UNIVERSITÀ DEGLI STUDI DI PADOVA

Sede Amministrativa: Università degli Studi di Padova

Dipartimento di Architettura, Urbanistica e Rilevamento

SCUOLA DI DOTTORATO DI RICERCA:
Studio e Conservazione dei Beni Archeologici e Architettonici

INDIRIZZO:
Scienze e Tecnologie per i Beni Archeologici e Architettonici

CICLO: XXIII

AN INTEGRATED APPROACH FOR RESTORATION AND CONSERVATION OF CULTURAL HERITAGE STRUCTURES: HISTORY, MATERIALS AND STRUCTURAL BEHAVIOUR. THE ARSENAL OF VENICE

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Sommario

Il restauro e la conservazione di beni storici è oggi un tema molto discusso ed in fase di evoluzione sia in Italia che all'estero. Il tema pone degli interessanti interrogativi circa la metodologia di approccio da seguire e gli obiettivi dai quali si deve partire per la definizione di un corretto progetto di restauro.

A livello normativo si stanno creando e affinando delle procedure di studio mediante la stesura di linee guida che hanno lo scopo di fornire una metodologia e una sensibilizzazione al problema a livello accademico, agli enti pubblici preposti alla conservazione e al restauro, e ai tecnici che lavorano in questo ambito. È infatti in atto un processo di "sforzo normativo" per colmare queste lacune mediante la produzione di documenti tecnici che si prefiggono di valutare il problema con maggior consapevolezza fornendo linee guida per il restauro di beni storici.

Attualmente la materia in esame vede due differenti tipi di approccio: da una parte il restauro conservativo, in cui viene data importanza alla forma ed alla materia e meno alla componente strutturale, e dall'altra un criterio più tecnicistico, che pone più riguardo alla struttura ma che, in molti casi, comporta degli interventi più massicci che rischiano di falsare la forma e la storia originaria del bene.

La tesi proposta si inserisce nel filone delle richieste normative, dei contenuti tecnici e dei documenti di riferimento quali le Linee Guida per i Beni Culturali e, a livello europeo, le Raccomandazioni ISCARSAH, ed intende fornire una procedura operativa per porsi nel giusto mezzo tra i due approcci sopra citati. Sono stati presi in considerazione vari documenti tecnici di riferimento ed è stato identificato un metodo integrato e mutidisciplinare per lo studio di beni storici, creato in funzione delle attuali normative e nel rispetto dei fondamentali criteri del restauro e della conservazione. Si è voluto rendere tale metodo il più generico possibile, senza fornire particolari informazioni inerenti specifiche tecniche di restauro, in modo tale che sia applicabile ad i vari tipi di beni storici.

Per validare tale approccio, la metodologia identificata è stata applicata a due differenti casi studio, entrambi presenti all'interno del sito ad elevato valore storico/sociologico/architettonico quale l'Arsenale di Venezia, che si differenziano tra loro per materiali, destinazioni d'uso e comportamento strutturale: la Sala Maggiore delle Sale d'Armi Nord e la gru idraulica Armstrong, Mitchell & Co. La scelta dei due casi studio è stata dettata da concrete esigenze della Soprintendenza B.A.P di Venezia e Laguna di recuperare i due beni riportandoli alla loro configurazione originaria sia materica che strutturale.

Abstract

Restoration and conservation of the cultural and architectural heritage is a very complex topic in continuous developing. The issue raises interesting questions about the methodological approach to be followed and the objective from which it must be start for the definition of a correct restoration project.

A regulatory-wide are creating and refining of a study procedures by means of the drafting of guidelines which intent is give a methodology and a sensitization at the problem at academic level, of public organs that control the conservation and the protection of the cultural heritage, and at the private technicians that work in this field. In Italy, a “normative effort” has been done by means of the production of technical documents that establish as evaluate the problem with more awareness. The aim is providing at the technicians the Guidelines about restoration of historical heritage. In the international field, from a general point of view, it is possible to refer to the ISCARSAH Recommendations, that give a preliminary approach to the topic.

Currently, in this field different approaches are used: (1) a criteria based on the conservation, which puts attention to the form and to the materials of the building but less at the structural components, (2) a technical method that is more interested at the structure but, in many cases, involves over-dimensioned interventions than the real requirements, altering the original form and history of the building.

The thesis is included in the current standards requirement and refers to the “Guidelines for evaluation and mitigation of seismic risk of cultural heritage” and the “Recommendations for the analysis, conservation and structural restoration of architectural heritage” ISCARSAH. Based on these documents, the study aims to providing an operative procedure able to balance the two approaches mentioned above. It has been identified a integrated and multi-disciplinary method for the study of cultural heritage, created in function of the currently codes and in respect of the fundamental standard of restoration and conservation. This method was made as generic as possible, without give particular information inherent specific restoration techniques, in such a way that is applicable at all the types of buildings with elevated historical-cultural valence.

The methodology was validate through its application to two paradigmatic case studies, both located within the highly historical / sociological / architecture site, the Arsenal of Venice. The case studies are: (1) the “*Sala Maggiore at the Sale d’Armi Nord*” (Main Room of the North weapons rooms) and (2) the Armstrong Mitchell & Co. hydraulic crane. The choice of the two examples has been dictated by the objective requirements of the Soprintendenza B.A.P. di Venezia e Laguna (local office of the ministry of fine arts responsible for the conservation of the cultural and landscape heritage of Venice and its lagoon), to intervene in these two constructions of great historical and architectural content, which differ in geometry, materials, structural behaviour and intended use.

Acknowledgements

First of all, I would like to thank Prof. Maria Rosa Valluzzi, tutor of my thesis, for the support received during these years.
This research has been done with the continuous collaboration of the *Soprintendenza BAP di Venezia e Laguna*. In particular I would like to thank the Arch. Claudio Menichelli and the Eng. Alberto Lionello for allowing me to work with cultural heritage structures of extraordinary interest, Umberto Basso who, thanks to his experience, has provided me with important teachings about the on-site work, the Alessandro Longega for his helpfulness.
Thanks to the Venice in Peril Fund, and in particular to Lady Clarke, for taking a special interest into the Armstrong, Mitchell & Co. crane project.
Thank to the Centro di Rilievo, Cartografia ed Elaborazione (CIRCE) (Centre for Surveying, Mapping and Processing) of IUAV, University of Venice.
I would like to thank Prof. Elena Francesca Ghedini for encourage me to deepen the study of English.
For his help and support I would sincerely thank Eng. Yaacov Schaffer.
To the Prof. Pere Roca for his advices and to giving me the possibility of developing part of my research in his university. My thanks also to Alvaro Viviescas who has patiently introduced me into the modelling world.
I am grateful to Prof. Claudio Modena for giving me the opportunity of being part of his research group and allowing me of taking part to important experiences that made me grow professionally.
I would also like to thank all my colleagues for their continuous collaboration and support, especially Francesca, Massimo, Marco, Filippo, Flavio, Luca, Matteo, Elena e Nicola. Thanks to Enrico for his continuous support, for his precious advices and for believing in this project. A special thank to my office colleagues, Giovanni e Bruno, for their support and for putting up with me during these years.
To my Venice colleagues, with whom I shared the experience of the on-site work and "joy and pains" of the Armstrong crane project.
I am also grateful to all the graduate students who gave their important contribution and helped me to achieve these results.
To my mum, my dad, Luisa and Giovanni for their continuous and fundamental support.
A particular thanks to a special person (B).

During these years I have taken part into many interesting activities. Among these, the most important one, both from a professional and from a personal point of view, is without doubt the work done in Abruzzo after the earthquake of the 6th of April 2009. I would like to thank the people that have shared this experience with me and in particular Fernando, Marcello, Corrado and the Vigili del Fuoco (Fireman).

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1 INTRODUCTION

1.1 Aim and methods of the research

Restoration and conservation of the cultural and architectural heritage is a very complex topic in continuous developing. The issue raises interesting questions about the methodology approach to be followed and the objective from which it must be start for the definition of a correct restoration project.

Nowadays, this topich has two different types of approach: on one side the conservative restoration, and on the other side the requirement of a technical criteria, often releated to heavy interventions.

The thesis is included in the current standards requirement and refers to the "Guidelines for evaluation and mitigation of seismic risk of cultural heritage" and the "Recommendations for the analysis, conservation and structural restoration of architectural heritage" ISCARSAH. Based on these documents, the study aims to providing an operative procedure able to balance the two approaches mentioned above.

Very often, in Italy, the restoration approach developed by public bodies that control the conservation and the protection of the cultural heritage is strongly related to conservative restoration, with particular attention at the degradation and at the recovery of the historical materials.

On the opposite side, technicians who aim providing adequate resistance to the structure according to the current standards, approach the problem in a too mechanic way, often with too invasive method and especially looking at the historical materials with a relatively low confidence to their structural qualities.

To remedy at this problem, in Italy, a "normative effort" has been done by means of the production of technical documents that establish as evaluate the problem with more awareness. The aim is providing at the technicians the Guidelines about restoration of historical heritage. In the international field, from a general point of view, it is possible to refer to the ISCARSAH Recommendations, that give a preliminary approach to the topic.

The first fundamental step for conservations and restoration of cultural heritage is the multidisciplinary approach. The investigation of historical structures requires the evaluations of different aspects and problems, having both technical-scientific and historical-critical profile. Therefore, a logical-intuitive deductive process is necessary, which requires a capacity of synthesis and understanding which only great experience and solid scientific-technical basis can ensure (Guidelines, 2006).

Therefore, it is of fundamental importance to establish a high-competence investigation team, able to incorporates a range of adequate skills for a complete preliminary study of the building and for the definition of interventions.

A planning program of interventions for conservation and restoration of historical heritage should be based on both qualitative methods, based on documentation and observation of the degradation of materials and structural damage, and quantitative methods based on experimental and analytic techniques, that take into account the effect of the various phenomena on the structural behavior.

This should lead the consolidation of historical structure in the restoration and conservation field respectiong the fundamental criteria such as: "reversibility", "compatibility" and "minimum intervention".

For a correct and complete analysis of an historical building it is necessary to draw a project based on the historical, geometrical, materical and structural knowledge of the structure. It must be designed not only to guarantee safety, but also to respect the context which surrounds them (Guidelines, 2006).

The *preliminary analysis* is a essential step for the study of historical structure and allows giving suited information for the following restoration interventions. If this stage is performed incorrectly, the resulting decisions could be arbitrary; poor judgment may result in either conservative, and therefore heavy-handed conservation measures, or conversely, inadequate safety levels (ISCARSAH, 2005).

The knowledge of the structural historic building is at the basis for a complete comprehension of the manufact. However, the main problem, in particular for the historical heritage, is the impossibility to know the structural and material transformations that developed during the centuries.

Therefore, there is the necessity of refining analysis and interpretation techniques of historical buildings by means of the accuracy of historical research, of the geometrical survey and of the evaluation of the damage of materials and of structural problems. The preliminary knowledge phase will be analyzed in function of the objectives of the project.

The purpose of the historical investigation is to understand the conception and the significance of the building, the techniques and the skills used in its construction, the subsequent changes in both the structure and its environment and any events that may have caused damage. Knowledge of what has occurred in the past can help to forecast future behavior and can be a useful indication of the level of safety provided by the current state of the structure (ISCARSAH, 2005).

This last point is developed in the stage of the geometrical, materic, structural and of the damage survey of the building. The main objectives of this stage are the identifications of the structural elements, the evaluation of the presence and intensity of deterioration in function of different types of materials, the evaluation of the structural damage in progress.

Also, It is important to discover how the environment may be damaging a building, since this can be exacerbated by poor original design and/or workmanship (e.g. lack of drainage, condensation, rising damp), the use of unsuitable materials, and/or inadequate (or non-existent) maintenance (ISCARSAH, 2005). For this type of analysis we make use of different means as the geometrical survey, the experimental tests, the maps of the damage and of the numerical models, for evaluating the global and/or local behavior of the structure at present.

The materials characteristics, in the majority of the cases, can be traced to degradation phenomena, may be reduced by decay caused by chemical, physical or biological action. For the evaluation of the characteristic of materials, in particular for the ones that present a strong degradations, it is necessary to do some tests in laboratory and/or in situ. The aim of the tests is identifying the mechanical, physical and chemical characteristic and the presence of possible discontinuities.

Non-destructive tests should be preferred to those that involve alterations to a structure. If these are not sufficient, it is necessary to assess the benefit to be obtained by drawing samples from the structure in terms of reduced structural intervention against the loss of culturally significant material (a cost-benefit analysis) (ISCARSAH, 2005).

The conclusive stage of the preliminary knowledge phases of historical buildings is the evaluation of the safety of the structure. Whilst the object of diagnosis is to identify the causes of damage and decay, safety evaluation must determine whether or not the safety levels are acceptable, by analyzing the present condition of both structure and materials. Therefore, it is must for the restoration interventions because it allows to evaluate if the corrective measurements are necessary. For these evaluations we should use qualitative, studied in the preliminary stage, and quantitative, based on mathematical models, approaches.

The finite element *numerical models* are the principal support both for the knowledge of the mechanical behavior of the structure and for the evaluation of the more appropriate and less invasive restoration interventions. Also, the modeling are an important instrument for the estimation of the efficacy of the interventions performed. The modeling presents a considerable problem concern the correct definitions of the geometrical and materical characteristic and the conditions which connect the various elements. For this reason, the scale of the model can be define on the basis of the information found during the diagnostic phase, according to the importance of the monument and the required level of detail.

All information collected and processed during the diagnostic phase allow to identify the critical points of the structure for planning a program of interventions.

For the historical structures, the use of the actual laws are not appropriate because, often, refer to a requirements to improve the strength that may lead to the loss of historic fabric or to changes in the original conception of the structure. A more flexible and broader approach, where calculations are not the only means of evaluation, needs to be adopted for historic structures to relate the remedial measures more clearly to the actual structural behavior according to the principle of minimum intervention (ISCARSAH, 2005).

In conclusion, the *intervention phase* must be jointed to the *monitoring* of the structure, in order to evaluate, step by step, the efficiency of the interventions. The collected data will also be represent the starting point for monitoring the possible further interventions, which has as like objective the evaluation of possible progressive damage phenomenas.

The previously mentioned methodology was validate through its application to two paradigmatic case studies, both located within the highly historical / sociological / architecture site, the Arsenal of Venice. The case studies are: (1) the "*Sala Maggiore at the Sale d'Armi Nord*" (Main Room of the North weapons rooms) and (2) the Armstrong Mitchell & Co. hydraulic crane. The choice of the two examples has been dictated by the objective requirements of the Soprintendenza B.A.P. di Venezia e Laguna (local office of the ministry of fine arts responsible for the conservation of the cultural and landscape heritage of Venice and its lagoon), to intervene in these two constructions of great historical and architectural content, which differ in geometry, materials, structural behavior and intended use.

For both case studies the main and final goal is the rehabilitation, paying particular attention to the re-proposing the construction techniques of the original construction period and to the selection of the minimum required interventions. The "Sala Maggiore" is intended to be reused as a museum. As for the Armstrong hydraulic crane the final goal is to achieved a conservative restoration of the asset, allowing the structure returning to its original configuration. This will alive the evidence of the evolution of the late nineteenth shipbuilding industry, allowing the tourists to enjoy the piece of history while visiting the Arsenal.

1.2 Thesis organization

The thesis is organized into 6 chapters, starting from the present introduction that resume the aim and the method of the research for the identification of a integrated methodological approach for historical heritage.

The chapter 2 presents a complete description of the methodology proposal in this thesis, specifying the fundamental steps and the most frequent problems actually present in the study of historical building. It also makes reference to the technical documents that actually regulate the restoration, in Italy and abroad.

The third chapter introduce the historical framework where the two case studies are located: the *Sala Maggiore* on the *Sale d'Armi Nord* complex and the Armstrong, Mitchell & Co. hydraulic crane. The aim is identifying the historical transformations that have characterized the Arsenale of Venice and the consequence which have influenced the story and the evolutions of the two examples.

The fourth chapter presents the first analyzed case study: the “*Sala Maggiore*” of the “*Sale d’Armi Nord*”. It is a building characterized by brick and Istria stone, wooden floors and wood and reinforced concrete roofs. The study of this case study starts with the historical analysis, from the transformations that the building underwent throughout the centuries to the geometric analysis using various survey techniques, and continuing also with the analysis of the degradation and material. The information obtained from the diagnostic phase are collected in forms able to synthesize the data. After completed the preliminary phase, it has been possible to evaluate the elements where the intervention is needed and, with the support of a global model of the structure and of detailed models of single elements, to define the interventions to be performed. The study of the techniques and the restoration of the various structural elements has been performed by using historical and traditional techniques used in the original construction period, in order to maintain the "construction history" of the building.

The fifth chapter presents the second analyzed case study: the Armstrong, Mitchell & Co. crane. This is the last remaining example in the world of a hydraulic crane of the British company Armstrong. The structure is composed by different materials such as Istria stone and stone on the base, whereas cast iron, puddle iron and wood were used for the upper part. In collaboration with expert technicians in the field, a restoration project of industrial architecture has been proposed. As for the previous case, after a preliminary analysis through a detailed historical analysis and a broad and thorough survey campaign, it has been possible to identify necessary interventions for the crane’s restoration, of both material and structure. The role of the numerical models, using advanced computer programs, has been crucial to simulate the behavior of the structure to define interventions able to affect the crane’s equilibrium with the minimum infuence.

In conclusion, Chapter 6 summarizes the main issues and conclusions highlighted and obtained in the work.

2 METODOLOGY

The research activity concerns the rehabilitation of complex constructive systems, through an integrated historical, material and structural approach.

The present work has as main objective the development and a pratical application of a method for a study of different structural typologies with a high historical-architectural content, that present different material, uses and structural behaviours.

The preservation of the historical heritage is argument topic that is increasingly sensitizing the public opinion, it constitutes a "intellectual debt" toward the rest of the world and plays a key role to preserve and emphasize the considerable economy resource that is cultural tourism and a cultural and economical patrimony for the future generations.

The conservation of architectural heritage is a very complex cultural operation, which was, and still is object of debates that greatly influenced – among many other aspects of the activity of the involved art historians, architects and engineers – the way how the crucial issue of "structural safety" is dealt with. Highly relevant aspects are: differentiation of the accepted safety level for different classes of existing structures; the use of qualitative evaluation of structural performances; the evaluation of safety based on pure equilibrium considerations; the limitations of interventions at the minimum level, depending of the required level of knowledge of the structure and of the use of appropriate investigation/monitoring techniques; the compatibility of materials and constructions techniques; the removability of the interventions. The situation is also stimulating the search for "new solutions" i.e. new materials, new techniques, new investigation tools, etc. which can offer new possibilities able to better satisfy and current design concepts. Among innovative materials highly durable stainless steel and even more FRPs (Fiber Reinforced Polymers) have high potential for possible application on historic structures, due to several well-know advantages (for FRP especially corrosion immunity, low weight, etc.) (Modena C., 2005).

The study methodology is based on the identification of the fundamental steps in order to obtain the correct knowledge of the structure (according to the study objectives) and to evaluate the extent of the required intervention. The preliminary identification phase of the subject is composed essentially by two steps: the unamnesium and the diagnosis.

The first step identifies the history of the building through a research of bibliographic sources and historical maps, the historical-constructive analysis of the manufact and the geometrical-architectural survey. Following the diagnosis, degradation, structural and material survey (study of the instability) were carried out.

This stage ends with the creation of "summarizing forms" with the intent that give immediate information for the identification of the building and its components. Also, it is a useful instrument for the future investigations and interventions, as memory of the actual situation. When completed the global knowledge phase of the structural elements of the manufact it is possible to switch to the intervention stage. It provides for the definition of intervention projects (interventions to provide safety conditions, conservation, reuse, improvement, stabilization,….) and the conservation program. For this type of study, the fundamental support are analytic and numerical structural simulations, with finite element software.

As mentioned in the "Recommendations for the analysis, conservation and structural restoration of architectural heritage" ISCARSAH (ISCARSAH, 2005) the knowledge of the structure requires information on its conception, on its constructional techniques, on the processes of decay and damage, on changes that have been made and finally on its present state. This knowledge can usually be reached by the following steps:

- a description of the structure's geometry and construction;

- definition, description and understanding of the building’s historic and cultural significance;
- a description of the original building materials and construction techniques;
- historical research covering the entire life of the structure including both changes to its form and any previous structural interventions;
- description of the structure in its present state including identification of damage, decay and possible progressive phenomena, using appropriate types of test;
- description of the actions involved, structural behaviour and types of materials;
- a survey of the site, soil conditions and environment of the building.

The visual and photographic investigations are the first step to characterize a building and to define the state of damage. Also, they allow to create a base point for the future interventions of restoration and conservation.

For the diagnostic operations it fundamental a preliminary geometric survey because it gives structural details and identifies the elements where it is necessary to do the investigations. In case there are not information it will be necessary to do a survey by means of traditional techniques, with instruments as measure, caliper, and of the innovative ones as laser-scanner, photogrammetry, etc.

Starting from a general survey, the investigation can be conduct in a more refined form for the localized parts of the structure, checking the materials and structural irregularities (Binda L. et. al., 1999). This analysis must to be done together with an study of the historical evolution of the structure to justify possible interventions over the centuries, actual discontinuities, heterogeneity and damage of the structure. This last phase is very important because in most of the monuments there are not previous exhaustive information and surveys.

In fact, history can be understood as a natural experiment occurred at true space and time scales. Historical investigations provides extremely significant information and must be considered as one of the most precious elements of the analysis (Roca P., 2005).

The information obtained until this point allows to assess the degradation to identify the structural layout of the asset. It actually helps to understand not only the structure state of damage, but also its possible causes and influences the choice of the type of surveys to be carried out afterwards, and the location of more detailed tests.

Tests can be grouped as it follows: Non-Destructive Tests (NDT), Minor destructive tests (MDT) and Destructive Tests (DT).

For a correct analysis of the degradation state of a building, nowadays are applied, whenever possible, non-destructive tests. In some cases they are applied together with destructive investigations such as material sampling for posterior physical-chemical tests performed in laboratory. In other cases, this technics are used as the only survey method.

In historical structures it is not always possible to do invasive test to characterize the structure. For this raison the more and more common trend is to use Non-Destructive Tests. These techniques allow to know the structural features about the material survey in order not to change the integrity, by keeping their use and without a sampling from the structure. It is necessary to underline that, generally, non-destructive tests do not give results directly correlated to the mechanic characteristics of the materials (stiffness, strength), whose knowledge is necessary for a good modelling of the structural behaviour of the object of the study. The Italian Circolare 617 of the 2nd February 2009 of the "Norme Tecniche per le Costruzioni" (D.M. 14/01/2008), which deals also about existing structures, reiterates the fact that non-destructive test methods are recommended to supplement those of a destructive or minor destructive nature, though not used in complete replacement of these.

When the design of the survey is previously available, then conclusions from the experimental investigation will bring to the diagnosis the real state of the structure. The following diagram (Fig. 2.1) shows which information can be available from in situ and laboratory surveys and how they can constitute the input data for the structural analysis. (Binda L. et. al., 2000).

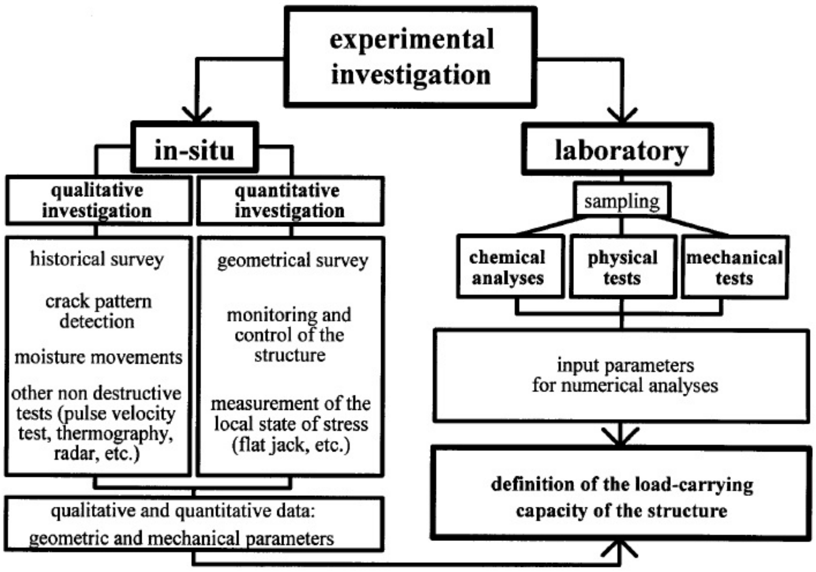


Fig. 2.1: Finalization of the experimental survey to the structural analysis (Binda L. et. al., 2000)

In the Guidelines for evaluation and mitigation of seismic risk of cultural heritage (2006), as in the Norme Tecniche per le Costruzioni (D.M 14/01/2008), the importance of the preliminar knowledge phase emerges in a direct way when it necessary define the reductive coefficient. In fact, less information there are, more elevated is the coefficient that lower the mechanical parameters of the materials. In fact, as stated on the norms (Guidelines), *"in relation to the depth of the geometric survey, the material and constructive survey, and the mechanical research regarding the terrain and the foundation, is assumed for the calculation a reductive factor that can influenced the properties of the material, reducing both the plastic model as well as the resistance or, in the case of rigid blocks, the confidence factor is applied directly to the structural capacity".*

In the following figure (F 2.2) is reported the definition (Tab. 4.1 of the *Guidelines*) of the level of the depth of research on diverse aspects of knowledge and relative partial confidence factors.

Geometric Survey	**Material and Construction Survey**	**Mechanical Properties of the Materials**	**Terrain and Foundations**
The Geometric survey has been completed F_{C1} = 0,05	Limited survey of materials and constructive elements F_{C2} = 0,12	Mechanical parameters deduced from available data F_{C3} = 0,13	Limited survey of terrain and foundations, in absence of Geological data or availability of information about the foundation F_{C4} = 0,06
The Geometric survey has been completed along with the graphic rendering of cracking and deformities	Extensive survey of materials and constructive elements F_{C2} = 0,06	Limited research of mechanical parameters of materials F_{C3} = 0,06	Geological data and information regarding the foundation structures is available; limited research on terrain and foundation F_{C4} = 0,03

$F_{C1} = 0$	Exhaustive survey of materials and constructive elements $F_{C2} = 0$	Extensive research of mechanical parameters of materials $F_{C3} = 0$	Extensive or exhaustive research on the terrain and foundations $F_{C4} = 0$

Fig. 2.2: definition of the level of the depth of research on diverse aspects of knowledge and relative partial confidence factors. (*Guidelines for evaluation and mitigation of seismic risk of cultural heritage*)

Tests about the structure integrity or the carrying capacity of a good involve some different aspects:
- valuation of safety coefficient;
- reuse of the construction or change of its use;
- evaluation of repair techniques efficacy applied to the structure or to the single materials;
- monitoring of material and structures behaviour for a long time.

The set of controls aimed to test a structure behaviour is defined monitoring. Monitoring is one of the most important activities to verify and to test the maintenance state of the historical – architectonical patrimony structures. By monitoring it is possible to obtain some information about the construction current state and about the possible development of its behaviour. Monitoring role in the study of a cultural patrimony buildings can be considered as something inside a larger application of a scientific methodology based on a multidisciplinary approach and involving the historic research, the surveys and structural modelling. (Roca P. et. al., 2008).

Monitoring is being more and more considered, in the field of cultural heritage buildings, as a key activity in order to increase the knowledge on the structural functioning of monuments. Therefore, it allows to have a deeper insight on their conditions, allowing to intervene with more confidence with a strengthening intervention, if needed, but also to prevent the execution of intrusive repair works, if they are not justified by an experimentally demonstrated worsening of the structural conditions of the structure (Casarin F. et. al., 2008).

All the information obtained and developed during the diagnostic phase allow to identify the critical points of the structure and prepare a program about material and structural intervention .

Decisions regarding the nature and extent of any interventions should be made by the team as a whole and should take into account both the safety of the structure and considerations of historic character, i.e. they must draw a balance between ensuring structural stability and preserving the cultural values of the fabric (ISCARSAH, 2005). Therefore the project must based on one of the fundamental aspect of restoration as he minimum intervention.

For the material restoration it is necessary draft a program of maintenance. Material decay is brought about by chemical, physical and biological actions and may be accelerated when these actions are modified in an unfavourable way (e.g. by pollution). The main consequences are the deterioration of surfaces, the loss of material and a reduction of strength. Stabilisation of material characteristics is therefore an important task for the conservation of historic buildings. A programme of maintenance is an essential activity because, while preventing or reducing the rate of change may be possible, it is often difficult or even impossible to recover lost material properties (ISCARSAH, 2005).

For the preparation of the essential structural intervention programs, and to calibrate them in an acceptable way, it is used structural modelling. The modelling strategy used in the intervention on historical assets can be summarized in three categories: (i) preliminary, which is essentially used to gather knowledge on the structure, (ii) detailed, used for the local and global assessment of the elements, and (iii) support to structural monitoring.

In the case of existing historic structures reliable and feasible models are difficultly available and in any case data on their mechanical properties are very scarce, generally insufficient to feed the models; on the other hand, substantial restrictions are imposed by conservation criteria to the possibilities of intervening to "increase" the safety (Modena C, 2005). But nowadays, quite a lot researches are oriented to obtain robust numerical tools capable of predicting the structural behaviour of the elements and structures, from the linear elastic range, through hardening, softening range and finally till its ultimate limit state. These numerical models, with the assumption that the all boundary and load conditions as well as parameters for their constitutive models are well determined, would make it possible to control the serviceability limit state, to fully understand the failure mechanism and asses the safety of the structures (Bosiljkov V., 2004).

The Finite Element Method (FEM) is usually adopted to achieve sophisticated simulations of the structural behavior and it is a powerful tool to study stresses and displacement in solids. it may be used to analyze localized areas or specific elements and with the complement of other techniques, may help in the structural assessment. It is not realistic to try to formulate constitutive models which fully incorporate all the interacting mechanisms of a specific material because any constitutive model or theory is a simplified representation of reality. (Bosiljkov V., 2004).

In a very complex and not yet completely defined field , as conservation and restoration of cultural heritage, it is necessary identify a general method applicable at different type of historical buildings and monuments. Currently, in this field different approaches are used: (1) a criteria based on the conservation, which puts attention to the form and to the materials of the building but less at the structural components, (2) a technical method that is more interested at the structure but, in many cases, involves over-dimensioned interventions than the real requirements, altering the original form and history of the building.

For this reason, this thesis propose to give a general operative procedure for the study, the restoration and the conservation of building with an elevated historical-cultural interests, referring to the Italian and European regulatory requirements and technical documents. The aim is strike a correct, complete and consistent balance between the two different approaches.

The proposed methodology requires an integrated and interdisciplinary approach, with the collaboration of different professional figures and private and public entities, in order to design and execute a restoration intervention based on correct scientific and conservation method. The study of ancient buildings and the design of any interventions must be carried out following procedures adequately supported by scientific insight and methodology. The scientific reasoning must include both theoretical concepts and experimental evidence. Previous experience resulting from scientifically acquired knowledge must be considered and integrated in the process leading to the diagnosis and the design of the interventions (Roca P., 2006).

The proposed methodology validation is made through its application to two complex case study with a high historical-architectonic value, very different between them, that present different materials, uses and complex structural behaviours.

The proposed paradigmatic case studies, both of them located within the Arsenale di Venezia are the Sale d'Armi Nord and the Armstrong Mitchell & Co Crane (Fig. 2.3). The Arsenale di Venezia is a location with a high historical\sociologic and architectonic value.

Fig. 2.3: View of the two paradigmatic case studies

3 THE VENICE ARSENAL IN THE HISTORY

3.1 The origin of the arsenal

The term "arsenal" has Arabic etymology meaning (*daras-sina'ah*), in Italian "the place of the work." For the Venetians, since always busy in the commercial trade with the East, this word get immediately the meaning of the place used for the construction of the boats, transforming itself following historical-linguistic process, at first, in *darzanà* then to *arzanal* and finally to the actual *arsenale*. From the dichotomy of the same term, the word “darsena” (dock) is also born pointing out the water's mirror inside the arsenal, where was developed the shipbuilding industry.

The origins of the arsenal in Venice coincide with those same of the creation of the city, even if regarding the precise dating of the first dockyard nucleus exist discordant opinions: in 1825, indeed, the Engineer Giovanni Casoni formulated a hypothesis on the date of foundation of the arsenal, based on the recovery, by himself, of a presumed medal (Casoni 1829) (Casoni 1847). The inscription affirmed that in the year 1104 the doge Falier would have founded the Arsenal of S. Martino di Castello. The authenticity of such medal is still today object of discussions, also because to us remains only a copy of the same one, made by Casoni. The first papery documents on the arsenal however, are not from before 1220, date which unable us with certainty to make reference for the presence of the first arsenal nucleus (Concina 1984).

Fig. 3.1.1: Chronologia Magna ab origine mundi (B.N.M., 1346)

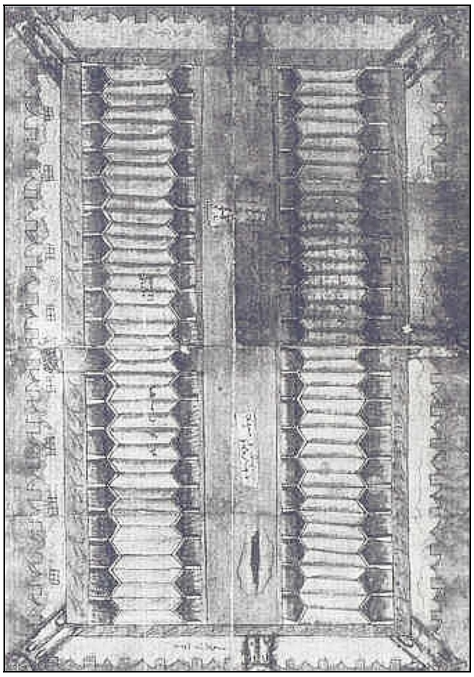


Fig. 3.1.2: Portrayal of an arsenal for eighty boats (A.S.V., 1391)

Connected to the port basin through an only channel, the Old arsenal (Arsenale Vecchio) opened in a rectangular basin on which they leaned out to comb the boatyards (called squeri), covered only afterwards, so that to assume the structure of " tesoni " (Fig. 3.1.2). The *Arsenale Vecchio* was symmetrically formed therefore from twenty-four ports open places around two twin islands "zimole", and with a wide central basin as large as the canal. Everything was closed by a wall that had only an entry toward the basin of S. Marco and that excluded in such way the north lagoon, for safety reasons (Fig. 3.1.3). In such way the arsenal is represented in the most ancient plant in Venice, that of 1346 (Fig. 3.1.1). Towards east, the lake of S.

Daniele was the most proper place for the future enlargements, with its ample half-moon bend facing the north of Venice (Pizzarello, Fontana 1983).

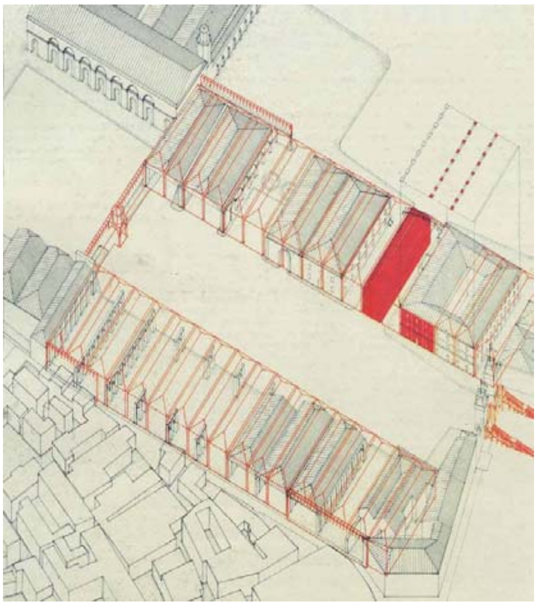


Fig. 3.1.3: Axonometric reconstruction of the original layout of Arsenale Vecchio (Gennaro, 1997)

3.2 First enlargement

The area of the complex suffers one modest addition between 1303 and 1304, when some marshy grounds were purchased on the east of the Vecchio Arsenale, bordering on the north with a mirror of water named the lake of St. Daniele. This way the New arsenal (Nuovo Arsenale) got established, where the Rio of the Stoppare (Rio delle Stoppare) opens for connecting the Old Dock (Darsena Vecchia) with the Lake of St. Daniele (Pizzarello, Fontana 1983).

The difference among the old one and the new shipyard is in a different organization of the work: in the *Vecchio Arsenale* the ships are planned and build on their port and then introduced in the central basin; in the *Nuovo Arsenale*, instead, the ships are planned and build only in the hull, and then moved in other moorings for the preparation and the armament (Albrizzi 1765). The first enlargement targets the improvement of the production of the boats‘ equipment material, without increasing the accommodation ability of the arsenal from the naval point of view (Concina 1984).

3.3 Second enlargement and the XV century

In 1326 the Municipality purchases from the monks the whole Lake of St. Daniele, widening then the New Dock (Darsena Nuova): while before the arsenal dealt an area of the dimensions comparable to those with S. Marco Square, now it articulates itself on a territory eight times greater than the one of the *Darsena Vecchia* (Bellavitis 1983). The new dock is then connected to the native one through the *Canale delle Stoppare*. In the inside they find position for the House of the Canevo (Casa del Canevo), a rowing workshop and some deposits spaces. As the previous area, also this one is contained by embattled boundaries and by turrets.

For about 150 years the arsenal doesn't suffer meaningful amplifications (Nani Mocenigo 1995) but the continuous changes and the fourteenth-century restructurings brought a strong renewal in the functions of storage, of arms building workshop, of periodic and seasonal maintenance and of reparation of the ships (Concina 1984).

Following the fall of Constantinople (1453), the Venetian Republic carries on politics of expansion of the arsenal, which assumes always the characteristics of military place. In 1456 are started some interventions of restoration of the *Arsenale Vecchio* and important works that interest the *Campagna* (area between the south bank of Darsena Vecchia), finalized to increase the weapons' production and the preparation of materials stocks (Concina 1984). They create besides in 1460 the Great Door (Porta Magna) (Fig. 3.3.1), the land's entry for the arsenal, which monument to the Republican naval victories, will be in the history enriched of bas-reliefs and commemorative statues.

In the 1463 Venice went to war in the Crusades against the Turks. It opens then a period of maximum running (Concina 1984).

Fig. 3.3.1: The "Porta di Terra" (Concina, 1984)

3.4 Third enlargement

In 1473, under the doge Marcello, the construction of the arsenal been decided Novissimo, in front of that New, toward the lagoon. This amplification was born as a result of the first great naval defeat that a Venetian never fleet had suffered aside Ottoman, the escape of Negroponte. The new dock had to constitute a fit base to maintain to disposition of the Serene one a permanent, specialized and homogeneous war fleet (Concina 1984).

The third enlargement (Fig. 3.4.1) was contemplated therefore to double the ability to welcome ships in the arsenal admitting to it a new zone of ports covered where they would have been able to find shelter seventy galee. The New arsenal was conceived, at least initially, according to an unitary consideration of functional and spatial logics, but the result was that of a series of interventions united only by the fact to be inside a same building circuit, and to be placed to the edges of an artificial dock (Nani Mocenigo 1995).

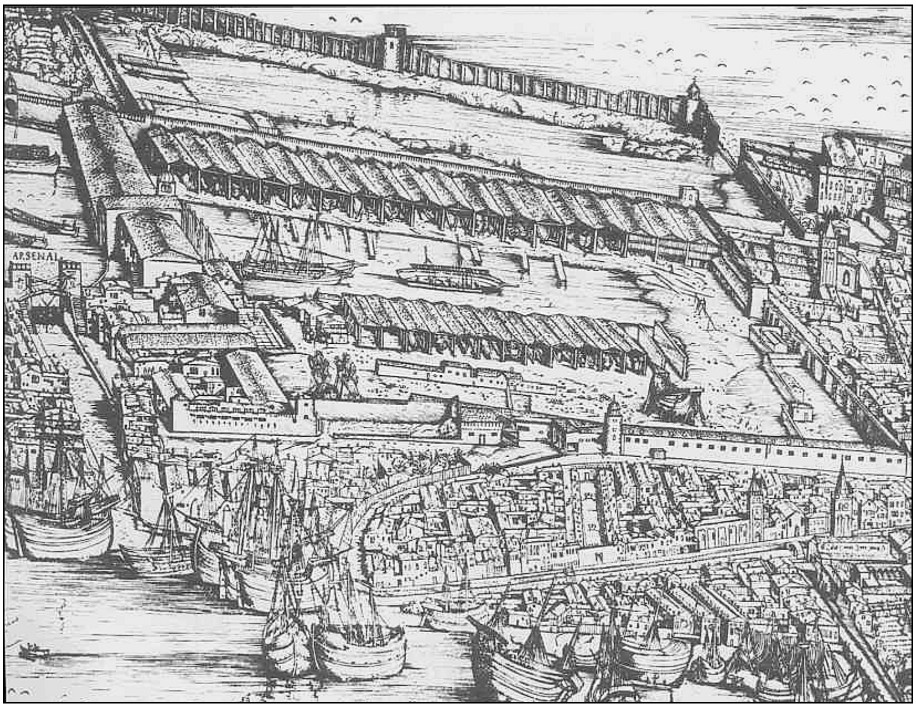


Fig. 3.4.1: View of Venice in 1500 by J. De Barbari (Bellavitis, 1983)

3.5 Fourth enlargement

The XVI century represents for the arsenal a period of acquisition of a decisive architectural configuration that places itself side by side with the strong desire of the Republic for some renewal of the *imperium maritimum* (maritime empire).

The motivations pushing to undertake new works inside the shipyard are of different nature. In primis, the Turks have begun the construction of a new great arsenal to Istanbul and, despite the fact that Venice has gotten the renewal of its privileges of commerce with the Ottoman Empire, it cannot fear other affronts. Besides, it is necessary to support the Venetian merchant politics and the new water openings in construction, will not only serve to the production of war vessels but also for commercial goals. Finally, however aspect of a notable importance, the execution of the works will employ the skilled shipyard workers of the arsenal, eluding by so, the risk of their emigration, dangerous for the escape of precious exclusive technical knowledge (Concina 1984).

While they are undertaking once again the works of the *tesoni*'s realization in the very New arsenal, three interventions of architectural character are locally performed: some demolitions are conducted in 1524 in front of the *Porta Magna*. The Field of the arsenal (Campo dell'Arsenale) is created, and goes to replace the general street (via Lata), as shown by the Canaletto in Fig. 3.5.1 and two important portals of classical style are being built (Zanelli1991).

Fig. 3.5.1: “Campo” of the Arsenale, painted by Canaletto (1743)

As regarding the fortifying aspect, between 1525 and 1528, the safety apparatus of the arsenal complex gets stronger, erecting the Tower of S. Andrea and the Angle Tower of the Den (Torre d’Angolo della Tana) (Concina 1984).

In 1535 a parcel of ground belonging to the Monastery of the Celestia is purchased, to install some millstones for the manufacturing of shooting powder and for some explosives stocks spaces (Concina 1984). In 1539 they proceeded, therefore, to perform the fourth enlargement of the arsenal (Nani Mocenigo 1995), with the above mentioned annexation of the land that modifies its perimeter. In 1569 a violent fire bursts in the new enclosure, destroying good part of the church of the monastery of the Celestia and definitely convincing the Senate to the total expulsion of the gun powders from the arsenal, which got transferred therefore to the island of Sant'Angelo di Contorta (Bellavitis 1983).

3.6 Fifth enlargement

In 1564, following the needs that had been argued to make the fourth enlargement, which we mentioned already, raised the necessity to proceed to the excavation of a channel that would unite itself together with the Old arsenal (Fig. 3.6.1): to such purpose is opened the channel of the Galeazze. This was the fifth enlargement and the last which it was decided by the Republic (Nani Mocenigo 1995).

In the final period of the XVI century the Venetian Republic needs to face once again the offensive of the Turks, who encircled Cyprus in 1570 conquering the island after eleven months. Spain of Phillip II united himself to the Pope Pio V and to Venice in a League Saint (Lega Santa), succeeding in destroying almost entirely the Ottoman fleet in the battle of Lepanto (1571). This victory however was not taken as an opportunity for architectural actions to proclaim formal magnificence inside the Venetian yards (Concina 1987).

In 1579 "the salt" Foreman Antonio da Ponte, was required to realize the reconstruction of the fourteenth-century Corderia of the Tana (Corderia della Tana): it is still today, despite the changes brought during the centuries, an exceptional space for grandeur and functional simplicity (Bellavitis 1983). With this work, the best Venetian engineer of that time magnificently concluded the sixteenth, time in which the architects intervened close to the internal dockers, but confined to follow customs and necessity, to apply forms where the technique and the rationality predominated on the invention.

Fig. 3.6.1: View of "Darsena Vecchia" and of the "Galeazze" channel

3.7 XVII century

After the great fervour of the previous century, the arsenal suffered during the Seventeenth century, a strong decrease of productivity, because of a bad politics of renewal organization, because of the war against the Turks and of the plague epidemic (Concina 1984); from the morphological point of view it seemed to settle down with its definitive form: maintenance, rebuilding of trusses and roofs that didn't damage its structure. In the 40's the building faced lacks effecting the dimensional adjustments of the boatyards of the Galeazze, and the enlargement of the weapons deposits according to the necessities. A real turn was found in the 60's, with the construction of the big war sailboats in the occasion of the conflict with the Turks for the Kingdom of Candia (1645 -1669).

In the 80's Venice owed to face different problems: the adjustment of a jail yard, the equipment system, the connection between the Arsenal and the port waters, the dimensional inadequacy of the covered ports, the scarce depth of the inside docks and the lagoon backdrops, the narrowness of the passages and of the Arsenal channels.

When, in fact, the transformations of the naval technique involved the introduction of squared sails vessels, the water entry was widened and the towers that held up the "rake" were reconstructed (1686): materials and forms got adjusted to the stylistic unity of the surrounded boundary wall, adopting a neo-Gothic language that would still have been used in the following times (Pizzarello, Fontana 1983).

3.8 XVIII century

The Most Serene Republic of Venice (La Serenissima) faces its last conflict with the Turks in 1714 and get defeated after four years of battle, after what follows a phase of total stagnation inside the arsenal. A first sign of wish of recovery is with the decision to face the problem of the Carenaggio's basins, difficult to realize due to the impossibility to do some excavations close to the foundations of the existing vaults. The selected solution, however, consists in the adjustment in relative flat forms of the hull's vessels to the low backdrops of the arsenal and the whole lagoon, consequently worsening their navigation's ability (Bellavitis 1983).

In the first years of the eighteenth century, initiatives of academic and scientific character are developed, like the creation of the School of Naval Sketch (Scuola di Disegno Navale) and the development of inherent studies, following new demands of the naval technique and the maritime traffic (Albrizzi 1765) (Pizzarello, Fontana 1983) that established a climate of notable turmoil inside the arsenal, despite the hostility manifested by the political men of the Republic toward the renovations.

The arsenal experience a phase of expansion of the commerce and the naval constructions in 1736, when the merchant harbour becomes subject to reforms of custom duty nature, so as to favour the preparation of the trade-ships called "*atte*", in order to be capable of autonomously defend themselves from the danger of the pirates. Together with the decision from the Republic to assume a neutral position in the war events of the time, such as the Seven years war, the Russian-Turkish wars and the American and French revolutions, the construction of the ships "*atte*" furnishes a new reason for activity to the arsenal, restoring the native role of the yard as for the construction of merchant boats (Bellavitis 1983) (Concina 1984).

In 1736, for the necessity to increase the stock spaces, and to improve the seasoning of the lumber, standing posts rather than stacked, the workshop "Officina degli Squadratori" is being built up. The native aspect of the building is no more legible because of numerous and imposing restructuring works at the end of the nineteenth century in the channel of the Galeazze (Bellavitis 1983) (Concina 1984).

The climate of renewal strikes again on all of the arsenal aspects. Indeed in 1768 is organized the first topographical survey of the complex and a program of restoration and rearrangement of the weapons deposits starts out. The spaces destined to the organization of the metals are being handled, and finally it is decided to act on the problem regarding the port exits of the boats, planning to renew the docks and realize a connection between the yard and the port's mouths (Bellavitis 1983). The arsenal's perspective which the Mediterranean lighthouse of the naval technique is being noticed by the mathematician Gian Maria Maffioletti, who in 1797 withdraws it as a yard in full activity in the suggestive perspective plant brought in the Fig. 3.8.1 (Bellavitis 1983) (Zanelli 2004).

During the initial phases of the arsenal's transformation, in a technically modern compound, in 1797 the general Napoleone Bonaparte, on the command of the Italian army, hands war to Austria passing for Veneto and declaring war to the death to the Republic of St. Mark from the Palmanova's fortress, after the only war action of the Venetians against French. On the 12 May of 1797 occurs the end of the Most Serene Republic of Venice (la Serenissma) and four days later the Bonaparte's French troops occupy the arsenal in Venice (Bellavitis 1983).

From this moment on, up to 1866, in Venice French and Austrian governments alternate (Bellavitis 1983) (Concina 1984).

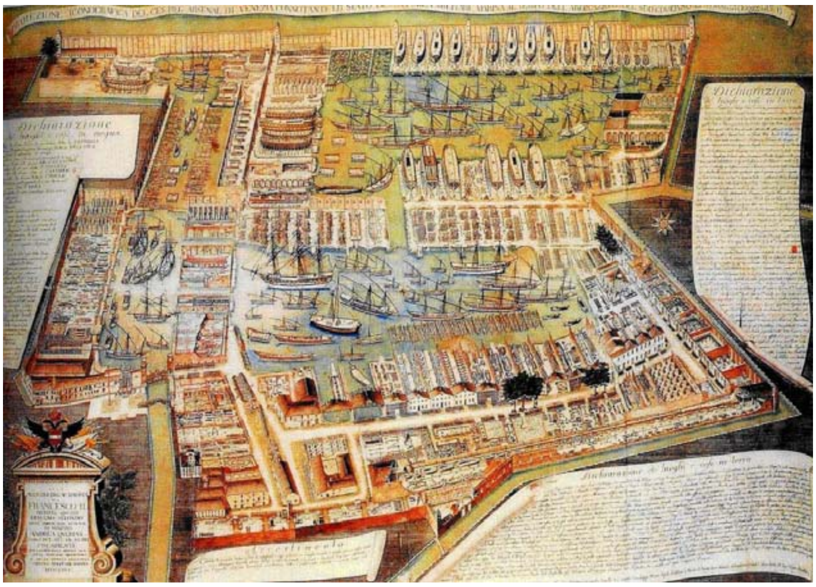


Fig. 3.8.1: Scenographic plan of the Arsenale by G. B. Maffioletti (M.S.N., 1797)

The following year, in fact, the city is assigned to Austria with the Campoformio treaty. During the Austrian occupation, that will last up to 1806, conditions are put to order in which pours the arsenal after the tragic seizure of the Frenchs without contributing however to its resumption limiting itself to restore the military character of the yard. The Austrians choose, indeed, to strengthen the commercial development in Trieste, neglecting instead the one of Venice (Bellavitis 1983) (Concina 1984).

3.9 Sixth enlargement

The resumption of the arsenal took place in 1806, when the Frenchs occupied Venice for the second time: the principal objective was to adapt the structures of the arsenal for the constructive formality of the French harbor.

In 1809 the Artillery Department got separated by the rest of the establishment, through the closing of the Portal of the Stradal Campagna: the access in the arsenal of Earth was only possible through a door that leaned out on a wooden bridge on the Canal S. Daniele (Concina 1984) (Pizzarello, Fontana 1983).

In 1810 the sixth enlargement of the arsenal is started: after the suppression of the religious orders the convent, the church and the garden of the Celestia were annexed; besides the New Door (Porta Nuova) was reopened, close to which the Tower Alberaria was built.

During the second French domination, besides the important physical transformation, the arsenal is interested into the presence of foreign technicians who impose the adoption of new constructive technologies, putting an end to the epoch of the overseer’s empiricism, quite extraordinary for quality and longevity (Zanelli 2004). Among the professional figures emerge those of engineers Prony and Sganzin. The first one is the author of the cadastral survey’s formulation in Venice, that for the first time, make visible all the connections between the arsenal and the surrounding building fabric, and of the exact plan, geodesic of the Venetian lagoon, where all the channels appear among the arsenal and the port of Malamocco.

The second French occupation concluded, in the 1815 Venice returned to belong to the Austrian domination, and in this period various activities effected tented to rearrange some activities, to complete some of the works still running and to the restoration of some of the arsenal parts. Besides, the mechanization of the activity was introduced, and the installations were adjusted towards the steam navigation system. Nevertheless, from the building point of view, this was mainly a handling phase, that left the arsenal in its native configuration.

The nomination of engineer Giovanni Casoni as Architect of the Maritime Factories and of the Hydraulic Works coincided with the resumption of the building’s interventions inside the arsenal: important works of consolidation in various parts of the compound, and in 1834 they were projected two new *tesoni* on the area of the Celestia, at the beginning of the Channel of the Galeazzes (Casoni 1847). A Room of weapons is reconstructed; a park of artillery is recomposed at the beginning of the Stradal Campagna, and the workshop Fabbri (Officina Fabbri) is built on the square close to the Foundries (Bellavitis 1983).

The engineer Casoni, architect at the Maritime Factories and engineer at the Hydraulic Works, will be for forty years the figure of reference for every architectural and hydraulic intervention at the arsenal (Zanelli 2004).

In 1835 in Venice a society of privates is constituted for the construction of a railroad that would connect the city to Milan, through the realization of a bridge on the Venetian lagoon, which will be concluded in the 1846. The work covers notable importance both for the city and the arsenal, which will definitely be shift back in comparison to the new line of traffics (Bellavitis 1983).

In full nineteenth-century climate of the independence’s wars, also Venice in 1848 knows its revolt, conducted by Daniele Manin. The insurrectionary leader, on the wake of the ancient power of the

Serenissima, individualize the arsenal as a crucial objective and drives an extreme resistance that brings the city to be the last one forced by the marshal Radetzky on August the 30th 1849 (Bellavitis 1983).

Again subdued to the Austrian government, Venice lives a period of economic resumption stimulated by the same emperor Francesco Giuseppe who allows the Venetian middle class to foundate the Venetian merchant Establishment to restore the movement in the port (Bellavitis 1983). In the arsenal the works of modernization of the yard are suspended, limiting the interventions of the Casoni to an ordinary maintenance and a static improvement, as it happens for the shed of the Department Novissimetta (Fig. 3.9.1) (Zanelli, 2004).

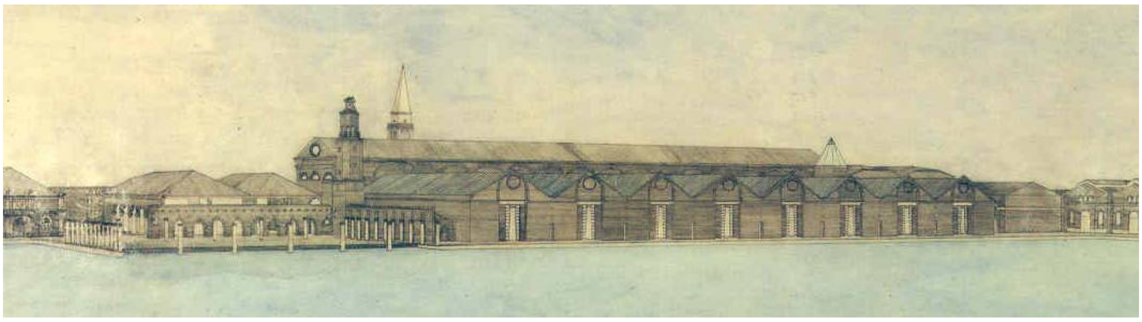


Fig. 3.9.1: View of the Novissima's buildings (Gennaro, 1997)

3.10 Seventh enlargement

In 1861 Vittorio Emanuele I becomes king of Italy, but the Veneto region remains up to 1866 to the Austrians, when the plebiscite decides for the annexation to the Kingdom. The permanent comity for the defence of the State establishes that Venice would be due to remain a fortress and a naval station of first importance; therefore it is decided to use the arsenal for the development of the maritime power of the Country (Bellavitis 1983).

At the end of the XIX century the condition of the Italian navy is rather behind in comparison to the rest of Europe, and inadequate towards the demands of the modern shipbuilding industry. The introduction of the motor propensity and the diffusion of the hulls in iron and of the armoured ships happened at the beginning of the nineteenth century; find in the arsenal an entirely inadequate yard deprived of special spaces and structures. In order to handle the necessities dictated by the evolution of the shipbuilding industry, new slipways and launching were to be realized for the ships with metallic hull, basins for the maintenance to the dry land of the boats, laboratories endowed with modern machineries, stock spaces, loading docks and above all a new wet dock with correct dimensions and suitable backdrops, since the new ships have in general greater dimensions in comparison to the traditional vessels (Menichelli 2004) (Martin, 1877a) (Martin, 1877b).

Taken knowledge of the lacks of the Venetian yard, in March of 1867 the general Chiodo introduces a project of motto for the rearrangement of the arsenal in Venice (Fig. 3.10.1). In the plan he foresees the achievement of a water basin of the dimensions of 330×230 meters through the union of the New docks (darsene Nuova) and the Nuovissetta docks, the construction of two basins of dry docks of 110 meters in the Novissimetta and of five ports, out of the enclosure of the arsenal, directly on the lagoon. The project involves the demolition of the Gaggiandres of the Squadratoris and of the brims of the islet. The new project is rejected by the Navy Department in how insufficient to the necessities the shipyard industry (Bellavitis 1983) (Menichelli 2004). The variation introduced by the general Chiodo foresees the reduction of the number of ports from five to two and that of the basins to one only. The solution designed offers a new connection among aqueous spaces, open and covered with earth, with a notable increase of these last mentioned in comparison to the first ones, underlining the change of the request of a modern arsenal contrary to the past ones (Menichelli 2004).

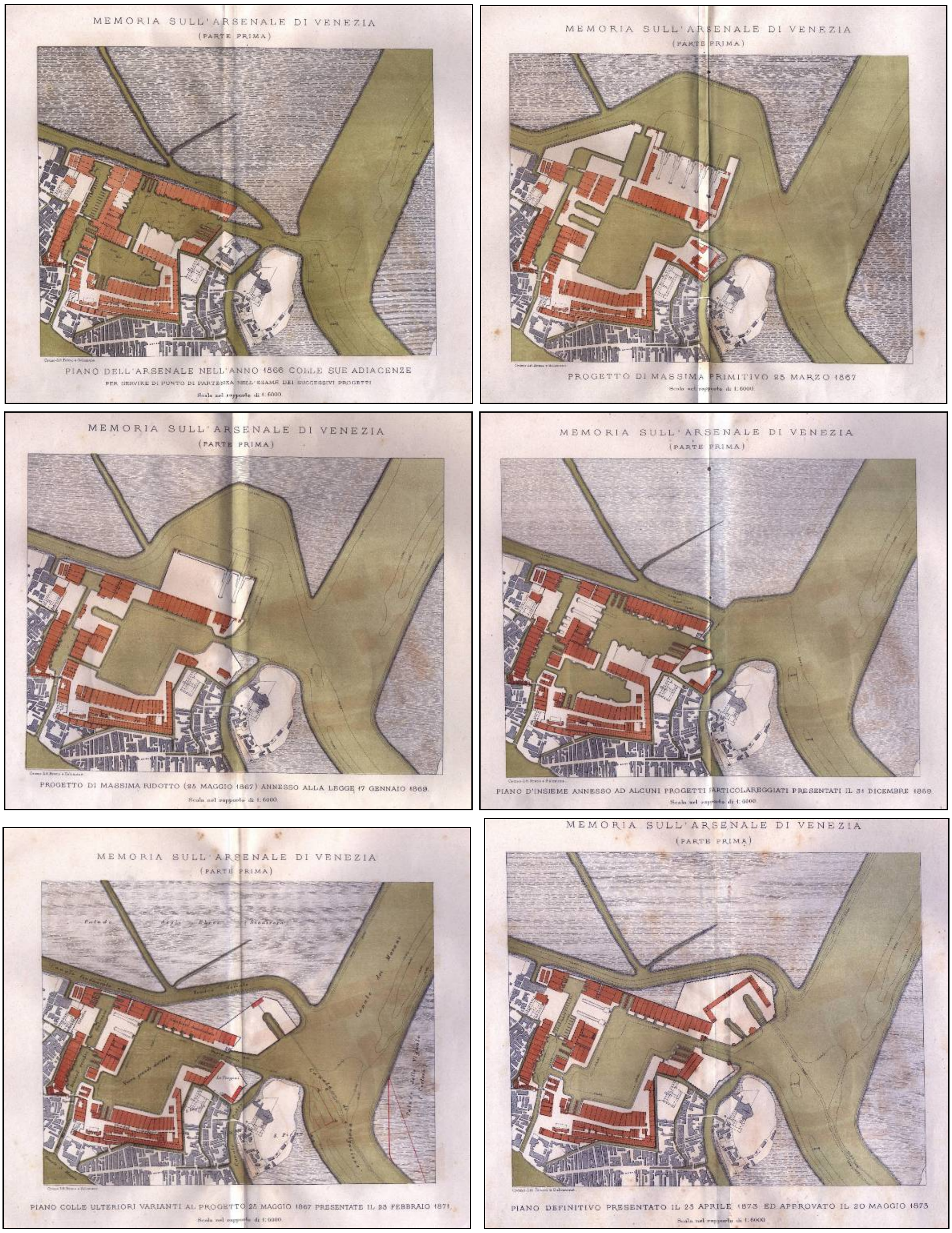


Fig. 3.10.1: Projects for the enlargement of the Arsenale of Venice (Martini, 1877a)

The variation is positively appraised by the special commission, constituted by the Consiglio Superiore of the Navy and by some members of the Genius Committee (Comitato del Genio), but in the executive phases it receives strong objections from the colonel Giani. As head of a direction extraordinary of the Venice Military Genius (Genio Militare a Venezia), he shows that the project of Chiodo is based on a wrong hydro-geological study (Bellavitis 1983) and exposes the risks for the stability of the buildings following the project of excavations of the dock of constant 10 meters of depth (Menichelli 2004). Giani introduces, therefore, two substitutive projects to be realized maintaining the strip of the islet as a separation of the two docks and limiting the excavations to 8,5 meters on the level of the sea for the central part and to 7,5 ms behind the loading docks. The proposal of Giani foresees the realization of a large dock to the north, outside the enclosure of the arsenal, the creation of a careening basin in the island of the Vergini and two open ports in the area of the Novissimetta (Bellavitis 1983) (Menicheli 2004).

Also the projects of the Giani are opposed and replaced by the ones of the colonel MorandI, took over with a new project proposing the two building-slips mounted among the Department Novissimetta and the Squadratoris, the only basin of careening on the island of the Vergini and the unification of the docks, bringing the depth to only 8,5 meters. The Department of the Navy (Ministero della Marina) understands the practicality of the proposal and definitely approves the plan in 1873, whose contract is given to the Venetian Society (Società Veneta) (Bellavitis 1983). May 10th 1873 the greatest Felice Martini compiles the general plan for the arsenal (Fig. 3.10.2), fixing definitely the lines of intervention in the yard. With the beginning of the works difficulties referring to the realization of the careening basin of the Vergini emerge soon, for which it is chosen in alternative the Swamp of the Hebrews (Palude degli ebrei), an area to be buried on the north east of the New Door (Porta Nuova). When the works are already started, it is decided to realize a second basin, smaller (Menichelli 2004).

Fig. 3.10.2: General plan by Felice Martini (1873) (Dina, 2004)

In 1875 begins the seventh enlargement of the arsenal (Bellavitis 1983). Some of the boatyards are demolished, the ones on the east of the Galeazzes, the one more to the south, for the construction of the first port and the northern portion of the building of the Squadratori, for the realization of the second open port. In 1876 it begins the stripping of the islet in the plan corresponding to the three east brim plants of the connection channel the two docks, that in 1880 will be completed because of the numerous difficulties met during the demolitions of the foundations' structures (Bellavitis 1983) (Menichelli 2004).

The works in the Square of the Basins (Piazzale dei Bacini) are made, with the construction between 1875 and 1878 of the two careening walled-up basins in the northern zone of the New Door's Tower (Torre di Porta Nuova) (Bellavitis 1983). These works represent an absolute novelty for Venice, for the considerable dimensions of the whole plant and for the unusual aspect of the great stone staircase, and made for welcoming the ships (Menichelli 2004), closed by the innovative device of the boat it brings which is presented on some sketches of project in the Fig. 3.10.3. The realization of the careening basins requests the construction of the brokes of the select zones through side walls, the formation of squares in which to insert them and the relative operations of burial around them. In such way, the whole area of the basins gets a strong characterization, also for the choice of the language neo-gothic employed in the building of the "machines of exhaustion" and in the turrets (Menichelli 2004).

In the same period, together with the ports, the basins and the unification of the two docks, several interventions contribute to the global redefinition of the arsenal compound. The works made for the improvement of the navigability and the general functionality of the compound refer to the raising of the ground floor of 1.20 meters on the level middle sea to face the problem of the high waters, to the formation of squares, to the widening of the various centers, to the formation of wharfs and the realization of the inside railway system and revolving bridges for speed the moves among the various departments. For the updating and the increase of the shipbuilding system, vapour machineries, the crane and the lifting machines are introduced.

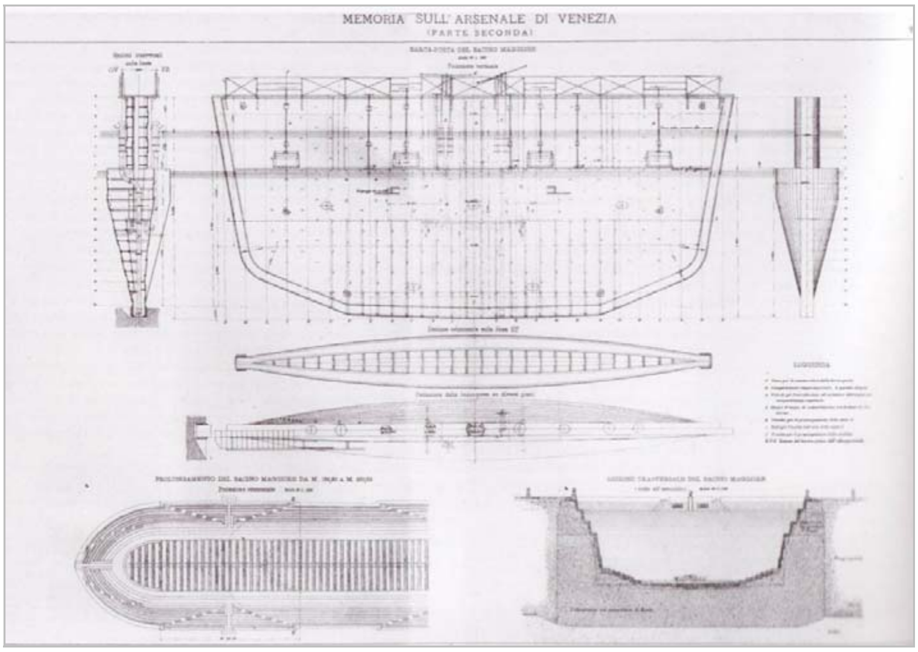


Fig. 3.10.3: Sections and projections of the boat located in the 19th century basin, by F. Martini (1897) (Dina, 2004)

In 1883 the hydraulic crane Armstrong Mitchell & Co. is installed on the oriental wharf of the Great Dock (Darsena Grande) (Fig. 3.10.4), today unused and only sample remained of the crane of the same typology, and symbol great technological innovation of the nineteenth century. New buildings are realized and they are done some works of general improvement of almost all the existing buildings, comparatively to the coverage and the building equipments (Menichelli 2004).

Fig. 3.10.4: The Armstrong, Mitchell & Co. hydraulic crane in the “Darsena Grande” of the Arsenale of Venice

3.11 XX century and eighth enlargement

In 1908 is made a further amplification of the arsenal toward north east for the realization of the third basin of careening, which will be concluded in 1915 and tested in the 1917. The work overhangs for a large ampleness the others two and represents the peak of the transformation’s process of the productive ability of the arsenal initiated in the 1870. In fact, for physical dimension, installation’s efficiency and topographical location, the added parts undertake a prominent and autonomous role from a functional point of view, limiting the Great Dock (Darsena Grande) to the simple function of a service and refuge area (Bellavitis 1983) (Menichelli 2004) (Pizzarello, Fontana 1983).

During the first world war the arsenal lives a period of intense activity. In 1916 the eighth and last enlargement is made, with the build-up of a “salt marshed” zone posted externally between the boundaries of the great Novissima (Novissima Grande) and those from the nineteenth-century raised to isolate the area of the two first careening basins. On this triangular area some buildings are being built lined up, the barracks, in order to give lodging to the ones from submarines in the epoch in which the arsenal still works as naval base of support. Subsequently, the barracks will be assigned to the Italian Navy Military staff in service in Venice.

Concluded the war, part of the arsenal is surrendered to the private industry, so as to develop the construction of the merchant shipping in substitution of the one lost during the war period. In 1920 a serious fire in the western brims of the Old arsenal destroys completely ancient manufactured artefacts that won't be rebuilt anymore. In 1921 the first official centre of the Naval Historical Museum is inaugurated in the old weapons rooms, next to the land entry (Bellavitis 1983).

In this period, the arsenal reduces its own activity, also due to the fact that the construction of the military ships is always more often contracted by the private yards. In 1932, in fact, the Military Navy gives in concession to the private shipbuilding enterprise CNOMV the areas of the careening basins for the construction of ships both merchant and military goals, while the areas not immediately connected to the Great Dock (Darsena Grande) assume a more marginal role of administrative management and representation. The functional distinction of the zones inside the arsenal remains unchanged during the second world war and becomes more radical in the fifties, when the command of the maritime military Department of the Alto Adriatico is transferred to Ancona, and in 1957 the granted areas to the CNOMV are expanded up to the sheds of the Nuovissima (Bellavitis 1983).

While the arsenal stops every productive activity, the Italian Military Marina withdraws itself in the south part and a big number of buildings is abandoned: it begins the slow impoverishment of the arsenal yard. In 1964, the entry inside the boundaries on the north of the Galeazze’s basin is opened, to allow the passage of the public transports (Bellavitis 1983).

In 1962 the Venetian commune gets equipped with General Regulator Plan, inadequate however to the resolution of the accessibility problem in the arsenal (Bellavitis 1983). Two years later, the Detailed Plan is adopted for the zone east Arsenal-Castle that foresees the retraining of this part of city through the opening of part of the arsenal for civil uses, locating there a series of public equipments of urban and extra-urban level. The new services planned for inside the arsenal, for instance university classrooms, laboratories of research, conferences rooms, and Congresses Building, theatres and others, would be installed in the buildings overlooking the Rio of the Galeazze. The north east part and the Nuovissima would have welcomed economic activity, and in the remaining parts would have been kept for military use (Bettiol 2007).

From 1982 the collaboration among corporate authorities like the Defence Offices, the Office of the Department of Internal Revenue, the Office of the Cultural Goods and the Commune of Venice have stimulated studies and projects of new handling and exploitation of the arsenal, understood as urban sector endowed with a very finely defined perimeter (Bellavitis 1983). In the immediate following years, in fact, the Venetian University institute of Architecture starts a complex interdisciplinary search on this theme, extremely delicate, for the necessary combination among memory and innovation (Dina 2004).

From 1983 the Superintendence B.A.P. of Venice and Lagoon is directly implied in the recovery of the architectural patrimony of the arsenal, through a program that has as priority the improvement of the roofs’ restoration without neglecting the other aspects of the maintenance. The Corderie (Rope making workshop) have been the first building in which the Superintendence has intervened, starting thay way a phase of study of the arsenal and programming. Some of the most representative architectural-sculptural manufactured articles in the arsenal have been subject to restoration: the Door of Earth (Porta di Terra) and the portals of the Artillery’s Department and the Weapons’ Rooms. From the end of the eighties interventions have been realized on the roofs’ structures of the Gaggiandre, of which a section is visible with mass’ works in safety in the Fig. 3.11.1, some Artilleries and of the northern and southern brims of the islet. In the overall restorations the roofs’ of the Superintendence have concerned over 20.000 mqs of structures. Currently the interventions of restoration are centred on the Weapons’ Rooms and on the hydraulic crane Armstrong Mitchell & Co. thanks to the interest of the “Venice in Peril” Fund (Menichelli 2004).

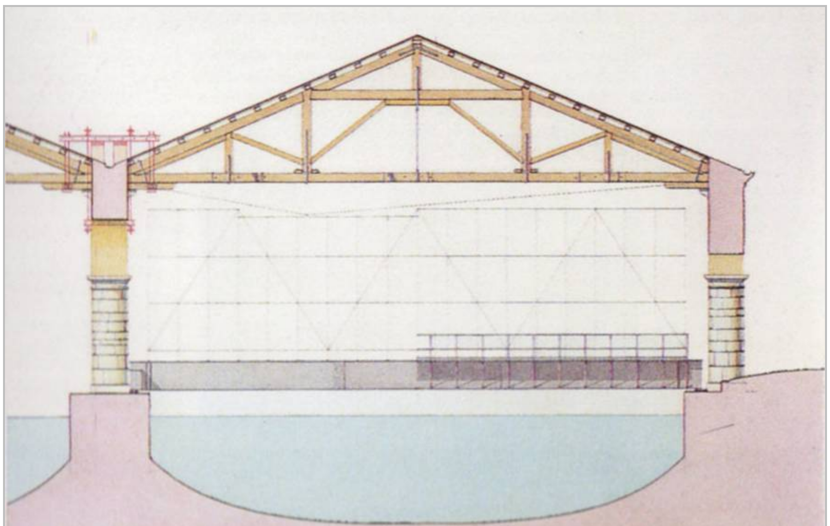


Fig. 3.11.1: Gaggiandre: plan of interventions to provide safety condition, by M. Piana (Dina, 2004)

At the beginning of the 90's of the last century, when in Venice we would experience the beginning of a crisis apparently with no return, also the future of the arsenal risks to get lost between uncertainties and inadequate assignments. Its close bond with the urban tissue, however, has allowed that, with the resumption of the city in the last fifteen years, we have been able to put the arsenal inside a project for Venice. The objectives to be realized relating to the arsenal compound consist in recognizing, in preserving and in developing its peculiar characteristics (Dina 2004). Currently the arsenal is object of a Detailed Plan whose priorities concern the relationship between maintenance and transformation, the individualization of the new functions and the system of the relationships (Dina 2004).

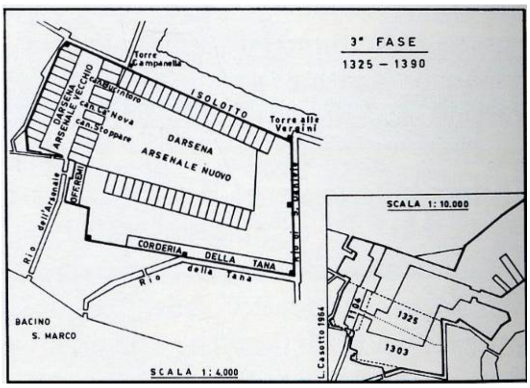
The Project Arsenal is undertaken in 1998 with the participation by the Venetian Commune to a ministerial proclamation that would have financed the activities of technical and planning support and of territorial recovery programs denominated PRUSST (Program of Urban Retraining and Sustainable Development of the Territory). It has been developed through a collective collaboration between local corporate body and subjects present in the arsenal, and it finds concreteness in the indication of a range of functions and their possible aggregations in poles today. The search pole on the environment themes, on the theme of maintenance and exploitation of the cultural patrimony and their communication, was inaugurated in the north area. The pole of the production is today active in the field of the shipbuilding business, in the north east area of the compound, the area of the careening basins. The pole of the culture, of museums and of the exhibition, activity mainly developed in the south area of the arsenal by the Biennial corporate organism. The pole of the Marina that welcomes in the west south area the competent activities for the Military Marina dealing with important institutions such as the Library and the school of Maritime Military Studies. Finally, close to the thematic areas simplified in the poles, the activities of support are individualized widespread, to the service of the future consumer, who can be visitors, workers, researchers or residents (Dina 2004).

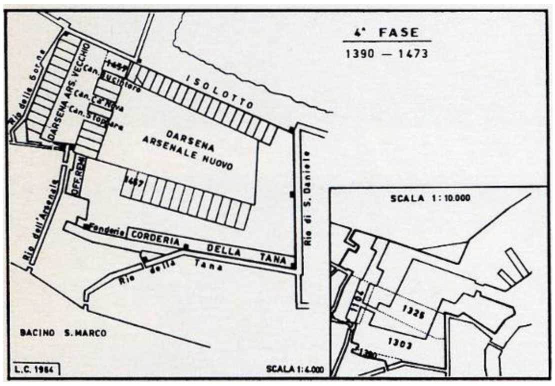
From the end of the 80's, without solution of continuity until to today, we have been facing the interventions on the roofs' structures for the Gaggiandres, the Artilleries, the north Brims of the islet, the south Brims of the islet and the North Weapons' Rooms (interventions in progress). Overall, the interventions of the Superintendence on the roofing, including those realized on the Corderie, have interested over 20.000 mqs of structures.

3.12 Conclusive remarks

The transformations undergone at the arsenal in Venice between the XII and the XXI century are been structured based on a deep historical search as well as from the sequence drawn by the photographic file from the Venetian commune presented in Fig. 3.12.1.

This last part contains a table summarizing the constructive phases of the arsenal in Venice.





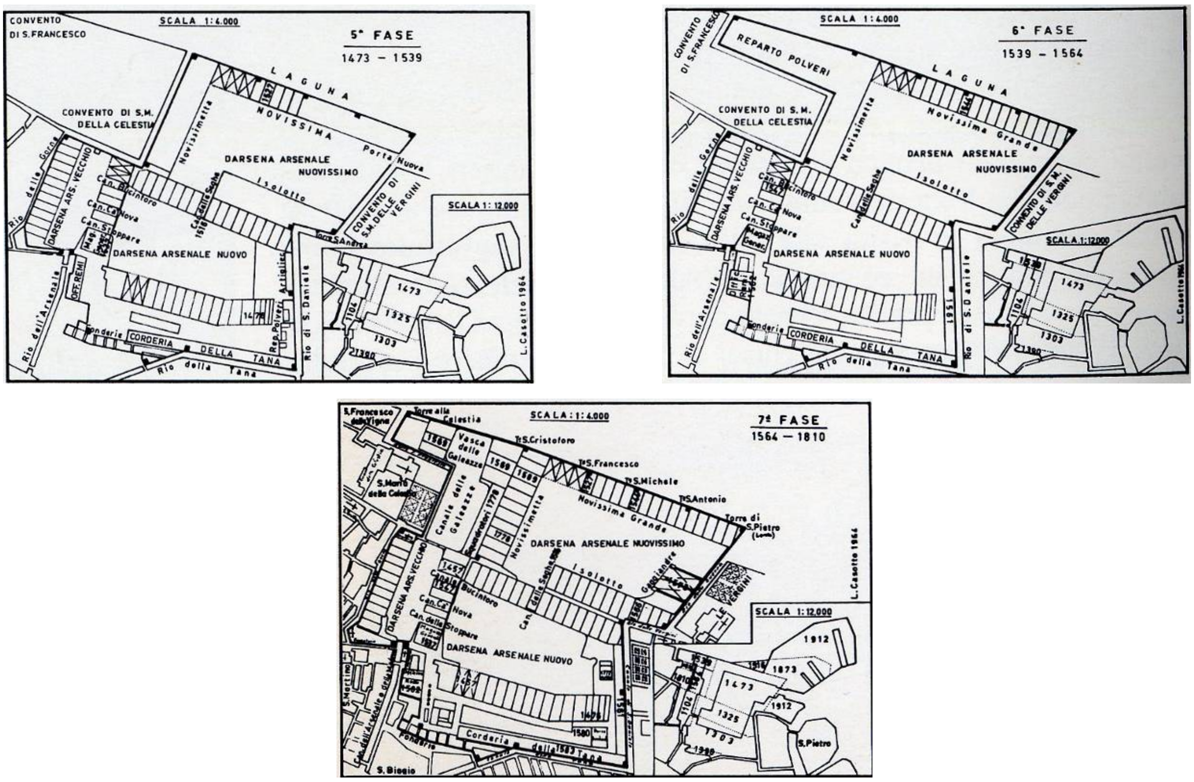


Fig. 3.12.1: Historical Sequences of the expansion and of enlargements of the Arsenale, from the origin, in 1104, to 1810, Photographic Archives – Municipality of Venice (Chirivi, 1976)


FASI COSTRUTTIVE DELL'ARSENALE DI VENEZIA
LEGENDA
ARSENALE VECCHIO
1303 PRIMO INGRANDIMENTO
1325 SECONDO INGRANDIMENTO
1473 TERZO INGRANDIMENTO
1539 QUARTO INGRANDIMENTO
1564 QUINTO INGRANDIMENTO
1620
1810 SESTO INGRANDIMENTO
1875 SETTIMO INGRANDIMENTO
1909
1916 OTTAVO INGRANDIMENTO

28

4 *SALA MAGGIORE* (MAIN ROOM) OF THE *SALE D’ARMI NORD* (NORTH WEAPONS ROOMS) OF THE VENICE ARSENAL

4.1 Foreword

With the aim of evaluating the effectiveness of the proposed methodology, referred to the interdisciplinary evaluation of cultural heritage building to properly tune the restoration interventions, the so called “West” or “Main” Room (Sala Ovest o Sala Maggiore), pertaining to the complex of “Sale d’Armi Nord” (North weapons rooms) in the Venice Arsenal.

The building is of relevant interest both for its location within the Arsenal of Venice, a complex of remarkably high historic/sociological/architectural value, and for the attention paid to it since several years by the “Soprintendenza B.A.P. di Venezia e Laguna” (local office of the ministry of fine arts responsible for the conservation of the cultural and landscape heritage of Venice and its lagoon), which is financing part of the described restoration interventions. The final goal of the *Soprintendenza* is the complete restoration of the building in order to make it accessible to visitors, with museum functions.

The *Sala Ovest* was subjected, in the past centuries, to several repair interventions and partial reconstruction, which changed the original lay-out of the building. The lack of maintenance of the last decades induced moreover local collapses of part of the roof which, in their turn, quickened the deterioration processes of most part of the structural elements.

The building is characterized by the presence of different building materials, like brickwork masonry, for the vertical elements, and wooden floors and roofs; a considerable portion of the roof is also supported by of reinforced concrete trusses.

For the study of the building, the key steps of the proposed methodology are followed. The first step for correctly comprehend the behaviour of a cultural heritage building is the historical analysis, which allows to understand the structural modifications intervened and the changes in the original configuration, caused by more or less intrusive structural interventions carried on in the centuries, necessary for the re-utilization of the building for different functions, or by intervened new construction techniques, collapses or fires.

The second step, in parallel with the historical analysis, is the evaluation of the present-day conditions of the building, which will allow to contextualize the construction within the Arsenale, and to identify the vertical and horizontal load bearing elements. This will be the starting point for the geometrical / material / damage / structural evaluation of the building.

The geometry of the building will be identified by means of traditional and up to date techniques: collected data will be then merged in order to obtain all of the relevant information necessary for the precise geometrical definition of the construction. On site surveys and, where possible, tests on materials are finalized to the identification of the main elements of the structure, to the evaluation of the decay of materials and structural damage, and to define the chemical and mechanical characteristics of materials.

At the end of the preliminary phase, for a complete but prompt comprehension of the studied building, some synthetic forms will be filled out, containing, besides the geometrical survey, detailed information on the individual structural elements and on their current conditions (e.g. level of deterioration, etc.).

All of the gathered information will allow to identify the critical spots of the structure, from a structural and material degradation point of view, and they will be the starting point for the definition of a conservative restoration design, which must includes the individual execution phases, the techniques to apply and the description of the interventions on the different elements by possibly using traditional techniques, used at the time of construction of the building, in order to maintain the *constructive history* of the same good.

All of this must be done according to the basic principles of restoration, as the reversibility of interventions and the minimal approach.

For the definition of a design intervention respectful of the ancient building methodologies it is necessary to deeply understand the traditional techniques used at the time of construction. A state of the art assessment tool is in any case to be used, and this can be done by using software programs based on the Finite Elements Method.

For the studied building it will be necessary to implement both global and local (single structural element or assembly) behavioral models. The firsts will describe the deformations and corresponding stresses of the building as a whole, whilst the seconds will permit to define in detail the stresses on the elements emerged as critical during the initial phase of knowledge. Corresponding structural verification will be then performed according to the applicable national and international standards, and the most suitable interventions for each studied case will be selected and numerically tested.

The choice of the type of intervention, both from use of materials and structural point of view, will be based upon the intended aim of the restoration intervention. In the case under evaluation, the final goal is to restore the building to its original configuration, however adapting the intervention to the new intended use of the building.

Three different restoration approaches will be applied in the case study under consideration, by however applying the same methodological evaluation process, which then can be generalized: reconstruction, conservative restoration and structural strengthening. For all of the three cases similar evaluation procedures will be set up, always paying attention to the re-use of original materials and to the original structural configuration of the building.

4.2 Historical backgrounds

The Sale d'Armi (Rooms of the Arms) could be defined as architectonical structures dedicated to the shelter and showing of the armoury and battle implements. These structures were settled in the urban area of the city of Venice during the period of the “Serenissima” Republic (XII th century). In order to rule over the seas and to maintain its maritime dominion, the manufacturing of special battle implements and different arms turned out so essential for the State. Therefore the Republic was compelled to locate shelter-structures where deposit, guard and preserve the battle and arms booty treasures. The historical surveys have brought us to identify three main settlements, dedicated to this aim:

- the Sale d'Armi located in the Palazzo Ducale (Doge’s Palace);
- the Sale d'Armi located in the area of the Arsenale Vecchio (Old Arsenal);
- the Sale d'Armi located in the area of the Arsenale Nuovo (New Arsenal).

The submentioned survey building is inside of the unit of the *Sale d'Armi* belonging to the Arsenale Vecchio.

It could be difficult to try underlining singly historical episodes of the *Sale d'Armi*, from all the events referred to the whole Arsenale and the city of Venice itself.

So, after a deep analysis of the historical-structural events of the whole complex of Arsenale, it was decided to include and produce other parallel details, which are not a matter of only the *Sale d'Armi*, but they are concerned with a particular zone so called: Artillery Detachment of the Land Arsenal (Reparto Artiglieria all'Arsenale di Terra) (Fig. 4.2.1).

It could be underlined that the reason why all few details we found about the Sale d'Armi is for their few historical backgrounds. After the first centuries of the history of the Arsenal, as matter of facts, emerging many problems concerning with its site-safety and with all the implements located inside it.

Therefore since 1500 A.C the whole Artillery Department located in the Arsenale di Terra (of Earth Arsenal), is protected jealously by secrecy, so for a long time is faded away from all the maps all over distributed, till the end of the eighteenth century when with Venitian view-painting (Vedutismo), the image-secret was broken.

Fig. 4.2.1: Localization of the buildings of the Artillery Department (Gambirasi, 2009)

4.2.1 The Artillery Department

The first written evidence that testifies the production of weapons within the site dates to 1322 AC, when along the southern rim of the Arsenale Nuovo arise the dealers' shops, the archers, lancers and cuirassiers, the so-called "*sagittarium stadium*"[1]. Towards the second half of the century, so that the Arsenale can hold the State monopoly in the manufacture of weapons and to guard the military secret about the new inventions of war, in this place is concentrated the embryo production of new weapons, "*guns and mortars*". These factors indicated that the first foundries were built in the *Campagna* (Concina,1984). *Campagna* means the area between the south bank of New Dock (Darsena Nuova) and the building *Corderie della Tana* of the fourteenth century. In this area, significant works have started in 1456, aimed at increasing the production of weapons and at doing ready stocks of materials necessary to implement a policy of strengthening the Arsenal, to which Serenissima Republic sets out after the fall of Constantinople (Concina, 1984).

[1] Concina E. (1984), "L'Arsenale di Venezia. Tecniche ed istituzioni dal Medioevo all'età moderna", Electa Ed., Milano, p. 36.

In the three rooms "*three restyled halls*", by January 1458, mentioned on the cards of the Senate, built for storage of different weapons including "*many fine copper mortars*" [2] and various materials useful armament of fifty "*galee*" (typical venician boat), you can identify the core of the system the old Rooms of the Arsenal, at the mouth of the *Stradal Campagna*, near the ground entrance (Concina, 1984).

Between 1524 and 1532, were built new forges and foundries, choosing the right area of the Old Foundries (Fonderie Vecchie) on *Campo della Tana*, since already closed on three sides and then fitted with the minimum expenditure, which prevents direct access to the city by the workers. From this point on, it is ordered that all the artilleries in bronze, to be done in the city, are carried out only in new buildings arranged in the Arsenale (Concina, 1984).

The story that leads between 1591 and 1592 (Concina, 1984) to the construction of the portal of the *Stradal Campagna*, visible in the drawing below (Fig. 4.2.2), a symbolic door-end of the Artillery Department, the most secret and jealous fence of the whole Serenissima Republic, began in 1518.

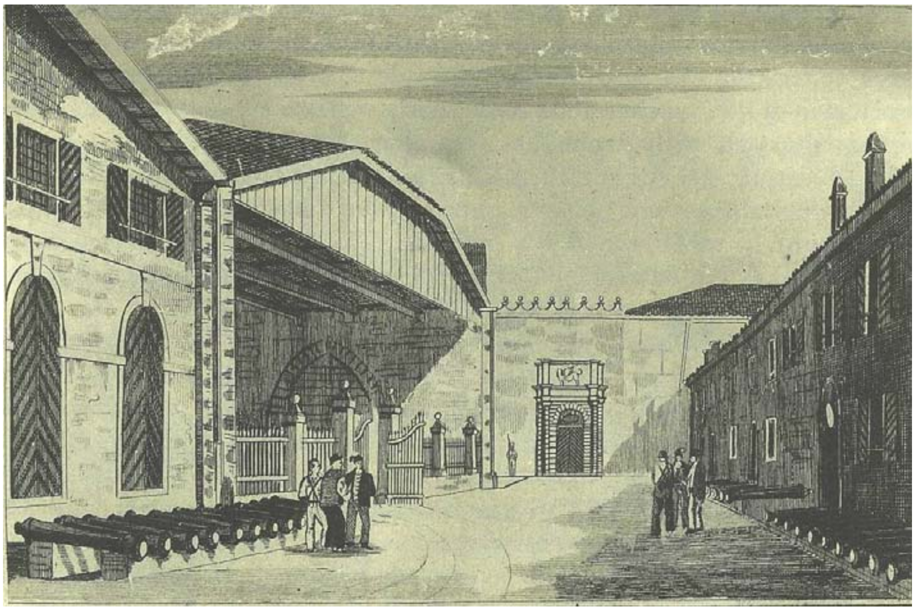


Fig. 4.2.2: Terminal part of the "*Stradal Campagna*" and of the portal of the Artillery Department, engraving of the beginning of nineteenth century (Zanelli, 1991)

Later, in 1538, the number of arms in *Palazzo Ducale* doubles and they are placed under the direction of specialized "*proti*" engineers. The relationship between the two Armouries is intensified when in 1541 it was decided to send, to those Rooms in the Arsenale, a huge number of old weapons in order to be fused to obtain new ones (Bellavitis, 1983).

The importance of the *Sale d'Armi* is growing: in 1549 is delivered to the "*Proti*" a frieze painted on canvas, from the new Library Hall of the *Palazzo Ducale*, to place it on their inside. The significance of this measure is clearly related to the fact that in1545 the College was created over the Maritime Militia chaired by the five judges who controlled all the military weapons of the State: the two Superintendents on the arms, the two Superintendents above the Arsenal, the Superintendent over the artillery (Bellavitis, 1983).

With this, the rooms of weapons in the Arsenal, with their "proto" specialized, directly submitted to the Superintendent over the Artillery, became the symbol and the epicenter of the huge flow of arms that fuels the strongholds of the State of land and sea, in addition to the fleet. These are practical reasons that led to the development between 1591 and 1592 the Door of Artillery, a classical masterpiece that identifies an area performed within a space in silent and secrecy of the Arsenale (Concina,1984) (Bellavitis, 1983).

[2] Concina E. (1984), "L'Arsenale di Venezia. Tecniche ed istituzioni dal Medioevo all'età moderna", Electa Ed., Milano, p. 51.

The south-east area of the Arsenale is made up of weapons deposits and gunpowder. The beginning of the safety problem of the site, started in 1476 with the explosion of mortars gunpowders, placed presumably in some building of the *Stradal Campagna*. Since 1500, these dangerous gunpowders have been allocated and moved into fence enclosed workshops, next to the walls of *San Daniele*, but in 1509, broke out a great fire, which destroyed all the walls and damaged most part of the department. The Senate decided, indeed, to keep here only the artillery or guns that were not subjected to fire damages, choosing to move gunpowders in the north -western part of the site (1539) (Concina, 1984).

Around 1561 the long building of Artillery was rebuilt along the eastern walls of New Darsena (New Dock), a work that recalls the architecture of Venitian Palladio's villas in Veneto for rhythm and order of the façade, made of exact cells, illuminated by two large doors and two windows. The arches are characterized by a slight rusticated embossed stone with lion heads on keystone (Bellavitis, 1983). The same finishing is applied to the four bodies of the Northern Halls of Arms, especially on the side to the Darsena (Dock), and in 1580 was erected the wall that surrounded the first open-air park of Artillery (Fig. 4.2.3).

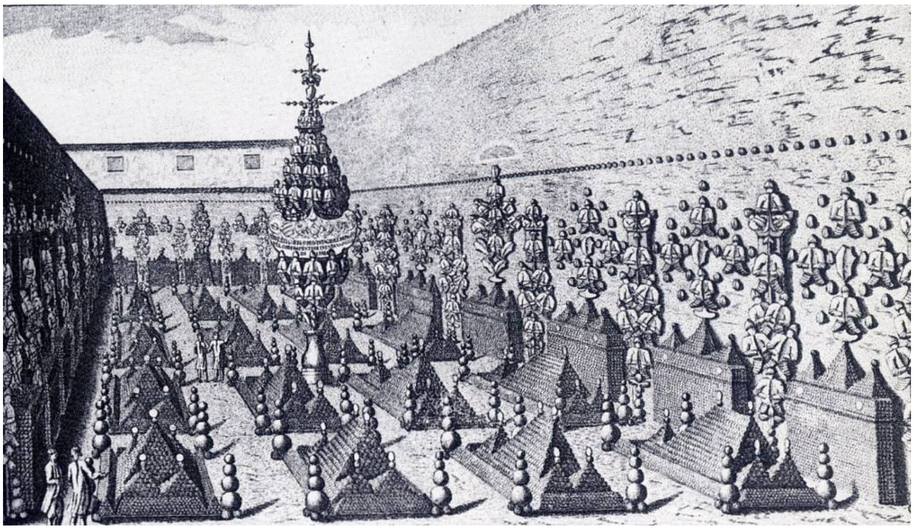


Fig. 4.2.3: The first open-air park of Artillery, engraving of the eighteenth century (Nani Mocenigo, 1995)

The entire Department of Artillery is completed in this way and orderly, while Arsenal is affected by the expansion of the fifth enlargement. In the late eighteenth century is made an additional main door in the Department, it is an arched doorway leading to the staircase to reach new weapon rooms (Bellavitis, 1983), they consist of six very large rooms (Nani Mocenigo,1995), as shown in Fig. 4.2.4.

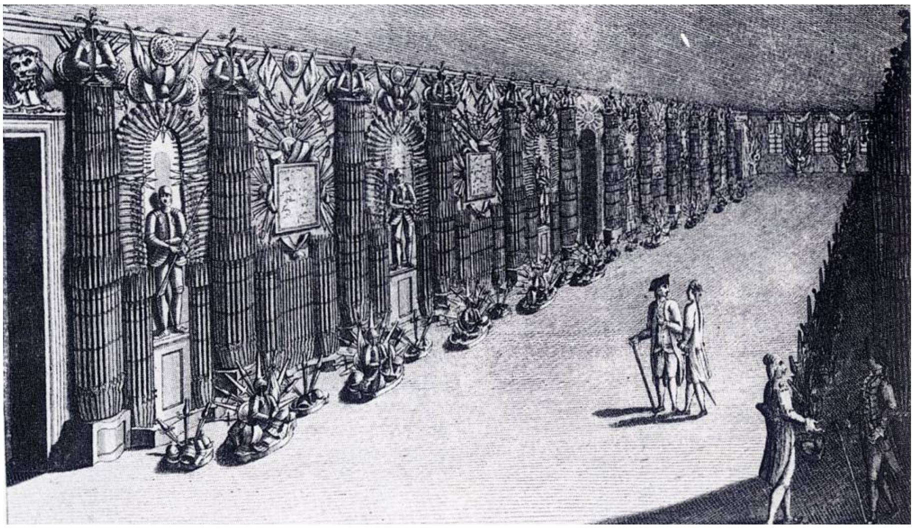


Fig. 4.2.4: Interior of one of the "*Sale d'Armi*", engraving of the eighteenth century (Nani Mocenigo, 1995)

The main entrance to the southern rooms of arms of the Arsenal is the only baroque architecture, and it was almost certainly designed by the architect Filippo Rossi, the main-door is marked with a sharp stone and rusticated embossed with a pattern on the lunette that recall a weapons spoils (Fig. 4.2.5) (Concina, 1984).

Fig. 4.2.5: Baroque portal at the entrance of the southern "*Sale d'Armi*" on the "*Stradal Campagna*"

Fig. 4.2.6: Entrance portal of the northern "*Sale d'Armi*" on the "*Stradal Campagna*"

In the middle of the eighteenth century, the Department became the headquarters of the Venitian Regiment of Artillery. At the end of the Serenissima Republic, the French came into Arsenal and sacked the precious collection of weapons, being brought to France and there probably fused (Nani Mocenigo, 1995).

In 1809, during the second French occupation, the Artillery Department is completely separated from the other part of the Arsenal and was made a special door on the boundary wall. Through a wooden bridge over the Rio di San Daniele, the Department may be connected to the city (Nani Mocenigo, 1995). The workshops and warehouses are used exclusively for the land-artillery and the industry changes its name to Arsenale di Terra (Earth Arsenal), being an independent organization inside the Arsenal itself.

In 1866 with the annexation of Venice to the Kingdom of Italy, the whole department is reconnected to the whole of the Arsenal and used for the construction and maintenance of small arms and artillery, small battle implements, as well as for service of underwater weapons (Nani Mocenigo, 1995). The buildings in this sector are the subject of restoration that maintain the look of the sixteenth century (Bellavitis, 1984). The history of the Artillery Department now becomes one with the Arsenale.

Currently the complex is in an important state of neglect and consequent deterioration, and is partly restored by the *Soprintendenza BAP di Venezia e Laguna*.

4.2.2 The *Sale d'Armi* inside the Artillery Department

The making of special rooms for the Arsenal weapons dating from the early sixteenth century and it is very interesting that a few decades later they made rooms for ammunition at the *Palazzo Ducale*, controlled by the

Consiglio dei Dieci (Council of Ten). The *Sale d'Armi* are architectural structures and for the deposit and showing of battle implements, according to storage systems, as shown in Fig. 4.2.7. The Halls of Arsenal are of considerable importance for the Republic since there will be placed huge amounts of artillery and weapons necessary to maintain the Venitian Serenissima State of Land and Sea, as well as for equipment and rigging of the Venetian fleet (Bellavitis, 1983).

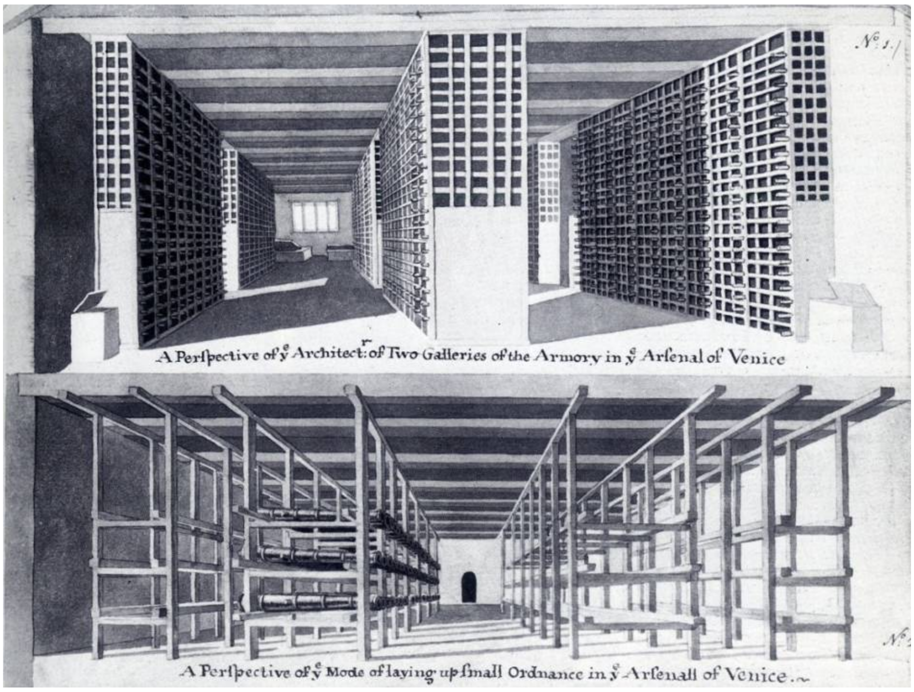


Fig. 4.2.7: Warehousing systems of arms and artilleries at the Arsenale of Venice, painting by E. Dummer (1686) (Concina, 1984)

The *Sale d'Armi*, as already mentioned, are located inside the Artillery Department, located, in its turn, in the south east side of the Arsenal. They develop into two distinct groups: *Sale d'Armi Sud* (Southern Halls of Arms) and *Sale d'Armi Nord* (Northern Halls of Arms) (Fig. 4.2.8).

Fig. 4.2.8: The "*Sale d'Armi*" north (green) and south (red) of the Artillery Department

The *Sale d'Armi Nord*

The complex of the *Sale d'Armi Nord* , which we can see an aerial view here below (Fig. 4.2.9), overlooks the south side of New Dock (Darsena Nuova). It is surrounded on two sides, east and south, by the *Stradal Campagna* and the third side to the west, is joined and faced to a maritime warehouse of the New Arsenal (Arsenale Nuovo). The buildings have solid brick perimeter walls, while inside, depending on the building, are pillars of different materials in support of the first floor. So, if the West Hall, called also Main Hall (Sala Ovest or Sala Maggiore), has brick and iron pillars to support floors, wooden and iron beams and brick vaults, the other three are characterized by iron pillars and concrete supporting floors made of hollow tiles and iron beams.

Fig. 4.2.9: South-east view of the complex of the "*Sale d'Armi Nord*"

The perimeter wall is made of brick; the doorways and windows, topped by round arches, are framed by blocks of Istrian stone. The first room is known as the *Sala Maggiore*, for the larger size than the other three. All structures, in length, are crossed on the ground floor by the inside railway track, used for warehouse functions and storage equipment and artillery.

It's very interesting to underline that the group of halls is a single complex till now connected.

Upstairs on the first floor, the four buildings are connected through the corridor, which is accessed from the lounge of the *Sala Ovest*, now dilapidated. The historiography said it was "*living room equipped with modern weapons, exquisite taste and decorum*" [3] where they received welcomed kings and notable personalities visiting the Department of Artillery of the Arsenal. Also on the ground floor, despite the large arches in the walls are now walled perimeter, the openings in the courtyard of the *Sala Ovest* use to achieve the adjacent and adjoining buildings.

The group of four rooms overlooking the *Darsena Nuova* is difficult to date for the contradictions that exist in the consulted sources. According to some, the *Sale d'Armi Nord* existed before 1458, the date we found written proofs about the existence of the *Sale d'Armi* of the *Arsenale Vecchio* (Concina, 1984), but were simply used as sheltered or indoor sites (Tosato, 1988). A memorial tablet on the façade facing the *Stradal Campagna* of the *Sala Ovest* of the complex bears the date 1476, together with three coats of arms that could indicate the construction date. This hypothesis needs to be treated carefully because, as that wall was clearly rebuilt, the stone was placed thereafter, may be in another time (Bellavitis, 1983). Important detail, however, on the age of construction of the complex, comes directly from the perspective map of Venice De Barberi of 1500 (Fig. 3.10.3), where instead of the *Sale d'Armi Nord* appear only two walls under

[3] Casoni G. (1829), "Guida per l'Arsenale di Venezia", Antonelli, Venezia, p. 112.

construction, thereby allowing them to go back to the monumental arrangement of the second half of the sixteenth century carried out by the proto architect Da Ponte.

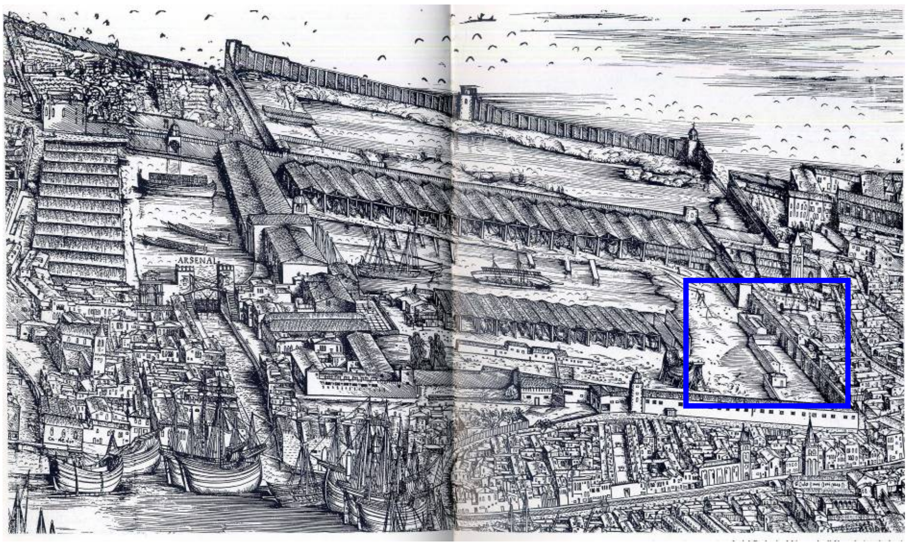


Fig. 4.2.10: View of Venice in 1500 by J. De Barberi (Bellavitis, 1983)

The Halls underwent first restoration work simultaneously with the reconstruction of Artillery in 1561. On the arches of access to the *Darsena* and on one arch to the *Stradal Campagna* we find the same finish with molded embossed lion heads, placed on the keystone (Fig. 4.2.11) (Bellavitis, 1983).

Fig. 4.2.11: Elevation of the "*Sale d'Armi Nord*"

The *Sale d'Armi Nord* are the subject of a reorganization for the managing of the Museum of Artillery, that in 1772 was set up in front of the complex, on the dock in front of the *Darsena Nuova*. On the detail of the scenographical projection of the Arsenal, made by Maffioletti, reported in Fig. 4.2.12, could be noted the museum and axonometric views on the Artillery Department, including the *Sale d'Armi*.

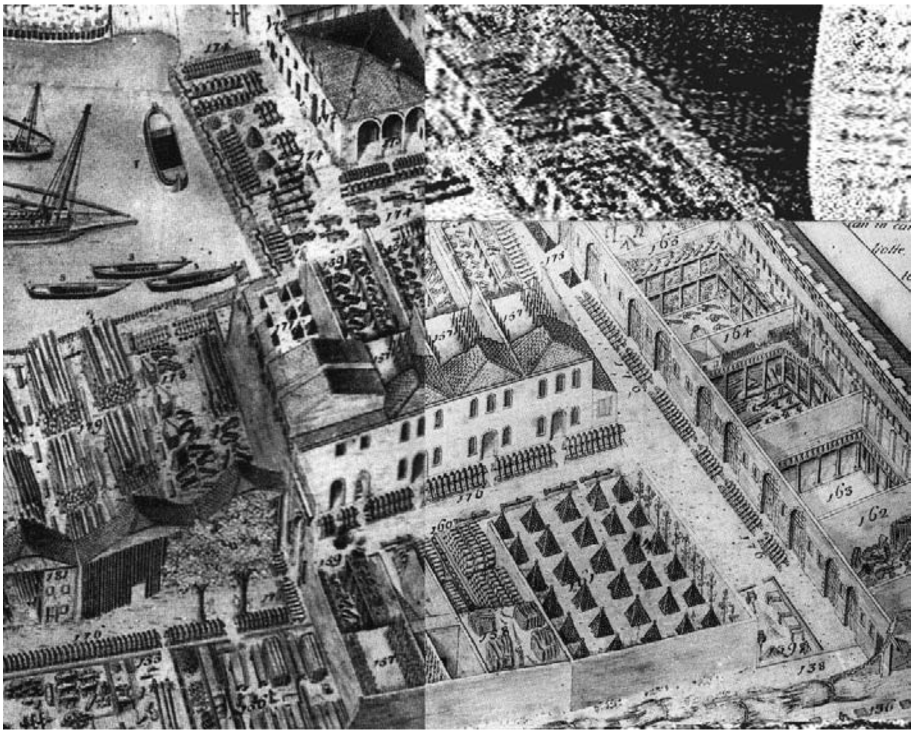


Fig. 4.2.12: Detail of the perspective map of the Artillery Department, by G. M. Maffioletti (1797) (Concina, 1984)

From the plan can be seen that the *Sala Ovest* is used for the storage of patterns of artillery, while in the other three rooms there are the armouries (Bellavitis, 1983). It is also clear that in this period, the *Sala Ovest* is different from the other three for the type of roofing. Of higher roofing and characterized by a pitched roof and a pitch on the façade for the *Sala Ovest*; of lower height and pavilion roofed for the remaining three halls. The unique shape of the roof of the *Sala Ovest* is likely due to the greater measures of the building and also, where they welcomed and received many kings visiting the Artillery Department.

On the late nineteenth century, with the refurbishing General Plan, reconstruction was done on the halls by changing strongly the structural setting: the intermediate floors are restored except the *Sala Ovest*, they inserted metallic columns on the ground floor (Tosato, 1988). The first floor rooms are divided into compartments of same size, used as offices of the Headoffice of Artillery. In the same works, they changed the southern façade of the *Sala Ovest*, aligning all of the rest of the buildings (Tosato, 1988). All the buildings of the Arsenal are the subject of restoration works of roofs and of brick masonry equipment, in which modern devices are adopted to improve the structural setting. These actions significantly change the structural model of the roofs (Menichelli, 2004). It could be, though not confirmed by any source, that the roofing of the *Sala Ovest* has been rearranged and adjusted conforming as type as the adjoining roofing of the adjacent rooms.

Currently, the roof structure of the *Sala Ovest* is made of wooden truss, sixteenth century styled, and reinforced by concrete structures, dating back to our Thirties.

The *Sale d'Armi Sud*

The three rooms, in some literature sources, are also known as Sale Nuove (New Halls) considered constructed before than the other halls. Arranged in front of the whole complex of the Sale Nord, on the other side of the *Stradal Campagna* with respect to the former halls, they border to the south on the *Corderie* building, as noted by the Fig. 4.2.1.

An interesting theory states that the original core of the *Sale d'Armi Sud* has nothing to do with what we see today. The plan, represented only in the plan of the Venice preserved at the National Library in Paris, is dated to the mid-seventeenth century, between 1653 and 1655: the rooms appear longitudinally oriented, in the same direction as the Corderie della Tana (Ropes Factory of the Tana) (Tosato, 1988). The complex would then have been destroyed by a fire in 1728. The *Sale d'Armi Sud* were thus reconstructed using the same axis of the northern complex and assuming its present appearance. But this information can not be regarded as true, because of traces of marble found in the sixteenth Park of Bombarde rather be assumed that the *Sale d'Armi Sud* also them could be dated back to the sixteenth century. This is also confirmed by the structure of buildings and the presence of barrel vaulted roofs and cross vaults. In the restoration of the arms depots, started in 1767, was built a "Big vaulted door, in order to stand out by proportioned ornamental decorum the Main Hall, used to welcome and serve foreign Princes" [4] (Concina, 1984), from which you have access to the six areas where are visible artistic compositions of arms (Nani Mocenigo, 1995). They too, like the Sale Nord at the end of the Serenissima used for storage of weapons (Bellavitis, 1983).

It is interesting to underline that most of the limited information on the *Sale d'Armi Sud* are derived from observation of the pictures of the Arsenal. So, if the complex currently consists of three buildings in the scenographical projection of the Maffioletti dating back to 1798 (Fig. 3.8.1) there are only two roofings, but in another source from the same author it could be noticed that the space between the two buildings is occupied by a stair leading to upper floors (Fig. 4.2.12). The central space, as matter of facts, remains open and unroofed until the end of the nineteenth century, when they made again the roofs for the other two rooms (Bettiol, 2007). According to other sources, the insertion of the new building, between the two from the eighteenth century, dates when Fascism provides reconstruction of roof structure of the building complex with a pavilion roof made of reinforced concrete beams and metal rods (Tosato, 1988), currently still existing. The unit consists of a perimeter wall of bricks; openings, topped by round arches are separated by brick elements interspaced by blocks of Istrian stone.

[4] Concina E. (1984), "L'arsenale di Venezia. Tecniche ed istituzioni dal Medioevo all'età moderna", Electa Ed., Milano, p. 227.

4.3 Present - day conditions

Along with historical analysis, was carried out a study and a research aimed at defining the present-day conditions of the *Sala Maggiore* of the *Sala d'Armi Nord*.

In particular, the vertical and horizontal elements have been identified and described, that characterize the building itself. Herebelow we show the geometrical survey of the building, containing plans, elevations and sections. A detailed description of horizontal and vertical structures of the *Sala Ovest* will follow. The study of the foundations did not focused since it was not considered relevant in the present study. In fact, the structure does not present any obvious damage from settlements and the loads transmitted to the ground are modest. We focused instead on the analysis of floors that have a state of advanced deterioration and to partial collapse.

4.3.1 Geometrical survey

In order to perform the necessary considerations for the identification of individual structural elements, for the knowledge of the building, should have a clearer picture dimensional of the structure. For this reason it was done a geometrical survey of the *Sale d'Armi* of the *Sala Maggiore*, through technique with laser-scanner.

The group of buildings *Sale d'Armi Nord* (Fig. 4.3.1), characterized by a total roofed area of approximately 2660 m^2, consists of brick buildings in the longitudinal walls, close to each other, such as the arsenal traditional type of "*tesoni*". Each building has its own plan, developed in a north-south direction, with differents size and geometry. The first three halls from the east side are the most similar. The plants, which occupy areas between 565 m^2 and 622 m^2, have a trapezoidal shape with a length of about 48 m, and an inside width varying between 9,80 m and 12,50 m.

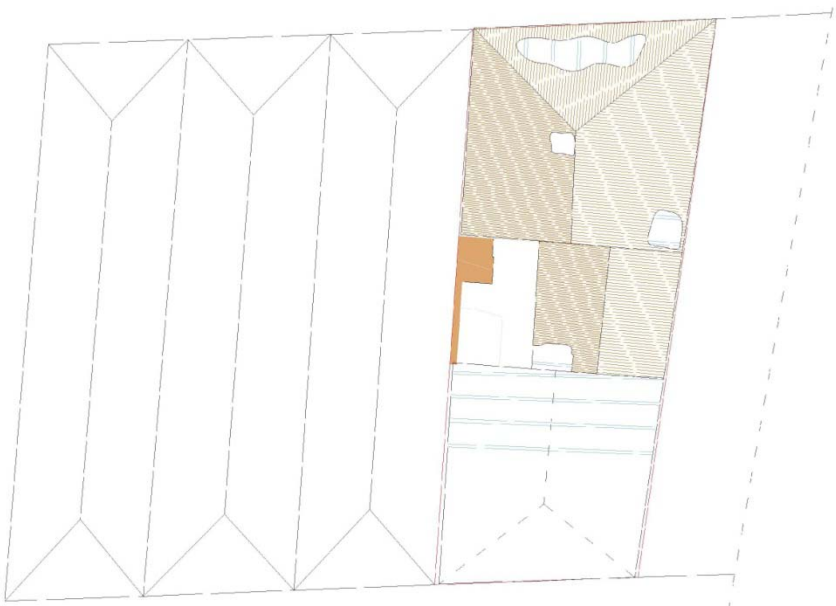


Fig. 4.3.1: The complex of the "*Sale d'Armi Nord*"

The last building, the *Sala Ovest*, has a larger floor plan, with around 875 m^2.

The building extends from the *Stradal Campagna* towards the *Darsena Nuova* for a length of about 48 m.

The plan is irregular in shape, next to a trapeze. The longitudinal walls, in fact, on the plan, move away from an inner width of 15,60 m on the northern side to an equal width of 19,10 m on the southern side.

The east side wall is shared with the next building of the *Sale d'Armi Nord*, while the west one with a building whose roof, which is at a lower level than that of the room, ends with a longitudinal skylight. Only in this room is possible to find an inner courtyard that leads to a sharp decrease in the width of the building but provides more natural light.
Below you can see the plans of the ground floor (Fig. 4.3.2) and those ones of the first floor (Fig. 4.3.3), elevations (Fig. 4.3.4) and sections (Fig. 4.3.5) of the building.

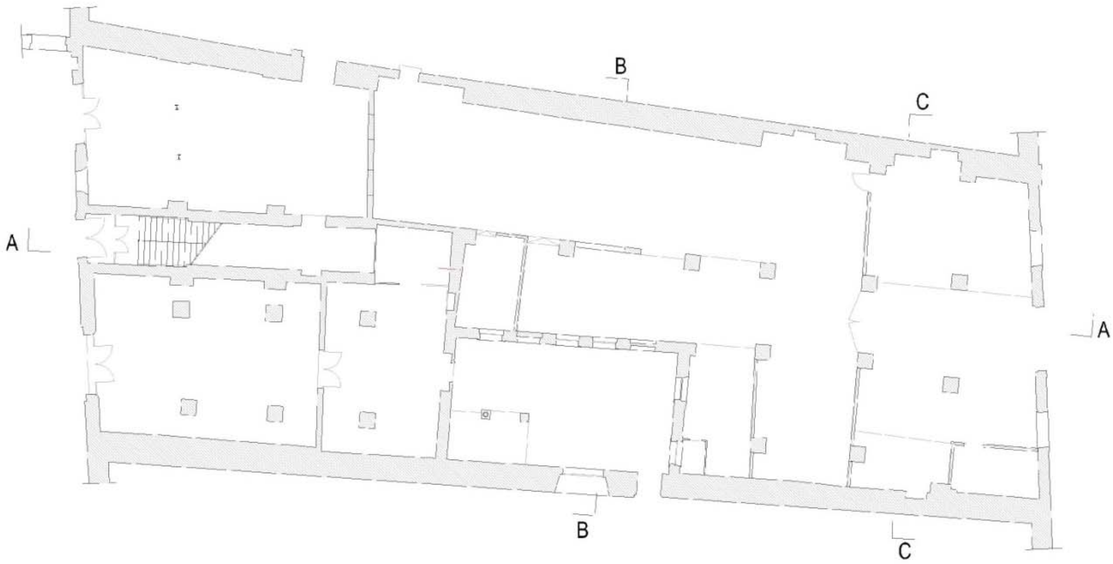


Fig. 4.3.2: Ground floor plan of the "*Sala Maggiore*"

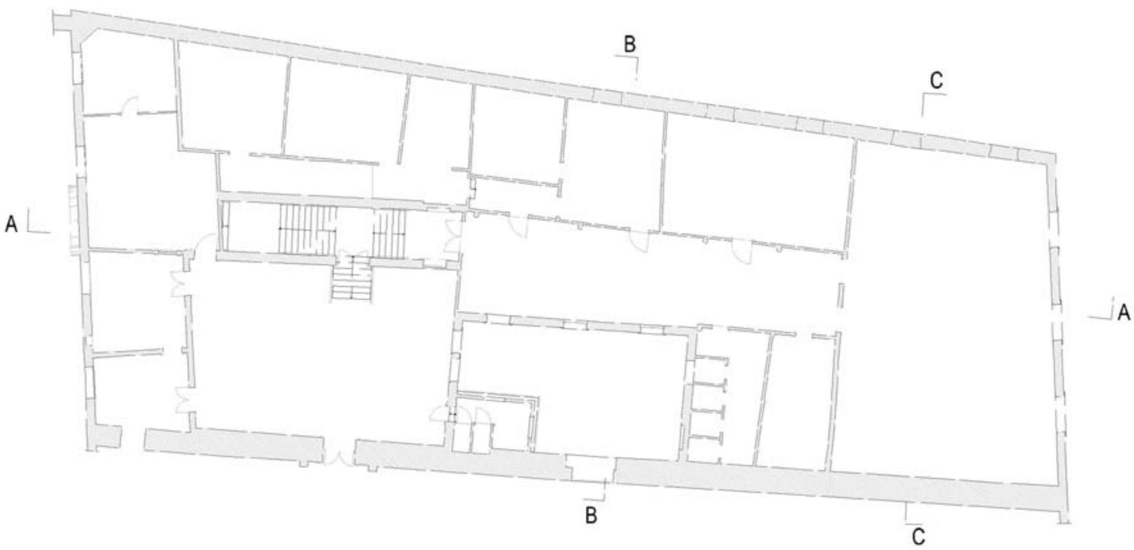


Fig. 4.3.3: First floor plan f the "*Sala Maggiore*"

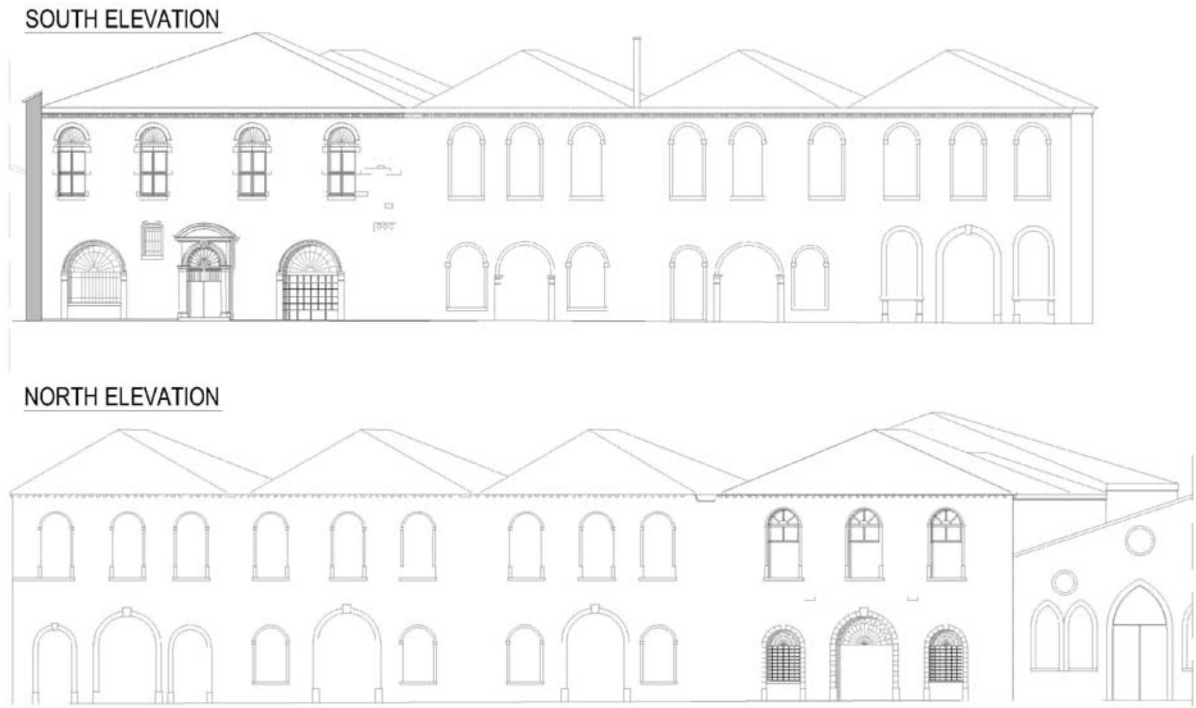


Fig. 4.3.4: North and south elevations of the "*Sala Maggiore*"

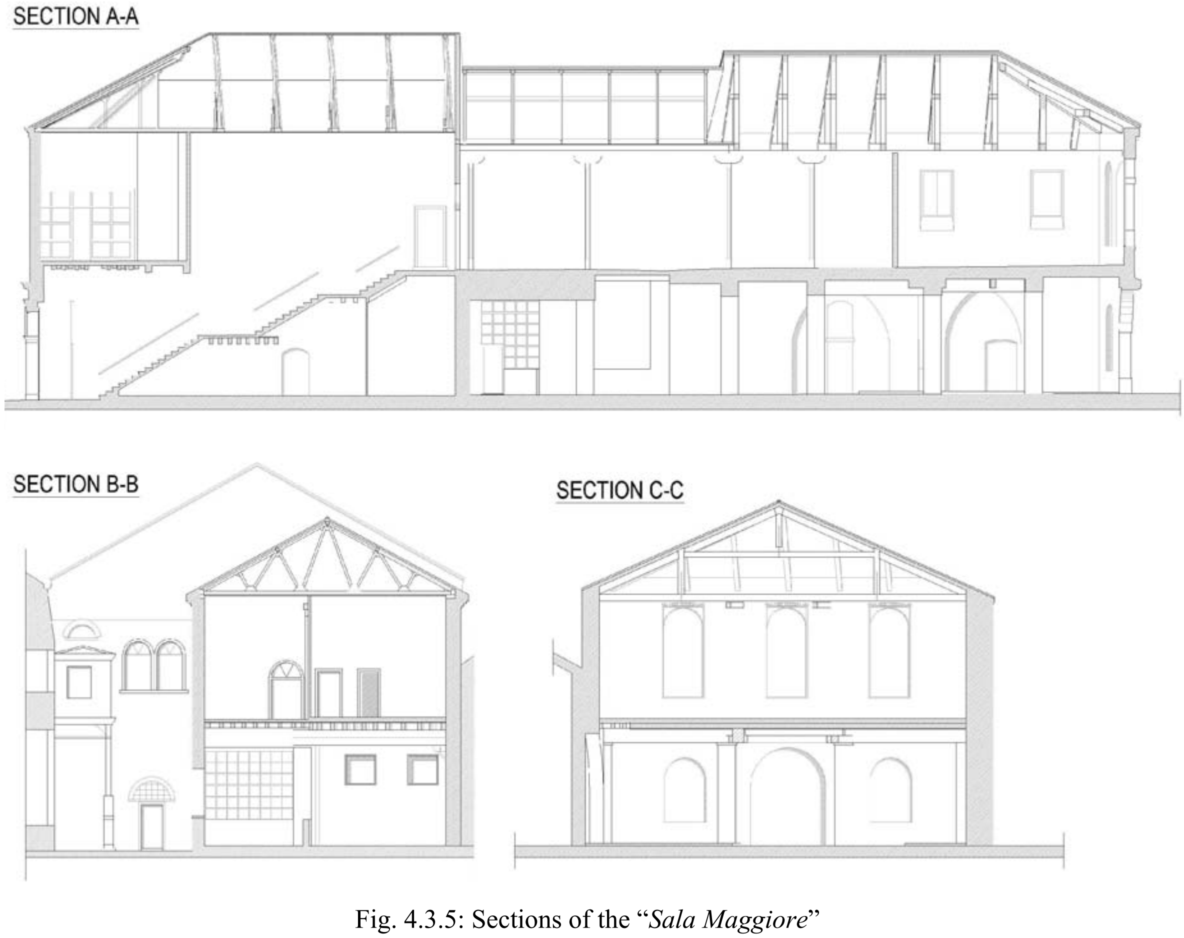


Fig. 4.3.5: Sections of the "*Sala Maggiore*"

From geometrical elevations and sections of the building we can state that, as already anticipated in the analysis of the present-day conditions, the building has three different heights. Fig. 4.3.6 e Fig. 4.3.7

respectively show an aerial view, where is is possible to see the different heights of the three roofs of the *Sala Maggiore* and the north elevation of the schematic with the levels of altitude above sea level. The central part of the building has a height of 15,50 m, highest one that overlooks the basin with 16,25 m. Finally, the maximum height of 17,80 meters characterizing the portion of the building facing towards the *Stradal Campagna*.

Fig. 4.3.6: View of the maximum heights of the "*Sala Maggiore*"

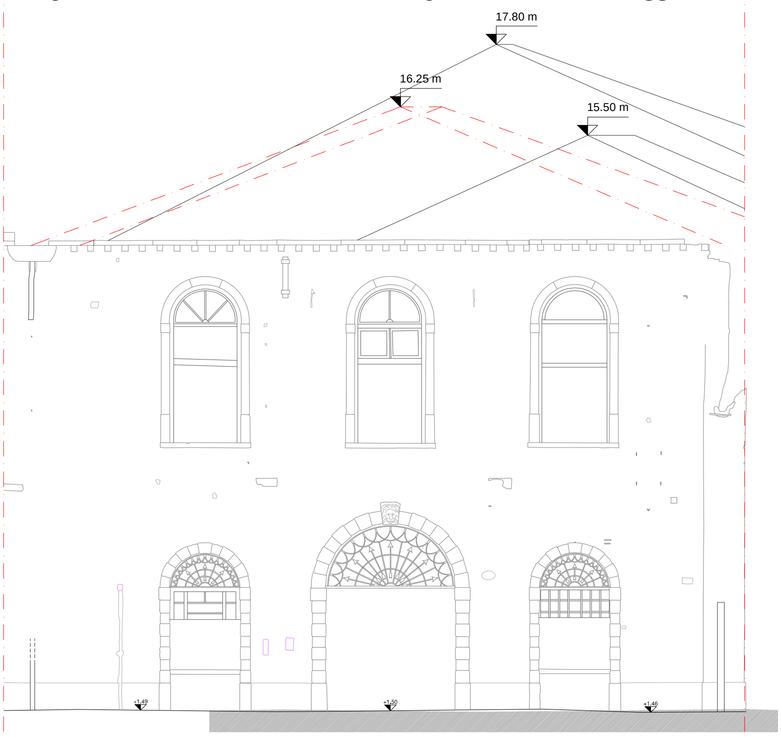


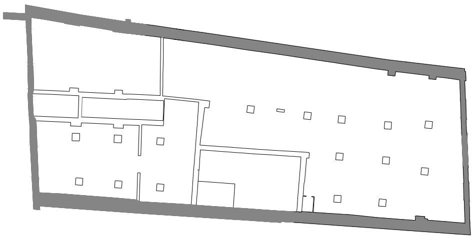
Fig. 4.3.7: Schematization of the maximum heights of the building

4.3.2 Vertical Elements

Vertical elements are defined as vertical load bearing structures, designed to support other structures and at the same time, to define the interior spaces, and not structural elements, the partitions. They include:

- perimeter walls of the buinding;
- internal walls of the buinding;
- pillars and columns;
- partition walls.

Perimeter walls of the buinding



Ground floor plan

First floor plan

The whole perimeter of the *Sala Ovest* is surrounded by partitions of brick masonry. Lengthwise, walls thickness approximately equal to 1,15 m separate the building to the east, from another room, and to the west from one of the Maritime Warehouses. The Hall communicates with the latter through large pointed arches, nowadays walled up. In addition to this, two walls of about 50 cm thick divide the interior from the courtyard. Openings on the ground floor and first floor allow to overlooking the inner courtyard and access into it.

The walls of the façade, masonry walls 50 cm thick, have a thickness of about 55 cm and they are characterized by openings on two levels.

It is interesting to note the differences between the two sides, both in size, both for material characteristics. The prospect to the south is longer than the northern one since the plan of the building is not rectangular. On the northern side are seen the remains of plaster and the significant presence of Istrian stone.

The walls of the façade have a thickness of about 50 cm and they are characterized by openings on two levels (Fig. 4.3.8, Fig. 4.3.9). It is interesting to note the differences between the two sides, both in size and for material characteristics. The south elevation is longer than the northern one since the plan of the building is not rectangular. On the northern side are seen the remains of plaster and the significant presence of Istrian stone.

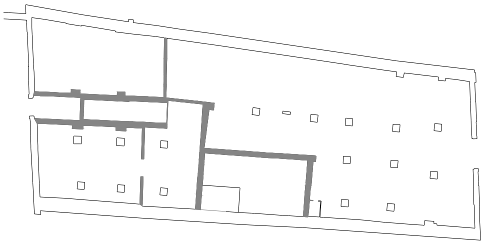
This is used for archivolts, jambs and sills of the three windows on the first floor. The rustication is used in the strengthening intervention as a reinforcement at the edge of the walls. To the south, however, the Istrian stone is limited to the archivolts and jambs of the three doors and to the sills of the four windows.

May be the difference of both façades is due to the stride forwards the south in the late nineteenth to align it with the rest of the buildings (Tosato, 1988). Any parts of plaster not being found in the elevation on the *Stradal Campagna* and considering its likely later than those parts of plaster found on the Darsena Nuova, it can be said that it has been maintained brickwork masonry.

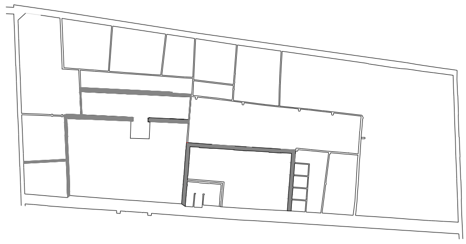
Fig. 4.3.8: North elevation

Fig. 4.3.9: South elevation

Internal wall of the building



Ground floor plan



First floor plan

The internal load bearing structures consist of walls which border on the stair that you have access from the south entrance on the *Stradal Campagna*. They grow longitudinally for about one third of the plan of the building and they have a height of about 11 m. They are made of masonry brick of varying thickness, equal to 50-60 cm on the ground floor and 40-50 cm on the first floor. On load bearing structures are relied the structure of the stair and the reinforced concrete trusses of the main span.

In the ground floorit is possible identified two walls of bricks masonry, which end at a height of about 5,50 m and have a thickness of 25 cm. This is the wall which separates the room where you enter from the northern entrance, supporting the the wooden floor joists, and that of the room partition, where you access from the *Stradal Campagna* into the two rooms, supporting also the iron floor.

Pillars and Columns

On the *Sala Maggiore* the single supports are represented by pillars and columns of different geometrical type and made of different material, which testifies the different periods of construction of the elements. On the ground floor there are brick pillars, arranged in two different structural patterns, which allow us to consider them as a combination of two different rooms. The first reached from the *Stradal Campagna*, the second from the northern entrance in front of the *Darsena Nuova*.

The room accessed by the *Stradal Campagna*, the only one from which you can reach the first floor, is divided into three main rooms, two of which are interconnected and characterized by six pillars arranged in two rows and other two against the wall the stairwell (Fig. 4.3.10).

Pillars, still partly plastered, about 5 m high, have a squared base of side approximately 80 cm, and supporting a floor in hollow tiles and beams. In the third room, there are two pillars leaning out for their half-thickness to the longitudinal partition. There are also two iron pillars, characterized by C-beams section, supporting the floor, made of masonry vaults and iron beams. (Fig. 4.3.11).

Fig. 4.3.10: Plastered masonry pillars

Fig. 4.3.11: I-Beams supporting brick vaults

From Northern entrance you can reach in a big room (Fig. 4.3.12), articulated in three main "aisles" of a double row of pillars in brick with a square base of side approximately 75 cm and a height of just more than 4.50 m.

Fig. 4.3.12: Ground floor entrance on the north side: room characterized by two rows of pillars

It could be also noted:
- Pillars against the longitudinal walls for mid-thickness to decrease the length of spans of the floor;
- Isolated pillars placed in a framed system with walls;
- Pillars placed at a distance slightly more than one meter from partitions.

The layer of plaster of the pillars shows moisture, considered failures at the ends. The irregularity of the mesh is immediately visible from an analysis of the plan and visiting the site. The elements, in order to hold the floor together with the partitions walls, were introduced maybe later into the structure of the building. The theory is partly confirmed by the historical texts that describe the *Sale d'Armi* such as deposits and by the presence of pointed arches, now walled up. To be noticed the implemented device in the construction of the beam-wall connection at the level of the pillars: the beams are not supporting directly on the wall but on a stone slab that isolates the ends of the element from moisture rise.

On the first floor single support blocks are represented by six vertical wooden elements, disposed on the longitudinal sides of the corridor leading to the stairwell. Four of these elements are aligned and hold the wooden beam that serves as supporting beam to the tie beam of roof trusses (Fig. 4.3.13). Between the pillar and the wooden beam, we observe the presence of the barbican of the same material, which increases the steadiness of the support (Fig. 4.3.14).

Fig. 4.3.13: Corridor on the first floor

Fig. 4.3.14: Wooden pillars with barbican supporting the wooden beam

Finally it is possible identify another type of supporting elements, made of cast iron blocks. In the inner courtyard (Fig. 4.3.15), two cast iron columns with squared brick base supporting the submentioned room from the first floor, likely added later to the the building (Fig. 4.3.16). On the first floor, two cast iron columns, nowadays collapsed, supported two wooden beams of the roof trusses, increasing their stability.

Fig. 4.3.15: North façade of the internal courtyard of the building

Fig. 4.3.16: Construction projecting supported by cast iron columns

4.3.3 The partitions

The vertical inner partitions have the purpose to divide vertically the inner spaces.

The internal subdivision of the ground floor room, accessible from the north, is obtained by low walls and wooden frames. Exploiting the mesh of pillars and perimeter walls, small rooms have been created.

The first floor is divided into rooms, of same dimensions, with walls of different sizes and thickness. This includes:

- masonry briks walls, 12,5 cm thick, plastered;
- wooden internal doors and windows;
- hollow bricks walls plastered with a thickness of 12 cm;
- wooden partition walls plastered with a thickness of 11 and 22 cm.

The brick walls, 12,5 cm thick, were used in four rooms constructed close to the bearing walls of the stairwell and to the southern and eastern perimeter walls (south and east). In one of these rooms was used a wooden frame wall to obtain an isolated room.

The hollow brick plastered wall characterized a portion of the wall of the corridor and a room.

The wooden partitions are used for a wall of the hall and several rooms. They consist of two parallel wooden curtains, in which they threw a heterogeneous mix of stone and recycled wooden material, and coated by wooden battens nailed and plastered. It is interesting to analyze the item of the wall parallel to the north façade along the length inside the building, in corrispondence to the wooden roof. Two thirds of the wall is made of wooden battens nailed and plastered and it has a thickness of about 23 cm. The remaining third of the wall is composed of hollow clay bricks, arranged to form a wall 12,5 cm thick, plastered.

In correspondence of the connection of the two parts of the wall a supporting beam was made (Fig. 4.3.17). Fig. 4.3.18 shows the remaining part of the partition made on hollow bricks, collapsed with the roof.

Fig. 4.3.17: Portion of the wooden partition covered by secondary beams nailed and plastered

Fig. 4.3.18: Detail of partition made of hollow bricks

4.3.4 Horizontal Elements

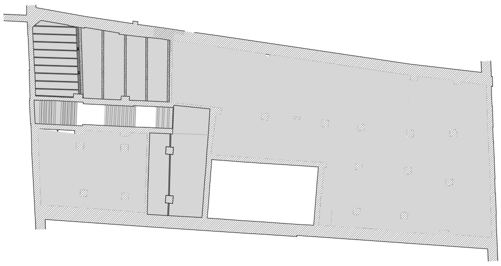
The horizontal structures can be divided into:

- ground floor;
- first floor;
- false ceiling;
- roofs.

Structural layout of the ground floor

The ground floor is made of reinforced concrete. The room which is accessed from the northern entrance is longitudinally crossed by the railroad tracks.

Structural layout of the ground floor



The first floor consists of three different types:

- wooden floor with composit frame: separating the room, which can be accessed from the northern entrance, from the first floor;
- traditional iron floor: they are in two separate rooms, accessible from the *Stradal Campagna*;
- reinforced concrete floor: it can be found in a room which is accessed from the *Stradal Campagna*.

The wooden floor is made of purlins and it is supported by main beams. The main beams are made of wood, concrete and iron. The purlin-beam connection is made by cross cornerstones.

The following figure (Fig. 4.3.19) shows the wooden floors collapsed. While they prevented from having access to some rooms on the first floor, thus limiting the information on the rooms,on the other hand, they made it possible to investigate the composition of the floor layer.

On this last it is possible to find two types of flooring. In most part of the first floor there are ceramic tiles. In other rooms there is the Venetian traditional flooring, a compound of polychrome marble chips in cement or slaked lime layed, with usually a thickness of 3-6 cm (Carbonara, 2004).

Fig. 4.3.19: A venetian traditional flooring emerged from the collapse of the wooden floor

Fig. 4.3.20: Floor in iron beams and brick vaults

The iron floors differ in the structure to fill in the gaps or interspaces between the beams. On the inferior flanges of the iron beams are set brick vaults (Fig. 4.3.20) or hollow tiles (Fig. 4.3.21), making the intrados plastered surface different. In both cases the heterogeneity of materials, iron and clay, and oxidation in the inferior flanges of the iron beams have caused cracks and detachments of the plaster. The iron floors are overlapping to the main iron beams with double I-section , in orthogonal directions, completely projecting to the intrados.

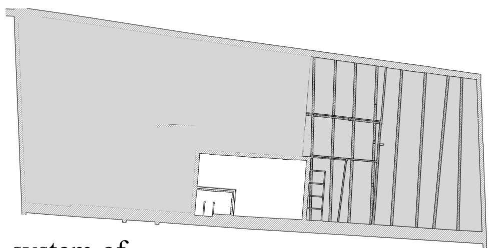
The reinforced concrete floor (Fig. 4.3.22) is made up of ribs protruding rectangular section, comparable, in terms of functionality, to the main beams of the wooden floors with composit frame.

The decay of the structure is underlined by the visibility of the ribs of the reinforcement, stirrups, longitudinal rods and slab, with a welded steel reiforcement mesh. On the intrados of the monolithic floor it is possible to see the marks of formwork used for the casting of the floor.

Fig. 4.3.21: Floor features by hollow tile placed in the flanges of the iron beams

Fig. 4.3.22: Reinforced concrete floor with ribs and slab

False ceiling



The first internal horizontal partition is observed in the room which can be accessed from the northern entrance, characterized by the presence of the wooden floor. Mostly of the space is covered by false ceilings, although now in poor condition, composed by two different types:

- plastered reed wattles false ceiling, linked to the floor through a system of secondary battens nailed on the sides of purlins;

- false ceiling made of wire metallic net nailed down to the purlins, which provides support to the plaster.

Proceeding in height, also the first floor is characterized by two different types of false ceilings whose position differs depending on the structure of covering:

- plastered reed wattles false ceiling, linked to the sixteenth century wooden roof through a system of battens, nailed to the tie beams of the trusses;
- false-ceiling type "Perret".

In the Fig. 4.3.23 it is possible to observe as, in correspondence to the wooden roof trusses, the ceiling is nowadays completely absent. It remains only the system of battens, on which the element was anchored to.

Where the roof of the building consists of reinforced concrete trusses, the false ceiling is made of hollow bricks tiles of type "Perret". Among these, there is a thin casting of reinforced concrete with a small reinforcement bar that is linked on inferior element of each truss, supporting in this way the ceiling.

The floor casting is not so thick to form a slab, since it does not support any load. For this reason, the false ceiling was undergo a huge number of collapses that have occurred in correspondance of the collapses of the roof structure, as noted in Fig. 4.3.24.

Fig. 4.3.23: System with secondary beam nailed at the tie beam for the anchorage of the false ceiling made with reed wattle

Fig. 4.3.24: Collapse of the"Perret" false ceiling, anchored at the reinforced concrete truss, as a consequence of the collapse of the tile roof

Structure of the roof

The roof of the *Sala d'Armi Nord Ovest* is a pavilion covering. The pitches have a longitudinal discontinuity in the central area due to the presence of an inner courtyard. The covering structure consists of trusses of different nature: there are, in fact, wooden trusses for a third of the area and other two-thirds made of reinforced concrete beams.

The wooden structure is located in the northern part of the building that overlooks the Darsena Nuova. It was partially affected in 2006 by a considerable collapse caused by the settlements of the supports of the trusses. It is the consequence of the poor state of preservation of the elements due to the moisture. The collapse has involved the wooden elements of the pavilion pitch and two trusses.

The remaining wooden trusses, still existing, are queen-post trusses (also called "Palladiana") (Bettiol, 2007).

The wooden trusses, still existing, loading on the perimeter walls of the building and have an intermediate point of support of the tie beam on the wooden supporting beam, placed on wooden pillars on the first floor. The trusses have a span of between 16 m and 17 m, and they are placed at a distance of wheel base between trusses of approximately 2,25 m. The measure of the span of those collapsed ranged from 15 m to about 16 m. The wooden trusses have a height of about 3 m. Because of the collapse, part of the building has no the tile roof, letting to be seen a double system of wooden purlins and wooden framework of a dormer (Fig. 4.3.25). The documentation provided by the *Soprintendenza* about the on-site surveys has been made before the collapse of the structure. It is possible to see a wooden plank placed on the purlins, on which there was, like the adjacent buildings, the tile roof.

Fig. 4.3.25: View of the partially collapsed wooden roof

The covering made of reinforced concrete, dating from the early twentieth century, concerns the remaining two thirds of the building. It is possible find other two systems, consisting of reticular trusses, dating to the Thirties, different for the span and the resulting height of the roofing.

From the opposite side of the *Stradal Campagna*, the span to be covered is between 17 m and 19 m. The five trusses have an heigth of 4,50 m. In the part of the building facing the courtyard, the span of the six trusses varies from 10,5 to 12,0 m. The slope of the pitches is less than that one of the adjacent wooden and concrete roofs, thus lowering the height of the ridge (Fig. 4.3.26).

For the irregular plan of the building, each reinforced concrete truss has different span from the others. The two different systems of trusses, with wheel base between trusses systems equal to 2,25 m, are all connected at the tie beams with a reinforced concrete edge beam, placed on the longitudinal perimeter walls so as to be visible externally.

The pavilion pitch is made through a system of struts, vertical and oblique tie beams and rods made of reinforced concrete.

Fig. 4.3.26: Different heights of the roof of the "*Sala Maggiore*"

Fig. 4.3.27: Reinforced concrete trusses and "Perret" false ceiling

The roof is made following the "Perret" style, as shown in Fig. 4.3.27, in which has been placed the tile roof composed by terracotta "Marseillaise" tiles. The "Perret", patented on the late nineteenth century

(Sorana, 2003), consists of hollow brick tiles connected with a casting of reinforced concrete. The tiles, just today called "Perret" tiles, have a thickness of 3,5 cm, a length of 41.5 cm and a width of 26 cm.

4.4 The knowledge phase

The knowledge phase plays a leading role for a suitable comprehension of the structural importance of the different elements, and it is the final step of the preliminary phase of the study the cultural memories of a building. Several on site surveys were carried out, with the aim of characterizing the buildings as a whole unit and the individual structural elements composing the system. Surveys were carried out both by laser scanner and traditional techniques (caliper, meter and distance meter).

Where possible, since not all of the areas were accessible due to the probable collapse of portions of the building, non destructive tests were carried out as well as chemical analyses on sampled materials, which allowed to estimate the mechanical and chemical properties of concrete, and the distribution of reinforcement bars within the elements of the r.c. trusses. Moreover, the study of the r.c. trusses was carried out – according to the current standards – comported the so called "simulated design", which consists in the execution of the design of the structures as they are, by using the standards in force at the time of construction of the structures. With reference to the studied building, since the construction period dates back to the thirties, the effective laws in force at that time was the Royal Decree (R.D.) 16/11/1939, nr. 2229.

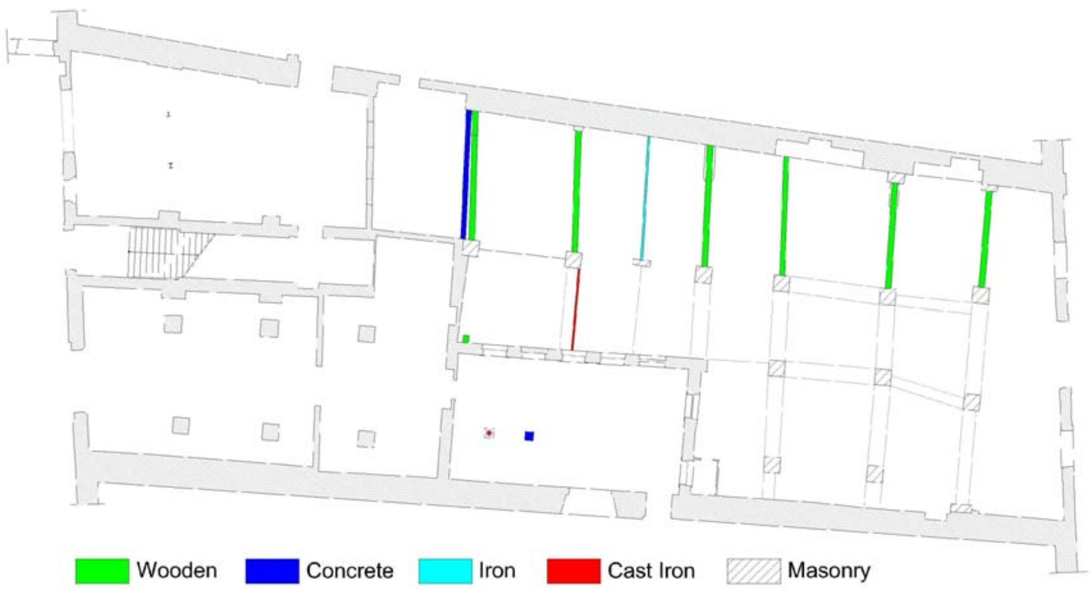
It was then possible to identify and to classify the structural elements of the building in terms of geometry, physical (mechanical, chemical) composition of concrete, possible reinforcement bars distribution and structural decay.

In the following subparagraphs, information regarding:

- Main beams;
- Secondary elements;
- Pillars;
- Roofs;

will be provided.

Fig. 4.4.1 contains a schematic distribution (plan view) of the main beams, of masonry elements and pillars.



- Fig. 4.4.1: Plan view: first floor (*Sala Maggiore*) - structural elements composition

Fig. 4.4.2 reports the plan view of the roof elements where different typology and materials are outlined.

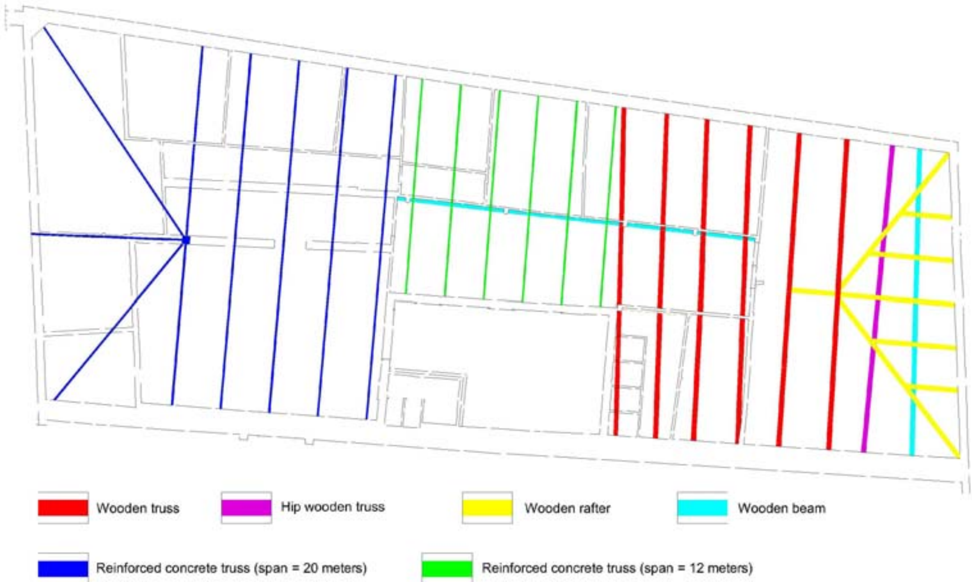


Fig. 4.4.2: Plan view: roof (*Sala Maggiore*) - structural elements composition

4.4.1 Main beams

From the plan view of the ground floor of the building it is possible to evaluate eight principal beams spanning on the masonry pillars and perimeter walls, sustaining the secondary elements. The most part of the floor elements are made of wood, even if metallic and r.c. beams (one by sort) are present, testifying the strengthening interventions of the 19th and 20th centuries.

Most part of the wooden main beams are coated by nailed wooden plank which prevents to take out information on the present-day conditions of the element. For those ones lacking in coating, we can evaluate now their state of preservation and they can be testified, due to the very high section, that they are made of two wooden overlapping beams. But the connection of the elements is not realized through particular anchorages, only in several cases, using metallic stirrups on the edges.

The geometric survey provides the basic height of the main beam and of the false ceiling rather than, in its absence, the height of the floor joist and of the wooden plank (Fig. 4.4.3).

This allows us to take easily the height of the beam section, as the difference between the detected height of the main beams and that one of the floor purlins, considering, if present, a false ceiling with a thickness of 3 cm. The dimension of the beam, on the contrary, is directly taken only in the absence of wooden coating.

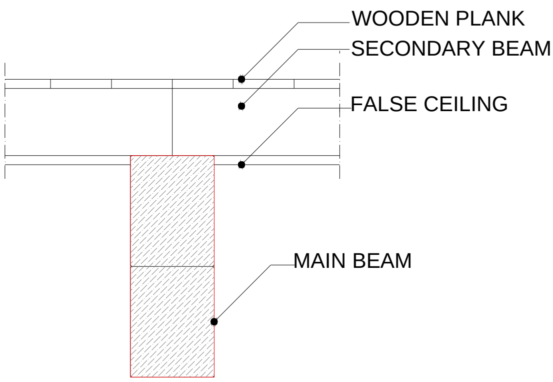


Fig. 4.4.3: Scheme of the transversal section of the "main beam – secondary beam" joint

The signs of decay visually detectable on main beams, and more generally, on the wooden floor, were classified into three types:

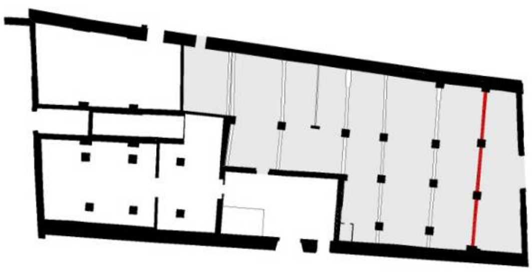
- load-bearing frame depression, which is manifested by the loss of horizontality of the element, due to the state of stress-caused by loads on the floor and then by deformation of the beams;
- settlements ofthe supports;
- material decay, due to attacks by fungus and xylophagous insects, to fire and to moisture (Carbone, 1995) (Munafò, 1990).

In the following plan (Fig. 4.4.4) are reported the denomination and the identication of the main beams of the floor.

Fig. 4.4.4: Denomination and identification of the main beams of the first floor

The main beams are numbered from 1 to 8, according to the order of survey observation from northern entrance of the building (Fig. 4.3.17).

BEAM 1



Material: wood

Structural layout

Beam with three spans on four supports: those central ones are represented by pillars 2 and 3, those ones from the side pillars 1 and 4, placed against the perimeter longitudinal wall in order to decrease the span width.

Description of span

- First span:	from pillar 1 to 2.	Span width	s_1= 5,14 m.
- Second span:	from pillar 2 to 3.	Span width	s_2= 5,10 m.
- Third span:	from pillar 3 to 4.	Span width	s_3= 5,28 m.

In the first span, the beam is coated below and on one side by a nailed plank. On the uncoated side, on the contrary, is it possible to see clearly the two overlapping beams to made up the main wooden beam (Fig. 4.4.5).

In the second span, there is the coating by nailed plank on all sides of the beam, thus preventing the observation of the same.

In the third span of the beam is visible from all sides, plastered except for the flange next to the wall, where there are many plaster detachments (Fig. 4.4.6).

Geometry of the section

The beams, that make up the beam number 1, have a rectangular section. The total height is determined by the difference between the detected height and that one of the main floor purlins. The base of the section is not classified for the first and second span because of the coating. The dimentions of the sections differ just a little from one span to another.

- First span b = unknown; h = 0,66 m;
- second span: b = unknown; h = 0,64 m;
- third span: b = 0,28 m ; h = 0,63 m.

Visible signs of decay

The edge of the beam restrained to the west wall has a plaster detachment due to the concentration of moisture rise in masonry and environmental humidity of the site. It helps, as it appears in this case, the attack of fungi.

Fig. 4.4.5: First span: connection between two wooden beams

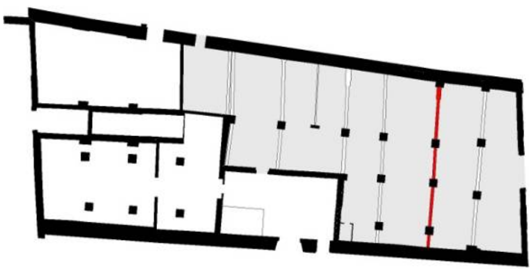
Fig. 4.4.6: Third span: decay of the end beam

BEAM 2

Material: wood

Structural layout

Four spans beam on five supports: the central ones are represented by pillars 5, 6 and 7, those of the side from the wall for one side and by the pillar 8, placed against the partition.



Description of span

- First span: from east wall to pillar 5. Span width s_1= 1,65 m.
- Second span: from pillar 5 to 6. Span width s_2= 4,62 m.
- Third span: from pillar 6 to 7. Span width s_3= 3,83 m.
- Fourth span: from pillar 7 to 8. Span width s_4= 5,70 m.

In the first span, the beam is coated by wooden nailed plank, but it is possible to see a wooden Barbican, 36 cm long, which further reduces the already limited span (Fig. 4.4.7).

In the second and third span, the observation of the beam is prevented by the presence of the nailed coating.

In the fourth span the beam is not coated: it is possible to observe the overlap of the two single wooden beams and the strengthening to the edge using lateral tackling with wooden element and metal stirrups, probably made up for the high span, increasing the supporting function to the edge (Fig. 4.4.8).

Geometry of the section

The elements that make up the beam number 2 has rectangular section. The total height, where the false ceiling remains, is calculated as the difference between the height of the room and that main one, added to those also the thickness of the false ceiling amount of about 3 cm. The base of the section can be measured only in the last span because the others are coated.

- First span: b = unknown; h = 0,67 m;
- second span: b = unknown; h = 0,67 m;
- third span: b = unknown; h = 0,67 m;
- fourth span: b = 0,28 m; h = 0,67 m.

The same measure were observed in height , it is reasonable to consider also for the base of the section the same measure in each span.

Visible signs of decays

After a direct observation, there were no special damages. It would be interesting to investigate more precisely about the ends of the beam, both restrained to the wall , as for the previously analyzed beam, and therefore exposed to moisture.

Fig. 4.4.7: Wooden barbican

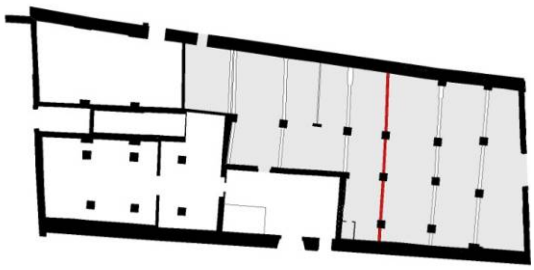
Fig. 4.4.8: Lateral wooden tackling and stirrups at the end beam

BEAM 3

Material: wood

Structural layout

Four spans beam on five supports: the central ones are represented by the pillars 9, 10 and 11, the side ones by partitions walls.



Description of span

- First span: from east wall to pillar 9. Span width s_1= 1,64 m.
- Second span: from pillar 9 to 10. Span width s_2= 4,61 m.
- Third span: from pillar 10 to 11. Span width s_3= 4,05 m.
- Fourth span: from pillar 11 to the west wall. Span width s_4= 6,07 m.

In the first span, the beam is reduced to the upper wooden single element, due to the reduced span (Fig. 4.4.9).

In the second and third span, the presence of wooden nailed plank prevents the observation of the item.

In the last span can be seen the overlapping of the two wooden beams. It is interesting to see carefully that the edge of the main beam, tied to the wall, is positioned over the point of the half-walled-up arch, loading its own weight on the vault and that weight of the floor, without presenting any strengthening intervention (Fig. 4.4.10).

Geometry of the section

The elements that constitute the beam number 3 have rectangular sections. The total height, where the false ceiling remains, is calculated as the difference between the height of the main room and that main one, adding up the thickness of the false-ceiling of about 3 cm. Because of coating, the base of the section cannot be measured in the following three spans.

- First span: b = unknown; h = 0,64/2 = 0,32 m;
- second span: b = unknown; h = 0,64 m;
- third span: b = unknown; h = 0,68m;
- fourth span: b = 0,23 m; h = 0,68 m.

Visible signs of decays

It was observed no particular signs of decay, unless at the head of the beam restrained to the west wall, where it is clear that the connection was achieved without any special care. The risk is that settlement of the support could happen , considering also the height of the fourth span. It is true also that the presence of moisture has helped decreasing the loading edge capacity, to which any strengthening intervention were applied.

Fig. 4.4.9: Detail of the degradation of the end beam

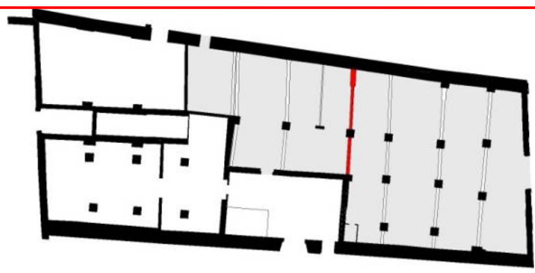
Fig. 4.4.10: The end of the beam located near the pointed arch of the west wall

BEAM 4

Material: wood

Structural layout

Two spans beam on three supports: the first one is the pillar number 12, the pillar number 13 is in the middle and the third is the west wall.



Description of span

- First span: from pillar 12 to 13. Span width s_1= 4,30 m.
- Second span: from pillar 12 to the west wall. Span width s_2= 6,20 m.

In the first span, the beam has a wooden nailed coating that prevents to investigate its section. From the support on the pillar 13 (Fig. 4.4.11), the beam is visible for the whole second span. The section consists of two wooden overlapping beams. It is possible to observe the edge strengthening intervention by side tackling with wooden elements and metal stirrups (Fig. 4.4.12). It has been made in order to increase the supporting edge beam capacity, due to the height of the span.

Geometry of the section

Twin elements, which make up the main beam, have a rectangular section. The total height, where the false ceiling still remains, is calculated as the difference between the height of the main room, to which we add the thickness, about 3 cm., of the false ceiling. The base of the section can not be directly measured in the first span, but looking out the support on the pillar number 13, is reasonable to believe that it could be as the same measure as for the second span:

- first span: b = 0,28 m ; h = 0,67 m;
- second span: b = 0,28 m ; h = 0,67m.

Visible signs of decay

It is possible to see the moisture attack to the wooden parts, used for tackling of the end of the beam. Again, the lack of attention to the connection beam-wall exposes the edge of the element to the danger of an attack by fungus and xylophagous insects.

Fig. 4.4.11: Wooden coating in the first span; support the beam on the pillar 12

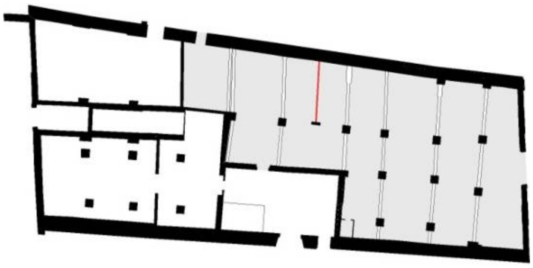
Fig. 4.4.12: Strengthening of one end of the beam through the application of side wooden planks connected to the original beams through metallic rods

BEAM 5

Material: steel

Structural layout

Beam fixed at the edges: the first one is restrained to a 26 cm thick wall, identified as pillar number 14, the second one fixed to west perimeter wall.



Description of the span

- Single span: from pillar 14 to the west wall. Span width s= 5,93 m.

Looking at the secondary wooden beams, the beam can be considered to be inserted into the structure with only function of supporting beam, due to its height span width between the main beam 4 and 6, about 6.20 meters long.

Seven elements of the floor, as matter of facts, are continuous from the beam 4 to 6, as shown in Fig. 4.4.13. One can also assume that its inclusion was made to rise a high masonry wall 25 cm. thick, aligned with the existing pillars. The other end is tied and fixed to the wall where the pointed arch was walled, as represented in Fig. 4.4.14.

Geometry of the section

I-Beam:

- flange width b = 131 mm ;
- beam depth h = 320 mm;
- web thickness d = 11,5 mm;
- flange thickness t = 17,3 mm.

Visible signs of dacay

It is evident the rust on the whole surface of the steel element. Corrosion is the type wet for the heavy concentration of umidity in the air due to the environmental peculiarities of the site. This features are more visible at the end of the element fixed to the west wall.

Fig. 4.4.13: The end of the beam restrained at the pillar 14

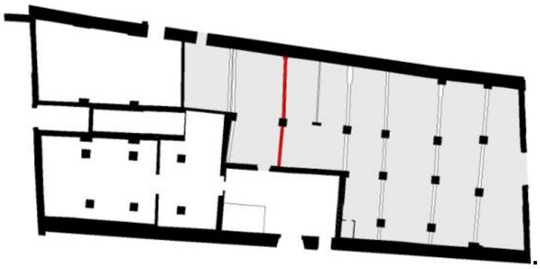
Fig. 4.4.14: End of the beam restrained at the west wall in the walled-up pointed arch

BEAM 6

Material: wood

Structural layout

Two spans beam on three supports: in the middle one is represented by pillar number 15, those lateral ones by walls.



Description of span

- First span: from east wall to pillar 15. Span width s_1= 4,28 m.
- Second span: from pillar 15 to the west wall. Span width s_2= 6,43 m.

Only in the first span it is possible to find the wooden nailed plank. Therefore, at the support on pillar number 15 can be seen the beam, which consists of two overlapping timber elements (Fig. 4.4.15). In the second span, where the two constituent elements of the beam are visible, the head edge, while restrained to the wall, is layed on a masonry corbel characterized by depth of 22.5 cm and width of 44 cm (Fig. 4.4.16).

Geometry of the section

The beam has rectangular section. For the presence of coating the base of the section in thefirst field can not be directly measured:

- first span: b = unknown; h = 0,84 m;
- second span: b = 0,28 m ; h = 0,67 m.

It can be noted that the height measured for the first section of the span is not justified on the first floor with a consequent possible difference on level.

Visible signs of decay

It is not possible to investigate on the beam-wall connection, but from the direct looking out the beam appears to be inserted in the partition without any device providing the necessary ventilation to the edge. This allows then to quantify the risk of edge decay with a consequent loss of load supporting-capacity due to the moisture rise, which can promote mycotic and xylophagous insects attack.

Fig. 4.4.15: Coating in wooden plank for all the span width of the first span

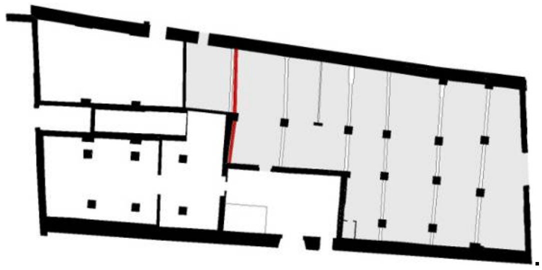
Fig. 4.4.16: End of the beam supported by masonry corbel

BEAM 7

Material: wood

Structural layout

Two spans beam on three supports: those ones on the sides are represented by walls, instead the middle one by the pillar number 16.



Description of span

- First span: from the east wall to pillar 16. Span width s_1= 4,52 m.
- Second span: from pillar 16 to the west wall. Span width s_2= 6,17 m.

For the first span is visible only the edge of the beam, since there is a false ceiling made of reed wattles, placed at the height of the main beam and it is connected to the same beam by wooden battens. It has to be observe, however, precisely on the support pillar 16 that the section consists of two overlapping beams.

In the second span, the beam is free of coating and it consisting of two overlapping wooden elements. Looking the timber Barbican beam, it is possible to see that it is anchored to the two above useful metal stirrups and supported by a wooden slope beam coming out from the wall. With a length of 1,15 meters, it has been designed to decrease the span, and then the stresses of the beam (Fig. 4.4.17).

The beam-wall connection to the support element on the pillar 16 is interesting for the use of a stone slab, which helps the insulated target of the wood, from moisture that perspires on the wall (Fig. 4.4.18).

Geometry of the section

The beam has a rectangular section in both spans. For the presence of the false ceiling the sectional base in the first span cannot be directly measured. The dimensions are:

- first span: b = unknown; h = 0,84 m;
- second span: b = 0,28 m ; h = 0,74m.

Again in this example, as previously described for beam 6, the measured height for the first section of the span is very high and not justified by any difference in level on the first floor.

Visible signs of the decay

The material decay, caused by moisture rise, interest both ends of the beam due to to the lacking of care in the realization of the connection beam-walls, and the ends resting on the pillar.

Fig. 4.4.17: view of the beam before the interventions to provide safety conditions of the wooden floor

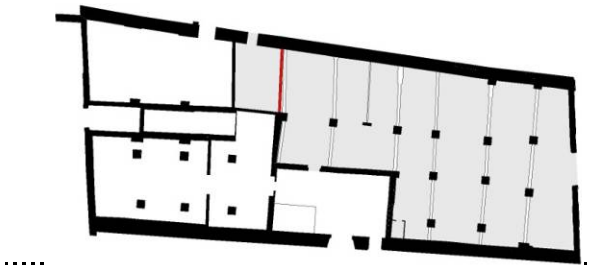
Fig. 4.4.18: Slab of stone for the connection beam-wall

BEAM 8

Material: reinforced concrete

Structural layout

Beam fixed at the edge-ends.



Span description

- Single span: from the east wall to the west wall. Span width *s*= 6,17 m.

The reinforced concrete element, intervention dated in the twentieth century, is located higher than that main wooden beam, which is supported and aligned, (Fig. 4.4.19). It is on first floor, in correspondence to the beam, a couple of steps, with a rise of 16 cm. After the on-site survey, it is possible to assume that the beam has been inserted during the construction underneath the existing floor: the concrete as matter of fact does not seem to be vibrated with skill, because the some aggregates can be observed.

The collapse of the floor (Fig. 4.4.20) permits to observe that the beam is located exactly at the beginning of covering made of reinforced concrete beams with the widest span in the building. It possible to hypothesis that the items are datable at the Fascist period (about 1930).

Section geometry

Dimension of the rectangular beam: b = 0,24 m ; h = 0,56 m.

Visible signs of decay

Compared to the above mentioned elements, the reinforced concrete beam has not a high level of decay. Detachments of material are visible on the element, caused by umidity.

Fig. 4.4.19: Reinforced concrete beam and wooden beams placed side by side

Fig. 4.4.20: Reinforced concrete beam

4.4.2 Secondary frame

The secondary frame of the wooden floor is made of joists which support the wooden plank on the top, laid at right angles, and the false ceiling below.

The purlins are on the structural layout as a continuous beam on two supports. In the central areas of the floor they load on wooden main beams. Those close to the wall on the façade are supported on the main beam and on the wall.

Looking at the floor near the walls of the eastern and western area, it is possible to notice that the edge beam, for its whole length, is close to the wall, thus creating a practical arrangement that helps the installation of the wooden plank and above all, the flooring (Munafò, 1990).

The beam-joists connection is made by overlapping to the load bearing main frame some elements. Neither the inspections nor the geometric surveys made it possible to investigate the presence of metallic elements for connecting the ends of the joists.

The joists of the floor, support the loads which come from two types of false ceiling above mentioned and from the floor over the wooden plank. The false ceiling made of reed wattles and coupled through a system secondary beams nailed on the sides of purlins. The one made of plastered metal net is nailed, on the contrary, directly to the low side of the elements. As described above, the floor on the wooden plank is made of Venitian traditional flooring or made of ceramics tiles, depending on the room is concerned with.

The secondary frame is the component of the wooden floor more damaged, mainly driven by the deterioration of the materials. There are partial or total collapse on the secondary frame caused by the presence of high environmental moisture, especially in areas corresponding to the collapse of the roof. Where the intrados of the floor is not visible for the presence of false-ceiling, it is not possible to deduce the conditions of decay from the floor upstairs. In accessible rooms, paved with tiles, is it possible to detect a deflection of the floor that leads to accommodate the deformation of the beams.

The signs of decay in the purlins were classified, as the main beams, in three types:

- depression of load bearing frame, which is shown by the deformation of the horizontal floor;
- supports settlements;
- material decay due to attacks by fungi and xylophagous insects, by fire and moisture (Carbone, 1995) (Munafò, 1990).

It was possible to draw the dimensions of the joists where they are directly obtained from the geometrical survey, the absence of the false ceiling, and where there were collapses of the structure. The height of the beam is obtained as the difference between the purlin and the plank.

In order to easy understand the survey, the space on the ground floor covered by wooden floor, was ideally divided into three main rooms, designated 0.1, 0.2, 0.3. Each room was further divided into floor sections, named with the letters A to F, almost identified with each mesh structure formed by the pillars structure and walls (Fig. 4.4.21).

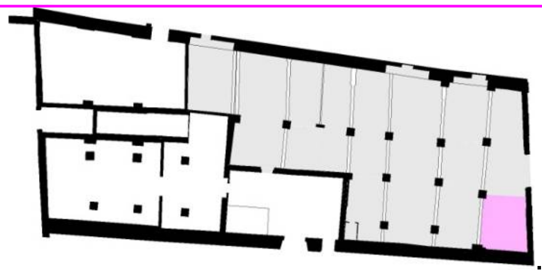
Fig. 4.4.21: Denomination and identification of the floor sections of the first floor

FLOOR SECTION 0.1.A

Structural layout

Two span beam

Mean span width $s = 4{,}80$ m



Description of the intrados

The false-ceiling, made of plastered reed wattles, stops at the measure of five purlins from eastern perimeter masonry. The purlins based on the main beam number 1 on one side and directly on the façade wall, on the other side (Fig. 4.4.22). It is not visible any device in the realization of the beam-wall connection.

Close to the wall of the facade, there is a couple of beams in the direction of the main ones and with same measures as those of the purlins. The couple is placed between the purlins and one beam placed at the corner. It is restrained to the wall for one edge and the other resting on the corner element (Fig. 4.4.23)

The measures of the purlins and wheelbase of the floor are not obtained from the geometric survey, just because the collapse of the part of the false-ceiling happened before the survey itself.

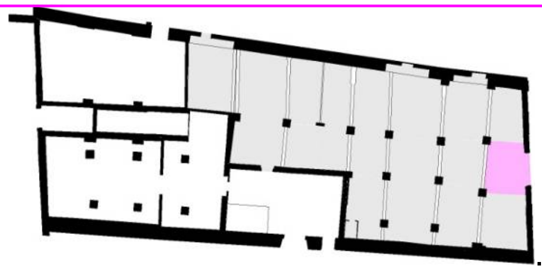
Visible signs of decay

The decay of the material, caused by the moisture rise of the masonry element, involves visible edges of the purlins and edge element.

Fig. 4.4.22: Purlins near the wall

Fig. 4.4.23: Corner beam

FLOOR SECTION 0.1.B



Structural layout

Two span beam

Mean span width s = 4,20 m

Description of the intrados

The floor structure is not visible due to the presence of the false-ceiling (Fig. 4.4.24). It can be true that, as in the next room, the purlins are based on one side on the main beam 1 (Fig. 4.4.25) and on the other side directly on the wall. Given the presence of the large entrance hole, it would be interesting to investigate whether within the wall has been placed a wooden sleeper.

The false-ceiling does not allow to obtain the measures of the frame.

Visible signs of decay

From the intrados the signs decay are not directly observable. Upstairs it is possible to detect a slight deflection of the ceramic floor tiles, the first symptom of the deformation of the purlins.

Fig. 4.4.24: False ceiling

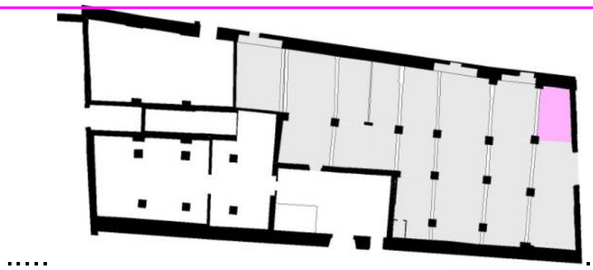
Fig. 4.4.25: Purlins supported by the mean beam 1

FLOOR SECTION 0.1.C

Structural layout

Two span beam

Mean span width s = 3,60 m



Intrados description

The collapse of the false-ceiling, made of plastered net, close to the perimeter walls, allowed to observe the system of purlins-plank (Fig. 4.4.26). The purlins rest on the main beam number 1 and on the façade wall, the same side where it is possible to see all their edges. Moreover, the first five beams are all visible from the point of the perimeter wall.

Visible signs of decay

The material decay, due to the moisture rise of the masonry, concern with a large part of the purlins and their edges. The edge beam presents a clear reduction in section (Fig. 4.4.27). All the frame shows colours fading.

Fig. 4.4.26: Collapse of the false ceiling next to the masonry

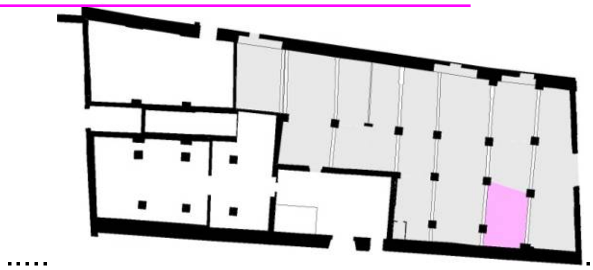
Fig. 4.4.27: Decay edge beam

FLOOR SECTION 0.1.D

Structural layout

Two span beam

Mean span width s = 4,30 m



Intrados description

The frame loads on main beams 1 and 2. The false ceiling,of which it is possible to see the net due to the deterioration of the plaster (Fig. 4.4.28), does not allow direct observation of the intrados of the floor Fig. 4.4.29). The false-ceiling does not allow to obtain the measures of the secondary frame.

Visible signs of decay

The observed decay of the plaster ceiling near the perimeter wall is caused by moisture rise, so it can be inferred that also the secondary frame of the floor is concerning with.

Fig. 4.4.28: View of the false ceiling where it is possible to see the metallic net

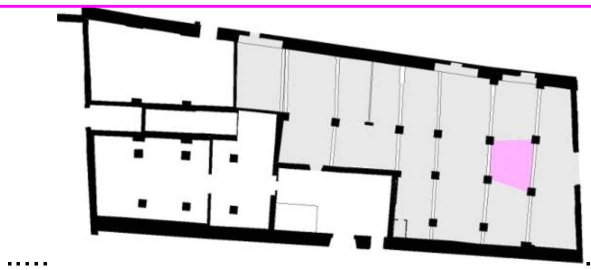
Fig. 4.4.29: Main beam 2 on wich rest the purlins

FLOOR SECTION 0.1.E

Structural layout

Three span beam

Mean span width s_1=2,60m; s_2=1,80m



Intrados description

The false-ceiling is plastered net type. Is it possible to observe the collapse of the floor surrounding the corresponding area on the upper floor to a cast iron column. The absence of floor and false-ceiling and the presence of the coating, made of nailed plank, could show two beams placed at the height of the main beam. The first of these beams is placed on the pillars 2 and 6, the second one is placed on the pillars 3 and 7. Both have half section compared to that one of the main beams (Fig. 4.4.30).

The two beams hold other two wooden elements, crosswise placed (transversely), which have a section that meets the height of the top edge of the main beams (Fig. 4.4.31). Above these ones, there are the wooden purlins of the floor, which have three supports: one in the central line and two at the edges, on the main beams 1 and 2, showing a strengthening intervention on the beam number 2.

It can be assumed that these last four described elements have been inserted later, after the edification of the floor, which corresponds to the placement of two cast-iron columns on the upper floor, in order to contain the stresses.

Visible signs of decay

The collapse of the floor is caused by the settlement of supports of some purlins, as a result of increased vertical loads and therefore shear stresses.

Fig. 4.4.30: Collapse of the floor

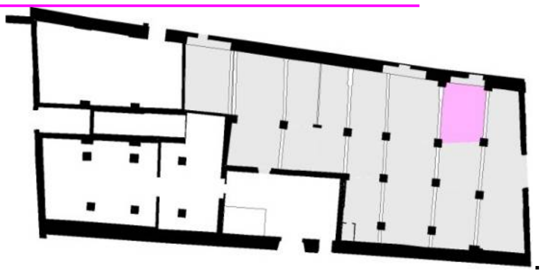
Fig. 4.4.31: Element supporting the purlins

FLOOR SECTION 0.1.F

Structural layout

Two span beam

Mean span width s = 4,50 m.



Intrados description

The false-ceiling is in reed wattles plastered style. It is connected to the floor through a system of battens, that is to say a system of secondary beams nailed on the side of purlins (Fig. 4.4.32). There is no false-ceiling near the perimeter wall. It is possible to observe the purlins and the above planking. The purlins load on the main beams 1 and 2. The edge beam is missing of a support on the edge (Fig. 4.4.33).

Visible signs of decay

The edge beam shows the settlement of the support on the main beam 2, which is caused by decay which is affected the main beam edge. The deterioration of wood, caused from moisture rise of the masonry, has spread all over the visible frame and over the plank, presenting a chromatic change.

Fig. 4.4.32: Purlins and wooden plank

Fig. 4.4.33: Collapse of the support of the edge beam

FLOOR SECTION 0.2.A

Structural layout

Two span beam

Mean span width s = 5 m

Intrados description

Reed wattles plastered false-ceiling, connected by battens nailed to the floor, has collapsed near the perimeter wall. Here one it is possible to see the purlins, resting on the main beams 2 and 3, the wooden planking of the floor and the edge beam (Fig. 4.4.33).

Visible signs of decay

The collapse of the false-ceiling near the wall is a symptom of the presence of moisture rise, which causes the decay of the material on visible elements. In addition, from the observation of the extrados on the first floor, there is the hollow of the floor along the perimeter wall (Fig. 4.4.35), that shows also a frame depression, resulting from the moisture.

Fig. 4.4.34: Reed wattle false ceiling joined to the purlins

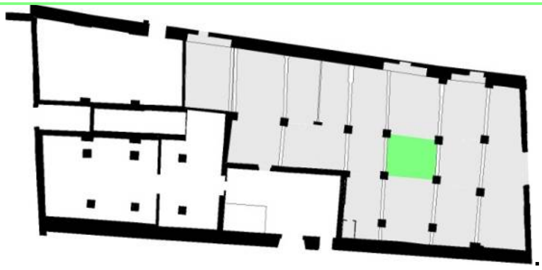
Fig. 4.4.35: Depression of the floor

FLOOR SECTION 0.2.B

<u>Structural layout</u>

Two span beam

Mean span width $s = 5$ m



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<u>Intrados description</u>

The false ceiling hides the structure of the floor (Fig. 4.4.36). On the sides of the room, it is possible to see the paneling plank of two beams: the first, not directly visible, is arranged on the pillar 6 and 10, the second is placed on the pillars 7and 11.

After the on-site survey it follows that they have a section of a height so high as the half of the main beam and they are supporting an additional element, of same dimensions, crosswise placed (transversally) near beam number 2, in order to increase the supporting area (Fig. 4.4.37). The floor purlins are based on the main beams 2 and 3.

<u>Visible signs of decay</u>

The false ceiling plaster has stains, symptoms of the presence of moisture. Walking on the extrados of the floor, it is possible to detect the depression of the tiles floor, due to frame-depression.

Fig. 4.4.36: False ceiling

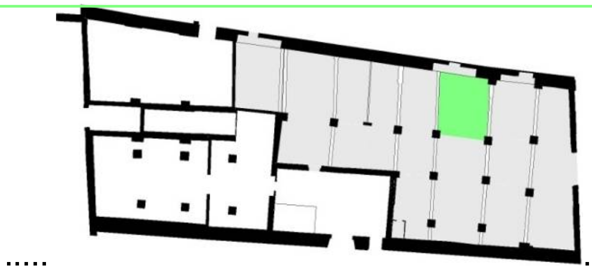
Fig. 4.4.37: Beam supported by the pillars 7 and 11

FLOOR SECTION 0.2.C

<u>Structural layout</u>

Two span beam

Mean span width $s = 5{,}10$ m



<u>Intrados description</u>

The false-ceiling is completely absent (Fig. 4.4.38). The purlins and the wooden plank are clearly visible. They load on main beams 2 and 3.

<u>Dimensional characteristics of the frame</u>

The floor is composed by 11 purlins.

- Mean section of the purlin: b = 0,15 m; h = 0,23 m.
- Wheel base between two adjacent beams: i = 0,53 m.

<u>Visible signs of decay</u>

The decay of the material affects the whole frame, wooden plank included (Fig. 4.4.39).

The decay is very pronounced for the edge beam, that suffers particularly of moisture rise, because it is near the wall. The floor extrados shows a depression of the frame.

Fig. 4.4.38: Secondary frame of the floor

Fig. 4.4.39: Purlins and plank moisture

FLOOR SECTION 0.2.D

<u>Structural layout</u>

Two span beam

Mean span width $s = 3{,}70$ m

<u>Intrados description</u>

The room is bordered on two sides by wooden frames, one for the eastern perimeter wall and for the second from the wall that separates it from the small courtyard inside of the building. For the complete absence of the false-ceiling, it is possible to see the wooden plank and the wooden purlins of the floor. The frame is loading on the main beam 3 with one edge and on the wall for the other. The connection beam-wall that, from the height of purlins, presents a reduced wall thickness in order to create a shelf. Here is placed a wooden joist on which rest the edges of the purlins (Fig. 4.4.40).

Furthermore, there is also a beam placed in the direction of the purlins at the height of 7 and 12 pillars: the element rests on the pillar 7 and on the main beam number 4, at a height of half section, which is higher than the section of the main beam (Fig. 4.4.41).

In the frame there are two iron sections, with function of supporting beam in order to allow the passage of the installations coming from upstairs.

Dimensional characteristics of the frame

The floor is constituted by 13 purlins.

- Mean section of the purlin: b = 0,16 m; h = 0,23 m.
- Wheel base between two adjacent beams:: i = 0,48 m.

Visible signs of decay

The purlins and the wooden plank are affected by the deterioration of the material, presenting the chromatic change. The moisture rise is very deep.

Fig. 4.4.40: Wooden joist used in the connection beam-wall

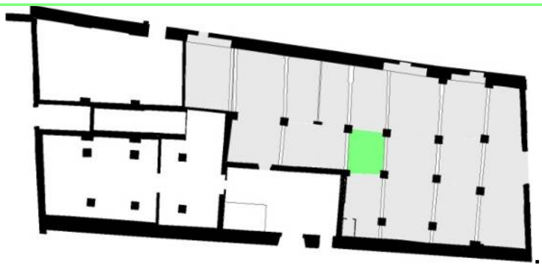
Fig. 4.4.41: Beam placed in the pillar 10 and in the main beam 4

FLOOR SECTION 0.2.E

Structural layout

Two span beam

Mean span width s = 3,70 m



Intrados description

The presence of the false-ceiling hides the structure of the floor (Fig. 4.4.42). It is reasonable to assume, for similarity to next rooms, that the purlins are loading on the main beams 3 and 4.

Visible signs of decay

For the spots on the false-ceiling it can be assumed that the plank is affected by moisture.The top surface of the floor confirms this, showing floor deflection , a symptom of depression of the frame, caused by the deterioration of the material.

Fig. 4.4.42: Floor frame hidden among the false ceiling

FLOOR SECTION 0.2.F

Structural layout

Two span beam

Mean span width s = 3,70 m

Intrados description

The false-ceiling is completely absent: the purlins rest on the main beams 3 and 4; and on them the wooden plank is lean (Fig. 4.4.43).

Dimensional characteristics of the frame

The floor is constituted by 12 purlins.

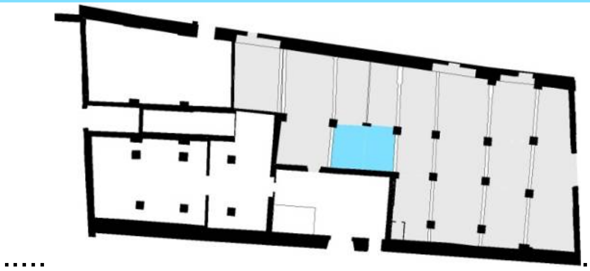
- Mean section of the purlin: b = 0,18 m; h = 0,27 m.
- Wheel base between two adjacent beams:: i = 0,53 m.

Visible signs of decay

The purlins and the plank are very deteriorated. Diffuse chromatics changes are observed all over the wooden plank. The floor of the extrados has some depressions near the wall, a sign that edge depression is more starting from the edge beam.

Fig. 4.4.43: Purlins supported by the main beam 5

FLOOR SECTION 0.3.A



Structural layout

Two span beam

Mean span width $s = 6{,}30$ m

Intrados description

The room has a false-ceiling, only for half part, while in the other one it is possible to see the purlins and the wooden plank (Fig. 4.4.44, Fig. 4.4.45). The purlins rest on the main beams 4 and 6, placed at constant distance.

Dimensional characteristics of the frame

The floor is constituted by 9 purlins.

- Mean section of the purlin: b = 0,145 m; h = 0,23 m.
- Wheel base between two adjacent beams: i = 0,52 m.

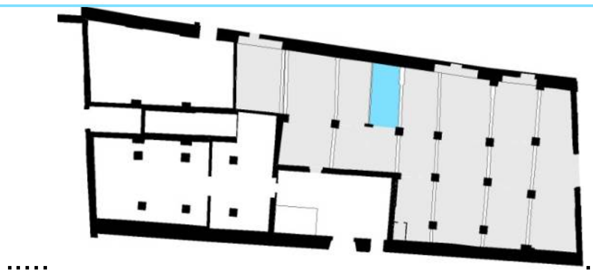
Visible signs of decay

The moisture rise affects the frame, where chromatic change are observed, and the false-ceiling, where some marks are visible on the plaster. The edge beam is affected in particular way. Also on the extrados it is possible to observe that floor is subject to depression due to loss of strength of the purlins.

Fig. 4.4.44: False ceiling of the first floor

Fig. 4.4.45: Floor wooden frame

FLOOR SECTION 0.3.B



Structural layout

Two span beam

Mean span width $s = 2{,}30$ m

Two span beam

Span width $s = 3$ m

Three span beam

Mean span width $s_1 = 3$ m

$s_2 = 3{,}30$ m

Intrados description

For the complete absence of the false-ceiling, it is possible to observe the plank and the floor purlins, for that a clear distinction is needed, based on their structural layout (Fig. 4.4.46).

The purlins next to the wall, the first six, resting for an edge on the iron beam number 5 and for the other edge they are restrained to a wooden supporting beam, embedded in the frame in order to remedy the lack of resistance of the edge of the beam number 4 and in order to unload the weight of the flooring part directly on the wall. The connection of the supporting beam with the purlins is of the half-thickness (Fig. 4.4.47, Fig. 4.4.48). A purlin stays on the beams 4 and 5. The remaining seven elements, which are characterized by the longer lenght, have the structure of continuous beam on three supports: the two at the sides are represented by the main beams 4 and 6, the central one by the beam 5.

Dimensional characteristics of the frame

The floor is constituted by 14 purlins, different for the structural layout.

Two span purlin (s = 2,30 m)

- Mean section: b = 0,20 m; h = 0,33 m.
- Wheel base between two adjacent beams: i = 0,30 m.

Two span purlin (s = 3 m)

- Section: b = 0,18 m; h = 0,33 m.
- Wheel base between two adjacent beams: i = 0,65 m.

Three span purlin (s_1= 3 m; s_2= 3,30 m)

- Mean section: b = 0,18 m; h = 0,33 m.
- Wheel base between two adjacent beams: i = 0,50 m.

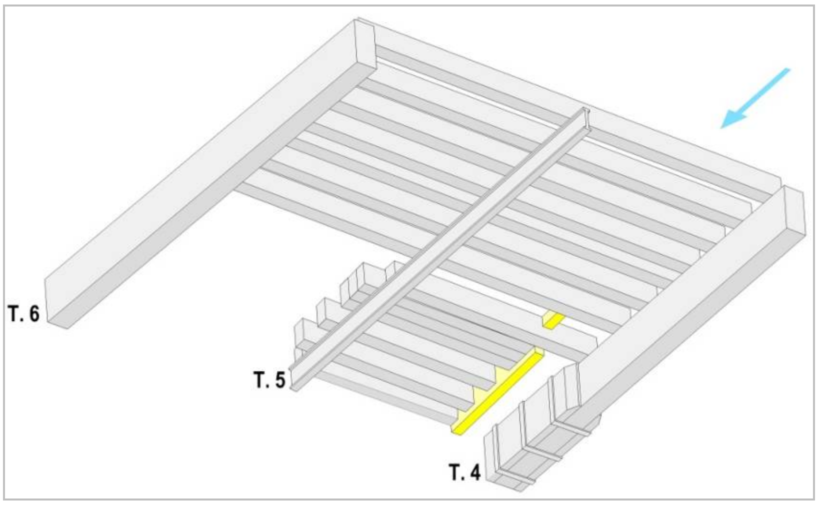


Fig. 4.4.46: Disposition of the purlins

Visible signs of decay

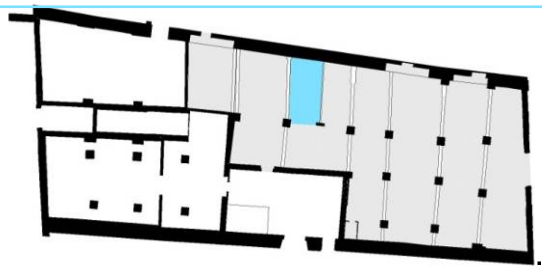
The material decay caused by moisture affects the purlins and the wooden plank.

Depression of the frame can be deduced from the extrados of the floor, where the floor aims at deflection in order to adapt itself to the deformation of the elements.

Fig. 4.4.47: Two span purlin rests on supporting wooden beam and on beam 5

Fig. 4.4.48: Supporting wooden beam to unload the weight of the end of the main beam 4

FLOOR SECTION 0.3.C



Structural layout

Two span beam

Mean span width $s = 3{,}30$ m.

Three span beam

Mean span width $s_1 = 3$ m;

$s_2 = 3{,}30$ m.

Intrados description

In the following floor, next to the preceding one, almost the same features could be found on the frame. Four purlins, next to the masonry, load on the main beams 5 and 6, presenting the structural layout of a beam on two supports (Fig. 4.4.49). The remaining seven joists rest their edges on the beams 4 and 6, with a central support represented by the beam 5 (Fig. 4.4.50).

On the frame there is an iron double I-section supported by nails framed on the purlins (Fig. 4.4.51). Without structural function, it is possible that it could be for the attachment of ropes or tools.

Dimensional characteristics of the frame

The floor is constituted by 11 purlins.

Two span purlin ($s = 3{,}30$ m)

- Mean section: b = 0,175 m; h = 0,33 m.
- Wheel base between two adjacent beams: i = 0,59 m.

Three span purlin ($s_1 = 3$ m; $s_2 = 3{,}30$ m)

- Mean section: b = 0,18 m; h = 0,33 m.
- Wheel base between two adjacent beams: i = 0,50 m.

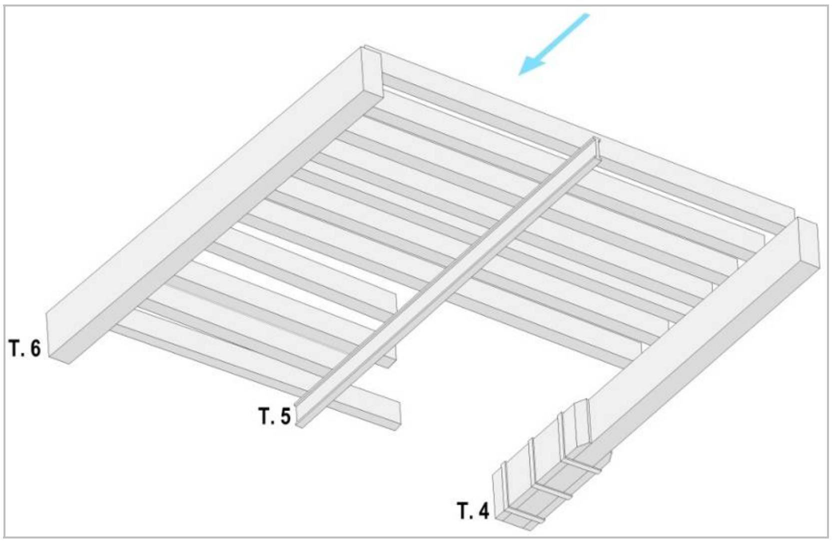


Fig. 4.4.49: Disposition of the purlins

Visible signs of decay

The deterioration of the material, caused by the presence of moisture, is visible both on the purlins and on the wooden plank. On the extrados of the floor leads to a deflection due to the moisture on the strength of the structural frame elements.

Fig. 4.4.50: Two span beam. View of the iron I-Beam

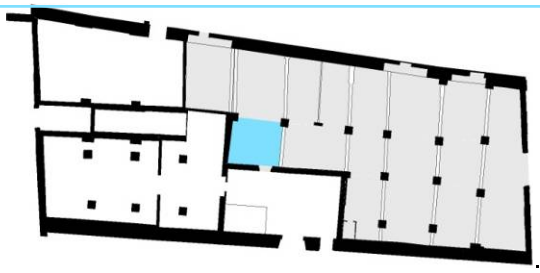
Fig. 4.4.51: Central support of the purlins of the floor

FLOOR SECTION 0.3.D

Structural layout

Two span beam

Span width   *s* = 4,90 m



Intrados description

The floor is almost completely hidden by the false ceiling made of plastered reed wattles. A partial collapse, however, close to the outside perimeter wall lets to see that the wall is connected to the main beam number 7, unlike from the other cases, by the wooden battens (Fig. 4.4.52). There are visible also the planks and the purlins that rest on the main beams 6 and 7.

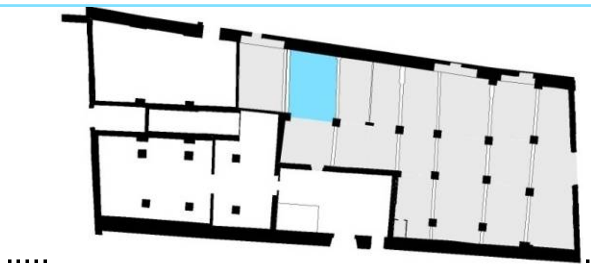
Visible signs of decay

The collapse of the false-ceilings show the presence of moisture rise (Fig. 4.4.53) due to the closeness to the wall. This is also the cause of depression of the frame, visible from the floor on the extrados.

Fig. 4.4.52: Collapse of the false ceiling near the wall

Fig. 4.4.53: Effects of the moisture rise in the false ceiling plastered

FLOOR SECTION 0.3.E



Structural layout

Two span beam

Span width s = 4,90 m

Intrados description

The floor upstairs is impassable for the suffered collapses (Fig. 4.4.54). This condition allows to investigate the composition of the layers above the plank. The Venitian traditional flooring can be observed. The purlins load on the main wooden beams 6 and 7.

Visible signs of decay

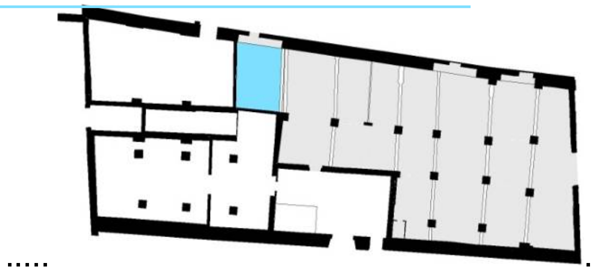
The degradation of the material throughout all the frame and on the plank is caused by high presence of moisture (Fig. 4.4.55). The depression of the floor has reached the collapse due to the cracks on the beams.

Fig. 4.4.54: Partial collapse of the floor

Fig. 4.4.55: Purlins and plank collapsed as a result of the material decay

F L O O R S E C T I O N 0 . 3 . F

<u>Structural layout</u>
Two span beam
Span width s = 4,60 m



<u>Intrados description</u>

The collapse of the floor is almost complete (Fig. 4.4.56). Only three wooden beams remain and almost no trace of the plank (Fig. 4.4.57). The frame of the floor loads on a masonry wall and on the concrete beam number 8. After the survey some walls on the upper floor without any support were found.

<u>Visible signs of decay</u>

The collapse of the floor was caused by the high level of moisture, to which has been exposed, given the total absence of the upper floors. The collapsed material from the upper floors has also got worse the load on the frame already affected by depression of the purlins concerned by a reduced loading capacity.

Fig. 4.4.56: Total collapse of the floor

Fig. 4.4.57: Materials decay of the frame

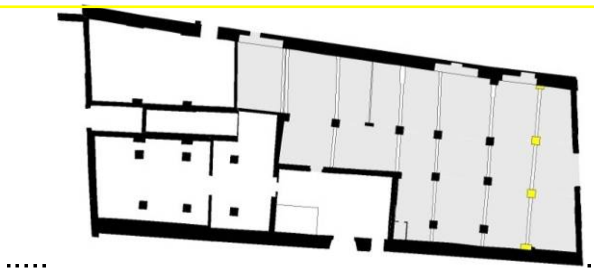
4.4.3 Pillars

The columns were numbered from 1 to 16, following the order of identification of the spans of the main beams, as shown in the Fig. 4.4.58. In some cases, the pillars are grouped in function of the main beam.

Fig. 4.4.58: Fig. 4.4.59: Denomination and identification of the pillars

PILLARS 1, 2, 3, 4

Represent the four supports of the main beam 1.



Description of the support and geometric features

These supports are plastered masonry; they have a squared base with a thickness of three rows of bricks, which is approximately 77 cm wide and 4,5 m high.

Pillars number 1 and 4, leaning against the wall, projecting for half thickness and have a minimum enlargement at the top that looks like a capital of pilaster (Fig. 4.4.60). The pillars 3 and 4 have sharp corners protected by metal sections, long about one meter from the ground (Fig. 4.4.61). All of these end with a stone slab for the support of the beam (Fig. 4.4.62).

Visible signs of decay

The pillars do not show structural damages. The plaster has some wide detachments caused by moisture rise on the brickwork masonry.

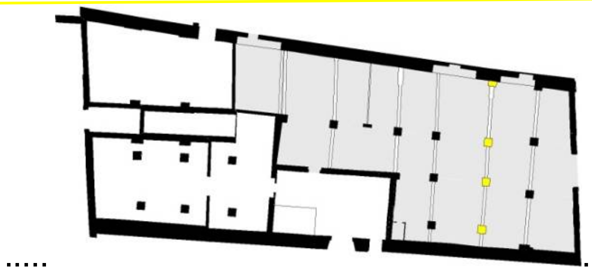
Fig. 4.4.60: Capital at the end of the pillar 4

Fig. 4.4.61: Metallic sections at the edges of the pillar 2

Fig. 4.4.62: Slab of stone at the edges of pillar 2

PILLARS 5, 6, 7, 8

Represent the supports of the main beam 2.



Description of the support and geometric features

The first three pillars have a squared base of side as long as three rows of bricks, while the fourth is placed against the perimeter wall, which protrudes about 50 cm. Rise from the ground about 4,5 m.

At the edge they all have a slab of stone for the beam-wall connection. The masonry pillars are coated by cement plaster, and now it has many detachments (Fig. 4.4.63). The pillar 5, while detached from the partition wall, it is only 1,25 m far from it (Fig. 4.4.64).

Visible signs of decay

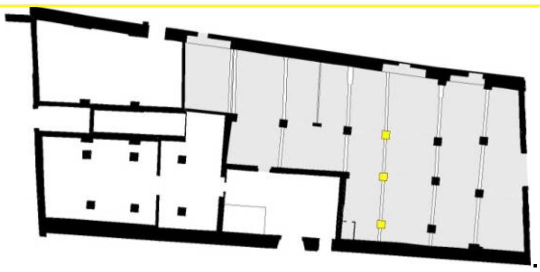
Structural damages are not observed. The plaster shows many detachments, caused by moisture rise in the brickwork masonry.

Fig. 4.4.63: Pillars 6 and 7 with diffuse detachments of plaster

Fig. 4.4.64: Pillar 5

PILLARS 9,10,11

Represent the central supports of the main beam 3.



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Description of the support and geometric features

They are characterized by a squared base, with a side as long as three bricks, and about 4,5 m high, and end with a stone slab, on which rests the wooden beam.

Due to the lacking of plaster, at the halfway up of the pillar 10, it is possible to see the slope of the rows of bricks, as a construction defect (Fig. 4.4.65).

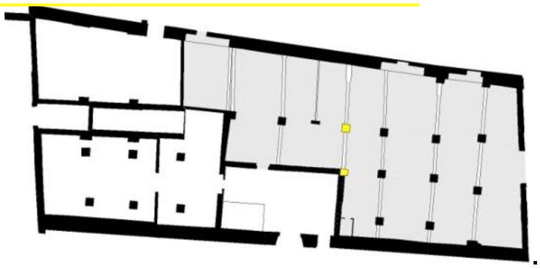
Visible signs of decay

There are no structural damages but large detachments of plaster, especially concentrated in the lower edge and caused by moisture rise. Powdery detachments of mortar are observed on the underside of the edges.

Fig. 4.4.65: Pillar 10 – inclination of the rows of bricks

PILLARS 12,13

Represent the supports of the main beam 4.



Description of the support and geometric features

The pillar number 12, leaned against the wall, protruding out the edge of this wall for about 26 cm, as long as a lenght of the brick. Unlike other similar elements has no capital, nor the stone slab to support of the beam (Fig. 4.4.66).

On the contrary, the pillar 13, is in the middle of the room and it has the same dimensions of the above mentioned elements. In the northern side of the pillar the plaster has been kept and in the half towards the aisle side has been detached (Fig. 4.4.67).

Visible signs of decay

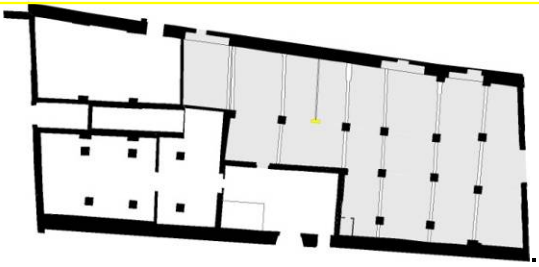
There are not structural damages. The detachments of the plaster are caused by the presence of moisture rise.

Fig. 4.4.66: Pillar 12 – absence of the slab of stone for the connection beam-wall

Fig. 4.4.67: Pillar 13 – presence of plaster in the upper part

PILLAR 14

At the pillar 14 is restrained the iron beam 5



Description of the support and geometric features

Actually, pillars 15 is a wall of 26 cm. The thikness is about 5 m for throughout the height of the room. Considering its connection to the iron supporting beam and the frame of the floor, it can be assumed that it could be inserted before than the structure of pillars and, with it, also the wall has been built connecting it to the pillar 15 (Fig. 4.4.68). In the element does not appear any device for the connection with the wooden purlins of the floor.

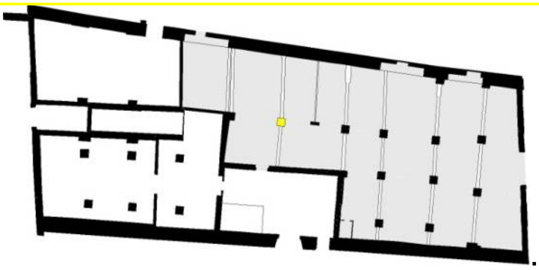
Visible signs of decay

The wall has no structural damages. The cement plaster has some detachments due to moisture rise, concentrated on the lower part of the element.

Fig. 4.4.68: Iron beam restrained at the pillar

PILLAR 15

It is the median support of the main beam 6



Description of the support and geometric features

The pillar is made of plastered wall, has a squared base of side as long as about three bricks and approximately 4,5 m high. It has a stone slab at the edge, where rests the main wooden beam (Fig. 4.4.69). It is connected to the pillar number 14 by a low wall (Fig. 4.4.70).

Visible signs of decay

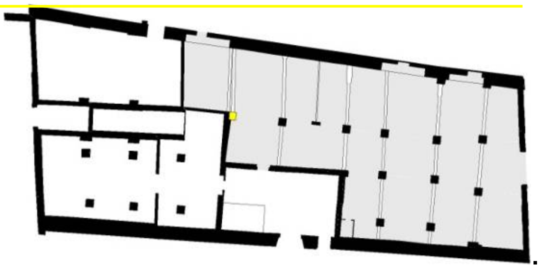
There are not structural damages. The plaster has concentrated detachments in the lower part of the element, due to the moisture rise, while it is in good condition in the upper part.

Fig. 4.4.69: Superior end of the pillar, made by a slab of stone

Fig. 4.4.70: Base of the pillar 15, connected with the pillar 14

PILLAR 16

It is the median support of the beam 7



Description of the support and geometric features

Pillar 16 is set against the outer perimeter wall, from which it protrudes out of 50 cm (Fig. 4.4.71). It has an hight of 4,50 m. At the end on which the wooden beam lean, there is a stone slab, that insulates the edge of the beam from moisture of the masonry (Fig. 4.4.72).

Visible signs of decay

There are no structural damages in the pillar. The plaster has wide detachments caused by moisture on the wall.

Fig. 4.4.71: Masonry pillar against the wall

Fig. 4.4.72: Slab of stone at the end of the pillar

4.4.4 Roofings

Currently, the roof structure looks like in the plan in Fig. 4.4.72. The area A is characterized by wooden trusses while the roof structure of the other two areas, B and C, is distinguished by reinforced concrete trusses.

Below, will be described in details the different covering structures of the building under study.

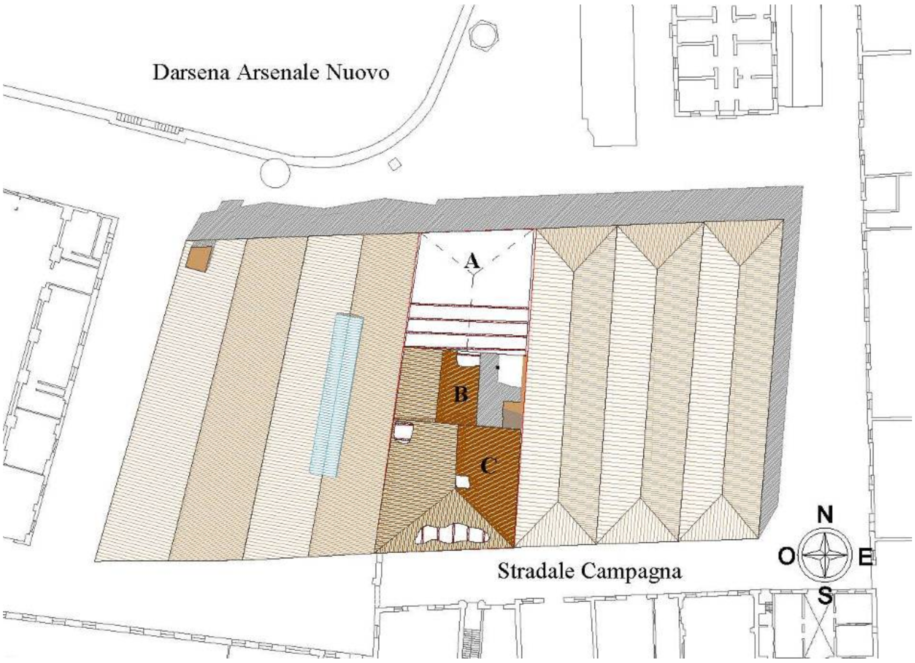


Fig. 4.4.73: Plan of the *Sale d'Armi Nord*

Area A: wooden trusses

The roof of the area A is characterize by wooden trusses. As mentioned in the previous paragraph, a part of the roof has recently collapsed due to the decay of the structural wooden elements. In particular, there was a strong degradation at the ends of the beams. The state of neglect of the building and the non-maintenance of the roof have caused local collapse of the surface roof thus allowing the atmospherics agents to penetrate in the wooden elements up to cause the loss of the load bearing capacity.

The roof structure after the collapse is schematized in Fig. 4.4.74.

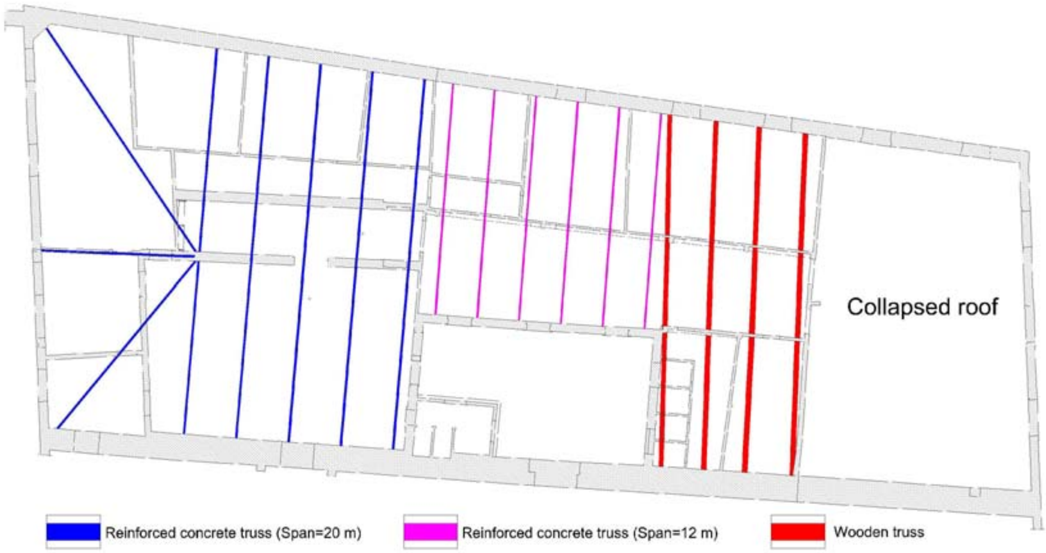


Fig. 4.4.74: plan of the roof situation at the beginning of the investigation

The not collapsed trusses are 4 (Fig. 4.4.75) and have a mean span of 16,50 meters. The wheel base between trusses is about 1,95 meters. The wood of the elements is larch, that is used in the most of the trusses present in the Arsenale of Venice. The roof of the area A has a pitch of 25° and the maximum height of the building is equal to 16,25 meters.

Fig. 4.4.75: View of the wooden trusses of the area A

The typology of the 4 trusses is queen-post truss (called "Palladiana") composed by two struts, a tie beam, a straining beam and two posts (Bettiol et. al., 2008). There is also a post in correspondence to the ridgepole. This post is not anchored at the straining beam and so it not influence the structural characteristics of the truss.

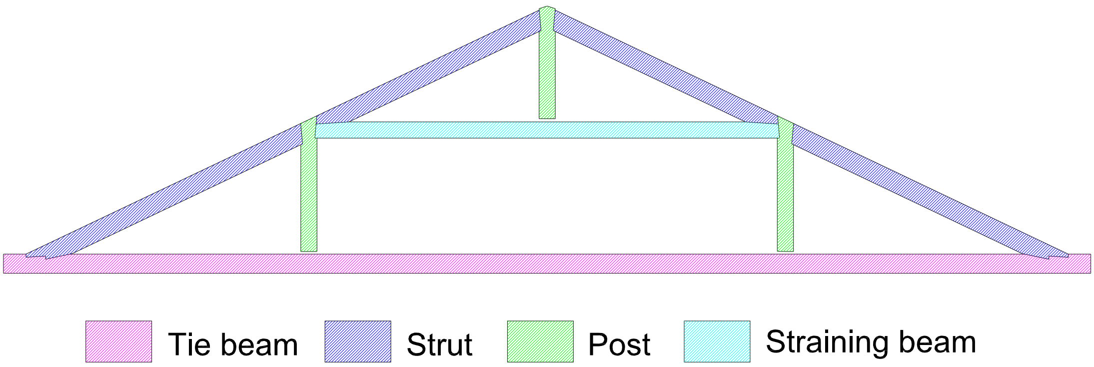
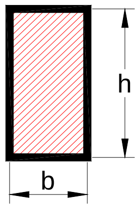
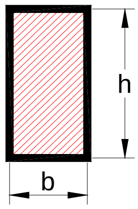
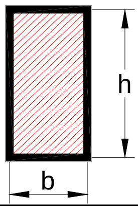
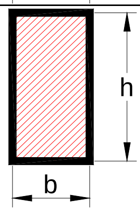
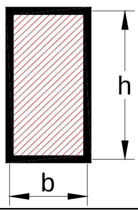


Fig. 4.4.76: "Palladiana" truss

This type of truss is characterized by an "open node" between the post and the tie beam. This constructive system was suggested by the need to not interfered in the tie beam where often were hung delicate wooden ceilings plastered and painted, or vaults made with plastered wooden centering (Barbisan, Laner, 2000). The non-joint between tie beam and struts is made to separate the structural elements in tension from the structural elements in compression, ensuring at the tie beam only the tensile stress. Even though the tie beam is not joined with the posts, the metallic stirrups braced the posts (Bettiol, 2007).

In the following table (Tab. 4.4.1) are reported the shape and the dimensions of the sections.

Tab. 4.4.1:Dimensioning of the elements of the wooden truss of the area A

ELEMENTS	TYPE OF SECTION	GEOMETRY OF THE SECTION	DIMENSIONS
TIE BEAM	Rectangular		b = 25 cm h = 30 cm
STRUT	Rectangular		b = 25 cm h = 30 cm
STRAINING BEAM	Rectangular		b = 25 cm h = 25 cm
POST	Rectangular		b = 25 cm h = 25 cm
UPPER POST	Rectangular		b = 25 cm h = 25 cm

From the study of plans and elevations of the building and the analysis of the not collapsed trusses and of the adjacent buildings it was possible reconstruct the situation before the collapse of the roof part.

Fig. 4.4.77 shows the plan of the wooden roof structure where are point out the collapsed and not collapsed trusses.

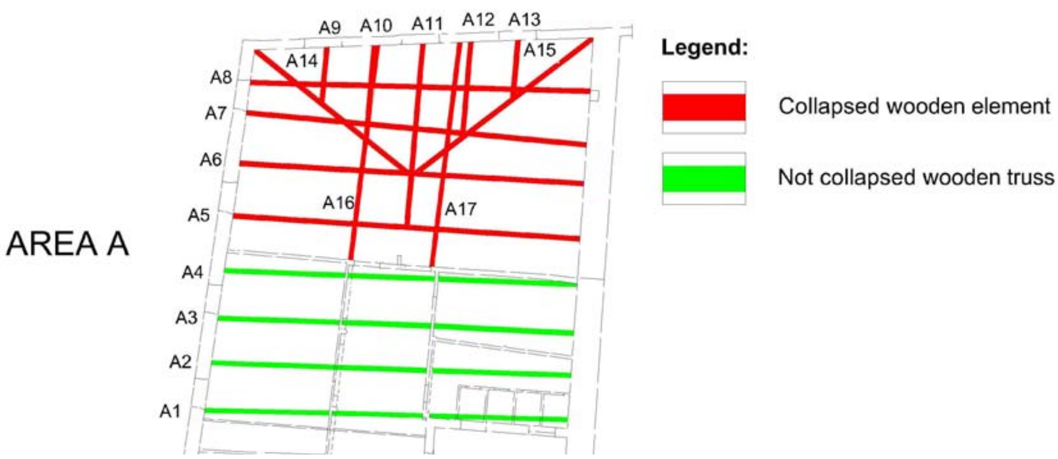


Fig. 4.4.77: Numbering of the wooden trusses (collapsed and not-collapsed)

As you can see from Fig. 4.4.77, the roof is pavilion type and it is characterize by a load-bearing frame composed by 6 trusses and one hip truss (called A7). In the truss A6 rest two rafters (A14 and A16), placed diagonally, which are connected also to the ridgepole giving the pitch at the anterior part of the roof. The façade principal frame is completed from the elements called A9, A10, A11, A12 and A13. They are composed by two wooden beam, one inclined of 28° from the horizontal and another disposed perpendicularly the wall.

The two inclined diagonally rafters and the central element of the principal frame are joined in the ridgepole of the roof, in correspondence to truss A6, and are supported by the straining beam of the hip truss.

The corresponding horizontal elements are rest in a single node in the centre of the tie beam of the truss A6. The other four raftes are connected to the façade wall and to the A14 and A15 elements. Finally the elements called A16 and A17 are the wooden supporting beams.

Below is reported a tridimensional reconstruction (Fig. 4.4.78) of the roof before the collapse.

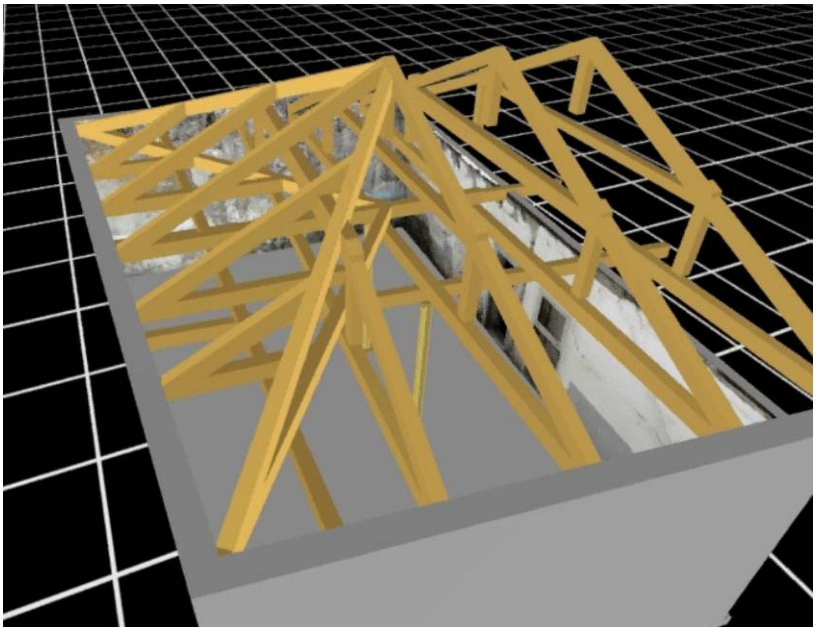
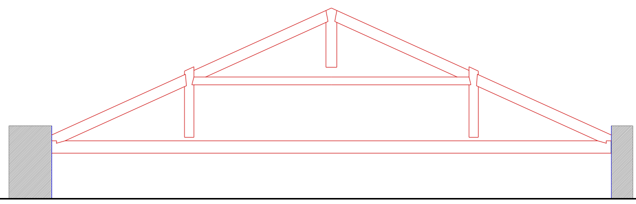
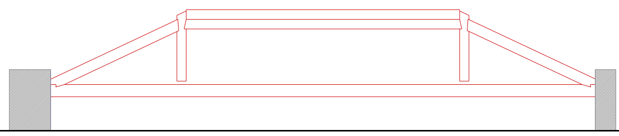
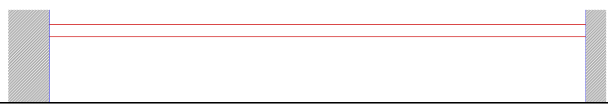
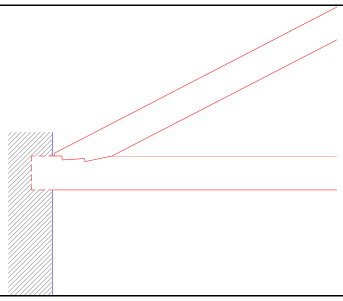
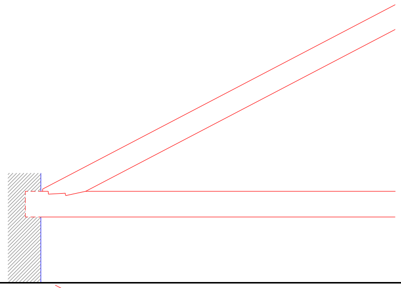
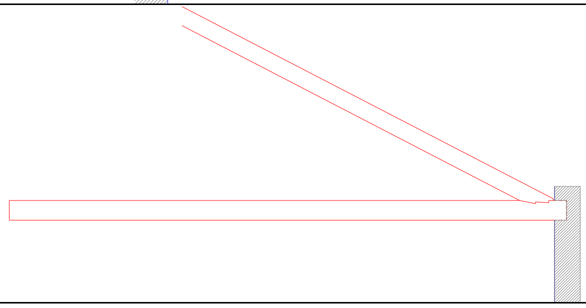


Fig. 4.4.78: Tridimensional reconstruction of the wooden roof before the collapse

In the following table (Tab. 4.4.2). is reported the geometry of the elements of the collapsed roof.

Tab. 4.4.2: Geometry of the elements of the collapsed roof

ELEMENT	TIPOLOGY	GEOMETRY OF THE ELEMENTS
A5-A6	Truss	
A7	Hip truss	
A8	Beam	

A9 – A13	Rafter	
A10 – A12	Rafter	
A11	Rafter	

Degradation of the roof

The not collapsed trusses have different and diffuse types of decay. The biotic degradation that involves the presence of insects and fungus, percolation from moisture and mechanical decay caused by cracking and breaking of the wooden elements (Fig. 4.4.79). The most diffuse damage is in the end of beams near the masonry interface, where there are high moisture zones (Fig. 4.4.80).

Also, there is a degradation layer in correspondence to the joints (Fig. 4.4.81 e Fig. 4.4.82) between the wooden elements of the trusses. For this reason it will be necessary a redevelopment intervention to give back the load bearing capacity at the roof structure.

Fig. 4.4.79: Detail of a breaking in the tie beam near the joint with the post

Fig. 4.4.80: Degradation at the ends of beams, near the anchorage

Fig. 4.4.81: Degradation of joints

Fig. 4.4.82: Degradation of joints and of the connection between masonry wall and tie beam

Below is reported the section of the present-day conditions of one of the 4 trusses. From the (Fig. 4.4.83), it is possible to see the considerable structural failures of tie beam, straining beam and the joints strut-tie beam. This section was made through a laser scanner survey.

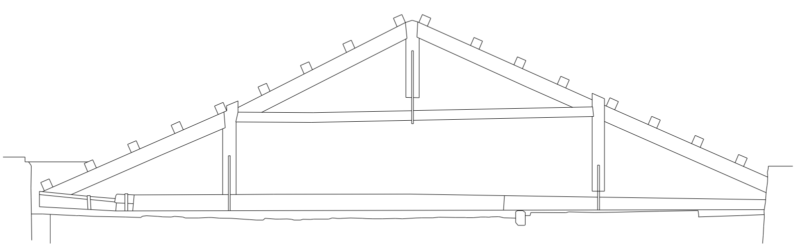


Fig. 4.4.83: Section of the present-day conditions of the truss called A2

Typology of roof

The secondary frame of the area A is constituted by purlins (section 15 x 15 cm and wheel base between two adjacent beams equal to 100 cm) and secondary beams (section 7 x 7 cm and wheel base between two adjacent beams equal to 25 cm).

Fig. 4.4.84: Detail of the secondary frame of the area A

Fig. 4.4.85 contains a schematic section of the roof elements of the area A. Besides trusses, purlins and battens, there are the wooden plank and the tile roof. The roof structure is similar at the others buildings of the Arsenale.

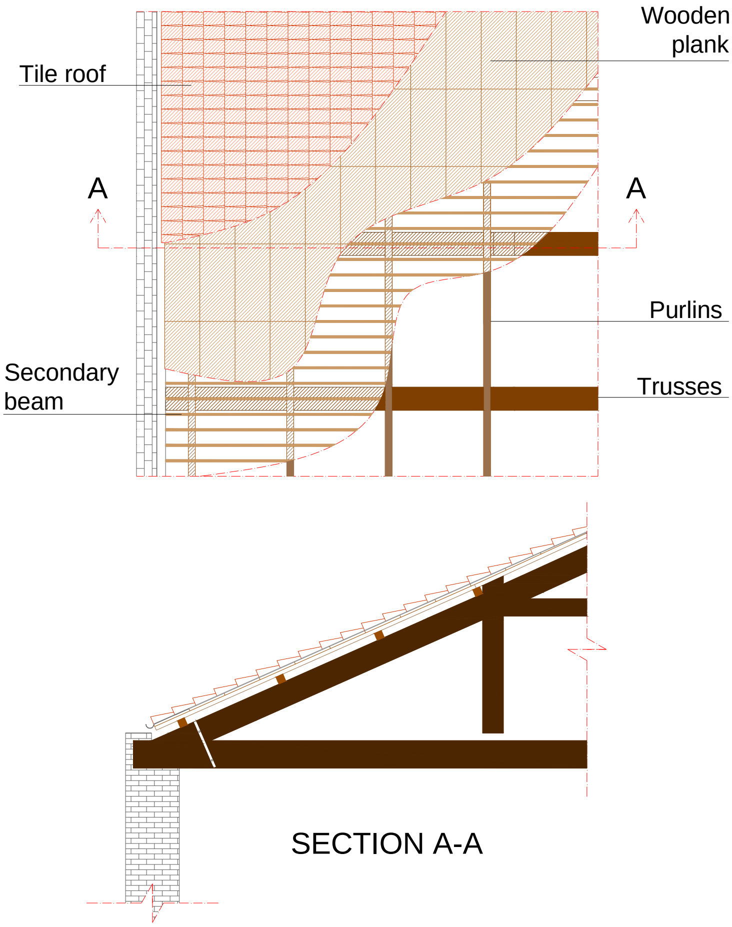


Fig. 4.4.85: Plan and section of the wooden roof

<u>**Area B: reinforced concrete trusses**</u>

The areas called B and C are characterized by a reinforced concrete roof (r.c). The trusses are different for type and dimension between one and the other area (Fig. 4.4.86).

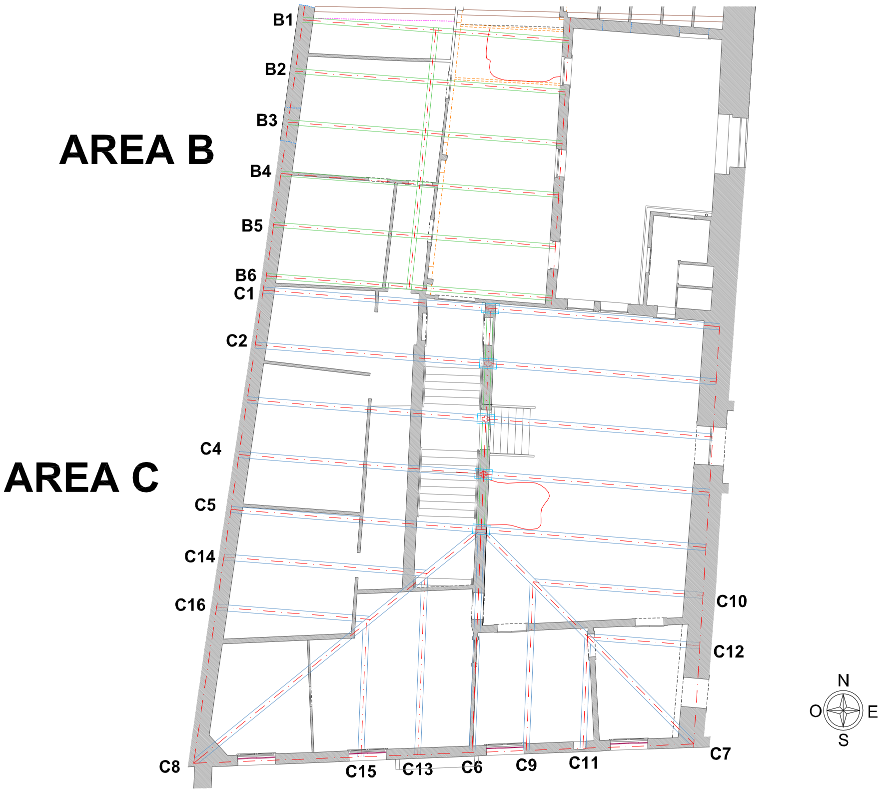


Fig. 4.4.86: Numbering of the reinforced concrete trusses

Below is reported the methodology for the characterization of materials and the geometry of elements of the area B (Fig. 4.4.73).

The Soprintendenza B.A.P. of Venezia e Laguna is financing the restoration of the trusses of this area. In order to test the trusses were conducted several on site inspections, from which has emerged a diffuse decay in all the elements of the roof.

It was necessary define the geometrical characteristics of the elements, the distribution of the reinforcement bars and the mechanical properties of reinforced concrete and steel . Therefore, it was fundamental make some on-site tests with Schmidt hammer and electromagnetic cover meter.

For this part of the building, the ridge height is equal to 15,50 meters above mean sea level (A.M.S.L). The trusses indentified in the survey are six (Fig. 4.4.87); the span of the trusses is variable between 11,00 m and 12,00 m due to the non-parallelism of the walls on which they are based. For this reason the struts of the trusses have a different length from one to another. The height of the trusses is about 3,20 m. The angle of the roof pitch change from 27° to 29°. The wheelbase between the trusses is approximately 2,10 m.

Fig. 4.4.87: View of the reinforced concrete trusses of the zone B

The trusses in the zone B are a part of the triangular scheme of reinforced concrete (r.c.) reticular trusses category, with inclined beams in tension and compression. In according to the literature the trusses can be dated to 1930. The dimensions of the components of the trusses are reported in Tab. 4.4.4 in which are contained the designs of the sections with the relative sizes. The denominations of the elements are reported in Fig. 4.4.88.

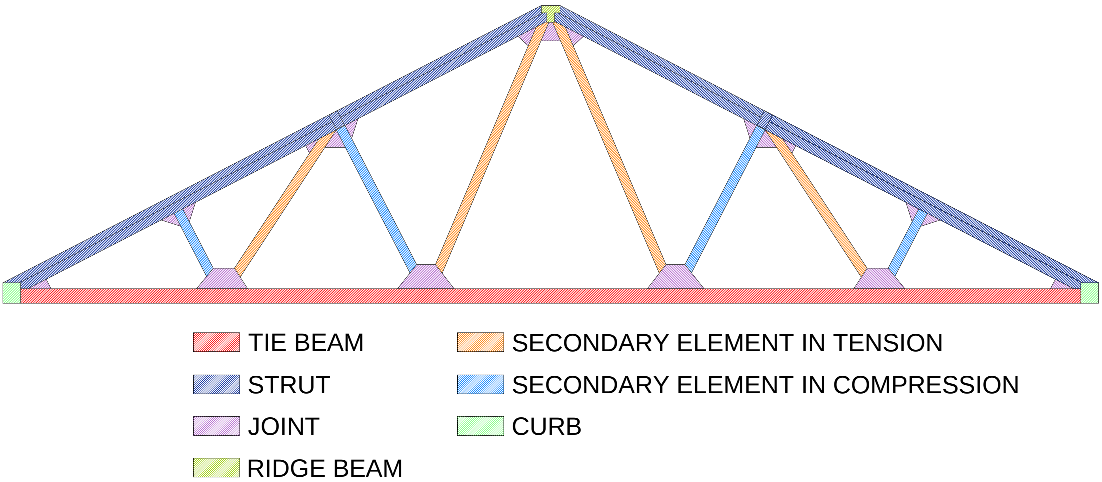
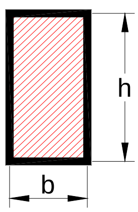
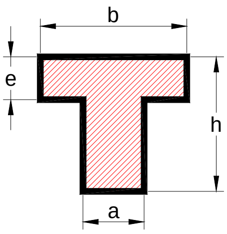
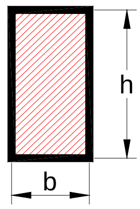
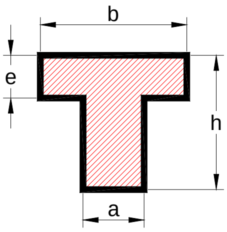
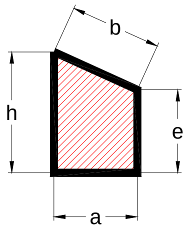
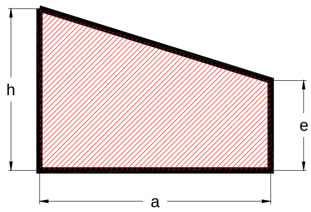


Fig. 4.4.88: Element Names of the truss in area B

Tab. 4.4.3: Dimensions of the elements of the reinforced concrete truss in area B

ELEMENT	TYPE OF CROSS SECTION	CROSS SECTION GEOMETRY	DIMENSION
TIE BEAM	Rectangular		b = 9,0 cm h = 15,0 cm

STRUT	I-Beam		a = 10 cm b = 20,5 cm e = 8 cm h = 18 cm
SECONDARY ELEMENTS IN TENSION AND COMPRESSION	Rectangular		b = 9,0 cm h = 10,0-12,0 cm
RIDGE BEAM	I-Beam		a = 8,5 cm b = 19,5 cm e = 9,0 cm h = 19 cm
PURLIN	Trapezium		a = 11,0 cm b = 12,0 cm e = 11,0 cm h = 16,0 cm
CURB	Trapezium		a = 40,0 cm h = 30,0 cm e = 15,0 cm

From on-site surveys it was possible hypothesized a constructive methodology of the trusses, made possible by observation of the joints. At the beginning were carried out the tie beams, afterwards were built the formworks for casting of the joints, indicated in Fig. 4.4.89, with the letters A, B and C (Fig. 4.4.90, Fig. 4.4.91, Fig. 4.4.92). The joints have a trapezoidal shape. They have a homogenous casting, which seems discontinuous nearness the attack of the beams. Subsequently were casted, within the formworks, the struts and the secondary elements in tension and in compression. In this case, the secondary elements that end directly in the struts and in the joints seem to have been casted in a successive phase. The joints D, E and F (Fig. 4.4.93, Fig. 4.4.94, Fig. 4.4.95) seem to be composed from concrete wedges that were designed to increase the stiffness of the elements. Tab. 4.4.4 shows the dimensions of joints (for the names of the joints we refer to Fig. 4.4.89).

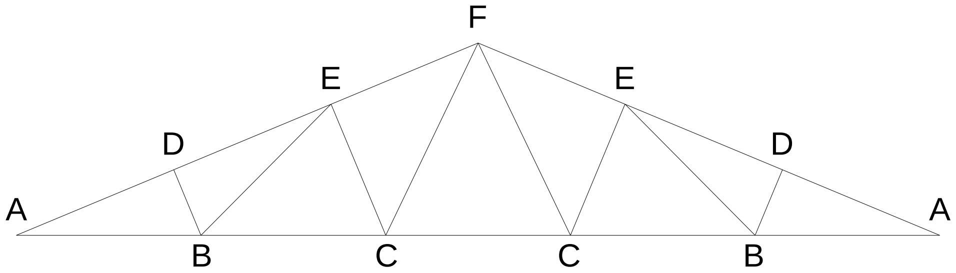


Fig. 4.4.89: Joints of the trusses in zone B

Fig. 4.4.90: Joint A

Fig. 4.4.91: Joint B

Fig. 4.4.92: Joint C

Fig. 4.4.93: Joint D

Fig. 4.4.94: Joint E

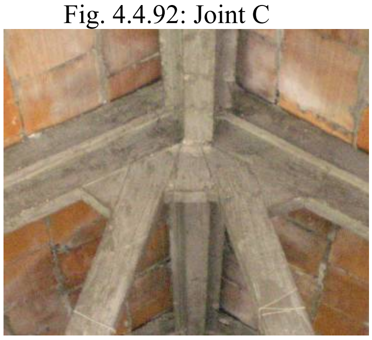
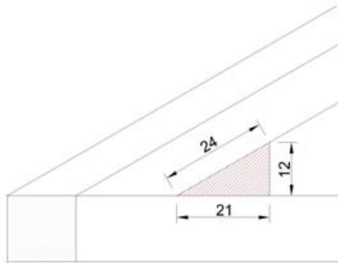
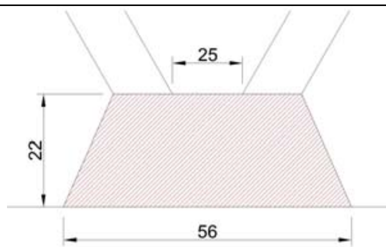
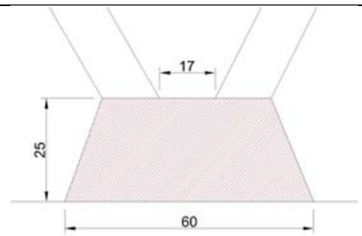
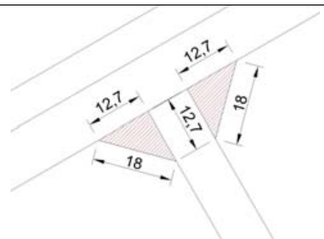
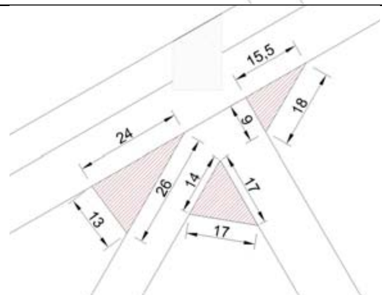
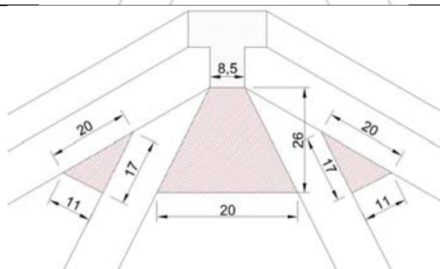


Fig. 4.4.95: Joint F

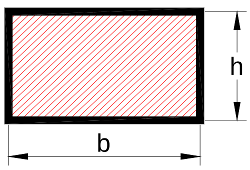
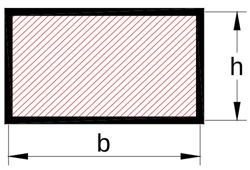
Tab. 4.4.4: Dimensions of joints of the reinforced concrete trusses in zone B

JOINT	**SHAPE**	**GEOMETRY OF THE JOINT (CM)**
A	Wedge	
B	Trapezium	

C	Trapezium	
D	Wedge	
E	Wedge	
F	Wedge	

Trusses are linked through ridge beam, purlins and reinforced concrete curbs that passing, respectively, in the joints F, E and A, in Fig. 4.4.89. The curb rests in a reinforced concrete joist. The section of the joists is rectangular and different in the two sides (Tab. 4.4.5).

Tab. 4.4.5: Dimensions of joists of the reinforced concrete trusses in zone B

Element	Type of cross section	Geometry of the cross section	Dimensions
Joist West wall	Rectangular		b = 70 cm h = 30 cm
Joist East wall	Rectangular		b = 55 cm h = 30 cm

The Fig. 4.4.96 and Fig. 4.4.97 show the shape of the support of the west wall. In the west wall, the joist has a thickness bigger than the wall, unlike the east wall where it is equal to the thickness of the wall.

Fig. 4.4.96: Detail of the curb

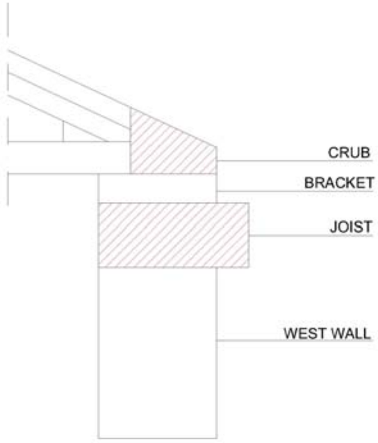


Fig. 4.4.97: Schematization of the support of the truss

The absence of the planning graphic maps of the constructive detail relating to the internal distributions of the reinforcement bars and the characterization of the mechanical properties of the materials required to make a test surveys. The impossibility to make mechanical tests directly on the reinforced concrete and on the steel reduces the level of the depth of research to LC1 (as defined in the Guidelines for evaluation and mitigation of seismic risk of cultural heritage (2006) and as in the Norme Tecniche per le Costruzioni, D.M 14/01/2008).

Knowledge survey

The information we have about the trusses of the zone B of the *Sala Maggiore* did not provide exhaustive indications about the mechanical characteristic of the reinforced concrete and about the distribution of the reinforcement bars and of the stirrups. For this reason were carried out non-destructive surveys in order to achieve an acceptable level of knowledge, that will be fundamental for the structural assessments of all the cover system.

For the evaluation of the mechanical characteristic of the materials it was decided to perform some tests with the Schmidt hammer since that was impossible make some core sampling given the reduced cross sections of the elements of the truss. The tests are:

- chemical analysis
- test with Schmidt hammer
- test with electromagnetic cover meter
- on site surveys with measurements by means of caliper, meter and distance meter.

Chemical analysis

Chemical analysis was carried out at the Arcadia Ricerche Srl, Venice, a laboratory of research and diagnostic. The test was made on a reinforced concrete specimen (Fig. 4.4.98) taken in the concrete cover in the core of the I-Beam section of the strut of the truss called B1.

Fig. 4.4.98: Concrete specimen

The information obtained concern the aggregates, the binder and the concrete mixture of the specimen. The aggregates are sedimentary origin, sandstone, and have a granulometry composition predominantly fine, because their size do not exceed 4 mm. The angular shape of the inerts is indicative of its good bond with the cement paste that assure a greater tensile strength. The presence of the ferrous oxide with a percentage equal to the 3% is justified by the presence of rust due to the oxidation of the specimen surface, caused by the widespread carbonation.

The binder is constituted by belite and celite, that are the two principal components of the idraulic Portland cement.

The concrete mixture has a areal ratio between aggregates and binders equal to 4/1. The matrix has a heterogeneous structure with a low percentage of pores, 15%. The concrete mixture can be defined in a good status of conservation.

From the chemical composition it is possible to hypothesize that it is a conglomerate with a good compression strength especially for the low porosity and the good conservation of the material. Due to the small size of the specimen it was not possible to do tests to evaluate the compressive strength of the concrete. For this reason the test with the Schmidt hammer was carried out.

Test with the Schmidt hammer

The test with the Schmidt hammer constitutes a non-destructive method able to give information about the quality of the concrete by measuring the penetration resistance capacity of the material. The method is used to estimate the cubical compressive strength of the concrete tested, to investigate the degree of homogeneity of the mechanical proprieties and to delineate zones or areas of low quality or decay. The methodology is not substitutive at the compression test on concrete specimens. The test must be carried out according to the norm UNI EN 12504-2 that defines the aim and the methodology of the test.

The Schmidt hammer is constituted by a cylindrical body fitted with a rod protruding from one end. The hammer measures the rebound of a spring loaded mass impacting against the surface of the sample. The test hammer will hit the concrete at a defined energy. Its rebound is dependent on the hardness of the concrete and is measured by the test equipment. By reference to the conversion chart, the rebound value can be used to determine the compressive strength. The bounce height is proportional to the superficial hardness of the concrete (the connection between hardness and resistance is empiric and probabilistic).

The test was carried out in the secondary element in tension of the truss called B3, in the secondary element in compression of the truss B2 and in the purlin situated between the trusses 1 and 2 (see Fig. 4.4.86 for the numbering of the trusses). In Fig. 4.4.99 is reported the map of the points in which the test was made.

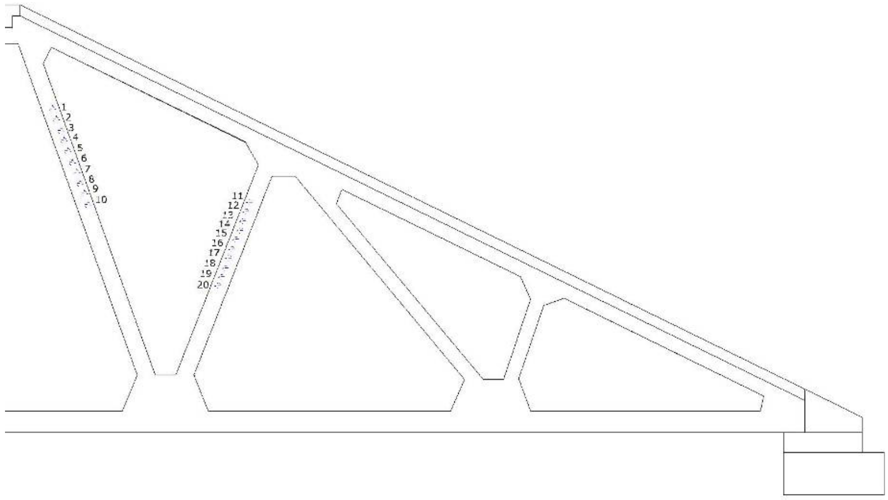


Fig. 4.4.99: Map of the points in which the Schmidt hammer test was made

Further to the processing data of the test with the Schmidt hammer it was possible to obtain the strength class of the concrete. It is included between 12 N/mm2 and 16 N/mm2.

The combination results of the chemical analysis and of the Schmidt hammer test has led to consider for the verification a concrete type C16/20. This value is compatible with the literature information, concerning to the compression strength of the concrete of the Thirties, period of the construction of the trusses under consideration.

Test with electromagnetic cover meter

Test with the electromagnetic cover meter allows to localize the reinforcement bars and the stirrups in the reinforced concrete and the size of the concrete cover. Also it gives qualitative indications on the diameter of the bars and the stirrups in relation to the difference in magnetic proprieties between them and the reinforced concrete. This non-destructive test consists in the passage of one probe on the surface of the elements. The data obtained are processed and visualized on a display and/or saved in the receiver's memory.

The test were made in different positions in order to localized the reinforcement bars and the stirrups (Fig. 4.4.100).

Fig. 4.4.100: Detail of the test with electromagnetic cover meter

The test was carried out in the secondary elements in tension (element 9) and compression (element 7) of the trusses 3 and 4. And also in the joints between the secondary elements in tension and compression (node VIII) of the trusses 4 and 5. Below are reported, in Fig. 4.4.101, the names of elements and joints that constitute the truss.

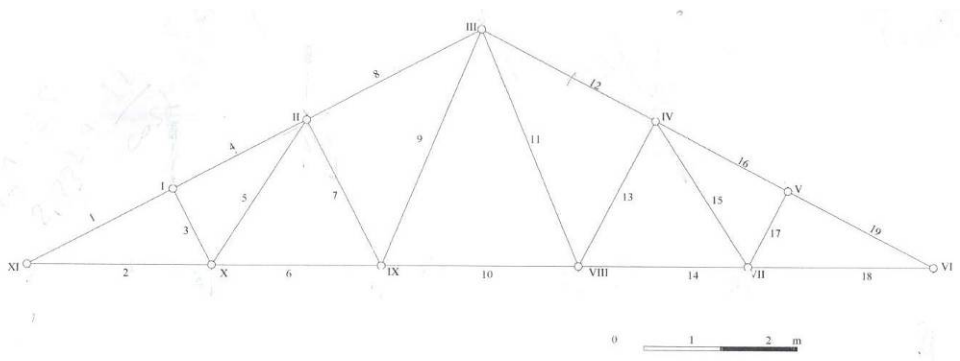


Fig. 4.4.101: Denomination of elements and joints of the truss

From the test it was possible to define the diameter and the position of the reinforcement bars and of the stirrups, and the thickness of the concrete cover. In Par. 4.6.4 are reported the distributions of the reinforcement bars and the stirrups of the elements of the trusses under consideration. The reinforcement bars are constituted by smooth bars with a variable diameter from Ø 6mm to Ø 10mm, depending of the type of element. The steel that will be using for the verifications is FeB22k.

Visual inspection

Given the presence of spallings on the concrete trusses it was possible measure the diameter and the pitch of reinforcement bars and of stirrups on sight, with the use of the caliper. So, it was confirmed the position of reinforcement bars and stirrups measured by electromagnetic cover meter, thanks to the visual observations (Fig. 4.4.102).

Fig. 4.4.102: Details of the actual conditions of the reinforced concrete trusses

Simulated project

The knowledge study of the reinforced concrete trusses of the area B was concluded with a simulated project.

The Circolare 617 of the 2nd February 2009 of the "Norme Tecniche per le Costruzioni" (D.M. 14/01/2008), in Annex C8A.1.B.3, provides the execution of the simulated project for define the knowledge level of buildings. "The simulated project is necessary, in absence of original constructive drawings, to define the quantity and the disposition of the reinforcement bars in all the elements with a structural function or of the joints characteristic. It is performed on the base of the technical rule in force at the time of the construction of the structure."

It was performed the calculation of the truss B5, using the rule of the Thirties, time of construction of the roof. It was used a simplified method. The rule used is the Regio Decreto 16/11/1939 n° 2229, "Norme per l'esecuzione delle opere in conglomerato cementizio semplice od armato". Fixed the allowable stresses, the procedure provides some verifying equations. They allow to determine the hypothetical dimension and the quantity of reinforcement bars used for the different elements.

The schematization of the truss B5 is reported in Fig. 4.4.103. The structural layout is approximated to a net-like structure with two span beam. Tab. 4.4.6 summarizes the span of the truss and the wheel base between trusses.

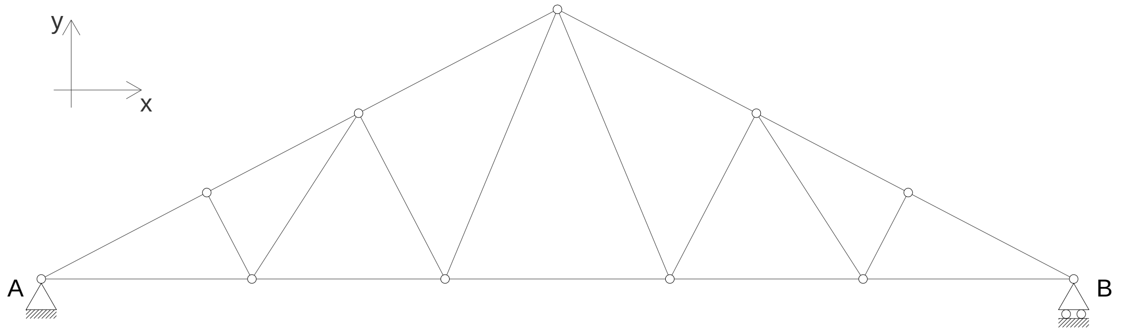


Fig. 4.4.103: Schematization of the truss B5

Tab. 4.4.6: Span and wheel base between trusses

SPAN	11,61 m
WHEEL BASE BETWEEN TRUSSES	2,10 m

Stresses were calculated by means of the effects superposition, combining the dead load of trusses and the live loads of wind and snow.

The loads used for the calculation of stresses are the weight of the truss, of ridge beam, of purlins, of tile roof and false ceiling. Fig. 4.4.104 shows the loads applied for a wheel base between trusses equal to 2,10 m. Tab. 4.4.7 reports the respective values. The load for unit of volume of the concrete is equal to 2500 [kg/m^3]. This value derives from the Art.20 of the Regio Decreto 16/11/1939 n° 2229. The loads for unit of area of the "Perret" tiles and of the Marseilles tiles were considered according to some calculation examples (Ciappi, 1935).

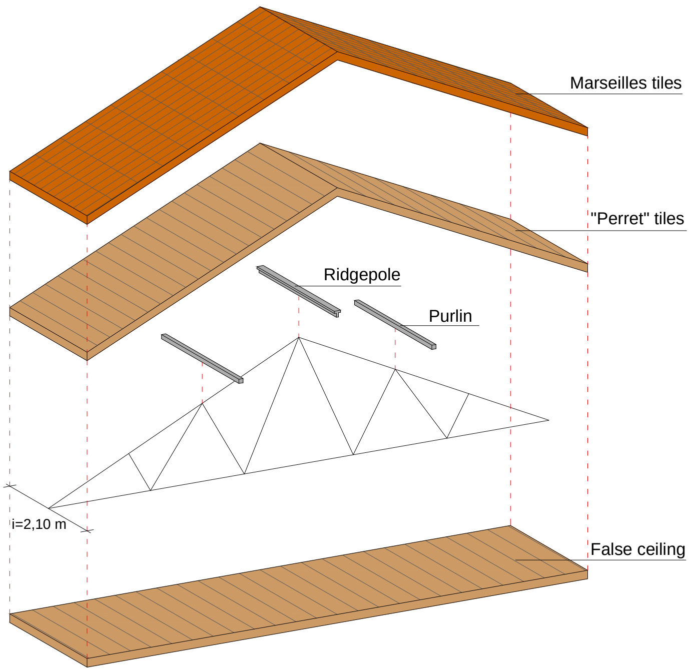


Fig. 4.4.104: Schematization of dead loads in the truss B5 (Barchi, 2009)

Tab. 4.4.7: Dead loads of the trusses of area B

LOAD	VOLUME O AREA	LOAD FOR UNIT OF VOLUME OR AREA	MASS [kg]
WEIGHT OF THE TUSS	0,64 [m^3]	2500 [kg/m^3]	1600
WEIGHT OF THE RIDGEPOLE	0,052 [m^3]	2500 [kg/m^3]	130
WEIGHT OF THE PURLIN	0,036 [m^3]	2500 [kg/m^3]	90
WEIGHT OF THE "PERRET" TILES	5 [m^2]	42 [kg/m^2]	210
WEIGHT OF THE MARSEILLES TILES	5 [m^2]	36 [kg/m^2]	180
WEIGHT OF THE FALSE CEILING	5 [m^2]	42 [kg/m^2]	210

In addition to the dead load, there will be calculated the live loads of wind and snow.

According to the literature, the symmetric vertical live loads were considered equal to 100 kg/m^2 (Santarella, 1934), resting on the struts of the truss.

Tab. 4.4.8 and Tab. 4.4.9 show the axial forces values, obtained with the effects superposition combining (dead and the live loads) of the struts, tie beam, and the secondary beams in compression and tension. In Fig. 4.4.101 is reported the denomination of the elements of the truss.

Tab. 4.4.8: Values of the axial forces in the strut and in the tie beam

STRUT	AXIAL LOAD [kg]	TIE BEAM	AXIAL LOAD [kg]
1	N_1=-7459	2	N_2=+6608
4	N_4=-6981	6	N_6=+5440
8	N_8=-5250	10	N_{10}=+3880
12	N_{12}=-5250	14	N_{14}=+5440
16	N_{16}=-6981	18	N_{18}=+6608
19	N_{19}=-7459		

Tab. 4.4.9: Values of the axial forces in the secondary elements in tension and in compression

SECONDARY ELEMENT IN COMPRESSION	AXIAL LOAD [kg]	SECONDARY ELEMENT IN TENSION	AXIAL LOAD [kg]
3	N_3=-922	5	N_5=+1392
7	N_7=-1680	9	N_9=+2005
13	N_{13}=-1680	11	N_{11}=+2005
17	N_{17}=-922	15	N_{15}=+1392

Depending on the axial stresses, the verifications performed on the beams of the truss show the distribution of the reinforcement bars, as represented in Fig. 4.4.105, and in detail in Tab. 4.4.10.

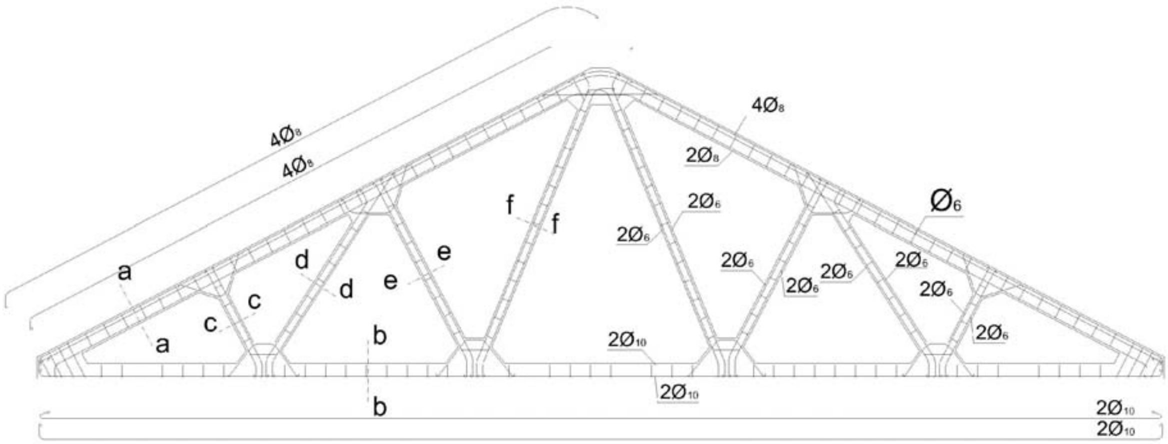
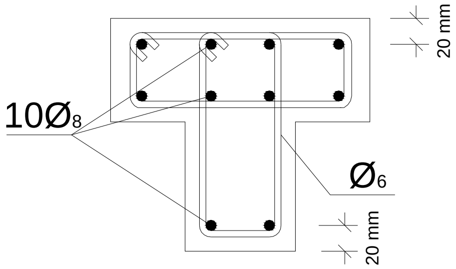
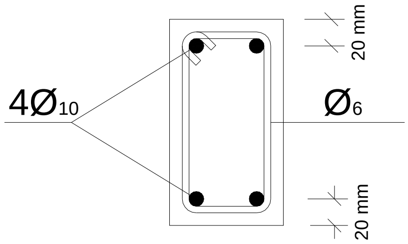
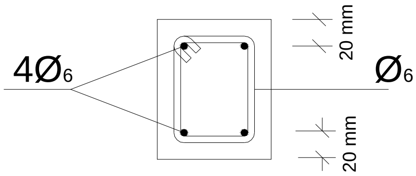


Fig. 4.4.105: Distribution of the reinforcement bars in the sections of the truss B5

Tab. 4.4.10: Reinforcement bars and stirrups in the sections of the trusses calculated in according to the Regio Decreto 16/11/1939

SECTION	DISTRIBUTION OF REINFORCEMENT BARS AND STIRRUPS (mm)
A-A	
B-B	
C-C D-D E-E F-F	

<u>Conclusive remarks</u>

The sections of the reinforced concrete truss most stressed are all verified in according to the Regio Decreto del 16.11.1939. Comparing the characteristics of the reinforcement bars calculated in according to the rule of the Thirties with those obtained through on-site investigations it is possible to see that the diameters and the numbers of bars are equal in some cases, and in other slightly higher.

In conclusion, the datas obtained with the analysis with the visual inspection, the test with electromagnetic cover meter and with the Schmidt hammer are reliable.

Degradation of the roofing

The building into consideration is located in a strongly saline environment and interested by an elevated percentage of moisture typical of the lagoonal ambient. These are the main causes of the diffused decay present in some parts of the r.c. trusses of the *Sala Maggiore*.

Numerous on-site inspections and photographic survey have permitted to identified some area of the roof in which have occurred significant collapses. In these zones the r.c. trusses are directly exposed at the atmospheric agents and, therefore, are subject to a strongly degradations. The trusses more damaged are the numbers 1 and 6, and the east part of the truss 3.

The structure is predominantly subjected at an elevated level of carbonation (Fig. 4.4.106) that, in some parts, it led to the spalling phenomenon (Fig. 4.4.107), that is the detachment of the concrete cover.

Fig. 4.4.106: Detail of the carbonation phenomenon in a joint of the r.c. truss of the area B

Fig. 4.4.107: Detail of the spalling phenomenon in the tie beam of the r.c. truss of the area B

In the r.c. trusses are also present, although in lower form, other types of decay:

- main and visible cracks in proximity of the parts where there are the detachments of the concrete cover (Fig. 4.4.108);
- poor concrete, especially in proximity of the joints (Fig. 4.4.109);
- concrete swelling that causing a superficial and localized raising of the material (Fig. 4.4.110);
- chemical attack due to algae patinas and mosses, identified by the presence of organic materials with a green colour (Fig. 4.4.111);
- presence of grout (Fig. 4.4.112);
- lacks that caused fall and loss of parts of material (Fig. 4.4.113).

Fig. 4.4.108: Detail of a crack in a strut

Fig. 4.4.109: Detail of a poor concrte in a joint

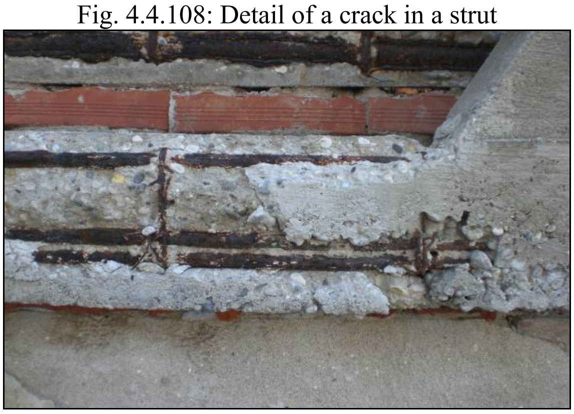


Fig. 4.4.110: Detail of concrete swelling in a tie beam

Fig. 4.4.111: Detail of a biological patina in a strut

Fig. 4.4.112: Presence of grout in a beam

Fig. 4.4.113: Detail of a lack in a strut

Typology of roofing

The roofing is characterize by a "Perret" tiles to a thickness of 3,5 cm with the addition of half centimetre of plaster, on which rest a marseilles tiles. The "Perret" system is applied both to realized the pitches of the roof (leans at the strut and at the purlins, as represented in Fig. 4.4.114) and also like false ceiling. It is fixed to the tie beam of the truss with hooks composed by wires that connect the bars of the "Perret" system from side to side of the tie beam, as reported in Fig. 4.4.115 and Fig. 4.4.116.

Fig. 4.4.114: "Perret" tiles leans in a strut

Fig. 4.4.115: Detail of the connection hook between false ceiling and the tie beam

Fig. 4.4.116: Detail of the false ceiling connected to the tie beam

<u>Area C: reinforced concrete trusses</u>

B elow is reported the methodology for the characterization of the reinforced concrete trusses of the area C (Fig. 4.4.73).

The surveys made for this area were not exhaustive due to the impossibility to reach at the roofing due to the collapse risk of the structure.

The maximum height of this part of the building is 17,80 m. The geometrical survey of the r.c. trusses was not made in a direct form but it was necessary proceed in a hypothetic way because the attic is not accessible due to the presence of the false ceiling and of the risk of collapse of the structure. In any case, it is possible to make out the trusses through some holes in the false ceiling and in the surface roof owing to localized collapses, as reported in Fig. 4.4.117 e Fig. 4.4.118.

Fig. 4.4.117: View of the trusses and of the pillar through a hole in the false ceiling

Fig. 4.4.118: View of the pillar of the truss through a hole in the surface roof

Through the hole it is possible to see a reinforced concrete pillar that not finds correspondence in the lower floors. It extends from the corbel in which leans, located in the masonry wall of the first floor, to the ridge of the roof, as shown in a longitudinal section reported in Fig. 4.4.119.

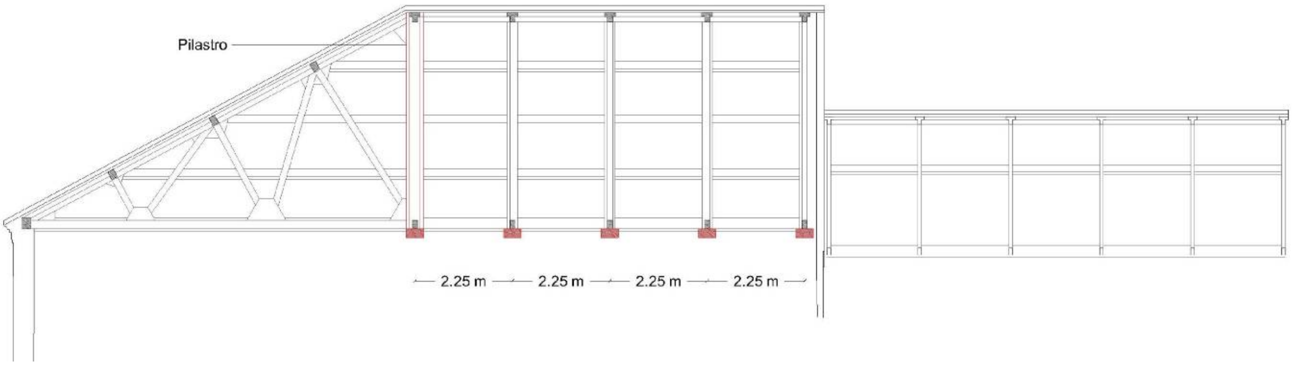


Fig. 4.4.119: Detail of the longitudinal section of the r.c. trusses of the area C

The trusses are five, as it is possible to note by the presence of 5 corbels, which have a regular spacing of approximately 2,25 m (Fig. 4.4.119). The corbels are located in the central wall of the building and are visible in Fig. 4.4.120 (inside the red circles).

Fig. 4.4.120: support corbels of the trusses

Differently from the typology of the trusses of the area B, it was here identified the presence of a vertical element (post) that joins the ridgepole to the tie beam in correspondence of the corbel.

In the area covered by a pavilion roof there are three hip trusses. The central hip truss follows the pavilion pitch in a perpendicular direction to the south façade of the building. The other two hip trusses are in between the longitudinal pitch and the pavilion one (diagonal trusses).

Fig. 4.4.121: View of the joints of the trusses in the area C

Fig. 4.4.122: Detail of the r.c. trusses in the area C

La Fig. 4.4.121 shows that other four beams lean to the south façade, as indicated in Fig. 4.4.86 with the names C9, C11, C13, C15. At these elements correspond other four beams (called C10, C12, C14, C16) arranged orthogonally at C9, C11, C13, C15, with a function as support for the longitudinal pitches.

Fig. 4.4.122 shows the detail of the trusses system that starts from the centre of the strut of the diagonal hip truss and extends in the direction of the two pitches (longitudinal and pavilion).

The span of the reinforced concrete trusses is variable from 18,00 m to 20,00 m due to the absence of parallelism of the two walls that support the roof. The tie beams have different span and are not symmetrical in respect to the central vertical rod because the east pitch is more extended that the west, as indicated in the red rectangle in Fig. 4.4.123. Therefore, the struts have different lengths not only from one truss to another but also for each truss. The trusses have a inclination of the struts variable from 26° to 27°. The diagonal trusses, called C7 and C8 (Fig. 4.4.86), have, respectively, an inclination equal to 21° and 18°. Finally, the hip trusses and the beams of the pavilion pitch have an inclination of about 28°.

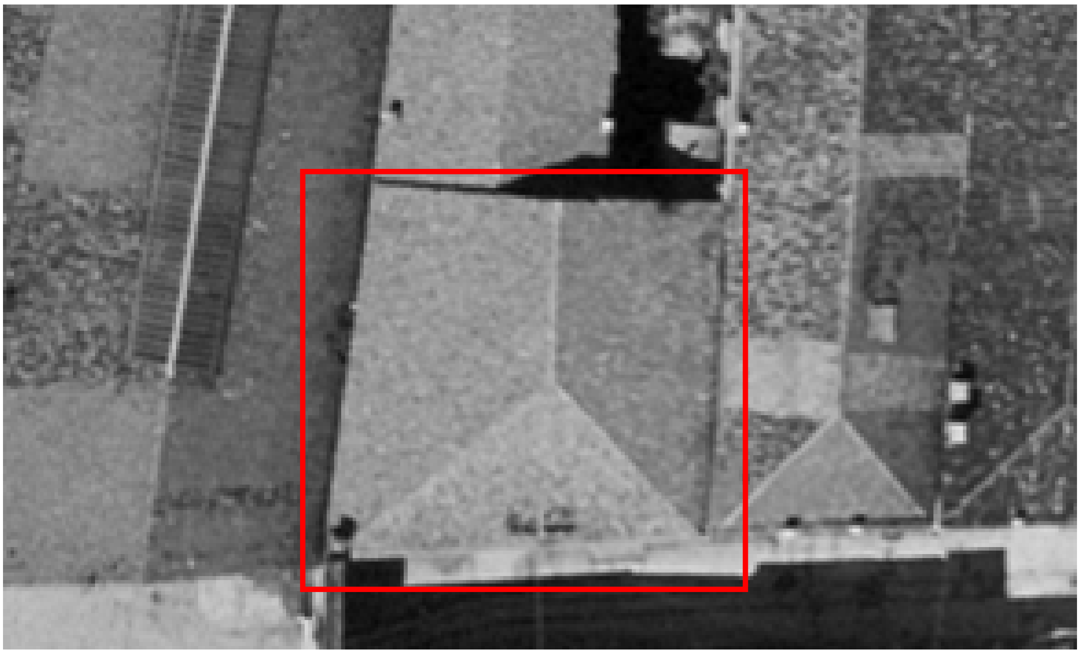


Fig. 4.4.123: Detail of the different inclination of the pitches of the area C of the *Sale d'Armi Nord*

The trusses have an height equal to 5,00 meters.

Due to the change of the lengths of struts and tie beam, the trusses have different dimensions.

The dimensions of the elements, reported in this project, that constitute the trusses of the area C are hypothetical because were derived from a global geometrical and photographic survey. The dimensions will be verified after the removal of the false ceiling, scheduled in the recovery project of the Soprintendenza B.A.P of Venezia e Laguna. In this phase of the knowledge the dimensions are defined by the comparison with the trusses of the area B.

Tab. 4.4.1 contains the hypothetical sections and dimensions of elements of the trusses. Fig. 4.4.124 shows the names of the elements that compose the truss.

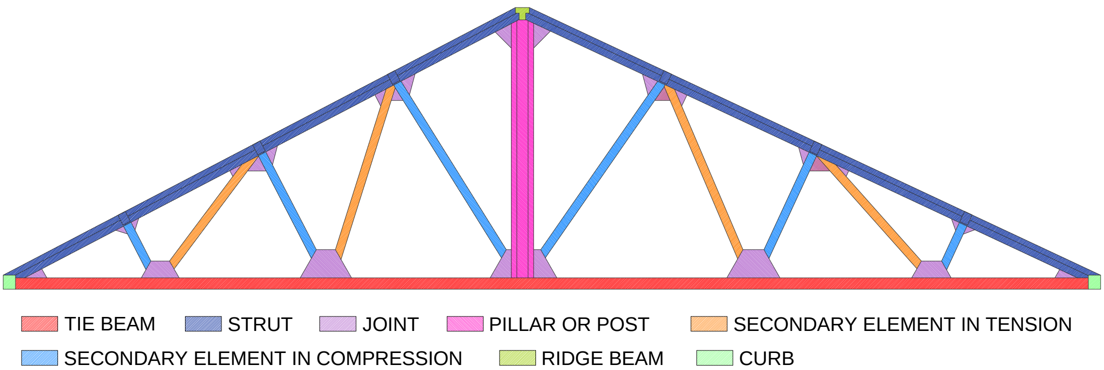
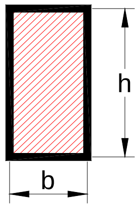
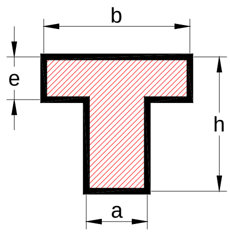
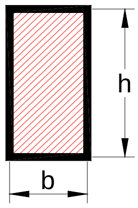
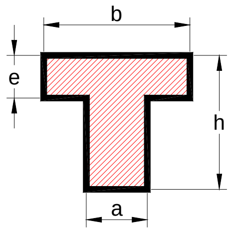
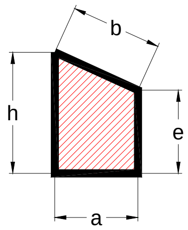
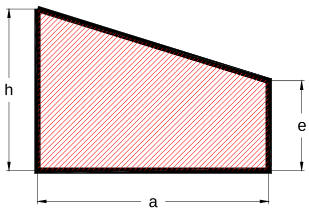
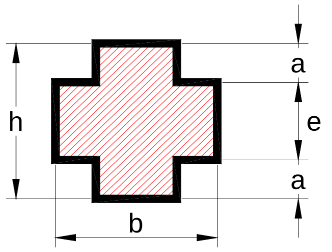
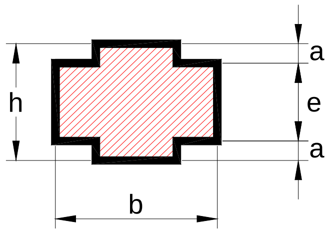


Fig. 4.4.124: Element names of the trusses in area C

Tab. 4.4.11: Dimensions of the elements of the reinforced concrete truss in area C

ELEMENT	**TYPE OF SECTION**	**GEOMETRY OF THE SECTION**	**DIMENSIONS**
TIE BEAM	Rectangular		b = 12 cm h = 20 cm
STRUT	I-Beam		a = 12 cm b = 25 cm e = 10 cm h = 22 cm
SECONDARY BEAM IN TENSION AND COMPRESSION	Rectangular		b = 12 cm h = 15 cm
RIDGE BEAM	I-Beam		a = 12 cm b = 25 cm e = 10 cm h = 22 cm
PURLIN	Trapezium		a = 15 cm b = 16,5 cm h = 22 cm e = 15 cm
CURB	Trapezium		a = 40,0 cm h = 30,0 cm e = 15,0 cm

PILLAR (TRUSS C5)	Cross		b = 40 cm h = 40 cm a = 7,5 cm e = 25 cm
POST	Cross		b = 30 cm h = 24 cm a = 6 cm e = 12 cm

The beams system is composed by the same elements of the trusses of the area B, with the exception of the pillar and of the post, as shows in Fig. 4.4.125.

Fig. 4.4.125: View of the beam system of the area C

Also the joints were not measured directly but by the photographic survey has been observed that the shape and the dimensions are similar to those of the trusses of the zone B. The joints between the secondary elements and the tie beam, indicated in Fig. 4.4.126 with the letters B, C and D, have a trapezoidal shape. The others between beams and struts, indicated in Fig. 4.4.126 with the letters E, F and G, and tie beam and strut have a wedge shape.

As for the section of the elements of the truss, the dimensions of the joints have been established using those of the area B and from the literature information. In Tab. 4.4.12 are reported the dimensions of the joints.

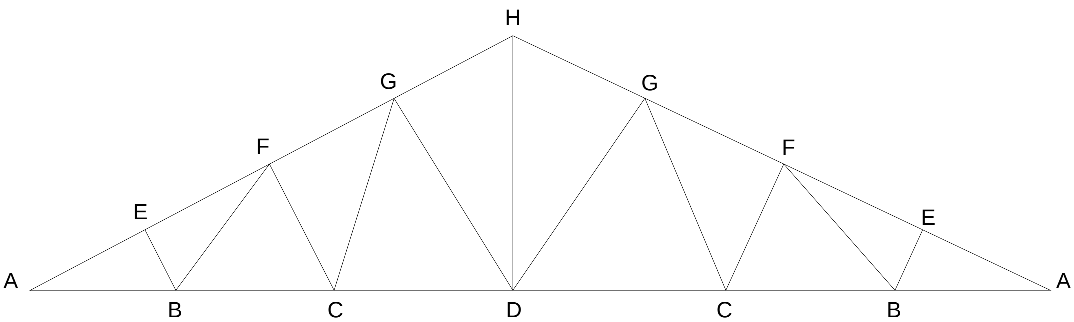
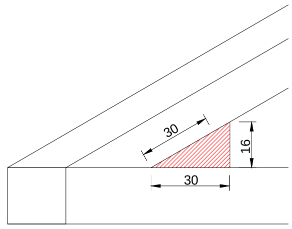
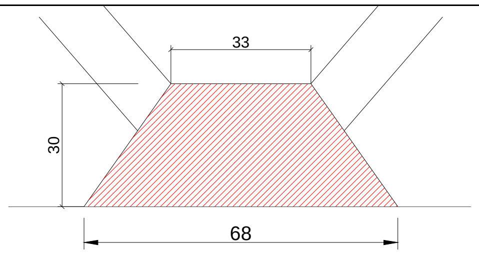
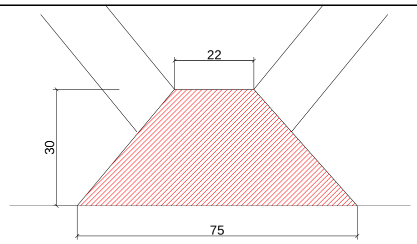
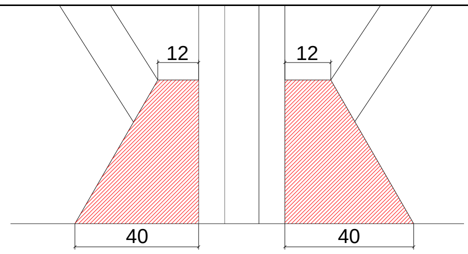
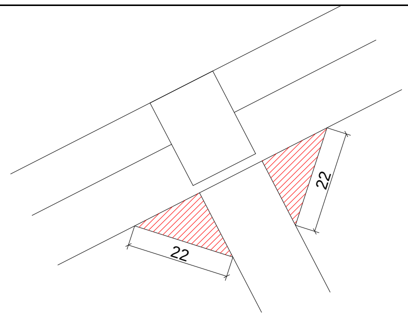
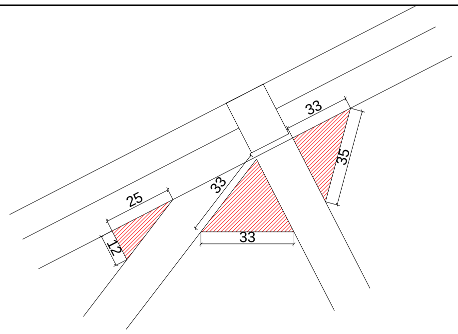
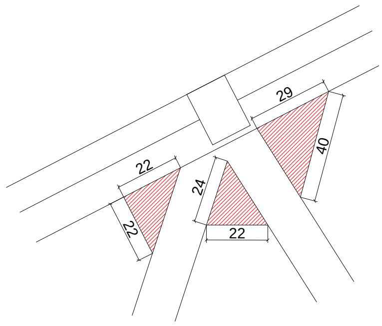
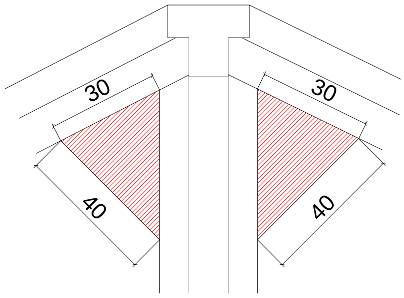


Fig. 4.4.126: Joints of the truss in zone C

Tab. 4.4.12: Dimensions of joints of the reinforced concrete trusses in zone C

Joint	**Shape**	**Geometry of the joint (cm)**
A	Wedge	
B	Trapezium	
C	Trapezium	
D	Trapezium	
E	Wedge	
F	Wedge	

G	Wedge	
H	Wedge	

There are not information about the geometry of the support of the truss on the wall, schematized in Fig. 4.4.126 with the letter A. However, it is possible to understand the distribution of the elements considering some photos, as the Fig. 4.4.127. The support of trusses is at a higher level, compared with the one of the area B, of about 60 cm., as is reported in the longitudinal section in Fig. 4.4.128. Moreover, there is a masonry eaves as for the B zone. For this reason, the support of the trusses can be considered similar to the B, whose representation is reported in Fig. 4.4.97. The presence of the joist in not assured.

Fig. 4.4.127: View of the roof of the areas B and C

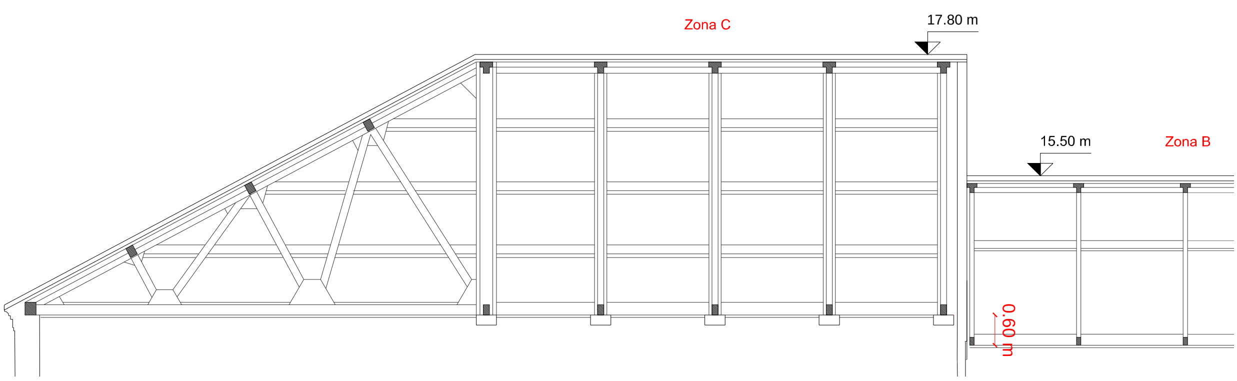


Fig. 4.4.128: Section of the reinforced concrete trusses of the zone C

The inaccessibility in this part of the roof not has allowed to make on-site tests to define the chemical and mechanical characteristics of materials and obtain information about the distribution and the dimension of reinforcement bars and stirrups.

Degradation of the roofing

The main degradation of the trusses in the area C is in correspondence of the localized collapses of the roof structure. Below the numeration reported in Fig. 4.4.86, the trusses that present the most damage are: C1 and C2, where the decay is concentrated at the end of the west strut, and C6, C9, C11, C13 and C15, where the degradation implicates all the element. For the other trusses the view is not allowed due to the presence of the false ceiling, but it is possible to suppose that the decay is similar to the five trusses of the area B, not interested by a diffuse damages.

Fig. 4.4.129 shows the presence of vegetation and the collapse of part of the false ceiling. It is not possible define precisely if there are other type of damages, but given the exposure of the trusses to the atmospheric agents, probably the structure is interested by the carbonation phenomena.

Fig. 4.4.129: View of the trusses C1 and C2

In the south part of the roof, the trusses most degraded are the ones that form the pavilion pitch (C6, C9, C11, C13 e C15). The conditions of the beams were detected through a hole in the false ceiling, as reported in Fig. 4.4.130, that shows the presence of algae patinas in the tie beam. As in the previous cases, it is evident the poor concrete and the diffusion of carbonation attacks. In the struts there are sediments of tile roof, as showed in Fig. 4.4.131.

Fig. 4.4.130: View of the truss C9

Fig. 4.4.131: View of the reinforced concrete beams that form the pavilion roof

In conclusion, in Tab. 4.4.13 is reported a degradation map of the roofing of the *Sala Maggiore*.

Typology of roofing

In the zone C was used both for the roof structure and false ceiling a "Perret" tiles system. Also, the roofing is characterized by marseilles tiles.

Tab. 4.4.13: Degradation map of the roofing of the Sala Maggiore

DEGRADATION OF THE ROOFING

LEGEND: Vegetation — Damaged roofing or sediments of material

4.5 Summarizing forms

The investigations permitted to obtain a complete summary of geometrical, material and damage characteristics and the structural evaluation of the building.

All the informations allowing a clear identification of the different elements composing the structure, are reported in summarizing forms, which represent an important tool both for a quick and plain comprehension of the structural functioning and for being an evidence of the present day conditions of the structural elements, for possible future studies and interventions.

For the main beam was created a summarizing table for each element. It contains the plan of the building on which is indicated the element into consideration, the most relevant photos, the structural frame, the typology of material and a description of the span of the element. Also, the main information about the degradation condition are reported.

The signs of decay visually detectable on main beams were classified into three types:

- load-bearing frame depression, which is manifested by the loss of horizontality of the element, due to the state of stress-caused by loads on the floor and then by deformation of the beams;
- settlements ofthe supports;
- material decay, due to attacks by fungus and xylophagous insects, to fire and to moisture.

Below an example of the summarizing table of the main beam called 1 is reported (Fig. 4.5.1).

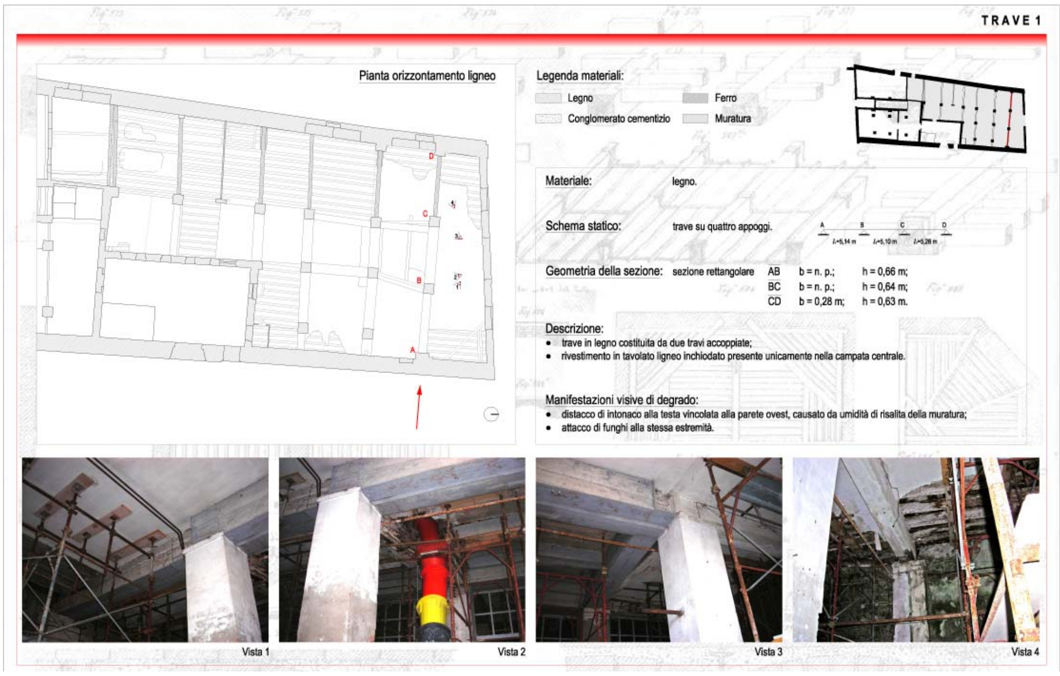


Fig. 4.5.1: Summarizing table of the main beam 1

The summarizing tables of the secondary frame contain the general plan of the floor on which are indicated the elements into consideration, some relevant photos, the material typology, the structural layout and the span width of the floor. Also, in the table the description of the intrados of the floor, the characteristic dimensions of purlins and degradation conditions are included.

The signs of decay were classified, as the main beams, in three types:

- depression of load bearing frame, which is shown by the deformation of the horizontal floor;
- supports settlements;
- material decay due to attacks by fungi and xylophagous insects, by fire and moisture.

In Fig. 4.5.2 an example of the summarizing table of the secondary frame is reported.

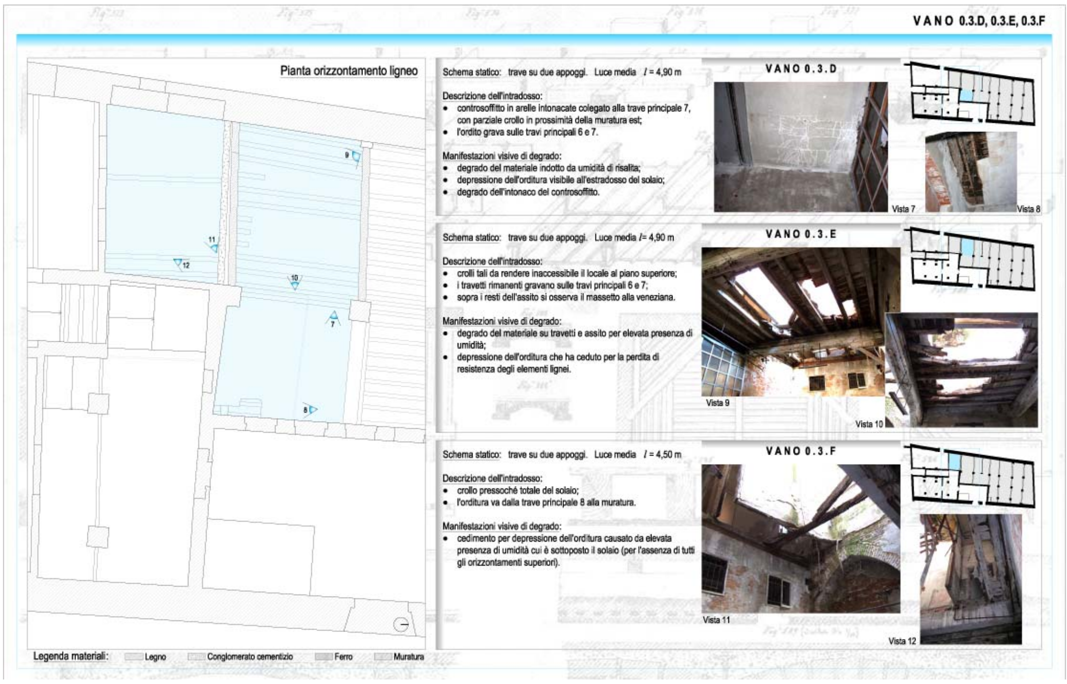


Fig. 4.5.2: Summarizing table of the secondary frame

The summarizing tables of the masonry pillars contain the plan of the floor on which are indicated the elements into consideration, some relevant photos, a description of the main characteristics of the pillars, including the base and the height derived from the geometric survey, and the degradation conditions. Decay is due in particular to the plaster detachments caused by moisture rise and pulverization of the mortar joints.

Below an example of the summarizing table of pillars is reported (Fig. 4.5.3).

Fig. 4.5.3: Summarizing table of pillars

For the covering structure were created the summarizing table contained the geometry of the sections. For the reinforcement concrete trusses were also indicated the distribution of the reinforcement bars and the degradation condition of the elements.

4.6 Methodological study and intervention phases

The preliminary phase of knowledge of the building led to the creation of a general overview of all the main aspects, issues and problems of the building into consideration. The historical analysis has allowed to understand the constructive evolution of the building and insert the historical transformations into a precise periods.

Information that were not present in the literature have been compensate by the study of the actual condition of the building. In particular, the geometrical survey have emphasize the complexity of the roof structure. It was been interested by numerous transformation, some dating back to last century, that have radically changed the original layout.

The weaknesses of the structure emerged from the geometrical, structural, degradation and materials survey. It has permitted to evaluate the entity of the damage and the type of intervention, both structural and material, to develop.

The main problem that characterizes this complex is the diffuse degradation of the carrying structures. In particular, the problems are concentrated in the horizontal elements such as floors and roof. These have in fact been subjected to partial collapses. The consequences were water seepages and humidity that have undermined the stability of the structure.

The preliminary analisys of the structure allowed to switch to the next step of identification of the intervention stages. Without these informations it would be difficult identified the intervention to be carried out without modify the structure with invasive or too much restrictive solutions.

To carry out the necessary verification and to know the global stresses of the structure and the local ones for each element numerical models were created. This give the opportunity to evaluate the stresses in the *Sale Maggiore* identifying the areas with the most risk to collapse.

The obtained informations were integrated with the degradation survey.

To test the metodology, proposed in this research, in different situations of damage, configuration and materials, are following suggested three different types of intervention, as reconstruction of the wooden trusses collapsed, according to the sixteenth-century building techniques, the conservation of those not collapsed and the structural reinforcement of the reinforced concrete roof.

In the verifications carried out, wasn't done the analysis of seismic behaviour of the building, since Venice has a very low seismic degree and geometry of the structure is regular, with limited height.

In effect, the seismic history of Venice shows that the seismic events that have affected the study area from 1065 until now were of limited size and did not cause major structural damage. From the official website of the INGV (National Institute of Geophysics and Volcanology) was extracted the graph (Fig. 4.6.1) of the history of seismic intensity (Is) which involved the area of Venice. It shows that the maximum intensity of earthquakes happened in Venice were of degree 7 on the Modified Mercalli Intensity (falling chimneys, damage to buildings), which occurred between 1200 and 1500 approximately.

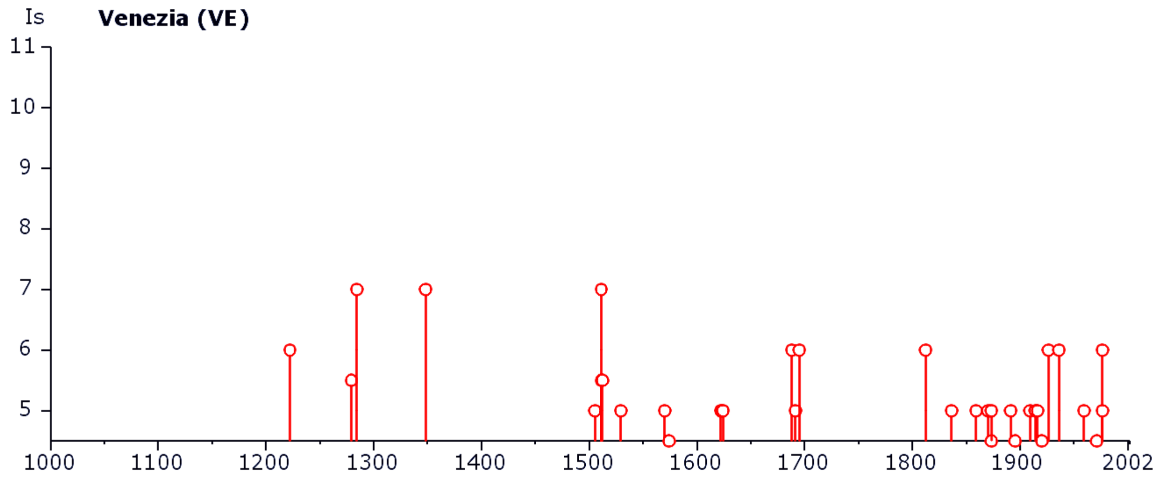


Fig. 4.6.1: Graph of the history of seismic intensity (Is) in the Venice area

In this paragraph are listed all the verifications carried out, by referring to the current regulations, and necessary actions to restore the structure to its original structural and aesthetical configuration.

4.6.1 Finite Element Modeling (FEM)

The Sala Nord Ovest has been interested, over the centuries, by many maintenance works rather than genuine restructuring, which have made it the sum of different construction techniques. Thus, in the building can be found structure elements of sixteenth century, such as wooden trusses of the roof, mixed together with the twentieth century reinforced concrete beams. The same trend is observed in the horizontal elements. The ground floor is separated from the first through floors made with different materials: wood, iron and concrete. Plausibly, these are results of the building maintenance done, simultaneously with construction of the covering, in the first decades of the twentieth century (Gambirasi, 2009).

The building under consideration belongs to the building complex of the *Sale d'Armi Nord*. The longitudinal perimeter walls of the Room separate it on the east side from another building and on the west side from the maritime warehouses of *Darsena Nuova*. In the Fig. 4.6.2 is reported a portion of a photomap where is possible to see the *Sale d'Armi Nord*, before the collapse of the roof of *Sala Ovest*, with the adjacent maritime warehouses. The façade walls are adjacent to those of the front of surrounding buildings.

The realization of the final numerical model of the *Sala Nord Ovest* will consider the aspects described above, namely heterogeneity of structural elements of the building and adjacency to other buildings. This will make possible to assess the general behaviour and the structural issues which apply to a building affected by actions of different nature. The Finite Element Model (FEM) refers to the structural configuration of the *Sala Nord Ovest* obtained by considering the wooden roof and floors before the collapse.

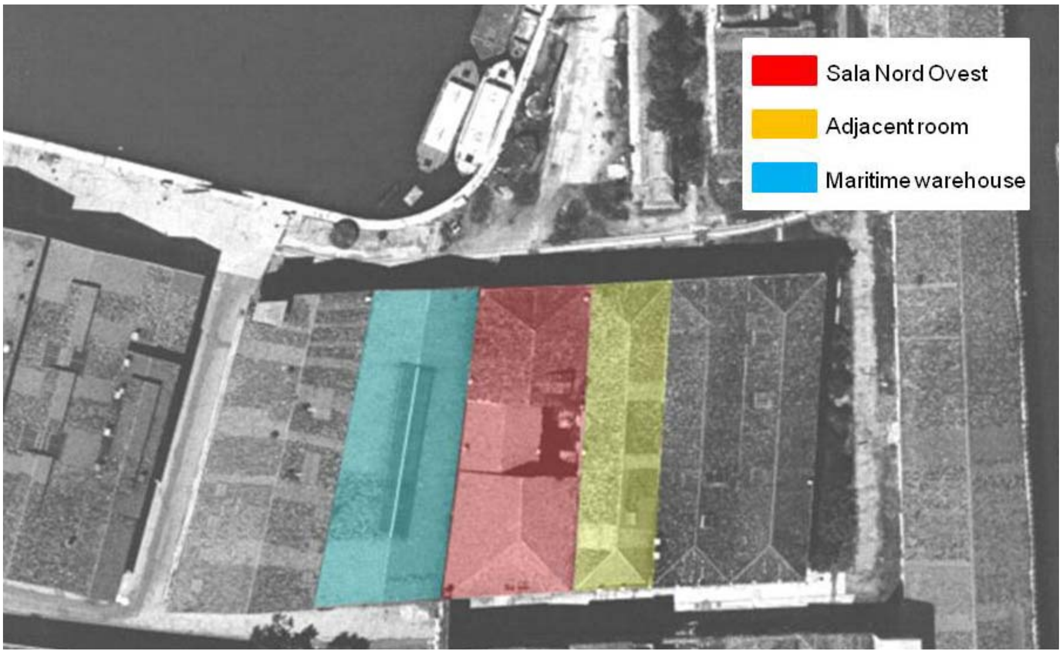


Fig. 4.6.2: Complex of the *Sale d'Armi Nord* and the maritime warehouses (the buildings under consideration for the modeling are highlighted with different colours)

<u>Main features of adjacent buildings</u>

The main features of the buildings adjacent to the *Sala Nord Ovest* have been identified through on-site and photographic surveys. The maritime warehouse is, as the name suggests, a one-level warehouse, with a gable roof made with "English" trusses (cambered howe truss) with a wheel base between trusses equal to 3 m, dated to around 1905 (Bettiol, 2007), and characterized by the presence of a skylight (Fig. 4.6.3). A pitch of the roof loads all the weight on the longitudinal western wall of the *Sala Nord Ovest*, at the height of 7,50 m, below the edge of the wall. The second pitch is consecutive to that of the roof of another adjacent warehouse. The "English" truss (Fig. 4.6.5) acts statically as a reticular structure, subjected only to loads applied at the nodes and solicited, therefore, only by the normal stress (Bettiol, 2007). Inside the warehouse, a loft along the longitudinal western wall is supported by steel columns.

Fig. 4.6.3: Internal view of the maritime warehouse roofing: "English" trusses and skylight

Fig. 4.6.4: Internal view of the ground floor of the building next to the *Sala Maggiore*

The room adjacent to the *Sala Nord Ovest*, from which is possible to observe the interior of the ground floor, has characteristics similar to the building under consideration. The ground floor is divided into rooms obtained with low wall elements and wooden window frames, as well as two rows of steel columns to support carrier warping of the first floor. The first floor is made of iron and it has double warping made of hollows tiles and beams. The roof is a gable roof with a terminal pavilion (Fig. 4.6.2). It is reasonable to consider that there are the same trusses'typology called queen-post truss ("Palladiana") identified in the *Sala Nord Ovest* (Fig. 4.6.6). The trusses load all the weight up one extreme on the longitudinal eastern wall of the building studied.

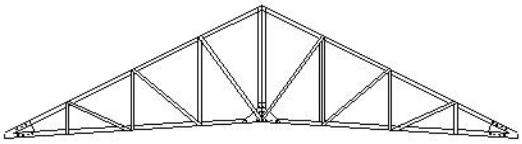


Fig. 4.6.5: "English" truss (Bettiol, 2007)

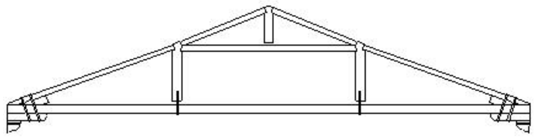


Fig. 4.6.6: Queen-post truss (called "Palladiana") (Bettiol, 2007)

<u>Numerical model construction</u>

The finite element model was created with the computer program Straus7 of HSH ® software (Finite Element Analysis System - Release 2.3.3). The model implemented includes:

- the *Sala Nord Ovest*;
- the adjacent room;
- the maritime warehouse.

Finally, it was decided to model a small portion of the façade masonries of the buildings nearby to the warehouse and the adjacent room to have the best structural understanding of the building complex.

In the modelling of masonry walls were used two-dimensional *plate*-type elements, distinguished into *quad4* and *tri3*, necessary for the arched openings' implementation. The two-dimensional *beam* elements were used to model the bearing frame of the floors, the pillars and the elements of roof trusses made in wood and reinforced concrete. The English trusses, depending on the operation of reticulate structure, were modelled through *truss* element, which ensures that only the axial efforts are transferred, cancelling all continuity of bending solicitation.

Each node placed at zero in the considered reference system was characterized by a joint constraint, preventing movements along the three Cartesian axes and rotations around the same axes. To simplify, were not considered the gradients from the ground altitude reported in the geometric survey. The static scheme of the main beams of the floor is a two-support beam. About the iron floors, in the traditional masonry structures iron beams are considered simple-support beams, and the same simplification can be extend to the floor of reinforced concrete, because is possible to exclude the existence of a curb that confers rigidity to the beam-wall connection. The nodes on the free vertical edge of the unfinished facade walls, movements in directions x and z are constricted (Fig. 4.6.7).

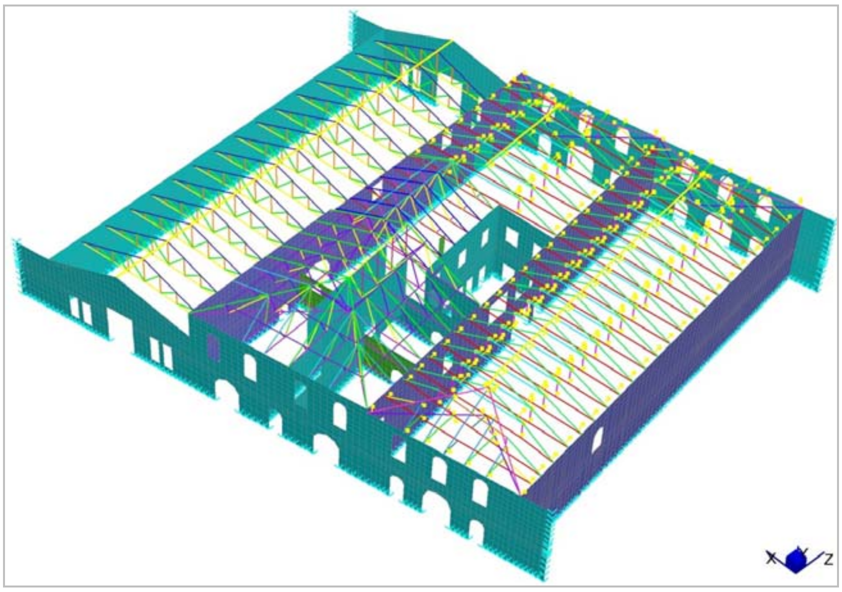


Fig. 4.6.7: Axonometric view of the global numerical model of the three buildings

The reinforced concrete beams, to the cause of the presence of the curb, are considered to be constrained to joint to masonry. To achieve this objective it is enough to ensure the connection of the node of the rafter's *beam* element to the *plate* element node of the wall. The steel trusses do not require the imposition of any restrictions on the use of truss elements, whereby the connection of the warehouse's roof to the walls is accomplished through a simple support. In the steel and wooden roofing was necessary to add a *link* of *Master/Slave* type to link all the strut-strut nodes to the trusses' ridge and ensures congruence to movements in the z direction of the elements considered. For the reinforced concrete roofs, this issue is guaranteed by the presence of the ridge beam and purlins of the same material (Fig. 4.6.8).

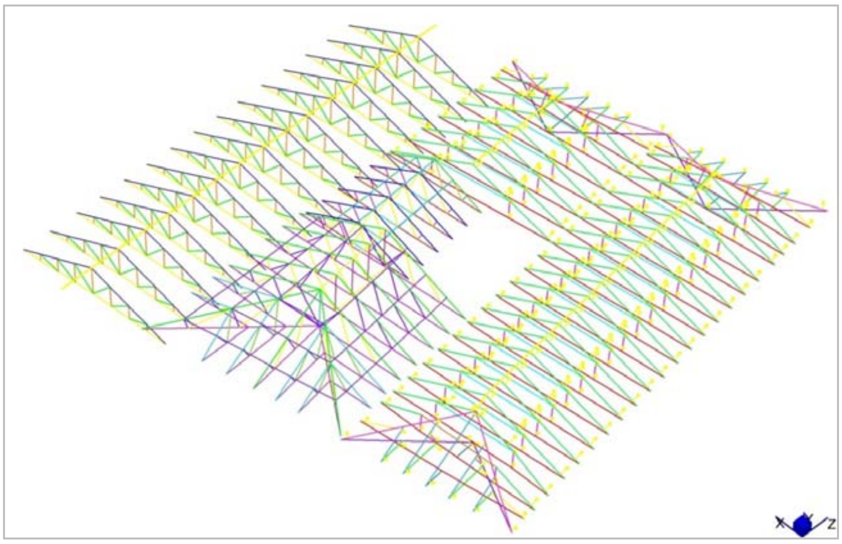


Fig. 4.6.8: Axonometric view of the roof numerical model of the three buildings

In the following tables (Tab. 4.6.1, Tab. 4.6.2, Tab. 4.6.3, Tab. 4.6.4, Tab. 4.6.5 e Tab. 4.6.6) are summarized the mechanical properties values for the materials of the structural elements composing the model.

Tab. 4.6.1: Mechanical properties of the masonry

Mechanical properties of masonry	
Young's modulus, E	2200 N/mm^2
Shear modulus, G	880 N/mm^2
Poisson's ratio, ν	0,25
Density, γ	1500 kg/m^3

Tab. 4.6.2: Mechanical properties of the wood using for the floor

Mechanical properties of the wood using for the floor	
Mean value of modulus of elasticity, $E_{0,mean}$	14500 N/mm^2
Shear modulus, G	910 N/mm^2
Density, γ	600 kg/m^3

Tab. 4.6.3: Mechanical properties of the wood using for the roof

Mechanical properties of the wood using for the roof	
Mean value of modulus of elasticity, $E_{0,mean}$	1200 N/mm^2
Mean value of shear modulus, G_{mean}	750 N/mm^2
Density, γ	600 kg/m^3

Tab. 4.6.4: Mechanical properties of the cast iron

Mechanical properties of the cast iron	
Young's modulus, E	$6,7 \cdot 10^4$ N/mm^2
Mean value of shear modulus, G_{mean}	$G=7,19 \cdot 10^{-1}$ N/mm^2
Poisson's ratio, ν	0,28
Mean density, r_{mean}	7250 kg/m^3

Tab. 4.6.5: Mechanical properties of the steel

Mechanical properties of the steel	
Young's modulus, E	206000 N/mm^2
Shear modulus, G	79231 N/mm^2
Poisson's ratio, ν	0,30
Density, γ	7850 kg/m^3

Tab. 4.6.6: Mechanical properties of the reinforced concrete

Mechanical properties of the reinforced concrete	
Young's modulus, E	20000-22500 N/mm^2
Poisson's ratio, ν	0,20
Density, γ	2500 kg/m^3

About the geometric characteristics of the modelled elements, the analysis and the calculation of dead and live loads and combinations of actions, refer to the thesis "*Analisi strutturale e modellazione della Sala Ovest nel complesso delle Sale d'Armi Nord all'Arsenale di Venezia*"- Gambirasi M.

Evaluation of the static linear analysis results

The resolution of the numerical model is linear static analysis (LSA), which provides the results for each load combination set.

The identified load-bearing walls are subject to different stresses and deformations, resulting from the typology of the structural elements.

In the walls, which the main structural elements are connected to, are analyzed the stress states in the symmetric configuration load because this is the most burdensome. The longitudinal eastern and western walls have a thickness of 1.16 m, unlike the others which all have a thickness smaller than 0.55 m. The interior partitions have an average thickness of 0.26 m.

The possible observations about the development of the model mainly concern the connection points of the walls of the three different buildings. Fig. 4.6.9 shows the global model in which there are all modelled walls.

With regard to the stresses in the global reference system (Fig. 4.6.9), is possible to observe a regular distribution for both front and longitudinal walls. Globally, the values of vertical stresses (compressive) remain moderately low, while the traction stresses are negligible.

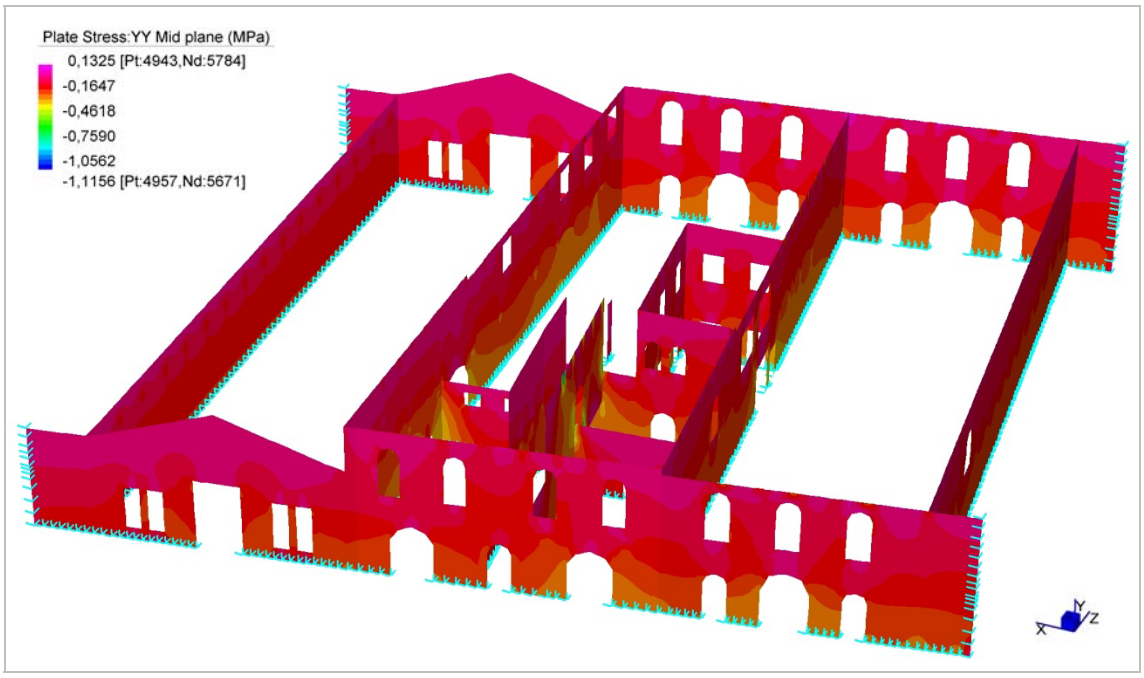


Fig. 4.6.9: Global model, vertical stresses (compressive) (MPa)

Considering the connection of heterogeneous structural elements to the walls, is possible to assess the influence of these last on the stress state of the walls. For the different floors is possible to observe that the maximum compressive stress values are very similar. This allows to state that the differences in type and material of the horizons do not actually influence the development of tensions in the wall, observation which can also apply to the presence of the roof of the adjacent warehouse. At the ground height, maximum compression tensions in fact vary between -0.20 MPa and -0.40 MPa. The wall which has little bigger stresses than the others is the eastern wall.

As an example, Fig. 4.6.10 shows the principal stresses for the east façade.

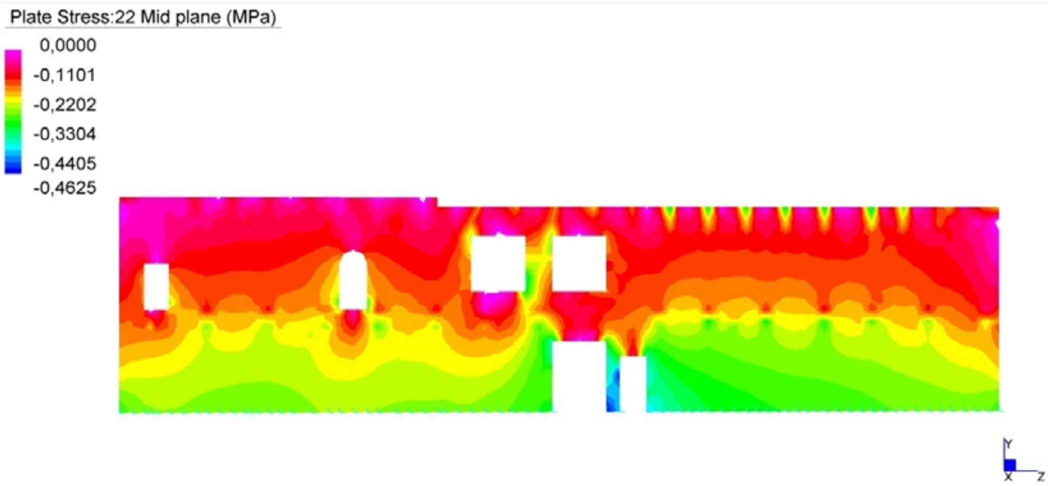


Fig. 4.6.10: East façade, pricipal stresses (MPa)

Conclusive remarks

Analyzing the global finite element model it can be concluded that the tension on the analyzed walls can be considered regular and content. Considering the entire walls system, there is a regular distribution of compressive stresses, with content maximum values.

After the surveys, analysis of the degradation and modelling it can be concluded that the walls do not not present a heavy situation of structural degradation. Therefore, was not considered necessary to focus on vertical structures. Below is reported the analysis of the most critical structural elements, i.e. the roofs of the complex.

4.6.2 Conservative restoration project of the wooden trusses

The wooden beams in the area A called A1, A2, A3 and A4 (Fig. 4.4.73) were not subjected to collapses. However, as previously described, they have a diffuse deterioration that has compromised the structural stability.

The preliminary phase of restoration, as described below, is the verification of the elements of the considered trusses, according to Eurocode 5.

In this paragraph, the project of restoration of the roof, where are indicated the interventions for the conservation of wooden elements, is reported. The criterion that have guided the planning choice for the static restoration of wooden trusses of the area A roofs is based on the principle that all wooden structures must be repaired according to criteria of restoration and strict respect of the formal, structural and aesthetic aspect. It was sought to preserve the original elements and materials and at the same time to not distort the original aesthetic, architectural and functional design of the elements.

It was therefore proceeded with a systematic and methodology approach following the design process involved in reconstruction of the original aesthetic and functional aspects by avoiding superfluous structural interventions made with materials different from wood that lead to significant alterations of original static scheme.

Verifications of the structural elements

The roof of the area A of the *Sale d'Armi Nord* is at present made by four Palladian simple-type trusses, with the post not firmly connected to the tie beam, defined as "open-node system".

For the purposes of the study of the trusses was created a numerical model using finite element software Straus7 ®. Defined the static scheme of the studied elements, the model is restrained taking into account two reference systems.

The truss is an isostatic system, always balanced. The two struts are subjected to compression while the tie beam is solicited to traction. This last is intended to prevent the outward displacement of the two struts.

The various elements that make up the trusses, such as struts, tie beam, straining beam and posts, are represented by so-called *beam* elements (element with six stress components) to which are assigned the characteristic properties of each element, the material type and the section geometry (Fig. 4.6.11).

Then, the loads acting were applied on the structure and were defined the different combinations of loads to calculate the stresses of bending moment, and shear and axial stress. The analysis is linear elastic type.

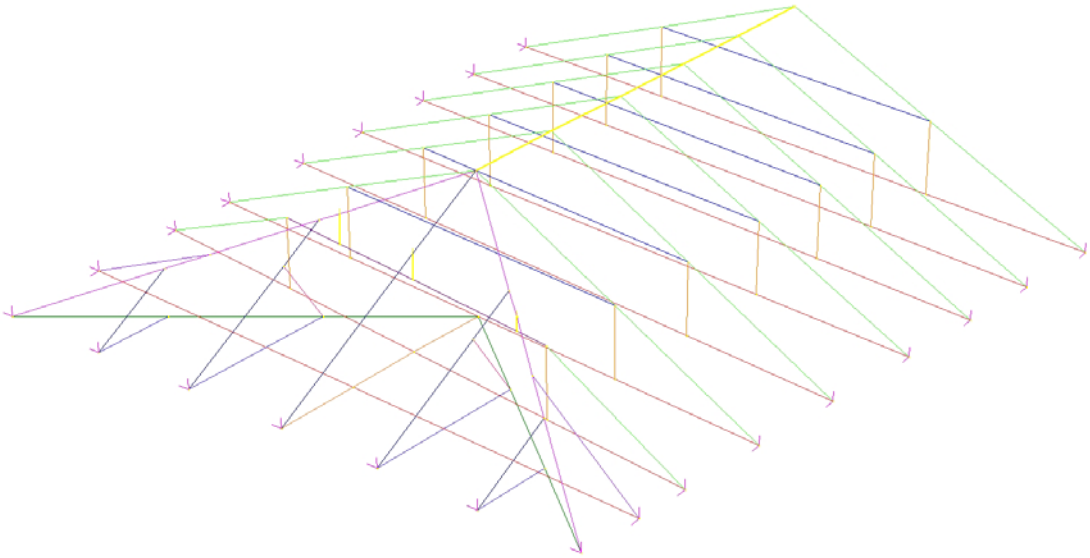


Fig. 4.6.11: View of the numerical model of the roof structure

The study of roof is done in according to the requirements of the "Nuove Norme Tecniche per le Costruzioni 14.01.2008" and of the "Decreto Ministeriale 09.01.1996" for the analysis of loads and overloads and Eurocode 5 for the verifications of the wooden elements.

The trusses have an average span of 16 meters and wheel base between two adjacent beams is 2.25 meters. Also, the angle between the strut and the tie beam is 25 degrees and the rafters' slope instead is 28°.

The material of the main structures is larch wood. In Tab. 4.6.7, the mechanical properties of the considered wood are reported.

Tab. 4.6.7: Considered mechanical properties of larch wood

MECHANICAL PROPERTIES	
CHARACTERISTIC BENDING STRENGTH $f_{m,k}$	32000 kN/m^2
CHARACTERISTIC TENSILE STRENGTH ALONG THE GRAIN, $f_{t,0,k}$	19000 kN/m^2
CHARACTERISTIC TENSILE STRENGTH PERPENDICULAR THE GRAIN, $f_{t,90,k}$	600 kN/m^2
CHARACTERISTIC COMPRESSIVE STRENGTH ALONG THE GRAIN, $f_{c,0,k}$	24000 kN/m^2
CHARACTERISTIC COMPRESSIVE STRENGTH PERPENDICULAR TO GRAIN, $f_{c,90,k}$	4000 kN/m^2
CHARACTERISTIC SHEAR STRENGTH, $f_{v,k}$	3200 kN/m^2
MEAN VALUE OF MODULUS OF ELASTICITY, $E_{0,mean}$	12000000 kN/mm^2

FIFTH PERCENTILE VALUE OF MODULUS OF ELASTICITY, $E_{0,05}$	8000000 kN/mm^2
MEAN VALUE OF PERPENDICULAR MODULUS OF ELASTICITY, $E_{90,mean}$	400000 kN/mm^2
MEAN VALUE OF SHEAR MODULUS, G_{mean}	750000 kN/mm^2
MEAN DENSITY (5 PERCENTILE), r_k	550 kg/m^3
MEAN DENSITY, r_{mean}	600 kg/m^3

Dead loads and live loads

The loads acting on the trusses under consideration are: dead loads (G_2) and live loads of snow (Q_{k1}). The wind loads were not considered, because its contribution in this combination is favourable to the verifications.

Dead loads

In Tab. 4.6.8 are reported the surface loads (kN/m^2) and the corresponding weights acting on beams G_2 (kN/m), by considering a constant wheel base between trusses (*i*).

Tab. 4.6.8: Values of dead loads acting on the wooden trusses of area A

LOAD	***i*** **[m]**	**SURFACE LOAD [kN/m^2]**	**G_2 [kN/m]**
TILE ROOF	2,25	0,800	1,800
LAYERS OF THERMAL INSULATION AND STIFERITE 4 CM	2,25	0,018	0,041
WATERPROOF LAYER, 4 MM	2,25	0,100	0,225
TERRACOTTA HOLLOW TILES, 2 CM	2,25	0,400	0,900
WOODEN SECONDARY BEAMS 7 X 7	2,25	0,120	0,270
PURLINS 15 X 15	2,25	0,130	0,293
WOODEN TRUSS ELEMENTS	2,25	0,600	1,350
			TOT = 4,88

The dead loads insisting on the struts (right pitch) are equal to:

G_2 = 4,88 kN/m.

In the left pitch of the roof the weight of the wooden elements located between trusses and purlins must be taken into account. Their function is to increase the slope to join the adjacent building roof.

In this case the dead load is equal to:

G_2 = 5,10 kN/m.

Live loads: snow

The loads of snow were calculated according to the directions of the Norme Tecniche per le Costruzioni currently in force and of Decreto Ministeriale 09.01.1996 in force during the project phase.

Since Venice is located in zone II with the parameter a_s (height sea level) < 200 m and a pitch of roof (α), measured from horizontal, $0° < \alpha < 30°$ are been obtained the following values of the snow loads (Tab. 4.6.9):

Tab. 4.6.9: snow loads

RULES	**SNOW LOAD q_s (kn/m^2)**
D.M. 2008	0,80
D.M. 1996	1,28

For the verifications is considered the most demanding snow load case, calculated with the D.M. 09.01.1996.

Maintenance live load

In the case of non accessibile roof (or accessibile just for maintenance) the live load to be considered corresponds to:

$q = 0.5\ kN/m^2$

Combination of actions for the Ultimate Limit State (ULS), according to the Eurocode

According to the Eurocode, it is necessary determine the values of the design effect of actions (E_d)

For the fundamental combination:

$$E_D = \sum \gamma_{G,j} G_{k,j} + \gamma_{Q1} Q_{k,1} + \sum_{i\geq 1} \gamma_{Q,j} \psi_{0,i} Q_{k,i}$$

where:

$G_{k,j}$ is the characteristic value of permanent action;

$Q_{k,1}$ is the characteristic value of leading variable action;

$Q_{k,i}$ is the characteristic value of the other variable actions;

$\gamma_{G,j}$ are the partial factors for a permanent actions in accidental design situations, equal to 1,35;

$\gamma_{Q,i}$ are the partial factors for leading variable action, equal to a 1,5;

ψ_1, ψ_2, ψ_3 are the combination factors for frequent value, quasi-permanent value and frequent value of a variable actions.

Considering the most demanding load case (calculated only on a single live load) the expression to use is:

$$\sum \gamma_{G,j} G_{k,j} + 1{,}5 Q_{k,1}$$

Considering the most demanding load case (calculated for all the live loads) the expression to use is:

$$\sum \gamma_{G,j} G_{k,j} + 1{,}5 \sum_{i\geq 1} Q_{k,1}$$

The 4 trusses have the same geometry and are subjected to the same external load combinations. Below are reported the verifications only for one truss, according to the Eurocode 5.

<u>Combined bending and axial compression</u>

For the verification of the combined bending moment and axial compression, the following expressions shall be satisfied (Expressions 6.19 and 6.20 of the Eurocode 5):

$$\begin{cases} \left(\dfrac{\sigma_{c0d}}{f_{c0d}}\right)^2 + \left(\dfrac{\sigma_{myd}}{f_{myd}}\right) + k_m \cdot \left(\dfrac{\sigma_{mzd}}{f_{mzd}}\right) \leq 1 \\ \left(\dfrac{\sigma_{c0d}}{f_{c0d}}\right)^2 + k_m \cdot \left(\dfrac{\sigma_{myd}}{f_{myd}}\right) + \left(\dfrac{\sigma_{mzd}}{f_{mzd}}\right) \leq 1 \end{cases}$$

Where:

$\sigma_{c,0,d}$ is the design compressive stress along the grain;

$f_{c,0,d}$ is the design compressive strength along the grain;

$\sigma_{m,y,d}$ and $\sigma_{m,z,d}$ are the design bending stresses about the principal axes;

$f_{m,y,d}$ and $f_{m,z,d}$ are the corresponding design bending strengths;

k_m parameter equal to 0,7 for rectangular section.

In the cases under consideration, calculating the most demanding load cases, we obtain:

$$[(N/A)/ f_{c,0,d}]^2 + (M/W)/ f_{m,y,d} \leq 1$$

Stability of members

For the Eurocode (Paragraph 6.3), in the case of combined bending and axial compression it is necessary do a check of the stability of members.

The following expressions shall be satisfied:

$[(N/A)/ (k_c \cdot f_{c,0,d})] + (M/W)/ f_{m,y,d} \leq 1$

Where:

k_c is a factor which takes into account lateral instability = $1/[k + \sqrt{(k^2 + \lambda_{rel}^2)}]$

k is the instability factor = $0.5 [1 + \beta_c (\lambda_{rel} - 0,5) + \lambda_{rel}^2]$

β_c is a straightness factor, equal to 0,2 for solid timber;

λ_{rel} is a relative slenderness ratio = $\sqrt{(f_{c,0,d} / \sigma_{c,crit})}$

$\sigma_{c,crit}$ is the critical compressive stress = $(\pi^2 \cdot E_{0,05})/ \lambda^2$ where λ is the slenderness ratio of the beam.

Combined bending and axial tension

For the verification of the combined bending and axial tension, the following expressions shall be satisfied (Expressions 6.17 and 6.18 of the Eurocode 5):

$$\begin{cases} \left(\frac{\sigma_{t0d}}{f_{t0d}}\right) + \left(\frac{\sigma_{myd}}{f_{myd}}\right) + k_m \cdot \left(\frac{\sigma_{mzd}}{f_{mzd}}\right) \leq 1 \\ \left(\frac{\sigma_{t0d}}{f_{t0d}}\right) + k_m \cdot \left(\frac{\sigma_{myd}}{f_{myd}}\right) + \left(\frac{\sigma_{mzd}}{f_{mzd}}\right) \leq 1 \end{cases}$$

Where:

$\sigma_{t,0,d}$ is the design tensile stress along the grain;

$f_{t,0,d}$ is the design tensile strength along the grain;

$\sigma_{m,y,d}$ e $\sigma_{m,z,d}$ are the design bending stresses about the principal axes;

$f_{m,y,d}$ e $f_{m,z,d}$ are the corresponding design bending strengths;

k_m parameter equal to 0,7 for rectangular section;

In the cases under consideration, calculating the most demanding load cases, we obtain:

$$[(N/A)/ f_{t,0,d}] + (M/W)/ f_{m,y,d} \leq 1$$

Shear

For the shear, in accordance with the equation 6.13 of the Eurocode 5, the following expression shall be satisfied:

$$\tau_d \leq f_{v,d}$$

Where:

τ_d is the design shear stress;

$f_{v,d}$ is the design shear strength for the actual condition.

In the cases under consideration we obtain:

$$T/A \leq f_{v,d}$$

In Tab. 4.6.10 the dimensional characteristics of the trusses are reported:

Tab. 4.6.10: Dimensional characteristics of wooden truss

ELEMENT	B · H (m)	W = (B · H^2)/6 (m^3)
TIE BEAM	0,25 · 0,30	0,00375
STRUT	0,25 · 0,30	0,00375
STRAINING BEAM	0,25 · 0,25	0,00261
POST	0,25 · 0,25	0,00261

The following table (Tab. 4.6.11) shows the maximum stresses determined for the most demanding load case.

Tab. 4.6.11: Maximum stresses for the most demanding load case of the wooden truss

ELEMENT	BENDING MOMENT (KN · M)	AXIAL STRESS (KN)	SHEAR STRESS (KN)
TIE BEAM	+ 14,97	+ 157,42	+ 3,57
STRUT	- 60,04	- 114,03	+ 32,92
STRAINING BEAM	- 12,04	- 70,95	+ 0,61
POST	0	+ 0,73	0

Below, the verifications for the elements of the truss under consideration are reported.

Combined bending and axial compression

The elements subjected to the combined bending and axial compression are struts and straining beam. In the following table (Tab. 4.6.12) the verification data are reported.

Tab. 4.6.12: Combined bending and axial compression for the trusses A1, A2 A3 and A4

ELEMENT	N/A (kN/m^2)	$f_{c,0,d}$ (kN/m^2)	M/W (kN/m^2)	$f_{m,y,d}$ (kN/m^2)	$[(N/A)/f_{c,0,d}]^2$+(M/W)/$f_{m,y,d}$	$[(N/A)/f_{c,0,d}]^2$+(M/W)/$f_{m,y,d}\leq 1$
STRUT	1520	14770	16011	19690	0,82	Verified
STRAINING BEAM	1135	14770	4613	19690	0,24	Verified

The verification for struts and straining beam are satisfied.

Stability of members

The elements that require the verification of stability are struts and straining beam. In the following table (Tab. 4.6.13) the verification data are reported.

Tab. 4.6.13: Stability of members for the trusses A1, A2 A3 and A4

ELEMENT	$L_{MIN\ EFFECTIVE}$ (m)	K_C	$[(N/A)/(K_C \cdot f_{c,0,d})]$ +(M/W)/$f_{m,y,d}\leq 1$	$[(N/A)/(K_C \cdot f_{c,0,d})$ +(M/W)/$f_{m,y,d}\leq 1$
STRUT	4,70	0,66	0,96	Verified
STRAINING BEAM	8,00	0,25	0,54	Verified

The verification for the struts and the straining beam is satisfied.

Combined bending and axial tension

The element subjected to the combined bending and axial tension is the tie beam.

The maximum tensile stresses are found at the ends of the element, in proximity of the tie beam-struts joint. For this reason the equation is calculated using the reduced section, because the tie beam is carved in that section. In this case, a carving of 0,10 m is considered. Therefore, the section has the area equal to A=bxh= 0,05 m^2 with h_{net} = 0,30 - 0,10= 0,20 m.

On the contrary, the maximum bending moment is found at mid span. Therefore, in the equation it is considered a resisting section corresponding to the gross area which is equal to A=bxh=0,25 x 0,30 m and W = (bxh2)/6 = 0,00375 m^3

In the following table verification data are reported.

Tab. 4.6.14: Combined bending and axial tension for trusses A1, A2 A3 and A4

ELEMENT	N/A (kN/m^2)	$f_{t,0,d}$ (kN/m^2)	M/W (kN/m^2)	$f_{m,y,d}$ (kN/m^2)	$[(N/A)/f_{t,0,d}]$ $(M/W)/f_{m,y,d}$	$[(N/A)/f_{t,0,d}]+$ $(M/W)/f_{m,y,d} \leq 1$
TIE BEAM	3148	11690	3992	19690	0,47	Verified

Maximum stresses in the tie beam are greater than the design loads

Posts present low values of bending and axial stresses. For this reason their verification is omitted.

<u>Shear</u>

The elements subjected to shear stress are the struts, because for tie beam, straining beam and posts the values are very low.

In the following table (Tab. 4.6.15) the verification data are reported.

Tab. 4.6.15: Shear stress for trusses A1, A2 A3 and A4

ELEMENT	T/A (kN/m^2)	$f_{v,d}$ (kN/m^2)	$T/A \leq F_{V,D}$
STRUT	439	1970	Verified

The verification for the struts is satisfied.

The verifications indicated that resistance of all of the analyzed elements of the trusses is greater than the design loads. Therefore, it is possible to consider just a conservative approach for the wooden trusses which survived the collapse.

<u>Conservative restoration</u>

The verifications carried out for the wooden trusses are satisfied. However, preliminary analysis revealed a situation of diffuse deterioration which is undermining the stability of the structure. For this reason, it was drawn up a conservative restoration project, which aims to maintain the original material, without increasing in any way its characteristics, and the structure, without altering then the current strength of the roof structure. The restoration done on the existing trusses was made reducing to a minimum level the interventions.

Also, have been made a series of inspections to assess the conservation status of wood trusses which have showed a diffuse decay, above all, at the end of the beams in correspondence of the strut-tie beam node (Fig. 4.6.12). Into the span width, in fact, the wood still appeared, in most cases, in good condition, while at the ends of the tie beams and strut, placed inside the perimeter wall of the building, appeared compromised by biological attacks. This degradation of the structures has significantly affected the stability of the entire structure.

Fig. 4.6.12: Wooden trusses before the intervention of conservation

The first stage of the project was the processing of the data obtained in the preliminary stage of knowledge. The survey of the present-day conditions and the study of wooden structures have permitted to obtain significant informations about the size of the structures, the geometry of the elements, with particular attention to the types of connections, also degrade. It was then obtained a full reading of present-day conditions, as previously described.

The intervention project involved the reconstruction of the deteriorated parts of the trusses, by the insertion of special wooden implants, respecting as much as possible the original static scheme and aesthetic aspect of the structures, with attention to the mechanical characteristics of the wood. The intervention is designed to not undermine the resistance characteristics of the wooden parts giving an adequate assurance on the load-bearing capacity and durability of the restoration.

Therefore, was done the repair and replacement of the deteriorated ends of the struts and tie-beams with special wooden elements.

The wood-wood connection thus obtained was then been joined to the sound part of wooden structure, after the careful elimination of all damaged parts.

Restoration of the trusses

For the restoration of the truss has proceeded with the execution of building works involving the flaring and opening of the support seats of the trusses on the wall to allow the restoration and create adequate manoeuvre space for the removal of the wooden structures (Fig. 4.6.13).


a)

b)

Fig. 4.6.13: a) Inserting the edge of the beam inside the masonry wall; b) detail of the material substitution at the beam's end

It was removed the metal bracket to release the node and allow the slight elevation of the strut than the tie-beam. Then, was performed resection of the terminal parts (Fig. 4.6.14) of the wooden truss needed to insert the new joints.


Fig. 4.6.14: Resection of the deteriorated sections

The reconstruction of the truss was made with the gluing of wooden implants fixed on the stumps that had been previously profiled.

Also, were inserted the oak wedges and replaced the metal brakets previously removed to allow the execution of the restoration (Fig. 4.6.15 e Fig. 4.6.16).

Fig. 4.6.15: Reconstruction of the tie beam-strut joint

Fig. 4.6.16: Structural interventions, coupling of new and original materials

This operation was repeated for each beam and each support that had to be restored to ensure the restoration of the original size and function of the elements. For the creation fo the new joints is very important to select the same wood species of existing trusses, as much as possible free from defects of the material. In addition, all the wood used for the various restored elements must be balanced to the same value of moisture, which must fall within the required compatibility limits.

After completion of conservative restoration (Fig. 4.6.17) the trusses are able again to act as a structural elements.

Fig. 4.6.17: Restored joints of the trusses of the area A

Completed the restoration of the trusses has been restored the support seat of the tie beam. The recovery of the walls was done with replacement of portions of deteriorated brick masonry using the "scuci-cuci" (masonry repair by substituting damaged units with new ones) technique.

The careful reconstruction near the support of the wooden elements has been carried out to ensure an adequate ventilation favouring an appropriate hygrometric balance.

In the support of the joint wall-tie beam was left an air space of about 5 cm, to allow a proper ventilation of the joint and facilitate the inspection of the wooden elements (Fig. 4.6.18).

Fig. 4.6.18: Restoration intervention in the end of the tie beam

The intervention on the trusses was concluded with the application of a rust-converter for the treatment of recovered and new ironmongery, the cleaning and the surface treatment of wooden trusses and of the purlins, and a curative treatment of the wood, both in deep and in surface.

<u>Restoration of the roof structure</u>

Completed the conservative-restoration works of the wooden elements, it was proceeded with the assembly of secondary beams (purlins) and surface roof.

Each element of the roof was inspected to identify the presence of any possible breakage and/or cracks in order to determine the possible reusability. The disassembly has concerned the pantiles, the layer of flat tiles, the secondary beams and the purlins. For these last, have been selected the elements yet efficient and reusable; was proceeded then at the removal from the surface of any kind of sediment, such as mosses and lichens.

Subsequently, the secondary frame was fixed by replacing with new elements the not usable purlins; have been installed the brick tiles of 2 cm thick, the bituminous sheath, the thermal insulation and a 4 cm-thick of stifferite layer. Finally, was proceeded the relocation of the tiles' surface to protect the roof.

4.6.3 Verifications and reconstruction project of the collapsed wooden roof

The reconstruction project of the wooden roof collapsed in the A area, in agreement with the *Soprintendenza B.A.P di Venezia e Laguna*, it was drew up proposing the original sixteenth-century configuration. It was then planned the reconstruction of two trusses and a hip rigid-link truss with pre-cambered of the tie beam, two diagonally-placed rafters that form the pitch watershed and other wooden elements that complement the main frame (Fig. 4.4.78).

The "rigid link" is implemented by a stirrup nailed to the post that wraps the tie beam. In this way the movement between the post and the tie beam is prevented. Since this latter is subjected to bigger stresses, was planed the construction of 5 cm-thick pre-cambered. This is done by lifting the tie beam and creating a connection between the post and the tie beam that for the latter assumes a curved configuration.

As described in the previous paragraph, was built a finite element model of the roofing system to evaluate the parameters of stress as a function of different load combinations, as indicated in Eurocode 5.

About the rigid link trusses, has been considered the internal link between the strut and the post on the top and the "strut – tie beam" node as a hinge while the "strut - straining beam – post" node for the present connection is schematized by a joint. The connection of the rigid link was represented through the creation of a internal hinge between the post and the tie beam.

As in the previous case, have been carried out the appropriate verifications, according to the instructions of Eurocode 5, considering the sections of the trusses and the hip truss equal to those of the portion of coverage not subjected to collapse. The sizes of the rafters were instead determined by comparing the geometric parameters of the structure, with the elements recovered after the collapse and the analysis of the adjacent buildings roofs.

Verification of the structural element

Trusses A5 and A6

Below the verifications of the trusses A5 and A6 are reported.

In Tab. 4.6.16 the dimensional characteristics of the trusses A5 and A6 (Fig. 4.6.19) are reported:

Tab. 4.6.16: Dimensional characteristics of trusses A5 and A6

ELEMENT	B · H (m)	$W = (B \cdot H^2)/6$ (m^3)
TIE BEAM	0,25 · 0,30	0,00375
STRUT	0,25 · 0,30	0,00375
STRAINING BEAM	0,25 · 0,25	0,00261
POST	0,25 · 0,25	0,00261

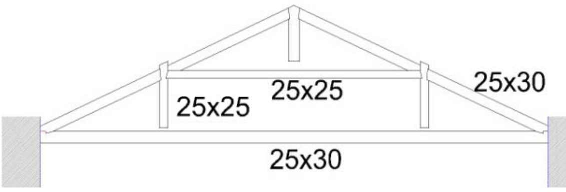


Fig. 4.6.19: Dimensions of the elements of the trusses A5 and A6

While the truss named A5 is subjected just to its self weight, and to the live and dead loads of the roof, the truss A6 must also act as a support for the rafters constituting the pavilion pitch of the roof. For this reason its tie beam is subjected to higher forces.

The following table (Tab. 4.6.17) shows the maximum stresses determined for the most demanding load case.

Tab. 4.6.17: Maximum stresses for the most demanding load case of wooden trusses A5 and A6

TRUSS	ELEMENT	BENDING MOMENT (KN · M)	AXIAL STRESS (KN)	SHEAR STRESS (KN)
TRUSS A5	TIE BEAM	+ 22,77	+ 112,13	- 7,55
	STRUT	- 26,72	- 167,32	- 29,38
	STRAINING BEAM	- 17,75	- 77,69	- 2,93
	POST	+ 24,33	+15,48	+ 13,13
TRUSS A6	TIE BEAM	+ 36,36	+ 131,20	- 15,74
	STRUT	- 26,28	- 159,02	- 28,60
	STRAINING BEAM	- 16,68	- 16,68	- 5,19
	POST	+ 22,39	+ 27,98	+ 11,75

Combined bending and axial compression

The elements subjected to the combined bending and axial compression are struts and straining beam. In the following table (Tab. 4.6.18) the verification data are reported.

Tab. 4.6.18: Combined bending and axial compression for trusses A5 and A6

TRUSS	ELEMENT	N/A (kN/m^2)	$f_{c,0,d}$ (kN/m^2)	M/W (kN/m^2)	$f_{m,y,d}$ (kN/m^2)	$[(N/A)/f_{c,0,d}]^2$+ $(M/W)/f_{m,y,d}$	$[(N/A)/f_{c,0,d}]^2$+ $(M/W)/f_{m,y,d} \leq 1$
A5	STRUT	2231	14770	7125	19690	0,38	Verified
	STRAINING BEAM	1243	14770	6801	19690	0,68	Verified
A6	STRUT	2120	14770	7008	19690	0,38	Verified
	STRAINING BEAM	1556	14770	6391	19690	0,34	Verified

The verification for struts and straining beam are satisfied.

Stability of members

The elements that require the verification of stability are struts and straining beam. In the following table (Tab. 4.6.19) the verification data are reported.

Tab. 4.6.19: Stability of members for trusses A5 and A6

TRUSS	ELEMENT	L_{MIN} EFFECTIVE (m)	K_C	$[(N/A)/(K_C \cdot f_{c,0,d})]$ $+(M/W)/f_{m,y,d} \leq 1$	$[(N/A)/(K_C \cdot f_{c,0,d})$ $+(M/W)/f_{m,y,d} \leq 1$
A5	STRUT	3,60	0,88	0,50	Verified
	STRAINING BEAM	8,00	0,25	0,68	Verified
A6	STRUT	3,80	0,85	0,52	Verified
	STRAINING BEAM	8,00	0,25	0,74	Verified

The verification for the struts and the straining beam is satisfied.

Combined bending and axial tension

The elements subjected to the combined bending and axial tension are tie beam and posts.

To calculate the area and section modulus of tie beam, refer to the considerations covered in the verification of the trusses not collapsed.

In the following table (Tab. 4.6.20) the verification data are reported.

Tab. 4.6.20: Combined bending and axial tension for trusses A5 and A6

TRUSS	ELEMENT	N/A (kN/m^2)	$f_{t,0,d}$ (kN/m^2)	M/W (kN/m^2)	$f_{m,y,d}$ (kN/m^2)	$[(N/A)/f_{t,0,d}]$ $(M/W)/f_{m,y,d}$	$[(N/A)/f_{t,0,d}]+$ $(M/W)/f_{m,y,d} \leq 1$
A5	TIE BEAM	2243	11690	6072	19690	0,50	Verified
	POST	248	11690	9322	19690	0,49	Verified
A6	TIE BEAM	2624	11690	9683	19690	0,72	Verified
	POST	448	11690	8579	19690	0,47	Verified

Maximum stresses in the tie beam and in the posts are greater than the design loads

Shear

The elements subjected to shear stress are the struts, tie beam and posts.

In the following table (Tab. 4.6.21) the verification data are reported.

Tab. 4.6.21: Shear stress for trusses A5 and A6

TRUSS	ELEMENT	T/A (kN/m^2)	$f_{v,d}$ (kN/m^2)	$T/A \leq F_{V,D}$
A5	STRUT	392	1970	Verified
	TIE BEAM	220	1970	Verified
	POST	210	1970	Verified
A6	STRUT	381	1970	Verified
	TIE BEAM	315	1970	Verified
	POST	188	1970	Verified

The verification for struts, tie beam and post are satisfied.

Hip truss A7

The dimensional characteristics of the hip truss called A7 are reported in Tab. 4.6.22:

Tab. 4.6.22: Dimensional characteristics of the wooden hip truss A7

ELEMENT	B · H (m)	$W = (B \cdot H^2)/6$ (m^3)
TIE BEAM	0,25 · 0,30	0,00375
STRUT	0,25 · 0,30	0,00375
STRAINING BEAM	0,25 · 0,30	0,00375
POST	0,25 · 0,25	0,00261

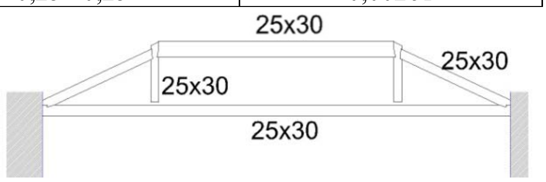


Fig. 4.6.20: Dimensions of the elements of the hip truss A7

The following table (Tab. 4.6.23) shows the maximum stresses determined for the most demanding load case.

Tab. 4.6.23: Maximum stresses for the most demanding load case of wooden hip truss A7

Element	Bending moment (kN · m)	Axial force (kN)	Shear force (kN)
Tie beam	+ 30,28	+ 152,00	- 35,25
Strut	- 27,13	- 205,48	- 29,94
Straining beam	- 34,00	- 153,43	- 26,73
Post	+ 20,10	+43,41	-25,28

Below the verifications of the hip truss A7 are reported.

Combined bending and axial compression

The elements subjected to the combined bending and axial compression are struts and straining beam. In the following table (Tab. 4.6.24) the verification data are reported.

Tab. 4.6.24: Combined bending and axial compression for hip truss A7

Element	N/A (kN/m^2)	$f_{c,0,d}$ (kN/m^2)	M/W (kN/m^2)	$f_{m,y,d}$ (kN/m^2)	$[(N/A)/f_{c,0,d}]^2+(M/W)/f_{m,y,d}$	$[(N/A)/f_{c,0,d}]^2+(M/W)/f_{m,y,d}\leq 1$
Strut	2740	14770	7332	19690	0,41	Verified
Straining beam	2046	14770	9067	19690	0,48	Verified

The verification for struts and straining beam are satisfied.

Stability of members

The elements that require the verification of stability are struts and straining beam. In the following table (Tab. 4.6.25) the verification data are reported.

Tab. 4.6.25: Stability of members for hip truss A7

Element	L_{MIN} EFFECTIVE (m)	K_C	$[(N/A)/(K_C \cdot f_{c,0,d})]+(M/W)/f_{m,y,d}\leq 1$	$[(N/A)/(K_C \cdot f_{c,0,d})+(M/W)/f_{m,y,d}\leq 1$
Strut	3,60	0,88	0,58	Verified
Straining beam	5,56	0,55	0,74	Verified

The verification for the struts and the straining beam is satisfied.

Combined bending and axial tension

The elements subjected to the combined bending and axial tension are tie beam and posts.
In the following table (Tab. 4.6.26) the verification data are reported.

Tab. 4.6.26: Combined bending and axial tension for hip trusses A7

Element	N/A (kN/m^2)	$f_{t,0,d}$ (kN/m^2)	M/W (kN/m^2)	$f_{m,y,d}$ (kN/m^2)	$[(N/A)/f_{t,0,d}]+(M/W)/f_{m,y,d}$	$[(N/A)/f_{t,0,d}]+(M/W)/f_{m,y,d}\leq 1$
Tie beam	3040	11690	8075	19690	0,67	Verified
Post	695	11690	7701	19690	0,45	Verified

Maximum stresses in the tie beam and in the posts are greater than the design loads

Shear

The elements subjected to shear stress are the struts, tie beam and posts.
In the following table (Tab. 4.6.27) the verification data are reported.

Tab. 4.6.27: Shear stress for hip trussA7

Element	T/A (kN/m^2)	$f_{v,d}$ (kN/m^2)	T/A ≤ $F_{V,D}$
Strut	399	1970	Verified
Tie beam	705	1970	Verified
Post	405	1970	Verified

The verification for struts, tie beam and post are satisfied.

Rafters

The dimensional characteristics of the elements of the rafters are reported in Tab. 4.6.28. For the denomination of elements refer to Fig. 4.6.21.

Tab. 4.6.28: Dimensional characteristics of wooden rafters

Rafter	Element	B · H (m)	W = (B · H^2)/6 (m^3)
A 9	Horizontal	0,20 · 0,20	0,00133
	Inclined	0,20 · 0,20	0,00133
A 10	Horizontal	0,20 · 0,20	0,00133
	Inclined	0,20 · 0,20	0,00133
A 11	Horizontal	0,25 · 0,30	0,00375
	Inclined	0,25 · 0,30	0,00375
A 12	Horizontal	0,20 · 0,20	0,00261
	Inclined	0,20 · 0,20	0,00133
A 13	Horizontal	0,20 · 0,20	0,00133
	Inclined	0,20 · 0,20	0,00133
A 14	Horizontal	0,25 · 0,30	0,00375
	Inclined	0,25 · 0,30	0,00375
A 15	Horizontal	0,25 · 0,30	0,00375
	Inclined	0,25 · 0,30	0,00375

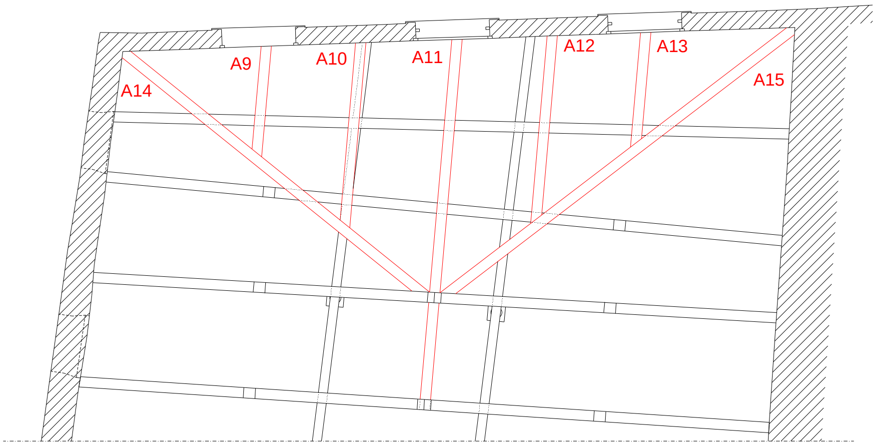


Fig. 4.6.21: Disposition of rafters

The following table (Tab. 4.6.29) shows the maximum stresses determined for the most demanding load case.

Tab. 4.6.29: Maximum stresses for the most demanding load case of wooden rafters

Rafter	Element	Bending moment (kN · m)	Axial stress (kN)
A 9	Horizontal	/	+41,15
	Inclined	+6,67	-52,25
A 10	Horizontal	+1,00	+17,25
	Inclined	-13,41	-27,85
A 11	Horizontal	+17,56	-21,06

	Inclined	+28,56	+35,95
A 12	Horizontal	+0,70	+12,66
	Inclined	+11,07	-23,33
A 13	Horizontal	/	+15,85
	Inclined	+6,50	-26,87
A 14	Horizontal	+9,29	-39,66
	Inclined	+16,61	+35,64
A 15	Horizontal	-13,84	+16,66
	Inclined	-14,55	-28,59

Below the verifications of rafters are reported.

Combined bending and axial compression

The elements subjected to the combined bending and axial compression are A9, A10, A12, A13 inclined rafters and A11, A14, A15 horizontal rafters. In the following table (Tab. 4.6.30) the verification data are reported.

Tab. 4.6.30: Combined bending and axial compression for rafters

ELEMENT	**N/A (kN/m^2)**	**$f_{c,0,d}$ (kN/m^2)**	**M/W (kN/m^2)**	**$f_{m,y,d}$ (kN/m^2)**	**$[(N/A)/f_{c,0,d}]^2+(M/W)/f_{m,y,d}$**	**$[(N/A)/f_{c,0,d}]^2+(M/W)/f_{m,y,d}\leq 1$**
A9 INCLINED	1306	14770	5015	19690	0,26	Verified
A10 INCLINED	696	14770	10083	19690	0,51	Verified
A11 HORIZONTAL	281	14770	4683	19690	0,24	Verified
A12 INCLINED	583	14770	8323	19690	0,42	Verified
A13 INCLINED	672	14770	4887	19690	0,25	Verified
A14 HORIZONTAL	529	14770	2477	19690	0,20	Verified
A15 HORIZONTAL	222	14770	3961	19690	0,20	Verified

The verification for all the elements are satisfied.

Stability of members

The elements that require the verification of stability are A9, A10, A12, A13 inclined rafters and A11, A14, A15 horizontal rafters. In the following table (Tab. 4.6.31) the verification data are reported.

Tab. 4.6.31: Stability of members for rafters

ELEMENT	**$L_{MIN\ EFFECTIVE}$ (m)**	**K_C**	**$[(N/A)/(K_C\cdot f_{c,0,d})]+(M/W)/f_{m,y,d}\leq 1$**	**$[(N/A)/(K_C\cdot f_{c,0,d})+(M/W)/f_{m,y,d}\leq 1$**
A9 INCLINED	2,00	1,50	0,31	Verified
A10 INCLINED	4,30	0,70	0,58	Verified
A11 HORIZONTAL	5,50	0,52	0,27	Verified
A12 INCLINED	4,30	0,70	0,48	Verified
A13 INCLINED	2,00	1,50	0,28	Verified
A14 HORIZONTAL	5,00	0,57	0,19	Verified
A15 HORIZONTAL	5,00	0,57	0,13	Verified

The verification for all the elements are satisfied.

Combined bending and axial tension

The elements subjected to the combined bending and axial tension are A11, A14, A15 inclined rafters and A10, A12 horizontal rafters

In the following table (Tab. 4.6.32) the verification data are reported.

Tab. 4.6.32: Combined bending and axial tension for rafters

ELEMENT	N/A (kN/m^2)	$f_{t,0,d}$ (kN/m^2)	M/W (kN/m^2)	$f_{m,y,d}$ (kN/m^2)	$[(N/A)/f_{t,0,d}]+(M/W)/f_{m,y,d}$	$[(N/A)/f_{t,0,d}]+(M/W)/f_{m,y,d}\leq 1$
A10 HORIZONTAL	431	11690	752	19690	0,08	Verified
A11 INCLINED	479	11690	7616	19690	0,43	Verified
A12 HORIZONTAL	317	11690	526	19690	0,05	Verified
A14 INCLINED	475	11690	4429	19690	0,27	Verified
A15 INCLINED	381	11690	3880	19690	0,23	Verified

Maximum stresses for all the elements are greater than the design loads

Bending

The elements A9 and A13 are subjected only to single axis bending.

For the verification of the single axis bending, the following expressions shall be satisfied (Expressions 6.11 and 6.12 of the Eurocode 5):

$$\frac{\sigma_{m,y,d}}{f_{m,y,d}} + k_m \frac{\sigma_{m,z,d}}{f_{m,z,d}} \leq 1$$

$$k_m \frac{\sigma_{m,y,d}}{f_{m,y,d}} + \frac{\sigma_{m,z,d}}{f_{m,z,d}} \leq 1$$

Where:

$\sigma_{m,y,d}$ e $\sigma_{m,z,d}$ are the design bending stresses about the principal axes;

$f_{m,y,d}$ e $f_{m,z,d}$ are the corresponding design bending strengths;

k_m parameter equal to 0,7 for rectangular section.

In the case under consideration, calculating the most demanding load cases, we obtain:

$$[(N/A)/ f_{t,0,d}] \leq 1$$

In the following table (Tab. 4.6.33) the verification data are reported.

Tab. 4.6.33: Bending fo the rafters

ELEMENT	N/A (kN/m^2)	$f_{t,0,d}$ (kN/m^2)	$[(N/A)/f_{t,0,d}]$	$[(N/A)/f_{t,0,d}]\leq 1$
A9 HORIZONTAL	1029	11690	0,09	Verified
A13 HORIZONTAL	396	11690	0,04	Verified

Maximum bending stress for the horizontal elements A9 and A13 is greater than the design loads.

Conclusive remarks

The verifications indicated that resistance of all of the analyzed elements of the trusses, hip truss and rafters is greater than the design loads. Therefore, it is possible to make a plan of construction of the new roof, in function of the original techniques of the sixteenth.

Reconstruction of the trusses

After the verifications were prepared the construction details containing the instructions necessary for the reconstruction of the trusses.

The trusses A5 and A6 (Fig. 4.6.22) are "Palladiana" trusses, are then formed by tie beam, two struts, straining beam and triple post. The sections of the various elements which constitute the new trusses and the various types of connections are reported.

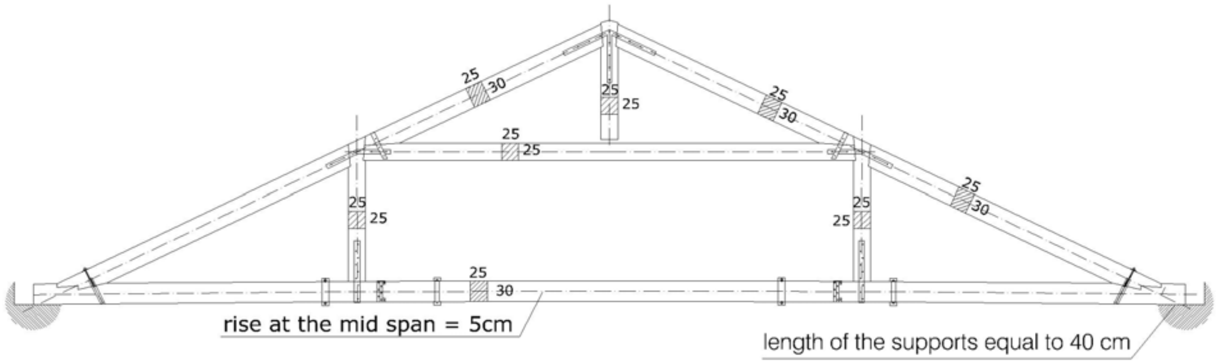


Fig. 4.6.22: Dimensioning of the wooden trusses A5 and A6

The truss A7 is different from the previous. It is in fact an hip truss (Fig. 4.6.23) reinforced at the top by the double straining beam. This latter is stressed by the wooden elements that make up the pavilion roof.

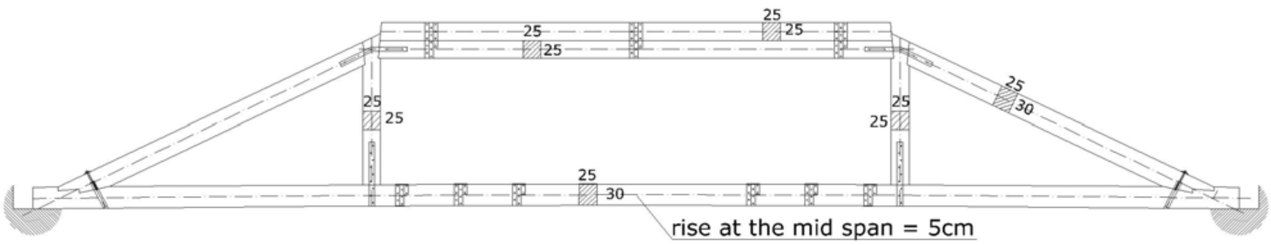


Fig. 4.6.23: Dimensioning of the truss A7

The maximum span in the timber trade is 8 meters, while the span of the truss is approximately 17,50 meters. Since the "dardo di giove" connection provides an overlap of sections of 2,40 meters was necessary to design the tie beam with two "dardo di giove". Executive drawing of the connection with the "dardo di giove" is reported below (Fig. 4.6.24).

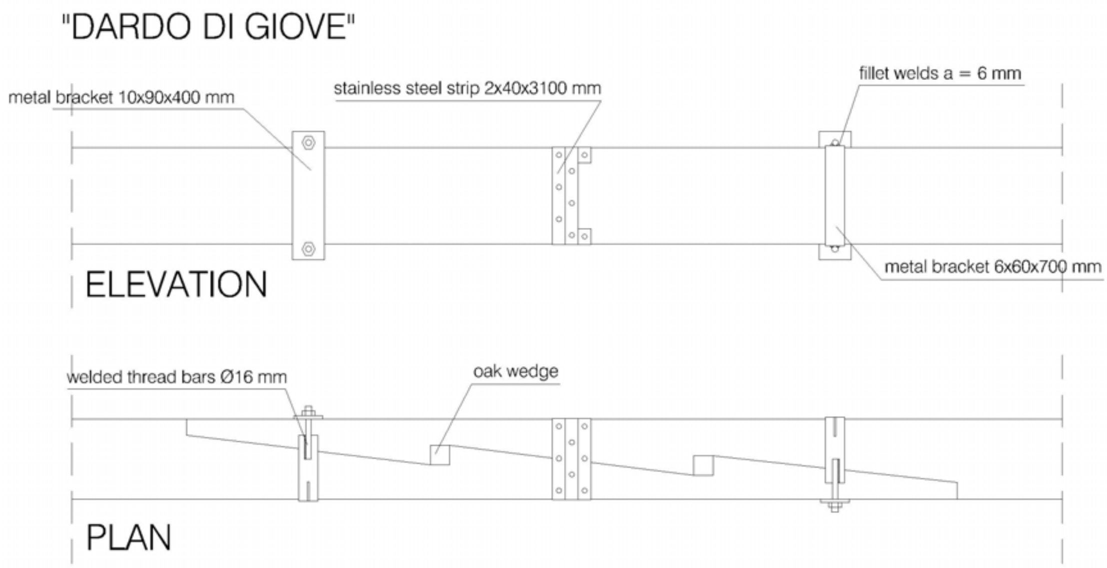


Fig. 4.6.24: Detail of the "dardo di giove"

It was also considered that the wood used was seasoned for several months before being cut in order to increase the performance of the material that is subjected, at the Arsenale di Venezia, to a strong degradation due to moisture present in the complex. The connections between strut and tie beam take up the venetian characteristic construction, which are present in other buildings of Arsenale, and so also for the "post – tie beam" elements that will be replicated with rigid link, through stirrups, with a maximum arrow of tie beam equal to 5 cm.

The construction details of the "tie beam – strut" (Fig. 4.6.25), "strut – post" (Fig. 4.6.26), "strut – post – straining beam" (Fig. 4.6.27) and "post – tie beam" (Fig. 4.6.28) connections are reported below.

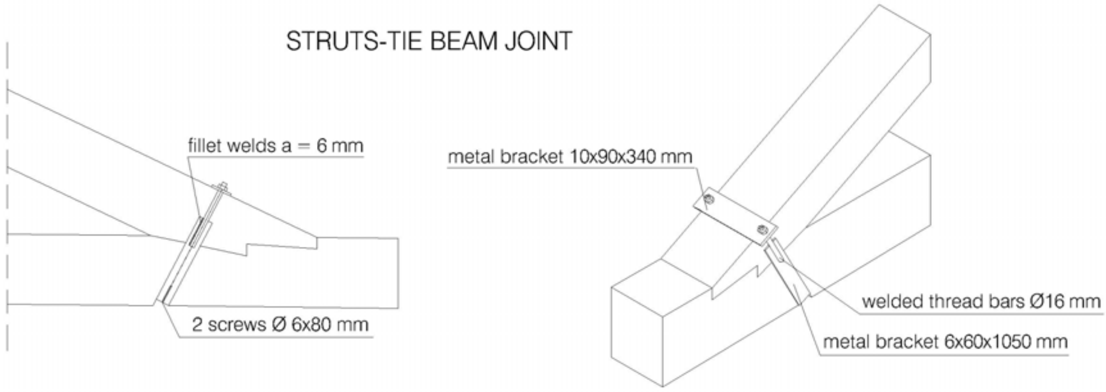


Fig. 4.6.25: Detail of the joint strut-tie beam

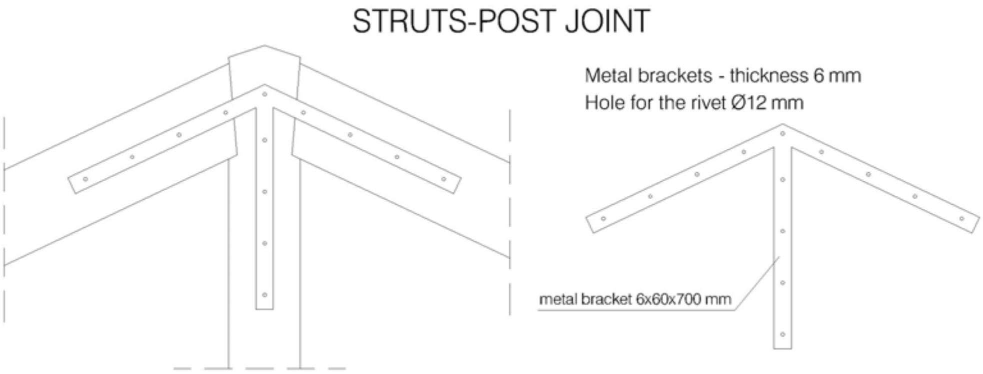


Fig. 4.6.26: Detail of the joint strut-post

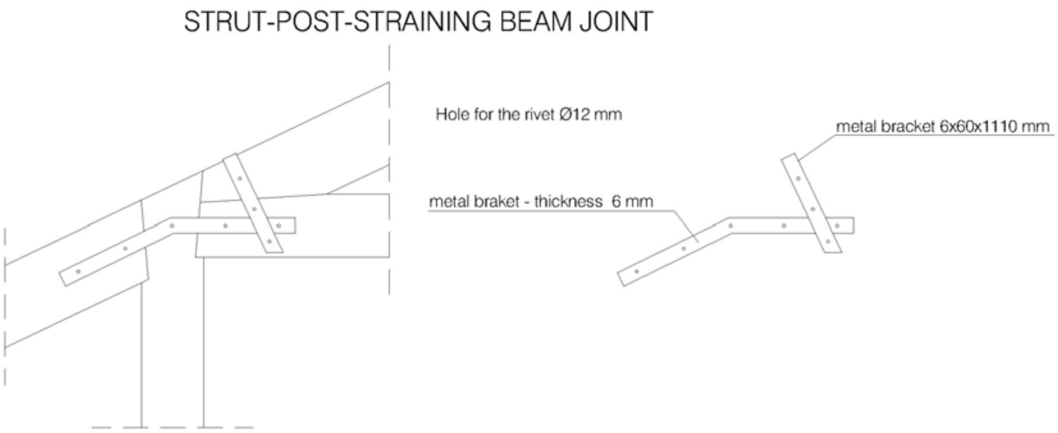


Fig. 4.6.27: Detail of the joint strut-post-straining beam

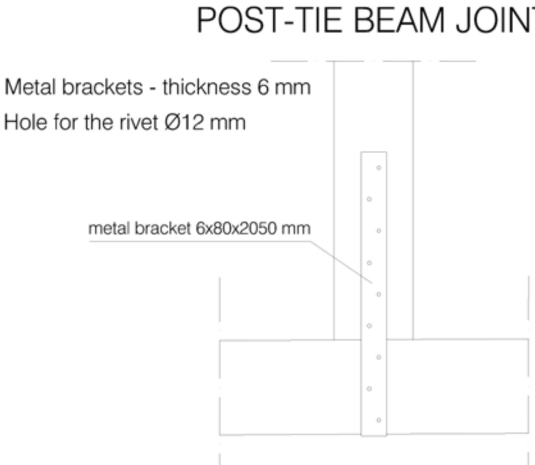


Fig. 4.6.28: Detail of the joint post-tie beam

The reconstruction interventions of the collapsed roof and restoration of the remained portion have been recently completed.

4.6.4 Restoration project of the reinforced concrete trusses

The restoration design of the reinforced concrete trusses is shown in the following paragraph.

The safety evaluation of historical structures, carried out according to the current standards' requirements, indicates, in many cases, unsatisfactory conditions

Imposed limitations on stresses and deformation, prescribed from the standards, are in fact unrealistic to be respected by cultural heritage buildings, since they are thought for new constructions. The Italian approach tries then to balance the two fundamental issues of safety and restoration principles, with the adoption of the "Guidelines for evaluation and mitigation of seismic risk of cultural heritage", technical regulations indicating the concept of structural and/or seismic improvement as a viable intervention in case of historical buildings, rather than to prescribe a "stronger" approach, like e.g. seismic retrofitting, which may alter the historical and cultural value of the structure. With such a perspective, minimal interventions, however improving the overall or local response of the structure, may result acceptable also from a structural point of view.

For the reinforcement concrete roof was designed an intervention of structural reinforce of trusses of the area B (Fig. 4.4.73) that currently present a diffuse decay. Also, it is expected to replace the original roof structure with one with different characteristics and an higher weight.

To verified the trusses was developed a preliminary studies, above mentioned, which has outlined a diffuse decay situation in all the elements of the 6 trusses.

In according to the currently Italian regulation, "Nuove Norme Tecniche per le Costruzioni, D.M. 14/01/2008" (NTC), were carried out the verification for the elements of trusses and was proposed a structural reinforced intervention.

<u>Materials characteristics</u>

Further to the determine of the concrete and steel characteristics during the preliminary analysis of the structure, are been considered the values of strength and stiffness of the trusses as if we had reached the level of knowledge LC1, limited knowledge (levels defined in the Circolare 07-03-2008 which refer to the "Norme Tecniche per le Costruzioni" according to DM 14-01-2008).

The data obtained by testing with the Schmidt hammer should not be considered completely reliable for the elements of the trusses. In fact, according to the specifications contained in the UNI EN 12504-2, the minimum cross section of a tested element should not be lower than 300 mm x 300 mm, and this was not the case of the trusses subjected to investigation. In any case, from the tests carried out it emerged a concrete characteristic resistance between 12 N/mm^2 and 16 N/mm^2. These observations, together with the indications of a possible good quality of concrete emerged from the chemical analysis, permitted to consider a concrete class equal to the maximum of the emerged range of values from the Schmidt hammer test (class C16/20).

Tab. 4.6.34 shows the strength and deformation characteristics for concrete class C16/20, according to NTC2008.

Tab. 4.6.34: strength and deformation characteristics for concrete C16/20, according to NTC2008

DENOMINATION	SYMBOL	ANALYTIC RELATION	VALUE
CHARACTERISTIC COMPRESSIVE CYLINDER STRENGTH OF CONCRETE AT 28 DAYS	f_{ck}	/	16 N/mm^2
CHARACTERISTIC COMPRESSIVE CUBIC STRENGTH OF CONCRETE AT 28 DAYS	$f_{ck,cube}$	/	20 N/mm^2
MEAN VALUE OF CONCRETE CYLINDER COMPRESSIVE STRENGTH	f_{cm}	$f_{cm} = f_{ck} + 8$ (MPa)	24 N/mm^2
MEAN VALUE OF AXIAL TENSILE STRENGTH OF CONCRETE	f_{ctm}	$f_{ctm} = 0,30 \times f_{ck}^{(2/3)} \leq C50/60$ $f_{ctm} = 2,12 \times \ln [1 + (f_{cm}/10)] > C50/60$	1,9 N/mm^2
CHARACTERISTIC OF AXIAL TENSILE STRENGTH OF CONCRETE (FRACTILE 5%)	$f_{ctk,0,05}$	$f_{ctk};0,05 = 0,7 \times f_{ctm}$ fractile 5%	1,3 N/mm^2
CHARACTERISTIC OF AXIAL TENSILE STRENGTH OF CONCRETE (FRACTILE 95%)	$f_{ctk,0,95}$	$f_{ctk};0,95 = 1,3 \times f_{ctm}$ fractile 95%	2,5 N/mm^2
CHARACTERISTIC OF FLEXURAL TENSILE STRENGTH OF CONCRETE	f_{cfm}	$f_{cfm}=1,2\, f_{ctm}$	2,28 N/mm^2
COMPRESSIVE STRAIN IN THE CONCRETE AT THE PEAK STRESS f_c	ε_{c1}	ε_{c1} (‰) $= 0,7\, f_{cm}^{0,31} < 2,8$	1,9 ‰
ULTIMATE COMPRESSIVE STRAIN IN THE CONCRETE	ε_{cu1}	/	3,5 ‰

For the assessments, for reinforcement bars and stirrups a steel FeB22k was considered. Tab. 4.6.35 shows the characteristics of the steel, according to NTC2008.

Tab. 4.6.35: Mechanical characteristics of the steel FeB22K, according to NTC2008

DENOMINATION	SYMBOL	ANALYTIC RELATION	VALUE
CHARACTERISTIC YIELD STRENGTH OF REINFORCEMENT	f_{yk}	/	215 N/mm^2
MAXIMUM CHARACTERISTIC YIELD STRENGTH OF REINFORCEMENT	$f_{yk,max}$	$f_{yk,max}=1,3\, f_{yk}$	280 N/mm^2
CHARACTERISTIC TENSILE STRENGTH OF REINFORCEMENT	f_{tk}	/	335 N/mm^2

Design compressive and tensile strengths (according to NTC 14.01.2008)

Design compressive strength

The value of the design compressive strength is defined as:

$f_{cd} = (\alpha_{cc} \cdot f_{ck}) / (\gamma_C \cdot FC)$

Where:

α_{cc} is the coefficient taking account of long term effects on the compressive strength and of unfavourable effects resulting from the way the load is applied, equal to 0,85;

γ_C is the partial safety factor for concrete, equal to 1,5;

f_{ck} is the characteristic compressive cylinder strength of concrete at 28 days;

FC is the Confidence Factor equal to 1,35.

$f_{cd} = (\alpha_{cc} \cdot f_{ck})/ (\gamma_C \cdot FC) = (0{,}85 \cdot 16) / (1{,}5 \cdot 1{,}35) = 6{,}72\ N/mm^2$

Design tensile strength

The value of the design tensile strength, f_{ctd}, is defined as:

$f_{ctd} = f_{ctk} / (\gamma_C \cdot FC) = 1{,}3 / (1{,}5 \cdot 1{,}35) = 0{,}64\ N/mm^2$

Where:

f_{ctk} is the characteristic of axial tensile strength of concrete;

γ_C is the partial safety factor for concrete, equal to 1,5;

FC is the Confidence Factor equal to 1,35.

Design strength of the steel

The value of the design strength of the steel, f_{yd}, is defined as:

$f_{yd} = f_{yk} / (\gamma_S \cdot FC)$

Where:

f_{yk} is the characteristic yield strength of reinforcement of steel;

γ_S is the partial safety factor for steel, equal to 1,15;

FC is the Confidence Factor equal to 1,35.

$f_{yd} = f_{yk} / (\gamma_S \cdot FC) = 215 / (1{,}15 \cdot 1{,}35) = 138\ N/mm^2$

Dead loads and live loads

The loads acting on the trusses under consideration are: dead loads (G_2) and live loads of snow (Q_{k1}). The wind loads were not considered, because its contribution in this combination is favourable to the verifications.

Dead loads

The previous roof structure was composed by "Perret" tiles and Marseilles tiles. To adapted the structure at a different use of the building, the new cover system will be more heavy.

In Tab. 4.6.36 the surface loads (kN/m^2) and the corresponding weights acting on beams G_2 (kN/m) are reported, by considering a constant wheel base between trusses (i).

Tab. 4.6.36: Values of dead loads acting on the reinforced concrete trusses

LOAD	*i* [m]	SURFACE LOAD [kN/ m²]	G_2 [kN/m]
HOLLOW TILES	2,10	24,0	0,504
WELDED MESH (DIAMETER 8 MM AND MESH 20 X 20 MM)	2,10	4,0	0,085
CASTING OF LIGHTWEIGHT CONCRETE	2,10	64,0	1,344
WATERPROOF LAYER, 4 MM	2,10	4,5	0,095
LAYERS OF THERMAL INSULATION AND STIFERITE 3 MM	2,10	1,0	0,021
WATERPROOF LAYER, 4 MM	2,10	4,5	0,095
TILE ROOF	2,10	60	1,260
			TOT = 3,40

The dead loads insisting on the struts are equal to:
G_2 = 3.40 kN/m for the central trusses;
G_2 = 1,70 kN/m for the lateral trusses (half wheel base).

Live loads: snow

The loads of snow were calculated according to the directions of the Norme Tecniche per le Costruzioni (Paragraph 3.4). Since Venice is located in zone II with the parameter a_s (height sea level) < 200 m and a pitch of roof (α), measured from horizontal, 0°<α<30° the load of snow q_s is equal to 0,80 kN/m^2.

Combinations of actions

According to D.M. 14.01.2008, for the assessments, is necessary to combine the actions, as reported in Tab. 4.6.37.

Tab. 4.6.37: Combinations of actions

LIMIT STATE DESIGN	LOAD COMBINATIONS	
ULTIMATE LIMIT STATE (ULS)	$Fd=\gamma_{G1}G1+\gamma_{G2}G2+\gamma_{Q1}Q_{k1}$	
SERVICEABILITY LIMIT STATE (SLS)	CHARACTERISTIC	$Fd=G1+G2+Q_{k1}$
	FREQUENT	$Fd=G1+G2+\psi_{11}Q_{k1}$
	QUASI-PERMANENT	$Fd=G1+G2+\psi_{21}Q_{k1}$

Where:
γ_{G1} is the partial factor for permanent actions of the structure, G;
γ_{G2} is the partial factor for permanent actions of the non-structural elements of the structure, G;
γ_{Q1} is the partial factor for variable actions;
$Q_{\psi 11}$, $_{\psi 21}$ are the factors defining representative values of variable actions (operant with the combination of actions).
In Table 2.5.I of the NTC2008 were chosen the factors defining representative values of variable actions. Tab. 4.6.38 shows the values considered.

Tab. 4.6.38: Factors defining representative values of variable actions and partial factors

COEFFICIENT	SYMBOL	VALUE
FACTORS DEFINING REPRESENTATIVE VALUES OF VARIABLE ACTIONS	ψ_{11}	0,0
	ψ_{21}	0,0
PARTIAL FACTOR	γ_{G1}	1,0 favourable 1,3 unfavourable
	γ_{G2}	0,0 favourable 1,5 unfavourable
	γ_{Q1}	0,0 favourable 1,5 unfavourable

Verification of trusses according to the D.M. 14.01.2008

The current standard, D.M. 14.01.2008, for the concrete structures, imposes to assess the elements with reference to the ultimate limit state (ULS) and the serviceability limit state (SLS).

For the evaluation of stresses of the elements of the truss was created a tridimensional (Fig. 4.6.29) finite elements model (FEM model) by means of the use of the software Straus7®. The modeling is performed with a static linear analysis and allowed to obtain values of the stresses required for verifications of the structure and for the assessment of the stability conditions of the roof. The elements of the trusses where is necessary make a recovery intervention were indentified.

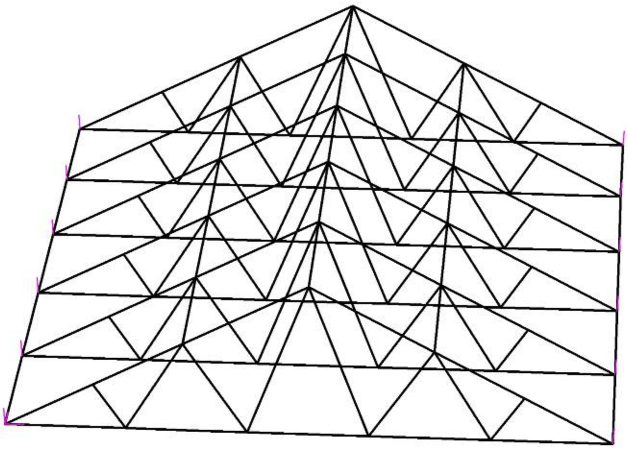


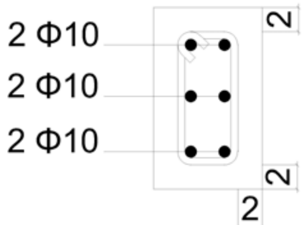
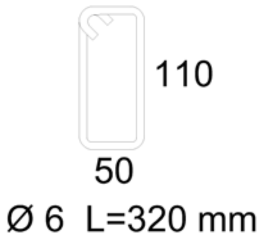
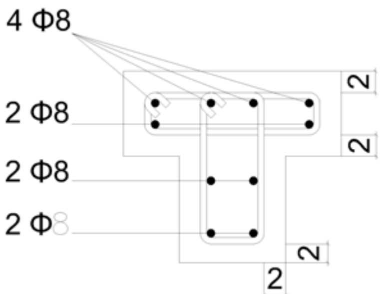
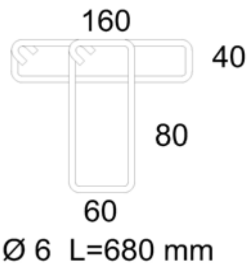
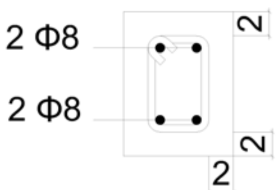
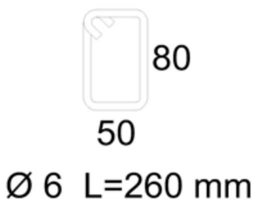
Fig. 4.6.29: Tridimensional numerical model of trusses

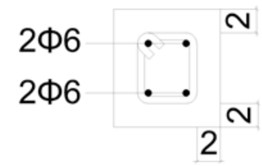
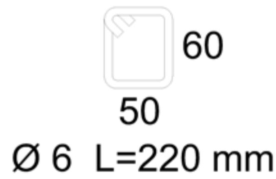
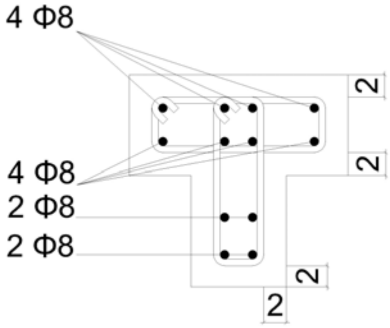
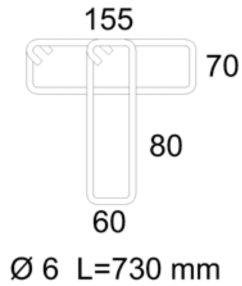
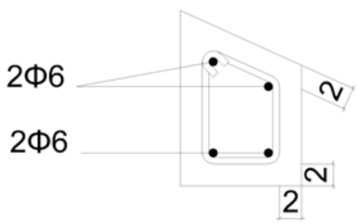
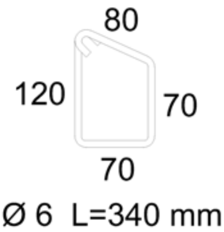
At beams loads of gravity, weight of the roof and snow loads were applied. The actions were then combined as previously shown in Tab. 4.6.37.

At the elements of the truss have been applied the material and geometric characteristics and the distribution and characteristics of the reinforcement bars and stirrups, obtained from on-site tests and survey.

Tab. 4.6.39 shows the characteristics of reinforcement bars and of stirrups for each element of the truss.

Tab. 4.6.39: Reinforcement bars and stirrups characteristics for each element of the truss

ELEMENTS	GEOMETRY OF THE REINFORCEMENT BARS	GEOMETRY OF THE STIRRUPS
TIE BEAM		
STRUT		
SECONDARY ELEMENTS IN TENSION 9-11 SECONDARY ELEMENTS IN COMPRESSION 7-13		

SECONDARY ELEMENTS IN TENSION 5-15 SECONDARY ELEMENTS IN COMPRESSION 3-17		
RIDGE BEAM		
PURLIN		

Verification of the ultimate limit state (ULS)

Design resistance to bending and axial force

According to the N.T.C. 14.01.2008, the stresses in the concrete and in the reinforcement bars must be deduced using the stress-strain diagrams. For the verification of the combined bending moment and axial compression, the following expression shall be satisfied:

$$M_{Rd} = M_{Rd}\,(N_{Ed}) \geq M_{Ed}$$

Where:

M_{Rd} is the calculated resisting moment corresponding to the axial load N_{Ed};

N_{Ed} is the design value of the applied axial force;

M_{Ed} is the design value of the applied internal bending moment.

The bending moment is considered positive if the inferior fibers are in tension and negative if the superior fibers are in compression. The axial force is considered positive in the case of compression and negative in the case of tension.

For each element of the truss were extracted from the FEM model the maximum and minimum values of bending moments and axial forces, reported in Tab. 4.6.40 and Tab. 4.6.41.

Tab. 4.6.40: M_{Ed} and N_{Ed} maximum for each element of the truss

ELEMENTS	M_{ED} (KN·M)	N_{ED} (KN)
STRUT	+2,8275	+111,7978
TIE BEAM	+0,6242	-93,1831
SECONDARY ELEMENTS IN TENSION 9 - 11	+0.0569	-26,4667
SECONDARY ELEMENTS IN TENSION 5 - 15	+0,0632	-15,2633
SECONDARY ELEMENTS IN COMPRESSION 7 – 13	+0,0423	+15,0832

SECONDARY ELEMENTS IN COMPRESSION 3 – 17	+0,0344	+7,4892
PURILN	+0,1076	-0,0423
RIDGE BEAM	+2,6715	-0,0033

Tab. 4.6.41: M_{Ed} e N_{Ed} minimum for each element of the truss

ELEMENTS	M_{ED} (KN·M)	N_{ED} (KN)
STRUT	-5,6398	+80,5410
TIE BEAM	-0.2655	-75,9061
SECONDARY ELEMENTS IN TENSION 9 - 11	-0,1146	-17,2544
SECONDARY ELEMENTS IN TENSION 5 - 15	-0,0487	-9,9182
SECONDARY ELEMENTS IN COMPRESSION 7 – 13	-0,0725	+24,4023
SECONDARY ELEMENTS IN COMPRESSION 3 – 17	-0,0992	+8,6055
PURILN	-0,1088	+0,0085
RIDGE BEAM	-3,2827	-0,0033

To calculate the bending moment corresponding to N_{Ed} (M_{Rd}) has been used a calculation program (VCA SLU) for the verifications of the ultimate limit state of reinforced concrete structures. The data entered are: geometry of the element, distribution and dimension of reinforcement bars and stirrups, characteristics of materials and maximum bending moment and axial force. Below it is reported (Fig. 4.6.30) an example (strut element) of the calculation program using.

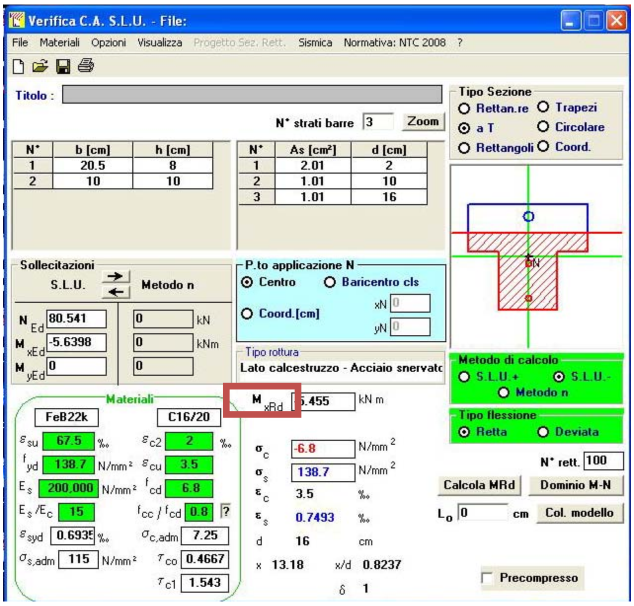


Fig. 4.6.30: Example of the program (VCA SLU) using to determine the value of Mrd

The verification is satisfied if the values of the M_{Rd} are higher or equal to the calculated value of the bending component of the action. Therefore, should be included inside the range of the actions and of the limit resistance range of the section in the plane M-N (domino M-N). As an example, in Fig. 4.6.31 the domino M-N in the case of the strut more stressed is reported.

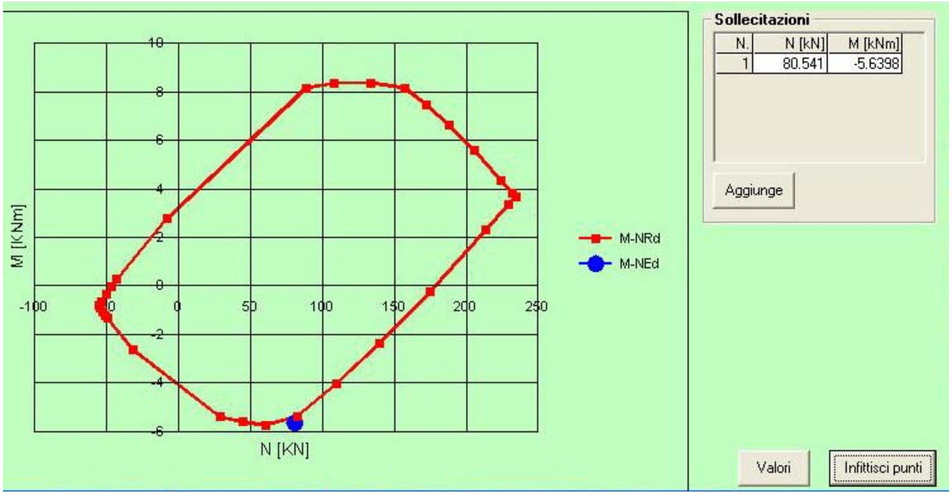


Fig. 4.6.31: Dominio M-N for the case of the strut more stressed

Tab. 4.6.42 shows the value obtained from the verification of combined resistance to bending and axial force.

Tab. 4.6.42: Verification of the combined resistance to bending moment and axial force

ELEMENT	M_{RD} (KN·M)	M_{ED} (KN·M)	VERIFICATION $M_{RD} \geq M_{ED}$
STRUT	-5,4550	-5,6398	Not verified
	+8,7190	+2,8275	Verified
TIE BEAM	Not verified	-0,2655	Not verified
	Not verified	+0,6242	Not verified
SECONDARY ELEMENT IN TENSION 9 - 11	-0,5398	-0,1146	Verified
	+0,0908	+0,0569	Verified
SECONDARY ELEMENT IN TENSION 5 - 15	-0,2655	-0,0487	Verified
	+0,0270	+0,0632	Not verified
SECONDARY ELEMENT IN COMPRESSION 7 - 13	-2,0850	-0,0725	Verified
	+1,8350	+0,0423	Verified
SECONDARY ELEMENT IN COMPRESSION 3 - 17	-0,8513	-0,0992	Verified
	+0,8175	+0,0344	Verified
PURLIN	-0,8497	-0,1088	Verified
	+0,8473	+0,1076	Verified
RIDGE BEAM	-5,9760	-3,2827	Verified
	+3,333	+2,6715	Verified

The verifications are not satisfied for the struts (in the case of negative bending moment), for the tie beam and the secondary elements in tension 5-15 (in the case of positive bending moment). For the tie beam was not possible calculated M_{Rd} because the reinforcement bars are insufficient. In fact, the maximum axial force permissible is equal to -65,3277 kN in the actual reinforcement bars situation of the element, lower than that actually present in the tie beam

The maximum bending moment of the struts is in proximity of the ridge node. While, for the secondary elements in tension 5-15 it is in correspondence to the mid span of the elements. As an example, in Fig. 4.6.32 and Fig. 4.6.33 are reported the bending moment and axial stress diagrams for the struts.

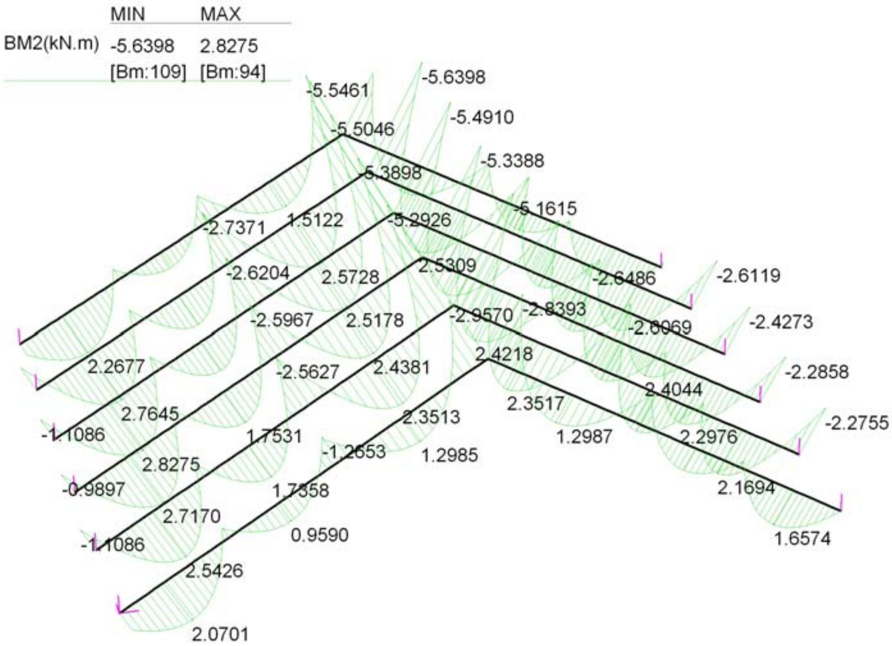


Fig. 4.6.32: Bending moment diagram of the struts (kN·m)

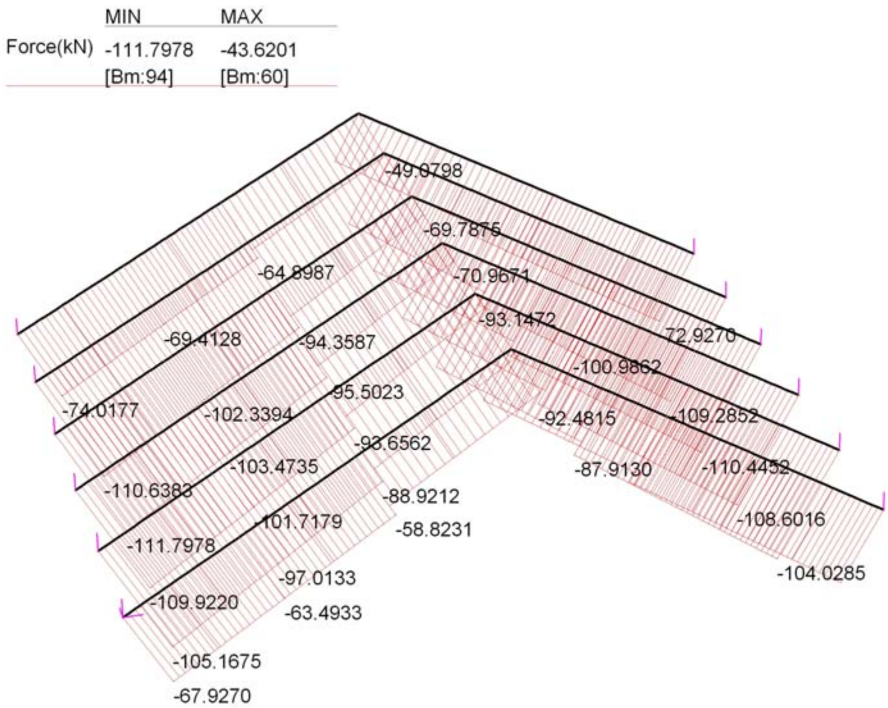


Fig. 4.6.33: Axial stress diagram of the struts (kN)

Since that the verification was carried out in the truss with the maximum bending moment, and it is not satisfied for a minimum percentage, the same verification was conducted for the struts of the other trusses of the structure into consideration. In Tab. 4.6.43 the results are reported.

Tab. 4.6.43: Verification of the combined resistance to bending moment and axial force for the struts of the trusses into consideration

STRUT ELEMENT	N_{Ed} (kN)	M_{Ed} (kN·m)	M_{Rd} (kN·m)	VERIFICATION $M_{Rd} \geq M_{Ed}$
TRUSS B1	49,6573	-2,9570	-5,6840	Verified
TRUSS B2	75,8512	-5,2926	-5,5560	Verified
TRUSS B3	80,1434	-5,3898	-5,4640	Verified
TRUSS B4	81,9128	-5,5046	-5,4220	Not verified
TRUSS B5	80,5410	-5,6398	-5,4550	Not verified
TRUSS B6	55,0968	-3,1815	-5,7360	Verified

The verifications are not satisfied for the struts of the trusses called B4 and B5. Therefore, a strengthening intervention is needed. Since it was observed a diffuse spalling of concrete (Fig. 4.6.34), the same intervention was applied to all of the struts.

Fig. 4.6.34: Detail of the decay of the joint struts-secondary elements

Similar considerations were done for the secondary elements in tension 5-15, since many of them did not pass the verifications. The selected strengthening interventions where applied to all of the corresponding elements. In Tab. 4.6.44 the results are reported.

Tab. 4.6.44: Verification of the combined resistance to bending moment and axial force for the secondary elements in tension 5-15

SECONDARY ELEMENT IN TENSION 5-15	N_{Ed} (kN)	M_{Ed} (kN·m)	M_{Rd} (kN·m)	VERIFICATION $M_{Rd} \geq M_{Ed}$
TRUSS B1	-9,9182	0,0478	0,2655	Verified
TRUSS B2	-14,4730	0,0627	0,0653	Verified
TRUSS B3	-14,8724	0,0636	0,4612	Not verified
TRUSS B4	-15,5590	0,0613	0,0125	Not verified
TRUSS B5	-15,2633	0,0632	0,0271	Not verified

TRUSS B6	-10,6213	0,0427	0,2369	Verified

Shear

According to the N.T.C. 14.01.2008, for the verification of shear (V_{Rd}) the following expression shall be satisfied:

$$V_{Rd} \geq V_{Ed}$$

Where:

V_{Ed} is the design value of the applied shear force.

The shear resistance of a member with shear reinforcement is equal to:

$$V_{Rd} = \min (V_{Rsd}, V_{Rcd})$$

The design value of the shear which can be sustained by the shear reinforcement is equal to:

$$V_{Rsd} = 0{,}9 \cdot d \cdot (A_{sw}/s) \cdot f_{yd} \cdot (ctg\alpha + ctg\theta) \sin\alpha$$

The design shear resistance of the members without shear reinforcement is equal to:

$$V_{Rcd} = 0{,}9 \cdot d \cdot b_w \cdot \alpha_c \cdot f'_{cd} \cdot (ctg\alpha + ctg\theta) / (1 + ctg\theta)$$

Where:

- d is the effective depth of a cross-section;
- b_w is the smallest width of the cross-section in the tensile area;
- A_{sw} is the area of the tensile reinforcement;
- s spacing between two stirrups;
- α is the angle between shear rewinforcement and the beam axis perpendicular to the shear force;
- f'_{cd} is the reduced design value of concrete compressive strength ($f'_{cd} = 0{,}5 \times f_{cd}$);
- α_c is a coefficient ≥ 1 depending on the axial force acting on the beam.

Tab. 4.6.39 shows the values of A_{sw}.

In Tab. 4.6.45 and Tab. 4.6.46 the values of V_{Rsd} and V_{Rcd} are reported.

Tab. 4.6.45: Values of V_{Rsd}

ELEMENT	d (mm)	A_{sw} (mm^2)	S (mm)	f_{yd} (MPa)	V_{Rsd} (kN)
STRUT	160	57	20	138,5	56,64
TIE BEAM	130	57	20	138,5	46,02
SECONDARY ELEMENTS IN TENSION 9 - 11	100	57	20	138,5	35,40
SECONDARY ELEMENTS IN TENSION 5 - 15	80	57	20	138,5	28,32
SECONDARY ELEMENTS IN COMPRESSION 7 – 13	100	57	20	138,5	35,40
SECONDARY ELEMENTS IN COMPRESSION 3 – 17	80	57	20	138,5	28,32
PURILN	140	57	20	138,5	49,56
RIDGE BEAM	160	57	20	138,5	56,64

Tab. 4.6.46: Values of V_{Rcd}

ELEMENT	d (mm)	bw (mm)	α_c	F'_{cd} (MPa)	V_{Rcd} (kN)
STRUT	160	100	1	3,36	48,38
TIE BEAM	130	90	1	3,36	35,38

SECONDARY ELEMENTS IN TENSION 9 - 11	100	90	1	3,36	27,22
SECONDARY ELEMENTS IN TENSION 5 - 15	80	90	1	3,36	21,77
SECONDARY ELEMENTS IN COMPRESSION 7 – 13	100	90	1	3,36	27,22
SECONDARY ELEMENTS IN COMPRESSION 3 – 17	80	90	1	3,36	21,77
PURILN	140	110	1	3,36	46,57
RIDGE BEAM	160	85	1	3,36	41,13

From the numerical model, the shear stress of all the elements of the truss were obtained. For the verification, these were compared with the minimum value between V_{Rsd} and V_{Rcd}, as reported in Tab. 4.6.47.

Tab. 4.6.47: Verification of shear stress for all the elements of the truss

ELEMENT	**V_{Rcd} (kN)**	**V_{Rsd} (kN)**	**V_{Ed} (kN)**	**VERIFICATION V_{Rd} (min (V_{Rsd}, V_{Rcd})) ≥ V_{Ed}**
STRUT	48,38	56,64	11,000	Verified
TIE BEAM	35,38	46,02	0,7634	Verified
SECONDARY ELEMENTS IN TENSION 9 - 11	27,22	35,40	0,1881	Verified
SECONDARY ELEMENTS IN TENSION 5 - 15	21,77	28,32	0,1745	Verified
SECONDARY ELEMENTS IN COMPRESSION 7 – 13	27,22	35,40	0,1867	Verified
SECONDARY ELEMENTS IN COMPRESSION 3 – 17	21,77	28,32	0,2519	Verified
PURILN	46,57	49,56	0,1036	Verified
RIDGE BEAM	41,13	56,64	3,7057	Verified

For all the elements of truss, the verification at the Ultimate Limit State is satisfied.

Verification of the serviceability limit state (SLS)

Stress limitation

The adopted standard cites: "Evaluated the internal actions in all of the parts of the structure, due to characteristic and quasi-permanent combinations, the maximum stresses are calculated both for concrete and steel. These values must be lower than the allowable values define by the rules."

Therefore, the compressive and tensile stresses in the elements of the truss were calculated, as combination of bending and axial force.

In Tab. 4.6.48 and Tab. 4.6.49 the values of compressive and tensile stresses for characteristic and quasi-permanent combinations are reported.

Tab. 4.6.48: Maximum compressive and tensile stresses for the characteristic combination

ELEMENT	**N_{Ed} (kN)**	**M_{Ed} (kN·m)**	**σc (N/mm^2)**	**σs (N/mm^2)**
STRUT	+55,4979	-3,8254	-8,238	+43,730
	+76,8260	+1,9281	-2,974	/

TIE BEAM	-52,3497	-0,2003	/	+111,146
	-64,0839	+0,4362	/	+136,000
SECODARY ELEMENT IN TENSION 9 - 11	-12,1481	-0,0878	/	+60,438
	-18,3198	+0,0435	/	+91,100
SECONDARY ELEMENT IN TENSION 5 - 15	-6,9375	-0,0373	/	+61,394
	-10,5083	+0,0461	/	+93,000
SECONDARY ELEMENT IN COMPRESSION 7 - 13	+16,7324	-0,0552	-1,396	/
	+10,4824	+0,0312	-0,863	/
SECONDARY ELEMENT IN COMPRESSION 3 - 17	+5,9048	-0,0674	-0,924	/
	+5,1337	+0,0235	-0,609	/
PURLIN	+0,0057	-0,0729	-0,358	+13,020
	-0,0282	0,0721	-0,353	+13,170
RIDGE BEAM	-0,0022	-2,2402	-4,062	+61,040
	-0,0022	+1,7700	-2,602	+110,800

Tab. 4.6.49: Maximum compressive and tensile stresses for the quasi-permanent combination

ELEMENT	N_{Ed} **(kN)**	M_{Ed} **(kN·m)**	**σc (N/mm²)**
STRUT	+41,6186	-2,7229	-5,927
	+57,1090	+1,3975	-2,180
SECONDARY ELEMENT IN COMPRESSION 7 - 13	+12,3501	-0,0540	-1,075
	+8,0805	+0,0284	-0,680
SECONDARY ELEMENT IN COMPRESSION 3 - 17	+4,3683	-0,0484	-0,676
	+3,7854	+0,0183	-0,455
PURLIN	+0,0039	-0,0497	-0,244
	-0,0187	+0,0492	-0,241
RIDGE BEAM	-0,0015	-1,6275	-2,951
	-0,0015	+1,1574	-1,701

<u>Maximum compressive stress in the concrete in the serviceability limit state</u>

For the verification of the maximum compressive stress in the concrete σ_c the following equations shall be satisfied:

$\sigma_c < 0{,}60\ f_{ck}$ for the characteristic combination;

$\sigma_c < 0{,}45\ f_{ck}$ for the quasi-permanent combination.

In Tab. 4.6.50 and Tab. 4.6.51 verification data are reported.

Tab. 4.6.50: Verification data of maximum compressive stress in the concrete for the characteristic combination (SLS)

ELEMENT	σc (N/mm^2)	f_{ck} (N/mm^2)	0,60 · f_{ck} (N/mm^2)	VERIFICATION $\sigma c < 0{,}60 \cdot f_{ck}$
STRUT	-8,238	16	9,6	Verified
	-2,974	16	9,6	Verified
SECONDARY ELEMENT IN COMPRESSION 7 - 13	-1,396	16	9,6	Verified
	-0,863	16	9,6	Verified
SECONDARY ELEMENT IN COMPRESSION 3 - 17	-0,924	16	9,6	Verified
	-0,609	16	9,6	Verified
PURLIN	-0,358	16	9,6	Verified
	-0,353	16	9,6	Verified
RIDGE BEAM	-4,062	16	9,6	Verified
	-2,602	16	9,6	Verified

Tab. 4.6.51: Verification data of maximum compressive stress in the concrete for the quasi-permanent combination (SLS)

ELEMENTO	σc (N/mm^2)	f_{ck} (N/mm^2)	0,45 · f_{ck} (N/mm^2)	VERIFICATION $\sigma c < 0{,}45 \cdot f_{ck}$
STRUT	-5,927	16	7,2	Verified
	-2,180	16	7,2	Verified
SECONDARY ELEMENT IN COMPRESSION 7 - 13	-1,075	16	7,2	Verified
	-0,680	16	7,2	Verified
SECONDARY ELEMENT IN COMPRESSION 3 - 17	-0,676	16	7,2	Verified
	-0,455	16	7,2	Verified
PURLIN	-0,244	16	7,2	Verified
	-0,241	16	7,2	Verified
RIDGE BEAM	-2,951	16	7,2	Verified
	-1,701	16	7,2	Verified

All elements are verified for compressive stress in the concrete in the serviceability limit state for characteristic and quasi-permanent combinations.

Maximum steel stress in the serviceability limit state

For the verification of the maximum steel stress σ_s the following equation, for the characteristic combination, shall be satisfied:

$\sigma_s < 0{,}8\ f_{yk}$ for the characteristic combination.

In Tab. 4.6.52 verification data are reported.

Tab. 4.6.52: Verification data of maximum steel stress for the characteristic combination (SLS)

ELEMENT	σs (N/mm²)	f_{yk} (N/mm²)	0,80 · f_{yk} (N/mm²)	VERIFICATION σs < 0,80 · f_{yk}
STRUT	+43,730	215	172	Verified
	/	215	172	/
TIE BEAM	+111,146	215	172	Verified
	+136,000	215	172	Verified
SECONDARY ELEMENT IN TENSION 9 - 11	+60,438	215	172	Verified
	+91,100	215	172	Verified
SECONDARY ELEMENT IN TENSION 5 - 15	+61,394	215	172	Verified
	+93,000	215	172	Verified
PURLIN	+13,020	215	172	Verified
	+13,170	215	172	Verified
RIDGE BEAM	+61,040	215	172	Verified
	+110,800	215	172	Verified

All elements are verified for steel stress in the serviceability limit state for the characteristic combination.

Deflection control

Verifica di deformabilità

The adopted standard imposes the calculation the deflections of elements. The D.M. 14.01.2008 cites: "*For the appearance and general utility of the structure, the sug of beams and floors subjected to quasi-permanent loads should not generally exceed span/250.*"

The verification was carried out comparing the value imposed by the standard with the maximum displacement, Dz, in the direction of the force of gravity (Dz). In Tab. 4.6.53 verification data are reported.

Tab. 4.6.53: Deflection control

ELEMENT	Span (m)	(1/250) · span (m)	Displacement Dz (m)	VERIFICATION (1/250) · span> Dz
STRUT	6,63	0,0265	0,0050	Verified
TIE BEAM	11,60	0,0464	0,0047	Verified
SECONDARY ELEMENT IN TENSION 9 - 11	3,45	0,0138	0,0046	Verified
SECONDARY ELEMENT IN TENSION 5 - 15	2,22	0,0089	0,0045	Verified
SECONDARY ELEMENT IN COMPRESSION 7 - 13	2,18	0,0087	0,0046	Verified
SECONDARY ELEMENT IN COMPRESSION 3 - 17	1,12	0,0045	0,0040	Verified
PURLIN	2,10	0,0084	0,0046	Verified
RIDGE BEAM	2,10	0,0084	0,0043	Verified

All elements of truss are verified for the deflection control in the serviceability limit state for the quasi-permanent combination.

Crack control

The crack control of the beams truss is verified thanks to the low stresses . However, the actual situation of the structure presents a diffuse spalling of concrete and the corrosion of the reinforcement bars. Therefore, a recovery intervention of the concrete cover is needed.

Intervention with high perform carbon fibers

Given the results of the verifications, a recovery intervention in some reinforced concrete elements of trusses is needed.

For all elements of trusses the verifications for the SLS condition and the shear for the ULS are satisfied.

The verifications of combined bending and axial force for the ULS are not satisfied for the struts (in the case of negative bending moment), tie beams (in the case of positive and negative bending moment) and secondary elements in tension called 5-15 (in the case of positive bending moment). Only for the lateral trusses the verification is satisfied. Since it was observed a diffuse spalling of concrete and a corrosion of reinforcement bars the reinforcement intervention was applied to all of the elements into consideration. For the tie beams the verification is not satisfied for insufficient reinforcement bars.

The choice to the technique to use was decided by the necessity to increase the resistance of the structure without modify the geometrical characteristics of the roof. In particular, the intervention should be minimum impact, reversible and must respect the existing material characteristics. In the case into consideration, the better solution is the application of high perform unidirectional carbon fiber for structural reinforcement. The carbon fibers are characterized by high Young modulus of elasticity as well as high strength.

In the following table (Tab. 4.6.54) geometrical and mechanical characteristics of the used carbon fibers are reported.

Tab. 4.6.54: Geometrical and mechanical characteristics of FRP

DRY FIBER (SINGLE FILAMENT)	
STRESS IN FRP REINFORCEMENT, σfibra	4900 N/mm^2
YOUNG'S MODULUS OF ELASTICITY OF FRP REINFORCEMENT, Efibra	240000 N/mm^2
STRAIN OF FRP REINFORCEMENT, εfibra	2,00%
DENSITY	1,80 g/cm^3
IMPREGNATED FABRICS (VALUES FOR THE CALCULATION)	
EQUIVALENT THICKNESS OF FRP, Teq	0,165 mm
CHARACTERISTIC STRENGTH OF FRP REINFORCEMENT, f_{fk}	3700 N/mm^2
YOUNG'S MODULUS OF ELASTICITY OF FRP REINFORCEMENT, Ef	230000 N/mm^2
STRAIN OF FRP REINFORCEMENT, εf	1,75%

For the assessments, the characteristics of the impregnated fabrics are used.

Flexural capacity of FRP-strengthened members subjected to bending moment and axial force

The Paragraph 4.2.2.4 of the "Guide for the design and construction of externally bonded FRP systems for strengthening existing structures. Materials, RC and PC structures, masonry structures" CNR-DT 200/2004, cites "Flexural design ULS of FRP strengthened members require that both flexural capacity, M_{Rd}, and factored ultimate moment, M_{Sd}, satisfy the following inequation:

$$M_{Sd} \leq M_{Rd}$$

However, the presence of axial force, N_{Sd}, needs to be taken into account when determining the member flexural capacity, M_{Rd}."

Struts

The struts of the roofing do not result verified to the combined bending and axial force, in the case of maximum negative bending moment. Therefore, it is proposed an intervention with FRP (Fiber Reinforced Concrete) whose minimum (or equal) geometrical and mechanical characteristics are reported in Tab. 4.6.54.

Since the difference between M_{sd} and M_{rd} is minimum (Tab. 4.6.55) it will be sufficient applied the minimum layout of fiber to verified the elements.

Tab. 4.6.55: M_{Rd} and M_{Ed} for the strut subjected to the highest forces

ELEMENT	M_{RD} (KN·M)	M_{ED} (KN·M)	VERIFICATION $M_{RD} \geq M_{ED}$
STRUT	-5,4550	-5,6398	Not verified

The beams into consideration are in compression, therefore the fiber should be applied in the upper part of the struts, along the whole length of the element, as indicated in Fig. 4.6.35. The fiber must have a width of 50 mm and a equivalent dry thickness of 0,165 mm.

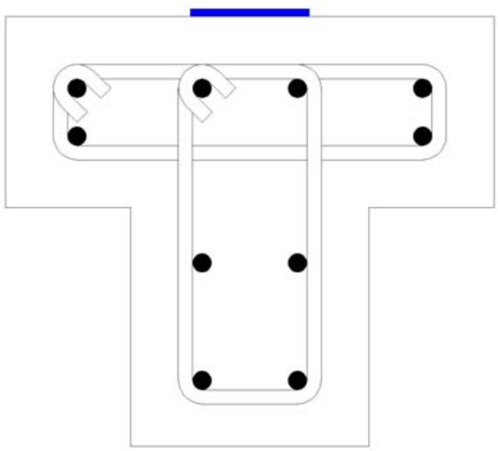


Fig. 4.6.35: Detail of distribution of FRP in the strut

Tie beam

The design of resistance to bending and axial force for tie beams is not satisfied because reinforcement bars are insufficient . Therefore, since the beam is subjected to traction, it is necessary applied two layers of carbon fibers disposed parallel to the reinforcement bars (Fig. 4.6.36).

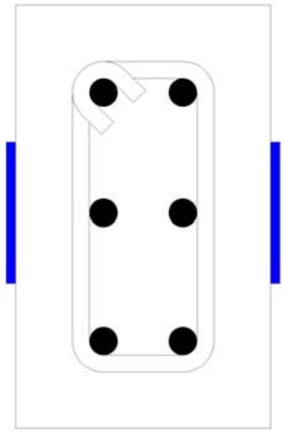


Fig. 4.6.36: Detail of distribution of FRP in the tie beam

In the following tale (Tab. 4.6.56), the calculated value of bending moment and axial force, for the tie beam, are reported.

Tab. 4.6.56: M_{Ed} and N_{Ed} for the tie beam with FRP subjected to the highest forces

TIE BEAM	M_{ED} (KN·M)	N_{ED} (KN)
MAXIMUM POSITIVE BENDING MOMENT	+0,6242	-93,1831
MAXIMUM NEGATIVE BENDING MOMENT	-0,2655	-75,9061

The intervention consists in the application of two layer of carbon fiber with a thickness of 0,165 mm and a width of 50 mm. The fiber is distributed along the whole length of the element. To verified that the proposal intervention is sufficient, it was created a dominio M-N in which are calculated the minimum values of M_{Rd} and N_{Rd}. Successively it was verified if the maximum stresses calculated are internal at the range of the dominio M-N. Since fibers should be applied in the central section of tie beam the bending moment will not change.

Therefore, to evaluate maximum positive and negative bending moments, in the program "Verification R.C. U.L.S." were insert geometrical and materials characteristics of the element into consideration, putting equal to zero the value of the axial force (Fig. 4.6.37).

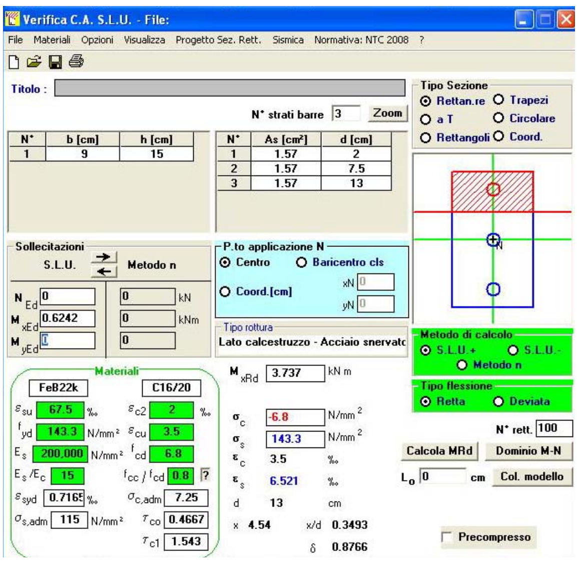


Fig. 4.6.37: Value of M_{Rd} for the tie beam, with axial force considered equal to zero

Below, the obtained values of maximum positive and negative M_{sd} are reported.

$M_{rd} = 3{,}737\ kN \cdot m$

$M_{rd} = -3{,}737\ kN \cdot m$

For the calculation of Nr, the section has been considered only reagent to tensile stress, with the contributions of reinforcement bars and of carbon fibers. The maximum FRP tensile strain, ε_{fd}, can be calculated, in according to CNR-DT 200/2004, as follows:

$$\varepsilon_{fd} = \min\{\eta_a \cdot (\varepsilon_{fk}/\gamma_f)\ ;\ \varepsilon_{fdd}\}$$

Where:

- ε_{fk} is the characteristic rupture strain FRP reinforcement;
- γ_f and η_a are the coefficients defined in Table 3-2 and in Table 3-4, respectively, of the CNR-DT 200/2004, and are equal to 1,10 and 0,85;
- ε_{fdd} is the maximum strain FRP reinforcement before debonding.

The value of the characteristic rupture strain, ε_{fk}, in the FRP system can be calculate as follows:

$$\varepsilon_{fk} = f_{fk}/E_f = 3330/207000 = 0{,}015$$

Where:

$f_{fk} = \alpha_{ff} \cdot f_{fibra} = 0{,}9 \cdot 3700 = 3330\ N/mm^2$

$E_f = 0.9 \cdot 230000 = 207000\ N/mm^2$

The value of the design strain, ε_{fdd}, in the FRP system can be calculate as follows:

$$\varepsilon_{fdd} = f_{fdd}/E_f = 437{,}80/207000 = 0.002$$

Where:

- design debonding strength of FRP reinforcement (mode 1)

$f_{fdd} = [1/(\gamma_{fd} \cdot \sqrt{\gamma_c})] \cdot [\sqrt{(2 \cdot E_f \cdot \acute{\Gamma}_{fk})/t_f}] = [1/(1{,}2 \cdot \sqrt{1{,}5})] \cdot [\sqrt{(2 \cdot 207000 \cdot 0{,}165)/0{,}165}] = 437{,}80\ N/mm^2$

- the characteristic values of specific fracture energy

$\acute{\Gamma}_{fk} = K_G \cdot K_b \cdot \sqrt{(f_{ck} \cdot f_{ctm})} = 0.003 \cdot 1 \cdot \sqrt{(16 \cdot 1{,}90)} = 0{,}165\ N/mm^2$

- K_b is a geometric coefficient depending on both width, b, of the strengthened beam and width, b_f, of the FRP system.

Therefore, the maximum FRP tensile strain is equal to:

$$\varepsilon_{fd} = \min\{\eta_a \cdot (\varepsilon_{fk}/\gamma_f)\ ;\ \varepsilon_{fdd}\} = \min\{0{,}012;\ 0{,}002\} = 0{,}002$$

For the considered section, with equilibrium considerations on translation it was obtained that:

$$N + \varepsilon_{s1} \cdot A_{s1} \cdot E_s + \varepsilon_{s2} \cdot A_{s2} \cdot E_s + \varepsilon_{s3} \cdot A_{s3} \cdot E_s + \varepsilon_f \cdot A_f \cdot E_f = 0$$

Imposing the value of the bending moment equal to zero is obtained that: $\varepsilon_{s1} = \varepsilon_{s2} = \varepsilon_{s3} = \varepsilon_f$

Furthermore, since that the distribution of the reinforcement bars (Fig. 4.6.38) is symmetric is obtained that: $A_{s2} = A_{s2} = A_{s3}$

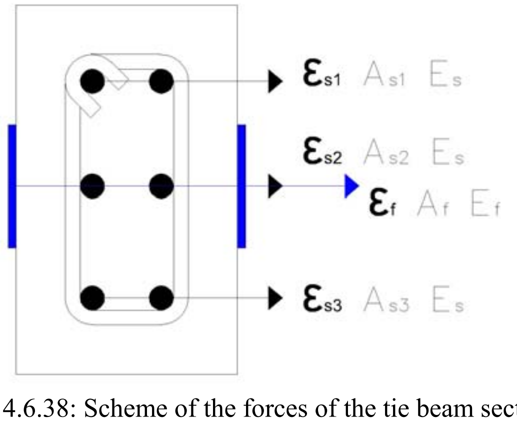


Fig. 4.6.38: Scheme of the forces of the tie beam section

Therefore, the value of N_{rd} can be calculate as follows:

$N_{rd} = - \varepsilon_f (A_f \cdot E_f + 3\ A_s \cdot E_s) = -204{,}65$ kN

Where:

- $\varepsilon = 0{,}002$ maximum strain of FRP and steel reinforcement;
- $A_f = (0{,}165 \cdot 50 \cdot 2) = 16{,}5$ mm^2 area of FRP reinforcement;
- $E_f = 0{,}9 \cdot 230000 = 207000$ N/mm^2 Young's modulus of elasticity of FRP reinforcement;
- $A_s = (2\text{Ø}10) = 157$ mm^2 area of steel reinforcement;
- $E_s = 210000$ N/mm^2 Young's modulus of elasticity of steel reinforcement.

The two layers of FRP with the geometrical and mechanical properties reported in Tab. 4.6.54 and with a length of 50 mm are sufficient to verified the flexural capacity of FRP-strengthened members subjected to bending moment and axial force if the following equation is satisfied:

$$(N_{sd}/N_{rd}) + (M_{sd}/M_{rd}) \leq 1$$

For the maximum positive bending moment:

$(-93{,}183/-204{,}65) + (0{,}624/3{,}737) = 0{,}622 < 1$ Verified

For the maximum negative bending moment:

$(-75{,}906/-204{,}65) + (-0{,}266/-3{,}737) = 0{,}442 < 1$ Verified

Fig. 4.6.39 shows the simplified "dominio M-N", where it is possible to see that the project values of stresses are inside the range.

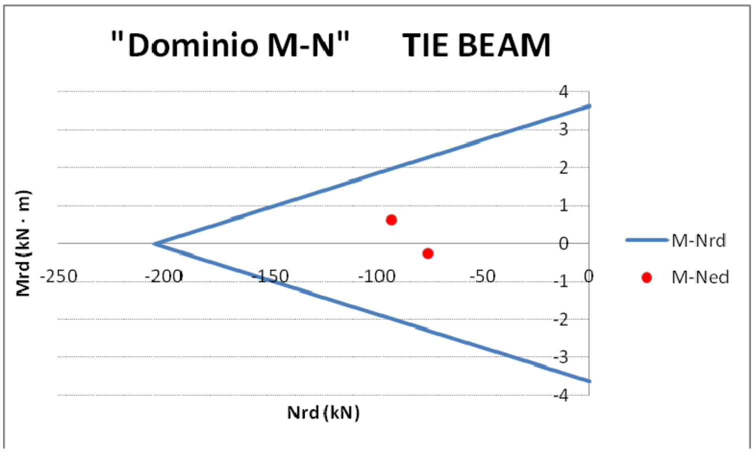


Fig. 4.6.39: "Dominio M-N" for the tie beam

The verification of flexural capacity of tie beams strengthened with FRP subjected to bending moment and axial force is satisfied.

Secondary beams in tension called 5-15

Secondary beams in tension, called 5-15, are not verified for the maximum positive bending moment. The behavior of these elements is the same as the tie beam. For this reason, the procedure used for the verification is analogous.

In Tab. 4.6.57 the values of bending moment and axial force for the secondary elements in tension subjected to the highest forces are reported. The verifications were carried out also for the maximum negative bending moment.

Tab. 4.6.57: M_{Ed} and N_{Ed} for the secondary elements in tension (called 5-15) subjected to the highest forces

SECONDARY BEAM IN TENSION 5-15	M_{ED} (KN·M)	N_{ED} (KN)
MAXIMUM POSITIVE BENDING MOMENT	+0,0632	-15,2633
MAXIMUM NEGATIVE BENDING MOMENT	-0,0487	-9,9182

The reinforcement intervention necessary to increase the resistance of secondary beams in tension (5-15) is composed by two fibers with a equivalent dry thickness of 0,165 mm and a width of 50 mm. The fiber is distributed for all the length of element.

The verifications, carried out with an analogous procedure to the case of the tie beam, are satisfied.

Constructive details

Struts, tie beams and secondary elements in tension (5-15) of the trusses into consideration must be reinforced, according to the diagram reported in Fig. 4.6.40.

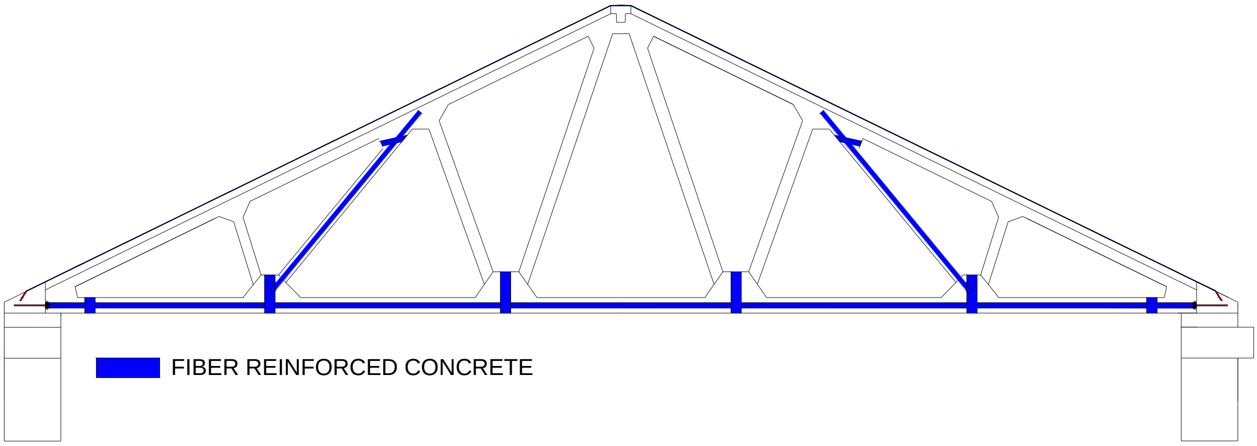


Fig. 4.6.40: Diagram of the distribution of FRP

The struts should be reinforced through a layout of FRP disposed in the upper part of the element. The minimum thickness of the layout is 0,0165 mm and the height equal to 50 mm (Fig. 4.6.41).

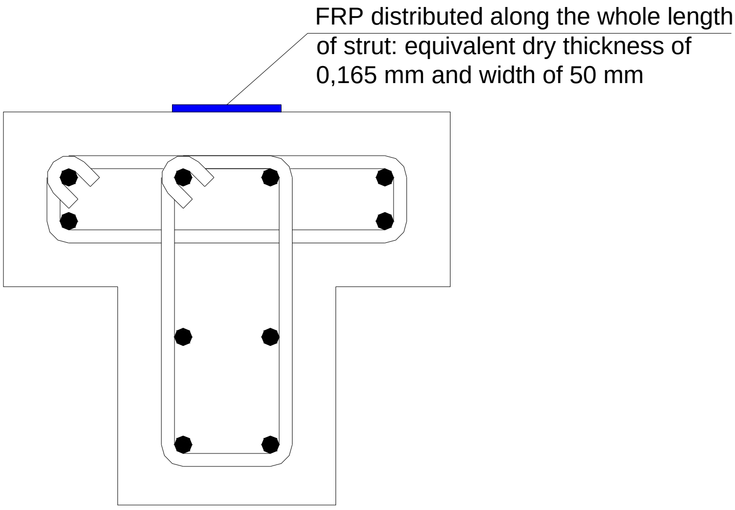


Fig. 4.6.41: Detail of the strut reinforced with FRP

In the ridge joint, to prevent the indent of the fiber, is necessary to level the ridge beam and the struts in order to guarantee a minimum bending radius of 50 mm (Fig. 4.6.42)

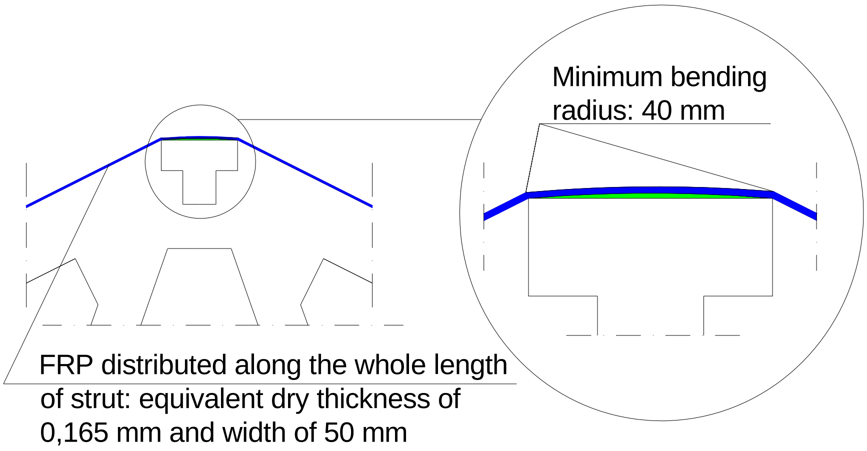


Fig. 4.6.42: Detail of the minimum bending radius (mm)

Tie beam should be reinforced through two layout of fibers disposed for all the length of the element (Fig. 4.6.43).

To avoid the problems of peeling and guarantee the correct behaviour of the FRP is necessary applied the fibers in the correspondence of the decay tie beam-secondary element joints. The FRP should have a equivalent dry thickness of 0,165 mm and a width of 100 mm (Fig. 4.6.44).

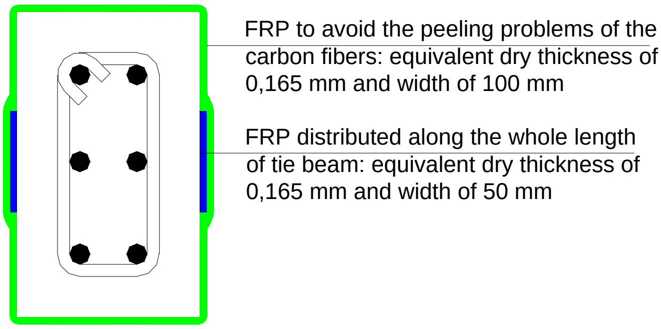


Fig. 4.6.43: Layout of the FRP reinforcement distribution in the tie beam

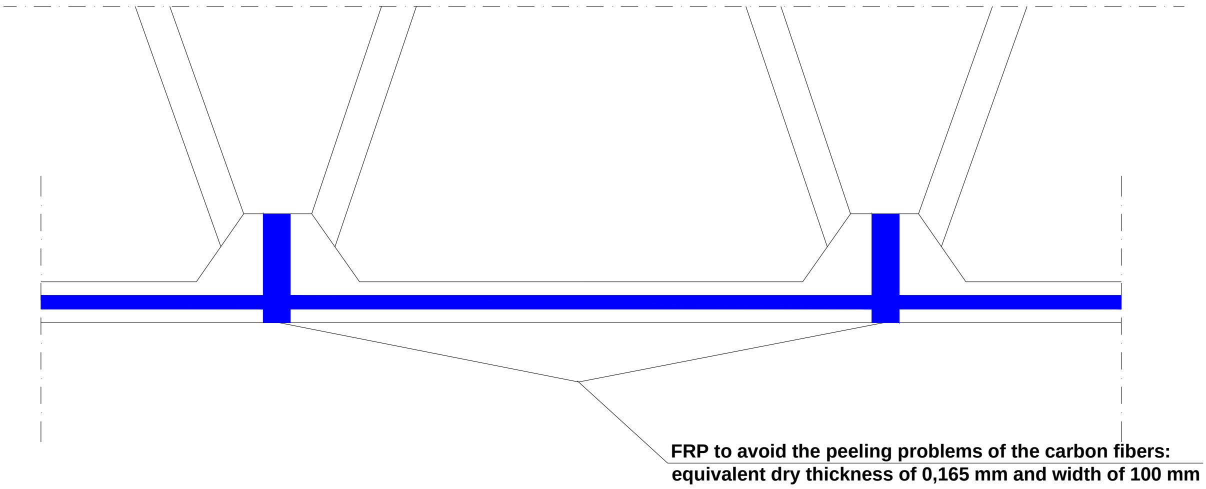


Fig. 4.6.44: Layout of the FRP distribution for the reinforce of the joints

The secondary elements in tension (5-15) must to be reinforced with two fibers disposed for all the length of beams. In the lower part of elements the FRP should be closed by FRP, as reported in Fig. 4.6.45.

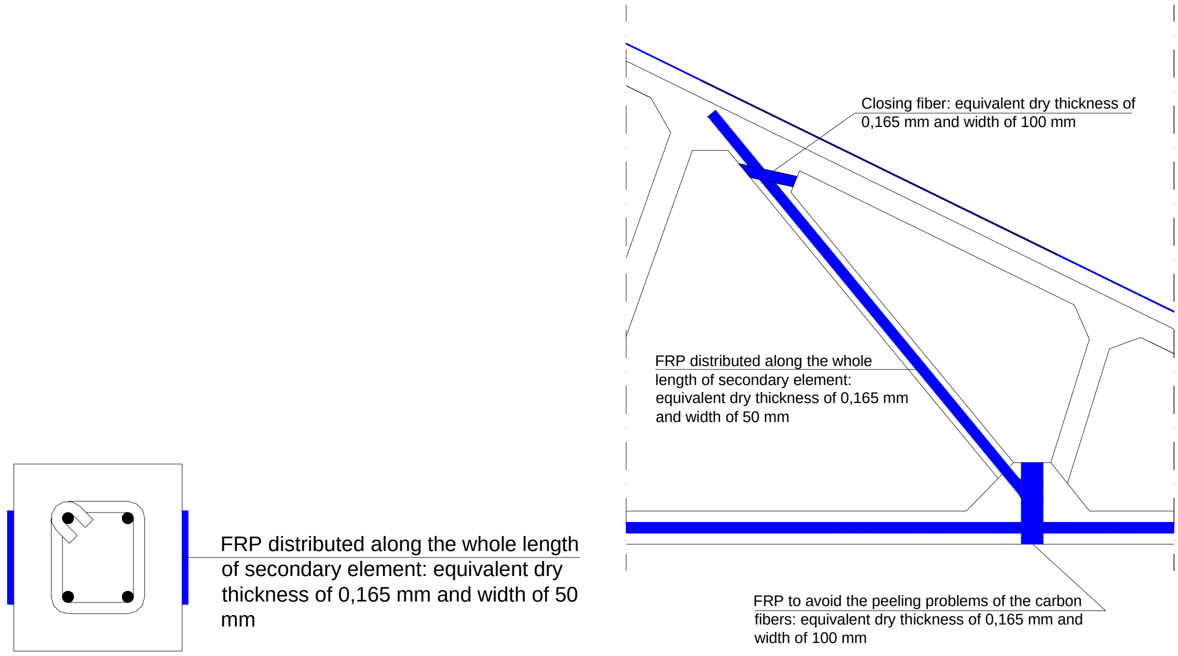


Fig. 4.6.45: Layout of the secondary element in tension (5-15) reinforced with FRP

The fibers applied in the tie beams and in the struts should be anchored to the curb, as showed in Fig. 4.6.46, to guarantee the correct functioning of the intervention.

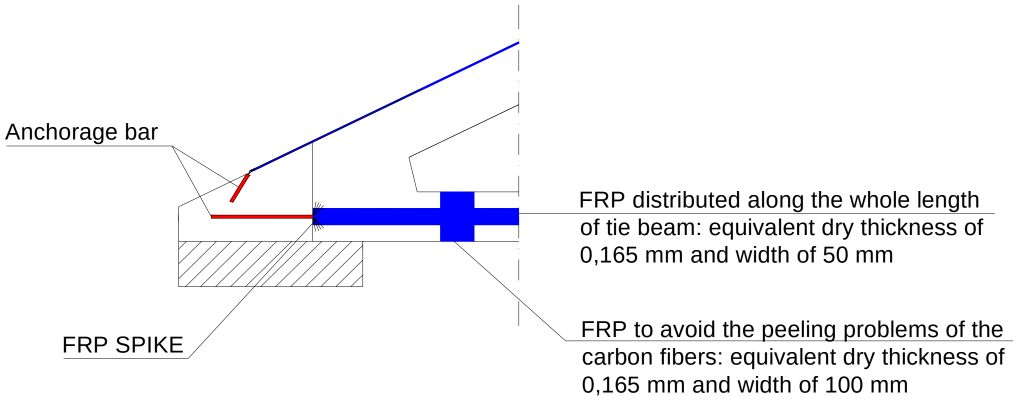


Fig. 4.6.46: Layout of the anchorage of the fiber to the curb

The FRP spike should be constituted by carbon or glass fibers inserted in a polyester net, for an overlapping length of 100 mm. For the intervention on the struts and on the tie beam, the anchorage length will be equal to 200 mm. The diameter of the rod will be of Ø 12 mm and the diameter of the hole equal to1,5 Ø rod (Ø 18 mm in the case into consideration).

<u>Restoration interventions of the reinforced concrete beams</u>

The interventions needed for the restoration of the elements are listed below.

- Removal of portions of detached, loose or otherwise carbonated concrete;
- mechanical treatment of the concrete surface to remove the detached parts, making the surfaces rough and increase the surface area in order to ensure the adherence of the integration material;
- elimination of the remaining powderye parts of concrete;
- cleaning of the metal elements of the frame until the total reduction of oxidation;
- cleaning of the reinforcement bars;
- protective treatment of the reinforcement bars;

- washing with under controlled pressure water for the removal of residues of previous work and the saturation of the concrete to avoid water to invade the integration mixture;
- closure of the lesions and cracks with reoplastic high fluidity mortar;
- reinstatement of the missing section with cement mortar after the establishment of a proper shuttering, prepared with the maximum care to ensure the original iron covers (which for all elements of the truss is approximately 20 mm).

4.7 Final remarks

The methodological approach described in the thesis was applied to the assessment of the *Sala Maggiore* (Main Room) of the Sale d'Armi Nord (Northern Weapons Rooms) of the Venice Arsenale.

The final goal achieved through the evaluation process was the restoration of the building for its successive re-use as museum, respecting its original features in terms of materials and structural configuration.

From the historical analysis related to the structural modifications of the building and from the evaluation of the present day conditions (geometry, structural elements) of the *Sala Maggiore*, emerged the causes which rendered the structure to its current state. For what concerns the bibliographic study, only the relevant information from a structural point of view was considered. For the geometrical description of the structure proved its effectiveness the laser scanner technique.

This first data collection was the starting point for the successive phases of geometrical, material, structural and elements degradation knowledge phase. For the geometry of the floors of the structure the information obtained via the laser scanner technique was not enough. It was then necessary to use traditional techniques to complete and refine the geometrical survey, as distance meter and caliper, in all of the areas of the complex.

The several surveys carried out allowed to identify the structural function of the different elements, and their deterioration. Non destructive on site tests were executed on the reinforced concrete trusses, where possible, to determine the chemical and mechanical characteristics of concrete, and to localize and quantify the reinforcing bars and stirrups. Given the impossibility of reaching all of the areas in safety conditions and the limitation imposed by the geometrical configurations of the structural elements, the investigation did not propose complete information. It was therefore executed, as imposed by the Italian standards on constructions (Technical Standards on Constructions, Ministerial Decree 14/01/2008), the so called "simulated design", which corresponds to the verification of structures by applying the regulations in force at the time of construction of the structure (Royal Decree nr.2229, 16/11/1939). This is a methodology useful for the estimation of the localization and quantification of the reinforcing bars: once given the maximum forces on structural elements (axial force, bending moment…), allowable (maximum) stresses defined by the reference standards are imposed to the reinforcing bars in order to have the possible number and diameter of them, as well as for the stirrups, once given the shear forces.

Collected information allowed to assess the present day conditions of the structure, both from a material degradation and structural point of view, and thus to identify its most critical areas. It was observed that the most critical conservation aspects of the complex are connected to the diffuse decay of the mechanical characteristics of materials, mainly due to lack of maintenance of the buildings in the last decades. Starting from the obtained results of the preliminary phase it was then possible to define an intervention program.

The most important tool used for the evaluation of stresses within the structural elements was structural modeling through FE codes. A comprehensive numerical model was implemented, also considering the influence of the adjacent buildings. Results of the analyses shown that masonry elements are not subjected to excessive stresses. The calculation outcomes, together with the absence of significant crack pattern in the walls or pillars, indicated that specific interventions on them were not requested. Detailed FE models of floors and roofs were then created, allowing the possible local verifications on the wooden and r.c. structural elements.

With reference to the chosen conservation approaches in different part of the building, three different intervention typologies were studied: conservative restoration, reconstruction and structural strengthening. Recently, a remarkable part of the wooden structure of the roof collapsed, due to the material decay of some structural elements. Out of the total number of trusses, just four remained standing. By means of the numerical model, the stress field in the elements of the remaining trusses, in their current state, was calculated, and the structural verifications were executed for the elements subjected to the highest forces.

All of the trusses passed the verifications, and for this reason it was decided to consider for these a conservational restoration approach, which foreseen the conservation of the original material. The connections at the nodes were restored (anchors, metal brackets..), protective coatings were applied to wooden elements and, only if strictly necessary, portions of original material in advanced state of decay were substituted.

The collapsed trusses were substituted by new ones. Following a deep and specific historical research concerning the traditional constructive techniques, a detailed study on the coverings of the Arsenal of Venice buildings constructed in the same period of the *Sala Maggiore*, and the retrieval of some parts of the collapsed trusses, it was possible to repropose the supposedly correct geometry of the original elements.

The reconstruction design was realized by considering the original constructive techniques of the 16th century, for what concerns both the connections between the different elements and the structural details. The prescribed verifications were executed, satisfying the standard requirements. The reconstruction works then started.

The design phase finished considering the intervention needed in the reinforced concrete trusses. As a matter of fact, the preliminary study indicated a diffuse material degradation, which may jeopardize their overall stability (spalling of concrete due to reinforcing bars oxydation). From the numerical models it was possible to define the stresses in the structural elements, and consequently to perform the compulsory standards' verifications. Part of the elements does not satisfy the standard requirements for the Ultimate Limit State. It was then necessary to proceed with a strengthening intervention. Between several possibilities it was chosen a minimal intervention methodology respectful of the original materials, geometry, and possibly reversible. FRP strips were then chosen, employed in order to increase the axial and bending resistance of the inadequate structural elements.

To the present day, financed by the *Soprintendenza B.A.P. di Venezia e Laguna*, the conservative restoration interventions on the wooden roofs (reconstruction of the collapsed trusses and conservation of the remaining ones) are finished, and the strengthening interventions on the r.c. trusses are in currently in progress.

Finally, the described methodology was applied to the analyzed case study, and it proved to be a valid tool for the planning of conscious conservative restoration projects, respectful of the original materials and structural configuration.

5 THE ARMSTRONG MITCHELL & Co. HYDRAULIC CRANE OF THE ARSENAL OF VENICE

5.1 Foreword

The big hydraulic crane of the Venice Arsenal was built by the English Company Armstrong, Mitchell & Co. in 1885, on the eastern wharf of the main shipyard. It represents a significant example of innovation in the English engineering of the late 19th century, and it is the last remaining crane of its kind, since all of the other ones built by the same company were demolished. Its uniqueness and the will of not losing the last remaining example of such masterpiece of industrial engineering requires then a restoration intervention, without pretending, however, that the crane may ever go back to its original functioning.

Interdisciplinary approaches were followed, aimed at the achievement of a satisfactory knowledge level on the structure, prior of the definition of any conservation design or provisional strengthening intervention. The first fundamental step was the knowledge of its history and of the interventions carried out on the crane from its construction.

All of its structural elements will be identified and analyzed from a constitutive, geometrical and structural point of view, paying attention to their function, to their connection with the other elements of the structure and to the composing material.

To deeply understand which are the main problems affecting the crane, the material deterioration causes were studied, and the structural assessment of the crane was carried out. All of the gathered information, allowing a clear identification of the different elements composing the structure, are reported in summarizing forms, which represent an important tool both for a quick and plain comprehension of the structural functioning of the crane and for being an evidence of the present day conditions of the crane's structural elements, for possible future studies and interventions.

All of the researches carried out, studies and reasoning led to the definition of a provisional strengthening intervention and subsequent restoration of the crane, passing through tests on materials, geometrical survey and structural modeling of the crane, in successive restoration steps.

The conservation design of the crane is based on both structural and construction material considerations. Structural analysis indicated unsafe conditions for the crane in its present-day state, and so the first operation to do is to provide it with an adequate structural safety factor. The needs of conserving the original material as much as possible should furthermore be compatible with the effectiveness of the necessary interventions, also avoiding significant alteration of the original structural configuration of the crane.

The principal tool for the evaluation of the present day structural conditions of the crane, for the definition of the necessary provisional strengthening interventions, and for the design of the final conservative restoration intervention, was the FE modeling of the structure.

A joint interdisciplinary workgroup was created to face the process, also considered the extent of the project and the different aspects which will be addressed, where public bodies, universities and private companies work together – each of those within its field of application – for the attainment of a satisfactory knowledge level prior of the definition of the conservative intervention. The final goal of the interventions will be to bring back the crane to its original configuration, both in terms of material and from a structural point of view, to keep alive the evidence of the shipbuilding industry evolution at the end of the 19th century, as well as for its fruition by the visitors of the Venice Arsenal.

5.2 Historical backgrounds

During the nineteenth century the deepest and most radical transformation of shipbuilding took place, from the origins of sailing to the present. The use of wood for the construction of the hull was given up for iron, and propulsion wind in favor of motors determined, in a remarkably short period of time, enormous changes in the features of boats. The pressing succession of significant developments in the field of armaments, moreover, affect the military shipbuilding industry with even more profound effects (Menichelli et. Al., 2006).

The places of production, housing and maintenance of ships, of course, had to adapt to these changes and, therefore, modifications of the boats meant remarkable mutations in shipyards and arsenals as well.

Keeping pace with the technological evolution of the arsenals, artillery and naval construction , the development of lifting machinery proceeded as a direct consequence. The firm founded by William George Armstrong was one of the main character of this revolution.

William George Armstrong started his research on applications of the hydraulic lifting equipment in the second half of the '30s, developing it during the subsequent years, along with his partner George Rendel. A preliminary draft of hydraulically operated crane was patented by him in 1846 on the platform of its plant in Newcastle; the crane was operated thanks to the hydraulic-power of the of the river running in the homonymous town. Armstrong subsequently applied their model to small and medium-sized cranes up to 30 tons crane. This meant a significant success, first in England and then in Europe.

The first major changes in its hydraulic cranes came with the introduction of the hydraulic accumulator, which made it possible to store a certain amount of fluid at constant pressure. In the early '70s, all the English ports were provided with hydraulic accumulator cranes. During those years new researches were performed, in order to produce large-scale cranes provided with swivel boom fixed to meet the largest needs of shipbuilding. These cranes, featuring load capacities between 100 and 160 tons, were characterized by a masonry basement of considerable thickness and stiffness, provided with a circular track of larger diameter, an boom of the reticular essential geometry and rigid, vertically by a counterweight to maximum distance from the crane and a lifting mechanism that used a large hydraulic cylinder, designed by George Rendel (Menichelli et. al., 2006).

Nine copies of cranes with such features were produced, between 1876 and 1905, and were installed in some of the most important arsenals in the world. The first was installed in La Spezia in 1876, the others were set up in Bombay in 1877, in Liverpool in 1881, in Malta in 1883, in Taranto and Venice in 1885, in Pozzuoli in 1887 and two in Japan in 1892 and 1905. The cranes of Bombay, Liverpool and one of the two installed in Japan had a capacity of 100 tons, while for all other specimens the capacity was 160 tons. All the documents found out so far during researches, mostly performed in Italian archives, mainly concern the crane installed in the Mediterranean ports. Nothing is known of the cranes installed in Japan, and there are few reports of other samples. However, it is known that most of them were destroyed during the Second World War, while those installed in La Spezia and Taranto were dismantled in recent times, respectively in 1969 and 1992. This is the result of many circumstances, mainly related to the functional needs of the shipyards, but also - especially as far as the last two demolitions in order of time regards - this is a result of the little attention paid during the recent last decades for the heritage of industrial culture.

The only surviving example is the Venetian one, and- as recently pointed out Norman Foster[5] - his recovery should be considered not only as a task of particular importance, but as a necessity. The salvation of

[5] "I write to express my extreme concern for the future of the great nineteenth century crane at the Arsenale. This iconic structure is not only aesthetically inseparable from its historic context, but it is a priceless part of the industrial heritage of Venice. It would be an unforgivable act of negligence if the permanence of this icon is not secured for present and future generations - locals and visitors alike." Sir Norman Foster

the crane in Venice due not only to the different historical importance of the site (declared under tutelage as a "Protected site" in 1986), but also to increasing interest, during recent years, towards the preservation and enhancement of the assets of Industrial Archaeology (Bovolenta, 2005).

The crane of La Spezia, the first large-scale of Armstrong, was born as an evolution of 80-ton cranes installed in 1875 in the real arsenal of Woolwich (Fig. 5.2.1), whose structural scheme is incorporated into La Spezia crane.

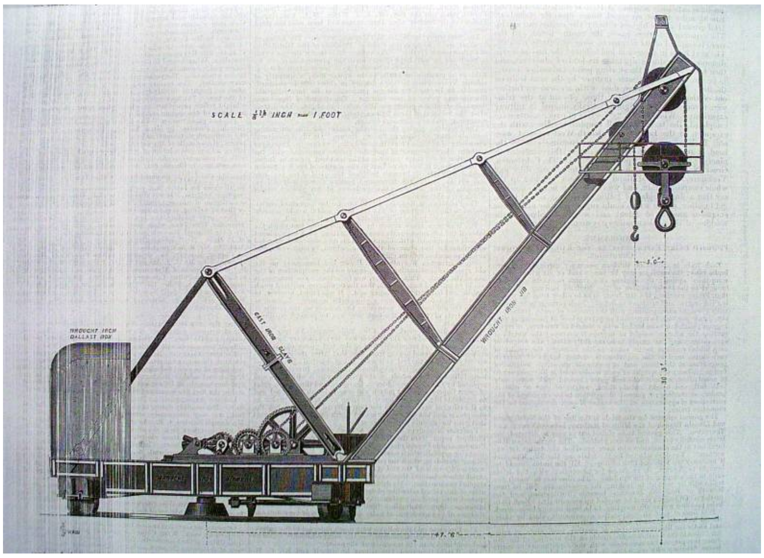


Fig. 5.2.1: 80 tons crane of the Arsenal of Woolwich, 1875 (The Engineer, 1875)

Further developments with respect to the reference model consist of the rotation system on sliding blocks and slewing ring, instead of coupled trucks, and the lifting system operated by an hydraulic cylinder. It was ordered by the Italian Navy in 1875, to move the 100 ton Armstrong guns to be installed in the "Duilio" class battleships - then under construction - and started to work in 1876 (Fig. 5.2.2) (Menichelli et. al., 2006).

All the typical elements of large cranes Armstrong feature also La Spezia one, such as the masonry basement, slewing ring, circular track, sliding blocks, the lattice boom consisting of puddle iron, the large metal box balancing the structure, the lifting system operated by an hydraulic cylinder.

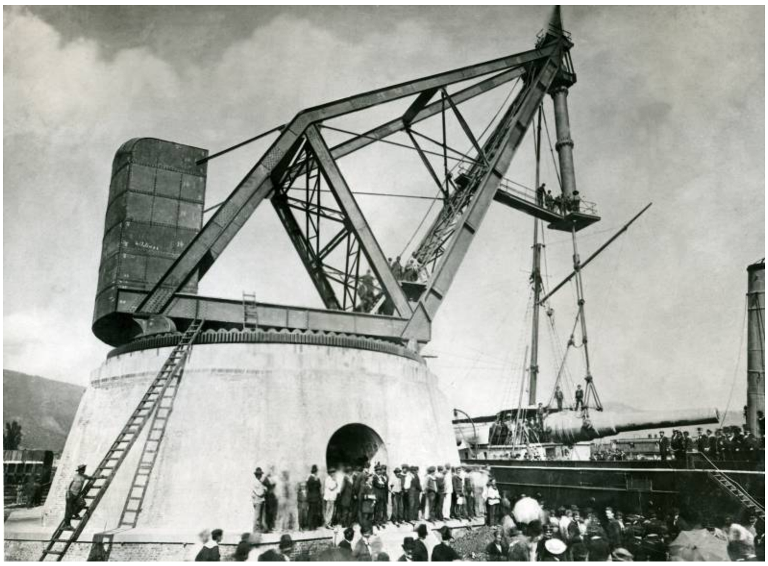


Fig. 5.2.2: Crane of La Spezia during the landing of the 100tons Armstrong cannon from Newcastle, 1876 (Bovolenta, 2005)

The sample installed in Bombay in 1877, 100 tons, had a major technological feature, consisting of the counterweight position, integrated into the network system of the boom as a vertical connection of the two upper tie rods with the corresponding horizontal bars of the platform. This structural change, increasing the compactness and rigidity of the crane, was subsequently used in all large-scale Armstrong crane (Menichelli et. al., 2006).

The 160-ton crane installed in Malta in 1883 (Fig. 5.2.3) represents the evolution and synthesis of the previous Armstrong cranes installed in La Spezia and Bombay and became the model for subsequent implementations, which do not differ substantially from it. The boom opening of Malta crane is 15,70 meters and the crane's height 27,3 m which is to say the longest one. A peculiarity of the crane of Malta, compared to earlier and later works, consists of all the machines were located outside the basement.

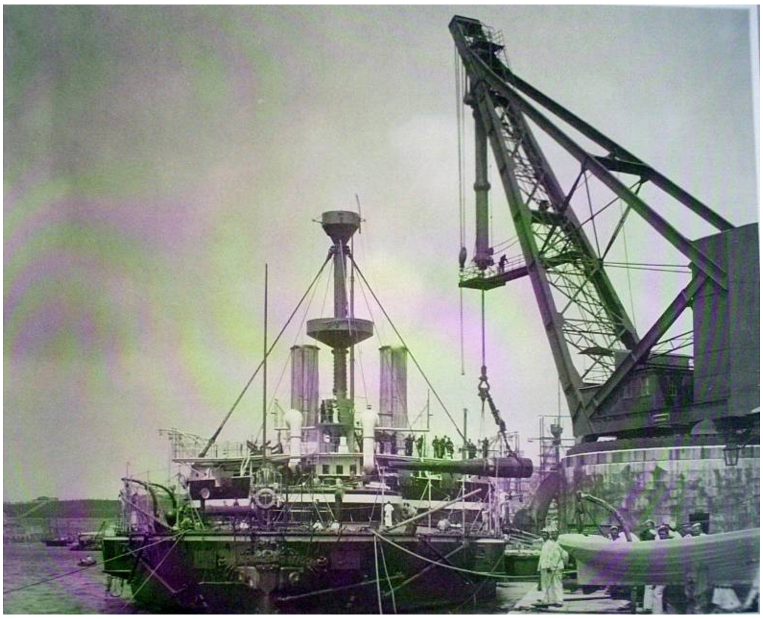


Fig. 5.2.3: The crane of Malta during the loading of a 67tons cannon on the ship Trafalgar, 1896 (B. Warlow e R.Ellis, 1989)

The crane installed in Taranto in 1885 (Fig. 5.2.4) is essentially identical to the Venice one.

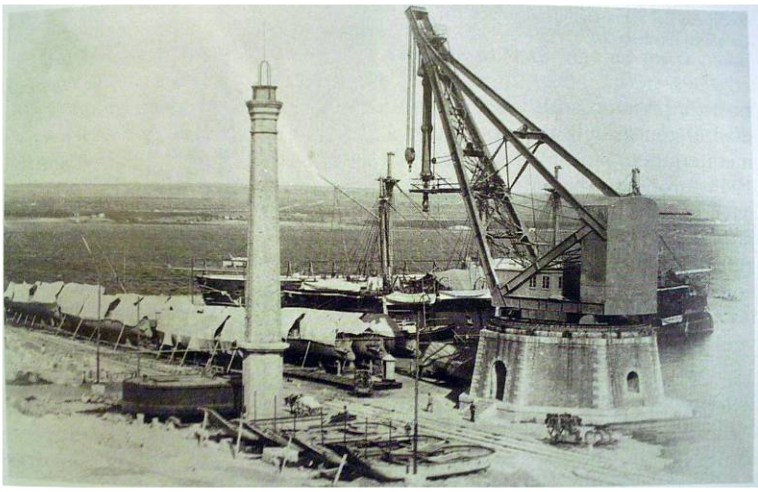


Fig. 5.2.4: The western bank of the Arsenal of Taranto and the 160tons hydraulic crane, 1894 (Rivista Marittima, 1966, vol IV)

The Arsenal of Venice undergone a modernization process: the adjustment of spaces and buildings was supported by the upgrading of equipment and machinery, installed inside of the buildings and on outdoor working areas. Lifting equipment were in fact provided: bridge cranes in the working spaces and warehouses and cranes of various sizes on the docks. Three of these old cranes still exist: a small one featuring an oblique boom located in the Nappe area (Fig. 5.2.5), the Fairbairn "gooseneck" 35 ton (Fig. 5.2.5) and the 160 tons Armstrong located in the dock large (Fig. 5.2.6) (Menichelli et. al., 2006).


Fig. 5.2.5: The oblique boom crane in the Nappe area and the 35 tons Fairbain goosneck crane on the Artillery zone

Fig. 5.2.6: The Armstrong, Mitchell & Co. hydraulic crane in the Artillery dock of the Great Basin of the Arsenal of Venice

When the construction of the battleship "Francesco Morosini", the first major warship in Venice provided with metal hull, was undertaken in 1881, a large scale crane was needed in Venice Arsenal.

In fact, four breech-loading guns Armstrong 106 ton armored tilt up towers had to be placed on the battleship. These had to be mounted in blocks, with precision and stability of operation (Bovolenta, 2005). In any case, this choice was part of the overall program of modernization of the Italian arsenals (was already implemented in La Spezia and was about to implement it also in Taranto) and, in some way, was part of trade agreements of the Italian Navy with Armstrong for the supply of guns, agreements that gave birth in 1886 in Pozzuoli, of a subsidiary branch of the British firm.

In 1883, the crane was commissioned directly by the Navy at the Armstrong, Mitchell & Co., of Newcastle-upon-Tyne and it was decided to install it on the eastern shore of the Grande Darsena (Great

Basin), where Fairbairn crane was already installed, placed in a more central position and closer to the *Porta Nuova* (Menichelli et. al., 2006).

Before the installation, the wall side of the dock – still not yet finished – was completed, in order to build the masonry work on which the crane was to be established. On the masonry basement consisting of an octagonal tower had to be placed the crane. It became necessary to demolish a section of the dock about 20 meters. The foundation works for the crane were very complex, as reported by Felice Martini in detail. The wood-basement for the foundations were placed at level -10 m. The wood basement consists of an octagonal crown of wood poles, 3,6 m long, filled with others wooden poles 2,6 m long. The foundation bedding was thrown into an octagonal pit which surface was 460 m^2 and thickness of 2,30 m. Then the brick masonry and Istrian stone basement was built. Work proceeded with the installation of structures and lifting equipment and, from 1885, the crane was able to begin to perform the important functions that had been assigned to. (Fig. 5.2.7) (Bovolenta, 2005).

Fig. 5.2.7: The Armstrong crane during the installation of a boiler on a ship (Bovolenta, 2005)

Around 1902, after the installation of the electrical power plant at the Arsenal, the crane operated using electricity, and many of its devices were modified.

The crane worked for about 30 years, until the outbreak of First World War, when it suffered the first damages. Since 1940, the hydraulic lift cylinder was already out of use. Afterwards it continued to work with the auxiliary hoist, damaged in 1946, but immediately repaired, and during the 50s, ceases to operate (Fig. 5.2.8) (Menichelli et. Al. 2006).

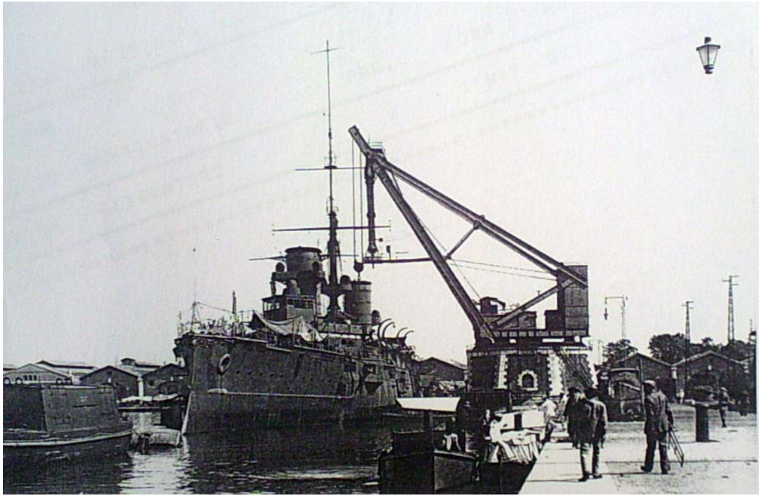


Fig. 5.2.8: The crane in function while loading a tank on the Varese cruiser back in 1916, by the mean of the auxiliary hoist (Bovolenta, 2005)

Currently, a conservation project for the crane - supervised by the Superintendence of Venice and financed by "Venice in Peril Fund" is under development. Crane Armstrong, Mitchell & Co. is unique not only because it is the only model surviving model, and a legacy of British engineering in the nineteenth century but also because it is an important part of the heritage of the city (Fig. 5.2.9).

Fig. 5.2.9: The Armstrong, Mitchell & Co. Crane inside the Arsenal of Venice

5.3 Present – day conditions

The first step in order to draw up a plan for the crane restoration is the knowledge of its status quo, which is to say, the identification and analysis of all elements of the structure.

The structure of the Armstrong. Mitchell & Co. crane can be divided into two well-defined and recognizable areas, consisting of the fixed lower stone basement made of Istria stone and brick and slewing ring and the movable upper part, consisting of the lattice boom equipped with the lifting device and counterweight (Fig. 5.3.1).

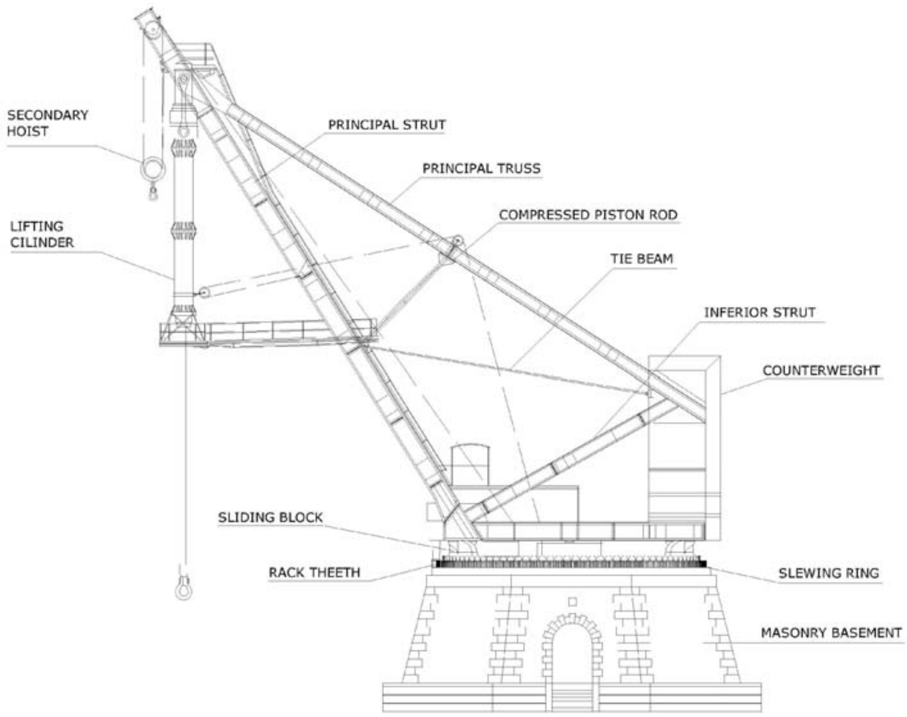


Fig. 5.3.1: Main elements of the Armstrong, Mitchell & Co. crane

5.3.1 The structure of the basement

The basement is built of masonry and Istria stone, shaped as a truncated pyramid featuring an octagonal basement (Fig. 5.3.2). The walls of the truncated pyramid are tilted about 10 degrees on the water level, except the one used to facilitate loading and unloading ships. On four of the eight walls there are three windows and the door giving access the dock. Two symmetrical Istrian stone stairs allow the access to the underside of the basement, completely covered in blocks of Istrian stone. Two iron stairs lead to the top of the basement, surrounded by an iron railing.

Fig. 5.3.2: The masonry basement of the crane

The upper part of the basement ends in a large circular platform with the higher profile slightly inclined in the concrete to allow the water drain (Fig. 5.3.3).

Fig. 5.3.3: The stone ashlar on the top of the basement

Within the walls there is a cylindrical chamber where the steam engine is installed. The steam engine empowered the hydraulic horns with its boiler, for the operation of mechanical parts.

The roofing system of the engine room consists of vault - diameter of 10,30 m – allowing the revolution of the devices, lowered, defined by Martini like a "cup" – 0,90 meters deep – on whose surface of there are four circular openings, running up the building mass. Through the openings escape the fumes (Fig. 5.3.4).

Fig. 5.3.4: The circular openings on the top of the basement

The central cap of the vault, with a diameter of 3 feet, is occupied by a massive stone block grant and, at the centre, is crossed by a metal pivot with an hollow section, allowing the passage of the pipes of supply of machines (Fig. 5.3.4). The pin is attached to the key stone of the vault by four long bolts with axes inclined to the vertical axis of rotation. The block emerges on the top surface (extradoss 0.70 meters) with a cylindrical projection; the frame metal collar bearing support of the crane is anchored to the outline of the cylindrical projection.

Fig. 5.3.5: The boiler

The Cornish boiler type (Fig. 5.3.5), with a single furnace of size approximately 1,50 x 3,60 meters, is installed on two trucks in cast iron. Outside, the chimney through which to escape the fumes produced by the boiler through the pipe underground is no longer present.

The adaptations of machine, switching from steam to electric motors, and the parts of it replaced are clearly recognizable (Fig. 5.3.6).

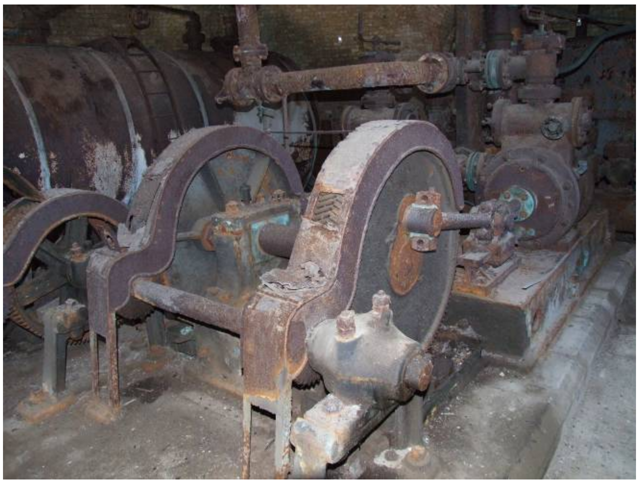



Fig. 5.3.6: The bronze parts replaced over time

5.3.2 The slewing ring

The slewing ring consists of a large rack teeth with a diameter of about 13 meters, with cast-iron elements, on which is installed the pinion gears that allows the rotation, and a system of 96 roller bearings of cast iron (Fig. 5.3.7).

The axes of rotation of the rollers are connected outside two by two, with overlapping metal strips. On the internal side the same axle ends are attached to a ring of steel-structured body, connected by rods, made of steel plates and L profiles and with a radial structure, to a metal collar concentric to the pivot pin of the crane (Fig. 5.3.7). The halo is divided into 16 "slices", each of which is the basic module, divided according to the structure of the slewing ring. Each "slice" includes: 6 rollers, a water drain hole on the plate of the counterweight, a connecting plate of the radial rods (major and minor) to the counterweight, a piece of the outer crown surface of cast iron.

Fig. 5.3.7: Ring gear, rollers and sheet ring of steel-structured body

5.3.3 The upper structure

The circular track platform (Fig. 5.3.8) is the part located at the basement of the boom that supports the counterweight.

Fig. 5.3.8: The circular track

Its structure consists of an array of beams that form a rectangle about 13 x 7 meters. It supports of the pinion tilt (Fig. 5.3.9) and the four large cast iron blocks, the length of 2,5 meters and a height of 0,80 m, allowing the rollers rotation system. On the back is connected to the counterweight box that “closes” the structure.

Fig. 5.3.9: The pinion tilt

The horizontal rectangular frame, which in fact is the platform, consists of two parallel longitudinal beams, characterized by a double T-section, consisting, itself, of a vertical profile plate, and two lateral “L” shaped upper and lower. The beam is 0,92 meters high and, at the flanges, is 0,24 meters deep. The wings are placed at a distance of 6,50 meters and connected by a series of angled rods. The beam is 0,92 meters high and, at the flanges, is 0,24 meters deep. The flanges are placed at a distance of 6,50 meters apart and connected by a series of orthogonal rods.

To reinforce the structure there are, internally, six C cross-beams nailed with triangular and L profiles to the longitudinal beams (Fig. 5.3.10).


Fig. 5.3.10: The reinforcements beams of the horizontal frame

Below the platform, in the front section, there is a metal large box where is deposited in the lifting chain used by the auxiliary hoist (Fig. 5.3.11).


Fig. 5.3.11: Box to contain the lifting chain used by the secondary hoist

The boom (Fig. 5.3.12) is shaped like a lattice pyramid and consists of large puddled iron beams with metal plates, angular and "L" shaped profiles attached with hot rivets.

The boom consists of these main elements:

- lower strut;
- main strut;
- main truss;
- compressed piston rods.


Fig. 5.3.12: The reticular boom showing its lattice pyramid shape

The lower struts consist of two parallel beams connecting the struts and tie beams of the main structure, installed on the counterweight and ending at the top front of the platform. They have a nailed double I-sections, which an height of 0,82 meters.

The section of the beams consists to vertical flat profile, four L-profiles (with a side about 10 cm long) distributed two by two on the side and two horizontal plates 0,34 meters long which enclose the double I-section at the top and at the bottom. The strut "spatial" is stiffened in its upper half, closer to the counterweight, with a lattice structure made, probably, using different L-coupled profiles, forming a I-section for the main beams and using flat profiles for the stiffening rods (Fig. 5.3.13).


Fig. 5.3.13: The lower strut and the stiffening system

The main struts are composed of two rods, starting from the top front of the rectangular platform, converging toward the center, in their vertical development, up to a distance of 2,4 meters from the two trusses in the top. It is the main compressed element of the lattice boom. (Fig. 5.3.14). The double I-section of the two girders, about 1 meter high, has a composition similar to that of the rods just described (central vertical profile and L side, terminating at both ends with two upper and lower horizontal sections 46 cm long) with four additional L profiles installed at each end. The lattice beam is divided into five areas consisting of connection crossed and nailed.

Fig. 5.3.14: The lattice boom of the crane showing its pyramid structure

The bracing beams (Fig. 5.3.15), unlike the other beams, are made with I-section profiles and stiffened with plates installed to form a cross section. They are connected to the main struts through a large plate riveted on the top and bottom. The connection of the two rods between the third and the fourth area, where the chain is place, is the only one obtained using a full double I-section beam.

Fig. 5.3.15: The bracing beams stiffening the two main struts

At the height of the node - where the connection with the two main trusses takes place - (Fig. 5.3.16), the two beams reducing their sections (becomes C-section) and develop in parallel to extreme close up to the top of the crane.

Fig. 5.3.16: Upper part of the main struts

The rods of the main trusses (Fig. 5.3.17) are devoid of a stiffening beams, not needed to support traction stresses the item should be adressed. The beam consists of two rods converging double T-section, consisting of a vertical flat profile (about 0,60 meters high) and four L-shaped profiles that create the flanges with two horizontal profiles with a length just under 20 centimeters.

The strained element is composed by the two beams. The only connection between the two beams is created through a double I-section element, at about half their length, where there are connecting rod and the lifting chain. This element is slimmer of the previous ones, and performs a structural function that best suits the characteristics of the material.

Fig. 5.3.17: Main tie beams

The compressed connecting rods (Fig. 5.3.18), located in an intermediate position of the structure, are considered elements stiffening the frame of the crane. The special shape of the element has been designed with an elegant form that fully comply to the structural function assigned to it. The connecting rod is made of four metal sheets with a slightly curved "L" profile crossing two by two on the triangular and trapezoidal plates in correspondance to the nodes.

Fig. 5.3.18: Connecting rods

Two bracing cross profiles, installed in the space between the two connecting rods, realize the system interconnection.

On both sides of the structure, at the top of the counterweight, it is possible to find two metal rods connected to the main strut. The function of these two elements was essential during the lifting of loads, to balance the crane (Fig. 5.3.19).

(a)

(b)

Fig. 5.3.19: (a)Detail of the joint between the tie beam and the caisson; (b) detail of the joint internal part of the tie beam

The counterweight (Fig. 5.3.20) is a big metal caisson - 8 meters high, 3,50 meters deep- filled with stones and iron with a total weight of 400 tons. The counterweight is not only a balance weight element but it also represents the closing element of the lattice boom. This particular represents a feature of the most advanced Armstrong crane technology, introduced for the first time with the crane located in Bombay in 1877.

Fig. 5.3.20: The counterweight

The caisson is consituted by plates with a thickness got smaller and smaller as you go up in height. The plates, as indeed the whole structure of the crane, are connected together by means of hot rivets. Inside there is a metal bracing system. In correspondence with the bracing crosses, metal plates are installed on the outer in order to provide greater stiffness to the most stressed areas (Fig. 5.3.21).

Fig. 5.3.21: The external strengthening system of the counterweight

Historical research has shown that the lower part of the element, up to the height of 1 meter, features seven metal beams that contribute to the stability of structure and create a system composed of "big boxes", and acting on the sand at the basement of the caisson, equalizing the load distribution (Fig. 5.3.22). Access to the inside part is given is by a manhole located at the top, closed by two bolted plates.

Fig. 5.3.22: Detail of the closing profiles on the bottom of the counterweight

The inspection holes allowed us to identify precisely the materials inside. The upper part is filled with trachyte and sand while the lower area with iron and sand.

The hydraulic cylinder (Fig. 5.3.23) - 35 tons - has a height of 14 meters with a diameter of 90 cm. It consists of four distinct elements connected by flanges and is suspended from the frame of the crane through two pairs of tie beams. This allows a certain possibility of movement along the main axis of the crane.

The cylinder of the lift system is still hanging in its proper position, but two important elements were lost: the two rigid pipes that fed the main cylinder, as well as most of the corner joints attached to the cylinder, and the lift rod of the cylinder, already absent in the photographs of the crane in 1916.

Fig. 5.3.23: The hydraulic cylinder

The auxiliary hoist (Fig. 5.3.24) is a traditional lift, operated by steam engines, for loads up to 40 tons.

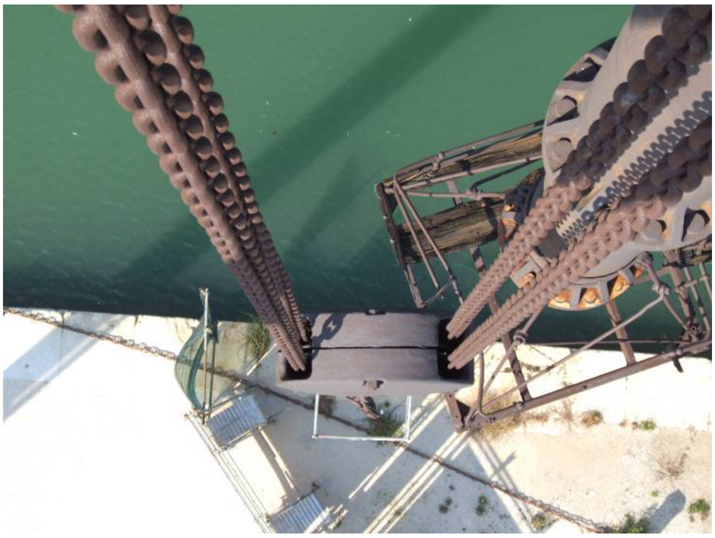



Fig. 5.3.24: The auxiliary hoist

The lifting height of the hydraulic cylinder is 18 meters, while the auxiliary hoist is 30 meters. The overall height of the crane from the floor of the dock is about 36 meters.

At the bottom of the cylinder there is only the iron structure of the platform that allowed to control the inlet and outlet valves. It was possible to access to it through an hinged steel bridge. This consists of wooden planks, installed in a plan surface and of a structure consisting, itself, of two C-section rods (Fig. 5.3.25).

Fig. 5.3.25: The brigde leading to the bottom part of the cylinder

On the platform rests a thick rectangular table on which it is possible to find the brake to stop the movement of the crane, the cylinders of the winding gear chain and other items used to operate the crane. Above it is possible to find what remains of the wooden cabin with a puddled iron structure from which the crane and the lifting cylinder were controlled (Fig. 5.3.26).

Fig. 5.3.26: The cabin and the gears needed for the crane’s operation

5.4 The knowledge phase

In order to design the activities to be performed for the safety, recovery and preservation of the crane it was of paramount importance the geometrical / material / structural knowledge of all the elemnts of the structure, as well as the current state of decay .

The techniques used to achieve this result have been different and allowed the creation of a geometrical survey and a detailed three-dimensional model, the geometric and structural characterization of all the beams and nodes connecting the crane, the chemical composition of metallic materials, and determining the stability of the structure during the phase of knowledge.

It was planned and carried out a major survey aimed to the reconstruction of the precise geometry of the crane, since in the past was not implemented any complete survey of the structure and in the literature were not find detailed information. The preliminary stage consisted in the arranging a series of operations to trace all the necessary measures and evaluation the best suited technical methodologies to achieve a rigorous and comprehensive final result.

Therefore, were carried out the photogrammetry, laser scanning and direct surveys. The elaboration and the union of the data led to the geometric reconstruction through planimetry, plan and elevations, and to creating a three dimensional model of the entire structure.

The multiplicity of techniques provides not only the fundamental representation of the crane but also, and above all, detailed elements regarding the construction methods used with regard to the geometric aspects. These are basic to assess the consistency of the crane materials with particular reference to the complex issues of conservation of metal structures and machines. In fact, laser scanning and photogrammetry allow to identify the three-dimensional coordinates used to build the graphic: the two techniques can be alternative or complementary. In this case, photogrammetry and laser scanning were used in a complementary way, with the support of direct survey, and allowed to redrawing almost entirely the crane.

The phases of the geometrical, photographic and macroscopi survey allowed to assess in detail the state of conservation of the constituent elements of the crane. Given materials difference the degradation analysis was performed in different ways for the basement and the lattice boom.

5.4.1 The geometric survey

In order to finalize the project to provide safety condition of the structure and subsequent recovery of the Armstrong Crane, it was necessary a careful measurements that could provide a complete reconstruction of the structure but also highlight the defects and the main points of structural weakness. It was so planned and carried out a project to reconstruct the precise geometry of the crane, since, previously, any complete survey of the structure was never performed, and therefore there was no geometrical survey. Each phase of the survey was carefully analized, considering the problems that could encounter, not only on-site but also during the restitution process. It was arranged a series of operations including all the necessary measures to assess what might be the most appropriate techniques and methodologies to achieve the best outcome.

For the geometrical survey the following techniques were used:

- Topographic survey
- Photogrammetric survey
- Laser scanning survey
- Direct measurement

The topographic survey

The first phase of the relief allowed the establishment of a high-precision topographic network (Fig. 5.4.1) which formed the general framework for all subsequent phases of relief and of representation made by the Centro di Rilievo, Cartografia ed Elaborazione (CIRCE) (Centre for Surveying, Mapping and Processing) of IUAV, University of Venice. The core network is connected to the topographic ones present in the Arsenal so that the cranes could be put in place within the existing measurement campaigns. Secondary and details networks were anchored to the core network, by the alignment of the laser scanner point clouds, thanks also to the support of the photogrammetric models and to the combination with the data of the direct measurement survey.

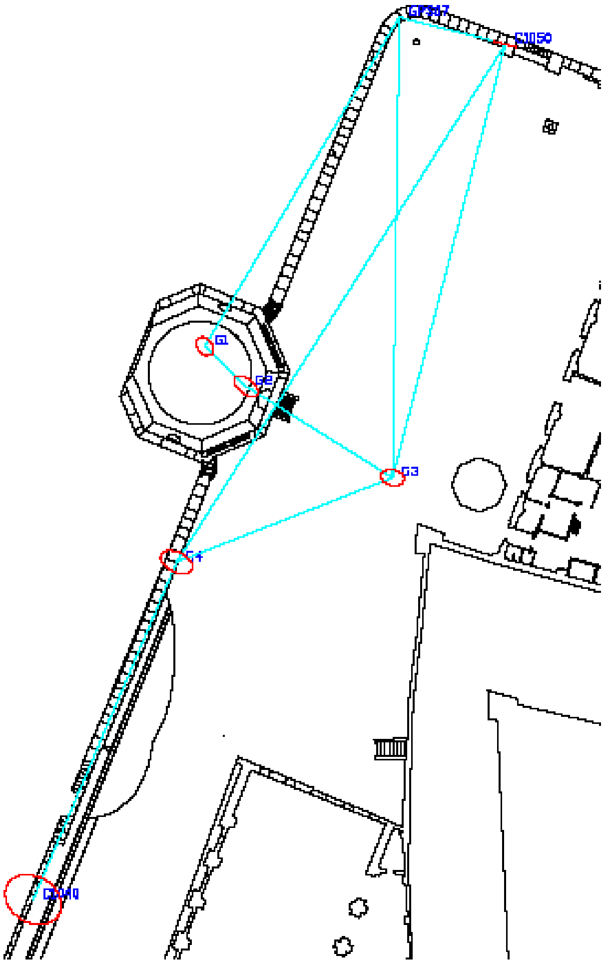


Fig. 5.4.1: Layout of the topographic core network

The photogrammetric survey

The photogrammetric survey, designed with 1:50 scale, has used stereoscopic photogrammetry models at the scale 1:200 with metric camera. For the implementation of the survey an aerial platform was used (Fig. 5.4.2). The models were oriented on signalized or natural points found by topographical surveys.

Fig. 5.4.2: Photogrammetric survey

The laser-scanning survey

A laser scanner survey was made which, along with other techniques, allowed us to build a "geometric" documentation as comprehensive as possible. The clouds of points were georeferenced to the topographic system used. From the clouds of points, the sections and the profiles were extracted, needed for the design and the construction of the model (Fig. 5.4.3).

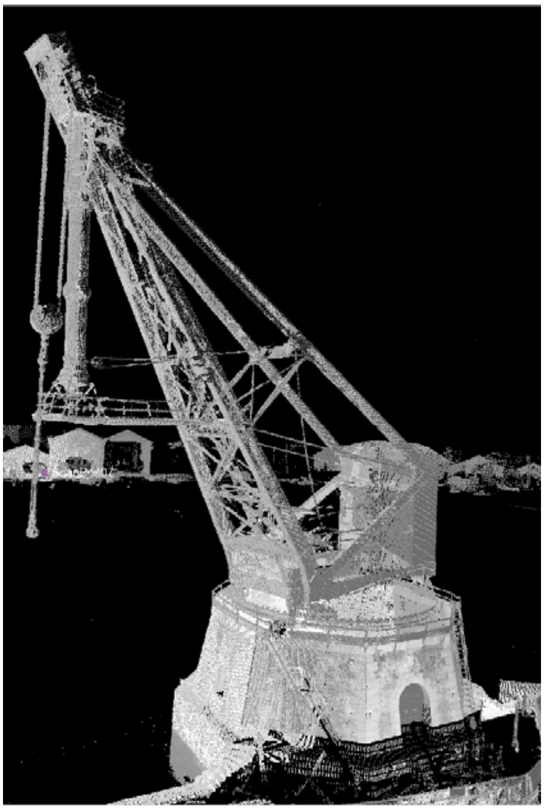


Fig. 5.4.3: Cloud of points obtained aligning the scans taken with the laser scanner

Direct technique

The direct survey was an essential support for the reconstruction of the model of the crane. The instrumental survey can ensure the uniformity of measurements and the overall consistency with the representation scale 1:50. The direct investigation has completed and integrated the photogrammetric and laser scanner surveys, deepening the detail to effectively describe all the aspects of the building, allowing a representation up to a scale of 1:10. It also allowed to perform a detailed photographic survey that was the main support for the degradation analysis.

The aim of the survey was to measure directly the main dimensions, the thickness and the pitchs of rods, joints, profiles, plates and rivets, in order to build a final model of the situation of the crane, including the possible decreases of the section due to degradation. It was also essential to identify the critical points of the structure, both for the degradation and structural point of view, and to promote the study and understanding of each element of the structural system of the crane.

For the purposes of work organization and final knowledge of the crane two summarizing forms have been designed: the first allowed the collection of data on the field during the direct survey and the second one allowed the total understanding of the structure, shape, geometry and size of the elements and their current condition. The direct survey was preceded by a careful study, which will be described in the following paragraph, concerning the nomenclature of all the constituent elements of the crane in order to make the clearest and fastest possible recognition of the elements during the on-site operations.

To carry out the survey calipers have been used for precise measurements, such as thickness of plates and profiles, pitch and diameter of the rivets, and meter and distance meter to measure lengths, such as height, length and distance between the connecting plates of rods and nodes. The photographic survey was also important to provide detailed documentation that serves as a support and a memory for the assessment of the current state of degradation, and the any progress of the decay over the years to come.

The method of survey was mainly directed to perform measurements not possible with other types of tecnique such as photogrammetry and laser scanner:

- the dimensional knowledge of the elements placed on the internal part of the structural nodes;
- the definition of the geometric section of plates and angle bars that make up the crane.

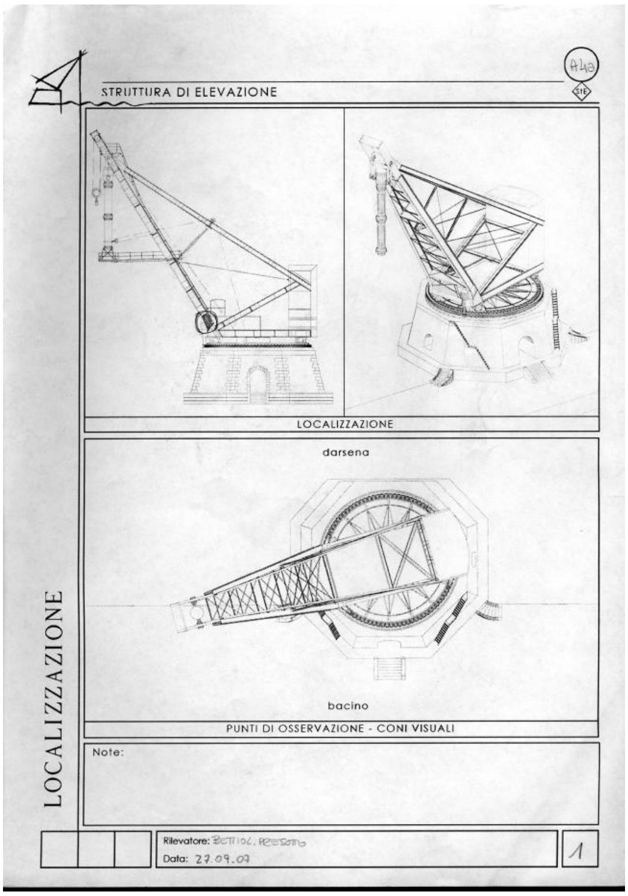
By means of the direct survey was possible to accurately measure the thicknesses and highlight any difference between elements of the same type, due to degradation.

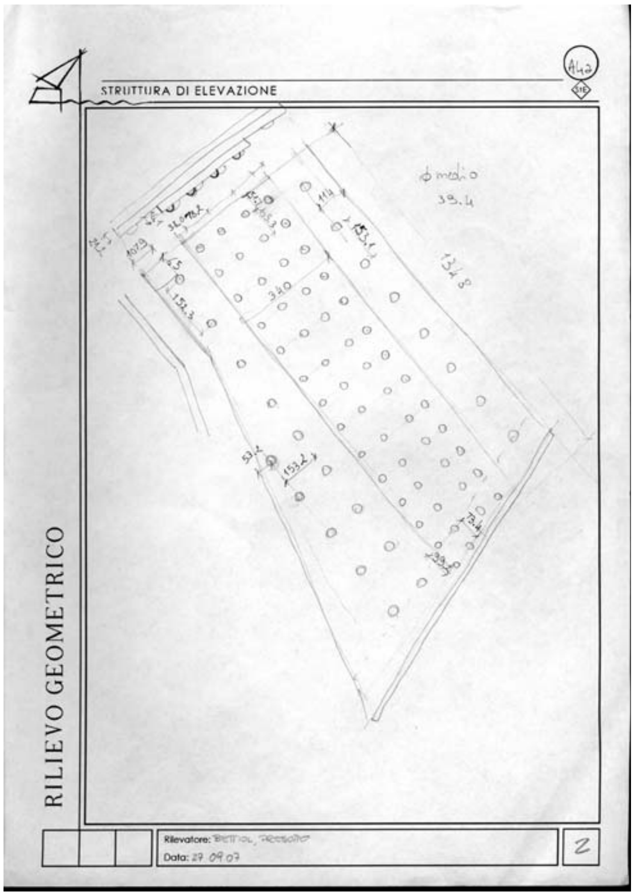
In order to define the geometry of the structure has been carefully surveyed the diameter and the interaction between the rivets on the surface of the crane. This verification was another important stage of the survey because it highlighted potential weaknesses in the structure due to a higher pitch (Fig. 5.4.4).

Fig. 5.4.4: Detail of the rivets pitch

The direct survey of overall heights and lengths of plates and angle rods had lead to the verification of information resulted by the photogrammetric tecnique and the description of elements otherwise difficult to determine.

All the information observed and registered during the measurement campaigns have been carefully recorded and collected in the form of sketches, two-dimensional and three-dimensional ones, to give a better understanding, and descriptive notes of the degradation. Below an example is reported (Fig. 5.4.5).





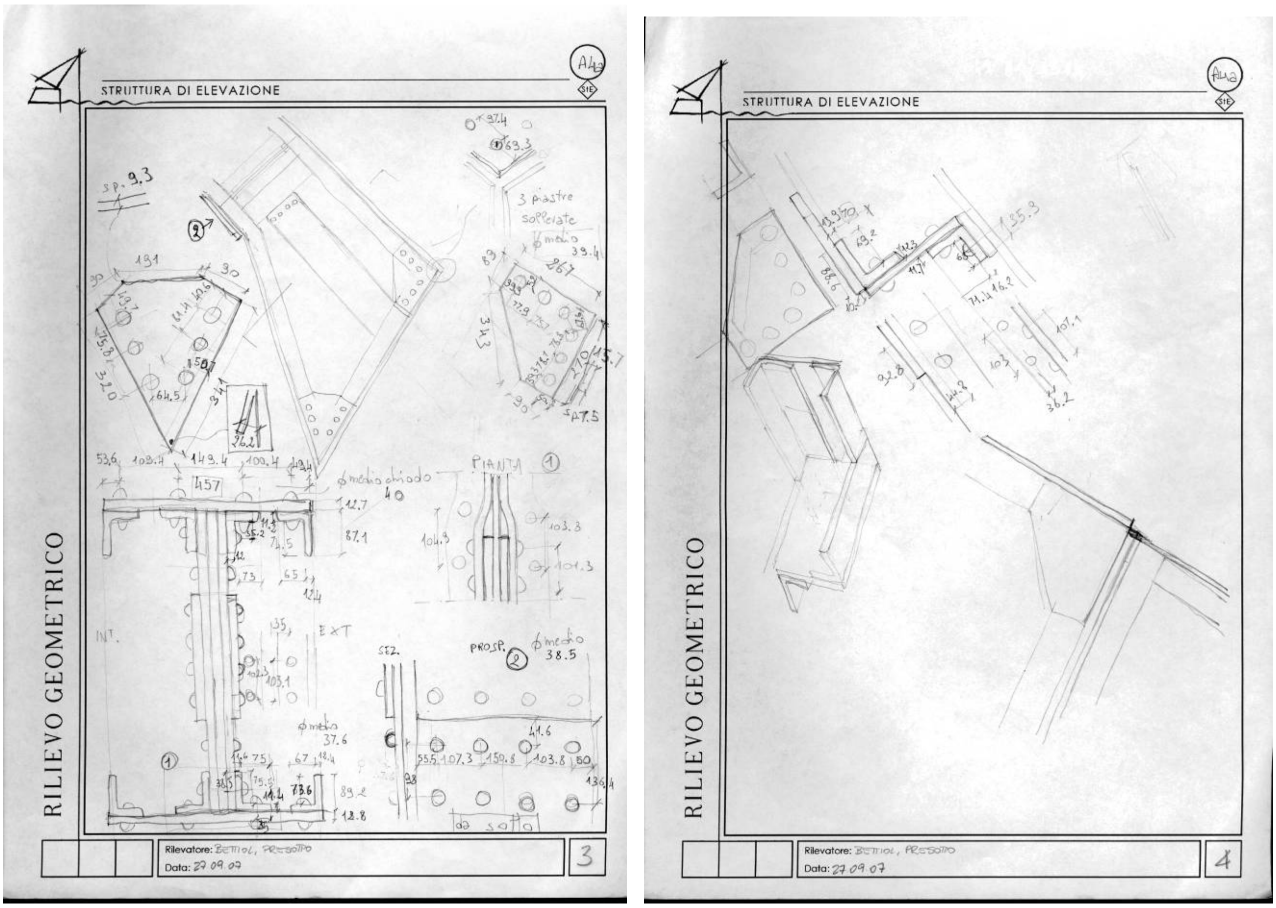


Fig. 5.4.5: Example of summarizing form of the on-site direct survey measuring

The final products of geometric survey

The data obtained from topography, photogrammetry, laser scanning and direct survey were elaborated and they yielded different products.

By integrating all of the methods used a tridimensional global model of the crane was constructed (Fig. 5.4.6).

Fig. 5.4.6: Model rendering (CIRCE)

With the same method, and through programs for data processing and design, the planimetry, plan and elevations have been extrapolated, arranged and rendered as closely as possible to reality (Fig. 5.4.7).

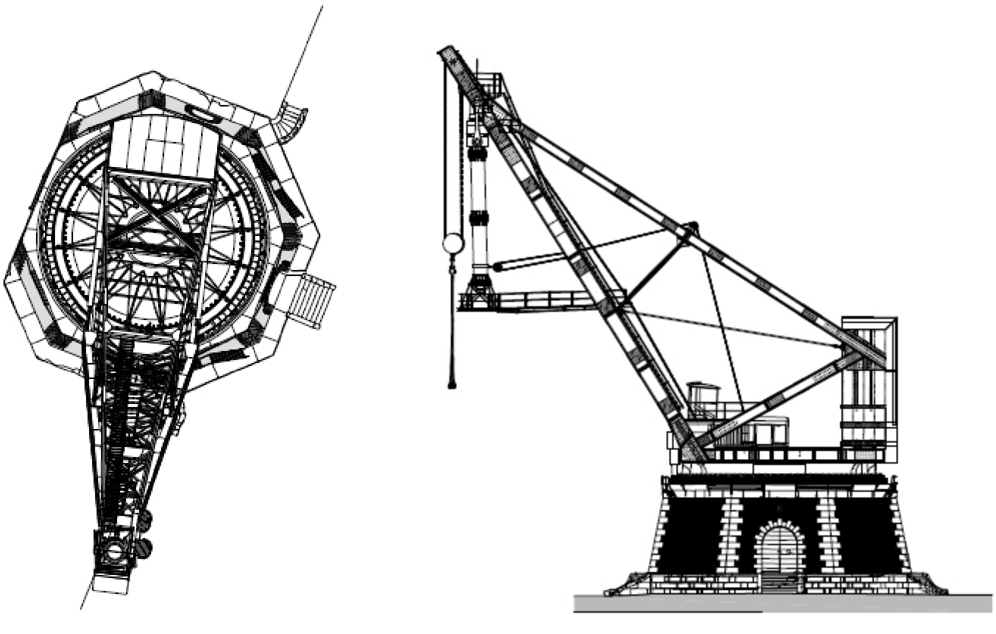


Fig. 5.4.7: Examples of the digital reconstruction of the crane (plant and elevation) (CIRCE)

By the measurements taken during direct survey and their subsequent processing showed that all dimensions are referable to the English system: inches.

A careful historical research was then carried out to discover and retrieve information regarding the types of profiles on the market back in late 1800 in England and the Armstrong Company's historical tables containing the construction details of the structure under consideration.

The most important information found useful in the redesign of the crane are reported below.

During the study of materials such as cast iron and puddle iron, it was essential to have a look at the related innovative construction systems which allowed the construction of architectures unthinkable before their advent. The usage of puddle iron in constructions involved a large number of structural elements, using bars, rods, sections, plates and sheets. In particular in reticular structures, flat and L profiles were individually bracing and connecting rods which, joined together by riveting, made up the double-tee section of the main beams (Fig. 5.4.8).

The usage of puddle iron in constructions has involved a large number of structural elements, such as rods, beams and plates. In particular in reticular structures, plates and L-profiles consituted the bracing and connecting rods which, joined together by riveting, made up the double I-section of the main beams (Fig. 5.4.8).

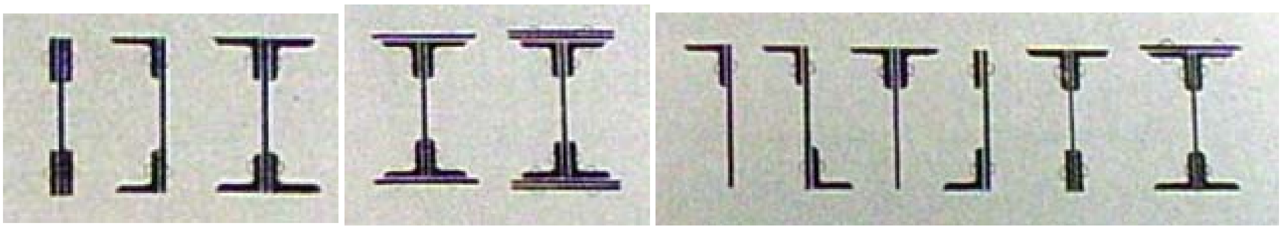


Fig. 5.4.8: Assemblage scheme of the composed sections[6] (Bovolenta, 2005)

Since the mid-'50s of nineteenth-century, the double I-section beam had become common use both in bridges, as a riveted composed beam, and in buildings, as a rolled beam.

This allowed to obtain greater strength and ease of construction compared to the "solid beams", whose maximum height for I-section beams was about 15 cm and for double I-section beams up to 40 cm. In the case of the crane the composed beams (Fig. 5.4.9) reaching an heigth up to a meter.

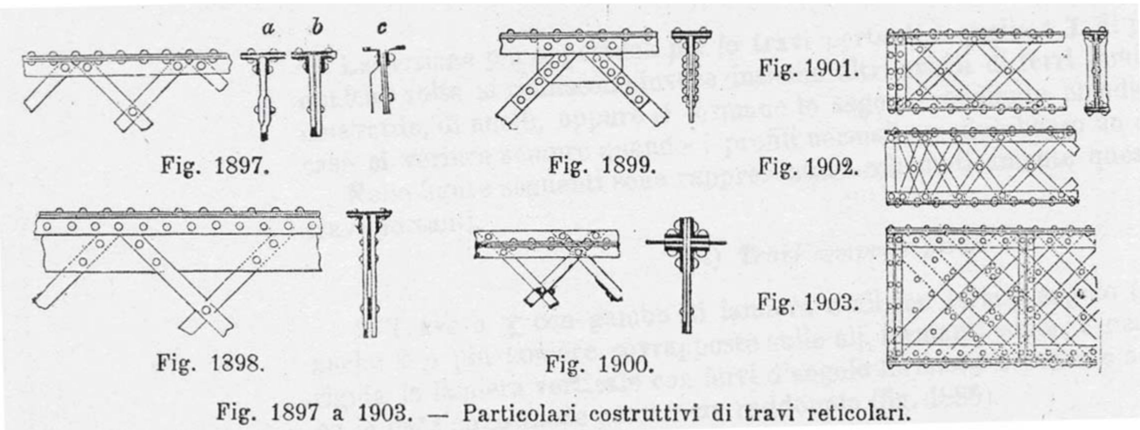


Fig. 5.4.9: Details of the lattice beams

The shaped elements present in Italy in the second half of the 800 was all imported from abroad. The components were made to be installed, just as they were, in the most different yards. It was the case of the crane's elements, supplied by the Armstrong Company, sent by the British shipyards and assembled on site. The carpentry was following production's standards and patterns peculiar United Kingdom.

The fact was proven even by historical documents found. In the English project table number 2919 (Fig. 5.4.10) of the crane of Venice, dated 1884 and preserved in Tyne and Wear Archives, Newcastle upon Tyne,

[6] Pio Chicchi, Corso teorico e pratico sulla costruzione di ponti metallici, Padova, Draghi Negro 1886.

dimensional indications are in English units as well as the entire design that is reproduced at a scale of ¼ inch to 1 foot (corresponding to a nominal scale of 1:48).

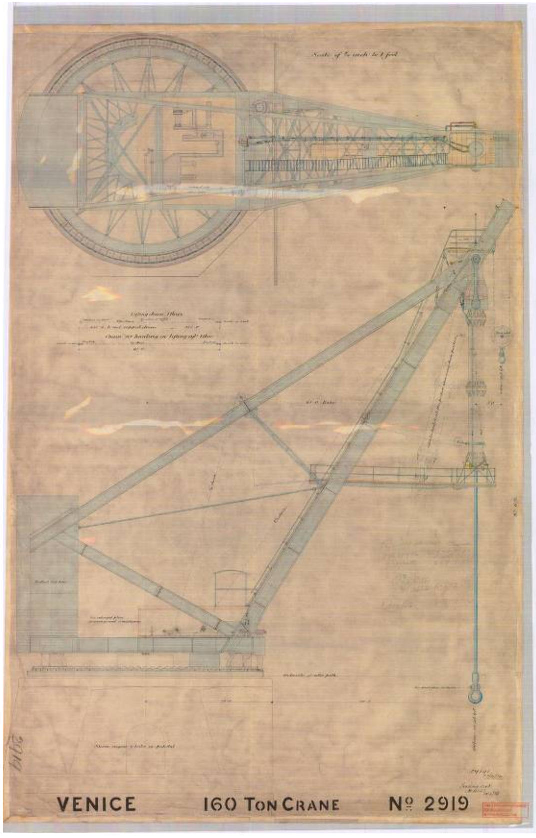


Fig. 5.4.10: Original table of plan and elevation of the Venice crane (1884)

Also in the drawings of the crane of Malta (Fig. 5.4.11) was found in the article of C. Colon, *The 160-Ton Hydraulic Crane at Malta Dockyard Extension Works*, contained in the journal Minutes of Proceedings of the Institution of Civil Engineers year 1876, that the main beams sections of the crane are in inches. The compositions of the beams and their rivets are the same as those in the Venetian crane, although the latter have a proportionately greater height.

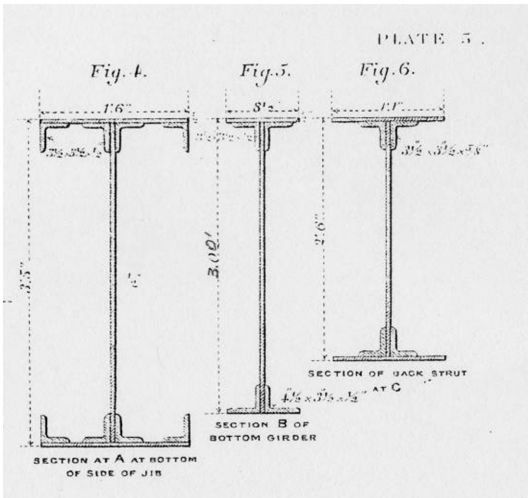


Fig. 5.4.11: Sections of the beams of the crane of Malta: platform, lower and main struts

A comparison between some of the Armstrong company's cranes was carried out (Fig. 5.4.12). Different graphic documentation was found, in particular in Malta, in Venice and in La Spezia.

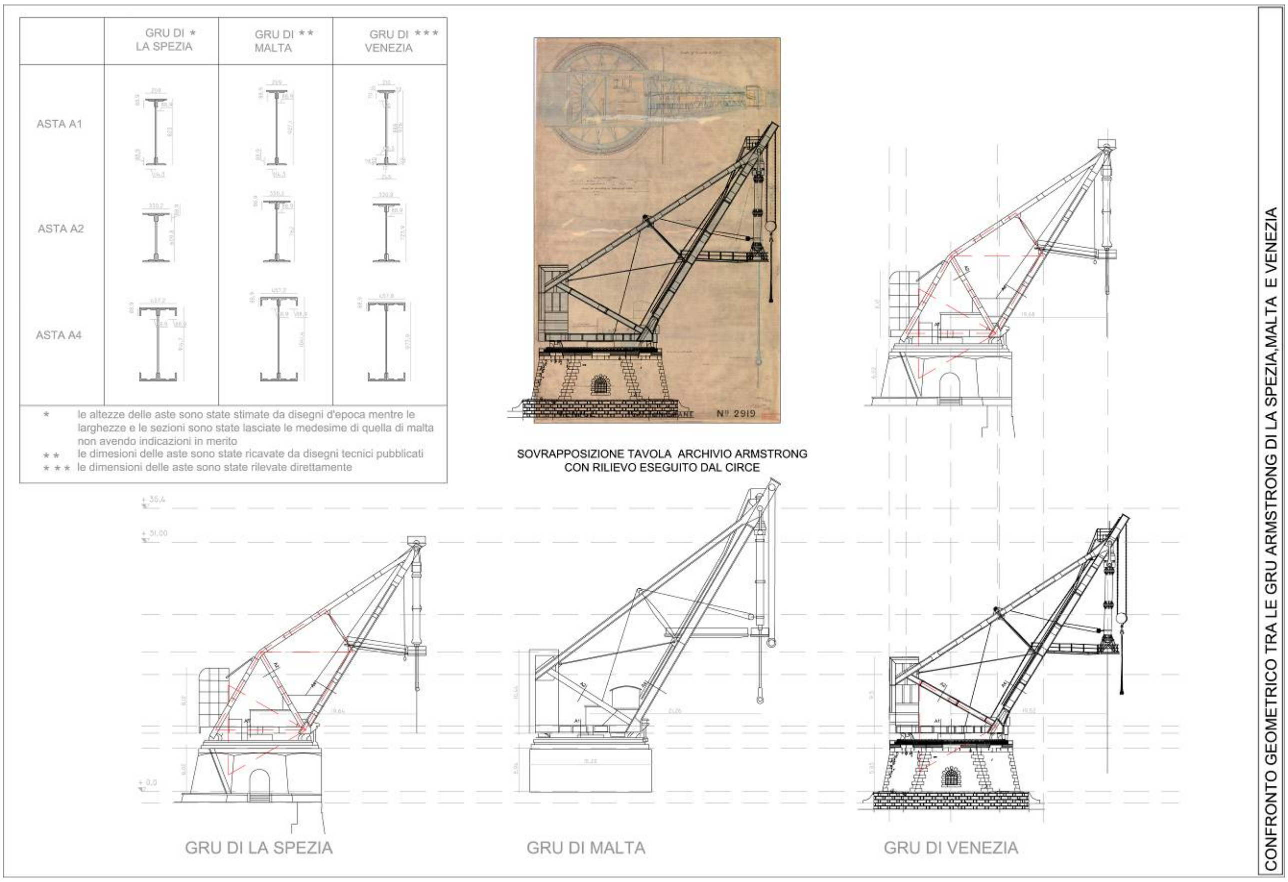


Fig. 5.4.12: Geometric comparison between Armstrong's cranes of La Spezia, Malta e Venezia

After the first direct measurements of the components of the crane of Venice, it was realized that the elements were producted using the inches system. Therefore, it was choose to describe metrically, in terms of graphic representation, the elements of the crane using English units. This choice was decided not only to facilitates the comparison, but also to make possible a reconstruction of Armstrong workshop's production during the years of its activity, to relate to the wide and varied production of English carpentry in that period.

Once the unit of measure has been settled, all the elements of the crane have been redesigned with the original size and dimension.

This has all been reported in summarizing form that will be described in the following paragraph.

5.4.2 Material and structural survey

Basement

The basement is perhaps the element what most differ in the various Armstrong cranes, especially from the architectural point of view, because it was the only part of the structure to be designed and manufactured locally, while the mechanical part was directly transported by sea, in pieces, from the headquarters of the British manufacturer.

While a calcareous sandstone typical of the site was used for the crane of La Spezia, a limestone and local granite of the highest quality was used for the crane of Malta. The obvious choice for the crane erected in the Arsenal of Venice (Fig. 5.4.13) was to use typical materials of the Venetian tradition: masonry and Istrian stone.

Fig. 5.4.13: Detail of the basement of the crane of Venice

While the analysis of the metal structure reveals the accuracy of nineteenth-century British metal carpentry, the basement reflects the working accuracy and the high quality of the workers of the Arsenal during the late nineteenth-century.

The building process has influenced the strength and the durability of the basement during the time, which is now in a satisfactory condition considering the total lack of maintenance for almost a century.

At this time the analysis of materials and their deterioration is the result of a mere on-site survey analysis, since there are no dangerous structural damages. The room inside the basement was left completely abandoned for over fifty years, but it shows no static instability.

Istrian stone elements

The stone (Fig. 5.4.14) is used in huge blocks as the podium's coating in direct contact with seawater because of its resistance to salt. Every single block has an height of 40 cm, and there is a regular alternation of square blocks and rectangular ones of 120 cm width.

Fig. 5.4.14: Detail of the Istrian stone basement

Even the stairs leading to the basement are made of Istrian stone. The two on the sides make continuous the perimeter line of the entire dock, made in plates of the same material, which continues interrupted, wrapping the base.

The main body of the basement, the stone outlines the corner elements of the frustum of pyramid, thereby nullifying the edges and determining the continuity between the two adjacent faces. The individual blocks decrease until they reach the upper crowning.

Moreover the stone, in regular segments, outlines the openings of the windows and the entrance to the engine room (Fig. 5.4.15). As for the podium, huge blocks are used, where the joints were carved on the surface to keep the formal regularity (Fig. 5.4.15).

Fig. 5.4.15: Details of the Istrian stone elements

The large circular track of the basement is made of blocks of stone with the perimeter of the elements finished off with a chisel.

The provenance of the Istrian stone used is not determined by microscopic analysis nor by chemical and physical characteristics of the material. The settling layers of the rock are evident, highlighted by the surface degradation, especially in the blocks above the podium. Most of them are correctly placed "in verse", that is following parallel planes along stratification.

One of the significant elements is the large ashlar on the upper platform that supports the circular track and the rotation movement. In the middle, the crane’s pivot is set, attached to the wall by 4 long bolts, which reach the center of the room. It is hollow because it can host the hydraulic lines, one for water under pressure and one for drainage (Fig. 5.4.16).

Fig. 5.4.16: Details of external and internal ashlar

The masonry

Probably the bricks have different backgrounds given the variety of color due partly to the different qualities of clay and partly to different cooking. In fact, depending on the weather and the cooking environment (oxidizing or reducing), the brick takes a different coloration ranging from red-brown to yellow-green.

Bricks (Fig. 5.4.17) have a quite heterogeneous composition and so-called ARF (Argillaceous Rocks Fragments) are clearly visible: they are red-brown nodules due to the vitrification of calcium or iron (found as debris in the clay), during cooking. They form when the time needed to maturing phase is not granted. This phase is carried out by leaving the clay exposed outside for about a year so that is affected by phenomena of freeze-thaw and it homogenizes.

Fig. 5.4.17: ARF's details present in eroded bricks

Bricks are still well made. Many of them have almost no surface degradation. Their resistance is due to the ancient method of brick crafting which induced a disordered macroporosity, less prone to capillary water and freezing. Also usually the old bricks have an elastic modulus lower than that of modern bricks, but their deformation is greater.

The adopted brick's size, despite the variety, is on average of 6,5 x 13 x 26 cm.

The bricks of the external face following the gothic disposition in regular horizontal courses with bricks laid alternately on a regular ABAB schema (Fig. 5.4.18).

Fig. 5.4.18: Detail of external masonry

The laying of the bricks was done to perfection while maintaining a uniform thickness of the joint never more than a centimeter high. The mortar used is composed of hydraulic lime, sand and pozzolana. This mortar has a high mechanical, physical and chemical strength because it has high impermeability, low porosity, low elasticity and therefore is able to be less vulnerable to freeze-thaw cycles and aggressive environments.

Inside the openings that get through the the wall's thickness the bricks are placed in the sill, and in the herringbone masonry on the top of the vault (Fig. 5.4.19).


Fig. 5.4.19: Detail of internal masonry

Inside the vault they are all placed as strechers (Fig. 5.4.20).

Fig. 5.4.20: Detail of masonry in the intrados of the vault

<u>Concrete</u>

The concrete fills the basement and forms the upper platform (Fig. 5.4.21). It has a compact structure, the chemical binder has not been analyzed, the aggregates are very small particle size and the reddish color of the surface is given by the deposits of corrosion of the above metal components.

Fig. 5.4.21: Detail of the concrete layer

The elevation structure

The materials that constitute and characterize the elevation structure of the crane above the basement are cast iron and puddle iron.

The elements of the crane made of cast iron are:

- the circular track with the external rack teeth;
- the 96 conical frustum shaped rollers;
- sliding blocks;
- the hydraulic cylinder.

All other metallic elements are made in puddle iron.

In this preliminary stage of knowledge of the structure the diagnostic phase of the metals is very important, to evaluate the safety of the structure and the choice of subsequent interventions.

Chemical analysis will also be crucial for the following additions or fusion of new elements and especially for the choice of cleaning and protection measures to be carried out in different ways depending on the chemical degradation and on the element type (structural or non-structural).

So far some investigations have been carried out on materials, more specific ones on the cast iron and more general ones on the ferritic material.

Initially macrographic analysis, chemical analysis such as SEM spectroscopic with microanalysis probe (EDS) and silicon metallographic replicas micrographs were carried out (Fig. 5.4.22).

Fig. 5.4.22: Micrographs with silicon metallographic replicas

Cast iron

The metallographic analysis performed on silicon replicas regarded the ring, a sliding block and a roller (Fig. 5.4.23) and shows the typical structure of a gray cast iron with lamellar graphite and ferritic-pearlitic matrix. In some areas there is the presence of ternary eutectic phosphorous (stead).

Fig. 5.4.23: Elements subjected to metallographic test (rack teeth, sliding block, roller)

On the cast iron micrographs and macrographs were also performed, by analyzing the same points of the previous metallographic campaign but in a post attack phase. It was found that the evaluations on the metallographic nature of examined components are confirmed.

Corrosion resistance of gray cast iron in marine environment is good. In any condition, the gray iron oxidizes in the air more slowly than soft steel.

The puddle iron

Regarding the composition and thus the chemical properties of puddle iron used in the construction of the crane there are not precise informations. The special fusion process lent important properties that distinguish it from other common ferrous metals: it can be soldered with ease, it has good fatigue strength and excellent corrosion resistance.

The percentage of carbon is around 0.05% and 0.2%, and silicon is present too.

This material, as witnessed by the cranes' conservation, shows a great resistance to corrosion compared to today's steels. The surface region, in fact, consists of a very tough ferritic layer, while the impurities during the solidification phase concentrate in the central thickness. In total lack of protection on the structures, the nature of the material can cause a dangerous particular deterioration, called exfoliation.

5.4.3 Deterioration survey

Basement: external deterioration

The deterioration suffered by the basement is due not only to the lack of maintenance, but also to its location within the Arsenale's dock, that is to its exposure to the elements, especially to the wind and to the marine environment, wet and rich in salts, which are also the reasons for the iron structure's corrosion.

Moreover there is the "disruptive" action of the vegetation, main reason for structural instability.

The degradation differs more between the four sides facing north-east and the four sides facing south-west (Fig. 5.4.24). This difference is evident by looking at the wall and its color. The faces that show a bright coloration, typical of masonry, are the most degraded ones in the southwest, that have a deep erosion, both in bricks and in mortar, while the faces in the north-east are brown, almost black, and the surfaces are well preserved, despite their major disruptions.

Fig. 5.4.24: On the left the north east side of the basement, on the right the south west side of the basement

Probably the winds inside Darsena Nuova blow strongly from the south. Their action accelerated the deterioration process, promoting the penetration of rainwater, increasing the wet/dry cycles and producing a surface erosion by removing material, so as to make them almost smooth (Fig. 5.4.25).

Fig. 5.4.25: Eroded surface's detail

On the bricks having less homogeneous structure the effect of disintegration or pulverization of the surface is clear, caused by the hygroscopicity of soluble salts present both in the material itself and in the environment, which brings to a volume increase, leading to a disruptive change in internal tensions.

On the same faces efflorescence (Fig. 5.4.26) is located mostly in the lower part of the wall, up to the openings and in sporadic locations at the top. In some places its thickness reaches a few millimeters. The stone of Istria basement and its high elevation above the water, if compared to high tides, has provided a good defense against the possible capillary rise of sea water. Their presence is closely related to the continuous and substantial water-lifts and wet environment. As one can notice there is no efflorescence on the surface covered by corrosion results or stains, because the resulting compact layer does not allow combination with the salts present in the bricks and does not offer chemical elements to bind to.

Fig. 5.4.26: Masonry’s detail showing that efflorescence is not present on the surfaces covered by corrision’s products and dark stains

Efflorescence phenomena is also undoubtedly due to the particular marine environment which presents an amount of sodium chloride (NaCl) in the air. These become hygroscopic salts when combined with other salts such as sulphates. In fact, chloride tends to crystallize in environmental conditions that can be defined as rare, that is at 25 ° C and 30% humidity, but they absorb a large amount of water and steam and keep them on the surface, so as to enable the solubilization of other salts.

The stains of corrosion deposits (Fig. 5.4.27) cover almost 100% of the faces on the northeast and the upper part of the entire crown. This is due to leaching of the products of corrosion of the metal parts above.

Fig. 5.4.27: Corrosive deposits stains

The brightest orange spots (Fig. 5.4.28) are those in which the action of the wind does not permit the combination of iron oxides with other elements, removing them continually. Generally the layer below the deposits, unless there was a strong action of atmospheric agents, is not degraded and in particular it seems to be consistent with the surface. The patina of rust, continually replenished, creates a layer of sacrifice to protect the underlying surface preserving the faces’ masonry. The very dark areas denote the formation of black crust caused by atmospheric particles rich in pollutants, which includes the exhaust fumes produced by the crane itself or by the vessels within the Arsenal, that laid on the surface of the material and that were smoothed by air humidity, slightly increased by the poor sunshine. The particles are very consistent to the substrate, the underlying surface is not flaked and there are no layers of plaster.

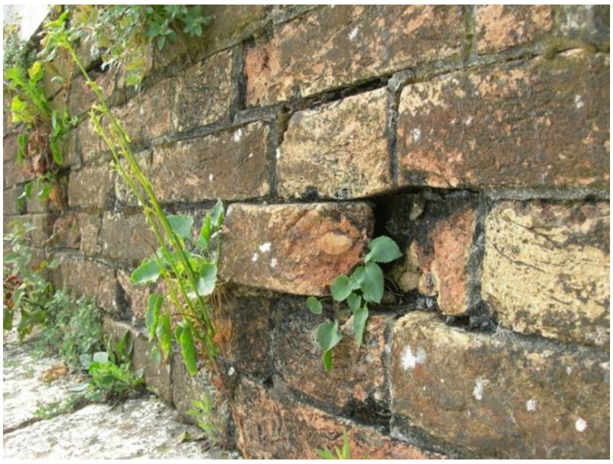



Fig. 5.4.28: Different types of stains

On the other hand the northeastern faces have some areas where large portions of bricks are missing. These are mainly concentrated at the top of the edge between the northwestern faces, accompanied by a vertical fracture and a disconnection of the masonry and its subsequent expulsion (Fig. 5.4.29).


Fig. 5.4.29: Masonry's disconnections

The masonry's disconnection may have been affected by the weight of the upper structure, composed of slewing ring, sliding blocks, rollers and lattice boom, but now it certainly is the settling of big deep weeds that brings to the disconnection of the rows of bricks. Fortunately the fractures are not passing and are not going to reduce the structural strength.

Fractures and cracks are the predominant degradation on the Istrian stone blocks. The main cause is the freeze-thaw phenomena amplified by the marine environment and the subsequent precipitation of soluble salts. In addition, within the fissures there are weeds whose deep and strong roots increase internal tensions between the edges of the cracks. The deeper cracks have caused the detachment and the loss of large parts especially on the podium.

The diversified degradation on Istrian stone is due, external factors being equal, to the different installation against the plans of settling of the blocks and to their cutting in the quarry. One may notice in fact that only the blocks of the face on the right side of the entrance, having plans of settling parallel to the compressive stresses to which they are subject, present surfaces exfoliation, a degradation which led to high loss of material, favored by environmental phenomena. The exfoliation phenomenon (Fig. 5.4.30) is limited only to areas with the presence of faults and with a homogeneous structure.


Fig. 5.4.30: Exfoliation example

Erosion is the predominant material degradation of the other blocks caused by environmental phenomena such as wind and the slow and continuous action over time of tide and the discontinuous one of waves.

The biological patina (Fig. 5.4.31) is widespread and it consists of several biodeteriogens, mostly lichens, present on the surface of the plates above the podium. In the area near the surface of the water a widespread and homogeneous band of seaweed is present. The degradation of the various micro-organisms, including microscopic algae, is of a chemical type (release of pigments) and of microclimate type (relative humidity increase on the surface), so that it holds water, which we know is a vehicle for pollutant and the cause of countless chemical reactions.

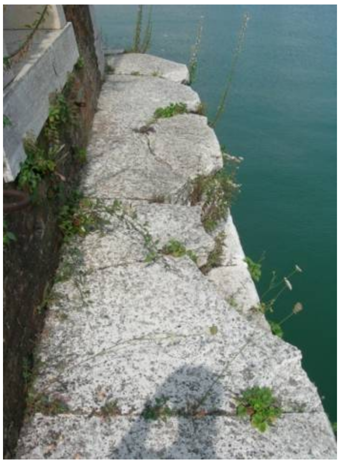



Fig. 5.4.31: Degradation cause by biological patina

The blocks of Istrian stone presenting the worst disconnections (Fig. 5.4.32) are the ones forming the three stairs leading to the basement below the podium where the mortar lost its cohesion.


Fig. 5.4.32: The two side stairs with disconnected blocks and missing parts

Istrian stones of the basement and of the upper crown are much less degraded than the others. This is due to a more accurate installation of the blocks and their finer grained and more compact structure.

The fractures are quite superficial and may also be due to the processing they were finished with. The stains (Fig. 5.4.33) are the predominant degradation due to the formation of corrosion deposits.

Fig. 5.4.33: Corrosion deposits stains on Istria stone

Basement: internal degradation

The high humidity carried in the air deposits then released on the surfaces, keeping constantly wet the faces of the bricks. Layers of salt efflorescence formed where they are not covered by washed rust (Fig. 5.4.34).

Fig. 5.4.34: Internal surface of the vault showing the light spots covered by efflorescence and the dark ones covered by oxides.

Almost the entire internal surface of the wall is covered with corrosion deposits, both in the bottom part and in the vault cover.

Oxides derived in part from corrosion of machinery and partly from the corrosion of the elements above the basement which penetrate by the 4 cylindrical holes made on the vault to expel the fumes of the machines. These holes (Fig. 5.4.35) were closed from the outside by circular iron plates with holes to avoid falling of foreign bodies. The steam coming out of these, during the crane operation, has certainly contributed to its corrosion.


Fig. 5.4.35:The basement's holes from which the superficial whasing products enter

The deposits on the bricks doesn't seem to be stuck to the substrate which was not subject to a significant alteration. The conditions of bricks and mortar joints could be called good, neither of them have subjected to removal of material.

Certainly the substantial thickness of the body of the basement has helped to maintain a constant internal hygroscopic air condition, with very high relative values at the same time.

<u>Basement: windows and external stairs</u>

The basement has a wooden gate with two doors and a second door inside with a wooden frame with a fanlight divided into segments (Fig. 5.4.36). The gaps were filled with glass now no longer present. The frames of the windows have two doors too, with an upper fanlight.



Fig. 5.4.36: The wooden frames of the basement

The iron stairs (Fig. 5.4.37), which leads up to the top of the basement, are affected by a degradation produced by localized corrosion. The entire surface is covered with cylindrical shaped craters having a uniform diameter of ± 1mm. The same corrosion on the thin iron plates, covering the small landings, has resulted in their total loss.

Fig. 5.4.37: Iron stairs to reach the upper part of the basement

Elevation structure: iron and cast iron

The crane completely lacked protection against corrosion for decades, but its condition is satisfactory. This is the result of a very careful structure construction and in particular of the rivet pitch always kept small in all the joints, which has limited the presence of the typical opern cracks of sheets and profiles junctions, between two rivets (Fig. 5.4.38).

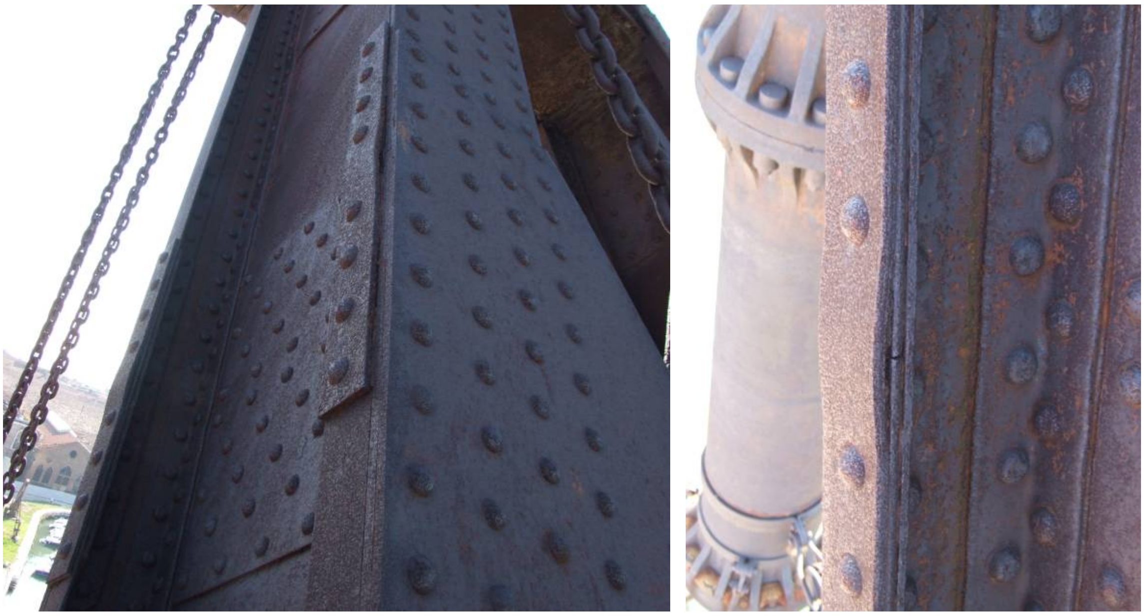


Fig. 5.4.38: Evident open cracks where the rivet pitch is wider and where there are joints.

Exfoliation is a very common effect of steel constructions corrosion.

It’s well known that the surface of these nineteenth century materials, unprotected, it’s much more resistant to corrosion than the modern structural steels (Fig. 5.4.39).

Fig. 5.4.39: The protective layer placed on the surfaces and not yet present on some small areas

However when the protective layer is destroyed by corrosion, the phenomenon of electrochemical corrosion generally occurs in plates and sections edges and especially in the surfaces cut in the workshop, and it is due to the numerous chemical and surface discontinuities therein present: the result is a rapid exfoliation throughout the entire thickness (Fig. 5.4.40).

Fig. 5.4.40: Examples of puddle iron parts exfoliation

The phenomenon is caused by the penetration of water in the interstitial layers of joints, which is favored, in the current joints, from the rivet pitch which is relatively high compared to the sheet thickness. Actually the phenomenon is much less evident in force joints, that is the structure's nodes, where the rivet pitch is smaller.

The corrosion that occurs in the joint levels, which is a typical case of corrosion by contact or in dead space, produces spongy oxides that hold water and promote a process of accelerated deformation of the plates, in which a decisive contribution is given by the formation of ice in winter months.

In other areas the corrosion is evident on localized spots of backwater, as on the water drains, present in the original project, but now obstructed by the corrosion products themselves because of lack of maintenance (Fig. 5.4.41).

Fig. 5.4.41: Drainage of the main ruster, blocked by the corrosion products.

The corrosion on the less ventilated areas, for example on the slewing ring below the elevation structure, is manifested in the form of bubbles and blisters indicating corrosion is taking place (Fig. 5.4.42).

Fig. 5.4.42: slewing ring's corrosion

The loss of materials is among the most dangerous consequences of corrosion on a structure such as this one. The destructive action of this degradation is very noticeable on thin sheets in which there is the complete disintegration of the surface (Fig. 5.4.43).

Fig. 5.4.43: Sheets corrosion

During direct survey portions of the rods proved to have suffered a reduction in cross section. Therefore it is likely that after the cleaning treatment by blasting, where the incoherent parts will be removed, the sections may reduce further. It will therefore be useful to calculate the minimum cross section required to meet the safety parameters.

The macrographic analysis showed heavy oxidation phenomena in all the metal parts of cranes, including the engine room inside the basement.

Visual examination of the driving machine shows that the components themselves are affected by the presence of exfoliation, even a deep one (Fig. 5.4.44).

Fig. 5.4.44: Engine room degradation

The rivets are missing just on a few spots (Fig. 5.4.45).

Fig. 5.4.45: Missing rivets

As for the deterioration that has affected the cast iron elements, the main cause is again corrosion.

The cast iron rollers are severely corroded, mainly due to backwater that caused the corrosion of the rolling surface, which it’s in direct contact with, and they present the most common degradation of this particular material, namely the exfoliation, on the bottom part which is constantly wet (Fig. 5.4.46).

Fig. 5.4.46: Rack's corrosion and rollers exfoliation

At the top of the rollers there is a ware along two intermediate layers, parallel to the sliding level, due to the rollers rubbing (Fig. 5.4.47).

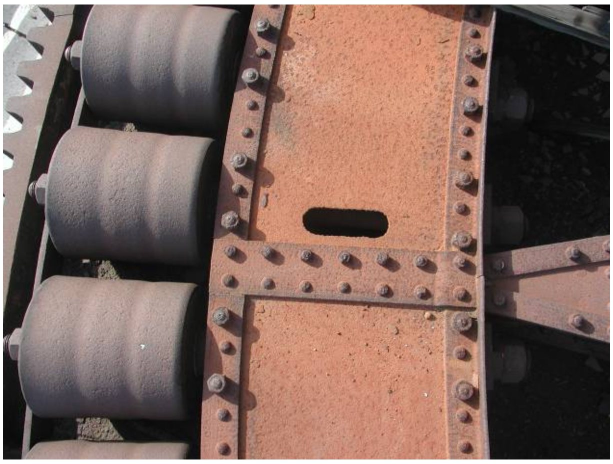


Fig. 5.4.47: Ware due to rollers rubbing

On the rollers edges and on the bottom parts of the sliding blocks there has been a loss of material that shows a breakdown due to corrosion and aggravated by wear (Fig. 5.4.48).

Fig. 5.4.48: Ware in rollers edges and in sliding blocks

The four cast-iron sliding blocks, which support the platform and slid on rollers, have been hit by a high corrosive state, aggravated in a decisive way by the "fatigue" sustained over decades.

The sliding blocks are indeed the most stressed elements of the whole structure of the crane, being the elements of support of the lattice boom. The two rear sliding blocks, placed below the counterweight, have suffered the most heavy damage, highlighted by obvious cracks, some of which are passing. In the past they have been subject to maintenance through the use of steel jackets, presumably put to stop, or at least slow down, the cracks progress and keep the two sliding block parts connected, so as not to diminish the reagent section (Fig. 5.4.49).

Fig. 5.4.49: Sliding blocks reinforcements

The condition of the sliding block placed under the counterweight is still critical because it is affected by a crack (Fig. 5.4.50).

Fig. 5.4.50: Crack on the sliding block below the counterweight

In the two front sliding blocks no evident sign of deterioration can be found, since the highest support reaction of these ones was reached in the past only in exceptional cases, resulting in no effect on the fatigue damage.

The presence of a large crack with relevant dimensions is detectable on the counterweight too, propagating through the rivets, as well as on the edges of the inspection holes on the walls, worsen by tensions within the load (Fig. 5.4.51).

Fig. 5.4.51: Crack on the rear of the counterweight and in particular on one of the inspection holes present in the box

5.5 Summarizing forms

The phases of the geometrical, photographic and macroscopic survey allowed to assess in detail the state of conservation of the elements composing the crane. All the information that came out from the study of the asset were collected in data sheets, for a clear understanding of the structure and as a database for further studies and interventions on the crane. The data sheets are also an important tool as a proof of the structure's present situation and they will be critical to assess, over the years, any geometrical alteration and in particular the material and structural degradation.

The data sheets have different layouts depending on if they're about the basement or about the top of the crane.

The first step was to split up the macro-structure in elements associated with different symbols:

Basamento
Struttura di elevazione

Asta : A (ultima A26)
Asta di collegamento/
irrigidimento: AC (ultima AC17)
Piastra/Piatto: P
Contrappeso: CP
Giunto cardanico: GC
Cilindro idraulico: Ci
Pattino: Pt
Paranco ausiliario: PA
Biella: B

Lamiera: l
Cantonale: L
Rinforzi a L: RL
Rinforzo a C: RC
Chiodatura: CH
Bullonatura: Bu

Nodo Strutturale (ultimo N49)
Giuntura: G (ultima G17)

Faccia

Following there is the nomenclature of each rods, which in turn are divided into sections. Below there are the elevations and three-dimensional views of the crane (Fig. 5.5.1, Fig. 5.5.2 e Fig. 5.5.3)

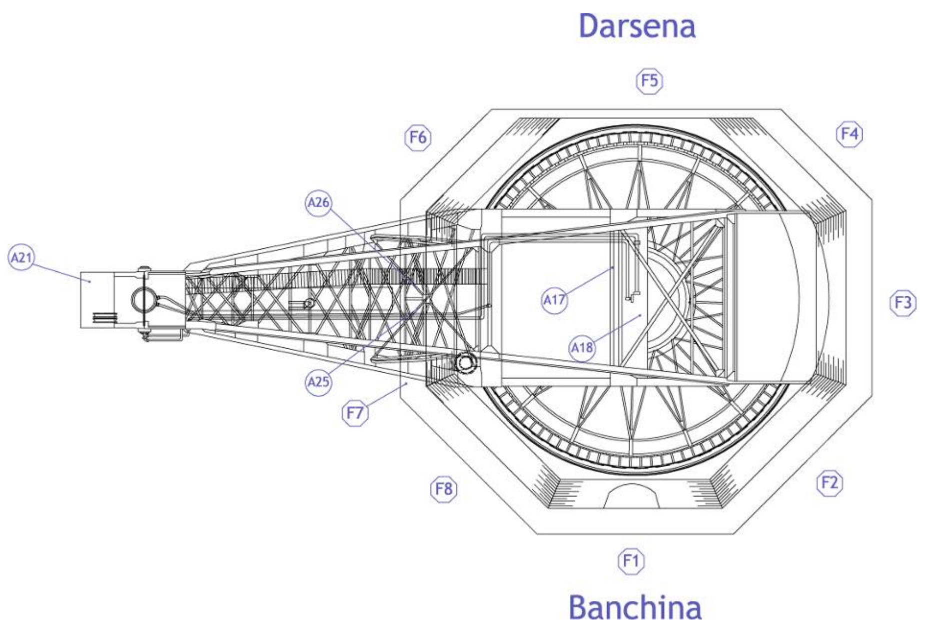


Fig. 5.5.1: Plant of the crane

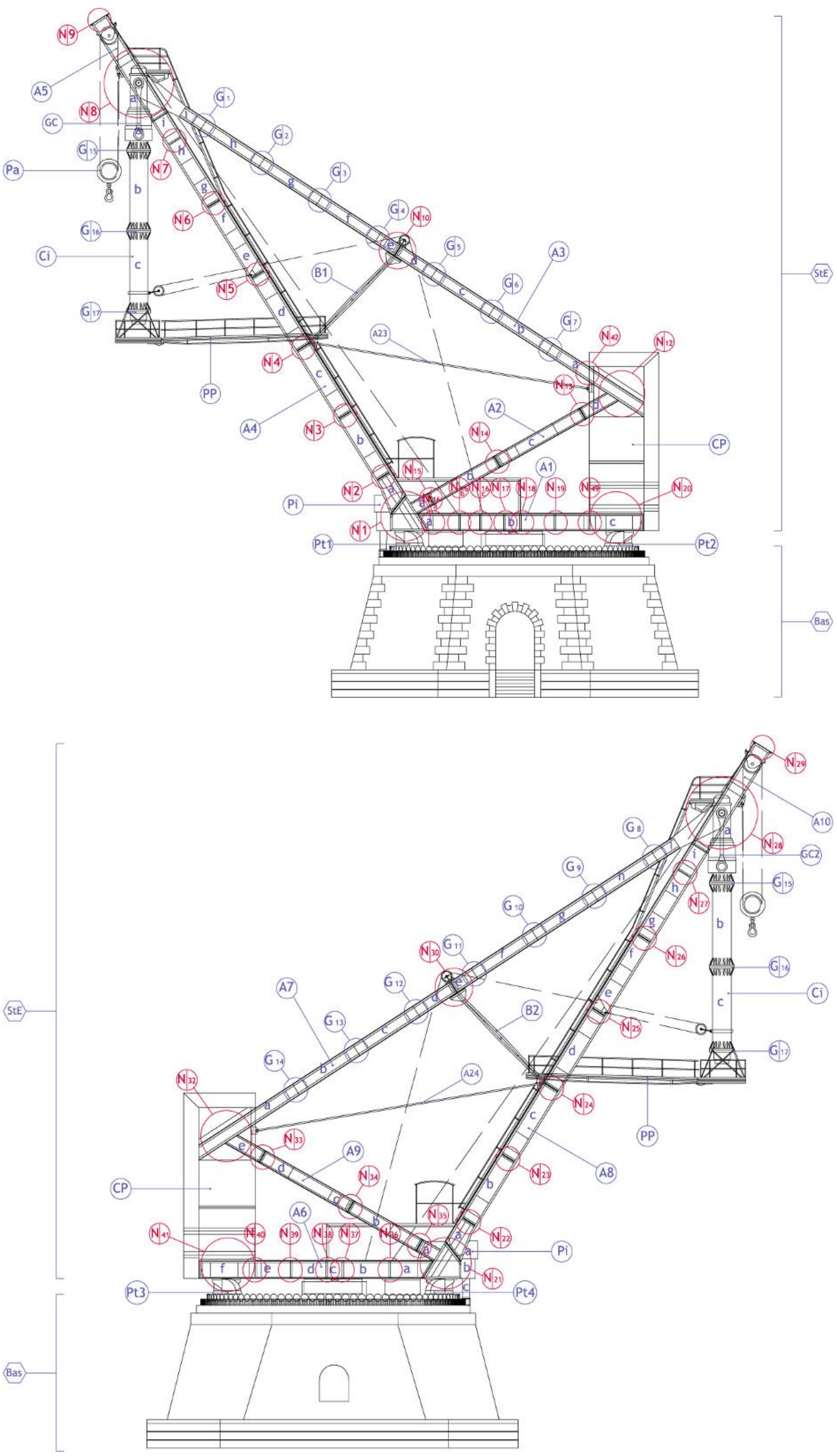


Fig. 5.5.2: Elevation of the crane, respectively basin and dock side

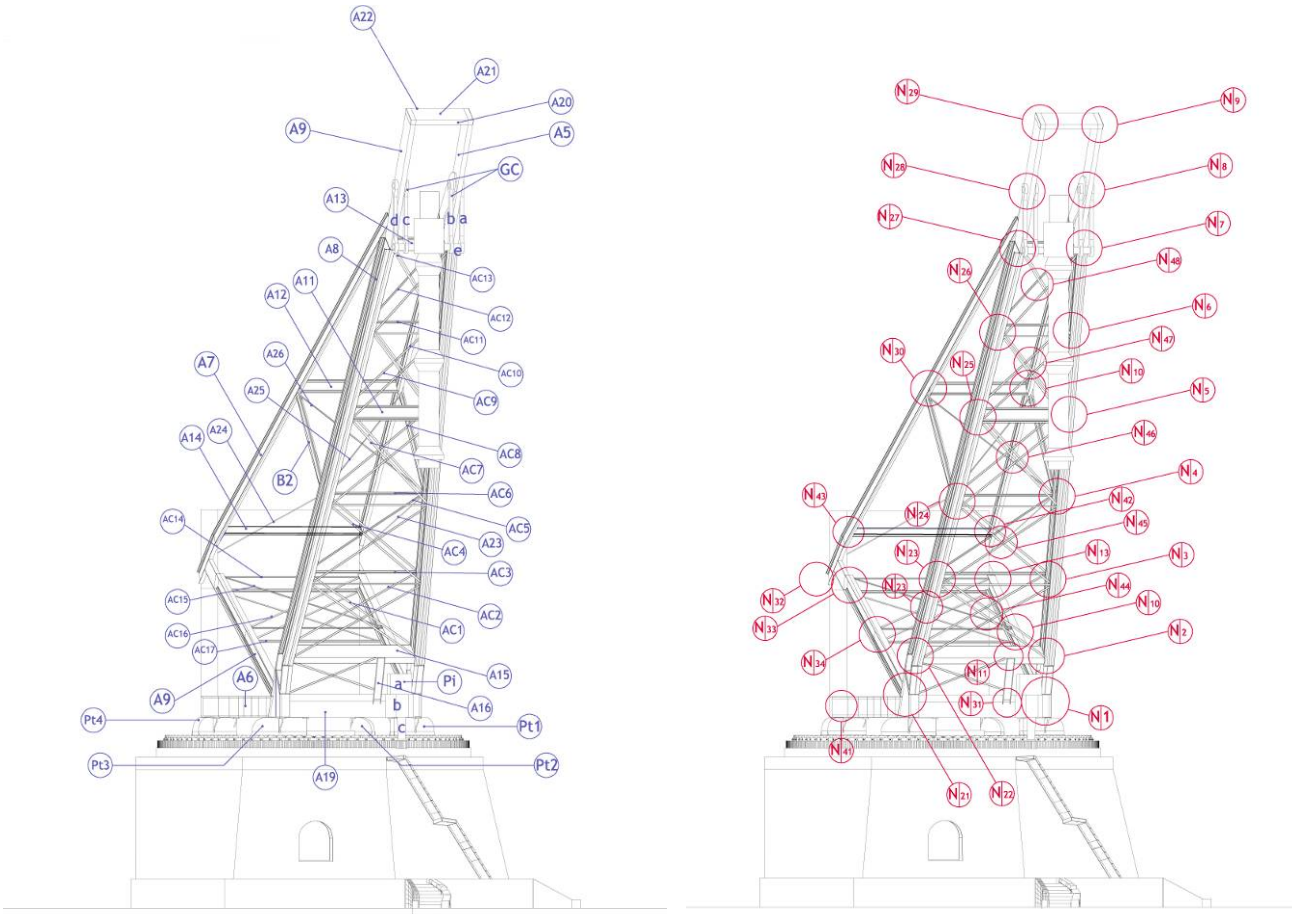


Fig. 5.5.3: Elevation of the crane with the main composing elements

The direct survey’s data sheets were simplified and made as open as possible to facilitate the surveyors operating in not easy conditions.

They were then organized into two pages:

- the first one contains the prospectus, the plant and the axonometric of the crane to quickly identify the coded name and the location of the detected item.

- the second one has room for the sketch to be implemented on site.

All this was reported in the data sheets aimed to summarize and to make easily readable all metric and photographic information collected during the survey, and further elaborated.

As for the bottom part, mainly built in masonry and Istrian stone, the degradation was shown in some graphic tables (Fig. 5.5.4) that contains the type of material, degradation and types of jobs to be performed for recovery and maintenance of the basement structure.

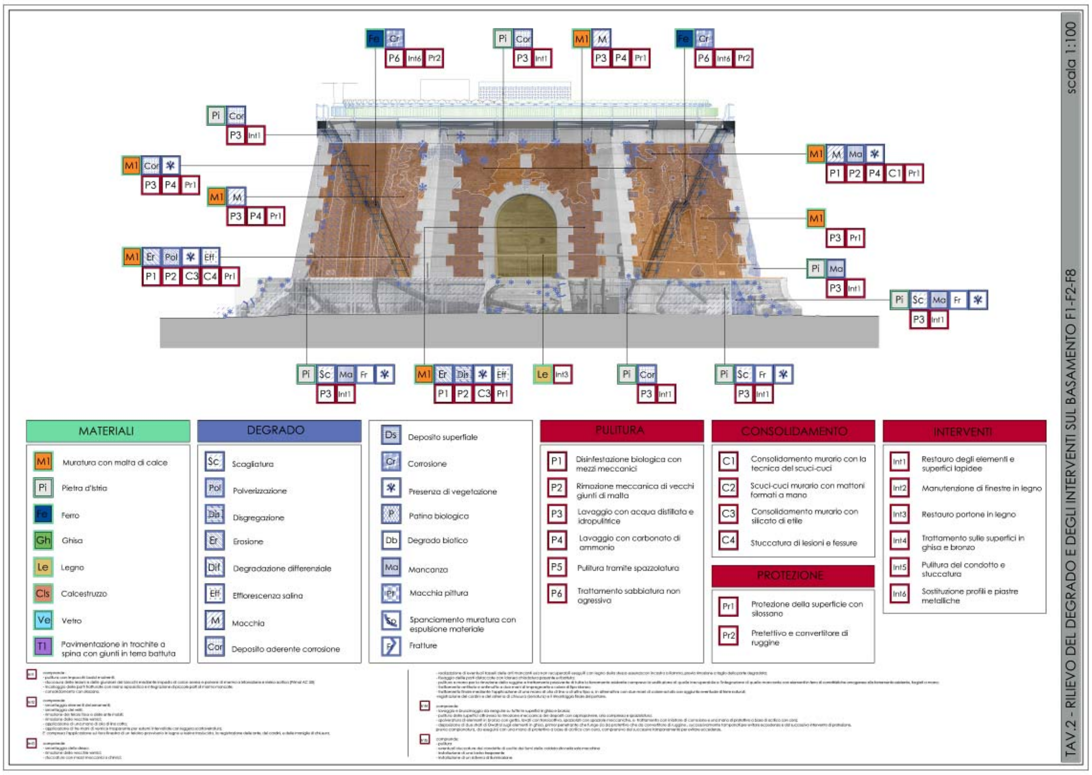


Fig. 5.5.4: Example of a graphical table concerning the degradation of the basement and the jobs to be performed for recovery

As for the elevation structure, mainly characterized by iron and puddle iron, the main causes of degradation were evaluated.

The final data sheets were divided into three sections to describe each item under three different aspects, namely:

- Through its spatial positioning by the element location (LOC);
- Through its appearance and consistency by a photographic survey (RF);
- Through its geometric shape by a geometric survey (RG).

Each of the three sections has unique identification codes for consultation, as reported in Fig. 5.5.5:

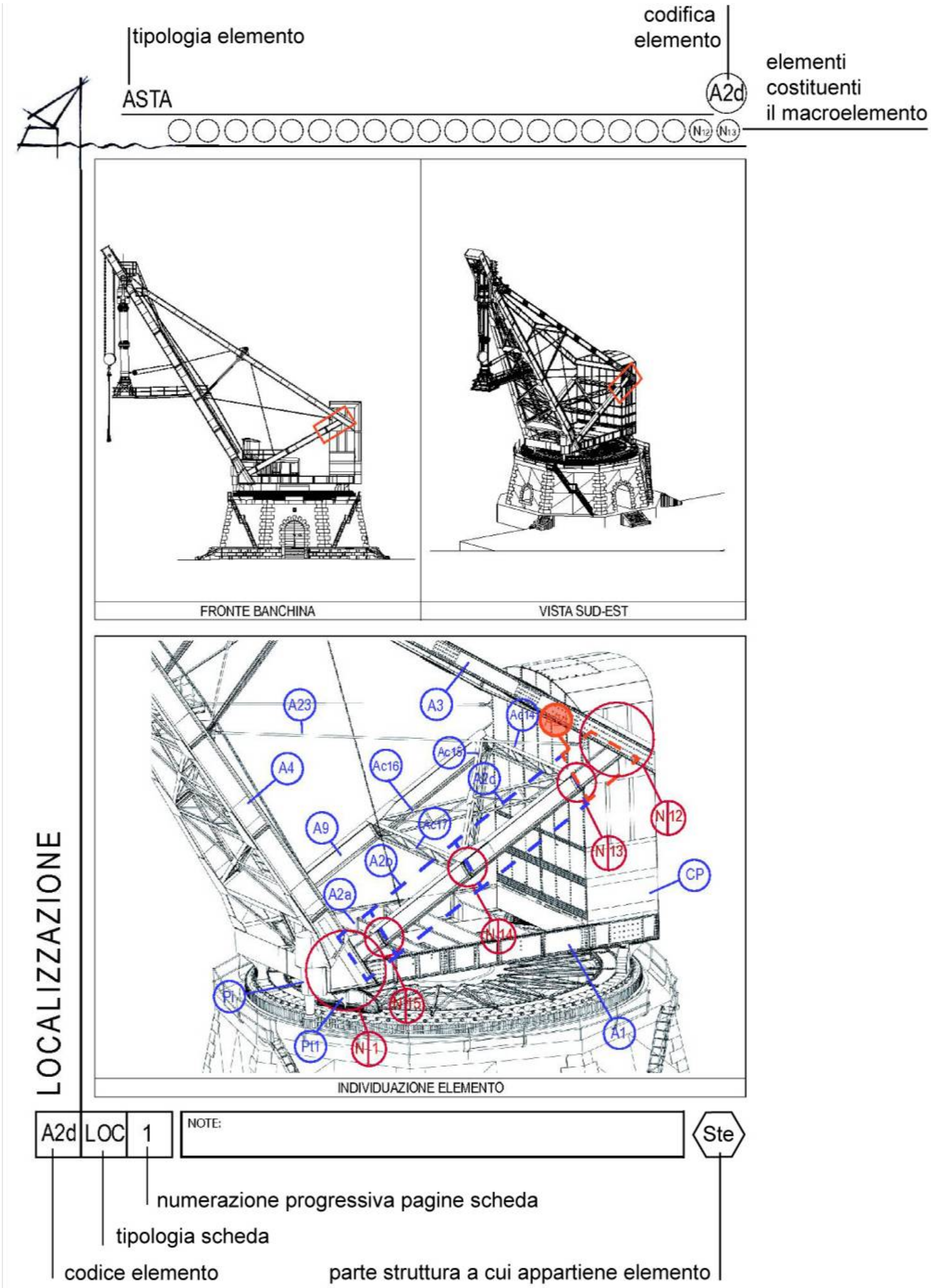


Fig. 5.5.5: Element's location - data-summarizing form

The location data sheet helps the understanding of the element positioning in the structure and its relation with its macro elements or with the junction nodes, which in turn are explained in a data sheet.

In the photographic survey data sheet (Fig. 5.5.6) there are some pictures that best describe the item's features. For a better understanding at the bottom of the data sheet there are the names of the file attachments, among which there are the many photographs taken during the survey.

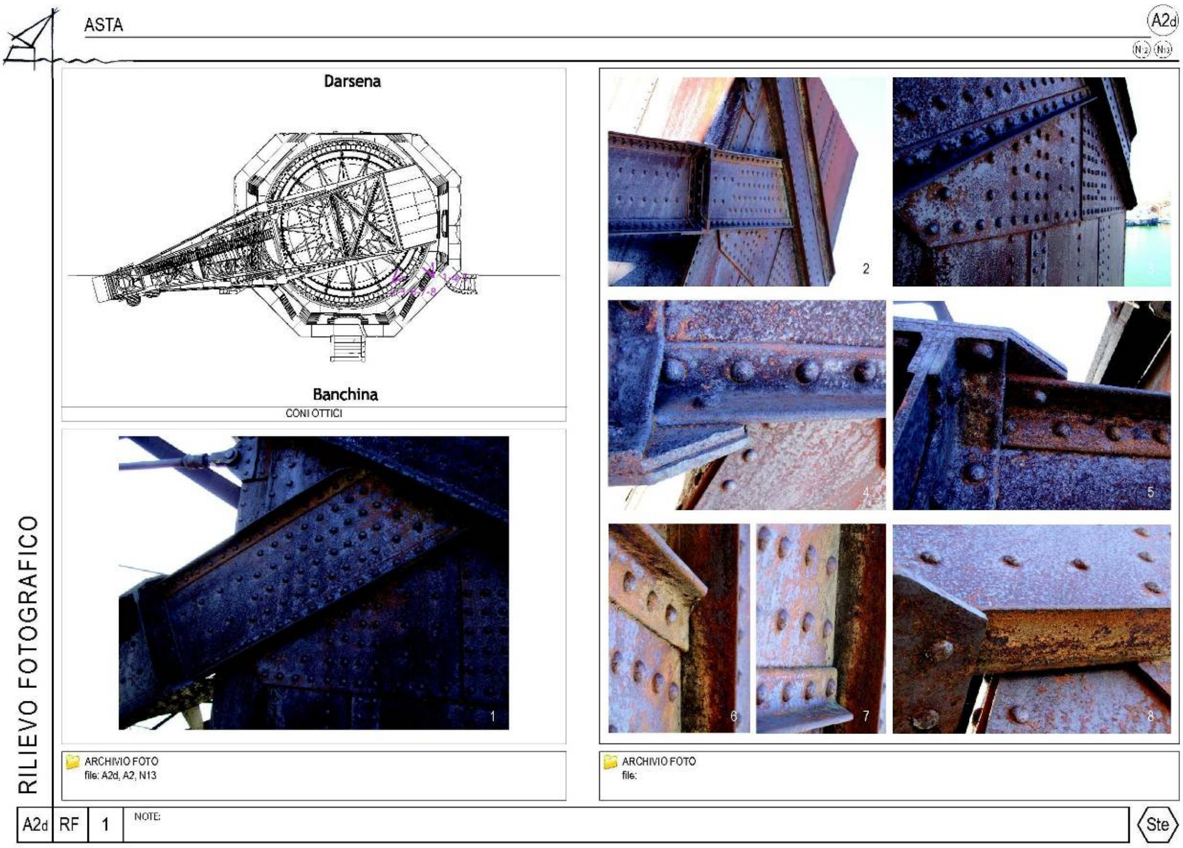


Fig. 5.5.6: Photographic survey data-summarizing form

The geometric survey data sheet (Fig. 5.5.7) schematically shows the desired element's position and the descriptive drawings with their sizes in English units. For sizeable items, such as the rods that make up the crane's boom, a scale representation of 1:50 was chosen, while for all other elements a smaller scale of 1:20 was reached. There's room for sections sizes, the rivets diameter and the rivet pitch.

In most cases the choice is to represent only the prospect of the current element, that is the one facing the quay.

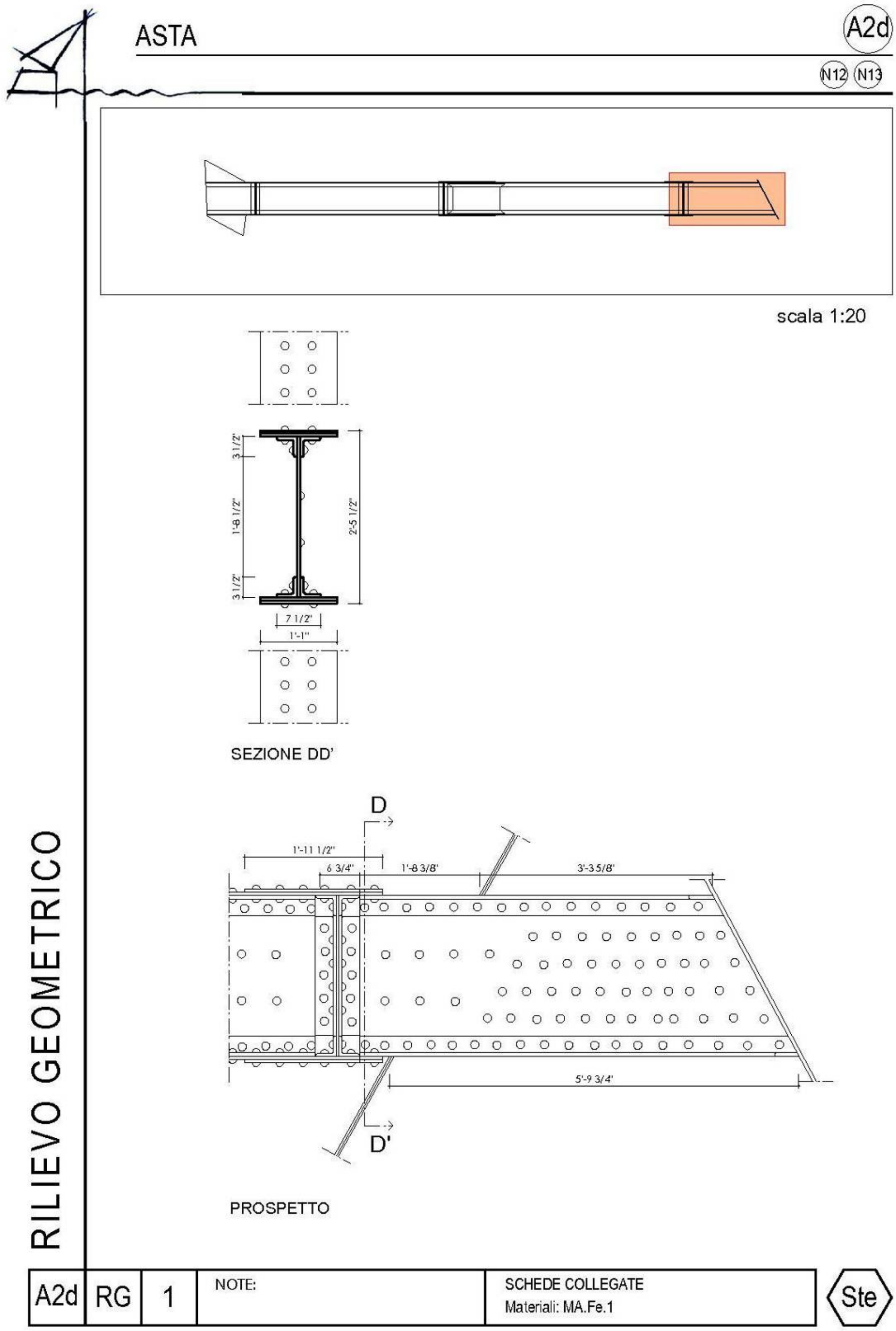


Fig. 5.5.7: Geometric survey data-summarizing form

5.6 FE modeling

The knowledge phase of the Armstrong, Mitchell & Co. Crane allowed to highlight the main problems both from a constitutive materials and structural point of view. It emerged that the crane present visible and dangerous cracks both in the balance weight and in the "rear" sliding blocks, symptoms of material decay and possible sources – if not properly treated - of critical problems for the stability of the whole structure.

The crane present remarkable dimensions: the *working* area can be inscribed in a cylinder with a diameter of 24 m and 37 m high, half on the wharf and half on water. The hoisting power corresponds to 160 tons, and structural masses of the crane are in the same range. The load of the balance weight corresponds to 400 tons, whilst the load of the upper part of the crane to 170 tons. Such loads are supported by 4 sliding blocks at the base of the crane. Also given the out of service of the crane for several decades, the most loaded supports, in the present conditions, are the "rear" ones, that is to say those located below the balance weight.

In the most stressed sliding blocks, the remarkably high load to whom they were (and still are) subjected, induced visible cracks on materials, some of them also passing through the thickness, which reduced the resisting area of the same blocks. It is then necessary to provide them with strengthening interventions, maybe also removing them, however avoiding to alter the equilibrium of the frame structure of the crane.

With the aim of assessing the effects on the original materials of alternative interventions on the crane and to complete the knowledge phase on the same from a structural point of view, FE numerical modeling was used. The modeling phase aims are several (see the following paragraphs): in any case the final goal of the numerical modeling aided evaluation is to monitor that in the different steps of the necessary interventions the field of stresses and strains in the original materials does not vary in a significant manner, particularly in the steel frame structure, in order to limit any possible disturbance to the structural equilibrium attained in the crane since several decades.

The description of the numerical model implementation and obtained results are reported in the following paragraphs. Both for the upper structure of the crane, made of iron trusses, and for the brickwork and stonework masonry basement, evaluation on present day stress patterns are drawn. With the goal of knowing the working conditions of the crane during its functioning in the last century, also a model considering the maximum live design load which the crane was able to lift (160 tons) was created.

5.6.1 Construction of the FE model

The FE model of the crane was implemented by using the software code DIANA ™ (TNO, Delft). Such code allows to visualize and work on the model (geometry and elements definition) before the calculation process (pre-process) and to visualize the emerged results after it (post-process). Several solver internal to the code may be chosen for different problems typology (linear, non-linear, phased, dynamic analyses…).

the Finite Elements Method is a standard procedure for problem solving in engineering. It is a widespread technique relying in the discretization of the continuous, where the analyzed structure is schematized through the construction of a number of 1D, 2D or 3D finite elements, connected at their nodes. The complete structure results then composed by the assembly of the interconnected elements, subjected, at their nodes to sets of internal and external forces. The degrees of freedom of the structure are represented by the displacements of the nodes, and they represent the unknowns of the problem. By using shape functions, the unknowns are calculated, and from these – once known the displacements of the nodes – the stresses are determined. Numerical modeling is a powerful tool in determining displacements and forces in solids by using suitable stress-strain relationships, which try to represent as close to the reality as possible the real behavior of materials, also using simplification however in reasonable agreement with the experiments.

The employed FE model of the Armstong Mitchell & Co. crane reproduces the geometry and the volumes of the structure, as emerged during the surveys carried out during the knowledge phase. The finite elements, grouped in functional sets, simulate the structural elements and their constitutive materials, with attribution of geometric and mechanical characteristics, as cross section for beams, stiffness, density, and strength values when non linear analyses are preformed.

The models used are three-dimensional models. Three different finite element typologies were selected with the aim of simulating different structural elements. Beam and brick elements were used for the trusses of the frame structure of the crane and for the balance weight system. Linear elastic schematization was adopted since non linearity is to be avoided in the metallic elements. The masonry basement was on the contrary simulated through brick elements with non linear constitutive laws.

Employed elements are:

- 2-nodes three-dimensional beam element, based on Timoshenko or Bernoulli theorems (L12BE). This element was used for the trusses of the frame structure of the crane (Fig. 5.6.1)

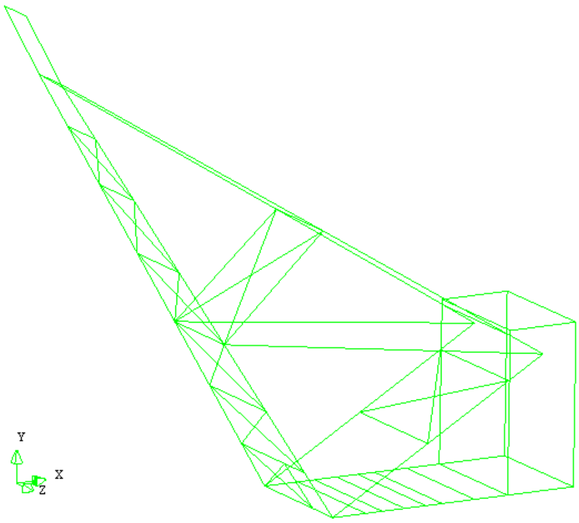


Fig. 5.6.1: Modeling of the upper part of the crane by means of beam L12BE elements

- 8-nodes brick linear element, based on linear interpolation and Gauss integration (HX24L). It was used for simulating the balance weight (Fig. 5.6.2).

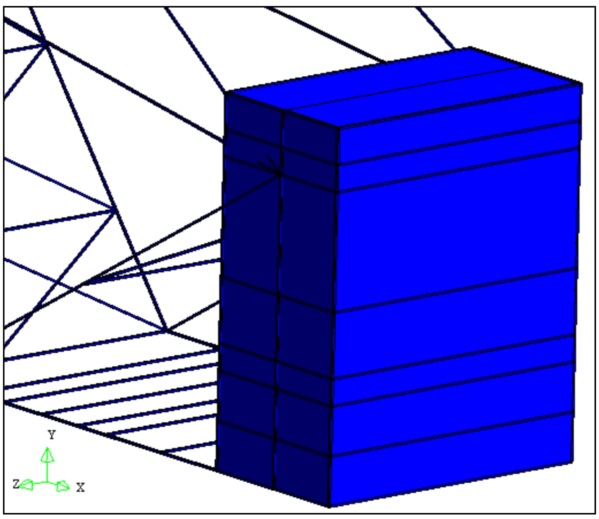


Fig. 5.6.2: Modeling of the balance weight of the crane by means of solid HX24L elements

- 4-nodes, three-sides isoparametric solid pyramid element, based on linear interpolation and numerical integration (TE12L). it was employed for the basement, the slewing ring an the circular track (Fig. 5.6.3 and Fig. 5.6.4).

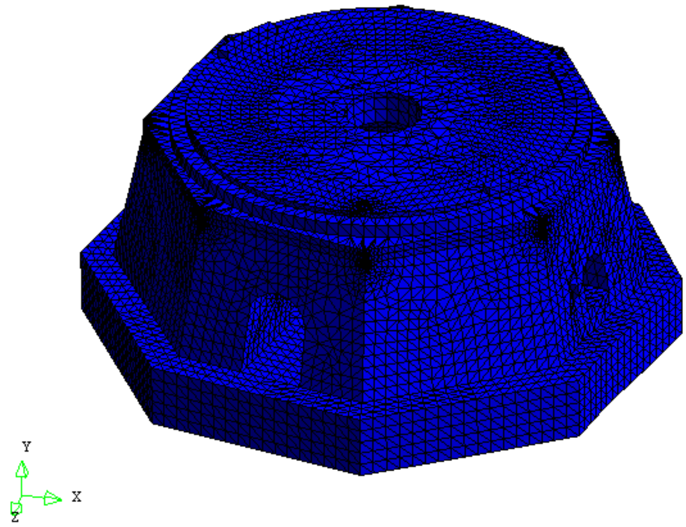


Fig. 5.6.3: Modeling of the basement of the crane by means of solid TE12L elements

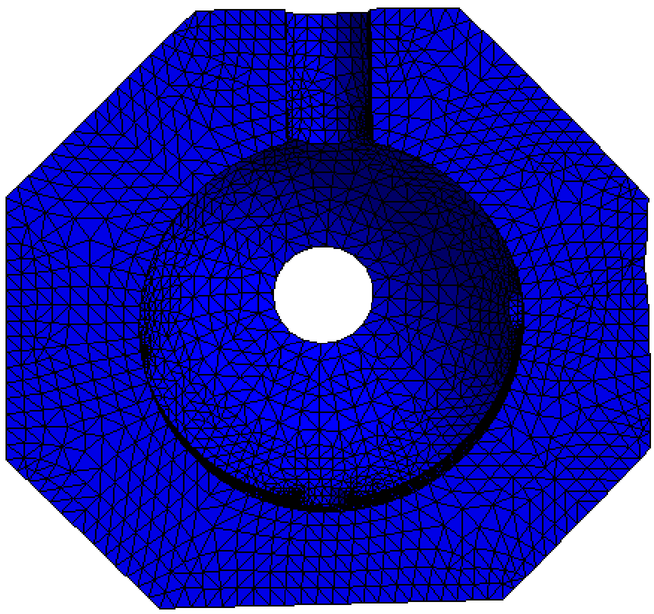


Fig. 5.6.4: Modeling of the vault of the crane's basement by means of solid TE12L elements

Geometric and material properties were applied to the beam elements constituting the frame structure of the crane. The balance weight is composed by a caisson made of sheet-iron, filled with two different types of materials, observed through the inspection holes. In its lower part, for a height of about 2,80 m, sand and iron debris are found, whilst the upper part, for a height of about 4,70 m, is characterized by the presence of sand and trachyte stone. When the caisson was opened from the top, lack of material for a depth of about 1,1 meters from the top was found. The density of trachyte stone and sand was considered equal to 1350 kg/m^3, whilst the density of iron debris and sand was estimated equal to 4000 kg/m^3, assuming a solid to void ratio equal to 1.

The total weight of the balance weight was finally calculated equal to 400 tons, value in good agreement with the information obtained from the bibliographical research (Fig. 5.6.5).

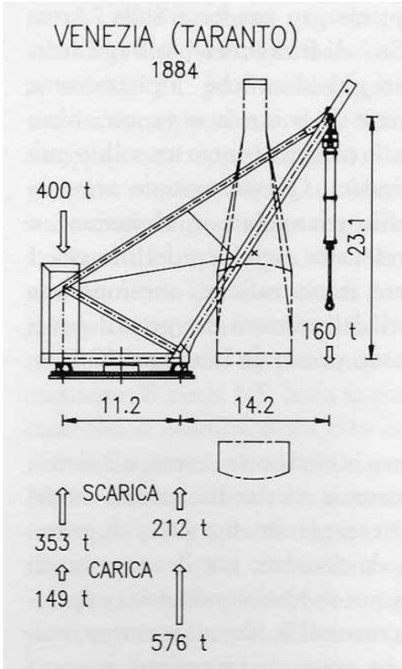


Fig. 5.6.5: Load in the balance weight caisson, corresponding to 400 tons (V. Nascè, M. Zorgno, 1994)

The numerical modeling considered different material typologies. From the on site surveys carried out, through topographic, photogrammetric and direct techniques, it was possible to precisely define the geometry of the balance weight, described in the following Fig. 5.6.6 and Fig. 5.6.7:

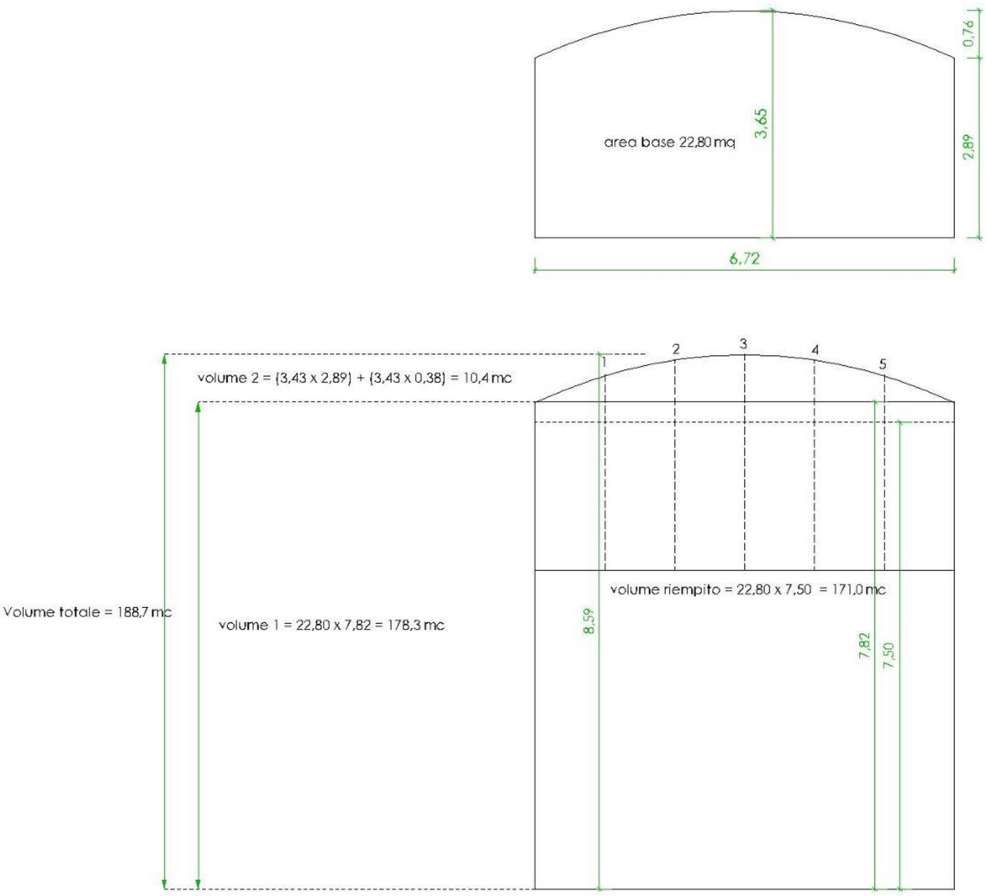


Fig. 5.6.6: Dimensions of the balance weight

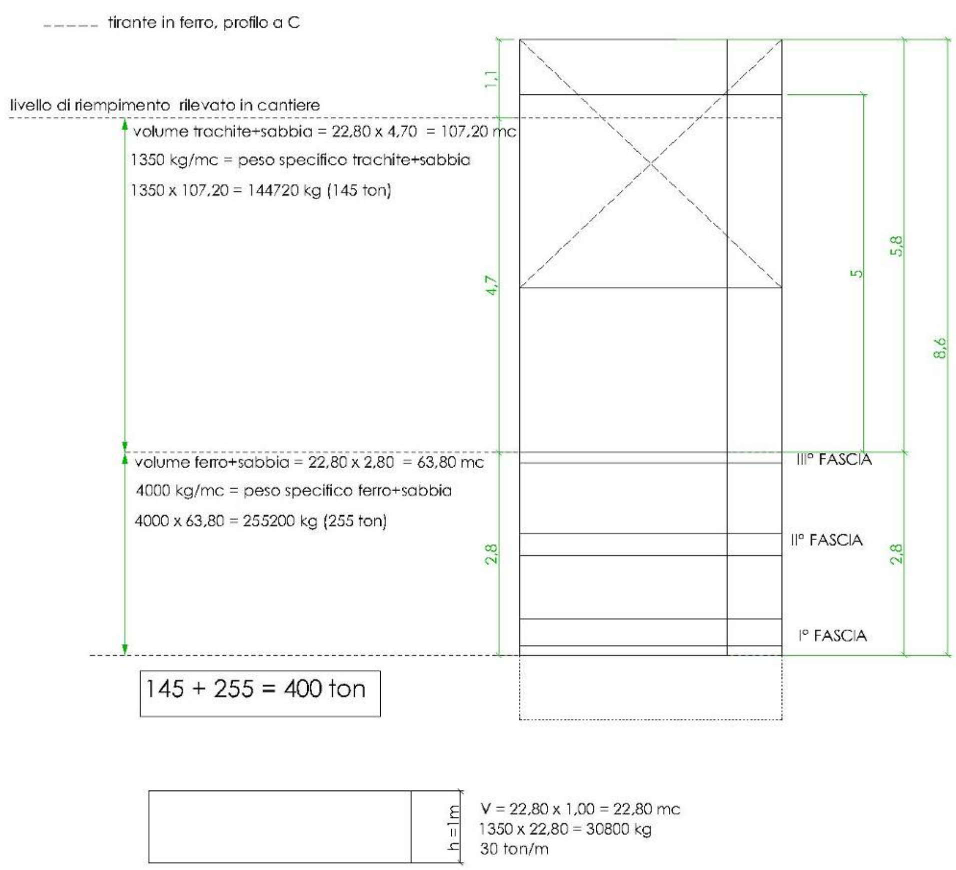


Fig. 5.6.7: Materials and loads of the filling of the balance weight caisson

To solid elements just the material properties were applied, since their geometry is already defined by the geometry of the same element. Numerical modeling of masonry is usually more complex with respect to metallic elements, since the hypotheses of isotropy, homogeneity and linear elastic behavior are generally non applicable. In fact, the stress strain relationship, for masonry, can be considered elastic just for a limited range close to the origin. Moreover, masonry possesses a very limited tensile strength.

In the case under study, also given the massive dimensions of the structure, a simplified macro-modeling approach was chosen, hence treating the material as a continuous isotropic composite.

In the following tables the characteristic mechanical values of materials and structural elements used in the model are summarized. The Istria stone masonry (Tab. 4.6.1) is located below the brickwork masonry basement, and in one layer of the slewing ring.

Tab. 5.6.1: Mechanical properties of the Istria Stone

MECHANICAL PROPERTIES OF ISTRIA STONE	
YOUNG'S MODULUS, E	2400 N/mm^2
POISSON'S RATIO, ν	0,25
DENSITY, γ	2240 kg/m^3

The bricks masonry (Tab. 5.6.2) is present in the basement of the crane.

Tab. 5.6.2: : Mechanical properties of the bricks masonry

MECHANICAL PROPERTIES OF BRICKS MASONRY	
YOUNG'S MODULUS, E	1500 N/mm^2
POISSON'S RATIO, ν	0,25
DENSITY, γ	1836 kg/m^3

The upper part of the basement, for a reduced layer, is characterize by concrete (Tab. 5.6.3).

Tab. 5.6.3: Mechanical properties of the concrete

MECHANICAL PROPERTIES OF CONCRETE	
YOUNG'S MODULUS, E	25490 N/mm^2
POISSON'S RATIO, ν	0,20
DENSITY, γ	2550 kg/m^3

The four sliding blocks on the slewing ring are in cast iron (Tab. 5.6.4).

Tab. 5.6.4: Mechanical properties of the cast iron

MECHANICAL PROPERTIES OF CAST IRON	
YOUNG'S MODULUS, E	200000 N/mm^2
POISSON'S RATIO, ν	0,20
DENSITY (USED FOR THE MODEL), γ	4000 kg/m^3

The lower internal part of the counterweight is characterize by sand and iron (Tab. 5.6.5).

Tab. 5.6.5: Mechanical properties of the sand and iron

MECHANICAL PROPERTIES OF SAND AND IRON	
YOUNG'S MODULUS, E	200 N/mm^2
POISSON'S RATIO, ν	0,27
DENSITY (USED FOR THE MODEL), γ	3660 kg/m^3

The upper internal part of the counterweight is characterize by sand and trachyte stone (Tab. 5.6.6).

Tab. 5.6.6: Mechanical properties of the sand and trachyte stone

MECHANICAL PROPERTIES OF SAND AND TRACHYTE STONE	
YOUNG'S MODULUS, E	200 N/mm^2
POISSON'S RATIO, ν	0,27
DENSITY (USED FOR THE MODEL), γ	1250 kg/m^3

The puddeled iron (Tab. 5.6.7) is present in all the elements of the lattice boom of the crane.

Tab. 5.6.7: Mechanical properties of the puddled iron

MECHANICAL PROPERTIES OF PUDDLED IRON	
YOUNG'S MODULUS, E	200000 N/mm^2
POISSON'S RATIO, ν	0,27
DENSITY (USED FOR THE MODEL), γ	7850 kg/m^3

The above introduced mechanical characteristics are related to the masonry properties as assembly of brick/stones units and mortar. The four sliding blocks were modeled by using TE12L three-dimensional elements, whilst the beams connected to them were introduced as simple beam two nodded elements (Fig. 5.6.8). Considered this simplification, for obtaining a realistic load distribution on the masonry elements positioned below the blocks, the stiffness of the elements representing the cast iron sliding blocks was fictitiously increased.

Fig. 5.6.8: Detail, connection sliding block – balance weight caisson

Fixed restraints were imposed at the base of the structure (Fig. 5.6.9) (translations in the three directions are prevented), schematizing the real restraints distribution.

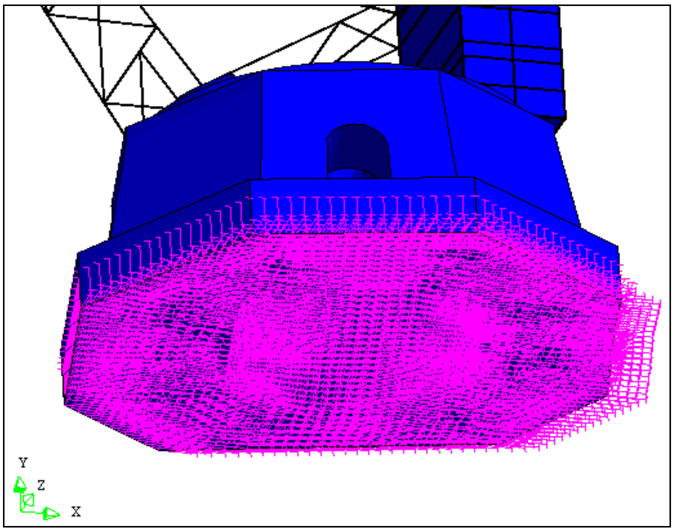


Fig. 5.6.9: Overall view of the model with the base restraints highlighted

The stiffening plates, located in the nodes of the steel frame, were not introduced in the FE models, but their weights were taken into account as equal value point loads in the corresponding nodes of the structure (Fig. 5.6.10).

Fig. 5.6.10: Detail, node of the crane where three trusses converge

Moreover, the cylinder positioned in the front/upper part of the crane (Fig. 5.6.11) was simulated by applying two point loads in the corresponding location, each of them with a value of 220 kN.

Fig. 5.6.11: Detail of the cylinder positioned in the front/upper part of the crane

Once completed the calibration of the model (Fig. 5.6.11), the analysis proceeded with the calculations execution and the results evaluation.

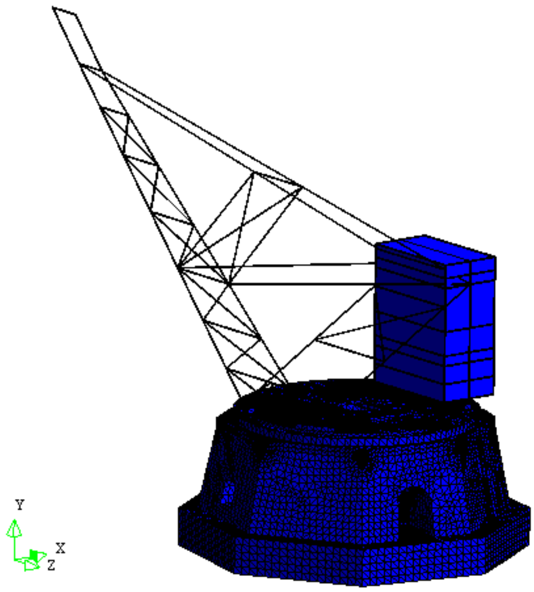


Fig. 5.6.12: FE global model of the crane implemented by using the DIANA ™ software code (TNO, Delft)

The implemented model was analyzed by using non linear material properties. Non linearity was introduced only in masonry elements, since the behavior of steel elements – according to the scope of the interventions design - must be kept within its elastic limit. To execute non-linear analyses, with the adopted constitutive laws for masonry elements (Total strain rotating crack, Feenstra et al., 1998), besides introducing density, Young's and Poisson's moduli, it is necessary to define the parameters describing the non linear material properties in terms of compressive and tensile strength (f_m, f_t), fracture energy value (G_f^I), and softening and hardening behavior.

5.6.2 FE models results

The analysis allowed to identify the deformations and stresses in the trusses of the upper part of the crane, as well as in the masonry basement, and to quantify the reactions at the base of the steel structures, forces which must be sustained by the sliding blocks.

All of the forces and bending moment acting on the trusses of the crane were taken into account, and the trusses were analyzed and assessed by using the standards' requirements, with the theory of the limit states.

Used units are Newton [N] for forces and meters [m] for distances.

The main trusses of the crane are subjected to relevant axial loads. Fig. 5.6.13 clearly shows the compressive and tensile forces in different elements. The trusses subjected to the highest tensile stresses are those positioned in the upper part of the crane, connecting the caisson of the balance weight with the upmost part of the crane. The axial force borne by these trusses corresponds to 336 kN. Those subjected to the highest compressive force are the struts, spanning from the topmost part of the crane to the front sliding blocks, with 719 kN of axial load.

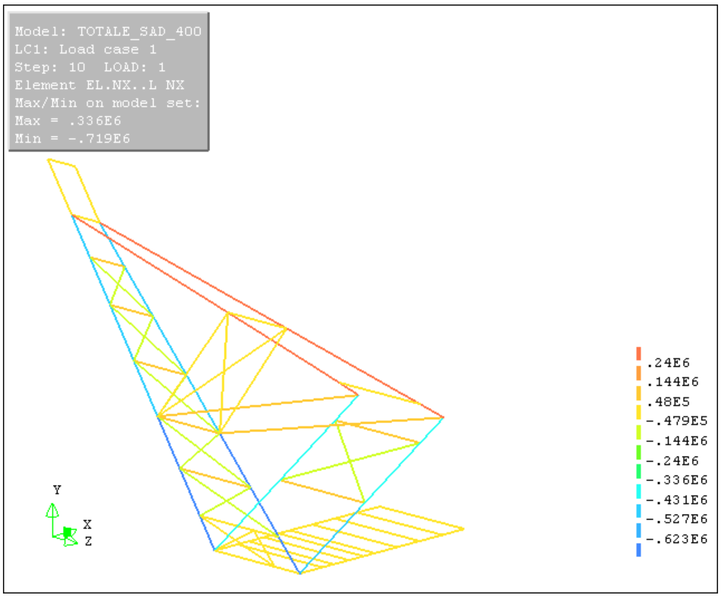


Fig. 5.6.13: Axial stress in the trusses of the frame structure of the crane (N)

In particular, this truss is subjected to a gradually increasing load towards node named N1 (Fig. 5.6.14). Such node, also depending on the observed material deterioration, resulted the most critical area of the upper structure. This is due to the remarkable forces to whom the node is subjected, since it represents the converging point of three of the four principal beams of the crane.

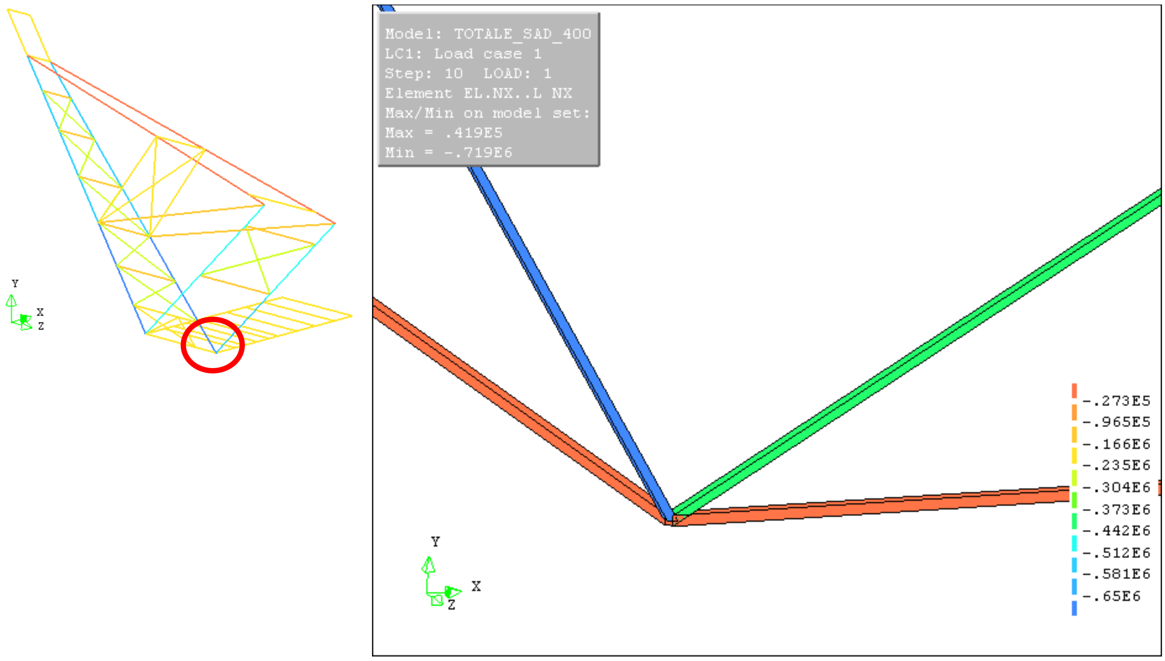


Fig. 5.6.14: Detail, axial load converging to node N1 (N)

The crane's trusses, besides being subjected to axial loads, must also bear non negligible bending moments (continuous elements). Bending moments (Fig. 5.6.15) remain however limited in almost all of the upper structure of the crane, except close to some nodes where several trusses converge. In particular, as for the case of the axial load induced stresses, the node subjected to the highest bending induced stresses is node N1.

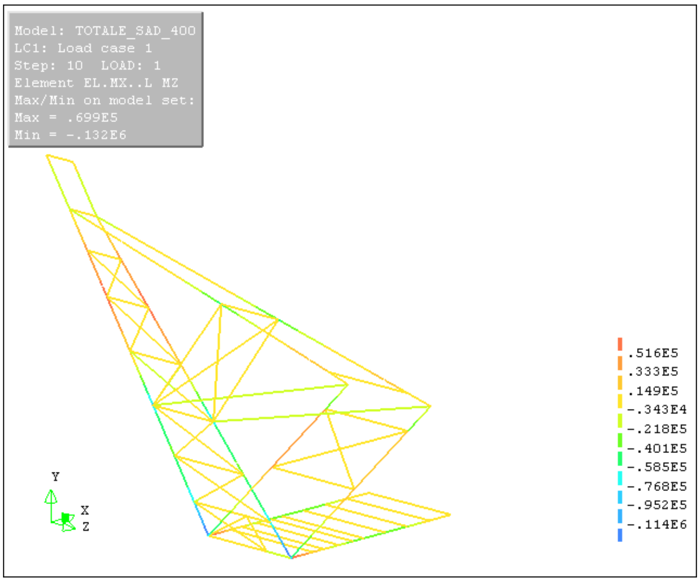


Fig. 5.6.15: Bending moment in the trusses of the frame structure of the crane (N·m)

The calculations to verify the capacity of the elements of the upper part of the crane with reference to the standards' requirements are illustrated in the following sections.

To assume realistic data for the mechanical characteristics of the puddle iron, a historic research was carried out. From the consulted documentation, the results of a tensile force test on puddle iron pertaining to a chain of the Armstrong Mitchell & Co. crane of Venice emerged. Test was carried out in the testing facilities of La Spezia (Italy) in 1948, and it resulted that the mechanical characteristics of the iron are very similar to those of a S235JR steel type. According to Table 11.3.IX of the national Italian standards on constructions (D.M. 14/01/2008), the yielding stress of such steel typology corresponds to 235 N/mm^2, a value very close to the one resulted from the 1948 test.

A partial safety factor on the strength of materials is in any case considered, and this is done according to the procedure defined in the Circular letter nr. 617 of February the 2nd 2009, explicative of the national Italian standards on constructions (D.M. 14/01/2008): the assumed "knowledge level" is equaled to LC2 (extensive knowledge), and thus a partial safety factor of 1,20 is adopted to conveniently reduce the design values of materials' strength.

The yield strength of puddle iron is assumed then, reducing its characteristics value:

$f_{yk} = 235/1{,}20 = 195$ N/mm^2

Ultimate tensile strength is equal to:

$f_{tk} = 360/1{,}20 = 300$ N/mm^2

the design strength of the puddle iron elements results equal to:

$f_{yd} = f_{yk}/\gamma_s = 195/1{,}15 = 169{,}57$ N/mm^2

where γ_s is the partial safety coefficient of steel, equal to 1,15.

The analysis results are successively evaluated in terms of deformations and stresses in the masonry elements of the basement. Accordingly to the classification of the materials composing the basement as brickwork and stonework (Istria stone) masonry, following the on site survey carried out, and in absence of direct evaluation of the mechanical characteristics of the two recognized masonries, the proposed strength and stiffness values of materials reported in the Circular letter nr. 617 of February the 2nd 2009 are assumed, divided by the Confidence Factor = 1,35 (partial safety factor related to "limited knowledge"), since the attained "knowledge level" is considered equal to LC1 (minimum).

According to Table C8A.2.1 of the above mentioned Circolar Letter, the mechanical values of brickwork and regular stone masonry, divided by the Confidence Factor (FC = 1,35) are reported here below:

For the basement materials, the following average strengths are considered:

- brickwork masonry with hydraulic lime mortar

$f_m = 2{,}40 / 1{,}35 = 1{,}77$ N/mm^2

- Stone blocks masonry

$f_m = 6{,}00 / 1{,}35 = 4{,}44$ N/mm^2

the Istria stone basement elements are subjected to limited value stresses, with maximum values in compression, localizing in the bottom areas at the interface with the wharf, equal to 0,37 N/mm^2, far below the considered limit strength (4,44 N/mm^2).

Fig. 5.6.16 shows the stresses in the Istria stone elements due to the upper positioned masonry bulk, indicating higher values in the areas below the balance weight, however definitely reduced.

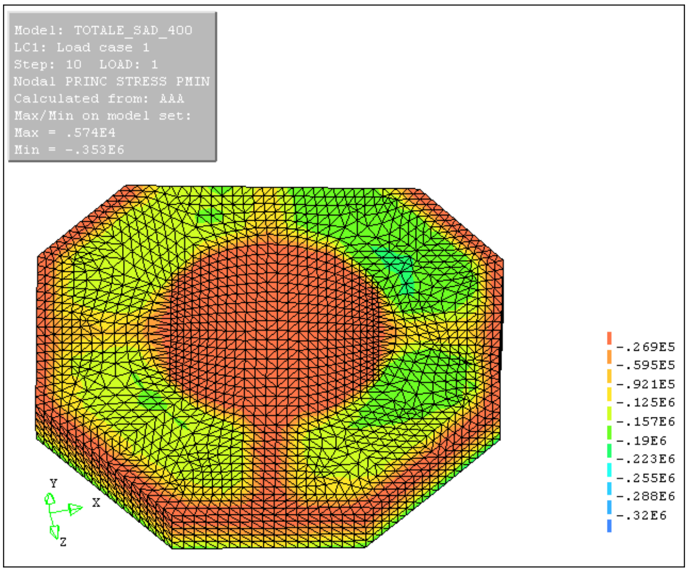


Fig. 5.6.16: Minimum principal stresses (compression) for the Istria stone elements of the basement (N/m^2)

Also the brickwork masonry elements of the basement are subjected to reduced stress values. Stresses in the external areas (Fig. 5.6.17) present higher components in the areas below the rear sliding blocks respect the front ones, in the range of 0,35 N/mm^2. In the perimeter of the basement the hydrostatic stress distribution induces maximum compression values at the foot of the structure, also here with maximum values around the value of 0,30 N/mm^2.

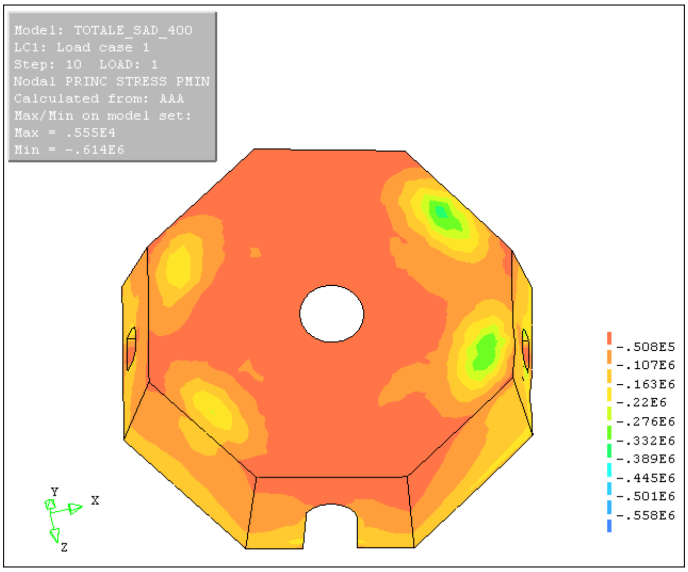


Fig. 5.6.17: Minimum principal stresses (compression) for the brickwork elements of the basement (N/m^2)

Inside the basement (Fig. 5.6.18), stresses in the central areas of the vault and close to the openings are limited, increasing getting closer to the vertical walls, with values in the range of those already found in the external parts. Highest stresses are found in very limited areas at the support of the vault, by consequence of the sudden change of thickness, reaching values of approximately 0,81 N/mm^2.

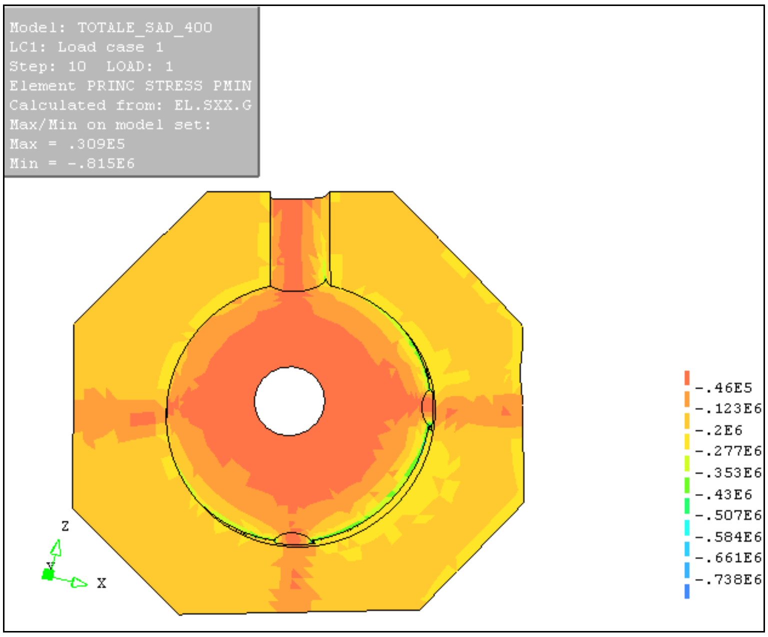


Fig. 5.6.18: Minimum principal stresses (compression) for the brickwork elements of the basement's vault (N/m^2)

Finally, the attention focuses on the upmost areas of the basement, composed by two layers of Istria stone and a finishing layer of concrete. From Fig. 5.6.19 it is possible to appreciate the stress *marks* due to the forces transmitted from the sliding blocks to the basement. High compressive stress values are found below the rear blocks, with average values of approximately 2,00 N/mm^2. Such values are higher than the assumed resistance of brickwork masonry (1,77 N/mm^2), and it is thus advisable to reduce such stresses, even if the strength of masonry is calculated with high safety factors, obviously reducing the load insisting on the rear supports of the crane. The front sliding blocks transmit to the below positioned masonry lower, although always significant, compressive stress values, in the range of 1,50 N/mm^2.

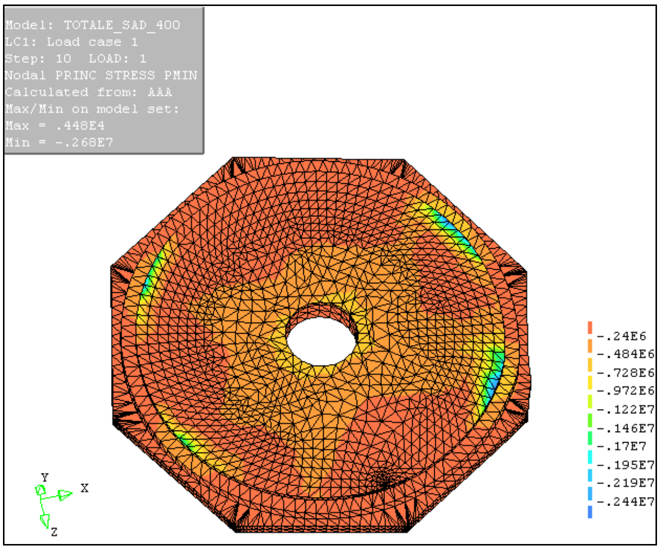


Fig. 5.6.19: Minimum principal stresses (compression) for the upmost brickwork elements of the basement (N/m^2)

Fig. 5.6.20 shows the global model of the basement where the principal minimum stresses (compression) are highlighted.

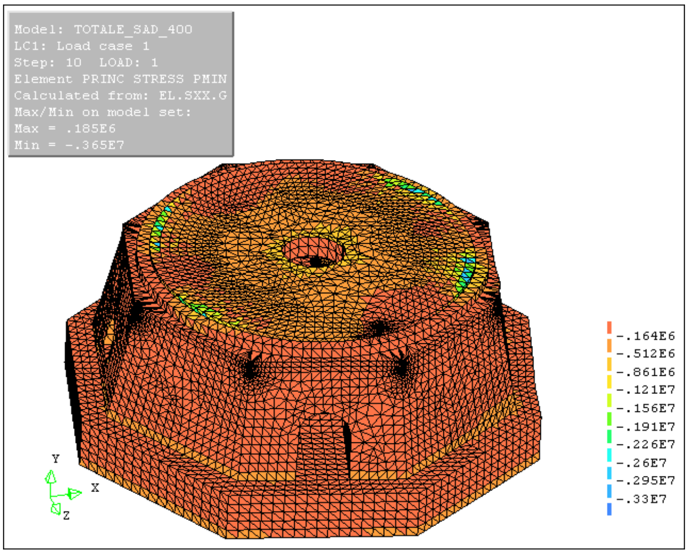


Fig. 5.6.20: Minimum principal stresses (compression) for the external brickwork elements of the basement (N/m^2)

The Armstrong Mitchell & Co. Crane is not functioning since several decades. The aim of structural modeling it is not hence the evaluation of the structural response of the crane when subjected to the original live loads, but to just its self weight, since the restoration's aim is the stabilization of the structure for conservative purposes.

The present day stress distribution was hence calculated, as well as the buckling conditions of the trusses of the upper part of the crane. For correctly representing the basement masonry behavior, non linear constitutive loads were selected, even if, considered the reduced stress fields emerged from the analyses carried out, its response remained limited within the elastic limits.

Finally, in the study of the global behaviour of the crane in its current conditions, the vertical reactions at the four restraints, corresponding to the sliding blocks, were evaluated (Fig. 5.6.21).

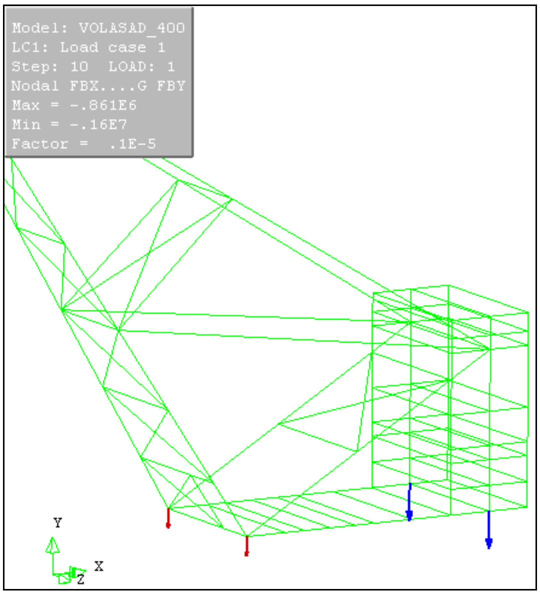


Fig. 5.6.21: Vertical reactions at the supports of the cranes, calculated values

In the following table (Tab. 5.6.8) the Y (gravity direction) reaction values are reported, naming with "rear sliding blocks" those positioned below the crane's balance weight, and "front sliding blocks" the two opposite to the first ones.

Tab. 5.6.8: Y reaction values (gravity direction) of front and rear sliding blocks

	Y REACTION VALUES (KN)
FRONT SLIDING BLOCKS	- 861
REAR SLIDING BLOCKS	- 1600

The crane must have been able to lift heavy loads through the hydraulic cylinder and the lifting hoist. As expected, the highest vertical reactions, in the current conditions of out of service, are in fact found at the rear supports of the crane, below the balance weight, with values almost double with respect to those of the front supports. For this reason (out of service of the crane, worsened by the effects caused by the removal of the piston, likely during the II World War), the rear sliding blocks manifest visible cracks, passing through the thickness. From the direct visual inspection the situation seems critical, with possible *partialization* of the blocks and thus reduced resisting area.

It was then necessary to introduce immediate provisional strengthening devices, until the beginning of the activities related to the final restoration intervention. The design of the temporary intervention includes the hooping of the balance weight caisson, and a partial emptying of the same, with the aim of balancing the reactions. To estimate the quantity of weight to be removed hand calculations, later on compared with the FE modeling results, were carried out.

The subsequent design phases included overall equilibrium considerations and the evaluation of stresses in the upper structure of the crane following the total removal of the material inside the caisson, for allowing the necessary final restoration intervention operations. Also here, the quantity of the reintroduced material was calculated in a way of equilibrating the reaction forces, having thus equal stresses in all of the four sliding blocks. To equilibrate the structure in all of the intervention steps, especially emptying the caisson, it emerged the necessity of introducing steel cables anchored to the masonry basement, preventing the global overturning of the crane. Several possible alternatives were considered and validated through FE modeling.

5.7 Provisional interventions

The results emerging from the general model of the crane together with the on site observations indicate the necessity of lightening the balance load, with the aim of reducing the stresses on the back sliding blocks, and consequently those on the masonry basement. A preliminary calculation was hence performed in order to evaluate the stability conditions of the upper part of the crane and of the mass to be removed to render equal the four vertical reactions at the sliding blocks. Verifications were performed by means of hand calculations and with the aid of the FE model.

The executed operations, related to the provisional interventions, are also described.

5.7.1 Analysis of the vertical reactions of the crane

The weight of the frame structure of the crane was assessed by considering the volumes of the composing elements in the three-dimensional model built after the several surveys carried out. The load of the hydraulic cylinder resulted from the assumption of an element thickness corresponding to 100 mm, referred to a total diameter of 900 mm. the total weight of the upper structure of the crane resulted equal to 171 tons. The coordinates of the center of mass of the frame structure are calculated with reference to a system of orthogonal axes whose origin is located at the front corner of the supporting base of the structure (Fig. 5.7.1).

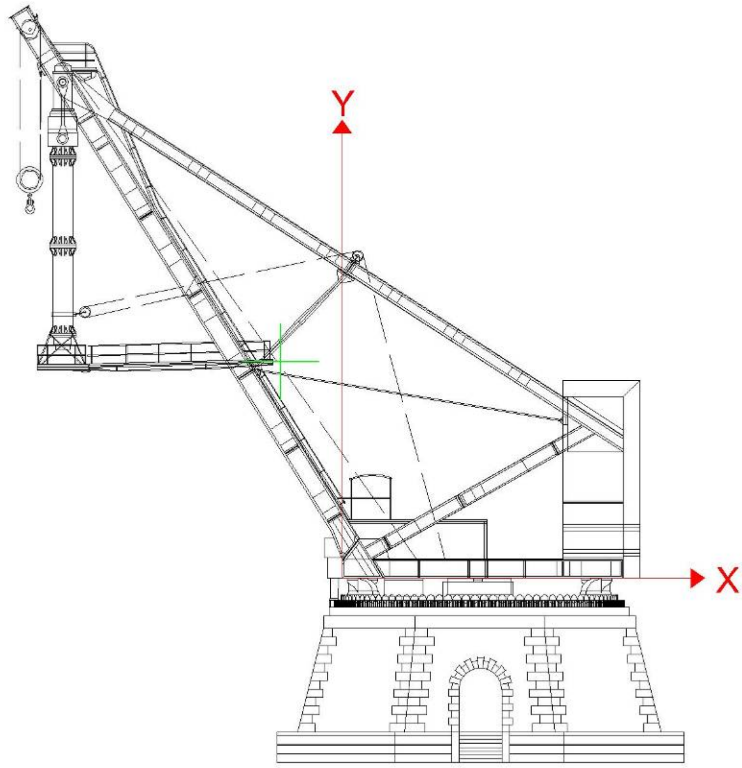


Fig. 5.7.1: Orhogonal coordinates with localization of the center of mass of the upper part of the crane

All of the centers of mass of allo f the elements were calculated, with the aim of obtaining the center of mass of the upper structure of the crane:

X = - 2,98 m

Y = + 10,53 m

The balance load mass necessary to equilibrate the vertical reactions on the four sliding blocks was then determined. To do so, the equilibrium of moments was calculated with respect to point C, center of gravity of the four sliding blocks. The total load of the upper part of the crane (P_v) is equal to 171 tons, and it is located at a distance of 2,98 m from the origin of the considered axes, A. The two couples of blocks, front and rear, have a reciprocal distance of 11,60 m. The wind load, defined by using the standard DIN 1055, part 4, was omitted in the calculations since it has a marginal effect on the general equilibrium. The unknown is then the value of the balance loads (named P_c) equaling the vertical reactions in the points A and B (Fig. 5.7.2).

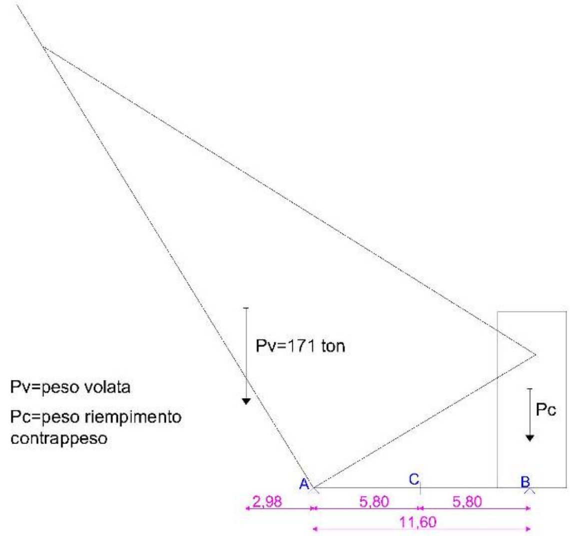


Fig. 5.7.2: Localization of the center of mass of the upper structure of the crane

Imposing the condition that the resultant force of the vertical reactions passes through point C, center of gravity of the four sliding blocks, a balance load of 260 tons (P_c) is necessary.

Through the equilibrium of forces was hence calculated the (minimum) limit value for the balance load mass to prevent the global overturning of the crane (rigid body assumption), resulting equal to 44 tons.

Considered the uncertainty on the actual balance load material density, it was considered a safe assumption to remove from the caisson, in a first phase, a mass equal to 100 tons, lower respect the 140 tons calculated for the equivalence of the vertical reactions, for in any case considerably reducing the compressive stress on the rear sliding blocks.

Equilibrium calculations through FE modeling

In order to validate the hand calculations based upon the equilibrium of rigid bodies, the FE global model of the crane was used. 100 tons were then removed from the balance load of the initial model (related to the current conditions), to evaluate the reduction of the vertical reactions in the rear sliding blocks (Fig. 5.7.3) and thus to appraise the effectiveness of the foreseen emptying operations.

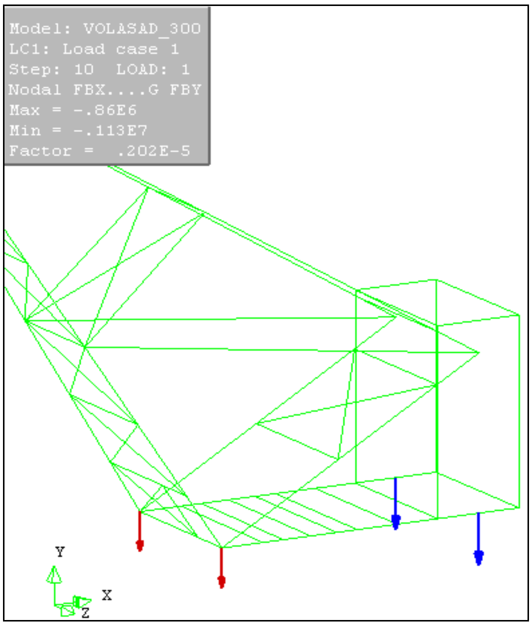


Fig. 5.7.3: FE model: vertical reactions values in the four blocks of the crane with a mass of 300 tons as balance load

As reported in Tab. 5.7.1, the vertical reactions of the rear sliding blocks remarkably decrease with the mass reduction of the balance load. By removing 100 tons of material in the caisson, the reactions in each of the below positioned blocks decrease of 500 (470) kN. As expected, the reactions of the front blocks do not show appreciable changes.

Tab. 5.7.1: Y reaction values (gravity direction) of front and rear sliding blocks

INTERNAL MASS OF THE COUNTERWEIGHT	Y REACTION VALUES (KN)	
	FRONT SLIDING BLOCKS	REAR SLIDING BLOCKS
400 TONS	- 861	- 1600
300 TONS	- 860	- 1130

It is then confirmed that the removal of 100 tons from the caisson are sufficient to remarkably reduce the vertical reactions in the rear blocks, pushing the resultant force of the supports of the crane close to their center of gravity. From numerical modeling the hand calculations are then confirmed. In fact, from the FE model it emerges that for having in all of the four sliding blocks equal reaction values it is necessary to remove a quantity of material from the caisson equal to 2433,86 kN (approximately 243 tons).

5.7.2 Interventio to provide safety conditions

As a result of the verifications was decide to removed 100 tons from the caisson. Before perform this intervention was be necessary insert reinforcement rings in the counterweight in order to work safety.

Reinforcment rings of the counterweight

The on-site survey has highlighted the presence of a diffuse degradations of the elements. For this reason, was decided the partial removing material inside the caisson (Fig. 5.7.4) to reduce the vertical reactions in the rear blocks.

Fig. 5.7.4: Counterweight of the crane

The counterweight was opened through a manhole located at the top, as represent in Fig. 5.7.5.

(a) (b)

Fig. 5.7.5: (a) Manhole located at the top counterweight; (b) inside of the counterweight

The internal situation of the lateral surfaces of the caisson and of the bracing cross profiles shows a advanced decay conditions (Fig. 5.7.5) as well as the extended cracks, one higher than one meter (Fig. 5.7.6).

Fig. 5.7.6: Crack in the north elevation of the caisson

The cracks have made timely and necessary an intervention to provide the safety condition of the crane with 12 temporary reinforcement rings (Fig. 5.7.7).

Fig. 5.7.7: Safety intervention with 12 temporary reinforcement rings

This intervention was evaluated appropriate since it was not possible to predict accurately the behavior of the counterweight during the partial emptying phase, a possible explosion or implosion caused by the removal of material.

Before of the application of the reinforcement rings were temporarily closed the inspection holes, in the south surface of the counterweight, by means of lead sheets (Fig. 5.7.8).

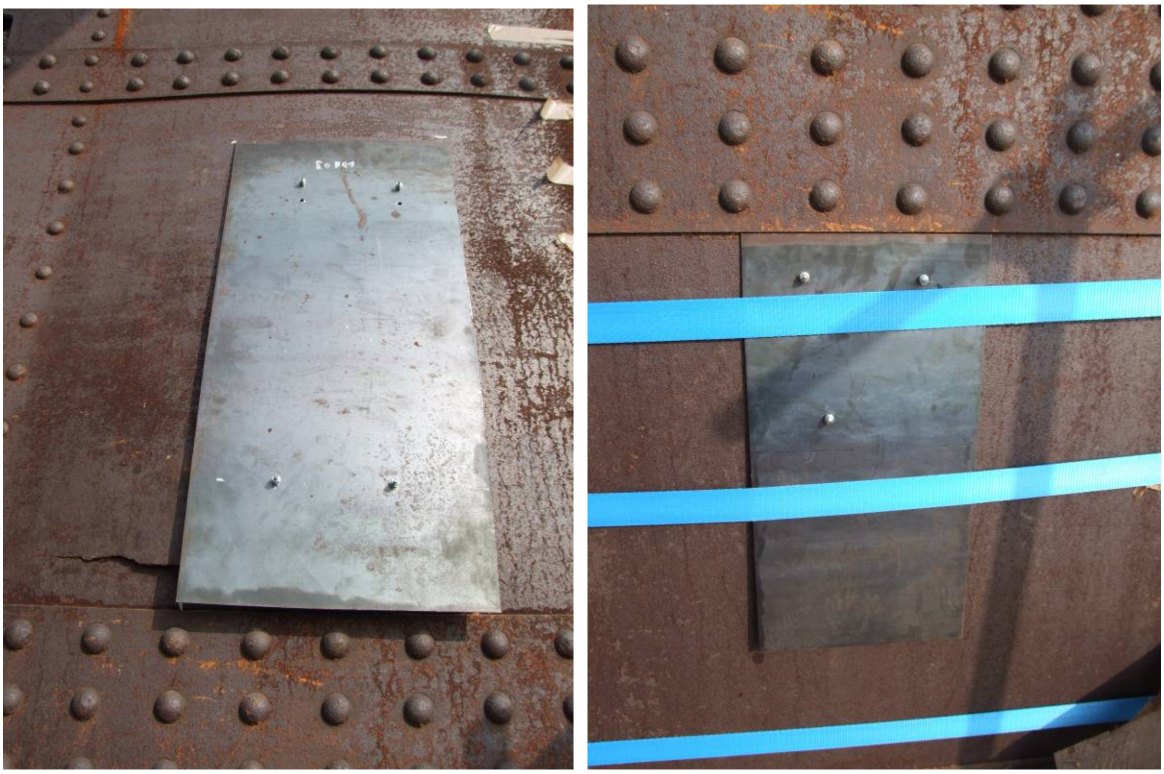


Fig. 5.7.8: Inspection holes closed with lead sheets

Partial emptying of the counterweight

Calculations and modelling showed that there was the necessity to decrease the weigth of the caisson equal to 100 tons. The removing of the material was done manually since any mechanical device would have cause the modifications of the internal stresses of the counterweight.

As mentioned above, the infilling at the moment of the opening was empty for an heigth of 1,10 meters from the upper manhole. With the emptying of 100 tons of materials the heigth of the infilling has been fallen by an additional 2 meters. This has permitted to evaluate the damage situation of the internal bracing system (Fig. 5.7.9).

Fig. 5.7.9: Bracing system

In the upper part of the caisson are present 5 bracing cross profiles. The central stiffening does not work any more in traction, due to a lateral breaking up of the two elements of the cross (Fig. 5.7.10).

Fig. 5.7.10: Central bracing cross profile

Opposite situation for the lateral bracing crosses in which there are profiles broken due to eccessive tensile stresses (Fig. 5.7.11).

Fig. 5.7.11: bracing cross with cracks due to excessive tensile stress

Finally, there are horizontal bracings and connection nodes between the horizontal and vertical profiles on the surfaces of the caisson in considerable state of deterioration, as reported in Fig. 5.7.12 e Fig. 5.7.13:

Fig. 5.7.12: Horizontal bracing

Fig. 5.7.13: connection node between horizontal and vertical elements

5.8 Methodological study and intervention phases

From the analyses carried out, as well from the results obtained through the investigation phase it was possible to define the achievable level of the restoration intervention, with particular reference to the foreseen integral conservation of the original constitutive material of the crane.

In the following sections, the operations connected with the obtainment of the equivalence of the vertical reactions in the supports of the crane, with the restoration of the balance load caisson, the conservation of original materials, the maintenance program and the successive fruition of the crane are described. Different subjects worked together (public bodies, universities and private companies) with their respective functions, for the successful final restoration design of the Armstrong, Mitchell & Co. crane.

The first analyzed phase corresponds to the study and evaluation of the most suitable methodology for balancing the upper part of the crane in the subsequent phases of emptying, restoration and successive re-filling of the caisson of the balance load. To do so, it was considered necessary the use of a FE model of the crane for simulating the behavior of the structure during the different phases. The analysis of the structure was performed starting to its present day conditions and successively decreasing the mass of the balance load, in order to determine the minimum load to maintain in order to avoid the global overturning of the crane.

Obtained values indicated the starting point for the analysis of the temporary balancing system, when the caisson will be completely emptied for restoration purposes. The choice of the most suitable methodology must fulfill the sought aim, trying to alter as less as possible the current equilibrium (stress distribution) in the trusses of the frame structure of the crane.

Furthermore, during the delicate phases of intervention it was foreseen, for controlling the stresses and deformations in the structure, the installation of a structural monitoring system devoted to the evaluation of the suitability of the applied interventions.

Once completed the structural restoration phase, the interventions considered for the material restoration are described in detail. The final goal of such a process is to bring back the crane in acceptably safe stability conditions, paying attention to the original material conservation issues, to render possible its fruition by the visitors of the Venice Arsenal.

5.8.1 Emptying and refilling the caisson

The most delicate operation in the activities of restoration of the crane corresponds to the repair of the balance load caisson. To do so it will be necessary to completely empty it, to substitute the deteriorated trusses, bracings included, and so to refill it again. Since the caisson serves both as balance load and structural element of the crane, during the intervention phases it will be necessary to design a temporary balancing system, in order to prevent the overturning of the crane. To assess the effectiveness and invasiveness of alternative solutions, FE models were employed.

As already described, a provisional intervention was already applied to strengthen the caisson of the balance weight. The operations consisted in the partial emptying (100 on 400 tons) of the caisson and the hooping, necessary to stop the progression of a wide crack, and to work with minimum safety conditions.

The first step was the assessment of the vertical reactions at the supports of the crane, when diminishing the mass of the balance load. Starting from the initial configuration, with a mass equal to 400 tons in the balance load, the stabilizing effect of the mass in the caisson was progressively reduced, with steps of 100 tons. From the evaluation of results, the following values for the gravity direction (Y) reactions in the four sliding blocks were obtained (Tab. 5.8.1).

Tab. 5.8.1: Y reaction values (gravity direction) of front and rear sliding blocks decreasing the weigth in the caisson

INTERNAL MASS OF THE COUNTERWEIGHT	Y REACTION VALUES (KN)	
	FRONT SLIDING BLOCKS	REAR SLIDING BLOCKS
400 TONS	- 861	- 1600
300 TONS	- 860	- 1130
200 TONS	- 860	-652
100 TONS	-860	-181
0 TONS	-860	+290

The above shown results indicate, as expected, that the reactions in the two front blocks of the crane are not subjected to variations with the mass diminishing in the caisson. The rear supports, on the contrary, located directly below the balance load, manifest remarkable stress reduction. From values in Tab. 5.8.1 it is possible to appreciate that , down to a balance load equal to 100 tons, the equilibrium condition are satisfied. With the caisson completely emptied, on the contrary, the equilibrium is lost, and a fictitious tensile reaction of 290 kN is found at the rear blocks (Fig. 5.8.1).

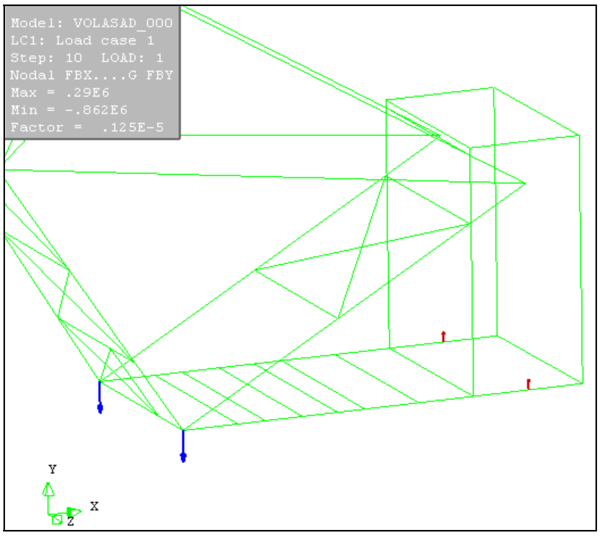


Fig. 5.8.1: Vertical reactions (N) in the four supports of the upper part of the crane, with the caisson completely empty

Vertical reactions in the rear sliding blocks, obtained with the caisson completely empty, represent the forces to be held by the temporary balancing system. With the aim of defining the minimum mass to be maintained in the caisson to still find the equilibrium, the reaction values in the sliding blocks were interpolated (Fig. 5.8.2).

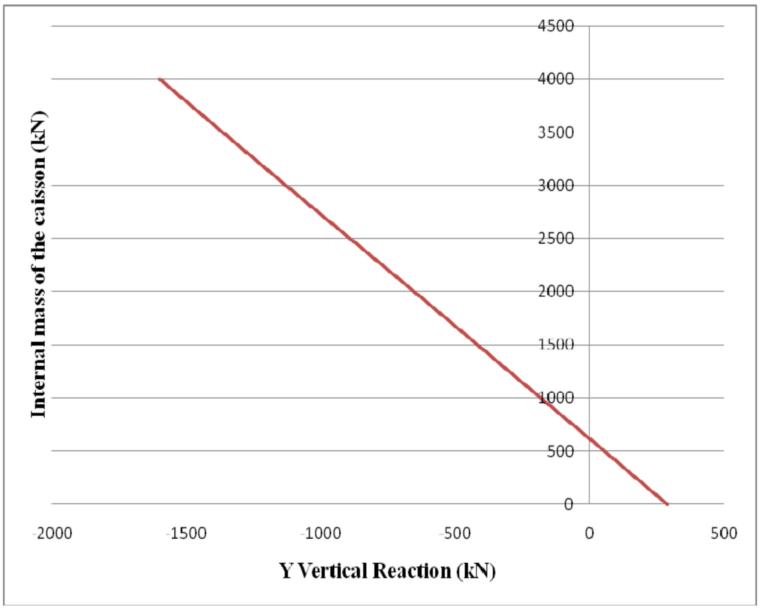


Fig. 5.8.2: Diagram with the decrease of the vertical reactions (Y) with varying mass in the caisson (kN)

The global overturning balance load limit of the crane, by FE modeling, corresponds to 620 kN (approximately equal to 62 tons).

Once obtained the forces to win, following the operations of total emptying of the caisson, the study of the temporary balancing system was carried out. The considered solutions were those possibly avoiding to alter the stress pattern in the trusses of the upper structure of the crane. To do so, the most suitable solutions seemed to be those considering the use of cables, to win the uplift forces, applied close to the balance load.

Cables must then act as tensile supports for the crane, preventing its global overturning. They will be connected to the caisson by means of two metallic beams, on the two sides, as shown in Fig. 5.8.3, in continuity with the trusses of the upper part of the crane, thus freeing the caisson from its direct structural role, and hence making possible its restoration. Such solution was also suggested by the conformation of the first Armstrong, Mitchell & Co. cranes, whose balance load did not played a direct *structural* role.

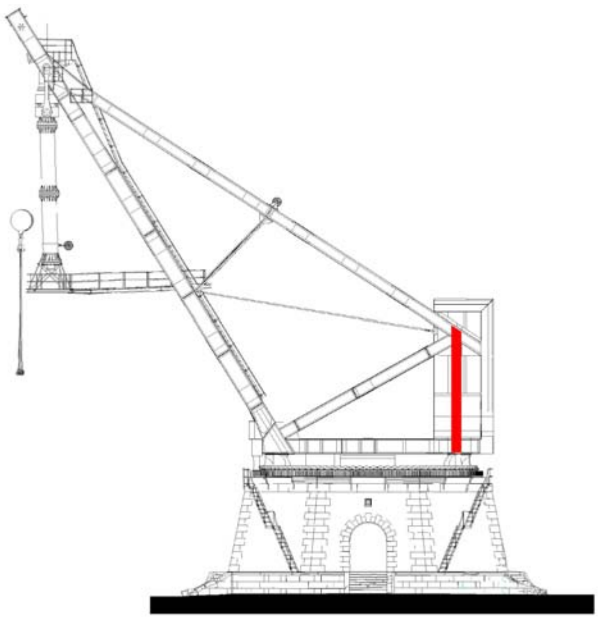


Fig. 5.8.3: Insertion of the metallic beams to be connected to the cables

In the FE model, the cables were modeled by means of L2TRU elements (three dimensional beams), with the following mechanical characteristics (Tab. 5.8.2):

Tab. 5.8.2: Mechanical properties of the steel cable

MECHANICAL PROPERTIES OF STEEL CABLE	
YOUNG'S MODULUS, E	220000 N/mm^2
POISSON RATIO, ν	0,30
DENSITY, γ	7900 kg/m^3

FE – linear - models were implemented, in order to realistically evaluate the tensile force in the cables, by removing the rear blocks. The vertical uplift force can only therefore be counteracted by the action of cables.

The first hypothesis (Fig. 5.8.4) considers the insertion, below the 4 sliding blocks, of cables anchored at the base of the basement masonry through stiffened stainless steel plates, with reduced inclination respect the vertical direction. Such solution aims at taking advantage of the all of the massive masonry bulk of the basement, since the cables are anchored almost at the base of it. The cables necessary to win the uplift vertical force related to the mass removal in the caisson are the rear ones. The resultant force in the cables – since they present an inclination with the vertical - can be decomposed on two components, vertical and horizontal. The horizontal component must be balanced by the cables positioned below the front blocks, which are therefore in their turn subjected to tension.

For the insertion of the cables in the basement it will be necessary to drill the masonry. For the insertion of the stiffened stainless steel plates in the basement, for anchoring of the cables to masonry, it will be also necessary to remove brick units at the foot of the basement.

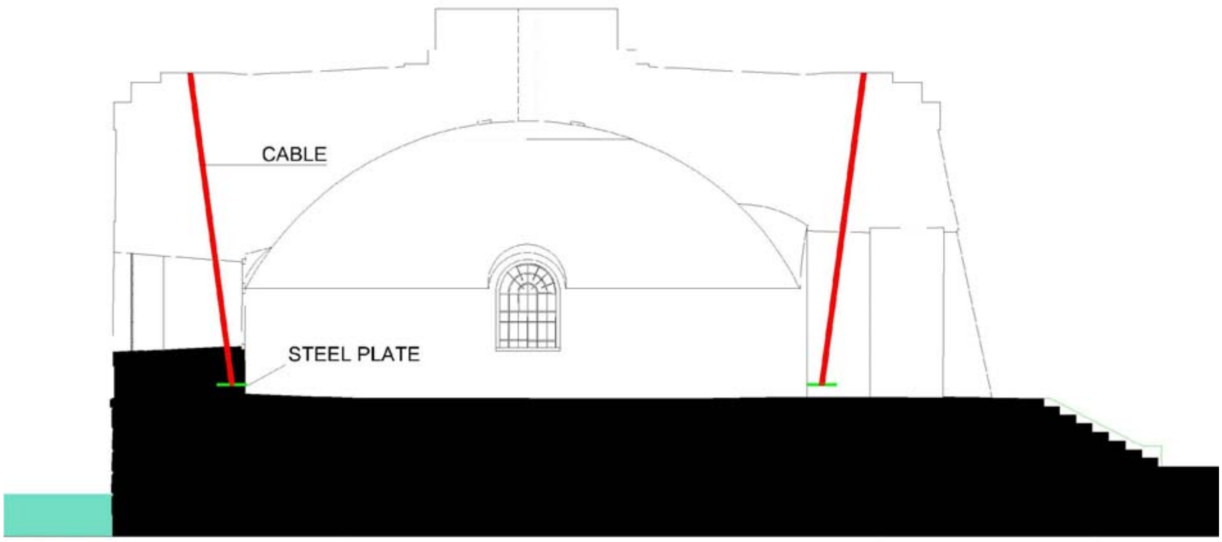


Fig. 5.8.4: Cables positioning layout at the foot of the basement

The intervention can be in any case considered reversible since, once finished the works and removed the temporary balancing system, holes will be filled with hydraulic lime mortar or kept open for possible future interventions or maintenance operations. Areas of masonry where bricks units were removed will be restored with the traditional intervention of "scuci-cuci" (masonry repair by substituting damaged or missing units with new ones), using the original brick elements.

The total mass to counterbalance, in the case of completely emptied caisson is equal to 290 kN per support, corresponding to a total uplift force of 580 kN. Cables will be anchored at the foot of the basement, using the total mass of it as balance load. The total mass of masonry constituting the basement portion used as balance load is equal to 5860 kN, approximately 10 times the uplift force to be won. Considered the significant force that cables must take, a couple of them per sliding block is necessary. They must be able to bear a tensile force of 145 kN each. According to UNI ISO 4309:2008, a safety factor of 5 has to be considered for the calculation of the design force in cables, thus dividing the break load by a factor of 5. The break load per cable should be then higher or equal to 725 kN. High resistance cables were then selected (steel strength equal to 2,20 kN/mm^2), with break load equal to 755 kN for a 30 mm diameter cable.

Selected cables will be anchored to masonry through stainless steel flanged plates, dimensioned for stress distribution on masonry. To evaluate the minimum dimensions of the plates, the already calculated maximum compressive stress of brickwork masonry are compared to the stresses determined by means of a FE detailed model. The plate was modeled (three dimensional model) by means of 2D plate elements, with the material properties indicated in Tab. 5.8.3.

Tab. 5.8.3: Mechanical properties of the steel flanged plate

MECHANICAL PROPERTIES OF STEEL FLANGED PLATE	
YOUNG'S MODULUS, E	196000 N/mm^2
POISSON RATIO, ν	0,28
DENSITY, γ	8000 kg/m^3

The FE model comprises the plate, the 4 stiffening flanges and the passing hole for the cable insertion. It was then applied the cable design force (145 kN) to the corresponding plate elements, and masonry was modeled as a bedding of brick elements with stiffness resulting from the standards indications (Fig. 5.8.5).

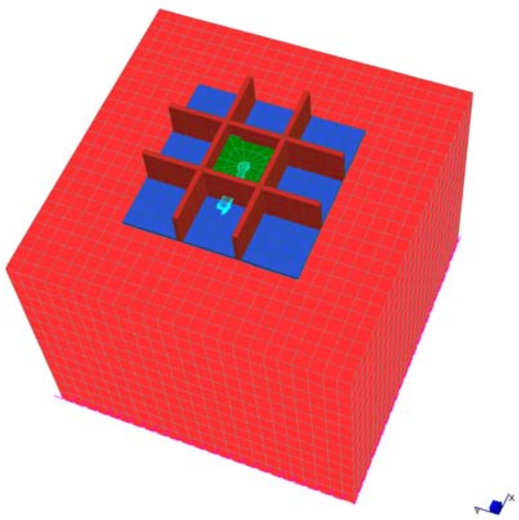


Fig. 5.8.5: Detail of the cables anchoring plate located at the foot of the basement

The stainless steel plate, considered as satisfactory by the FE modeling and the successive calculations, presents sides of 540 mm x 540 mm, and thickness 20 mm. As shown by Fig. 5.8.6, stresses in the steel plate are far lower than the design yield strength.

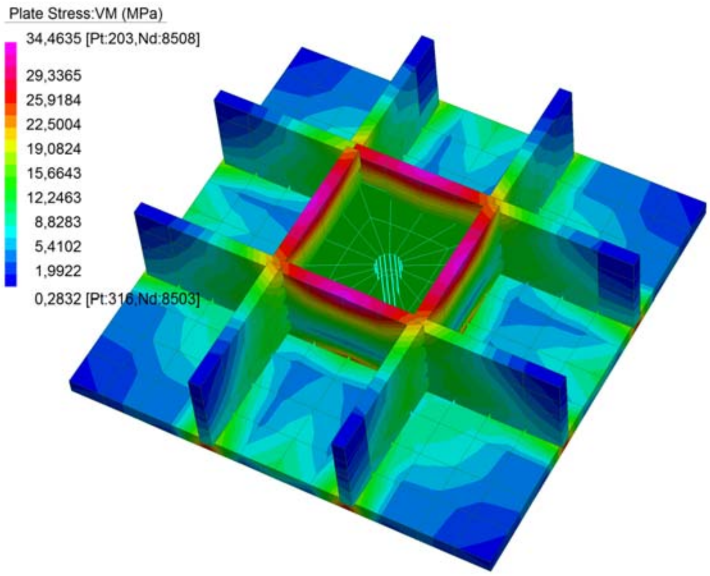


Fig. 5.8.6: Von Mises stresses in the anchoring steinless steel plate (MPa)

Stresses at the interface with masonry present limited values, around 0,9 N/mm^2, with maximum components close to the center of the plate, and decreasing with the distance (Fig. 5.8.7).

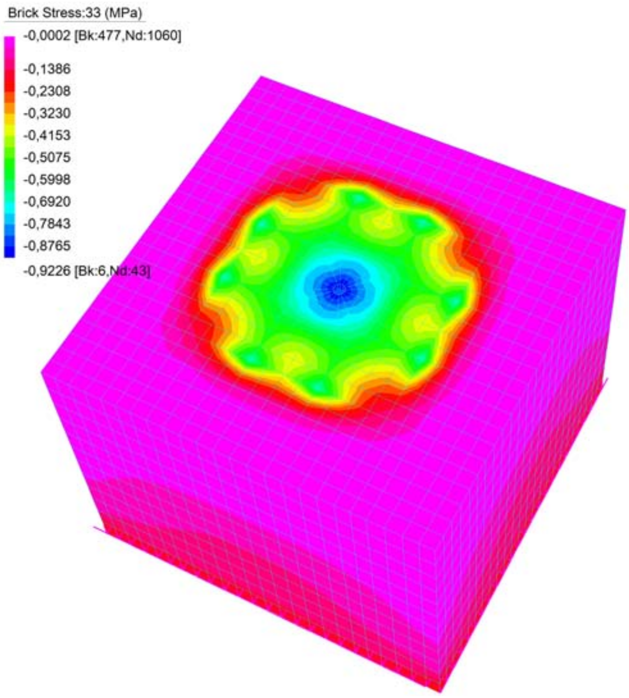


Fig. 5.8.7: Compressive stresses at the interface between steel anchoring plate and masonry (N/mm^2)

Once verified that stresses in the steel plate comply with the standards requirements, structural details were drawn (Fig. 5.8.8). Stiffening flanges present a thickness equal to 16 mm.

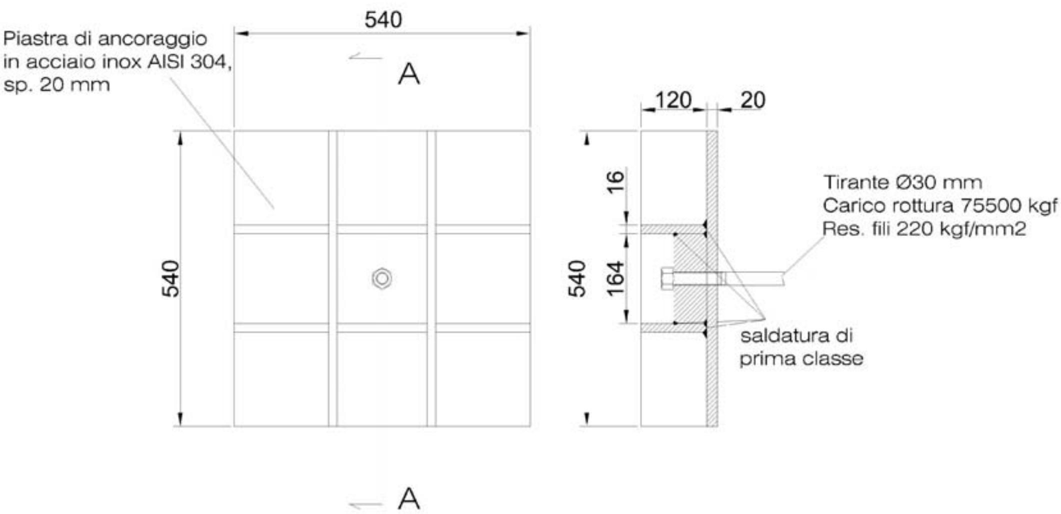


Fig. 5.8.8: Detail, cables anchoring stainless steel plate (mm)

A second design hypothesis was considered, in order to prevent from a deep masonry core drilling: cables are now anchored to the vault, in the upper part of the basement (Fig. 5.8.9).

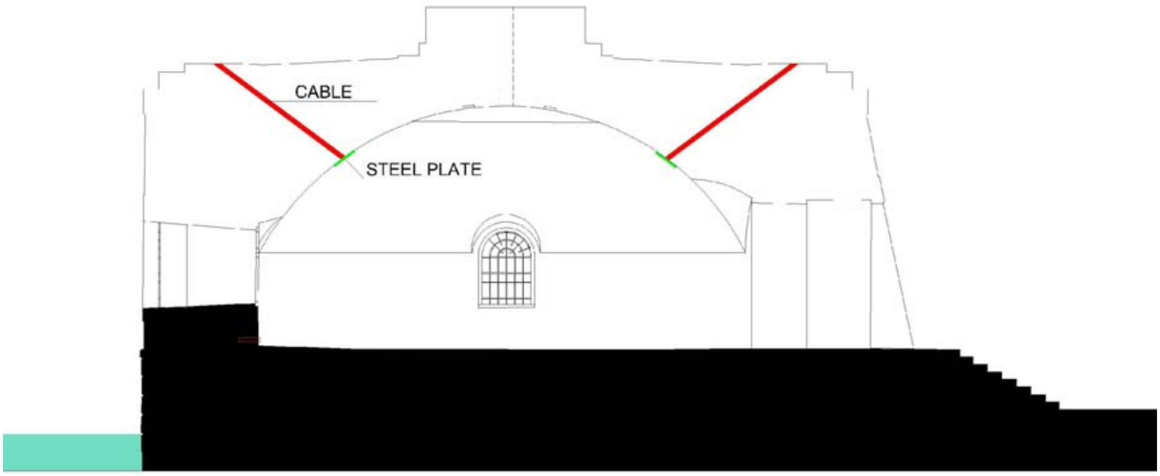


Fig. 5.8.9: Layout of the positioning of the cables anchored to the vault in the upper area of the basement

The aim is in fact to insert the cables in the supply ducts for the machineries of the slewing ring, boring through the vault of the basement (Fig. 5.8.10).

Fig. 5.8.10: Detail, water drains

As for the previous case, cables were inserted below the rear sliding blocks: the vertical uplift force induces also here a horizontal component in the cables, counterbalanced by the cables positioned in the front area. The length of the cables is lower than before, but the angle with the vertical of the same cables is remarkably increased, with a value of 32°. This inclination generates relevant stresses in the cables (Fig. 5.8.11)

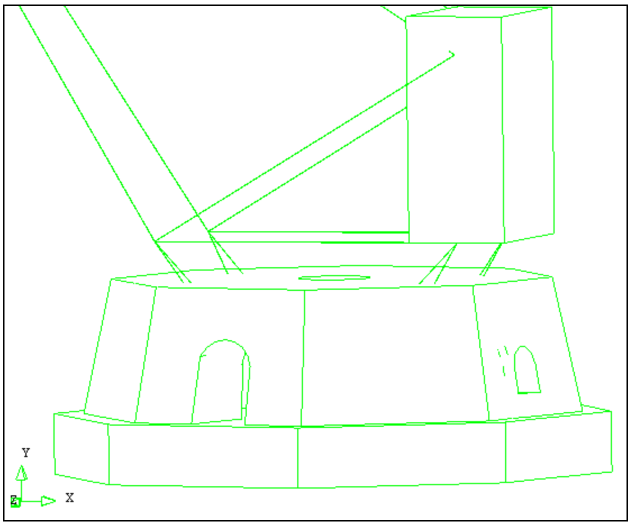


Fig. 5.8.11: Localzation of the cables in the vault

From the FE model results, also considered the remarkable inclination of the cables and the consequent force increase, it emerges that a higher number of cables – respect the previous hypothesis - is now needed.

The two proposed solutions have pro and contras. The first solution, with the cables anchored at the foot of the masonry basement, considers slightly invasive operations as coring in the brickwork masonry, and the removal of part of masonry units for anchoring the steel plate. On the other hand the balancing force needed results to be moderate, given the reduced inclination of the cables, and then two cables per support are needed.

The second solution, with the cables fixed at the intrados of the vault, does not necessitate invasive interventions in the masonry elements, since the water drain holes will be used for the cables passing through the masonry. However, besides the necessity of having an increased number of equal diameter cables because of the higher resultant force, non negligible compressive stresses are then induced to the horizontal trusses at the base of the upper structure of the crane, due to the horizontal component of the force in the anchoring cables. Since one of the goals of the structural assessment is to find a stabilizing system able to reduce to minimum the alteration in the stress fields of the crane's elements during the temporary emptying of the caisson, the selected solution is the one considering the anchoring of the steel cables at the foot of the basement, also considered the deterioration in the metallic trusses at the base of the upper structure of the crane.

The installation of a dedicated monitoring system, composed by strain gages, displacement and force transducers, as well as accelerometers, is scheduled during the successive steps of the interventions, in order to ascertain the stresses and strains in the existing metallic elements of the crane and for balancing the tensile forces in the newly positioned cables.

Once installed the provisional crane stabilizing system and the monitoring system, it will be possible to remove the filling material in the caisson. As for the provisional stabilization interventions phase, the material removal will be done by hand, without aid of excavators or other electric tools, for not excessively stressing or damaging the sheet metal parts of the caisson. Then, the restoration or, where necessary, the substitution of deteriorated material will take place, in the internal and external parts of the caisson. The metallic elements to be restored will be cleaned and/or lightly sanded, and the damaged rivets will be substituted.

As previously described in the chapter related to modeling, to equal the vertical reactions in the four supports, a mass of 250 tons is requested as balance load. The material currently filling the caisson is not however acting just as load, but also to prevent buckling in the metallic elements. So, for not modifying the original arrangement of the balance load, filled up to a height of 7,60 m, it was considered the possibility of using a reduced density material respect to the one currently employed, for leaving, after the restoration interventions, the same volume of material inside the caisson. The most suitable material emerged to be blocks of lightweight concrete and sand. Such filling material can also be more easily removed in future maintenance or repair interventions.

5.8.2 Material test and monitoring

Material tests and monitoring will be done in order to mechanically and chemically characterize the crane's materials and to evaluate the effectiveness of the recovery.

The tests on materials will just concern puddle iron and cast iron (Fig. 5.8.12).

Fig. 5.8.12: Details of iron and cast iron elements

The basement's masonry does not show a severely degraded situation, so that cleaning treatments along with consolidation and protection of the external will be enough. These jobs does not require an extensive knowledge of the chemical and mechanical characteristics of bricks and of Istrian stone.

The slewiong ring and the boom of the crane consist of metal alloys and, in some significant areas of the structure, there is a clear decrease in the cross section of the profiles mainly due to oxidation, stripping (Fig. 5.8.13).

Fig. 5.8.13: Details of the metal elements degradation

The tests in the laboratory will require the taking of samples and their preparation, including cutting, withdrawal of the metallographic section and any embedding, polishing and metallographic attack. The withdrawal includes the removal of rivets in different parts of the structure (rods, nodes and counterweight). The resulting data will be used to define the critical areas where it will be necessary to replace some or all of the rivets (Fig. 5.8.14).

Fig. 5.8.14: Details of missing or degraded rivets

To evaluate the thickness of the cross section, of the nodes and of some critical points of the main rods, and to know the mechanical properties of cast iron, it is advisable to do optical microscopy tests with image analysis, chemical analysis of metal alloys and scanning electron microscopy with microanalysis. Some of these tests were carried out directly on site during the knowledge phase.

To get a complete picture of the situation and to create a finite element model that best approximates reality, other fundamental studies are accelerated corrosion tests, that allow to estimate the corrosion resistance in saline environment of metallic materials with or without protective layers.

Some rollers (Fig. 5.8.15) of the training system serve as support for the four sliding blocks and are affected by heavy oxidation. The removal of one cast iron roller will be done in order to perform laboratory tests aimed to assess the strength, to assess if there are micro cracks inside and to write a recovery program of the slewing ring in the event of a future restore of the crane’s operation.

Fig. 5.8.15: Detail of the rollers upon which one of the four sliding blocks lies

In addition to laboratory tests, there will be a test on site with a portable dynamometer to determine the strength of metallic materials of the structure.

Furthermore, to determine the effectiveness of the intervention of temporary balance of the crane and its efficient operation, a monitoring system will be installed, that consists of accelerometers, electrical strain gauges and displacement transducers. These instruments provide information on changes and stresses acting on the structure that allow to calculate their deformations. They are therefore necessary in order to assess the tension of each metal rod and to balance the boom of the crane during the counterweight recovery phase.

The assessment of rigid movements and stability of the boom of the crane’s structural elements is determined by monitoring with high precision inclinometers. The latter measure changes in the inclination of both the horizontal and vertical planes. The absolute and relative movements of the boom of the crane will be measured through a precise leveling difference, in order to obtain its complete mapping.

The last two monitoring systems described above will remain operational for a longer period than the recovery of the structure to assess the correct behavior of the crane over time.

5.8.3 Treatments and metal components replacement

The analysis of the degradation of the crane’s metal components showed a diffuse and generalized corrosion process, having different levels of intensity and severity depending on the constituent material (steel or cast iron) and to exposure conditions of the surfaces. Similar considerations can be done about the rivets.

Therefore an intervention program was planned that includes the preliminary survey with overall cleaning by compressed air. This operation is aimed to assess the actual building’s condition in each part, especially in the more recondite one (rolls, sliding blocks, and the bottom part of the boom of the crane), to identify the surfaces containing original layered paintings (Fig. 5.8.16), to map the degradation and to identify critical points where the degradation has produced the worst surface damage and / or structural damage.

Fig. 5.8.16: Different layers of paintings

In order to prevent the degradation progress, filling by epoxy resin will be done on the cracks present in the cast iron sliding blocks and in the metallic elements where parts are missing. Such intervention should also be done in cases where there is a gap between the joints of metal profiles (Fig. 5.8.17) in order to prevent water infiltration.

Fig. 5.8.17: Separation between the cracks of the profiles

All surfaces should be treat by sand blasting, to clean them enough not to see rust and foreign substances, except for the parts with paintings, on which will be done sand blasting by equipment and by abrasives suitable to perform a non-aggressive cleaning of the treated material, or a cleanup by protective methods in order to prevent the removal of traces of the original layered paintings.

After the cleaning phase a penetrating primer will be laid that is both a protective agent and a rust converter. The final step will be the protective action of metallic structure (steel or cast iron) through the application of a facing.

For the consolidation of the boom of the crane, especially the upper part of the counterweight, which includes internal bracing system, stairs, landings (Fig. 5.8.18) and balustrades the replacement of degraded metal profiles is expected.

Fig. 5.8.18: Stairs and landings degradation

The rivets to be replaced must be tailor-made preserving the original size in inches.

5.8.4 Wooden elements treatments and replacement

The wood in the structure under study is present on a part above the slewing ring, which served both as protection for the upper engine room, and as a stiffening for some profiles of the lower rods of the boom of the crane.

A visual analysis shows a clear need to remove and replace some wooden beams that are unrecoverable and/or rotting (Fig. 5.8.19) with well-seasoned wood beams of the same essence and size of the existing ones.

Fig. 5.8.19: Detail of unrecoverable wooden beams

In order to preserve the crane's history the wooden structure located above the engine room will be rebuilt. The cabin in fact burned in a fire around the 70s of last century (Fig. 5.8.20).

Fig. 5.8.20: Conditions of the cabin after the fire

Currently the damaged material has been removed but the reconstruction is vital for the protection of the machinery (Fig. 5.8.21).

Fig. 5.8.21: Current condition of the engine room

The structure will be rebuilt following the original design (Fig. 5.8.22), but using a different type of timber to show that it is not the real one.

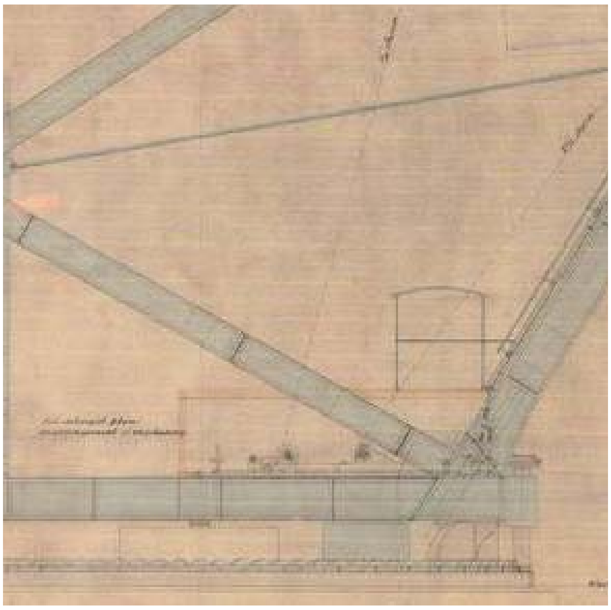


Fig. 5.8.22: Extract from Armstrong Archives

Beams and timber in good condition (Fig. 5.8.23) will be cleaned and the wooden surfaces will be treated, by brushing and washing with a surface acting agent.

Fig. 5.8.23: Beam detail that needs cleaning and processing

5.8.5 Masonry and Istrian stone treatments and interventions

The crane's basement is built in masonry and Istrian stone. A visual analysis allowed to evaluate the level of degradation, which is superficial. On a structural level there aren't special and important issues, thanks to the skill in carrying out the work and to the thickness of the wall, that is more than 2.00 meters at the basement (Fig. 5.8.24).

Fig. 5.8.24: View of the basement and of the wall’s thickness

The interventions for the recovery of the foundation are not invasive and involve only the superficial part of the basement.

The first thing to do is to wash the whole affected area. Following there is the removal of lichens and mosses to be carried out by mechanical instruments and subsequent dry or wet brushing. As for the bulging of the wall with ejection of material, a replacement of walls will be done using the unstitch-stitch technique. The joints between bricks should have the same characteristics as the existing ones. The new bricks to be used during the unstitch-stitch phase will need to have size, color, mixing and consistency similar to existing ones.

All lesions and cracks will be filled and the formation of a substrate is included, to be carried out with hydraulic white desalinated lime and inert materials having particle size, mechanical and colors similar to the existing mortar.

Once all the work listed above is completed, the Istrian stone and the masonry will be cleaned from surface deposits. The wall surface’s recovery will be finished by applying a protective agent.

The stone elements (doors, door jambs, moldings, architectural frames and Istrian stone surfaces) will be cleaned to remove stains of paint color, fatty deposits and overlapped treatments. The following jobs will be done: grouting of cracks and of the joints of the blocks, gluing fractured parts with epoxy, integration of small missing pieces of marble.

In the inner surface of the basement a cleanup operation from surface deposits will be done.

Further steps that are required to maintain a good climate and a low level of moisture in the engine room, located inside the basement, consist in the maintenance of wooden windows and the restoration of the entrance door.

Fig. 5.8.25: Details of the windows and of the wooden door of the basement

Another needed intervention is the cleanup and the eventual filling of the boiler's flue gas pipe (Fig. 5.8.26) located in the engine room and the subsequent installation of a transparent sheet and a lighting system to make it visible and inspectionable.

Fig. 5.8.26: Details of boiler's flue gas pipe

5.8.6 Recovery of cast iron and bronze machinery of the crane

The machines, which allowed the operation of all crane's elements, are located both inside the basement and above the slewing ring, and they are made of cast iron and bronze (Fig. 5.8.27).

Fig. 5.8.27: Cast iron and bronze machinery

Currently they are in a remarkable state of degradation caused by lack of maintenance. The deterioration has so much altered the structure that is unthinkable for them to operate again. To keep alive the history of the crane and its operational techniques a treatment program is necessary. For both materials a surface cleaning by removal of deposits and a treatment with corrosion inhibitor and protective agent are expected.

5.8.7 Draining system

Further problems found in the crane are the deterioration and the decay of the outside of the vault. In fact, near to the laying level of the slewing ring the original slopes are no more present and water deposits formed on some areas (Fig. 5.8.28). In fact the water no more flows through the draining holes. For this reason it was decided an operation to fix the rainwater draining system of the slewing ring, by the mean of lightweight concrete casting, and its restoration through the installation of collection and drainage networks.

Fig. 5.8.28: Backwater problems at the top of the vault

5.8.8 Valorization and fruition

The aim of restoration is to give to the future visitors of the Arsenal the crane in his full value as machinery, as witness and as monument, making it somehow usable. The crane will have to evoke what it has been for the history of Venice and the Arsenal, and this can be achieved with extensive illustrations, making its parts as open as possible and letting the visitors know also its “dynamic”. We are aware that this would necessarily make it meet much stricter safety standards and that this probably means to make changes and installations that threaten its old material.

Some measures could be aimed at making open the interior of the basement, where machines are. It would also be interesting to allow visitors to climb where the slewing ring is. To let this happen iron railings should be integrated, to avoid falling objects and people, with possible installation of protective nets. The vertical connection systems between the base and the top of the slewing ring should also meet the safety standards.

5.9 Final remarks

The *knowledge* phase and the structural assessment of the Armstrong, Mitchell & Co. Crane at the Venice Arsenal allowed to obtain a complete and satisfactory picture of the historical evolution, of the geometrical and constitutive materials configuration, as well as of the current safety state of the structure.

In parallel with the bibliographical research on the cranes built by the Armstrong, Mitchell & Co. company around the world, specifically on the Venice Arsenal's one, the investigation on the present-day conditions of the crane allowed to identify the elements and the materials constituting the basement and the upper part of the structure. Starting from this approach it emerged the necessity to obtain a brand new complete geometrical survey, not found in literature. Different technologies were then applied for doing so, from the most up to date methodologies as the laser scanner technique to the more traditional ones, comporting the use of meters and calipers. The merging of the obtained results led to the creation of plan views, elevations, cross sections and of a three dimensional model.

Once obtained the basic geometrical information, it was possible to investigate the material decay and to study the structural configuration. The peculiar characteristic of a machinery is movement, and it is thus necessary a continuous and costly maintenance to guarantee the efficiency of all of its parts, especially in outdoor maritime conditions. Decay for oxidation processes is in fact more harmful to the gears rather than to the metallic structure itself. Similar conclusions can be drawn for rollers. Some of them present lack of material, particularly those supporting the sliding blocks, and so the entire upper structure. Only assumptions can be made on their ultimate bearing capacity, since their mechanical characteristics and inner compositions are unknown. The crane will however remain in its original location and this signifies that it will be continuously exposed to the same external phenomena which caused – in the last decades – its decay. It will be thus necessary to schedule a maintenance plan, especially referred to the surface protection of metallic elements.

The difficulties and operating particular nature of restoration interventions were determined by the structural complexity of the crane and its peculiarities as moving machinery. It was in fact noted that the "out-of-service" of a machinery is far more harmful than for a building. The steadiness leads to concentrated loads, ovalization of rotating parts and to possible collapse for fatigue phenomena.

For the main beams and bracings the material deterioration does not seem critical from a structural point of view. The most decayed areas are those less subjected to the principal winds, thus, by consequence, those where salts and particles find a spot to settle. The basement is well preserved, being a state of the art traditional construction made of thick brickwork masonry walls whose connections at the corners are improved by the use of white Istria stone. The most worrying deterioration phenomenon is the presence of shrubs which are causing the breakage and consequent fall of bricks.

As it clearly emerged during the study carried out, a major problem localize in the balance weight. It is in fact subjected to a huge crack, one meter long and several centimeters wide, which may undermine the balance system and thus the whole crane, with particular reference to the possible prosecution of the crack and related leakage of material (sand, pebbles…).

From a structural point of view, the most worrying problems are connected to the material decay in the sliding blocks and in the balance weight structure. This last was in fact necessitating a rapid provisional strengthening intervention, prior to the execution of the definitive conservative restoration. Preliminary calculations were then carried out on the equilibrium of the crane, with the aim of increase its safety conditions, by removing part of the material contained in the caisson, and so decreasing the compressive stress in sliding blocks and bearing rollers.

The final part of the work consisted in the definition of the interventions and maintenance program, always having in mind the reversibility issues, one of the fundamental concepts of restoration. Between the foreseen operations, the most delicate consists in the total emptying out of the caisson, in the strengthening of the deteriorated beams, and in the successive filling of it, always guaranteeing the equilibrium to the structure. For doing so, it was found a solution – after several discussions and the evaluation of a number of different alternatives - which does not sensibly alter the equilibrium in the reticular elements of the upper part of the crane during the successive restoration phases of the balance weight. The chosen system consists in the application of a series of cables deeply anchored to the masonry basement of the crane, preventing the global overturning caused by the balance weight removal.

Crucial for the evaluation of the effects of the intended interventions on the structure was the use of software FEM codes modeling, since it allowed to determine the variation of force at the reactions (=sliding blocks) during the phases of removal of the balance weight, and to simulate alternative systems for the provisional balancing of the upper part of the crane, enabling to identify the most suitable solution.

The design finished with the material restoration proposals on the different elements composing the crane. The will of preserving as much as possible the original materials, indicated the possible approach in the case of minor damage, e.g. detected in the metallic elements, that is to say rather local material strengthening than substitution.

6 CONCLUSIONS

Restoration and conservation of cultural heritage is now a topic much discussed and in evolution both in Italy and abroad. A regulatory-wide are creating and refining of a study procedures by means of the drafting of guidelines which intent is give a methodology and a sensitization at the problem at academic level, of public organs that control the conservation and the protection of the cultural heritage, and at the private technicians that work in this field.

In this context, it is insert the study of this research. Were taken into account several standards and has been identified a integrated and multi-disciplinary method for the study of cultural heritage, created in function of the actually regulatory and in respect of the fundamental standard of restoration and conservation, described in the introduction of this thesis. This method was made as generic as possible, without give particular information inherent specific restoration techniques, in such a way that is applicable at all the types of buildings with elevated historical-cultural valence.

To validate this approach, the identified methodology has been applied a two different case studies that differ in geometry, materials, structural behavior and intended use: the Sala Maggiore (Main Room) of the Sale d'Armi Nord (Northern Weapons Rooms) and the Armostrong, Mitchell & Co. hydraulic crane. The choice of the two case studies was dictated by the concrete requirements for restore the monuments by the side of the Soprintendenza B.A.P. di Venezia e Laguna. The aim is to recover the two case studies with the restoring to their original materials and structural behavior.

For the definition of a project of restoration and conservation dealing different aspects of the problem, a group of study has been created with different persons according to the competences. The various professionals have collaborated in an integrated way, uniting and elaborating the specific knowledge's of the proper reality, as well as preparing the intervention project.

The first fundamental step has been the historical search. In fact, even if for different motivations, for both the cases study there were no information about the transformations and the motivations that have brought to the construction of such works. It has started by a search on the arsenal in Venice with the purpose of knowing the various changes suffered by the whole complex during the centuries. Subsequently a search has been developed more deepened for each structure. From an ample bibliographical research the connected information to the objective of the project of restoration are been identified, eliminating the superfluous information, and appraising the reliability of the sources, looking therefore to reconstruct the structural history of the patrimony. It must be remembered that, in the greatest part of the cases, the historical documents were written for different goals from those of the structural engineering, and for such motive there can be some wrong technical information brought and/or some information can be omitted or structurally distorted from the key facts or from the meaningful events (ISCARSAH 2005). It is therefore necessary, in this phase, to have a clear and complete idea of the information retrieved, in order not to depart from wrong or irrelevant data from the structural point of view.

The study has then been concentrated in the description of the present-day condition and in particular way in the geometrical, material, structural and of the degradation knowledge of the elements constituting the two cases study. The analysis of the present-day condition has the purpose to identify all the constituent elements the structure, their structural function and the materials. In the study of historical buildings, it is easy to find different types of materials from different times following the various interventions that the manufact has suffered during the centuries.

The transformations can be due to the evolution of the construction techniques, to destructions, fires, bombardments, collapses, or often to changes of use destination. Important base for the start of the diagnostic phase is the geometric survey. It has been developed, for both case studies, both using innovative techniques and traditional ones, placing side by side the direct measurement, to techniques such as photogrammetry and laser scanner. Particularly, for complex structures like in the case of the crane, the actual techniques even if sophisticated, are not able to investigate completely the manufact. There is therefore the risk to lose important information. It is opportune to unite the data drawn by these methods with those more punctual of the direct survey. In the specific case of the crane the measurements manually developed have allowed the identification of the profiles' sections, underlining diminutions and discrepancies, fundamental information to value the actual state of the building.

The identification of the constituent materials, the structural analysis and the analysis of the deterioration foresees a deep and accurate knowledge of the obtainable building through inspection in site and, where possible, tests for the determination of the chemical and mechanics characteristics of the materials. In the historical assets it is trying to avoid, as far as possible, destructive tests that can alter the structure. Therefore non destructive tests and/or partially destructive tests were prefered. In the specific case (for ex. For the reinforced concrete trusses covering the Main Room in the Venice arsenal - Sala Maggiore dell'Arsenale di Venezia), the tests developed for the identification of the mechanical characteristics and of the distribution of the reinforcement bars of reinforced concrete trusses. In the case in which the information of the tests on the materials were not exhaustive to the goals of the verifications and to the following project of intervention has been developed, in according with the current Italian standards on construction (D.M. 14/01/2008), the simulated design. It corresponds verification to the verification of structures by applying the regulations in force at the time of construction of the structure. Regarding therefore the prescriptions imposed by the codes, it is possible to calculate, for example, the necessary quantities of reinforcement bars that, integrated with the geometric and structural survey, furnish the necessary data for a satisfactory characterization of the structure. In the present research an example of the case of reinforced concrete trusses of the *Sala Maggiore* is reported.

The obtained information has been gathered in summarizing tables for a complete and immediate understanding of the building, and for a base of future studies. The tables contain the general survey of the structure and the constructive details about geometry, materials, state of degradation and structural layout.

The preliminary knowledge phase has allowed us to underline a complete picture of the principal problem of the structure, the first step for the definition of the restoration and conservation project. The fundamental support for the structural evaluation and the identification of the developed interventions has been the modelling. Regarding the importance of the building and the objectives to reach, a global and detail numerical finite element models are created. For the *Sala Maggiore* the global model of the structure has furnished information on the structural behaviour of the masonry that, together with on-site surveys, has brought to exclude the necessity to focus the attention in such areas. The models of detail have been used however for the study and the verifications of the roofs that have precarious conditions due to a diffused state of degradation. For the crane Armstrong modelling has served to evaluate the global behaviour and the stresses in the various elements of the structure, allowing to develop the opportune considerations for both the masonry basement and Istria stone and for the lattice boom and the counterweight in cast iron and puddled iron. Besides it has been a useful tool for the definition of the intervention phases of the restoration allowing, through various simulations, to identify the most opportune solution. This way it has avoided making invasive or wrong intervention projects that could have altered in meaningful way the behaviour and the equilibrium of the crane.

Following all the phases above mentioned, it has been possible to draw a program of interventions in the respect of the fundamental principles of the conservative restoration, such as the maintenance of the material-structural history of the building into consideration, the compatibility with the original materials and the minimum intervention. For the project of the *Sala Maggiore* three different types of intervention have been developed: reconstruction, conservative restoration and structural reinforcement. The reconstruction has been concerned a portion of the roof with a wooden trusses that recently suffered a collapse. The intervention has been thought to rebuilding the trusses, the connection joints of the elements and the stirrups of reinforcement according to the traditional techniques of the sixteenth-century. Before the layout of this type of project, a technical preparation is necessary on the past construction methods obtained through the bibliographical search and through a study on the constructive historical indications. For the not collapsed trusses, once verified all the elements' sections constituting the trusses, a project of conservative restoration has been made. The connections at the nodes were restored, portions of original material in advanced state of decay were substituted and protective coatings were applied setting particular attention to the elevated umidity conditions of the site. Finally for the study of the reinforcement intervention in the further covering area, the trusses in reinforced concrete were considered. They have a strong state of degradation of the materials due to, in a particular way, to the exposure to atmospheric agents following the partial collapses of the surface roof. The opportune verifications imposed by the standars have been done and, in the cases on which they were not satisfy the requests, an intervention of reinforcement has been proposed. The choice of the technique to be used for increasing the resistance of the structure has been dictated by the basic principles of the adopted methodology, such as the maintenance of the geometry, of the aesthetics and of the original material configuration, the minimum intervention and its reversibility. One of possible solution, in the specific case, that can satisfy such requisite is the application of Fiber Reinforced Polymers (FRP). Since the model from which the maximum stresses were obtained, desplite having applied the corrective coefficients, did not keep into consideration the decay. For this reason it has been decided to intervene, even if the verifications were satisfied, in the most critical sections emerged from the degradation analysis, developed in the preliminary phase of knowledge of the structure. It is confirmed therefore the importance of the continuous integration among the technical-scientific and historical-critical data.

For the Armstrong crane, following the preliminary phase, a critical degradation situation has emerged. It had needed an intervention to provide safety condition of the structure immediate and effective, while waiting for the definitive one. The conditions of decay of the counterweight and of the sliding blocks to connect the lattice boom to the basement were worrying. For this reason a project of emptying the caisson has been done. The actual critical situation is due to the inactivity of the machine for several decades and creates a consequent degradation state of the elements that currently support the greatest part of the weight. With the support of the numerical model and the calculations, the quantity of weight to be removed from the caisson has been estimated. Considering the fragility of the actual structure, the on-site operations have been studied in the detail so as to operate in the most respectful way of the actual equilibrium conditions, reached in the years by the structural elements of the crane. In fact it has been observed that the removal of the material could not done through mechanics ways since the vibrations would have risked to jeopardize the situation of the counterweight. Some alternative solutions have therefore been valued that have allowed to reduce to the minimum the risk . Concluded the provisional interventions it was passed to the study of the structural intervention that consists in the emptying, the restoration and the following filling of the counterweight. This operation has pointed out the necessity of a provisional balancing system that has been chosen, following various modelling simulations, paying attention to the minimum alteration of the structure's equilibrium. In this operation, it has been needed a structural monitoring system that allows to study step by step the intervention and to appraise possible corrections during the progress of the project, as well as to validate on the long-term the planning choices made. The project was concluded with the techniques and the products to use for the conservative restoration of the crane, foreseeing a series of preventive chemical and mechanical tests on materials.

In collaboration with the *Soprintendenza B.A.P. di Venezia e Laguna* (local office of the ministry of fine arts responsible for the conservation of the cultural and landscape heritage of Venice and its lagoon), the interventions of reconstruction and conservative restoration of the wooden trusses have been termined. Instead, the operations of structural reinforcement of the reinforced concrete trusses are in progress. For the crane Armstrong have been concluded theprovisional interventions and while waiting for financings the restoration project will be honed.
Concluded the interventions of restoration and recovery of cultural heritage, it is important, also from an economic point of view, to establish a program of organized maintenance. As schematically underlined in the following figure (Fig. 4.6.2) the degradation of a structure proceeds with non linear course (indicatively exponential) in comparison to the time evolution and to the lack of maintenance for long periods it creates a notable economic loss. Instead, with a program of preventive maintenance, the structure will maintain the expected configuration following the interventions of restoration and following structural stabilization, with investments contained and diluted in the time.

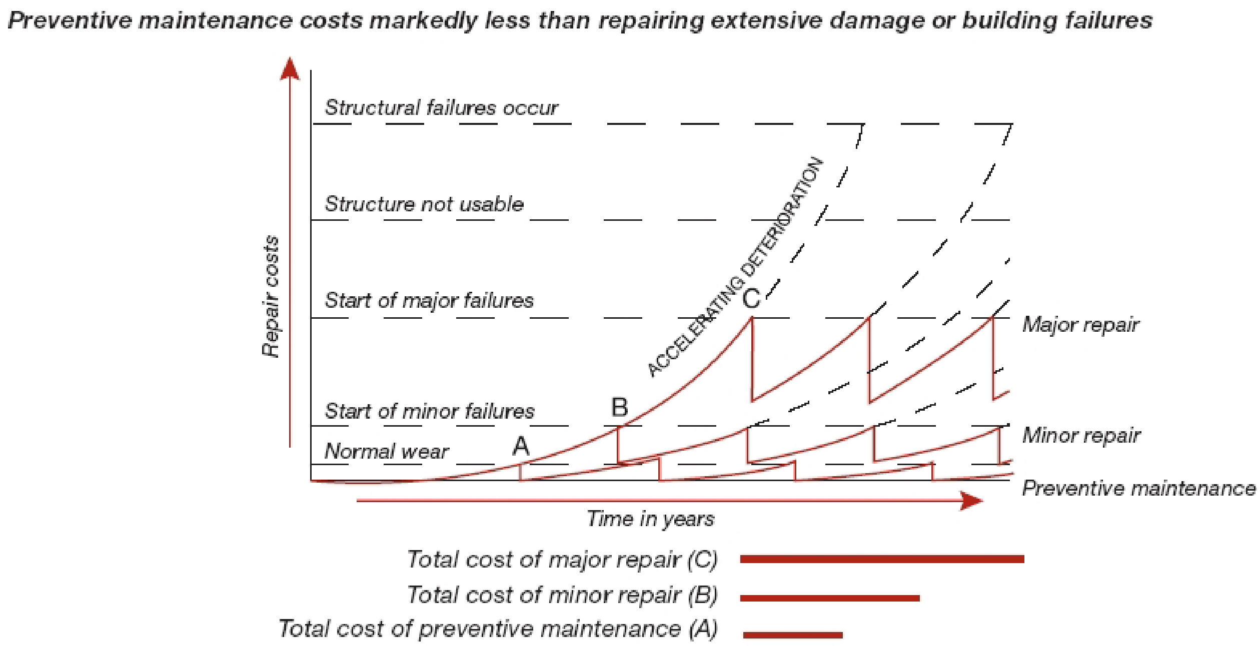


Fig. 5.9.1: Diagram from preventive maintenance of buildings (Van Nostrand, 1991)

The analysis process developed is proposes as tool for the application of an integrated methodological approach of structure of historical-cultural interest, both for the preliminary study and for the phases to be followed for the editing of the interventions' project. In this sence is evidence also a special versatility that makes the proposed tools destined both to public bodies that control the conservation and the protection of the cultural heritage, as *Soprintendenze*, and for single qualified planners that have to face such types of problematics.
Both for the Sala Maggiore and for the Armstrong crane the methodological path has been made which has allowed us to draw some intervention projects of maintaining unchanged as much as possible the structure in all of its aspects and of guaranteeing, besides, a suitable structural safety. The good quality of the results is also thanks to an integrated study of the analyses and the planning solutions of restoration and maintenance from a qualified work group, in which every figure has collaborated for the scope of its own competence. The integrated approach, many times underlined, is in fact one of the key points for the layout of a good project.

Concluding, the objectives and the developments of the research have delineated a possible methodology of analysis for the study and the restoration of cultural heritage structure, which get integrated within the development of scientific approaches for the achievement of results of restoration the most adherent to the conservative approach even though finalized to the accomplishment of an acceptable structural safety level of the buildings into consideration.

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ABBREVIATIONS

A.S.V.: Archivio di Stato di Venezia.

B.N.M.: Biblioteca Nazionale Marciana, Venezia.

M.S.N.: Museo Storico Navale, Venezia.

Codes & Standard

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