OPERATIONAL METHODS FOR THE ASSESSMENT OF EXISTING STRUCTURES

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FOREWORD

The Leonardo da Vinci Project Pilot Project CZ/11/LLP-LdV/TOI/134005: "*Vocational Training on Assessment of Existing Structures*" addresses the urgent need to train students, young engineers and professionals in the assessment of existing structures. The future of the construction industry is namely moving from new constructions towards maintenance, repair and rehabilitation of existing structures. The safety assessment of existing structure plays thereby an important role.

Assessment of existing structures is an urgent issue of a great economic significance in most countries around the world as more than 50% of all construction activities concern existing buildings, bridges and other civil engineering works. Presently the Eurocodes which will be used in all CEN Member countries are primarily focused on the design of new structures. Additional operational rules for existing structures are still missing. The international standard ISO 13822 provides only general principles for the assessment of existing structures which should be further developed for their effective operational use in practice.

The current project addresses the importance for implementing principles of the assessment and verification of existing structures in practice in the Czech Republic and other partner countries. The project is supported by the Czech Chamber of Chartered Engineers (ČKAIT). The project consortium, under the leadership of the Klokner Institute of the Czech Technical University in Prague (KI CTU), consists of the Secondary Technical School of Civil Engineering (CZ) and the research institutions and universities from four EU Member States (DE, ES, IT, NL) and one associated country (TR). All researchers of the partnership are involved in research projects dealing with reliability assessment of existing structures. They participate in the national and international standardization activities within organizations CEN and ISO.

The project outcomes include vocational training materials based on documents of the international research organization Joint Committee on Structural Safety JCSS and international research projects, selected outcomes of the previous project of the Leonardo da Vinci programme (developed by 5 partners of the present consortium in 2008-2010) and also on background documents to new European and international standards.

A basic project outcome is the already published Handbook 1 "*Innovative Methods for the Assessment of Existing Structures*" which focuses on methodologies to assess and evaluate the condition of existing structures. The methodologies provided are independent from type of structure and material and are compatible to the background methodologies used in the Eurocodes. Practical techniques for the assessment of existing structures and associated case studies based on the methodologies of Handbook 1 are presented in this Handbook 2 of this project entitled "*Operational Techniques for the Assessment of Existing Structures*".

Handbook 2 consists of ten chapters. Chapter 1 gives a brief overview of the basic concepts and important terminology used in the assessment of existing structures. Chapter 2 summarizes currently used techniques including non-destructive testing, inspection and monitoring and provides quantitative information for their implementation. Chapter 3 reviews the basic aspects of structural modelling and analysis and discusses them in an application dealing with a masonry building; thereby an equivalent frame model, a finite element model and a kinematic model are compared. Chapter 4 describes the verification procedure compatible to the Eurocodes by using updated partial factors and design values and based on the methodological aspects discussed in Handbook 1. Practical applications of updating are then shown in Chapter 5. They deal with the reinforcement steel strength, with proof load and with updating of earthquake action parameters. Chapter 6 deals with concrete structures; it summarizes first practical codes, standards and recommendations. It also includes two case studies dealing a) with strengthening of concrete columns and b) with construction of additional storeys i.e. with implementation of additional loading in an existing concrete building.

Assessment of existing metal structures is presented in Chapter 7. General aspects are provided and illustrated in the reassessment of an 115-year old truss bridge. The implementation of operational techniques in the assessment and rehabilitation of existing timber structures is shown in Chapter 8. The degradation of timber properties is discussed and illustrated in a characteristic case study of a building in Lucca (Italy). Particular attention is paid to heritage buildings which are treated in a separate chapter namely Chapter 9. Basic aspects of investigations techniques are provided. Three case studies of heritage structures in Italy are analysed in detail a) the rehabilitation of a historical sanctuary near L'Aquila, b) the requalification of an old bell tower in Pisa and c) the repair of a masonry arch bridge in Carrara. Finally Chapter 10 addresses the important issue of existing structures in seismic zones. The seismic retrofit of the structures is summarised, basic techniques are presented together with their implementation in two case studies concerning the strengthening of a r.c. school in Denizli (Turkey) and the repair of an earthquake damaged r.c. residential building in Molise (Italy).

It is believed that the material of this Handbook 2 is presented in an understandable way for the practicing engineers reflecting the experience of the authors and supported with many case studies described herein in detail. The methodologies presented in Handbook 1 are referred to and in addition many references are provided for background material and further study.

Pisa and Prague 2013

CONTENTS

INTRODUCTION

Assessment of existing structures is in many aspects different from that taken in designing a new structure. The effects of the construction process and subsequent life of the structure, during which it may have undergone alteration, deterioration, misuse, and other changes to its as-built (as-designed) state, must be taken into account. In general actual variation in the basic variables describing actions, material properties, geometric data and model uncertainties should be taken into account. Taking into account these documents the main principles for assessment of existing structures may be summarized as follows:

- available scientific knowledge and know-how including currently valid codes should be applied; historical practice and provisions valid when the structure was built (designed), should be used as guidance information only, especially to simulate the original design process;
- actual characteristics of structural material, action, geometric data and structural behaviour should be considered; the original documentation including drawing should be used as guidance material only.

The most important step of the whole assessment procedure of existing structures including evaluation of inspection data and updating of prior information concerning strength and structural reliability, described in detail in Handbook 1, are summarised in this Handbook in a condensed and operational form.

Beside technical considerations, normative provisions for the assessment of existing structures should take into account:

- the reasons why the assessment of the existing building is necessary;
- the possibility that in some Country safety and reliability levels of existing structures are compulsorily fixed by the law, depending on the intervention.

Concerning the motivations for reassessing the structure, in principle, it is crucial to distinguish two relevant cases:

- case nr. 1: where the re-assessment of the existing structure is dictated essentially by repair needs, as the structural scheme and the vertical actions on it remain substantially unchanged;

- case nr. 2: where the existing structures is modified to such an extent that its behaviour and the required performances are deeply toggled, like it happens when:

  - the structure is superelevated;
  - the structure is enlarged in such an extent that the structural behaviour is modified;
  - permanent and/or imposed loads are significantly increased, for example in consequence of variation of building category or addition of intermediate floors;
  - in consequence of the interventions the structural scheme is significantly changed, so that the structural response of the original structure is substantially different from that resulting after the completion of retrofit works;
  - systems for seismic isolation are adopted.

The above mentioned distinction is very relevant, because variable and seismic actions given in modern Codes could result incompatible with the actual structural performance and different intervention strategies can be envisaged, depending on the actual case classification.

In case nr. 1, the retrofit should be addressed to restore or improve the original reliability of the structure, especially under horizontal loads (wind, earthquake etc.): depending on the particular situation, it is could be also necessary to reduce or limit vertical loads, imposing limitations in the use of the structure itself, like variation of building category and so on. This is a typical situation when considering historical buildings or bridges, where the needs of preservation of historical heritage supersede the structural requirements.

In case nr. 2, instead, the existing structure is deeply modified and in principle the structural performance required in the final situation are more severe than those required before the intervention. For this reason, the reliability to be required to the "new" resulting structure are very similar to those required for a new construction, since the possibility to assure a suitable reliability level after the retrofit is one of the premise and a key issue in the evaluation of the feasibility of the planned intervention. In comparison of new constructions, in this case a reduction of the required target reliability level could be envisaged in some case, but this possibility should be carefully evaluated and justified in the framework of a global engineering judgement.

Particular attention must be paid in Countries where the reliability levels to be achieved with the retrofit are mandatory and ruled by the law. In this case the type of intervention should be preliminarily classified in order to determine if, according to the compulsory prescriptions, the design strategy is to improve simply the actual reliability level or to achieve a well-defined target reliability.

On the base of the aforementioned considerations, it must be underlined that the techniques for the assessment of existing structural as well as the case studies discussed in the present Handbook must be duly interpreted and/or modified according to the different situation that can occur in design practice.

CHAPTER 1: BASIC CONCEPTS AND TERMINOLOGY

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1 INTRODUCTION

1.1 Background documents

Three International Standards ISO 2394 [1], ISO 13822 [2] and ISO 12491 [3], related to the assessment of existing structures, have been recently developed. Moreover, ISO 13822 [2] contains an annex focused on heritage structures. Additional information may be found in a number of scientific papers and publications, for example in [4], [5] and [6]. Examples of practical procedures and technique are presented in recent papers [7] and [8].

1.2 General principles

Assessment of existing structures is becoming a more and more important and frequent engineering task. Continued use of existing structures is of a great significance due to environmental, economic and socio-political assets, growing larger every year. These aspects are particularly relevant to heritage buildings that always constitute a great historical, social and economic value.

General principles of sustainable development regularly lead to the need for extension of the life of a structure, in majority of practical cases in conjunction with severe economic constraints. That is why assessment of existing structures often requires application of sophisticated methods, as a rule beyond the scope of traditional design codes. Nevertheless, apart from few national codes, three International Standards ISO 2394 [1], ISO/CD 13822 [2] and ISO 12491 [3], related to assessment of existing structures, have been recently developed.

The approach to the assessment of existing structures is in many aspects different from that taken in designing the structure of a newly proposed building. The effects of the construction process and subsequent life of the structure, during which it may have undergone alteration, deterioration, misuse, and other changes to its as-built (as-designed) state, must be taken into account.

However, even though the existing structure may be investigated several times, some uncertainty in the basic variables and structural behaviour shall always remain. Therefore, similarly as in design of new structures, actual variation in the basic variables describing actions, material properties, geometric data and model uncertainties are taken into account by partial factors or other code provisions.

In general, an existing structure may be subjected to the assessment of its actual reliability in case of:

– rehabilitation during which new structural members are added to the existing load-carrying system;
– adequacy checking in order to establish whether the existing structure can resist loads associated with the anticipated change in use of the facility, operational changes or extension of its design working life;

- repair of a building, which has deteriorated due to time dependent environmental effects or which has suffered damage from accidental actions, for example, earthquake;
- doubts concerning actual reliability of the structure.

In some circumstances assessments may also be required by authorities, insurance companies or owners or may be demanded by a maintenance plan.

2 PRINCIPLES OF ASSESSMENT

Two main principles are usually accepted when assessing existing buildings:

- currently valid codes for verification of structural reliability should be applied, codes valid in the period when the structure was designed should be used only as guidance documents.
- actual (estimated) characteristics of structural materials, actions, geometric data and structural behaviour should be considered, the original design documentation including drawings should be used as guidance documents only.

The first principle should be applied in order to achieve similar reliability level as in case of newly designed structures, taking only account of economic aspects as indicated below. The second principle should avoid negligence of any structural condition that may affect actual reliability (in favourable or unfavourable way) of a given structure.

Most of the current codes are developed assuming the concept of limit states in conjunction with the partial factor method. In accordance with this method, which is mostly considered here, basic variables are specified by characteristic or representative values. The design values of the basic variables are determined on the basis of the characteristic (representative) values and appropriate partial factors.

It follows from the second principle that a visual inspection of the assessed structure should be made whenever possible. Practical experience shows that inspection of the site is also useful to obtain a good feel for actual situation and state of the structure.

As a rule the assessment need not to be performed for those parts of the structure that will not be affected by structural changes, rehabilitation, repair, change in use or which are not obviously damaged or are not suspected of having insufficient reliability [2].

In general, the assessment procedure consists of the following steps (see the flow chart in [2]):

- specification of the assessment objectives required by the client or authority;
- scenarios related to structural conditions and actions;
- preliminary assessment:
  - study of available documentation;
  - preliminary inspection;
  - preliminary checks;
  - decision on immediate actions;
  - recommendation for detailed assessment;

- detailed assessment:
  - detailed documentary search;
  - detailed inspection;

  - material testing and determination of actions;
  - determination of structural properties;
  - structural analysis;
  - verification of structural reliability;

- report including proposal for construction intervention;
- repeat the sequence if necessary.

When the preliminary assessment indicates that the structure is reliable for its intended use over the remaining life a detailed assessment may not be required. Conversely if the structure seems to be in dangerous or uncertain condition immediate interventions and detailed assessment may be necessary.

3 INVESTIGATION

Investigation of an existing structure is intended to verify and update the knowledge about the present condition (state) of a structure with respect to a number of aspects. Often, the first impression of the structural condition will be based on visual qualitative investigation. The description of possible damage of the structure may be presented in verbal terms like: 'unknown, none, minor, moderate, severe, destructive'. Very often the decision based on such an observation will be made by experts in a purely intuitive way.

A better judgement of the structural condition can be made on the basis of (subsequent) quantitative inspections. Typically, the assessment is a cyclic process when the first inspection is supplemented by subsequent investigations. The purpose of the subsequent investigations is to obtain a better feel for the actual structural condition (particularly in the case of damage) and to verify information required for determination of the characteristic and representative values of all basic variables. For all inspection techniques, information on the probability of detecting damages if present, and the accuracy of the results should be given.

The statement from the investigation contains, as a rule, the following data describing

- actual state of the structure;
- types of structural materials and soils;
- observed damages;
- actions including environmental effects;
- available design documentation.

Proof loading is a special type of investigation. Based on such tests one may draw conclusions with respect to:

- the bearing capacity of the tested member under the test load condition;
- other members;
- other load conditions;
- the behaviour of the system.

The inference in the first case is relatively easy; the probability density function of the load bearing capacity is simply cut off at the value of the proof load. The inference from the other conclusions is more complex. Note that the number of proof load tests needs not to be restricted to one. Proof testing may concern one element under various loading conditions and/or a sample of structural elements. In order to avoid an unnecessary damage to the structure due to the proof load, it is recommended to increase the load gradually and to measure the deformations. Measurements may also give a better insight into the behaviour of the system. In general proof loads can address long-term or time-dependent effects. These effects should be compensated by calculation.

4 BASIC VARIABLES

In accordance with the above-mentioned general principles, characteristic and representative values of all basic variables shall be determined taking into account the actual situation and state of the structure. Available design documentation is used as a guidance material only. Actual state of the structure should be verified by its inspection to an adequate extent. If appropriate, destructive or non-destructive inspections should be performed and evaluated using statistical methods.

For verification of the structural reliability using the partial factor method, the characteristic and representative values of basic variables shall be considered as follows:

(a) dimensions of the structural elements shall be determined on the basis of adequate measurements. However, when the original design documentation is available and no significant changes in dimensions have taken place, the nominal dimensions given in the documentation may be used in the analysis;
(b) load characteristics shall be introduced with the values corresponding with the actual situation verified by destructive or non-destructive inspections. When some loads have been reduced or removed completely, the representative values of these loads (actions) can be reduced or appropriate partial factors can be adjusted. When overloading has been observed in the past it may be appropriate to increase adequately representative values;
(c) material properties shall be considered according to the actual state of the structure verified by destructive or non-destructive inspections. When the original design documentation is available and no serious deterioration, design errors or construction errors are suspected, the characteristic values given in original design may be used;
(d) model uncertainties shall be considered in the same way as in design stage unless previous structural behaviour (especially damage) indicates otherwise. In some cases model factors, coefficients and other design assumptions may be established from measurements on the existing structure (e.g. wind pressure coefficient, effective width values, etc.).

Thus the reliability verification should be backed up by inspection of the structure including collection of appropriate data. Evaluation of prior information and its updating using newly obtained measurements is one of the most important steps of the assessment.

5 EVALUATION OF INSPECTION RESULTS

Using results of an investigation (qualitative inspection, calculations, quantitative inspection, proof loading) the properties and reliability estimates of the structure may be updated. Two different procedures can be distinguished:

(a) updating of the structural failure probability;
(b) updating of the probability distributions of basic variables.

Direct updating of the structural reliability (procedure (a)) can be formally carried out using the following basic formula of the probability theory:

$$P(F|I) = \frac{P(F \cap I)}{P(I)} \tag{1}$$

where P denotes probability, F local or global failure, I inspection information, and $\cap$ intersection of two events. The inspection information I may consist of the observation that the crack width at the beam B is smaller than at the beam A. An example of probability updating using equation (1) is presented e.g. in [6].

The updating procedure of a univariate or multivariate probability distribution (procedure (b)) is given formally as:

$$f_X(x|I) = C \cdot P(x|I) \cdot f_X(x) \tag{2}$$

where $f_X(x|I)$ denotes the updated probability density function of X, $f_X(x)$ denotes the probability density function of X before updating, X a basic variable or statistical parameter, I inspection information, C normalising constant, and $P(I|x)$ likelihood function.

An illustration of equation (2) is presented in figure 1. In this example updating leads to a more favourable distribution with a greater design value x_d than the prior design value x_d. In general, however, the updated distribution might be also less favourable than the prior distribution.

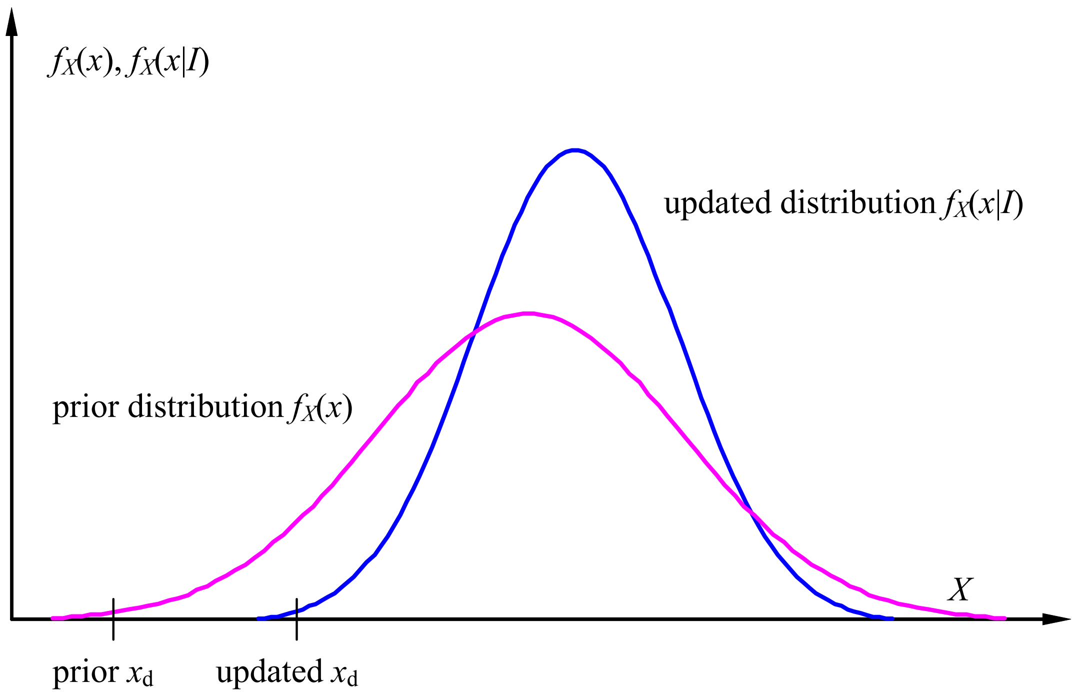

Figure 1: Updating of probability density function for an expected variable *X*

The updating procedure can be used to derive updated characteristic and representative values (fractiles of appropriate distributions) of basic variables to be used in the partial factor method or to compare directly action effects with limit values (cracks, displacements). More information on updating may be found in ISO 12491 [3].

Once the updated distributions for the basic variables $f_X(x)$ have been found, the updated failure probability $P(F|I)$ may be determined by performing a probabilistic analysis using common method of structural reliability for new structures. Symbolically it can be written

$$P(F|I) = \int_{g(X)<0} f_X(x|I)\,dx \tag{3}$$

where $f_X(x|I)$ denotes the updated probability density function and $g(x) < 0$ denotes the failure domain ($g(x)$ being the limit state function). It should be proved that the probability $P(F|I)$, given the design values for its basic variables, does not exceed a specified target value.

A more practical procedure is to determine updated design values for each basic variable (procedure (b)) that is discussed in Chapter 5.

6 STRUCTURAL ANALYSIS

Structural behaviour should be analysed using models that describe actual situation and state of an existing structure. Generally the structure should be analysed for ultimate limit states and serviceability limit states using basic variables and taking into account relevant deterioration processes.

All basic variables describing actions, material properties, load and model uncertainties should be considered as mentioned above. The uncertainty associated with the validity and accuracy of the models should be considered during assessment, either by adopting appropriate factors in deterministic verifications or by introducing probabilistic model factors in reliability analysis.

When a structure is analysed, conversion factors reflecting the influence of shape and size effect of specimens, temperature, moisture, duration-of-load effect, etc., should be taken into account. The level of knowledge about the condition of components should be also considered. This can be achieved by adjusting the assumed variability in either the load carrying capacity of the components or the dimensions of their cross sections, depending on the type of structure.

When deterioration is observed, the relevant mechanisms shall be identified and a deterioration model predicting the future performance of the structure shall be determined on the basis of theoretical or experimental investigation, inspection, and experience.

7 VERIFICATION

Reliability verification of an existing building shall be made using valid codes of practice, as a rule based on the limit state concept. Attention should be paid to both the ultimate and serviceability limit states. Verification may be carried out using partial safety factor or structural reliability methods with consideration of structural system and ductility of components. The reliability assessment shall be made taking into account the remaining working life of a structure, the reference period, and changes in the environment of a structure associated with an anticipated change in use.

The conclusion from the assessment shall withstand a plausibility check. In particular, discrepancies between the results of structural analysis (e.g. insufficient safety) and the real structural condition (e.g. no sign of distress or failure, satisfactory structural performance) must be explained. It should be kept in mind that many engineering models are conservative and cannot be always used directly to explain an actual situation.

The target reliability level used for verification can be taken as the level of reliability implied by acceptance criteria defined in proved and accepted design codes. The target reliability level shall be stated together with clearly defined limit state functions and specific models of the basic variables.

The target reliability level can also be established taking into account the required performance level for the structure, the reference period and possible failure consequences. In accordance with ISO 2394 [1], the performance requirements for assessment of existing structures are the same as for design of a new structure. Lower reliability targets for existing structures may be used if they can be justified on the basis of economical, social and sustainable consideration (see Annex F to ISO/CD 13822 [2] and numerical example in [8]).

An adequate value of the reliability index β should be in general determined [2] considering appropriate reference period. For serviceability and fatigue the reference period equals the remaining working life, while for the ultimate limit states the reference period is in principle the same as the design working life specified for new structures (50 years for buildings). This general approach should be in specific cases supplemented by detailed consideration of the character of serviceability limit states (reversible, irreversible), fatigue (controllable, incontrollable) and consequences of ultimate limit states (economic consequences, number of endangered people, loss of the cultural heritage value).

8 ASSESSMENT IN THE CASE OF DAMAGE

For an assessment of a damaged structure the following stepwise procedure is recommended:

1) *Visual inspection*

It is always useful to make an initial visual inspection of the structure to get a feel for its condition. Major defects should be reasonably evident to the experienced eye. In the case of very severe damage, immediate measures (like abandonment of the structure) may be taken.

2) *Explanation of observed phenomena*

In order to be able to understand the present condition of the structure, one should simulate the damage or the observed behaviour, using a model of the structure and the estimated intensity of various loads or physical/chemical agencies. It is important to have available documentation with respect to design, analysis and construction. If there is a discrepancy between calculations and observations, it might be worthwhile to look for design errors, errors in construction, etc.

3) *Reliability assessment*

Given the structure in its present state and given the present information, the reliability of the structure is estimated, either by means of a failure probability or by means of partial factors. Note that the model (structural analysis) of the present structure may be different from the original model. If the reliability is sufficient (i.e. better than commonly accepted in design) one might be satisfied and no further action is required.

4) *Additional information*

If the reliability according to step 3 is insufficient, one may look for additional information from more advanced structural models, additional inspections and measurements or actual load assessment.

5) *Final decision*

If the degree of reliability is still too low, one might decide to:

- accept the present situation for economical reasons;

- reduce the load on the structure;
- repair the building;
- start demolition of the structure.

The first decision may be motivated by the fact that the cost for additional reliability is much higher for existing structure than for a new structure. This argument is sometimes used by those who claim that a higher reliability should be generally required for a new structure than for an existing one. However, if human safety is involved, economical optimisation has a limited significance.

9 FINAL REPORT AND DECISION

The final report on structural assessment and possible interim reports (if required) should include clear conclusions with regard to the objective of the assessment based on careful reliability assessment and cost of repair or upgrading. The report shall be concise and clear. A recommended report format is indicated in Annex G to ISO/CD 13822 [2].

If the reliability of a structure is sufficient, no action is required. If an assessment shows that the reliability of a structure is insufficient, appropriate interventions should be proposed. Temporary intervention may be recommended and proposed by the engineer if required immediately. The engineer should indicate a preferred solution as a logical follow-up to the whole assessment in every case.

It should be noted that the client in collaboration with the relevant authority should make the final decision on possible interventions, based on engineering assessment and recommendations. The engineer performing the assessment might have, however, the legal duty to inform the relevant authority if the client does not respond in a reasonable time.

In the case of heritage structures minimisation of construction interventions is required in rehabilitation and upgrades, but sufficient reliability should also be guaranteed. When dealing with the preservation of heritage buildings, it may be difficult to propose construction interventions that respect all requirements for preservation of the heritage value. Modern principles of interventions seem to include the following aspects:

- Unobtrusiveness and respect of the original conception,
- Safety of the construction,
- Durability of materials,
- Balance between costs and available financial resources,

and in some cases also:

- Removability,
- Compatibility of materials,
- Indoor environment quality including aspects of comfort, security and accessibility.

10 CONCLUDING REMARKS

The main principles for assessment of existing structures are:

- Currently valid codes for verification of structural reliability should be applied, codes valid in the period when the structure was designed, should be used only as guidance documents;

- Actual characteristics of structural material, action, geometric data and structural behaviour should be considered; the original design documentation including drawing should be used as guidance material only.

The most important step of the whole assessment procedure is evaluation of inspection data and updating of prior information concerning strength and structural reliability. It appears that a Bayesian approach can provide an effective tool.

Typically, assessment of the existing structures is a cyclic process in which the first preliminary assessment is often supplemented by subsequent detailed investigations and assessment. A report on structural assessment prepared by an engineer should include a recommendation on possible intervention. However, the client in collaboration with the relevant authority should make the final decision concerning possible interventions.

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CHAPTER 1 – ANNEX A

TERMINOLOGY

(Extracted from ISO CD 8930.2 [9])

structure	Organized combination of connected parts designed to provide resistance and rigidity against various actions.
structural elements or *structural components*	Physically distinguishable parts of a structure, including structural members (such as columns, beams, slabs, shells) and also joints.
structural system	The system formed by the structural elements of a construction works, and the way these elements function together.
maintenance	The routine activities to be performed during the working life of a structure in order to preserve fulfilment of requirements for reliability. ***Note:*** *to restore the structure after an accidental or seismic event is normally outside the scope of maintenance.*
assessment (of the reliability of a structure)	Total set of activities performed in order to find out if the reliability of the structure is acceptable or not.
compliance	Fulfilment of specified requirements
risk	Danger that an undesired event represents for humans, environment or properties. ***Note:*** *risk can be expressed in terms of possible consequences of the undesired event, and associated probabilities.*
failure	Insufficient load-bearing capacity or inadequate serviceability of a structure or structural element
capacity	Ability of a structure (or a part of it) to withstand without failure. For instance: deformation capacity, rotation capacity, load-bearing capacity.
robustness	Ability of a structure to withstand events (like fire, explosion, impact) or consequences of human errors, without being damaged to an extent disproportionate to the original cause.
design criteria	Quantitative formulations describing the conditions to be fulfilled for each limit state.

limit states	States beyond which a structure no longer satisfies the design criteria. These boundaries between desired and undesired performance of the structure are often represented mathematically by "limit state functions".
limit state function	A function of basic variables, whose attainment of the '0' value characterizes a limit state.
ultimate limit states	States associated with collapse, or with similar forms of structural failure. ***Note:*** *they generally correspond to the loss of load-carrying capacity of a structure or structural element.*
serviceability limit states	States corresponding to conditions beyond which specified service requirements for a structure or structural element are no longer met. ***Note:*** *they are related to user's comfort, risk of deterioration, or intended maintenance.*
irreversible serviceability limit states	Serviceability limit states where some consequences of actions exceeding the specified service requirements will remain when the actions are removed.
Reversible serviceability limit states	Serviceability limit states where no consequences of actions exceeding the specified service requirements will remain when the actions are removed.
serviceability criterion	Design criterion for a serviceability limit state.
serviceability constraint	Limit value for a particular serviceability criterion.
reliability or structural reliability	Ability of a structure (or a structural element) to fulfil specified requirements - for safety, serviceability, and durability - over the design working life. It may be evaluated as the probability that the structure will not attain a specified limit state during a specified reference period.
element reliability	Reliability of a structural element which has one single dominating failure mode.
system reliability	The reliability of a structural element which has more than one relevant failure mode, or the reliability of a system of more than one relevant structural element.
probabilistic methods	Calculation methods in which the relevant basic variables are treated as random. ***Note:*** *this term covers both reliability index methods and fully probabilistic methods.*
reliability index	A substitute for the failure probability P_f, defined by $\beta = -F^{-1}(P_f)$, where F^{-1} is the inverse standardised normal distribution.

target reliability level	The level of reliability required ensuring acceptable safety and serviceability.
reliability class	Class (of structures or structural elements) for which a particular specified level of reliability is required
reliability differentiation	The socio-economic optimisation of the resources to be used to build construction works, taking into account all the expected consequences of failures and the cost of the construction.
structural safety	Ability (of a structure or structural element) to resist, with a specified level of reliability, the expected actions (and also specified accidental phenomena) during its construction and anticipated use. ***Note****: the structural safety is related to the ultimate limit states*
serviceability	Ability (of a structure or structural element) to show a specified level of reliability during its normal use. ***Note:*** *the serviceability is related to the serviceability limit states*
limit states method	Calculation method in which the intention is to prevent the structure from exceeding specified limit states.
basic variables	A specified set of variables representing physical quantities which characterise actions and environmental influences, geometrical quantities, and material properties (including soil properties).
primary basic variables	A specified set of basic variables, whose variability is of primary importance in design.
model uncertainties	Uncertainties related to the accuracy of a model. For instance: physical uncertainties, statistical uncertainties.
statistical uncertainties	Uncertainties related to the values of statistical parameters, or to the choice of the statistical distributions of the basic variables.
method of partial factors	Calculation method in which allowance is made for the uncertainties and variability assigned to the basic variables by means of representative values, partial factors and, if relevant, additive quantities.
reliability elements	Numerical quantities used in the partial factor format, by which the specified degree of reliability is assumed to be reached. ***Note:*** *the reliability elements are normally partial factors and additive quantities.*
Importance factor	Factor by which the importance of the possible consequences of failure of a given structure is taken into account.

characteristic value	Value (of an action or a material or a geometrical property) chosen – either, on a statistical basis, so that it has a prescribed probability of not being exceeded towards unfavourable values – or, on a non-statistical basis, for instance on acquired experience or on physical constraints (i.e. nominal value)
design value	Value (of a basic variable) used in a design criterion. ***Note:*** *this value is obtained* – *either by multiplying or dividing a characteristic value by a partial factor (in case of an action or a material property)* – *or by applying an additive or subtractive element (to a geometrical data)* – *or by assessment on the basis of tests.*
nominal value	Value fixed on a non-statistical basis, for instance on acquired experience or on physical constraints.
deterministic method	Calculation method in which all basic variables are treated as non-random.
design working life	Duration of the period during which a structure or a structural element, when designed, is assumed to perform for its intended purpose with expected maintenance but without major repair being necessary.
durability	Ability of a structure or a structural element to maintain adequate performance for a given time under expected actions and environmental influences.
life cycle	Total period of time during which the execution and use of a construction works takes place.
remaining working life	The period for which an existing structure is intended/expected to operate with planned maintenance.
design situation	Set of conditions under which the design is required to demonstrate that relevant limit states are not exceeded during a specific time interval.
persistent design situation	Design situation that is relevant during a period of time of the same order as the design working life of the structure. ***Note:*** *generally it refers to conditions of normal use, including possible extreme loading from wind, snow, imposed loads, earthquakes in areas of high seismicity, etc.*
transient design situation	Design situation which is relevant during a much shorter period than the design working life of the structure, and which has a high probability of occurrence.

***Note:** it refers to temporary conditions of the structure, of use, or exposure, e.g. during construction or repair.*

accidental design situation

Design situation involving possible exceptional conditions for the structure – in use or exposure -, including flooding, fire, explosion, impact or local failure.

seismic design situation

Design situation involving the exceptional conditions when the structure is subjected to a seismic event.

hazard

Exceptionally unusual and severe event, e.g. an abnormal action or environmental influence, insufficient strength or resistance, or excessive deviation from intended dimensions.

environmental influences

Chemical, biological, or physical influences on a structure. They may deteriorate the materials constituting the structure, which in turn may affect its reliability in an unfavourable way.

Action

– a set of concentrated or distributed forces acting on a structure (direct action),

or

– - a set of deformations or accelerations imposed on a structure or constrained within it (indirect action).

–

individual action

(or single action)

Action which can be assumed to be statistically independent in time and space of any other action acting on the structure.
***Note:** an individual action may consist of several components, partially correlated together; for example a thermal action may have a uniform component and a gradient component, a traffic load has vertical and horizontal components.*

permanent action

Action which is likely to act throughout a given reference period of time, and for which the variation in magnitude with time around its mean value is negligible, or for which the variation is monotonic (i.e. always in the same direction) until the action attains a certain limiting value.

variable action

Action which is likely to act during a given design situation, and for which the variation in magnitude with time is neither negligible nor monotonic.

accidental action

Action which is foreseeable but unlikely to occur with a significant value during the design working life of the structure.

fixed action

Action that has a fixed distribution and position over a structure (or a structural element). This means that the magnitude and direction of each individual force (or deformation or acceleration) are determined unambiguously for the whole structure when determined at one point of it
For instance: a static water pressure.

free action	Action that may have any spatial distribution over the structure within given limits. for instance : load of persons on a floor, vehicles on a bridge
load arrangement	Identification of the position, magnitude and direction of a free action.
dynamic action	Action that causes significant acceleration to a structure (or a structural element).
static action	Action that does not cause significant acceleration to a structure (or structural element).
quasi-static action	Static action representing a dynamic action including its dynamic effects.
bounded action	Action which cannot exceed a certain value (exactly or approximately known) .
sustained action, transient action	A qualitative distinction, referring to the duration of actions: e.g. the weight of the furniture on a floor is a sustained action, whereas the weight of persons on the floor is a transient action.
self weight	***Note:*** *one should avoid the expression "dead load" on account of its ambiguity.*
prestress	Permanent action resulting from the application of controlled forces to a structure and/or of controlled deformations to it.
geotechnical action	Action transmitted to the structure by the ground, fill or groundwater.
seismic action	Action that arises due to earthquake ground motions.
imposed load	***Note:*** *one should avoid the expression "live load" on account of its ambiguity.*
construction load	Load specifically related to execution activities.
reference period	A chosen period of time used as a basis for assessing the design value of variable and/or accidental actions.
representative values of an action	Representative value of an action: a value assigned to the action for a specific purpose, for instance the verification of a limit state.
characteristic value of an action	The principal representative value of an action. It is chosen – either, when a statistical base is available, so that it can be considered to have a prescribed probability of not being exceeded (towards unfavourable values) during a reference period, – or from acquired experience

– or on physical constraints.

Note: *the "reference period" shall take into account the design working life of the structure and the duration of the design situation.*

combination value of a variable action	Value chosen for an action in combination with others - in so far as it can be fixed on statistical bases – so that the probability that the effects of the combination will be exceeded is approximately the same as when only the characteristic value of the action is present. This 'combination value' may be expressed as a part of the characteristic value by using a factor $\psi_0 \leq 1$.
frequent value of a variable action	Value determined – in so far as it can be fixed on statistical bases – so that either the total time, within the reference period, during which this value is exceeded is only a small given part of the reference period, or the frequency of this excess is limited to a given value. This 'frequent value' may be expressed as a part of the characteristic value by using a factor $\psi_1 \leq 1$.
quasi-permanent value of a variable action	Value determined so that the total period of time for which it will be exceeded is a large fraction of the reference period. The 'quasi-permanent value' may be expressed as a part of the characteristic value by using a factor $\psi_2 \leq 1$.
load case	A set of actions (including load arrangements and imposed deformations) and imperfections, taken into account simultaneously for a particular verification.
combination of action (or load combination)	Set of the design values of different simultaneous actions used for the verification of the structural reliability for a particular limit state.
fundamental combination of actions	Combination of permanent actions and variable actions (the leading action plus the accompanying actions) used for studying an ultimate limit state.
accidental combination of actions	Combination for accidental design situations, involving either an explicit accidental action (e.g. fire or impact) or the situation after an accidental event.
characteristic combination of actions	Combination of permanent and variable actions used for studying a service limit state, where one of the variable actions has its characteristic value.
frequent combination of actions	Combination of permanent and variable actions used for studying a service limit state, where one of the variable actions has its frequent value.

quasi-permanent combination of actions	Combination of permanent and variable actions used for studying a service limit state, where all the variable actions have their quasi-permanent value.
strength	Property of a material indicating its ability to resist mechanical actions. ***Note:*** *it is usually given in units of stress.*
characteristic value of a material property	A specified fractile of the statistical distribution of the material property in the supply produced within the scope of the relevant material standard
conversion factor, conversion function	Factor (or function) which converts properties obtained from test specimens to properties corresponding to the assumptions made in calculation models.
design value of a material property	Value obtained – either by dividing the characteristic value by a partial factor γ_M, – or by direct determination.
geometrical imperfections	Deviations from the intended geometry of a structure or a structural component
characteristic value of a geometrical quantity	The characteristic value of a geometrical quantity corresponds to – usually the dimension specified in the design – where relevant, a prescribed fractile of the statistical distribution of the quantity.
design value of a geometrical quantity	The design value of a geometrical quantity corresponds to – usually a nominal value – where relevant, a prescribed fractile of the statistical distribution of the quantity. ***Note:*** *the design value of a geometrical property is generally equal to the characteristic value. However, it may differ in cases where the limit state under consideration is very sensitive to the value of the geometrical property, for example when considering the effect of geometrical imperfections on buckling. In such cases, the design value will normally be established as a value specified directly, for example in an appropriate European Standard or Pre-standard. Alternatively, it can be established on a statistical basis, with a value corresponding to a more extreme fractile (i.e. a rarer value) than applies to the characteristic value.*
resistance	Capacity of a structural element or a cross-section of a structural member to withstand actions without mechanical failure. For instance: tension resistance, bending resistance, buckling resistance.

design resistance	Value of a resistance incorporating partial factors
effects of actions (or action effects)	The effects of actions (or action effects) on structural elements (e.g. internal force, moment, stress, strain) or on the whole structure (e.g. deflection, rotation).
structural analysis	Determination of the effects of actions in a structure or part of it. A distinction is generally made between global analysis (considering the whole structure), member analysis (e.g. about buckling), and local analysis (e.g. a cross-section, a connection, a weld).
structural model	An idealisation of the structure, used for the purposes of analysis, design and verification.
calculation model	A simplified description of a physical reality, suitable for calculation. For instance: model for actions, structural analysis model, behaviour model.
damage	Unfavourable change in the condition of a structure that may affect structural performance
deterioration	A process that adversely affects the structural performance including reliability over time due to: – naturally occurring chemical, physical or biological actions – normal or severe environmental actions – repeated actions such as those causing fatigue – wear due to use – improper operation and maintenance of the structure
deterioration model	A mathematical model that describes structural performance as a function of time taking deterioration into account
inspection	On-site non-destructive examination to establish the present condition of a structure.
investigation	Collection and evaluation of information through inspection, document search, load testing and other testing.
load testing	Test of the structure (or part of it) by loading to evaluate its behaviour or properties, or to predict its load bearing capacity
material properties	Mechanical, physical or chemical properties of structural materials
monitoring	Frequent or continuous, normally long-term, observation or measurement of structural conditions or actions.

repair (of a structure)	Improvement of the condition of a structure by restoring or replacing existing components that have been damaged.
safety plan	Plan specifying the performance objectives, the scenarios to be considered for the structure, and all present and future measures (design, construction, or operation, - e.g. monitoring) to ensure the safety of the structure.
structural performance	A qualitative or quantitative representation of the behaviour of a structure (e.g. load bearing capacity, stiffness, etc.) in terms of its safety and serviceability.
upgrading	Modifications to an existing structure to improve its structural performance.

CHAPTER 2: NON-DESTRUCTIVE TESTING, INSPECTION TECHNIQUES AND MONITORING

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1 INTRODUCTION

The first prerequisite to intervention in existing buildings is a comprehensive understanding of each and every one of its characteristics. The engineer or architect responsible must be in possession of information on items including the period of construction, construction methods and building codes in place and materials in use at the time, and the history of the building, including any remodelling or retrofitting and events such as earthquakes or fire.

A second essential consideration is a survey of the building in its present state, which must focus on geometry and architectural features. The structure must be carefully identified and the non-structural elements characterised. One of the most important questions to be explored in this type of survey is the existence of symptoms of instability. Crack patterns and leaning walls, for instance, must be carefully identified and depicted in the survey drawings, together with data on crack width and extension, angle of inclination and similar. Such inspections should be geared to establishing the nature of the instability and its causes with a view to determining the most suitable intervention measures to eliminate the causes and halt progression.

Other major aspects that need to be reviewed to complete the knowledge about the building include:

- structural system;
- dimensions of the structural members;
- mechanical properties of materials;
- response of the structure or some of its members to loads;
- pattern of variation in possible instability or deterioration.

The behaviour of the structural materials may be determined by testing a certain number of samples cored from the structural members. While this method reliably characterises mechanical properties, for obvious reasons it cannot be used extensively on a number of samples large enough to ensure statistical significance. Alternative techniques have consequently been developed to assess mechanical strength by measuring other related properties in which sample coring is not necessary, i.e., non-destructive techniques.

In any case, measurement inaccuracy should be carefully considered in the determination of the characteristic value of mechanical properties. In §A.9 of the Annex A to the present chapter two examples are given dealing with this relevant topic.

Knowing the mechanical behaviour of all or part of a structure is imperative to verifying model results in both new and existing structures. That information is attained by applying static or dynamic design loads to the structure and monitoring the resulting deflection or deformation: a sort of short-term monitoring.

In buildings exhibiting deterioration, in turn, information is needed on whether the condition has stabilised or is progressing and, if the latter, at what rate. To that end, the effects of instability must be monitored for periods of time long enough to establish foreseeable future behaviour.

This chapter addresses the assessment of geometric properties with the most widely used non-destructive techniques for inspecting steel bars (section 2) and concrete quality in reinforced concrete structures (section 3). Section 4 discusses the standard inspection techniques used to determine mechanical behaviour and durability in reinforced concrete and masonry structures. Section 5 describes static and dynamic load tests for structural verification. Short- and long-term monitoring are briefly reviewed in section 6, while some tips on the statistical assessment of test results are given in the Annex.

2 UPDATING OF GEOMETRIC PROPERTIES

When slid across the surface of RC members (fig. 1), cover meters detect the presence and indicate the position of steel rebar, even when at a considerable depth (fig. 2). Moreover, where the bars are spaced at some distance from one another, their diameter and distance from the concrete surface can also be estimated.

Figure 1: Use of cover meter

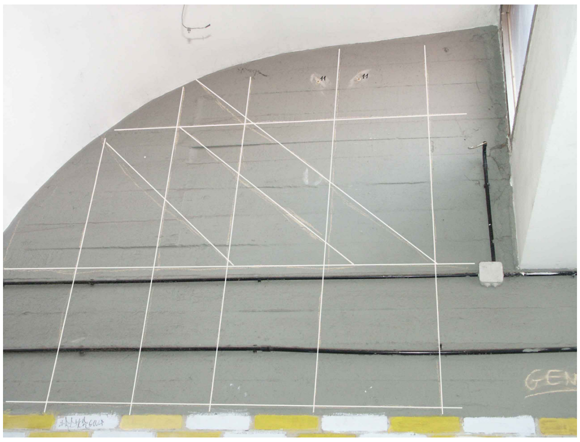

Figure 2: Stake-out of reinforcing bars detected

3 UPDATING OF MECHANICAL PROPERTIES OF CONCRETE

3.1 Compression test on cores

Concrete quality is normally defined in terms of its compressive strength. In structures under construction, strength is determined by means of compression tests conducted to failure on an appropriate number of samples, made with the same fresh concrete as the target structures.

In existing buildings, compression tests can usually be performed on samples cored from the structure, although this operation may entail certain complexities and weakens the member. Such semi-destructive procedures consequently tend to be limited to a short number of samples. Concrete quality assessment is therefore supplemented with a suitable number of non-destructive tests.

The samples used for compression testing are cored from a reinforced concrete (RC) member with a drilling machine fitted with a rotating diamond ring that may range in diameter from 20 to over 200 mm (figs. 3 and 4). The diameter of the core should be greater than 3 times the maximum size of the aggregate.

Cores are compression tested as laid down in European standard EN 12504-1 (fig. 5). The compressive strength of the in situ concrete is found by processing the test findings as described in European standard EN 13791.

Figure 3: Coring concrete samples

Figure 4: Drill hole in RC member

Figure 5: Compression testing of a core sample

The core strength depends mainly on:

– the core diameter, that affects both the strength measured and its variability: the strength of a 100-mm diameter, horizontally drilled core with a length-diameter ratio of l/d = 1 is equivalent to the strength of a 150-mm cubic specimen, while variability is generally greater in cores with diameters of under 100 mm and l/d = 1. Hence, three times as many 50-mm cores as 100-mm diameter cores may have to be tested to obtain results reliable enough for rectilinear interpolation between the two types of samples;
– the diameter-to-maximum aggregate size ratio: coefficient of variation rises as the ratio declines: for cores with a diameter smaller than 50 mm (micro-cores), specific procedures must be followed;

- the length/diameter ratio: the measured strength declines with *l/d* ratios greater than one and rises with ratios smaller than one; this effect can be attributed primarily to the restraint induced by the test frame platens;
- presence of rebars: cores used to measure the concrete strength should not contain reinforcing bars; when this cannot be avoided, the strength measured with a core containing steel (other than along its axis) must be expected to be lower than in cores without rebars; however cores containing reinforcing bars at or close to the longitudinal axis are not suitable for testing strength.

Since drilling cores in a structural member may damage or weaken the element itself, the number of samples should be limited and the assessment of in situ compressive strength supplemented with indirect, generally non-destructive, methods.

Evidence exists linking some physical properties of concrete to its compressive strength. Indirect methods that measure such physical properties can therefore be used to subsequently compute compressive strength from the respective correlation law.

While the values of these measurements are sufficiently reliable, the correlations are less so, given the many factors that may intervene. Consequently, although indirect tests are non-destructive, faster and less expensive than core drilling, the results do not consistently deliver accurate estimates of compressive strength. As a rule, better results are obtained by combining two or more indirect methods, since the factors affecting the results tend to offset one another.

In conclusion, in situ compressive strength is most suitably assessed with extensive indirect testing to identify groups of structural members with similar characteristics so that a limited number of members in each group may then be chosen for core drilling. The compression test findings for the cores can then be used to validate the correlation between the property measured and compressive strength.

The most widespread indirect methods include the rebound hammer test, the ultrasonic pulse velocity test, and a combination of the two, commonly known as the SonReb test.

3.2 Rebound hammer test

This method is based on the principle that the rebound of an elastic mass depends on the hardness of the surface impacted.

The test consists of striking the concrete surface with a given mass at constant energy and measuring the rebound distance. Part of the impact energy is absorbed by the concrete in the form of permanent inelastic deformations and part is returned to the mobile mass, which bounces off the surface. Therefore, the greater the strength of the material, the lower is its permanent deformation and the greater the rebound distance (fig. 6).

The assessment is intrinsically limited to measuring the hardness of a surface layer no more than approximately 10 to 30 mm thick. Moreover, that layer need not always be representative of the entire member, due to the alterations induced by environmental factors.

All measurements must be taken in an area of about 200x200 mm, free of surface defects, at a considerable distance from reinforcing bars and properly prepared and cleaned to smooth surface roughness.

The concrete must be struck at least 10 times and the rebound distance calculated as the average of the measured values. The standard deviation must also be taken into consideration: if the values are too scattered, the test findings are unreliable.

Since the dispersion of the results for any given material is fairly small, rebound tests can aptly detect the uniformity of concrete properties in structures.

When determining the strength of concrete in situ, however, the values recorded are affected by many factors, such as type of cement, type of aggregate, type of surface, surface moisture, carbonation, age of the concrete, curing conditions and consolidation. The effects of these factors may not therefore be overlooked when interpreting rebound test findings.

The method for finding concrete strength from rebound text values recommended in European standard EN 13791 involves the use of a basic correlation curve for each group of uniform RC members, calibrated against core strength values (fig. 7).

Figure 6: Rebound hammer test

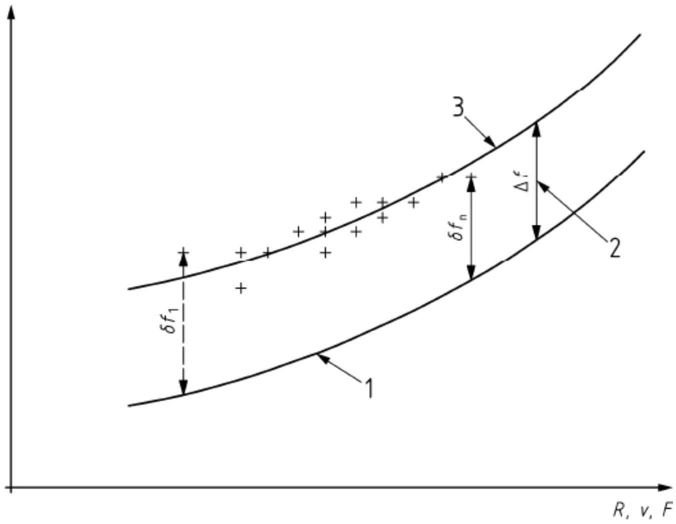

Figure 7: Rebound test correlation curve

The following correlation has been proposed [1, 2]:

$$f_{is,cyl} = 0{,}009\, I_r^2 + 0{,}77\, I_r - 11 \tag{1}$$

where:
$f_{is,cyl}$=in situ core strength [N/mm^2]
I_r=rebound number.

3.3 Ultrasonic pulse velocity method

This method is based on the principle that the velocity with which vibrational pulses propagate in a medium, V_L, depends on the elastic properties of the medium (dynamic modulus of elasticity, E_d, and Poisson ratio, v_d) and its density, ρ:

$$V_L = \sqrt{\frac{E_d}{\rho}\frac{1-\nu_d}{(1+\nu_d)(1-2\nu_d)}} \tag{2}$$

where:
V_L=velocity of propagation of the ultrasonic waves [m/s]
E_d=dynamic modulus of elasticity [N/m^2]
v_d=dynamic Poisson ratio
ρ=density [kg/m^3]

Velocity is related directly to the elastic modulus but not to strength. Nonetheless, since generally the elastic modulus increases with strength, the existence of an indirect relationship between strength and velocity can be expected. In addition, many factors affect the ultrasonic pulse velocity in concrete, such as voids, shape and size of the aggregates and the presence of reinforcing bars. Hence, strength estimates using this method are not always reliable.

Figure 8: Ultrasonic pulse velocity test

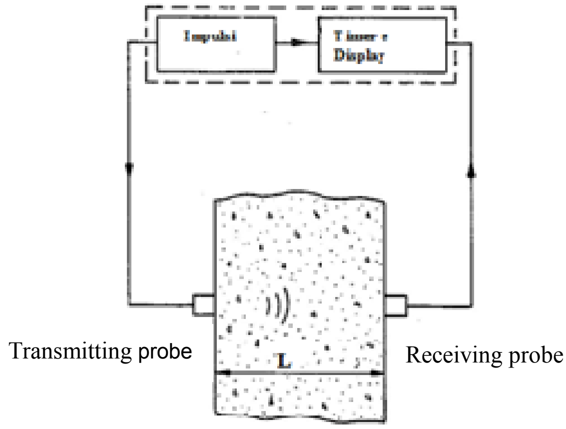

Figure 9: Test set-up for UPV testing

As with the rebound test, finding situ strength from ultrasonic pulse velocity values involves the use of a basic correlation curve for each group of uniform RC members, calibrated against core strength values (fig. 10).

The following correlation has been proposed in the literature [3]:

$$f_{is,cyl} = 1{,}88 \cdot 10^{-21}\, V_L^{6{,}184} + 0{,}77\, I_r - 11 \quad (3)$$

where:

$f_{is,cyl}$=in situ cylindrical strength [N/mm^2]

V_L=ultrasonic pulse velocity [m/s].

3.4 Combined method: SonReb

Inasmuch as the two methods described above are affected by different factors, more reliable estimates can be obtained by performing and then comparing the two types of measurements to identify possible discrepancies.

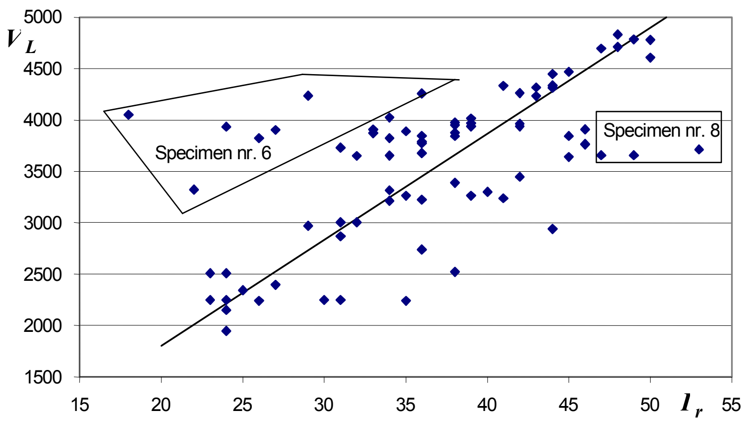

Figure 10: I_r vs V_L [m/s]

By way of example, the plot in figure 10 illustrates the correlation between the rebound number, I_r, and the ultrasonic pulse velocity, V_L. Most of the data are fairly closely correlated, with the exception of two groups: the first, *specimen nr. 6*, refers to a high quality concrete whose surface was deteriorated by acids, and the second, *specimen nr. 8*, to a concrete whose surface was hardened by carbonation. In such cases, outliers should be rejected.

When the results of the two methods are consistent, the correlation known as the SonReb method [4], which has proven to deliver fairly reliable results, is applicable (fig. 11):

$$f_{is,cyl} = 7{,}695 \cdot 10^{-11}\, I_r^{1{,}4}\, V_L^{2{,}6} \tag{4}$$

where:
$f_{\text{is,cyl}}$=in situ cylindrical strength [N/mm^2]
I_r=rebound number
V_L=ultrasonic pulse velocity [m/s]

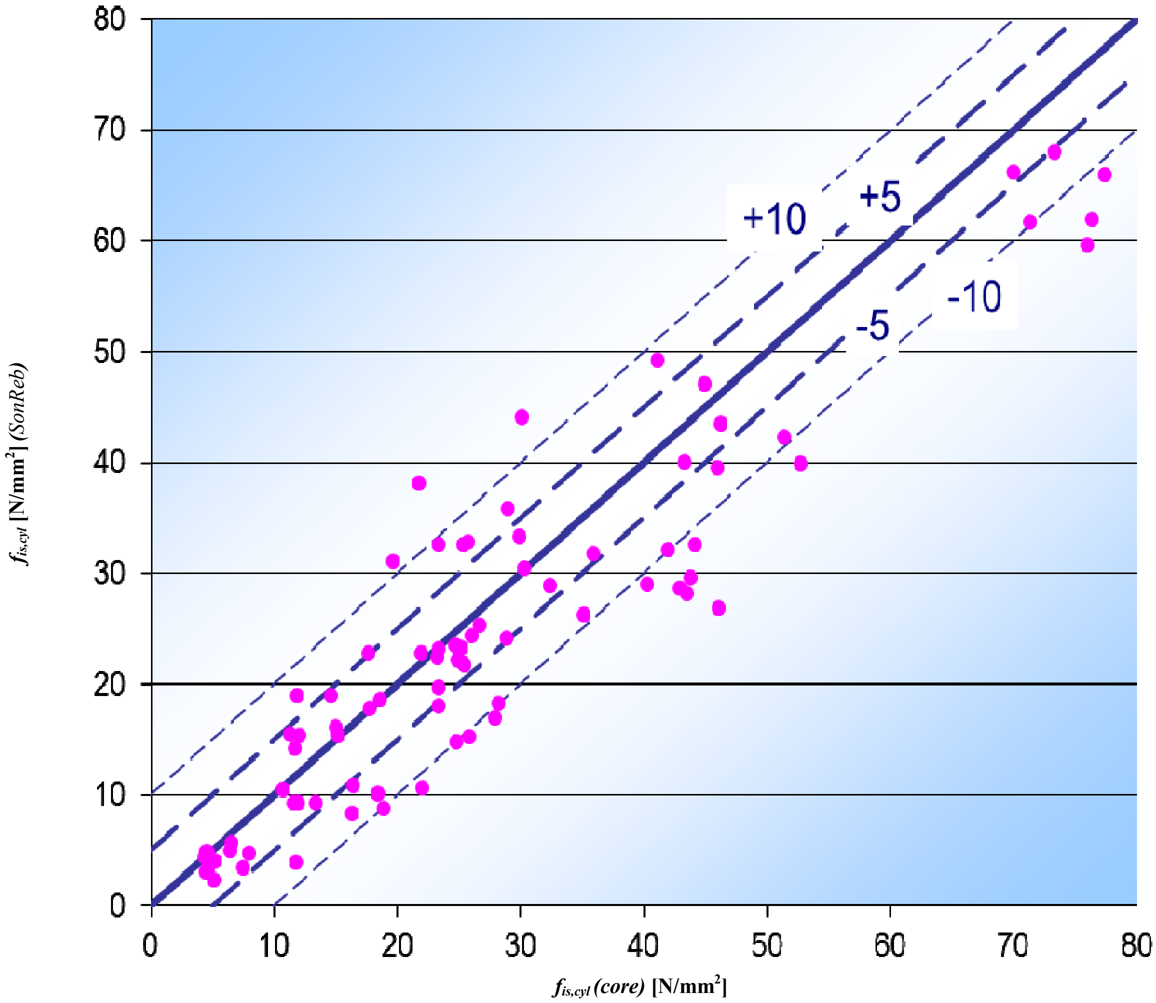

Figure 11: Core strength versus SonReb estimates

4 SURVEY OF STRUCTURE

4.1 In-situ measurement of corrosion parameters in reinforced concrete structures

4.1.1 Measurement of corrosion rate

The corrosion rate is defined as the amount of steel corroded per unit of surface and time [5]. The amount of oxides generated is directly related to concrete cover cracking and weakening of the steel/concrete bond, while any decline in the steel cross-section significantly affects the load bearing capacity of the structure. The corrosion rate is therefore a proxy measurement for the loss of load carrying capacity. The main structural consequences of corrosion are [6]:

- declining cross-section in reinforcing steel;
- loss of steel ductility;
- cover cracking and consequently decline in concrete cross-section;
- loss of the steel/concrete bond.

In addition to calculating rebar cross-section losses, the corrosion current may be used to identify corroded zones and evaluate the efficacy of repair techniques [7].

The corrosion current, I_{corr}, is measured by a reference electrode which determines the electrical potential, in conjunction with a counter electrode, which supplies the current. In on-site measurements, in addition to the central circular counter electrode, a second electrode (guard ring) is used to confine the current to a limited area in the reinforcement. The aim of this guard ring is to balance the electrical field produced by the central auxiliary electrode (fig. 12).

The polarisation resistance, R_p, method is the most widespread of the various electrochemical methods proposed to measure corrosion-related electrical parameters. This method is based on the application of a small electrical current to the steel with a counter and a reference electrode. Providing the electrical signal is uniformly distributed across the reinforcement, R_p is defined as [5]:

$$R_p = \left(\Delta E / \Delta I\right)_{\Delta E < 20\ \mathrm{mV}} \tag{5}$$

where:
ΔE=corrosion potential-induced polarisation
ΔI=polarisation current.

The corrosion current, I_{corr}, is inversely proportional to R_p:

$$I_{corr} = B / R_p \tag{6}$$

where:
B=constant; for in-situ tests, usually taken to be 26 mV [7].

In on-site measurements, the location of the measuring points is crucial, for they must be representative of the deterioration process. The location may be selected on the grounds of a hypothetical grid with fixed spacing, although supplementary techniques such as the corrosion potential or resistivity may be used prior to the grid procedure.

The corrosion rate meters used must be able to accurately determine current dispersion over a given distance. That calls for applying the modulated confinement technique (controlled guard ring, see fig. 12) or the potential attenuation method. One of these two operating modes must be used, for otherwise an error of one or two orders of magnitude may be incurred [7].

When no electrochemical techniques are available, a mean corrosion rate value may be calculated by dividing the measured loss in rebar diameter by the number of years that corrosion has been propagating [5].

Other aspects and parameters related to the environment of the concrete itself that must be taken into account in on-site corrosion rate measurements include:

- corrosion morphology (general or pitting);
- macrogalvanic effects;
- chloride content;
- moisture content;
- temperature.

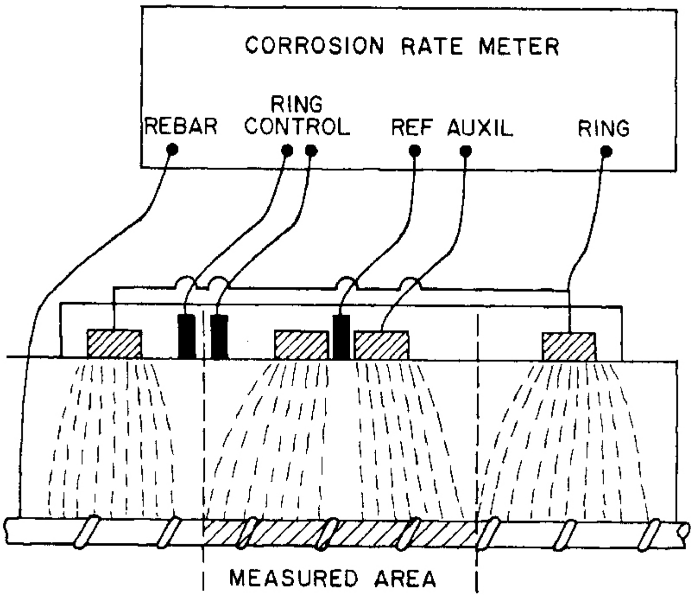

Figure 12: Modulated current confinement method [5]

4.1.2 Electrical resistivity

Electrical resistivity, ρ ($\Omega \cdot m$), which is the inverse of conductivity, is a volumetric measurement of electrical resistance, R_e (Ω): according to Ohm's law, the ratio between voltage and current ($R_e = V / I$). Resistivity is the property whereby a porous medium is able to convey an electrical charge [8].

Since the potential difference or the current applied by two electrodes is carried across the aqueous phase of the concrete pore system by electrical carriers (ions), the electrical resistivity of water-saturated concrete is an indirect measure of concrete pore connectivity. Electrical resistivity provides insight into pore connectivity and therefore into the resistance of the material to penetration by fluids. In other words, resistivity indirectly measures the key properties that determine reinforcement durability [8].

Age, water saturation level and temperature are factors that affect resistivity values. Concrete resistivity increases with time due to pore structure refinement. Porosity declines with cement phase hydration, raising both mechanical strength and resistivity. Resistivity, ρ, varies with the degree of water saturation in the pore system because in semi-saturated conditions ions are conducted across the layer of water adsorbed onto the walls of the pores [9]. Temperature also has a substantial impact on resistivity, which declines as temperature rises. This effect can only be generalised, however, if the ρ values are normalised to a reference temperature, usually 25 ºC [10, 11]. Variations in concrete resistivity with changes in environmental factors such as moisture condition and temperature must consequently be taken into consideration in on-site measurements (section 4.1.4).

Concrete resistivity can be measured in any of three ways:

–

- directly on the surface of the structure;
- on cores drilled from the structure;
- using embedded sensors.

Since this chapter focuses on inspection methods, which are mainly referred to on-site testing, only direct measurements on the surface of the structure are considered. The most widely used techniques for direct on-site measurement of concrete resistivity are the four-probe method (four electrodes) and the disc method (one electrode).

The four-probe method is based on Wenner's technique [12], originally developed for geophysical prospecting but subsequently applied to concrete. This method uses four equally spaced point electrodes in contact with the concrete surface [13]. The electrode tips should be moistened with a conducting liquid to ensure good contact with the concrete.

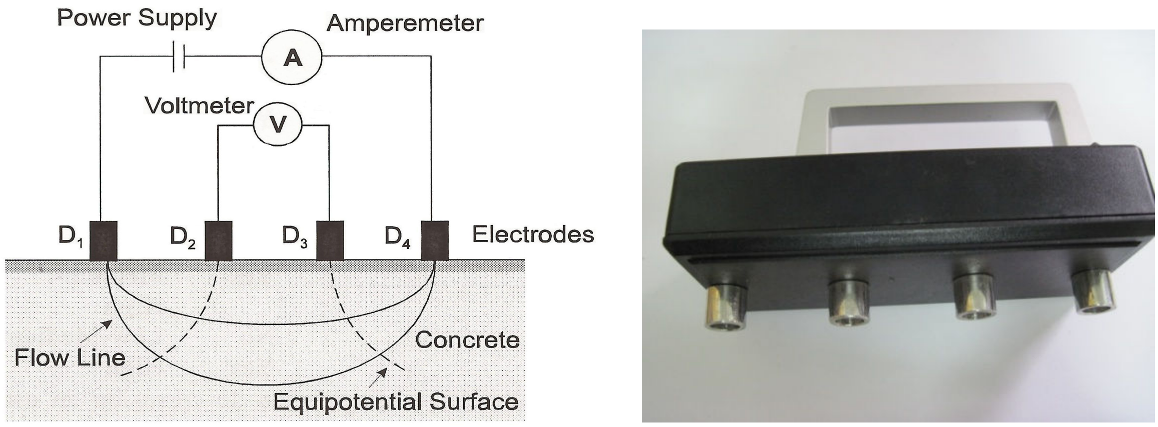

Figure 13: Wenner four-probe method resistivity measurement set-up and sensor [7]

A known alternating current (generally with a frequency of 50 to 1000 Hz) is passed between the two outer electrodes while measuring the potential difference between the inner electrodes (fig. 13). Resistivity is found as a function of voltage, current and distance between tips:

$$\rho = 2 \cdot \pi \cdot a \cdot R \tag{7}$$

where:
a=distance between electrode tips
R=resistance as directly measured.

The disc (one electrode) method, based on Newman's studies, was fully developed by Feliú, González and Andrade [14] to estimate the ohmic drop between a small disc placed on the surface of an electrolyte and a much larger counter electrode placed at "infinity". Theoretically, if the contribution by the counter electrode to the total resistance is negligible, the electrical resistance can be said to depend on electrode resistivity.

The device consists of a disc made of conductive material, a potentiostat and a reference electrode (fig. 14). Once good contact is made between the electrode and the concrete, a galvanostatic pulse is applied and the instantaneous ohmic drop is recorded. As in the Wenner four-probe method, disc and rebar must be at a certain distance, normally twice the disc diameter, to obtain an accurate measure of bulk resistance. The resistivity obtained is:

$$\rho = 2 \cdot R_e \cdot \phi \tag{8}$$

where:
ϕ = disk diameter.

4.1.3 Potential measurement

Steel corrosion leads to the coexistence of passive and corroding areas in the same bar, forming a short-circuited element in which the corroding area is the anode and the passive surface the cathode. The main purpose of measuring potential in a structure is to locate areas where the reinforcement has become depassivated and may corrode when suitable amounts of oxygen and moisture are present [7]. Nonetheless, potential measurements may also be taken to:

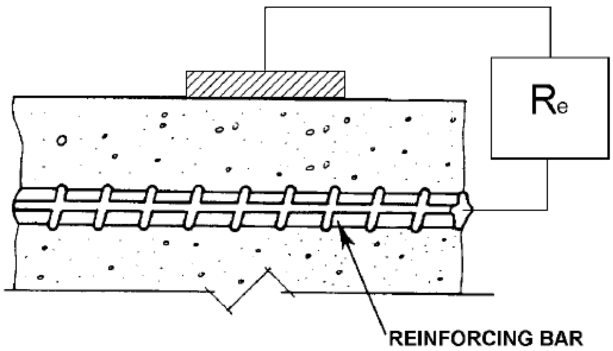

Figure 14: Disc method for measuring resistivity [7]

- locate and define areas where other kinds of tests should be conducted to obtain more accurate and cost-effective information about the condition of the structure;
- evaluate the efficacy of repair works by controlling the corrosion state of the rebar;
- design preventive measures such as cathodic protection or electrochemical restoration techniques.

Before the areas where the reinforcement is corroding can be identified, a system of coordinates must be devised to correlate readings and measuring points. The accuracy of the measurements depends on the size of this grid. While a good electrical connection to the reinforcement must be established to measure the half-cell potentials in a structure, direct contact should not be made if the reinforcing steel is connected to an exposed steel member. An external reference electrode is positioned on the concrete surface (with a wet sponge between them to ensure good electrolytic contact) to furnish the high impedance voltmeter with further data (fig. 15). The electrical continuity of the reinforcing steel must be assured by measuring the resistance between separate sections of rebar. Resistance values of less than or equal to 0.3 Ω denote electrical continuity.

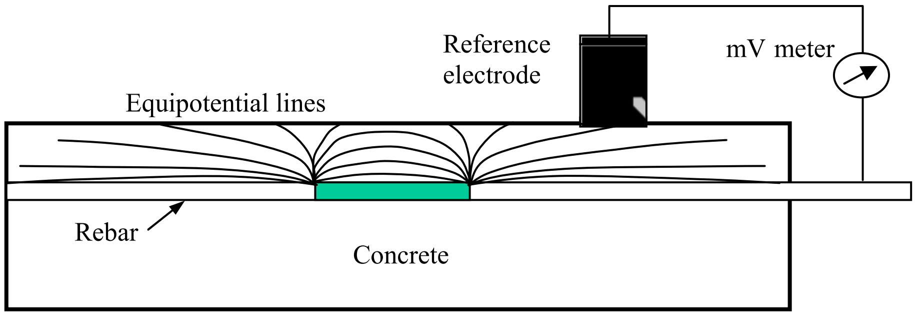

Figure 15: Measuring potential in concrete rebars with one reference electrode [7]

Potential measurements can be performed with a single electrode or with one or several wheel electrodes. The latter are used to check potential in structures such as large bridge decks or car park deck slabs, for they can cover up to 300 m^2 per hour when connected to microprocessor-controlled data-loggers.

Once the data are gathered, optimal representation depends on the amount of data and structure type. The information may be set out in anything from tables to a coloured grid map of the potential field, where every individual potential reading can be identified as a small cell and a contour line map can be drawn with algorithms for interpolating values between measurement points. Potential measurements can also be represented with standard statistical tools, of course, such as frequency distributions or histograms. The representational design should be determined by the kind and depth of the study [7].

Corrosion potential measurements are affected by a wide range of factors, including: concrete moisture content, chloride content, concrete carbonation, cover thickness, polarisation effects, oxygen content and type of reference electrode used, and are very dependent on the environmental conditions, temperature and humidity, during measurements. For these reasons, corrosion rates determined with the abovementioned methods result generally very scattered. Therefore, additional inìvestigations and suitable cross checks are necessary to improve the reliability of the predicted corrosion rate values.

Aiming to determine the evolution of corrosion during the structural life, it is necessary to introduce service life models taking into account the variation of corrosion rate with time. Generally, it is assumed that corrosion rate varies linearly with time, but in many cases, especially when the influence of environmental conditions is particularly significant, this assumption results unrealistic and more complex models need to be implemented.

4.1.4 Environmental parameters

The chief environmental parameters to be taken into consideration when assessing corrosion-deteriorated structures include moisture, temperature, chloride content and carbonation.

Concrete moisture content, the environmental parameter with the greatest impact on corrosion, depends on environmental humidity and affects electrical resistivity and oxygen availability around the rebar.

Fig. 16 plots the corrosion rate versus concrete moisture content. When the pores are fully saturated, resistivity is at its lowest, but oxygen access is limited because it needs to dissolve in the pore water. Consequently, the corrosion rate may decline due to scant oxygen diffusion. As the pores start to dry, oxygen has readier access to the rebar and corrosion rises accordingly. When the concrete dries completely, however, resistivity increases, again curbing corrosion. Therefore, the corrosion rate peaks when the concrete is nearly saturated and the effects of oxygen availability and resistivity are counterbalanced [7].

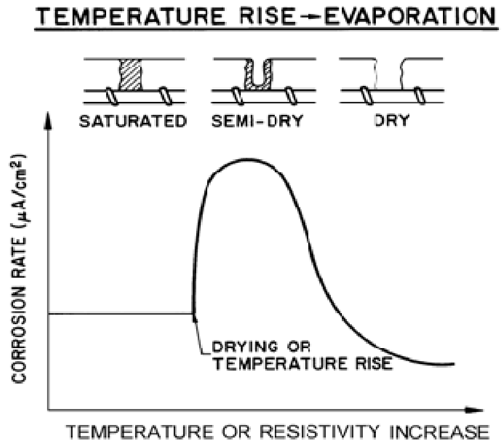

Figure 16: Corrosion rate versus concrete pore moisture content and effect of temperature on moisture content [7]

Temperature may hasten or retard corrosion (fig. 16). When it rises, pore water evaporates and oxygen is released from the pore solution. Consequently, although corrosion is stimulated by higher temperatures, that effect may be counterbalanced by the rise in resistivity (evaporation) and the decline in oxygen levels (oxygen is less soluble at higher temperatures). The opposite effect is induced by declines in temperature in semi-dry concrete due to the resulting condensation. In a nutshell, the effect of daily and seasonal variations in relative humidity and temperature on the corrosion rate cannot be easily quantified [7].

Chloride attack is the result of the presence of chloride ions in the concrete, either as a component of admixtures, water or aggregates, or, more commonly, as an outside agent in structures exposed to marine environments or de-icing salts.

Chlorides induce local damage to the passive layer on the steel, generating pits or localised attacks. Cracking may or may not ensue, depending on whether the corrosion is widespread or localised. Rebar has been known to corrode in submerged members with no cover cracking. Chlorides penetrate submerged or fully saturated concrete by diffusion. In members exposed to the air or cyclical conditions (de-icing salts), by contrast, ingress may be governed primarily by capillary absorption.

A number of methods are in place to determine the total chloride content in hardened concrete. The samples for these tests may be taken from dust in core holes or scratched off the structure at different depths, measured from the surface. Fragments for chemical analysis may also be removed from cracks or spalled areas. The aim is to establish the chloride gradient or profile from the concrete surface inward and identify the chloride threshold that induces depassivation. Chloride profiles can be also obtained from cores drilled from the structure and subsequently scratched layer by layer [7].

The quantab- and rapid chloride (RCT) tests are the methods most often used to determine the total chloride content in field surveys. Other more accurate analytical methods can be performed under laboratory conditions.

In the absence of chlorides, *carbonation* is the agent that affects concrete most aggressively. Atmospheric carbon dioxide reacts with the calcium and alkaline hydroxides present in the cement phases, lowering the pore solution pH to values near neutrality. This process results in the depassivation of the steel in contact with the carbonated zones.

The concrete moisture content is the factor with the greatest impact on the penetration rate of the carbonation front. This rate peaks in concrete in semi-dry conditions (central region in fig. 16).

A freshly exposed concrete surface is needed to determine the carbonation depth. The depth of the carbonation front is found by spraying the concrete surface with an acid-base indicator (phenolphthalein) that changes colour with the pH value. At least four measurements of depth at which the indicator is colourless must be taken and the maximum and minimum values recorded to obtain a representative mean value. Carbonation depth can be measured on cores drilled for mechanical strength or in the drill holes. If no core can be drilled or concrete removed, a hammer drill can be used to obtain a freshly exposed concrete surface [7].

Once the environmental data are known, the parameters of the service life models describing the evolution of degradation and corrosion and the deterioration of material properties can be calibrated, allowing more precise prediction of the structural performance decay with time.

4.2 Masonry structures

4.2.1 General

Masonry consists of courses of block, such as natural stone or bricks, bonded together with mortar. Masonry is usually a highly variegated material, a combination of regular or irregular units and frequently built as multi-wythe brick or stonework. Typologies and materials vary widely over time and from one region to another (Figs. 17–20). The mechanical characteristics of masonry required for structural analysis cannot be directly correlated to the properties of its components (brick, stone, mortar) except for brick and stone masonry made with regular units and joints. Therefore, information on the properties needed to define the structural behaviour of masonry can only be obtained where the material is tested as a composite [15].

Testing the strength and stress-strain behaviour of masonry in the laboratory is extremely difficult because significant specimens can simply not be sampled from existing masonry structures. The only viable alternative would appear to be in-situ testing on the masonry as a composite material.

The behaviour of masonry walls depends less on the characteristics of the block and mortar components than on how they are laid, i.e., the amount of mortar, the presence of voids, the inter-course connections and similar (figs. 21, 22). The first step in the survey, then, is to identify the type of component materials and their arrangement. For the outer wythes, removing the plaster may suffice to identify the type of material, block shape and dimensions, the thickness of mortar joints, and the bond patterns. When the plaster cannot be removed or in inner wythes, one of two techniques may be used: thermographic or endoscopic surveys.

Figure 17: Cyclopean masonry

Figure 18: Ashlar masonry

Figure 19: Stone masonry

Figure 20: Brick masonry

Figure 21: Arrangement of blocks and mortar across the wall thickness

Figure 22: Presence of voids

4.2.2 Thermographic surveys

Thermographic cameras can identify the bond patterns in masonry walls, block shapes and dimensions, the presence of internal voids such as chimney flues, the presence of concrete curbs and similar (figs. 23, 24).

Thermographic surveys are speedy and non-invasive. While they can detect voids very reliably, they can only reveal inner bond patterns when the wall is in the bright sun and the temperature is favourable.

Figure 23: Photograph

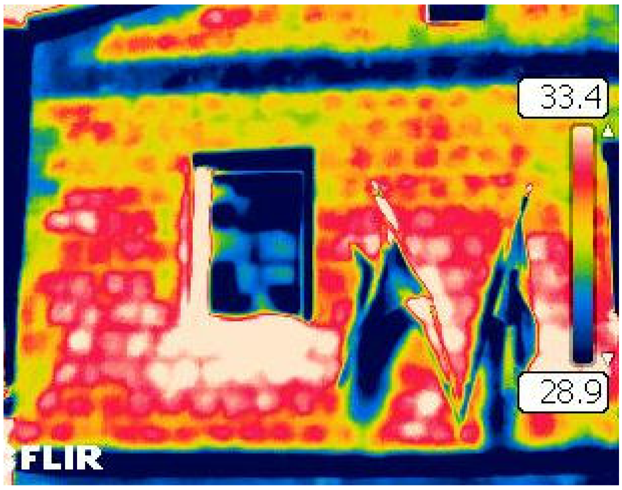

Figure 24. Thermograph

4.2.3 Endoscopic surveys

By inserting an endoscope into a hole drilled in a masonry wall (fig.25), the thickness of the wall can be determined, as well as:

- the presence of layers of different materials;
- the nature of the materials;
- the dimensions of the blocks;
- the amount of mortar;
- the presence of voids;
- the presence of gaps between wythes.

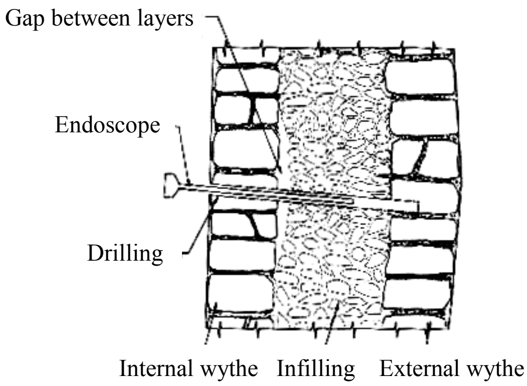

Figure 25: Endoscopic survey set-up

Endoscopic tests are speedy, inexpensive and only minimally invasive, inasmuch as the hole needed is only a few millimetres in diameter (figs. 26, 27).

Figure 26: Endoscopic survey

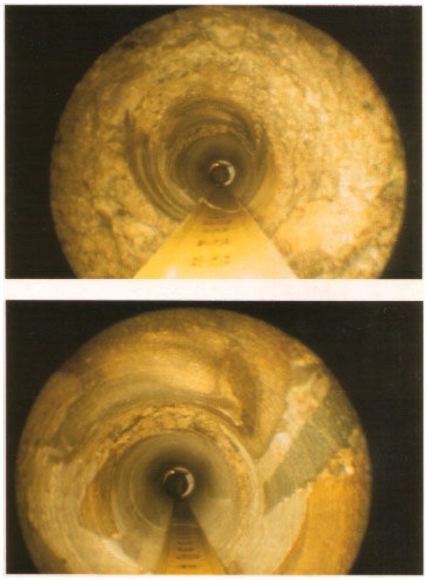

Figure 27: Endoscopic images

4.2.4 Flat jack tests

4.2.4.1 Introduction

Flat jack testing is a non-destructive, in-situ procedure for evaluating the stress in masonry. The flat jack technique [16] may be applied in two different ways, depending on the purpose:

– single flat jack testing is used to determine the local stress state in masonry;
– double flat jack testing is used for the in-situ determination of the mechanical behaviour of masonry under compression.

Flat jacks are devices consisting of two, generally semi-circular, steel plates welded together around their entire perimeter, except at two points that house injection tubes (figs. 28, 29). The flat jack is inserted in a cut made in the masonry with a circular saw (figs. 30 and 31). Oil is injected under pressure into the interspace between the two plates with a hydraulic pump (fig. 32), thereby applying pressure against the surfaces of the cut.

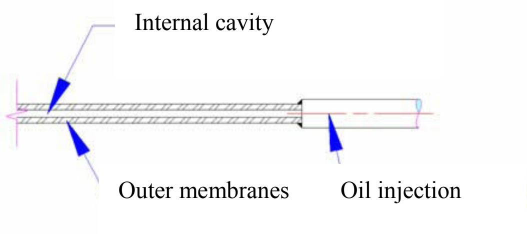

Figure 28: Flat jack cross-section

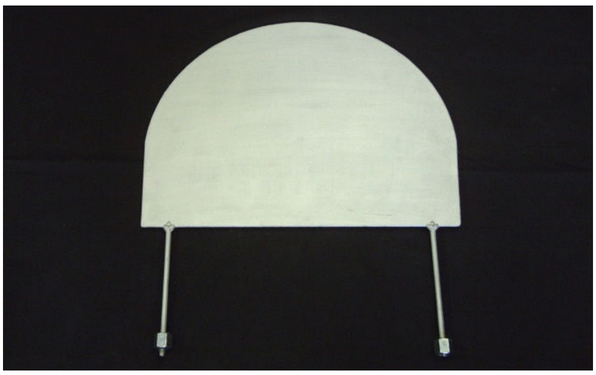

Figure 29: A flat jack

Figure 30: Circular saw

Figure 31: Inserting the jack into the cut

Figure 32: Hydraulic pump

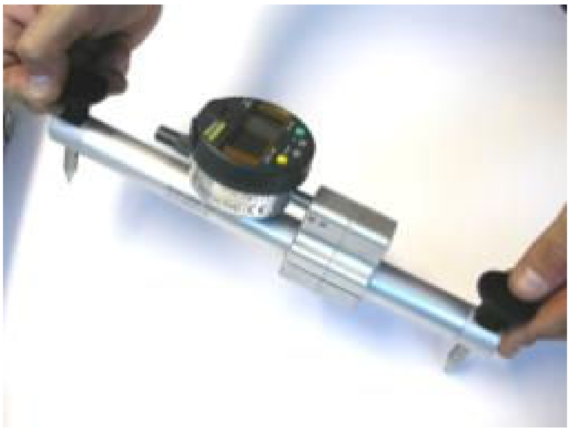

Figure 33: Deformometer

4.2.4.2 Single flat jack test

Before making the cut, the distance between three pairs of reference points straddling the cut is measured with a deformometer (figs. 33, 34). These distances decline after the cut is made because of the stress released, but return to the original values when the pre-existing stress is restored by inserting a flat jack in the cut opened in the wall. The oil pressure existing in the jack circuit (less a calibration constant) when the distance between the reference points is restored provides a measure of the initial stress level in the masonry wall.

Knowing the value of local stress in a masonry wall is useful when verifying the results of theoretical analyses or detecting differences in stress from one course to another when a structure is out of plumb or bears eccentric loads (fig. 35).

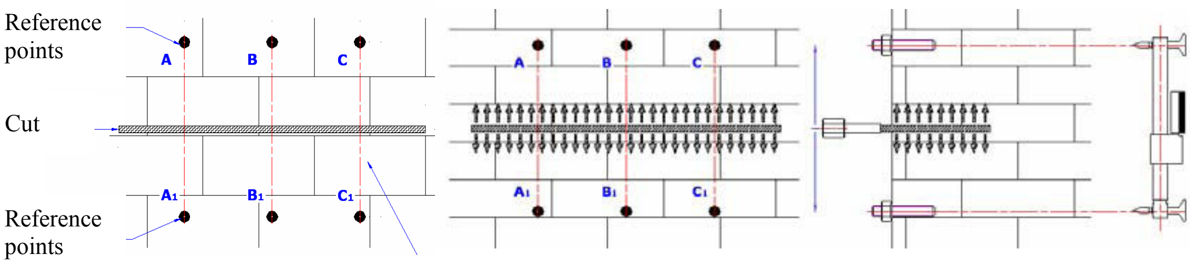

Figure 34: Single flat jack test instrumental set-up

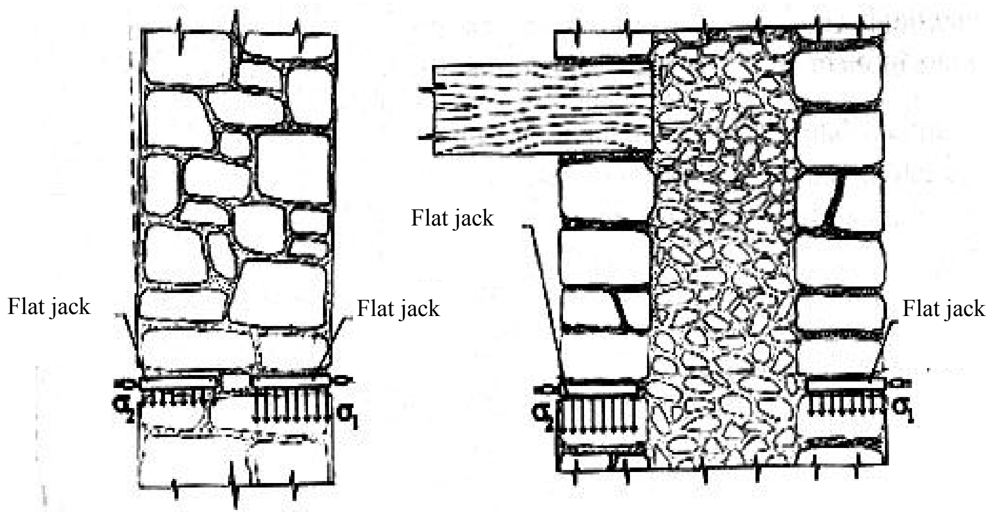

Figure 35: Application of results of the single flat jack test

4.2.4.3 Double flat jack test

In this test, the wall is cut at two sites about 50 cm apart and aligned vertically. Two flat jacks connected to the same pump are inserted into the cuts, while 3 pairs of reference points are determined on the surface of the wall in the vertical direction, and a fourth horizontally (fig. 36). The change in the relative distance between the two points in each pair is measured with a deformometer as the pressure in the jack is raised. The data gathered are used to plot the stress-strain diagrams for the masonry between the two cuts (fig. 37).

Under suitable conditions, the pressure may be increased until the masonry between the jacks is crushed to obtain a diagram showing the elastic modulus, the Poisson modulus and the mechanical strength of the masonry.

Flat jack tests are moderately destructive, and especially for stone masonry, fairly time-consuming (about one day per test). Moreover, the cut position must be very carefully determined.

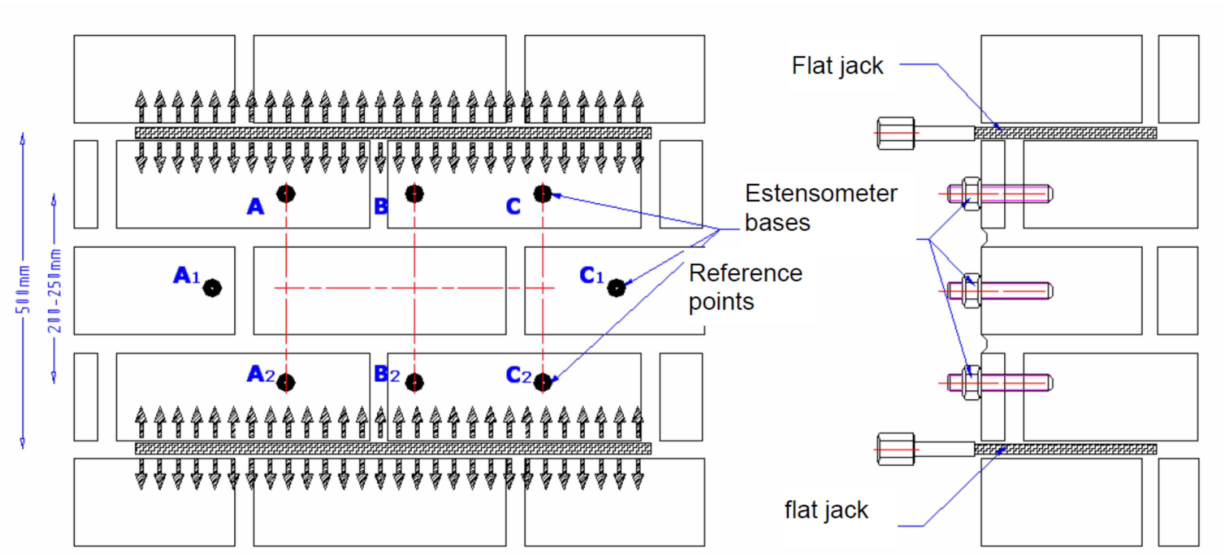

Figure 36: Double flat jack test instrumental set-up

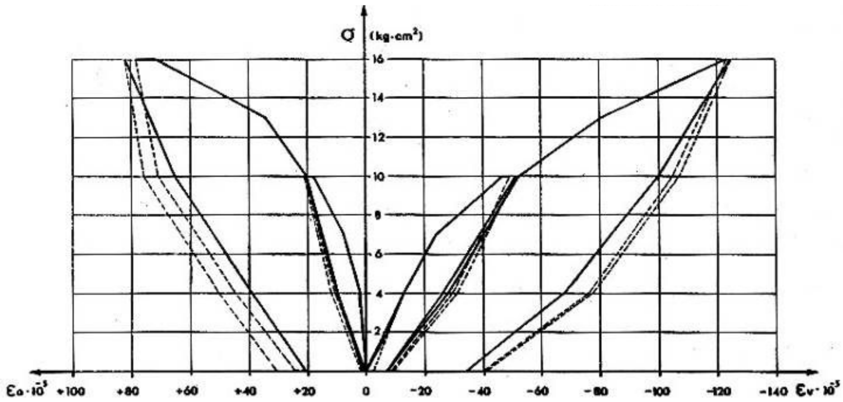

Figure 37: Example of a double flat jack test: masonry stress-strain diagram

Figure 38: Typical flat jack test sequence

4.3 Structural surveys with ground penetrating radar (GPR)

4.3.1 System description

Ground penetrating radar is a non-destructive inspection method used world-wide in applications ranging from mining to geophysical or structural surveys.

Basically, GPR is an electromagnetic technique based on the difference in the dielectric behaviour of materials. An antenna located on the surface of the structure emits a short pulse of electromagnetic energy. When that energy strikes an interface between layers of materials with different dielectric properties, part of the wave is reflected and the rest continues to the next interface. The splitting rate of this energy is determined by the relative dielectric properties of the media. The energy reflected can be detected by an antenna and analysed to identify and characterise features not externally visible.

The equipment required for radar surveys normally comprises three elements (fig. 39).

The control unit powers the antenna and receives and processes the reflected signals detected. The processed signal is subsequently transferred to the data-logger where it may be printed out in real-time if connected to a printer.

The antenna is the key component in the radar system. A number of types of antenna can be used, depending on survey requirements. Smaller high frequency antennae (500 MHz-2,6 GHz) are useful for low depth (to 0,5 m), high-resolution work, such as the location of rebar. Low frequency units (200–900 MHz) penetrate more deeply into the structure surveyed (1–8 m) but afford much lower resolution. As a rule, the receiving and transmitting circuits are built into in the same antenna (monostatic mode), although bistatic systems with two antennas, one for transmission and the other for reception, are also used.

The data-logger features real-time signal reception and display, as well as data recording. A video monitor is normally needed for signal adjustments.

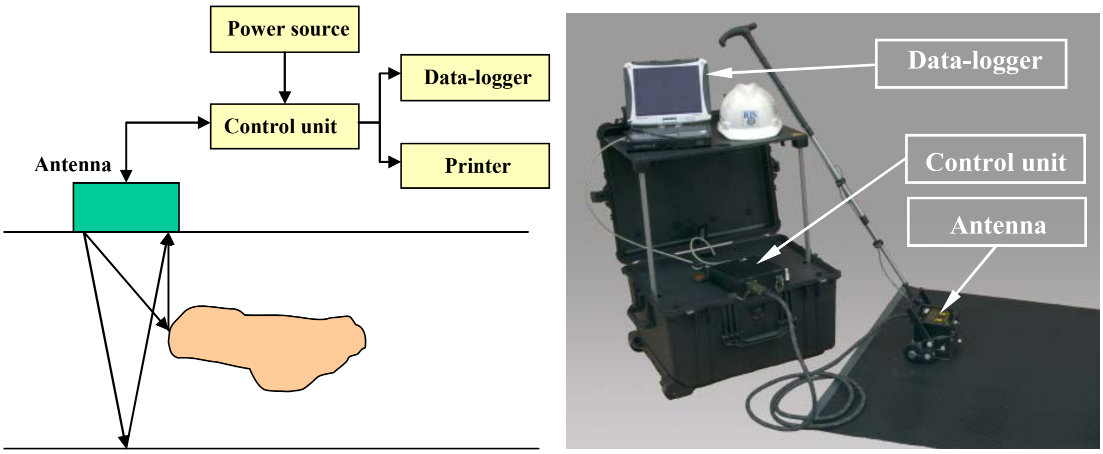

Figure 39: GPR flow chart (left) and radar equipment (right) used in non-destructive structural analysis

4.3.2 Applications

Radar systems can help solve many survey problems in a wide variety of fields, including structural engineering, railway engineering, road engineering, geological and environmental engineering, archaeology, the cultural heritage, forensic engineering, and reliability assessments. More specifically, in the non-destructive analysis of structures and buildings, GPR is used to:

- locate rebar and prestressing tendons in concrete structures;
- measure slab thickness;
- detect voids and other structural defects;
- identify wall cavities;
- detect the presence of water;
- explore the reconstruction of inner wall structures;
- locate pipes and other objects in walls and floors;
- inspect structural members in bridges, monuments, towers, tunnels, car park deck slabs and balconies.

4.3.3 Advantages and drawbacks compared to conventional techniques

The main advantages of GPR are:

- multiple applications;
- the innocuousness of wave frequencies for the subsurface, environment or people in the surrounds;

- multiple size options for convenient storage, shipping and use almost anywhere with scant limitations;
- wide range of antenna with frequencies of 200 to 2600 MHz to meet resolution and depth requirements.

Its most significant drawbacks are:

- performance limitations due to signal scattering in variable conditions;
- relatively high energy demand, which can pose problems in lengthy field surveys.

5 STATIC AND DYNAMIC LOAD TESTS FOR STRUCTURAL IDENTIFICATION

5.1 General

Static or dynamic load tests are usually performed to determine a building's response to environmental actions or applied loads.

Since load tests are not usually conducted through collapse, the information that can be drawn from them is directly related not to strength, but to structural stiffness. Nonetheless, if the stiffness findings concur with the output from a mathematical model, the strength calculated by the model more than likely also reflects the actual strength.

For existing buildings, "designing the load test" is of vital importance. In other words, the test preliminaries include:

- performing an accurate survey of the structural members to be tested;
- assessing the mechanical characteristics of the materials;
- modelling the structural members;
- verifying that the sum of the permanent and test loads is smaller than the expected member limit strength;
- evaluating the deflection or deformation induced by the test loads in the structural members.

The first four operations are geared to ensuring structural safety during the test, while the fifth addresses the use of suitable instruments in terms of accuracy, measuring range and real-time verification of the results.

5.2 Static load tests

A static load test consists of applying a load on a structure, usually a floor or a balcony, and measuring the resulting deflections.

While it is generally preferable to load bare structures, this is often an impossible aspiration in existing buildings, where non-structural elements such as flooring or partitions may collaborate with the structure tested, altering stiffness results. A similar situation arises when only a portion of a floor is loaded: the adjacent portions collaborate in bearing the load. In all such cases, collaboration must be factored into the respective model.

Since tests usually call for loading structures vertically, weights can be used for this purpose (figs. 40, 41). Hydraulic jacking would constitute an alternative procedure (fig. 42); in such cases, care needs to be taken to distribute the load on an appropriate portion of the slab to avoid punching. The strength of the upper floor, which acts as reaction structure, must be verified to ensure it will be able to resist the load applied upwards.

Figure 40: Load test with water containers

Figure 41: Load test with concrete blocks

Figure 42: Load test with hydraulic jacks

Figure 43: Electronic deflectometers

During load tests, structural deflection or sag is measured with mechanical or electronic instruments known as deflectometers (see fig. 43) as the relative distance between the structure and a reference point that remains in place during the test, such as the floor below. Deflection at any given load step is the difference between the measurement at that step and the initial value. Instruments must be placed at the points of maximum expected sag and, depending on the constraints, at intermediate points as well.

The maximum load is usually equal to the service load, further to the model results. Step loading is recommended to be able to verify the results in real time and prevent unexpected premature failures.

After the preliminary instrument readings are recorded, loads are applied step by step, monitoring the readings at each step until the maximum load is reached. The load should be increased gradually but not too slowly to prevent other factors such as temperature from affecting the results. The maximum load may be applied for a longer interval, around 15 minutes for instance, before unloading.

The final measurements are taken once the structure is fully unloaded. The outcome of a load test is positive when all of the following conditions are met:

- no damage has occurred.
- the results are consistent with the forecast.
- deformation rises more or less linearly with the load.
-

The magnitude of the residual deflection after unloading is a fairly small proportion (about 10%) of maximum deformation.

As noted in Chapter 8 of Handbook 1 [17], survival of a proof load test indicates only that the minimum bearing capacity of the structure tested is greater than the applied load. It does not ascertain the actual strength of the structure, nor does it provide a meaningful measure of structural safety. Nonetheless, proof load test results can be used with probabilistic analysis to obtain reliability estimates. An example of the procedure is discussed in Chapter 5 of this Handbook.

5.3 Dynamic tests

The aim of dynamic testing is to detect the dynamic characteristics of a structure: natural periods and stiffness (assuming that the mass is known), modes of vibration, damping and in some cases, the amplitude of the vibrations induced by environmental or artificial causes.

Dynamic tests may be performed instead of static tests, particularly to determine lateral stiffness or when the structure is expected to be exposed to significant dynamic actions.

In a dynamic test, environmental actions such as wind, traffic or earthquakes or forced actions such as impact or vibrators induce vibrations in the structure tested, measured by electronic instruments (accelerometers, seismometers and the like) and recorded on data-loggers (fig. 44).

The output of a dynamic test consists usually in a series of graphics recording the acceleration, velocity or displacement of the points of the structure where instruments are installed.

In order to obtain useful information, the data recorded are subsequently processed, by means of Fourier transform for instance, to define the dynamic characteristics of the structure (fig. 45).

In the Fourier spectrum of the recording of vibrations, each peak of amplification corresponds to a natural mode of vibration of the structure: so, for each peak the corresponding frequency is a natural frequency and the amplitude of the diagram allows evaluating the damping ratio of that natural mode of vibration.

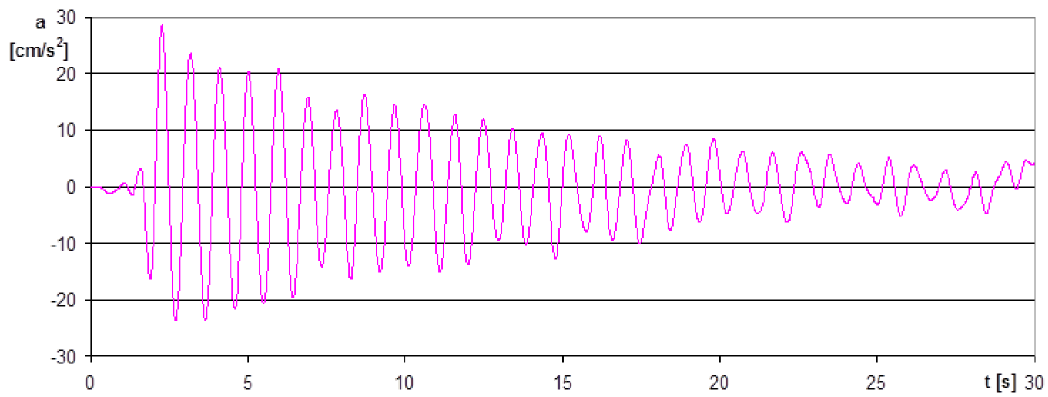

Figure 44: Acceleration recorded during a dynamic test

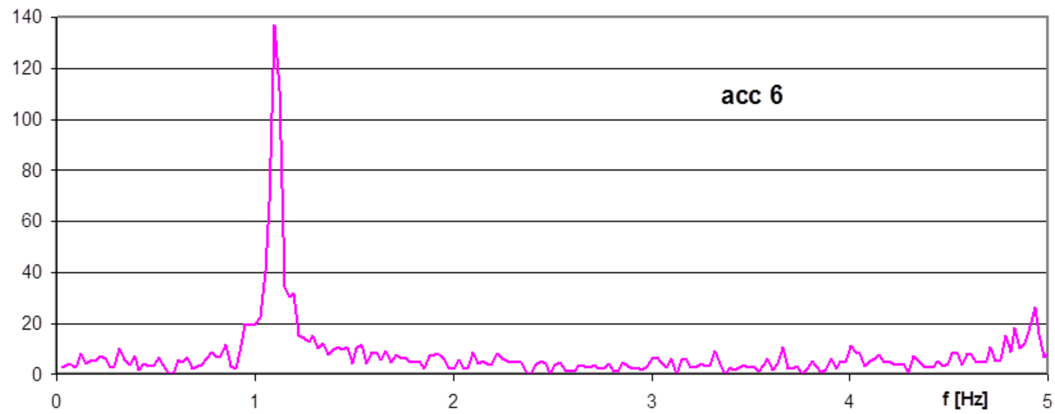

Figure 45: Fourier amplitude for the data in figure 44

The results of a dynamic test, frequency and damping ratio of the first natural modes, may be used not only to identify the model of the structure and thus to improve the evaluation of the response of the structure to loads and external actions, but also the occurrence and the evolution of damage in structural members or in the whole structure.

6 MONITORING

6.1 Monitoring failures in RC and masonry structures

6.1.1 Introduction

The variations over time in building failure should be monitored for a number of reasons. Steep rises in instability-induced damage calls for speedy repair, inasmuch as equilibrium can be expected to be lost entirely. Inversely, if no variations are observed, the cause of instability has in all likelihood plateaued, which would justify repair work at that time. Information on variability patterns is, then, essential to decision-making.

Structures must be monitored for quite a long time, at least a full year, to distinguish the actual variations in instability from changes induced by cyclical events such as seasonal variations in temperature.

6.1.2 Monitoring instruments

Since the visible effects of instability in buildings generally consist of cracks or rotations, variations can be determined by monitoring crack width or rotations.

Crack monitoring refers not to measuring the crack width, but its variations over time. Inexpensive and convenient mechanical instruments may be used for this purpose, although electronic instruments featuring continuous monitoring, remote control data storage and other functions are also available.

The simplest and least expensive instrument for crack monitoring is the crack meter (fig. 46). It consists of two plastic plates positioned on either side of the crack. One plate has a reference mark and the other a measuring grid. When crack width varies, the distance between the two plates varies accordingly and can be measured on the grid.

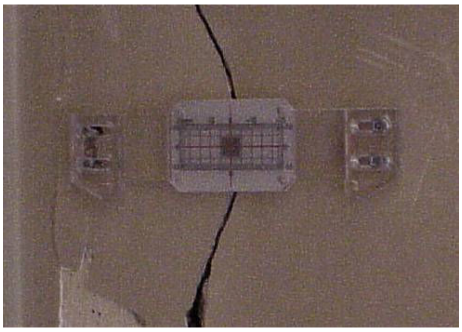

Figure 46: Crack meter

Another mechanical instrument for monitoring cracks is the removable deformometer (fig. 47), which measures the relative movements between reference pins set on each side of the crack (fig. 48). Such deformometers are non-invasive and accurate (to a precision of 1/100 mm) and can be used long-term monitoring, providing the pins are not removed.

Figure 47: Removable deformometer

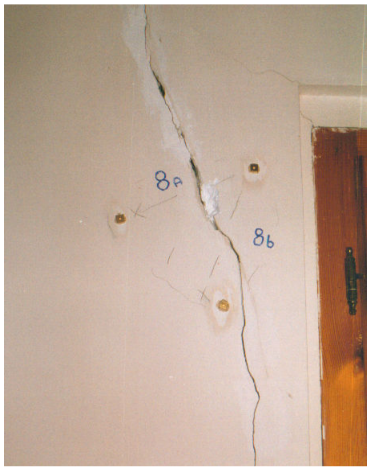

Figure 48: Reference pins for removable deformometer

Electronic monitoring equipment generally includes:

- transducers:
  - displacement transducers;
  - inclinometers;
  - special transducers;
  - temperature transducers;

- a data acquisition system.

Crack width is monitored with inductive displacement transducers (fig. 49), whose frame houses an electronic circuit and a sliding core. Such instruments measure the relative displacement between the position of the frame and the position of the core; if the frame and core are set on either side of a crack, it acts like a crack meter (fig. 50).

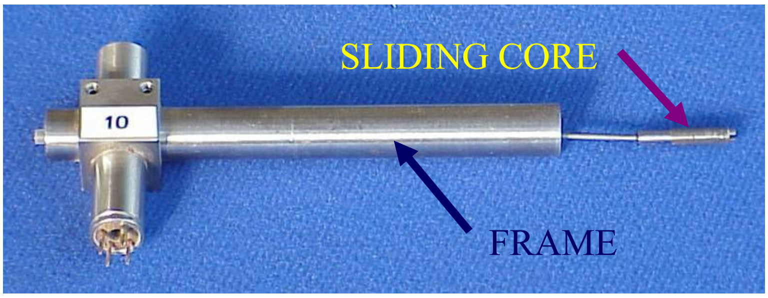

Figure 49: Inductive displacement transducer

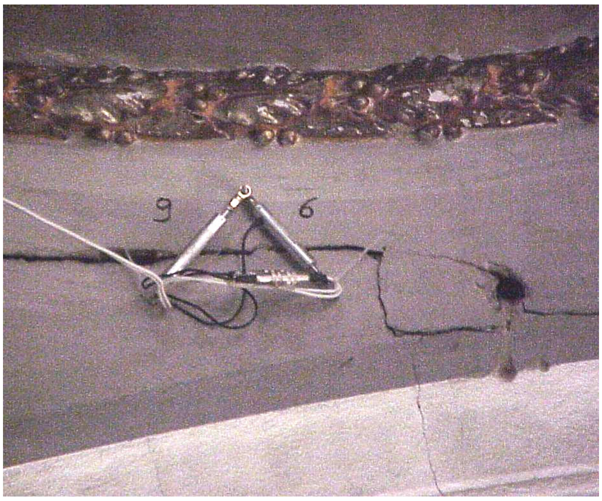

Figure 50: Displacement transducers used to monitor two-axis variations in crack width

6.1.3 Results of monitoring

Monitoring data must be collected and processed at a frequency in keeping with the rate of variation. If no signs of imminent danger are observed, the results can be analysed over fairly long periods of time, about a year, to record cyclical events such as the effect of temperature.

The graphs in figures 51, 52 and 53 plot the crack width data collected for one year. The blue line depicts the width measurements, which appear to indicate cyclical variations, while the probable effect of instability on width is shown by the red line. The graph in figure 51 is typical of a stabilised system: repair work may be performed or monitoring may continue with less frequent measurement readings.

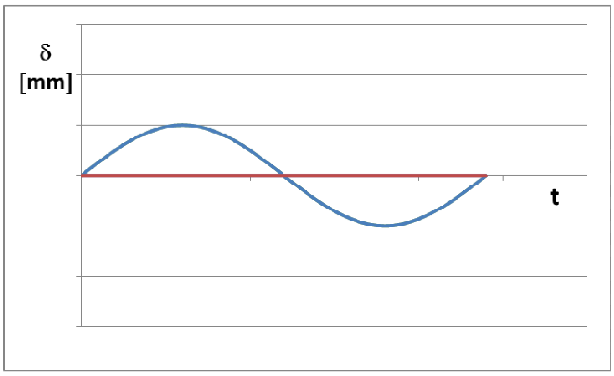

Figure 51: Crack width over one year: stable situation

Figure 52 plots steadily rising crack width. In such cases, depending on the scale of the increase, short-term action may be required to eliminate the causes of the crack and repair the building, or simply continue monitoring. The graph in figure 53 represents a situation that is gradually stabilising and calls for ongoing monitoring.

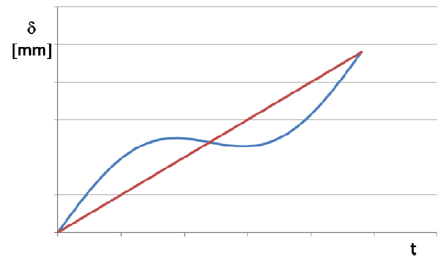

Figure 52: Crack width over one year: upward trend

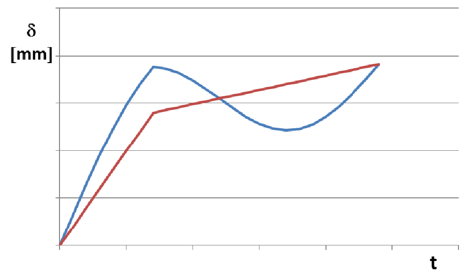

Figure 53: Crack width over one year: gradual stabilisation

6.2 Remote radar interferometric measurement of displacement and vibrations

6.2.1 Overview

Microwave interferometry is an innovative technology for remote static and dynamic monitoring of bridges and structures such as buildings, historical monuments and towers. Interferometric radar equipment can be operated remotely and need not be in direct contact with the monitoring target. The radar generates an ongoing flow of deformation maps as opposed to the periodic information provided by current contact sensors. Measurement speeds and accuracy are also higher than in conventional techniques. The displacement response of several points on a structure can be measured simultaneously with a precision on the order of 0,01 mm and a maximum sampling frequency of 100 Hz. Such high sensitivity is the result of interferometric techniques that measure displacement by comparing subsequent readings of the waves reflected off the object.

6.2.2 System description

The system consists of the following elements (fig. 54):

– sensor module fitted with a signal transmitter and receiver, view finder and horn antenna;
– tripod and 3-D rotating head;
– processing unit;
– power supply.

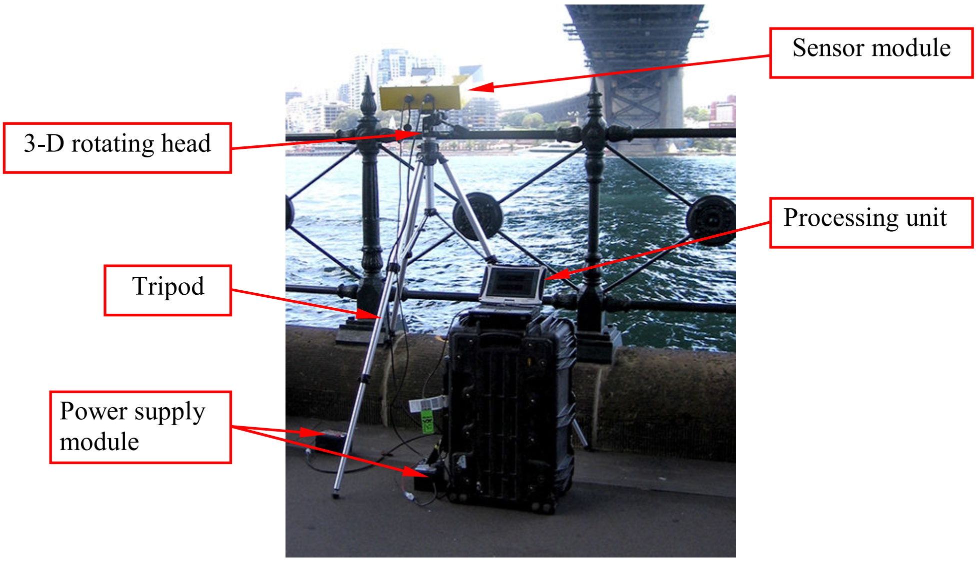

Figure 54: Microwave interferometer

The sensor module emits an electromagnetic wave that first strikes and is then reflected off all the targets radiated by the antenna. The information received from the reflected wave is used to record the variation in the position of the measuring point with respect to the preceding reading (fig. 55). Microwave interferometers measure line of sight displacement (d_{los}). Assessment of the actual displacement calls for information on the acquisition geometry and the real direction of movement.

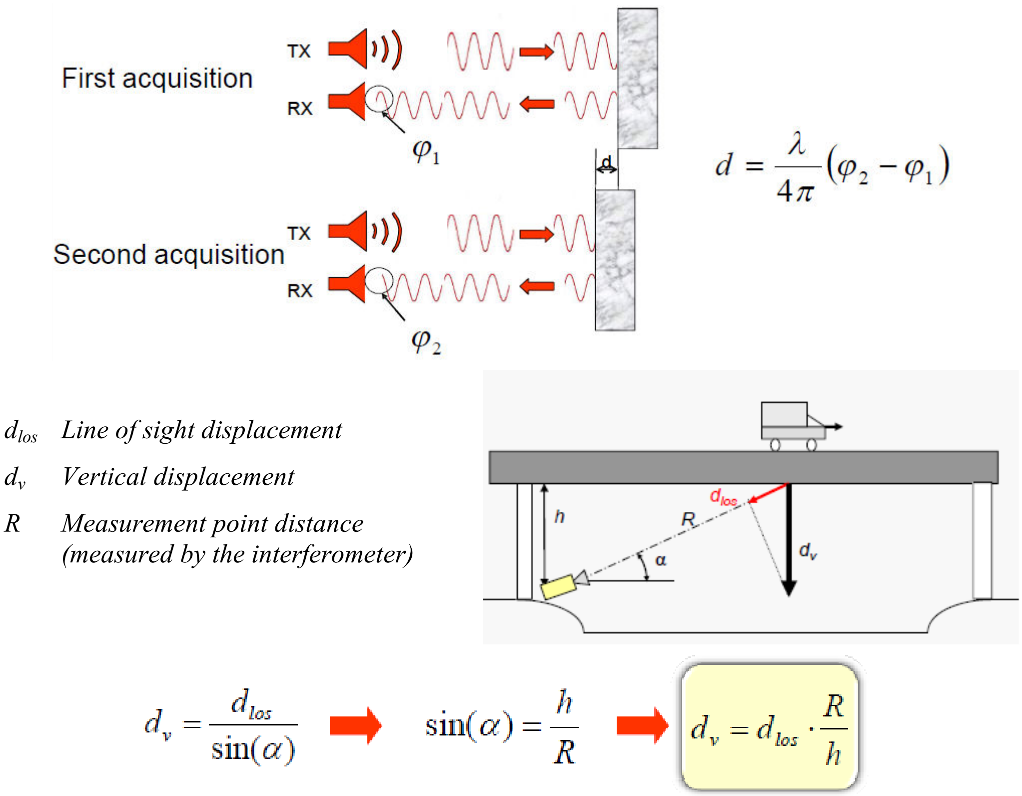

Figure 55: Remote radar interferometric monitoring of structural movements and deformation [18]

The fields of application for remote monitoring based on radar interferometry include:

static problems:
- structural load testing;
- structural displacement and collapse hazards;
- conservation of the cultural heritage;

dynamic problems:
- structural resonance frequency measurement;
- modal shape analysis of the structure;
- deformation monitoring in real time.

6.2.3 Advantages and drawbacks compared to conventional techniques

The main advantages of the microwave interferometer are:

- remote sensing at large distances (up to 1 km) with no need for equipment to be installed on the monitored structure;
- measurement precision of up to 0,01 mm;
- real-time simultaneous deformation mapping;
- simultaneous static and dynamic monitoring;
- structural vibration sampling up to 100 Hz;
- 24/7 operation, regardless of weather conditions;
- delivery of direct displacement measurements, not derived quantities;
- remote operation, surmounting accessibility difficulties and obviating any need to modify the service conditions of the structure.

The primary drawback is the high initial cost of such gear.

6.3 Fibre optic sensor technology

6.3.1 Introduction and background

Fibre optic sensors operate on the principle according to which variations in external parameters can induce changes in the properties of the light guided by the optic fibre. Such external parameters may include strain, displacement, pressure or temperature, among others [19].

From a number of standpoints, fibre optic sensors are ideal for infrastructure monitoring. Their durability, flexibility, stability and immunity to external perturbations make them especially suitable for the long-term monitoring of any kind of infrastructure. In contrast, the small size and relative fragility of bare fibres make such sensors apparently incompatible with the hostile environment prevailing in most infrastructures. Such incompatibility is only apparent, however, because with suitable sensor and accessory design and protection, fibre optic sensors can improve the performance of conventional approaches in terms not only of measurement resolution but also of service life.

6.3.2 Sensor configuration and sensor types

Optical fibres, which usually consist of three concentric layers - fibre core, cladding and jacket -, are dielectric devices used to confine and guide light. Most optical fibres used in sensing applications have silica glass cores and claddings. The refractive index is lower in the cladding than in the core to confine the propagation of light along the core fibre. The core diameter varies from 10 to 100 μm, while the cladding diameter may measure up to 125 μm. The plastic or metal outer layer of a fibre optic sensor, called the jacket, which usually measures 250 µm in diameter, provides the fibre with appropriate mechanical strength and protects it from damage and moisture.

A wide variety of fibre optic sensors is available for civil engineering applications: interferometric sensors (Fabry-Perot sensors and low-coherence interferometry), distributed fibre sensors (Rayleigh, Raman and Brillouin scattering), grating-based systems (Bragg and long period) and luminescent and plastic fibre sensors. Each optical fibre sensor is characterised by specific sensing attributes and each has advantages and drawbacks. The most suitable sensor for each application depends on a number of factors, including the sensitivity required, manufacturing cost, and the need or otherwise for absolute measurements (i.e., direct relationship between the sensor signal and the measurement) [20]. The intrinsic characteristics of fibre Bragg grating (FBG) technology have proven to be particularly well suited to structural monitoring, making it one of the most widespread optical technologies, in civil structures as well as in other applications [21].

Fibre Bragg gratings are obtained by creating periodic variations in the refractive index of the core of an optical fibre. When light travels down the fibre at what is known as the Bragg wavelength, the light reflected by the varying zones of refractive indices is both in phase and amplified [21].

Figure 56, adapted from [21], shows the transmitted and reflected spectrums generated by the FBG fibre optic system used by [22] to take strain measurements along the length of embedded bars, thereby monitoring the load transfer from the bars to the concrete in pull-out tests.

Figure 56 also shows the variation in the wavelength reflected (blue line) off the FBG sensors. When strain is induced in an FBG sensor due to mechanical stress, thermal expansion or a combination of the two, its grating pitch shifts in such a way that its reflected wavelength varies proportionally to the strain. Therefore, strain can be found by measuring the variation in the wavelength reflected [23].

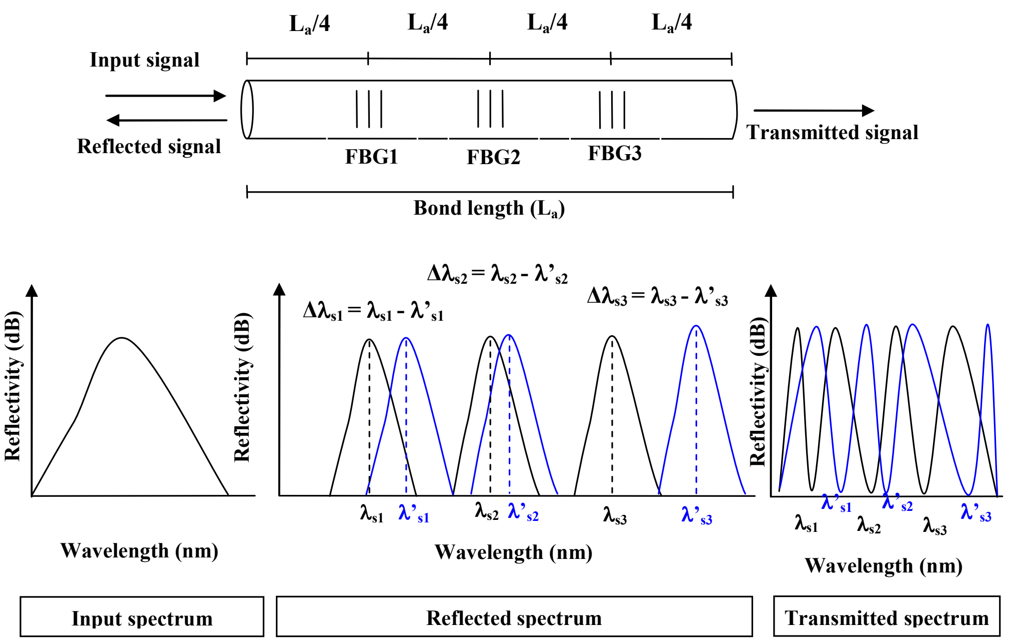

Figure 56: Transmitted and reflected spectrums generated by the FBG fibre-optic sensing system used in [22], adapted from [21].

6.3.3 *Monitoring applications*

Fibre optic sensors have been successfully applied to many civil engineering structures to monitor a series of parameters. Bridges, especially concrete bridges, are the civil structures most widely monitored by fibre optic sensors. Several types of fibre optic sensing solutions and different network strategies have been used both in research and industrial applications for bridge monitoring systems. A number of bridge parameters, such as strain, displacement, pressure, load, acceleration, rotation, temperature, concrete cracking and reinforcement corrosion monitoring have been measured with embedded or externally installed fibre optic sensors [19].

One of the parameters most commonly measured in bridge behaviour assessment is strain. FBG sensors have been the standard solutions for monitoring local strain in bridges, by installation on steel bars embedded in concrete or critical structural elements such as prestressed tendons, FRP reinforcement, cables, ties or bracing bars [21].

Pipelines, tunnels and dams are other examples of infrastructures where fibre optic sensors have been used to monitor the behaviour of their various components. These sensors have also been successfully applied to buildings to measure relative displacement, strain and temperature.

6.3.4 *Advantages and drawbacks compared to conventional techniques*

The main advantages of fibre optic sensor technology are that they [21]:

– can measure different parameters such as strain, displacement, vibration, pressure and temperature;
– are compact, lightweight and, in general, minimally invasive;
– act as both sensing elements and signal propagation conduits;
– are immune to electromagnetic interference and ground loops;
– are water- and corrosion-resistant;
– have excellent measuring resolution and range;
– can be multiplexed, i.e., several sensors can be used in the same fibre.

Several drawbacks to fibre optic sensors limit their range of application. Optical fibres are fragile and may break during packaging, shipping and, especially, installation on host structures [24]. A further significant disadvantage is the need for costly optoelectronic devices.

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CHAPTER 2 - ANNEX A

EVALUATION OF RESULTS

A.1 GENERAL

The evaluation of statistical data representing a random sample taken from a particular population is frequently the first step in the assessment of existing structures. In the following discussion, the notions of a general population and the random samples taken from it are introduced, along with the definitions of standard sample characteristics. Emphasis is placed on moment characteristics that usually provide the initial background information for the specification of a theoretical population model. The sample characteristics normally used in engineering and science describe the central tendency, dispersion, asymmetry (skewness) and kurtosis (peakedness) of the distribution of statistical data. The general rules and computational techniques used for determining the characteristics of a single random sample, as well as for the combination of two random samples, are illustrated in the examples below.

The notions of population and random sample are extremely important for the due interpretation and analysis of statistical data. The population, or “the universe”, is the totality of items under consideration. A population may be finite (N sampling units) or infinite. Rather than examining the entire group of N units, a small fraction of the population, i.e., a sample of n units, may be examined instead. Although the precise definition of a population is often difficult to obtain, it must be established if the outcome of statistical research is to be correctly interpreted [A.1, A.2]. An excellent description of the basic technique is given in [A.3, A.4] and a short review is provided in [A.5]. The correct terminology and procedures are available in international standards [A.6, A.7, A.8].

A sample is one or more units taken from a population to obtain information on that population. It may serve as a basis for decision-making about, or for determining the process that generated, the population. The term “random sample” refers to samples that are taken from a population in such a way that all possible units have the same probability of being taken. The number of sampling units, called sample size, abbreviated as n, may vary considerably. As a rule, samples are classified as very small (n<10), small (n<30), large (n>30) or very large (n>100). Obviously, the larger the size, the more representative the sample. The sampling procedure is as important as size, however.

If a sample is representative of a population, meaningful conclusions can often be inferred about the population by analysing the sample. This area of statistics is called inductive statistics, or statistical inference. The area that seeks only to describe and analyse a given sample is called descriptive, or deductive, statistics. This annex addresses the latter.

Example A.1

A structure consists of 70 members of the same type. A random sample of 10 members is taken from the population of 70 units using a table, or a generator of random numbers ranging from 1 to 70. The sample is then chosen by taking the units whose serial numbers are equal to the ten random numbers generated.

A.2 MEASURES OF CENTRAL TENDENCY

The basic measure of the central tendency of a sample is the sample mean, m_X, given by

$$m_X = \frac{1}{n}\sum_1^n x_i \tag{A.1}$$

where:
x_i = sample units.

If the sample units are ranked in ascending order, the subscript i is generally written as (i), and the units are denoted $x_{(i)}$.

Another measure of central tendency is the median, $\tilde{m}_X$, defined as the point that separates the ordered sequence of data into two parts, such that one half of the data are less and the other half greater than the median value.

Example A.2

A random sample of ten concrete strength measurements yields the following values:
x_i = {27; 30; 33; 29; 30; 31; 26; 38; 35; 32}(in MPa) or, ranked by value:
$x_{(i)}$ = {26; 27; 29; 30; 30; 31; 32; 33; 35; 38}(in MPa).

The sample mean m_X and the median $\tilde{m}_X$ are defined as follows:

$$m_X = \frac{1}{n}\sum_1^n x_i = 31{,}1\ \text{MPa}; \quad \tilde{m}_X = \frac{1}{2}\left(x_{(5)} + x_{(6)}\right) = 30{,}5\ \text{MPa}$$

A.3 MEASURES OF DISPERSION

The basic measure of dispersion is called the variance:

$$s_X^2 = \frac{1}{n}\sum_1^n \left(x_i - m_X\right)^2 \tag{A.2}$$

In practice, its square root, known as standard deviation, s_X, is more widely used.

Another measure of dispersion, the coefficient of variation, is often applied in engineering and science:

$$v_X = \frac{s_X}{m_X} \tag{A.3}$$

This coefficient is a measure of relative dispersion normalised to the sample mean, m_X. It is used in engineering when m_X is not very small. Where the sample mean is relatively small, standard deviation is more suitable.

Yet another measure of dispersion is sometimes used for very small samples ($n \leq 10$). Called the sample range, it is simply the difference between of the largest and smallest sample unit values, $x_{(n)} - x_{(1)}$.

Lastly, the mean or average deviation, MD, defined as the mean of the differences $|x_i - m_X|$ may also be used:

$$MD = \frac{1}{n}\sum_{i=1}^{n}|x_i - m_X| \tag{A.4}$$

Example A.3

The variance of the sample given in Example A.2: x_i = {27; 30; 33; 29; 30; 31; 26; 38; 35; 32}(in MPa) is:

$$s_X^2 = \frac{1}{n}\sum_{1}^{n}(x_i - m_X)^2 = 11{,}69\ \text{MPa}$$

The standard deviation is therefore $s_X = \sqrt{11{,}69} = 3{,}42\ \text{MPa}$

Example A.4

The coefficient of variation for the data in the random sample given in Example A.2, x_i=27; 30; 33; 29; 30; 31; 26; 38; 35; 32} (in MPa) is:

$$v_X = \frac{3{,}42}{31{,}1} = 0{,}11 = 11\%$$

Example A.5

Ranking the Example A.2 data in ascending order, $x_{(i)}$ = {26; 27; 29; 30; 30; 31; 32; 33; 35; 38}(in MPa), the range of variation and the mean deviations are:

$$x_{(n)} - x_{(1)} = 38 - 26 = 12\ \text{MPa};\ MD = \frac{1}{n}\sum_{i=1}^{n}|x_i - m_X| = 2{,}72\ \text{MPa}$$

A.4 MEASURES OF SHAPE

Asymmetry or skewness and kurtosis or peakedness (the extent to which a frequency distribution is concentrated about the mean) are used less frequently than the central tendency (mean, m_X) and dispersion (variance, s_X^2) measures. They nonetheless provide valuable information about the nature of the sample, notably the distribution of observations to the left and right of the mean and the concentration of observations around the mean. This information may be extremely useful for determining the appropriate theoretical probability distribution.

The following are the most widely used measures of shape. The coefficient of asymmetry is defined in terms of the third order moment, i.e.:

$$a_X = \frac{1}{ns_X^3}\sum_{i=1}^{n}(x_i - m_X)^3 \tag{A.5}$$

Similarly, the coefficient of kurtosis is related to the fourth order moment:

$$e_X = \frac{1}{ns_X^4}\sum_{i=1}^{n}\left(x_i - m_X\right)^4 - 3 \tag{A.6}$$

Note that the coefficients of asymmetry and kurtosis should be close to zero for samples taken from populations characterised by a normal distribution.

The coefficient of asymmetry is positive when more sample data is on the left of the mean and negative when more data is on the right. The coefficient of kurtosis is positive when the sample data is located mostly in the vicinity of the mean and negative when the data is distributed more uniformly. Both these characteristics (skewness, a_X, and peakedness, e_X) are heavily dependent on abnormal deviations of some sample units (outliers), or errors, particularly in the case of small samples (n < 30). In such cases their interpretation may be highly uncertain (and subject to statistical uncertainty due to limited data).

Example A.6

Again using the data from Example A.2 where x_i = {27; 30; 33; 29; 30; 31; 26; 38; 35; 32} (in MPa), the coefficients of asymmetry and kurtosis are:

$$a_X = \frac{1}{ns_X^3}\sum_{i=1}^{n}\left(x_i - m_X\right)^3 = 0{,}46$$

$$e_X = \frac{1}{ns_X^4}\sum_{i=1}^{n}\left(x_i - m_X\right)^4 - 3 = -0{,}44$$

The positive coefficient of asymmetry indicates that more observations are on the left of the mean (in fact, 6 of the 10 values are on the left of the mean). The slightly negative coefficient of kurtosis indicates low peakedness (the observed values seem to be distributed slightly more uniformly than in a normal distribution). Note that the sample used is very small (10 values only) and the values obtained for a_X and e_X may be inaccurate.

An empirical relationship (known as the Pearson coefficient of skewness) exists between skewness, a_X, the mean, m_X, the median, $\tilde{m}_X$, and standard deviation, s_X, as follows:

$$a_X \approx 3\left(m_X - \tilde{m}_X\right)/s_X \tag{A.7}$$

Using the results from examples A.2 and A.3, $m_X = 31{,}1\,\text{Mpa}$, $\tilde{m}_X = 30{,}5\,\text{MPa}$ and $s_X = 3{,}42\,\text{Mpa}$, it follows that:

$$a_X \approx 3(31{,}1 - 30{,}5)/3{,}42 = 0{,}53$$

This seems to be reasonably close to the coefficient of skewness found earlier (a_X=0,46). It also proves the intuitive expectation that if the median, $\tilde{m}_X$, is less than the mean, m_X, skewness, a_X, should be positive. Consequently, more data is located left than right of the mean.

A.5 ORDINARY AND CENTRAL MOMENTS

Most of the sample measures described above form part of the so-called moment characteristics, which are based on ordinary or central moments of the distribution. The ordinary moment (about the origin) of order l (l = 1, 2, 3, …) is defined as the arithmetic mean of the sum of l-powers:

$$m_l^* = \frac{1}{n}\sum_1^n x_i^l \tag{A.8}$$

The central moment (about the mean) of order l is:

$$m_l = \frac{1}{n}\sum_{i=1}^n \left(x_i - m_X\right)^l \tag{A.9}$$

The moment characteristics can be then defined as follows.

$$m_X = m_1^* \tag{A.10}$$

$$s_X = \sqrt{m_2} \tag{A.11}$$

$$a_X = \frac{m_3}{m_2^{3/2}} \tag{A.12}$$

$$e_X = \frac{m_4}{m_2^2} - 3 \tag{A.13}$$

The following relationships between the ordinary and central moments may proved to be useful in numerical calculations:

$$m_2 = m_2^* - m_X^2 \tag{A.14}$$

$$m_3 = m_3^* - 3m_X m_2^* + 2m_X^3 \tag{A.15}$$

$$m_4 = m_4^* - 4m_X m_3^* + 4m_X^2 m_2^* - 3m_X^4 \tag{A.16}$$

When computers are used to assess statistical samples, equations (A.14) to (A.16) are not directly used.

A.6 COMBINATION OF TWO RANDOM SAMPLES

Two random samples taken from the same population may on occasion have to be combined on the grounds of their characteristics only because the original observations, x_i, are not available. Note that only uniform samples of the same origin (taken from a single population under the same conditions) can be combined. Failure to observe this important rule would lead to incorrect results.

Assume that a first sample of size n_1 is characterised by m_1, s_1, and a_1, and a second sample of size n_2 by m_2, s_2, and a_2. Only three basic characteristics are considered here (the coefficients of kurtosis are rarely available for combined samples).The characteristics for the combined sample of size n can be determined from the following expressions:

$$n = n_1 + n_2 \tag{A.17}$$

$$m = \frac{n_1 m_1 + n_2 m_2}{n} \tag{A.18}$$

$$s^2 = \frac{n_1 s_1^2 + n_2 s_2^2}{n} + \frac{n_1 n_2}{n^2}(m_1 - m_2)^2 \tag{A.19}$$

$$a = \frac{1}{s^3}\left[\frac{n_1 s_1^3 a_1 + n_2 s_2^3 a_2}{n} + \frac{3 n_1 n_2 (m_1 - m_2)(s_1^2 - s_2^2)}{n^2} - \frac{n_1 n_2 (n_1 - n_2)(m_1 - m_2)^3}{n^2}\right] \tag{A.20}$$

Note also that standard deviation, s, depends not only on the standard deviations of the two initial samples, s_1 and s_2, but also on their means. Similarly, skewness, a, also depends on the lower order measures (means and standard deviations). The relationship for kurtosis is not shown, for it is not normally used.

If the original data are available for both samples, they can be analysed as if they were only one; relationships (A.17) to (A.20) can then be used to verify the results. The most important consideration is to ensure that both samples are taken from the same population.

Example A.7

An example of the practical application of equations (A.17) to (A.20) is shown below.

Samples	***n***	***m***	***s***	***a***	***v***
Sample 1	10	30.1	4,4	0,5	0,15
Sample 2	15	29,2	4,1	0,5	0,14
Combined	25	29,56	4,25	0,53	0,14

Varying the number of sample units may affect the characteristics of the resulting combined sample. An Excel spread sheet has been developed for such cases.

On occasion, the size of one of the samples, n_1 for instance, may not be known, and information may be available only for the other two characteristics, m_1 and s_1. This situation typically arises when earlier data with m_1 and s_1 are to be updated with new observations from a sample with size n_2 and characteristics m_2 and s_2. Here, the Bayesian approach may be used to assess the unknown value, n_1, and the respective degree of freedom ν_1. Guidelines on how to proceed in such cases are discussed below very generally, with no mathematical justification.

Pursuant to the Bayesian concept [A.1, A.3], unknown value n_1 and degree of freedom ν_1 may be found from the relationships between the coefficients of variation for the mean and standard deviation $V(\mu)$ and $V(\sigma)$ (the parameters μ and σ are regarded as random variables in the Bayes procedure), for which the following equation holds:

$$n_1 = \left[\frac{s_1}{m_1 V(\mu)}\right]^2, \quad \nu_1 = \frac{1}{2V(\sigma)^2} \tag{A.21}$$

Both unknown variables n_1 and ν_1 may be assessed independently (generally $\nu_1 \neq n_1 - 1$), depending on previous experience respecting the degree of uncertainty applicable to the estimator of the mean, μ, and the standard deviation, σ, of the population. Note that for a new sample, $\nu_2 = n_2 - 1$.

When sample size n_1 and degree of freedom ν_1 are estimated, the degree of freedom ν is [A.6, A.9]:

$$\nu = \nu_1 + \nu_2 - 1 \;\; \text{if } n_1 \geq 1, \;\; \nu = \nu_1 + \nu_2 \;\; \text{if } n_1 = 0. \tag{A.22}$$

The resulting size of combined sample n with mean m is found with equations (A.17) and (A.18); standard deviation s is found with a modification of equation (A.19), namely:

$$s^2 = \frac{1}{\nu}\left(\nu_1 s_1^2 + \nu_2 s_2^2 + \frac{n_1 n_2}{n^2}(m_1 - m_2)^2\right). \tag{A.23}$$

The above relationship may be easily applied using an Excel spread sheet or other software tools.

Example A.8

Assume the following information on the strength of a previous batch of a given type of concrete:

m_1=30,1 MPa; $V(\mu)$=0,50; s_1=4,4 MPa; $V(\sigma)$=0,28.

Further to equation (A.21), unknown characteristics n_1 and ν_1 are:

$$n_1 = \left(\frac{4{,}4}{30{,}1}\frac{1}{0{,}50}\right)^2 \approx 0\,;\;\; \nu_1 = \frac{1}{2 \times 0{,}28^2} \approx 6\,.$$

Consequently, the values used hereafter for the size and degree of freedom for that sample are: n_1=0 and ν_1=6.

New specimens from the same type of concrete are tested to verify its quality, with the following results:

n_2=5, ν_2=n_2–1=4, m_2=29,2 MPa, s_2=5,6 MPa.

Using equations (A.17), (A.18), (A.22) and (A.23), the updated values are:

n=0+5=5

ν=6+4=10

$$m = \frac{0 \cdot 30{,}1 + 5 \cdot 29{,}2}{5} = 29{,}2\,\text{MPa}$$

$$s^2 = \left[6 \cdot 4{,}4^2 + 4 \cdot 5{,}6^2 + \frac{0 \cdot 5}{5}(30{,}1 - 29{,}2)^2\right]/10 = 4{,}5^2\ \text{MPa}^2$$

Using the previous information, then, the standard deviation for the new measurements could be lowered from s=5,6 MPa to s=4,5 MPa.

Nonetheless, combining previous with current information may not always lead to favourable results. If, for instance, the coefficients of variation are $w(\mu)$=0,2 and $w(\sigma)$=0,6, according to equation (A.21), unknown characteristics n_1 and ν_1 are:

$$n_1 = \left(\frac{4{,}4}{30{,}1}\frac{1}{0{,}20}\right)^2 \approx 1,\;\; \nu_1 = \frac{1}{2 \times 0{,}6^2} \approx 1$$

In this case:

n=1+5=6

ν=1+4–1=4

$$m = \frac{1 \cdot 30{,}1 + 5 \cdot 29{,}2}{5} = 29{,}35\,\text{MPa}$$

$$s^2 = \left[1 \cdot 4{,}4^2 + 4 \cdot 5{,}6^2 + \frac{1 \cdot 5}{6}(30{,}1 - 29{,}2)^2\right] / 4 = 6{,}03^2\ \text{MPa}^2 .$$

In this second case, the mean rises slightly from 29,2 to 29,35, with a substantial increase in the standard deviation, from 5,6 to 6.03. This is an extreme case, however, resulting from the unfavourable estimates of n_1, ν_1 and ν further to equations (A.21) and (A.22). In practical applications these equations should be applied with caution, particularly in extreme cases such as the above example. In connection with this warning, an important condition mentioned at the beginning of this section should be stressed. Only samples that are irrefutably taken from the same population can be used to combine or update statistical data; otherwise the results of the combination of two random samples may lead to incorrect results.

A.7 NOTE ON TERMINOLOGY AND SOFTWARE

Standards such as ISO 3534 and Excel, Mathcad and Statistica software use slightly different terminology and definitions for basic moment characteristics.
Two alternative expressions are commonly found for dispersion.

– The measure called “the sample standard deviation”, “the standard deviation of a sample”, or “the population standard deviation” (when n is the population size), is defined as follows:

$$s_X = \sqrt{\frac{1}{n}\sum_1^n (x_i - m_X)^2}\ . \tag{A.24}$$

– The standard deviation estimated from the sample, known as the point estimate of the population standard deviation and denoted by the symbol $\hat{s}_X$, is sometimes called the “sample standard deviation”:

$$\hat{s}_X = \sqrt{\frac{1}{n-1}\sum_1^n (x_i - m_X)^2}\ . \tag{A.25}$$

Expression (A.24) is the restatement of equation (A.2) for the sample standard deviation. Expression (A.25) represents a point estimate for standard deviation derived from the mean of the distribution of the sample variance (based on the χ^2 random variable).

Similar modifications of sample characteristics are also available for skewness and kurtosis. “Sample skewness”, a, defined here by equation (A.5) can be re-written in simplified form as:

$$a_X = \frac{m_3}{m_2^{3/2}} = \frac{1}{n s_X^3}\sum_{i=1}^n (x_i - m_X)^3\ . \tag{A.26}$$

Statistica, Excel, Mathcad and other software provide a point estimate of population skewness, $\hat{a}_X$, as:

$$\hat{a}_X = \frac{n^2}{(n-1)(n-2)} \frac{1}{n\hat{s}_X^3} \sum_{i=1}^{n} (x_i - m_X)^3 = \frac{\sqrt{n(n-1)}}{(n-2)} a_X \,. \tag{A.27}$$

Note that the estimated standard deviation for the population, $\hat{s}_X$, is used in equation (A.27). If the sample standard deviation is used, the estimate of population skewness would be:

$$\hat{a}_X = \frac{n}{(n-1)(n-2)} \frac{1}{s_X^3} \sum_{i=1}^{n} (x_i - m_X)^3 = \frac{n^2}{(n-1)(n-2)} a_X \,. \tag{A.28}$$

The factor that modifies sample skewness, a_X, in equation (A.28) (the fraction containing sample size n) is slightly greater than the analogous factor in equation (A.27) (for n>30 the difference is less than 5 %); that difference declines with increasing sample size, n.

Similar modifications of sample characteristics are in place for kurtosis based on the fourth order central moment (see equation (A.6)). The formulae involved can be found in the help function of the respective software. In practice, however, kurtosis is seldom evaluated and only in very large samples (n>100).

A.8 GROUPED DATA, HISTOGRAM

Where n is large, the data may be grouped for readier handling into a short number of classes, k (usually $7 \leq k \leq 20$); the number of units in each class, n_i (i=1,2,…k), is called class frequency ($\Sigma n_i = n$). Each class is represented by a class mark, x_i^*, which is the midpoint in the class interval, defined by the upper and lower class limits.

The data so grouped are often depicted as a histogram, i.e., a bar graph showing the frequency, n_i, or relative frequency, n_i/n, for each class. Histograms are very useful graphical tools for obtaining an overview of the sample and its key characteristics.

The mean, m_X, is given by the first order ordinary moment (A.8), which for grouped data is written as:

$$m_X = m_1^* = \tfrac{1}{n} \sum_{i=1}^{k} n_i x_i^* \,. \tag{A.29}$$

The central moments (about the mean) of order l for grouped data are:

$$m_l = \tfrac{1}{n} \sum_{i=1}^{k} n_i (x_i^* - m_X)^l \,. \tag{A.30}$$

The moment characteristics of grouped data can be determined using equations (A.11) to (A.13). The relationships between the ordinary and central moments provided by equations (A.14) to (A.16) can also be used in the numerical assessment of grouped data.

Example A.9

The results for n=90 concrete strength tests are grouped into k=9 classes as indicated in the table below and the histogram in figure A.1. A visual review of the histogram reveals that the sample is well-ordered (no outliers), symmetric (skewness is expected to be close to zero) and slightly less spiky (more flat) than a normal distribution (some negative kurtosis is expected).

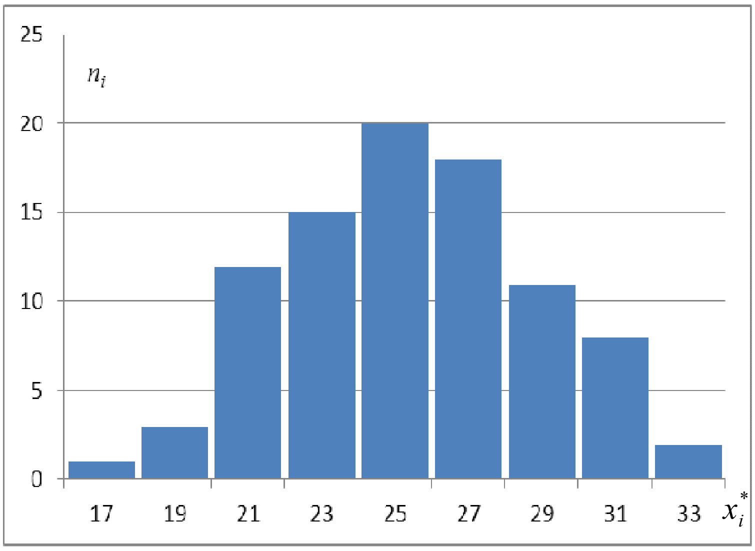

Figure A.1: Histogram for Example A.9 grouped data (90 observations of concrete strength)

The table gives the class intervals, class marks, x_i^* (in MPa), frequency, n_i, and the products $n_i x_i^*$ and $n_i(x_i^* - m_X)^2$ used to calculate the first order ordinary moments, and the second order central moment. The third and fourth order moments would be needed to calculate skewness, a_X, and kurtosis, e_X.

Class i	Class interval in MPa	Class mark x_i^* in MPa	Frequency n_i	Product $n_i x_i^*$	Product $n_i(x_i^* - m_X)^2$
1	16 to 18	17	1	17	71,309
2	18 to 20	19	3	57	124,593
3	20 to 22	21	12	252	237,037
4	22 to 24	23	15	345	89,630
5	24 to 26	25	20	500	3,951
6	26 to 28	27	18	486	43,556
7	28 to 30	29	11	319	139,062
8	30 to 32	31	8	248	246,914
9	32 to 34	33	2	66	114,173
Sum	-	-	90	2290	1070,222

It follows from equations (A.8) and (A.11) and the numerical results in the bottom row of the above table that the sample mean and standard deviation are:

$$n_X = m_X = m_i^* = 2290/90 = 25{,}44\ \text{MPa}\ ; \quad s_X = \sqrt{m_2} = (1070{,}222/90)^{0{,}5} = 3{,}45\ \text{MPa}\ .$$

The relatively high coefficient of variation, v_X=3,45/25,44≈0,14, indicates that the quality of the data is fairly low. The other moment characteristics can likewise be found using the higher order central moments and equations (A.12) and (A.13). These calculations show that sample skewness is almost zero, a=0,03, and that kurtosis, e, is -0,53. Consequently, the sample is actually symmetrical and slightly more uniform than a normal distribution.

A.9 INFLUENCE OF MEASUREMENT INACCURACY

Measurement inaccuracy influences esperimentally determined determined characteristic values, so it must duly taken into account elaborating test results.

Example A.10

Consider the case that the strength X has an unknown mean value but a known scatter s_X. A number of n measurements is carried out with a device having a zero bias and a standard deviation equal to s_m. In that case the characteristic values are given by:

$$X_K = m_X - 1{,}64\sqrt{\frac{s_m^2}{n} + \frac{s_X^2}{n} + s_X^2}\ . \tag{A.31}$$

if the measurement error is independent for each measurement

$$X_K = m_X - 1{,}64\sqrt{\frac{s_X^2}{n} + s_X^2 + s_m^2}\ . \tag{A.32}$$

if the measurement error is fully dependent for each measurement.

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CHAPTER 3: STRUCTURAL MODELLING AND ANALYSIS

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1 INTRODUCTION

As recalled in HB1 [1], modelling and analysis are key aspects in safety assessment of existing buildings, also in view of design of interventions.

Of course, the building reflects the state of knowledge at the time of its erection, therefore it is necessary to distinguish between existing buildings, designed using formal design approaches and based on recognized theories and/or normative prescriptions, and existing buildings designed on empirical bases, like the historical ones.

Modeling and analysis of formally designed existing buildings can be performed, when relevant, adopting the original design methodology, if known, or methodologies commonly used when the building was designed, so arriving to the so called *simulated original design* , or using more modern and sophisticated approaches, like for new buildings, allowing to explore more deeply the structural behaviour of the considered building.

As the information about geometry, dimensions and details of the structure depends on the available original documentation as well as on the level of the surveys, the *simulated original design* is a very powerful tool in studying the building, since it allows to enlarge significantly the knowledge about dimensioning and mechanical performances of structural members achieved through the survey.

It must be observed that linear and non-linear, static and dynamic, modeling and analysis methods for formally designed existing buildings are not dissimilar from modeling and analysis methods adopted in new building design, therefore they will not be treated here. Anyhow, some significant aspects concerning the non-linear behavior of framed structures under seismic actions will be discussed in Annex A to the present chapter.

On the contrary, modeling and analysis of existing buildings designed on empirical bases requires ad hoc studies, especially when masonry buildings subject to horizontal actions, wind or earthquake, are to be analyzed.

To investigate masonry building, several methodological approaches have been suggested, depending on the structural scheme and on the floor stiffness; between them particularly relevant are the equivalent frame analysis, the finite element model and the kinematic analysis, which is based on the kinematic theorem of the limit analysis, and that is widely treated here. Often, different aspects of the structural behaviour are captured by different methods so that, in general, comprehensive answers can only be obtained through a combined approach.

2 THE EQUIVALENT FRAME METHOD

The structural analysis of masonry buildings under vertical and horizontal loads structural analysis can be performed using a simplified scheme, where the 3-D masonry structure is subdivided in the two main horizontal directions in strips similar to 2D frames, called equivalent frames [2], [3], [4], [5], [6].

The load-bearing masonry walls belonging to the building strip represent the columns of the equivalent frame, while the floors and the masonry fasciae over the masonry wall openings, spandrels, represent the beams (fig. 1). An alternative approach foresees the introduction of rigid links at the ends of the frame elements to better simulate the actual stiffness (fig. 2).

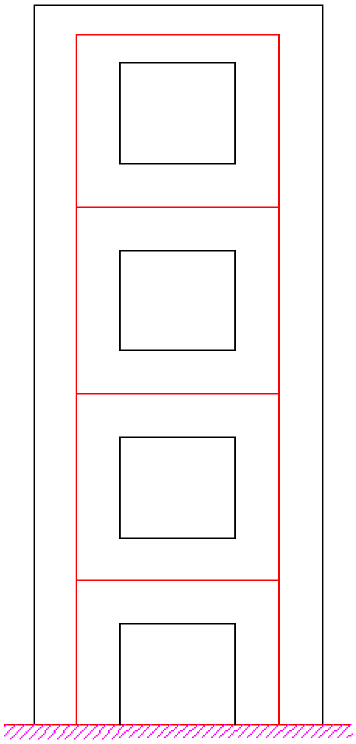

Figure 1: Equivalent frame and masonry wall

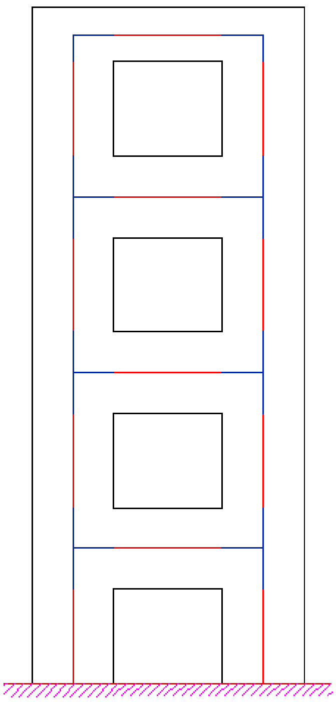

Figure 2: Equivalent frame with rigid links and masonry wall

The equivalent frame approach looks very simple and it can be adopted to analyze the structural behaviour both for vertical and horizontal actions, especially when the in-plane stiffness of the floors is small, so that each frame can be considered individually as planar frame. The model can be further improved, adopting, instead of classical beam elements, based on the Saint-Venant theory, Timoshenko beam elements, taking into account also shear deformations.

On the other hand, when the wall geometry is more complex, like when the openings are not aligned or are characterized by different dimensions, or when the columns are connected by arches, the method requires particular skills, as the identification of equivalent beams and columns is not trivial.

In principle, the method could be extended modeling with equivalent frames, characterized by different orientations, the whole structure, also in case the in-plane floor stiffness is sufficient to grant a box behaviour, redistributing the horizontal forces between different frames. But in this case, it necessary to check carefully the validity of the results, since it can happen that the axes of one column belonging to two differently oriented frames does not coincide, so modifying the structural answer, in particular under horizontal loads. The significance of this remark is stressed comparing figures 3 and 4, where the frame equivalent 3D model of a complex masonry building (fig. 3) is compared with the real aspect of the building itself.

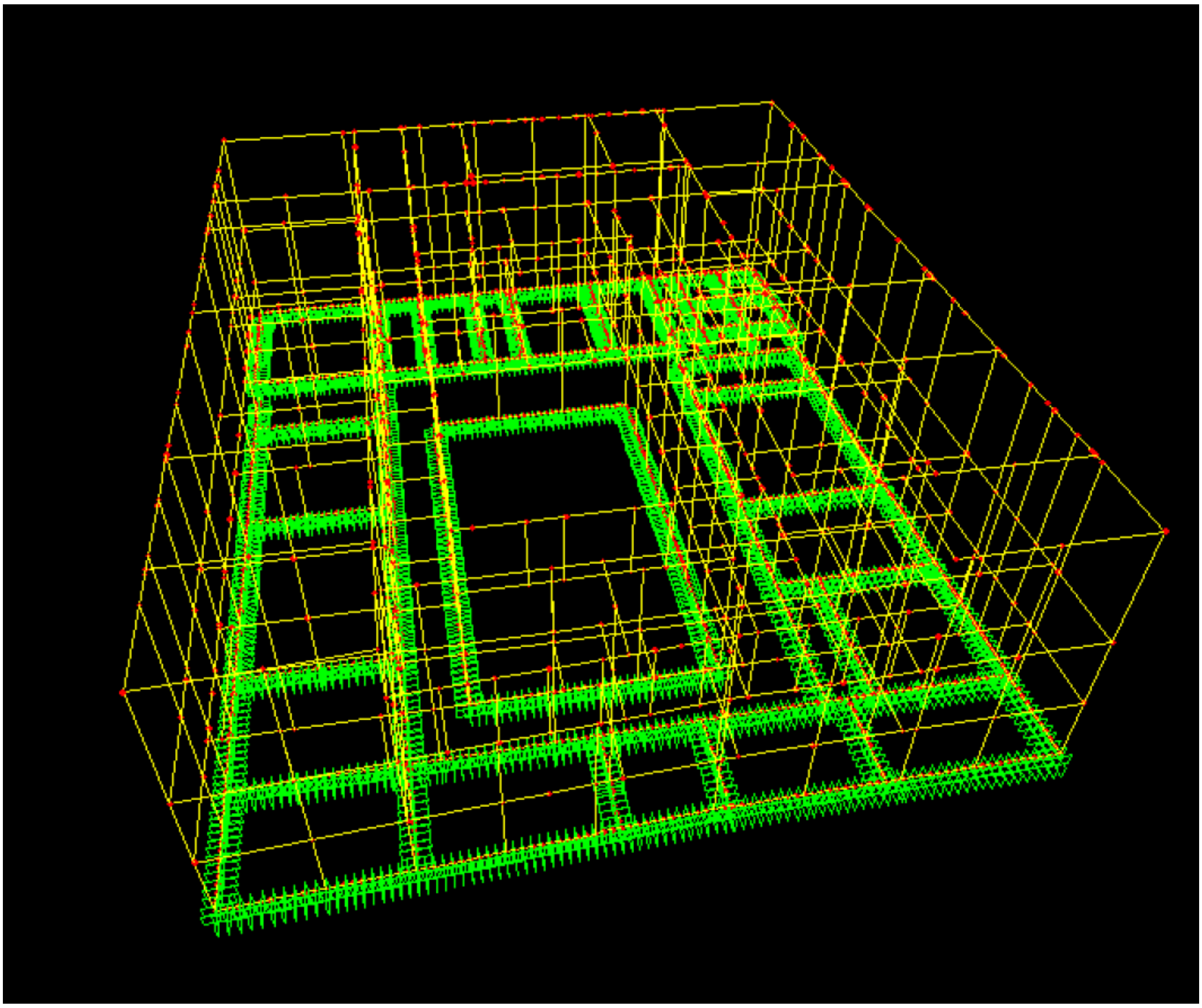

Figure 3: 3-D equivalent frame model of a masonry building

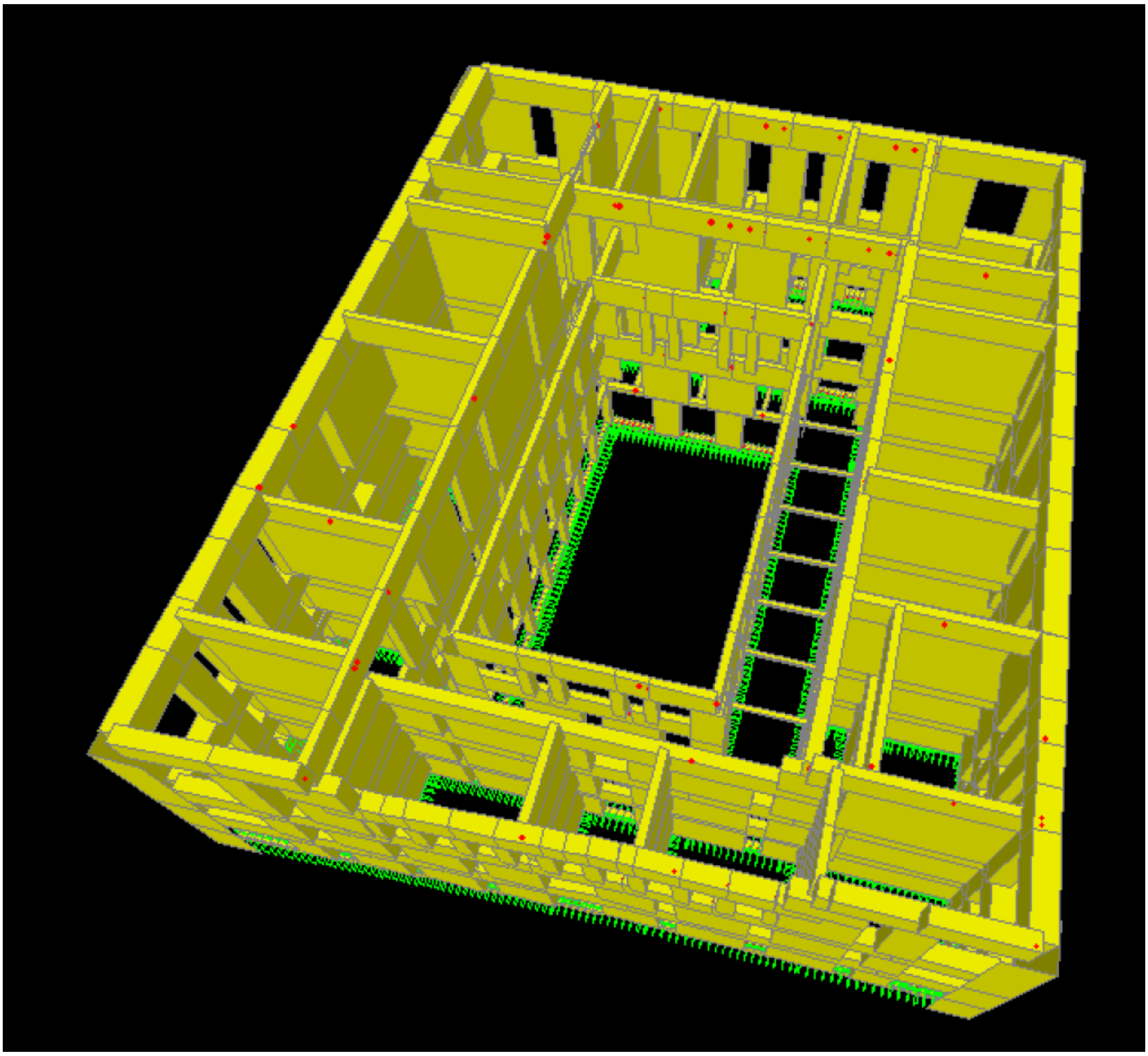

Figure 4: Actual masonry building and 3-D equivalent frame model

3 FINITE ELEMENT METHOD

Also in case of existing buildings, the most general approach is the finite element method, which allows to consider very complex geometries and to perform any kind of analyses, depending on the mesh refinement, on the element types adopted to represent the various parts of the structure, on the constitutive laws, linear or non-linear, implemented for the materials and on the type of analysis, static or dynamic, carried out. Clearly, the illustration of the finite element method is out of the scope of the present handbook and the interested reader could refer to the extensive literature on this relevant topic, see for example [7], [8].

Of course, the equivalent frame method illustrated before is a particular case of the FEM, where the finite element type is forced to be a 2-D beam or, more generally, a 3-D beam.

It must be highlighted that the FEM can be used also as additional tool to refine the kinematic analysis as discussed in the following. The modal analysis, in fact, could allow to identify the mode shape and the corresponding natural frequency associated with the considered macro-elements, so that the appropriate spectral acceleration is pick up from the response spectrum.

As an example, in figure 5 it is shown the finite element mesh, obtained using 3D shell elements, regarding the masonry building considered in §2, while the mesh detail for the masonry arches of the inner portico at the ground floor is illustrated in figure 6.

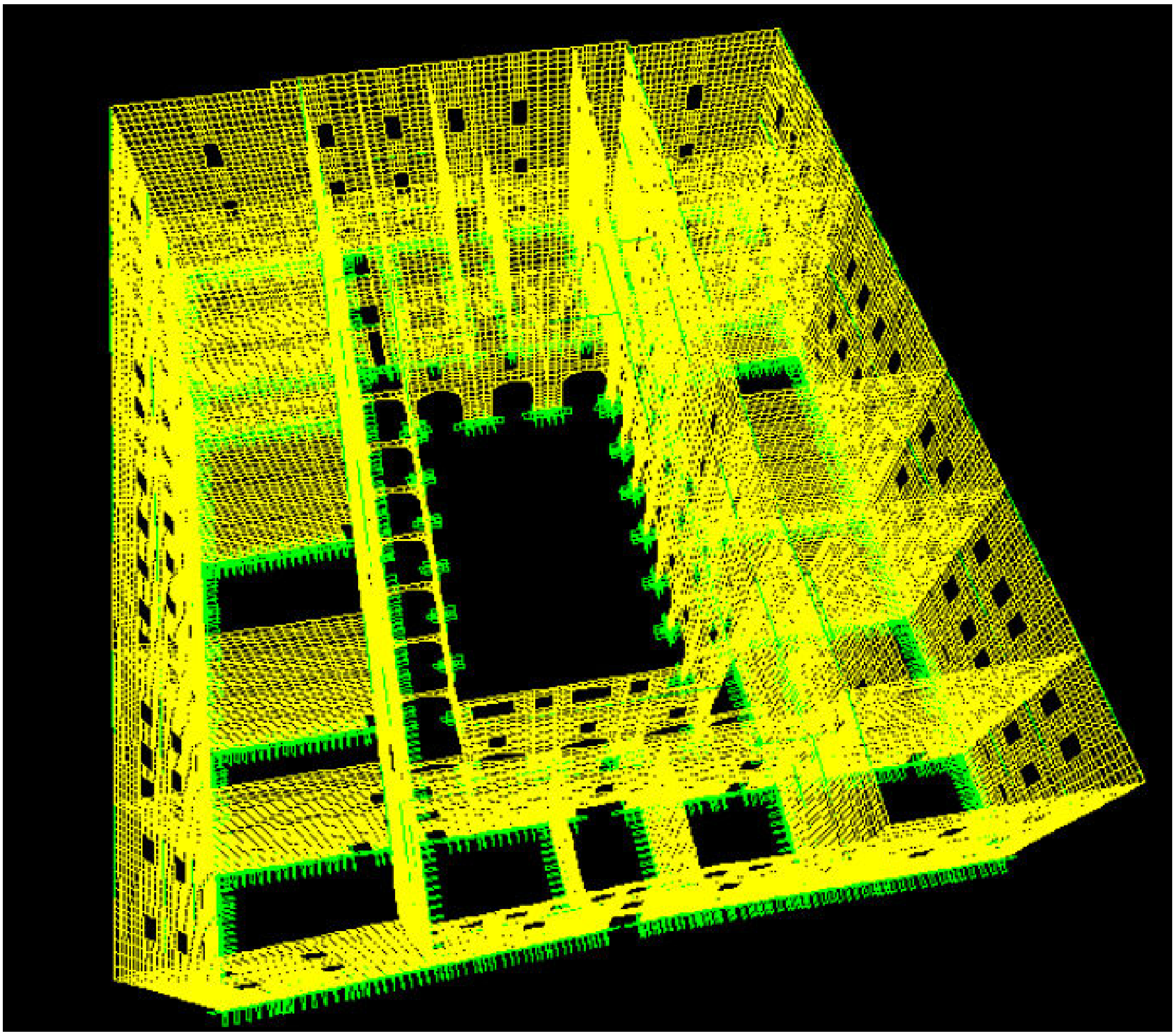

Figure 5: 3-D shell model of the masonry building considered in §2

Comparing modal shapes and natural frequencies, a refined finite element model could be also used to calibrate simpler model, in such a way that very complicated and time consuming investigations can be performed on the latter. For example, in case of not-negligible in-plane stiffness of the horizontal floors, the comparison between the first mode shapes of the equivalent frame model (fig. 7) and of the shell finite element model (fig. 8) and the corresponding natural frequencies confirms that the equivalent frame model underestimates the actual stiffness: in fact, the fundamental frequency of the equivalent frame model is 1,56 Hz, with a participating mass of 12% in the x-direction, while the fundamental frequency of the shell model is 3,05 Hz, with a participating mass of 14%, so justifying the need of calibration.

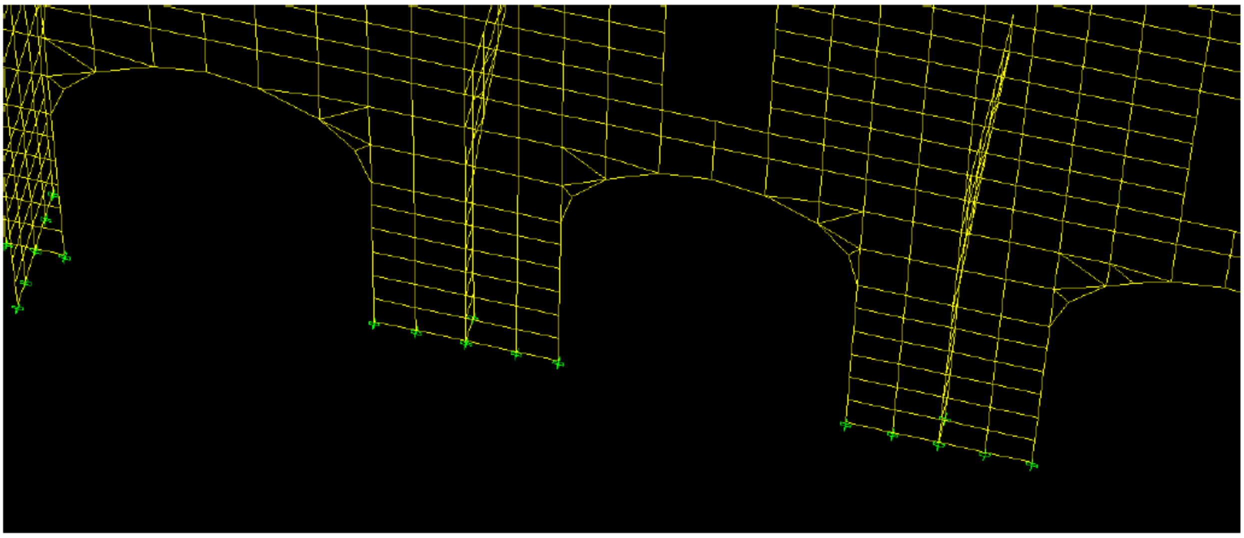

Figure 6: 3-D shell mesh detail for masonry building considered in §2

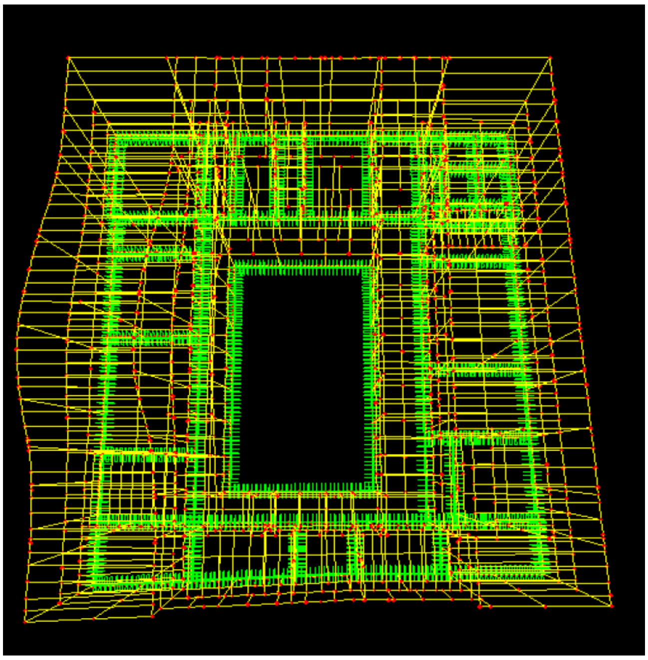

Figure 7: 1st mode shape of equivalent frame

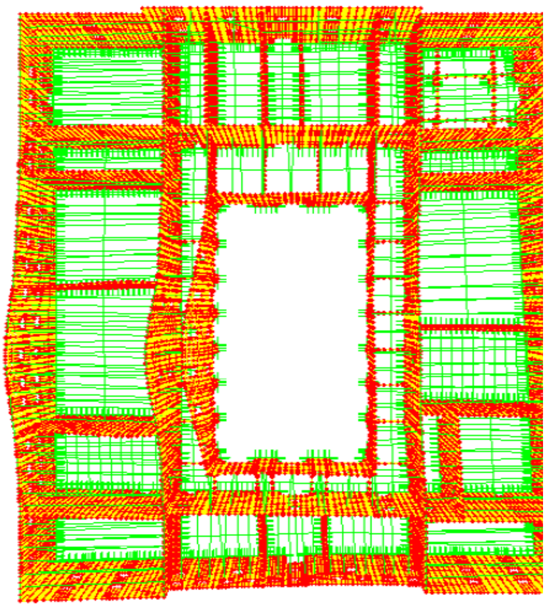

Figure 8: 1st mode shape of shell model

4 KINEMATIC ANALYSIS

The damages detected in masonry buildings as a result of seismic actions show that the earthquake select the weakest structural parts: therefore the analysis of the structural organization building enables to predict the possible future damage or collapse.

Unlike what happens in frame structures, a deficiency or lack of connection between the components of masonry made without of specific rules, enables the occurrence of partial collapses, generally corresponding to the loss of equilibrium of portions of the structure, usually called “macro-element”.

In the extreme case, the masonry structure does not show a clear global behaviour, but rather tends to react to the earthquake as a set of subsystems (local mechanisms): in this circumstance, a global model does not comply with the actual seismic behaviour of the structure and the seismic assessment can be reduced to a comprehensive set of local checks.

In existing masonry buildings, therefore, the study should be performed, appropriately considering, as well as the global seismic behavior, also these local collapse mechanisms, which are generally less resistant and less ductile than those involving the response of the whole of the building [9-14].

The first step of the analysis concerns the recognition of conditions that predispose the activation of the local damage or collapse mechanism, and subsequently the evaluation of the need to perform local kinematic analysis, instead of considering a global analysis.

A masonry wall, hit by an earthquake, may exhibit different damage mechanisms, conventionally classified into two basic failure mode categories, depending on the direction of the horizontal forces:

- 1st failure mode category: in this case the mechanism is activated by seismic forces acting orthogonally to the wall plane, like, for example, in case of the overturning of a portion of wall, not sufficiently connected;
- 2nd failure mode category in this case the the mechanism is activated by seismic forces acting parallel to the wall plane, inducing, for example, diagonal shear cracks (fig. 10).

Figure 9: Cracks indicating overturning of the building facade

Considering that the mechanism of the 1st failure mode category are often the first to become active during an earthquake, their study is very important, also in view of the design of interventions for improvement and/or seismic retrofitting of the buildings, since they should firstly increase the safety factor against this type of collapse.

Figure 10: Diagonal shear cracks in a masonry wall

4.1 Types of analysis and basic assumptions

The kinematic analysis is based on the kinematic theorem of the limit analysis method.

According to the kinematic approach, the application of virtual works theorem allows to estimate the load multipliers corresponding to the local collapse mechanisms.

The analysis must be performed on a limited number of mechanisms, which are recognized to occur on the building and are identified as the result of damage induced by an earthquake or the results of specific investigations, depending on the workmanship of the construction, the erection techniques, the analysis of crack patterns and so on, on the considered building or on other similar buildings.

Generally, two different kind of analyses are foreseen:

- linear kinematic analysis, leading to the definition of the load multiplier or the peak ground acceleration (PGA) activating the local mechanism, or
- non-linear kinematic analysis, leading to the definition of the (PGA) corresponding to the local collapse.

The method applied is essentially based on the Heyman model [14], based on the following assumptions:

- masonry tensile strength equal to zero;
- lack of sliding between the blocks;
- infinite masonry compressive strength.

For each significant local failure mode identified, the following procedure should be applied:

- *transformation of a building part in a mechanism (kinematic chain),* through the identification of rigid bodies, delimited by the fracture planes and able to rotate or to scroll through them according the damage and collapse mechanisms;
- *assessment of the α_o loads multiplier* corresponding to the mechanism activation (damage limit state);
- *assessment of the evolution of horizontal loads multiplier* until the annulment of the horizontal seismic force, as a function of the increase of the displacement d_k of a suitable control point of the kinematic chain, point that is usually chosen near the center of gravity of the considered part;
- *transformation of the curve so obtained in a capacity curve*, expressing the liaison between the spectral acceleration a^* and the spectral displacement d^*, and evaluation of the ultimate displacement, corresponding to failure (ultimate limit state);
- *safety assessment*, by controlling the compatibility of displacements and/or strengths required by the analysis with those offered by the structure.

However, in many cases, in order to arrive to a more realistic simulation of the actual behaviour, it is appropriate to consider, at least in an approximate form:

- the sliding between the blocks, considering the friction presence;
- the connections, also of limited strength, between the masonry walls;
- the presence of metal chains;
- the limited masonry compressive strength, considering the hinges set back with respect to the edge of the section;
- the presence of disconnected wall facades.

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4.2 Linear kinematic analysis

The linear kinematic analysis consists in the evaluation of α_0 load multiplier that determines the local damage mechanism activation and it is mainly used to assess serviceability limit states (SLS).

Each macro-element is considered as a single degree of freedom body mechanism, connected to the remaining part of the building by a suitably hinge. The seismic loads are assumed to be static horizontal forces amplified by a kinematic multiplier α_0; while the vertical gravity loads act generally like stabilizing forces (fig. 11).

The kinematic multiplier can be evaluated using the Virtual Work Theorem.

Referring to figure 11, the following systems of forces must be taken into account:

- a system of vertical forces W_i and P_j, being W_i the weight of i-th masonry block, applied in its centroid, and P_j the vertical loads transmitted to the blocks by floors, roofs, vaults and other masonry elements not considered in the structural model;
- a system of horizontal forces, proportional through α_0 to the vertical loads carried by each block, provided that they are not effectively transmitted to other parts of the building;
- internal horizontal forces Q_k transmitted by structural elements, like the thrusts of arches, domes and vaults;
- horizontal component of stabilizing external forces F_h, such as those transmitted by metal chains or FRP tie, or stabilizing internal forces, like those offered by the connections with adjacent walls.

Assigned a virtual rotation θ_i to the i-th masonry block, the virtual displacements δ associated with it can be easily determined as a function the geometry of the structure.

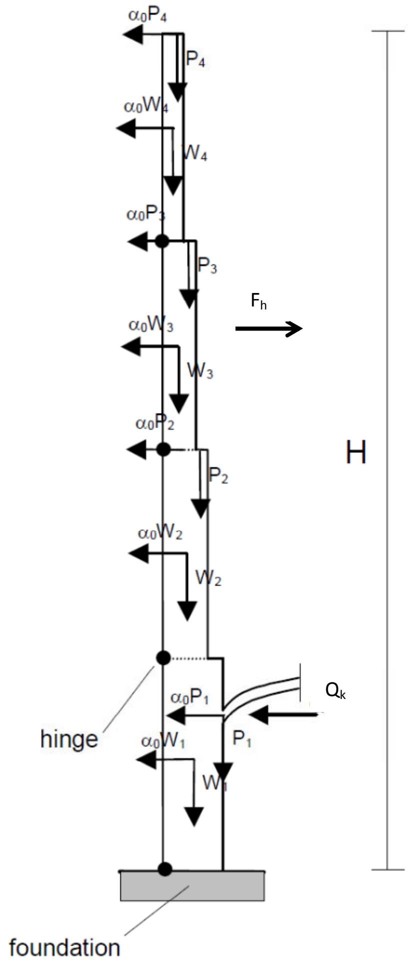

Figure 11: Kinematic analysis of a macro-element

The multiplier α_0 is obtained by applying the Principle of Virtual Work, in terms of displacements, equaling the total work performed by the external and internal forces applied to the system at the act of virtual motion:

$$\alpha_0\left(\sum_i W_i\delta_{xi} + \sum_j P_j\delta_{xj}\right) + \sum_k Q_k\delta_{xk} - \sum_i W_i\delta_{yi} - \sum_j P_j\delta_{yj} - \sum_n F_h\delta_{xn} = L_{fi}\,, \qquad (1)$$

where δ_x are the virtual displacements of the horizontal forces, δ_y are the virtual displacements of the vertical forces and L_{fi} the virtual works of other internal forces.

The participating mass M^*can be estimated considering that the virtual displacements of the points of the macro-elements could be assumed to be a simplified representation of its fundamental mode shape, so that it results

$$M^* = \frac{\left(\sum_i W_i\delta_{xi} + \sum_i P_j\delta_{xj}\right)^2}{g\left(\sum_i W_i\delta_{xi}^2 + \sum_j P_j\delta_{xj}^2\right)}, \qquad (2)$$

where the sums are extended to all the applied weights W_i and vertical loads P_j associated with the macro-element under examination.

From (1) and (2), it can be definitively determined the spectral acceleration a_0^*

$$a_0^* = \frac{\alpha_0 \left(\sum_i W_i \delta_{xi} + \sum_i P_j \delta_{xj} \right)}{M^*}, \tag{3}$$

which can be seen as the acceleration to be applied to the participating mass M^* to obtain the horizontal inertia forces activating the mechanism. In other words, a_0^* is the maximum spectral acceleration to which the macro-element can resist before the mechanism activation.

Once the peak ground acceleration a_g is known, the spectral acceleration demand can be derived as function of the fundamental period T [s] of the structure. In particular, in EN1998-1 [15] the elastic response spectrum is given by

$$\begin{aligned}
&0 \le T < T_B && S_e\,(T) = a_g \cdot S \cdot \eta \cdot \left[2{,}5 \frac{T}{T_B} + \frac{1}{\eta} \left(1 - \frac{T}{T_B} \right) \right] \\
&T_B \le T < T_C && S_e\,(T) = 2{,}5\, a_g \cdot S \cdot \eta \\
&T_C \le T < T_D && S_e\,(T) = 2{,}5\, a_g \cdot S \cdot \eta \cdot \left(\frac{T_C}{T} \right) \\
&T_D \le T && S_e\,(T) = 2{,}5\, a_g \cdot S \cdot \eta \cdot \left(\frac{T_C \cdot T_D}{T^2} \right)
\end{aligned} \tag{4}$$

where $S=S_s \cdot S_T \ge 1{,}0$ depends on soil category and on topography and T_B, T_C and T_D, expressed in s, are the values of the fundamental period corresponding to the starting points of the constant acceleration branch, of the constant velocity branch and of the constant displacement branch of the elastic response spectrum, respectively.

In eqs. (4) the coefficient η takes into account the effects of damping coefficient ξ, expressed in per cent. The general expression for η is

$$\eta = \sqrt{\frac{10}{5+\xi}} \ge 0{,}55 \tag{5}$$

but in the present case it can be set, as usual, equal to one.

The verification of the macro-element is satisfied if the following two conditions are satisfied:

1. the spectral acceleration activating the mechanism, a_0^*, is higher than the spectral acceleration to the ground: $a_0^* \ge a_g(P_{VR})S$, where $a_g(P_{VR})$ is the peak ground acceleration which can be exceeded with the probability P_{VR} in the reference period, and S is defined before;
2. the spectral acceleration activating the mechanism, a_0^*, is higher than the peak acceleration demand at the height Z, $a_0^* \ge S_e(T_1)\psi(\mathrm{Z})\gamma$, where $S_e(T_1)$ is the value of the elastic spectrum (4) corresponding to the fundamental period T_1 of the structural mode shape activat-

ing the mechanism, γ is the modal mass participation factor and $\psi(Z)$ is the amplitude of the mode shape at the height Z of the centre of gravity, normalized with respect to the maximum mode shape amplitude, occurring at the top height, H, of the structure.

A sound procedure to determine the fundamental period T_1 associated with the mechanism is to perform a refined modal analysis of the whole building, in order to identify the mode shape which is relevant for the considered mechanism, that is the one best fitting locally the rigid body motions.

4.3 Relevant failure modes and associated elements

As said, kinematic analysis preliminarily requires the identification of the relevant failure modes and the associated macro-elements.

The investigation made until now on existing building and especially on the basis of post-earthquake surveys on damaged building allowed to identify recurrent failure modes, which are illustrated in Annex B to the present chapter.

4.4 Non-linear kinematic analysis

The non-linear kinematic analysis is devoted to investigate the displacement capacity of the structure when the collapse occurs through the considered mechanism.

The horizontal load multiplier α can be calculated not only on the initial configuration, like described before for linear analysis, but also on the macro-element displaced configurations: in this way the evolution of the mechanism can be exhaustively defined, through a relationship $\alpha=\alpha(d_k)$ linking the load multiplier α with the displacement d_k of a control point of the macro-element [16]. The analysis will be terminated when the load multiplier becomes zero, indicating with $d_{k,0}$ the corresponding displacement of the control point.

The expression can be obtained analytically, graphically or numerically, applying the Virtual Work Theorem in the deformed configurations, and considering the effect of the displacements on the load configuration.

If the forces (weight forces, external or internal force) are not depending on the displacement, the $\alpha=\alpha(d_k)$ curve can be approximated by a straight line expressed by

$$\alpha = \alpha_0\left(1 - \frac{d_k}{d_{k,0}}\right) \tag{6}$$

where α and d_k are the load multiplier and the corresponding displacement in a generic displaced configuration; $d_{k,0}$ can be derived from (6) setting $\alpha=0$.

The spectral displacement d^* can be derived from d_k as

$$d^* = d_k \frac{\alpha_0\left(\sum_i W_i \delta_{xi}^2 + \sum_i P_j \delta_{xj}^2\right)}{\delta_{x,k}\left(\sum_i W_i \delta_{xi} + \sum_i P_j \delta_{xj}\right)} \tag{7}$$

where $\delta_{x,k}$ is the virtual horizontal displacement of the control point and the other symbols have been defined before (§4.2).

The ultimate spectral displacement d_u^* is finally given by 0,4 d_0^*, where $d_0^* = d^*(\alpha = 0)$ or, if smaller, by the maximum displacement dictated by fulfillment of structural requirements.

The seismic assessment consists in checking that d_u^* is bigger than the required spectral elastic displacement.

The application of the non-linear kinematic analysis to a practical case is illustrated in Annex C to the present chapter.

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CHAPTER 3 - ANNEX A

NON-LINEAR ANALYSIS OF R.C. FRAMES

A.1 INTRODUCTION

In modern design codes, to avoid uneconomical and/or unfeasible interventions, assessment of existing buildings is mainly controlled by ultimate limit states, devoted to prevent overall or partial collapse, rather than serviceability limit states. In fact, while the level of the accepted risk of human life or major material losses, due to structural collapse, should be similar for existing structures (after the intervention) and for new structures, the accepted probability that the structure is permanently damaged or goes out of service is higher in existing structures than in new ones. Non-linear structural analysis is then a powerful tool to assess existing building, especially under horizontal actions.

In the present appendix, non-linear analysis of existing framed r.c. structures under earthquake action is shortly discussed.

A.2. NON-LINEAR BEHAVIOR OF STRUCTURES

The general principles of earthquake resistant design are generally based on the following needs:

- prevention of any damage on structural and non-structural elements of buildings under low-intensity earthquakes;
- limitation of damage in structural and non-structural elements to repairable levels under medium-intensity earthquakes;
- prevention of overall or partial collapse of buildings in high-intensity earthquakes.

In seismic design codes, force based elastic analysis methods are often considered. In these methods seismic forces induced by high intensity earthquakes are calculated first considering elastic building systems, and subsequently reduced taking into account a suitable structural behavior factors q, depending on the structural scheme and on the energy dissipation capability of the structure, depending on the accepted level of damage. In other words, the acceptance of some structural damage under high-intensity earthquakes allows to perform seismic assessments taking into account lateral forces suitably reduced than those evaluated for elastic system.

When damage occurs, the structural behavior becomes clearly non-linear and cannot be precisely described adopting traditional linear methods. In seismic design codes, linear analysis is generally accepted also for ULS assessments and the capability of the structure to sustain plastic deformations is ensured by some additional rules, like strong columns-weak beams, and detailing provisions about critical sections of the structural elements.

This kind of force based elastic design approach is very clear and easy to use, therefore it is generally preferred in designing new structures. On the contrary, the assessment of existing structures often requires inelastic analysis methods, able to take into account not only the strength capacity but also ductility and energy dissipation capability (damageability) of the individual members and of the whole structure.

A typical representation of elastic and inelastic response curves of the structure is sketched in figure A.1. Building loaded beyond the elastic strength capacity undergoes plastic (permanent) deformation without collapse. Higher plastic deformations also indicates that increasing damage levels.

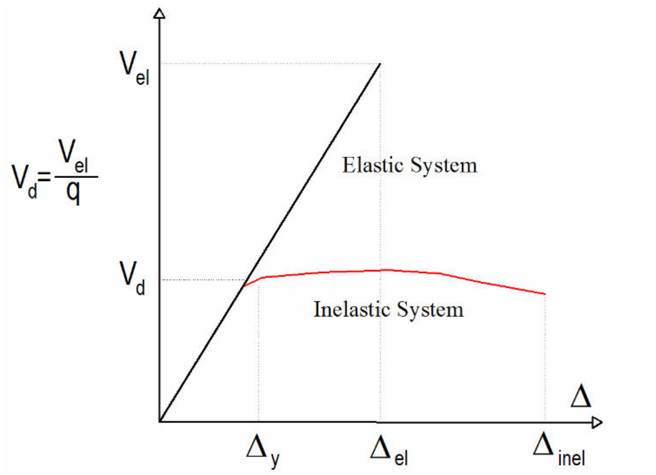
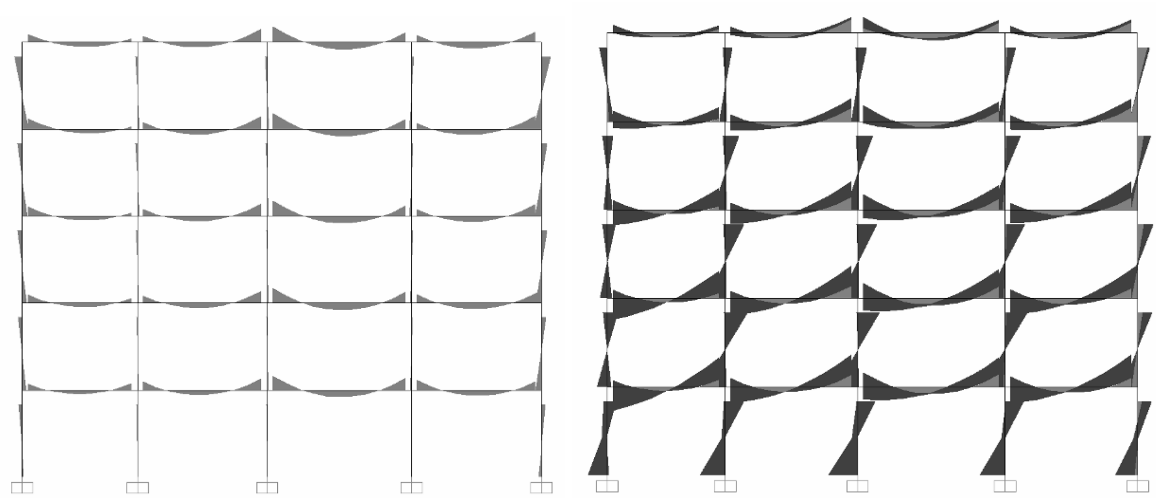


Figure A.1: Elastic and inelastic force and displacement response of buildings

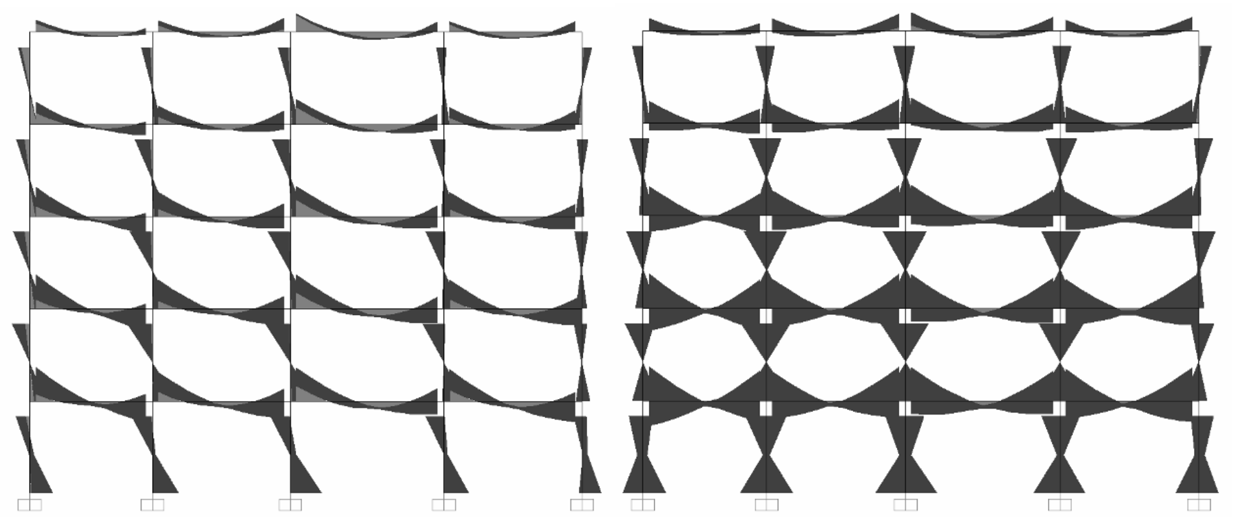
A.3 NONLINEAR MODELING OF STRUCTURES

The study of inelastic behaviour of buildings requires applying displacement based analysis methods rather than force based methods.

One of the most relevant problems to be tackled in assessing the inelastic response of the structure is the calculation of displacements, which are indication of the damage level: to do this, it is necessary to determine the critical regions subjected to inelastic deformations. These critical regions, where the damages are accumulated, are called as plastic hinge regions. Plastic hinges generally occur around the connection joints and/or near the member ends, since the maximum reactions occur in these regions. Bending moment diagrams of a typical frame structure under the effect of vertical dead and live loads and horizontal earthquake actions are sketched in figure A.2. The envelope diagrams presented in the figure clearly confirm that the connections and/or the member ends are the most critical regions, where plastic hinges can occur.



vertical dead and live loads *dead and live loads & seismic loads (+X)*



dead and live loads & seismic loads (-X) *Failure envelope*

Figure A.2: Bending moment diagram envelopes of typical frame structure

Strength and deformation capacity of plastic hinge regions can be determined by using moment-curvature analyses. Increasing tension and compression strains, depending on the increasing curvatures, determine the stress level in concrete and steel.

Evolution of the inelastic deformations at critical section of the cantilever column, located at its base, is represented schematically in figure A.3.a, being the internal forces carried by steel and concrete in equilibrium with the external ones (*N* and *M*) for each given value of the curvature (fig. A.3.b).

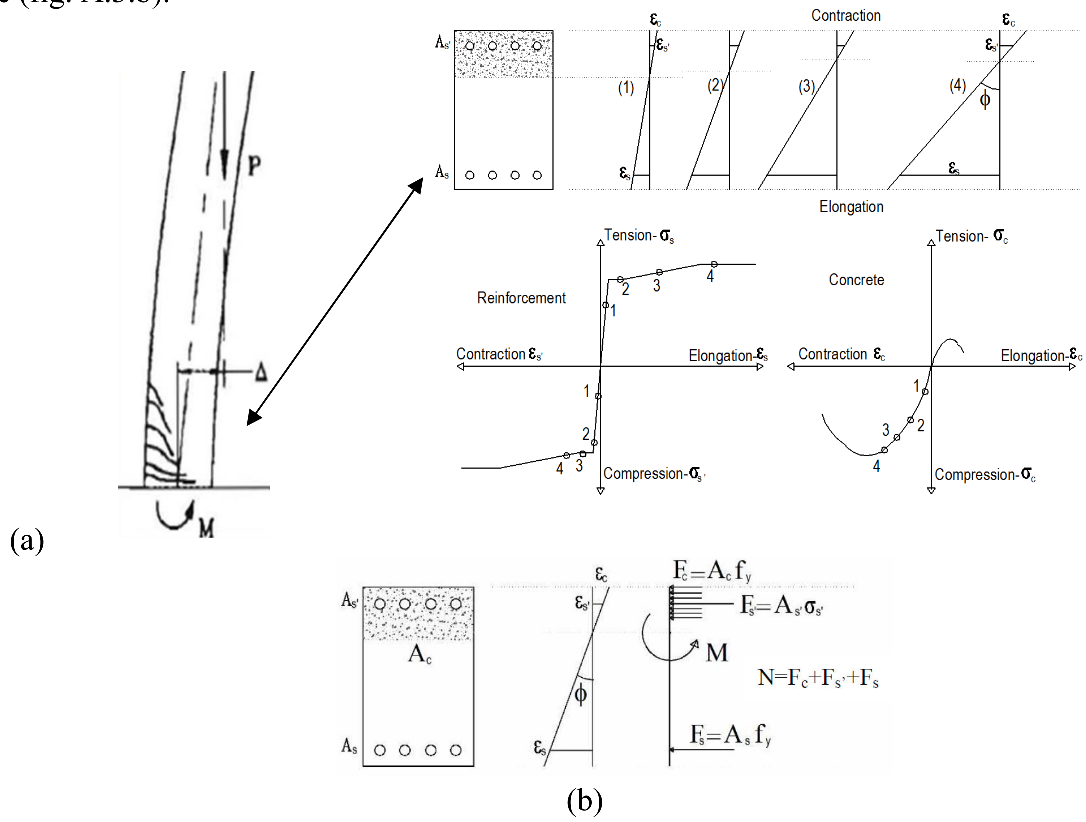


Figure A.3. Moment-curvature response at the critical section of cantilever column

Obviously, the bending moment capacity of the plastic hinge corresponds to the bending moment capacity of this section. Stepwise representation of displacements occurred at the top of cantilever column and the corresponding moment and curvature profile along the member height are shown in figure A.4. Accumulation of plastic curvatures within the plastic hinge region (L_p) is also represented on the figure.

Typical damages induced by earthquake at the base of cantilever type precast columns are shown in figure A.5. These photographs, taken from real buildings damaged during Marmara earthquake (1999), clearly explain and support the theoretical explanations.

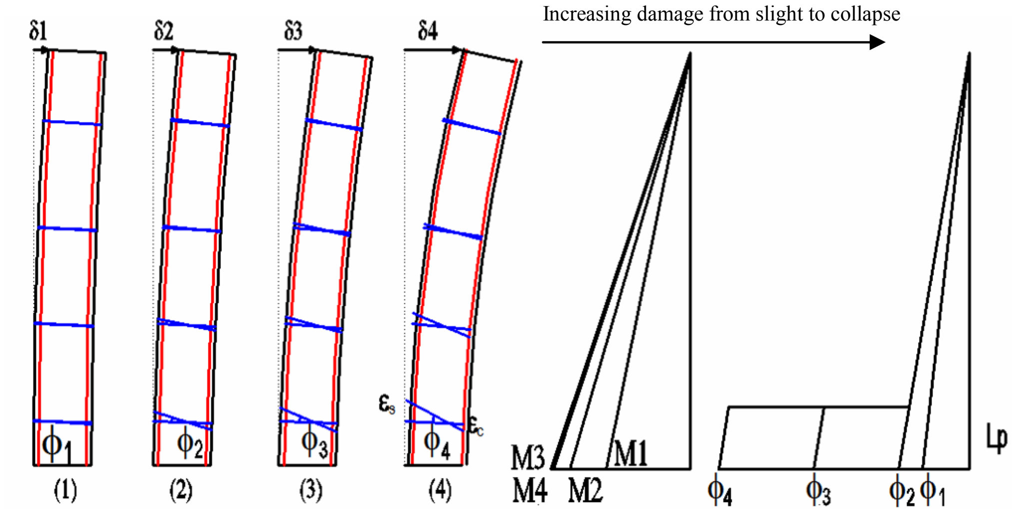


Figure A.4: Representation of displacements corresponding to increasing curvatures

Figure A.5: Damage in cantilever type precast columns (Marmara earthquake 1999)

The aforementioned analysis procedure (moment curvature analysis) can be extended to more complex structures, which have a lot of plastic hinge regions, in order to evaluate the moment and deformation capacity demand of each plastic hinge, so assessing the effective strength and deformation capacity of the structure.

A typical outcome of this kind of non-linear analysis is reported in fig. A.6, where the damage induced by increasing seismic actions in the plastic hinges of a frame structure are shown.

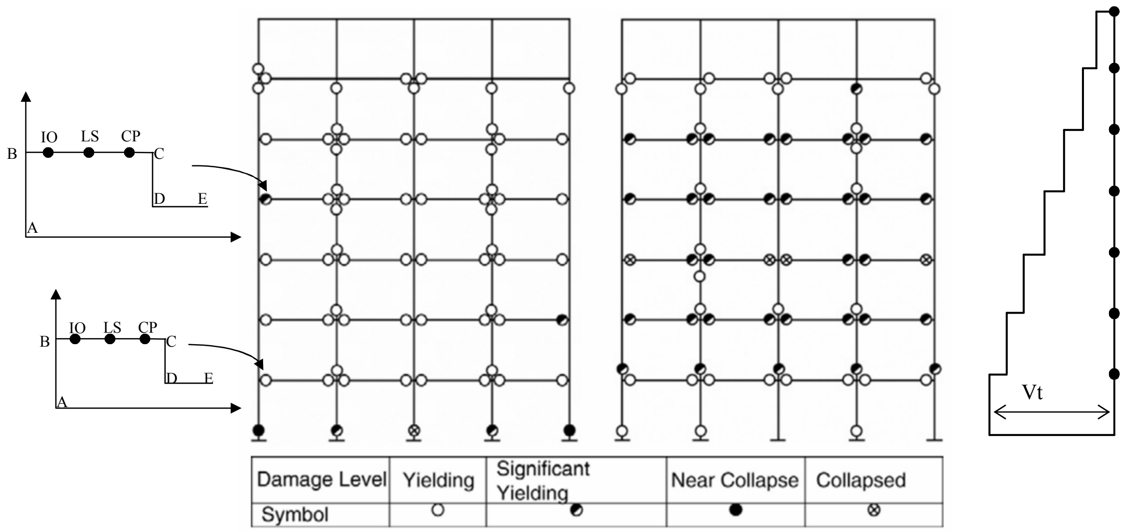
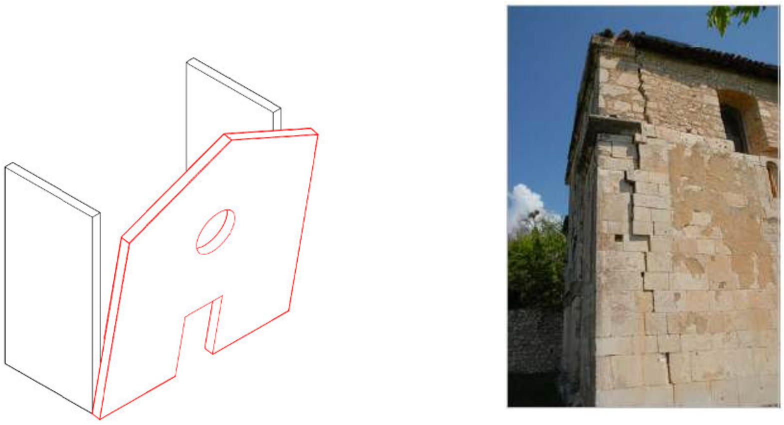


Figure A.6: Damage levels occurred in plastic hinges under different seismic demand levels

CHAPTER 3 - ANNEX B

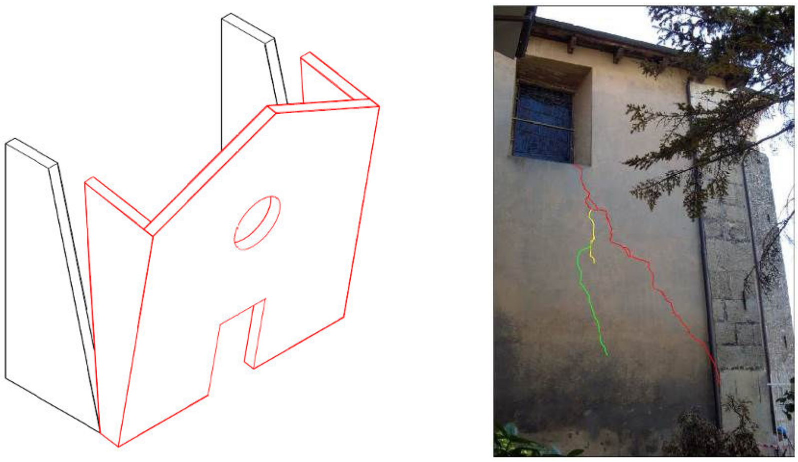
ILLUSTRATION OF RELEVANT FAILURE MODE AND MACRO-ELEMENTS

In the following, relevant failure modes and associated macro-elements are illustrated for churches and mansions.



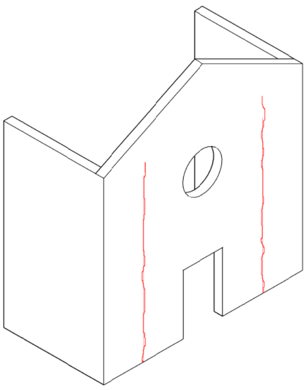
Note: *The façade and the transversal walls are disconnected along a vertical plane.*
The plan of the façade overturns around a horizontal cylindrical hinge located at the base of the façade.

Figure B.1: Detachment and overturning of the façade



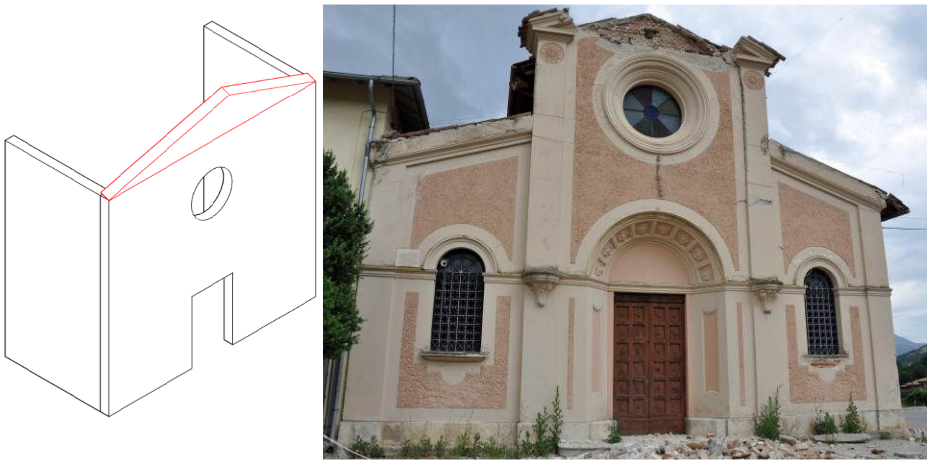
Note: *The façade overturns, together with triangular wedges of transversal walls limited by inclined cracks, around a horizontal cylindrical hinge located at the base of the façade.*

Figure B.2: Global overturning of the façade



Note: *Vertical cracks occur on the façade due to out of plane bending of the façade itself.*

Figure B.3: Vertical cracks caused by the out of plane bending of the façade

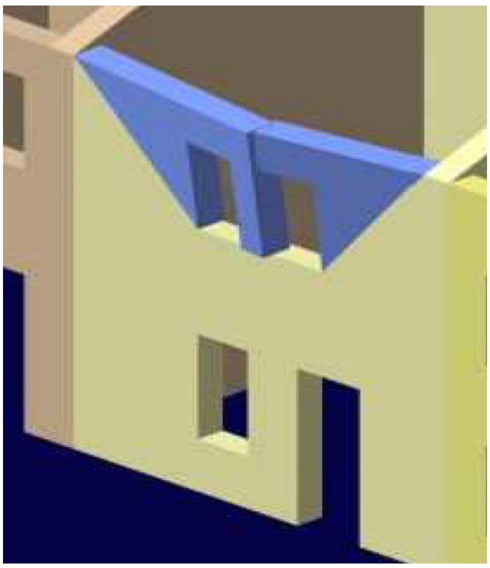


***Note:** The tympanum overturns around a horizontal cylindrical hinge located at the base of the the tympanum itself.*

Figure B.4: Overturning of the tympanum

***Note:** V-shaped pattern of cracks can be induced by out of plane bending: weak points, like openings or variations of the wall thickness can facilitate the occurrence of these cracks. When relevant resisting arch mechanisms can be excited (see figure B.6)*

Figure B.5: V–shaped pattern of cracks in a church caused by out of plane bending



***Note:** see note to figure B.5.*

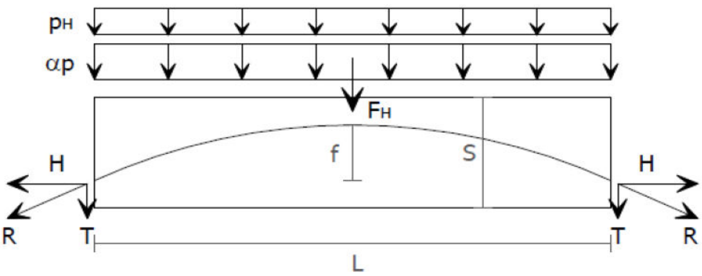
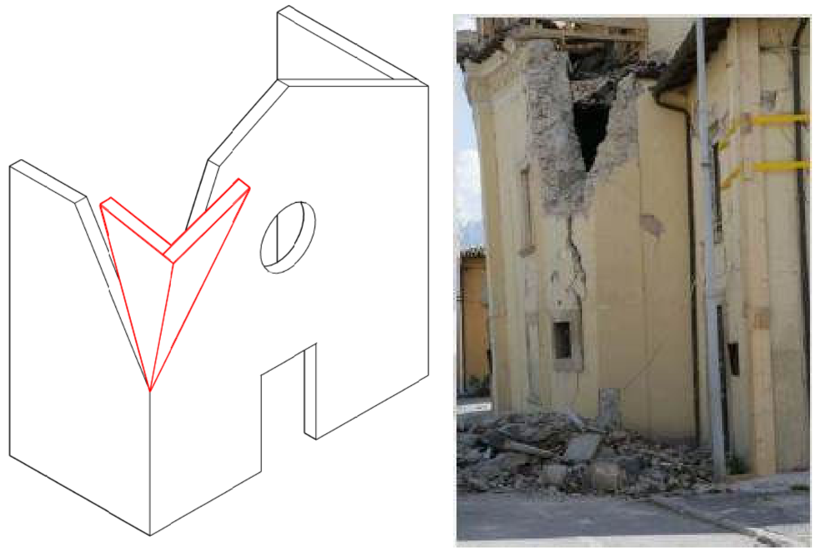
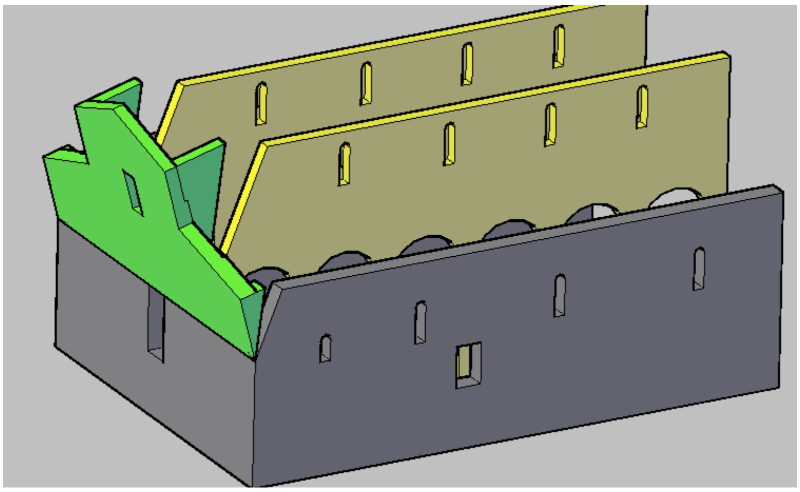


Figure B.6: V –shaped cracks pattern in a mansion caused by out of plane bending and resisting equivalent arch model



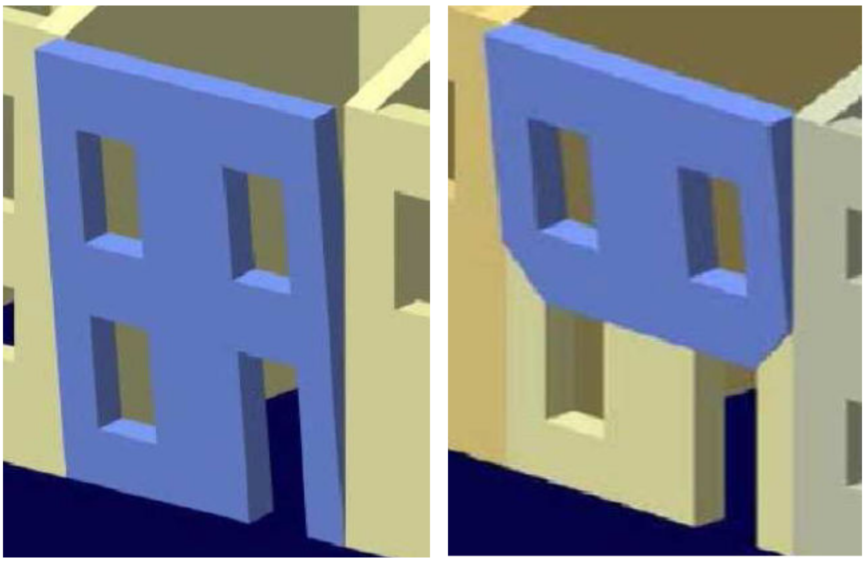
Note: *A wall corner, limited by inclined cracks overturn around a spherical hinge located in its vertex.*

Figure B.7: Overturning of the wall corner



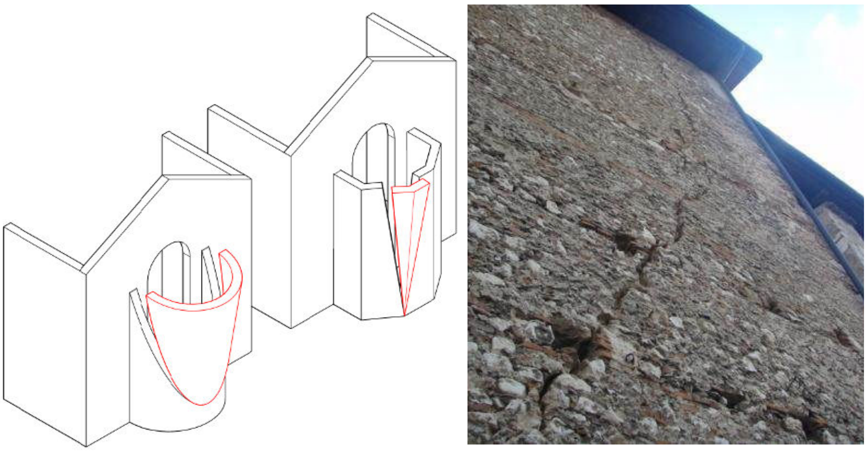
Note: *The façade overturns around a horizontal cylindrical hinge located at a certain height according to fig. B.1 or fig. B.2 mechanisms.*

Figure B.8: Partial Overturning of the façade



Note: *Part of the wall overturns around a horizontal cylindrical hinge located at its base or at a certain height.*

Figure B.9: Global and partial overturning of the wall



Note: *Apse parts in form of wedges, limited by inclined cracks, overturn around spherical hinges*

Figure B.10: Apse overturning

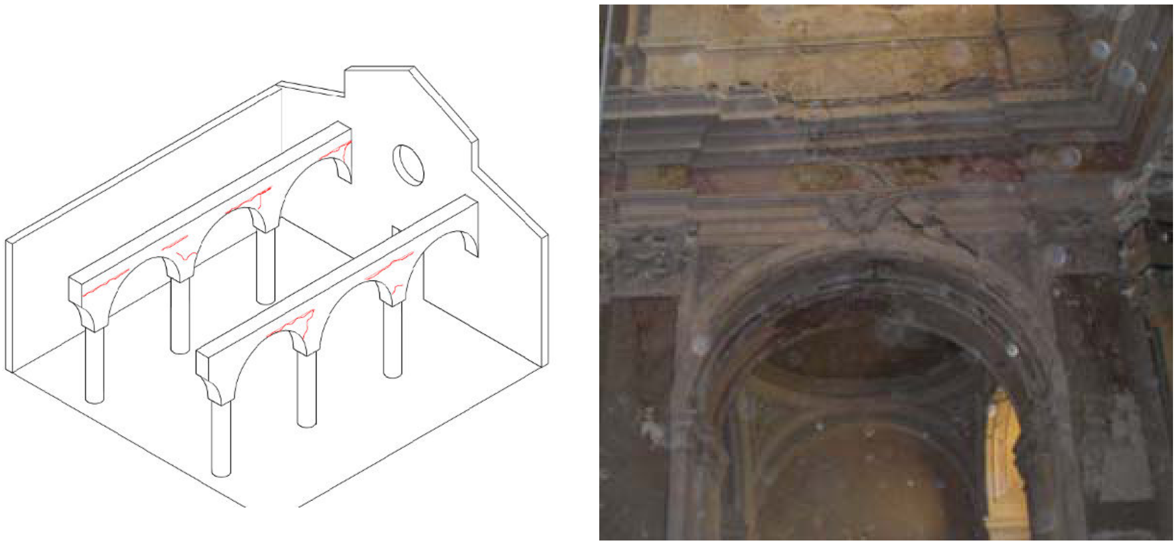


Figure B.11: Colonnade – Crack pattern in the arches

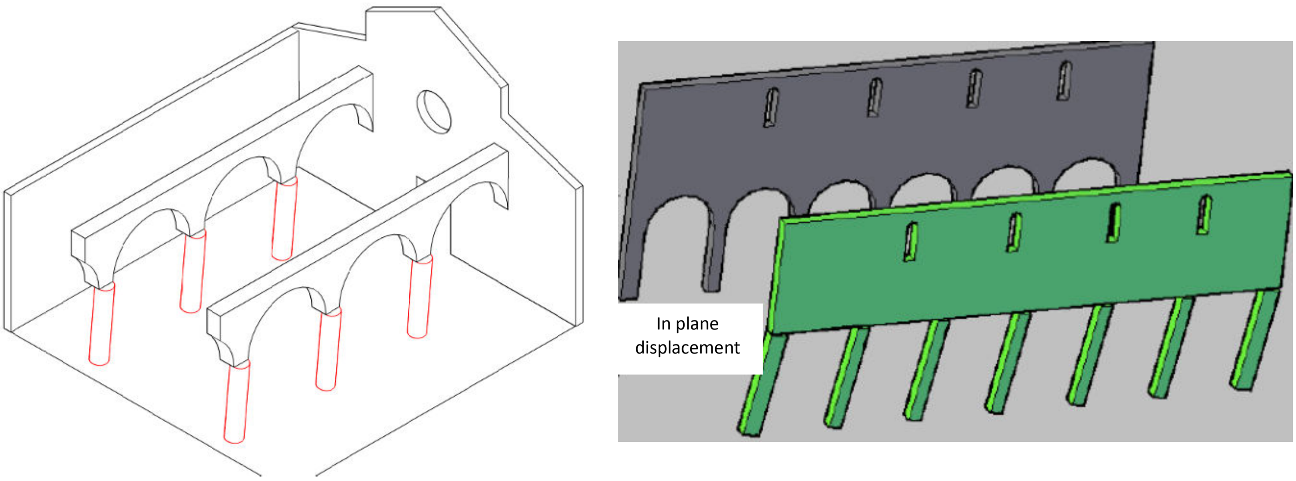


Figure B.12: Colonnade – Out of plane and in-plane displacements

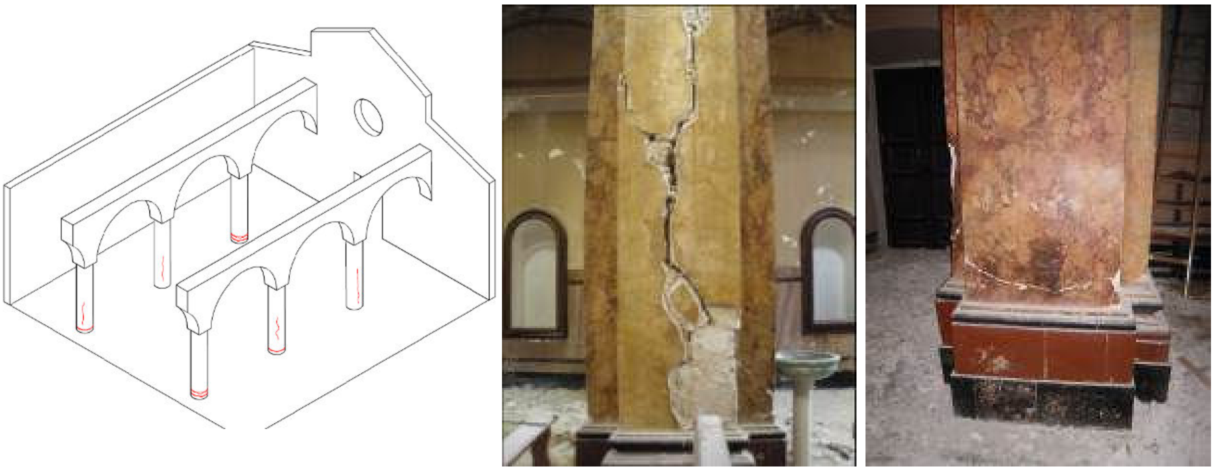
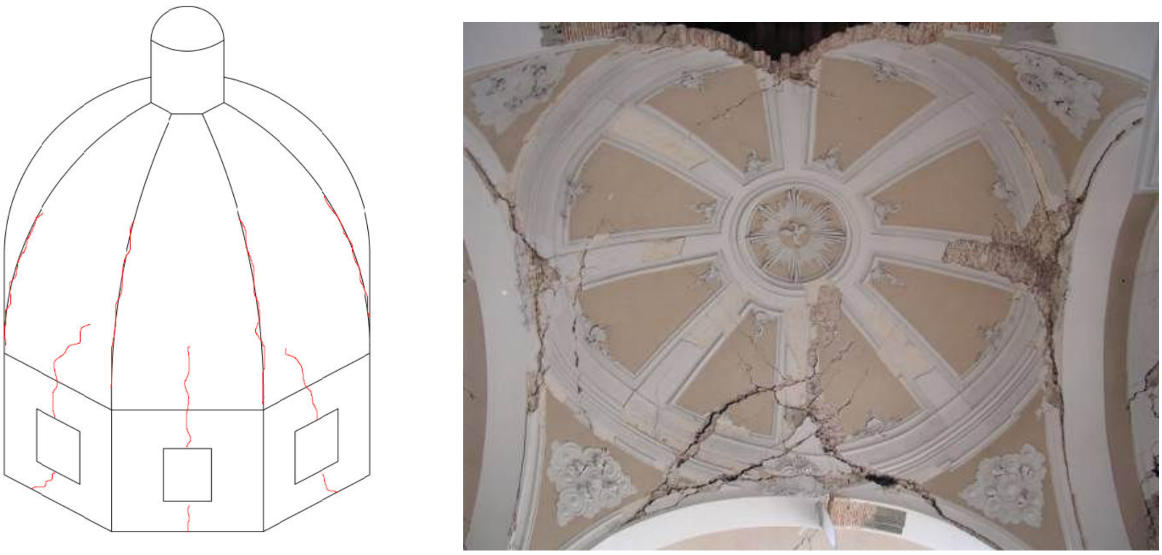


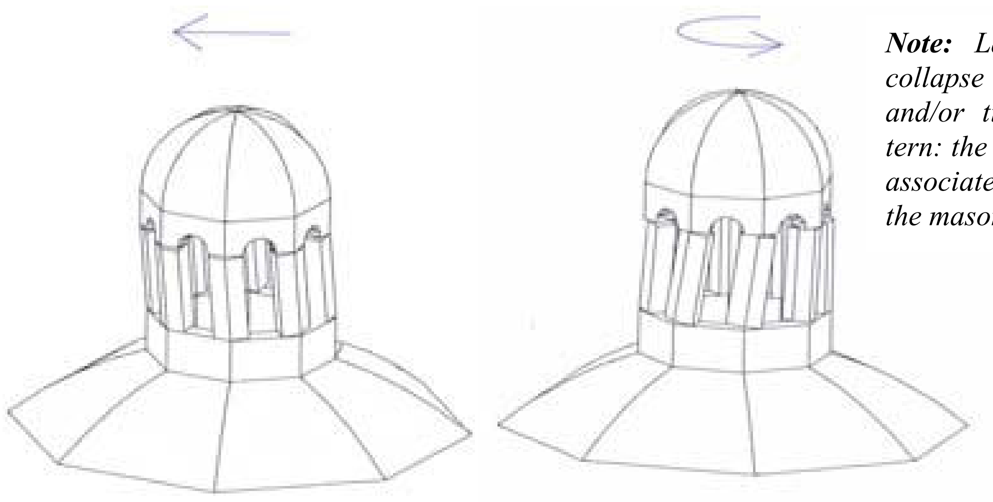
Figure B.13: Colonnade – Compression breaking of the columns

Note to figures B.11, B.12 and B.13: *In the colonnade several collapse mechanism can occur, even simultaneously. Cracks in the arches can be the results of the diagonal crushing of the masonry as well as of the mechanism connected with the in plane displacement of the colonnade. Vertical cracks in the columns are the results of crushing of the columns themselves due to excess of vertical loads: in this case the phenomenon can be emphasized by out of plane displacements of the colonnade.*



Note: *Typical cracks patterns in domes are meridian and tambour cracks. These cracks, caused by high tensile stresses, are more relevant when the tambour is present, due to the poor confinement granted by the tambour. Additional cracks are due to horizontal loads or to unsymmetrical vertical loads.*

Figure B.14: Dome – Meridian and tambour cracks

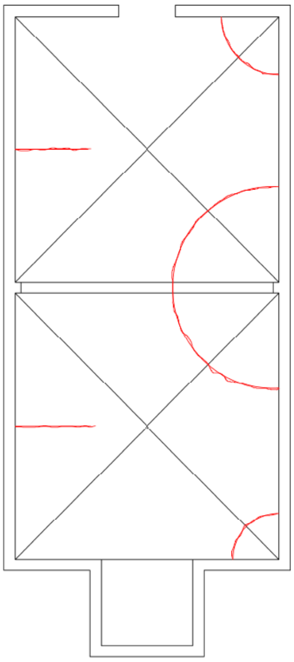


***Note:** Lantern piers can collapse due to sliding and/or tilting of the lantern: the mechanism can be associated to crushing of the masonry of piers.*

Figure B.15: Dome – Sliding and tilting of the lantern piers

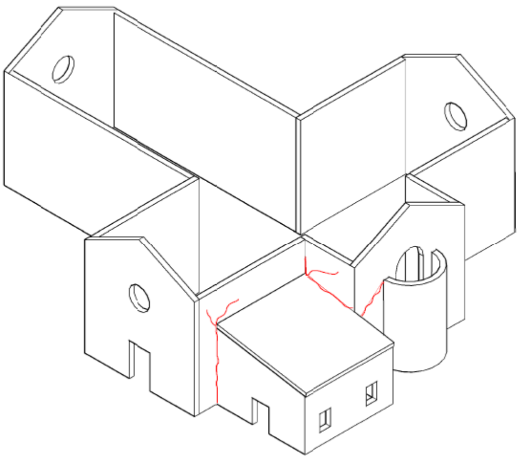
***Note:** Collapse of triumphal arches of churches is often associated with in plane mechanisms. But it is necessary to highlight that the activation of the in plane mechanism determine the formation of highly stressed masonry struts near the arch keystone. Since the dimensions of these struts are very small, crushing of the struts can occur, so determining an early failure of the arch.*

Figure B.16: Triumphal arch - in plane mechanism



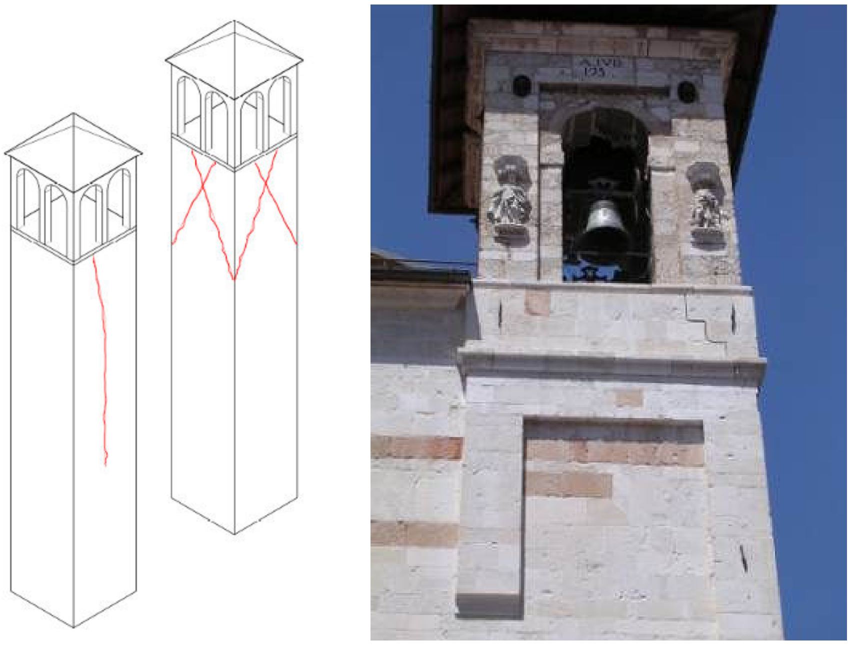
Note: *Typical and large cracks patterns can occur in vaults due to horizontal loads and/or unsymmetrical vertical loads and/or activation of failure mechanism of the supporting structures.*

Figure B.17: Vault – Large crack pattern



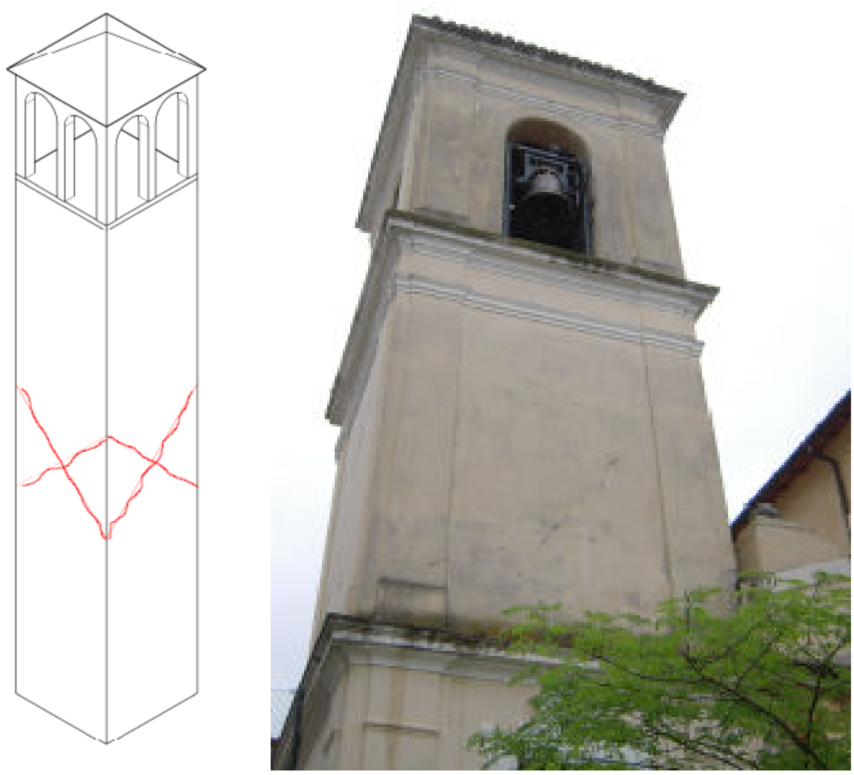
Note: *Hammering between adjacent parts due to out of phase vibrations can determine local damage and even collapse of the wall, when the gap between the two parts is lower than the relative displacement induced by the earthquake.*

Figure B.18: Hammering between adjacent parts



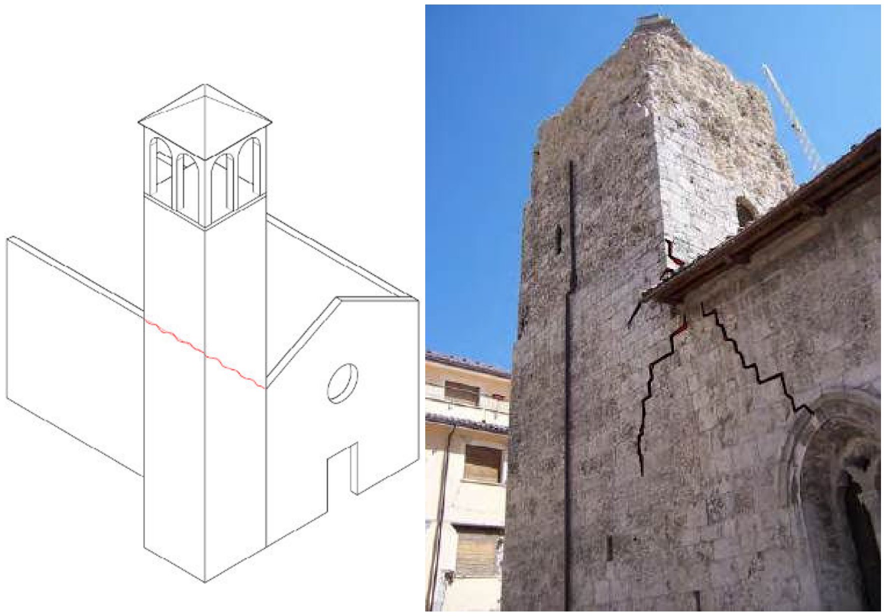
Note: *At the connection with the belfry, the top of the bell tower shows diagonal or vertical cracks, which open under the action transmitted by the belfry, when the top of the bell tower is not suitably connected in the horizontal plane.*

Figure B.19: Bell tower – Sending off of the sides and/or of the corners



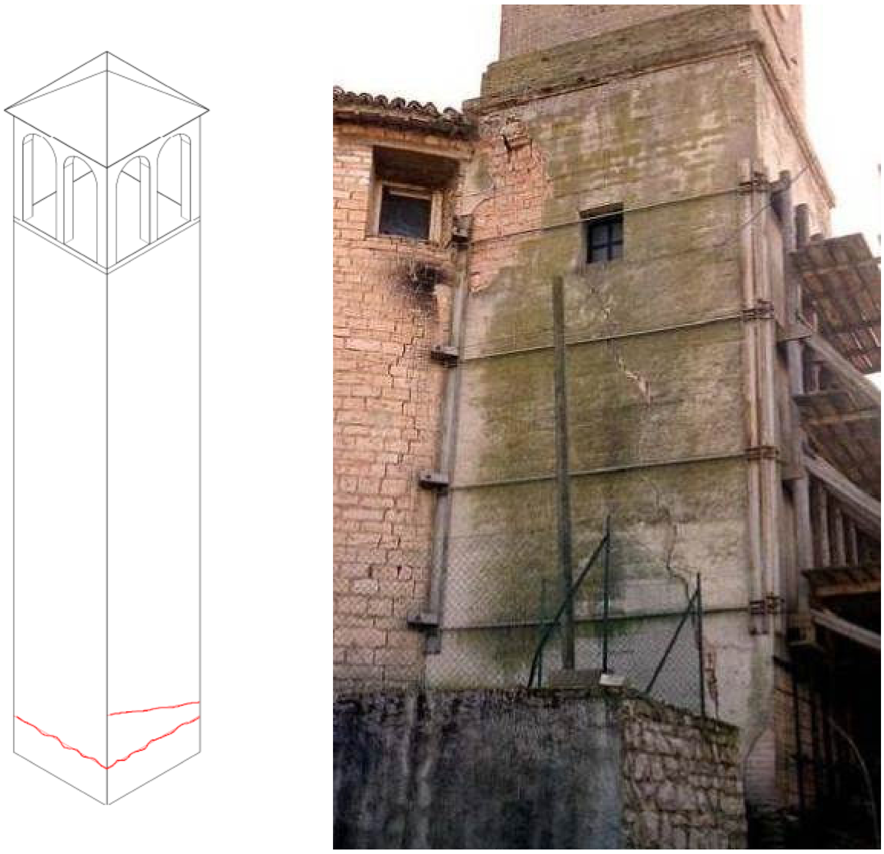
Note: *Diagonal cracks in the bell tower can be induced by shear failure or by sliding.*

Figure B.20: Bell tower – Shear or sliding cracks



Note: *Partial overturning of the bell tower involves the upper part of the bell tower which rotates around a horizontal cylindrical hinge or a spherical hinge located at a certain height: generally located the bell tower is connected to the main body of the church.*

Figure B.21: Bell tower – Partial overturning



Note: *Global overturning of the bell tower involves the bell tower in whole, rotating around a horizontal cylindrical hinge or a spherical hinge located at the base level of the bell tower itself.*

Figure B.22: Bell tower – Global overturning

Note: Belfry can collapse due to sliding, tilting or buckling of the piers.

Figure B.23: Belfry – Sliding, tilting or instability of piers

Note: *Arch cracks and in plane mechanism induced by relative translation and/or relative rotation between the belfry and the bell tower.*

Figure B.24: Belfry – Arch cracks and in plane mechanism

Note: *Steeple can collapse due to sliding, tilting and in plane mecganism.*

Figure B.25: Steeple – Sliding, tilting and in plane mechanism

CHAPTER 3 - ANNEX C

NON-LINEAR KINEMATIC ANALYSIS – PRACTICAL EXAMPLE

The present appendix is devoted to illustrate, referring to a significant practical example, the application of the non-linear kinematic analysis, illustrated in §3.4.

The considered mechanism is the global overturning of a church façade around a horizontal cylindrical hinge located at bottom edge of the front door, previously summarized in figure B.2 (see fig. C.1).

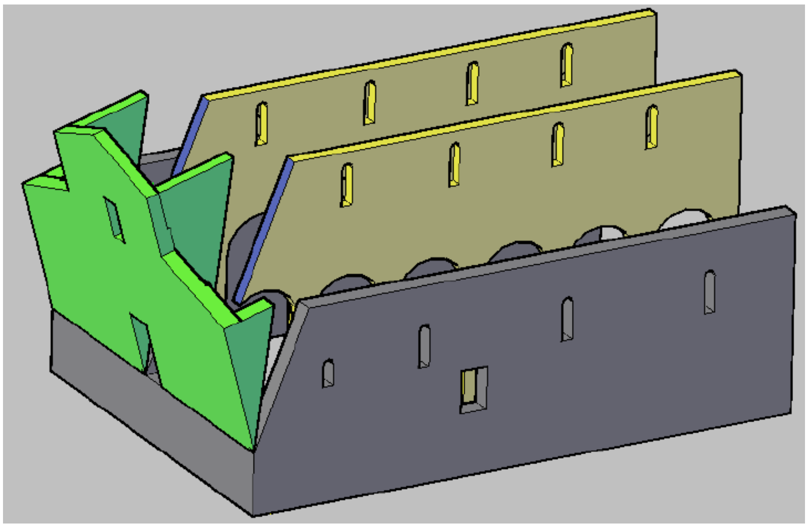


Figure C.1: Global overturning mechanism of the façade considered in non-linear analysis

Referring to figure C.2, we can consider the façade composed by three bodies, whose centers of gravity are indicated are G_1, G_2 and G_3, respectively.

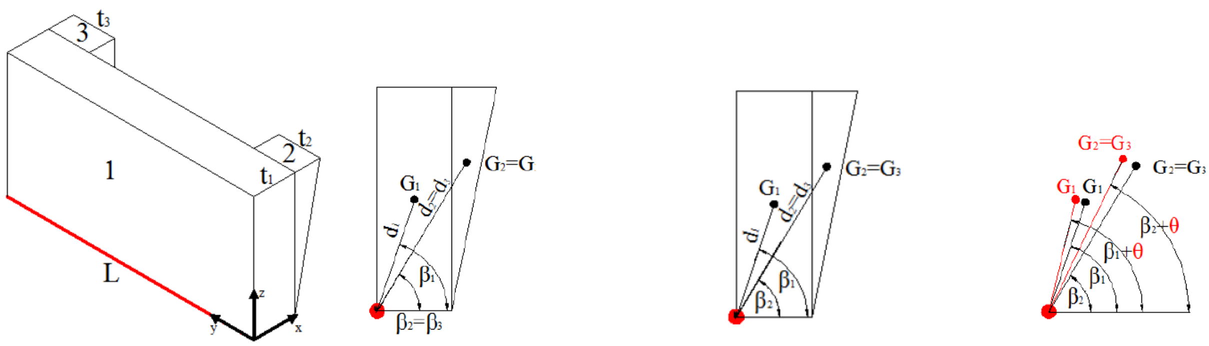


Figure C.2: Problem geometry and relevant parameter

Said π_1, π_2 and π_3 the planes containing the hinge axis and G_1, G_2 and G_3, respectively, be β_1, β_2 and β_3, β_2=β_3, the angles formed by the horizontal plane with these planes, and θ the angle describing the rotation of the façade.

The coordinates of the center of gravity can be then expressed by

$$G_1 \equiv \begin{pmatrix} d_1 \cos(\beta_1+\theta) \\ \frac{L}{2} \\ d_1 \sin(\beta_1+\theta) \end{pmatrix}; \; G_2 \equiv \begin{pmatrix} d_2 \cos(\beta_2+\theta) \\ \frac{t_2}{2} \\ d_2 \sin(\beta_2+\theta) \end{pmatrix}; \; G_3 \equiv \begin{pmatrix} d_3 \cos(\beta_3+\theta) \\ L-\frac{t_3}{2} \\ d_3 \sin(\beta_3+\theta) \end{pmatrix} \qquad \text{(C.1)}$$

where L is the length of the façade, t_i is the thickness of the i-th body and d_i is the distance of G_i from the hinge axis.

To perform non-linear kinematic analysis it is possible to consider a step-by-step procedure where increasing at each step the angle θ by a constant and suitably small increment θ^*: in this way the positions of the centers of gravity G_i at the n-th steps are

$$G_{1-n} \equiv \begin{pmatrix} d_1 \cos(\beta_1+n\theta^*) \\ \frac{L}{2} \\ d_1 \sin(\beta_1+n\theta^*) \end{pmatrix}; \; G_2 \equiv \begin{pmatrix} d_2 \cos(\beta_2+n\theta^*) \\ \frac{t_2}{2} \\ d_2 \sin(\beta_2+n\theta^*) \end{pmatrix}; \; G_3 \equiv \begin{pmatrix} d_3 \cos(\beta_3+n\theta^*) \\ L-\frac{t_3}{2} \\ d_3 \sin(\beta_3+n\theta^*) \end{pmatrix} \qquad \text{(C.2)}$$

which are displaced by $\delta G_{i\text{-n}}$ from the positions occupied at the (n-1)-th step:

$$\delta G_{i-n} \equiv \begin{pmatrix} d_i \left[\cos(\beta_1+n\theta^*) - \cos(\beta_1+(n-1)\theta^*)\right] \\ 0 \\ d_i \left[\sin(\beta_1+n\theta^*) - \sin(\beta_1+(n-1)\theta^*)\right] \end{pmatrix}. \qquad \text{(C.3)}$$

Applying the Virtual Work Principle (eq. (1)) at each step, it is possible to derive the corresponding load multiplier, so that a suitable $\alpha=\alpha(d_k)$ function can be derived, where d_k is the horizontal displacement of a suitable control point, generally corresponding to the center of gravity of the whole macroelement. The procedure ends when α=0.

For the considered example, the α-d_k curve is reported in figure C.3, while the a^*-α curve is shown in figure C.4, being a^* the spectral acceleration.

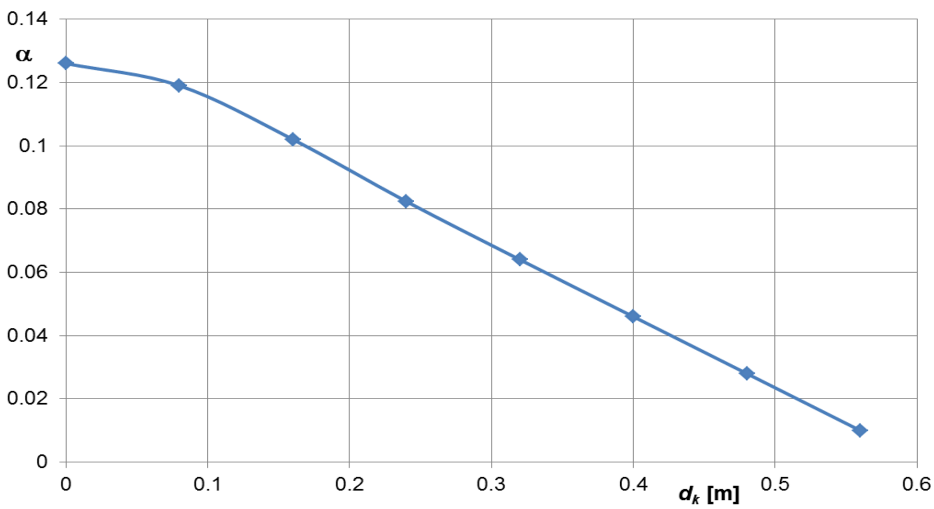


Figure C.3: α-d_k curve for the considered example, as deduced from non-linear analysis

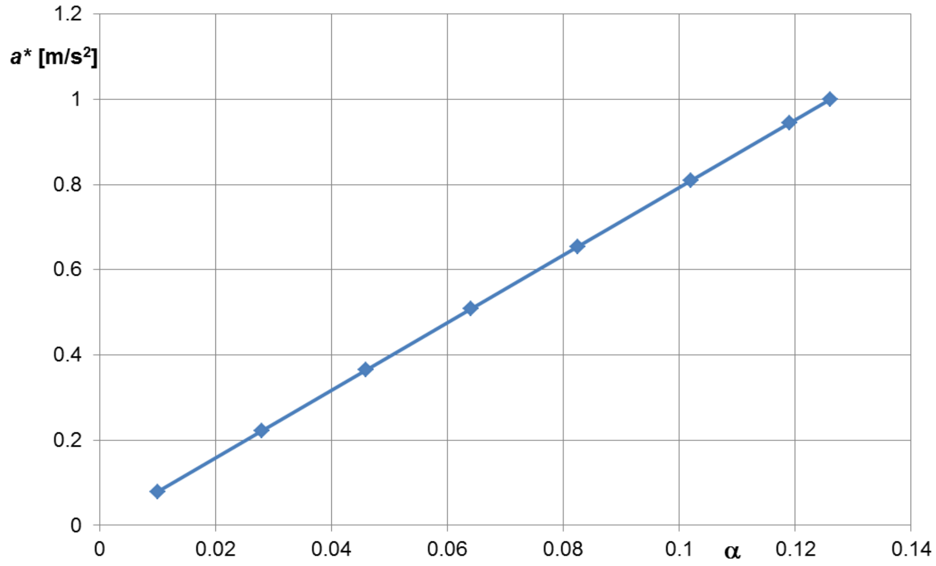


Figure C.4: a^*-α curve for the considered example, as deduced from non-linear analysis

Combining the two diagrams, the a^*-d_k curve shown in figure C.5 can be finally derived, so that all the information regarding the structural resistance are known and the seismic assessments can be performed.

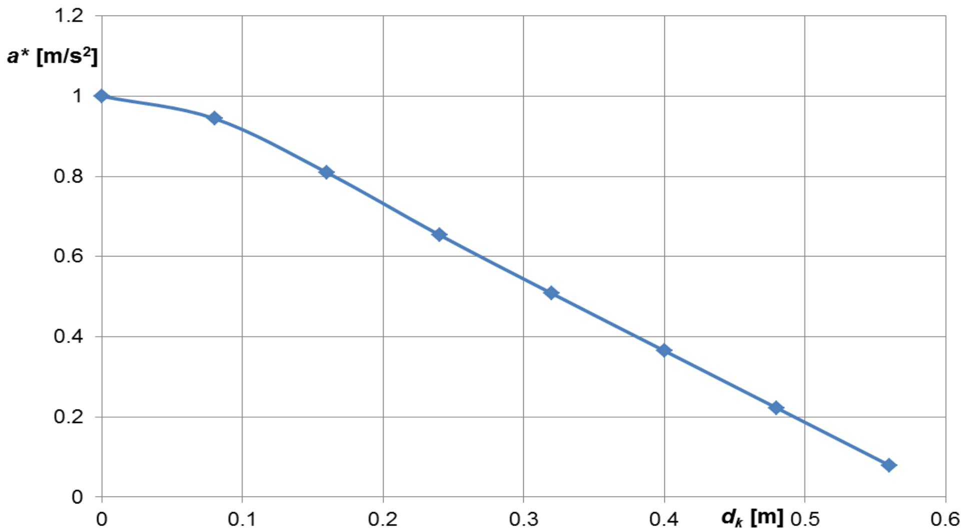


Figure C.5: a^*-d_k curve for the considered example, as deduced from non-linear analysis

SLS verifications

According to §4.2 and with the same symbols, SLS verifications are satisfied if the two following conditions are simultaneously satisfied:

1. the spectral acceleration activating the mechanism, a_0^*, is higher than the spectral acceleration to the ground: $a_0^* \geq a_g(P_{VR})S$;
2. the spectral acceleration activating the mechanism, α_0^*, is higher than the peak acceleration demand at the height Z, $a_0^* \geq S_e(T_1)\psi(Z)\gamma$.

In the present case:

Condition nr. 1 is not satisfied as it results:

$$a_0^* = 0{,}989\ \mathrm{m/s^2} < a_g\ S = 1{,}294\ \mathrm{m/s^2}\ . \qquad \text{(C.4)}$$

Condition nr. 2 is not satisfied too, in fact:

$$a_0^* = 0{,}989\ \mathrm{m/s^2} < S_e(T_1)\psi(\mathrm{Z})\gamma = 0{,}171\ g\ 0{,}164 \cdot 1{,}1 = 1{,}132\ \mathrm{m/s^2}, \qquad \text{(C.5)}$$

being T_1=0,65 s; $S_e(T_1)$=0,171 g; γ=1,1 and $\psi\,(Z)$=0,614. As usual, in eq. C.5, $\psi\,(Z)$ has been evaluated considering an approximately linear mode shape, so that $\psi\,(Z)=Z/H$ (Z=9,6 m and H=15,7 m).

ULS verifications

The ULS verification is satisfied if $d_u^* \geq S_{De}(T_s)$, being $S_{De}(T_s)$ the elastic displacement spectrum.

In the present case, the above mentioned condition is fulfilled, in fact:

$d_0^* = 0{,}171\ \mathrm{m}$;

$d_u^* = 0{,}301\ \mathrm{m}$;

$d_s^* = 0{,}4\ d_u^* = 0{,}12\ \mathrm{m}$;

$a_s^* = 0{,},874\ \mathrm{m/s^2}$;

$$T_s = 2\,\pi \sqrt{\frac{d_s^*}{a_s^*}} = 2{,}33\ \mathrm{s}$$

$d_u^* = 0{,}301\ \mathrm{m} > S_{De}(T_s) = 0{,}167\ \mathrm{m}$.

CHAPTER 4: VERIFICATION PROCEDURES

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1 INTRODUCTION

General requirements on performance of (existing) structures may be found in the International standards EN 1990 [1], ISO 2394 [4] and ISO 13822 [6] and in other recommendations such as [2, 3, 6]. In addition to these general documents, individual countries may have special codes for steel and/or concrete bridges.

On a European level basic requirements on construction works including existing structures are given in the Construction Product Requirements. It is stated there that construction works shall be designed, built and maintained in such a way that the loadings which are liable to act on them during their execution and use will lead to adequate reliability that:

- the collapse of the whole or part of the construction works
- an inadmissible degree of deformations
- damage to other parts of the works or to fittings or installed equipment as a result of major deformation of the load-bearing structure
- damage by an event to an extent disproportionate to the original cause.

The basic requirements on construction works are also given in EN 1990 [1] where basic provisions how to achieve adequate structural resistance, robustness, serviceability and durability are provided. However, the Eurocodes are elaborated for design only. Main differences between design and assessment is that existing structures should and can be judged on the basis of actual (updated) material properties, action effects and environmental influences and that for socio-economic reasons deviating reliability levels may be acceptable. Also the residual working life may differ from the design working life, while on the other hand the preceding life of the structure should be taken into account.

2 BASIC CONCEPTS

2.1 Residual life time

The notion of a residual working life is useful for:

- the selection of design actions (e.g. imposed load, wind, earthquake etc.) and the consideration of material property deterioration (e.g. fatigue, creep) in reliability verification.
- comparison of different design solutions and choice of materials, each of which will give a different balance between the initial cost and cost over an agreed period - life cycle costing will need to be undertaken to evaluate the relative economics of the different solutions.

– evolving management procedures and strategies for systematic maintenance and renovation of structures.

In general the residual lifetime for an existing structure shall be shorter than the design life time for new structures.

2.2 Design situations

Environmental influences and structural properties, which occur throughout the remaining working life of a structure, should be considered by selecting distinct situations representing a certain time interval with associated hazards.

Four design situations are classified in EN 1990 [3] as follows:

(a) persistent situations refer to conditions of normal use. These are generally related to the design working life of the structure. Normal use can include possible extreme loading conditions from wind, snow, imposed loads, etc.
(b) transient situations refer to temporary conditions of the structure, in terms of its use or its exposure, e.g. during construction or repair. This implies the use of a time period much shorter than the design working life; one year may be adopted in most cases.
(c) accidental situations refer to exceptional conditions of the structure or of its exposure, e.g. due to fire, explosion, impact, local failure. This implies the use of a relatively short period, but not for those situations where a local failure may remain undetected.
(d) seismic situations refer to exceptional conditions applicable to the structure when subjected to seismic events.

These design situations should be selected so as to encompass all conditions which are reasonably foreseeable or occurring during the anticipated use of the structure. For example a structure after an accidental design situation due to actions like fire or impact may need a repair (short time period of about one year), for which the transient design situation should be considered. In general a lower reliability level and lower partial factors than those used for persistent design situation might be applicable for this period of time. However, it should be mentioned that the repair should be designed considering all the other foreseeable design situations.

2.3 Limit states

According to the concept of limit states it is considered that the states of any structure may be classified as either satisfactory (undamaged, serviceable) or unsatisfactory (failed, unserviceable). Distinct conditions separating satisfactory and unsatisfactory states of a structure are called limit states. Thus, the limit states are those beyond which the structure no longer satisfies the performance criteria. Each limit state is therefore associated with a certain performance requirement imposed on a structure. Limit states may be sharp as well as gradual. In the last case often a sharp definition is often used for convenience. Usually two different types of limit states are recognised (a) ultimate limit states and (b) serviceability limit states.

Ultimate limit states are associated with:

(a) loss of equilibrium of the structure or any part of it, considered as a rigid body;
(b) failure of the structure or part of it due to rupture, fatigue or excessive deformation;
(c) instability of the structure or one of its parts;

(d) transformation of the structure or part of it into a mechanism;
(e) sudden change of the structural system to a new system (e.g. snap through).

Time dependent structural properties, such as fatigue and other time dependent deterioration mechanisms reduce the strength of a structure and can initiate one of the above mentioned ultimate limit states. In this respect it is useful to distinguish two types of structures: damage tolerant (i.e. robust) and damage intolerant (sensitive to minor disturbance or construction imperfections). Effects of various deteriorating mechanisms on the ultimate limit states should then be taken into account according to the type of the structure.

The serviceability limit states are associated with conditions of normal use. In particular they concern the functioning of the structure or structural members, comfort of people and appearance of the construction works. For existing structures the judgement regarding service-ability limit states can often be considered given the actual behaviour of the structure. For durability aspects a further evaluation may be necessary.

2.4 Global Failure and Robustness

In order to minimize the likelihood of failures many current building codes consider the need for robustness in structures and provide some strategies for their achievement. In Eurocode EN 1990 [3], the basic requirement concerning robustness is given in Chapter 2 where it is stated that "*a structure should be designed and executed in such a way that it will not be damaged by events such as: explosion, impact, and the consequences of human errors, to an extent disproportionate to the original cause*."

In general it is considered as extremely expensive to improve the robustness properties of an existing building. In EN 1990 [3] the existing structures are classified in three different consequence classes, according the consequence of the collapse and of the malfunction of the structure itself. In terms of loss of human life and economic, social and environmental impacts, low consequences are foreseen in case of failure of consequence class 1 (CC1) structures; medium consequences are foreseen in case of failure of consequence class 2 (CC2) structures; high consequences are foreseen in case of failure of consequence class 3 (CC3) structures.

For CC1 and CC2 structures additional measures are not recommended, while for CC3 structures a risk analysis may show which measures are necessary in order to keep an adequate safety level.

3 UPDATING

Characteristic values in the assessment procedure should be reflect as much as possible the real existing situation. Where possible probability distributions of material and geometric properties should be brought in accordance with the as built situation, later modifications and possible effects of time and loading. The findings at inspection and measurements should be incorporated using appropriate updating procedures as will be discussed in chapters 4 and 5.

In some cases also loadings may be subject to updating, in particular the permanent loads, but also the models for time varying loads may contain time-invariant parts that can be subject to measurement and adjustment (see chapter 5).

4 PROOF LOADING

The extreme inspection is a proof loading. Based on such tests one may draw conclusions with respect to:

- the resistance of the tested member;
- the resistance of other similar members;
- the resistance under other conditions;
- the behaviour of the system.

Based on the proof load results the reliability estimate of the structure can be updated. The inference in the first case is relatively easy. The probability density function of the load bearing capacity is simply cut off at the value of the proof load. The inference of the other conditions is more complex. It should be noted that the number of proof load tests does not need to be restricted to one.

In case of a proof loading it is necessary to start with a calculation as far as possible. Based on that the test set up and the objectives should be formulated. Loads should be increased slowly and the behavior of the structure during the test should be monitored carefully in order to avoid unnecessary damage or collapse. Apart from that, all measured data may help to get a better understanding of the structural properties and load bearing abilities. As proof loading usually have a short duration, long term effects should be taken care of in another way.

The failure probability given a survived proof load level, should of course fulfill the same reliability requirement as in the case of an analytical verification:

$$P\left(R < S \middle| R > S_{proof}\right) < \Phi\left(-\beta_{\text{target}}\right), \tag{1}$$

where R is the resistance, S the load, S_{proof} the maximum proof load level, $\Phi(.)$ the standard normal distribution function and β_{target} the target reliability index, that may have been reduced for economic reasons. In most cases this requirement will be translated to partial factor format.

5 PARTIAL FACTORS

The theory behind partial factors has been discussed in Handbook 1. The basic formula according to ISO 2394 for example for a resistance factor in case of a normal distribution can be written as:

$$\gamma_m = \frac{x_k}{x_d} = \frac{\mu - k\sigma}{\mu - \alpha\beta\sigma}, \tag{2}$$

where x_d is the design value for X, x_k the characteristic value, μ the mean value, σ the standard deviation, α the probabilistic influence coefficient, β the target reliability index and k=1,64 (5%–fractile of the standardized normal distribution) is usually used. For other distributions similar equations may be derived.

Note that values may be different compared to design. First of all statistical parameters like mean and standard deviation should be based on the updating procedures. Usually the mean will be higher and the standard deviation smaller. Next the target reliability may have changed for economic reasons. It may take quite some efforts to improve the safety of an existing structure. Small improvements usually should be avoided for that reason. As a guideline one could drop the reliability index by 0,5 or 1,0. Rules for this reduction should be found in the national regulations.

As an example, consider a resistance having a coefficient of variation of 10%. For CC2 the standard target reliability index for design is 3,8. For α=0,8 we then arrive at γ_m =1,20. Reducing the target beta form 3,8 to 2,8 leads to γ_m =1,08. If measurements are available, a change in the mean does not affect the partial factor (only the characteristic values) but a change in the observed scatter may help. If the updated coefficient of variation drops from 10 to 6% , the gamma value reduces further from 1,08 till 1,04.

In the case of proof loading many uncertainties existing in analytical verification are no longer present. In particular the self-weight has no uncertainty and the same holds for the stresses raised by the self-weight. This means that the partial factor be taken a 1,0, in other words, there is no need to increase the self-weight artificially value 10 or 20 % higher value. A similar argument holds for the resistance Variable loads should of course be introduced with their design values, although also here the safety margin for the structural model may be excluded. Note that there may be reasons to adjust the standard α-values in the formula for the design values.

The above arguments hold only for the elements and mechanisms present in the test. If one for instance wants to stretch the conclusions of a proof loading to other elements, load configurations or mechanism one should again consider safety margins for the corresponding uncertainties. In other chapters examples will be considered.

6 UPDATED DESIGN VALUES

Instead of using partial safety factors one may apply directly updated design values based on the well-known FORM results. Common verification methods take basis in design equations from which the reliability verification of a given design may be easily performed by a simple comparison of resistances and loads and/or load effects. Due to the fact that loads and resistances are subject to uncertainties, design values for resistances and load effects are introduced in the design equations to ensure that the design is associated with an adequate level of reliability. When deriving design values one may consider:

- costs of safety measures;
- remaining lifetime;
- degree of information available.

A verification is considered to be sufficient if the limit states are not reached when the design values are introduced into the analysis models. In symbolic notation this is expressed as:

$$e_d < r_d \, . \tag{3}$$

This is the practical way to ensure that the reliability index β is equal to or larger than the target value.

The design values of action effects e_d and resistances r_d should be defined such that the probability of having a more unfavourable value is as follows :

$$P(E > e_d) = F_x(e_d) = \Phi(+\alpha_E \beta) \tag{4.a}$$

$$P(R < r_d) = F_x(r_d) = \Phi(-\alpha_R \beta) \tag{4.b}$$

where:

- F_x is the cumulative distribution function;
- β is the target reliability index;
- α_E and α_R are the sensitivity factors.

The design values are obtained as:

$$E_d = F_x^{-1}(\Phi(+\alpha_E\beta)) \quad (5.a)$$
$$R_d = F_x^{-1}(\Phi(-\alpha_R\beta)) \quad (5.b)$$

The expressions provided in the following table should be used for deriving the design values of variables with the given probability distribution.

Table 1. Design values for common distributions

Distribution	**Design values**
Normal	$\mu - \alpha\beta\sigma$
Lognormal	$\mu \exp(-\alpha\beta \mathrm{V})$ for $V = \dfrac{\sigma}{\mu} < 0{,}2$
Gumbel	$u - \dfrac{1}{\mathrm{a}}\ln\{-\ln[\Phi(-\alpha\beta)]\}$ where $u = \mu - \dfrac{0{,}577}{a}$; $a = \dfrac{\pi}{\sigma\sqrt{6}}$

The reliability index β can be taken from the target values as recommended chapter 9 of Handbook 1. Examples for verification are shown in other chapters of the present Handbook.

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CHAPTER 5: APPLICATIONS OF UPDATING

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1 INTRODUCTION

Updating information is one of the most important tasks in the assessment of existing structures, for the reliability of the evaluation depends on the degree of uncertainty associated with structural analysis variables. Information for the intents and purposes of assessment may be updated in a number of ways and may include:

- material properties determined by non-destructive or destructive testing;
- geometric characteristics and permanent actions determined by component dimensions measured during inspection;
- environmental effects identified during inspection;
- damage and deterioration detected during inspection;
- actual load carrying capacity estimated by proof loading.

Probabilistic methods may be used to combine prior information about a variable with test results and measurements. In a fully probabilistic procedure, the so-called prior distribution function must first be established for the unknown distribution parameters of a random variable. The distribution should reflect all the available information about the parameters. Such prior distributions, together with the statistical data obtained from tests and measurements, can be used to derive a posterior (updated) distribution for the random variable [1]. The posterior probability density function (PDF) for the distribution parameters can be obtained applying the Bayes theorem, which calls for weighting the available information [1].

2 UPDATING REINFORCING STEEL YIELD STRENGTH

2.1 Statement of the problem

The assessment of an existing RC structure entailed updating the characteristic value and the distribution function parameters for the yield strength of the reinforcing steel used in its construction. Prior information on the distribution parameters was available from a series of tests performed at an earlier stage of the assessment.

2.2 Prior information

The mean yield strength found with the four tests comprising the trial was 268,8 MPa, with a standard deviation of 15,64 MPa. Based on these data, the prior probabilistic model for reinforcing steel yield strength, f_y, was assumed to be log-normally distributed, with the aforementioned uncertain parameters, whereby:

$$Y = \ln(X) \equiv N(\mu,\sigma) \tag{1}$$

is normal. Given the equivalence described in (1), all the expressions obtained for normally distributed variables (e.g., *Y)* can be applied for the log-normal variable $X = \exp(Y)$ [2].

According to [1, 2], prior information on distribution parameters can be characterised by four estimators:

n_0, prior sample size
m_0, prior logarithmic sample mean
w_0, prior logarithmic sample standard deviation
c_0, prior number of degrees of freedom for w_0

Further to the available information, the values for the prior distribution estimators in this case study were as follows: $m_0 = 5{,}592$, $w_0 = 0{,}0686$, $n_0 = 4$ and $c_0 = n_0 + 1 = 4 + 1 = 5$.

2.3 Evaluation of tests results

The second stage of structural assessment included an additional data acquisition campaign, in which tensile tests were conducted on five cores drilled in the structure to determine the yield strength of the steel. The findings were: f_{ys} = (270, 274, 277, 268, 281) N/mm^2.

The parameter values obtained for the sample were as follows:

- sample size, $n = 5$
- logarithmic sample mean, $\overline{y} = 5{,}613$
- logarithmic sample standard deviation, $s_y = 0{,}019$.

2.4 Updating PDF parameters

The aforementioned prior information was combined with the results of *n* observations to obtain the respective posterior estimators, n_1, m_1, w_1 and c_1. A detailed explanation of this procedure can be found in [2]. The expressions for the updated values of the logarithmic mean, $\mu_{\ln X,act}$, the logarithmic standard deviation, $\sigma_{\ln X,act}$, and the characteristic value were:

$$\mu_{\ln_{X,act}} = m_1 = \frac{n_0 m_0 + n\overline{y}}{n_1} \tag{2}$$

$$\sigma_{\ln_{X,act}} = \sqrt{\frac{c_1}{c_1 - 2}} w_1 \quad \text{with } c_1 > 2 \tag{3}$$

$$X_{k,act} = \exp\left(m_1 - T_{c_1} w_1 \sqrt{1 + \frac{1}{n_1}} \right) \tag{4}$$

where:
m_1 = the posterior logarithmic mean
n = the sample size
n_1 = the posterior sample size
w_1 = the posterior logarithmic standard deviation
c_1 = the posterior number of degrees of freedom for w_1
T_{c_1} = the coefficient of the Student distribution for c_1 degrees of freedom and the assumed fractile (5 %).

The posterior distribution estimators were obtained from:

$$n_1 = n_0 + n \tag{5}$$

$$c_1 = c_0 + n \tag{6}$$

$$c_1 w_1^{\,2} = c_0 w_0^{\,2} + n s_y^{\,2} + \frac{n_0 n}{n_1}\left(m_0 - \bar{y}\right)^2 \tag{7}$$

Pursuant to (2), (5), (6) and (7): $n_1 = 9$; $c_1 = 10$; $m_1 = 5,604$ and $w_1 = 0,0513$.

The updated characteristic value of the steel yield strength and the updated distribution parameters were: $f_{ys,k,act}$ = 249,8 N/mm^2; $\mu_{fys,act}$ = 271,4 N/mm^2; $\sigma_{fys,act}$ = 13,2 N/mm^2; and $CoV_{fys,act}$ = 0,049.

The findings for this example are illustrated in Figure 2, which shows the variation in the information on steel yield strength as a result of combining the uncertain prior information and new test results. The effect of the latter proved to be quite significant. The updated parameters for this basic variable may be used for performing subsequent structural assessments [3].

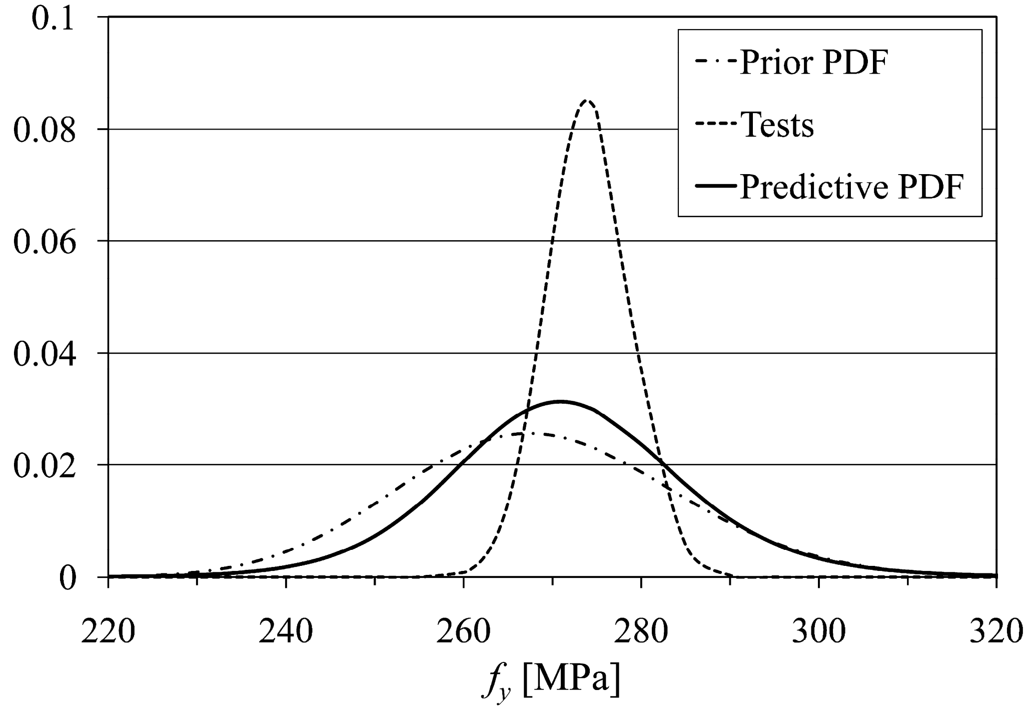

Figure 1: Prior probability density function (PDF), test results and predictive PDF for reinforcing steel yield strength

3 UPDATING THE OCCURRENCE RATE

The probability p of at least one event (accidental action, earthquake) during a period T can be derived by assuming that accidents follow a Poisson distribution, i.e.:

$$p = 1 - \exp\left(-\nu T\right) \tag{8}$$

where:
v = occurrence rate
T = reference time period (in existing structures, for instance, the desired residual service life).

The rate of occurrence of accidents is found with statistical analysis. Under the classical approach, if n events are observed over a given observation period t:

$$v = n / t \tag{9}$$

In a Bayesian approach the mean annual occurrence rate can be obtained by assuming a uniform prior distribution and applying the methodology described in [4, 5] as:

$$vT = -\ln\left[1/\left(1+T/t\right)^{n+1}\right] \tag{10}$$

with T = 1 year. The difference between expressions (9) and (10) is large only when the data available are very scant. For instance, if 0 events were observed in the last 5 years, expression (9) would yield an occurrence rate of 0 and expression (10) a rate of 0,182. If 2 events were observed in 5 years, the occurrence rate found with (9) would be 0,40 and with (10), 0,55. Where 0 events were observed in the last 50 years, the rate of occurrence according to expression (9) is 0 and according to (10), 0,02. If 2 events were observed in 50 years, expression (9) would calculate a rate of 0,04 while (10) would return a rate of 0,06. If 20 events were observed in 500 years the occurrence rate yielded by (9) would also be 0,04, whereas the value found with (10) would be 0,042. In other words, the longer the observation period, the closer are the results found with (9) and (10).

Since additional information on an existing structure, such as events occurring its past lifetime, is available during reassessment, the occurrence rate can be updated based on the aforementioned procedure.

4 PROOF LOADING

4.1 Survival under high loads

A critical analysis of the behaviour of a structure during its past lifetime provides useful insight into its present condition. Significant information might include, for instance, evidence that the structure successfully bore an extremely high load, E_E, during the time interval studied. The probability of failure in the redesign stage can be evaluated by using the conditional probability expression:

$$p_f = P\left[R - E \le 0 \mid R' - E' > 0\right] \tag{11}$$

where:
R' = resistance at $t = t_E$
E' = total loading effect at $t = t_E$.

Equation (11) can be rewritten as:

$$p_f = \frac{P\left[R - E \leq 0 \cap R' - E' > 0\right]}{P\left[R' - E' > 0\right]} \tag{12}$$

Condition (12) has a greater impact if the load successfully borne was high.

If the failure functions in (12) are assumed to adopt a simple fundamental two-dimensional form:

$$Z = R - E \tag{13}$$

and E_E is deterministic, the resistance distribution $f_R(r)$ can be truncated as:

$$f_{R'}(x) = \frac{1}{1 - F_R(E_E)} f_R(x) \qquad \text{for } x > E_E \tag{14}$$

where:

$f_R(x)$ = original strength distribution.

Assuming that the strength is normally distributed with a mean of μ_R and a standard deviation of σ_R, the following can be defined:

$$\lambda = \frac{E_E - \mu_R}{\sigma_R} \tag{15}$$

The mean and standard deviation of the calibrated strength distribution $f_{R'}(r)$ are obtained as follows:

$$\mu_{R'} = \mu_R + \frac{\varphi(\lambda)}{1 - \Phi(\lambda)} \sigma_R \tag{16}$$

$$\sigma_{R'} = \left[1 + \frac{\lambda \varphi(\lambda)}{1 - \Phi(\lambda)} - \left(\frac{\varphi(\lambda)}{1 - \Phi(\lambda)}\right)^2\right]^{1/2} \sigma_R \tag{17}$$

where:

φ (.) = probability density function for the standardised normal variable

Φ (.) = standard normal integral.

If load E_E is not deterministic but random, function $f_{R'}(x)$ can be evaluated numerically with the probability density function $f(E_E)$ from:

$$f_{R'}(x) = \int_0^\infty \frac{f_R(x)}{1 - F_R(E_E)} f(E_E)\, dE_E \tag{18}$$

Considering, for example, the statistical parameters for resistance R to be given by:

- μ_R = 29,3 N/mm^2
- σ_R = 4,0 N/mm^2

and taking the weighting factor for resistance to be α_R = 0,8, and the target safety factor to be β = 4,2, the design value found is as shown below, subject to the approximation that R is normally distributed:

$$x_R^* = \mu_R - \alpha_R \beta \sigma_R = 29,3 - 0,8 \cdot 4,2 \cdot 4 = 15,8 \text{ N/mm}^2 .$$

Assuming that the structural element survives a loading effect of magnitude E_E, characterised by a deterministic value of 28 N/mm^2, the new resistance design value should be E_E. To allow for a safety factor, however, a fictitious design resistance value is calculated based on the statistical parameters (updated mean value and standard deviation) for the truncated distribution.

Expressions (15), (16) and (17) yield:

- $\lambda = -1,32$
- $\mu_{R'}$ = 31,7 N/mm^2
- $\sigma_{R'}$ = 2,7 N/mm^2

With α_R=0,8 and β=4,2 and approximating R' to be normally distributed, the following design value is obtained:

$$x_{R'}^* = \mu_{R'} - \alpha_R \beta \sigma_{R'} = 31,7 - 0,8 \cdot 4,2 \cdot 2,7 = 22,8 \text{ N/mm}^2$$

which is lower than E_E = 28 N/mm^2 and therefore on the side of safety.

This mathematical procedure can be repeated to analyse different cases by varying the value of E_E. Raising the value of the extreme load would be expected to yield higher design resistance values, because known past performance with satisfactory results would heighten confidence in structural soundness. Conversely, a lower extreme load would provide no new knowledge about structural e resistance, and would have only a marginal effect on the updated design value.

Entering E_E=24 N/mm^2, for instance, in the aforementioned procedure, yields:

- $\lambda = -0,33$
- $\mu_{R'}$ = 30,0 N/mm^2
- $\sigma_{R'}$ = 3,4 N/mm^2

For α_R = 0,8 and β = 4,2, the conservative updated design value obtained is:

$$x_{R'}^* = \mu_{R'} - \alpha_R \beta \sigma_{R'} = 30,0 - 0,8 \cdot 4,2 \cdot 3,4 = 18,6 \text{ N/mm}^2$$

This example shows how statistical parameters can be updated and how the results can be used to obtain conservative reassessments.

4.2 Floor structure in a residential building

4.2.1 Overview

The structure tested was a reinforced concrete floor in a residential building completed in 2007. Due to problems observed during construction, the proof load test was conducted before the permanent loads were applied. A total area of 61.9 m^2 was tested. The plan view of the tested area and the longitudinal and cross sections of beam V-23, whose resistance was evaluated by load testing, are shown in figure 2. The reliability assessment has been simplified here for the intention is not to discuss case-specific details, but to illustrate the effect of proof loading survival on structural reliability.

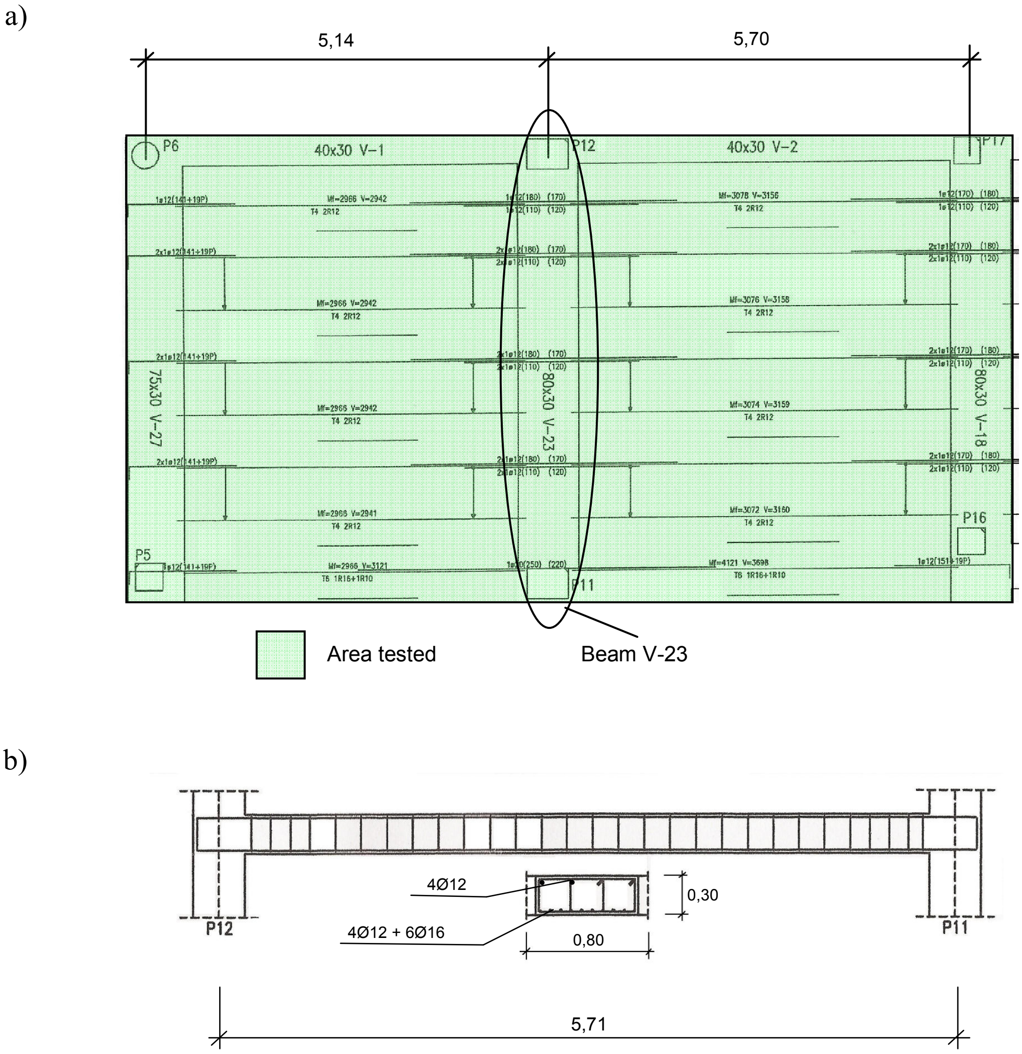

Figure 2. a): Plan view of area tested; b) longitudinal and cross-section of beam V-23 (dimensions in m).

4.2.2 Pre-test probability of failure

Beam V-23 reliability was first verified using the design information available about the structure. Code VaP 3.0 [6] was used to calculate the failure probability, p_f, and the reliability index, β. The Limit State Function (LSF) for mid-span bending failure was deduced from the design rules for reinforced concrete structures laid down in Spanish codes [7, 8, 9]. The LSF can be written as:

$$G = \xi_R \left[A_s \cdot f_y \cdot d - 0,5 \frac{\left(A_s \cdot f_y\right)^2}{b \cdot f_c} \right] - \xi_E \left[M_{G_c} + M_{G_p} + M_Q \right] \tag{19}$$

The basic LSF variables set out in (19) and their respective probabilistic models are listed in Table 1. Assuming a 50-year reference period, the probability of failure calculated from the LSF equation (19) was $p_f = 2,54 \times 10^{-4}$ and the reliability index $\beta = 3,48$. The probabilistic models for the variables in Table 1 are consistent with the models listed in the Probabilistic Model Code [10] and represent the state of uncertainty associated with the design rules for reinforced concrete structures laid down in Spanish codes [7, 8, 9]. The procedure used for and results obtained in developing probabilistic models for structural design variables are discussed in [11]. The (nominal) representative values for the basic variables given in Table 1 were determined from the design documentation on the structure.

Table 1. Probabilistic models for basic variables

Variable	Symbol	Unit	Distribution type	Mean value	Standard deviation
Concrete compressive strength	f_c	N/mm^2	LN	31	5,6
Reinforcement yield strength	f_y	N/mm^2	LN	560	29,7
Reinforcement area	A_s	mm^2	N	1659	33,2
Effective depth	d	mm	N	275	11
Width of cross-section	b	mm	N	800	24
Resistance model uncertainty	ξ_R	-	LN	1	0,05
Moment due to the self weight of concrete	M_{G_c}	kN·m	N	68,48	2,74
Moment due to permanent loads (except self weight)	M_{G_p}	kN·m	N	44,18	4,418
Moment due to the imposed load	M_Q	kN·m	Gumbel	30,0	7,81
Load effect uncertainty	ξ_E	-	LN	1	0,1

The probabilistic model for concrete compressive strength, f_c, was updated with the results of destructive tests on six cores drilled in the structure itself. The updated model for concrete strength was found to be log-normally distributed with a mean value of 24,9 N/mm^2 and a standard deviation of 4,17 N/mm^2. Using this model, the estimated reliability index, β, declined to 3,361 while the probability of failure rose to $3,885 \times 10^{-4}$.

4.2.3 Updated probability of failure

The reinforced concrete floor was proof load-tested to determine its structural integrity. The test consisted of gradually raising the proof load to a maximum of 5,55 kN/m^2, or 92,5 % of the sum of the design imposed load, Q_d, plus the design permanent loads except self weight, G_{pd}. The deflection measured during the test in the mid-span section was 2,19 mm, i.e., under the 4,66-mm ceiling laid down in [9].

Updating the previous estimate of structural reliability on the grounds of the successful proof load test [12] led to the following implication:

$$H = \xi_R \left[A_s \cdot f_y \cdot d - 0,5 \frac{\left(A_s \cdot f_y\right)^2}{b \cdot f_c} \right] - \xi_E \left[M_{G_c} + M_{Q_{PL}} \right] > 0 \quad (20)$$

where:
$M_{Q_{PL}}$ = proof load bending moment.

The proof load data, Q_{PL}, could then be used to update the probability of failure, p_f'', from the following conditional probability expression:

$$p_f'' = \left[G \le 0 \mid H > 0\right] = \frac{P\left[G \le 0 \cap H > 0\right]}{P\left[H > 0\right]} \tag{21}$$

The probabilistic models for the basic variables set out in equation (21) are defined in Table 1. Three probabilistic models for the proof load bending moment were used to determine the post-test reliability index and probability of failure, thereby estimating the sensitivity of the results. The applied probabilistic models are listed in Table 2.

Table 2 Probalistic models for the proof load moment, $M_{Q_{PL}}$

Variable	Symbol	Unit	Distribution type	Nominal value	Mean value	Standard deviation
Proof load moment	$M_{Q_{PL}}$	kN·m	Gumbel	122,6	83,37	21,67
		kN·m	N	122,6	122,6	6,13
		kN·m	Det	122,6	-	-

In light of the satisfactory performance of the structure in the proof load test, the updated reliability index, β'', and probability of failure, p_f'' were calculated using equation (21) and three probabilistic models for the proof load moment (Table 2). The results for β'', p_f'' and the probability of failure during load testing, $P(\bar{H}) = P(H < 0)$, are summarized in Table 3. As expected, the findings showed that the greater the scatter of proof load intensity, the greater was the updated probability of failure, but the lower the probability of structural failure during the proof load test.

Table 3. Post-test reliability analysis. Proof load effect results for three probabilistic models

Distribution type	β''	p_f''	$P(\bar{H})$
Gumbel	3,45	$2,772\times10^{-4}$	$1,385\times10^{-2}$
Normal	4,51	$3,301\times10^{-6}$	$6,779\times10^{-2}$
Deterministic	4,63	$1,820\times10^{-6}$	$6,239\times10^{-2}$

The effect of proof load intensity was assessed by assuming that the structure successfully passed two new tests under proof loads equal to 1,2 and 1,5 times G_{pd} plus Q_d. The reliability of the reinforced concrete beam was updated taking the proof load effect, $M_{Q_{PL}}$, to be deterministic.

Table 4 gives the findings for the updated reliability index, β'', the updated probability of failure, p_f'', and the probability of failure during the proof load test, $P(\bar{H})$. As expected, these findings showed that raising the proof load intensity led to significantly lower probability of failure values, but a significantly higher likelihood of proof load-induced failure.

Table 4. Results of post-test reliability analysis for hypothetical proof load intensities

Proof load intensity	β''	p_f''	$P(\bar{H})$
$Q_{PL} = 1,2\left(G_{pd} + Q_d\right)$	5,88	$2{,}042\times10^{-9}$	0,3126
$Q_{PL} = 1,5\left(G_{pd} + Q_d\right)$	7,54	$2{,}323\times10^{-14}$	0,7652

5 LOWERING THE SEISMIC REQUIREMENTS FOR EXISTING STRUCTURES

Peak ground acceleration (*PGA*), *a*, is the most widely used design parameter in structural engineering, given its immediate relationship to the induced seismic forces that lay at the heart of current structural seismic design procedures [13].

The curve that plots earthquake intensity (*PGA* = *a*) at a given location against the respective probability of exceedance is called site-specific seismic hazard curve, generally expressed as:

$$p = c_1 A^{-c_2} \tag{22}$$

where:
p = probability of exceedance limit
A = peak ground acceleration in g
c_1 and c_2 = site-specific constants.

This curve has to be plotted for each site. The city of Cosenza, Italy, whose hazard curve fits the expression (23), is assumed as the reference site, [14]:

$$p = 0{,}0001 A^{-2{,}218} \tag{23}$$

In current practice, the lifelong limit state must be calculated for an earthquake with a 475-year return period. The 50-year probability of exceedance is therefore 10 % and the annual probability, p = 0,0021. The associated design value is consequently a_d = 0,27g. The design value is related to the reliability index β (see Chapter 3 of this Handbook) as follows:

$$P\left(A > a_d\right) = \Phi\left(+\alpha_a \beta\right) \tag{24}$$

Assuming a sensitivity factor of α_a= −0,7 the reliability index comes to β=4,09.

For existing structures, the cost of achieving a higher reliability level is usually higher than for structures that are still in the design phase. The target level for existing buildings should therefore be lower (see [15] or Handbook 1 for a fuller discussion). Assuming the target β value is reduced by 0,5, lowering the consequence class established in [16] and hence the safety requirements for existing structures, the reliability index can be defined as:

$$\beta_{\text{existing}} = \beta - \Delta\beta = 3{,}59\,.$$

The annual frequency of exceedance, p, equal to $\Phi[-\alpha\beta]$ for the normal distribution (where β = 3,59 and $\alpha_a = -0.7$), is equal to $\Phi[-0{,}7 \times 3{,}59] = 0{,}006$. The associated 170-year return period is considerably lower than the design period. It follows from equation (22) that the design value for earthquake acceleration in the redesign stage is $a_d = 0{,}16\ g$.

Standard ASCE 41-06 recommends that existing buildings should be evaluated and possibly downgraded to lower hazard levels than new buildings [17]. With that procedure, the reduction in the safety requirements leads to an earthquake with a 20% probability of exceedance in 50 years (related to a return period of 225 years) instead of the present 10% in 50 years. That would bring the redesign stage design value for the case study discussed here to $a_d = 0{,}18\ g$.

The reduction of earthquake safety requirements for existing buildings, illustrated in the above simple example, is based on the following basic premises.

- The cost of retrofitting an existing building to achieve the same performance as a new building may be disproportionately high for the benefit attained.
- The desired residual lifetime of an existing building is usually lower than the service life of a new building.

6 FINAL REMARKS

This chapter illustrates the role of updating information in the assessment of existing structures. Examples are given of updating for reinforcing steel yield strength, load carrying capacity as determined by proof loading, and actions and their effects. The conclusions drawn for the above discussion are as follows:

- Updated statistical parameters for basic variables, obtained as a result of the combination of uncertain prior information and new test results, may be used in structural assessments based on the partial factor method.
- Information on the occurrence of past events in the life of an existing structure can be used to update the occurrence rate of accidental actions.
- Statistical parameters relating to structural resistance variables can be updated on the grounds of data on satisfactory structural performance under extreme loading and can be used to obtain conservative reassessments.
- The application of higher proof load intensities lead to significantly lower probability of failure values, but also to a significantly higher probability of failure during proof loading.
- Assuming the proof load intensity to be a deterministic variable lowers the updated probability of failure but raises the likelihood of structural failure during the proof load test more than the normal or Gumbel probabilistic models.
- Safety requirements for existing buildings can be relaxed based on two premises: the cost of attaining higher reliability level usually amounts to an inordinate proportion of the cost of the structure; and the desired residual lifetime of existing buildings is usually lower than the service life of new buildings.

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CHAPTER 6: CONCRETE STRUCTURES

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1 INTRODUCTION

According to ISO13822 [1], an existing building requires structural reassessment when:

- its reliability is inadequate, also due to misuse or human errors;
- the structure is modified and/or enlarged;
- the category of use of the structure is improved and/or its design working life is increased;
- the structure has been damaged or deteriorated by environmental, chemical or biological, attack or by more general time dependent effects;
- the structure has been damaged by accidental loads, e.g. earthquake or explosion, or by settlements or by other unintentional events like impacts, vibrations, water losses and so on.

According to the flow charts reported in Handbook 1 [2], the investigation process involves the acquisition of all relevant information concerning:

- the original design and structural conception of the building, as well as the reference structural codes, if any;
- the sequence of structural modifications during its life, addition or demolition of structural parts and/or deep maintenance interventions;
- actual material properties;
- actual damage and/or crack patterns;
- required performance level.

In figurative sense, the definition of structural interventions requires anamnesis, diagnosis, prognosis like the definition of a medical treatment for an ill living organism.

Once established that repair or strengthening of the structure is necessary, it must be decided type and extent of interventions. In some case, the intervention can be localized and limited to some structural elements, in other cases it could be more general and even extended to the whole structure.

As rule, the intervention cannot reduce the present structural reliability, while in some circumstance it could be necessary to improve it, attaining a prescribed reliability level, which can reach even the target reliability prescribed for new structures. In principle, reliability and durability of repairs should be the same required for new structures, but, in any case, reliability and durability of the repaired structure should not be affected by repairs.

Clearly, as discussed in chapter 9, in reassessing historical or monumental buildings, weaker structural requirements must be satisfied, as structural needs are tempered by the necessity to grant the preservation of the historical heritage.

In the following two different case studies concerning intervention on r.c. buildings are presented, but preliminarily it is necessary to summarize the main characteristics of the reinforced concrete, the history and the evolution of reinforced concrete structures as well as the most relevant problems which can affect concrete structures.

1.1 Evolution and properties of reinforced concrete structures

As known, the first modern reinforced concrete structures where built after 1850 and it recognised that the first r.c. building was erected by Coignet in 1853 in the surroundings of Paris.

Originally, the reinforced concrete was conceived as a composite material, where the compression strength is mainly granted by concrete, while the steel reinforcement is intended to counterbalance the reduced tensile strength of the concrete assuring at the same time a sufficient ductility. In fact, the first reinforcing schemes were mainly devoted to resist tensile strength, paying less attention to many relevant questions, like the confinement of the compressed concrete, the durability, the long term behaviour and so on, which are common requirements in modern concrete structures [3], [4].

In the first half of the 20^{th} century, a big improvement was achieved introducing the pre-stressing techniques, to increase the load-bearing capacity of the structure. Starting from 1940-1950, thanks to the availability of high strength steels which allowed to counteract the loss of pre-stress linked due to creep and shrinkage, and to the technical evolution pre-stressed concrete structures have so widely used, in particular to extend the field of application of r.c. structures.

Parallel to the evolution of the knowledge about the structural behaviour, relevant improvements have been achieved not only regarding the mechanical properties of the reinforcing steel and the pre-stressing steel, but also regarding the mix design, the preparation and the curing of concrete.

The yield stress of the reinforcing steel was increased from around 200 MPa to 450-500 MPa, while the lateral surface, originally smooth, is now ribbed, so improving considerably the bond -slip performances.

The yield strength of pre-stressing steel varies between 900 MPa, for pre-stressing bars to around 1600 MPa for wires, ropes and strands

The compressive strength of the concrete, that was originally around 10-15 MPa, now can attain currently values around 50 MPa, with maxima of 100-120 MPa.

For long time, it has been a common belief that reinforced concrete was not affected durability problems, but as it was aware corrosion and decay of steel and concrete and the influence of rheological phenomena, more refined constructive details have been introduced to increase quality and durability of the structures.

It is necessary to stress, at this point, that, from the origin till the issue of the first handbooks [4-11] and to the first technical codes, that in Italy for example was issued in 1907, reinforced concrete structures executed, were executed on the basis of patented systems, like the Hennébique system or the Monier system, based on an instinctive perception of the structural behaviour rather than codified theories. That implies that structures built at the end of the 19^{th} century, even if still surviving must be carefully approached, since it is not possible to refer to standards.

On the contrary, structures designed according some kind of standard or some kind of codified theory can be studied simulating in some way the procedure originally followed by the designer, also in order to better address the in-situ investigations, devoted to ascertain the geometry and the static scheme of the structures, the dimensions of the structural elements, the material properties, type and position of the reinforcing bars, extension of decay and corrosion and so on.

In-situ investigations should take into account the most relevant failure or decay modes in r.c. structures like: cracks, carbonation, chloride attack, sulphite attack, alkali-silica reaction, corrosion of reinforcing bars, briefly described in the following.

Cracks

Tensile cracks depend, on size, type, position, distance and concrete cover of rebars, on the stress in the rebars themselves, as well as on the mix design and on the appropriate curing of concrete.

When the crack opening exceed the limit values, the concrete cannot protect the bars so that moisture and aggressive agents can penetrate corroding the reinforcement.

Cracks parallel to compressive principal stresses indicate crushing of concrete.

Carbonation or neutralisation

Carbonation is the chemical reaction between the carbon dioxide which is in the air and the calcium hydroxide, which gives calcium carbonate, decreasing the alkalinity of concrete and so reducing the rebar protection, especially when the concrete cover is too small.

Concrete covers foreseen by structural codes are generally sufficient to grant the rebar protection. Carbonation can be checked treat the cut surface of a fresh drilled hole in concrete with phenolphthalein indicator solution, that turn pink in contact with alkaline concrete.

Chloride attack

When chloride concentration is sufficiently high, corrosion of embedded steel rebars is induced. Corrosion can be localized (pitting corrosion) or generalized and can be avoided limiting the chloride concentration in the water or in the aggregates.

It must be stressed that chlorides are typically presents in sodium chloride, with is usually used for roadway de-icing.

Sulphate attack

The sulphates (SO_4) which can be present in the soil or in the groundwater can attack the Portland cement, causing the formation of expansive products, like ethringite or thaumasite.

Alcali-silica reaction (concrete cancer)

If the aggregates contain amorphous silica, a sufficient quantity of hydroxyl ions (OH-), is available from the cement pore solution and the concrete is characterized by a relative humidity (RH) above 75% relative humidity (RH), the silica (SiO2) dissolves and dissociates in alkaline water, and the dissociated silicic acid reacts with the portlandite originating expansive calcium silicate hydrate, causing tensile stress and cracking. The phenomenon can be controlled using suitable aggregates.

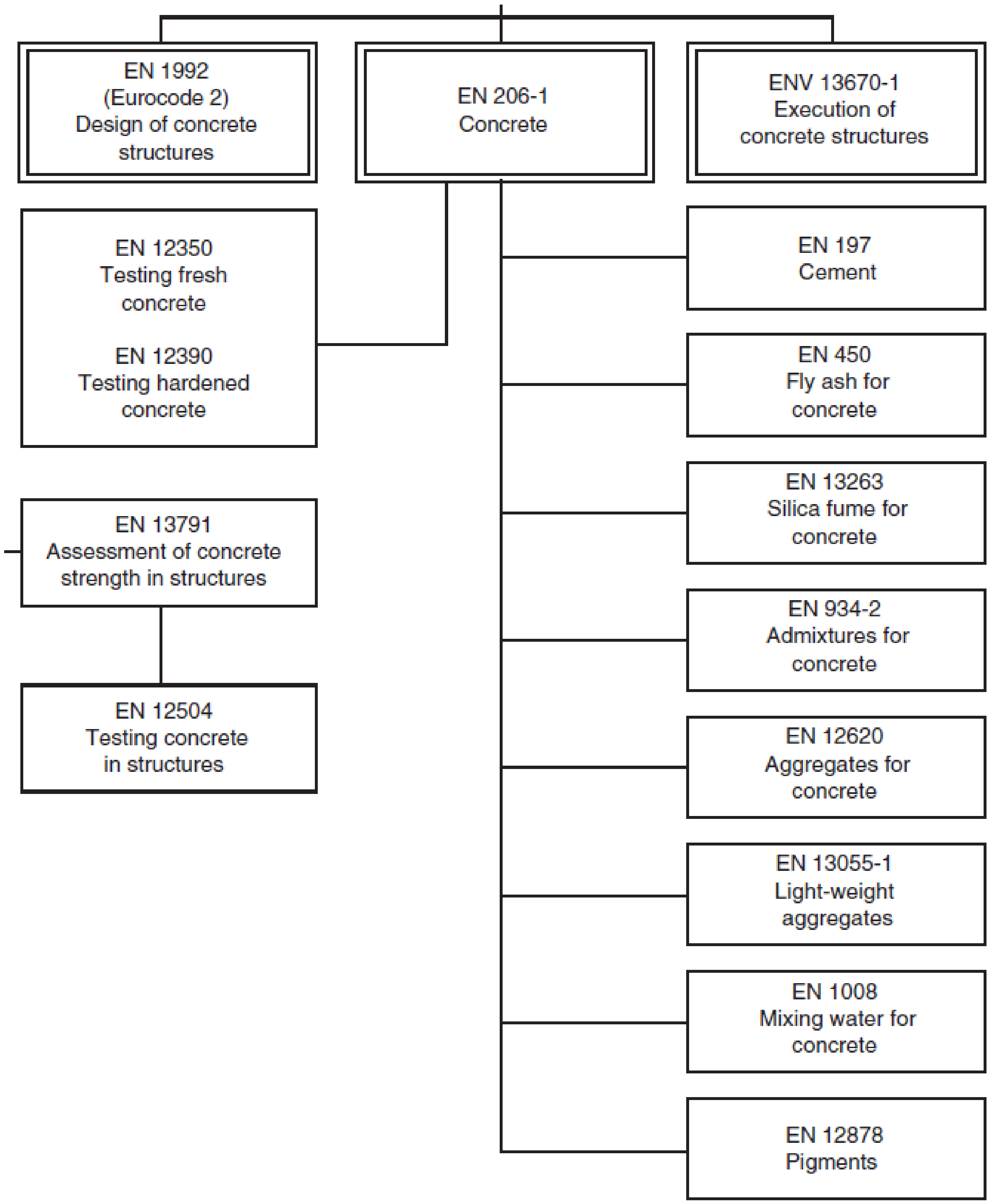
Corrosion of reinforcing bars

The products of the corrosion of carbon steel rebars (iron oxides) increase the volume of the steel reinforcement, which can induces spalling of concete, while the effective area of the steel reinforcement reduces.

If the concrete is properly prepared and cured, the risk of steel corrosion and concrete degradation is reduced; this objective is generally achieved following appropriate standards.

The structure of the EN standards referring to concrete and reinforced concrete is summarized in table 1.

Table 1: Standards for testing and assessment of concrete structures



2 CASE STUDY N. 1 – STRENGHTENING OF R.C. COLUMS

The first case study concerns the localized strengthening of r.c. columns in a residential building.

The building on pilotis, which has been built in the late '70s, is characterized by a reinforced concred framed structure, (figs. 1 and 2) with r.c. and brickwork floors. The columns of the first order stilt are seriously damaged by corrosion of the reinforcing bars, which has determined significant reduction of the rebar area (fig. 3) as well as cracking of the concrete cover (fig. 4).

Figure 1: General view of the building

Figure 2: Damaged columns

Figure 3: Corroded rebars

Figure 4: Cracking of concrete cover

The aim of intervention should be to restore at least the original strength of the columns, avoiding, at the same time, significant alterations of the static and dynamic behavior of the structure, especially under horizontal loads, like wind or earthquake.

According to these objectives, it was decided to strengthen all the columns of the stilt, independently on the specific level of deterioration, in such a way that the relative stiffness of each column is preserved, as indicated in figure 5.

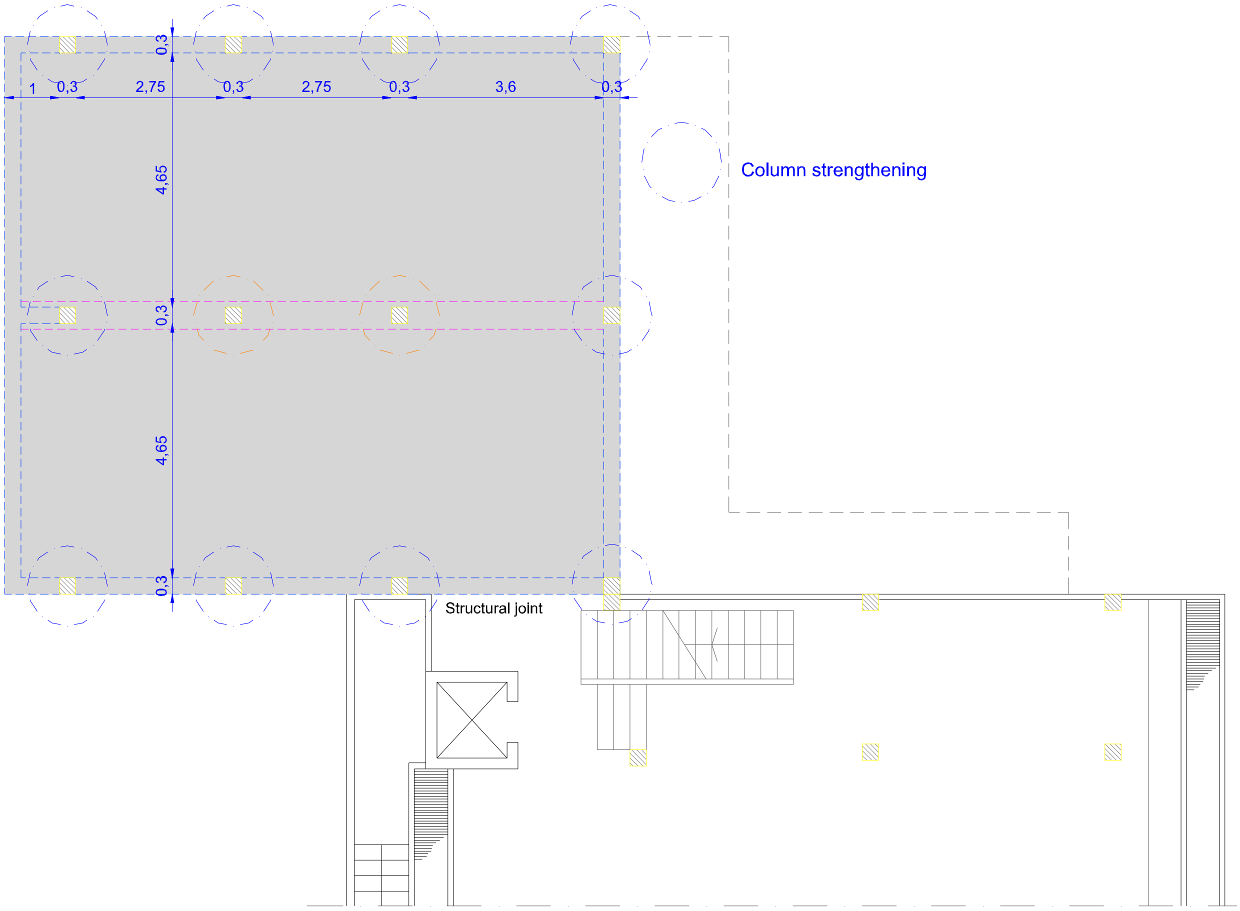


Figure 5: Building plan with indication of columns to be strengthened

Usually, two different techniques are used in the rehabilitation of damaged r.c. columns, according as FRP or steel is used as reinforcing material. In these cases two aspects must be considered in the choice of repair technique:

- the needs of a good confinement od the concrete, which can be assured by both techniques,
- and the efficiency of the reinforcement in transferring the stresses from the original column core, which carries the permanent loads, and the new parts, which can be obtained in a much more easy and reliable way using steel. In fact, while the additional steel reinforcement can be mechanically bonded to the existing concrete column using shear connectors, the FRP longitudinal reinforcement should relies on the surface grip between the adhesive agent and the concrete, which can fail due to peeling or delamination.

The scheme of the column strengthening intervention is summarized in figures 6 and 7.

The strengthening is performed according to the following procedure, which is illustrated through the sequence reported in figures 8-1 and 8-2

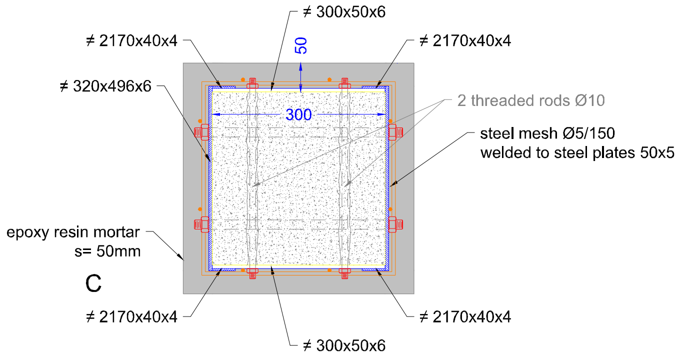


Figure 6: Cross section of the strengthened column

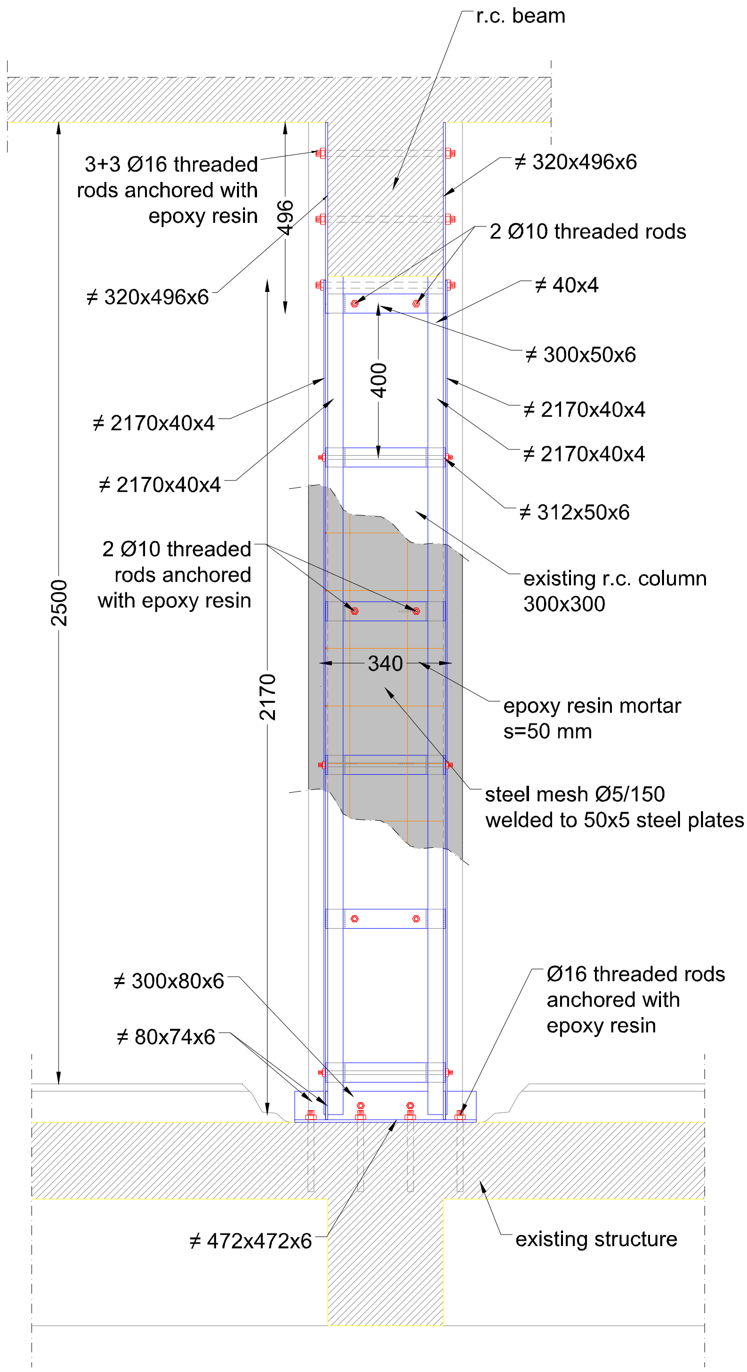


Figure 7: Elevation of the strengthened column

- concrete cover removal (fig. 8.a);
- rust removal and passivation of the reinforcing bars (fig. 8.b);
- execution of holes in concrete necessary to allows the passage of connecting devices (threaded rods); (fig. 8.c)
- positioning of transverse steel plates and of connecting devices using epoxy resin to anchor the rods and epoxy mortar to regularize the surface (fig. 8.d);
- positioning of the longitudinal reinforcement and of the end joints, devoted to connect the reinforcement to the foundation and to beams (figs. 8.e, 8.f, 8.h and 8.h);
- welding of steel mesh to the steel plates (fig. 8.i);
- execution of the additional epoxy mortar layer (s=50 mm) (figs. 8.j and 8.k);
- surface finish (fig. 8.l).

Fig. 8.a

Fig. 8.b

Fig. 8.c

Fig. 8.d

Fig. 8.e

Fig. 8.f

Figure 8-1: Phases of the column strengthening

Fig. 8.g

Fig. 8.h

Fig. 8.i

Fig. 8.j

Fig. 8.k

Fig. 8.l

Figure 8-2: Phases of the column strengthening

3 CASE STUDY N. 2 – EXECUTION OF ADDITIONAL STOREYS

The second case study refers to the expansion and the execution of additional storeys in a residential building used as hotel. The building, represented in figure 9 was originally designed around 1960 according to an old structural code, where seismic actions were not taken into account.

The expansion and the execution of the additional storeys impose a complete reassessment of the structure according to the new standards, duly considering seismic actions, in order to reach the same reliability of new buildings. The new structural scheme and some information regarding structural interventions are summarized in figures 10, 11 and 12, referring to the longitudinal cross section of the building and to the plan of the foundations and of the first floor.

Figure 9: View of the existing building

The first phase of the reassessment was the achievement of the necessary knowledge about the existing structure: original design, static scheme, effective geometry, material properties and so on. Some of the preliminary in situ investigation phases is illustrated in figure 13.

In designing the intervention it was considered that the existing structure cannot be made stronger and ductile enough to withstand the horizontal seismic actions, so that it was decided to entrust to the existing r.c. frames, duly reinforced, only the vertical loads, inserting an appropriate set of shear walls, suitably connected to the existing structure, devoted to sustain horizontal loads due to wind and earthquake. From the seismic resistance point of view, as the shear wall system is much more stiff of the spatial frame existing (about 90% of the total horizontal stiffness of the building is due to the shear walls), columns and existing beams act as secondary elements, which are not stressed by horizontal loads and for which no special performances are required in terms of ductility and/or energy dissipation, so avoiding that they govern the collapse of the building.

For the same reason, the connections between the columns of the additional storeys and the existing structure are carried out through some kind of hinged connection (see figure 14) in order to leave practically unchanged the bending moments due to vertical loads in the existing structure.

As just said the structure has been reassessed considering it as a new structure, considering as not mandatory the fulfilment of specific new detailing rules for the existing structural elements. The prior investigation phase reassessment has been preceded

Obviously, actions on foundations are considerably increased, especially under the shear walls, so that the existing foundation has been enlarged and additional micropiles have been executed(see figs. 15 and 16). A system of shear connectors inserted in holes drilled in the existing foundation ensures the transfer of stresses between it and the new added parts of the foundation.

The strengthening of the existing beams is performed by means of steel stirrups mechanically connected to their webs, which allow, if necessary, also the insertion of additional longitudinal steel rebars. The reinforcement is completed by welded meshes and by additional epoxy mortar layer, like in case of columns (see fig. 17).

Some other significant construction phase is represented in figures 18 and 19.

Finally, in figure 20 it is represented the new building at the end of the works.

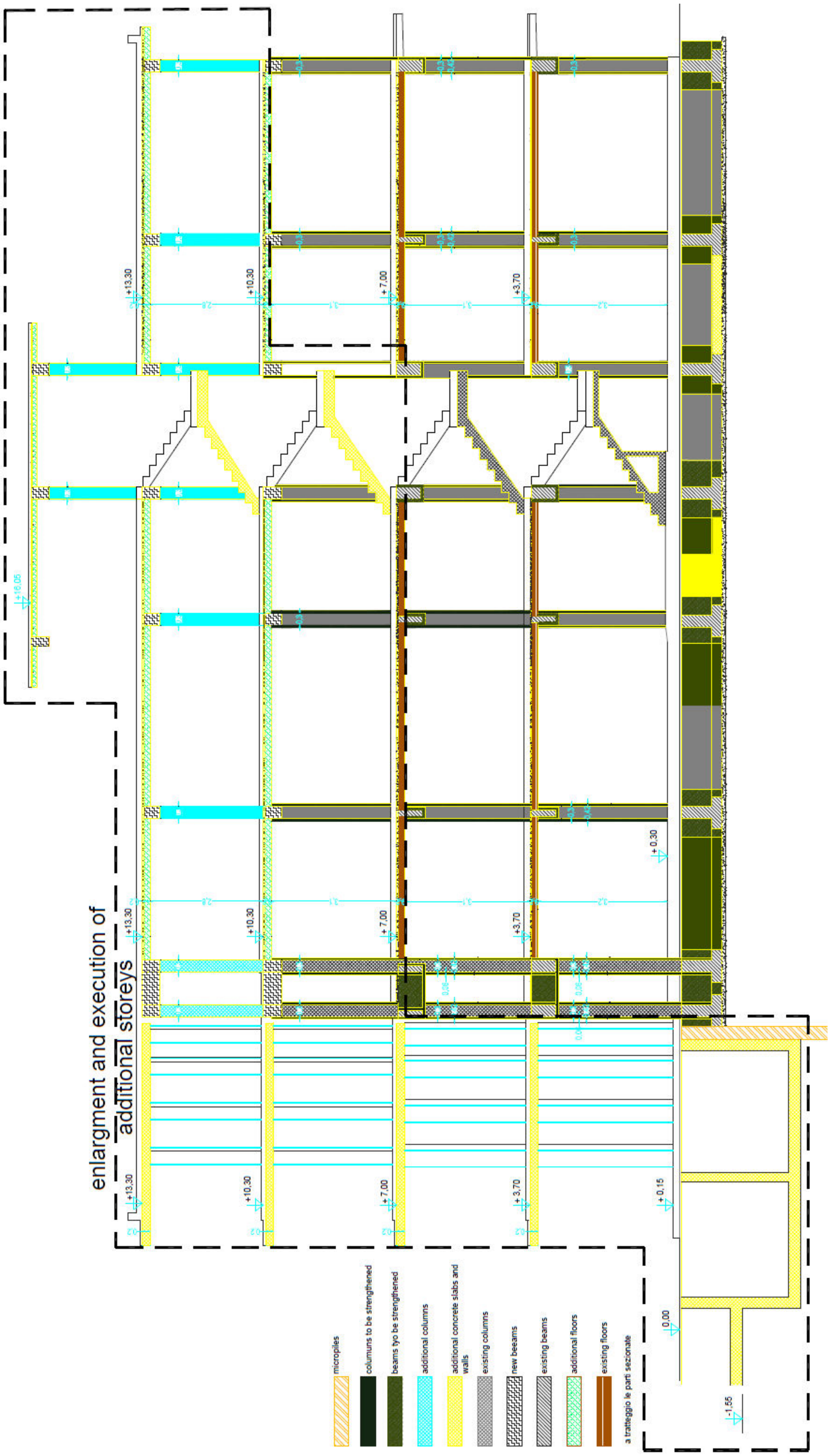


Figure 10: Summary of the additions and of interventions needed to reassess the building (longitudinal cross section)

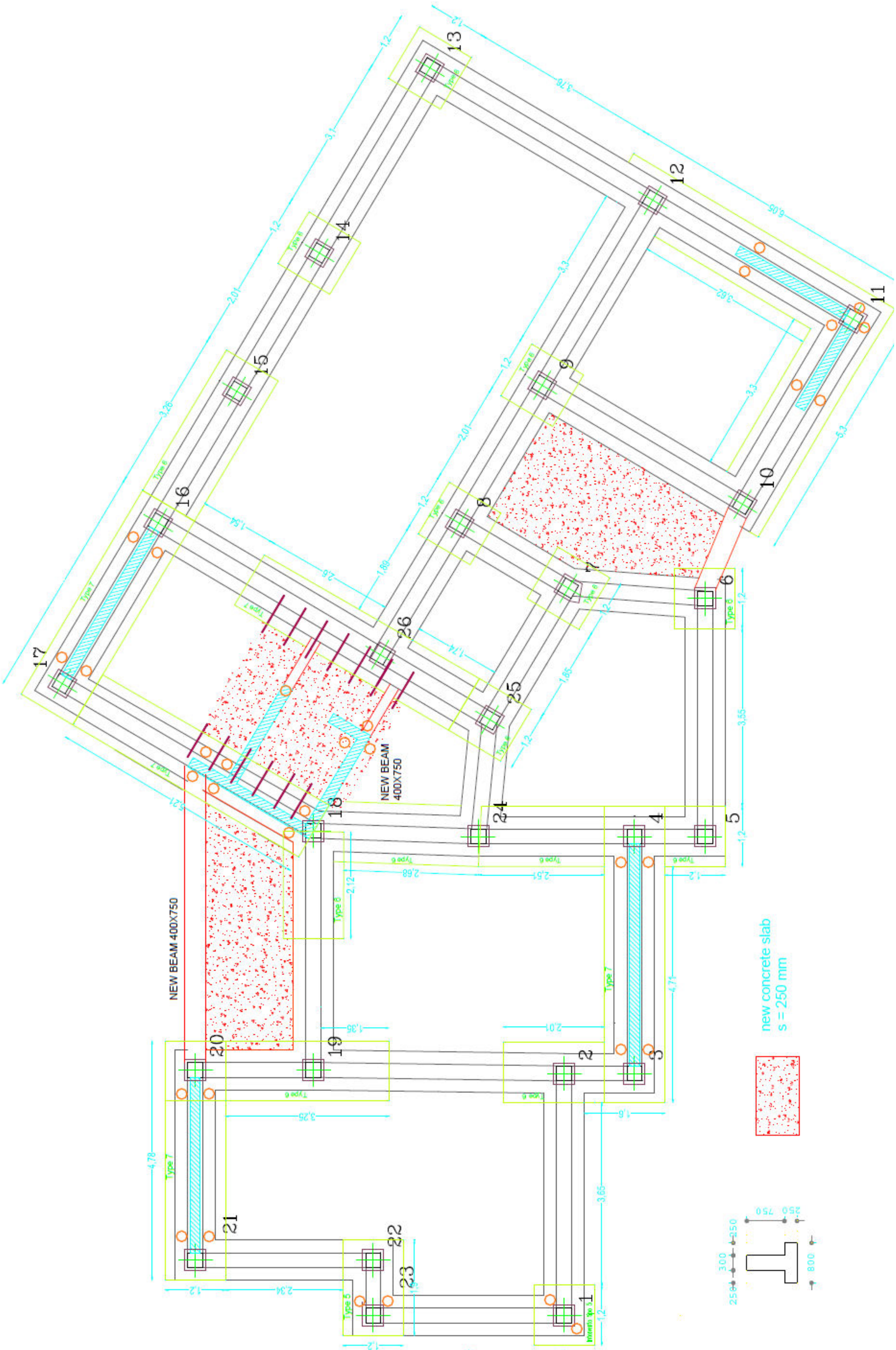


Figure 11: Summary of the additions and of interventions needed to reassess the building (foundations)

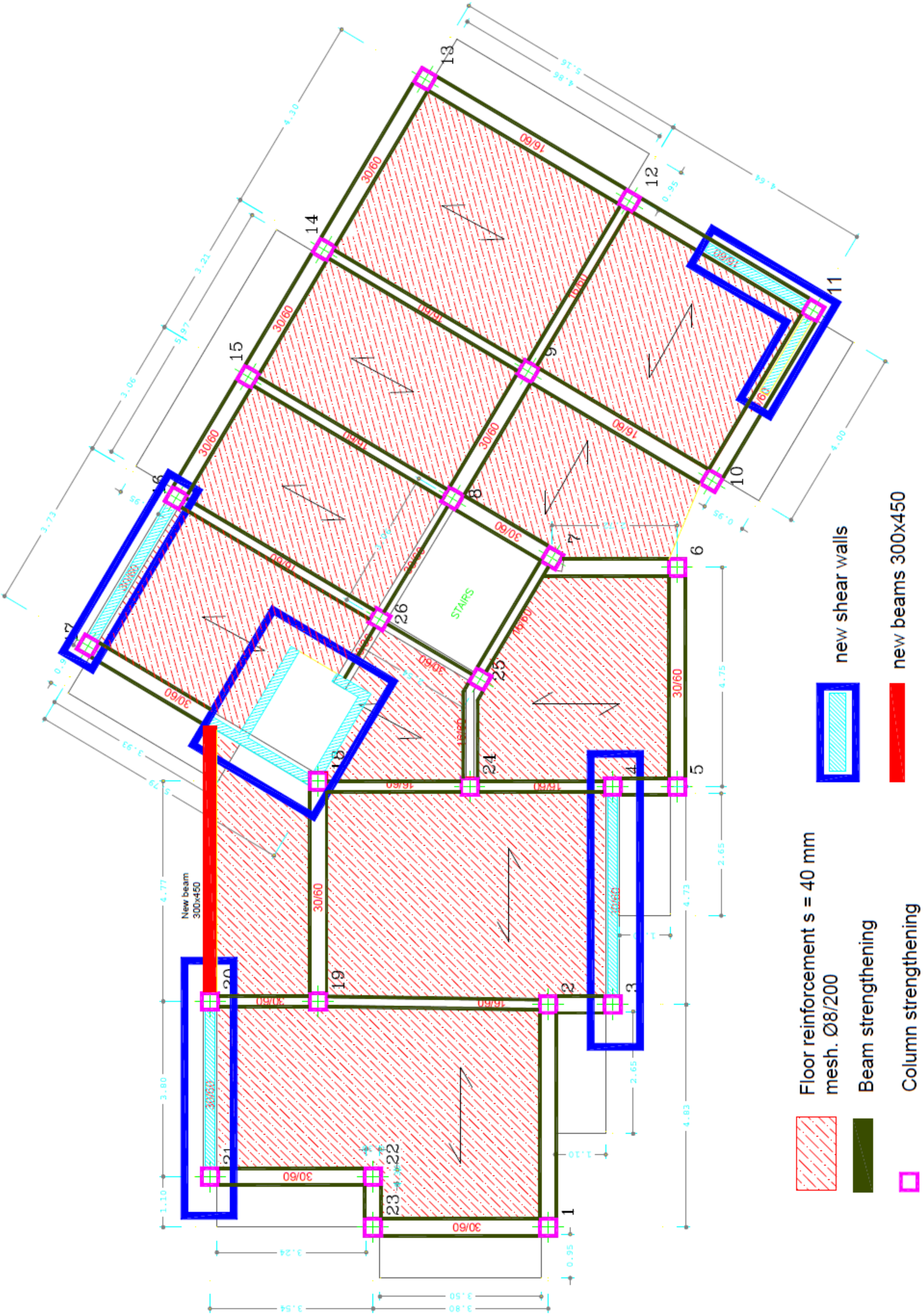


Figure 12: Summary of the additions and of interventions needed to reassess the building (first floor)

Figure 13: Preliminary investigation phase

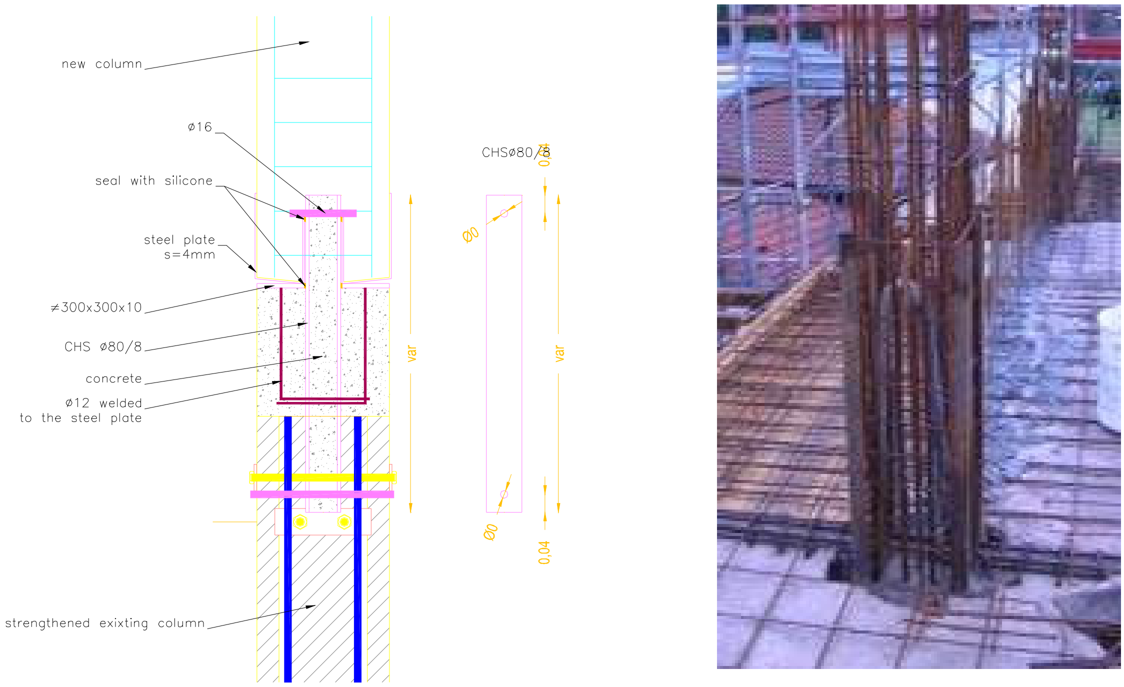


Figure 14: "Hinged" connection between the new and the existing column

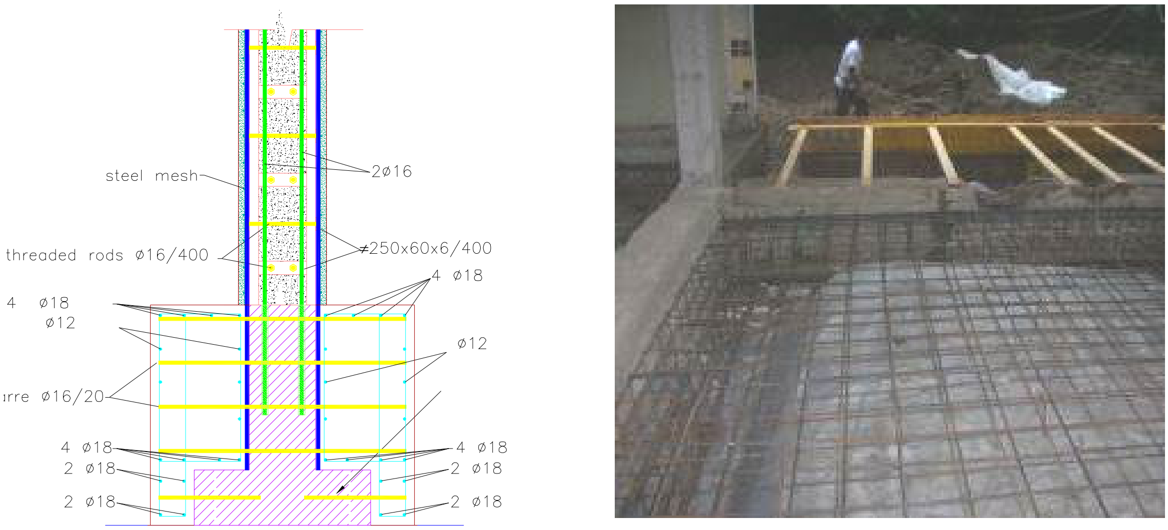


Figure 15: Foundation enlargement

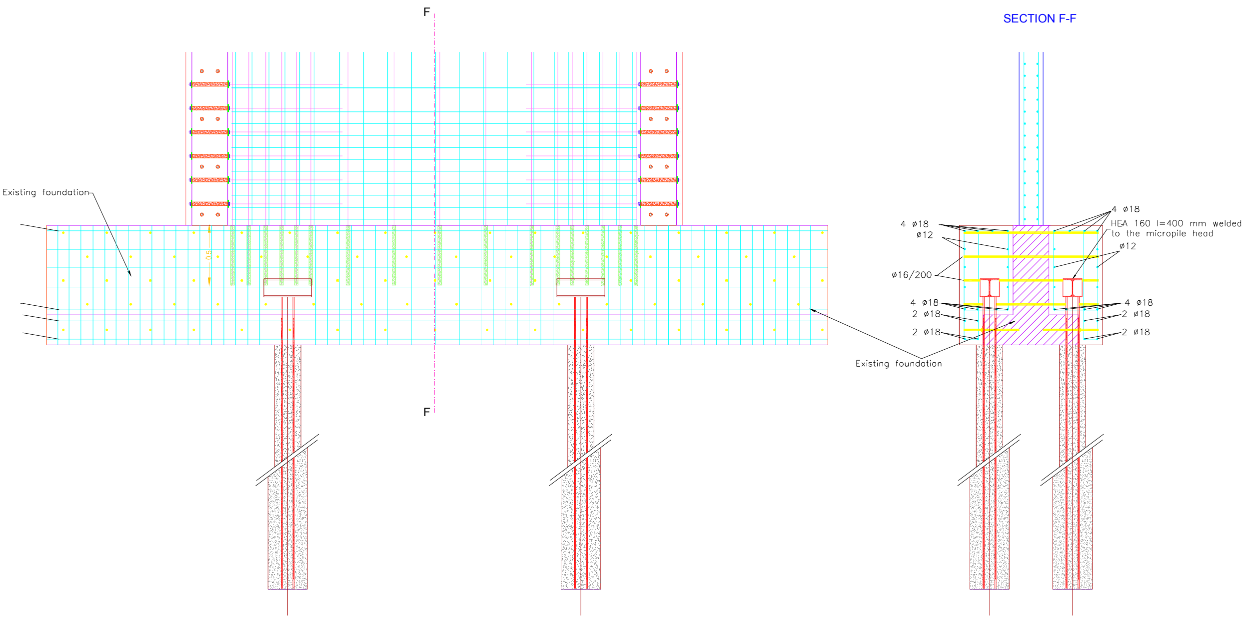


Figure 16: Micropiles and shear wall

Figure 17: Beam reinforcement

Figure 18: Some picture during the execution

Figure 19: Some picture during the execution

Figure 20: The building at the end of the works

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CHAPTER 7: METAL STRUCTURES

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1 INTRODUCTION

The level of accuracy for the load and resistance models, which are needed for the assessment of an existing structure, can be increased for example by visual inspection, material testing or field testing. It is always possible to improve these models by collecting more data about the assessed structure. However, the updating of information by collecting site data may result expensive, time consuming or even ineffective if the choice of the test programme is not made to suit the characteristics of the structure under investigation and if the updated information can not easily be introduced in the calculation models used for the assessment. Tests should therefore carefully be planned, executed and evaluated.

This chapter deals with the evaluation of the structural safety of a 115 year old wrought iron truss-girder bridge. The relationship between planning, execution and evaluation of tests is emphasized.

2 DESCRIPTION OF THE BRIDGE

The bridge investigated crosses the Duero river in Zamora, Spain, and was built around 1895. It is a continuous riveted wrought iron truss-girder bridge over five spans (43,2; 54; 54; 54; 43,2 m) with a total length of 248,4 m. The two main girders beams consist of parallel horizontal U-section members and crossed diagonals (Figure 1). The platform is composed of a wrought iron framework which supports the deck, consisting of a wrought iron sheeting, a sand fill and an asphalt layer. At the moment of the evaluation, the main girder bottom U-section members were affected by severe corrosion due to poor detailing and reduced maintenance in the past. For this reason, the bridge evaluation was initiated.

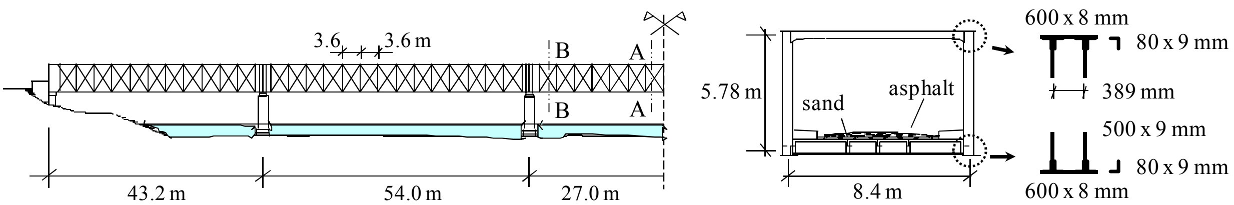

Figure 1: View and cross-section of the investigated truss bridge.

3 EVALUATION PROCEDURE

The assessment of the structural safety is carried out applying a staged procedure. Figure 2 shows the concept of the staged evaluation procedure and its relation to the collection of site data by inspection, material- and field testing.

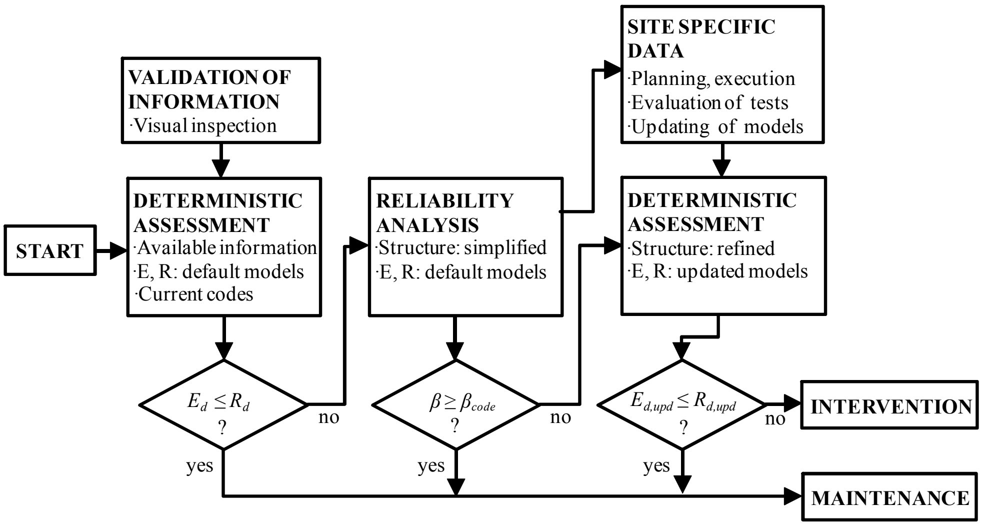

Figure 2: Staged evaluation procedure and its relation to the collection of site data.

In a first step, a preliminary deterministic assessment is carried out, using the verification criteria defined in the current Spanish bridge design code at the moment of evaluation [1]. For this, the calculation models are based on the available information about the structure, validated by visual inspection. No further evaluation is necessary for the members for which structural safety is verified in this first step.

For the most critical area, identified in the first step, a simplified structural model can be established that permits a reliability analysis using default probabilistic models of action effects and resistance. If the structural safety of this area is not verified, further evaluation is possible based on improved load and resistance models. The improvement of these models is possible through the collection of site data. The aforementioned reliability analysis aids the planning of site data collection: from the results it can be deduced which parameters can be most effectively updated.

The site data can be used to calibrate updated deterministic models of action effects and resistance. For the calibration, reliability methods are applied to the simplified structural model mentioned above. The updated deterministic models of action effects and resistance are then used for a detailed deterministic assessment using a more refined structural model.

For the structural members for which safety is not verified by deterministic assessment with updated models, a reliability analysis could be used for a more accurate assessment of structural safety. However, due to the large number of different structural elements, nodes and riveted connections, a full reliability analysis is not considered viable for the investigated bridge. An intervention must be planned for the members for which safety is not verified by any of the aforementioned assessment methods.

4 COLLECTION OF SITE SPECIFIC DATA

4.1 Critical areas

4.1.1 Validation of information

The available information about the structure is validated by a first visual inspection before carrying out the preliminary deterministic assessment. The most important findings can be summarised as follows:

- Important eccentricities exist at main girder nodes, not visible from the geometry of the original plans (fig. 3).
- Advanced global corrosion of the truss girder bottom U-section member can be observed, facilitated by its channel like geometry. A large number of holes with dimensions of the order of 200 - 400 mm exist (fig. 3).
- Buckling is found of the slender "web plates" (with a height to thickness ratio of 55 and a free edge as can be seen from figure 1) of top and bottom U-section compression members.
- Fatigue cracks in truss top lateral sway frames are observed, spreading out from rivet holes (this finding is important with a view to the evaluation of fatigue safety and the planning of maintenance and inspection strategies [2, 3]; however, fatigue and brittle fracture are not further considered in the present paper).
- The foundations are in a very good state.

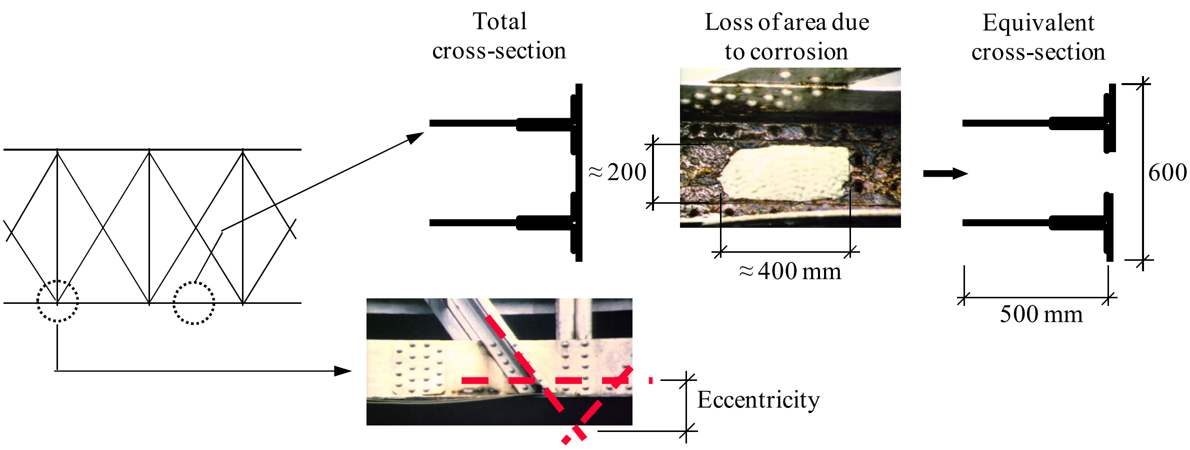

Figure 3: Validation of information.

4.1.2 Preliminary deterministic assessment

Structural safety is evaluated by applying the verification criteria defined in the relevant design standards. The action effect, *E*, is calculated by using actions and load factors according to [4] and by introducing in the structural model the aforementioned eccentricities at main girder nodes. The corrosion of the truss girder bottom U-section members is taken into account by adopting an equivalent cross-section according to figure 3 for the calculation of the resistance, *R*. Information about the material properties of wrought iron is available from literature [2], and resistance factors are adopted from [1]. The structural safety can be expressed by a rating factor, *r*:

$$r = \frac{R/\gamma_R}{E_d} \tag{1}$$

R resistance;
E_d design load effect;
γ_R partial factor for resistance (=1,1).

If r is greater than or equal to 1,0, the investigated member or connection reaches the required structural safety level according to the Spanish codes. If the rating factor is less than

1,0, then structural safety is not verified and there is a need to perform a more accurate evaluation. The preliminary deterministic assessment reveals that the governing elements regarding load carrying capacity of the bridge are the top and bottom U-section compression members at midspan and over the piers, respectively (sections A-A and B-B, respectively, in figure 1). Quite a number of these elements do not reach the required safety level. The minimum value for the rating factor *r*, equal to 0,57, is obtained for the main girder top U-section member at midspan (fig. 1, section A-A).

4.2 Importance of different variables for safety

4.2.1 Simplified structural model

Once the compression members at midspan and over the piers are identified as the critical areas, it can be assumed that the structural behaviour is brittle and that there is no significant system redundancy. Therefore, the failure of the most critical member leads to the failure of the system. Consequently, the failure probability for the bridge is governed by the failure probability of the most critical member [5].

Due to the aforementioned eccentricities at main girder nodes, the most critical member is subject to combined bending and axial compression. Although the "web plates" of the U-section are slender (fig. 1), the governing combination of bending moment, *M*, and axial compression, *N*, which defines the Ultimate Limit State of the critical member, leads to a loss of stiffness due to plate buckling of the order of only 18,5%. Therefore, the ends of the member are not free to rotate in the plane of buckling (plane of the girder, fig. 4). According to [1], the buckling length, l_p, of a truss girder top compression member corresponds to the length of a "pin ended" member which has the same buckling resistance. In the present case it can be assumed that $l_p = 0{,}9\ l$. The reliability analysis can now be carried out for the simplified structural model, consisting of a "pin ended" member with a length of 0,9 *l* which is subject to combined bending and axial compression according to figure 4.

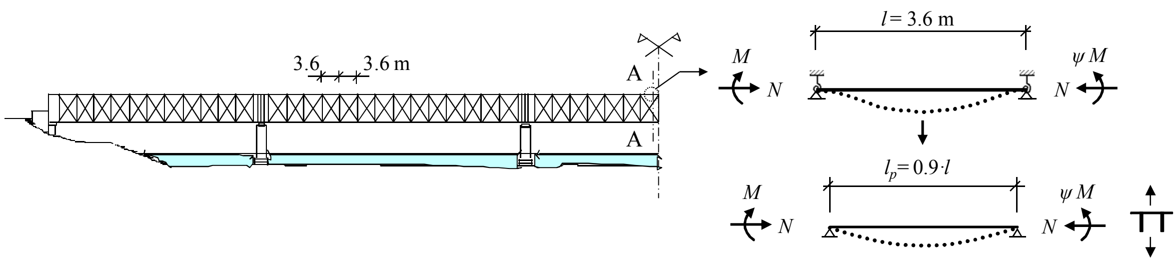

Figure 4: Simplified structural model for reliability analysis.

4.2.2 Reliability analysis

Basic variables which are considered for the assessment of structural safety are associated with uncertainty. The safety of a structure can therefore be measured in terms of, for example, its reliability which takes account of uncertainty and is represented by a probability of failure.

The safety of a structure is expressed in terms of the basic variables by the Limit State Function (LSF). The simplest LSF defines safety as the requirement that resistance, *R*, is greater than or equal to the total action effect, *E*:

$$R - E \geq 0 \tag{2}$$

The probability of failure, p_f, is thus equal to the probability that *E* is greater than *R*.

Different numerical or analytical reliability methods exist for the analysis of structural safety. First Order Second Moment (FOSM) method [6] introduces for example a reliability index, β, for which a direct link to the failure probability exists. Even though the FOSM reliability method only produces an estimate of failure probability, the resulting errors are small if it is used to compare the failure probabilities for a given LSF and varying basic variables. This is what the FOSM method is used for in the present study: going out from the axiom that a correct application of the current codes results in a safe structure, the verification of structural safety of an existing structure consists of three steps [7]:

- Dimensioning of the existing structure according to a consistent set of codes,
- Calculation of the reliability index, β_{code}, related to the dimensions obtained in the first step, considering the parameters (mean value, standard deviation, probability distribution) of the variables assumed to lie behind the rules of codes,
- Calculation of the reliability index, β, related to the actual structure using default probabilistic models of action effects and resistance.

The structure may be considered safe if

$$\beta \geq \beta_{code} \,. \tag{3}$$

In the case of the investigated truss bridge, in the first step the main girder top U-section member at midspan (Figure 4) is to be dimensioned according to the current codes [1, 4]. The analysis reveals that a rolled profile HEB 300 is required with a specified nominal yield strength of f_y=235 N/mm^2. Such a main girder top member at midspan may be considered safe according to the aforementioned axiom.

In the second step the reliability index, β_{code}, of the above safe member is to be calculated. The LSF which is used in this reliability analysis is derived from the Spanish code [1] for the verification of structural safety of members subject to combined bending and axial compression:

$$f_y - \frac{\left(N_a + N_s + N_p + N_q\right)}{\chi A_{eff}} - \frac{k\left(M_a + M_s + M_p + M_q\right) + e\left(N_a + N_s + N_p + N_q\right)}{W_{eff}} = 0 \,, \tag{4}$$

f_y	elastic limit of structural steel (or wrought iron);
N_a	axial compression due to the self weight of the steel;
N_s	axial compression due to the sand fill;
N_p	axial compression due to the asphalt layer;
N_q	axial compression due to the traffic actions
M_a, M_s, M_p, M_q	moments due to the different aforementioned actions;
A_{eff}	effective area of the cross-section when subject to uniform compression;
W_{eff}	effective section modulus of the cross-section when subject only to moment about the relevant axis;
χ	reduction factor for the relevant buckling mode;
e	shift of the relevant centroidal axis when the cross-section is subject to uniform compression
k	factor which takes into account the distribution of the moments and the characteristics of the cross-section.

The parameters of the variables involved in the LSF that are assumed to lie behind the rules of the codes are taken from the literature [5]. This LSF and the parameters of the variables (mean value, standard deviation, probability distribution) may now be introduced in a computer program [8] which handles the variables in accordance with the method from [6] and calculates the FOSM reliability index β_{code}. In the present case we obtain β_{code}=4,06.

The third step of the verification consists of the calculation of the reliability index, β, of the actual member. *A priori* values for the parameters of the variables (Table 1), are either taken directly or interpreted from [2, 5, 9, 10] and introduced in the LSF (4). The FOSM reliability index is calculated to be β=1,12.

Obviously, according to the inequality (3), the member under consideration is not safe. Site data should therefore be collected in order to improve the load and resistance models for the continuation of the evaluation (Figure 2).

Table 1: Assumed values of the parameters of the variables for the estimation of β and results of FOSM analysis

Variable	Type	Bias	CoV	Nominal value	Mean	Standard deviation	Influence coefficient	Design value
		μ_X/X_{nom}	μ_X/σ_X	X_{nom}	μ_X	σ_X	α_X^*	X^*
f_y	LN	1,195	0,115	220 N/mm^2	263	30,3	0,826	234,8
N_a	N	1,01	0,03	234 kN	236,3	7,1	-0,025	236,5
N_s	N	1,20	0,25	273 kN	327,6	81,9	-0,29	354,3
N_p	N	1,20	0,25	82 kN	98,4	24,6	-0,087	100,8
N_q	Gumbel	0,88	0,125	1070 kN	941,6	117,7	-0,45	980,9
M_a	N	1,01	0,03	5,9 kN·m	6,0	0,18	-0,004	6,0
M_s	N	1,20	0,25	9,5 kN·m	11,4	2,85	-0,061	11,6
M_p	N	1,20	0,25	2,8 kN·m	3,4	0,85	-0,018	3,42
M_q	Gumbel	0,88	0,125	43 kN·m	37,8	4,7	-0,094	37,48
A_{eff}	N	1,02	0,01	14061,6 mm^2	14342	143,4	0,034	14340
W_{eff}	N	1,02	0,01	$1{,}68\cdot10^6$ mm^3	$1{,}71\cdot10^6$	$1{,}71\cdot10^4$	0,038	$1{,}71\cdot10^6$
χ	N	1,05	0,024	1,0	1,05	0,025	0,081	1,048
e	N	1,02	0,01	82,5 mm	84,2	0,84	-0,025	84,22
k	N	1,04	0,02	1,15	1,20	0,024	-0,025	1,2

4.2.3 Conclusion

In addition to the reliability index, β, the method according to [6] provides the design values, X^*, and the importance factors, α_X^*, corresponding to the variables involved in the LSF (table 1). The design values X^*, correspond to the most probable set of values of the variables at failure. The importance factor is a function of the relative importance of a given basic variable within a given LSF. The greater the absolute value of α_X^* (the importance factor is negative for variables which have an unfavourable effect on safety), the bigger the influence of the variation of the corresponding variable on the reliability index. In the above example the yield strength of wrought iron, f_y, and axial compression due to traffic actions, N_q, are most critical. For these variables, updating efforts would be most effective.

4.3 Collection of site data - Planning and execution

4.3.1 Overview

The definition of a test program includes the choice of the parameters which should be updated, the definition of the method of observation and recording, the selection of test specimens, test conditions and arrangements, the number of tests and the method of evaluation.

The execution of tests should be in accordance with the planning, and the measurement techniques in accordance with the required tolerances. For the evaluation of the test results, methods should be used which enable an easy introduction of the updated information in the calculation models. In the present case, according to §4.2.3 updating is carried out for the wrought iron yield strength and the traffic actions. For two reasons it is also decided to carry out measurements of the actual dimensions of wrought iron member cross-sections: the influence of corrosion is to be assessed and the assumed dimensional variation in the reliability analysis (§4.2.2) corresponds to modern welded steel elements, for which fabrication tolerances are very small, and not to wrought iron members.

In the following, some information about the planning and execution of site data collection is given. Section 4.4 contains some thoughts on test evaluation, and the obtained site specific data is summarised in table 2.

4.3.2 Material properties

Material properties are determined from miniaturised specimens, which can be drilled from structural members without reducing their resistance [11]. In the present case for example, the dimensions of the cylindrical specimens for tensile tests are: 40 mm of total length and 3 mm of diameter. Chosen test temperatures are room temperature (20°C) and −20°C corresponding to the lowest service temperature expected to occur within the intended remaining life of the structure.

Test samples should be representative and a sufficient number should be taken in order to determine variability with adequate certainty. In normal daily practice, however, only a limited number of tests can be carried out for economical reasons. In the present case for example, the number of tensile tests is eight. In section 4.4, the influence of the number of tests on the characteristic value of the wrought iron yield strength is discussed.

4.3.3 Cross-section area

The influence of the severe corrosion of the truss girder bottom U-section members is directly taken into account in the corresponding resistance model by introducing an equivalent cross-section (fig. 3). The influence of the dimensional variation due to corrosion and fabrication tolerances on the structural resistance of the other members is to be assessed. This is done by extensive measurement of the actual dimensions of wrought iron cross-sections.

4.3.4 Traffic actions

For economical reasons, neither vehicle surveys nor measurements of the effects of vehicle actions on the bridge with a view to obtaining data describing traffic actions are possible in the present case. Only traffic counting can be carried out: a daily traffic volume of 10059 vehicles, of which 12,5% are Heavy Goods Vehicles, is physically measured. This means that an average of 1257 HGV per day cross this urban bridge. Furthermore, frequent traffic jams are observed due to the traffic lights situated at both ends of the bridge. It is also known that the percentage of overloaded HGV in Spain is around 25% [12]. The effects of traffic actions on road bridges is described by a certain frequency distribution which determines the extreme action effects to be considered during the assessment of structural safety [5]. These effects may be obtained based on numerical simulations by generating random traffic actions for the considered traffic type [5, 9].

4.4 Evaluation of tests

If only a limited number of tests on material samples are available, as normal in daily practice, the evaluation of test results according to standard statistical methods may lead to unrealistic low characteristic or design values [13]. This drawback can be avoided, if the evaluation of test samples with a limited number of tests is carried out according to statistical models which permit the introduction of prior knowledge. Based on knowledge about the distribution of the investigated variable, a posterior distribution is derived in combination with the obtained test results. Such an approach is applied in the present study. In the case of the wrought iron strength, for example a mean value of the yield strength m_{fy}=225 N/mm^2 and a standard deviation of s_{fy}=17,1 N/mm^2 are obtained from the sample of eight tensile tests. The corresponding characteristic value, which is based on a 5% fractile with a confidence level of 75%, evaluated with standard statistical methods [13], is f_{yk}=187,5 N/mm^2. It is known from previous experience that for the yield strength of wrought iron a lognormal distribution can be expected. Furthermore, the sample standard deviation, s_{fy}, underestimates the standard deviation of the whole population, σ_{fy}, depending on the sample size. Taking into account this prior information, the estimate for the characteristic value of the yield strength is f_{yk}=196,8 N/mm^2.

5 INTRODUCTION OF TEST RESULTS IN THE CALCULATION MODELS

5.1 Overview

As mentioned in section 3, a full reliability analysis is not considered viable for the investigated bridge. A simplified deterministic method should therefore be used. The aim of a deterministic assessment of structural safety is to verify that the inequality (2) is satisfied by using nominal values of basic variables and partial factors in order to obtain the values that they would have at the design point in a reliability analysis [5]. The link between reliability concepts and deterministic methods is the design point which is the most probable failure point on a limit state surface [5]. The relation between the design point, partial factor and nominal value is given by

$$X^* = \gamma_X \cdot X_{nom}, \tag{5}$$

X^* value of the basic variable at the design point;
γ_X partial factor;
X_{nom} nominal value of the basic variable.

The Limit State Function is the same for both methods (reliability and deterministic), only the representation of the variables is different. Partial factors, which are introduced in a deterministic analysis, are therefore attributed individually to the variables in the LSF and vary according to the degree of uncertainty and the importance of the variable within the LSF. The aim of the collection of site specific data is the reduction of the uncertainty associated with the variables. The influence of this change cannot be considered explicitly in a deterministic assessment (only changes in the mean value of a variable can be accounted for). As mentioned in section 3, the site specific data is therefore used to calibrate updated deterministic models of action effects and resistance, by applying reliability methods to the simplified structural model according to 4.2.1.

5.2 Calibration procedure

According to the axiom mentioned in §4.2.2, the calibration procedure consists of the following five steps:

- dimensioning of the existing structure according to a consistent set of codes;
- calculation of the reliability index, β_{code}, for this structure;
- calculation of the reliability index, β_{upd}, for the actual structure using the updated parameters of the variables. β_{upd} may be greater or smaller than β_{code}, depending

mainly on the state of the structure (corrosion) and the aggressivity of the actual traffic;
- find the required actual resistance, $R_{upd,req}$, by multiplying the actual resistance, R_{upd}, by a factor, κ_R, in a way that results $\beta_{upd}=\beta_{code}$ for the actual effect of actions, E_{upd} (fig. 5);
- derive partial factors, in analogy with equation (5), which can be applied to the nominal values of basic variables (E_{nom} for action effects and R_{nom} for resistance) in a deterministic assessment:

$$\gamma_{E,upd} = \frac{E^*_{upd}}{E_{nom}}, \tag{6}$$

$\gamma_{E,upd}$ updated partial factor for action effects;
E^*_{upd} updated action effect at the design point;
E_{nom} nominal value of the action effect;

$$\gamma_{R,upd} = \frac{\kappa_R R_{nom}}{R^*_{upd,req}}, \tag{7}$$

$\gamma_{R,upd}$ updated partial factor for resistance;
$R^*_{upd,req}$ updated required resistance at the design point;
R_{nom} nominal value of the resistance;
κ_R factor for the calculation of the required actual resistance.

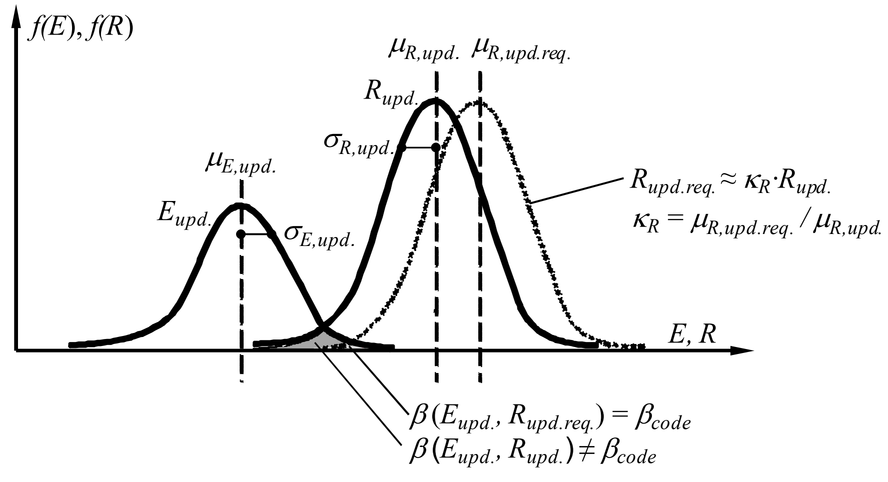

Figure 5: Calibration of updated load and resistance models.

The updated partial factors, which take into account the influence of a change in uncertainty associated with the variables and are attributed individually to the basic variables in a LSF, can now be used in a deterministic assessment (using a more refined structural model) of structural safety, together with the nominal values of action effects and resistance. The requirement for structural safety can therefore be derived from the inequality (2) and is expressed by the following condition:

$$\gamma_{E,upd} \cdot E_{nom} \leq \frac{R_{nom}}{\gamma_{R,upd}}, \tag{8}$$

5.3 Case study

The first two steps of the calibration procedure correspond to the first two steps of the reliability analysis from 4.2.2. Therefore, the reliability index according to the current codes is: β_{code} = 4,06. The collection and evaluation of site data according to 4.3 and 4.4 results in updated parameters of the corresponding variables, listed in Table 2.

Table 2: Updated parameters of the variables

Variable	Type	Bias $\frac{\mu_{X,upd}}{X_{nom}}$	CoV $\frac{\sigma_{X,upd}}{\mu_{X,upd}}$	Mean $\mu_{X,upd}$	Standard deviation $\sigma_{X,upd}$
f_y	LN	1,023	0,079	225 N/mm^2	17,7
N_q	Gumbel	0,80	0,125	856 kN	107
M_q	Gumbel	0,80	0,125	34,4 kN·m	4,3
A_{eff}	N	1,013	0,023	14249,4 mm^2	336,6
W_{eff}	N	1,013	0,023	$1{,}71\cdot10^6$ mm^3	$3{,}9\cdot10^4$

Table 3: Updated partial factors

Action effects				Resistance
Iron $\gamma_{Ga,upd}$	Sand $\gamma_{Gs,upd}$	Asphalt $\gamma_{Gp,upd}$	Traffic $\gamma_{Q,upd}$	$\gamma_{R,upd}$
1,01	1,45	1,3	1,4	1,06

For the other variables of the LSF (4), the parameters from Table 1 are adopted. The calculation of the FOSM reliability index for the actual structure gives β_{upd}=0,493. This value is even lower than the one calculated in §4.2.2 using default probabilistic models of action effects and resistance. This is mainly due to the fact that in the bridge under investigation the elastic limit of the wrought iron is lower than usual values for this type of material. For the aforementioned factor, κ_R, a value of κ_R=1,484 is found. The values of the basic variables of the LSF (4) at the design point, $X^*_{upd.(req)}$, result from the FOSM analysis, carried out for E_{upd} and $R_{upd.req}$. These values are then used to derive updated partial factors according to the equations (6) and (7). The obtained results are listed in Table 3. In a detailed deterministic assessment with updated models of action effects and resistance (according to (8)) it is now possible to determine the structural elements, nodes and riveted connections which need to be strengthened (fig. 2). The proposed solution for the strengthening is presented in [12].

6 CONCLUSIONS

A proper assessment of an existing bridge based on incomplete or defective information may be completely wrong. Therefore, correct updating of data is probably the most important step in a bridge evaluation. For the choice of the test and inspection programme some guidelines should be observed:

- the expected structural behaviour, loading and environmental conditions should be investigated by a qualitative analysis;
- based on the results of the preliminary analysis, the objectives of the tests can be formulated and correct choices for the test programme are possible;
- the tests should be undertaken following the established plan;
- the evaluation of test samples with a limited number of tests should be carried out taking into account prior knowledge in order to avoid unrealistic low design values.

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CHAPTER 8: TIMBER STRUCTURES

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1 INTRODUCTION

Since the antiquity timber structures are currently used around the world.

As known, the physical and mechanical properties depends not only on the species, but also on grade and condition of the wood: these three important factors must be duly considered when an existing structure is examined. In addition, it is necessary to consider the intrinsic anisotropic nature of the wood, which results in properties along longitudinal, tangential, and radial directions which differ significantly, while wood properties can vary greatly also within a single species.

Approaching an existing timber structure, the methodology to be used is the nearly the same used approaching a new one, provided that the wood conditions and the wood degradation are duly taken into account. For this reason, it is possible to refer, in terms of testing methods or definition of relevant properties to standards for new buildings [1], [2], [3], where the procedure for the derivation of the main physical and mechanical properties are given.

1.1 Physical and mechanical properties of the timber

Physical and mechanical characteristics of wood are controlled by specific anatomy and to a lesser extent, mineral and extractive content.

For structural purpose, the most relevant physical properties are: humidity (moisture content), density, permeability, shrinkage, thermal conductivity, coefficient of thermal expansion.

Moisture contents

Moisture content, expressed in mass percentage, is one of the most relevant properties, because it influences not only the other physical and mechanical properties, but also the durability and in service performances. For dried processed wood the moisture content ranges 9-14%.

Until the moisture content is above the fibre saturation point, the wood is dimensionally stable, otherwise shrinkage occurs, mainly in transversal or radial direction.

Thermal conductivity and coefficient of thermal expansion

Transverse thermal conductivity is important to wood processing: in fact all treatments because drying, curing and conditioning are performed in one step, by heating.

The coefficient of thermal expansion in longitudinal direction is about $5 \cdot 10^{-6}$ while in the transverse direction ranges $30 \cdot 10^{-6}$- $70 \cdot 10^{-6}$.

Mechanical properties of wood, in particular strength and elastic modulus, depend not only upon species, but also on loading direction, fibre orientation, moisture content, size and location of knots and other natural defects, loading rate.

Generally, the mechanical properties in longitudinal direction are significantly greater than in transverse direction.

1.2 Timber degradation

Timber degradation is a continuous process induced both by abiotic and biotic attacks [4].

Abiotic attack

Abiotic attack can be induced by heat, strong acids and strong basis, organic solutions, salt solutions, sunlight and mechanical wear.

The heat darkens the wood and reduces its strength: when the temperature raises about 220 °C combustion ensues.

Strong acid reduce the strength; strong bases degrade the lignin, whitening the wood, while organic or salt solutions act on the cellulosic matrix, affectingthe mechanical properties.

Exposure to the ultraviolet light attacks the wood polymers, which release free radicals.

Biotic attack

Biotic attack is due to a variety of biological agents, like fungi, insects, borers, birds, animals and bacteria and it occurs provided that favourable microclimatic conditions are in place, that are: temperature ranging 0-40°C, presence of water, oxygen and food sources.

Fungi, which are one of amongst the most important wood-degrading organisms, generally belong to ascomycetes and deuteromycetes: molds, stainers, soft rotters, brown rotters, and white rotters can are wood-degrading fungi, characterized by the attack patterns.

Several insects attack wood, including termites, which are the most relevant, beetles, bees and ants.

Borers, and especially marine borers can cause significant damages.

Finally, bacteria are capable to produce minor damages, connected in some cases with cell wall decay.

1.3 Wood protection

Wood protection can be performed in several ways, depending on the attack to be faced.

One of the most effective method is the adoption of durable and/or decay- or insect-resistant species. Alternative or additional protection techniques can be implemented by spraying, dipping, soaking, or making pressure treatment with preservatives, as well as limiting the moisture exposition.

2 CASE STUDY - ANALYSIS OF THE STATE OF DAMAGE AND STRENGTHENING OF A BUILDING IN THE HISTORICAL CENTER OF LUCCA (ITALY)

2.1 Description of the building

The building in question is located along the Fillungo street, the main road of the historical center of Lucca; it has medieval origins and has been subjected to changes and interventions over time. It consists of four floors in the south part, while the north has the typology of a six floors tower (figs. 1, 2).

On the ground floor there is the entrance and the common stairs, and three premises used for commercial exercises; below the two stores in the northern part, there is a basement floor. On the mezzanine there are service premises for the stores below; the upper floors are used as dwellings.

As part of the study of the building, a survey was carried out in all the rooms that are part of the building, including the basement, in order to identify the structural elements and all the phenomena of instability present. It was then carried out the detailed survey of the stairs and around the second and third floors. In the other floors, the survey was executed roughly, both for the presence of fixed furnishings that have hindered the execution, and the apparent absence of elements of interest, such as irregularities in the structure or presence of signs of instability.

Figure 1: The building front

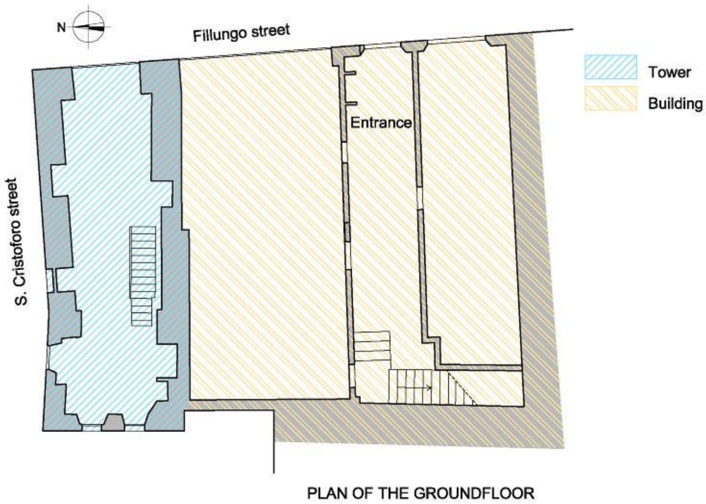

Figure 2: Location of the building

2.2 Description of the structure

The building is composed of two parts, different from each other, both for the typology and for the structure.

The northern part of the building has a tower type, with the plan elongated in the direction perpendicular to the Fillungo street; by observing the arrangement of the load-bearing walls, the differences in the materials used and the eaves height, we can identify two parts, probably built in different periods. All load-bearing walls are very thick, the arris of the façade and the walls in east-west direction are in ashlar masonry up to the fourth floor, in mixed brick-rubble masonry above; this type is also found in the back façade.

In the southern part of the building, the load-bearing structures are organized in the direction orthogonal to the façade on Fillungo street, on three lines: the south wall, in common with the adjacent building, the central wall and the wall on the north side which is part of the tower (figs. 3, 4).

The central wall is made up of two pillars forming part of the façades, P1 and P3 (fig. 3), which are in ashlar masonry at the ground floor, and solid and well-organized brick masonry at the upper floors. Inside, in the center, there is a pillar, P2, of ashlar masonry, which runs from the basement to the mezzanine (fig. 5). From the first floor up to the roof, the central wall is supported by a wooden frame, consisting of a central pillar in continuation of the pillar P2 (fig. 6), and wooden beams at each floor. Between the wooden pillars and the beams, there are brick masonry walls which, on the first floor, have a thickness about 140 mm, including the plaster on both sides, while on the second about 180 mm.

On the first floor, the wooden frame appears as in figure 7: the central pillar, PA, supports a cantilever (MA) which, in turn, supports two beams (TA and TB); the structural joint is completed by the pillar of the upper level (PB). The two beams TA and TB support the beams TC, TD and TE of the upper floor (fig. 4).

Concerning to the type of the floors of the south part of the building, it was possible to detect with accuracy only the one between the first and the second floor. This floor is wooden, with the principal beams directed parallel to the façade on Fillungo street, span about 5,70 m, and placed at different distances. Supported by the beams, there are wooden joists, of long span (up to 4,40 m), and above, the planking and the tiling.

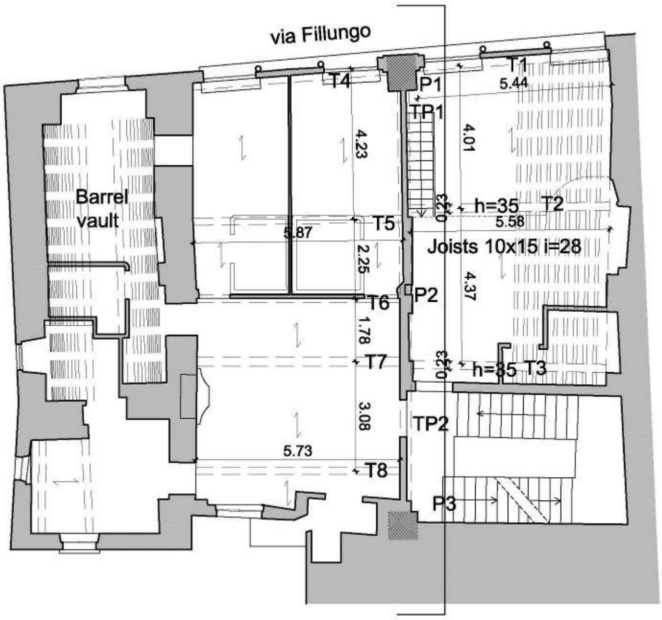

Figure 3: Schematic plan of the structures (1st floor)

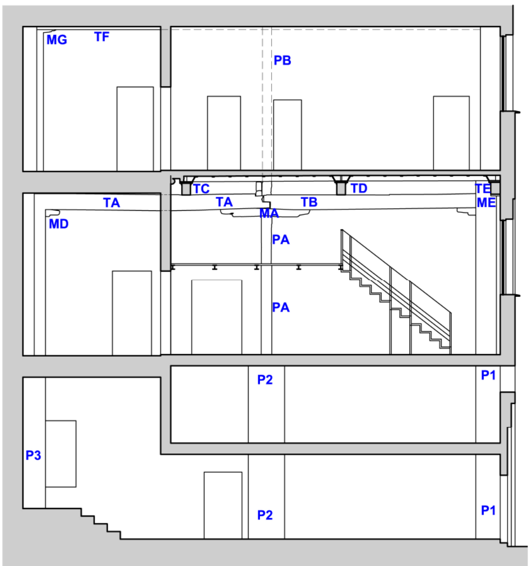

Figure 4: Section of the building

Figure 5: The central pillar P2 on the ground floor

Figure 6: The central pillar PA on the first floor

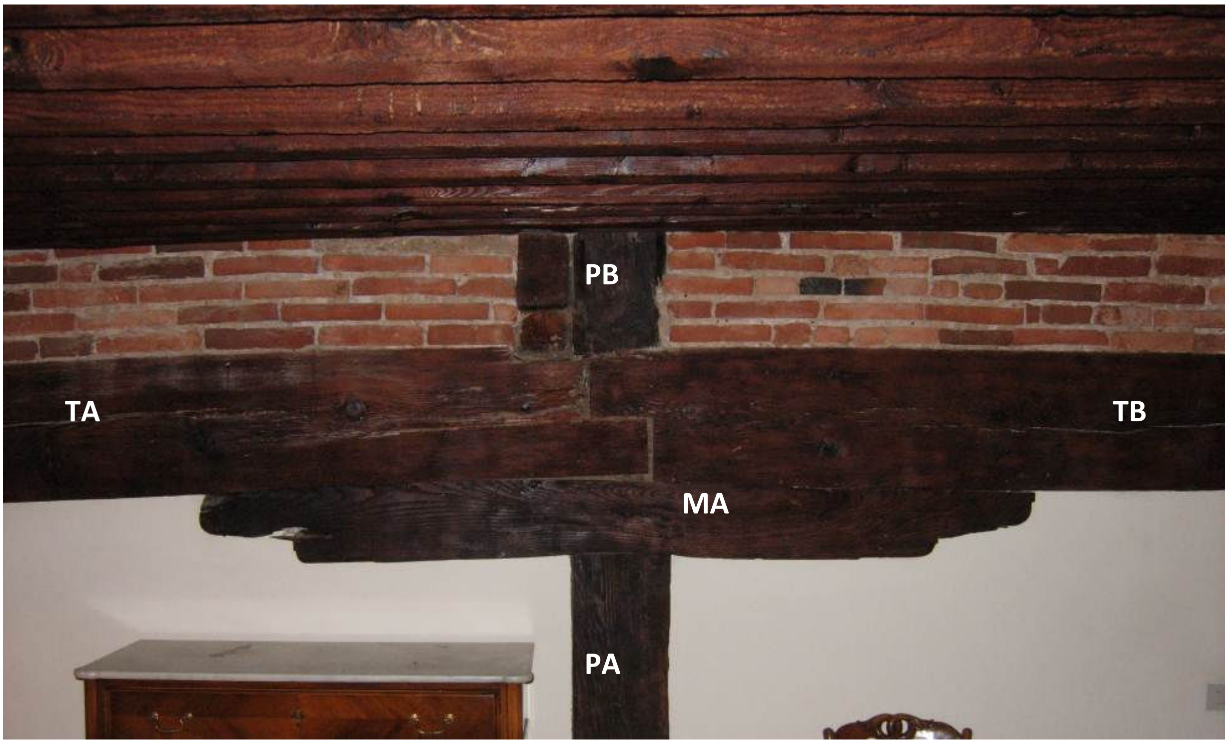

Figure 7: The wooden frame on the first floor

2.3 Description and analysis of damages

From what has been possible to ascertain during the surveys, it emerges that in the basement floor, ground, mezzanine and first floor of the building there are no significant signs of instability; the same for the floors above the second, which are basically developed in the tower.

The manifestations of instability are in fact concentrated on the second floor and extend more or less for the whole storey (fig. 8). They consist essentially of:

- cracks, also of considerable amplitude, of internal partitions, B, I (figs. 9, 10);
- cracks of the central load-bearing wall, C (fig. 11);
- depressions of the floors, N (fig. 12);
- detachment of the ceilings from internal and outside walls, D, M, H;
- cracks in load-bearing walls of the tower, O.

In figures 9-12 are illustrated with diagrams and photos, some typical damage phenomena: in the schematic drawings are given approximately the trend lines of the cracks, their amplitude and the direction of the relative displacement between their edges.

The observation and analysis of the cracks pattern detected, leads to the following considerations.

In the tower, there is a crack pattern of little importance: there are cracks in the wall O with vertical slope, and in the ceilings, in a direction parallel to the main façade, with continuation in the architrave of a door.

It must be highlighted that tn the southern part of the building, the phenomena are more valuable, and in rapid progress.

The crack patterns observed in partitions (B, I,...) are compatible with the high deformability of wooden floors below; these floors are in fact made of joists of fir wood, with spans up to 4,40 m, section 0,1×0,15 m, 0,28 m spacing, supported by the beams T1, T2, T3, of white fir wood, span about 5,60 m, section 0,23×0,35 cm. Both the joists and the beam T2 are rather slim, compared to the load which they must carry; however, they are not even stiffened by partition walls on the lower floor, which were demolished during recent internal renovations.

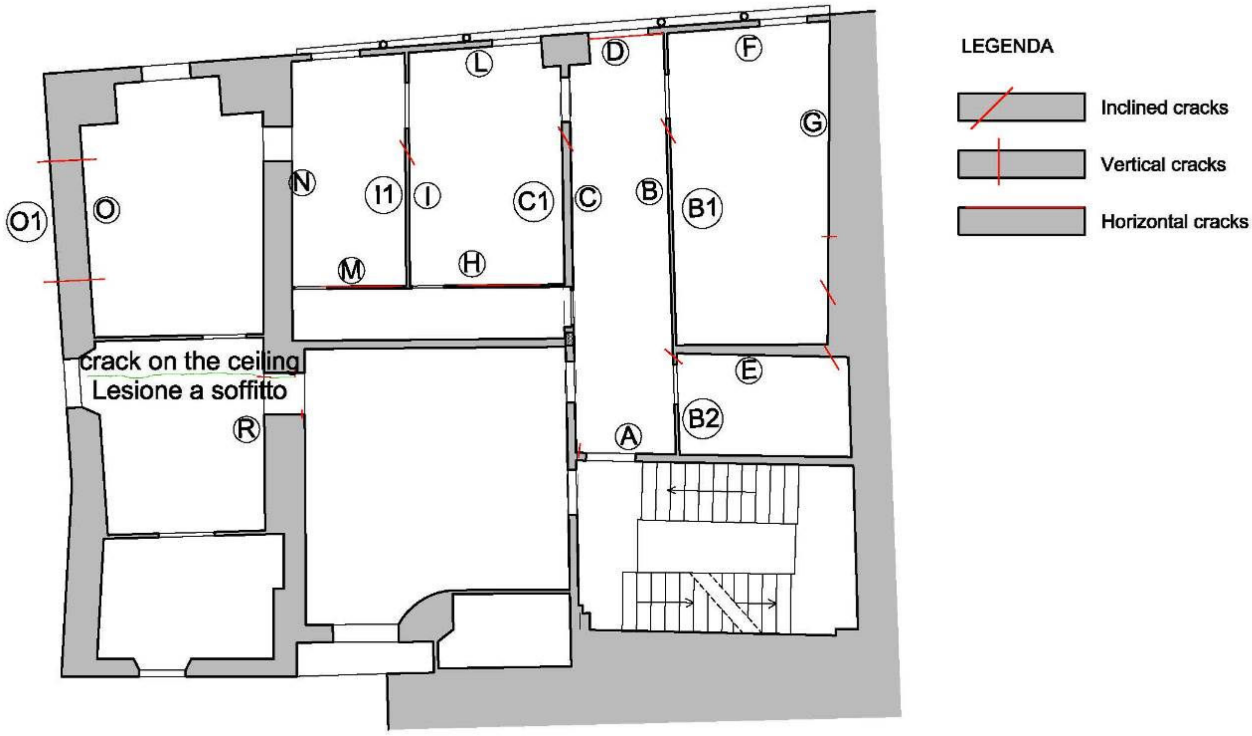

Figure 8: Overview of failures on the second floor

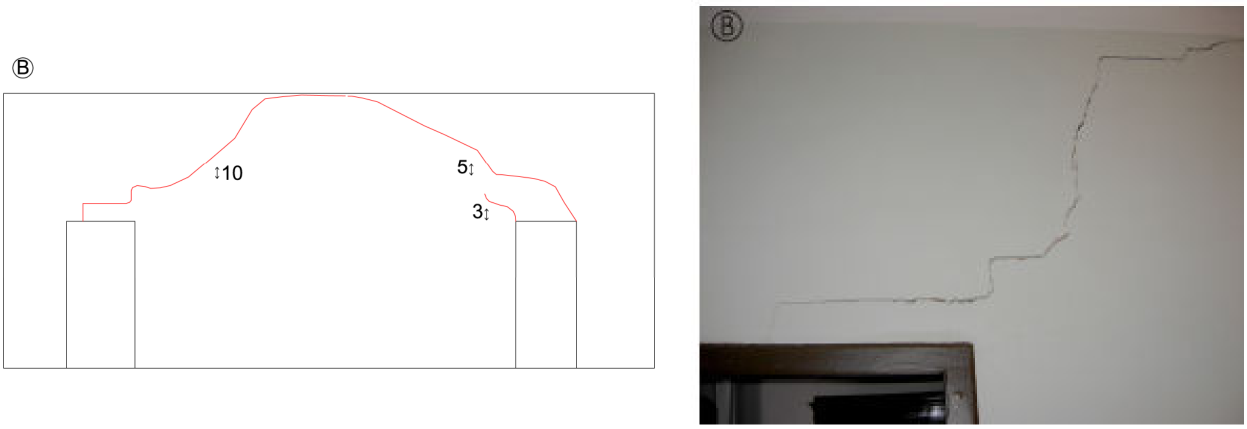

Figure 9: Crack pattern in partition wall B

A similar crack pattern is present in the wall C, which is part of the central bearing wall of the building. Since this phenomenon is not due to the deformability of the floors, the cause was sought in the state of wooden elements that form the frame of the bearing wall. At a first visual inspection, it became clear that both the beams and the base of the pillar were significantly deteriorated (fig. 13), so that the pillar and the above wall were sunk into the beams, and hence the cracking of the wall.

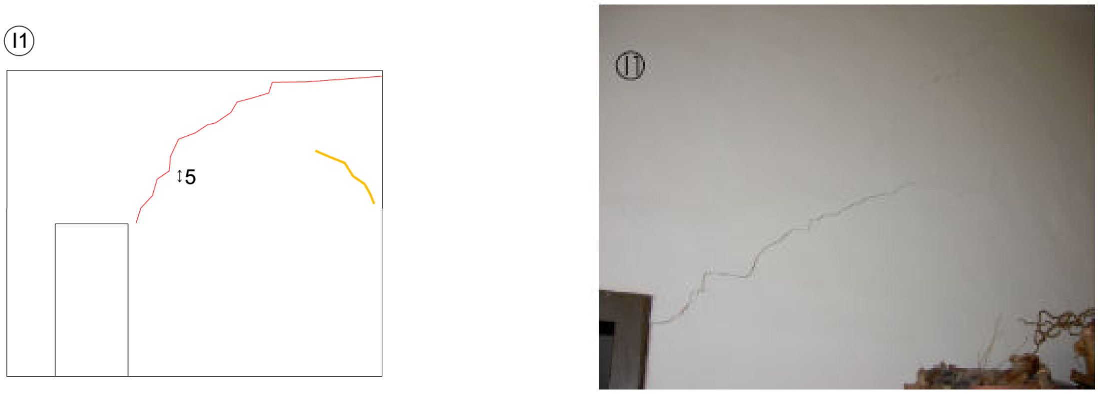

Figure 10: Crack pattern in partition wall I

Figure 11: Crack pattern in central wall C

Figure 12. Depression of the floor, N

2.4 Diagnostic surveys

In order to assess the state of conservation and the residual load-bearing capacity of the wooden elements that are part of the floor between the first and second, were performed diagnostic investigations which have involved the central wooden pillar, up to the floor of the second level, the cantilever and the two portions of beams supported by that pillar, the main beams of the floor which rest on the two aforementioned beams and finally the portion of the wooden pillar that continues to the upper level (figs. 7, 13).

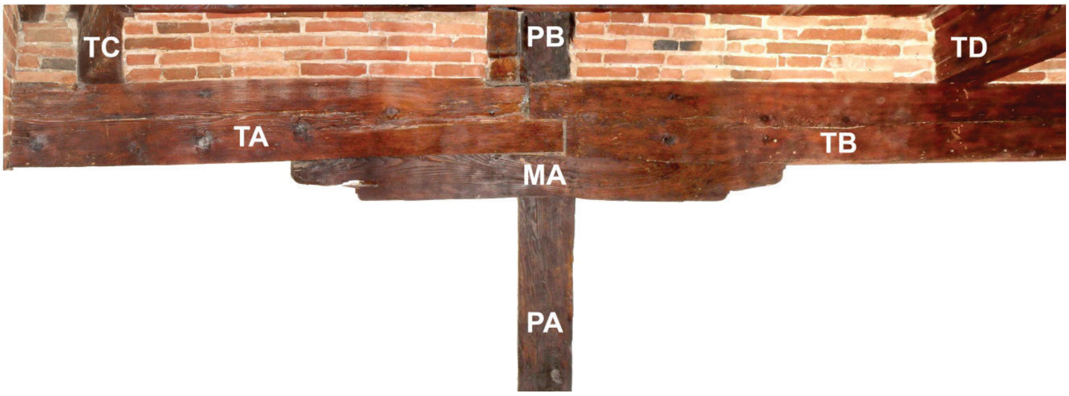

Figure 13: Joint between wooden beams and pillars on the first floor

The investigations were carried out, where possible, in accordance with the methodology described by the italian standard UNI 11119:2004. This involves making a diagnosis by visual inspection, supplemented by an investigation with a special tool. The visual inspection objectives are the determination of species, the assessment of defects aimed at structural classification, the in situ estimate of wood moisture to determine the risk of fungal attack. The inspection performed by a special drill named Resistograph, allows to identify the presence of hidden attacks by pathogens (insects and/or fungi) and to estimate its size, in order to determine the residual resistant section.

The investigations carried out have provided the following results.

The pillars PA and PB are made of chestnut wood, section 280 x 300 mm about. The investigations with Resistograph have not found the presence of degradation nor at the base or at the head of the element PA, while in the element PB showed the presence of some degradation at the joint with the beams TA and TB, due to attack by termites (fig. 14).

The cantilever MA, made of chestnut wood, is broken by bending. The fracture occurred for the presence of a strong degradation internal to the cantilever itself in its distal portion, as evidenced by Resistograph profiles (fig. 15).

The beams TA and TB are made of silver fir, average section of 320 x 400 mm about.

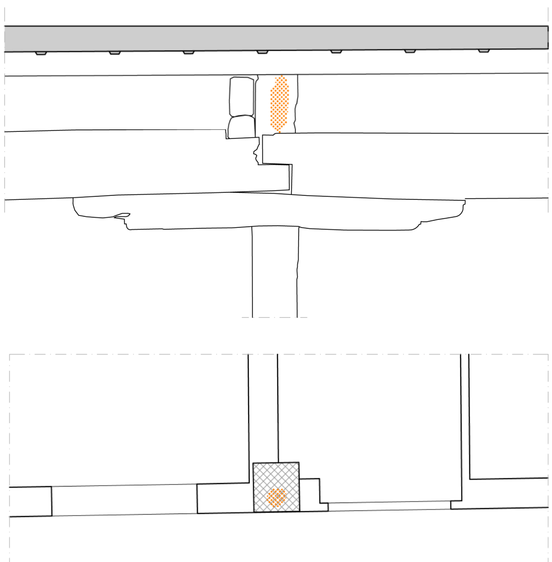

Figure 14: Extension of degradation on the pillar PB (elevation and horizontal section)

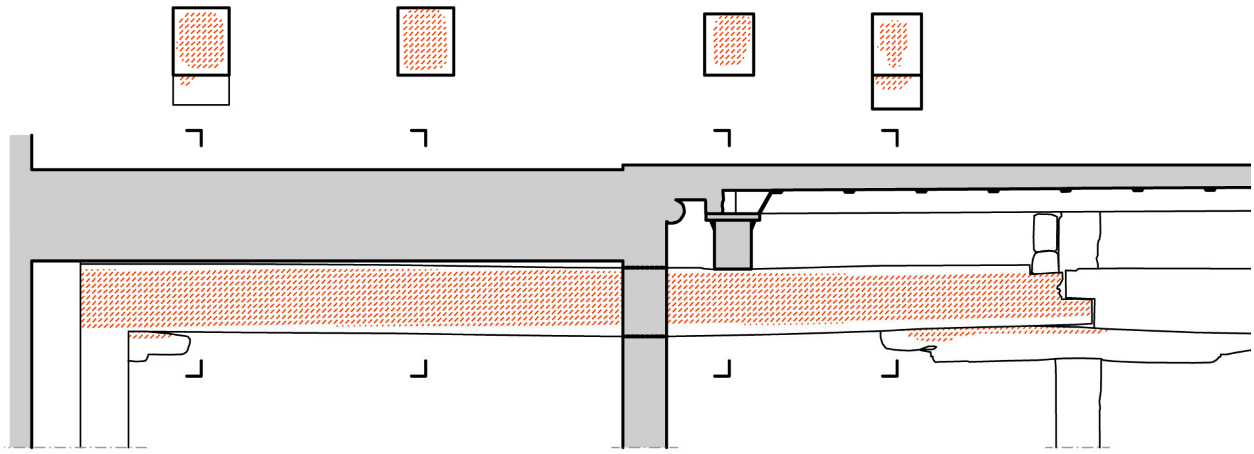

Figure 15: Extension of degradation on the cantilever MA and the beam TA

The beam TA shows a marked deformation both at the joint with the beam TB, and near the support of the beam TC, which is a clear indication of the presence of a strong degradation inside the beam. The Resistograph analysis confirm a degradation extended between 75% and 100% of the section of the element, caused by an attack of termites, no longer in place (Fig. 15).

For the beam TB, the Resistograph profiles show the presence of a significant degradation in the area of the connection with TA, where degradation affects 50-75% of the section concerned (fig. 16).

For the beams that support the ceiling (indicated by the letters TC, TD and TE) and the joists, were determined the type of wood, silver fir, and the structural classification; in general, they are in good condition and still fit for structural use, except for the end of the beam TD, where it is present a cavity already filled in the past with cement mortar with no structural characteristics.

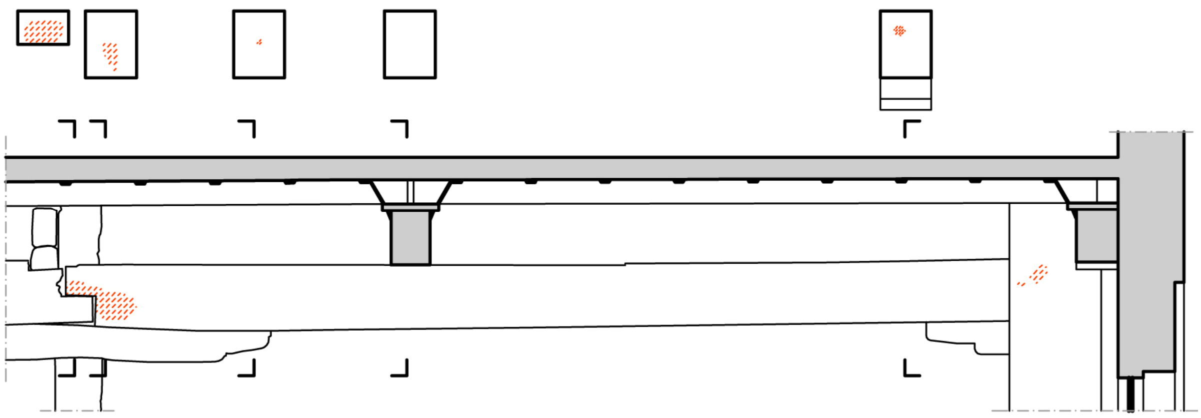

Figure 16 - Extension of degradation on the beam TB

2.5 Deficiencies of the building and consolidation works

Investigations and surveys carried out have highlighted two main problems: the first regarding the elements of wood in the central bearing wall, severely deteriorated and in highly precarious equilibrium, and the other on the wooden structure of the second floor, where both the beams and the joists have very small dimensions of the sections in relation to their span, which produce an inadequate safety level and an excessive flexibility that induces cracks in the partition walls above.

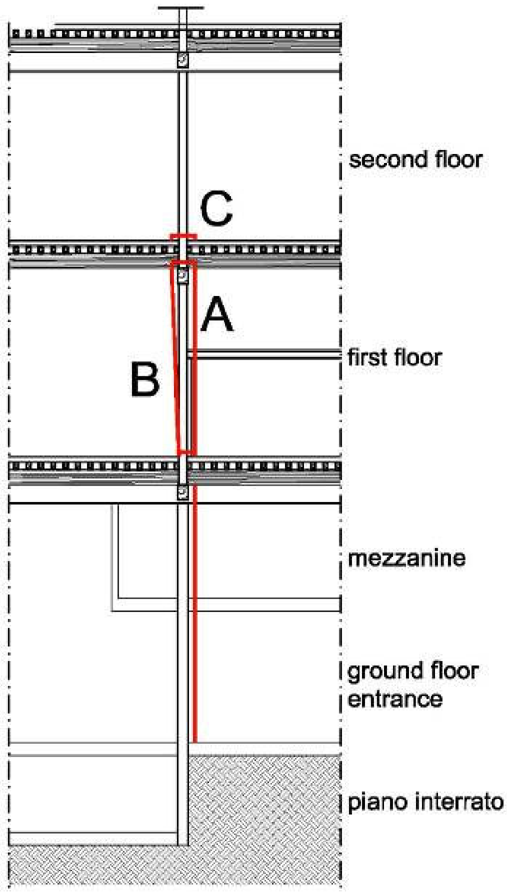

Figure 17: Scheme of the propping **Figure 18: The props A**

The first operation performed consisted in the restoration of the wooden elements, part of the central bearing wall. This requested to prop the wall above the beams TA and TB that had to be reclaimed (fig. 17). Such supporting was performed, on one face of the wall (A), in a direct manner by conveying the loads to the ground (fig. 18), while, on the other (B), due to the presence of false ceilings and the need of maintaining the full accessibility at the ground floor, the supporting had to be made by conveying the load on the first floor near the central wall (fig. 19).

The wooden pillar on the second floor (C) was supported by two steel profiles fastened with bars, and resting, through blocks of wood, on the floor, which was supported by props (fig. 20).

The restoration of the beam TA consisted in almost total reconstruction with the insertion of a reinforcement, composed of longitudinal bars, inserted in the adjacent structures, and stirrups, and the subsequent filling with epoxy mortar of suitable composition.

The restoration was conducted piece by piece by doing the following:

1. removal of the bricks above the portion of the beam;
2. cleaning of the interior of the beam from the corroded wood (fig. 21);
3. waterproofing treatment of the inner surfaces of the beam (fig. 22);
4. inserting of the reinforcement made of galvanized steel (fig. 23);
5. casting of epoxy mortar into the resulting cavity of the beam (fig. 24).

The process of cleaning the interior of the beams confirmed, as highlighted by the instrumental surveys, that the wood was heavily damaged due to attack by termites; termites, in fact, dig tunnels in the spring portion of the beams, determining the severe damage and the danger of sudden collapse (fig. 25).

Figure 19: The props B

Figure 20: The props C

In a similar way were restored the degraded portions of the beam TB and of the pillar PB.

When completed, the elements have retained their appearance, because the outer layer of wood was kept and used as formwork for the casting of the grout (fig. 26).

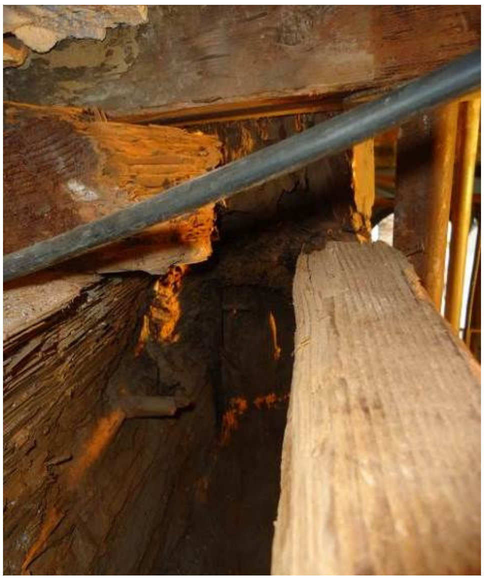

Figure 21: Beam TA: phase 2

Figure 22: Beam TA: phase 3

For the cantilever MA, which, in addition to degraded, appeared broken for bending, it is operated as follows: the cantilever has been removed from its position (fig. 27); it has been practiced a vertical cut in a median position, in which it was inserted a steel plate to which crops of steel bars were welded (fig. 28); finally, the steel element was fixed with epoxy resin and the beam repositioned.

Figure 23: Beam TA: phase 4

Figure 24: Beam TA: phase 5

Figure 25: Wood deteriorated by termites

Figure 26: Beam TA after restoration

Figure 27: Removal of the cantilever MA

Figure 28: Reinforcement of the cantilever MA

The wooden floors between the first and second floors were reinforced in different ways, for functional reasons (fig. 29). In the central area, two steel beams, RT1 and RT2, were added in the middle of the length of the joists, then covered with wooden boards. This action has proved of rapid execution and especially has allowed the use of the accommodation on the second floor during the whole duration of the work. In the adjacent area, in which the presence of a loft on the first floor prevented the insertion of reinforcement for the already too low height of interplane, it is opted for the intervention of reinforcing the extrados.

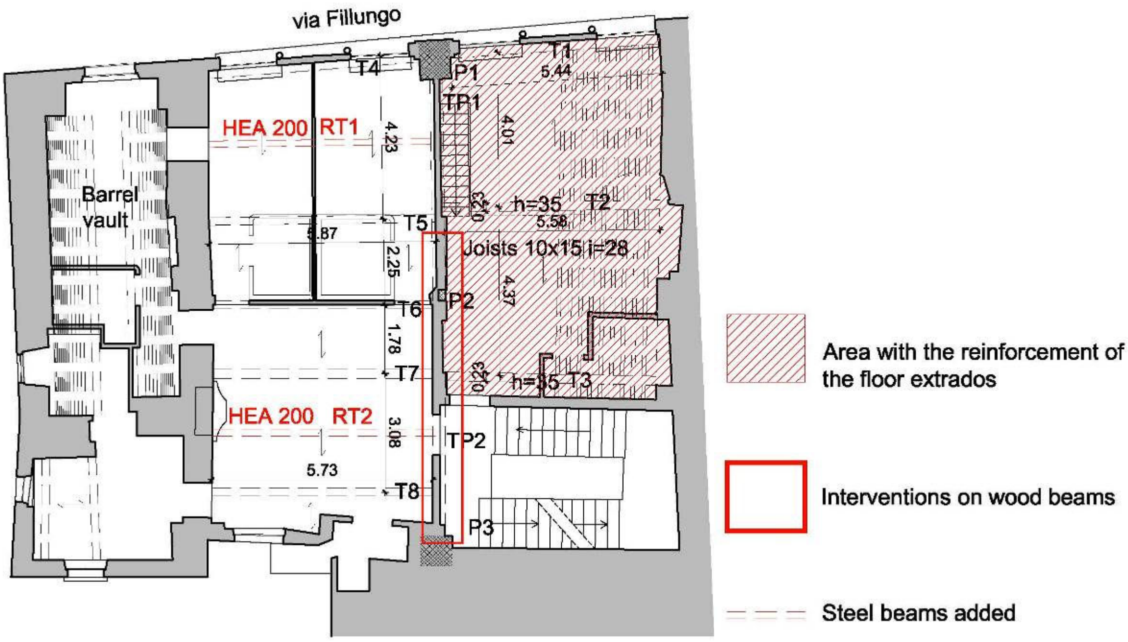

Figure 29: Interventions on the first floor. Are indicated: RT1, RT2, steel beams added; hatch, area with the reinforcement of the floor extrados

The intervention included:

– the demolition of the partitions and disassembly of the bathroom, the removal of tiling, screeds and ceiling of the second floor;
– the insertion of metal connectors along the joists and beams (fig. 30);
– the realization of a reinforced lightweight concrete slab;
– the reconstruction of partitions, ceilings, bathroom, and general restoration of the pre-existing situation.

3 FINAL REMARKS

In this chapter, it has been illustrated the state of degradation of the structure of an ancient building in the historical center of a medieval town in Italy and the interventions of restoration that have been implemented.

As usually happens when operating in existing buildings, the first problem is to understand which is the cause of instability, and this is not always simple, because the origins may be unusual and/or hidden.

The other problem is to find the right solution for the intervention which is always much more complicated than in new buildings, because of the conditioning situations. In the case here examined, the requirements were to discontinue using the premises for the shortest possible time; there were problems of propping, too, and handling of long or heavy elements.

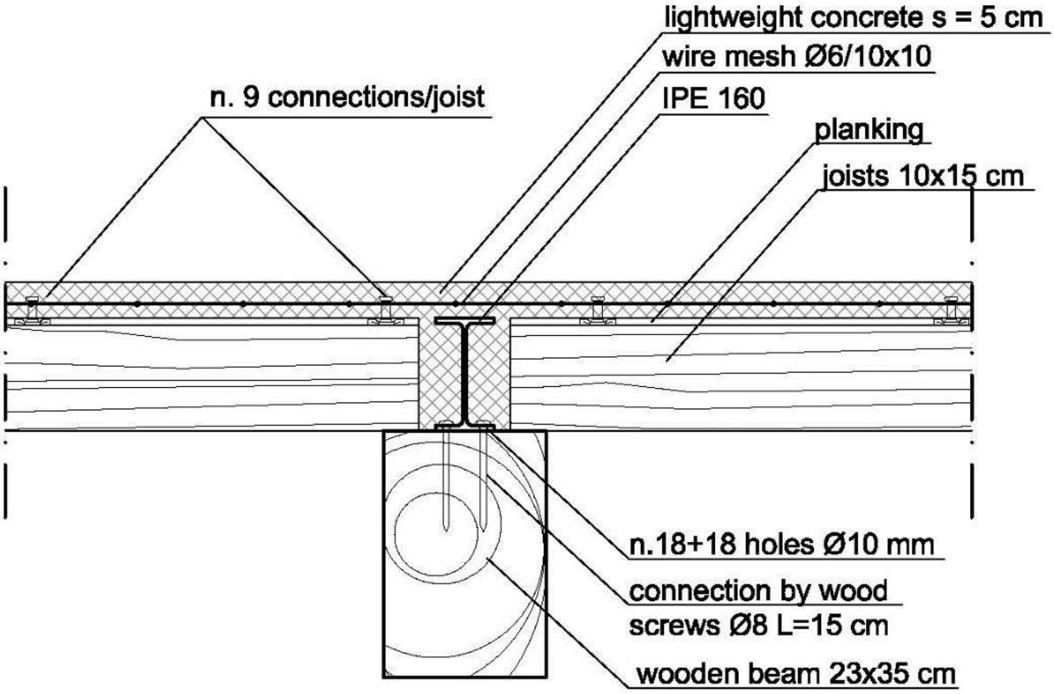

Figure 30: Reinforcement of the wooden floor

Finally, the solution of the intervention was studied in every detail so as to meet the needs of safety as well as the functional and logistical needs.

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CHAPTER 9: HERITAGE STRUCTURES

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1 INTRODUCTION

Heritage buildings and particularly monumental buildings deserve special considerations regarding their structural assessments. The term "heritage buildings or structures" covers monumental historical buildings such as castles and churches, and also bridges, industrial heritage structures as well as master-pieces of modern architecture. ICOMOS (International Council on Monuments and Sites) issued the principles of conservation which are agreed upon by the International Charters require a careful respect for the integrity of heritage buildings and oblige avoidance of methods of investigation and interventions that might entail a loss of authenticity of the heritage building [2]. In general, the most developed societies perceive necessary to maintain architectural heritage [3]. However, the need for rehabilitations of heritage structures is often confined by severe economic constraints. Structural strengthening is then considered as the most sensitive aspect of the rehabilitations since it may conflict with the heritage value [1]. That is why assessment of heritage structures often requires application of sophisticated methods, as a rule beyond the scope of traditional design codes.

Nevertheless, apart from few national codes, three international standards ISO (ISO 2394 [4]; ISO 13822 [5]; ISO 12491 [6]) are available concerning the assessment of existing structures. For example, ISO 13822 [5] provides general requirements and procedures for the assessment of existing structures (buildings, bridges, industrial structures, etc.) based on the principles of structural reliability and consequences of failure. This International Standard is also applicable to heritage structures provided additional considerations shown in Annex I are taken into account. Additional information may be found in a number of scientific papers and publications like [7] and [8].

According to ISO 13822, the assessment of a heritage structure will include two aspects: that concerning its structural performance, familiar to engineers, and that concerning its value as a cultural resource. These two aspects shall both be taken into account in any decision involving possible structural interventions.

In general a heritage structure may be subjected to the reliability assessment [1] in case of:

- rehabilitation during which new structural members are added to an existing load-carrying system;
- adequacy checking in order to establish whether the heritage structure can resist loads associated with the anticipated change in use, operational changes or extension of its working life;
- repair of a heritage structure, which has deteriorated due to time dependent environmental effects or which has suffered damage from accidental actions, for example earthquake;
- doubts concerning actual reliability of the structure.

Under some circumstances assessments may also be required by authorities, insurance companies or owners or may be demanded by a maintenance plan.

The approach to assessment of a heritage structure is in many aspects different from that taken in structural designing. Difficulties in assessments of heritage structures may arise from the complexity of geometry, variability of material properties, different construction techniques, limited knowledge on structural conditions including the damage from past actions, and from interventions restricted by heritage value and excessive costs [7]. Methods of experimental mechanics and numerical simulation approach have been rather recently introduced in professions which had been for long reserved only for humanities or arts. Conservation of cultural heritage belongs among such fields.

However, even though the heritage structure may be investigated several times, some uncertainty in the basic variables and structural behaviour shall always remain. Therefore, similarly as in design of new structures, actual variation in the basic variables describing actions, material properties, geometric data and model uncertainties are taken into account by partial factors or other code provisions.

The chapter summarises general principles of the structural assessment of heritage structures and provides several examples.

1.1 Principles of assessment

Two main principles are usually accepted when assessing heritage structures [1]:

(1) Currently valid codes for verification of structural reliability should be applied; historic codes valid in the period of structural design should be used only as guidance documents.
(2) Actual characteristics of structural materials, actions, geometric data and structural behaviour should be considered, the original design documentation including drawings should be used as guidance only.

The first principle should be applied in order to achieve a similar reliability level as in case of newly designed structures. The second principle should avoid negligence of any structural condition that may affect actual reliability (in a favourable or unfavourable way) of the structure.

Most of the current codes have been developed assuming the concept of limit states in conjunction with the partial factor method. In accordance with this method, which is mostly considered here, basic variables are specified by characteristic or representative values. The design values of basic variables are determined on the basis of the characteristic (representative) values and appropriate partial factors.

It follows from the second principle that a visual inspection of the assessed structure should be made whenever possible. Practical experience shows that inspection of the site is also useful to obtain a good feel for actual situation and state of the structure.

As a rule, quantitative assessments need not to be performed for those parts of the structure that will not be affected by structural changes, rehabilitation, repair, change in use or which are not obviously damaged or not suspected of having insufficient reliability, ISO 13822. In general the assessment procedure consists of the following steps (see the flow chart in ISO 13822):

- specification of the assessment objectives required by a client or authority;
- scenarios related to structural conditions and actions;
- preliminary assessment: study of available documentation, preliminary inspection, preliminary checks, decision on immediate actions and recommendation for detailed assessment;

- detailed assessment: detailed documentary search, detailed inspection, material testing and determination of actions, determination of structural properties, structural analysis, verification of structural reliability;
- report including proposal for construction intervention;
- repeat the sequence if necessary.

When the preliminary assessment indicates that the structure is reliable for its intended use over the remaining life, a detailed assessment may not be required. Conversely if the structure seems to be in dangerous or uncertain conditions immediate interventions and detailed assessment may be necessary.

In accordance with Annex I of ISO 13822 structural assessment of a heritage structure should be carried out in collaboration with a multidisciplinary team of engineers, architects, archaeologists, historians, material scientists and possibly other specialists. While the structural engineer should deliver a specific structural evaluation report, the ramifications of this report should be discussed within the multi-disciplinary team and decisions should be generally reached by consensus.

1.2 Investigation

Assessment of existing structures should be based on the actual as-built conditions concerning geometry, material properties, loading and environmental conditions. Investigation of a heritage structure is intended to verify and update the knowledge about the present condition (state) of the structure with respect to a number of aspects [1]. Often, the first impression of the structural condition will be based on visual qualitative investigation. The description of possible damage of the structure may be presented in verbal terms like: '*unknown, none, minor, moderate, severe, destructive*'. Very often the decision based on such an observation will be made by experts in a purely intuitive way.

A better judgement of the structural condition can be made on the basis of (subsequent) quantitative inspections. Typically, the assessment is a cyclic process when the first inspection is supplemented by subsequent investigations. The purpose of the subsequent investigations is to obtain a better feel for the actual structural condition (particularly in the case of damage) and to verify information required for determination of the characteristic and representative values of all basic variables. For all inspection techniques, information on the probability of detecting damage if present, and the accuracy of the results should be given.

The statement from the investigation contains, as a rule, the following data describing:

- actual state of the structure;
- types of structural materials and soils;
- observed damage;
- actions including environmental effects;
- available design documentation.

Proof loading is a special type of investigation. Based on such tests one may draw conclusions with respect to:

- the bearing capacity of the tested member under the test load condition;
- other members;
- other load conditions;
- the behaviour of the system.

The inference in the first case is relatively easy; the probability density function of the load bearing capacity is simply cut off at the value of the proof load. The inference from the other cases is more complex. Note that the number of proof load tests needs not to be restricted to one. Proof testing may concern one element under various loading conditions and/or a sample of structural elements. In order to avoid unnecessary damage to the structure due to the proof load, it is recommended to increase the load gradually and to measure the deformations. Measurements may also give a better insight into the behaviour of the system. In general, proof loads can hardly address long-term or time-dependent effects. These effects should be analysed by calculation.

1.3 Structural analysis and verification

Structural behaviour should be analysed using models that describe actual situation and state of a heritage structure. Generally the structure should be analysed for ultimate and serviceability limit states using basic variables and considering relevant deterioration processes [1].

All basic variables describing actions, material properties, load and model uncertainties should be considered as mentioned above. The uncertainty associated with the validity and accuracy of the models should be considered during assessment, either by adopting appropriate factors in deterministic verifications or by introducing probabilistic model factors in reliability analysis.

When a structure is analysed, conversion factors reflecting the influence of shape and size effect of specimens, temperature, moisture, duration-of-load effect etc., should be taken into account. An example is minor destructive drilling of masonry units. The level of knowledge about the condition of components should be also considered. This can be achieved by adjusting the assumed variability in either the load carrying capacity of the components or the dimensions of their cross sections, depending on the type of structure.

When deterioration is observed, the relevant mechanisms shall be identified and a deterioration model predicting the future performance of the structure shall be determined on the basis of theoretical or experimental investigation, inspection and experience. Even considerably simplifying assumptions may yield realistic results as shown for degradation of reinforced concrete structures in [9].

Reliability verification of a heritage structure shall be made using valid codes of practice, as a rule based on the limit state concept. Attention should be paid to both the ultimate and serviceability limit states. Verification may be carried out using partial safety factor or structural reliability methods with consideration of structural system and ductility of components. The reliability assessment shall be made taking into account the remaining working life of a structure, the reference period, and changes in the environment of a structure associated with an anticipated change in use.

The conclusion from the assessment shall withstand a plausibility check. In particular discrepancies between the results of structural analysis (e.g. insufficient safety) and the real structural condition (e.g. no sign of distress or failure, satisfactory structural performance) must be explained. It should be kept in mind that many engineering models are conservative and cannot be always used directly to explain an actual situation.

The target reliability level used for verification can be taken as the level of reliability implied by acceptance criteria defined in proved and accepted design codes. The target reliability level shall be stated together with clearly defined limit state functions and specific models of the basic variables.

The target reliability level can also be established taking into account the required performance level for the structure, the reference period and possible failure consequences. In accordance with ISO 2394 the performance requirements for assessment of existing structures are the same as for design of a new structure. Lower reliability targets for existing structures may be used if justified on the basis of economic, social and sustainable considerations; more details are provided elsewhere [10] [11]. A simple model for specifying the target reliability of heritage structures was proposed in [12]. In accordance with Annex I of ISO 13822 it is important to realise that the protection of heritage value may require the acceptance of a different reliability level, lower than that in design codes. Fundamental differences between structural design and the assessment of existing structures and protection of heritage values should be considered.

2 CASE STUDY N. 1 – THE SANCTUARY OF OUR LADY OF THE CROSS IN POGGIO DI ROIO

Immediately after the earthquake that hit the Abruzzo 6 April 2009, the Italian Ministry of Culture has launched a campaign of studies on a large number of damaged monumental buildings, aimed at interpreting the mechanisms of damage and disruption triggered by the earthquake. This was a preliminary activity devoted to perform a critical analysis of the effectiveness of any prior interventions of consolidation to improve guidelines for conservation and seismic improvement of such important heritage.

As part of these initiatives, the group from the University of Pisa dealt with the analysis of a Sanctuary of Our Lady of the Cross stands in the main square of Poggio di Roio in the town of L'Aquila (fig. 1), which was severely damaged

Figure 1: General view of the Sanctuary of Our Lady of the Cross before the 6 April 2009 eartquake

2.1 Historical background

The Sanctuary was built in 1625, on the existing St. Leonardo Chapel (dating from 1200), to provide accommodations to a statue of the Madonna, which was found in the place where a shepherd found his flock thanks to the miraculous apparition of the Virgin.

The Sanctuary preserves a portion of the original building, which is the altar area and the vault, painted where a noticeable sixteenth-century fresco depicting Our Lady of Hope.

2.2 Typological and dimensional data

The original plan of the sanctuary was a Greek cross, with two vestries at the sides of the altar and the bell tower in the west side (fig. 2). In the last century, it was added the rectory, connected to the east side chapel attached by a covered walkway. Currently the sacristy west also serves as access to the Our Lady of the Cross Institute.

The internal length of the nave is about 15 m and its internal width is about 6,40 meters, while the transects are 4,20 m long and 6,00 m wide.

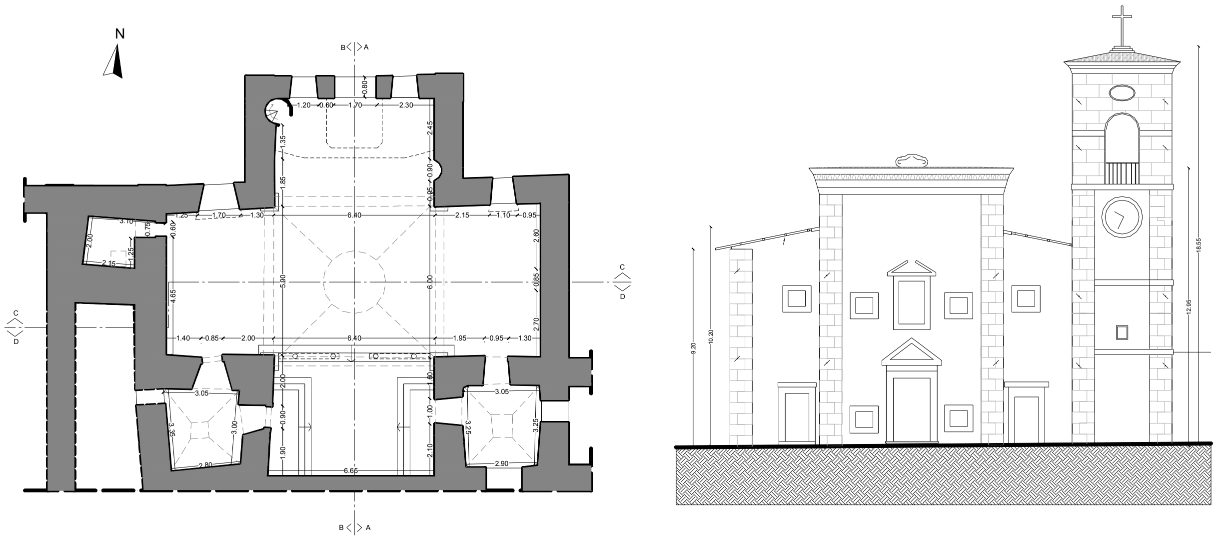


Figure 2: Plan and north front of the Sanctuary

The limbs of the cross are covered by barrel vaults, whose keystone height is 9,30 m, ending in a cross vault height of 9,70 m in key in correspondence of the central part, while the lateral sacristies, square in plan with sides of 3,00 m about, are covered with ribbed vaults.

The bell tower has a square plan with a 4.70 m side, while its height is about 18,20 m.

It must be noted that, approximately every 4 meters in elevation, there are steel chains in both directions, so that four levels of chains are in the bell tower and two levels of chains are in the main body of the Sanctuary.

2.3 Type of structural elements

From the survey carried out, the structure resulted constituted by masonry walls, whose thickness was remarkable and varying between 0,8 m and 1 m. The masonry, which can be classified as chaotic and mixed, presents irregular texture, with stone elements irregular again, characterized by joints of considerable size, filled with mortar of very poor quality (see figs. 3 and 4). However, even if it was not possible to inspect directly the roof because of safety reasons, the investigations carried out demonstrated that the quality of the masonry of the central vault as well as of the barrel vaults is quite satisfactory.

The cantonal of the building are also made by chaotic masonry, covered by stone slabs 0,20 m thick, arranged in the vertical plane.

Figure 3: Detail of masonry at the north-east corner

Figure 4: Masonry of the North face of the East limb

The two pitched roof is characterized by a wooden structure, being the central ridge beam perpendicular to the main façade.

2.4 In situ testing campaign

In order to better understand the mechanisms of damage triggered by the earthquake, an ad hoc in situ testing campaign has been carried out on the building to recognize the state of damage of the structure. The survey regarded the main body of the Sanctuary and the two lateral sacristies.

The connection between the sanctuary and the adjacent convent was inaccessible, while, again for safety reasons, it was not possible to reach the top floors of the bell tower.

2.5 Microseismic survey for the estimation of the elastic parameters of the masonry

To estimate the mechanical parameters of the masonry non-destructive micro-seismic investigations were performed on the walls of the church façade, in the positions indicated by T1 and T2 in figure 4.

Figure 4: Micro-seismic investigated zones

In each location measurements were elaborated according to different methods: seismic profiles, profiles type MASW (multichannel analysis of seismic waves), tomograms by refraction, in such a way to compare and to refine results obtained with different techniques.

By elaborating seismic profiles it was possible to conclude (see, for example, Fig. 5) that the average speed of propagation of P-waves was of about 1000 m/s for one of the walls and of about 1400-1500 m/s for the second: these average speeds correspond to dynamic elastic moduli of the order of 1000 MPa and 1600 MPa, respectively, which are typical of heavily decayed walls.

The application of the MASW technique allowed to determine, for the second wall, a velocity profile for the S-waves variable between 350 m/s and 590 m/s, which yields to a value of Poisson's ratio around 0,4.

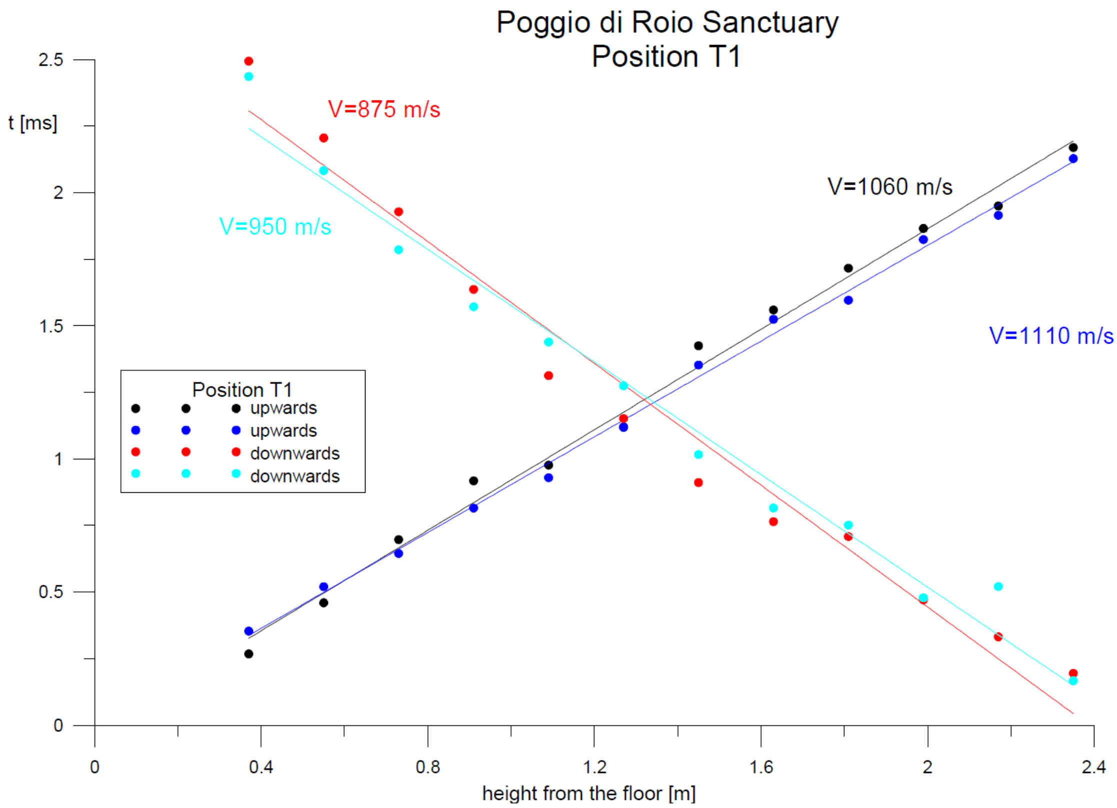


Figure 5: Seismic profile – T1 position

2.6 State of damage survey

During the in situ investigation, the detailed crack patterns and the level of the damaged were carefully recorded. To emphasize the graphical representation of the crack pattern, cracks were grouped, according to the classification of the damage Scale of the European Macroseismic EMS98, in three different classes, depending on their approximate width. In the following figures cracks whose width is less than 1 mm are indicated in green, cracks whose width is between 1 and 3 mm are in yellow and those having greater width are in red.

The main wall facade (figs. 6 and 7) has vertical cracks at the centerline of the tympanum, which depart from the top of the mosaic frame and extend up to the end marble fillet. A series of major injuries is detected also near the chains of the second order as well as around the second row of windows. In addition we observe a significant detachment of the stone cover in the upper left corner of the tympanum, as well as part of the marble cornice.

Additional cracks, less relevant in terms of amplitude, but very meaningful in the diagnosis, run horizontally, where the tympanum surpass the main body of the Sanctuary.

Inside the church, significant gaps are present between the main façade and the intrados of the vault of the nave (fig. 8), the window of the west limb and the gate of the eastern limb, where the cracks correspond to the external ones.

In the organ area additional cracks confirm that the clamping between the eastern wall and the main façade is not satisfactory.

Further significant cracks are present above the eastern window of the first order as well as and both side doors.

Side walls of the façade are characterized by diagonal cracks starting from the openings and descending towards the east. The crack patterns is more significant on the right wall.

Full-thickness diagonal shear cracks and vertical cracks near the corner cover indicate the mobilization of overturning mechanisms of the façade walls (fig. 9).

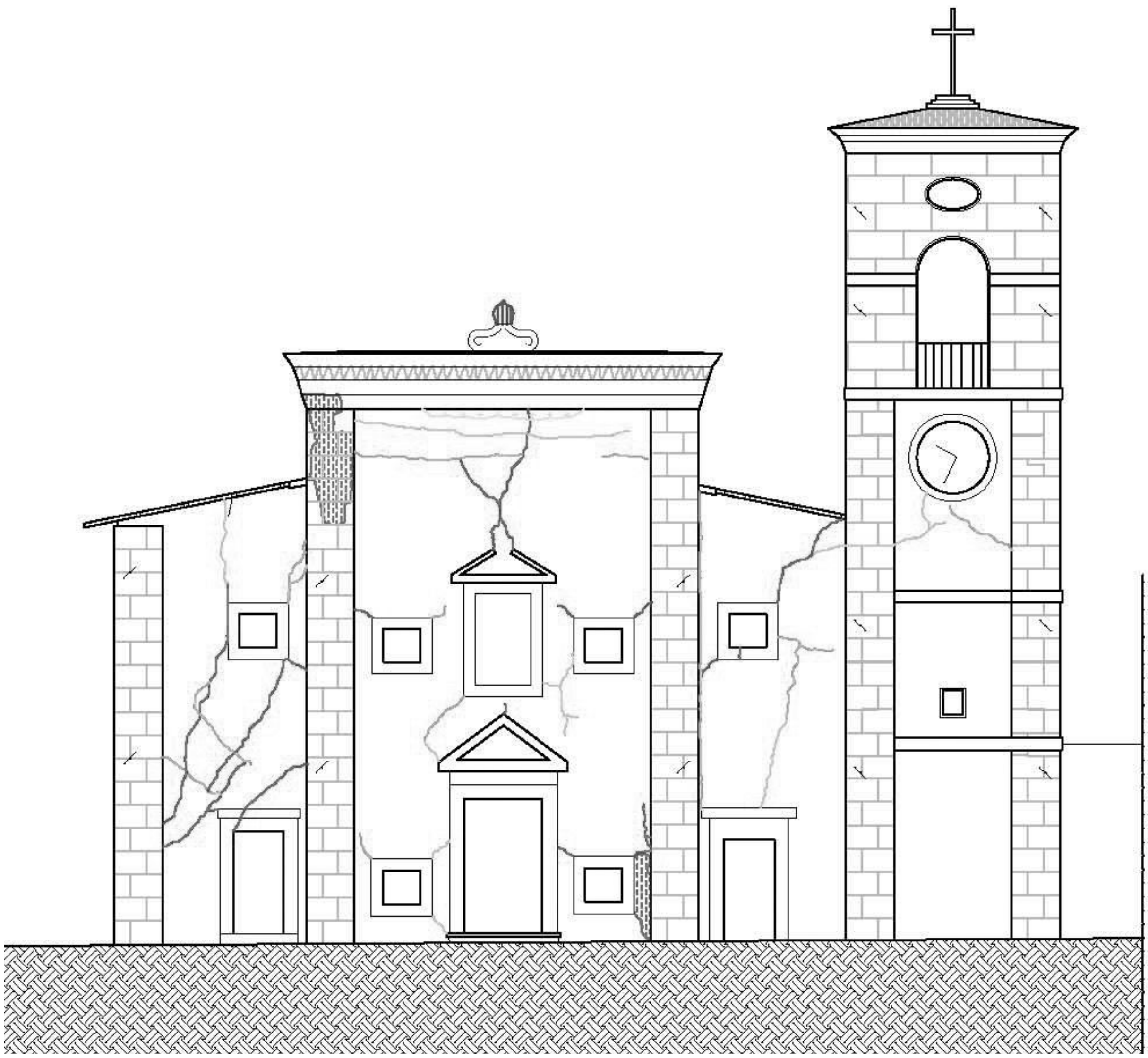


Figure 6.a: Crack pattern of the main façade (external view)

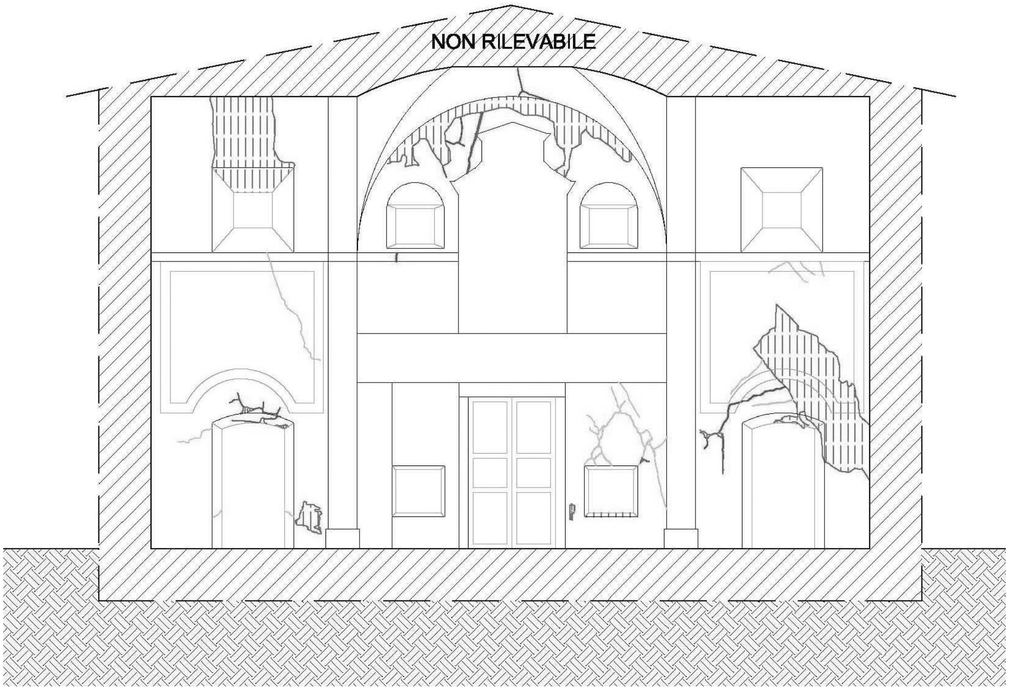


Figure 6.b: Crack pattern of the main façade (internal view)

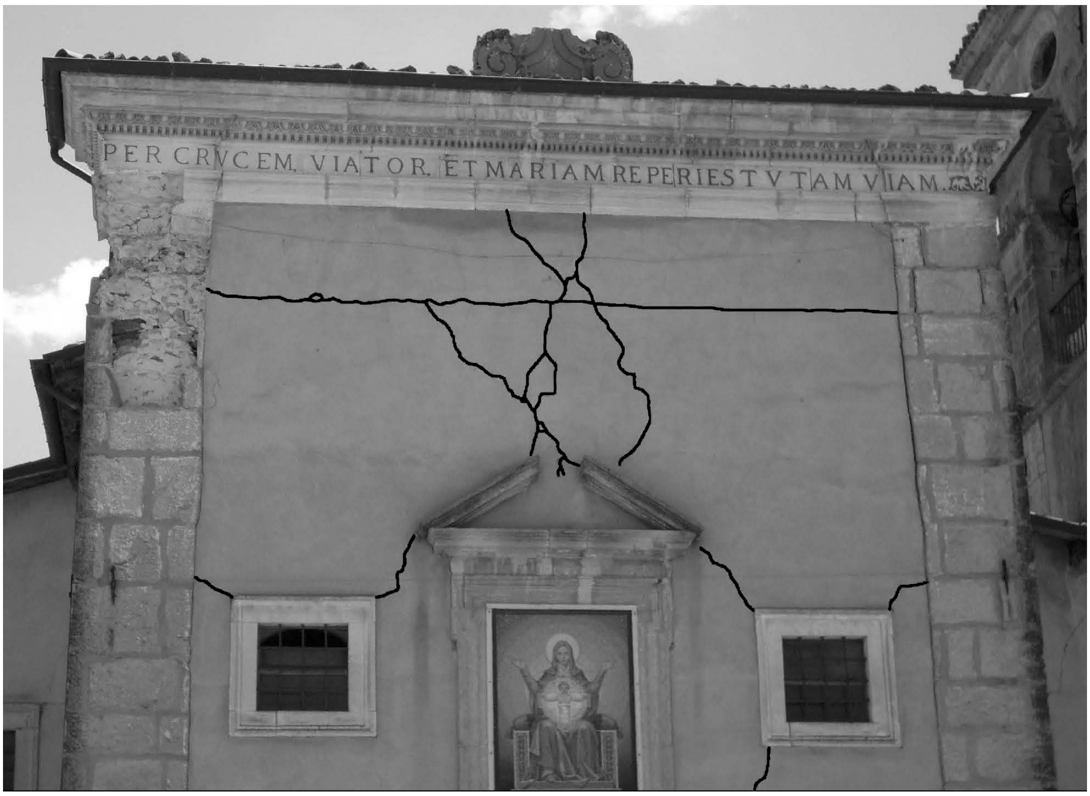


Figure 7: Crack pattern near the tympanum of the main façade

The crack patterns at the attachment with the adjacent buildings and in the back wall of the altar are less pronounced.

Finally, in the bell tower horizontal cracks start from the connection with the main body of the Sanctuary and propagate up to the clock.

The overall crack pattern is summarized in the graphs illustrative of the main failure mechanisms that have been activated, which are discussed below.

Figure 8: Crack between the main façade and the vault

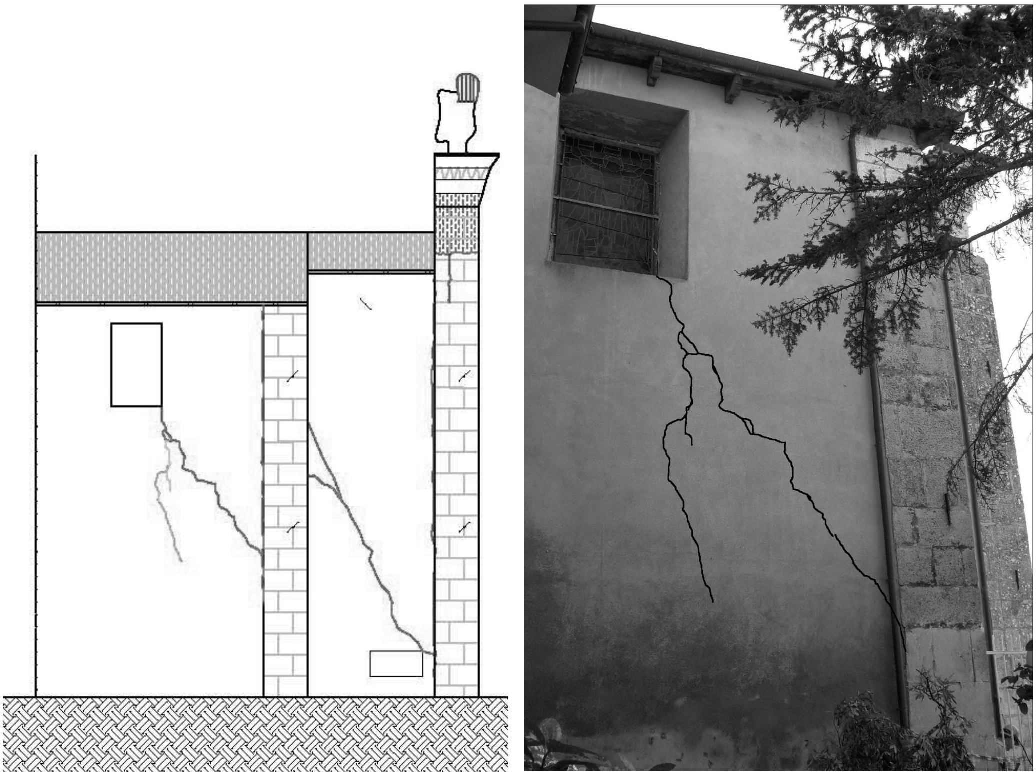


Figure 9.a: Crack pattern of the eastearn façade

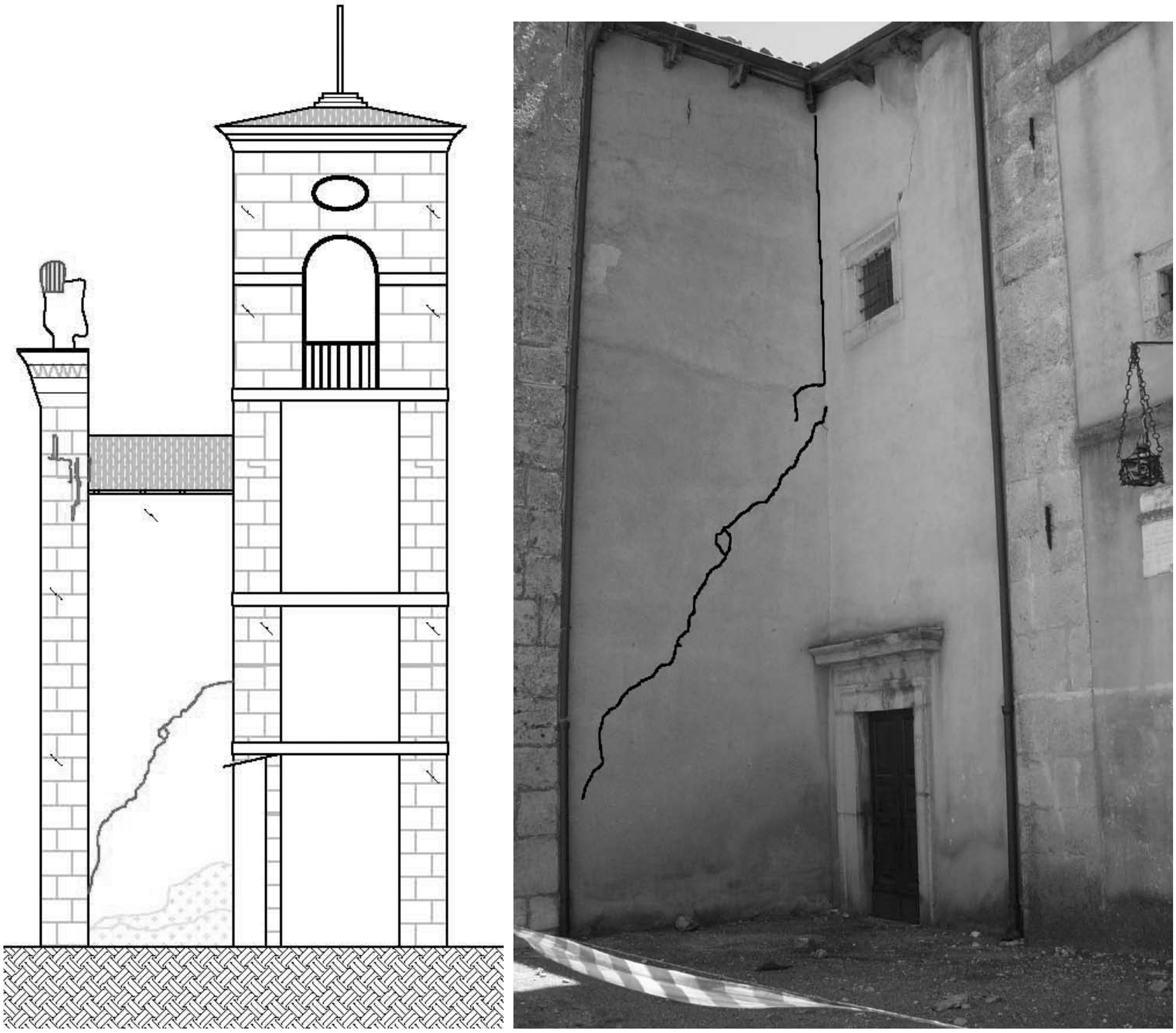


Figure 9.b: Crack pattern of the western façade

Internally, the vaulted structures exhibit a widespread damage, summarized in figure 10. The barrel vault of the west limb appears to be the most affected, with a considerable detachment of material in the neighborhood of the window. But also the central vault is characterized by considerable damage with cracks originating from the central ring and spreading toward the imposts of the vault itself. A significant crack occurred also at the connection with the lateral barrel vault (fig. 11).

2.7 Damage to painted walls

The plaster peeling off caused significant damage to the frescoes, as it can be seen from figures 11, 12 and 13, illustrating the damage to the vault (fig. 11), to the sixteenth-century fresco depicting the Our Lady of Hope (fig. 12) and the south vault (fig. 13).

2.8 Failure mechanisms

The failure mechanisms (see chapter 3) that have been activated by the earthquake are obviously influenced by the presence of adjacent buildings. In fact, the southern part of Church is constrained by The Our Lady of the Cross Institute, the western by a wing of the convent and by the bell tower, the eastern part by the covered walkway, connecting the ground floor of the sanctuary to the rectory.

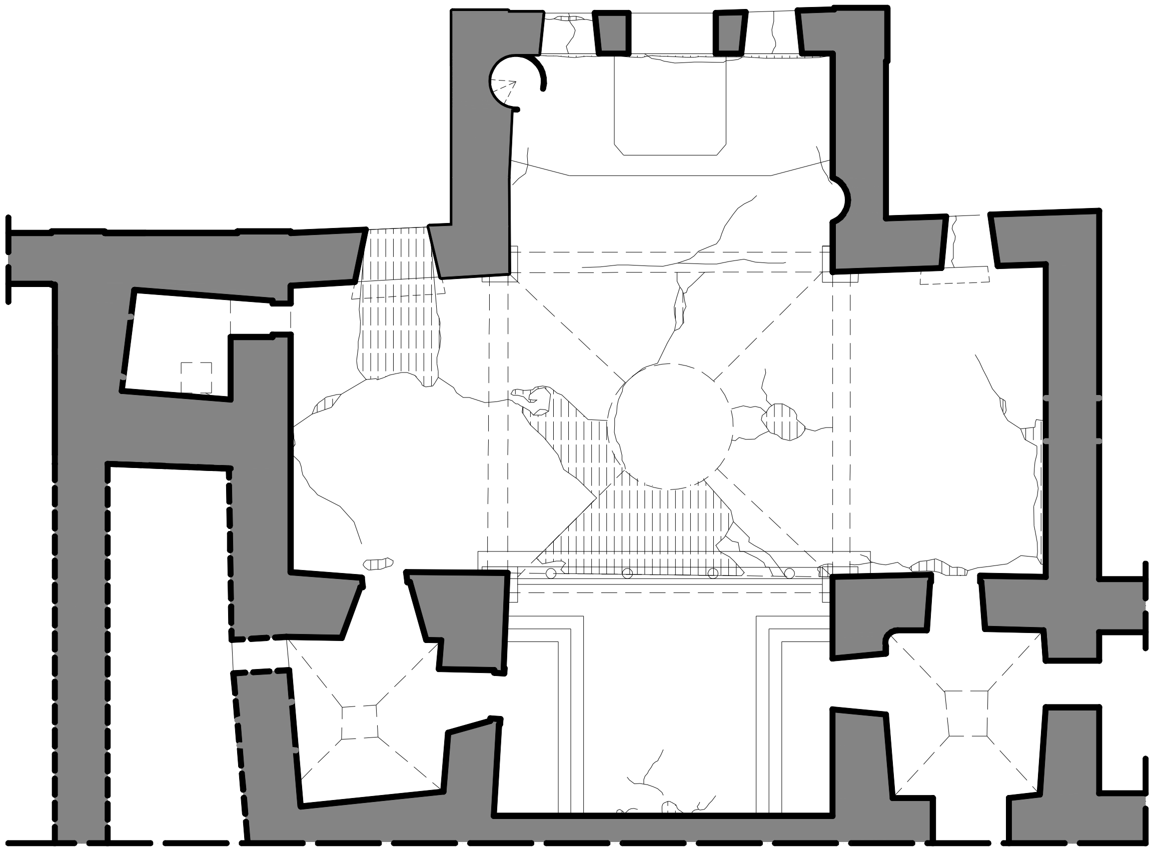


Figure 10: Crack pattern of the vaults

Figure 11. Damage of the central cross vault

Figure 12: Damage of the south limb barrel vault

Figure 13: Damage of the east limb barrel vault

These constraints, together with the presence of the vestry in the south-east and south-west corners, of the, make south and west side of the Sanctuary considerably stronger and stiffer than the remaining part, so that the activated mechanisms, identified analyzing the crack patterns, mainly concern free walls.

2.9 Façade failure mechanisms: tympanum overturning

Under seismic actions, the upper part of the façade (tympanum), which is higher than the main body of the Sanctuary, may overturns, rotating around a horizontal line located at the top of the church (type A mechanism - fig . 14), or located at the level of the upper chains (type B mechanism - fig 15). The type B mechanism can be explained because the poor clamping between the façade and the transverse walls, also due to the rounded shape of the stone elements and to their reduced size.

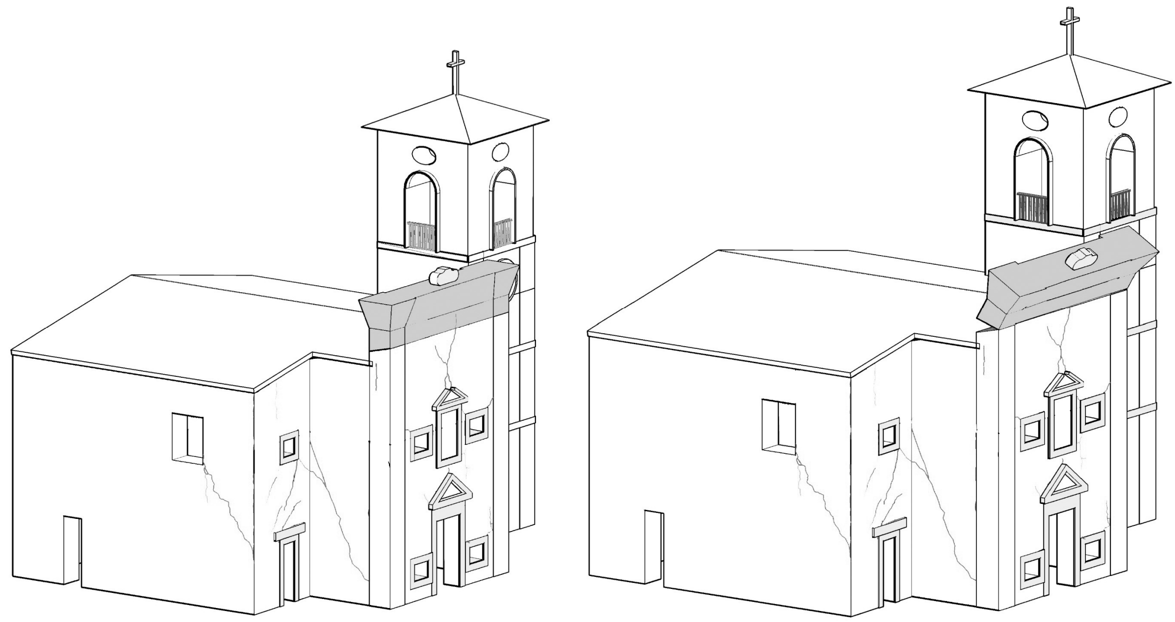


Figure 14: Type A mechanism – Overturning of the top of the tympanum

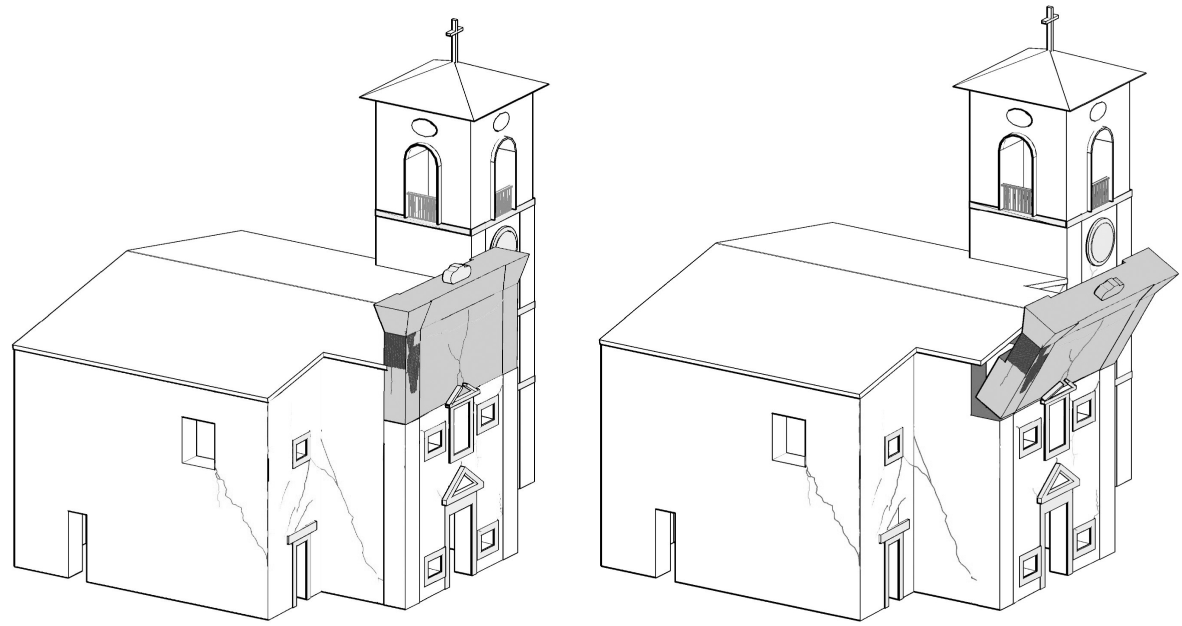


Figure 15: Type B mechanism – Overturning of the tympanum

The activation of these two mechanisms is clearly demonstrated by the crack pattern of the façade.

The steel chains seem to have only partially avoided the overturning of the façade, nevertheless they were not able to prevent the activation of a more general façade mechanism, indicated, in the following, as C mechanism, involving also a part of the side walls.

The presence of horizontal cracks lead to conclude that there was significant hammering between the roof and the main façade.

2.10 Overturning of the façade and the north-east corner

The diagonal shear cracks present on the external sides of the northern limb denounce the activation of the overturning mechanism of a wedge formed by the facade and by portions of the transverse walls (type C mechanism - fig 16).

Similar overturning mechanism is activated in the eastern limb, with rotation toward the north-east, around the vertex of a wedge formed by the upper parts of the corner (type D mechanism - fig 17).

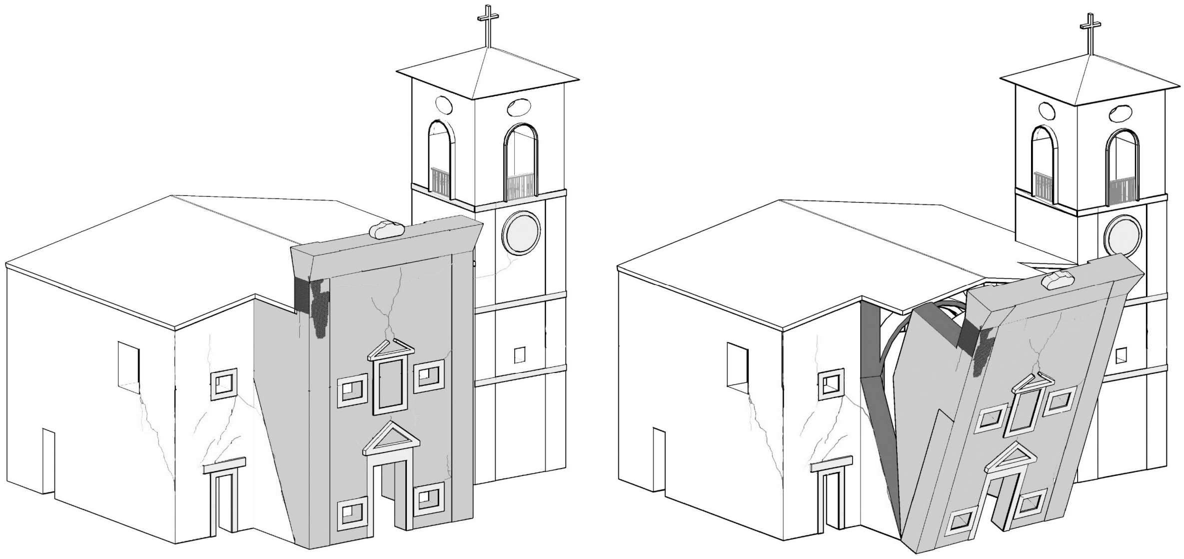


Figure 16. Type C mechanism – Overturning of the façade in the N-E direction

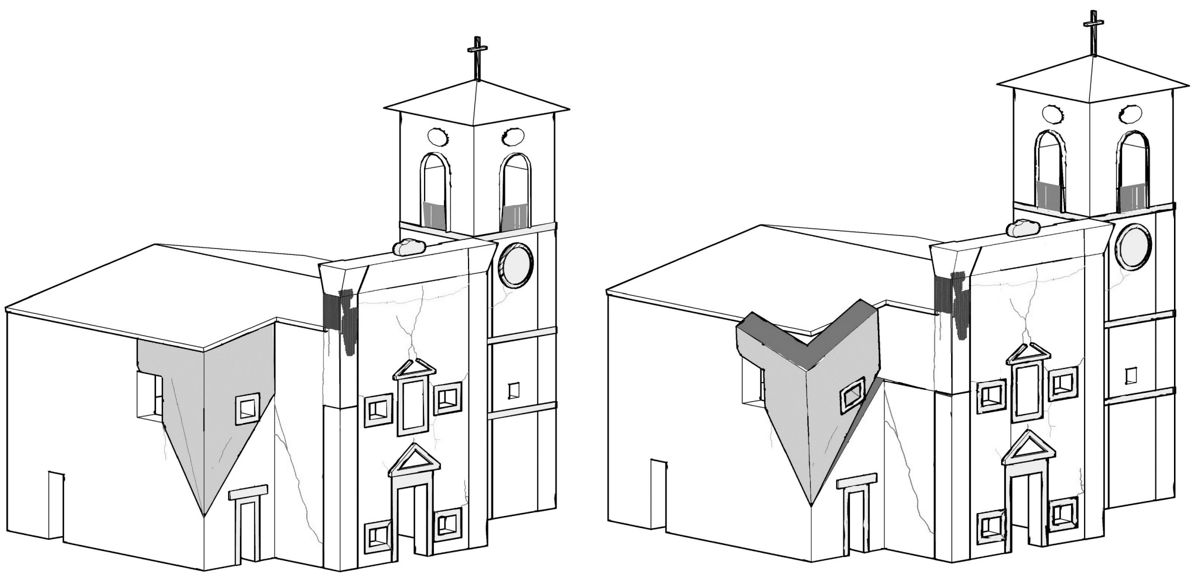


Figure 17: Type D mechanism – Overturning of the corner in the N-E direction

2.11 Failure of the central vault

Following the shift to the north-east of the north-eastern edge of the central vault, there was a lowering of the arch thrust, resulting in the disconnection of the vault itself from the façade with cracking of the vault ribs.

2.12 Evaluation of mechanism activation acceleration and rotation capacity

For each identified mechanisms it has been calculated the value of the horizontal acceleration able to activate the mechanism, while for type B mechanism it has been also studied the non-linear relationship between horizontal acceleration and rotation till the total collapse

2.13 Calculation of the acceleration for mechanism activation

To determine the acceleration needed to activate the mechanism, it was assumed zero tensile strength for the masonry and infinite compressive strength, according the kinematic mechanism theory. The effect of steel chains has been disregarded, considering the extremely small dimensions of their anchorages.

The following data were considered in the calculations:

- foundation plan 2 m below the ground level;
- weight of the roof: 0,8 kN/m^2 (referring to the horizontal projection of the surface);
- weight of the unit volume of the masonry walls: 19 kN/m^3;

The spectral accelerations needed for the activation of different mechanism are:

$$a_0^* = \frac{\alpha_0 \cdot P_{top}}{M^*} = 0{,}464g$$

for the type A mechanism;

$$a_0^* = \frac{\alpha_0 \cdot \left(P_{facciata} + P_{trave}\right)}{M^*} = 0{,}185g$$

for the type B mechanism;

$$a_0^* = \frac{\alpha_0 \cdot \left(P_{facciata} + P_{trave} + P_{lati}\right)}{M^*} = 0{,}159g$$

for the type C mechanism;

and, finally,

$$a_0^* = \frac{\alpha_0 \cdot \left(P_{sommità} + P_{lat}\right)}{M^*} = 0{,}356g$$

for the type D mechanism.

3 CASE STUDY N. 2: DYNAMIC CHARACTERIZATION OF THE BELL TOWER OF ST. NICCOLA CURCH IN PISA

Modern methodologies for monitoring and control of the structures described in chapter 3 are largely employed to appraise the safety of historical buildings in the context of preservation of historical heritage.

The study of masonry buildings dynamic identification techniques is generally very complex. In fact, because of the high structural stiffness of masonry structures, the study of their dynamic behaviour under periodic or casual excitations requires to consider that time histories are generally of limited amplitude and often contaminated by noise. For this reason the ability of recognition of the natural frequencies guaranteed by the different methods of signal analysis depends both on the adopted algorithm and on the nature of the dynamic excitation of the structure. It is therefore evident that a correct methodology of dynamic characterization asks for the use of different signal analysis techniques, even in combination. In this case study it is illustrated the numerical and experimental studies carried for the dynamic characterization of the bell tower of the church of St. Niccola in Pisa (fig. 18).

Figure 18: The bell tower of the church of St. Niccola in Pisa

Thanks to their relatively simple static scheme and to the predictability, at least in qualitative terms, of their dynamic answer, towers and bell towers are relevant examples to test the possibilities of modal identification techniques.

Looking more deeply into the specific case studies, it results that the use of the classical methods of analysis, based on the fast Fourier transform (FFT) can be difficult due to stress concentrations induced by the inclination, by the presence of openings and by the the variations of the cross section, so that a more modern operational methodology of dynamic identification of the structure has been setup, based on the wavelet transforms [13] [14], integrated by sophisticated FEM investigations.

The proposed combined methodology, corroborated by the experimental results, allows not only to verify the reliability and the field of application of different signal analysis technique, but also to calibrate the values of the main geometrical, physical and mechanical parameters governing the dynamic behaviour of the structure.

3.1 The bell tower of S. Niccola in Pisa

The bell tower of the church of St. Niccola to Pisa, considered a masterpiece of the Romanic-pisan style, it is a construction of notable beauty and great historical interest, characterized by a particularly complex architectural structure. The shape of the cross section, in fact, change from circular, to octagonal in the higher orders and to hexagonal in correspondence of the bell cell, whose roof is a hexagonal pyramid.

The bell tower, that raises from the left of the façade of the homonym church, is well preserved and doesn't show evident cracks. Due to soil plasticity, it shows a settlement of about 1,0 m, and a slope of 1°13' with respect to the vertical (fig. 19).

The dating and the attribution of the bell tower are not certain and different hypotheses have been made: Ragghianti attributes it to Diotisalvi around 1170; Nannicini and Testi-Cristiani attests it between 1230 and 1250, while Frey hypothesizes two or three erection phases, going up from 1173 till to 1230-1250.

The bell tower, originally isolated, was added to the adjacent convent of the Augustinian friars around 1295.

Vasari attributes the erection of the tower to Nicola Pisano, furnishing an accurate and eulogistic description of it, saying that "*Niccola erected many other buildings and churches in Pisa, and it was the first one that use wood pile foundations to reduce the settlements, like in the church of St. Michael in Borgo. But the most beautiful, the most prestigious and more capricious architecture that Niccola ever made, was the bell tower of St. Nicola of Pisa, where Augustian friars are: he is octagonal outside and round inside, with a spiral staircase inside the wall which is open on the inner part and sustained by columns placed around. This kind of capricious invention was also drawn subsequently, with more relevant dimensions and decorations by Bramante in the Belvedere, for pope Giulio II; and by Antonio da Sangallo, in the St. Patrice well in Orviet, for pope Clemente VII.*"

As recalled by Vassari, internally in the bell tower it is present a spiral stone staircase consisting of a helical gallery of elegant rampant arches sustained by mullions (figs. 20 and 21).

3.2 Dynamic tests

In the framework of the experimental campaign for the dynamic identification of the bell tower, a wide series of dynamic tests have been performed, adopting three different exciting functions: namely, an environmental excitation, provided by the normal road traffic flow, a sinusoidal one, caused by the motion of the bells and an impulsive one, provoked by the impact on the road surface of the street of the the wheels of a calibrated lorry transiting on a concentrated step, 10 cm in height, placed on the roadway, perpendicularly to the road axis of St. Maria street (fig. 22).

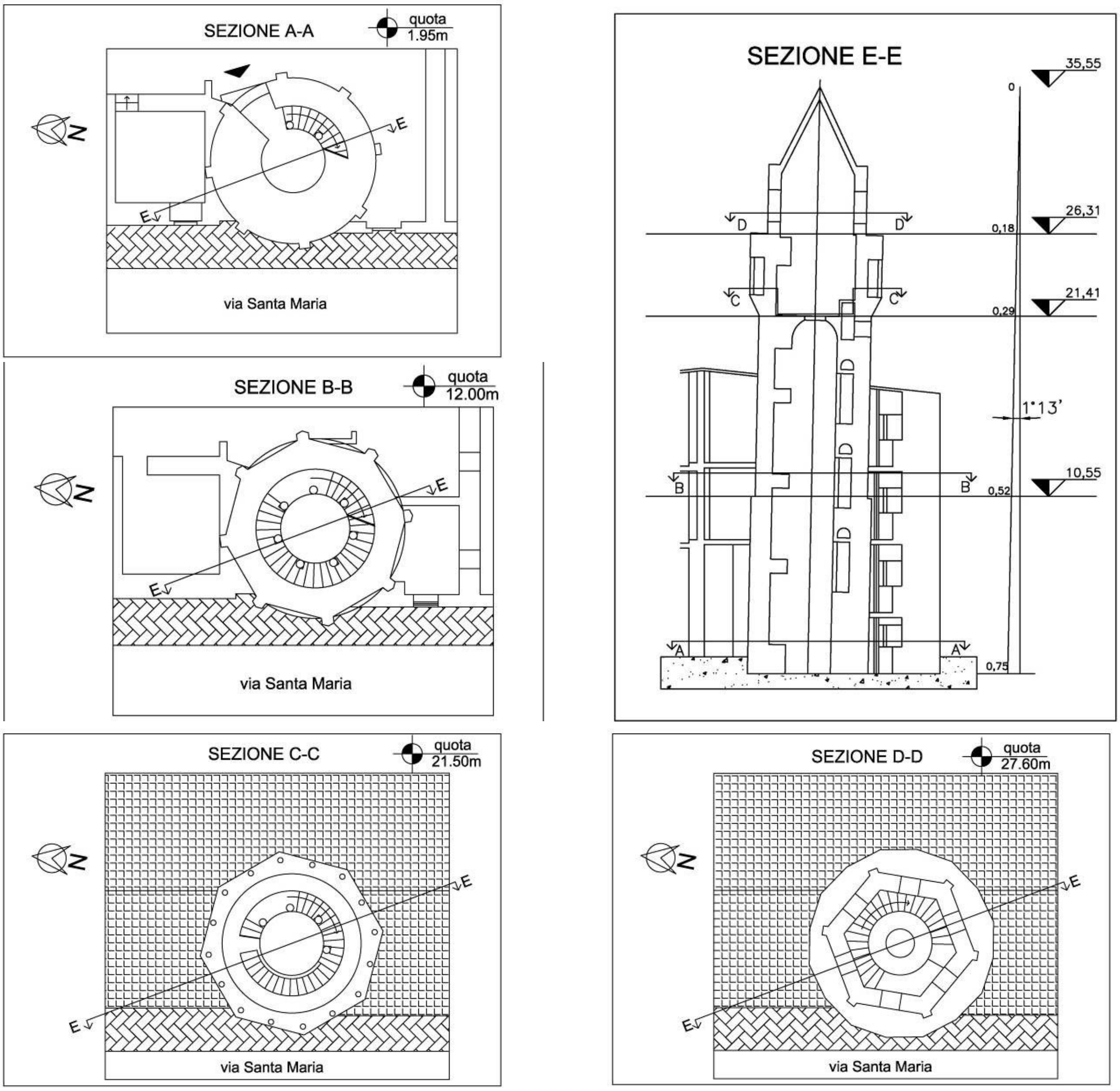


Figure 19: Cross sections of the bell tower - to) circular, b) octagonal, c) hexagonal

Figure 20: Helical stairs seen from below

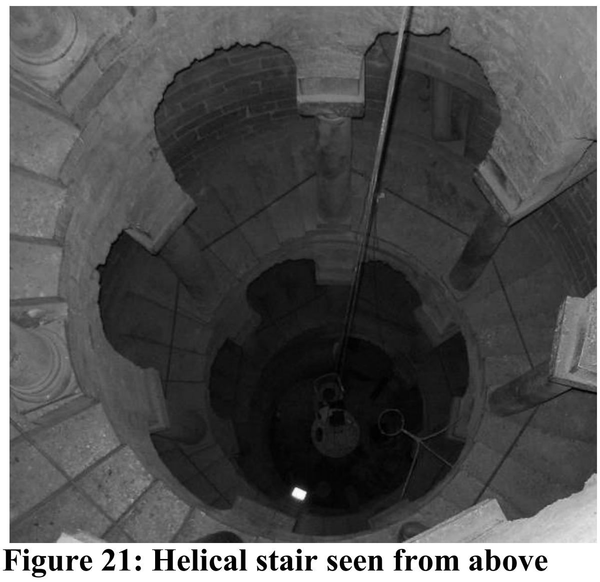


Figure 21: Helical stair seen from above

Figure 22: Transit of the calibrated lorry on the 10 cm height step to induce impulsive excitation

It is important to notice that, since the interaction with the walls of the convent in which the tower is incorporated modifies the rigidity and the dynamic answer of the bell tower, compared with the isolated structure case, test results are complex and the interpretation of the results more difficult than for the isolated tower.

During the experimental campaign, 41 tests have been performed in total: 20 with environmental excitation, 10 with sinusoidal excitation and 11 with impulsive excitation.

In each test, the time histories have been recorded at different heights, both in terms of displacements and accelerations. The measuring apparatus consisted of two seismometers and six accelerometers, whose positions and orientations were suitably modified to obtain additional data.

The position of the instrumentation is sketched out in figure 23.

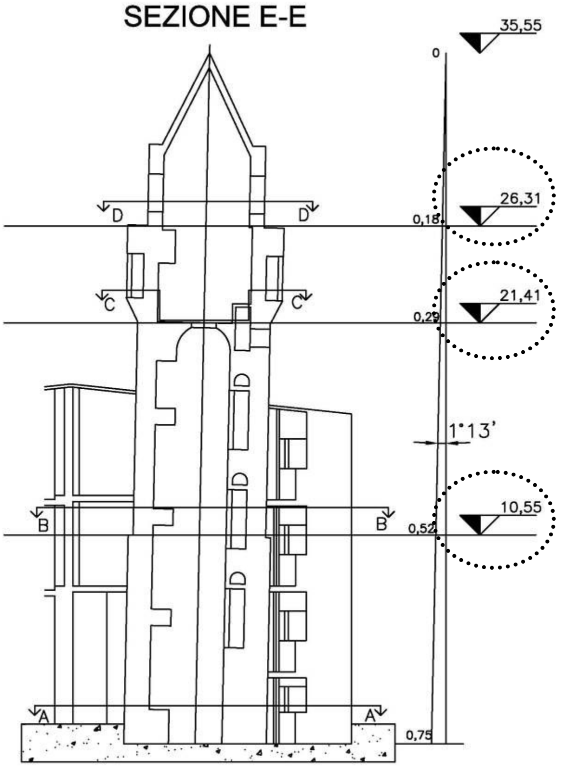
3.3 Signal analysis

As just said, the signals experimentally acquired can be analysed using different techniques, but the techniques commonly used in the structural field are those classical, based on the fast Fourier transform.

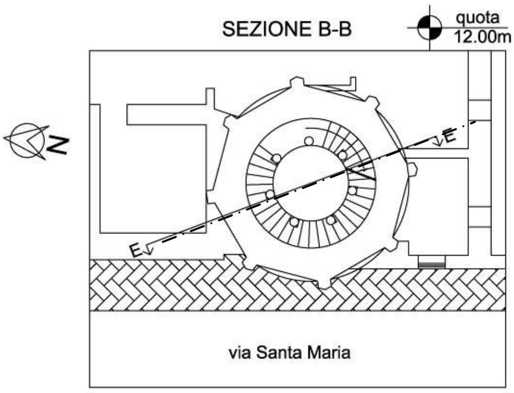
The wavelet transform, which is commonly used in different physical fields, like analysis of climatic data, financial indexes, cardiac monitoring, statistical fluctuations in turbulent motions, characterization of fracture surfaces, image compression and so on, is an alternative technique for dynamic identification of structures.

The wavelet analysis uses compact (*let*) oscillatory functions (*wave*), deriving their name from the form of a fundamental function Ψ, the "mother" wavelet, used to build such "children" functions (usually referred to as atoms).

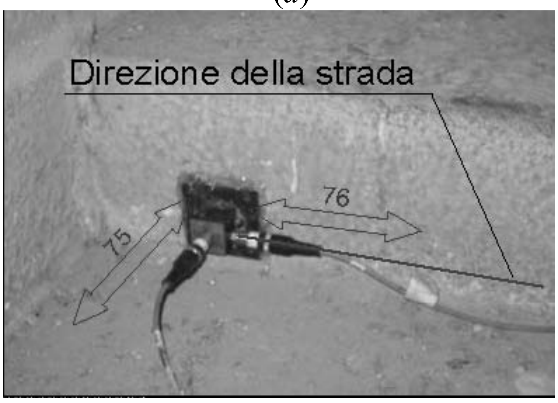
Unlike other basis functions, the wavelets are produced by translation and expansion of a single function, called mother wavelet (fig. 24). That is to say wavelets are basis functions, of real or complex variable, generated by translation and expansion of the mother wavelet, Ψ, which is a regular function, defined on a limited interval and equal to zero outside the interval, characterised by null average and finite energy [13]. Wavelet transform associates to a function defined in a short time interval, a function defined in an analogously short interval in the transformed domain while Fourier transforms associates to a finite temporal impulse an infinite spectrum in the frequency domain. These mathematical properties make wavelets particularly effective in the analysis of non periodic, intermittent, transitory or noisy signals.



(*a*)



(*b*)



(*c*)

(*d*)

Figure 23: a) scheme of the measurement apparatus. b) directions of measurements c) accelerometers d) seismometers

In analogy with Fourier transforms, we can distinguish continuous (CWT) and discrete (DWT) wavelet transform.

The continuous wavelet transform (CWT) of a function $f(t)$ is defined in eq. 1 in which a is a scale parameter, b represents the translation on the time axis, $w(a)$ a normalization function assuring that the wavelets have the same energy for each value of the scale parameter and the star means the complex conjugate . The meaning of scale parameter is analogous to the cartographic scale: high values of the parameter give global information on the signal, associated with low frequencies, while small values give local information on the signal, associated with the highest frequencies.

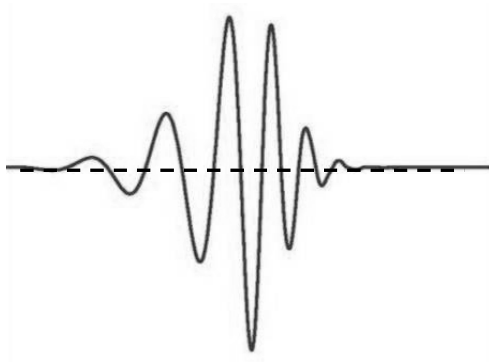


Figure 24: Mother wavelet

$$T(a,b) = w(a)\int_{-\infty}^{+\infty} f(t)\psi^{*}\left(\frac{t-b}{a}\right)dt \tag{1}$$

Gabor wavelet is considered in the present work (figg. 25 and 26). Gabor wavelet is a complex valued function obtained by modulating a Gaussian window. Its analytical expression, in time domain, is given in eq. 2.

$$\psi(t) = e^{i\eta t} \cdot \frac{1}{(\pi\sigma^2)^{\frac{1}{4}}} \cdot e^{-\frac{t^2}{2\sigma^2}} \tag{2}$$

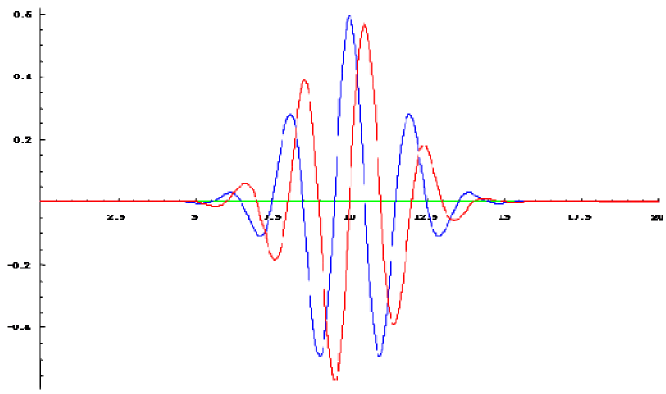


Figure 25: Gabor Mother Wavelet (σ=1,6, η=π), real and complex part

where η is the modulation frequency, σ is the standard deviation of the Gaussian window and t is time. It is $\|\psi\|^2 = 1$.

More about the properties of this function can be found in the paper of Simonowski and Boltežar [15], while Slavič and al. [16] deal with the use of Gabor wavelet transform to extract resonance frequencies and damping coefficients.

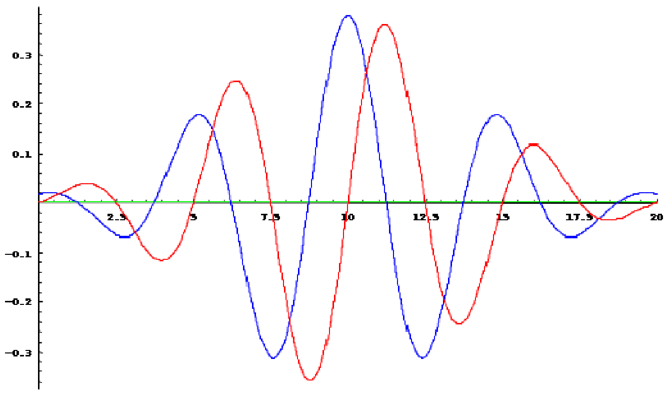


Figure 26: Gabor atom (a=2,5), real and complex part

3.4 Analysis of experimental data

Because of the high stiffness of the tower bell, the recorded signals are characterized by small amplitude and low signal-noise ratio, as underlined in figure 27, where the time is in second and the displacements in mm.

In figures 28, 29 and 30 are represented, for instance, the time histories induced by environmental, sinusoidal and impulsive excitations, respectively, and the results of the analyses performed using the FFT transform (case a), a triangular windowing of the FFT transform (case b), or the wavelet transform (case c).

The examination of the data underlines that:

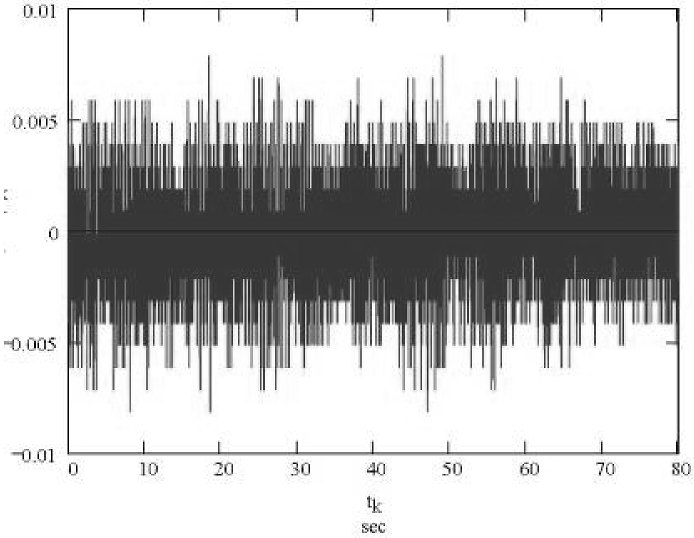
- the Fourier analysis of this type of signals, weak and strongly contaminated by noise, allows to obtain satisfactory results only in case of sinusoidal excitation, when wavelet transform result less satisfactory;
- on the contrary, when the source of excitation is environmental or impulsive, the identification capability of the FFT reduces considerably, whereas the wavelet transform is particularly efficient;
- the efficiency of the Fourier transform doesn't improve resorting to windowing techniques, like the triangular one adopted here.

Concerning the structural response signals taken into account, the analysis demonstrated that in case of impulsive excitation acceleration time histories give more satisfactory and more clearly interpretable results than displacement time histories, while in case of periodic excitation displacement time histories seem to be preferable.

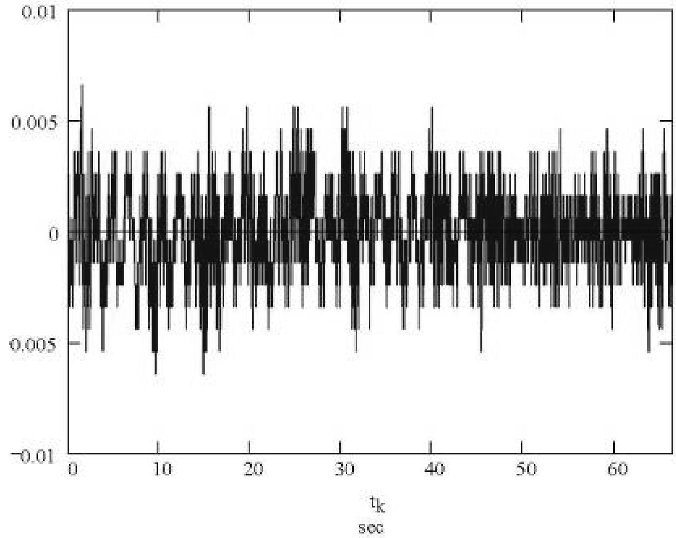
In table 1 the natural frequencies deduced from the analysis of recorded data are summarized. Data are grouped on the basis of the type of excitation, in order to underline its ability to excite the natural frequencies of the bell tower.

Aiming to eliminate spurious frequencies only natural frequencies recorded in more than 30% of the tests have been retained.

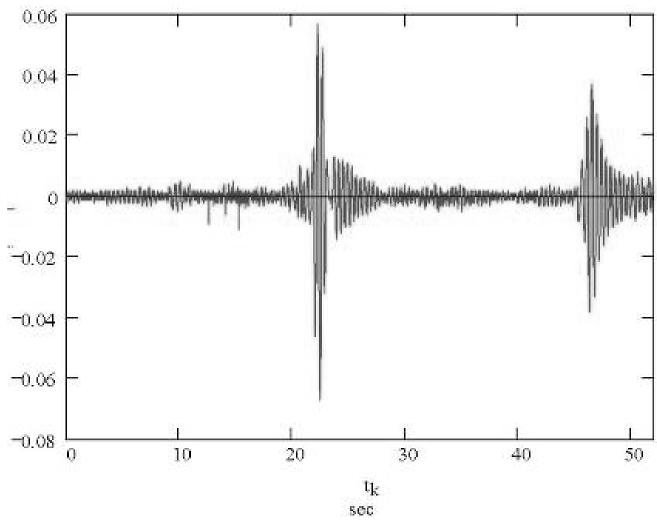
The results in table 1 show that the first four mode shapes are primarily flexural: the second mode shape is contained in the plane parallel to St. Maria street, while the others three are in the plan perpendicular to the road axle. This is not surprising because the stiffness of the bell tower is sensibly different in the above mentioned plans, because of the interaction of the bell tower itself with the walls of the Augustian friars convent.



a) Environmental excitation

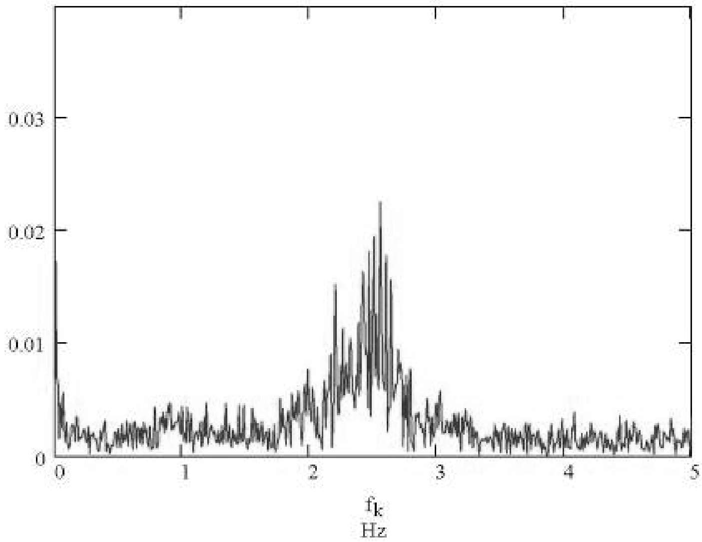


b) Sinusoidal excitation

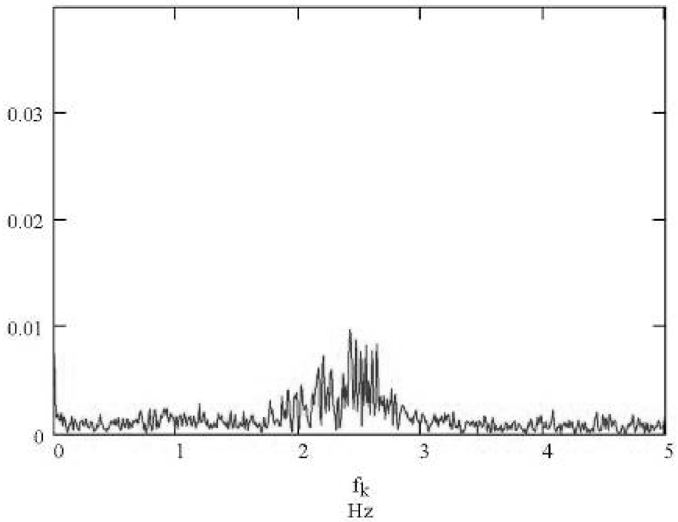


c) Impulsive excitation

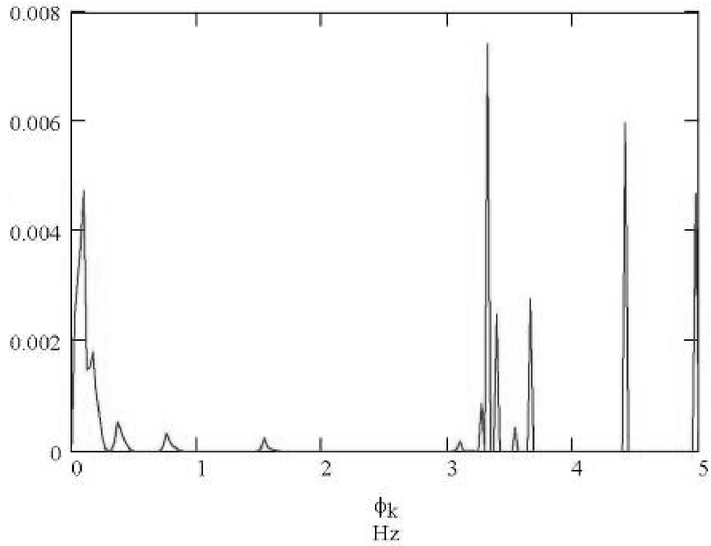
Figure 27: Time histories recorded during the tests



a) Fourier transform

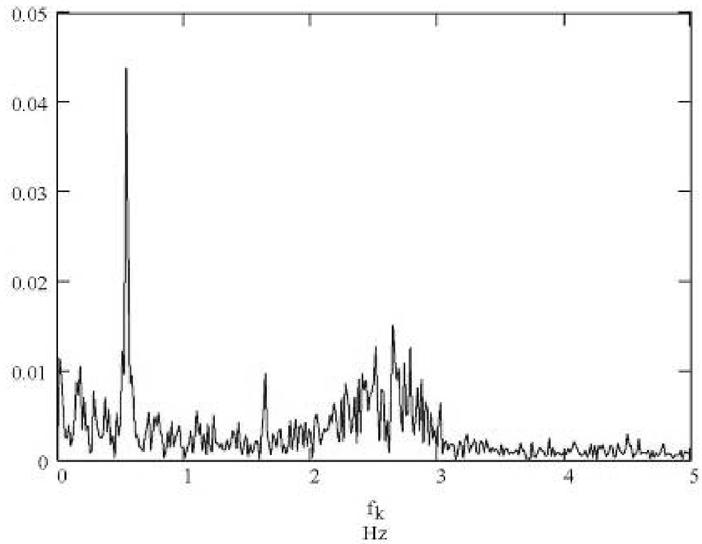


b) Fourier transform with triangular window

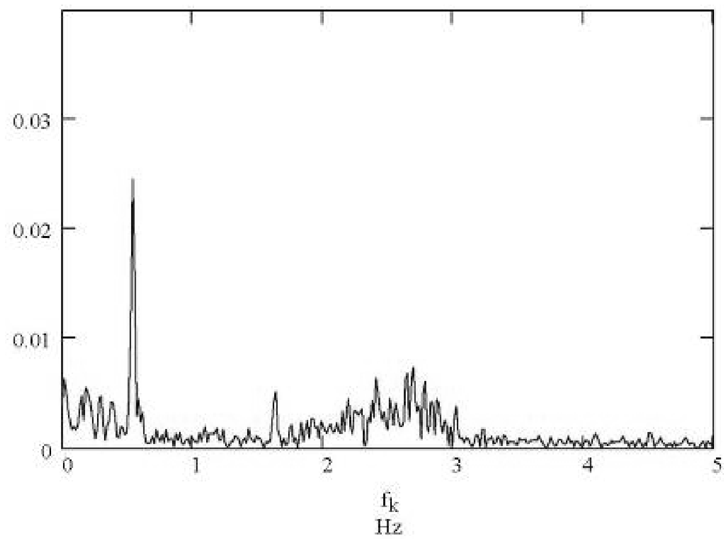


c) Wavelet transform

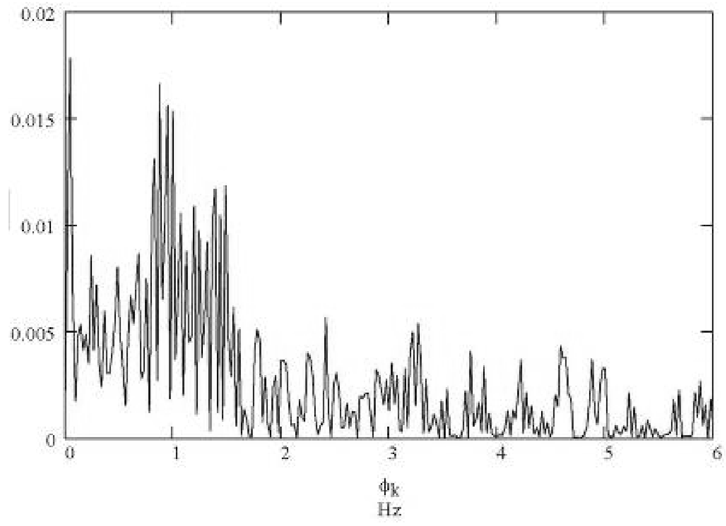
Figure 28: Environmental excitation



a) Fourier transform

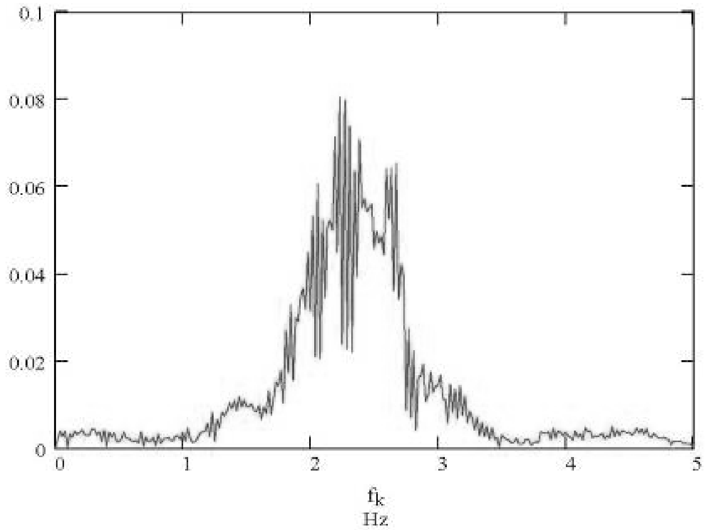


b) Fourier transform with triangular window

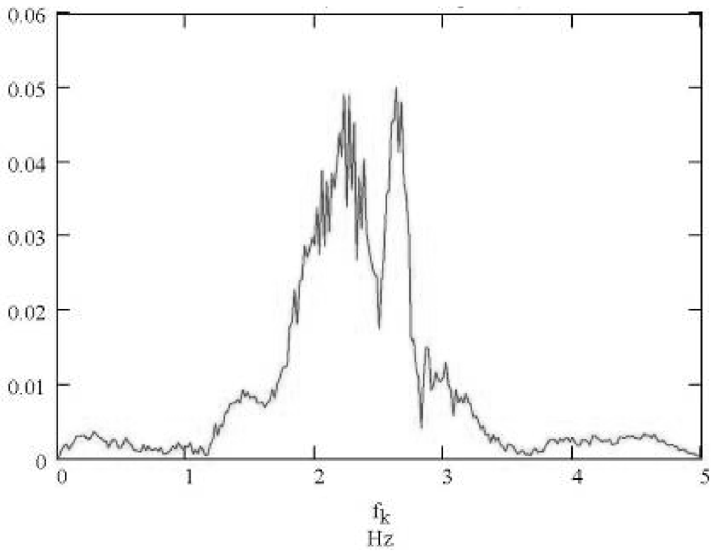


c) Wavelet transform

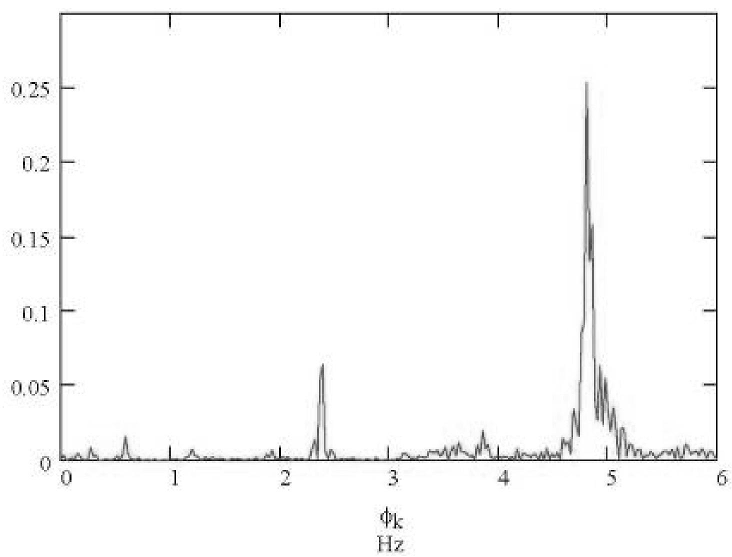
Figure 29: Sinusoidal excitation



a) Fourier transform



b) Fourier transform with triangular window



c) Wavelet transform

Figure 30: Impulsive excitation

Table 1: Natural frequencies and shapes experimentallydetected

Measured (Hz) frequencies				Direction of vibration
Mode nr.	*Environmental excitation*	*Sinusoidal Excitation*	*Impulsive excitation*	
1	/	0,545	0,561	*perpendicular to the road axle of St. Maria street*
2	/	1,633	1,834	*parallel to the road axle of street S. Maria*
3	3,198	/	/	*perpendicular to the road axle of street S. Maria*
4	3,338	/	3,32	*perpendicular to the road axle of street S. Maria*

The comparison of the results underlines that, as it was reasonable to wait him, the forcing environmental doesn't have enough intensity to excite the first proper ways, correspondents to the frequency natural lower, confirming, besides, that the based methods on you transform her wavelet they are more effective than the based methods on transforms her/it of Fourier, because they allow to also individualize the frequencies of the superior ways.

3.5 Numerical analysis and comparison of the results

In a subsequent phase, the static and dynamic behaviour of the bell tower has been numerically studied with a refined finite element analysis, performed using the COSMOS/M FE software.

The finite element model, that is constituted by around 160000 8-node solid elements (SOLID elements of the COSMOS library), is very accurate and it reproduces accurately all the details of the bell tower, in particular the present openings and the inside helical staircase, as well as the soil-foundation interaction as well as the walls of the adjacent convent (fig. 31).

Since it was not possible to perform direct tests in situ, the mechanical properties of the bell tower materials have been initially derived from the existing literature regarding similar buildings, and they have been subsequently refined on the basis of the recorded data, in order to fit the mode shapes and the natural frequencies reported in table 1, arriving finally to the mechanical properties reported in table 2.

Table 2: Mechanical characteristics of builiding materials adopted in FE analysis

Material	Elastic modulus [MPa]	Specific weight [kN/m^3]	Elements
Conglomerate	25000	24,0	*Foundation*
Conglomerate	30000	24,0	*Floor at the colonnade leve*
Stone	50000	27,0	*Outer and inner face of the masonry, bell cell and staircase*
Filling	5000	14,0	*Infilling of the masonry*
Masonry	10000	18,0	*Pyramidal roof*

The numerical results obtained via the FE model are compared with the experimental ones, obtained analysing the responses under environmental, periodic, and impulsive excitation, respectively, in table 3.

In table 3 the relative errors

$$E_{\omega_i} = \frac{\left|\omega_{num_i} - \omega_{sp_i}\right|}{\omega_{num_i}} \cdot 100 \tag{3}$$

are also reported, being ω_{num} and ω_{sp} the natural pulsations determined with the FE model and experimentally, respectively, which are generally less than 5%.

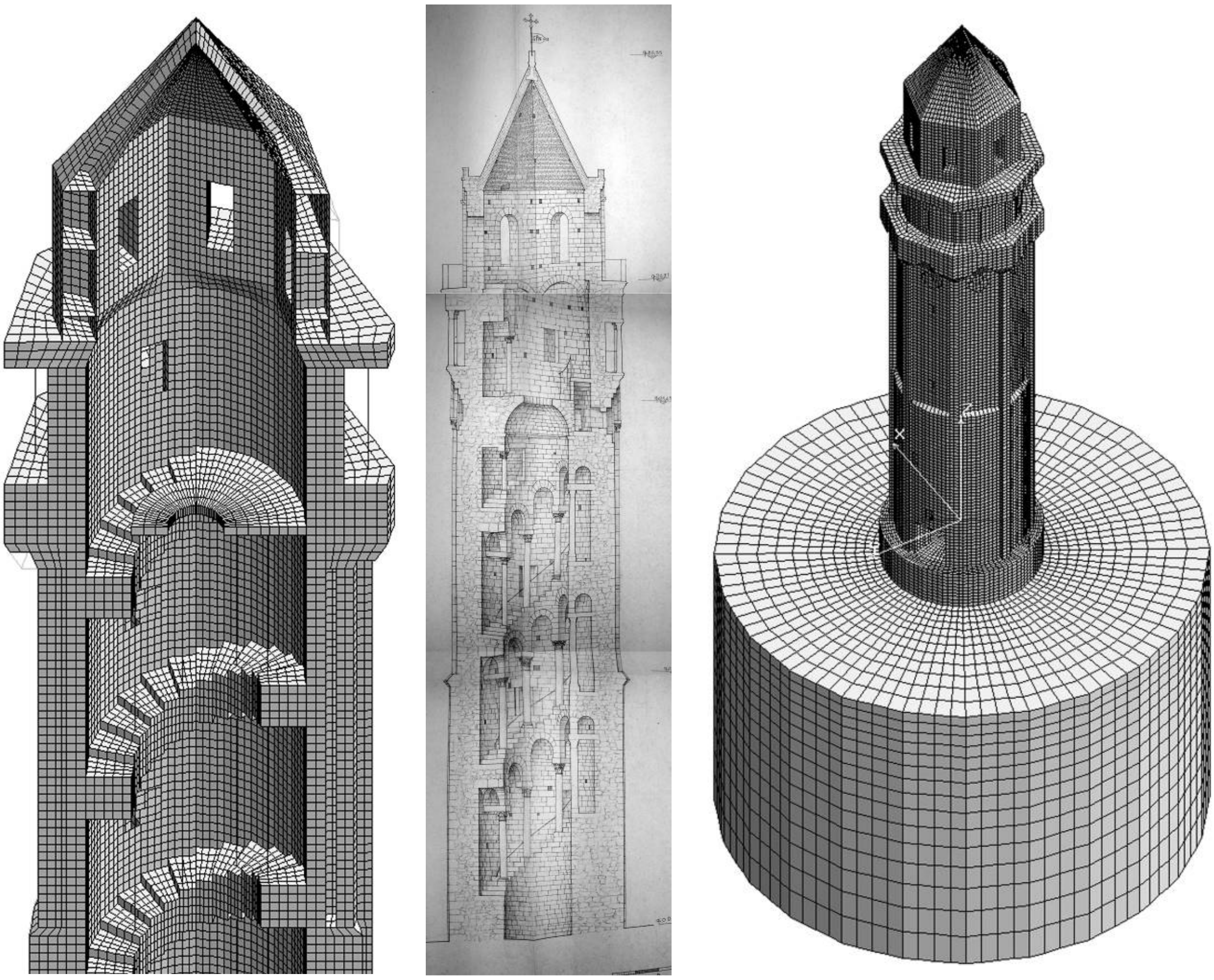


Figure 31: The finite element model compared with the section of the bell tower

Table 3: Comparison between measured and numerical natural frequencies of the bell tower

Mode nr.	Natural frequency [Hz]				E_{env} *(%)*	E_p *(%)*	E_{imp} *(%)*
	Environmental	*Periodic*	*Impulsive*	*FE model*			
1	/	0,545	0,561	0,544		0,18	3,13
2	/	1,633	1,834	1,903		14,19	3,63
3	3,198	/	/	3,067	4,27		
4	3,338	/	3,32	3,141	4,17		4,74

The satisfactory agreement between theoretical and experimental data confirms the correctness of the dynamic identification methodology adopted, stressing the importance of the data refinement, based on the experimental results.

3.6 Concluding remarks and future developments

The results confirm that the dynamic identification capability of different methods depends on the type of excitation, so that an effective technique of dynamic characterization should foresee the combined adoption of methods based on the Fourier transform and methods based on wavelet transform, integrated by refined finite element models, where the material properties that govern the dynamic behaviour of the structure are calibrated according experimental results.

In effect, in dynamic identification, Fourier transform is very effective when the excitation is periodic and the structural answer is not contaminated by noise, while, in case of weak or contaminated signals the modal identification ability of the wavelet transform is sensibly superior.

Although some applications are already known in the field of the dynamic identification of bridges in c.a. [17] [18], the high sensibility to the low frequencies, the ability of demodulation, the accuracy in the manipulation of the transient and non-periodic signals make the wavelet transform an interesting tool for dynamic identification of very stiff structures and particularly of historical masonry constructions.

Future developments should aim to widen the field of application of the proposed combined technique, also referring to possible applications on more complex structures, like masonry bridges as described in the following.

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4 CASE STUDY N. 3: THE VARA VIADUCT IN CARRARA

4.1 The Vara Viaduct in Carrara and its history

The Vara Viaduct (fig.32) was built inside the marble caves close to Carrara between 1887 and 1890 and it became soon a symbol for its region.

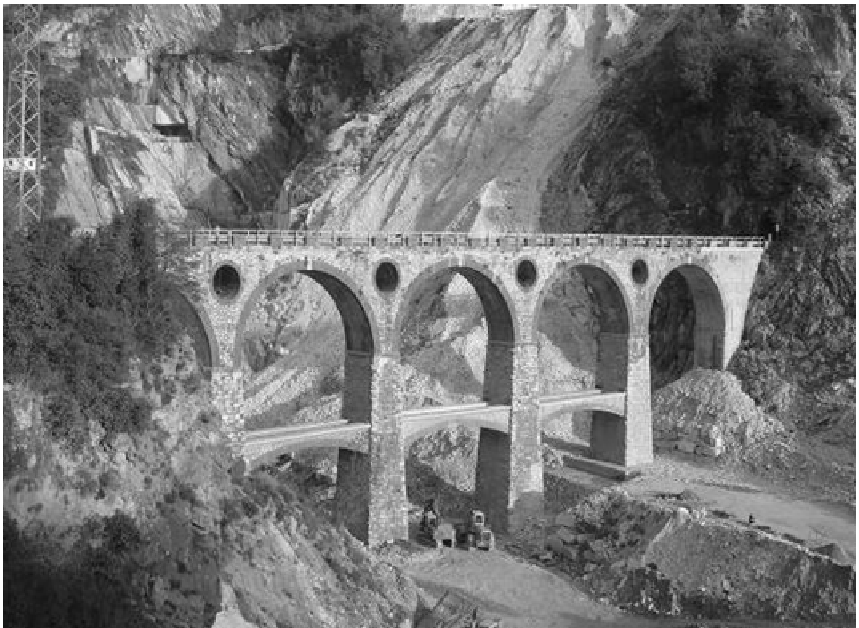

Figure 32: The Vara viaduct

The viaduct is a five spans masonry arch bridge. Each arch spans about 16m for a global length of about 100m. The deck is 40m high over the ground; it curves and it slopes about 7%. The lower arch in the middle spans were built in 1932 during the restoration works due to the failure of the fourth pier for replacing the temporary wooden props (see fig. 33) [19].

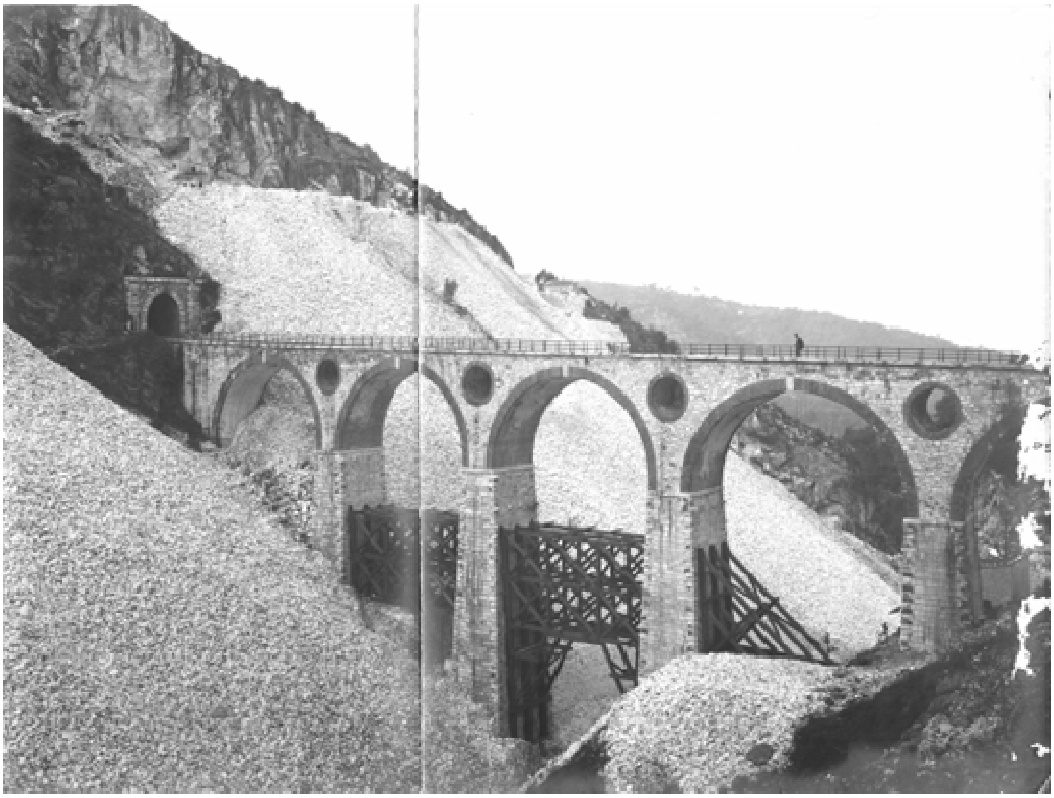

Figure 33: The pier propped up after the failure (1913) *(Historical Record Office Carrara)*

It was originally a railway bridge but it was changed to a road bridge about in 1960.

In recent years the fourth pier failed again and it was necessary to close the bridge to traffic for allowing new restoration works. For evaluating its seismic safety a finite element model was developed and the modal analysis was performed.

Due to the structural complexity and the uncertainties about the mechanical properties of the material used for the construction, again an experimental validation of the numerical results was necessary, so that dynamic identification tests described in the next paragraph were performed.

4.2 Experimental test setup

Two sets of experimental dynamic tests were performed, the first before the bridge was strengthened (January 2002), the second after the strengthening has been carried out (October 2006).

The proof were outlined both as shown in fig. 34, so that a meaningful comparison of the results was possible.

Dynamic loads were produced by a weighted lorry driving on a transversal step and impacting on the deck, see figure 35.

The truck axis weighted 71,0 kN (the front one) and 186,8 kN (the rear one). The geometrical features of the transversal step are shown in figure 36.

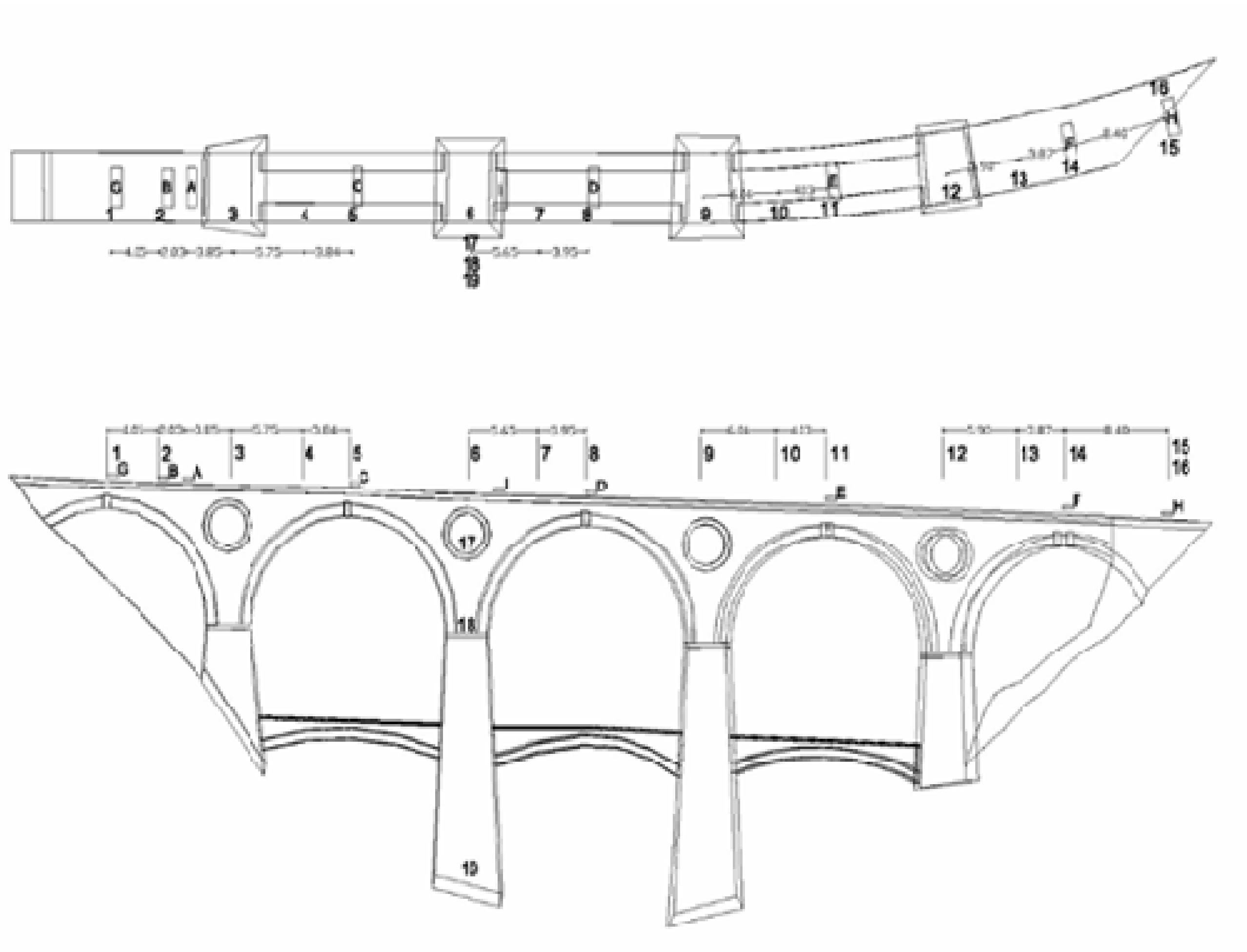

Figure 34: Outline of instruments location on the bridge (letters denote the step position, numbers the accelerometers positions).

Figure 35: The lorry "launched" for the jump

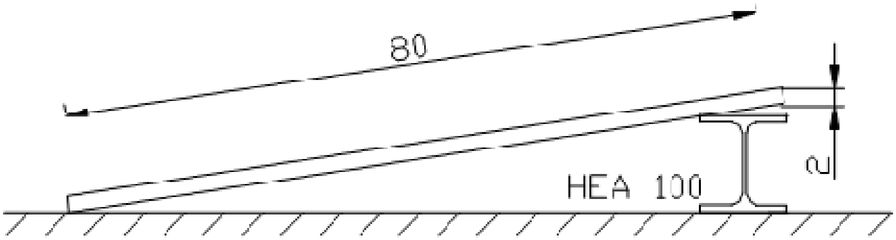

Figure 36: Step geometrical features

During the experimental tests the bridge was dynamically excited in different positions, moving the step as shown in figure 34 (positions denoted by letters A to I). For each test the time histories were recorded simultaneously in three sections (sections 1 to 19 in fig. 35) in three directions (vertical (V), transversal (T) and longitudinal (L) referred to the roadway axis).

4.3 Numerical analysis and experimental results

The bridge was studied via a sophisticated finite element model realized with Cosmos/m v. 2.7 software.

The model, shown in figure 37 is made up of 13160 nodes and 8895 solid elements.

The first six modes of vibrations were calculated together with their respective modal participating factor: figure 38 shows as an example the second mode shape of the viaduct.

The numerical model used for the studies carried out before the strengthening was validated with experimental results achieved after the first experimental tests.

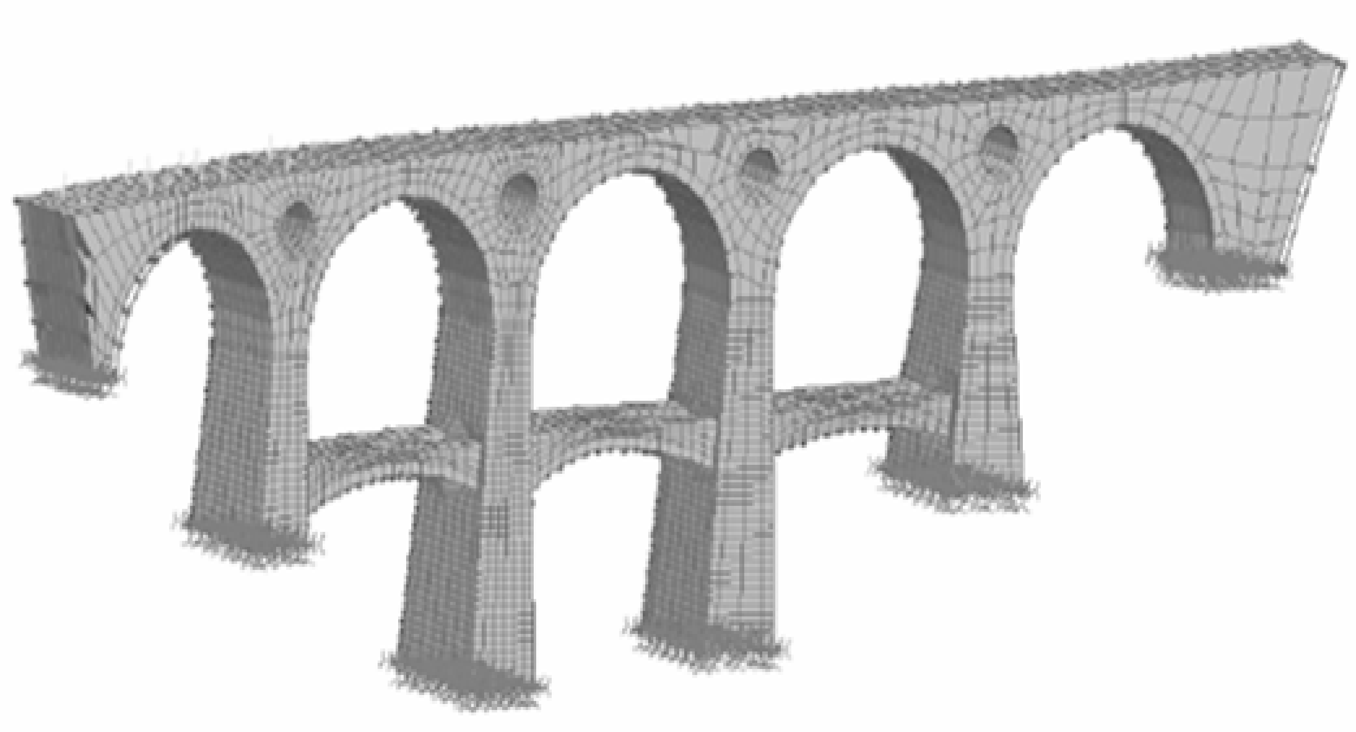

Figure 37: The finite element model of the bridge

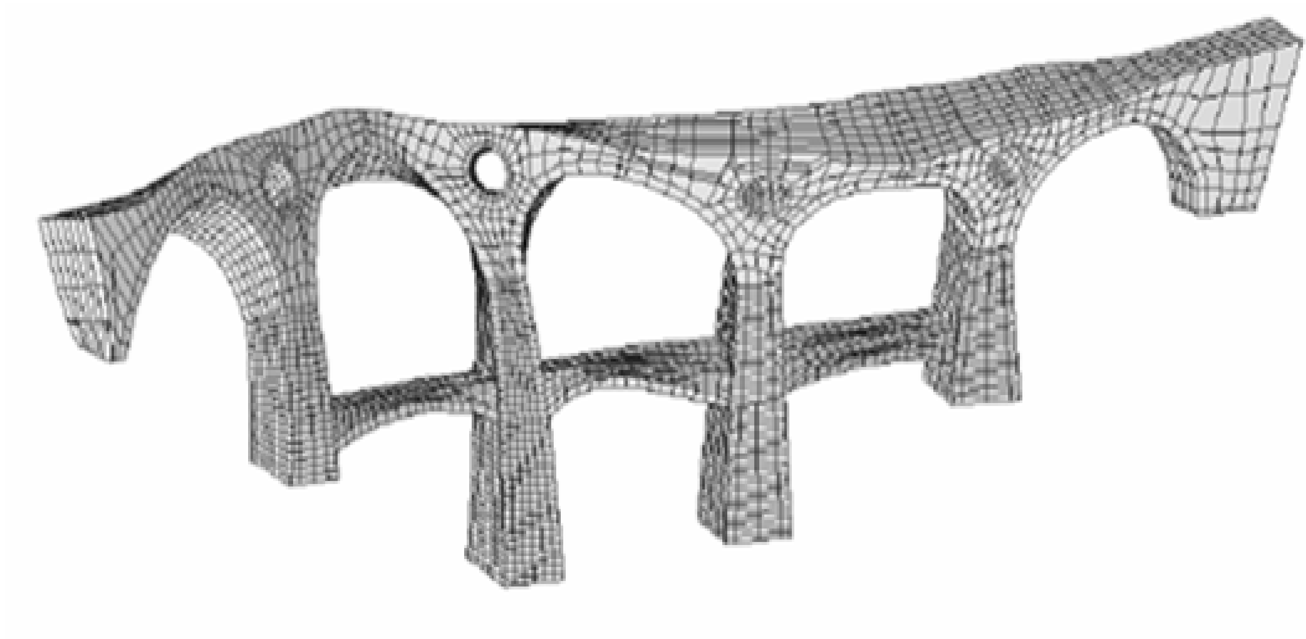

Figure 38: The second mode shape of the viaduct

The natural frequencies and the damping coefficients experimentally derived on the strengthened bridge are listed in table 4 together with the experimental and numerical results obtained in year 2002, before the bridge strengthening.

The comparison highlights a slight difference in modal shapes (modes 3 and 4) and natural frequencies. This results can be interpreted as the stiffening effect of the retrofitting on the structure. The strong variation in the damping values needs a more careful discussion.

Table 4: Comparison between experimental (2002 and 2006 tests) and numerical results

Mode nr.	2002			2006			FEM 2002	
	dir	f [Hz]	ξ (%)	dir	f [Hz]	ξ (%)	dir	f [Hz]
1	/	/	/	T	2,45	0,98	/	/
2	T	3,55	16,60	T	3,65	1,14	T	3,53
3	L	4,88	17,82	T	5,41	1,48	L	4,75
4	T	5,51	16,80	L	6,17	1,64	T	5,54
5	T	5,92	14,83	--	------	------	T	5,86
6	L	6,54	14,76	--	------	------	L	6,26
7	T	6,97	16,66	--	------	------	T	7,16

The high values detected in 2002 can be explained by the level of damage of the bridge, but the very low values detected during the tests on the strengthened bridge cannot be considered as representative of global structural damping, which reasonably ranges, for this kind of structures, in the interval 5% to 10%.

Plotting the envelope curve of a 1% damped signal on a registered single frequency time history (see fig. 39) a good matching is observed, so demonstrating that the global bridge response is decaying very fast, and that the sensor registered a local response (that is lasting for a longer time).

The local response is further and better highlighted in figures 40 and 41.

Frequency value increases with time while damping value decreases with time. That mean the frequency of motion decreases while the signal amplitude (e.g. the signal energy) increases.

The frequency-energy dependence of free oscillations is a typical dynamical feature of nonlinear oscillations with softening characteristics.

The nonlinear behaviour is also confirmed by the damping-energy dependence. Fig. 41 confirms that modal viscous equivalent damping assumption is not necessarily the most appropriate representation of the physical dissipation phenomenon, in fact, when dry friction effects (bricks sliding with respect to each other) or hysteretic behaviour are not negligible, structural energy dissipation is essentially a nonlinear phenomenon (see, for example, [20] and [21]).

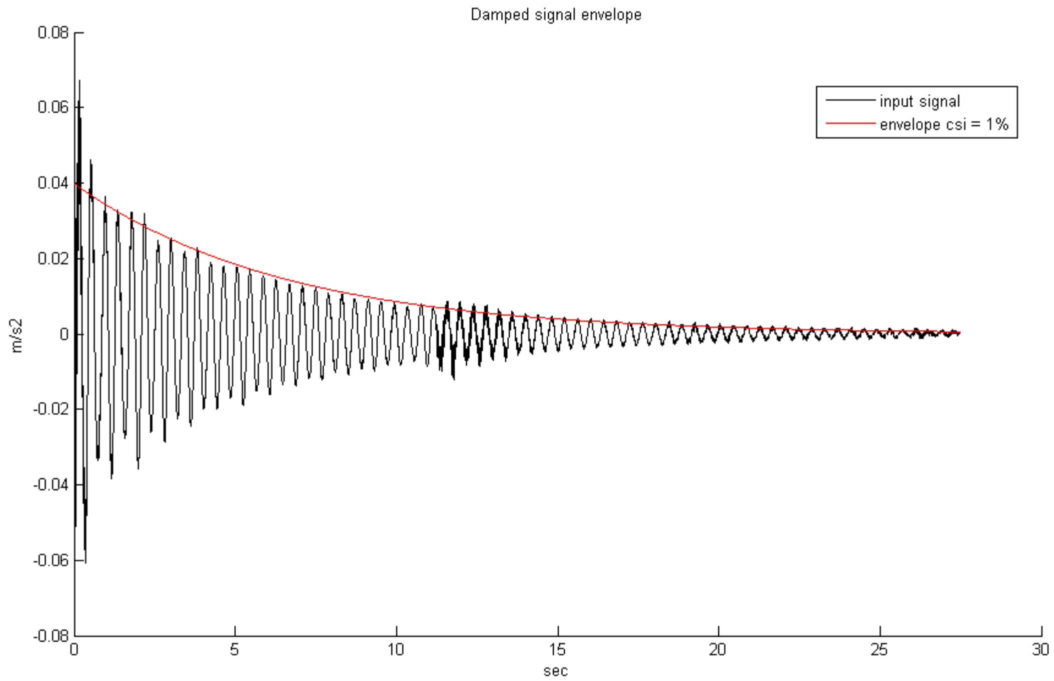

Figure 39: A registered single frequency signal vs 1% damping envelope curve

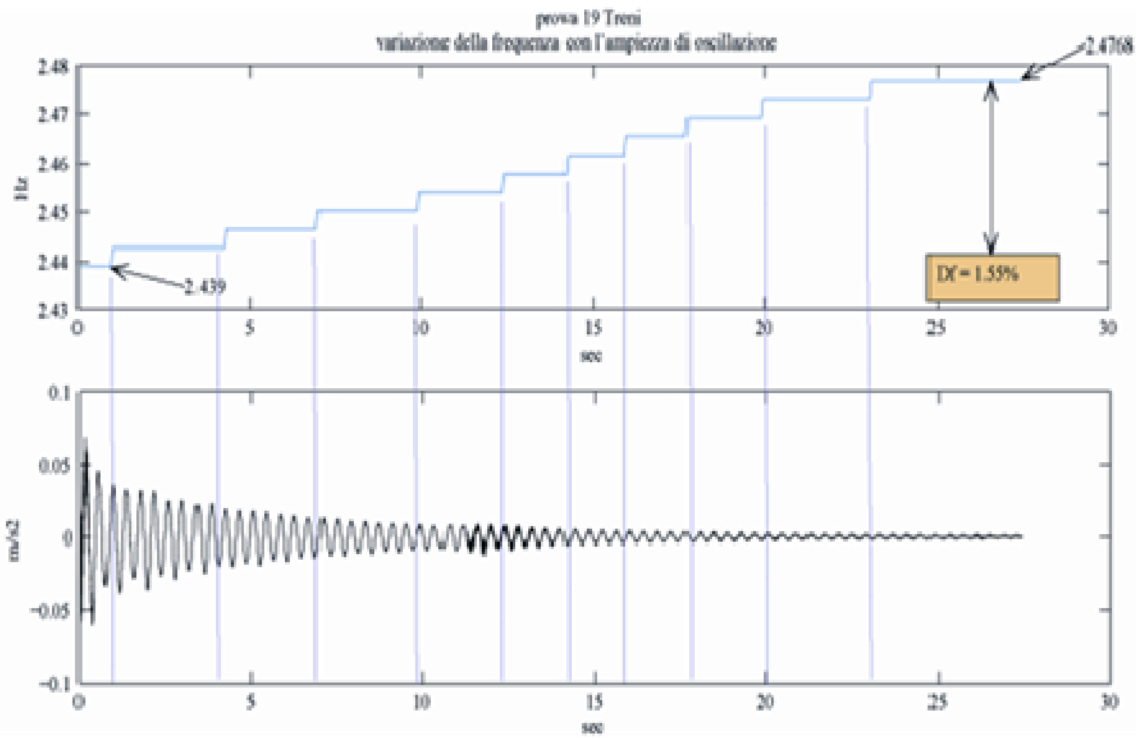

Figure 40: Frequency variation with signal amplitude

The analysis of recorded displacements time histories resulted in the same non linear effects in frequency and damping. Figure 42 shows some fitting attempts for the characterization of the nonlinear effect, considering polynomial and periodical variations' laws. It seems that the fifth degree polynomial law is the one that minimize the least squares regression, while higher order polynomial laws result numerically unstable.

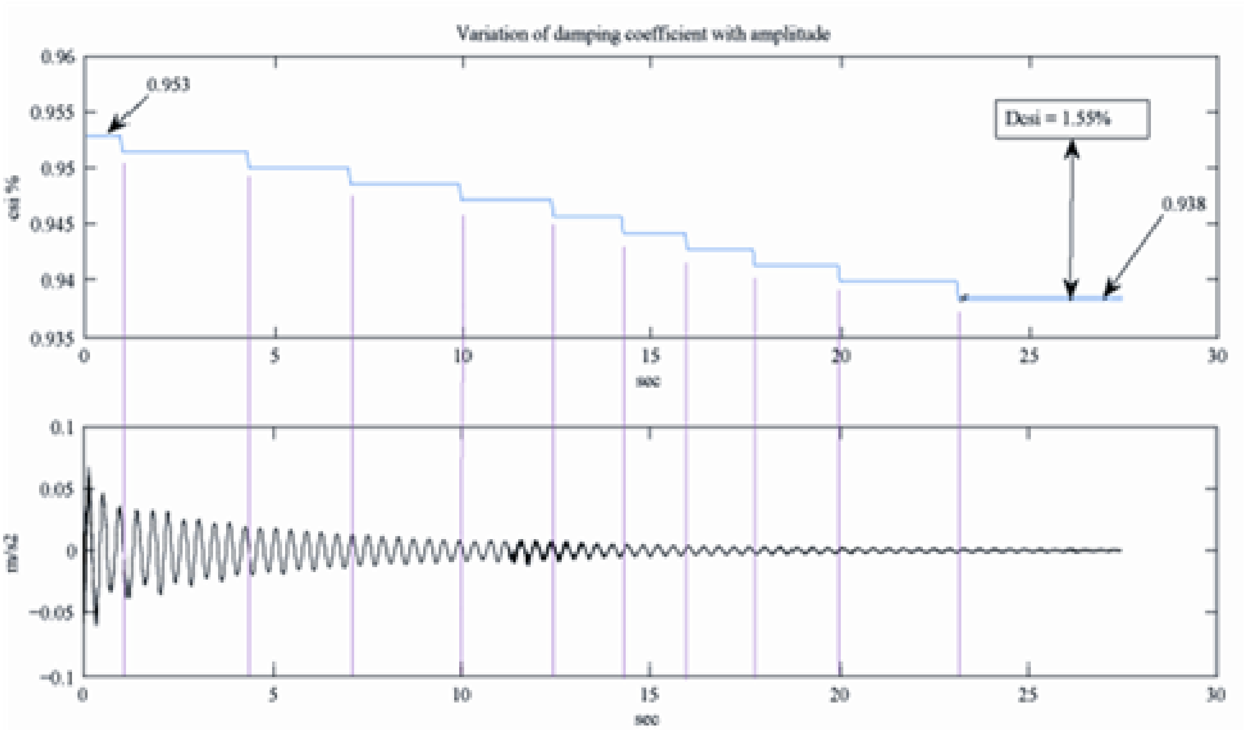

Figure 41: Damping variation with signal amplitude

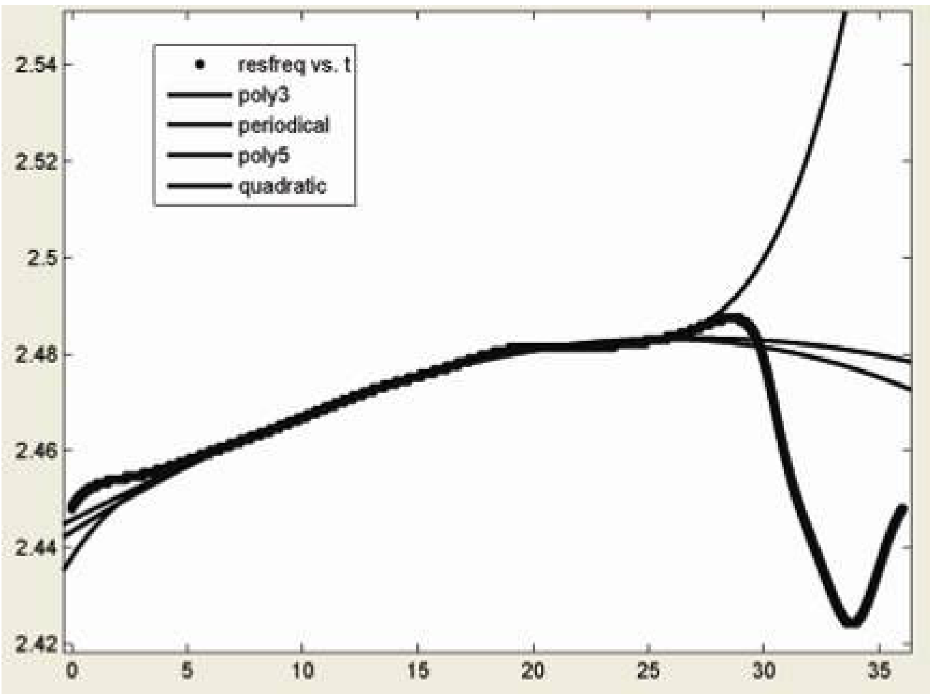

Figure 42: Characterization of non linearity via curve fitting method.

At this stage the interpretation of these experimental evidences is not still fully understood. The time variability of natural frequencies is very often related to structural non linearity. However in the present case study the load used for exciting the bridge was not severe enough to involve material non linearity or huge geometrical displacements.

For this reason the most probable explanation lies in some cracks that open and close under dynamic loading, but more investigations are needed for the characterization of these nonlinear effects and the complete comprehensions of damping dissipation in this masonry kind of masonry structures.

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CHAPTER 10: SEISMIC RETROFIT OF BUILDINGS

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1 INTRODUCTION

Especially after Second World War increasing demand to new buildings and facilities has occurred and in this time period a lot of structures and buildings were constructed. During this rapid construction period many critical issues which can play important roles in the durability of these structures have not been considered sufficiently. Environmental effects and loads, seismic forces etc. can be counted among these effects. These kinds of problems require enforcement of some additional regulations and rules for both design calculations and construction details. Therefore, in many countries building codes were published and distributed by the national authorities.

In the last three decades, especially the codes of seismically active countries exposed to substantial changes depending on the considerable improvements and findings in seismic assessment and designing fields. Updated codes and improved design requirements were applied for the design and construction of newer buildings. It can be roughly said that seismic design forces considered in the calculation of new buildings have been increased almost two times during the evolution of most modern codes.

Improved design requirements can be expected to reduce damage of newer buildings to acceptable levels during a moderate to strong earthquake. However, the older buildings designed by older codes which have not adequate safety, are likely to be vulnerable to severe damage or collapse under strong seismic event. Devastating earthquakes during last several decades have caused remarkable number of casualties and heavily damaged or collapsed buildings. Evidence of damages and poor behavior of existing buildings emphasize the need to improve the ability of especially existing buildings to withstand seismic forces. For this reason, assessment of existing structures is an urgent issue of a great economic significance in most countries around the world as more than 50% of all construction activities concern existing buildings as stated in Handbook 1 [1]. The international standard ISO 13822 provides general principles for the assessment of existing structures [2]. In recent years a lot of studies were performed to investigate and to improve the assessment and retrofit methods of existing buildings.

2 PERFORMANCE ASSESSMENT OF EXISTING BUILDINGS

Current documents and the investigations commonly agreed that performance improvement studies for existing buildings should generally involve a three step procedure; screening, evaluation and retrofit. The first two stages; screening and evaluation are performed to assess the seismic performance of existing building or buildings.

2.1 Screening

The first stage, screening, is generally applied for the assessment of group of structures rather than a single structure. It can be estimated that the majority of existing building stock is composed of older buildings which were constructed before modern code regulations. However, this situation does not imply that the all of the former code buildings are at risk. Therefore, a kind of preliminary stage should be used in order to minimize both costs and works required for the retrofit studies. This preliminary stage is called as screening. By this method, it is decided that whether building should be included for the detailed investigation stage or not. In other words, screening stage entails assessing buildings to ascertain their level of seismic risk following a simplified procedure whose main objective is to determine if the building should or should not be subjected to a more detailed investigation. This means that economical applicability of large scale retrofit studies directly related with the efficiency of screening method since the size of the problem is affected from the methods applied in this stage. There can be found some methods suggested for the screening stage in the literature such as Hazus, FEMA-154 and Japan seismic index method [3-5].

In these methods easily available structural properties such as type of structural system (i.e., reinforced concrete, masonry, steel etc.), number of stories, vertical and plan irregularities (i.e., soft storey, short columns, pounding effect, and heavy overhangs), location of building and age of building and apparent quality of the building which affect the seismic performance of buildings are collected and assessed. After the evaluation process, buildings are generally classified according to their quality levels such as poor, moderate and good. Buildings behind the expected quality level are selected for detailed investigation stage.

2.2 Performance evaluation

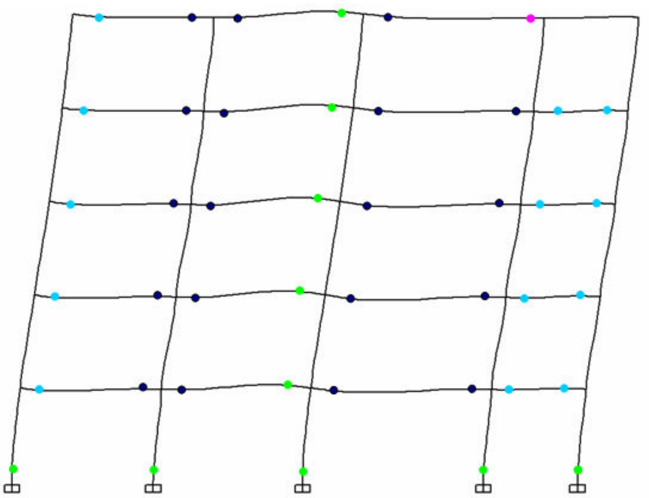
In the evaluation process, a detailed investigation is performed on buildings with medium to high priority as a result of the screening process. The objective of a performance evaluation is to identify the vulnerability of the structural and non-structural systems and their components to seismic loads. Seismic retrofit becomes necessary if it is shown that, through a seismic performance evaluation, the building does not meet minimum requirements up to the current codes and may suffer severe damage or even collapse during a seismic event.

In order to reveal the realistic situation of the building, constructing the mathematical modeling of structure is required. Guidance for modeling is given in Chapter 3. Performance of building is then verified by comparing the demand and capacity in terms of displacements or forces. It can be said that calculation methods enforced in the codes for the design of new buildings are based on linear elastic behavior assumption and therefore they can be called as force based methods. The results of linear methods can be very inaccurate when applied to buildings with highly irregular structural systems. Furthermore, linear procedures do not represent the actual building response under severe earthquake which requires displacement capacity rather than strength capacity. Calculation of displacement capacity, on the other hand necessitates the use of nonlinear methods rather than force based linear methods.

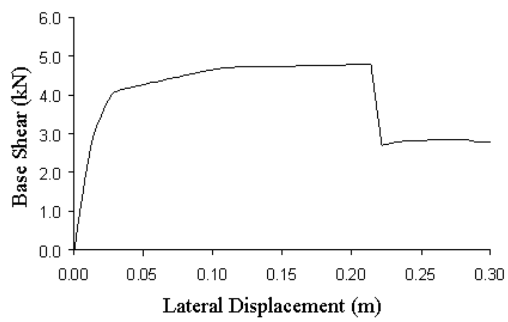
Therefore, majority of performance evaluation methods proposed in the last two decades focuses on the nonlinear structural response. It can be stated that preliminary studies were triggered after several damaging California Earthquakes in USA and seismic code regulation program for existing buildings was vigorously undertaken. Federal Emergency Management Agency of USA played important role in the organization of these studies. Consequently a lot of documents and reports were prepared and published by various researchers. Two basic documents; FEMA 310 [6] and FEMA 356 [7] can be counted as a result of FEMA studies. Especially in FEMA 356 document non-linear assessment methods are highly advocated and this document is intended to become a nationally recognized standard. This assessment approach affected the future studies and shaped the earthquake retrofit practice.

Application of non-linear analysis methods requires the determination of force and displacement response of individual elements at critical sections. Force and displacement capacities of sections are affected from material and detailing quality which should be determined by site investigations. The critical regions, where the damages are accumulated, are called as plastic hinge regions. Plastic hinges generally occur around the connection joints and/or near the member ends since the maximum reactions occur in these regions. Strength and deformation capacity of plastic hinge regions are determined by using moment-curvature analyses as mentioned in Chapter 3. Existing models for unconfined and confined concrete and typical steel stress–strain model with strain hardening for steel can be used for moment-curvature analysis [8 and 9]. Calculated member properties are then assigned to each elements to complete overall structural model. At this stage strength and stiffness contribution of each element (beams, columns, partition walls, etc.) should be considered and modeled to obtain reliable analysis results. As can be seen that construction of non-linear analysis model necessitates are detailed and time consuming studies.

Most favorable and widely used method to obtain strength and deformation capacity of buildings is static pushover analysis method. By using this method it is possible to investigate the level of damages occurred in members and building depending on the increasing displacements. This kind of analysis gives valuable information about the probability of partial or total collapse existence and the distribution of most vulnerable members in building. Figure 1b presents a typical capacity curve of a 2-D frame building obtained by pushover analysis. Horizontal and vertical axes shown on this figure represents respectively the displacement of roof level and base shear capacity of the building. As can be seen from this figure plastic deformations are occurred in building after the base shear capacity is reached and the level of building damage increases depending on the increasing displacements.



a) 2-D 5 storey frame building subjected to lateral forces



b) Typical capacity curve

Figure 1: Typical representation of non-linear response of 2D frame building

Figure 1.a shows a 2-D 5 storey frame building subjected to lateral forces considering first mode shape at each storey levels, relatively. When the target displacement was reached, the distribution of damages on structure was obtained. Figure 1 designated distribution of plastic hinges in structural members and the level of damage attained in each plastic hinge (slight, moderate, extensive damage), so giving an idea about the sample damage distribution in building. It can be seen from the figure that it is important to understand damage distribution both on the basis of system and structural members. Moreover, this situation will be helpful in later stages of study for retrofitting and strengthening strategies of the structure.

Pushover and time history analyses are carried out using existing programs such as SAP2000 [10], Opensees [11] and Perform-3D [12]. It must be highlighted here that descending branches of shear capacity curves often causes numerical problems in algorithms of existing software, therefore in many cases it is necessary to ascertain that the FE programme available is capable to do this. For instance, among the mentioned programmes, descending branch of base shear-lateral displacement can be explored in pushover analysis with SAP2000.

Non-linear analysis of the structures is a wide and open research area and determination of non-linear response of buildings are investigated by various researches in literature [13-18].

3 SEISMIC RETROFIT OF STRUCTURES

The fundamental documents on the strengthening of existing buildings are FEMA 172 - *NEHRP Handbook of Techniques for the Seismic Rehabilitation of Existing Buildings* [19], FEMA 273 - *NEHRP Guidelines for the Seismic Rehabilitation of Buildings* [20], FEMA 356 - *Prestandard and Commentary for the Seismic Rehabilitation of Buildings* [7], FEMA 547 - *Techniques for Seismic Rehabilitation of Existing Buildings* [21], and EN 1998-3 – *Eurocode 8 – Design of Structures for Earthquake Resistance – Part 3 Assessment and Retrofitting of Buildings* [22].

Seismic retrofit becomes necessary if it is shown that, through a seismic performance evaluation, the building does not meet minimum requirements up to the current building code and may suffer severe damage or even collapse during a seismic event. Once a building is decided to be retrofitted, there are several aspects to be considered:

- specification of the assessment objectives required by a client or authority;
- technical considerations;
- nontechnical considerations.

Technical considerations include identification of deficiencies and developing appropriate retrofit technique for these deficiencies. Compatibility of solution with the existing building load resisting system is required to be considered for a proper retrofit. Nontechnical consideration is related to cost of retrofit, time and disruption of occupants, the effects of retrofit on functionality and aesthetics of the building after retrofit.

Seismic retrofitting can be performed through several techniques with various objectives such as increasing the load, deformation, and/or energy dissipation capacity of the structure. This section is intended to describe the most common seismic rehabilitation techniques. “Technique” is used to describe a local action consisting of insertion of a new lateral force-resisting component or enhancement of the seismic resistance of an in-situ component in an existing building. A complete seismic retrofit may consist of the use of several techniques.

3.1 Seismic deficiency categories

Regardless of the seismic evaluation method used, failure to meet the specified performance criteria identifies certain seismic deficiency/deficiencies in existing buildings. It is convenient for the purposes of discussion and for developing retrofit strategies to categorize these deficiencies, which can affect:

- global strength;
- global stiffness;
- configuration and static scheme;

- diaphragms stiffness and/or strength;
- effectiveness of load paths;
- component and joint detailing;
- foundations.

Global Strength

Global strength typically refers to the lateral strength of the vertically oriented lateral force resisting system. A deficiency in global strength is common in existing older buildings either due to lack of seismic design or a design per an early code with inadequate strength requirements. In certain cases, the strength and expected inelastic displacement is related each other added strength may reduce nonlinear demands into acceptable ranges.

Global Stiffness

Although strength and stiffness are often controlled by the same existing elements or the same retrofit techniques, the two deficiencies are typically considered separately. Failure to meet evaluation standards is often the result of a building placing excessive drift demands on existing poorly detailed components. Global stiffness refers to the stiffness of the entire lateral force resisting system although the lack of stiffness may not be critical at all levels. For example, in buildings with narrow walls, critical drift levels occur in the upper floors. Conversely, critical drifts most often occur in the lowest levels in frame buildings. Stiffness must be added in such a way that drifts are efficiently reduced in the critical levels.

Configuration and static scheme

This deficiency category is related to plan and vertical irregularities that adversely affect performance of the building. Plan irregularities may often cause torsional response resulting in unusual demands on elements. Vertical irregularities are created by uneven stiffness and/or mass distribution between floors. In older existing buildings, such irregularities were seldom taken into consideration in the original design stage and therefore normally require retrofit measures to mitigate.

Diaphragms stiffness and/or strength

The primary purpose of diaphragms in the overall seismic system is to act as a horizontal plate spanning between lateral force resisting elements. In existing documents [14], deficiencies affecting this primary purpose, such as inadequate shear or bending strength, stiffness, or reinforcing around openings or re-entrant corners, are placed in this category.

Effectiveness of load paths

A break or inadequate strength in the load path prevents the positive attributes of the lateral load resisting system from being effective. The load path is typically considered to extend from each mass in the building to the supporting soil. For example, for a panel of cladding, this path would include its connection to the supporting floor or floors, the diaphragm and collectors that deliver the load to components of the primary lateral force-resisting system (walls, braces, frames, etc.), continuity of these components to the foundation, and finally the transfer of loads between foundation and soil. If the connection of the cladding panel or exterior wall fails and the element falls away from the building, the adequacy of the balance of the load path is moot. Similarly, the strength and stiffness of an added new shear wall element to the exterior of a building as a retrofit measure have no effect if it is not connected adequately to the floor diaphragms.

Component and joint detailing

Detailing refers to design decisions that affect a component's or system's behaviour beyond the strength determined by nominal demand, often in the nonlinear range. The most common example of a detailing deficiency in existing older buildings is poor confinement in concrete columns. The expected drifts from the design event will exceed the deformation capacity of such columns, potentially leading to degradation and collapse. Although the primary gravity load design is adequate, the post-elastic behaviour is not, most often due to inadequate configuration and spacing of stirrups. Identification of detailing deficiencies is significant in selection of mitigation strategies because acceptable performance often may be achieved by local adjustment of detailing rather than by adding new lateral force-resisting elements.

Foundations

Foundation deficiencies can occur within the foundation element itself, or due to inadequate transfer mechanisms between foundation and soil. Element deficiencies include inadequate bending or shear strength of spread foundations and grade beams; inadequate axial capacity or detailing of piles and piers; and weak and degrading connections between piles, piers, and caps.

3.2 Seismic retrofit measures

A systematic understanding of the expected seismic response of the existing building and all of its deficiencies is essential for an efficient retrofit scheme. In the traditional sense of improving the performance of the existing structure, there are three basic *classes* of measures taken to retrofit a building:

- add elements, usually to increase strength or stiffness;
- enhance performance of existing elements, increasing strength or deformation capacity;
- improve connections between components.

In addition to improving the strength or ductility of the existing structural elements, there are less traditional methods of improving the performance of the overall structure. These methods can be categorized as follows:

- reduce seismic demand, removing upper floors or other mass from the structure, adding damping devices to reduce displacements or using seismic isolation;
- remove selected components, removing or weakening the selected components to prevent damaging interaction between different systems.

Adding elements

Adding elements is the most common class of retrofit measures. In many cases, new shear walls, braced frames, or moment frames are added to an existing building to mitigate deficiencies in global strength, global stiffness, configuration, to reduce span of diaphragms.

Large lateral deformations induced in the structure due to ground shaking, impose high ductility demand on structural components. Besides flexible structures with components having inadequate ductility behave poorly. It is essential that such structures be stiffened at a global level.

Existing documents [7, 19-22] propose the addition of new braced frames or shear walls within an existing structure for increasing the stiffness. While some existing structures have inadequate strength, which result into inelastic behaviour at very low levels of earthquake forces and cause large inelastic deformation demands throughout the structure. By strengthening the structure, the threshold of lateral force at which the damage initiates, can be increased. Moment resisting frames can be provided as they are more flexible and add strength to the structure without significantly increasing its stiffness, as per these two documents. Eurocode 8 [22] suggests addition of new structural elements like bracings or infill walls; steel, timber or reinforced concrete belts in masonry construction; etc. or addition of a new structural system to take the seismic action.

Enhancing performance of existing elements

Some of existing buildings with a sufficient level of strength and stiffness at the global level may have some members (or components), which lack adequate strength, stiffness or deformation capacity to satisfy the performance objectives.

An appropriate strategy for such structures may be to perform local modifications of inadequate members. Local modifications that can be considered include improvement of component connectivity, component strength, and/or component deformation capacity. This strategy tends to be the most economical rehabilitation approach when only a few of the building's components are inadequate.

Existing documents [7, 19-21] explain that the component is allowed to resist large deformation levels without failure by improving the deformation capacity or ductility of the component, without necessarily increasing the strength. For example, placement of a jacket around a reinforced concrete column to improve its confinement increases its ability to deform without spilling or degrading reinforcement splices. As per FEMA 273 [20], the cross section of selected structural components can be reduced to increase their flexibility and response displacement capacity. According to Eurocode 8 [22], local or overall modification of damaged or undamaged elements (repair or strengthening) can be done, considering their stiffness, strength and/or ductility. It also suggests full replacement of inadequate or heavily damaged elements.

Current research on advanced materials in civil engineering is mainly concentrated on high performance concrete and steel, and fiber reinforced plastic (FRP) composites. FRP composite materials have experienced a continuous increase of use in structural strengthening and repair applications around the world in the last fifteen years. High specific stiffness and specific weight combined with superior environmental durability of these materials have made them a competing alternative to the conventional strengthening methods. It was shown through experimental and analytical studies that externally bonded FRP composites can be applied to various structural members including columns, beams, slabs, and walls to improve their structural performance such as stiffness, load carrying capacity, and ductility.

Improving connections between bomponents

The class of retrofit technique is almost exclusively targeted at mitigation of load path. deficiencies. With the exception of collectors, a deficiency in the load path is most often created by a weak connection, rather than by a completely missing link. However, some poor connections, particularly between beam and supporting column, are not directly in the primary seismic load path but still require strengthening to assure reliable gravity load support during strong shaking.

Reducing seismic demand

For the existing older buildings with relatively weak lateral system and excess space or a site where supplementary space can be constructed, removal of several top floors may prove to be an economical and practical method of providing acceptable performance. In many cases, little or no retrofit work may be required on the lower floors, although due to a shortened period, the acceleration response of the base may be increased.

Reducing demand by modification of dynamic response of a structure is also considered in this class. Perhaps the most notable example is seismic isolation. An overall advantage of base isolation is reduction in demands on the elements of the structure. This technique is most effective for relatively stiff low rise buildings with large mass compared to light, flexible structures. However, base isolation is technically complex and costly to implement and can be considered for special and historic structures. The existing documents [7, 19-22] propose base isolation as an option for seismic rehabilitation. However, they generally refer to specialist literature for details of analysis and design.

Removing selected components

Deformation capacity can be enhanced locally by uncoupling brittle elements from the deforming structure, or by removing them completely. Examples of this procedure include placement of vertical saw-cuts in unreinforced masonry walls to change their behaviour from shear failure to a more acceptable rocking mode and to create slots between spandrel beams and columns to prevent the column from being a "short column" prone to shear failure.

Building code requirements for seismic forces in seismically active countries have increased by almost up to 100% since the early 1970's. Improved design requirements can be expected to reduce damage of newer buildings to acceptable levels during a moderate to strong earthquake. However, the older buildings, designed by codes that are now known to provide inadequate safety, are likely to be vulnerable to severe damage or collapse under strong seismic event. Devastating earthquakes during last several decades have caused remarkable number of casualties and heavily damaged or collapsed buildings. Evidence of damages and poor behavior of existing buildings emphasize the need to improve the ability of existing buildings to withstand seismic forces.

Improvement of an existing building to withstand seismic forces involves a three-step process; screening, evaluation and retrofit.

Screening entails assessing buildings to ascertain their level of seismic risk following a simplified procedure whose main objective is to determine if the building should or should not be subject to a more detailed investigation. In the evaluation process, a detailed investigation is performed on buildings with medium to high priority as a result of the screening exercise. The objective of a performance evaluation is to identify the vulnerability of the structural and non-structural systems and their components to seismic loads. Seismic retrofit becomes necessary if it is shown that, through a seismic performance evaluation, the building does not meet minimum requirements up to the current building code and may suffer severe damage or even collapse during a seismic event.

According to the above mentioned modern seismic codes [17-22], seismic retrofitting can be performed through several methods with various objectives such as increasing the load, deformation, and/or energy dissipation capacity of the structure. Conventional as well as emerging retrofit methods are briefly presented in the following subsections.

3.3 Techniques for seismic retrofit of existing buildings

Existing documents [3-6] provides detailed information about identification of seismic deficiencies and solution for possible retrofit techniques.

One of the most complete document for seismic retrofit of existing buildings is FEMA 547 [21] which provides problem definition and solutions for model buildings with different types of lateral load resisting systems, such as wood frame, steel moment frame, reinforced concrete moment frame and reinforced concrete frame with infill masonry walls.

This section illustrates some examples of relevant tables given in FEMA 547, where further information and more retrofit detailing can be found.

Tables 1-4 list deficiency and retrofit techniques for wood frame, steel moment frame, reinforced concrete moment frame and reinforced concrete frame with infill masonry walls buildings, respectively. For foundations see, for instance, chapter 23 of FEMA 547.

Table 1: Seismic deficiencies and recommended retrofit techniques for wood frame buildings

Deficiency		Retrofit Technique				
Category	*Deficiency*	*Add new elements*	*Enhance existing elements*	*Improve connections between elements*	*Reduce demand*	*Remove selected components*
Globalstrength	Insufficient in-plane wall strength	Wood structural panel Shear wall Steel moment frame Steel braced frame	Enhance woodframe shear wall	Shear wall uplift anchorage and compression posts	Replace heavy roof finish with light finish	
Global stiffness	Insufficient in-plane wall stiffness	Wood structural panel Shear wall Steel moment frame Steel braced frame	Enhance woodframe shear wall	Shear wall uplift anchorage and compression posts		
Configuration	Missing or inadequate cripple wall bracing	Add woodframe cripple wall Add contiuous foundation and foundation wall	Enhance woodframe cripple wall			
	Open front	Wood structural panel Shear wall collector Moment frame	Enhance woodframe walls perpendicular to open front Detailing of narrow woodframe shear wall piers			
	Hillside	Wood structural panel Shear wall	Enhance woodframe shear wall	Anchor base level diaphragm to uphill foundation		
Load path	Inadequate shear anchorage to foundation			Anchorage to foundation		

Table 1: Seismic deficiencies and recommended retrofit techniques for wood frame buildings (cont'd)

Deficiency		**Retrofit Technique**				
Category	***Deficiency***	***Add new elements***	***Enhance existing elements***	***Improve connections between elements***	***Reduce demand***	***Remove selected components***
Load path (continued)	Inadequate shear wall overturning load path		Supplement framing supporting woodframe shear wall	Shear wall uplift anchorage and compression posts		
	Inadequate shear transfer in wood framing			Enhance load path for shear		
	Inadequate collectors to shear walls		Enhance existing collector	Add collector		
Component detailing	Unreinforced and unbraced chimney		Infill chimney Brace chimney		Reduce unsupported chimney height	Remove chimney
Diaphragms	Inadequate in-plane strength and/or stiffness		Enhance diaphrams Diaphragm overlay		Replace heavy roof finish with lighter finish	
	Inadequate chord capacity		Enhance chord members and connections			
	Excessive stresses at openings and irregularities		Enhance diaphrams detailing			
	Re-entrant corner		Enhance diaphrams detailing			

Table 2: Seismic deficiencies and recommended retrofit techniques for steel moment frame buildings

Deficiency		**Retrofit Technique**				
Category	***Deficiency***	***Add new elements***	***Enhance existing elements***	***Improve connections between elements***	***Reduce demand***	***Remove selected components***
Globalstrength	Insufficient frame strength	Moment frame Braced frame Concrete/masonry shear wall Steel plate shear wall	Strengthen beams, columns, and/or connections		Seismic isolation Supplemental damping	
Global stiffness	Excessive drift	Moment frame Braced frame Concrete/masonry shear wall Steel plate shear wall	Strengthen beams, columns, and/or connections		Supplemental damping	
Configuration	Soft Story	Moment frame Braced frame Concrete/masonry shear wall Steel plate shear wall				
	Re-entrant corner	Moment frame Braced frame Concrete/masonry shear wall Collector	Enhance detailing			
Load Path	Missing collector	Collector				
	Inadequate shear, flexural, and uplift anchorage to foundation		Embed column in to a pedestal bonded to other existing foundation elements	Provide steel shear lugs or anchor bolts from base plate to foundation		
	Inadequate out-of-plane anchorage at walls conneted to diaphragm			Tension anchors		

Table 2: Seismic deficiencies and recommended retrofit techniques for steel moment frame buildings (cont'd)

Deficiency		Retrofit Technique				
Category	*Deficiency*	*Add new elements*	*Enhance existing elements*	*Improve connections between elements*	*Reduce demand*	*Remove selected components*
Component detailing	Inadequate capacity of beams, columns and/or connections		Enhance beam-column connection Add cover plates or box members Provide gusset plates or knee braces Encase columns in concrete			
	Inadequate capacity of panel zone		Provide welded continuity plates Provide welded stiffener or doubler plates			
	Inadequate capacity of horizontal steel bracing	Provide additional secondary bracing	Strengthen bracing elements Reduce unbraces lengths	Strengthen connections		
Diaphragms	Inadequate in-plane strength and/or stiffness	Collectors to distribute forces Moment frame Braced frame Concrete/masonry shear wall Steel plate shear wall	Concrete topping slab overlay Wood structurul panel overlay at flexible diaohragms Strengthen chords	Add nails at flexible diapragms		
	Inadequate shear transfer to frames			Provide additional shear studs, anchors, or welds		
	Inadequate chord capacity	Add steel members or reinforcement				
	Excessive stresses at openings and irregularities	Add reinforcement Provide drags into surrounding diaphragm				Infill opening

Table 3: Seismic deficiencies and recommended retrofit techniques for concrete moment frame buildings

Deficiency		**Retrofit Technique**				
Category	***Deficiency***	***Add new elements***	***Enhance existing elements***	***Improve connections between elements***	***Reduce demand***	***Remove selected components***
Globalstrength	Insufficient number of frames or weak frames	Concrete/masonry shear wall Steel braced frame Concrete or steel moment frame Steel moment frame	Increse size of columns and/or beams		Remove upper story or stories Seismically isolate Supplemental damping	
Global stiffness	Insufficient number of frames or frames with inadequate stifness	Concrete/masonry shear wall Steel braced frame Concrete or steel moment frame	Increse size of columns and/or beams Fiber composite wrap of gravity columns Provide detailing of all other elements to accept drifts		Supplemental damping	Remove components creating short columns
Configuration	Soft Story or weak story	Add strength or stiffness in story to match balance of floors				
	Re-entrant corner Torsional layout	Add floor area to minimize effect of corner Add balancing walls, braced frames, or moment frames		Provide chords in diapragms		
	Incidental walls failing or causing torsion	Add balancing walls, braced frames, or moment frames	Uncouple incidental walls Convert incidental walls to lateralelements walls			Remove incidental walls

Table 3: Seismic deficiencies and recommended retrofit techniques for concrete moment frame buildings (cont'd)

Deficiency		**Retrofit Technique**				
Category	***Deficiency***	***Add new elements***	***Enhance existing elements***	***Improve connections between elements***	***Reduce demand***	***Remove selected components***
Load path	Inadequate collector	Add or strengthen collector				
Component detailing	Lack of ductile detailing--general		Perform selected improvements to joints		Seismic isolation	
	Lack of ductile detailing: Strong column-weak beam		Jacket columns			
	Lack of ductile detailing: Inadequate shear strength in column or beam		Fiber composite wrap Concrete/steel jacket			
	Lack of ductile detailing: Confinement for ductility or splices		Fiber composite wrap Concrete/steel jacket			

Table 4: Seismic deficiencies and recommended retrofit techniques for concrete frames with masonry infill walls buildings

Deficiency		**Retrofit Technique**				
Category	***Deficiency***	***Add new elements***	***Enhance existing elements***	***Improve connections between elements***	***Reduce demand***	***Remove selected components***
Global strength	Inadequate length of exterior wall	Interior concrete walls Interior steel braced frames	Concrete wall overlay Fiber composite wall overlay			
	Excessive sized openings in infill panels	Interior concrete walls Interior steel braced frames	Infill selected openings Fiber composite wall overlay			
	Inadequate columns for overturning forces		Add confinement Add tensile capacity on outside surface of column			
	Weak or deteriorated masonry	Interior concrete walls Interior steel braced frames	Point outside and/or inside wythes of masonry Inject wall with cementitious grout Fiber composite overlay			
Global stiffness	*See Global strength*					
Configuration	Soft or weak story	Interior concrete walls Interior steel braced frames				
	Torsion from one or more solid walls	Balance with interior concrete walls Balance interior steel braced frames				Remove selected infill panels on solid walls
	Irregular Plan Shape	Balance with interior concrete walls Balance with interior steel braced frames				

Table 4: Seismic deficiencies and recommended retrofit techniques for concrete frames with masonry infill walls buildings (cont'd)

Deficiency		Retrofit Technique				
Category	*Deficiency*	*Add new elements*	*Enhance existing elements*	*Improve connections between elements*	*Reduce demand*	*Remove selected components*
Load Path	Out-of-plane failure of infill due to loss of anchorage or slenderness of infill		Provide surface wall supports Shotcrete overlays Fiber composite overlay			Remove infill
	Inadequate connection of finish wythe to backing		Add connections			
	Inadequate collectors	Add steel collector or concrete collector				
Component detailing	Inadequate columns splice for tension due to uplift force induced by infill			Add splice plates Provide splice through added reinforced concrete encasement		
	Inadequate beam column connection to resist compression thrust			Strengthen connection in shear with steel or fiber composite		

4 CASE STUDIES

In the present section, two case studies concerning seismic retrofitting of building are presented.

The two cases taken into account refer to an undamaged reinforced concrete school building, located in Turkey, and to a residential building damaged by earthquake, located in Italy.

It must be emphasized that the preliminary studies in both cases have been carried out in accordance with the investigation techniques and the assessment methods described in the present Handbook and in the Handbook 1 [1], as well as the criteria for the choice of the interventions are in agreement with the suggestions given in the above-mentioned seismic regulations for retrofit of structures.

The assessments were performed adopting for actions and reinforcing materials the same design values as for new structures, in such a way that, once the retrofit is completed, the reliability of the repaired structure is the same required for new buildings.

4.1 Strengthening of an undamaged r.c. school building in Turkey

A typical template design for high school buildings is used to illustrate application of a retrofit example. The building is located in Denizli as being in high seismic region of Turkey. The 5-story school has 890 m^2 floor area and 24 classroom capacity: plan view of the building is shown in figure 2.

The selected building has reinforced concrete (RC) moment resisting frame in longitudinal direction while the load resisting system is RC moment resisting frame with shear walls in transverse direction. The semi-buried base story has band windows resulting in short columns. In-place concrete strength was determined as 13 MPa by core sampling and laboratory testing.

Seismic performance of the selected buildings has been evaluated considering nonlinear behaviour of reinforced concrete components.

The capacity has been determined by nonlinear static analysis using SAP2000 [10] that is a general-purpose structural analysis program. In the analysis beam and column elements have been modelled as non-linear frame elements with lumped plasticity, characterized by plastic hinges at both ends.

Seismic evaluation according 2007 Turkish Earthquake Code [23] pointed out that the building had inadequate strength, stiffness and displacement capacity, also because the presence of short columns, susceptible to brittle shear failures.

In order to overcome global strength, stiffness and deformation capacities, additional shear walls in both longitudinal and transverse directions have been introduced for seismic retrofit of the school building. Further element improvement has been obtained increasing the ductility of columns at different story levels. using FRP wrapping.

Finally, beside the reduction of demand granted by the introduction of shear walls, also the shear capacity and the ductility of short columns in the base story have been enhanced, by means of FRP wrapping.

The capacity curves of school building before and after retrofit in both longitudinal and transverse directions are shown in figure 3. In the diagrams, the vertical axis plots the shear strength coefficient, that is the base shear normalized by the building seismic weight, while the horizontal axis plots the global displacement drift, that is lateral displacement of building at the roof level normalized by the building height. The improvement in base shear capacity and stiffness in both directions is obvious. The increase in strength and stiffness decreases the displacement demands and improves the displacement and ductility capacity of the structure.

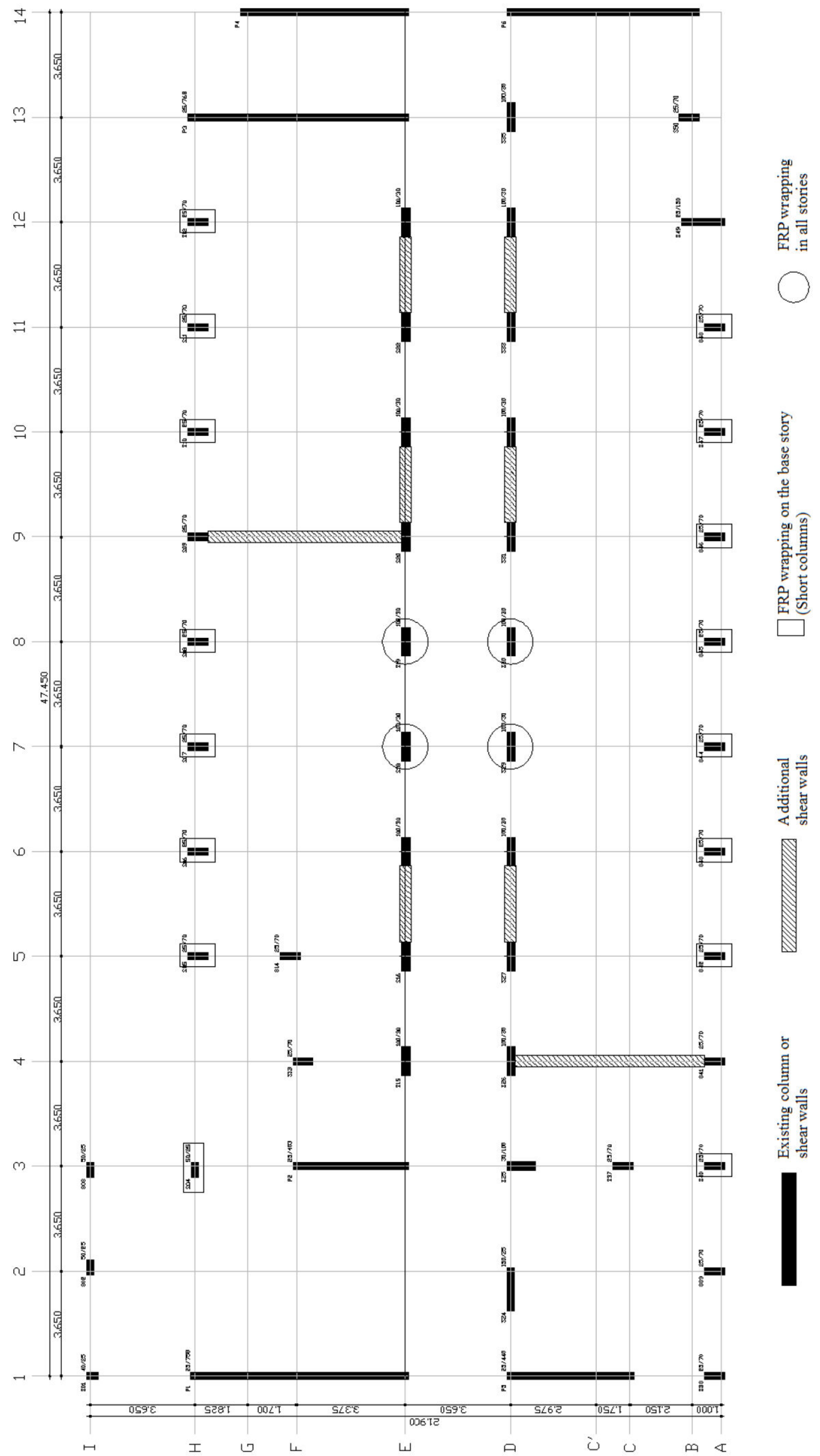
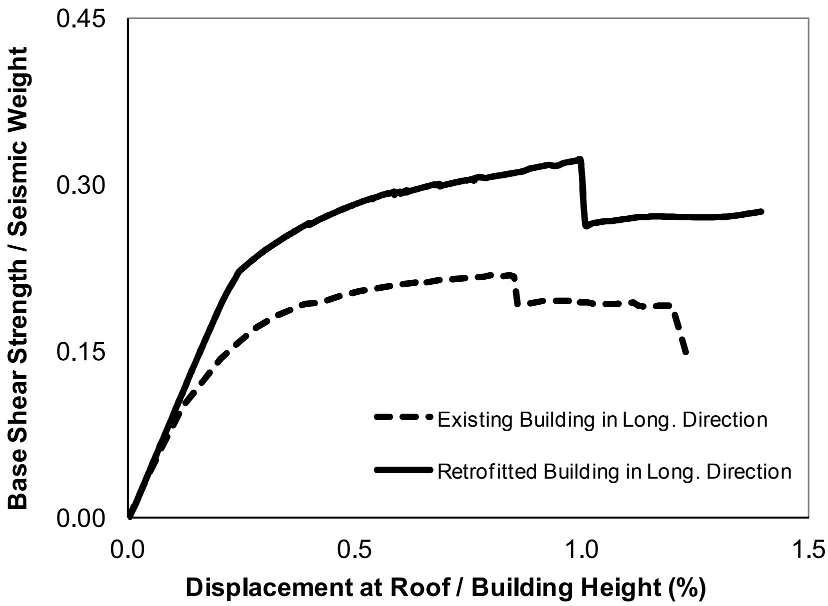
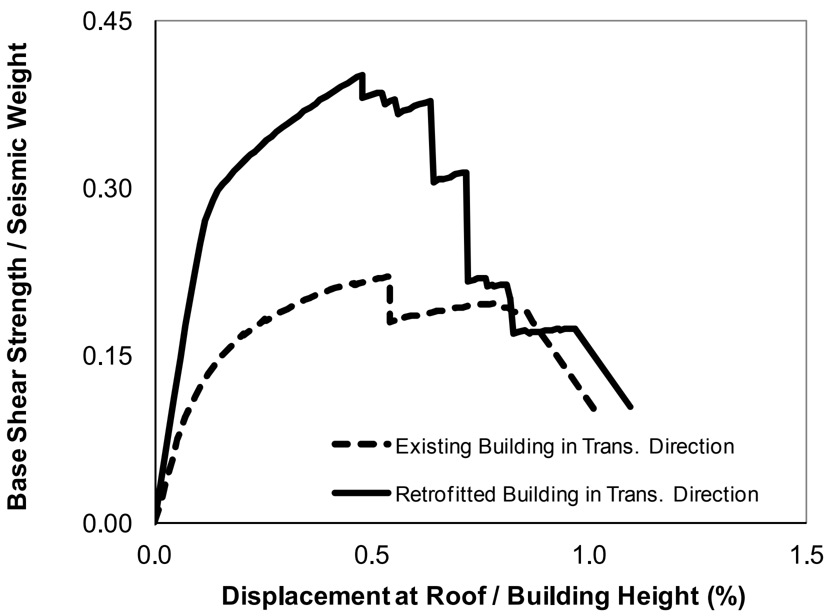


Figure 2: Plan view of a typical template design for high school buildings located in high seismicity region (Denizli)



(a)



(b)

Figure 3: Capacity curve of existing and retrofitted school building (a) in longitudinal direction (b) in transverse direction

4.2 Repair and strengthening of a damaged r.c. residential building

The second case study refers to the repair and strengthening of a three storey building heavily damaged by the Molise (Italy) earthquake of 31st October, 1st November 2001.

In the plan view, the building configuration is not regular, since it is defined by three rectangular bodies displaced by an extent of about 25% of the total size of the building in the corresponding direction. The original structure was a 3D RC. frame (see figs. 4, 5 and 6). The frames are oriented along a principal direction (North-South, while) connecting beams in the direction orthogonal to the frame plane are virtually absent since the structure does not have adequate resistance to horizontal seismic actions in the east-west direction.

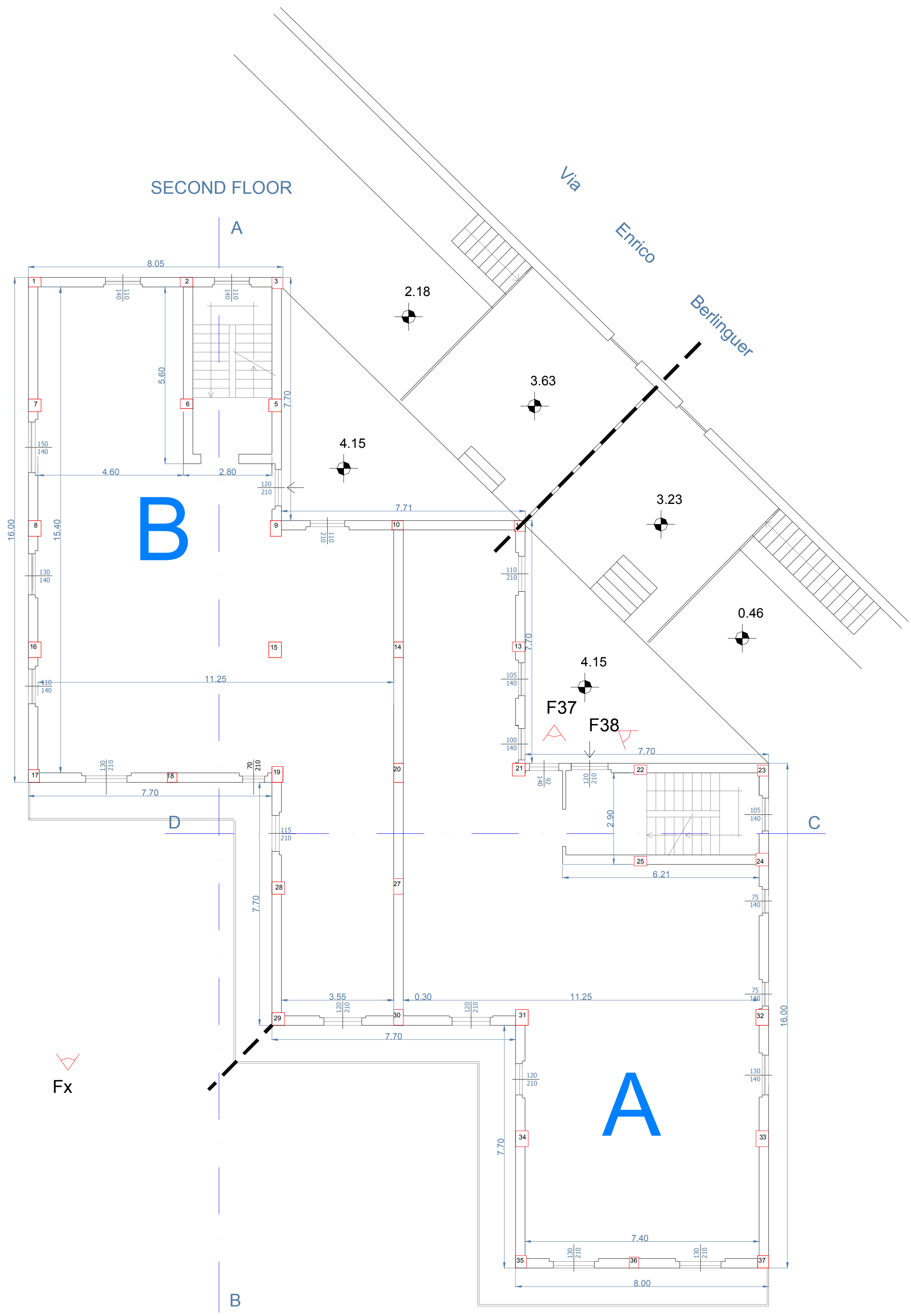


Figure 4: Plan of the 2nd floor

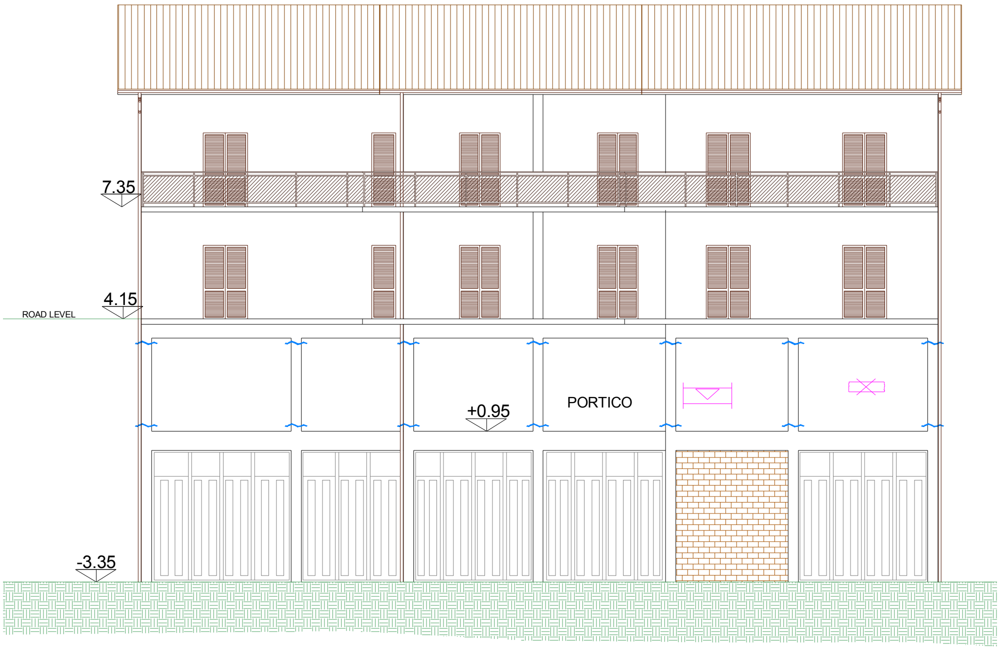


Figure 5.a: Front view

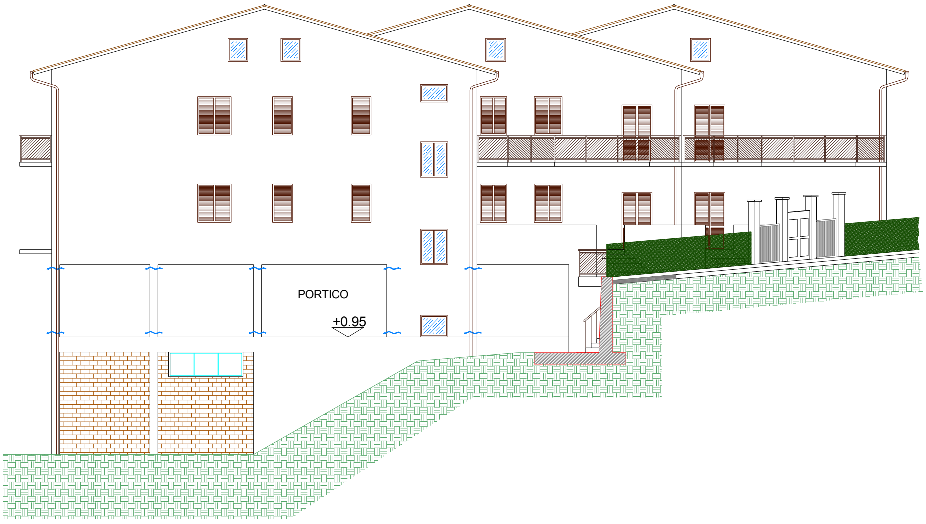


Figure 5.b: Side view

Figure 6: Views of the existing building

In height, the distribution of mass and stiffness is not constant due to the presence of the portico at the intermediate level, corresponding to the first floor. The presence of this soft storey implies the involvement of a small number of structural elements of a single plane to the dissipation of seismic input so that an unfavourable shear-type collapse mechanism is involved (fig. 7). This "strong beams and weak columns" frame conception clearly determines formation of plastic hinges at the ends of the columns of the soft storey

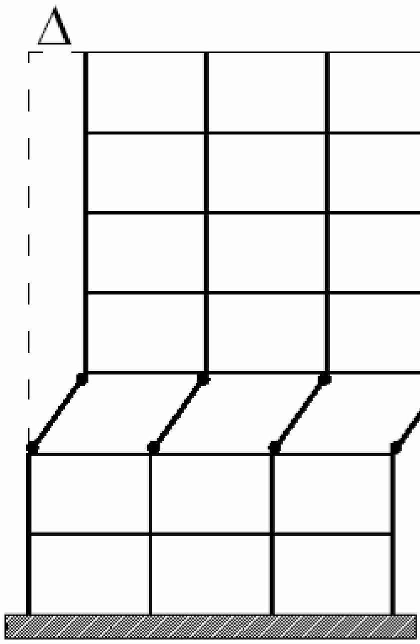


Figure 7: Shear type collapse mechanism

This building was designed around 1980 according to an old structural code, where again seismic actions were not taken into account. The damage suffered by the structure after the Molise earthquake confirmed the structural weaknesses set out above. In effect, the main damage was localized at the level of the first floor (soft storey) (fig. 8) where plastic hinges were detected on more than 50% of the columns with buckling of the longitudinal bars and yielding of the stirrups (fig. 9).

Figure 8: Damages induced by earthquake

Figure 9: Damages at the ends of the columns

The design of repair and strengthening interventions aims to reduce the seismic actions on the existing structure, which was originally inadequate and which was damaged by the earthquake, introducing an ad hoc system of shear walls. (figure 10).

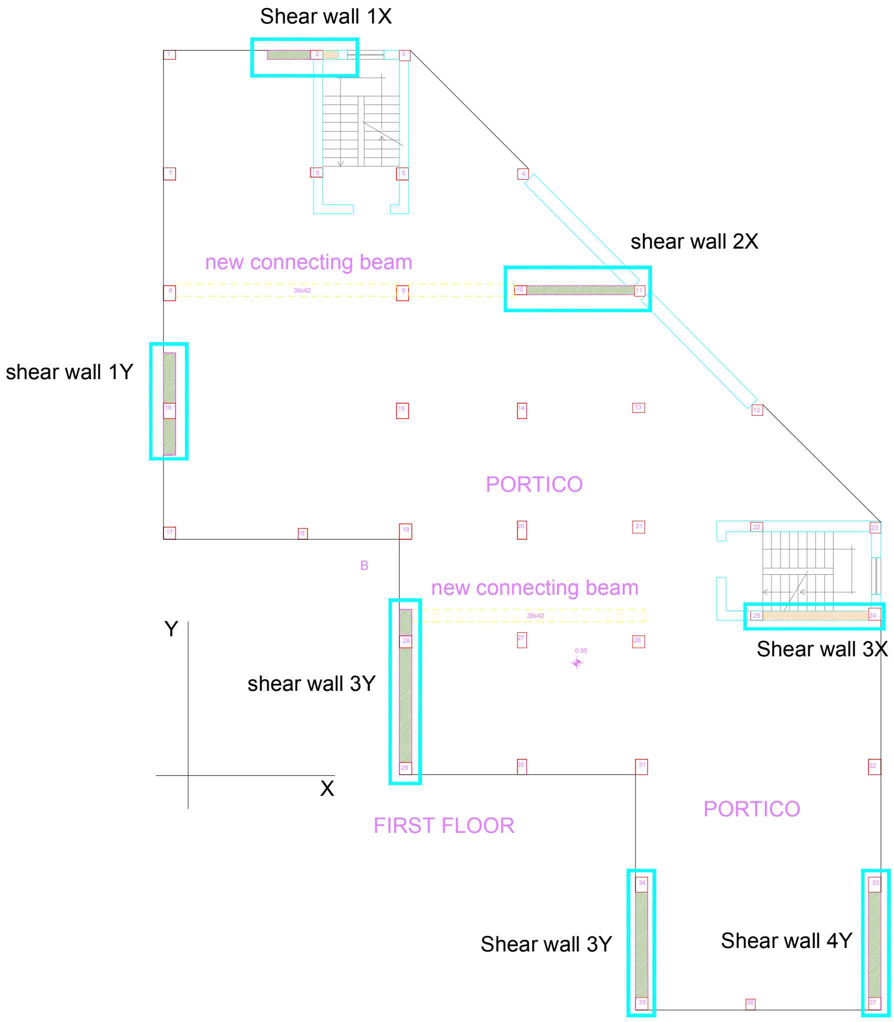


Figure 10: Shear wall system

The shear walls were inserted at strategic points of the structure, on both the perimeter and inside of the plant, trying to limit any torsional effects induced by seismic actions without modifying the elevations of the building. The shear wall, as said, are able to withstand all the seismic actions foreseen for new buildings, driving the horizontal stresses directly to the soil by means of a suitable foundation disposed on two or four pairs of micro-piles ϕ 220 mm. (some more details in given in short in figs. 11 and 12).

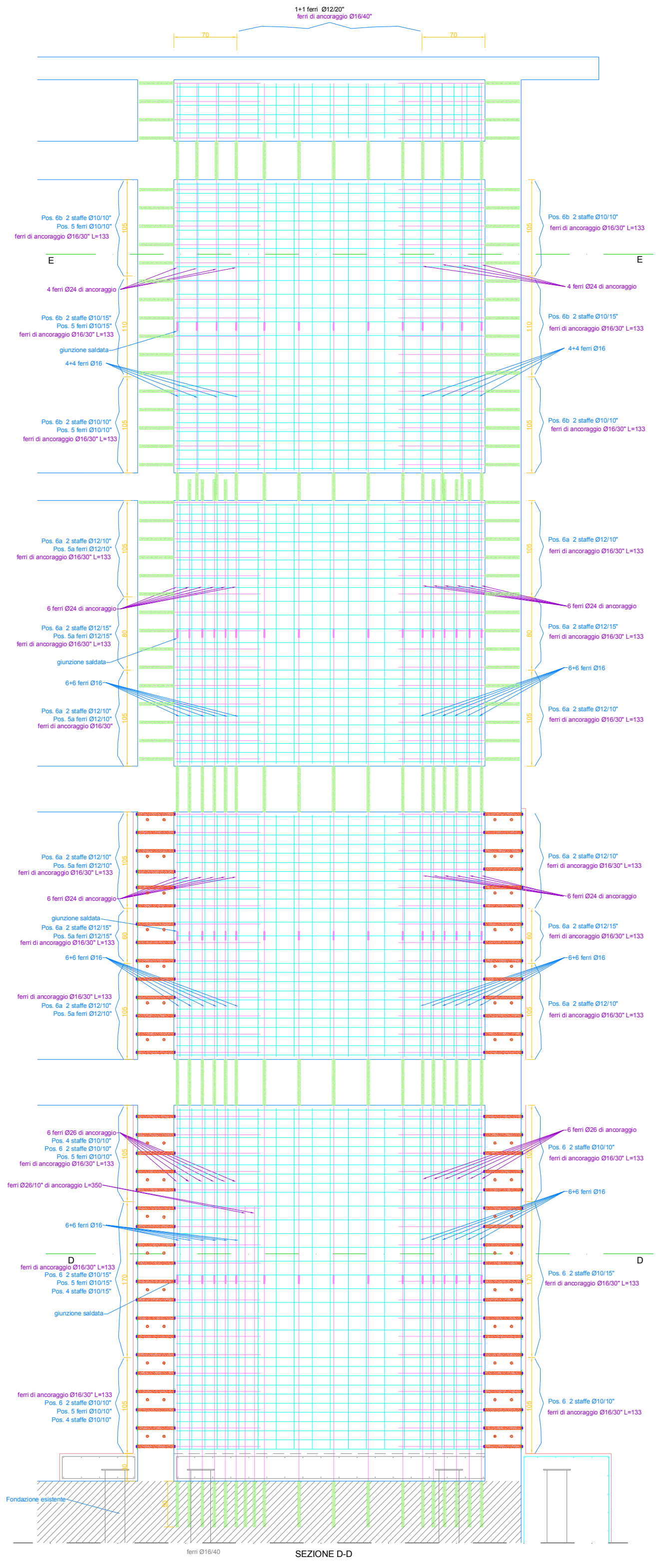
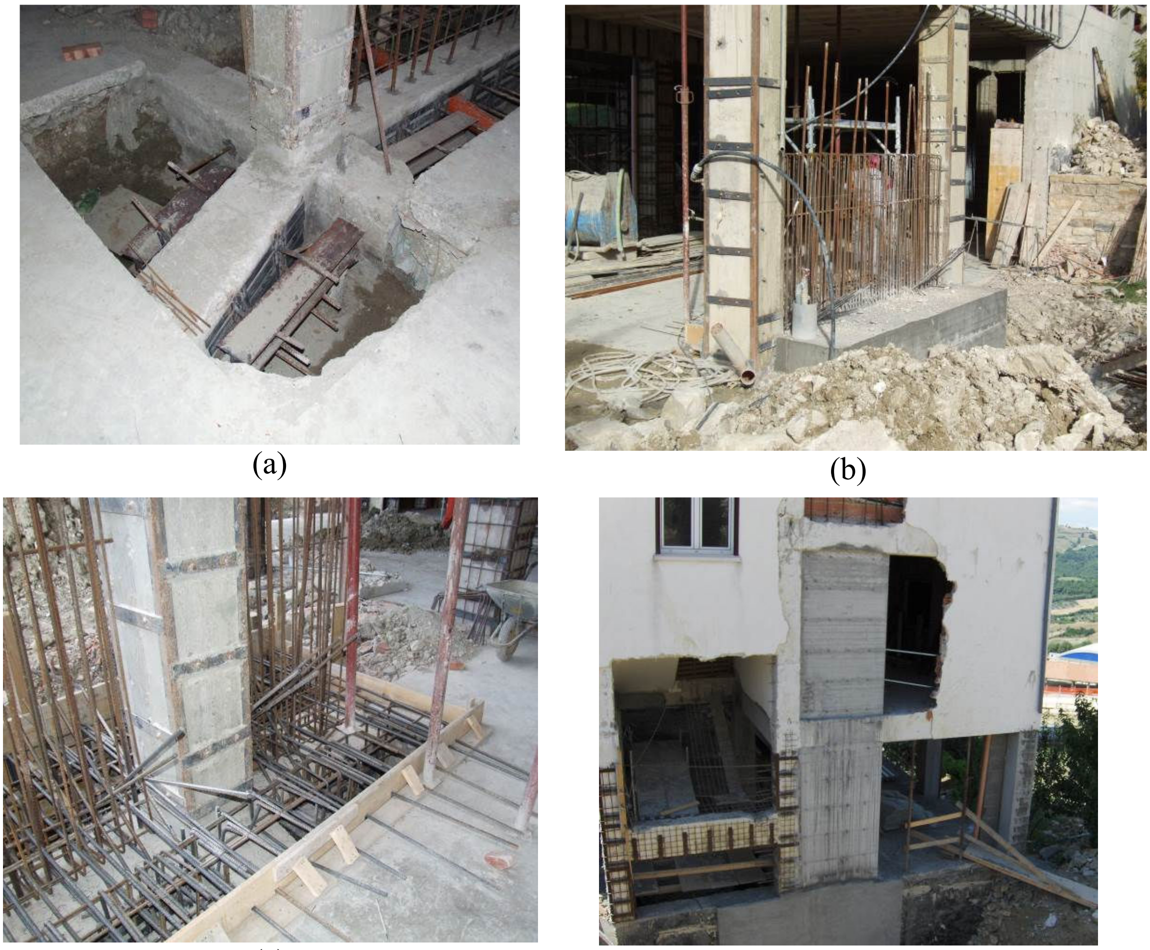


Figure 11: Vertical section of a shear wall



(c) (d)

Figure 12: Execution of the shear walls

The most damaged beams and columns has been reinforced according to the previously described philosophies and techniques, while the node confinement was improved using steel plates and steel profiles joined to the existing r.c structure by means of connectors made by threaded rods (fig. 13).

(a) (b)

Figure 13: Execution of columns and beams repairs

The interventions have been concentrated on the structures below the soft storey, as the soft storey itself “protected” the upper part of the building during the earthquake, so that for beams and columns of the upper levels no heavy intervention were necessary.

The improve the efficiency of the RC and brickworks floors in transferring horizontal actions new connecting r.c. have been inserted (fig. 14): to minimize the costs of such intervention, which inevitably involves the partial demolition of the floors, it was decided to use steel beams HEB220 instead, when it was necessary to preserve the finishings. Finally in figures 15 and 16 there are some view of the building at the end of the works-

(a)

(b)

Figure 14: Execution of connecting beams

(a)

(b)

Figure 15: View of the building at the end of the works

Verification example of Shear Wall 1X

Materials

- *Structural steel* S355

  yield stress f_{yk}=355 MPa
  partial factor γ_S=1
  design stress $f_{yd}=f_{yk}/\gamma_S$=355 MPa

- *Steel bolts:* class 8.8

- *Reinforcing steel* FeB44k (equivalent toB450C)

  yield stress f_{yk}=430 MPa
  partial factor γ_s=1,15
  design stress $f_{yd}=f_{yk}/\gamma_S$=379 MPa

- *Concrete* class C25/30

  concrete strength f_{ck} =25 MPa
  partial factor γ_C=1,6 (actually γ_C=1,5)
  design strength $\sigma c_d=\alpha_{cc} f_{ck}/\gamma_C=0{,}85\, f_{ck}/\gamma_C$=13,3 MPa.

Assessment of shear wall 1X

- *Shear wall cross sections dimensions*

  length l=2,30 m
  thickness b_0=0,25 m

The most severe membrane stress distributions in the shear wall are induced by earthquake acting in the shear wall direction, x, and are represented in figures 16 and 17. More precisely, figure 16 refers to normal stresses, σ_z, being z–axis vertical, and figure 17 to shear stresses τ_{zx}.

Suitably integrating the above mentioned distributions, we obtain the design forces

M=991,88 kNm
N= 230 kN
V=365,13 kN

The main longitudinal reinforcement of the shear wall is placed in the confined zones at the ends of the shear wall, whose length is approximately 0,2 l=0,46 m and consists of 8 steel bars ϕ16, so that $A_s=A'_s$=8 ϕ16=1608 mm^2.

The shear reinforcement is made by steel stirrups ϕ10, 150 mm spaced.

The ultimate limit state verifications of the shear walls so give:

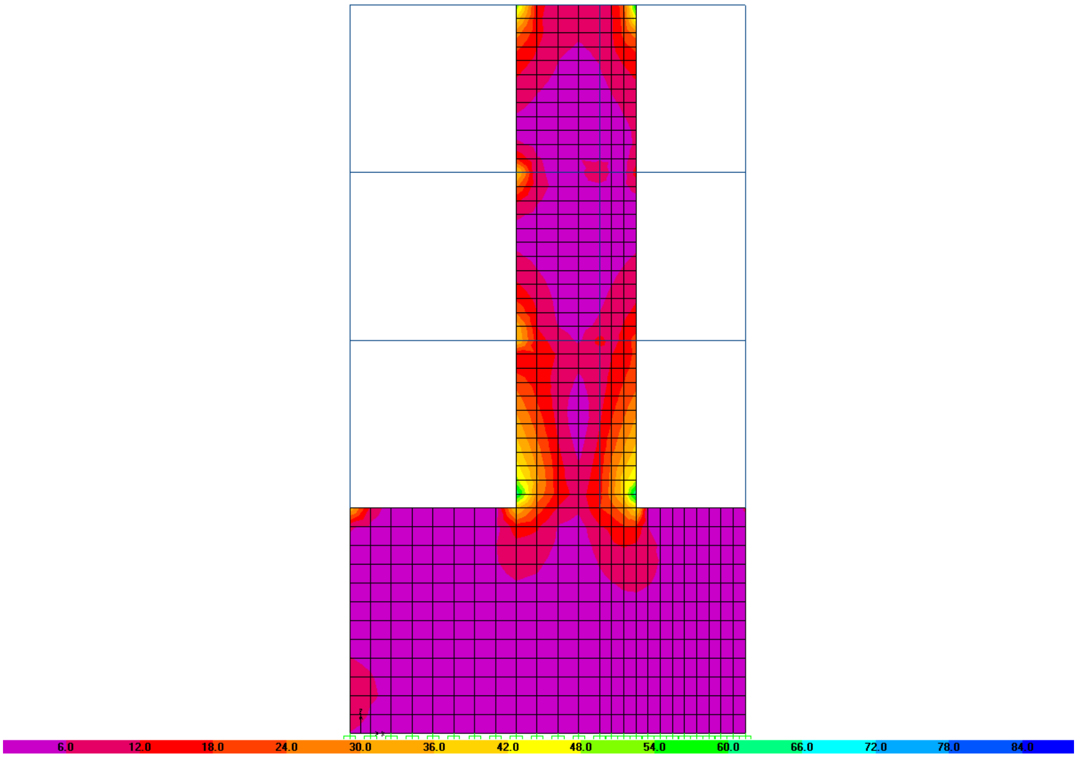


Figure 16: σ_z (z vertical) membrane stress distributions in the shear wall (x seismic combination) (MPa 10)

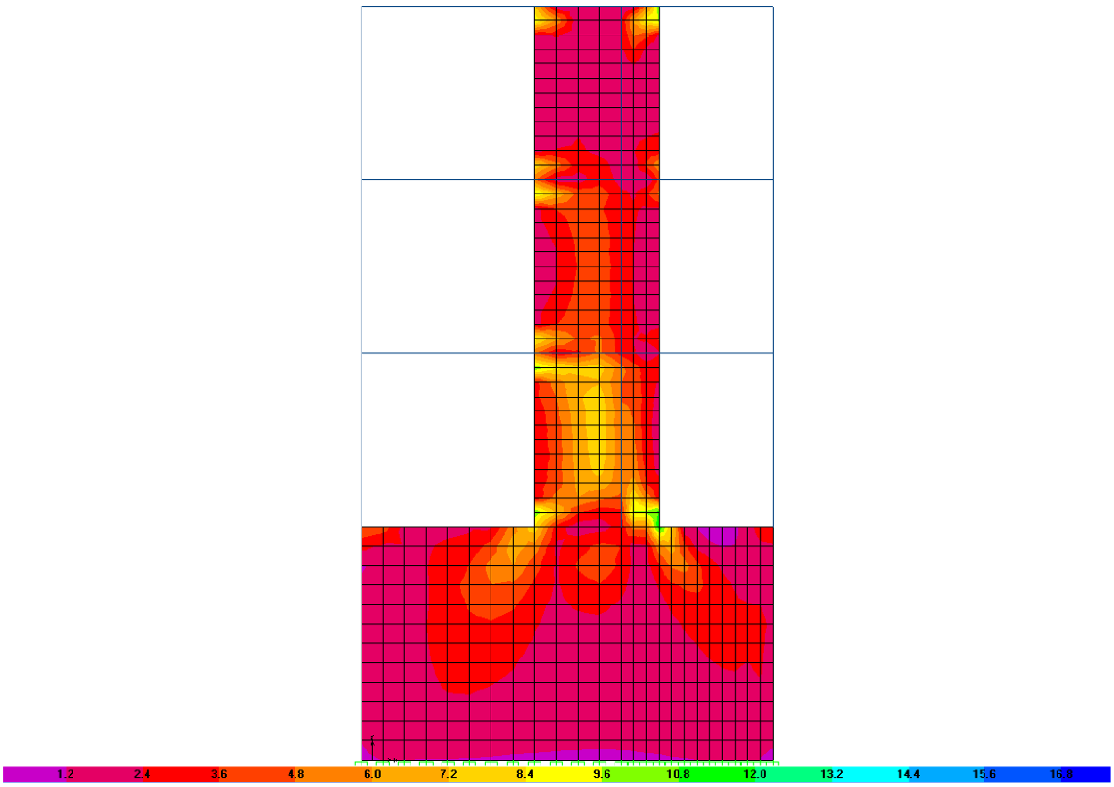


Figure 17: τ_{zx} shear membrane stress distributions in the shear wall (x seismic combination) (MPa 10)

Normal stresses

Results are summarized in figure 18, while the *M-N* resistance domain is reported in figure 19.

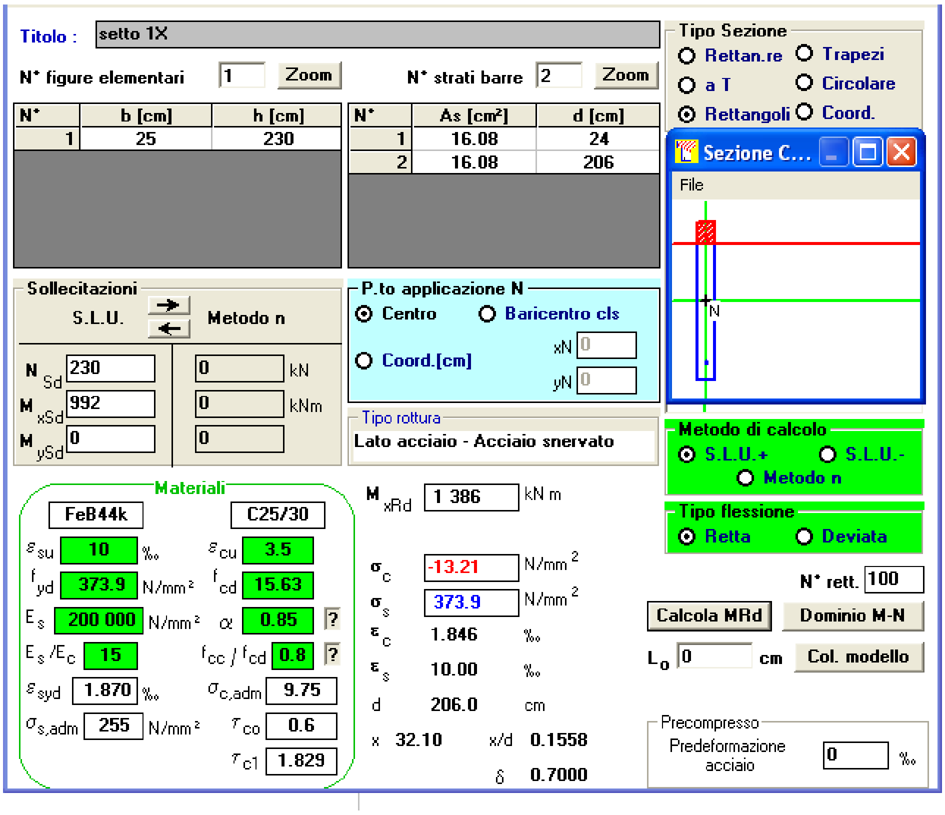


Figure 18: Cross section verification

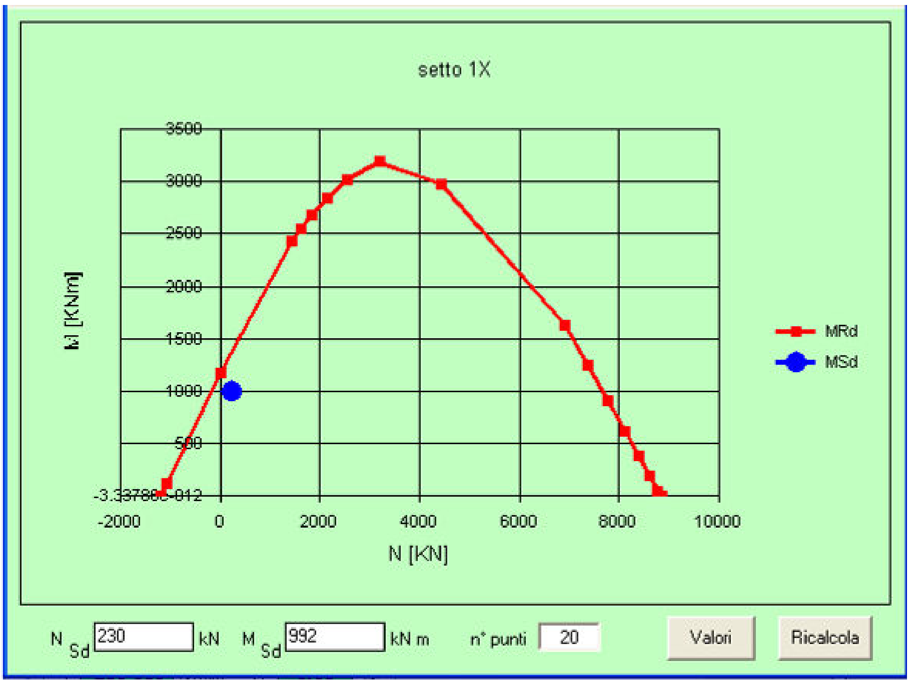


Figure 19: *M-N* resistance domain

Shear stresses

Concrete strut verification:

$$V_{Rd2} = 0{,}4 \cdot 0{,}8\, l \left(0{,}7 - \frac{f_{ck}}{200\ \mathrm{MPa}} \right) f_{cd} b_0 = 1401\ \mathrm{kN} > 365{,}13\ \mathrm{kN}$$

Steel stirrup verification:

Longitudinal steel reinforcement: 16 ϕ16

Longitudinal reinforcement ratio ρ

$$\rho = \frac{A_s}{b_0\, l} = \frac{3{,}216 \cdot 10^3}{5{,}75 \cdot 10^5} = 5{,}593 \cdot 10^3$$

*Steel stirrups area (*2 ϕ10/0,15 m*):*

157 mm^2/150 mm

$$V_{Rd} = 839{,}10\ \mathrm{kN} > 365{,}13\ \mathrm{kN}$$

Sliding verification:

Longitudinal steel reinforcement: 16 ϕ16

Height of the compressed zone of the base cross section: l_c=0,32 m

Contribution of the longitudinal steel reinforcement to sliding resistance

$$V_{dd} = 0{,}25\, f_{ywd}\, A_s = 300{,}63\ \mathrm{kN}$$

Contribution of the compressed zone to sliding resistance (friction)

$$V_{fd} = 0{,}25\, f_{cd}\, l_c\, b_0 = 265{,}63\ \mathrm{kN}$$

$$V_{Rds} = V_{dd} + V_{fd} = 566{,}3\ \mathrm{kN} > 365{,}13\ \mathrm{kN}\,.$$

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