on RC jacketing, has been assessed. The main construction phases of the rehabilitation intervention are reported in Figure 33.

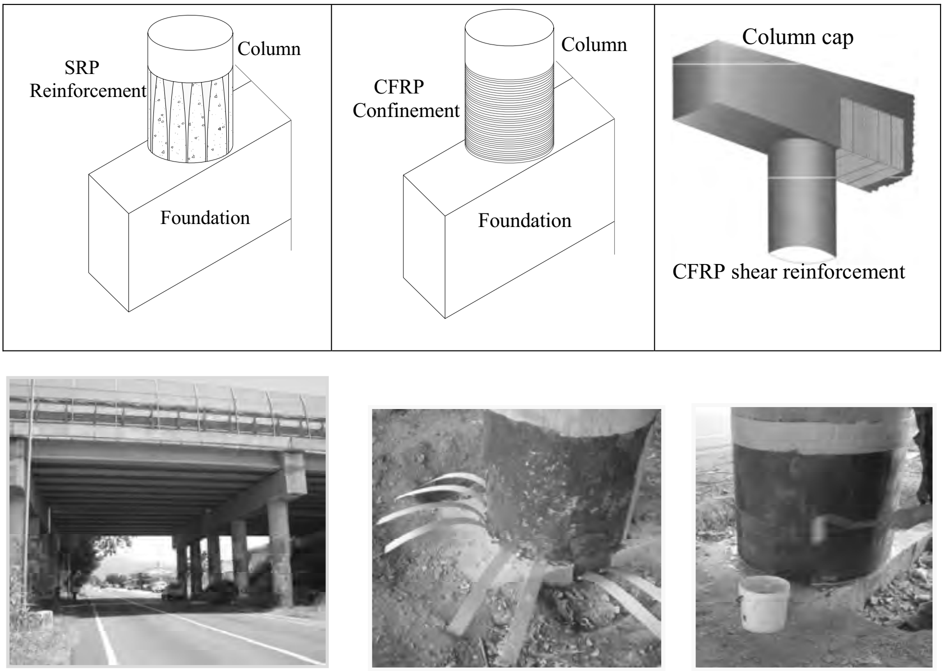


Figure 33. Details of the designed rehabilitation system.

4.7 Task 8.7: The design criteria for the seismic retrofit of masonry structures with FRP

The main design problems concerning the seismic vulnerability mitigation of masonry structures are the strengthening of masonry panels for in plane and out of plane actions, the improvement of FRP action by means of clamping and connection mechanical devices and procedures for the definition of the reinforcement layout in seismic analyses.

In order to test the design methods proposed by CNR DT200/2004, several seismic studies of complex monumental masonry buildings have been completed by carrying out detailed linear dynamic seismic analyses and FRP retrofit design. In the design several combinations of strengthening techniques were explored in order to point out the real applicability of the cited guidelines.

The obtained results cover the following areas:

a) Verification of the feasibility of a FRP design through simple modifications of the normal design activity;
b) Definition of the feasibility bounds for this retrofit technique in case of monumental buildings;
c) Detail types and level of detailing required in real applications;
d) Open FRP design problems not covered by the guidelines or lacking of necessary information;
e) Comparison of different modelling techniques making use of plate, shell, beam, truss elements to represent the masonry structure;
f) Comparison of different material constitutive assumptions for the non linear analysis of strengthened walls and buildings.

The main results of the completed research are summarized in the following:

a) The FRP reinforcement net design is easily implemented by a simple modifications of the normal design activity. The only needs are: integration of the stress distributions in order to obtain the stress resultants and preparation of a verification sheet including the design rules for bending and shear of a FRP reinforced masonry panel. Actually many companies have implemented DT 200/2004 rules in freeware software which can be helpful in this activity.
b) The feasibility bounds for this retrofit technique checked for three monumental buildings analysed, are very different if stuck or mechanically fastened reinforcement is used. In fact, only slight diffuse increase of resistance is obtained by using externally bounded FRP nets. Owing to increase significantly the safety of the building, mechanical devices are mandatory. In this last case true building retrofit is possible.
c) The set of the detail types needed for practical applications is very large and many connection types are not yet fully investigated although practically employed. Fibre ropes, bars, fasteners, metallic inserts are mixed in a way almost never covered by existing experiments and guidelines.
d) Open FRP design problems include cement matrices, thermal cycling, delamination in compression, real bond of the regularization primer to rough masonry surfaces, behaviour of the mechanically fastened FRP elements after internal delamination.
e) Several modelling techniques have been employed in order to carry out the seismic studies. Plate elements in linear dynamic analysis allow for both in plane and out of plane evaluation, but non linear static analysis with this type of discretisation is not a viable solution. Studies on discrete representation of masonry walls by using a refined truss structure to represent the compressive load paths inside the masonry panels showed that this type of discretisation is very simple, allows for complicated constitutive laws, allows easily equilibrium checks, and produces very reliable load – displacement curves.
f) The basic choice of associate Mohr – Coulomb or Drucker – Prager elastic plastic constitutive laws can be effective only in limited cases, where dilatancy is not dominant. More effective analyses require a non associate zero dilatancy rule, which is however not common in the professional engineering software. More refined damage rules are actually under way, but not for normal design activity. Truss elements however, allow to introduce complex behaviour by means of geometric discretisation, and by this way, crack tracking can be pursued, even if in a very rough representation.

4.8 Task 8.8: The strengthening of masonry structural elements with FRP systems

The research activity has been performed through experimental tests and theoretical study. In particular, the experimental program has been carried out with reference to masonry panels strengthened with FRP and with SRG and subjected to in plane loads, while the theoretical study concerns the modelling of masonry walls strengthened with FRP.

Masonry panels and building strengthened with FRP

Several experimental tests have been performed with reference to masonry panels strengthened with different FRP materials (carbon and glass) arranged according to various configurations.

In-plane shear–compression tests have been performed on full-scale tuff masonry panels consisted of two-layered walls with the inner part filled with mortar and chips from yellow tuff blocks, considering different FRP materials and strengthening configurations. In particular, two sets of panels have been strengthened with grid pattern carbon fibre unidirectional strips (CFRP), made with three horizontal and vertical strips on each face and two sets of panels have been strengthened with the same layout, but doubling the number of plies. Similarly, four sets of panels have been symmetrically strengthened with a grid pattern on both sides of the panels, but with glass bidirectional fibre strips (GFRP). A further set of panels has been made selecting a different geometric configuration by arranging FRP laminates along the diagonals of both sides of the panels and considering both a CFRP and a GFRP cross layout with either one or two FRP plies.

Important experimental evidences have emerged from the performed tests underlying the role of the configuration of the FRP strengthening system on the failure mechanism of the tested panels. In fact, while in some cases it has been observed that the debonding of the FRP strips has been the main responsible of the panels failure, in few cases the tensile rupture of the FRP strips has occurred. The different observed failure mechanisms have particularly influenced the behaviour of the strengthened panels both in terms of strength increase and post-peak behaviour (fracture energy).

Further tests have concerned square masonry panels composed of clay bricks, strengthened with different FRP configurations and subjected to compression diagonal load. In particular, some of the panels have been strengthened considering both vertical and horizontal CRFP and GFRP strips, while other panels have been strengthened arranging FRP strips along diagonal directions of the panels. Different levels of the strength increase and different failure mechanisms have been observed. In particular, while localized cracks pattern have characterized both the case of un-strengthened panels and the case of panels strengthened by FRP only on one side, a more diffuse crack distribution has been observed in the case of panels strengthened on the two opposite sides. In several cases the debonding of the FRP strips has been also observed.

From the experimental studies conducted on FRP strengthened masonry panels, considerations useful for improving the Document CNR-DT200/2004 have been deduced. In particular, it has been observed that for the evaluation of the design shear strength of FRP strengthened masonry panels (eqn. 5.16 of the CNR-DT200/2006), the contribution of the masonry component can be evaluated through the eqn. 5.17 (used in the case of reinforced masonry elements) only if the FRP shear strengthening is coupled with FRP vertical elements fixed both at the base and at the top of the panel. For this reason it has been suggested to include in the CNR-DT200 specific design indications concerning this aspect. In fact, it is important to underline that, when the FRP shear strengthening system is not coupled with a flexural FRP strengthening system with efficient anchor elements, the contribution of the masonry material in terms of shear strength is the same of the case of un-reinforced masonry elements.

A further aspect deduced with reference to tests on masonry panels strengthened by FRP has concerned the case of fibres arranged along to the diagonal directions of the panel. Form the experimental evidences and from global analyses it has been possible to affirm that the contribution of the FRP can be evaluated only considering the component parallel to the shear load.

A scale model of a typical tuff masonry buildings has been constructed and tested on the seismic simulation shaking table at the structural laboratory of CESI, Bergamo, Italy. In particular, the test procedure has consisted in three phases: in the first phase dynamic tests have been conducted on the un-strengthened prototype in order to induce some damages; in the second phase the prototype has been repaired by selecting GFRP strips arranged according

to the Italian Code (Ordinanza 3431/2005, CNR DT 200/2004); in the third phase, dynamic tests have been performed on the strengthened prototype.
Experimental evidences have shown that the applied GFRP strips allow to perform fast repair interventions in order to make operational masonry structures severely damaged by earthquake and at risk of aftershocks. In fact, the repaired prototype has showed reduced openings at the horizontal joints (about one-third of the maximum opening observed in the case of un-strengthened prototype) and an increasing of the lateral strength (about +34%). A further aspect emerged from this tests has concerned the importance to provide effective anchorages for FRP strengthening elements in order to avoid the delamination phenomenon which is particularly influenced by cyclic actions.

Masonry panels strengthened with SRG

Beside the “traditional” Fiber Reinforced Polymers (FRP), the research investigated the possibility of application of innovative composite materials, called Steel Reinforced Grout (SRG), based on high strength steel wires (Ultra High Tensile Strength Steel) forming that are assembled into a fabric and embedded within a cementitious grout. This application in fact could combine, to the traditional advantages proper of FRP, the performances of this new material, reducing installation and material costs, and inducing an increase of ductility. Both composites, FRP and SRG, can be used with perforated or solid brick to form a new strengthening system, called LATLAM ring – beam, that can be used to effectively construct the roof ring beams of a masonry structure. This new system, that can be also subjected to a pretension force. The following conclusions may be drawn from the developed research:

- the analytical model developed to determine the mechanical behaviour of LATLAM ring – beams has shown good agreement with experimental results and can be incorporated in design provisions;
- the experimental tests, performed on full – scale prototypes of LATLAM ring – beams, demonstrated good results in terms of load carrying capacity;
- LATLAM ring – beams proved to be a good substitute, either under a technical and economical perspective, of “traditional” reinforced concrete ring – beams.

Modelling and analysis of masonry walls strengthened with FRP

A further subject of the task research activity has concerned the modelling and the analysis of masonry elements strengthened with FRP. Indeed, recent codes extended the use of displacement-based design methodologies, such as the pushover analysis, to the case of masonry buildings. Thus, different modelling approaches able to capture the structural behaviour of masonry panels strengthened with FRP have been examined: a macro-micro modelling approach; a macro modelling approach; a frame model approach.
The first model relies on a homogenization approach combined with limit analysis suitable for the evaluation of the collapse loads and failure mechanisms of FRP reinforced masonry panels. The application of FRP strips on masonry has been treated adopting a simplified multi step approach. In the first step the un-reinforced masonry, regarded as a periodic heterogeneous material, has been substituted with a homogeneous macroscopic material using a homogenization technique. In particular, an estimation of the homogenized unreinforced masonry strength domain has been obtained by means of a micro mechanical model based on the lower bound theorem of limit analysis. In the second step, FRP strengthening has been introduced on the already homogenized masonry material.

The second model is based on the use of both 2D or 3D nonlinear behaviour finite elements and interface elements. In particular, special yield criteria coupled with nonlinear constitutive laws characterized by softening response have been selected in order to simulate the behaviour of masonry material both in tension and compression. A special nonlinear constitutive law has been also considered for the interface elements in order to simulate the debonding mechanism of FRP which characterizes in several cases the failure mechanism of masonry elements strengthened with FRP.

Finally, the third modelling approach consists of a simple but effective 1D frame element able to predict the response of masonry structures, eventually reinforced with FRP materials. In fact, the proposed elements presents some peculiarities both for converting the geometry of a masonry panel in the geometry of the equivalent frame and for accounting the nonlinear behaviour of masonry and FRP materials. Special components have been also provided in the developed model in order to account for the shear failure modes.

Several numerical applications have been carried out using the proposed modelling approaches and considering experimental cases deduced from literature. The obtained results, regarding both un-strengthened and FRP-strengthened simple panels and masonry façades, have shown the reliability of the proposed models to reproduce the experimental behaviour of masonry elements underlying some peculiarities due to the presence of the FRP strengthening system. In fact, the micro-macro and the macro modelling approaches have particularly underlined the role of the FRP strengthening system to change the stress path and, consequently, to induce different damage states characterized by a diffuse crack path along the strengthened elements. This aspect has revealed the important role of the FRP system to increase the energy dissipation capacity of masonry structures. The increase both in terms of strength and ultimate deformation capacity of strengthened elements has been confirmed by all the modelling approaches evidencing the capability of the simple frame model to capture the global response of masonry elements strengthened with FRP and consequently the possibility of using this simple model in a design process of the FRP strengthening system for masonry structures.

4.9 Task 8.9: The strengthening of masonry vaulted with FRP systems

The aim of the research task is the development of models and procedures for the analysis of vaulted structures reinforced with FRP. In particular the behaviour of arches, vaults and domes subjected to seismic action is studied in order to investigate the behaviour of these typologies as built and strengthened with FRP materials.

The study is performed with simplified analytical procedure and with numerical methods. Experimental tests are further expected for the validation of the models.

The research activity has been focused on the experimental tests on the topic available in literature for a comparison with the analytical and numerical results.

On the basis of the acquired data, a simplified analytical model for the evaluation of the ultimate load of arches and portal frames reinforced at the intrados and/or extrados has been developed.

The main results are related to the definition of a methodology for the evaluation of the ultimate load of one-dimensional vaulted structures reinforced with FRP and subjected to vertical and horizontal forces.

Starting point of the procedure is the assessment that the FRP presence does not allow the formation of the collapse mechanisms, which characterise the ultimate behaviour of the un-reinforced structure. As a consequence the cinematic approach of the limit analysis, usually adopted for this kind of structure cannot be applied. The proposed analytical model, starting from the analysis of the ultimate behaviour of the un-reinforced structure, identifies the location of the hinges, up to make the structure statically determined, and then to be solved with equilibrium condition. Obviously in this case and contrarily to the limit analysis, the mechanical characteristics of the masonry and of the FRP and mainly the debonding phenomenon of the composite material significantly influence the structural response.

The model has been developed and applied to assigned geometries of arches and portal frames, subjected to static vertical and horizontal loads, with FRP reinforcement at the intrados. The obtained results have been expressed in terms of interaction domains in the plane of the vertical and horizontal forces.

Afterwards, the procedures for the evaluation of the ultimate behaviour of masonry arches reinforced with FRP have been extended to other typologies, such as the masonry portal frames reinforced at the intrados or extrados.

In particular has been defined the formulation of a secant linear relationship defined by the maximum load and by the related displacement at the brittle failure, to be adopted for the estimate of the seismic behaviour of the structure. This model, even if simplified, can be suitable for simulating the response of vaulted structures subjected to vertical and horizontal loads, strengthened with FRP sheets. In this case, indeed, if the FRP is applied at the intrados, the failure is due, generally, to its delamination, while if the composite material is glued at the extrados, the failure is due to compression of shear crisis of the masonry. Anyway the global behaviour of the structure is often characterised by an almost linear response up to the maximum load, ad generally with a brittle failure.

Numerical FEM procedures in bidimensional field, have been further developed. The comparison between the results obtained with the two methodologies has allowed the validation of the simplified models.

Parallel to the analytical-numerical models, an experimental program has been set up, on vaulted structures reinforced with FRP.

Three circular arches, subjected to a key load, have been tested. The first one represents the reference un-reinforced, the other two have been strengthened with FRP at the intrados. The tests have been performed in displacement control, with the aim of evaluating the post-peak behaviour and the softening branches.

The results obtained have allowed the preliminary validation of the model related to arches reinforced at the intrados.

The last phase of the research was devoted to the validation of the developed models, through experimental tests on masonry vaulted elements. In particular, besides the preliminary tests in masonry arches reinforced with FRP at the intrados, a test has been performed on a masonry portal frame, in full scale, reinforced with FRP and subjected to vertical and horizontal loads. The obtained results allowed the validation of the analytical models and assessed the validity of the assumed simplified hypotheses.

In the Laboratory of Structures and Materials of the Department of Civil Engineering of the University of Rome “Tor Vergata”, has been realized a masonry portal frame, constituted by two columns with rectangular section (24cm x 49cm) and height of 2m, and an arch with internal radius R_i = 130cm and square section with side equal to 24 cm.

The portal frame was constructed with masonry block on mortar beds, and then reinforced with a layer of FRP at the intrados of the arch and on the internal surface of the columns. No anchorage of the composite has been given at the foundation level.

A constant vertical load (v) has been applied on the arch key and two horizontal increasing loads, with equal intensity (H/2) are given at the arch abutments, which simulate a seismic action.

Figure 34. Portal frame under construction. Figure 35. Portal frame during the test.

The experimental results on both the arches and a portal frame appear to validate the simplified models and the procedures developed in this research program for the design and check of masonry vaulted systems reinforced with FRP.

4.10 Task 8.10: The quality control and monitoring of FRP applications to existing masonry and RC structures

Several masonry and reinforced concrete structures have been utilized for the application of fiber reinforced composite materials, with the aim of carrying out quality control and monitoring tests, in accordance to CNR DT 200/2004. For each structure, special working sheets have been developed for a proper characterization of the building from a geometrical, mechanical and logistic point of view. On these structures semi-destructive tests, non-destructive tests and delamination tests have been performed.

Semi-destructive tests

This test consisting in pull-off and shear tearing tests conducted on different types of FRP materials, applied on the in-situ structures. Moreover, pull-off tests and shear tearing tests have been conducted in the laboratory of the Department of Structural Engineering of University of Calabria on concrete elements as well, reinforced by carbon fiber sheets and laminates. The latter tests results have been utilized for a comparison with a number of experimental tests conducted at the Universities of Bologna, Naples, Sannio and Milan, in the framework of Task 8.2 Round-Robin tests, with the aim of obtaining a standard test procedure for delamination tests and evaluating the scattering between experimental results derived from tests conducted on specimens realized by the same worker, but tested in different laboratories. The tests have been conducted using the same device used for outdoor tests. Pull-off tests, used to assess the properties of the strengthened substrate, have been carried out attaching a thick circular 75 mm diameter steel plate to the FRP and isolating it from the surrounding FRP with a core drill, taking particular care in avoiding heating of the FRP system while a 1-2 mm incision of masonry substrate was achieved. The test consists in pulling off the steel plate by means of an ad hoc device (Figure 36-a), obtaining the ultimate pull-off strength value expressed in kN (Figure 36-c). Whereas, shear tearing tests are used to assess the quality of bond between FRP and masonry substrate. These tests can be conducted only when it is possible to pull a portion of the FRP system in its plane located close to an edge detached from the masonry substrate. The tests have been carried out using the same ad hoc device used for pull-off tests. In particular, metallic elements have been set up onto the masonry wall and through the FRP strip, with the aim of connecting the entire test device. Then, the FRP element has been tightened until collapse (Figure 36-b), obtaining the failure tearing force, expressed in kN (Figure 36-d). For what concerns in situ tests, 16 reinforced concrete structures and 17 brick and stone masonry building have been considered, for a total number of more than 300 tests. The FRP materials have been applied in the form of strips having the dimensions of 500x200 mm and 50x200 mm in the case of r.c. structures and the dimensions of 500x300 mm e 50x300 mm in the case of masonry structures for the execution of pull-off and shear tearing test, respectively. Both carbon, glass and natural fiber composites have been utilized.

According to CNR DT 200/2004, FRP application may be considered acceptable if at least 80% of the tests return a pull-off stress not less than 10% of masonry support compressive strength, or not less than 0.9-1.2 MPa in the case of reinforced concrete structures, provided that failure occurs in the support itself. For what concerns shear tearing tests, FRP application may be considered acceptable if at least 80% of the tests return a peak tearing force not less than 5% of masonry support compressive strength, whereas it has to be not less than 24 kN in the case of reinforced concrete structures.

For what concerns masonry structures, taking into account the compressive strength of the support, both pull-off and shear tearing experimental results respect the limit values suggested in CNR DT 200/2004. In the case of concrete structures, pull-off results are in accordance to CNR DT 200/2004, whereas shear tearing test results the limit value of 24 kN suggested for reinforced concrete structures has never been reached. This is probably due to the small dimensions of the FRP strips. In fact, CNR DT 200/2004 Guidelines don't provide any instruction about the FRP dimensions that should be utilized to reach the above mentioned limit value.

In the case of pull-off tests, failure has always occurred in the substrate, for each type of FRP material applied, as expected.

The semi-destructive tests conducted in the laboratory on concrete elements reinforced by CFRP sheets and laminates have been realized. From mechanical characterization tests, a value of 25 N/mmq for concrete compressive strength was found. The Research Unit of University of Calabria tested 3 prisms reinforced by 3 CFRP strips and CFRP laminates applied on their surfaces. Pull-off and shear tearing tests have been conducted, with the aim of studying the failure mode and debonding of FRP from the substrate.

The results of pull-off tests conducted on both strips and laminates respected the limit values suggested in CNR DT 200/2004, and also in these cases failure occurred in the substrate, showing the effectiveness of the FRP application. On the other side, shear tests showed a different behaviour of the composite materials for strips and laminates. In particular, in the case of laminates, the collapse occurred suddenly and the whole composite debonded from concrete, with an ultimate value of the shear force higher than 24 kN (limit value suggested in CNR DT 200/2004). In the case of CFRP strips, a partial delamination of the composite occurred and the limit value was never reached.

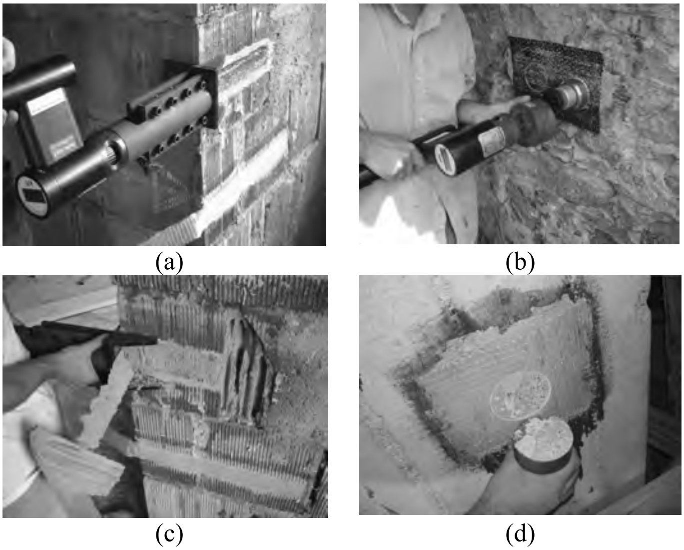


Figure 36. Pull-off test (a) and shear tearing test (b); Pull-off test result (c) and shear tearing test result (d).

Non-destructive tests

Non-destructive tests, consisting in thermographic tests were conducted on both brick masonry elements built in the laboratory and on the same real masonry structures used for the above mentioned semi-destructive tests, reinforced with different FRP materials. These tests are usually carried out to characterize the uniformity of FRP application. Both, the active and the passive thermography technique have been adopted, in which thermal energy is applied externally onto the test object or the natural infra-red radiation emitted by the object due to a sufficient exposure to sun light can be utilized, respectively.

The first Infra-Red thermographic tests have been conducted in the laboratory of the Department of Structural Engineering of University of Calabria on brick masonry macro-elements, which have been reinforced by carbon fiber strips placed in different directions onto the specimen surfaces. From the test a mortar joint was clearly visible due to the non plane substrate surface.

The in situ masonry and reinforced concrete structures have been utilized for thermographic tests as well, to verify the quality of bond between FRP and the substrate, before and after the conduction of the described semi-destructive tests. For instance, some tests have been conducted on a reinforced concrete structure on which FRP strips have been previously applied for shear tearing tests. From the thermograms, a crack could be noticed corresponding to the strip subjected to the semi-destructive test. Such a technique is then useful for the detection deterioration and damage in the structures.

Influence of roughness surface on the debonding force of FRP

In order to investigate the effect of the concrete surface preparation method on the roughness surface and debonding force of FRP, an experimental campaign has been carried out. The specimens have been produced with different formworks (staves, panels) and different compaction types (beating, vibration). Moreover, two different concrete strengths have been used in realizing the specimens in order to evaluate also their influence on the surface preparation method efficacy. Thirty 15 cm-length standard cubes have been also poured and used to evaluate the mechanical properties of concrete (according to Italian standards). Mean compressive strength (R_{cm} = 15 or 20 MPa) from the compression tests has been obtained by standard cube at an age of 50 days, corresponding to same period of the first profilometer tests. The number of specimens considered in the present experimental campaign and the two different casting processes are shown in Table 1.

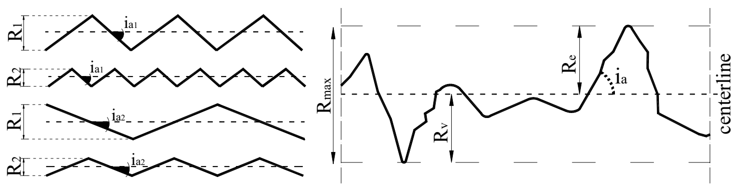
After curing, four different methods for surface preparation have been applied on concrete prisms in order to study the effect of the treatment on the FRP-concrete bond strength: *Grinding* – the upper surface of the concrete block has been grinded with a stone wheel to remove the top layer of mortar, just until the aggregate was visible; *Sand Blasting* – the concrete surface has been sand blasted in order to remove the whole mortar over the aggregates, so obtaining a very rough concrete surface; *Brushing* – the surface has been brushed with a twisted steel cord bonded to a rotating disc; *Scabbling* – impacting the substrate at variable angle with a metallic tip to create a chipping and powdering action. The driving mechanism is compressed air.

In order to examine exhaustively the concrete surface, an extensive campaign of laser profilometer analysis was carried out before and after the surface preparation. The profilometer used is the DRSC produced by Miami University, which gives quality and quantity information. This instrument using laser striping, highlights the rough concrete surface by thin slits of red laser light at an angle of 45 degrees, and the surface is observed at 90 degrees by an high-resolution (tiny) board CCD camera (Figure 38*a*). The video image of the laser stripes is digitized with a PCMCIA frame-grabber. The projected slit of light appears as a straight line if the surface is flat, and as a progressively more undulating line as the roughness of the surface increases. Lasers with one toeleven stripes were used.

The roughness degree can be identified by several parameters obtained by laser profilometry and each one can give specific information on roughness and its particular properties. They can be classified in amplitude parameters (R_i of Figure 37) and slope parameters (i_a of Figure 37*a*). The first group is sensitive to roughness morphologies, where the surface is either stepped or slotted and might be described as a discontinuous roughness, the second group is insensitive to roughness morphologies and is more useful for characterizing continuous roughness. Figure 37*b* shows some output parameters provided by the profilometer: R_{max} – maximum peak and valley, rough measure of the vertical distance between the highest peak and the lowest valley; R_e measures the vertical distance between the highest peak and the centerline of the profile; R_v measures the vertical distance between the lowest valleys and the centerline of the profile; R measures the average of all individually measured peak to valley heights, R_p – roughness profile index, defined as the ratio of the true length in the fracture surface trace to its projected length in the fracture plane; i_A is the micro-average inclination angle, representing the average of the pixel to pixel angles of the stripe profile.

Table 1. Specimens and summary of test sample.

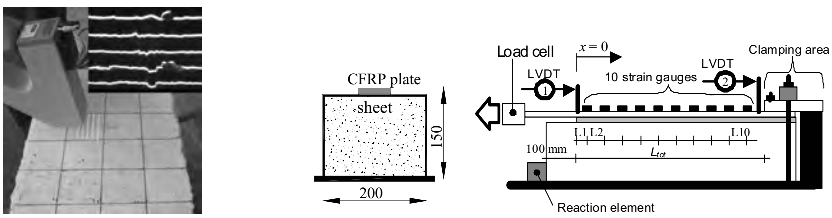
N° Specimen	Dimension (mm)	Mean Compressive Strength (N/mm^2)	Formwork (wooden)	Type of Compaction
20 ?	160 x 400 x 600	15	staves	beating
20?		20	panels	vibration



(*a*) (*b*)

Figure 39. Roughness parameters.

In order to define a unique parameter for describing the surface roughness, profilometer parameters were analyzed and correlated. For each parameter given by the profilometer, average value and covariance have been calculated for evaluating the quantity and quality factors of roughness. The covariance can provide for information on surface homogeneity; both quantities are interesting especially regarding the efficiency of concrete surface preparation methods. In the following, the roughness is described by coefficient $I_R = R \cdot i_a$, where R and i_a have been described before. The parameter I_R is used to give information on the absolute value of the roughness and on its specific shape. Profilometer analysis allows to correlate the casting methods with various degrees of roughness. In fact, the specimens casted with staves are more rough because the disconnection of the staves increase the surface irregularity. The specimens compacted by means of the vibration are more rough due to different positions of aggregates and the presence of vacuum produced by air bubbles.



(*a*) (*b*) (*c*)

Figure 37. (a) Laser profilometer, (b) geometry of FRP-strengthened prism and (c) experimental set-up.

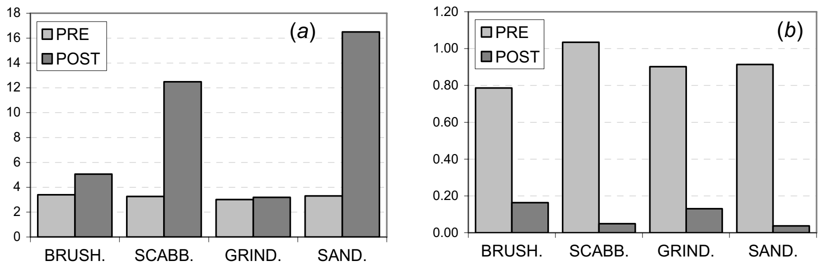


Figure 38. Average values (a) and covariance (b) of IR parameter for different surface preparations.

The roughness of the concrete surface has been investigated before and after the preparation; its difference is a way to evaluate the efficiency of each surface preparation method. In Figure 39. *a,b* are shown the average values and the covariance of the I_R parameter for all the different surface preparation methods. Figure 39. *a* shows that all the surfaces prior to the treatments have very similar roughness while after them the mean value of the I_R parameter is particularly high in the case of scabbing and sand-blasting. On the contrary, all the surface preparation methods strongly reduce the statistical dispersion of I_R parameter (Figure 39. *b*); the scabbing and sand-blasting provide for the higher level of homogeneity.

Delamination tests

Delamination tests have been carried out on bricks reinforced with FRP materials, with the aim of studying both the collapse load value and the failure modes that can occur if FRP materials applied during strengthening interventions collapse. In particular, delamination tests have been conducted on two half bricks placed inside a properly designed steel frame and connected between them by means of two carbon, glass or natural FRP strips glued on both sides of the specimens. The frame has been designed in such a way as to avoid any hindrance to the specimen collapse. The specimens have been subjected to uniaxial tensile tests under displacement control by means of an electro-mechanic testing machine, with a capacity of 100kN, connected to a personal computer. Some specimens have been also monitored by means of unidirectional strain gauges applied both on the brick surface and on the fibres.
From the experimental tests, the specimens failure mode has been analysed. Collapse has occurred for delamination in almost all cases. However, some specimens have reached failure due to fibres crack, as in the case of glass fibers and natural ones due to the different stiffness of the FRPs. More than 30 specimens have been tested. In the case of specimens reinforced by carbon and glass fibers, the load-displacement curve relative to failure for delamination obtained from the strain gages applied on FRP and brick surfaces was almost linear until failure which occurred suddenly and for debonding of the fabric from the reinforced bricks, with consequent removing of part of the substrate surface. The presence of relevant traction stresses at the attachment of the FRP strip with the bricks where the delamination phenomenon begins.
For what concerns the tests conducted on bricks reinforced by natural fibers, the results, obviously, cannot be compared to the ones derived from brick reinforced by CFRP or GFRP; however, the tests have shown interesting properties of natural fibers for not bearing applications such as on ancient masonry structures where FRP mechanical behaviour sensibly affects the global behaviour of the structure.
Finally, for each tested specimen, an accurate study of the substrate after failure has been conducted for the exact definition of the delaminated surface dimensions; the crack begins at a depth of about 10 mm and this distance obviously depends on the experimental equipment, whereas the depth and the form of delaminated substrate appears almost constant and dependent on the testing modality (Figure 40).

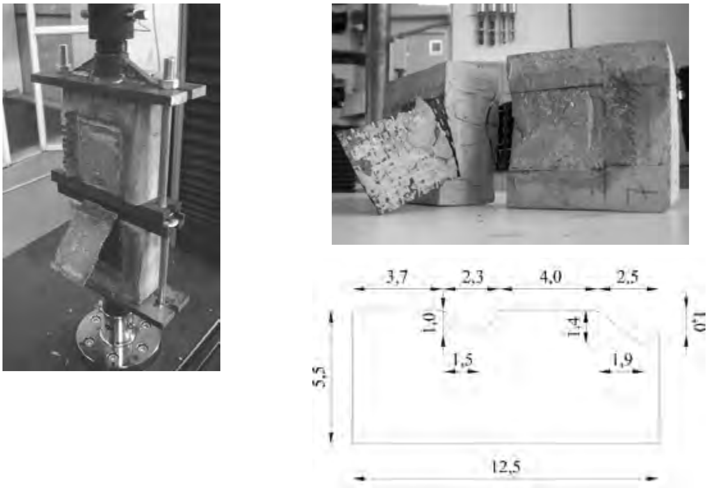


Figure 40. Failure mode of bricks reinforced with CFRP strips subjected to delamination tests.

From both in-situ and laboratory tests, important information were obtained regarding composite materials behavior and experimental procedures. In particular, it was noticed that shear tearing tests are extremely affected by instrumental errors that can take place during the test conduction. In fact, while FRP laminates allow a perfect alignment of the composite with respect to the hydraulic pull machine, in the case of FRP strips the applied force is not perfectly aligned with the FRP – concrete interface, and peeling stresses are generated, so causing a significant reduction of the debonding force.

Then, on the basis of the above described experimental results and in accordance to workshops organized during the Research Activity, some variations to the CNR DT 200/2004 guidelines, with particular regard to shear tearing limit value and experimental procedure, were proposed. In particular, shear tearing test should be carried out following two different procedures, namely direct and indirect procedures. The direct procedure is preferred if the load can be directly applied on the FRP glued onto the specimen surface, especially in the case of laminates. If the FRP system is made of strips to be prepared in-situ, the indirect test procedure is preferred, in which load is applied by means of steel plates glued onto the FRP surface.

5 DISCUSSION

In this research task a large number of experimental tests has been performed in order to validate the design equations provided by CNR-DT200/2004 and to calibrate some coefficients included in these equations to better fit theoretical predictions and experimental results.
Experimental tests have been carried out on both real scale or scaled members by using traditional FRP systems or innovative typologies of FRP systems and advanced materials. The experimental tests have been focused on both masonry and reinforced concrete members. In particular, advanced FRP materials made of basalt fibers (bonded with traditional or inorganic cement matrix) have been used for the confinement of RC and masonry columns. In addition, the mechanical properties of several FRP systems at fixed environmental conditions and/or under cyclic actions, the behaviour of RC members confined or strengthened in flexure and shear as well as of beam-column and beam- foundations joints have been deeply analyzed. Design criteria for the seismic retrofit of masonry members and structures have been also provided. The outcomes of experimental activities have, in most cases, confirmed the reliability of CNR-DT200/2004 provisions, especially for RC members. Regarding to masonry members, test results indicate that some coefficients of theoretical expressions provided by CNR-DT200/2004 have to be refined and some other experimental results could be necessary to derive theoretical relationships specifically targeted at different masonry substrates and failure modes.

6 VISIONS AND DEVELOPMENTS

Innovative materials for the vulnerability mitigation of existing structures have been largely analysed in the research activity. Further, new typologies of FRP systems, also made of several fibers and matrices types are spreading; on these new typologies further investigations are strongly necessary in the future. The study of the feasibility of using composite systems made of inorganic matrix strengthened with natural fibers fabrics is needful. In fact, the idea of using natural fibers is due to economic and environmental sustainability suggested by the use of such materials. These studies could allow new FRP systems to be inserted in the actual guidelines in order to increase their use in the civil engineering applications.
The indications provided by CNR-DT 200/2004 for the vulnerability mitigation of existing structures have been accepted by recent Italian guidelines, but further experimental tests will need, especially to mitigate the vulnerability of existing masonry structures.

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STRUCTURAL HEALTH MONITORING AND EARLY WARNING

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1 INTRODUCTION

The general objective of structural health monitoring and seismic early warning is to improve the capacity of implementation of actions aimed at real time decrease of seismic risk and to increase the effectiveness of post emergency actions.
A significant progress toward these main goals was achieved through:

- the development of innovative methodologies for the evaluation of dynamic response of strategic structures using micro tremors and environmental noise;
- the improvement of a simplified structural monitoring system for damage detection of public strategic buildings, following the lines proposed by the Italian Department of Civil Protection (DCP). This system is able to provide a rough assessment of the general conditions of the building through the comparison of dynamic parameters calculated before, during and after the earthquake.
- the preparation of guidelines for the permanent monitoring of civil structures and infrastructures;
- the definition and solution of some problems of interface between the early warning seismic information and the engineering components, that are crucial to improve in real time the defence of structures and the safety of infrastructures and life-lines against earthquakes. In fact the development of a "few seconds engineering", meant as the utilization of the lead time given from an early warning system, can have different applications, each one with its own peculiarity;
- the development of decision models based on the expected site ground motion and referred to structures equipped with active or semi-active control systems for seismic insulation and attenuation;
- the evaluation, of the reliability of the SEW system existing in Campania;
- the evaluation of the applicability of an EWS to hospitals of Regione Lombardia, considering the seismo-tectonic context, the distribution of seismic sources relative to large inhabited areas and to power stations and the involved pre-warning times;
- a feasibility study, with experimental validation, of an innovative system for the protection of statues or monuments with prevalent vertical development. This includes the study and validation of non invasive semi-active EWS for the protection of master-works.

2 BACKGROUND AND MOTIVATION

The assessment of the aged structures and infrastructures existing in Italy entails an overwhelming effort. The occasional collapse of historical buildings (Venice and Pavia bell towers, Noto Cathedral, Cuneo S. Mary of the Angels Church among the others) and the damage produced to historical centres by moderately high magnitude earthquakes (e.g. the 1997-98 Colfioritoand the recent Abruzzo sequences) remind that our unique Italian architectural heritage is in perpetual danger. The current practice of periodic visual inspections for the safety evaluation of structures and infrastructures appears to be more and more inadequate, especially if the purpose is to provide a reliable evaluation of seismic risk.

The Italian Department of Civil Protection (DCP) proposed the development of a health structural monitoring network constituted by 105 public buildings and 10 bridges equipped with a complete seismic monitoring system able to rebuild the motion of all storeys and of the surrounding ground. This lead to the planning of a new simplified health monitoring network of public strategic building selected all over the Italian seismic areas. The new network is characterised by a simplified monitoring system able to rebuild the motion only of the top floor and of the surrounding ground. The system evaluates the safety of the monitored building starting from the dynamic parameters evaluated before, during and after the earthquake and provides an approximated estimate of the general conditions of the building.

The main aims of the simplified structural monitoring system are:

1. periodical health monitoring of the buildings;
2. monitoring of anomalous vibrations;
3. post earthquake alarm;
4. post earthquake health monitoring;
5. assessment of the post earthquake structural condition;
6. advancement of the technical-scientific knowledge.

The critical examination of both complete and simplified structural monitoring systems for damage detection proposed by DCP evidenced the need to get to some in depth scientific analysis in order to verify the real feasibility of the system and the possibility to improve their performances. This requires:

- the estimation of the effectiveness of instrumental parameter for experimental damage detection;
- the definition of strategies for using the information relevant to the damage measured on a single building in order to evaluate the damage scenario and losses estimation;
- the definition of guidelines for permanent health monitoring of structures and infrastructures.

A further step forward the health monitoring of structures is modelling its dynamic response using natural sources, such as tremors and environmental noise.

Health monitoring systems can be integrated by seismic early warning systems. The latter are based upon the differences between the velocity of seismic wave propagation and that of analogue (digital) signals transmitted via radio (or cable). Depending on the distance from the source area of a strong earthquake, information about its location, magnitude and predicted levels of strong ground motion can reach sites that are “potentially at risk” a few seconds to tens of seconds before the arrival of the seismic waves of largest amplitude. The time available for risk mitigation actions (lead-time) is the difference between the estimated time of arrival of the potentially destructive waves (S waves; surface waves) at the target and the time of the first P wave recorded at a seismic network deployed in the earthquake source region.

In spite of the small lead time, potential advantages of the application of earthquake early warning systems (EEWS) is recognized at international level. This is due to the ever increasing urban populations, especially in the larger cities, which are becoming hotspots of global risk change. This urban explosion takes place predominantly in the developing world, where the population of large cities doubles every 15 years, while that of informal settlements doubles every 7 years. By 2015, around 420 million people are expected to reside in cities with a population of 2 million or greater. About 350 million will be in developing countries. In Europe, although the growth of urban population is smaller, nonetheless cities face increasing levels of risk, making them hotspots of risk change, because of the high level of industrialisation and the ever greater networking of infrastructures, lifelines and economies, Therefore the developing world faces an increase of risk mostly in terms of increasing loss of human life, while European cities face increasing risk mostly in terms of increasing financial and infrastructure losses. It must be stressed out that, as a result of globalisation (economic and social), the vulnerability to any destructive events occurring in any place of the world cannot be considered as confined to the area of immediate physical damage.

The importance of EW in general has been highlighted in a number of international documents at various governmental levels. Within the context of the United Nations, EW was emphasised within the Hyogo Framework for Action, where it was identified as one of the five priorities for action. A report requested by the then UN Secretary General, Kofi Annan, provides a global assessment of the EW capabilities, gaps and opportunities. Three international conferences dealing with EW, all under the sponsorship of the United Nations, have continued to emphasise the importance and necessity of EW systems.

Earthquake hazard cannot be reduced, but risk can be. Early Warning methods have the potential to be used to decrease damages in real time. Earthquake Early Warning raises many scientific and technical challenges. In the high-risk areas of Europe, the lead times are of the order of only 10's of seconds. Such a short time window means that many actions, particularly those associated with industrial processes, would need a high degree of automation, raising legal issues regarding liability in the event of false or missed alarms. Nonetheless, even such short lead times are a step towards more reliable safety measures. Consider for example the cases of LNG and nuclear power plants. Such facilities are required by regulation to have an earthquake alarm system for safe-shut-down. However, the alarms systems in question are essentially located on-site and the alarm/warning time is too short for fully robust safety measures to be made. A regional EEW system on the other hand would potentially increase the warning times, with the existing EW system serving as a redundancy.

In Japan, EEW systems have been practically implemented for more than 20 years. Applications of high social value include the system of the JR (Japanese Railways) for the automatic shut-down of moving “Shinkasen”, the very high-speed bullet trains, when a seismic ground motion of amplitude exceeding a pre-determined threshold is detected. An excellent successful example comes from the 23rd October Niigata large earthquake (M = 6.8), when a “Shinkasen” moving exactly over the epicentral region stopped automatically without fatalities. The system ha recently been applied also to the Tokyo Underground. A similar system has been in use for more than 15 years by Tokyo Gas, a company that distributes gas to more than 6 million people in Tokyo. The operational centre of Tokyo Gas is capable of shutting down gas supply as soon as a seismic ground motion with an amplitude exceeding a pre-determined threshold is detected. An amendment to the “Weather Service Law” issued on December 2007 gives Japan Meteorological Agency the duty and responsibility to issue earthquake early warning whenever it is deemed necessary.

In Mexico, large earthquakes occurring in the Oaxaca seismogenic segment along the west coast can be detected by the EW system in the Mexico city, about 350 km from the earthquake source. This allows warning times of about 30 seconds before the catastrophic seismic surface-waves arrive, with the information being sent to civil protection agencies, fire brigades, as well as to schools.

There are a number of projects being sponsored in Europe, both by the European Commission and national governments. In terms of EEW, the most prominent is the “Seismic eArly

warning For EuRope" (SAFER) project. SAFER brings together specialists in earthquake seismology and engineering, with 23 partner institutions from 14 countries taking part, including one from Japan, one from United States of America and one from Taiwa. The project started in June, 2006, and will continue until June 2009. It is primarily a research project, whose products are intended to be integrated into current and future seismic networks during the project itself and several years after irs completion (www.saferproject.net).
Generally speaking, a seismic Early Warning System (EWS) is a set of actions that can be taken from the moment when a seismic event is triggered by a sensor network with significant reliability to the moment the quake strikes in a given location. The researchers involved in the present study are analysing the potential exploitation of EWSs from different perspectives:

- a fundamental approach, where EWSs are looked at as tools to locate in real time an earthquake and to estimate its magnitude, as well as to describe ground-shaking maps. The research units (UR) Department of Physical Sciences of the University of Napoli Federico II (UNINA-DSF) and Osservatorio Vesuviano of the Istituto Nazionale di Geofisica e Vulcanologia (INGV-OV) are working from this perspective;
- a natural evolution of the exploitation of EWSs, dealing with the possibility of issuing warning or alarm signal when the located earthquake is considered potentially dangerous. The UR Department of Structural Engineering of the University of Napoli Federico II (UNINA-DIST) and European Centre for Earthquake Engineering (EUCENTRE) are following this research path;
- a further evolution of EWSs, finalized to exploit information gathered ahead of time about an incoming earthquake in order to either activate protection systems for the cultural heritage, as proposed by the UR Ente Nazionale per l'Energia e l'Ambiente (ENEA), or modifying in near real-time the dynamic behaviour of a structure so as to better withstand the incoming ground motion, as proposed by the Department of Technologies of the University of Napoli "Parthenope" (UNIPARTHENOPE).

3 RESEARCH STRUCTURE

The activity of Line 9 is organized in two Tasks:

- Task 1: Health Monitoring of strategic structures and infrastructures;
- Task 2: Prototypes of early warning systems for strategic structures, infrastructures and cultural assets.

The achievement of the various research goals required the integration of activities of several research groups. Considering the background and expertise of the researchers involved, each University was designated as a research unit and had peculiar research objectives to achieve, as summarized in Table 1.

Table 1. UR participating to Line 9.

Acronym	Institution	Scientific Leader	Tasks and Role
UNINA-DIST	Università di Napoli Federico II Dipartimento di Ingegneria Strutturale	Gerardo VERDERAME	Task 2 Cost benefit analysis and implementation of EEW systems
UNIMOL	Università del Molise Dipartimento SAVA	Giovanni FABBROCINO	Task 1 Structure monitoring in Molise, and Guidelines
POLITO	Politecnico di Torino Dipartimento di Ingegneria Strutturale	Alessandro DE STEFANO	Task 1 Dynamic response Modelling and guidelines of health monitoring
EUCENTRE	EUCENTRE Pavia	Carlo G.LAI Barbara BORZI	Task 2 Feasibility of EEW for hospitals
UNIPARTH	Università di Napoli Parthenope Dipartimento di Tecnologia	Antonio OCCHIUZZI	Task 2 Semi active control systems
ENEA	ENEA Centro Ricerche Casaccia	Gerardo DE CANIO	Task 2. EEW applied to cultural heritage
INGV – OV	INGV Osservatorio Vesuviano	Giovanni IANNACCONE	Task 2 Real time shake maps
UNIBAS	Università della Basilicata Dipartimento di Strutture, Geotecnica, Geologia applicata all'ingegneria	Felice C. PONZO	Task 1 Health structure monitoring and guidelines
UNINA DSF	Università di Napoli Federico II Dipartimento di Scienze Fisiche	Aldo ZOLLO	Task 2 Fast determination of earthquake parameters for EEW application

4 MAIN RESULTS

4.1 Health monitoring of structures

The main results in this field have been:

a) the development and testing of a simplified structural monitoring system following the lines proposed by Civil Protection;
b) the development of innovative methods for evaluation of dynamic response of structures using natural tremors and environmental noise;
c) the theoretical assessment and the in-field application of a strategy based on of permanent (on-line) structural health monitoring through distributed, miniaturized, wired or wireless sensing;
d) the preparation of Guidelines for the health monitoring of structures.

a) The improvement of a simplified structural monitoring system for damage detection of public strategic building able to provide a rough assessment of general conditions of the building through the comparison of dynamic parameters calculated before, during and after the earthquake was pursued starting from an analysis of the effectiveness of the instrumental parameters based on experimental and numerical investigations.
The effectiveness of the methodological approach proposed for the evaluation of the structural damage detection has been calibrated and verified through the results' combination of the experimental experiences and non-linear numerical analysis.
It was observed that the maximum acceleration of the last floor, obtained by the numerical analysis of the regular buildings under low earthquake intensities, is the better correlated parameter for the evaluation of the maximum inter-story drift. The other parameters become significant for the maximum drift determination when PGA increase up to the strongly non-linear behaviour of the structure. These results confirmed the experimental data obtained in the second year of activities.
The fundamental parameters considered in the methodology (maximum acceleration, fundamental frequency variation and variation of the equivalent viscous damping) have been evaluated for both principal direction of the structures and for all acceleration profiles.
Then, the structural characteristic constants required by the polynomial relationship for the evaluation of the maximum expected drift have been obtained through a regression analysis of the outcomes of non-linear analyses carried out considering all earthquake input (Ponzo et al. 2007).
The last step of the procedure, started but not yet completed due to the large number of elaboration to be carried out, is the estimation of the correlation degree between structural characteristic constants and the geometrical parameters and of the constituent materials of the buildings. This step, fundamental in the practical applications because it allows getting the constants for an assigned structural typology without performing any complex numerical analysis, has been carried out during the third year considering several structural typologies obtained by varying the number of floor and the presence or absence of infill panels and of the soft story. The first results have emphasized a non linear dependence between the structural characteristic constant and the investigated geometric parameters, generally, consistent with a second degree polynomial equation.
The constants of the polynomial equation have been estimated by means of statistical approach considering a great numbers of cases, which need further analyses in order to statistically make acceptable the correlation.
The next step was a feasibility analysis of the system based on experimental investigation. The specimen for the experimental verifications of the health monitoring method has been conceived in order to allow a rapid interchange of its elements and vary the geometric configuration, the number of floor, dimensional ratio among the sides (Figure 1).
Moreover, the particular conception of the experimental model allows to take into account of torsional effects through a non symmetrical location of the additional masses respect to the stiffness centre or, in alternative, non symmetrically distribution in plant of the column and beam stiffness. In particular, aluminium alloys or steel filleted bars, with constant or tapered diameter, have been designed at the end of beam and column elements, in order to calibrate different beam-column strength and stiffness ratios (Figure 2).
A theoretical study for the set up of a new method of distortions effects evaluation on the experimental results of scaled models has been carried out in order to correctly interpret the experimental outcomes. This method is based on the seismic input distortions implemented starting from the evaluation of known quantity (stiffness and strength ratios), evaluated by

means of the comparison of capacity curves of both full scale and scaled down models (Ponzo *et al.* 2009).

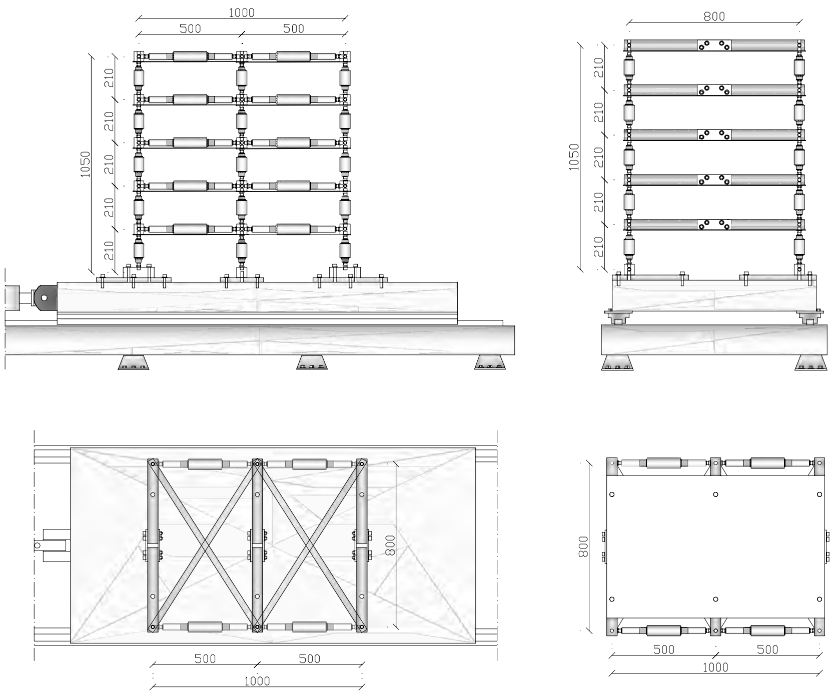
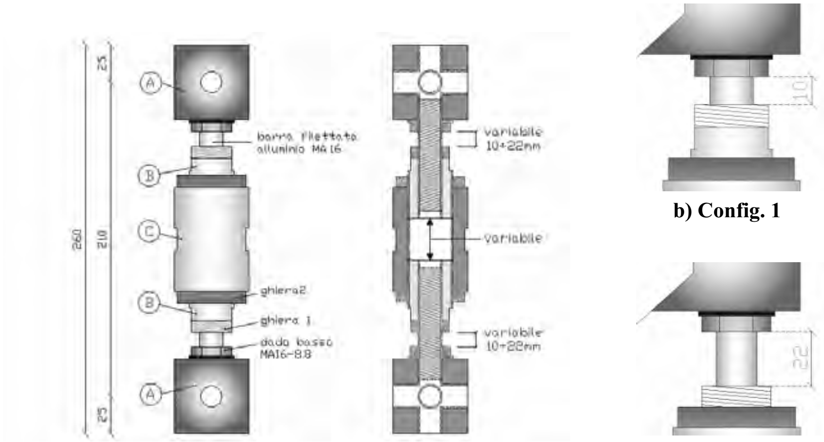


Figure 1. Plan and lateral view of the experimental model assembled on the shaking table.

The dynamic response of the distorted scaled models can be brought back to that related to the perfectly scaled model through simple mathematical expressions, function of the same factors used for modifying the input. Before said relationships have been obtained through linear regression of the results of over 4500 non-linear dynamic analyses carried out on SDOF and MDOF numerical models (from 1 to four d.o.f.) characterized by different distortion typologies conventionally assigned. Then, the reliability of the method has been verified through non linear dynamic analysis on 3-D numerical models (Ponzo *et al.* 2009).



b) Config. 1

a)

c) Config. 2

Figure 2. a) Components and section of the column element; particular view of the column fluent device.

Starting from instrumental data recorded on the monitored structure utilized in the proposal simplified procedure, the optimization of the algorithms for the evaluation in real time of damage building level has been started during the second year and defined in the third year.

The optimization procedure has concerned different aspects, as following shown:

- the optimal location of the last storey instrumentation, in order to identify both displacement and rotation of the structure;
- the trigger levels for automatic activation of data acquisition system, as function of the typology of the monitored structure, and the manual activation (on-demand);
- the Filter typologies (passband, etc.) for signal processing, as function of the monitored structural typology.
- the executive procedures for in situ installation of the system, after a preliminary characterization of the building frequencies and of the geometrical and mechanical characteristics to determine the building constants sets. The characteristic constants for regular buildings are described in the software library derived from the statistical evaluations. In case of strategic buildings (for example central nuclear, schools, hospitals, police office and civil protection structures) or particular complex structures, the characteristic constants must be evaluated for each case through accurate non linear dynamic analysis and valid accelerograms for the considered site.

The automatic procedure of the instrumental parameters extraction at the last floor (maximum acceleration in the two directions, variation of the proper frequencies and the values of relative modal damping), for the evaluation of the interstorey drift and the determination of monitored building characteristic constants starting from the geometrical and mechanical characteristics, have been completely implemented in Matlab, then compiled in C++, in order to run on every operating system (Windows, MAC, Linux). The software and relative user manual are made as a final product of the research activity.

An algorithm for fully automated modal identification in output-only conditions has been developed and tested against simulated and actual records. Several techniques for automated damage detection are available in the literature. They can be classified into two main classes: a first group of techniques, the so-called "modal-based" algorithms, aims at tracking changes in structural response directly or indirectly related to the mechanical characteristics (such as natural frequencies, etc.) of the structure before and after damage. Conversely, the second approach is based on the post-processing of measurement data to detect anomalies from measurements (ARMAV modelling, wavelet decomposition, etc.). In both cases, the trend is in using methods able to automate the detection process by taking advantage of the recent advances in information technologies.

One of the main limits of the so-called global, or modal-based, algorithms is related to the continuous availability of information about the dynamic characteristics of the monitored structure: in fact, the techniques for modal parameters extraction in output-only conditions usually require interaction with an expert user. The developed algorithms has been integrated with a continuous automated modal parameters tracking procedure developed during the second year of research activity so that an integrated fully automated system for modal parameter monitoring has been obtained. In fact, the automated modal identification procedure provides the information (mode shapes) needed by the tracking software, thus allowing a frequent estimation of modal parameters after the start-up phase represented by the former algorithm, which is more time consuming and, as a consequence, not suitable to fast on-line data processing. Since both algorithms are based on a reliable identification procedure such as the Enhanced Frequency Domain Decomposition, they are robust also against non-stationary signals. Application of this data processing system to the School of Engineering Main Building SHM system data has provided promising results, since the software is able to detect changes in the modal parameters without user intervention and it is characterized by a low computational burden, thus allowing extraction of the main modal parameters in a reduced time interval. (Rainieri *et al.*, 2008 a,b)

b,c) The theoretical research on dynamic identification in the framework of the Theory of Complexity, allowed the development of optimization techniques for managing uncertain and cumbersome information from large numbers of sensors. At the same time a number of innovative sensors (accelerometers and tiltmeters) were developed and implemented at the Turin Olympic Footbridge for dynamic evaluation and permanent health structural monitoring (Figure 3).

Figure 3a. SHM of the 2006 Olympic Games Footbridge in Turin.

Figure 3b. TMD system.

Figure 3c. Some of the sensors employed on the footbridge.

Furthermore dynamic identification techniques were experimentally tested and validated on a prototype steel structure meant to become an experimental benchmark case in the fields of identification, diagnostic and measurement techniques (Figure 4) and model-updating techniques through multi model and data mining were implemented.

Figure 4. FEM model of the benchmark structure.

The structure is conceived to obtain a strong coupling between the two first vibration modes (flexural and torsional), a peculiar issue in dynamic system identification and model-updating. Coupling is granted thanks to the possibility of separately controlling the two modes by changing the transverse location of masses lumped at the end transverse beam.
Two further flexibilities of usage are associated to the said one: loosening the bolts of diagonal elements and bracings allows simulating a multitude of damage scenarios (partial or total, single or multiple) while inserting damping material (of visco-elastic or hysteretic types) inside the slotted holes of bolted joints permits to realize a variety of dissipation mechanisms. Due to such characteristics, the structure lends itself to be assumed as a cast study object for the application of a number of experimental investigation techniques connected with structural monitoring and will be proposed, in ways still to be defined, for the fulfilment of the following goals:

- validation of dynamic identification and damage detection techniques for structures characterized of coupled flexural and torsional modes;
- verification and comparison of low-cost devices and measurement systems (accelerometers, fibre optic sensors etc.).

The experimental dynamic identification of the structure is under way. From the preliminary results, it appears necessary to modify some aspects of the experimental setup in order to grant the maximum invariance of the constraint conditions, for instance the adoption of a controlled tightening of the bolts and the adoption of a rigid link of the foundation to the floor. The procedure allows furthermore the validation of a variety of alternative linear identification techniques implemented by the Research Unit in MATLAB codes, such as ERA, time-frequency domain techniques, etc, and in next months of robust model-updating methods now under development.
The increasingly high safety and efficiency demand addressed from the Country to its own structural and infrastructural heritage can be tackled today, thanks to the wonderful development in information technology, electronics, material and structural engineering, with a new tool, effective and competitive: the structural health monitoring (SHM).
Initially proposed (and commonly intended) as a complement, partially or totally automatic, to the simple periodic visual inspection, aimed at the assessment of the degradation of the infrastructures with the ultimate purpose of an optimal allocation of resources within a management and maintenance strategy, the structural monitoring can actually accomplish a number of tasks, including in recent years the seismic protection. A report on the permanent monitoring system installed on the 2006 Turin Olympic Games footbridge is provided.
d) With the contribution from the other RUs involved in Line 9 Task 1, UNIBAS and UNIMOL, POLITO UR has conceived and produced the document "Guidelines for the monitoring of structures and infrastructures oriented to seismic protection" (in Italian).

Looking at current international standards on structural monitoring as at the natural, unavoidable and significant reference, these new guidelines aim at synthesizing the common bases and the most remarkable points, focusing on the issues connected with a use with seismic protection purposes. In order to bridge the gap between scientific knowledge and operative practice, the text is meant for the structural engineer who still does not possess a direct experience or a deep knowledge of this sector. (De Stefano *et al.*, 2009)

4.2 Earthquake Early Warning

The main results obtained on the assessment of the potential of EEW for real time risk mitigation in Italy are based on the experience gained by the regional ISNET early warning seismic network, which was implemented in the Campania Apennines by AMRA Scarl, the University of Napoli and INGV-Osservatorio Vesuviano. Fast processing algorithms of earthquake data and preliminary feasibility studies of engineering applications were performed in the framework of a project supported by Civil Protection of Regione Campania.

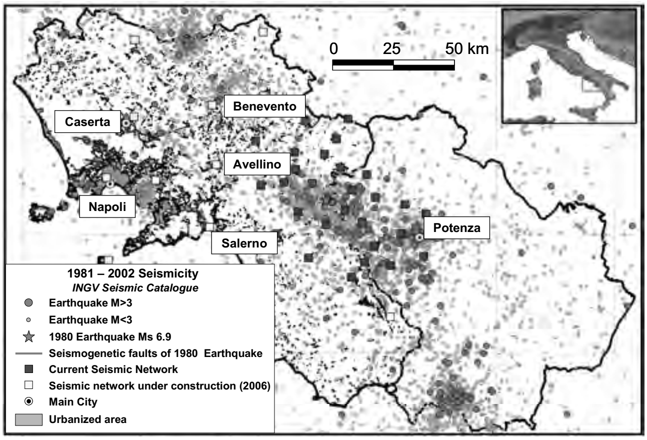


Figure 5. The Campania Earthquake Early Warning network and background seismicity.

The seismological activities performed in the RELUIS framework aim at complementing and better finalising the activities to engineering applications responding to the following objectives:

- assessment of the reliability of real time estimates of earthquake source parameters for engineering applications;
- development of an innovative real time procedure for real time production of ground shaking maps.

A technique for location has been formulated (RTLoc) that provides the best possible constraints for an earthquake hypocentre position immediately after its identification and it improves this estimate continually with time. This constraint is expressed in the form of a probability density function (PDF) for the location of the hypocentre in a three-dimensional space. The evolutionary probabilistic location is a critical element for an early warning message because it allows specific decisions to be taken on the basis of the interval of possible distances and directions from the source (Satriano *et al.*, 2008).

A real time evolutionary algorithm was developed also for magnitude estimation. It was based on a magnitude predictive model and a Bayesian formulation (RTMag). (Lancieri and Zollo, 2008). It is aimed at evaluating the conditional probability density function of magnitude as a function of ground motion quantities measured on the early part of the acquired signals. The predictive models are empirical relationships correlating the final event magnitude with the logarithm of quantities measured on first 2-4 seconds of record. These models were prompted and validated through the analysis of 256 shallow crustal events in the magnitude range 4 - 7.1 located over the entire Japanese archipelago. The peak displacement measured in a 2 seconds window from the first P-phase arrival correlates with magnitude in the range M = [4 − 6.5]. While a possible saturation effect above M about 6.5 is observed, it is less evident in an enlarged window of 4 seconds. (Zollo and Lancieri, 2008).

The developed method estimates the probability density function (PDF) of the predicted magnitude, at each time step after the onset of the event. The predicted magnitude value corresponds to the maximum of PDF, while its uncertainty is given at the 95% confidence level. The method has been applied to the 2007 M = 6.9 Noto-Hanto and 1995 M = 7.3 Kobe earthquakes. The results of this study can be summarized as follow:

- The probabilistic algorithm founded on the predictive model of peak displacement vs final magnitude is able to provide a fast and robust estimation of the final magnitude.

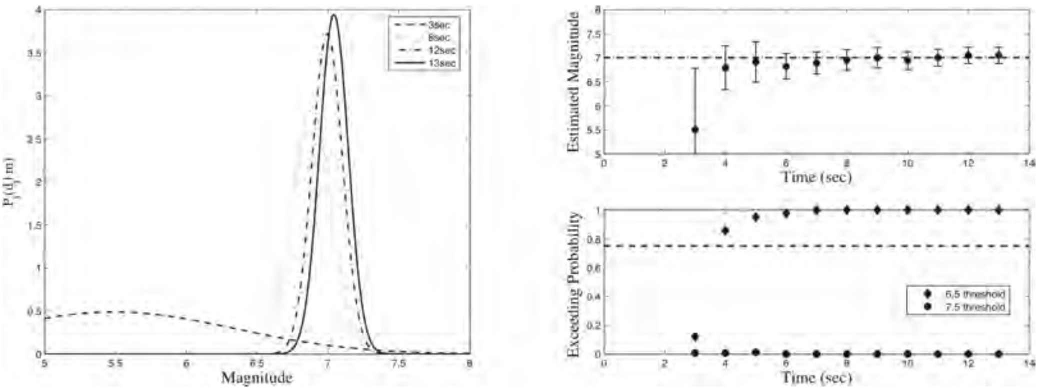


Figure 6. Application to the M=7 synthetic event. Left panel PDF distribution at several time step measured from the first P-phase picking. Right top, magnitude estimation with uncertainties in function of time. The dashed line is referred to the actual magnitude value, the errors represent the 95% of confidence bound evaluated as cumulative PDF integral in the 5-95% range. Right bottom, probability to exceed magnitude 6.5 and magnitude 7.5 thresholds in function of time.

- The information available after a few seconds from the first detection of the P phase at the network can be used to predict the peak ground motion at a given regional target with uncertainties which are comparable to those derived from the attenuation law.
- The near-source, S-phase data can be used jointly with P data for regional early warning purposes, thus increasing the accuracy and reliability of magnitude estimation.

Furthermore, a performance evaluation of the Early Warning system installed in southern Italy was carried out. To this purpose the effect of extended faulting processes and heterogeneous wave propagation on the early warning system capability to predict the peak ground velocity (PGV) was investigated in the southern Apennines (Italy) (Zollo *et al.*, 2009). Simulated time histories at the early warning network have been used to retrieve early estimates of source parameters and to predict the PGV, following an evolutionary, probabilistic approach. A real-time, probabilistic evolutionary algorithm for early warning was implemented whose main components are the automatic first-P picking, event location, magnitude estimation and prediction of ground motion intensity measure at a given target site. (Zollo *et al.*, 2006, 2007, Satriano *et al.*, 2008).

The system performance is measured through the Effective Lead-Time (ELT), i.e., the time interval between the arrival of the first S-wave and the time at which the probability to observe the true PGV value within one standard deviation becomes stationary, and the

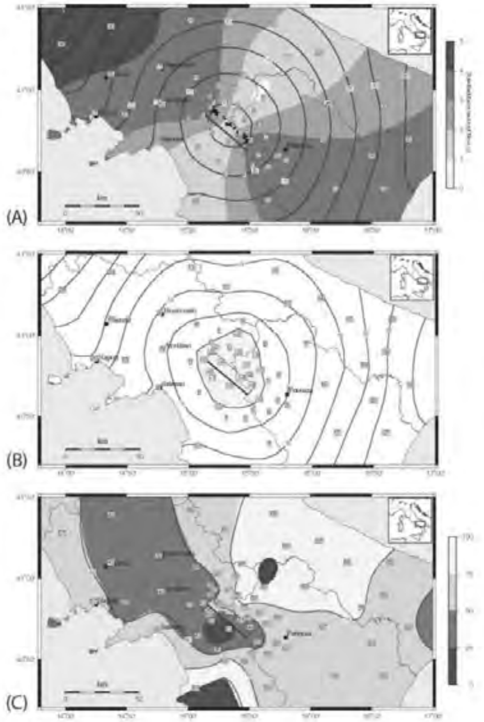


Figure 7. Regional maps of Early Warning System performance indicators. The maps are computed for 300 earthquake scenarios for an M 6.9 occurring inside the network. (a) Distribution of average Maximum Lead-Time (MLT) in seconds (isolines) and the associated range of variation (shaded scale). (b) Distribution of the Effective Lead-Time (ELT) in seconds. The shaded area inside the network indicates a zone with negative ELTs, where S-waves arrive before the distribution of PE becomes stable. (c) Distribution of PPE, the Probability of Prediction Error on parameter log(PGV) (see text for details). Shaded areas are obtained from a discrete representation of PPE, where lighter regions indicate a better efficiency of the EEWS to predict the PGV relative to darker regions.

Probability of Prediction Error (PPE), which provides a measure of PGV prediction error. The regional maps of ELT and PPE show a significant variability around the fault up to large distances, thus indicating that the system's capability to accurately predict the observed peak ground motion strongly depends on distance and azimuth from the fault. (Fig. 7)

To analyze the performances of the Campania early warning system in the case of moderate-to-large magnitude earthquakes a Matlab/Simulink® simulator was developed. It allows to represent the behavior of the EEWS without recorded signals, although this implies some intrinsic limitations. The flow chart of the simulator may be represented by three main bocks:

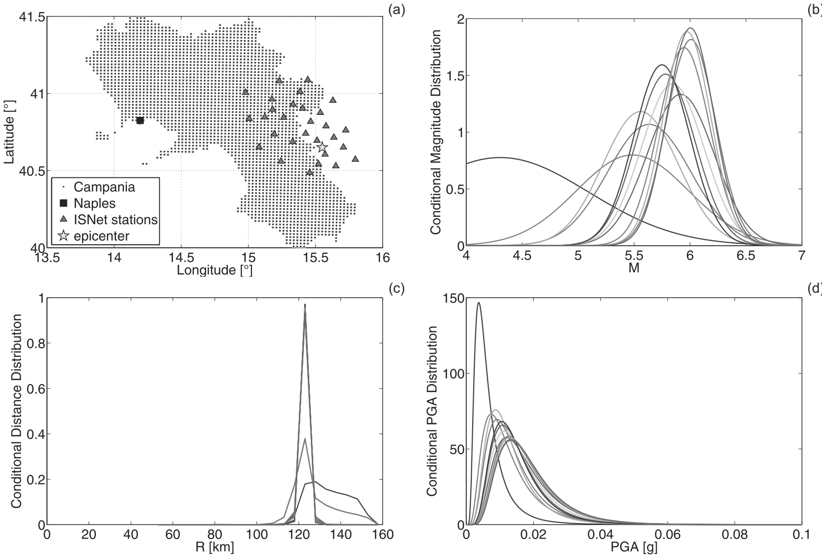
acquisition, computation, and alarm issuance decision. These blocks allow to represent the way the Campania EEWS operates in real-time probabilistic seismic hazard analysis (RTPSHA) (Iervolino *et al.*, 2009).

In the adopted approach, the estimation of critical ground motion becomes stable only after a number of stations have recorded the early signal of the event, therefore, there is a trade-off between the lead-time and the level of information for the alarm issuance. In fact, different definitions of lead-time are considered each of those correspond to a different number of stations triggered (i.e., a different level of information on magnitude and location of the event). For each of these proposed definitions, the minimum, maximum and average lead-times associated to each point (*j*) in Campania, have been computed using an appropriate regional 1D velocity model. According to this model P and S velocities, v_P and v_S respectively, are function of hypocentral depth and have a constant of 1.68 ratio (Satriano et al., 2008). In this framework the lead-time, T_k^j, for each point may be computed as:

$$T_k^j = T_S^j\left(h\right) - T_P^k\left(h\right) - \Delta t$$

where, $T_S^j\left(h\right)$ is the S-wave arrival time at observation point (*j*) and depends on the source depth *h*; $T_P^k\left(h\right)$ represents the time to trigger the first *k* stations and also depends on the source depth *h* because of the velocity model; Δt is the required processing time (assumed equal to 5 seconds and includes the required P-wave data recording at each station). $T_S^j\left(h\right)$ and $T_P^k\left(h\right)$ can be approximated using travel-time curves as a function of distance from the seismic source and *h.*

As an example, results of simulation for an *M* 6 event located within the network are given. The target site, used for the simulation, is Naples, capital of the Campania Region. The epicenter of the simulated event is 124 km far from Naples (Figure 8a). The probabilistic estimations of the magnitude and source-to-site distance (epicentral) are given in Figure 8b and 8c in terms of PDFs. In Figure 8d the real-time hazard is given in terms of PGA for the considered site (the used attenuation relationship is from Sabetta and Pugliese, 1996). Curves in Figure correspond different numbers of triggered stations during the developing seismic event, i.e., are function of time. Note that the hazard increases as time flows (more stations measures are available because more stations are triggered), consistently with the magnitude estimation method, which tends to underestimate *M*, especially at the beginning of the event (Iervolino *et al.*, 2006).



_______ t = 7s, n = 2; _______ t = 8s, n = 5; _______ t = 9s, n = 9; _______ t = 10s, n = 11;

_______ t = 11s, n = 14; _______ t = 12s, n = 16; _______ t = 13s, n = 18; _______ t = 14s, n = 20;

_______ t = 15s, n = 24; _______ t = 16s, n = 26; _______ t = 17s, n = 28; _______ t = 18s, n = 29.

Figure 8. Results of RTPSHA for Naples in the case of a simulated M 6 event (adapted from Iervolino et al., 2009).

To better visualize how estimations of specific PGA values evolve with time, in Figure 9a the exceedance probabilities are given for different possible values of PGA_c of Eq. (1). The plot in the figure refer to 100 averaged simulations (i.e., the stations measurements were simulated 100 times for the same earthquake). It appears the probability of exceedance does not change after 10 sec – 13 sec, i.e., after on average 11-18 stations have measured τ. In Figure 9b, the coefficient of variation (CoV, the ratio of the standard deviation to the mean) of the PGA is given for different conditions, as the number of triggered stations increases. In particular, the CoV of the PGA is computed, via the hazard integral (Iervolino *et al.,* 2006), in the following cases: (i) considering both PDFs of *M* and *R*; (ii) considering only the mode of *R* without its full PDF; and (iii) considering only the mode of *R* and the mean of *M* (Convertito *et al.*, 2008). This allows to assess the effects of uncertainty in estimation of *M* and *R* on the distribution of the PGA.

It appears that uncertainty on distance is negligible in respect to the prediction of PGA (CoV of PGA does not vary significantly considering or not uncertainty on distance); also uncertainty on magnitude, although larger than distance, is small if compared to that of the attenuation law, at least when several stations measures are available.

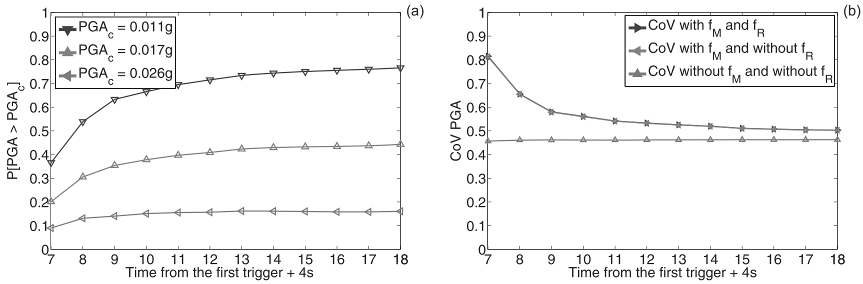


Figure 9. RTPSHA uncertainty analysis' results (adapted from Iervolino et al., 2009).

Figure 9 shows that there is a trade-off between the lead-time and the level of information based on which the alarm issuance is decided. Consequently, lead-time has been computed for k = 4, k = 18 and k = 29 considering as possible epicenters those occurring in the rectangular area which within the EW network, while the focal depth may be up to 12 km.

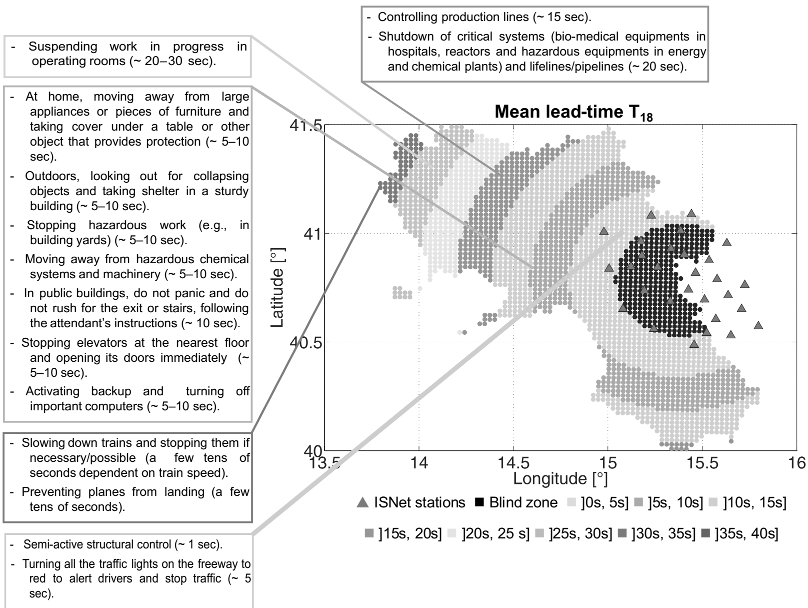


Figure 10. 18-stations lead-time map and possible risk-reduction actions (adapted from Iervolino et al., 2009).

Fig. 10 is similar to Fig. 7, with the difference that lead times are computed for a point source. P-wave arrivals are detected with an average delay of 1.5 s after the earthquake onset (delays ranging from 1 to 4s) from 1 station and with an average delay of 7.5 s (ranging from 5 to 11 s) from 18 stations and finally with an average delay of 12 s (ranging from 8 to 16 s) from all the 29 stations of the network The 18 stations lead time map for the Campania territory is reported in Fig. 10, where the "blind zone". i.e. the area where no warning is possible, is defined and the possible real-time risk reduction actions and the time they require are indicated.

A common way to evaluate the efficiency of a control system is the analysis of the false and missed alarms rates (Iervolino *et al.*, 2006). According to the decisional rule described by Iervolino *et al. (*2009) a false alarm occurs when the alarm is issued but the true intensity at the site PGA_T (T subscript means "true", indicating the realization of the PGA to be distinguished from the prediction PGA_C) is below the threshold value contrarily to the predicted intensity PGA_C. A missed alarm corresponds to not launch the alarm when it is, in fact, needed, Eq. (1).

$$\begin{aligned} &Missed\ Alarm: \left\{ No\ Alarm \cap PGA_T > PGA_C \right\} \\ &False\ Alarm: \left\{ Alarm \cap PGA_T \leq PGA_C \right\} \end{aligned} \tag{1}$$

Information and the uncertainties on earthquake location and magnitude are dependent on the number of stations triggered at a certain time. Therefore, in principle, the decisional rule may be checked at any time since the first station has triggered. Consequently also the false and missed alarm probabilities are time dependent. Figure 11 reports the missed (MA) and false alarm (FA) probabilities as a function of time from the first trigger for the three PGA_C values according to which 9a was computed.

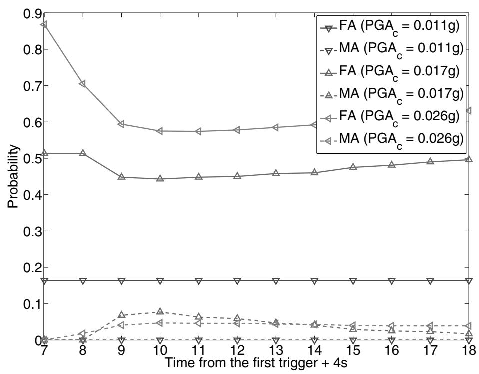


Figure 11. False and missed alarm probabilities for the PGA_C values and event of Figure.

A novel procedure for rapid estimation of ground-shaking maps after moderate-to-large earthquakes was developed in the project. The technique uses two different approaches to cover the data domain, one for the area within the seismic network, and the other for the area outside the network. The former uses an optimal data gridding scheme designed to account for bi-dimensional features of strong ground-motion fields, such as directivity, radiation patterns and focal mechanisms, to which most damage can be correlated. The basis of the mapping technique is a triangulation procedure to locally correct predicted data at the triangle baricenters where their vertices correspond to seismic stations. Furthermore, site-specific coefficients depending from the local geology have been introduced, to be used for correcting the estimates of peak ground-motion quantities at rock sites. The software package for ground-shaking maps computation and displaying was named GRSmap (Convertito et al, 2009). GRSmap has been tested off-line by the 23 November 1980 Irpina M 6.9 earthquake. The results have highlighted the ability of the procedure to produce ground-shaking maps containing bi-dimensional source effects, such as directivity or focal mechanism, with respect to the estimates performed using classical attenuation relationships. Furthermore, the obtained ground-shaking maps reproduce quite well the features of the complex fault mechanism that characterized this event, although the data was recorded at a sparse seismic network.recorded by a sparse network.

All the activities developed both in this project and in the project carried out for the Civil Protection of Regione Campania lead to the implementation of the ERGO (EaRly warninG demO) visual terminal. It was installed in the office of the dean of the school of engineering of the University of Naples Federico II and at the Civil Protection of Regione Campania. ERGO processes in real-time the accelerometer data provided by a sub-net of the Campania EEW network, performs RTPSHA and eventually issues an alarm in the case of events occurring with magnitude larger than 3 in the southern Appennines region. (Figure 12).

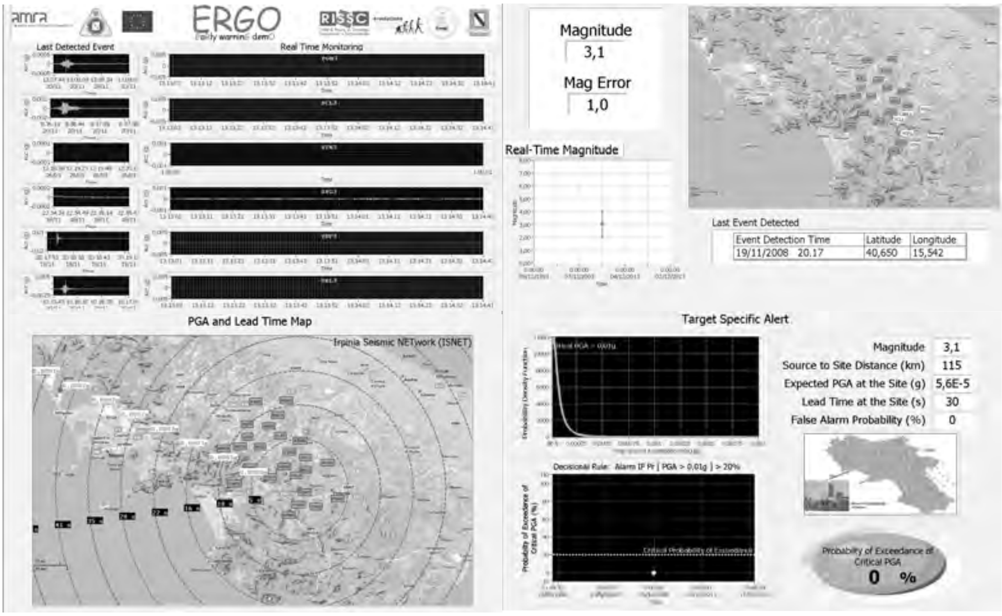


Figure 12. The ERGO EEW terminal.

Seismic signals produced by the Campania ISNet network can be utilized for automatic protection of structure and infrastructures. Each application requires a feasibility study and the implementation of reliable active or semi-active control devices.

The case of a reinforced concrete bridge was studied in this project. The bridge, characterized by all equal piers, has been modelled by a FEM computer program and analyzed considering different possibilities of seismic retrofit as base isolation and energy dissipation. Both strategies need a design approach based on a preliminary assumption about the intensity of the seismic action expected. A different strategy is based on the addition of a variable-damping, semi-active magnetorheological (MR) device (semi-active control). The properties of the damper, in this case, do not need to be fixed at the design stage, but could be varied according to the intensity of an incoming earthquake provided by the SEWS with the aim of ensuring the optimal structural response.

Two prototype semi-active MR dampers have been tested. Both devices, supplied by the German company Maurer Söhne, have the same nominal technical characteristics. Two

different types of dynamic tests were designed: imposed harmonic displacement tests and imposed constant velocity tests. Sinusoidal displacement cycles of constant amplitude were imposed: six different frequencies (0.5, 1.0, 1.5, 2.0, 2.5 and 3.0 Hz) with two different displacement amplitudes (±10 and ±20 mm) and eight different current levels (0.0, 0.5, 1.0, 1.5, 2.0, 2.5, 3.0 and 4.0 A) were considered.
A set of 17 European far field records has been used for non linear dynamic analyses of the bridge. The results in terms of relationship among the estimated PGA values and some structural response parameters have been provided.(De Iuliis *at al.*, 2008)
The result allows to assume that the PGA is a good parameter to consider when control strategy is based on the use of semi-active techniques in combination with SEWS and represents a good way to assign through a mathematical relationship, the mechanical properties of MR devices and to optimize the seismic response for different intensity of the earthquakes. (Occhiuzzi *et al.*, 2008)
In conclusion, this first attempt to exploit seismic early warning system for structural control using MR devices appear to be a promising technique, to be further enhanced in the future.

A feasibility study of an Earthquake Early Warning System (EEWS) in the Lombardia Region was carried out with the aim to determine whether this methodology is actually significant in reducing the seismic risk in the hospitals of this region. The design objective consisted, once determined the location and energy of the incoming earthquake, in issuing the hospitals with a warning which has to be transmitted sufficiently ahead before the arrival of damaging ground shaking in order to allow the medical staff to start the shut-down operations of medical equipments and all the necessary security operations. Albeit the efficacy proved in many applications around the word by an EEEWS, and in spite of the interest proved by the hospital officials and medical staff, the analysis demonstrates that the application of such an EEW system to the hospitals of Lombardia has important limits and it is not convenient on a cost/benefit criterion.

The used procedure is the following. After an exhaustive study of the geologic and seismotectonic context of Northern Italy, we individuated the seismogenic sources which may potentially affect Lombardia territory. In particular, the goal was to individuate earthquake scenarios potentially damaging for the health structures of the region. Within this context, and in order to carry out the most realist analysis, we defined two earthquake scenarios based on historical events which stroke Lombardy in the recent past (Figure 13):

- 1996, Reggio Emilia earthquake, Mw 5.4 (ZS 913);
- 2000, Salò earthquake, Mw 5.0 (ZS 907).

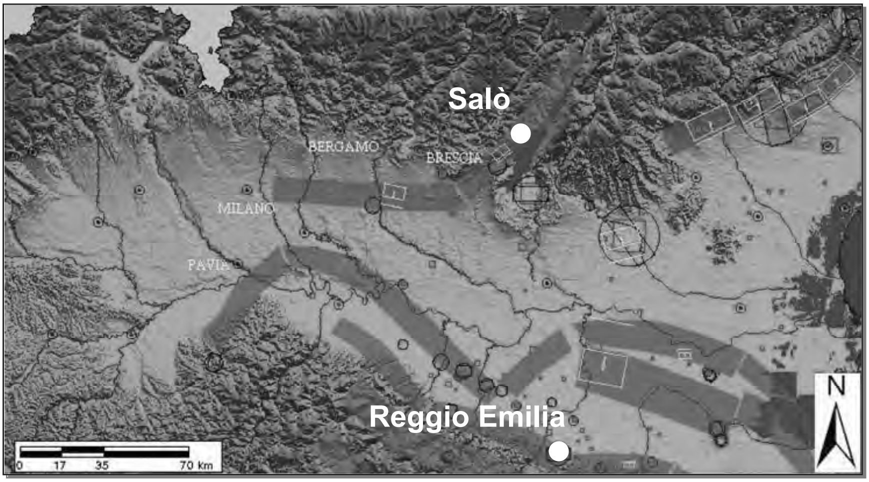


Figure 13. Seismogenic sources in Northern Italy (from the DISS 3.0 database - http://www.ingv.it/DISS/).

These two scenarios were used to evaluate potentialities and limits of an EEW system. A numerical simulation of ground shakings virtually generated at main towns through the territory. The simulation was carried out with the code proposed by Hisada and Bielak (2003). It is based on a cinematic source model in a stratified medium. The elastodynamic problem is solved by means of the generalized reflection and transmission coefficients (Chen, 1993; Hisada, 1994, 1995). In order to refine and verify the reliability of the solution, a second computation code was employed (Herrmann 2002). This latter computes the displacement generated at the surface by deep dislocation sources in a stratified medium on the basis of the Haskell (1964) model. The simulation procedure was then definitely validated by comparisons of the calculated signal with the signals recorded at seismic stations for the simulated events. As shown in Figure 14, real and simulated data show a good agreement, in particular for horizontal motion components, which are the most burdensome. Although the simulation codes do not allow for reproducing the high frequencies of real recordings, a spectrogram analysis allowed for verifying that highest peak motions of recordings were at frequencies that are included in the synthetic spectrum, with good agreement in terms of temporal phases. This procedure allowed us to estimate the ground shaking levels and the warning times theoretically issuable to the sensitive objectives through Lombardia. Sensitivity analysis of the model to main parameter variation was carried out through. With this aim, we simplified the entire procedure by substituting a simplified geometric ray model to the rigorous simulation. The simplified model is based on Snell's law, with plane waves propagating in a layered media. We found good agreement between the rigorous procedure and the simplified model only for high epicentral distances, whilst in the near field, as expected, the two models are not comparable. The geometric ray model yields therefore reasonably good results in the far field, whilst in the near field it is too simplistic to exhaustively reproduce the ground motion. This makes it unserviceable to our purposes. The geometric ray model can however be employed in order to identify the variables which influence the most the ground motion generated at a given point by an earthquake scenario, at least in the far-field. In this sense, shear wave velocity of the bedrock is the most important parameter affecting the results. Thickness of the shallower layers plays a quite insignificant role.

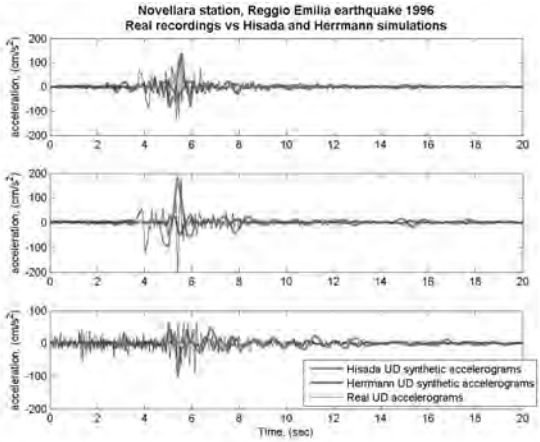


Figure 14. Comparisons between simulation results of the Hisada and Herrmann codes (red and pink lines, respectively) with the real recordings (blue lines) at Novellara seismic station, during the 1996 Reggio Emilia earthquake (INGV Milan courtesy).

We also evaluated directivity effects by assuming a circular configuration of virtual receivers (see Figure 15). This feature is particularly critical and imprescriptible when setting an early warning system and subsequent emergency interventions.

Once the procedure for estimating ground shaking levels and warning times at main Lombardy towns was settled and verified, we identified a future possible realistic seismic scenario, critical for Lombardy region. We evaluated the ground motion expected at given sites due to the reactivation of the "Orzinuovi" seismic source, located in the central zone of the Central and Southern Alps seismogenic area. Parameters of the source were retrieved from the database of seismogenic sources DISS 3.2 (2006). The simulation technique illustrated above allowed the calculation of early warning times and levels of motion expected at major urban centres and the assessment of possible directivity effects (see Figure 15 left and Table 2).

For the described seismic scenarios, site effects due to shallow soil characteristics were evaluated, referring to the soil categories reported in EUROCODE 8. The effect of wave travelling through soft shallow materials yields an amplification of ground shaking motion with respect to a seismogram recorded on rock site. The intensity of ground amplifications are function of the natural frequency of the system, with peaks up to 5 times the motion on a rock profile.

In order to quantify the deviation between the theoretical and the expected pre-alarming times that can be issued to a particular location taking into account the "technical" time, we simulated different network configurations and associated algorithms. The so-called "technical time" includes the time necessary to compute location and magnitude of the incoming event (which is function of the seismic network configuration and the used algorithm) and transmission time for the alarm issue (which depends on the employed communication technology). It therefore represents the difference between the time at which first seismic waves reach the receiver and the alarm activation at sensitive objectives.

Table 2. Theoretical warning times and expected shaking levels issuable to main town locations in Lombardy region following Orzinuovi seismogenic source activation. Shaking levels are computed both, with the numerical scheme described in the text, and the attenuation relationship proposed by Sabetta and Pugliese (1996).

Loction	Epic. Dist. (km)	PGA horiz. (g)	PGV horiz (cm/s)	PGA vertical (g)	Theoretical warning time (s)	Sabetta & Pugliese attenuation relationship 1996: horiz. PGA (g)	Sabetta & Pugliese attenuation relationship 1996: horiz. PGV (cm/s)
Crema	**24.1**	**0.07**	**6.8**	**0.04**	**3**	**0.080**	**6.10**
Bergamo	**34.0**	**0.05**	**4.8**	**0.02**	**5**	**0.057**	**4.39**
Milano	**60.9**	**0.02**	**3.0**	**0.006**	**10**	**0.030**	**2.47**

In order to estimate the most efficient seismic network configuration, different geometries have been considered. They are listed and discussed in the following.

- "*Regional early warning*", or "*front detection*": a net of seismometers is located close to the earthquake source area and alarms are issued to more distant urban areas. Earthquake characteristics are estimated using first seconds of recordings at numerous stations (the number depending on the desired accuracy and algorithm chosen).
- "*On-site early warning*": seismic stations are located next to the objectives. This system is sometimes preferred to the previous one since it is faster, it does not need a large number of seismic stations, and probability of false alarms is reduced.
- "*Hybrid system*": it combines information coming from the recordings at seismic stations close to the seismic source with heuristic information on the location of seismic faults, available crust model, etc. This system allows for more precise estimates, with reduced estimation times with respect to the "*Regional*" system, since the algorithm converges faster due to the additional information.

Lombardia hosts more than 200 health centres, therefore an "*On-site*" configuration of early warning cannot be applied readily.

Conversely, technical times expected for "*Regional*" EEWS are too high to make this kind of system really useful: the "blind" zone, in which no warning can be issued, is too large for making such economic investment interesting. We have therefore selected a "*Hybrid*" system, which merges together information contained in the recordings of the incoming event at triggered stations near the seismic source with heuristic information from not-yet-activated stations, a-priori knowledge on the location of seismic faults, available crust models, etc. This allows for faster algorithm convergence and reduction of the blind zone.

Figure 15 (right) shows a comparison between the blind zone amplitude for a *Regional* EEW configuration and for a *Hybrid* EEW configuration. In the first case the blind zone amplitude ranges between 24 and 30 km, while in the second case this is reduced to 13 to 19 km wide.

Figure 15. (Left) Directivity effect of the Orzinuovi seismic source, from simulations by Hisada and Bielak 2003 code. (Right) Comparison between blind zone amplitudes in case of Orzinuovi seismic source re-activation for Regional EEW configuration (in blue) and for Hybrid EEW configuration (in orange).

Seismic sources affecting Lombardy outside the regional territory are characterized by small to medium magnitude potential and they would generate ground shaking levels of low intensity (comparable with micro-seismicity), at the major urban centres due to the large epicentral distances (if not accounting for ground amplification). In this case the installation of an EEWS would be unessential. In light of this, it is therefore of great interest to focus the attention on seismic sources inside the regional territory which could be capable of generating ground shaking potentially more damaging for biomedical equipment and dangerous for patients undergoing surgery.

We have estimated the theoretical warning times and shaking levels we expect at sensitive objectives. In order to assert the real efficiency and utility of an EEWS in Lombardy Region, we have to compare such results with the needs of the final users. A positive result of this comparison will definitely establish the importance and the usefulness of such a system. It will allow, moreover, to fully take advantage of the potentialities the EEW system can offer.

We made contact with biomedical engineers (in order to establish minimal damaging shaking levels for medical equipments and the most sensitive systems) and with medical staff (in order to identify critical and particularly sensitive medical processes as well as the estimated necessary safety measures and timings related to them).

The biomedical engineers indicated as the most critical systems in case of incoming earthquake, for which an EEWS may be essential for safety, all those processes related to gas transport, especially combustible gases such as oxygen. Note that an oxygen network distribution is deployed through the whole hospital surface. An early warning would allow for activating the localized secondary distribution systems and to isolate each department from the rest. Procedures of safety, shut-down and compartment isolation could therefore be activated in advance on the incoming ground motion.

Examples of equipments most sensitive to vibrations are radiology, surgery, ophthalmological laser, TAC, or microelectronic equipments. In these cases, however, an EEW system is not an appropriate safety measure since an automatic shut down of these equipments would engender more damage than the vibration itself would provoke (if we consider the earthquake magnitude and shaking levels expected for Lombardy Region). In addition, quite high shaking levels would be necessary to actually damage the machinery, except for the case of particular aim equipments.

On the other hand, in the cases of medical equipments and systems to which an extraordinary vibration would cause damages to the patient under treatment, an early warning before beginning of the shaking, would allow to perform minimum safety measures for the patient. These processes include surgery, ophthalmological laser, hanging equipments, orthopaedic processes, diagnosis equipments etc.

In all described cases, the intervention of technical staff is however strongly advisable. Expected times necessary for staff intervention in security arrangements of medical equipment are indicated of about 15 minutes. These are incompatible with an EEWS system possible warning times.

As regarding to safety measures addressed to patient under treatment, times needed for immediate and first safety actions are of about 1-2 minutes. These are of the same order of magnitude as expected EEW times.

Finally the conditions needed for protecting a relevant Cultural Heritage structure such as the statue of David by Michelangelo, using very low damping passive insulators and locked in normal condition and activated by a SEW triggered signal were studied using the ENEA shaking table facilities. A study the configuration of a dedicated net of single-station systems within the AMRA regional early warning network to activate the fire protection system of the “campo Oli” in Val D’Agri, Basilicata, Italy.

5 DISCUSSION

The activity developed during the project allowed the achievement of all the main expected outcomes, which are indicated in the Introduction of this report.
In particular in Task 1 for a better organization the deliverables indicated in the original project have been merged into three main deliverables:

1. Development of innovative methodologies for the evaluation of dynamic response of strategic structures using micro tremors and environmental noise.
2. Definition of guide lines for permanent health monitoring of structures;
3. Design and implementation of health monitoring systems (including software) applied to different structural typologies for acquisition, processing and transmission of synthetic data on the damage level of a structure;

Some of the activities which were to be implemented in Molise have been applied also on structures in Regione Campania to test its integration with EEWS.
The main objectives of Task 2 were relative to the feasibility of several engineering applications of EEWS. The main outcomes have been:

1. the existing Campania EEW seismic network is fit for engineering applications for real time risk mitigation of several infrastructures and lifelines. A general procedure for selecting the proper actions to be performed has been developed and it is ready for implementation to any specific case in any context. A visual alert system was developed and implemented;

2. a novel procedure for rapid ground shake maps production from the data recorded at the Campanian seismic network has been produced;
3. a feasibility study of the applicability of EEWS to real time mitigation of hospitals in Lombardia indicated that EEWS are of scarce usefulness for mitigation of inconvenience related to the occurrence of an earthquake;
4. an evaluation of the performance of magneto-rheological devises as interfaces for real time mitigation of risk of structures has indicated good perspectives of application.

6 VISIONS AND DEVELOPMENTS

In the development and implementation of innovative methodologies of Health Monitoring of Structures, Italy is the forefront both in research and application.
Future development in this field must include:

- the integration of structural monitoring capabilities and early warning technologies for real time damage assessment during an earthquake and for possible real time risk reduction actions;
- the passage from global information on the state of damage of a structure to detailed specific information;
- the implementation of procedures for inferring actions consequent to a given damage level.

The situation for Earthquake Early Warning is more complex. Italy is ready from a seismological point of view, both as regards the distribution of seismic stations and the development of very fast processing techniques. In the RELUIS and in other projects the theoretical procedures for engineering application have been developed. However, implementation of EEWS to specific and general cases is lacking and Italy is lagging behind countries like Japan and Taiwan, but also other European countries such as Turkey and Romania. An overview of the global situation is available in Allen *et al.*(2009), Kanamori (2006), Gasparini *et al.*(2008).
In some countries, like Japan and Mexixo, EEW is used both to protect some infrastructures and to give general rapid notification of an impending earthquake to people. The latter application requires a long term education to individual and social responsibilities. It can be reached in Italy (and in many other European countries) only after a long time from now. This activity can be started in limited cases (schools, industries, dam, power plants, etc.). Anyway EEW is going to have a growing role in Europe and in the world, mainly for the protection of infrastructures. Critical infrastructures consist of those physical and information technology facilities, networks, services and assets that, if disrupted or destroyed, would have a serious impact on the health, safety, security or economic well-being of citizens or the effective functioning of governments. Critical infrastructures extend across many sectors of the economy, including banking and finance, transport and distribution, energy, utilities, health, food supply and communications, as well as key government services. Indeed, it is not only the protection of critical infrastructures that is a goal of EEW systems, but also allowing them to remain operational during a post-earthquake period. It is obvious that some infrastructures, such as hospitals, would in fact be even more necessary during such a period, and therefore should receive particular attention in developing actions that will ensure their operational status.
In some well documented cases, the largest proportion of fatalities, casualties and economic losses may result from secondary events triggered by an earthquake, e.g. tsunamis (Lisbon, 1755, Sumatra, 2004), fires (Tokyo, 1923; San Francisco, 1906), industrial accidents (Izmit,

1999) and landslides (Neapolitan Earthquake, 1857, El Salvador, 2003) and both tsunamis and landslides (Okushiri, 1993). Triggered or cascade effects will become more important, owing to the increasing interconnection and complexity of modern societies. EEW systems can play a major role in lessening the consequences of these secondary events. Gas pipelines would be cut off to reduce the likelihood of fire, while the examples given above dealing with trains and bridges would lessen the likelihood of derailments and additional traffic casualties in the event of bridge or tunnel failures. Of particular importance would be the integration of EEW systems with those dealing with tsunami and landslide hazard.

EEW techniques are being developed where an earthquake's characteristics, namely its epicentre, focal depth and magnitude, are determined with a high degree of accuracy even before the earthquake has finished. Such information is of crucial importance in the generation of near real-time scenarios of the expected geographic distribution of damage. Techniques of this type, known under the general term *shake maps*, have proved to be very useful for the timely mobilization of the civil protection groups and rescue teams. There are currently available products that generate such maps that are then made available on-line with special products made available to specific end users such as the media.

EEW would also play a crucial role after an event, with respect to aftershock detection, Aftershocks always occur following a substantial earthquake, and are a source of considerable worry, not only because they keep under stress an already traumatised population, but because they hinder rescue operations, and cause additional damage to the weakened built environment. Therefore, part of EEW is the development of methodologies to obtain some predictive capacity for aftershock behaviour, allowing a greater degree of flexibility for rescue operations and to keep the general public informed.

EEW is not only a scientific and engineering endeavour, but has an essential societal aspect. In fact public education and understanding of the social attitudes of a population are required to fully exploit and adapt an EEW system to a given population.

Even considering only technical aspects of EEW, this issue is complicated by the fact that there is no possibility of simply transferring an EEW methodology from one location to another, as the actions appropriate for one region, a function of the geology, nature of the seismicity, available resources and social attitudes, may be totally inadequate for another.

Early Warning methods require customized systems for each situation. Nonetheless they are based on technological components and information systems with some common requirements:

- Low cost and high space density sensor networks;
- Robust and reliable signal transmission systems able to operate under extreme conditions;
- Capability of fast data processing and modeling (minutes or less);
- Information and preparation strategies;
- Solution of legal problems.

Application to complex situations is hampered by insufficient knowledge on the physics and time/space evolution of hazard sources (earthquakes, eruptions, floods, mass movements…), by a need of optimization of monitoring and real time data processing systems, of assessing the reliability of warning (false alarms minimization) including consequence based analysis, Other issues, such as the development of structural control systems and interface with EWS, people education and information, development of monitoring and signal transmission systems resistant to extreme conditions, are also relevant the application of these methods.

The efficient implementation of EEW in Italy requires the accomplishment of the following objectives:

1. Interoperability and data fusion from all the major, local and national accelerometric networks;
2. Continuation of the feasibilities studies, such as those performed for Campania and Lombardia in the present RELUIS project, at least for the highest seismic risk areas of the Italian territory;
3. Extensive cost-benefit analysis of each potential application, following the lines developed in the present RELUIS project;
4. Improvement of the automatic methods for fast detection and determination of the focal parameters of the earthquake to better than a few seconds from the first P wave arrival;
5. Improvement and validation of innovative fast active and semi-active control systems for structures and infrastructures;
6. Improvement and implementation of methods for rapid and accurate impact assessment;
7. Identification and resolution of legal problems (e.g. liability in case of false or missed alarms).

The latter point is of utmost importance when the warning affects public utilities which can be restored after long time. In fact “early warnings” are not considered in the legislation of Italy (and perhaps of Europe).

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DEFINITION AND DEVELOPMENT OF DATABASES FOR RISK EVALUATION, EMERGENCY PLANNING AND MANAGEMENT

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1 INTRODUCTION

The development of databases, vulnerability maps and risk maps meets two fundamental demands of the Department of Civil Protection (DPC): *a)* developing a strategy of seismic prevention at the national scale and at the local scale, *b)* planning the emergency and managing it in the post-event phase.

The project involved a work of systematization, improvement, updating and experimentation in different fields, based on a large amount of available data, provided by a wide set of researches in recent years.

The research activity was subdivided into the following Tasks:

1) Ordinary buildings;
2) Public and strategic buildings;
3) Infrastructures;
4) Urban systems and historical centres;
5) Monuments;
6) Emergency planning and management;
7) Development of databases and GIS.

The research was developed in coordination with the project lines devoted to the assessment of existing structures, in particular with the Line 1 "Assessment and reduction of vulnerability of masonry buildings" and the Line 2 "Assessment and reduction of vulnerability of existing RC buildings". The Project was also developed in close coordination with the DPC.

2 BACKGROUND AND MOTIVATION

2.1 Ordinary buildings

The evaluation of vulnerability and risk to buildings represents an essential tool to plan the intervention in a framework of limited resources. Several tools and products available at the beginning of the project were characterized by significant heterogeneity, and the systematization of the available databases – many of which developed within the framework of LSU Projects, and other derived from post-earthquake investigations – started since a few years.

This systematization, started in the years 2002-2004 within the framework of the Project GNDT-SAVE, allowed an advanced utilization of these databases, yielding updated maps of vulnerability and risk to ordinary buildings, schools and hospitals of Southern Italy. It allowed also to improve the methods for the estimation of vulnerability and to develop models for the estimation of indirect losses and socio-economic losses caused by earthquakes.

An integration has been carried out of the existing databases with the survey of vulnerability and damage in consequence of the Molise 2002 earthquake. These surveys represented also an important test of AeDES and MEDEA forms, as well as of integrated procedures for the vulnerability survey of centres and the development of GIS for risk analysis and scenarios simulation.
The risk maps produced within the framework of the Project GNDT-SAVE were developed using the hazard maps published in 2001 by USSN and those produced by the DPC, based on a calibration procedure consisting of: 1) correlation between the number of dwellings and number of buildings; 2) assignation of vulnerability class; 3) definition of a vulnerability parameter at the urban scale.
Based on the damage data collected in several post-earthquake surveys, USSN developed a representation of the propensity of buildings to physical damage through Damage Probability Matrices as function of macroseismic intensity, through fragility curves as function of PGA and, for RC buildings, as function of the spectral ordinate of the first mode. The analysis of these databases allowed also to perform a statistical evaluation of the consequences of the physical damage in terms of usability and economic losses.
Methods and codes for the evaluation of reconstruction costs have been developed as well, based on the forms for damage survey and usability.
In 2001, USSN published an update of the risk map of Italian territory, accomplished in 1996 for the DPC. This update was determined not only by the availability of new hazard maps, but also by the evaluation of new Damage Probability Matrices and new fragility curves, in terms of either macroseismic intensity or parameters of ground motion (PGA, EPA, Arias Intensity, etc.). A CD-ROM was produced, with the data relevant to hazard, seismic zoning, territorial characteristics, seismic risk of the 8100 Italian municipalities.
The CNR-DAST (now CNR-ITC) had a wide experience in the field of collection and elaboration of vulnerability data, in advice and research in the emergency and post-emergency phase, as well as in the prevention of seismic risk at the regional scale. In particular, elaborations have been carried out on the LSU census in seven regions of Southern Italy on public buildings, samples of ordinary buildings, cultural heritage, urban centres. The elaborations of the samples of ordinary buildings allowed to identify the building types, both the most numerous ones and those with highest vulnerability, yielding to the “regionalization” of the vulnerability characteristics. Other activities were: estimation of the seismic risk at the urban scale, with a procedure developed by a Working Group GNDT-SSN, using the data of the LSU inventory; analysis of the data relevant to the reconstruction in Marche Region after the 1997 earthquake, in particular for ordinary masonry buildings and Programmes of Urban Restoration; experience in the emergency phase of the Umbria-Marche 1997 and the Pollino 1998 earthquakes.
UNIGE developed two methods for vulnerability analysis: a macroseismic method and a mechanical method, both applied in the Project “Scenario analysis in western Liguria and solutions for the preservation of historical centres”
The macroseismic method, to be used for risk analysis when the hazard is defined in terms of macroseismic intensity, was formulated on the basis of the definitions of the EMS-98 applying the classical probability theory and the fuzzy sets theory, describing the vulnerability of individual buildings and of categories of buildings in terms of a vulnerability index and a ductility index.
The mechanical method was proposed for hazard analyses whose results are available in PGA or spectral ordinates. The method is based on the Capacity-Spectrum, with capacity curves associated to each of the building types in the classification. Both approaches allowed to associate a mean behaviour to each building type; variations could be considered when

evidence was available, at the regional scale, of weakness of a particular type or, on the contrary, of a better response compared to the mean.

Over the last two decades, POLIMI developed the analysis of damage and decay of masonry originated by chemical-physical and mechanical causes, with reference to construction types. Despite the variety of situations which can be encountered in the constructive practice, and the consequent difficulty to describe the masonry types within a unitary framework, a classification had been proposed according three fundamental types, corresponding to as many types of interface between the leaves. These types had been enriched by subsequent surveys on masonry cross sections in several Projects CNR-GNDT, CNR-MIUR and PRIN.

The analyses carried out by UNIPD on the historical centres of Umbria-Marche and Veneto highlighted the typological scatter of existing buildings (isolated buildings, rows and complex buildings), and consequently the need for suitable criteria for structural analysis, particularly for complex aggregates, in relation to specific issues as: porches, large rooms, offsets between vertical and horizontal elements, buildings on slope, etc.

As regards RC buildings, they often showed a unsatisfactory behaviour during past earthquakes, particularly when they had been designed considering only vertical loads (Irpinia 1980, Japan 1995, Turkey 1999, Greece 1999).

At the beginning of the project, many international guidelines were available, e.g. in Japan (JBDPA, 1977) and in the USA (FEMA, 1992). The software HAZUS was available as well (FEMA-NIBS, 1999), developed in the USA, whose results have been determined from the observation of the behaviour and damage in local earthquakes, and therefore cannot be directly used in Italy because they refer to structural types different from Italian ones.

The literature on the seismic vulnerability of RC buildings is very rich, both in the national and in the international field (e.g. Calvi and Recla, 2000; Cosenza *et al.*, 1999; Fardis, 1998; Kappos, 2001; Kunnath *et al.*, 1990).

The modelling potential and the theoretical-experimental knowledge now available on the seismic behaviour of RC elements (Fib, 2003; Cosenza *et al.*, 2002), allow to apply evaluation methods based on mechanical modelling. In fact, the application of methods based on the Damage Probability Matrices is problematic for RC buildings: the databases on damage caused by earthquakes are relative scarce, and the buildings are usually classified in one vulnerability class. For these reasons the research is being developing evaluation procedures based on mechanical models, more or less simplified (Calvi, 1999; Cosenza *et al.*, 2004; Cosenza *et al.*, 2005). They have the advantage that the seismic capacity of the buildings can be compared with the hazard, expressed in physical quantities. Moreover, it is possible to estimate the effect of structural interventions (Modena *et al.*, 2000) and a possible mitigation of the scenario at the urban scale. Even though this approach was relatively recent at the beginning of the project, some applications for the vulnerability evaluation at the urban scale had been carried out by UNINA-DIST (Cosenza *et al.*, 2005b; Faccioli and Pessina, 1999).

UNIBAS carried out investigations, partially funded by DPC-SSN, on the seismic vulnerability of representative types of existing RC buildings, with moment resisting frames designed for vertical loads.

A first study on 2-D structural types built after 1970 (Masi, 2001; Masi, 2003), highlighted a set of typological elements: (i) the almost exclusive presence of frame buildings; (ii) the presence of frames in only one direction, typically the longitudinal one, with the exception of perimetric frames; (iii) the substantially symmetrical distribution of stiffness in the transverse direction, because of the position, usually centred, of the stairs.

A method for vulnerability evaluation was purposely developed, based on simulated design according to the regulations and the practice of the construction period, yielding curves of

damage vs. intensity with a consequent assignment of the vulnerability class according to the EMS-98.
This study was subsequently extended to the most widespread 3-D types, made of assemblage of plane frames, with reference to structures built after 1970, designed for vertical loads (Masi and Vona, 2004a). The seismic vulnerability was also evaluated of 2-D structures, similar to those analyzed by the first study but built between 1945 and the early '70s (Masi and Vona, 2004b).

2.2 Public and strategic buildings

Some recent earthquakes showed that many public and strategic buildings are very vulnerable to seismic actions. In Italy, the OPCM 3274 started a mid-term program for the evaluation of vulnerability and risk of every public and strategic building.
The level of international care is high as well. For instance, the OECD organized a meeting of an "ad hoc Experts' Group on Earthquake Safety in Schools", Paris, 2004, which produced recommendations to be submitted to its Council for further and more incisive prevention actions.
The Italian studies on the seismic vulnerability of buildings received a significant impulse after the 1980 earthquake, with different approaches and different detail level (Braga *et al.* 1982, 1987; Baratta, 1985; Benedetti and Petrini, 1984, Bernardini and Modena, 1987; Giuffrè and Carocci, 1996). The methods developed for ordinary buildings have been applied also to public and strategic buildings; however, their results are sound in the mean, and not for individual buildings (Cherubini *et al.*, 1999).
The Project GNDT-SAVE (2002-2004) characterized the most widespread structural types of schools and hospital, made a more accurate vulnerability evaluation on a limited sample of schools, set up "analytical" methods for vulnerability evaluation, compared the vulnerability evaluations at different levels of detail.
Evaluations for individual structures, rather than for classes of structures, require new methods, operating at a definition level higher than the methods currently used for groups of ordinary buildings. At the same time, methods of very high detail, as those used for assessment, cannot be used because of the enormous number of public and strategic buildings to be evaluated. In fact, it is possible to foresee that this evaluation will be made through phases with different levels of detail, in order to identify and select, in the first phase, the buildings with highest risk, estimate the economic expense, and proceed to the subsequent widening.
For these reasons, the Project GNDT-SAVE focused on methods which use accurate data on geometry and material properties, and yield relatively robust and reliable results through simple models. Considering RC moment resisting frames, and in particular those designed for vertical loads, it can be observed that they usually have low reinforcement ratio, so that collapse usually occurs according to a storey mechanism. In these cases, the positive contribution of the infills is often decisive, and is taken into account through simplified formulations.
The method of analysis, developed by UNIBAS, was essentially based on the behaviour factor, within the framework of a force-based approach. A small value was usually assigned to the behaviour factor, depending on the compression level of the piers and on their possible shear failure (Dolce *et al.*, 2003b; Dolce and Moroni, 2005). After the Molise 2002 earthquake, this method was extensively applied to schools in Basilicata, Molise and Tuscany. This method can be applied to a wider class of buildings, designed only for vertical loads. Moreover, the application of the method to buildings with different mechanisms yields conservative evaluations.

A similar method was developed for masonry buildings with rigid diaphragms, and in-plane shear mechanism of the walls.

2.3 Infrastructures

The management of individual structures, or sets of structures, requires a systematic approach ensuring their reliability, a suitable condition state and an acceptable risk level, within the limits of the available resources. Any maintenance model, either deterministic or probabilistic, aims at predicting the future behaviour of the structure under study. However, the present and future states of the structure are associated to different levels of uncertainty, which make the probabilistic approach usually compulsory. Over the last forty years, several studies have been published on optimal maintenance models, with particular reference to modelling (Sherif and Smith, 1981; Cho and Parlar, 1991; Dekker, 1996).

With reference to the bridge management at the network level, the primary objective is that to provide the responsible agencies with the tools for optimal resource allocation, maintaining at the same time a suitable level of safety and usability of the whole stock. The prioritization criteria make use of concept as the Condition State (CS), safety, maintenance planning, cost. Nowadays, most management systems are typically based on the evaluation of the CS through visual inspection, as PONTIS (Thompson *et al.* 1998) o BRIDGIT (Hawk and Small 1998). In these systems the prioritization is based on cost minimization, while safety is implicitly assumed as correlated with the CS. A weak point of this approach is that the influence of the defects on the reliability of the bridge is neglected, and no evaluation is performed of the load bearing capacity. The theoretical research on bridge management according to reliability criteria received a strong development in the last decade, covering issues as: "condition ranking prioritization" (Stewart *et al.* 2001, Akgul and Frangopol 2003), optimal inspection strategies (Sommer *et al.* 1993; Onofriou and Frangopol 2002), optimization of maintenance and repair interventions (Augusti *et al.* 1998; Frangopol *et al.* 1997; Frangopol and Estes 1999; Kong and Frangopol 2003). In most cases, the optimal program of inspection and repair is based on the minimization of cost over the lifetime, maintaining at the same time an acceptable degree of reliability. Branco and Brito (1995) proposed a decision system based on the definition of a Cost Effectiveness Index (CEI) for each option, assuming the possibility to define the benefit of an intervention in monetary terms. More recently, Frangopol and Neves (2004) proposed an approach based on CS, safety and cost. The prioritization is based on the minimization of an objective function accounting for these three quantities, and provides a set of optimal solutions. It is worth mentioning that decision systems based on multi-objective optimization apply to a wider field of problems in civil engineering (e.g. Chunlu and Hammad 1997; Lounis and Vanier 2000, Augusti *et al.* 1994, Augusti and Ciampoli 1998).

In the past, structural monitoring was intended as an extraordinary intervention, justified by the importance of the structure (economic and/or strategic importance, high historical and cultural "value"), or by the contingent necessity to assess its precarious safety state. Nowadays the trend is to consider the monitoring system as integrating part of the design, in the case of new structures, or of the retrofitting intervention, in the case of existing structures. This philosophical change can be explained on one hand with the recent development of instrumentation and methods of analysis; on the other because the responsible agencies become aware of the necessity to base the judgment on the structural safety on detailed and updated information; some structural collapses accelerated this process. The criteria of data collection and interpretation changed as well. In the past, the investigation techniques had local character (Shepherd and Charleson, 1971; Tanaka and Davenport, 1983; McLamore *et al.*, 1971). More recently, the research focused on the possibility to obtain information on the integrity state of the structure on the basis of vibration measures (Farrar *et al*,. 1994; Maeck *et* *al.*, 2000). The idea at the base of the method is that modal parameters are function of the physical properties of the structure; therefore, variations of the physical properties cause variations on modal parameters (Doebling e Farrar, 1998). One of the advantages of this method is that a local measure can provide information on the global behaviour.

2.4 Urban systems and historical centres

Previous researches of the DPC led to the development of a method to study the urban systems, based on the so-called "strategic system", i.e. the set of the physical and functional component of the town which must warrant the functionality after a seismic event. The essence of the strategic system, which shall not be confused with a simple sum of strategic buildings, is the concept of "Minimal Urban Structure" (MUS). This concept, born to transfer the interventions for risk reduction into town planning, showed itself to be a useful tool for the study of urban systems. At the beginning of the project, the method was tested in the urban centres of Rosarno, Melicucco, Nocera Umbra and Reggio Calabria.

A method was developed by the DPC to analyze the reliability of a road network interacting with the system of buildings and the with the emergency system. In particular, the method accounted for the possible capacity reductions of the roadway caused by total or partial collapses of the buildings, and to props. Forecasts could be made in the short term, determining the possibilities to connect the different strategic points of the urban system, or at mid- and long-term, predicting the service conditions of the network.

At the beginning of the project, the contributions of the DPC to the evaluation of vulnerability and risk to historical centres came from the utilization of forms for the survey of urban vulnerability, according to an approach set up in Emilia Romagna in 1999. This approach provided evaluations for homogeneous zones of historical fabric through a synthetic vulnerability index. The analyses highlighted that the "structural aggregate" was the fundamental element of the vulnerability analysis.

The following activities were carried out, in cooperation with the MiBAC, aimed at the evaluation of the risk of the cultural heritage through the GIS of the DPC:

- database "Atlas of the historical centres exposed to seismic risk" (GIS including the list of about 23000 historical centres provided by the ICCD-MiBAC, integrated by the method developed in cooperation with UNIROMA3 for Central Italy, Eastern Sicily and Reggio Calabria);
- model for the analysis of seismic risk in terms of "cultural loss" of the historical centres, on the basis of the DPC GIS and the "Atlas of the historical centres exposed to seismic risk";
- CSRS WEB form "Historical centres – Seismic risk", with corresponding software, developed integrating the "LSU-Parks form for historical centres" and the historical analysis UNIROMA3/MiBAC-De Rosa, for the extension of the database "Atlas of the historical centres exposed to seismic risk" to the Italian territory;
- institutional network Region-Province-Municipality (Metadato form).

The results of the research of UNIGE within the framework of the GNDT Project "Western Liguria" highlighted that the application of vulnerability models to historical centres shall account for a number of factors which are not considered for ordinary buildings: the aggregate context of the buildings, the anti-seismic elements, the local rules of the art which guide the construction and the possible transformations and superfetations over the building lifetime. These factor had been identified for Western Liguria and their influence on the seismic response quantified in terms of behaviour variations, compared to the mean behaviour, within the framework of both mechanic and macroseismic methods, for each of the considered building types.

An extensive investigation carried out by POLIMI on four historical centres (Montesanto, Roccanolfi, Campi and Castelluccio) within the framework of a GNDT 2000 Project yielded the collapse mechanisms and the typical damage patterns of multiple-leaf masonry structures when the lack of maintenance and/or prevention weakens the response to seismic actions.
A database was developed within the frame of the GNDT project and it is now available on the POLIMI server. Very soon it will contain also the results of the investigation carried out in Sulmona together with CNR-ITC. Collapse mechanisms and damages were also surveyed on reinforced buildings, verifying that some types of interventions were mainly aimed at seismic protection than at seismic improvement, also because of calculation models not suitable to the structures and the masonry studied. An abacus was set up of the collapse mechanisms, a database was developed (Grasso *et al.*, 2000), and verifications through macroelements were carried out (Valluzzi *et al.*, 2001). Investigations were also carried out on techniques compatible with structural and masonry types: injections of hydraulic grouts (Binda, 2002; Binda *et al.*, 1994; Binda *et al.*, 1997), confinement through reinforced repointing (Binda *et al.*, 2001; Modena *et al.*, 2001), deep repointing (Baronio *et al.*, 2001).

2.5 Monuments

The data on vulnerability and damage to churches collected by UNIGE after the Umbria 1997 earthquake allowed to define a macroseismic vulnerability model which was re-defined in the framework of the GNDT-SAVE Project for three possible knowledge levels, from the check list to the in situ survey through a specific form. Moreover, a form was set up for the survey of vulnerability and damage, and a mechanic model, with reference to the macroelements of the churches, based on the capacity curves of the most plausible collapse mechanisms.
Within the framework of the same project, UNIBAS collected over 600 forms on the monuments of Basilicata, damaged by the Irpinia-Basilicata 1980 earthquake.
The analysis firstly focused to the churches. The elaborations consisted of statistical evaluations, based on the Damage Probability Matrices (DPM's) method. For each MCS intensity, the distribution was determined of damage in five levels, and the frequency distribution was modelled according to a binomial distribution.
A first comparison was carried out with the DPM's of the churches of Umbria and Marche, determined after the earthquakes of 1997, observing similar behaviours.
Subsequently, the database was enlarged with the data collected within the framework of a project aimed at the vulnerability evaluation of the churches, based on the data of (Ministero per i Beni Culturali ed Ambientali, 1994). The new DPM's presented slight disagreements, more pronounced for the MCS intensities VII and IX. A further comparison was carried out considering the damage data of ordinary buildings in vulnerability class A in 41 municipalities stricken by the 1980 earthquake (Braga *et al.*, 1982), both in terms of mean damage and DPM's, highlighting a similar vulnerability for the MCS intensities V, VI and VII, whilst for the intensity VIII the churches showed higher vulnerability.
Starting from the cooperation with the MiBAC, the DPC developed databases and models for the evaluation of risk of the cultural heritage through the DPC GIS, and in particular:
• georeferentiation of monuments, in cooperation with the ICR and inserted into the WEB form CSRS;
• USSN-CEI-ICCD L0 form for churches.

2.6 Emergency planning and management

Damage scenarios provide a territorial picture of the stricken area, with important information on the element at risk (population, building stock, transportation network, lifelines, etc.) and the corresponding expected losses, with obvious implications on the Civil Protection

activities, related to emergency planning and management. Tools are available for the evaluation of the impact of one or more events, which allow to establish the response in terms of human resources and equipment.
At the starting of the project, several studies (e.g. Barbat *et al.*, 1996; Esteva, 1997; Fah *et al.*, 2001), and research projects, both national (Catania Project, Potenza Project, GNDT-SAVE) and international (RISK-UE, ENSeRVES, RADIUS), dealt with the issue of seismic scenarios.
In the USA, the Federal Emergency Management Agency (FEMA) and the National Institute of Building Standards (NIBS) provided the software HAZUS (FEMA-NIBS, 1999) for the estimation of losses at the regional scale consequent to different disasters, and operating in GIS (Whitman *et al.*, 1997).
Among the Italian projects, particularly prominent was the Catania Project (Faccioli, 1999), funded by the DPC and developed within the framework of GNDT. The main objective of the project was that to attain to damage scenarios for the city of Catania. The hazard evaluations and the maps of damage to buildings were two prominent results of the project.
The RISK-UE Project (An advanced approach to earthquake risk scenarios with application to different European towns), essentially aimed at adapting the HAZUS software to the European situations accounting for the different building types, funded in 2001 by the European Community, concluded its activities in 2004 (www.risk-ue.net). Several research centres in seven European countries (France, Italy, Romania, Spain, Greece, FYROM and Bulgaria) took part in the project with the objective to develop a modular method of general validity for the determination of seismic scenarios for the European towns. After evaluating the peculiar features of European buildings, the method was applied to seven towns. Moreover, an example of Risk Management Plan had been set up, accounting for the rescue agencies, the civil protection and other public authorities engaged in the mitigation of seismic risk.
The RADIUS Project (Risk Assessment Tools for Diagnosis of Urban Areas against Seismic Disasters) was launched in 1996 by the Secretariat of the International Decade for Natural Disaster Reduction (IDNDR 1990-2000), with the technical and financial support of the Japanese Government. It was aimed at promoting worldwide activities for the reduction of the effects of seismic events in urban areas, with particular reference to emerging countries. Nine cities were selected and examined as case studies to develop damage scenarios and intervention programmes aimed at reducing the seismic risk, with the participation of the political, administrative, social and scientific communities, and the media. Based on the experience of the case studies, operative tools were developed for the estimation of seismic damage, and a comparative study was carried out to evaluate the seismic risk at the urban scale worldwide.
The ENSeRVES Project (European Network on Seismic Risk, Vulnerability and Earthquake Scenarios) was funded in 1997 by the European Community within the framework of the INCO-Copernicus Programme and promoted by the European Association of Earthquake Engineering. The ENSeRVES Project engaged researchers of different disciplines (seismologists, geologists, engineers, planners, etc.) involving 11 important institutions in the field of seismology and earthquake engineering coming from 10 European countries. The main objectives of the ENSeRVES Project were: (i) comparing the procedures for the estimation of seismic hazard, vulnerability and damage adopted by the different countries; (ii) improving and extending the procedures for the evaluation of the vulnerability of the building stock through the integration of different approaches; (iii) developing general shared procedures for vulnerability evaluation; (iv) comparing and developing methods for the

evaluation of seismic scenarios; (v) examining problems of seismic protection at the urban scale (Dolce *et al.*, 2002, 2003a).

The DPC is equipped with methods, procedures and software for the evaluation of post-earthquake damage scenarios for the national territory, with different resolution scales and different refinement levels. The following software were available at the starting of the project.

- SIGE: given the severity and the localization of the event, provides a territorial picture of the stricken area, calculates the losses (expected number of buildings collapsed, unusable, damaged, casualties, homeless) with resolution at the municipality scale within 50 km from the epicentre.
- QUATER: software for the generation of query, report, analysis and visualization of SIGE results.
- FACES: extension of SIGE; it contains widening on source mechanism, with particular attention to anisotropy factor and directivity; it allows analysis with resolution at the scale of census tract.
- ESPAS: extension of SIGE; it represents an evolution of the foregoing codes and accounts for the intrinsic randomness of the phenomenon through the uncertainties of the model variables. The system allows the dynamic updating of the estimations, based on the elaboration of expert data and/or data which become available during the emergency. Beyond the estimation of losses to dwellings, the software can perform evaluations of losses to strategic buildings, historical centres and monuments, where the vulnerability of these elements at risk is available.
- SCETER: software for the definition of the reference events for emergency plans at the inter-municipality scale; the code provides the estimation of the expected losses as function of the return period, accounting for the seismogenic structure. The results of the software can be used within an expert evaluation, which can define different impact thresholds with increasing gravity and/or return period, corresponding to different activation levels of the emergency plan.
- SCECOM: software for planning at the municipality scale: it estimates the losses (collapses, casualties) of the municipality, given the exceedance probability of the event. The difference between SCETER and SCECOM lies the different definition of the reference events, coming from their different planning objectives. The former assumes as reference event that which maximizes the losses over a super-municipal territory; the latter provides the losses corresponding the three significant events for the municipality.

Immediately after a seismic event, the fundamental activity for the resettlement of population in dwellings is the survey of usability of the constructions. It requires a high degree of standardization of the procedures for the management of surveys and inspections. Within the Augustus method, the inventory is carried out of damage to persons and property, and the usability check of buildings and other structures is made at the territorial scale. This objective is pursued in different phases, using a method of survey and evaluation, in deep cooperation with the Region and the Local Bodies.

These activities were reported in the "Manual of the 1st Level Form for Damage Survey, First Intervention and Usability for Ordinary Buildings in the Post-Seismic Emergency" developed by DPC and GNDT with reference to the AeDES 2000 form, used and tested in several Italian seismic events, as well as in the "Manual for the Management of the Technical Activity of the COM", implemented in the SET software.

The DPC promoted and organized a set of initiatives aimed at the formation of technical personnel, belonging to public Administrations and to the private sector. Scientific and technical support was provided to several Regions, with the objective to release shared and

standardized criteria of evaluation. Cooperation agreements were also promoted with the professional associations in the field of technical management of seismic emergency, with the aim to have at disposal a significant number of technical personnel able to accomplish usability evaluations in the post-event, and finally to arrange a national list of “technical personnel for usability evaluations”.
The method used in Italy for the evaluation of usability and damage was also illustrated in California and Japan. The usability form is reported in some Californian web sites and in a volume on the Japanese standards.
As regards the definition of updated standards for the collection of data in the emergency phase, a specific multimedial software, entitled MEDEA was produced (Manual of Training on Damage and Usability for ordinary buildings) and an analysis on damage mechanisms was tested in cooperation with UNINA, to be used as integration of traditional forms which represent the damage only in terms of extension and severity.
The “Structural interventions and emergency works” Office published the “Tools for the preliminary quantification of the damage and technical documents for very urgent works in consequence of significant events”, in relation to the consequences of disasters as landslides and meteorological events. The main objective was the development of a survey system as simple as possible, but at the same time standardized and sufficiently exhaustive for a preliminary and extensive quantitative evaluation of damage. Forms were set up, to be filled by the surveyors, and a software was developed for data processing. In this framework the ME.RI.D.I.A.NA. Programme (Method of Damage Survey in Natural Environment) started. A glossary was developed. At the starting of the project, a further development of the whole system was under progress in the framework of the European Programme 2004-2006 INTERREG III “DAMAGE”.
In the first phase of the post-earthquake emergency, one of the most delicate problems regards the provisional works on strongly damaged structures (demolitions, props, tie-rods, barriers, etc.) (SSN-GNDT 1996). These interventions are often characterized by great urgency and are necessary to prevent the progression of damage, which can be caused by the aftershocks, and/or to protect people safety and restart the normal socio-economic activities.
The most complex issues of these interventions regard the choice of the most suitable type of intervention, its correct execution and the optimization of costs. The last issue, in particular, has a considerable importance, because of the number of interventions after a significant seismic event, which will be generally removed in the subsequent phase of definitive repair. The timber props, once removed, cannot be reused, whilst the tubular elements are usually rent, and, considering the technical and administrative times to accomplish final repairs, weigh significantly upon the total cost. The choice of the most suitable type of intervention is often made with reference to the usual interventions for vertical loads, without taking into account the possible dynamic phenomena caused by the aftershocks, nor the interferences with the road network in historical centres which are consequent to the adoption of some encumbering types of intervention. In general, the solutions adopted are often inefficient, anti-economic or excessive, and in some cases unsafe.
The state-of-the-art (Di Pasquale and Dolce, 1999; Falsini *et al.*, 1994; Mastrodicasa, 1993) and of the practice was the starting point for a scientific and technical study on the subject. The National Seismic Survey published the volume “The provisional works in the seismic emergency” (Bellizzi, 2000), which represents a preliminary study for the design of provisional interventions. A particular emphasis was devoted to cultural heritage, reproposing the model of intervention of the Ballardini-Doglioni Document, approved in 1996 by the National Committee for the Prevention of the Seismic Risk to Cultural Heritage. From the analysis of the state-of-the-art, instructions were drawn for the design of these interventions.

In 2001, the Project OPUS (Urgent Provisional Post-Earthquake Interventions) was started by UNIBAS, funded by the National Seismic Survey, for studies and researches on the subject. Project OPUS studied the behaviour of the post-earthquake provisional interventions in a systematic way, with the aim to provide design criteria which could be easily applied. Starting from a set of examples, and qualitative and quantitative critical analyses, the study attained to a functional classification of the provisional interventions, and to improvements of their performance under seismic actions. Experimental investigations were carried out, aiming at checking the validity of the techniques examined through tests on materials (steel rods and strips, polyester belts), structural elements reinforced through rings (masonry pillars), real-scale props and tie-rods. For these systems, pseudo-dynamic test simulating the earthquake were carried out. These activities allowed to determine the safety levels which can be reached, to compare the efficiency of different interventions, to validate simplified calculation methods and to improve their performance.

Another issue in the post-earthquake phase is that of emergency housing. Over the last two decades, the issue of emergency housing became more and more important within the framework of the activities of civil protection, overcoming the traditional limits of the emergency and considering new demands, such as that of house mobility related to the increasing phenomena of extra-European immigration. However, the Umbria-Marche 1997 earthquake highlighted a number of problems related to a correct emergency management. The issue is extremely complex, because involves questions of organization and management, and technical aspects related to performance. Firstly, it should be observed that provisional dwellings have no consolidated tradition, and many of the problems only arose in the last century. Among the different constructive techniques, metal structures, and in particular cold-formed systems, found widespread applications. Cold-formed systems are very light constructive systems, thanks to shape optimization, and which can be prefabricated, thanks to the use of innovative connection techniques. Despite technological progress, containers are typically handled through rudimental devices, which are not properly designed. The possibility to overcome some of the intrinsic limits of these constructive types – e.g. the non adaptability to different soil conditions and the low comfort level related to the direct contact between prefabricated modules and soil – represents one of the open problems.

Despite many studies on architectonic issues (Bologna and Terpolilli, 2005; Bologna, 2007; Falasca, 2000; Mango and Guida, 1988; Cecere *et al.*, 1984; Donato *et al.*, 1983; Pedrotti, 1998; Latina, 1986; Future Systems, 1990; Bohtlingk, 1998; Della Corte *et al.*, 2003), at the starting of the project, the literature lacked specific studies on the structural performances of these constructions.

2.7 Development of databases and GIS

The activities of data collection, at the national scale, with the aim of evaluating the seismic risk in Italy, began in 1992-3.

The sources of territorial data are national agencies, public or not, which validate the data: CNR, ENEL, ISPESL, Ministry of Public Works, Ministry of Agriculture and Forests, Ministry of Health, Ministry of Environment, National Technical Surveys. Moreover, databases produced by the National Seismic Survey are available.

The cartographic database (at two scales: 25,000 and 250,000) contains 80 vector databases arc/info (for a total of over 500 coverages), georeferenced in the UTM system, fuse 32, and meeting the following requisites:

• they describe the whole national territory with the same level of detail;
• they are certified, i.e. come from an official source which provides their validation;
• each database has the data necessary to be integrated by other databases;

• each database has its metadatation.
These databases are integrated by the raster national coverage produced by the IGM at different scales, and the national orthophotographic georereferenced coverage.
The Geographical Information System (GIS) is the ideal environment for multi-disciplinary studies, for which it is essential to cross the data and verify, from many points of view, specific territorial phenomena.
The GIS represent a strategic factor in the development of analyses, studies and procedures for the evaluation of urban and territorial risk. These researches involves different competences, and requires a system of data synthesis coming from different sources, with the objective to determine and visualize the risk probabilities of the territory.
The GIS set up by UNISANNIO for the Traiano Project has a multi-level architecture. In particular, it has topological networks with different size (from 5 km to 50 m) which allow the interface and the comparison of different data and the elaboration of damage scenarios.
A database was developed within the frame of the GNDT project and it is now available on the POLIMI server. Very soon it will contain also the results of the investigation carried out in Sulmona together with CNR-ITC.

3 RESEARCH STRUCTURE

3.1 Ordinary buildings

UNIGE developed a macroseismic methodology for the analysis of the seismic risk to constructions derived from the EMS-98. The method was developed and specified with reference to databases, on the inventory of the buildings, of different levels. Two different procedures were defined, in particular for a regional and a urban scale, and some case-studies were analyzed (Abruzzi Region and Sulmona (AQ) for the regional and urban scale, respectively). The activity of this RU on this subject and the specific applications has been performed with the cooperation of CNR-ITC.
UNINA-DIST updated the maps of seismic risk at the national scale on the basis of the ISTAT 2001 Census on population and buildings. The RU enlarged the database on observed vulnerability with the insertion of about 20,000 buildings. Finally, the RU developed a new procedure for the interpretation of ISTAT “poor” data, aimed at the estimation of the distribution of typological vulnerability classes, and at the calibration through comparisons with the surveyed vulnerability data.
UNIPV analyzed the data on observed damage to buildings, collected in Italy during the last 30 years, to determine vulnerability curves of the building stock (Colombi *et al.*, 2008).
The post-earthquake damage surveys from the most important earthquakes that have occurred in Italy have been utilised: Irpinia 1980, Eastern Sicily 1990, Umbria-Marche 1997, Umbria 1998, Pollino 1998 and Molise 2002. The relationships proposed by (Meroni *et al.*, 2000) were employed to estimate, from the data present in the ISTAT 2001 Census survey forms, the number of undamaged buildings classified as a function of the period of construction, number of storeys and the vertical structural type within each municipality.
A common data classification scheme, which could be applied to all of the databases, has been identified, and includes: ISTAT code, number of storeys above ground, building typology (based on the vertical structure) and damage to the vertical structure. The buildings have been classified as reinforced concrete (RC), masonry, and buildings with both RC and bearing masonry walls (referred to as mixed).
For each aforementioned building class, the number of buildings with each level of damage was related to the intensity of ground motion to which they were subjected to produce

fragility curves. These curves have been compared with the curves calculated according to mechanical methods: DBELA (Crowley *et al.*, 2004) and SP-BELA (Borzi *et al.*, 2008a,b).
The objective of the research activity of CNR-ITC is to develop tools for the survey and analysis for typological characterization and vulnerability and damage evaluations for masonry and RC buildings, to develop the associated databases, to produce risk maps and scenarios (Beolchini *et al.*, 2007; Cifani *et al.*, 2007a,b,c; Di Grezia *et al.*, 2007; Mannella *et al.*, 2008; Lemme *et al.*, 2008; Parodi *et al.*, 2008).
A revision has been carried out of the operative tools for the vulnerability survey on ordinary buildings, with the aim to define unified forms, which can be used with different methods of different levels. The 2nd level forms, used in previous studies, have been updated, as well as the "Level 1 Form for damage survey, expeditious intervention and usability in the post-seismic emergency for ordinary buildings (AeDES), attaining to a proposal of an integrated system of data collection which allows to formulate coherent multi-level evaluation models.
A multi-level method has been developed and tested, aimed at the survey of vulnerability of ordinary buildings, at the evaluation of seismic risk and at the calculation of damage scenarios at the territorial and urban scales. This study has been carried out in cooperation with UNIGE, with the objective to attain to an organic development of tools and procedures, coherent with the definitions of vulnerability and damage of the EMS-98. The method, developed by UNIGE on the basis of the "macroseismic" approach to operate on several knowledge levels, has been set up and tested through:

a) an application at the territorial scale for Abruzzi Region;
b) an application at the urban scale for the case study of Sulmona (AQ).

The method has been developed at a first level, defined as Level 1, substantially based on the ISTAT 2001 data, integrated by additional information which allows a more realistic interpretation in a given area. At Level 1, the vulnerability analysis is made at the scale of census tract.
Subsequently, Level 2 has been developed, based on an updated version of the AeDES form, and coherent with Level 1. At Level 2, the vulnerability analysis is made at the scale of individual buildings. The availability of survey data, carried out previously with a similar expeditious form, but different from the modified AeDES form (see Task 6 – Emergency planning and management), suggested the opportunity of a specific application highlighting that the macroseismic approach is potentially applicable to a detailed description of the building stock, as in the analysis of a homogeneous historical centre.
A second objective of the research activity of CNR-ITC is to attain to a database on vulnerability, damage, usability and interventions on the buildings of Regione Marche stricken by the 1997 earthquake, starting from the database "Tellus".
The database contains the data of about 8,000 intervention designs, including the corresponding variants and STAP (Design Technical Form). Preliminary elaborations, aimed at eliminating the duplicates and the buildings in aggregates, singled out 5218 buildings. Starting from the analysis of data relevant to vertical structures, 4833 masonry buildings were singled out. The remaining buildings can be classified as buildings in RC, steel or mixed structure. Analyses were carried out on masonry buildings, aimed at determining the constructive and dimensional characteristics for typological classification. The data allow vulnerability evaluations, also of higher level. In particular, for each building, the possibility has been studied to determine the vulnerability classes associated to the data of the 2nd level GNDT form. The information on observed damage is reported into the STAP through 6 levels, relevant to some types of structural damage (damage to masonry panels, separation cracks, damage to horizontal structures, subsidings, poundings, collapses, other). This requires the definition of conversion models to the scale conventionally used for vulnerability

analysis. A first conversion has been carried out in order to obtain the damage index according to the definition of GNDT form, and some correlations between the data have been highlighted.
The third objective consists of developing technical forms for the identification of damage and collapse mechanisms, their calculation models and intervention techniques. The models are updated at the Technical Regulations for Constructions (NTC 2008).
UNIPD developed and validated an updated version of the procedures Vulnus and c-Sisma, according to the performance requirements of Ordinance 3431/2005, for the systematic evaluation of vulnerability of existing masonry buildings, based on the application of mechanical models able to describe at the global and local levels the mechanisms related to the loss of equilibrium of structural macroelements.
In the historical centre of Sulmona, a survey based on forms has been carried out in cooperation with other RU, in particular CNR-ITC. After having defined the representative types of historical residential buildings (buildings in single and double row, court buildings, blocks) a sample of aggregates has been selected which have been surveyed in detail; for some units the masonry has been completely identified, also through non destructive and moderately destructive tests. The vulnerability evaluation has been carried out through the procedures Vulnus and c-Sisma, also with the aim to validate and calibrate the results of expeditious analyses of other RU.
On the basis of the available surveys for some historical centres of Umbria stricken by the 1997 earthquake, and characterized by different typological building distributions, UNIPD, in cooperation with POLIMI, completed the analyses of the seismic vulnerability and damage of simple building types (isolated buildings and rows) and more complex types (aggregates). The buildings analyzed were object in the past of interventions (replacements of timber floors with RC floors, insertion of RC string-courses, insertion of tie-rods, etc.).
The research of UNIBAS is aimed at developing ad hoc methods for the vulnerability evaluation of RC buildings, based on fragility curves, in a sufficiently accurate way and, at the same time, not too much expensive.
The seismic performances of RC buildings have been evaluated through non linear dynamic analyses using a purposely set-up procedure initially proposed in (Masi, 2003).
Structures widely present in the Italian building stock and representative of low- mid- and high-rise building types designed only to vertical loads, have been considered (Masi *et al.*, 2007a,b; Masi *et al.*, 2009). They were carefully designed on the basis of the codes in force, of the available handbooks and of the current practice of the period (simulated design). Investigations on the Italian construction standards before and after the 1971 have been undertaken in order to design buildings than can be considered representative of the “as built” in Italy.
Very often, non seismically designed buildings present proper frames only in one direction (typically the longitudinal, longer building direction). In the other “weak” direction, the frame effect is possible if exterior frames are present, while internally is guaranteed only by the contribution of the floor slabs spanning between the columns (No Beam, NB).
As for the “weak” direction is concerned, the typical characteristics of the Italian building stock show that, because of the presence of masonry infill walls, it is very common to find edge beams spanning between the columns of the two exterior frames. The edge beams can have different stiffness factor since both conditions of beams within the floor slab thickness (Flexible Beam, FB) and emergent beams (Rigid Beam, RB) are very common in the construction standards. The cases of buildings having large (Type 1) and small (Type 2) plan area, that is made up of 4 and 6 frames, have been also analysed.

Further, by recognizing the major role of masonry infills in the seismic behaviour of buildings without earthquake resistant design, buildings without infills (BF type), regularly infilled (IF type), and with pilotis (PF type) have been considered.
A mechanical approach has been used to obtain the intensity vs. damage relationship for each structural type. A macro-modeling based on lumped plasticity has been adopted using the computer program IDARC-2D. Non linear and degrading behaviour, typical of the structures under consideration when subjected to high seismic loads, has been evaluated using the three parameter hysteretic Park model.
For vulnerability assessment, a type recognition and classification is required to assign the structural type to each individual building. The fundamental information is the period of construction of each building as, in the procedure, vulnerability is defined on models detailed according to a simulated design based on the codes in force, the available handbooks and the current practice at the time of construction.
Results obtained applying the procedure have been compared to those ones provided by other approaches, such as the SP-BELA method (Borzi *et al.*, 2007) showing reasonably well agreement (Borzi *et al.*, 2008). Further, it is worth noting a wide comparison of the results from several approaches adopted in Italy for vulnerability estimations carried out in the framework of the USGS-PAGER Project devoted to rapidly assess the consequences of severe earthquakes in the world. Beyond UNIBAS, several other Italian RUs participated to the PAGER Project: DPC, UNIGE, UNINA, UNIPD, UNIPV. In (Goretti *et al.*, 2008) the contribution of the Italian RUs to the PAGER project is presented, providing an insight into the used methodologies and a comparison of the results.
The objectives of the research activity of UNINA-DIST are:

a) improving and testing the survey form for RC buildings;
b) attaining to the evaluation of seismic vulnerability through the method of class-scale risk assessment.

The existing survey forms for RC buildings – originally intended for the sole survey of geometric and mechanic parameters of buildings directly related to their seismic behaviour – were improved through the completion of the section relevant to structural damage, which was also equipped with a series of images representing different types of damages for reinforced concrete elements and also for non structural parts (e.g. infills).
The forms were tested in Arenella district in Naples. The building stock in this area (approximately 500 aggregates/blocks corresponding to more than 1500 buildings) is constituted mainly of mid- and high-rise RC moment resisting frames built over the twenty years following the 2nd World War, before seismic regulations were introduced for the city of Naples (pre-code buildings). Sidewalk pre-screening allowed to identify single buildings from aggregates (more than 1400 buildings were identified) and the relative construction typology (RC, masonry, other). Cartographic and exterior surveys, extended to all the identified RC buildings (more than 850), were complementary in the acquisition of global dimensional and morphologic data, as well as of aspects related to possible deficiencies (irregular infill distribution, soft storey, etc.). Internal inspection of a limited number of randomly extracted buildings, together with the consultation of a number of architectural drawings, allowed to gather more detailed information, such as stair type, slab way and thickness, resisting element dimensions, structural mesh organization etc.
The latter objective was pursued through the implementation of a method for risk evaluation at class-scale (Iervolino *et al.*, 2007). Based on this method, a risk analysis at territorial scale was carried out using the data collected in the Arenella district.

3.2 Public and strategic buildings

The objectives of the research activity of CNR-ITC are:

a) development and analysis of a database on the seismic assessment on schools of Regione Molise;
b) development of a procedure for the computerization of the results of the seismic assessment of public buildings according to the Ordinance 3274/2003.

As regards the former objective (Dolce *et al.*, 2007a,b; Martinelli *et al.*, 2008), the available information from the designs was included in a general database, according to the Guidelines of CNR-ITC. The elaborations are relevant to the typological and vulnerability characteristics. For masonry buildings, the collapse PGA calculated by the designers were compared with those calculated according to the evaluation method of the SAVE Project, which uses data of GNDT 2nd level.

As regards the latter objective, a procedure was developed, in coordination with the DPC, for the filing and transfer to the DPC itself of the data of the "Synthetic form on seismic verification at level I or level II of strategic buildings to the purposes of Civil Protection, or prominent in the case of seismic event". The procedure was initially developed with reference to the OPCM 3431. The approval of the Technical Regulations for Constructions in January 2008, and their adoption by many Regions, made necessary its updating, integrating the possibility of computerization and data management with the two regulations (OPCM 3431 or NTC).

In strategic buildings, RC stair-lift cores, of high stiffness and strength, are often present. In the case where there is a single core in eccentric position, torsion arises, which is equilibrated by the combined contributions of moment resisting frames and core. When many RC cores are present, even if in eccentric and/or asymmetric position, the response to torsion improves. In order to account for such effects, RC stair-lift cores have been implemented by UNIBAS in the procedure VC for the evaluation of vulnerability and risk of RC buildings.

3.3 Infrastructures

The main objective of the UNITN RU is to design and develop a technical system able to perform real-time evaluations of seismic risk and reliability of an infrastructure network (Zonta *et al.*, 2006a,b,c; Zonta and Pozzi, 2007; Pozzi *et al.*, 2007; Zonta *et al.*, 2007; Zonta *et al.*, 2008a,b,c; Pozzi *et al.*, 2008; Zonta *et al.*, 2009).

This study includes three sub-objectives:

A. Improvement of an existing Bridge Management System (BMS), in order to include seismic vulnerability information at the network level.
B. Design and production of smart elements, able to provide on-line information on the structural response of monitored bridges.
C. Development of Bayesian algorithms for real-time condition and damage assessment of single structures and of a whole infrastructure network

As regards the first objective, correlation has been investigated between bridge typology and condition state to seismic vulnerability, in order to assign a prior distribution for seismic risk to each element in the network. The BMS has been accordingly integrated by a vulnerability model consistent with the HAZUS model.

As regards the development of smart elements, a technology has been developed and validated in the laboratory, based on fiber optic strain interferometric sensor suitable to be embedded into Prestressed Reinforced Concrete elements.

A Bayesian algorithm has been developed, in order to interpret the large amount of data recorded by the system, which allows to take into account the *a priori* information, including material properties, environmental conditions and functionality of sensors.

3.4 *Urban systems and historical centres*

The objective of the task – coordinated by the CNR-ITC and in which DPC, INGV-ROMA, POLIMI, UNIBAS, UNIGE, UNINA-DIST, UNIPD and UNISANNIO took part – is the definition of a model, based on the analogy with neural networks, for the analysis of the response of urban systems under a given seismic action (Cherubini *et al.*, 2008; Working Group of Task 4, coordinated by Cherubini A., 2008).

The neural model is defined by:

- systems which can be activated by a given simulated seismic event, representing an individual seismic sequence; a sequence of relations is then built, which yields a "neural attractor";
- the seismic action (main shock and aftershocks) is represented in a time-duration axis in logarithmic scale;
- for each system, the initial response capacity C_{0j} is defined (normalized value) through a response threshold;
- the systems are then activated for an individual sequence with evolution modes (modifications of the capacity of the same system) or correlation (modifications of the capacity of a system because of other systems);
- reaching of the reduced capacities of the systems C_{0j} corresponding to the neural attractor;
- capacity restoring of some systems because of:
  - external interventions increasing capacity (e.g. Civil Protection interventions);
  - internal resilience of the urban system.

The model has been developed at two levels:

- evaluating in a synthetic way (from national databases) the total capacity loss for a high number of urban centres, determining the worst risk conditions and deciding priorities of further investigations (level 0);
- evaluating, through detail parameters, the total capacity loss of an urban centre, based on expeditious surveys and investigations (level 1).

The Task has been organized in Working Groups, with the following objectives:

- WG1 (CNR-ITC): attaining to the evaluation of level 0", on the basis of ANCITEL data, considering also the urban-typological characterization of the centres through typical models, and implementing the data relevant to Cultural Heritage;
- WG2 (CNR-ITC, POLIMI, UNIPD): survey, in situ investigations and systematization of collected data, data interpretation through the concept of "minimum unit", not smaller than the census tract, and "inductive" approach for the investigated sub-systems;
- WG3 (UNIPD): simplified capacitive analyses using "poor" data or data from vulnerability forms, using data provided by the WG2;
- WG4 (DPC): method for the analysis of "value" and "social" components in the minimum unit.

CNR-ITC proposed the municipality of Sulmona as sample centre for testing the methods and the evaluation procedures of vulnerability, risk and emergency management. Sulmona is a town of small-medium size of inland Abruzzi, with a significant historical centre. Thanks to previous investigations, carried out in collaboration with the Abruzzi Region, CNR-ITC has a large knowledge base of the buildings in the historical centre, implemented in GIS.

The exposure of the historical centre, in terms of resident population, has been acquired and implemented in the GIS, as well as an investigation on recent buildings, outside the historical centre.

The set of available information, organized in databases and GIS, has been placed at disposal of all the RU involved in the Task.
The research activity on the geologic, geomorphologic, geotechnical and geophysical databases (INGV-ROMA) focused on survey forms, on the simplified parametrization of data of the studied areas, on the integration of data within the framework of the analysis of urban systems according to the approach of neural network. The aim is that to attain to an evaluation of seismic risk based on integrated analyses and, in the post-event, to more detailed analyses on observed damage in relation to the possible activation of local effects.
INGV-ROMA contributed to the implementation of engineering database for the analysis of urban systems (level 0), providing data relevant to base and local hazard available in the literature, for all the 8101 Italian municipalities. Moreover, the geologic data for the historical centre of Sulmona have been elaborated, with reference to the analyses at level 1.
POLIMI investigated the building types and the constructive elements in the historical centre of Sulmona, attaining to a typological-morphological classification of the buildings, and to an inventory of the constructive elements (Binda *et al.*, 2009a,b,c).
The historical information on the town of Sulmona has been acquired from the literature, vintage photographs, ancient and modern cartography, documents relevant to territory planning and transformation.
The lack of reliable and sufficiently extended documentation on the ground floor of the individual residential cells of the town of Sulmona, steered the investigation on the classification of the types of the aggregates, rather than of the types of dwellings.
UNIPD, in collaboration with POLIMI and CNR-ITC, and within the framework of the WG2 (survey and data collection, re-elaboration of the forms of the urban centres, networks and infrastructures for levels I and II) organized the operations and set up the tools for the survey of the historical centre of Sulmona: these operations allowed the collection and the systematization of the typological data of the buildings in the historical centre, and the information on the materials, interventions and damage mechanisms for subsequent evaluations of vulnerability.
Within the framework of WG3 (collection of synthetic models of structural behaviour in terms of capacity loss of buildings and infrastructures, in masonry or other types), a study has been carried out for the definition, through detail parameters, of the seismic capacity, and the functions describing the capacity loss of masonry buildings, using data from expeditious investigations and surveys, coherently with level I. The capacity of a generic building accounts for the possible physical damage; the capacity losses associated to "evolution" have been then defined, in relation to the damage caused by a seismic event.
The data on the historical centre of Sulmona, provided by the WG2, have been used to calibrate the results of the new analyses in terms of capacity loss.
UNINA-DIST determined fragility curves for existing RC building classes in the historical centre of Sulmona. These curves were used to determine the capacity loss for different limit states for varying scenarios.
The historical centre of Sulmona town consists mainly of masonry buildings. Nevertheless, there are 60 RC buildings that were studied at the Level 1 of the task. The information concerning the buildings derives from survey forms compiled by CNR-ITC. The data utilised for classification are the number of stories, the type of structural system and the presence or not of a soft storey (caused by the absence of infills at the first level). All the RC buildings, being constructed after the first classification of Sulmona as a seismic town, were designed according to "old generation" seismic codes, not containing indications on capacity design. Considering the aforementioned information, and accounting for dimensional data retrieved by available cartography, the buildings of an approximately regular shape (the sole studied)

were subdivided according to: number of stories, infills typology (strong or weak), dimensional intervals of the base plant. A separate treatment was performed for buildings with irregular distribution of infills in elevation (those with bare first storey): these are few (6) buildings, 4 of which built consecutively along the same road, with porch at the first storey and a large first inter-storey height, all having weak infills at the upper stories. For each of the buildings examined, to be considered representative of a sub-class, fragility analysis was performed. Finally, the capacity loss corresponding to earthquakes with return period of 50 and 475 years was evaluated.

3.5 Monuments

The objectives of the research activity of UNIBAS, partially in common with Line 1, are:

a) the evaluation of vulnerability and expected damage on a sample of monuments of Basilicata;
b) the evaluation, on a reduced sample, of the damage mechanisms, possibly caused by recent interventions, and the development of intervention protocols.

As regards the former objective, the vulnerability, together with the seismic and non-seismic damage, have been surveyed for one hundred monuments, representative of the Cultural Heritage of Basilicata Region, subdivided into 86 churches, 9 palaces and 6 castles (Liberatore *et al.*, 2009a,b).

The information on the interventions adopted in consequence of the Irpinia-Basilicata 1980 earthquake derives from an investigation on the designs of the interventions of seismic improvement at the Superintendence for the Architectural Heritage and Landscape of Basilicata. A database has been developed, collecting the information from the in situ investigations and the analysis of the designs.

The latter objective, relevant to damage mechanisms, recent interventions and development of intervention protocols, represents a widening of the foregoing study. A comparative analysis has been carried out on 10 monuments, with the aim to study the structural weaknesses, possibly caused by the interventions adopted in consequence of the Irpinia-Basilicata 1980 earthquake (Liberatore and Speranza, 2009). These interventions were investigated thanks to the wide documentation available at the Superintendence, including designs, photographs of the damage and photographs taken during the works. The interventions were studied in detail, singling out those ones which may have produced detrimental modifications of the structural behaviour.

POLIMI has filled in the forms and carried out the investigation on 10 churches in the Brescia province after the 2004 earthquake and studied the centre of Morgnaga (Binda *et al.*, 2006; Anzani *et al.*, 2007a,b; Binda *et al.*, 2007a; Cardani *et al.*, 2008).

A first research activity of CNR-ITC consisted of data collection on seismic vulnerability of the churches of the historical centre of Sulmona. INGV-ROMA collected also “geological” data on the churches sites using a specific survey form developed in this project.

A second activity, developed taking the opportunity of an European research project INTERREG/CARDS PHARE entitled “RECES modiquss – The network of small historical centres as model of urban quality and sustainable development”, consisted of a research entitled “Vulnerability and risk analysis of a sample of churches of the ‘Baronia di Carapelle’, according to the ‘Guidelines for the evaluation at level 1 and the reduction of the seismic risk of Cultural Heritage’ (G.U. 29 January 2008). Within the framework of this activity, the survey has been carried out of 30 churches in 6 municipalities of the territory of Gran Sasso, near l’Aquila, through a form considering 28 mechanisms.

The research, coordinated by INGV-ROMA, and in which POLIMI, UNIGE and Regione Molise took part, has the objective to investigate the effects of seismic amplification for topographic causes, which may have affected the churches damaged by some of the Italian earthquakes.

The novelty of this study consist of a method of analysis which, starting from the observation of the effects on the structure, and from the survey of the geologic, geomorphologic, geotechnical and geophysical characteristics of the site, compares the observed damage with the expected damage – calculated on the basis of statistically reliable data – attaining to the quantification, also through numerical modelling of the local seismic response, to the influence that site morphology may have exerted on the increment of seismic input and, as a consequence, of the observed damage (Di Capua *et al.*, 2006; Di Capua *et al.*, 2007; Compagnoni *et al.*, 2007).

In consequence of the Molise 2002 earthquake, a survey of the damage and vulnerability of the churches has been carried out, through forms widely tested and acknowledged at the national level (G.U. 7 March 2006). The mean observed damage was compared with the mean expected damage, calculated through vulnerability curves, highlighting that in some cases the damage cannot be completely ascribed to the vulnerability of the structure. In particular, the analysis of site morphology showed that, when the building is located on a ridge, the observed damage is always higher that the expected damage.

With the objective to account for the hazard increase because of topographic effects within the framework of vulnerability analysis, two approaches have been proposed: macroseismic and mechanic. They have been validated through a study of the geologic, geomorphologic, geotechnical and geophysical characteristics of the sites, utilizing a form of new generation (Compagnoni *et al.*, 2009; Di Capua *et al.*, 2009), through modelling of the local seismic response (estimation of amplification factors and of response spectra) and of the seismic response of the macroelements damaged by the reference event (linear and non linear kinematic analyses).

The study considered a sample of 72 churches damaged by the earthquakes of Irpinia-Basilicata (1980), Abruzzi Apennines (1984) and Molise (2002). In particular, 18 churches are located in Basilicata (16 in Potenza Province and 2 in Matera Province), 54 in Molise (6 in Isernia Province and 48 in Campobasso Province). For each church, the form for churches has been filled in, which considers 28 damage mechanisms (Lagomarsino *et al.*, 2005).

At the same time, the survey has been carried out of the geologic, geomorphologic, geotechnical and geophysical characteristics of the sites, in situ at the technical offices and from the literature, through a “geologic” form, developed by INGV-ROMA. Within the framework of the activities of INGV-ROMA, a database has been developed in Microsoft ACCESS, named “SELSE 1.0”, for data computerization of the “geologic” form.

The seismic risk of the churches studied has been calculated according the ‘Guidelines for the evaluation and the reduction of the seismic risk of Cultural Heritage’ (G.U. 29 January 2008).

The sample was then studied with the aim to evaluate the effects of seismic amplification caused by site morphology, for the churches having foundation on the rock. The analysis, carried out for 18 churches located on ridges, with morphological characteristic which satisfy the EC8 parameters (inclination of the slope $\alpha \geq 15°$ and height $H \geq 30$ m) with homogeneous lithologic characteristics, such that the response can be assumed elastic, highlighted that the particular topographic location of the buildings affected their seismic response and damage level (Di Capua *et al.*, 2006; Di Capua *et al.*, 2007). With the aim to define a damage scenario accounting for the effects of seismic amplification caused by site morphology, the macroseismic and the mechanic approaches have been utilized (Compagnoni *et al.*, 2007).

3.6 Emergency planning and management

The problem of temporary housing of the population originated by natural or anthropic disasters represents a test particularly demanding for the civil protection system. Within a long-term programme of expansion of emergency means, the DPC decided to develop a Special Regulation defining the characteristics and the minimum levels of performance required to temporary housings.

The research programme coordinated by UNINA-COSTRARC, and in which UNIFI/UNIBAS took part, on the basis of the requirements of the DPC, had as main objective the development of this Special Regulation for emergency housing.

The Regulation provides the minimum technical requirements, according to the most recent advances in technological and scientific fields, that emergency housing has to guarantee in order to satisfy specific demands, while the description of methodologies for evaluating the system performance should be referred to the design codes. According to this, the Special Regulation should define the minimum requirements that living units have to guarantee over a short-to-medium period. The evaluation of performance is entrusted to the manufacturer, who should certify the requirements satisfactory by adequate certifications.

The Regulation, in very general terms, aims at defining the characteristics of the modular prefabricated living units for civil protection, overcoming the descriptive approach for “civil protection containers”, whose qualities, but also intrinsic limits, have been highlighted during the most recent emergencies.

The Regulation defines rigorous “performance requirements” for the structural and functional characteristics of prefabricated units, with the aim to guarantee a high quality of the products; these requirements represent also the criteria for the evaluation of the offers of different producers.

The Special Regulation defines the minimum requirements, independently from the technology and the constructive type adopted, of the living units in the short-to-medium period (1 week - 12 months). The units, installed between 72 hours to 7 days from the event, should allow the activities for the restoration of “normal” conditions of life. These constructions should satisfy the requirements of short transportation time, execution time and performance levels in terms of safety, comfort and environmental sustainability.

The Special Regulation follows a performance-based approach, according to the Technical Regulation for Constructions (D.M. LL.PP. 14 January 2008), the Eurocodes, the regulations for fire safety engineering. In particular, the adoption of European regulations has the twofold advantage to involve a larger number of producers, and at the same time to allow Italian products to be used in the management of emergencies on the whole European territory.

CNR-ITC and UNIBAS developed an updating of the “1st level form for damage survey, first intervention and usability of ordinary buildings in the post-seismic emergency (AeDES).

A further research activity of UNIBAS regards the study of the response of “active” props, i.e. props than can be prestressed in order to make them effective even for low seismic actions. Active props have been studied in previous investigations, which showed their good performance, and encouraged to further investigate their behaviour.

3.7 Development of databases and GIS

The research activity of UNISANNIO has the objective to develop a GIS procedure for the implementation of the neural approach to the evaluation of the seismic risk of urban systems.

In particular, UNISANNIO focused to a deliverable useful to the DPC and with possibility of further developments, thanks to its intrinsic modularity.

The activities were carried out at two levels of detail:

- level 0, which performs an analysis at the national scale, starting from the ISTAT data, aggregated on municipalities, for which an query interface has been developed;
- level 1, performing local analysis which uses the detail data of the area.

4 MAIN RESULTS

4.1 Ordinary buildings

The macroseismic methodology, developed by UNIGE for the analysis of the seismic risk to constructions, is based on fuzzy Damage Probability Matrices implicit in the EMS-98 (Bernardini *et al.*, 2007a) for each vulnerability class (A to F), which are the basis to develop the DPM for the different construction typologies, whenever it is possible to attribute a probability distribution to the different vulnerability classes over which they are distributed. To these ends, preliminary information for both masonry and RC types is implicitly suggested by the definitions of the types considered in the EMS98 scale. Moreover, a Bayesian procedure was indicated for updating those frequencies whenever more information was acquired relative to each typology (modifiers to seismic behaviour).
The mean value of each damage parameter (described by a function of the 6 damage degrees defined by the scale) could then be calculated by way of the identification of opportune extreme distributions, which supply the upper and lower bounds (Bernardini *et al.*, 2007b).
For monotonic damage functions, the extreme distributions used are the upper and lower cumulative distributions derived from the upper and lower bounds of the ordered single degrees of damage. It is therefore determined to be convenient to supply an approximate, analytical and parametrical version for each distribution, based on a unique parameter V (the "Vulnerability Index") ranging from 0 to 100 and independent from macroseismic intensity. The model established the use of the beta distribution in a discrete form (six levels of damage), depending on V and macroseismic intensity.
The methodology, previously calibrated with reference to ISTAT 1991 Census of the residential units (Bernardini *et al.*, 2007b), has been firstly specified with reference to the more reliable data supplied by the ISTAT 2001 Census for each building. As for vulnerability purposes, the ISTAT 2001 catalogue allows one to identify the building in terms of structural typology (masonry, reinforced concrete buildings with and without infill at ground storey, other), age of building (seven age ranges), height (number of floors), state of maintenance (excellent, good, common, bad) aggregation conditions (isolated, adjacent with another building on one side or two or more sides).
The identification of the EMS-98 type, both for masonry and RC buildings, has been solved assigning probabilities of membership to the EMS-98 types as a function of the year of construction; for each municipality the seismic classification of the area and the level of seismic protection imposed by the actual rules in the year of the construction have been taken into account.
With reference to RC buildings, the ISTAT Census distinguishes between regular infilled frames, and frames with irregular (not infilled) ground storey, while EMS-98 considers general frames and walls, designed according to three different levels of seismic protection. Splitting of the probability distribution derived by EMS98 for RC frames has been obtained by expert judgment (Figure 1). Three levels (absent, moderate, high) of earthquake-resistant design have been identified through a correspondence to the seismic classification of the area in the year of construction of the building (for example, after 1981, respectively: unclassified or 3, 2, 1).

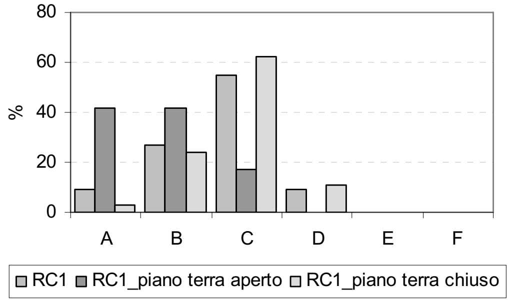

Figure 1. Probability distributions for regular infilled frames and frames with irregular (not infilled) ground storey, without earthquake-resistant design.

Moreover, in the application at regional level (Abruzzi), some available information on masonry units and techniques in different areas (in the application the coastal areas and the mountainous area) has been considered in order to recognise the EMS-98 typologies from the information given by the ISTAT 2001 catalogue (Figure 2).

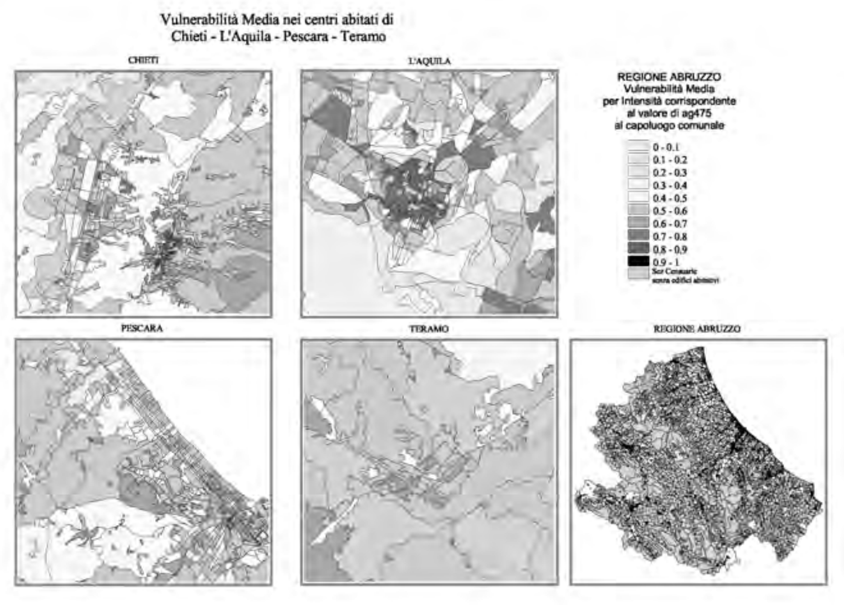

Figure 2. Mean vulnerability in the census tracts of the main town centres and in Abruzzi region.

In the application to the more restricted area in Sulmona (Figures 3, 4) the 14th ISTAT national census data have been compared with a much more detailed and reliable information collected in a field survey performed by CNR-ITC. The comparison shows a good agreement between the two inventories.

Figure 3. Aerial photo of the historical centre of Sulmona (AQ).

Figure 4. Masonry buildings in the historical centre of Sulmona.

The proposed macroseismic methodology and the results obtained suggest that data of different level to describe the inventory of buildings (from the minimum of the ISTAT inventory to the detailed survey of each building) could be combined, in the analysis of the vulnerability, with any other information available for the regional or urban area of interest to reduce the uncertainty.

A hierarchy of coherent databases of different levels of detailing can be used to derive a hierarchy of coherent descriptions of the vulnerability of the building stock and of the expected damage conditional to macroseismic intensity.

Particular attention has been devoted to the AeDES form for the post-emergency survey of buildings. For the application of the macroseismic general approach, specific rules have been proposed to recognise the EMS-98 typologies from the information given by this form and to calibrate the vulnerability modifiers on the basis of the judgements about the considered vulnerability factors.

A similar methodology has also been defined with reference to the forms proposed by the CNR-ITC for masonry and RC buildings, and used in the survey of the building stock in the urban area of Sulmona. Also in this case, for the application of the macroseismic general approach, specific rules have been proposed to recognise the EMS-98 typologies and to calibrate the vulnerability modifiers.

The methodology has been applied to the building stock in Sulmona with reference to two specific earthquake macro-seismic intensities (Figure 5).

Finally, by grouping the results for each census tract, a comparison with the results derivable from the application of the procedure based on the ISTAT data has been performed.

UNINA-DIST updated the vulnerability and risk maps at the national scale. The estimation of the vulnerability distributions has been carried out by searching the possible correlations between the vulnerability typological classes and the building characteristics reported in the ISTAT database.

Two different procedures have been set up and applied in sequence to evaluate the reliability of the results. This was necessary because the disaggregated database was made available by ISTAT only recently. In the meantime, the study was based on the non disaggregated data of 2001 census.

Procedure A utilizes the survey data (damage and vulnerability), the general database for census tract with aggregate data and the regional database with disaggregated data.

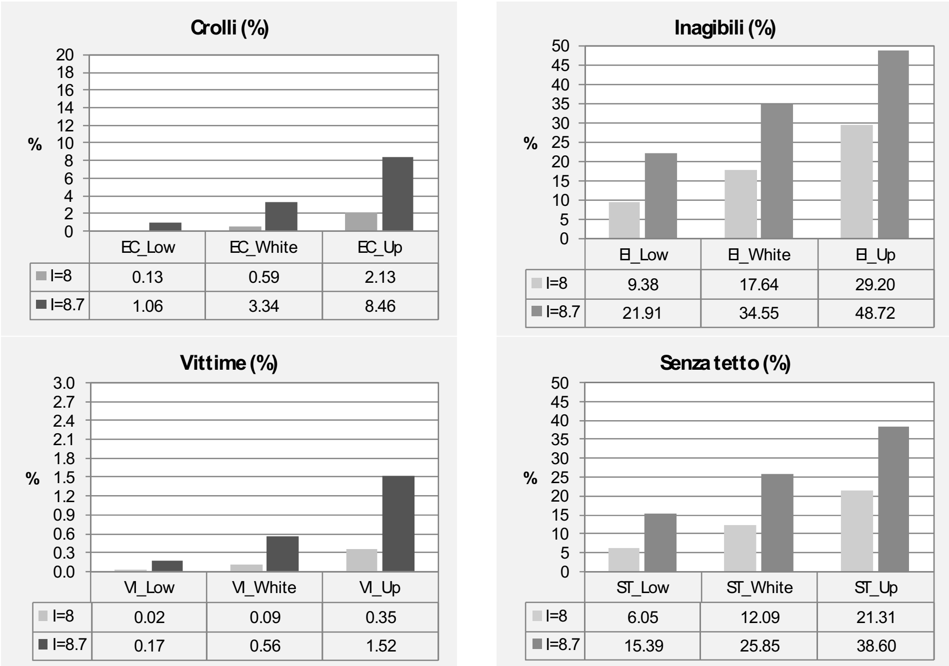

Figure 5. Damage scenarios in Sulmona for I = 8 and for I = 8.7 (return period = 475 years).

The correlations between age, number of storeys and structural types have been determined by analyzing the regional database with disaggregated data. Subsequently, 49 categories of buildings have been defined, as function of the construction period and of the demographic class of the municipality. From the vulnerability database of the surveyed buildings, the distribution has been calculated of the vulnerability classes for each of the 49 categories. In Figure 6, there are shown, as example, two of the 49 distributions found.

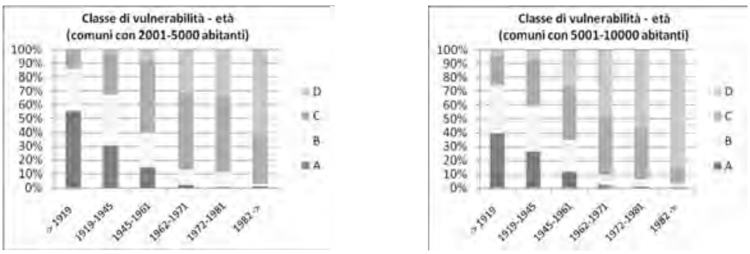

Figure 6. Examples of distributions of vulnerability classes.

By applying these distributions to the buildings of the ISTAT database, the distribution of the vulnerability classes has been estimated for each municipality. It is worth noting that the number of residential buildings reported in the ISTAT database is often not reliable, because the definition of "building" of the ISTAT instructions is some different from that commonly adopted in the macroseismic vulnerability analyses. Therefore, a correction procedure has been set up, which utilizes the number of dwellings and statistics determined on the survey database.

Procedure B utilizes also the database for census tract with disaggregated data. These data are not available for the whole national territory, but only for some municipalities where surveyed vulnerability data were available. The vulnerability data have then been compared with the ISTAT data, and the existing correlations have been singled out. The available information is relevant to the municipality and to the individual building.
Correlation statistics have been searched in this database, as well as recurring characteristics of the buildings as function of the characteristics of the municipality. It is worth to highlight the importance of disaggregated data, which makes possible the extraction of data with *ad hoc* aggregation combinations.
Possible classifications of the buildings in categories have been singled out, utilizing different combinations of the available parameters. The first possibility examined is the combination, as discussed formerly, between age and demographic class, which yields 49 building categories. Other combinations have been singled out, as for instance age + number of storeys + altimetric position, or demographic size + structural type + age + geographic position. For all the building categories present in each combination, a statistic of vulnerability distribution has been calculated from the survey database. By applying these statistics to the ISTAT database, the distributions have been estimated of the typological classes in all the municipalities studied.
The distributions determined resulted independent on the particular combination of parameters chosen. A sensitivity analysis has then been performed, with a set of comparisons between the estimated distributions and those, known, of the survey. This yielded the parameter combination which allows to estimate the vulnerability distributions with minimum errors.
For each municipality, a synthetic vulnerability index has been determined through the procedure already adopted in the GNDT-SAVE Project, which can be described as follows:

- the distribution of expected damage is determined for three hypothetic events of intensity VI, VII, and VIII, respectively, through the Damage Probability Matrices (DPM's);
- for each of the three damage distributions, the mean $S_{pd}(I)$ is determined;
- the synthetic vulnerability index is calculated as mean value of the three $S_{pd}(I)$; from the dimensional point of view, this index coincides with a damage, and its value ranges from 0 to 5.

The DPM adopted is the same that subsequently is utilized to compile risk maps; however, it is worth to note that the vulnerability index does not represent a scenario, but only a comparative scale of the municipalities; therefore, the choice of a particular DPM does not affect in decisive way the result. The intensities VI, VII and VIII have been chosen because they represent the most statistically robust data on damage, as damage data are lacking at high and low intensities.
As regards seismic hazard, the classification of the Project INGV-DPC 2004-2006 has been utilized.
By combining the inventory with the hazard map, and utilizing the DPM recalibrated within this project, the risk maps have been compiled with exceedance probability lower than 10% in 50 years for each of the 8101 Italian municipalities.
The maps report:

- expected number of partial or total collapses (D4+D5);
- expected number of unusable buildings;
- expected number of unusable dwellings;
- expected number of dead;

- expected number of injured;
- expected number of homeless.

Finally, with regard to the evaluation of casualties, it has to be stated precisely that light recalibrations of the parameters utilized in the GNDT-SAVE Project have been adopted.

The research of UNIPV relates the observed damage to buildings to the intensity of the shaking. The displacement spectral ordinates, in correspondence to the period of vibration of the buildings, were considered to represent seismic action. To this end, the attenuation relationship proposed by (Faccioli *et al.*, 2007) was employed, as this prediction equation has been directly developed in terms of spectral displacement and the data used for the regression analysis of this equation were from digital records, and thus considered to be more reliable in terms of displacement prediction.

The vulnerability curves, derived through the activities reported herein, are described in detail in (Colombi *et al.*, 2008), and were compared with analytical counterparts derived with mechanics-based procedures; DBELA (Crowley *et al.*, 2004) and SP-BELA (Borzi *et al.*, 2007a,b). Similar curves were observed for masonry buildings whilst significant differences were seen in the comparison for reinforced concrete buildings, but this was justified by a number of shortcomings related to the database of survey forms.

In the mechanics-based procedures, random populations of buildings for a given building class are first generated with Monte Carlo simulation based on probabilistic distributions of the material and geometrical properties, which are defined *a priori*. Simplified pushover curves of the buildings based on the input data on the material and geometrical properties are then generated for each building, which allow the period of vibration and displacement capacity at three different limit states to damage to be obtained. This information is used to compare the limit state displacement capacity with the limit state displacement demand from a displacement response spectrum anchored to a certain value of PGA to see which limit state is exceeded. Once this has been repeated for all buildings within the random population, the probability of exceeding each limit state for a given value of PGA is obtained. This is repeated for increasing levels of PGA and the probability of limit state exceedance is plotted against PGA; regression analysis in then applied to define a lognormal curve which fits the data (Figure 7).

One of the mechanics-based methodologies (DBELA) has been used in the calculation of seismic risk maps for Italy based on the most up-to date seismic hazard data in Italy. Seismic hazard data for Italy in terms of spectral acceleration for different response periods have been computed within the INGV-DPC S1 Project (2007). The results have been computed for various annual frequencies of exceedance (the reciprocal of the return period) and the results are given as the percentiles of the distribution of all possible values resulting from the logic tree. In particular, the 16th, 50th and 84th percentile maps have been produced for the whole of Italy using the 0.05° grid presenting the spectral ordinates in acceleration for various response periods from 0.1 to 2 s and for return periods varying from 30 to 2500 years, leading to 90 maps of seismic hazard. These maps have been used to produce uniform hazard spectra for a number of return periods for each of the 8101 municipalities in Italy.

The influence of site conditions on the uniform hazard spectra has been accounted for by amplifying the spectra within each municipality as a function of the percentage of different site classes that can be found within that municipality. Site classification at a national scale in terms of the three site classes A, B, C found in Eurocode 8 (CEN, 2004) has been obtained from the map produced by (Amato and Selvaggi, 2002).

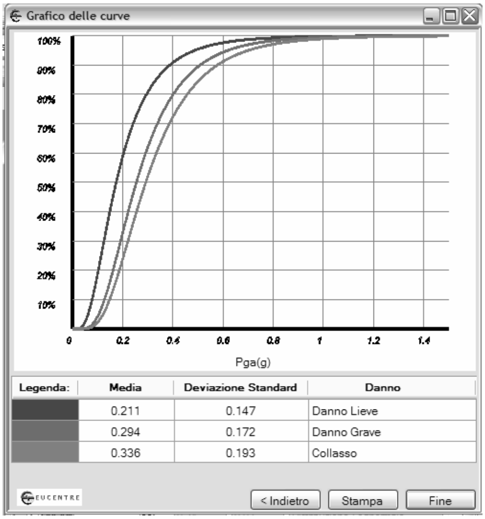

Figure 7. Vulnerability curves and lognormal distribution parameters.

The general characteristics of the Italian building stock for the exposure model have been obtained from the 13th General Census of the Population and Dwellings (ISTAT 1991). The Census data in 1991 was collected in terms of dwellings; however, within the Census form, each dwelling was classified as being located within a building with a certain number of dwellings (from 1 to >30), of a given construction type (RC, RC with pilotis, masonry, other), and with a given number of storeys (1-2, 3-5, >5). Hence, based on the Census forms compiled for all dwellings within each census tract/municipality, Meroni *et al*. (2000) have estimated the number of buildings classified according to the period of construction, number of storeys and the vertical structural type within each municipality. The 1991 Census also includes the surface area and number of residents for each dwelling, from which the surface area and population of each building class (in terms of period of construction, number of storeys and vertical structural type) has been estimated in the same way as described above for the buildings. The volume of each building class has also been estimated by multiplying the surface area by an average storey height of 3 m.

Based on this inventory data for Italian buildings, 29 building classes have been defined as a function of the construction material, seismic design and number of storeys. The geometrical properties of RC buildings which are required include the storey height, the column depth, the beam depth and the beam length; the statistics of these values have been obtained from a sample of reinforced concrete buildings processed by Marino (2005). For masonry buildings, statistics related to the storey height, the pier height and the interstorey drift capacity at different limit states are required in DBELA. All of these data have been collected from various sources including the GNDT 2nd level assessment forms which have been used to assess around 42,000 masonry buildings in Italy (Martinelli and Corazza, 1999).

The exposure, hazard and vulnerability have been convolved to calculate the seismic risk and the following maps have been obtained:

- Total risk in terms of buildings, inhabitants, dwellings, volume;
- Risk related to seismically designed RC buildings in terms of buildings, inhabitants, dwellings, volume;

- Risk related to gravity-load designed RC buildings in terms of buildings, inhabitants, dwellings, volume;
- Risk related to masonry buildings in terms of buildings, inhabitants, dwellings, volume.

For each type of risk reported above, the results can be presented in terms of absolute and percentage risk for different time windows (1, 50 and 100 years) or for scenarios with different return periods (72, 475 and 2475 years).
All of the results (that are saved within a Microsoft Access environment) have been uploaded into a GIS platform and a tool has been coded in Visual Basic which allows the aforementioned seismic risk maps (as well as the hazard and site condition maps) to be easily selected and visualised (Figure 8).

(a) (b)

Figure 8. Screenshots of the GIS platform for the visualization of the seismic risk maps.

The multilevel evaluation method set up by CNR-ITC, together with the updated survey tools, is an innovative proposal for the definition of a unified system for vulnerability and risk analysis of ordinary buildings, open to the integration with other researches on specific issues, as the vulnerability evaluation of RC buildings (UNINA-DIST) or masonry aggregates according to a mechanic approach (UNIPD), and represents one of the contributes to the development of methods for the analysis of urban systems.
The applications at Level 1 with ISTAT 2001 data, and at Level 2 with the proposed forms, produced promising results which, beyond the necessary widening and calibration, highlighted issues relevant to the improvement of the procedures and their further development.
The Tellus database on the damage, usability and vulnerability of the building in the Regione Marche stricken by the 1997 earthquake, and subsequently strengthened, contains the data of the Design Technical Form (STAP), which summarizes the information relevant to building characteristics, damage, interventions and costs. The strength of masonry buildings is expressed through the coefficients C_{conv} e C_{fin}, or their difference ΔC, calculated according to the simplified model of the GNDT form which assumes a storey mechanism.
The results of the study have been utilized in a wider study, in collaboration with Regione Marche, and published in the volume: "Inventory of damage mechanisms, intervention techniques and costs for masonry buildings (AA.VV., 2007).
The database of masonry buildings was organized in ACCESS format and has been fitted out of a report containing the description of the structure, the analyses on the typological characteristics, vulnerability and damage, and some evaluations on the intervention costs deduced from the design.

As regards the forms for the identification of damage and collapse mechanisms, their models and intervention techniques, the following mechanisms have been considered:

- simple overturning of monolithic wall, two leaves wall and multi-storey wall;
- composed overturning with diagonal wedge, double diagonal and multi-storey;
- overturning of joint angle;
- vertical bending of monolithic wall, two leaves wall and multi-storey wall;
- horizontal bending of confined wall, non-confined wall, monolithic wall and two leaves wall;
- pounding of the tympanum.

The forms provide criteria for the recognition and limit analysis of the mechanisms. They describe: the boundary conditions and vulnerability associated to the mechanisms, symptoms of activation, variants. In particular, they include:

- a synthetic description;
- schematic drawings and photographic references;
- the main factors which allow the recognition;
- the analytical formulation of the problem and the calculation model;
- the main interventions opposing the mechanisms;
- a scheme of calculation procedure with an example of the "CINE" software.

The calculation of the collapse multipliers is performed through EXCEL spreadsheet.

Another deliverable, elaborated in collaboration with POLIMI and UNIGE, is represented by the 1st level form for the classification and the evaluation of masonry quality. It is based on surveys and design of interventions, fitted out of a procedure for the attribution of mechanic parameters according to the OPCM 3431/2005 and the Draft of Instructions to the NTC, DM 14 January 2008. The form is based on the recognition of the masonry fabric and on qualitative evaluations The form has been conceived also to be utilized for the identification of the masonry types within the framework of vulnerability surveys.

UNIPD set up and validated updated versions of the procedures Vulnus and c-Sisma. The procedure Vulnus performs vulnerability evaluations, for individual buildings or groups, with extension to the predictions of damage (fragility curves) according to the EMS-98; the procedure c-Sisma performs assessment (SLD and SLU, in the two simplified procedures and with capacity spectrum), in relation to the updating of the seismic analysis of the OPCM 3431/2005

Probabilistic relationships (Fragility Curves, FCs) of some structural types are obtained by UNIBAS starting from the assigned Vulnerability Class. Actually, real buildings, made of plane frames having different dynamic characteristics, strengths and available ductility, show a complicated non linear dynamic behaviour. 3-D types representative of real RC buildings widely present in Italy, made up of some plane frames and considering low-rise mid-rise and high-rise structures, were considered to account for the interactions among the different plane types. Further, an application of the proposed methodology to the residential RC buildings of the urban centre of Potenza was carried out, also comparing the obtained results to those ones provided by the classical Damage Probability Matrix approach.

Tentative Fragility Curves relevant to the above described structural types have been obtained. Four damage states, beyond the null state, have been considered, according to the damage classification of the EMS-98.

Making reference to the methodology provided in the RISK-UE Project (Spence and Le Brun, 2006), each fragility curve is characterized by the median value and the lognormal standard deviation of the selected seismic parameter. The Housner Intensity (HI) has been chosen as seismic parameter because, as it is shown in some studies (e.g. Masi *et al.*, 2008), an integral seismic parameter, such as HI, is more effective than peak or spectral parameters in representing the damage potential of a ground motion.

In Figure 9, a suite of FCs relevant to some structural types is displayed. It has to be underlined that they can be considered as a first proposal of FCs to be possibly updated in the future.

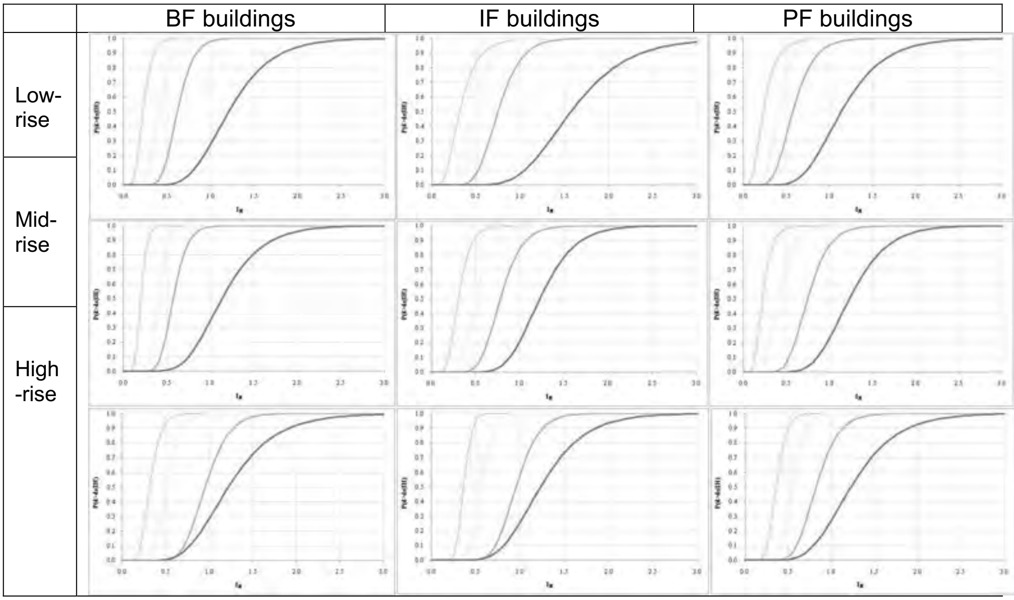

Figure 9. Fragility Curves for Italian RC frame buildings designed only to vertical loads: low-, mid- and high-rise buildings, without infills (BF), regularly infilled (IF) and with pilotis (PF).

UNINA-DIST set up a form for the survey of RC buildings which includes a more complete section on structural damage. In particular, damage patterns not originated by a seismic event were introduced in order to collect information on the building state prior to a seismic event and help planning the post-earthquake intervention. A suitable algorithm for survey form digitalisation on pocket PC was developed, with automatic check procedures to minimize compiling errors. The form was tested in Arenella district in Naples, constituted by more than 1500 buildings.

A second deliverable is related to the method for class-scale risk assessment. The vulnerability of a building class is analysed starting from push-over analyses performed for virtually all the buildings belonging to such class. In particular, a series of subsequent steps are applied: (a) perform building inventory; (b) generate a sample of building models through simulated design process (Verderame *et al.* 2007), perform push-over analyses and determine global capacity parameters for each one of them; (c) run Monte Carlo analysis extracting random model input parameters from the relative statistics (corresponding to generic building within the class); (c') calculate the capacity by local regression from the capacities of sample buildings; (c'') compare capacity with demand, in a capacity spectrum framework, to determine fragility curves for slight, moderate, extensive and complete damage.

The suitable re-elaboration of the vulnerability data for RC built environment of Arenella district in Naples allowed to perform a risk analysis at the territorial scale. In particular, for the three building classes RC1 (RC frame buildings having 1 to 3 storeys), RC4 (4 to 6 storeys), and RC7 (buildings with more than 7 storeys) (Figure 10), the fragility curves, previously determined in terms of spectral displacement, were re-determined in terms of peak ground acceleration. These curves refer to the probability of attaining to four different limit states, as defined in HAZUS (FEMA-NIBS, 1999): slight damage, moderate damage, extensive damage and complete damage (collapse). In order to allow the utilization of the available information on local seismic hazard (INGV-DPC S1, 2007), the fragility analysis was performed scaling the elastic spectra provided by the actual code for Naples town at the coordinates of Arenella district. The obtained fragility curves may be utilised both to perform scenario analyses and risk analyses. By using the hazard data for $T = 0$, given by INGV-DPC for a 50 years time interval, the area risk was computed, i.e. the probability that the buildings belonging to the different classes attain to failure in the considered time period.

Figure 10. Classification of RC buildings in Arenella district in Naples.

4.2 Public and strategic buildings

CNR-ITC developed and analyzed a database on the seismic assessment of schools of Regione Molise, including:

1. localization of the individual structural units within the school complexes;
2. technical-administrative data on the intervention designs;
3. typological and dimensional data;
4. graphical survey;
5. vulnerability and structural weak points;
6. test results;
   a. strength of steel and concrete, determined through tests on cores and SonReb method;
   b. strength of masonry, mainly determined through flat jacks tests;
   c. dynamic characterization through ambient vibration tests;
   d. load tests on floors;
   e. characterization of the soils;

7. results of the simplified assessment through the models VC and VM (Dolce and Moroni, 2005), or other types of analysis, in terms of collapse PGA and corresponding return period;
8. final judgement on the vulnerability and risk conditions;
9. preliminary proposal of intervention.

The database on the schools assessed was linked with the database of the Ministry of Education, University and Research (MIUR).

A second deliverable consists of a procedure for the transfer to the DPC of the data of the "Synthetic form on seismic verification at level I or level II of strategic buildings to the purposes of Civil Protection, or prominent in the case of seismic event", fitted out of user's manual of the software and support to filling in. The procedure was released nationwide and will allow the systematic collection of the assessment data, and the development of a centralized database at the DPC.

UNIBAS developed an updated version of the procedure VC for the analysis of buildings with stair-lift cores. The cores are modelled as vertical cantilevers under a concentrated load at 2/3 of their height. The strength calculation is carried out according to the NTC, accounting for the brittle shear failure and the ductile bending failure.

4.3 Infrastructures

UNITN developed a Bridge Management System (BMS) integrated by a vulnerability model consistent with the HAZUS model. In this model, the seismic vulnerability is expressed through fragility curves. They represent the probability that the structure will reach a predefined damage state as a function of seismic intensity. Fragility curves are obtained comparing the seismic demand (i.e. the structural performance required by the seismic action) with the structural capacity, that can in general be represented in terms of displacement or strength. The seismic demand is evaluated using the spectral method, while the structural capacity is obtained by considering different possible damage scenarios. From the observation of the consequences of past seismic events, it can be seen that the most vulnerable elements of a bridge are: piles, abutments and bearings. The system considers two damage scenarios: damage to the piles because of horizontal actions and damage to the deck caused by sliding on bearings.

Consistently with HAZUS paradigm, and according with the NTC (DM 14 January 2008), the model considers five different condition states and four performance levels (minor damage, moderate damage, major damage and collapse).

The HAZUS model, defined by the geometrical and typological information collected into the database, represents the prior fragility curve. For instrumented bridges, the posterior fragility curve is then estimated based on the response history of the embedded instrumentation using Bayesian logic. In a similar manner, using Bayesian inference, the information is extended to the whole stock.

As regards the development of smart elements, the sensor technology adopted is a multiplexed version of the standard SOFO fiber optic strain interferometric sensor, where in-line multiplexing is obtained by separating each measurement field through broadband FBGs, coupled with traditional sensors, including metal-foil strain gauges and thermocouples. The optical sensing system is prepared in the form of a 3-field smart composite bar. An instrumented element in prestressed concrete, of size $3.8 \times 0.5 \times 0.3$ m^3, has been manufactored and tested. The specimen was post-tensioned with real-time control using a load cell inserted within the anchorage system. The load protocol includes a sequence of load-unload cycles, repeated under different values of pre-stressing and maximum vertical load. The experiment aimed at identifying the response of the sensors to different damage conditions artificially produced on the element, including cracking, cover spalling and reinforcement corrosion. In particular, the intent was to correlate the response of the embedded sensor to different prestressing levels on the beam. Indeed, the release of the prestress caused on the specimen the appearance and progressive propagation of cracks.

The prototype deployment in the laboratory demonstrated how the procedure to fabricate smart elements is perfectly feasible, and can be applied at industrial level to the production of full-scale precast element for bridges.

To exploit appropriately the large amount of measurements recorded by the system, a Bayesian approach is applied to interpret measurement data while also allowing proper handling of all prior knowledge, including material properties, environmental conditions and sensor performance. This methodology allows to identify not only the most likely values of the unknown damage parameters (such as type, position and extent) but also their posterior probability distribution. The aim of experimental validation is to recognize the loss of prestressing based on sensor data: the resulting estimate can be compared with the actual preload recorded by the load cell applied on the prestressing bar. The laboratory test shows that an occurrence such as a loss of prestressing can be recognized early with a high degree of reliability based on the strain data acquired. It is interesting to observe that, in the test analysis, when prestressing is partially released, the probability of the damage scenario becomes immediately very high, independently of the precision of the damage parameters identified. Despite the fact that the validation given in the test is very specific, the general approach adopted is not problem dependent, and can be extended to a broader class of problems, including manifold scenarios, model or material uncertainties, prior knowledge of parameters distribution.

The approaches and the results of the first two objectives has been merged together into a Decision-Supporting-System that works at network level. This tool allows taking rational decisions on a stock of infrastructures following to the occurrence of a seismic event.

According to the system approach, a set of representative structures are supposed to be instrumented and monitored in real-time. The data flow deriving from this set is processed for a twofold aim. On one hand it allows to update the vulnerability curves of the monitored structures, on the other it allows projecting the results onto the whole stock.

According to HAZUS paradigm, the fragility curves depend heavily on physical parameters related to material properties and structural behavior. When no measurement is available on a specific structure, the curves are derived from prior information taken from literature. For instance, Figure 11 shows the distribution of the probability of exceeding an operational limit state in the APT network for a design earthquake with a return time of 475 years, using the HAZUS model. Each bridge is represented by a dot, the colour of which (green, yellow, orange and red) represents increasing probability values, being green lower than 10^{-5} and red greater than 10^{-3}.

On a monitored structure it is possible to update in real-time the distribution of some relevant parameters and, in turn, the fragility curves, following the approach of the second objective. This process is generally efficient even during the ordinary environmental condition, as some parameters play a key role both in daily response and in the seismic one: for example an estimation of the dynamic mass of a structure can be derived from the ordinary vibration and employed in the evaluation of the seismic ultimate capacity. However the response during a seismic event is much more relevant for the estimation of key parameters involved into the curves computation. Hence basically the method requires: i) to analyze the seismic response, ii) to update the distribution of the relevant parameters, iii) to compute the corresponding curves, including the posterior uncertainties. Of course, for the sake of estimating the damage related to seismic event, the sensor measurements can be directly employed: e.g. a sensor placed on the interface between deck and pier can directly measure a possible relative displacement. However, the updating of the parameters based on the measurements is essential to project the estimation of the whole stock. The data taken on the monitored structures can be linked to the capacity analysis of other bridges: in facts, those measurements can be equivalently interpreted as deriving from a set of full-scale seismic tests on similar structures. It is worth noting that these data are related to the seismic excitation whose consequences the user is exactly interested on.

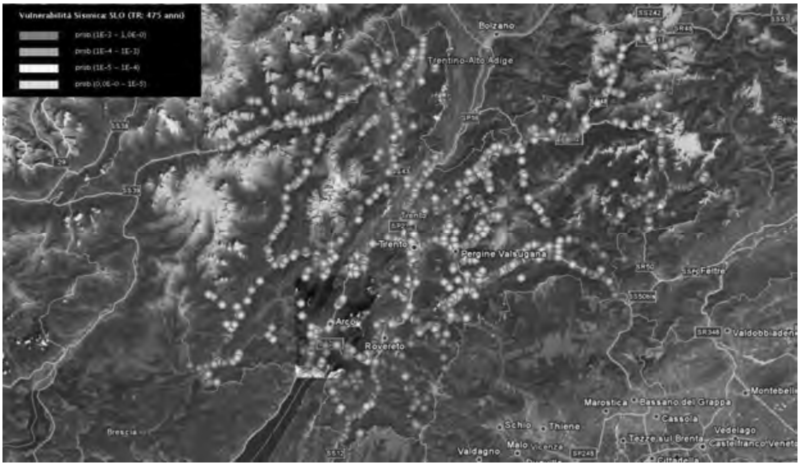

Figure 11. Operation scenario of the APT network after a 475-year return time earthquake.

The way in which the tool works can be illustrated by an example. Let us suppose a series of highway bridges is built in the same period, adopting the same typology and, possibly, the same dimensions. Let us further assume that the seismic capacity is directly related to the friction coefficient between deck and abutment. The value of this coefficient in different bridges can be model as probabilistically related, deriving from the same distribution. Hence, the estimation of the friction on the monitored bridges provides information even of that of the similar ones.

As it is shown by the example, it is key to define the probabilistic correlation between the set of monitored bridges and the unmeasured ones. Once these correlations are established, the updating of the curves and, in turn, of the seismic capacity, is possible.

4.4 Urban systems and historical centres

A first deliverable, elaborated by CNR-ITC, DPC, UNIBAS, UNISANNIO and INGV-ROMA, consists of the GIS software for "level 0" analysis. It calculates the total capacity loss of an urban centre for a given seismic input. This software can be applied to all the Italian municipalities; an example of application is provided for the municipalities of Abruzzi.

The components and the sub-products of the software are: the base hazard (CNR-ITC and INGV-ROMA), drawn from the INGV database for all the Italian municipalities, and referred to their centre; the local hazard accounting for lithologic/morphologic characteristics of Italian municipalities (INGV-ROMA); PGA values for different return periods for all the Italian municipalities; the capacity losses because of unusability or structural collapse (DPC), determined from the ISTAT 1991 data through damage probability matrices for all the Italian municipalities; the capacity losses of urban centres (CNR-ITC, UNIBAS e DPC), determined through elaborations on the data of ANCITEL 2000 census on Italian municipalities, through parameters and indicators of the components of sub-systems, to describe:

- casualties and capacity losses of "objects" stricken by the earthquake (buildings, infrastructures, streets, etc.);
- capacity losses (direct and indirect) of the minimum sub-systems necessary for the physical and functional organization of urban systems: residential, social, cultural heritage, economic, infrastructural, networks and emergency.

A second deliverable, elaborated by CNR-ITC, UNIBAS, DPC, INGV-ROMA, UNINA, UNIPD, POLIMI, UNIGE and UNISANNIO, is the software for the analysis of "level 1" model. This software is constituted of a set of interfaced GIS, which calculate the capacity losses (direct and indirect) of the different components of the sub-systems, given the seismic input. This software can be applied to all the urban centres where an expeditious survey has been carried out according to pre-determined criteria and adopting suitable forms; an example is provided for the historical centres of Sulmona.

This deliverable has the following components or sub-products:

- damage scenarios, in term of capacity loss for the historical centre of Sulmona (CNR-ITC, UNIGE), based on the methods for ordinary buildings from the data of expeditious survey;
- evaluation of local hazard, with reference to the historical centre of Sulmona (INGV-ROMA);
- method for evaluating the capacity loss, with reference to a given earthquake, for masonry buildings (UNIPD and POLIMI), determined on the basis of expeditious survey;
- method for evaluating the capacity loss, with reference to a given earthquake, for RC buildings (UNINA), determined on the basis of expeditious survey;
- evaluation of the capacity loss for Cultural Heritage and historical buildings of the centre of Sulmona (DPC and UNIGE), based on survey data on churches (CNR-ITC);
- method for evaluating the capacity loss of streets (CNR-ITC and UNISANNIO), based on expeditious survey of the historical centre of Sulmona (CNR-ITC).

As regards the geologic, geomorphologic, geotechnical and geophysical databases, the following deliverables have been accomplished (INGV-ROMA):

1. database of the 8101 Italian municipalities, containing information on base and local hazard, in EXCEL format.
2. "Geologic" forms of I and II level for the "Evaluation of local effects in the sites of individual buildings (ordinary, strategic and monumental buildings)", with instructions for filling in, and Local Seismic Hazard Index (LSHI), based on the data of the I level form.
3. Expeditious microzonation of the historical centre of Sulmona.
4. "Geologic" forms for the "Evaluation of local effects in the sites of individual buildings", relevant to 16 churches to the historical centre of Sulmona.

POLIMI, based on the survey of the building types and constructive elements of the historical centre of Sulmona, drew up an abacus of the types of aggregates and a typological map of the centre.

In collaboration with UNIPD and CNR-ITC, the following deliverables have been accomplished.

1. Recognition of the building types in the historical centre of Sulmona, based on the PRG map and in situ investigation.
2. Survey of masonry types, based on the "Form for the evaluation of masonry quality", set up within the framework of Line 1, starting from the previous "Form for the

typological survey of seismic damage to buildings and evaluation of their vulnerability" (POLIMI), including the contribution recently proposed for the definition of a Quality Index (Binda *et al.*, 2007b; Acito *et al.*, 2008).
3. Critical revision of the form and ensuing modification, in collaboration with CNR-ITC.
4. Application of the "Form for the typological survey of seismic damage to buildings and evaluation of their vulnerability" to representative aggregates of the historical centre of Sulmona: block 69 in via Acuti, Palazzo Sardi, Palazzo Meliorati and building in Corso Ovidio. For these buildings, the following deliverables have been accomplished:
   - geometric and cracks survey (Lombardo and Sgobba, 2008; Restelli and Rossini, 2008);
   - analysis of toothings;
   - characterization of masonry quality through ND and MD tests (Acito *et al.*, 2008; Cardani *et al.*, 2008);
   - vulnerability analysis, in collaboration with UNIPD (Ferrario *et al.*, 2008).

The survey of the building types in the historical centre of Morgnaga (Gardona), stricken by the 2004 earthquake, has been carried out as well (Anzani *et al.*, 2007b).
The data collected through the "Form for the typological survey of seismic damage to buildings and evaluation of their vulnerability" will be implemented in an accessible database on the server of POLIMI. The foreseen updates will regard the historical centres of: Ponte di Cerreto (Umbria), Bajardo, Taggia, Bussana, (Liguria), Sulmona (Abruzzi), Morgnaga (Lombardy) (Penazzi *et al.*, 2000; Valluzzi *et al.*, 2001; Binda *et al.*, 2003; Anzani *et al.*, 2004; Binda *et al.*, 2004a,b,c,d; Binda *et al.*, 2005a,b; Valluzzi *et al.*, 2005; Binda *et al.*, 2007c; Binda and Piccarreta, 2007).
In the future, the database could be linked with similar databases provided by other RU, referred to forms of levels 0 and 1.
UNIPD developed the following deliverables:
1) Definitive version of the procedures, updated in relation to the performance based requirements of the Ordinance 3431/2005, for the systematic evaluation of vulnerability for existing masonry buildings, at global level (Vulnus) and local level (c-Sisma), based on the analysis of local mechanisms of structural macroelements.
2) Report on the vulnerability analyses, utilizing the updated procedures Vulnus and c-Sisma, on some historical centres of Umbria, based on available surveys (Munari *et al.*, 2009a,b; Valluzzi *et al.*, 2009). These centres, stricken by the 1997 earthquake, are characterized by different typological distributions of buildings: Campi Alto di Norcia (PG) is characterized by simple types of masonry buildings, isolated or in rows, Castelluccio di Norcia (PG) is characterized by more complex aggregates. The analyses yielded:
   Vulnus: vulnerability evaluations, individual or for groups, with extension to damage prediction (fragility curves) according to the EMS-98;
   c-Sisma: assessment (SLD and SLU with simplified procedure and procedure based on the capacity spectrum), quantification of the improvements corresponding to structural interventions, in relation to the models of OPCM 3431/2005.
3) Report on the investigations on the blocks 27, 39, 48 (Palazzo Sardi), 69, 87 (Palazzo Tabassi), 92 (Palazzo Meliorati) in the historical centre of Sulmona. The method of analysis consisted of:

a) in situ survey through forms (of buildings and masonry) of a significant sample of the buildings, which includes different types (buildings in simple and double row, court buildings, blocks) – in collaboration with POLIMI;
b) for some units, complete identification of the masonry types, also through ND and MD tests – in collaboration with POLIMI;
c) data collection (interventions, damage mechanisms) and vulnerability evaluation of the building systems in the historical centre through the procedures for vulnerability analysis Vulnus and c-Sisma.

4) Report on the evaluation method at level 1 (synthetic/expeditious investigations and surveys) of the residential sub-system in masonry of the urban system of Sulmona, evaluating the capacity loss because of a given seismic event, in order to single out the parts with higher risk, aimed at the implementation of neural models for the risk evaluation of urban systems. The study consisted of:
   a) definition of the physical-mechanical parameters describing the initial resisting capacity;
   b) development of simplified mechanical models of typical masonry buildings;
   c) static non linear analyses (push-over) yielding capacity curves;
   d) determination of the capacity loss: damage analysis and estimation of expected damage;
   e) comparisons with other methods.

For each of the RC building examined in the centre of Sulmona, UNINA-DIST performed a fragility analysis. In particular, the methodology introduced in (Iervolino *et al.*, 2007) and utilized in (Polese *et al.*, 2008) was applied, with some important differences because the buildings studied are infilled.
The behaviour of infilled buildings was studied through pushover analysis, modelling the infills as equivalent struts acting in compression. The characterization of such struts is strongly dependent on the mechanical properties of the infill panel; in order to account for uncertainties in the characterization of infills, a suitable variability of the cracking strength and of the elastic normal and tangential moduli E and G, respectively, of the infills was introduced. Moreover, the differences between strong and weak infills were suitably accounted for by changing the mean strength and the correlated elastic moduli.
For what regards the determination of the seismic demand, the approach introduced in (Dolsek and Fajfar, 2004) was utilized; this method reduces the elastic demand as a function of a set of strength and ductility parameters. The fragility analysis, for the limit damage states of damage and near collapse, was performed scaling the elastic spectrum provided by the present seismic code for the Sulmona centre, and deriving the relative curves in terms of peak ground acceleration. Finally, the capacity loss was evaluated for the peak ground acceleration corresponding to the 50 and 475 return periods scenarios.

4.5 Monuments

UNIBAS surveyed one hundred monuments of Regione Basilicata through three forms: the check list, the masonry form and a vulnerability/damage form, purposely developed. The last form consists of three sections: the first section is devoted to seismic vulnerability, the second to damage, either of seismic or non seismic origin, the third to decay and interventions. The damage examined is that relevant to the Irpinia-Basilicata 1980 earthquake. Each section is particularized to the individual macroelements.
The analysis of the database yielded the damage probability matrices, as function of the MCS intensity, for the main damage mechanisms. With the aim to account for elements increasing the vulnerability, or at the contrary of anti-seismic elements, multiple regression models have

been utilized, determining the factors which influence the vulnerability of each macroelement. Finally, the correlations between the different macroelements have been analyzed.
A detailed analysis on 10 monuments showed that, similarly to the minor building stock, the interventions on the Cultural Heritage of Regione Basilicata utilize the same intervention techniques on buildings characterized by different structural systems. The structural weak points at the present state have been highlighted, in many cases originated by the interventions themselves, which may play a detrimental role in the case of future seismic event. The critic elements of the different buildings have been analyzed in a comparative way, for types of structural element or macroelement (bell tower, vault, chapels, etc.), also accounting for the present cracking state.
Based on the results obtained, conclusions are drawn on static and seismic safety levels of the monuments studied, as well as on the effectiveness or harmfulness of the post 1980 interventions. Finally, provisional intervention protocols have been developed, generally based on tie-rods and rings.
CNR-ITC surveyed the vulnerability of about 30 churches of the territory of Gran Sasso, near L'Aquila, through the form for churches, with 28 mechanisms. The churches investigated belong to six municipalities in L'Aquila Province: Barisciano, Calascio, Castelvecchio Calvisio, Castel del Monte, Carapelle Calvisio, S. Stefano di Sessanio. This territory is one of the cultural-tourist districts of the National Park of Gran Sasso and Monti della Laga, named 'Terre della Baronia'. The churches were studied according to a vulnerability analysis at Level 1, aimed at testing the Guidelines for the Cultural Heritage, also in the more general framework of the new seismic Regulation. This activity may represent a useful contribution to the monitoring of the application of the Guidelines, in order to define priorities and strategies for restore and preservation.
INGV-ROMA coordinated a study, to which POLIMI, UNIGE and Regione Molise took part, investigating the relation between damage to churches and the morphologic characteristics of the sites, according to the macroseismic approach. With the aim to evaluate the importance of local amplification effects, observed damage has been compared with mean expected damage, calculated on the basis of surveyed vulnerability. Based on this comparison, for the whole sample examined, it was evident that the observed damage after the Molise 2002 earthquake is higher than the mean expected damage. The geologic, geomorphologic, geotechnical and geophysical data of the sites show that the influence for lithologic causes can be neglected; therefore the major effect was ascribed to topography. Within the framework of a vulnerability analysis through the macroseismic approach, it was preferred accounting for the topographical amplification through the definition of an added "vulnerability", through a suitably defined modifier for local morphology (Di Capua *et al.*, 2006). The modifier has been calculated, for each church, by evaluating the vulnerability increment required to match the mean expected damage and the observed damage. The results show that the topographic amplification is affected by the combined values of relief slope (α) and height (H). In particular, the effect are proportional to slope and inversely proportional to height.
The analysis according to the mechanic approach evaluated the influence of topographic amplification, considering the increment of seismic input according to OPCM 3431/2005. For the sites studied, a numerical 2D modelling of the local seismic response was performed, through a code based on the boundary element method (Brebbia, 1984). It allowed to calculate the amplification factors (Pergalani *et al.*, 2003a,b) as ratio between the output and the input spectral intensities in the period intervals 0.1-0.5 s and 0.5-1.5 s (Housner, 1952) and as elastic response spectra. The amplification factors obtained are in agreement with the regulations (OPCM 3431/2005; EC8, 2003). Moreover, by studying the amplification factor as function of the morphologic parameters of the sites, it is possible to draw conclusions

similar to those of the macroseismic approach, highlighting the substantial equivalence of results of the two approaches.
For the churches of San Michele Arcangelo at Campolieto (CB) and San Pietro in Vincoli at Castellino del Biferno (CB) (Figure 12), the activation acceleration and the capacity curve of façade overturning (mechanisms of global overturning and partial overturning of the upper part) were calculated according to limit equilibrium analysis. These two churches, damaged by the Molise 2002 earthquake, were chosen because they appear to be interested by topographic amplification of different degrees, which may be highlighted by the approaches developed. According to the "Guidelines for the assessment and the reduction of seismic risk to Cultural Heritage, with reference to the Technical Regulations for Constructions" (point 5.2), the assessment has been performed analyzing local models based on the concept of macroelement (Doglioni *et al.*, 1994). In particular, the out-of-plane mechanisms of the façade have been studied, according the method of linear kinematic analysis (Guidelines, point 5.2.2) and non liner kinematic analysis (Guidelines, point 5.2.4). The analyses have been performed either considering or neglecting topographic amplification. The results of the linear kinematic analysis highlight that for the Church of San Pietro in Vincoli the acceleration of the 2002 earthquake is largely higher to the spectral acceleration of the macroelement, and the increment associated to topographic amplification is in better agreement with the observed damage, which is near the structural collapse (level 4 – EMS-98). On the contrary, for the Church of San Michele Arcangelo, the demand of the earthquake is lower to that corresponding to the ultimate limit state, coherently with the low damage level observed in the macroelement. The influence of topographic amplification appears not significant, also in relation with its low value.
The results of the non linear kinematic analysis, in relation to both mechanisms studied, are less conservative, compared to linear kinematic analysis. In particular, the verification of the mechanism of global overturning is always satisfied for the two churches, whilst the verification for the overturning of the upper part is not satisfied for the Church of San Pietro in Vincoli, when topographic amplification is taken into account. The better accuracy of non linear kinematic analysis yields a damage scenario which is in good agreement with the observed damage, in relation to both mechanisms studied.

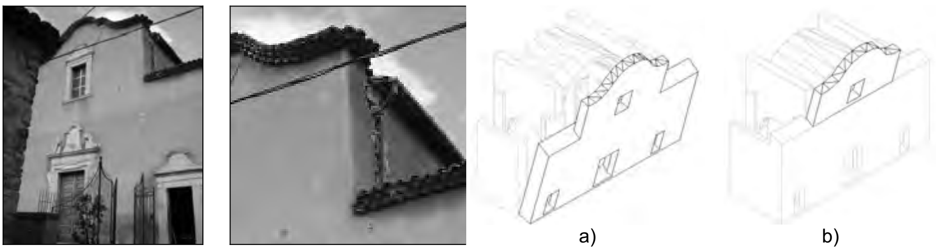

Figure 12. Damage associated to the overturning of the façade tympanum of the Church of San Pietro in Vincoli, Castellino del Biferno (CB), for the Molise 2002 earthquake, and collapse mechanisms studied: a) global overturning, b) overturning of the upper part.

The comparisons performed validate the meaning of the vulnerability modifier within the framework of the macroseismic approach, as well as the reliability of the amplification factor derived from the "geologic" form. In this way it is possible, in absence of an explicit modelling of topographic amplification, to utilize the values calculated according to the "geologic" form, or to the macroseismic approach, for the evaluation of topographic amplification effects. This quantities are suitable for evaluations at the territorial scale and in the cases of limited knowledge. The results, in terms of pseudo-acceleration spectrum, have been utilized within the framework of the mechanic approach at the level of individual structural element, and therefore are suitable for detailed analyses in the cases of high knowledge levels (Compagnoni *et al.*, 2007; Di Capua *et al.*, 2007).

4.6 Emergency planning and management

The Special Regulation for emergency housing, drawn by UNINA-COSTRARC and UNIFI/UNIBAS, is organized according to the following index:

I. General aspects;
II. Technical specifications;
III. Documents for certification;

Annex A – Declaration of offered performance and certification.

In Section I, the principal reference codes and standards, together with general principles, are defined.

In Section II, the techical specification are presented. The specification form is divided in two parts: in the first part the requirement is identified; in the second part the contents and the prescriptions to be evaluated in the design and costruction phase are specified.

In Section III, the documents are defined for the certification that the contractor has to present in order to certify the performance of his product.

Finally, in the Annex A, the certification forms that have to be filled by the producer are presented.

CNR-ITC and UNIBAS developed an updated version of the "1st Level Form for Damage Survey, First Intervention and Usability for Ordinary Buildings in the Post-Seismic Emergency" (AeDES), which improves the section of masonry building type, and fills the gap of the previous version for RC buildings through a descriptive section with 7 parameters which can be easily observed. Moreover, an estimation is introduced of damage level to structural components and to the whole building, based on the classification of the EMS-98.

The additions and modifications introduced make the form more complete, and make the data congruent with those of 2nd level forms for the survey of typological characteristics, vulnerability and damage to ordinary masonry and RC buildings.

As regards the study on active props, UNIBAS performed a parametric analysis on statically determined propping systems with steel ties, varying interstorey height, masonry thickness, distance between the boundary and the wall, type of boundary (raised or not). The design of the propping system has been performed according to either the OPCM 3274 or the NTC 2008. A software has been implemented for expeditious design of the provisional interventions, interfaced with the form for damage survey and active mechanisms, finalized to emergency management and costs estimation.

UNIBAS set up a procedure for the evaluation of economic losses caused by damage in the building stock, based on an estimation of the repair cost conditional upon the suffered damage level and the building type (Dolce *et al.*, 2006).

The estimation of the repair cost was carried out by (Di Pasquale and Goretti, 2001) using the data collected with the GNDT-SSN usability survey form after the Umbria-Marche 1997 and the Pollino 1998 earthquakes, thus obtaining a database with more than 50,000 buildings. Based on the information drawn from this form, typical repair interventions were selected, their extent and cost were computed, and the global repair costs were evaluated. As a result, curves of repair costs as a function of damage level and building vulnerability class were proposed. Specifically, an economic damage index $C_{r,r}$ (relative repair cost) was evaluated, equal to the ratio of cost of repair to cost of replacement of the building.

The curves obtained by (Di Pasquale and Goretti, 2001) indicate that $C_{r,r}$ depend only on damage level, whereas the dependency on the building vulnerability class, i.e. on the type of the structural system, can be neglected. As $C_{r,r}$ is a random variable whose values are bounded between 0 and 1, the curves of the relative repair cost as function of the damage level (referred to the mean damage of the vertical structures) can be described by a standard beta distribution.

The proposed procedure provides the mean value μ and standard deviation σ of the distributions, together with the relevant values of the parameters q and r of the beta-distribution, obtained in closed form.

On the other hand, the DPM's provide the probability to observe the different damage levels L_d for each vulnerability class, given the seismic intensity. By combining these probabilities with the Cumulative Distribution Functions CDF's of the relative repair costs, the total probability can be obtained for each considered value of repair cost $C_{r,r} > 0$, given the seismic intensity I. If an inventory of buildings is available, the total repair cost caused by a given seismic event can be computed by applying the proposed procedure as a fraction of the total cost of replacement of the entire building stock.

The proposed procedure was applied to some case studies, such as some small towns damaged during the 1998 Basilicata earthquake. A comparison between the total repair cost, computed assuming a suitable average replacement cost applied to the total area of the building stock of each town, and the total economic requirements fixed for government grant has been carried out confirming the good prediction capability of the procedure.

4.7 Development of databases and GIS

The GIS architecture developed by UNISANNIO simulates the interactions between the individual urban sub-systems through the techniques of spatial analysis: each element of a specific sub-system interacts with the elements of other sub-systems depending on its position; the interactions between individual elements are then simulated through a spatial analysis procedure which activates when indicators thresholds, which imply the capacity loss of the whole sub-system, are reached.

For the system which represents the input level of the system, that is the level of local geology, it is necessary to define the hazard, to be associated to the model for the evaluation of risk and losses, both in emergency and at the steady-state. The residual capacity of each individual sub-system is then determined in the different phases of the emergency until the return to the normal steady-state. For a correct definition of the model, and in order to complete the implementation, suitable algorithms have been set up to calculate the concatenation of neural effects among the different sub-systems.

In particular, the model represents each sub-system as an individual neuron, and the relations between sub-systems as synapses linking the different neurons. The system is structured at different informative levels, representing the individual sub-systems/neurons, and algorithms are implemented for the representation of different relations/synapses between the sub-systems.

The interaction between buildings and road network has been represented for the historical centre of Sulmona. To this end, the graph of the road network has been drawn through a linear information level where the dimensional characteristics of the arcs (stretches of road) and of the joints (crossings) are reported. The "buffer" functionality allows to deduce useful information on the probability of obstruction of the roadway in the case of building collapse (Figure 13).

The present GIS, based on neural logic, can be further developed, in relation to the implementation of new functions for the preventive evaluation of the vulnerability of an urban centre.

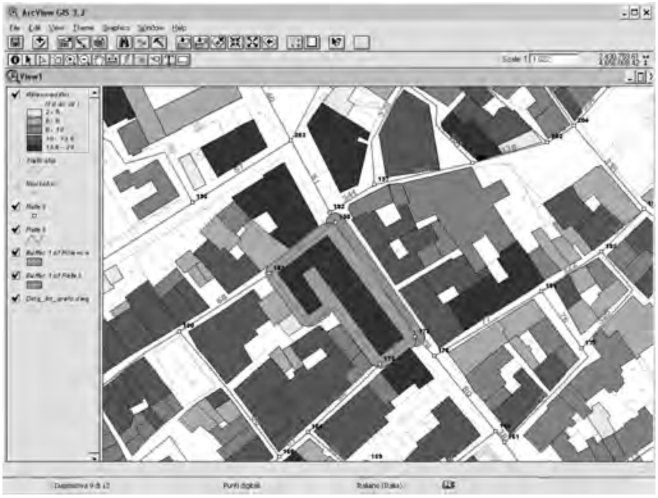

Figure 13. Interaction between building and roadway in the case of collapse.

A set of available databases were collected and analyzed by UNIBAS. Data contained in these databases were collected using either the 1st level GNDT90 inspection form for vulnerability and damage evaluation, or the AeDES survey form for usability and damage of buildings.
The main characteristics of these databases are reported in the following:

1) Potenza 1990. 41 municipalities were surveyed after the 1990 Potenza earthquake. In 21 municipalities, including Potenza, a systematic survey of all buildings was carried out. Vulnerability and damage data on about 50,000 masonry and RC buildings were collected, about 10,000 buildings in Potenza. The 1st level GNDT90 survey form was used.
2) Pollino 1998. 27 municipalities were surveyed after the 1998 Pollino (Southern Italy, Basilicata) earthquake. In 3 municipalities (Lauria, Rivello and Castelluccio Superiore) a systematic survey of all buildings was carried out. Vulnerability and damage data on about 20,000 masonry and RC buildings were collected. An old version of the AeDES survey form was used (1997 AeDES form).
3) Val D'Agri 2002. 9 municipalities were surveyed in Val D'Agri zone, within the framework of a project funded by the Basilicata Region. In all the municipalities (Viggiano, Tramutola, Spinoso, Paterno, Montemurro, Marsico Vetere, Marsico Nuovo, Moliterno, Grumento Nova) a systematic survey of all buildings was carried out. Only vulnerability data on about 11000 masonry and RC buildings were collected. An upgraded version of the AeDES survey form was used (2000 AeDES form).

Globally, building data for 77 municipalities are currently available (out of 131 municipalities globally present in the region) as shown in Figure 14.

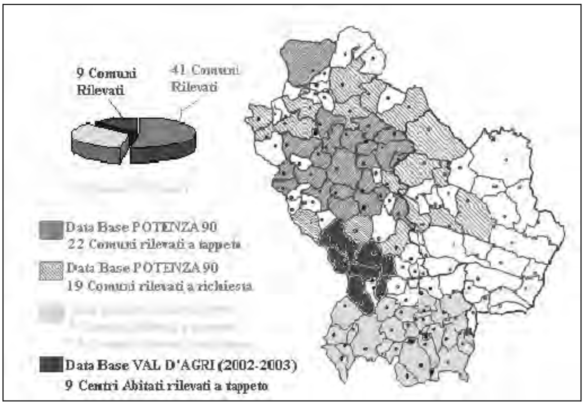

Figure 14. Databases available for the Basilicata Region.

Data already available were almost exclusively relevant to towns located in Potenza province. For this reason, during the 3rd year of the Project, some villages located in Matera province have been surveyed. Data have been collected using the Interview Protocol and an updated version of the AeDES survey form for usability and damage of buildings.

5 DISCUSSION

In the following there are reported the objectives declared at the starting of the projects, and the analysis of the deviations from plans.

5.1 Ordinary buildings

1) Inventory, characterization and vulnerability evaluation for ordinary buildings, finalized to the calibration of ISTAT data.
2) Improvement of the forms for damage survey in emergency, with definition of a damage index, and possibly of a vulnerability index, in order to establish correlations for risk analysis.
3) Development of innovative methods for vulnerability survey; for RC buildings, through aided acquisition and “expert” control of data.
4) Critical analysis of the methods for vulnerability evaluation.
5) Development of a mechanic method for the evaluation of seismic risk of populations of masonry and RC buildings; validation with other methods and with damage scenarios observed after seismic events.
6) Risk evaluation at the national scale, based on calibrated ISTAT data.
7) Definition of criteria for the evaluation of economic and social losses, based on the observation and/or estimation of physical damage.

All the objectives have been accomplished.

Moreover, the additional objectives have been accomplished:

8) Development of a multilevel method for the survey, vulnerability and risk analysis, damage scenarios at the regional and urban scales.
9) Development of a “geologic” form at levels I and II for the “Evaluation of local effects in the sites of individual buildings (ordinary, strategic and monumental buildings)” and definition of a seismic hazard index based on the data of the I level form.

10) Development of a 1st level form for the evaluation of masonry quality.

5.2 Public and strategic buildings

1) Systematization of the database of public buildings, data collection of the assessments finalized to an updated databases.
2) Improvement of the methods for the evaluation of the seismic vulnerability of RC and masonry buildings, based on a simplified model requiring a reduced data set, and on non linear static analysis. Introduction, for masonry buildings, of out-of-plane mechanisms.
3) Development of a method for the evaluation of the seismic vulnerability of mixed masonry-RC buildings, based on a simplified model, requiring a reduced data set, and on non linear static analysis

The objectives 1-2 have been accomplished.
In alternative to objective 3, the following objective has been accomplished:

4) Development of technical forms for the identification of local damage mechanisms and corresponding models.

5.3 Infrastructures

1) Improvement of the forms for the vulnerability census of networks and lifelines, finalized to the expeditious evaluation of seismic risk.
2) Definition of the basis for the development of a technological system for real-time evaluation: (a) of seismic risk and (b) of the reliability of a bridge stock with the aim, on one hand, to check in real-time the safety and the practicability after a seismic event, on the other, to define the priorities of intervention on the stock.

Objective 1 has not been accomplished.
Objective 2 has been accomplished.

5.4 Urban systems and historical centres

1) Widening of methods for expeditious evaluation of risk at the urban scale.
2) Improvement of forms for the survey of vulnerability of urban centres, finalized to the expeditious evaluation of seismic risk.
3) Development and improvement of methods for the evaluation of modifications of the infrastructural network as consequence of an event, finalized to single out the best route for rescue or evacuation.
4) Development and improvement of methods for planning the sites to be reserved to emergency logistic, to be preferably located in sites with high accessibility to zones at risk.
5) Evaluation of the seismic vulnerability of historical building types.

The objectives 1-3 and 5 have been accomplished.
Moreover, the additional objective has been accomplished:

6) Development and validation of a model, based on the analogy with neural networks, for the study of the response of urban systems under a given seismic action.

Objective 4 is included in the “neural” model, through the analysis of the transportation sub-system.

5.5 Monuments

1) Definition of forms for the survey of vulnerability and damage for types different form churches (palaces, castles, towers, etc.).

2) Widening of the methods for the evaluation of vulnerability and "value" of the different types of Cultural Heritage.

Objective 1 has been reached.
Objective 2 has been partially reached, in relation to the methods of vulnerability evaluation. In alternative, the following objective has been accomplished:

3) Development of protocols for provisional interventions.

5.6 Emergency planning and management

1) Development of methods for the evaluation of post-event scenarios at the regional and urban scales.
2) Development of methods and procedures for post-event planning and management.
3) Definition of "objective" criteria for usability evaluations, in order the reduce the subjective effects of the surveyor.
4) Development of methods and procedures for the design of provisional interventions (props, rings, etc.), based on damage mechanisms, also finalized to the organization of the operations in the post-event and to cost estimation.
5) Development of a database of constructive systems for emergency housing, finalized to typological, technological and structural classification, to the definition of the requirements of kits, and to cost evaluation.

All the objectives have been accomplished.
As regards objective 5, a Special Regulation has been drawn for emergency housing, which represents a deliverable more directly usable by DPC.

6 VISION AND DEVELOPMENTS

6.1 Ordinary buildings

The evaluation of vulnerability and risk of ordinary buildings, thanks to the multilevel methods for data collection and analysis developed in the project, allows to account for and combine data characterized by different level of detail. In the near future, it is possible to foresee applications based on these methods, both in data collection and in database organization. It is also possible to foresee, at the highest level, a further development of the analysis of damage mechanisms, which allows to correlate macroseismic models to mechanic models.
As regards the maps at the national scale, important updates have been accomplished, utilizing either the macroseismic or the mechanic method, and accounting for the new INGV-DPC hazard data. The main issues still present are relevant to the utilization of ISTAT 2001 data, to the inventory and characterization of the typologies. In particular, the utilization of the ISTAT 2001 data, disaggregated at the level of census tract, is possible only for some selected municipalities. This leads to calibration analyses, where disaggregate data are available from previous vulnerability/damage surveys. It is also suitable to state a definition of "building" coherent with that commonly adopted in the vulnerability analyses. As regards the inventory and characterization of typologies, significant advances have been accomplished, for both masonry buildings (studies on masonry types, analyses of aggregates), and RC buildings (fragility curves). Notwithstanding these advances, a further systematization of the present knowledge is desirable. To this end, multilevel methods may provide the conceptual framework for the organization of information, of different origin and detail level. It appears also suitable to focus on typologies whose behaviour is still less known, as mixed masonry-RC buildings, and reinforced masonry buildings.

6.2 *Public and strategic buildings*

As regards public and strategic buildings, the collection of data has been started on seismic assessment, carried out according to the OPCM 3274. The completion of data collection, and relevant analyses, are the logic continuation of this activity, together with the development of criteria to determine the intervention priorities.

6.3 *Infrastructures*

As regards the monitoring of infrastructures, and in particular bridges and viaducts, the project registered significant improvements in the development of methods and technologies for real-time identification of damage. The application of these to real cases appears a natural development.

At the same time, it is desirable to develop simplified methods for the evaluation of risk to infrastructural systems at the territorial scale.

6.4 *Urban systems and historical centres*

An innovative method has been developed for the evaluation of risk to urban systems and historical centres, based on the analogy with neural networks. This method provides a conceptual framework to account for the complex interactions between the different sub-systems of an urban system stricken by a given earthquake. It also accounts for the interventions of Civil Protection, allowing to take decisions on a rational basis. The method is integrated by a GIS which accounts for the interactions between the different sub-systems, based on spatial analysis. Application of the method to Sulmona has been carried out.

The developments of the method may regard:

a) improvement of quality and quickness of survey, eliminating some unnecessary data, and including additional information on urban resilience, economic and productive activities;
b) calibration and improvement of the model, developing the relations between sub-systems and relevant indirect losses;
c) involvement of Administrations, with links to regional projects, in order to acquire interview protocols and expert advices.

6.5 *Monuments*

The analysis of observed damage to monuments yielded damage probability matrices for several mechanisms. This results can usefully employed for the prediction of damage and the adoption of intervention, either provisional or definitive. It is necessary, in the near future, to evaluate the vulnerability of monuments subjected to interventions, which may be increased, in several cases, by erroneous interventions.

The analysis of topographic effects, carried out on churches, yielded significant results. It appears desirable, in the near future, to validate these results on ordinary, public and strategic buildings.

6.6 *Emergency planning and management*

As regards emergency planning, one of the results of the project is the Special Regulation for emergency housing, which represent a tool of undoubted interest for the DPC.

From the point of view of the provisional post-seismic interventions, improvements of the design of active props have been accomplished. It appears suitable that the recent innovations will be transferred to the practice, through the formation of technicians, the production and the preventive supplying of special components.

In the near future, it is desirable to start the study and the application of innovative systems for damage survey in emergency (e.g. based on satellite images), finalized to optimal rescue allocation.
It is also suitable finalizing loss analysis to the adoption of insurance cover by developing methods to estimate the maximum coverage corresponding to an individual event affecting a given building stock distributed over the territory.

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RESEARCH NEEDS IN EARTHQUAKE ENGINEERING HIGHLIGHTED BY THE 2009 L'AQUILA EARTHQUAKE

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1 THE 6TH APRIL 2009 L'AQUILA (ITALY) EARTHQUAKE

On April the 6th 2009, at 1.32 a.m. UTC, the city of L'Aquila and the surrounding Appennines areas comprising 80 municipalities located in one of the regions with the highest seismic hazard in central Italy, were shaken by a large shallow focal depth earthquake ground motion. Such motion initiated in the Earth outer crust, at a depth of about 10-12 kms. The focal mechanism was normal fault: the ground rupture moved upwards, or *up-dip* (towards the city centre of L'Aquila), and travelled from North-West to South-East, towards the Aterno valley. The magnitude of the main shock was estimated as M_L=5.8 by the Italian National Institute of Geophysics and Volcanology (INGV) and M_w = 6.3 by the United States Geological Survey (USGS). The 6th April earthquake was the strongest of a sequence of seismic events that initiated a few months earlier as shown pictorially in Figure 1, where the 23 earthquakes of M_w>4 occurred between 30/03/09 and 23/04/09, including an M_w 5.6 on 07/04 and an M_w 5.4 on 09/04, are reported.

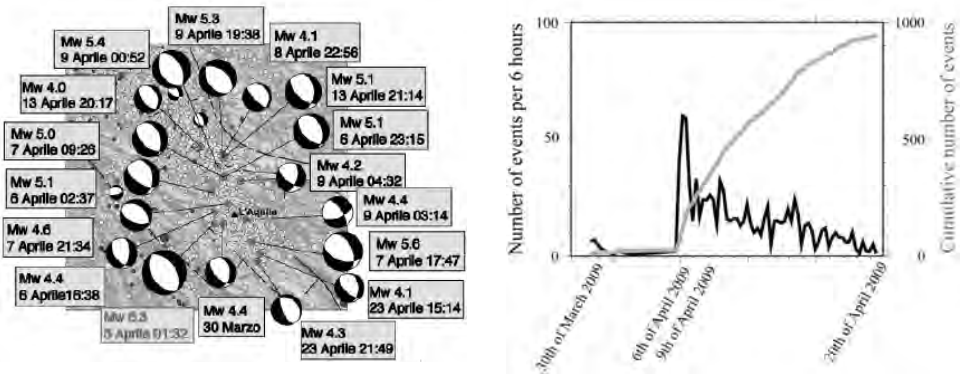

Figure 1 . Sequence of seismic events occurred in L'Aquila and surrounding municipalities (Abruzzo region) during April 2009 (*source http://www.ingv.it*).

The earthquake-affected zones were densely instrumented at the time of the event because of the high seismicity of Central Appennines Mountains; 57 of the approximately 300 modern digital strong-motion instruments of the National Accelerometer Network (RAN, Rete Accelerometrica Nazionale), managed by the Department Civil Protection (DCP), recorded the main shock (e.g. Cosenza *et al.*, 2009) and allowed the estimation of the peak ground acceleration (PGA) distribution as provided in Figure 2.

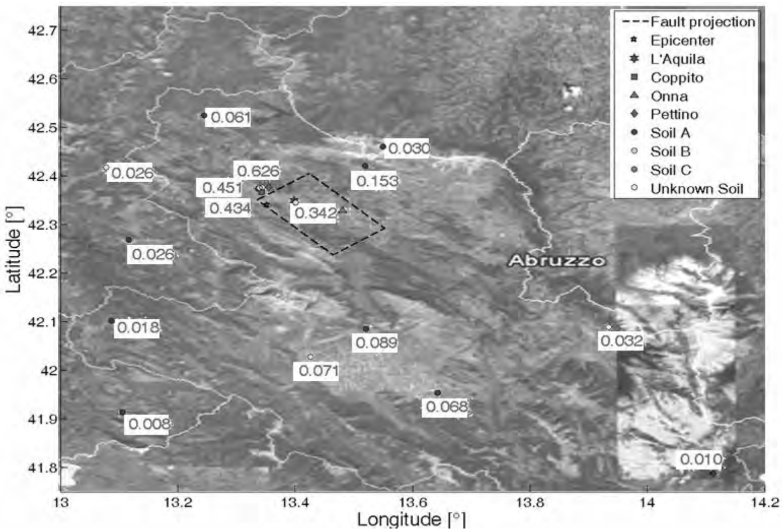

Figure 2 . Peak ground horizontal acceleration distribution in the suburbs of L'Aquila (*values are expressed in g*).

The computed values of horizontal PGAs exceeded 0.35 g in many stations, especially in the sites located in North of L'Aquila; the corresponding values of peak ground velocities varied between 24.50cm/sec and 26.68cm/sec and the observed maximum horizontal ground deformations were greater than 20cm, as confirmed by the interferometry provided by the satellite image in Figure 3. Local amplifications due to site effects were also observed in many locations, especially between the boroughs of Coppito and Pettino, in the North of the fault trace. Vertical component of ground motions may also have had significant ground and structural effects; such effects are still under investigations.
The human and financial losses caused by the 6th April 2009 earthquake are extremely significant. The main shock occurred at L'Aquila caused the death of more than 300 people, injured 1,500, destroyed or damaged an estimated 10,000-15,000 residential buildings, prompted the temporary evacuation of 70,000-80,000 residents, and left nearly 25,000 homeless. The majority of the visual vulnerability assessment of the built environment was carried out by the experts of ReLUIS consortium in collaboration with a DCP delegation. The surveyed damage was widespread to old and modern constructions. Several masonry buildings either collapsed or showed irreparable damage caused by the lacking of seismic conceptual design and detailing (*see* reconnaissance reports available on http://www.reluis.it). Historical, monumental, government buildings, basilicas and churches, especially in L'Aquila city centre, were destroyed by the seismic events occurred during April 2009. Reinforced concrete multi-storey buildings showed extensive damage chiefly to the non-structural components, such masonry infills and interior partitions (Figure 4).

Figure 3 . Interferometry of the region affected by the 6th April 2009 earthquake (*source http://www.ingv.it*).

Figure 4. Example of damage to masonry infills in RC multi-storey buildings.

Critical facilities were shut down primarily because of the lost of functionality caused the failure of non-structural components and building contents. Lifelines failed in all the areas affected by the earthquake ground motion. Damage at bearing and at connections were detected in many bridges and viaducts. Geotechnical and ground effects were also widespread; landslides caused the interruption of several main roads.

A quick emergency response was managed by the DCP in collaboration with several other government and non-government organizations, institutions and professionals. ReLUIS consortium has played a role of paramount importance in the post-earthquake emergency activities.

The 6th April 2009 L'Aquila earthquake has taught a number of lessons that are essential for seismic risk mitigation and preparedness. Although some of them are related to topics which were already addressed in the ReLUIS 2005-2008 project (see other papers in this same book), further aspects and crucial research needs emerged as far as earthquake engineering is concerned and they are briefly discussed in the following.

2 RESEARCH NEEDS IN EARTHQUAKE ENGINEERING

The recent 6th April 2009 L'Aquila (Italy) earthquake highlighted the need for deepening of research topics are still pending and they should be addressed in the forthcoming ReLUIS research program. There are numerous engineering seismology issues, non-structural component response and effects, loss assessment, critical buildings assessment and design for operability that require further thorough understanding and investigation (both experimentally and numerically) to mitigate the seismic risk at a societal level. The lessons learnt from the L'Aquila earthquake showed the need of focusing in the near future on the following research thrust areas illustrated hereafter. Reconnaissance reports relative to the L'Aquila earthquake (available on http://www.reluis.it) have also shown that geotechnical effects may be devastating and further research is need (Simonelli *et al*., 2009). Additionally, seismic vulnerability of large earth and reinforced concrete dams should considered because they failure and/or collapse may undermine the safety of large societal communities.

2.1 Near-source Effects

The 6th April L'Aquila (M_L=5.8) earthquake caused extensive damage in the triggered areas because of the nature of the fault rupture and measured ground displacements (Figure 5). Significant directivity effects were observed and high acceleration were estimated for both horizontal and vertical components of ground motions (Chioccarelli *et al*., 2009). Directivity of fault rupture, site effects, dispersion and incoherence may be of paramount importance for the seismic effects at near fault locations (Chioccarelli and Iervolino, 2009).

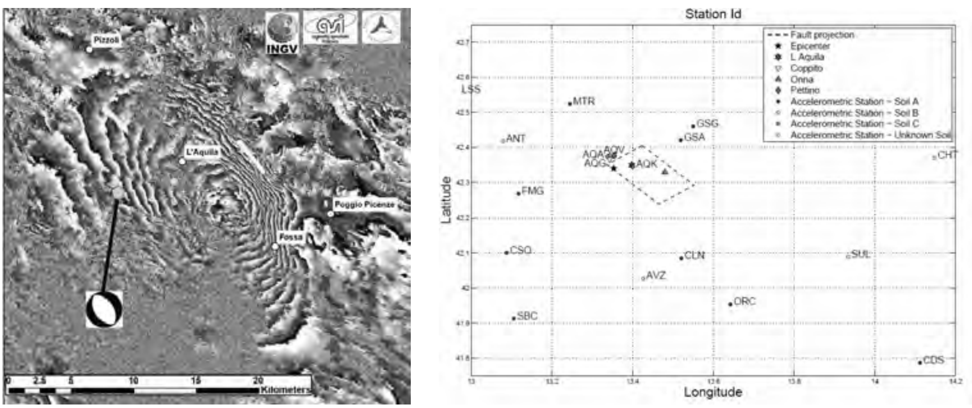

Figure 5. Directivity of the fault rupture: satellite interferometry (*left*) and geographical distribution of the main earthquake shocks.

Existing attenuation relationships should be re-assessed and compared with the results derived from the earthquake records. The effects of the vertical component of the earthquake motions on near-fault stiff structures, e.g. low-rise masonry dwellings, should be further investigated.

2.2 *Effects of non-structural components*

Numerous surveys carried out in the aftermath of the L'Aquila earthquake showed that the vast majority of the existing RC multi-storey buildings did not collapse. However, non-structural damage was extensive (Figure 6). Masonry infill panels failed primarily with out-of-plane mechanisms because of the weak connections between the interior and exterior walls (eg. Verderame *et al*., 2009). In several cases, the connections were absent and the brick walls possessed high slenderness. The effects of the masonry infills on the local and global stiffness, strength and ductility of the framed systems should be investigated by performing static and/or dynamic cyclic experimental tests. Analytical models should be calibrated on the basis of the test results and parameter analyses carried out to provide sound design for next generation seismic codes of practice. It is necessary that the masonry infills should be adequately designed and accounted for in the seismic analysis of building structures.

Figure 6. Damage to masonry infills in RC multi-storey framed buildings.

It is of vital importance to protect the occupants from the collapse of ceilings and other suspended components in ordinary buildings, schools, fitness and computer centre. The functionality of several buildings was impaired by the failure of architectural features that are typically used for building structures (Figure 7). The response of such components should be further assessed experimentally and design rules provided for the reliable sizing of their connections to the supporting structure.

Figure 7. Failure of ceiling panels.

Handbooks with comprehensive illustrative examples should be issued for the practical use by architects and interior design. Detailed design rules for non-structural components should be formulated and incorporated in the next generation seismic design codes; similarly for the equipments and building contents.

2.3 Critical facilities: hospitals

Hospital buildings are critical facilities that should remain operational in the aftermath of moderate-to-high magnitude earthquakes. They are essentials for all the injured people that need medical care and assistance. The performance of all major hospitals located nationwide is undermined by the failure of building contents and non-structural components (Magliulo *et al*., 2009) under minor-to-moderate earthquakes (Figure 8). Moreover, traditionally based hospital buildings, i.e. even those fulfilling the capacity-design rules, exhibit structural damage when subjected to high-magnitude earthquakes. The use of behaviour (or response modification) factors greater than unity leads to the occurrence of structural damage under moderate-to-large earthquakes.

Figure 8. Damage observed in the Saint Salvador Hospital, L'Aquila.

It is thought that enlightened policy makers, health managers and all stakeholders should rethink the design of hospital buildings and other health care facilities. The functionality of such buildings is of paramount importance for societal communities in the aftermath of an earthquake. Adequate design rules should, therefore, be formulated on the basis of the observed damage and interruption of functionality. Innovative technologies and design should

be promoted to enhance the seismic performance of non-structural components, building contents and medical and high-tech equipments which are seriously damaged during earthquakes.
Handbooks with practical guidelines for the design of non-structural components, fixing and seismic detailing should be issued to help architects and engineers to design safe hospital buildings.

2.4 Industrial plants

The assessment of the seismic risk of industrial plants is a key issue to protect communities from earthquake-induced catastrophes. Chemical power plants located in or near urban centres should be protected adequately and cost-effective retrofitting strategies proposed for those thanks that do not possess sufficient resilience (Figure 9). In so doing innovative technologies may be employed. However, the assessment of the seismic vulnerability of the existing industrial plants should be investigated and prioritization scheduled according to the available funds. Vulnerability functions should be formulated for different types of industrial plan facilities. The limit states for adequate seismic performance should be established.

Figure 9. Damage of metal tanks between the municipality of Onna and L'Aquila.

Simplified methods of analyses should also be implemented in codes of practice to allow a quick estimate the safety of the existing industrial facilities.

2.5 Lifelines

The existing national lifeline systems are highly vulnerable to earthquake ground motions. The assessment of the as-built systems is an essential step to estimate the effects of the rupture of gas pipelines, sewage, electrical and telecommunication networks on social communities. Table 1 provides the statistics of a recent survey estimating the Italian major lifelines. It is noted that the dimensions of the networks are a few thousands kilometres (from 4.342 for the oil pipelines to 21.872 of the Terna electrical power network).
The transportation systems (highways with their 6.544kms and railways with their 16.295kms) exhibit high seismic risk. Methods and types of retrofitting may be analysed on a national scale. Comparative interventions schemes may be performed to provide cost-effective solutions to the transportation managers. Early warning systems and methodologies may be employed to ensure the safety of the hazardous lifelines, such as oil and gas pipelines, for electrical power networks and high-speed railways.

Table 1. Statistics of the Italian major lifelines (Source ISAT – Terna).

Lifeline (type)	*Dimension (in kms)*	*Owner*
Electrical power network	21.872	Terna (since 2006)
Oil pipelines	4.342	Ministry of Infrastructure and Transportation (since 2008)
Gas pipelines	8.479 (main pipelines) 22.410 (secondary pipelines, only ENI network)	
Highways	6.544	
Railways	16.295	

2.6 Small historical centres

The 6th April L'Aquila earthquake demonstrated the weakness of the existing historical buildings, which represent the living legacy of a glorious past. A number of damage reports showed that the historical city centres of L'Aquila, Pettino and Onna were severely affected by structural collapses. There are, however, numerous small historical centres (*see* for example Figure 10), generally located on hilly places or in the suburbs of large urban areas. The analysis of seismic risk of such centres and its mitigation is an essential task that is required to ensure safety to the local populations and to preserve the existent cultural heritage. The evaluation of the seismic risk may be carried out initially at a macro-scale level and then considering the building aggregates and single units of the centres.

Figure 10. Aerial view of the small historical centre of Tempera.

In addition post-earthquake evacuation plans should be assessed on the basis of the easy of access to the sites and the surrounding transportation networks.

2.7 Cultural heritage and monumental buildings

The death of nearly 300 people was a great human loss caused by the 2009 L'Aquila earthquake. However, earthquake damage resulting in the collapse of monuments, historical palaces and places of worship and stately buildings (*see* for example Figure 12) represents an irreplaceable loss in terms of cultural heritage, while their restoration costs is a serious threat of the gross national product. Vulnerability assessment of the cultural heritage and monumental buildings is a fundamental task than the earthquake engineering community should consider as an urgent need. Sound guidelines should be formulated using the experimental data and numerical results achieved in the 2005-2008 ReLUIS-DCP research projects. Additional experimental and numerical work is deemed necessary to enhance the reliability of the structural models, the definition of appropriate limit sates, deformation and strength thresholds for structural components and systems. Health monitoring of landmark structures may be adopted as a viable solution to monitor and prevent the occurrence of structural damage during minor to moderate earthquakes. Retrofitting strategies should be formulated in compliance with the requirements prescribed by the conservation thrusts.

Figure 11. Typical collapse of the dome of the L'Aquila Duomo.

Post-earthquake first aid interventions should be assessed and practical rules formulated for practical applications. In so doing, the reversability of the interventions should be adequately accounted for.

2.8 Emergency management

Post-earthquake emergency management is a crucial task that government and non-government institutions and organizations which are in charge of the civil protection need to assess responsibly. The technical support provided by the ReLUIS researchers in the emergency management in the aftermath of the 6th April L'Aquila earthquake showed the effectiveness of a team working. The ReLUIS researchers worked closely with the DCP delegates to coordinate the damage surveys in the areas affected by the earthquakes. Expert groups were formed and visited thousands of buildings, bridges, industrial sites and lifelines. The damage surveys were crucial to establish the safety of structures subjected to the earthquake. The structures were tagged according to the level of observed structural and non-structural damage (Figure 12). The technical checks were based on visual inspections of experts. Databases were compiled using the data collected on site for the inspected structures. It is thought that more efficient intervention plans may be assessed. In so doing, the format of the existing damage collection data should be improved. A further critical issue that should be developed in the future is the temporary housing. Urban and architectural planning, structural types, material of constructions, constructability, maintenance and re-use of temporary houses should be assessed.

Figure 12. Visualization of the damage scenario of large urban centres.

2.9 Quick damage evaluation

The evaluation of the damage caused by earthquakes is of paramount importance for the scheduling of the retrofitting or the reconstruction of the structures. This approach was experimented during the 2009 L'Aquila earthquake with regards to the cultural and historical buildings. For such buildings cost estimates were carried out using an approximate approach that allows the evaluation of the total sum needed for the repair of a single structure. The repair costs was estimated chiefly for churches, palaces and monuments. The cost was derived by the interventions needed to repair the failure caused by the earthquake. It is essential to extend the quick estimation of the cost of the damage repair to other constructions. Back-analyses are also deemed necessary to establish the soundness of the approach adopted in the estimations. Parameter analyses may be carried out to assess the cost-effectiveness of alternative retrofitting schemes.

2.10 Safety and sustainability

Structural safety is essential for building occupants but is should be achieved using sustainable materials and technologies. The reconstruction initiated after the 2009 L'Aquila earthquake was based on green approaches. The project C.A.S.E. (Complessi Antisismici Sostenibili Eco-Compatibili, Seismic Resistant Blocks Sustainable Eco-Friendly) which is financially supported by the Italian Government is aimed at providing 150 residential buildings to homeless people. The buildings employ base isolation systems and are built with green technologies in compliance with the most recent environmental regulations. The conceptual approach of green buildings (Figure 13) may be further developed and application is seismic zones promoted.

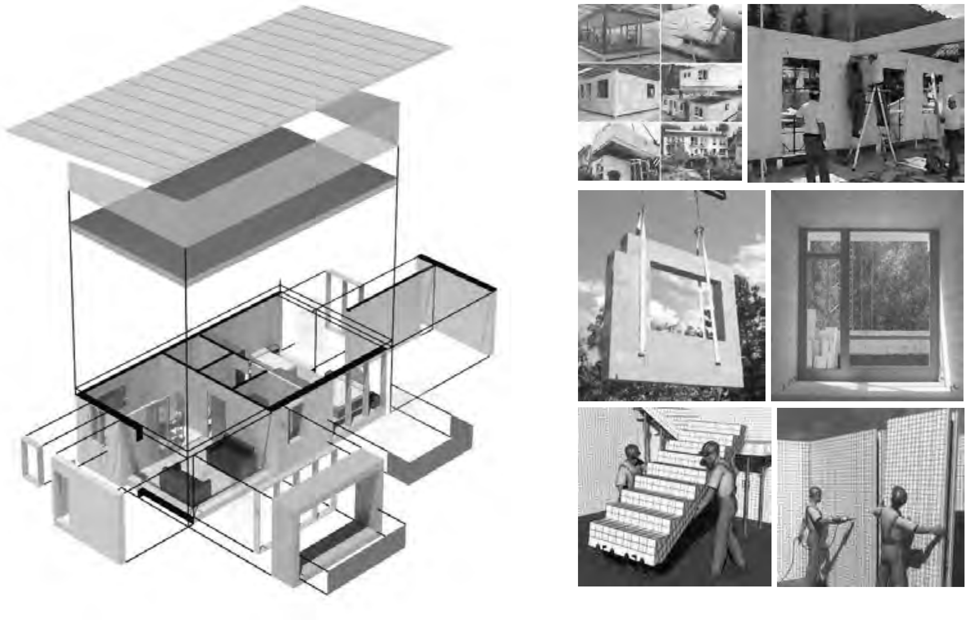

Figure 13. Safe "green" building structure.

The environmental efficiency should be used along with the cost-effectiveness of prefabricated modular type building systems which may minimize the construction time and hence are optimal solution particularly in the post-earthquake reconstruction.

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