DECLARATION

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Title of the Msc Dissertation:	Inventory of Repair and Strengthening Methods with Iron and Steel
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I hereby declare that all information in this document has been obtained and presented in accordance with academic rules and ethical conduct. I also declare that, as required by these rules and conduct, I have fully cited and referenced all material and results that are not original to this work.

I hereby declare that the Msc Consortium responsible for the Advanced Masters in Structural Analysis of Monuments and Historical Constructions is allowed to store and make available electronically the present MSc Dissertation.

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Date:	July 12,2009
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"A structural engineer, looking at a Gothic cathedral, will see, not a massive array of nave piers, but the skeletal structure formed by the centre-lines of those piers; not a thick vault, but a thin doubly-curved sheet spanning between the mathematical centre-lines of the ribs."

Jacques Heyman

Starting from my early childhood years, learning, discovering new things and challenges have always been the driving force for my personal development. It was one of my dreams to study in Europe after graduating the high school which turned out to be a dream which would not be realized. In years, with the projects I got involved, I was able to strengthen my connections not only with Europe but with other parts of the world more and more, yet studying and living in a country , experiencing the real life, traditions and culture is totally different then visiting for certain durations.

I had to suspend and change many things in my life to be the part of this study but I will never regret. It was really a big challenge after all the achievements in my life to show the willingness to take this "exam"- starting from zero point, accepting the apprenticeship which is the unique antidote to cure and improve the personality. It will stand as a hinge in the middle of my life representing many things, connecting my past with my future as a strong device furnishing my personality with necessary resistance and tolerance I shall need for the rest of my life.

It was a unique experience to stay at a studio apartment in Padova in an absolute isolation mood and concentrating on one specific issue which was a real luxurious situation in my life as a person who had to deal with many things at a time, as woman and as a mother. It was a great amusement to walk along the streets of Barcelona, exploring the architectural texture of the city in between the sessions of searching and typing the report. It was a great experience to see and compare the different cultures, traditions, habits of different societies, nations. I will never forget how I was deeply affected when I saw the people from 7 to 77 years old were dancing and singing perfectly and naturally during Feria de Abril in Barcelona. I will never forget how people still was discussing vehemently in the piazzas of Padova on top of a stool as a person from a country where come-togethers are treated suspiciously. I have definitely acquired a knowledge which could only be conceived by living.

In the light of all these feelings, I would like to dedicate this "period – SAHC advanced master study" to all the achievements of human beings, to all the marvelous persons inspired us with their creations, to fearlessness, to intelligence, cognition, supremacy of knowledge and enlightenment,

And to my dearest brother *Mahomet Emin Ay-my role model* who passed away when he was only 26 shortly after his graduation as a civil engineer.

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I would like to acknowledge the support I got from my supervisor Professor Climent Molins, who guided me with his comments throughout the dissertation period.

I would like to express my deepst gratitude to the Msc Consortium and the Erasmus Mundus Programme for giving me the opportunity to participate in this very enriching study and the financial support.

As indicated as the motivation and objective of this study on SAHC ‘s web site, Europe is a world leader in the generation of knowledge, methodology and technology applicable to the conservation and restoration of the architectural heritage. The large investment made during the last years lead to significant advances in experimental and numerical techniques applied to the conservation of architectural heritage structures and I was proud to take the challenge of being the part of this perfectly organized study which combined the diversity of expertise at leading European universities in the field, offering education oriented to a multidisciplinary understanding of structural conservation through the involvement of experts from complementary fields (engineers, architects, materials scientists and others). The programme fulfilled its promise by facing us top level structural analysis knowledge in a research oriented environment, with close cooperation with the industry and a focus on problem solving.

I would like to sincerely thank to all the friends I made in the university of Padova and Barcelona (Ahmed, Murat, Thomas, Peter, Emmanuel, Tasos, Justin, Jiri, Yee, John, Patricia, Logan, Dechen, Ziba and Vanesse).

My sepcial thanks to Elisa Trovo who was always available to help with her smiling face.

I would also like to thank to my friends in FIP INDUSTRIALE S.p.A(Paolo Baldo, Gabriella Castellano, Schuster Astrid) who supported me throughout my study, for their incomparable friendship and hospitality.

I cannot pass without mentioning the patience of my dearest children Setenay and Oğuz for their encouraging manners towards me and their courage of practicing and experiencing life without their mother. I am grateful to my daughter Setenay and her friend Didem for their help to organize the text to fit in the template.

After all these years of dealing with many different types of engineering projects, it is not an exageration to say that it is this study which gave me a unique tool of conceving an engineering structure as a whole with its idiosyncratics.

Thank you......

ABSTRACT

This study aimed at producing a report (inventory) on stabilization / repair / strengthening materials, methods and technologies used for the structural restoration of structures including cast iron, wrought iron and steel members. A detailed literature survey was carried out by reading as much document as possible using different database including UPC library and after all the courses studied throughout this advanced master study it may be concluded that regardless of the technique decided to be applied in a restoration project, it is very important that any repair or strengthening must be based on a correct diagnosis of the problem by determining the real cause of the observed defects, and if the cause is still active by eliminating it. Otherwise the repairs will be short-lived. There is a parallel in the field of medicine: any treatment must be preceded by a correct diagnosis.

Masonry is one of the oldest construction material and tracing back the use of masonry will take us to the history of the civilization. It is not an exaggerate to say that masonry represents civilization. From the first ruble walls of the Stone Age to contemporary brick veneer systems, it is masonry the architects and engineers turn to when performance, solidity, aesthetics are sought the evolution of which is parallel to the one of civilization.

The problem of conserving and structural restoration of architectural heritage and is complex involving different challenges. It is is multidisciplinary task carried out with the involvement of different competencies like architects, engineers, arcehologiests and historians. It is very important to follow a methodology which includes survey, diagnosis, saftey evaluation and the selection of the correct and effective intervention technique fitting best with the strengthening requirements for the specific building with respect to it's original conception and the historic value.

This is a difficult task because of the complexity of the geometry of historic monuments, the variable and often unpredictable characteristics of original materials, the different building techniques, the absence of knowledge on the existing damages occured throughout their life due to several reasons, and the lack of applicable codes. In addition, restrictions in the inspection and the removal of specimens in buildings of historical value, as well as the high costs involved in inspection and diagnosis, often result in limited information about the internal structural system or the properties of existing materials. Without a thorough understanding of the mechanisms of decay and deterioration conservation skills can not be increased to prolong the life of cultural property for future generations. consideration of these aspects is not simple and calls for qualified persons capable of combining advanced knowledge in the field with engineering reasoning, as well as a careful, time-consuming approach

Although there are a lot of materials and intervention techniques available for structural restorations recent philosophy underlying restoration recommends to prefer prefer techniques that are fullfilling the criterias like minimum intervention, compatibility, durability, reversibility or substitutability. In principle

this philosophy is correct, but one should not forget that any decission may not always be error-free and it is therefore unwise to exclude the possibility of exploiting better techniques and materials if and when they are developed. conservation requirements .

In chapters 5, 6 and 7 of this report, different repair/strengthening techniques using iron and steel are discussed according to the possible causes of damages of the structures. Post-tensioning, strengthening with ant-seismic devices like shape memory alloy devices and shock transmittion units in combination with other strengthening techniques are discussed as innovative techniques. Application examples can be found in chapter six of the report which explains in detail the techniques.

RESUMEN

Este estudio tiene como objetivo elaborar un informe (inventario) sobre la estabilización / reparación / el fortalecimiento de los materiales, métodos y tecnologías utilizadas para la restauración estructural de las estructuras incluidas hierro fundido, hierro forjado y acero miembros. Un detallado estudio de la literatura fue llevada a cabo por la lectura tanto como sea posible el documento base de datos utilizando diferentes bibliotecas UPC y después de todos los cursos estudiados a lo largo de este maestro de avanzada estudio se puede concluir que independientemente de la técnica decidió que se aplicarán en un proyecto de restauración, se es muy importante que cualquier reparación o fortalecimiento debe basarse en un diagnóstico correcto del problema mediante la determinación de la verdadera causa de los defectos observados, y si la causa sigue activa, por su eliminación. De lo contrario, la reparación será de corta duración. Hay un paralelo en el campo de la medicina: cualquier tratamiento debe ser precedido por un diagnóstico correcto.

Albañilería es uno de los más antiguos materiales de construcción y el rastreo de nuevo el uso de albañilería nos llevará a la historia de la civilización. No es exagerar decir una mampostería que representa la civilización. Desde el primer rublo paredes de la Edad de Piedra hasta la chapa de los sistemas de ladrillo, es de mampostería los arquitectos e ingenieros a su vez, cuando el rendimiento, la solidez, la estética se busca la evolución de la que es paralela a la una de la civilización.

El problema de la conservación y la restauración estructural del patrimonio arquitectónico y es complejo que participen diferentes desafíos. Es tarea multidisciplinaria se llevó a cabo con la participación de las diferentes competencias, como arquitectos, ingenieros, arqueólogos e historiadores. Es muy importante seguir una metodología que incluye el estudio, diagnóstico, evaluación de la seguridad y la selección de la correcta y eficaz de intervención técnica de montaje con los mejores requisitos para el fortalecimiento de la construcción con respecto a su concepción original y el valor histórico.

Esta es una tarea difícil debido a la complejidad de la geometría de los monumentos históricos, la variable y, a menudo impredecibles características de materiales originales, las distintas técnicas de construcción, la ausencia de conocimientos sobre la actual se produjeron daños a lo largo de su vida debido a varias razones, y la la falta de códigos aplicables. Además, las restricciones en la inspección y la eliminación de los especímenes en los edificios de valor histórico, así como los altos costos implicados en la inspección y el diagnóstico, a menudo como resultado información limitada acerca de la estructura interna del sistema o las propiedades de los materiales existentes. Sin una profunda comprensión de los mecanismos de la decadencia y el deterioro de conservación de las competencias no puede ser aumentado a prolongar la vida de los bienes culturales para las generaciones futuras.

consideración de estos aspectos no es sencillo y requiere personal cualificado capaz de combinar un conocimiento avanzado en el campo de la ingeniería con el razonamiento, así como una cuidadosa, el tiempo que consume enfoque. Aunque hay una gran cantidad de materiales y técnicas de intervención disponibles para restauraciones estructurales reciente restauración de la filosofía subyacente a preferir prefiero recomienda técnicas que se cumplen los criterios de intervención, como mínimo, la compatibilidad, duración, reversibilidad o de sustitución. En principio, esta filosofía es correcta, pero no hay que olvidar que cualquier decisión puede no ser siempre libre de errores y, por tanto, es prudente descartar la posibilidad de explotar las mejores técnicas y materiales, siempre y cuando sean desarrollados. los requisitos de conservación.

En los capítulos 5,6 y 7 de este informe, los diferentes reparación / el fortalecimiento de las técnicas que utilizan el hierro y el acero se discuten en función de las posibles causas de los daños de las estructuras. Post-tensado, con el fortalecimiento de hormiga sismológicas dispositivos como dispositivos de memoria de forma de aleación y las unidades de choque de transmisión en combinación con otras técnicas de refuerzo como se discuten las técnicas innovadoras. Ejemplos de aplicación se pueden encontrar en el capítulo seis del informe, que explica en detalle las técnicas.

RIASSUNTO

Questo studio mira a produrre una relazione (inventario) sulla stabilizzazione / riparazione / rafforzamento dei materiali, i metodi e le tecnologie utilizzate per il restauro strutturale di strutture tra cui ghisa, ferro battuto e acciaio membri. Una dettagliata indagine letteratura è stata effettuata con la lettura di un documento di più come possibile utilizzando diversi database tra cui UPC biblioteca e dopo che tutti i corsi di studi avanzati in questo studio master si può concludere che, indipendentemente dalla tecnica ha deciso di essere applicata in un progetto di restauro, è è molto importante che ogni riparazione o il rafforzamento deve essere basata su una corretta diagnosi del problema da determinare la vera causa del osservati difetti, e se la causa è ancora attivo, eliminando esso. In caso contrario, la riparazione sarà di breve durata. C'è un parallelo nel campo della medicina: ogni trattamento deve essere preceduto da una corretta diagnosi.

Muratura è uno dei più antichi materiali da costruzione e per rintracciare l'utilizzo di muratura ci porterà alla storia della civiltà. Non si tratta di un esagerare a dire che in muratura rappresenta civiltà. Dal primo rublo mura di pietra per la contemporanea mattone impiallacciato sistemi, è in muratura gli architetti e gli ingegneri a sua volta, quando le prestazioni, solidità, estetica sono chiesta l'evoluzione di cui è parallela a quella della civiltà.

Il problema della conservazione e restauro strutturale del patrimonio architettonico è complessa e coinvolge diverse sfide. E 'compito multidisciplinare è effettuata con il coinvolgimento di diverse competenze, come architetti, ingegneri, archeologi e storici. E 'molto importante seguire una metodologia che comprende indagine, la diagnosi, la valutazione della sicurezza e la selezione del corretto ed efficace intervento tecnica di montaggio in modo ottimale con il rafforzamento requisiti specifici per l'edilizia con il rispetto alla sua originale concezione e il valore storico.

Questo è un compito difficile a causa della complessità della geometria dei monumenti storici, la variabile e spesso imprevedibile caratteristiche dei materiali originali, le diverse tecniche di costruzione, mancanza di conoscenze sulle esistenti danni verificatisi nel corso della loro vita a causa di vari motivi, e la mancanza di codici applicabili. Inoltre, le restrizioni durante l'ispezione e la rimozione di esemplari, in edifici di valore storico, così come i costi elevati in materia di ispezione e diagnosi, spesso sfociano in numero limitato di informazioni sulla struttura interna del sistema o le proprietà dei materiali esistenti. Senza una profonda comprensione dei meccanismi di degrado e deterioramento tecniche di conservazione non può essere aumentata a prolungare la vita dei beni culturali per le generazioni future. considerazione di questi aspetti non è semplice e richiede persone qualificate in grado di coniugare conoscenze avanzate nel campo di ingegneria ragionamento, così come una attenta, in termini di tempo approccio.

Anche se ci sono un sacco di materiali e tecniche di intervento a disposizione per gli interventi strutturali recenti restauri filosofia che sottende il restauro raccomanda a preferire preferiscono tecniche che sono rispondenti ai criteri di intervento, come minimo, la compatibilità, la durata, la reversibilità o la sostituibilità. In linea di principio, questa filosofia è corretta, ma non bisogna

dimenticare che qualsiasi decisione non può essere sempre esente da errori ed è quindi sconsigliabile per escludere la possibilità di sfruttare meglio le tecniche e materiali se e quando si sono sviluppate. esigenze di conservazione.

Nei capitoli 5,6 e 7 della presente relazione, diversi riparazione / rafforzamento utilizzando tecniche di ferro e acciaio sono discusse secondo le possibili cause dei danni delle strutture. Post-tensionamento, rafforzando con ant-sismica dispositivi in lega a memoria di forma come i dispositivi di trasmissione e shock unità in combinazione con altre tecniche di rafforzamento sono discussi come tecniche innovative. Esempi di applicazione possono essere trovate nel capitolo sei della relazione, che spiega in dettaglio le tecniche.

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CHAPTER 1

1.1 INTRODUCTION

1.1.1 The aim

The concern of this thesis report is to discuss as detailed as possible on the structural repair/ strengthening / restoration techniques of masonry buildings with iron and steel. It's intention is to describe, explain and comment on different techniques, their effectiveness, Advantages, disadvantages, limitations in the light of the modern conservation principles.

1.1.2 The search

The search has involved library sleuthing, evaluation of theories, alternative possibilities,

on practical applications and medium or long term performance (possible side-effects and

undesirable consequences), with focus to real cases; field work consisting of the analysis of real case studies; discussion on possibilities, advantages, defects, compliance with conservation criteria; conclusions and best approaches and corresponding applications. It was observed that although there are considerable amount of scientific papers, and various publications, those specifically focused on repair and strengthening techniques with iron and steel by mentioning all the relevant aspects are almost do not exist or not sufficient to conceive the issue as a whole.

1.1.3 The Methodolgy

The conservation of cultural properties constitues a single, interprofessional discipline coordinating a range of aesthetic, historic, scientific, and technical methods. It is a rapidly developing field that , by its very nature, is a multidisciplinary activity, with experts respecting each other's contributions and combining to form an affective team.

The obove paragraph underlines the phiolosphy considered throughout the study of this thesis report which defines the method and the logic it is organized. Any technique described in the report was given as a part of a whole project which has various elements in connection with the philosophy of this paragraph.

Although the chapters 2,3 and four are devoted to other aspects of conservation and masonry which are indispensable in the sense of seeing a strengthening project as a whole, chapters 5 and 6 intensified on the structural aspects of repair & strengthening by using iron and steel.

It is known that some structural engineers in the past have considerably damaged the character of some historic buildings, by introducing strengthening devices that counteracted the inherent mechanics of the original structure, in order to create a structural system conforming to their conventional textbook patterns and the letter of the codes of practice. There should be a compromise between the architect and the engineer realizing that unless the structure of the building is sound, its architectural and artistic glories are bound to perish which will help the architect to understand that the laws of gravity and structural mechanics apply to all structures, regardless of their antiquity or historic significance; conversely, the engineer should conceive that older buildings have unique caharacteristics, which are not often found in modern structures and for the successful operation of structural restoration techniques, it is essential that the engineer becomes familiar with these symptoms.

CHAPTER 2

THE PRINCIPALS OF CONSERVATION

2.1. General

The modern principles that govern the organization and application of conservation interventions have taken centuries of philosophical, aesthetic, and technical progress to articulate. The problem of conserving and structural restoration of architectural heritage and is complex. Even in a scientific age that has developed the technology of space travel and atomic power, the solution to local environmental problems still presents a challenge to the present and the future. The continuous changes in materials and construction techniques are far ahead from traditional practice, and the challenging technical and scientific developments, create new possibilities available for all the agents involved in the preservation of the architectural heritage. These are the key aspects in the division between the science of construction and the art of conservation and restoration. Withoout a thorough understanding of the mechanisms of decay and deterioration conservation skills can not be increased to prolong the life of cultural property for future generations. consideration of these aspects is not simple and calls for qualified persons capable of combining advanced knowledge in the field with engineering reasoning, as well as a careful, time-consuming approach

The conservation of cultural property demands wise management of rersources and a good sense of proportion. Perhaps above all, it demands the desire and dedication to see that cultural property is preserved. In this sense, the familiar maxims are pertinent: “Prevention is better than cure” and “ A stich in time saves nine”. Modern long term conservation policy concentrates on fighting the causes of decay. Natural disasters such as floods and earthquakes cannot be prevented, but by forethough the damage can be greatly reduced. Industrial life cannot and should not be halted., but damage can be minimized by combating waste, uncontrolled expansion, economic exploitation, and pollution. Conservation is, therefore, primarily a process leading to the proplongation of the life of cultural property for its utilization now and in the future.

2.2 Conservation Methodology

The conservation of cultural properties constitues a single, interprofessional discipline coordinating a range of aesthetic, historic, scientific, and technical methods. It is a rapidly developing field that , by its very nature, is a multidisciplinary activity, with experts respecting each other’s contributions and combining to form an affective team.

Despite the difference in scale and extent of intervention, the underlying principles and procedureal methods remain the same for the conservation of movable and immovable cultural property. There are however, important logistical differences.

First, architectural work entails treatment materials in an open an virtually uncontrollable environment. Whereas the museum conservator/restorer can genrally rely on good environmental control to minimize further deteriroration, the architectural conservator cannot. He must allow for the effects of time and weather.

Second, the scale of architecturla operations is much larger, and in many cases methods used by museum conservators/restorers may be found impracticable because of the size and complexity of the architectural fabric.

This, and again because of the size and complexity of architectural conservation, contractors, technicians, and craftsmen must actually perform the various conservation functions, while the museum conservator/restorer may do most of the treatment with his own hands. Communication and supervision, tehrefore, are important considerations for the architectural conservator.

Lastly, because the architectural fabric has no function as a structure, resisting its own dead weight and applied live loadings, tehre are further differences between the practice of architecturala and museum conservation. Architectural conservation must be within the context of historic structure, which also incorporates its site, setting and physical environment.

The values assigned to cultural property come under three major headings:

- Cultural Values: documentary value, historic value, archeological and age value, aesthetic value, architectural value, scientific value, symbolic or spiritual value, townspace value, landscape and ecological value.
- Use Values: functional value, economic value, social political value.
- Emotional Values: wonder, identity, continuity

The cost of conservation may have to be allocated partially to each of the above values in oerder to justify the total to the community. There may be conflicts between some of the values. In certain cases architectural values will predominate. In other cases, artistic or historical considerations will prevail, while in yet others practical and economic considerations may modify the scope of conservation. Sound judgement based upon wide cultural preparation and mature sensitivity gives the ability to make corrrect value assessments.

2.3 Concepts of Intervention

When a treatment is being planned, the following three general concepts regarding an object or a structure's condition are considered.

- Damage suffered by the object

- Insecurity of the object
- Disfigurement sufferred by the object
- During all conservation treatments, the following standards of ethics must be rigorously followed:
- The condition of the object , and all methods and material used during treatment, must be celarly documented.
- Historical evidence should be fully recorded, it must not be destroyed, falsified, or remove.
- Any intervention must be the minimmum necessary
- Any intervention must be governed by unswerving respect for the aesthetic, historical, and physical integrity or cultural property.

Intervention should:

- Be reversible, if technically possible.
- Not prejudice a future intervention whenever this may become necessary.
- Not hinder the possibility of later access to all evidence incorporated in the object.
- Allow the maximum amount of exisitng material to be retained.
- Be harmonious in color, tone, texture, from and scale, if additions are necessary, but the less noticable than original material, while at the same time being identifiable.
- Not be undertaken by conservators who are insufficiently trained or experienced, unless they obtain competent advice. However, it must be recognized that some problems are unique and have to be solved from first principles on a trial-and-error basis.

2.4 Inventories and Documentation

At the national level, conservation procedures consist first of making an inventory of all cultural property in the country. This is a major administrative task for the government. It involves establishing appropriate categories of cultural property and recording them as thoroughly as possible, graphically and descriptevly.

A preliminary written study of each object or building is necessary in order to know and define it as awhole, which, in the case of architecture, includes its setting and environment. The present condition of the building or object must also be recorded. Documentation of these studies must be full and conscientous. Records and archives must be searched. In some countries, reliance may have to be placed on oral traditions, which should be recorded verbatimand included in the dossier created fır each object or building.

2.4.1 Documentation

Complete recording is essentail before, during, and after any intervention. In all works of preservation, repair, or excavation of cultural property there must always be precise documentation in the form of analytical and critical reports, illustrated with photographs and drawings. Every stage of the work of cleaning, consolidation, reassembyl, and reintegration, including all materials and techniques used, must be recorded. Technical and formal features identified during the course of the work ahould also be included in the odcumentation. This record should then be placed in the archives of a public institution and made available to research workers. Finally, if the intervention can in any way serve to be broaden general knowledge, a report must be published. Often in large projects it may take several years to write a scholarly report, so a preliminary report or series is desirable to inform the public and to maintain popular support.

Documentation is essential because it must be remembered that the building or work of art will oıtlive the individuals who perform the interventions. To ensure the maximum survival of culturla property, future conservators/restorers must know and understand what has ocureed in the past.

2.5 Interventions

Intervention should be the minimum necessary. The techniques used depend upon the conditions of climate to which cultural property is likely to be subjected. These fall into three groups:

- Natural climatic and microclimatic condtions, which vary greatly and are virtually uncontrollable.
- Modified climatic conditions, such as those found in a normal building that forms an environmental spatial system with a partially self-adjusting modified climate.
- Conditions where humidity and temperature are controlled artificially to minimize dangerous variations. Ideally, the climatic control has been designed for the safety of the objects, rather than the confort of the visitor.

Interventions practically always involve some loss of a “value” in cultural property, but are justified in order to preserve the objects for the future. Conservation involves making interventions at various scales and şevels of intensity that are determined by the physical condition, the causes of deterioration, and the probable future environment of the cultural property under treatment. Each case must be considered individually and as a whole, taking all factors into account.

Always bearing in mind the final aim, principles, and rules of conservation, seven degress of intervention can be identified. However, in any individual conservation treatmenti several degress may take place simultaneously in various parts of the whole. The seven degress are:

- Prevention deterioration
- Preservation
- Consolidation
- Restoration
- Rehabilitation
- Reprodcution
- Reconstruction

2.6 Prevention Deterioration

Prevention entails protecting cultural property by controlling its environment, thsu keeping agents of decay and damage from becominga ctive. Neglect must also be prevented.

Therefore, prevention includes control of humidity, temperature, and light, as well as measures of preventing fire, arson, theft, and vandalism. In the industrial and urban environment, it includes measures for reducing atmospheric pollution, traffic vibrations, and ground subsidence from many causes, particularly abstractions of water.

2.7 Preservation

Preservation deals directly with cultural property. Its objects is to keep it in the same state. Damage and destruction caused by humidity, chemical agents, and all types of pets and microorganisms must be stopped in order to preserve the object or sturcture.

Maintanance, cleaning schedules, good housekeeping and good management aid preservation. Repairs must be carried out when necessary to prevent further decay and to keep culturla property in the same state. Regular inspections of cultural property are the basis of prevention. When the property is subjected to an uncontrollable environment, such inspections are the step in preventive maitenace and repair. [25]

2.8 Consolidation

Consolidation is the physical addition of adhesive or supportive materials into the actual fabric of cultural property in order to ensure its continued durabilty or structural integrity. In the case of immovable cultural property, consolidation may entail, for exmple, the injection of edhesives to secure a detached mural painting to the wall. Movable cultural property, such as weakend canvas paintings and works on paper, are often backed with new supportive materials.

With buildings, when teh strenght of structural elements has been so reduced that it is no longer sufficient to meet future hazards, the consolidation of the existing material is necessary, and new material may have to be added. However, the integrity of the sturctural system must be respected and its for preserved. No historical evidence should be destroyed. Only by firts understanding how a historical building as a whole acts as a “spatial environment system” is it possible to make adjustments in favor of anew use, introduce new techniques satisfactorily, or provide a suitable environment for objects of art.

The utilization of traditonal skills and materials is of essential importance, as these were employed to create the object or building. However, where y-traditional methods are inadequate, the conservation of cultural property may be achieved by the use of modern techniques that should be reversible, proven by experience, and applicable to the scale of the project and its climatic environment. In buildings made of perishable materials, such as wood, mud, brick, or rammed earth, traditional materials and skills should be used for the repair or restoration of worn or decayed parts.

2.9 Restoration

The object of restoration is to revive the original concept or legibility of the object. Restoration and reintegration of details and feature occur frequnetly and are based upon respect for original materials, archeological evidence, original design, and authentic documents. Replacement of missing of decayed parts must integrate harmoniously with the whole, but on close inspection must be distinguishable from the original so that the restoration does not falsify artistic or historical evidence.

Contributions from all periods must be respected. All later additions that can be considered as historical documents, rather than merely previous restoration, must be preserved. When a building include supreimposed work of different periods, revealing the underlysing state can be justified only in exceptional circumstances: when the part removed si widely agreed to be of little interest, when it is certain that the material brought to light will be of great historical or archeological value, and when it is clear that its staet of preservation ,s good enough tı justify the action. Restoration also entails superficial cleaning, but with full respect for the patina of age.

2.10 Rehabilitation

The best way of preserving buildings is to keep them in use, a practice that may involve what French call “mise en valeur”, or mornization and adaptive alteration.

Adaptive reuse of buildings, such as utilizing a medieval convent in Venice to house a school and laboratory for stone conservationor turning an eighteen-century barn into a domestic dwelling, is often the only way that historic and aesthetic values can be made economically viable. It is also often the only way that historic buildings can be brought up to contemporary standards by providng modern amenities.

2.11 Reproduction

Reproduction entails copying an extant artefact, often in order to replace some missing or decayed parts, generally decorative, to maintain its aesthetic harmony. If valuable cultural property is being damaged irretrievably or is threatened by its environment, it may have to be moved to a more suitable environment and a reproduction substituted in order to maintain the unity of a site or building. For example, Michelangelo's 'David' was removed from the Piazza della Signoria, Florence, into a museum to protect it from the weather, and a good reproduction took its place. Similar substitutions have been undertaken for the sculpture of the cathedrals of Strasbourg and Wells. [25]

2.12 Reconstruction

Reconstruction of historic buildings and historic centres using new materials may he necessitated by disasters such as fire, earthquake or war. Reconstruction cannot have the patina of age. As in restoration, reconstruction must be based upon accurate documentation and evidence, never upon conjecture. The moving of entire buildings to new sites is another form of reconstruction justified only by over-riding national interest. Nevertheless, it entails the loss of essential cultural values and the generation of new environmental risks. The classic example is the temple complex of Abu Simbel (XIXDynasty), Egypt, which was moved to prevent its inundation following the construction of the Aswan High Dam, but is now exposed to wind erosion.[25]

2.13 ICOMOS Recommendations

The International Scientific Committee for the Analysis and Restoration of Structures of Architectural Heritage (ISCARSAH) has prepared recommendations2, intended to be useful to all those involved in conservation and restoration problems. These recommendations contain principles, where the basic concepts of conservation are presented, and Guidelines, where the rules and methodology that a designer should follow are discussed. More comprehensive information on techniques and specific knowledge can be found elsewhere. In addition, normative and pre-normative are gradually becoming available, at least with respect to seismic rehabilitation, which is a major concern.

2.13.1 Principles

A multi-disciplinary approach is obviously required in any restoration project and the peculiarity of heritage structures, with their complex history, requires the organization of studies and analysis in steps that are similar to those used in medicine. Anamnesis, diagnosis, therapy and controls, corresponding respectively to the condition survey, identification of the causes of damage and decay, choice of the remedial measures and control of the efficiency of the interventions. Thus, no action should be undertaken without ascertaining the likely benefit and harm to the architectural heritage. A full understanding of the structural behavior and material characteristics is essential for any project related to architectural heritage. Diagnosis is based on historical information and

qualitative and quantitative approaches. The qualitative approach is based on direct observation of the structural damage and material decay as well as historical and archaeological research, while the quantitative approach requires material and structural tests, monitoring and structural analysis. Often the application of the same safety levels used in the design of new buildings requires excessive, if not impossible, measures. In these cases other methods, appropriately justified, may allow different approaches to safety.

Therapy should address root causes rather than symptoms. Each intervention should be in

proportion to the safety objectives, keeping intervention to the minimum necessary to guarantee safety and durability and with the least damage to heritage values. The choice between “traditional” and “innovative” techniques should be determined on a case-by-case basis with preference given to those that are least invasive and most compatible with heritage values, consistent with the need for safety and durability. At times the difficulty of evaluating both the safety levels and the possible benefits of interventions may suggest “an observational method”, i.e. an incremental approach, beginning with a minimum level of intervention, with the possible adoption of subsequent supplementary or corrective measures.

The characteristics of materials used in restoration work (in particular new materials) and their compatibility with existing materials should be fully established. This must include long-term effects, so that undesirable side effects are avoided.

Finally, a most relevant aspect is that the value and authenticity of architectural heritage cannot be assessed by fixed criteria because of the diversity of cultural backgrounds and acceptable practices.

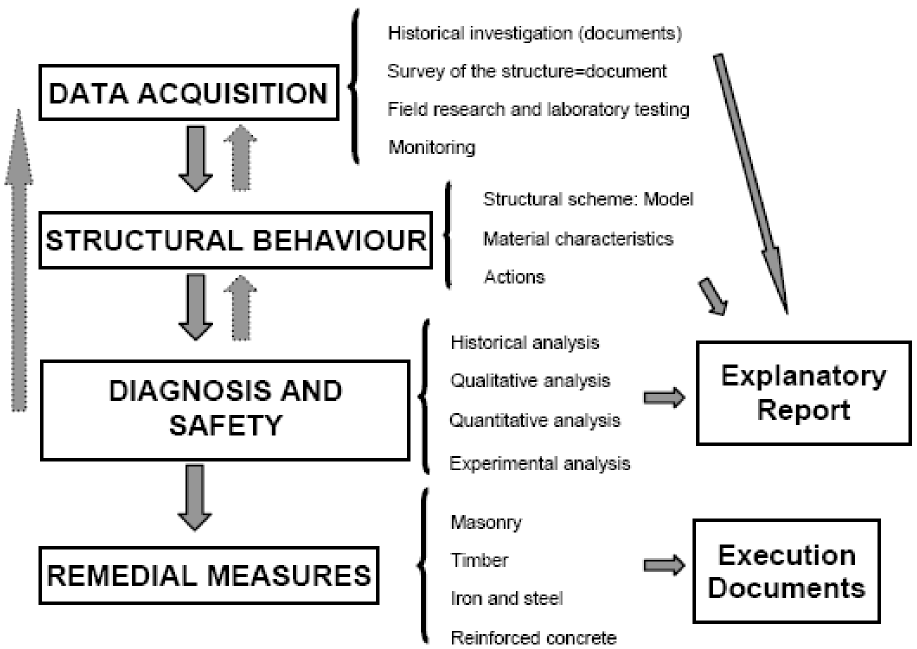
2.13.2 Guidelines

A combination of both scientific and cultural knowledge and experience is indispensable for the study of all architectural heritage. The purpose of all studies, research and interventions is to safeguard the cultural and historical value of the building as a whole and structural engineering is the scientific support necessary to obtain this result. The evaluation of a building frequently requires a holistic approach considering the building as a whole, rather than just the assessment of individual elements.

The investigation of the structure requires an interdisciplinary approach that goes beyond simple technical considerations because historical research can discover phenomena involving structural issues while historical questions may be answered from the process of understanding the structural behavior. Knowledge of the structure requires information on its conception, on itsconstructional techniques, on the processes of decay and damage, on changes that have been made and finally on its present state.

The recommended methodology for completing a project is shown in Figure 1, where an iterative process is clearly required, between the tasks of data acquisition, structural behavior, and diagnosis and safety. In particular, diagnosis and safety evaluation of the structure are two consecutive and related stages on the basis of which the effective need for and extent of treatment measures are determined. If these stages are performed incorrectly, the resulting

decisions will be arbitrary: poor judgment may result in either conservative and therefore heavyhanded conservation measures or inadequate safety levels. Evaluation of the safety of the building should be based on both qualitative (as documentation, observation, etc.) and quantitative (as experimental, mathematical, etc.) methods that take into account the effect of the phenomena on structural behavior. Any assessment of safety is seriously affected by the uncertainty attached to data (actions, resistance, deformations, etc.), laws, models, assumptions,etc. used in the research, and by the difficulty of representing real phenomena in a precise way.



CHAPTER 3

HISTORIC PERSPECTIVE FOR MASONARY STRUCTURES

3.1 Historical Background

Masonry is one of the oldest construction material and tracing back the use of masonry will take us to the history of the civilization. It is not an exageration to say that masonry represents civilization. From the first ruble walls of the Stone Age to contemporary brick vener systems, it is masonry the architects and engineers turn to when performance, solidity, aesthetics are sought the evolution of which is parallel to the one of civilation.

Dictionary definition of masonry is as the “work of a mason”. It includes several loosely related materials-brick, stone, concrete, and the tile-that are manufacture as relatively small units and bonded together in the field, usually by mortar. All these various kinds of masonry can provide strenght, protection from weather, security, and fire and sound resistance, although their structural behavior is slightly different.

The distinctions between design and construction, particularly in the past, have not always been clear; the two were at times mutually inclusive. For, until Renaissance, the master mason and master carpenter served in two capacities, at once)in modern terminology) both artchitect and contractor. Consequently, it is sometimes difficult to differentiate the two functions. Therefore, there are many situations which should be treated in a sense to consider that the practical issues have influenced the engineering or architectural design. A gothic cathedral was designed by a man who was both architect and engineer-or, of course, by a succession of such men, if the building campaigns spread over decades (Heyman, The Stone Skeleton). The “master of the work” had survived the long training of apprentice to journeyman to the caree grade of master, and had been one of those few outstanding masters who ere put to school again in the design office, before finally achieving control of a major work. This educational path contrasts strongly with that of modern Western European practice, which is based upon Rennaissance concept of the “gentlemen” architect, for whom considerations of history and aesthetics are divorced tos ome extent from those of engineering structure. If the building is complex, then the architect must work hand in hand with a technical adviser. There was not this division in the thirteenth century (or in the sixth, when Justinian employed two outstanding Greeks, Anthemios and Isidorus, to design Hagia Sofia). The architect then knew, in the fullest technical sense, how to build, as well as how to give hşs building an “architectural” design. Harvey (1958) has discussed this: The Gothic rules were so complicated that no one who had not served a long apprenticeship and pent years of practice could master them; whereas the rules of Vitrivius were so easy to grasp that even bishops could understand them, and princes could try their hand at design on their own”

Vitrivian rules, however, find no place for the flying buttress or for the rib vault. These two structural elements, which may perhaps be thought to represent the essence of Gothic, would seem to demand a long apprenticeship indeed for the mastery of their design. The medieval rules, th secrets of the lodges, ensured that the sturcture was effective; “decorative developments could then take place safely. The rules shaped the skeleton, and were themselves subject to evolutionary change; the sekeleton, and were themselves subject to evolutionary change;the skeleton once fixed, however, could be fleshed in a wide variety of forms.

To take a single exmaple, the great range of vaults, from simple quadripartite through the lierne to the vault, have a skeletal “shell” very much in common, and their basic structural action is the same. This basic action stems from the nature of masonry as a material, and to understand the action it is necessary to construct a structural theory which incorporates the curious properties of masonry. Above all, it is necessary to state clearly the question that is being posed when an engineer undertakes a structural analysis: what is the problem that requires solution? (Heyman, The stone skeleton)

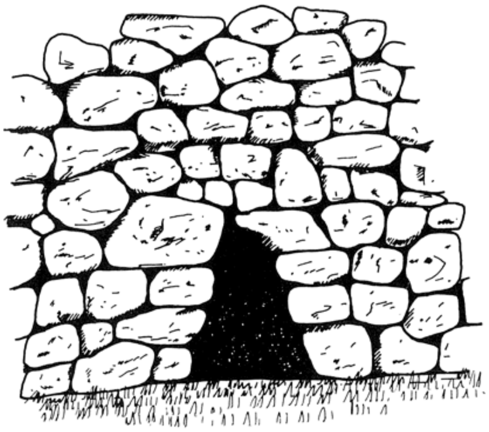
The outstanding feature of masonry buildings, even those erected many years ago, is their ability to survive. Minor failures have occurred, and there have been major catastrophes, but the masonry structure is essentially extremely stable. Two severe earthquakes only slightly damaged Hagia Sophia, Istanbul , the bombardments of World War II often resulted in a slightly damaged medieval cathedral surrounded by a modern city that was totally destroyed, as at Cologne. At a much less severe level of disturbance, the continual shifts and settlements of foundations experienced over the centuries seem to cause the masonry structure little distress, although, as will be seen, there may be an initial high-risk period of about a generation.

3.1.1 Evolution of Structural Shapes

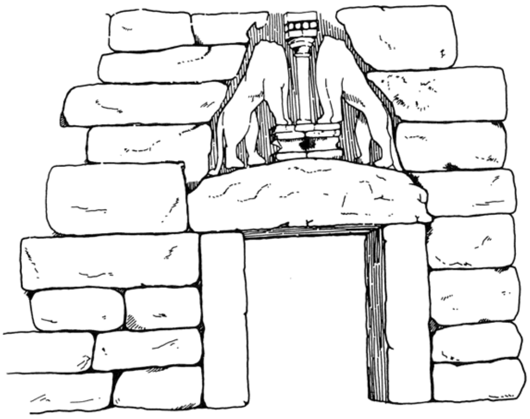
Horizontal space is most easily spanned when placing a beam across an opening. This occurs naturally when a tree trunk falls across a creek or small river. Primitive men used the same technique by balancing a stone across two other stones. In this post and lintel form of construction the long or the balanced stone bends to produce compressive stresses at the top and tensile stresses at the bottom. Because the tensile strength of rock is low and it is prone to crack, the stone beam or lintel had a relatively large cross section and span over only a short distance. Post and lintel construction is a fundamental form of masonry construction used all over the world. Stone lintels were also used to support the masonry above openings walls. In addition to the simple corbelled arch, in the 13th century BC the Lion’s Gate at Mycenae (Greece) was constructed with a lintel spanning just over 3 m above a triangular slab placed, enabling to see the beginnings of the arched behaviour that would dominate the following millennia, Figure 2.1. Arches do not exist in nature: it is an invention, which appeared in Babylon perhaps 6.000 years ago (Aztecs and Incas built in masonry for centuries without knowing the arch).

The technique of arching with blocks placed as corbels one on top of the other has been applied to build both false domes and arches, much older than the true arches and domes systematically constructed in the Roman period. With this brief background, it is clearly seen how the change from linear to curved structures, e.g. arches and vaults, represented a significant structural advance, which allowed to replace stone and timber lintels in walls, with stone or

brick masonry spanning wider openings. Indeed, in curved elements, it is usual to find only compressive stresses in a given section and consequently no tensile on resistant materials are required, Figure 2.1b.



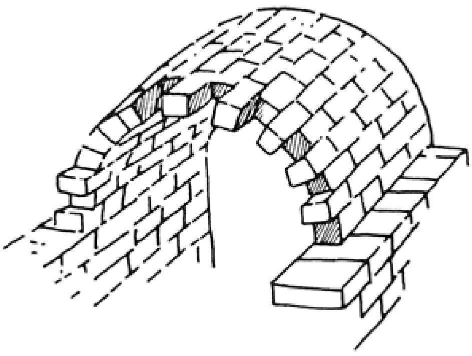
(a)



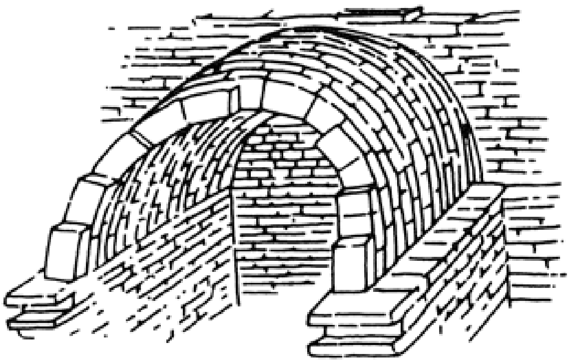
(b)

Figure 2.1*: Two samples of the first masonry conceptions in Greece: (a) Corbelled arch in wall Tiryns (c.600 B.C); (b) The Lion Gate (c.1250 B.C.).*

Figure 2.2b, shows an alternative design of a circular vault, which can be built from tiles without the use of centring; this method was developed in Egypt and Assyria, and can be found in some Byzantine work. The tiles laid back at an angle (starting from a vertical end wall), and each course acts as permanent formwork for the next course to be laid.



(a)



(b)

Figure 2.2*: True arch construction, construction of vaults (a) requiring temporary support; (b) using end walls and previous construction for stability Turing construction.*

Although in others cultures such as the Islamic architecture, the shape of arches and domes was often dictated by architecture intentions than by structural efficiency. The arch was the structural element that best exploited the characteristic of masonry and for two thousand years it was also the dominant feature of bridges.

Vaults and domes

Vault represents the three dimensional extension of the arch in space, although their initial beginnings, its techniques of construction and materials are different. Potentially the vault has a big advantage in relation to the arch: its two dimensional behaviour, which may be considered as an individual series or arches (meridians) and parallels in domes with bending moments.

Another structural element that encloses space is the dome. This can be thought of as the shape formed by the rotation of an arch about its vertical axis. Once again the earliest domes were formed by the corbelling technique which leads to a fairly pointed dome. In this case, each course of masonry forms a horizontal ring in which, to some extent, each masonry units is prevented from unbalancing

by adjacent unit forming a compression ring. Great ingenuity was often shown in combining barrels vaults to create pleasing buildings. The culmination of this development was the cross vault, or groin vault, so named after the lines created by its intersecting surfaces. Examples of vaults

are shown in Figure 2.3.

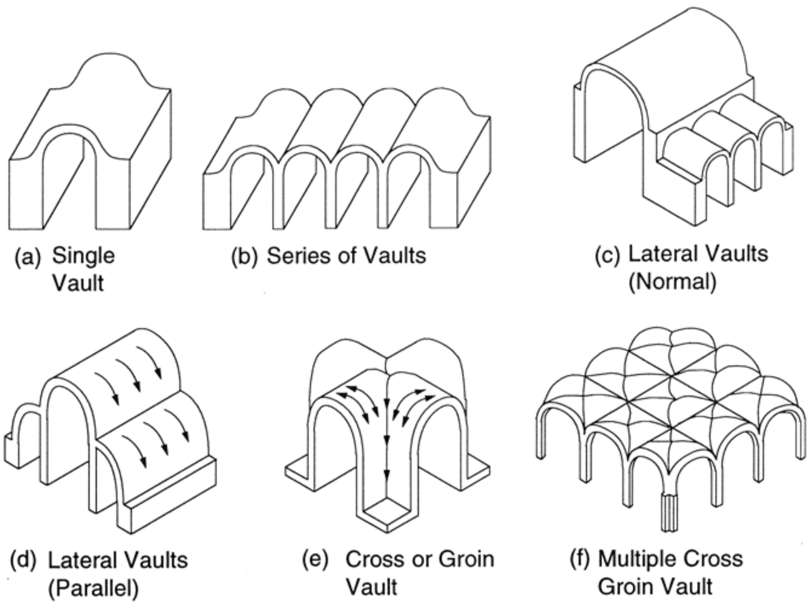
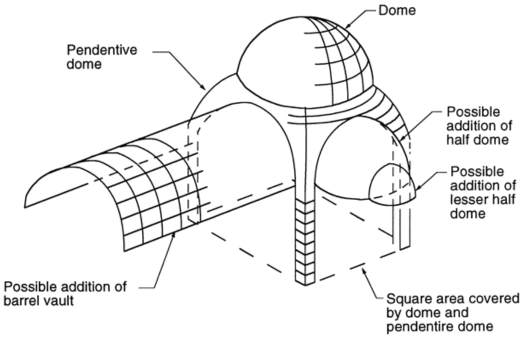


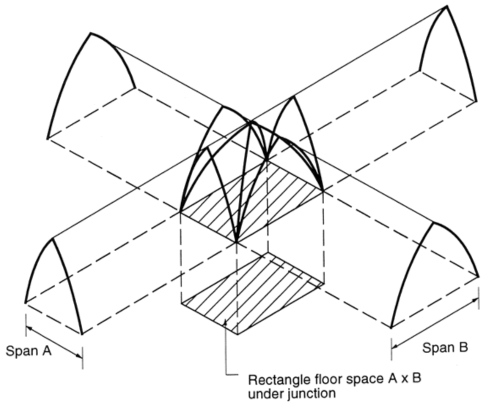
***Figure 2.3**: Examples of combined barrels: (a) single vault; (b) series of vaults; (c) lateral vaults (normal); (d) lateral vaults parallel; (e) cross or groin vault and (f) multiple cross groin vault.*

Hemispherical domes were set upon cylindrical walls or more interestingly over polygonal or even square supporting walls. In latter cases, the supporting walls were made thick enough to contain the base of the dome or thinner walls used with lintels or arches spanning across the corners to support the dome. There is another elegant solution, to support a dome over a square plan area, using portions of another hemispherical surface called a pendentive dome. In this solution the top dome had a diameter equal to the side of the square area below. The pendentive dome was a hemisphere encompassing the square below, but truncated at the base of the top dome and the vertical side of the square as indicated in Figure 2.4a. In domes where stresses in the parallel directions exceed the weak tensile strength, cracks along the meridians occur, and as the cracks increase, the behaviour approaches that of independent arches, defined by the arches along the meridians, the bending moments may then become significant and unstable situations

may occur (Croci 1998). Interesting and efficient roof structures can be generated on a rectangular grid by using variations of this junction illustrate in Figure 2.4b.



(a)



(b)

***Figure 2.4: (a)** combinations of domes and vaults; **(b**) intersections of a pointed vaults.*

3.2 Structural Aspects of Historic Masonry Buildings

Masonry is a collection of dry stones (or bricks etc), sometimes squared and well-fitted, sometimes left unworked, that are placed one on another to form a stable structure. Mortar may be used to fill interstices, but this mortar will have been weak initially, may have decayed with time and cannot be assumed to add strength to the construction. Thus an arch built by assembling wedge-shaped stones (voussoirs) upon centering will stand when the centering is removed. Gravity ensures that each stone presses on the next, and a state of compression exists throughout the masonry. If the arch were inverted it would fly apart, unless there were strong adhesive (mortar) between the stones that would help them to resist the disintegrating tensile forces. Stability of a whole masonry structure is assured by the compaction under gravity of the various elements; a general state of compressive stress exists, but only feeble tensions can be resisted (Jacques Heyman)

Masonary is a brittle composite material which is weak in tension due to very limited bonding between mortar and units, but much stronger in compression. Masonry and stone will normally crack when subjected to tensile stress. Visible patterns of cracking particularly in vaulted masonary buildings indicate the nature of internal action. These cracks indicate the direction of the principal compressions, which are parallel to the cracks because cracks in masonary occur close to the right-angles to the principal tensile stresses. In spite of cracks masonry structures may stand safely if a resisting scheme involving only compression forces (as an embedded arch or system of arches) is developed. Cracking in masonry do not necessarily point to structural problems.

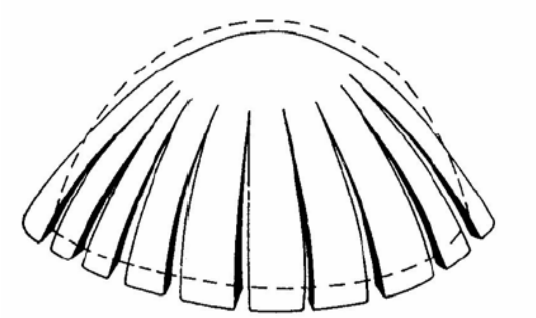


***Figure 3.2.1**. Cracking in masonry do not necessarily point to structural problems*.

The three main structural criteria are those of strengthe, stiffness and stability. The structure must be strong enough to carry whatever loads are imposed, including its own wight; it must not deflect unduly; and it must not develop large unstable displacements, eheter locally or overall. If these three criteria can be satisfied, then the designer can run through a check list of secondary limit states to make sure that the structure is otherwise serviceable. An immediate, and paradoxical dificulty arises when the idea of strength, stiffness and stability are applied to masonry. Ancient sturctures-the Roman Pantheon, for example, or a Greek temple- seem intuitively to be strong enough; they are still standing, and evidently the loading (self-weight, wind, earthquake) has not, over the centuries, caused failure to occur by fracture of the material. It is a fact that the mean stresses are low in a typical masonry structure, cracking and local spalling may be seen, but these seem hardly to affect the structural integrity of the whole (Heyman, The stone skeleton).

The behaviour of the masonry structure can be examined in the light of three simplifying assumptions, each one of which is not strictly true, and each of which must in any case be tested in the light of contrary experience with a particular building. The three assumptions are that masonry has no tensile strength, that stresses are so low that masonry has effectively an unlimited compressive strength and that sliding failure does not occur. The first assumption will clearly lead to an underestimate of the strength of a structure, but it is not unduly conservative. Individual blocks of stone may be strong in tension (and corbelled construction relies on this), but mortar between stones is indeed weak. An attempt to impose tensile forces would pull the work apart. The assumption that the material has unlimited compressive strength will be approximately correct for average stresses. Stress concentrations can arise, however, which will lead to local distress evidenced by surface spalling (splintering), but the overall structure will not fail. (Jacques Heyman)

It is not possible to avoid all cracking in masonary because, even in the abscence of extreme events, some cyclic movements continue to take place. The sources inducing crackings may also be movements due to thermal and moisture changes, fire or introduction of central heating in the historic buildings. Therefore, crackings should be controlled by means of appropriate reinforcing materials yet the correct choice of materials in a particular case should be made by considering the local and existing conditions.

The correct choice of materials for structural consolidation is different. In any wall or pier it is desirable that the core should be as strong and stiff as the facings and should be well bonded to them. Except in the case of some solid brick walls of solid ashlar masonry and most Roman concrete walls, this is rarely the case. Short of completely reconstructing such walls and piers with weak rubble cores and the like, grouting is the only available technique, possibly assisted by the introduction of a limited amount of reinforcement. Stronger mortars for this purpose are desirable, but Portland cement mortar again must be criticized on the grounds of excessive hardness, lack of elesticity and impermeability.

With uniform loading on ground that offers uniform support, a building should settle uniformly even if the ground loading is excessive by modern standards. In the case of and historic building, such settlement will usually have taken place long ago and now matters little. Even a linearly varying settlement, perhaps as a result of a continuous variation in the support conditions over the length or breadth, matters little if the resulting bodily tilt of the building is not excessive in itself or in relation to its height.

Inclinations of 1% are quite common. Differential setlements of a non-linear kind, such as a greater setlement in the centre of a building, are the ones that matter most, since they can be absorbed only by weakining deformations of the superstructure.

Different groups of historic buildings can be classified, ranging from the simplest early huts to much more complex forms for following the intricate pattern of interrelated developments of elements and complete buildings and gaining an insight into the main characteristics of each structural form.

If we discuss the elements in broad groups we may classify them in four as follows:

- Beams, Arches, Vaults (barrel vault, groin vault, rib vault, fan vault) , domes, spires
- Trusses and Frames
- Walls, Piers and Columns,

- Foundations

Classifications and general understandings of the essential characteristics and typical maladies of different structural groups are a helpful tool to understand the individual yet one must not forget that they should not be regarded as more than a starting point and each building must be considered as a unique individual structure to care it properly.

Each sturcture must be considered as a whole before making an assesment because, for instance considering the stability of an arch without considering its supports and the stability of the ground on which they rest will not be sufficient to diagnosing the maladies of the building.

Arches

Arches must be considered in the context of the wall or arcade in which they are situated and of the thrust they exert in which they are situated and of the thrsut they exert on their abutments. Pointed and parapolic arches are special cases. The hinging effects is most relevant when the depth of the voussoirs is small in relation to the span. When voussoirs are deep in relation to span, arches are able to absorb deformations generally by slipping, as in the case of the Colosseum. In all cases the condition of the abutments is vital to arches. If these supports are sliding, sinking or rotating, the stability of the arch itself is suspect; therefore, the abutments always need most careful consideration.

The sources of stress in arches come from the weihgt of the structure, dead loading and applied loads. In long-span masonary arches the dead load sometimes caused collapse when the shuttering was removed. Stresses can be increased by the settlement of an abutment.

Loads can be calculated by graphical means and then lines of thrust can be shown. Such diadrams are very helpful when investigating stresses in an arched construction.

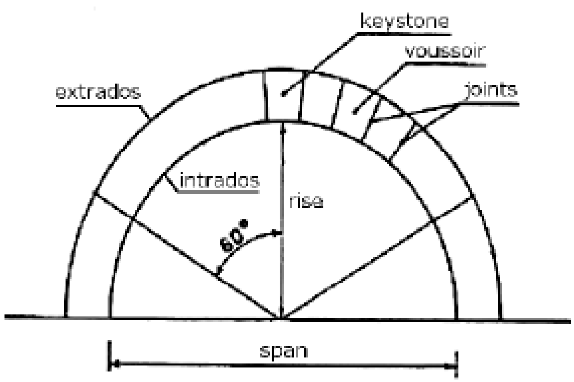


Figure 3.2.2

The masonary has a well known neglgible tensile strength, so the safety condition for masonary arches is a chieved when the line of thrust, coincident with the funicular polygon, is kept inside of each section of the arch itself. When the resultant of the internal forces moves outside the central core, the section partialises and a phase of high deformations starts (Heyman 1982)

According the Heyman (1982) hypothesis the behaviour of arches and vaults can be explained as follows:

1-Masonary has no tensile strength; although stone and brick exhibit a tensile strength, the mortar between the voussoirs could be deteriorated or the coussoirs be laid dry.

2- Masonary has an infinite comspressive strength; although a local crushing of the masonary could be observed, overall collapse due to masonary crushing is unrealistic, because the stresses are usually low in comparison to the material's compressive strenght.

3-Sliding failure can not occur; it is assumed that friction between voussoirs is high enough, or stones are effectively interlocked, so that they can not slide one on another, although it is possible to find occasional evidence of slippage in a masonary structure.

According to lower-bound theorem with terms applicable to masonary:

"If a thrust line can be found, for the complete arch, which is in equilibrium with the external loading (including self weight), and which lies everywhere within the masonary of the arch ring, then the arch is safe"

The importance of this theorem lies in the fact that the thrust line found in this way need not to be the actula thrust line.

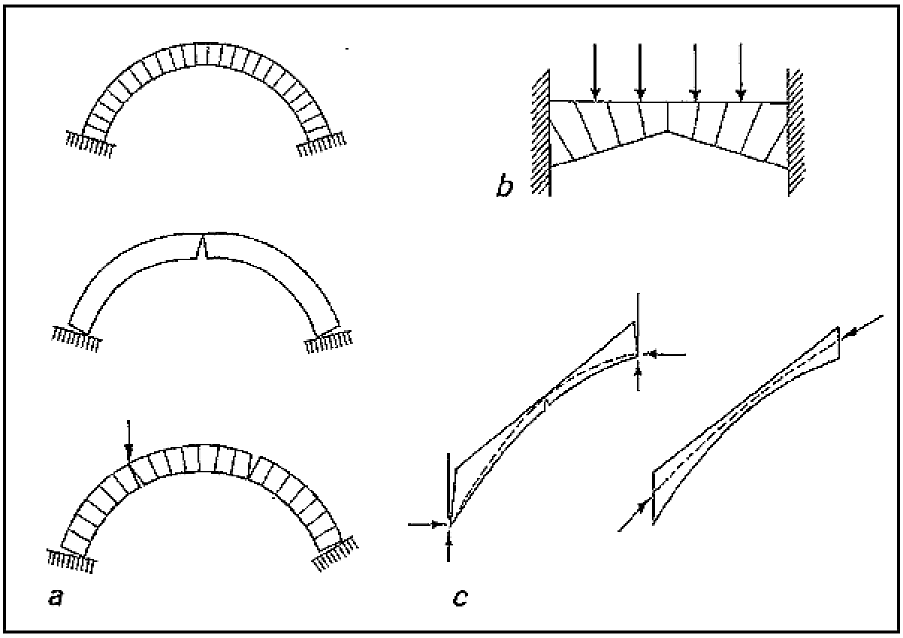


Figure 3.2.2 *Masonry arches: (a) (from top to bottom): fitting perfectly between abutments; cracking to accommodate an increased span; at collapse by the formation of four 'hinges'; (b) flat arch; (c) flying buttress: (left) passive state; (right) working state*

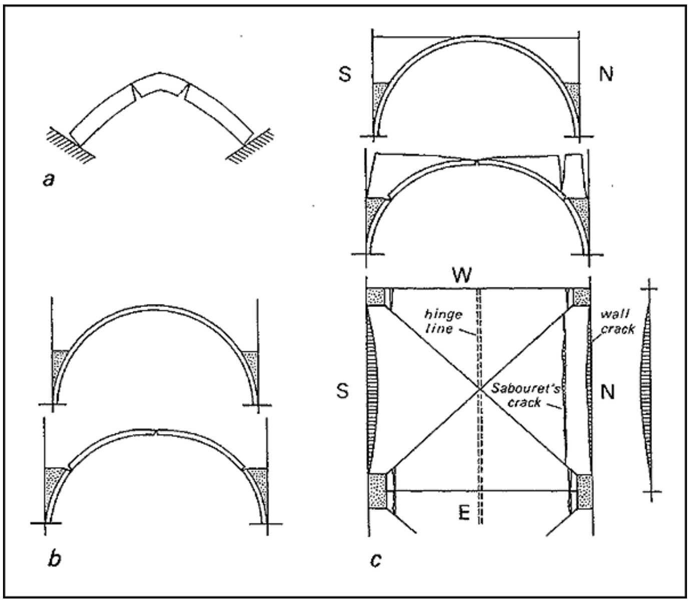


Figure 3.2.3 *Crack development: (a) pointed arch; (b) barrel vault; (c) quadripartite vault*

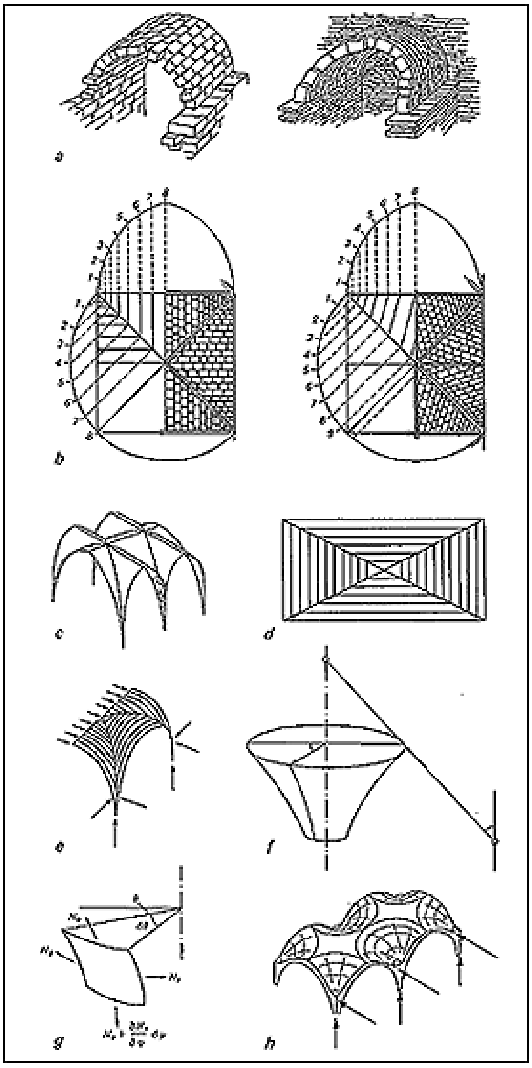


Figure 3.2.4
Masonry vaults: (a) semicircular barrel vault: (left) built with supporting formwork; (right) built without support; (b) stonework coursing for vaults: (left) French way; (right) English way; (c) quadripartite vault seen from above; (d) rectangular vault thought of as a series of arches; (e) forces necessary to maintain the equilibrium of a quadripartite vault; (f) basic shell of revolution defining the surface of a fan vault; (g) stress resultants acting on an element of a fan vault; (h) basic assembly of a complete fan fault

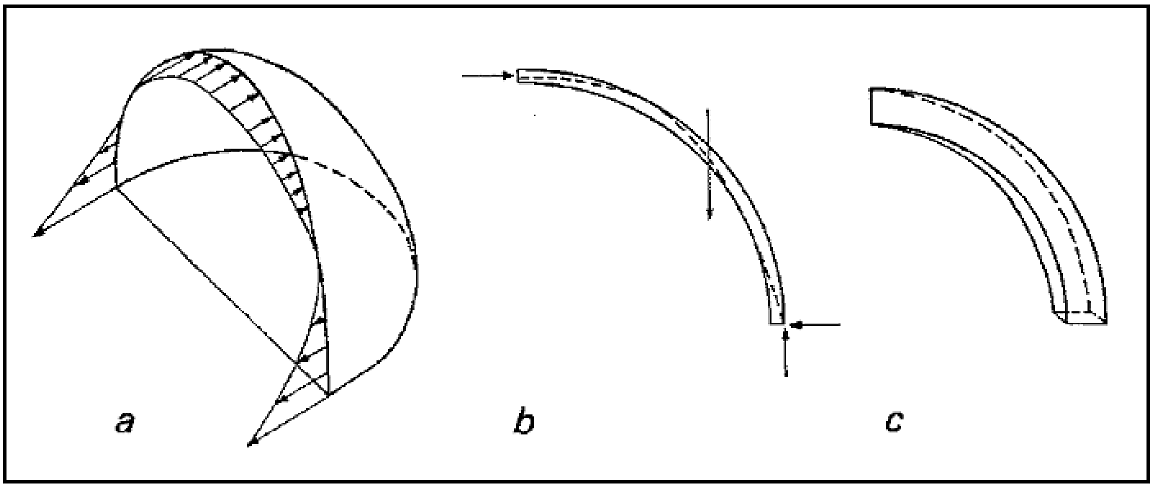


Figure 3.2.5 *Masonry domes: (a) membrane hoop stresses acting in a thin hemispherical dome; (b) minimum thickness for a sliced masonry dome; (c) an 'arch' isolated from a dome by meridional cuts*

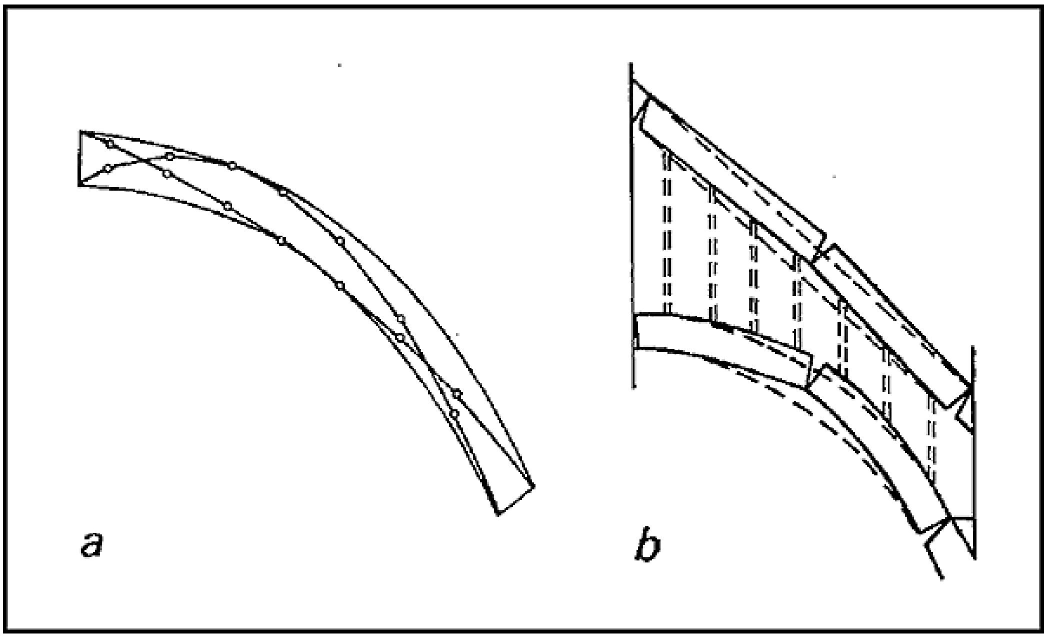


Figure 3.2.6 *Flying buttresses at Amiens Cathedral: (a) limiting positions for the thrust lines in the lower rib; (b) possible buckling mode of failure*

3.3. Materials Used in Historic Buildings

Bernard M Fielden, in his book- Conservation of Historic Buildings, gives a description for **"an historic building"**

"Briefly, an historic building is one that gives us a sense of wonder and makes us want to know more about the people and culture that produced it. It has architectural, aesthetic, historic, documentary, archaeological, economic, social and even political and spiritual or symbolic values; but the first impact is always emotional, for it is a symbol of our cultural identity and continuity—a part of our heritage. If it has survived the hazards of 100 years of usefulness, it has a good claim to being called historic."

It is, therefore, essential to study Its structural and material condition which means investifating the different phases of construction of the building complex, interventions throughout its life , any internal or external peculiarities and the environmental context of the surroundings of the building.

The two main structural materials found in historic buildings are timber and masonry. Members of timber systems can resist tension as well as compression, and the joints can resist tension, but the strength of the total system is limited by the strength of the joints. The span is limited by the length of timber available. Known timber structural systems include, for walls, solid timber tree trunks set vertically or horizontally, post and lintel systems, box frames and cruck framing, and for roofs, simple beams, rafters and tie beams, trussed beams, arch braced beams, hammerbeam trusses and framed trusses. Masonry structural systems include simple post and lintel systems, as for timber, but the commonest sort of masonry system is mass construction with pierced openings; stone skeleton frames were devised for sophisticated Gothic buildings with pointed arch and flying buttress, through which the lines of thrust were intended to flow within the physical limits of the masonry. In contrast to timber, the total strength depends more on the compressive strength of the materials rather than on the joints.

In the case of stone, its strength, quarrying facilities, lifting and transportation were also vital considerations. The longest stone beams known are in Fukien, China. They are 23 m (75 ft) long, are of granite and were used to build a multi-span bridge. Calculations show that the tensile strength of the stone is just sufficient to support this length. The length of wooden beams is limited by the length of available timbers. [25]

If we compare the main structural materials used in historic buildings (stone, brick, mass concrete and timber) with cast iron and wrought iron, structural steel and aluminium alloy with respect to the tensile and compressive strength, density and stiffness , striking differences are found. Except for timber, historic materials are all very weak in tension, whereas the materials used in the nineteenth century and later, with the possible exception of cast iron, are strong in tension and exceptionally so in compression. Wrought iron, even the weakest of them, exceeds the best stone in compressive strength.

All the main materials available now in use are derived from the same basic materials that have been used for thousands of years—earth, rock, timber and metals—but have been improved in utilization and manufacture. Other new materials, such as plastics, synthetic resins and aluminium, lack stiffness in practice, so play a secondary role in construction, because for most structural purposes stiffness is as important as strength. In the context of historic buildings, reinforced concrete must be considered a new material.

3.4 Causes of Decay in Materials and Structure

Of the causes of decay in an historic building, the most important and universal ones are:

- Gravity and the structural actions resulting from gravity, (Gravity is both the force that keeps buildings standing and the major cause of their destruction).
- Actions of man
- Diverse climatic and environmental effects—botanical, biological, chemical and entomological.

when analysing the causes of deterioration and loss in an historic building, the following issues must be studied:

(1) What are the weaknesses and strengths inherent in the structural design and the component materials of the object?

(2) What are the possible natural agents of deterioration that could affect the component materials?

How rapid is their action?

(3) What are the possible human agents of deterioration that could affect the component materials or structure? How much of their effect can be reduced at source.

Man-made causes of decay are serious and can only be reduced by forethought and international co-operation. Neglect and ignorance are possibly the major causes of destruction by man, coupled with vandalism and fires.

CHAPTER 4

EVALUATION & ASSESMENT of EXISTING MASONARY BUILDINGS

4.1 Investigation

In order to diagnose the causes of decay and propose an effective cure that involves only the minimum intervention, it is an essential preliminary before embarking upon historical research or any further study of the structure such as structural analysis, including soil mechanics, investigations to carry out a proper assesment & investigation works which should be reported to include the following main items:

- Initial report based upon visual inspection, listing all defects and describing and studying the building.
- A maintenance plan. Approximate itemized estimates for immediate, urgent, and necessary repairs and other desirable works.
- Historical research and analysis supported by photographic records.
- Recording of the initial state of the building; soil mechanics, humidity studies and opening up suspect parts.
- Further studies. Structural analysis.
- Final estimates and proposals with specifications and full report for submission for governmental
- grant covering all the above factors, as they modify each other.

4.2 Decision Making

Clear recommendations for action must follow from an initial investigation.

Recommendations must be scaled to what is possible in the context of the situation. They should be given in order of priority as follows:

(1) *Immediate work* is what must be done straight away to deal with work necessary for the safety of the fabric or its users.

(2) *Urgent work* is that required to prevent active deterioration, i.e. attack by insect or fungus or penetration by rain water.

(3) *Necessary work* is that required to the ‘standard’ appropriate for the building and its present or proposed use in the context of the client’s resources and includes items of preventive

maintenance. This category can be subdivided into ‘good housekeeping’, ‘rolling programme’ and ‘major works’.

(4) *Desirable work* is what is recommended to enhance the use or appearance of the building or what is necessary for re-evaluation or adaptive use of the building.

(5) *Items to be kept under observation* are, for example, active movements and roofs or installations that are nearing the end of their life and may need renewal within 10 or 15 years.

Where remedial interventions are considered to be necessary, they should respect, as far as possible, the philosophy of compatibility(i.e. compatibility with the character and integrity of the original structure). The materials similar to the original ones, should be utilized, as far as possible. Where different materials are substituted, their physical characteristics should be compatible with the original, particularly with regard to porosity and permeability, and care should be taken not to introduce elements of excessive strength or stiffness into a structure which will usually be less stiff and more accommodating to long-term movements than contemporary structures. The final choice of the approach to be adopted should be made only after a proper appraisal (consistent with the scale of operations and the resources available) of alternatives, with some eye to the future such that an action of intervention may be substituted with a better technique in the future.

CHAPTER 5

CONSERVATION AND STRUCTURAL STRENGTHENING OF MASONRY STRUCTURE

5.1 General

Conservation or restoration of historic buildings involves different challenges. It is is multidisciplinary task carried out with the involvement of different competencies like architects, engineers, arcehologiests and historians. It is very important to follow a methodology which includes survey, diagnosis, saftey evaluation and the selection of the correct and effective intervention technique fitting best with the strengthening requirements for the specific building with respect to it's original conception and the historic value.

This is a difficult task because of the complexity of the geometry of historic monuments, the variable and often unpredictable characteristics of original materials, the different building techniques, the absence of knowledge on the existing damages occured throughout their life due to several reasons, and the lack of applicable codes. In addition, restrictions in the inspection and the removal of specimens in buildings of historical value, as well as the high costs involved in inspection and diagnosis, often result in limited information about the internal structural system or the properties of existing materials.

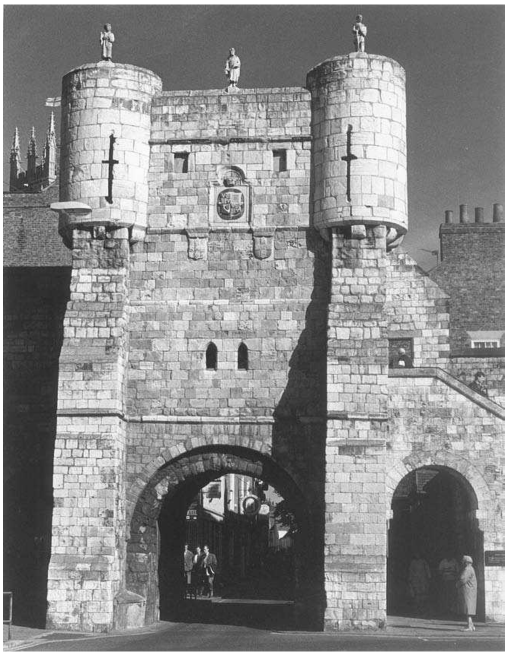


Figure 5.1.1 *Bootham Bar, York: Mediaeval limestone masonry, largely built as two skins of dressed stone with a core of rubble and mortar. Being originally built as a fortification, the heavy construction of this gate-tower has ensured that it is still intact despite the quite severe weathering of the surface.*

Although there are a lot of materials and intervention techniques available for structural restorations recent philosophy underlying restoration recommends to prefer prefer techniques that are fullfilling the criterias like minimum intervention, compatibility, durability, reversibility or substitutability. In principle this philosophy is correct, but one should not forget that any decission may not always be error-free and it is therefore unwise to exclude the possibility of exploiting better techniques and materials if and when they are developed. conservation requirements .

A full understanding of the structural behaviour and material characteristics is essential for any project related to architectural heritage. Diagnosis is based on historical information and qualitative and quantitative approaches. The qualitative approach is based on direct observation of the structural damage and material decay as well as historical and archaeological research, while the quantitative approach requires material and structural tests, monitoring and structural analysis. Often the application of the same safety levels used in the design of new buildings requires excessive, if not impossible, measures. In these cases other methods, appropriately justified, may allow different approaches to safety.

5.2 Stabilization & Repair & Strengthening Techniques with Iron and Steel

An early reference to iron ties is given in Richard Brown, Sacred Architecture. This book by Professor Brown, who designated himself an architect, is uneven and not very significant for our purpose [35]. But on p.107, in speaking of the construction of Beauvais Ctahedral in the thritieenth century, he says: “At this time, the pillars being placed too far apart, the vaulted roof threatened to fall in, which actually took place after means had been adopted to support it by iron braces and chain, to hold the side walls together”.

Lethaby (King’s Craftsmen, pp.145 f.) speaking of the “new work” of Henry III at Westminster Abbey, which began with the eastern and three northern bays of the apse.

In his later book (Westminster Abbey Reexamined, pp.103-5 and notes, p 297) Lethaby has more to say about the use of iron in the Chapter Houses at Salisbury and Westmisnter, and includes a perspective drawing of the interior of the latter with the tie-rods showing. He notes (p.105) that beside the eight ties mentioned, the windows are threaded by three tiers of strong iron bars, which are still wholly or largely original. One may assume thet they link up the angles and form bands right around the octagon. Scott found, from the fact that the western window originally had four lights like the others.

It appears from these and similar accounts that iron was used far more extensively to tie together the masonry members in the great English Chruches than it was in those of France. Westminster would seem to indicate that English builders progressed from wood ties (probably because the were so much cheaper than iron) , to continuous through-the pier iron ties, to eyed iron ties attached to projecting iron hooks, to no ties at all: as though the the builders proficiency grew along with their increasing confidence.

With repect to the presence of ties that were placed in churches at the time of their construction and that remain in place to this day, a sharp distinction should be drawn netween those of western Europe and of Eastern Mediterranean area. Lethaby (King's Craftsmen, p.361) seeks to establish the widespread use of ties throughout the ages by recalling that "In old schools of architecture the use of iron and wood ties for binding arches was quite general. The most important Byzantine Buildings S.Sophia, Constantinapole-today-Istanbul, and S. Demetrius, Salonika- have such ties throughout. Of the early Arabic school, the Dome of the Rock and the Aksa mosque have series of ties at the springing of the arches.

The arch ties of Byzantine and other eastern churches are conspicous and practically universal features of these buildings, large and small alike, because the lands in which they are found are subject to earthquake tremors. Hence, from the start as well as ever since, these buildings have needed to be strutted by ties that are resistant to both tensile and compressive stresses (cf. Choisy, L'Art de batir chez les Byzantine, cahp. X, 'Les Chainages',pp.115-22).

Figure 5.2.1 *A 1950 artist's impression of what the Westminster Cathedral will appear when the decoration is finally complete, from* Westminster Cathedral: From Darkness to Light

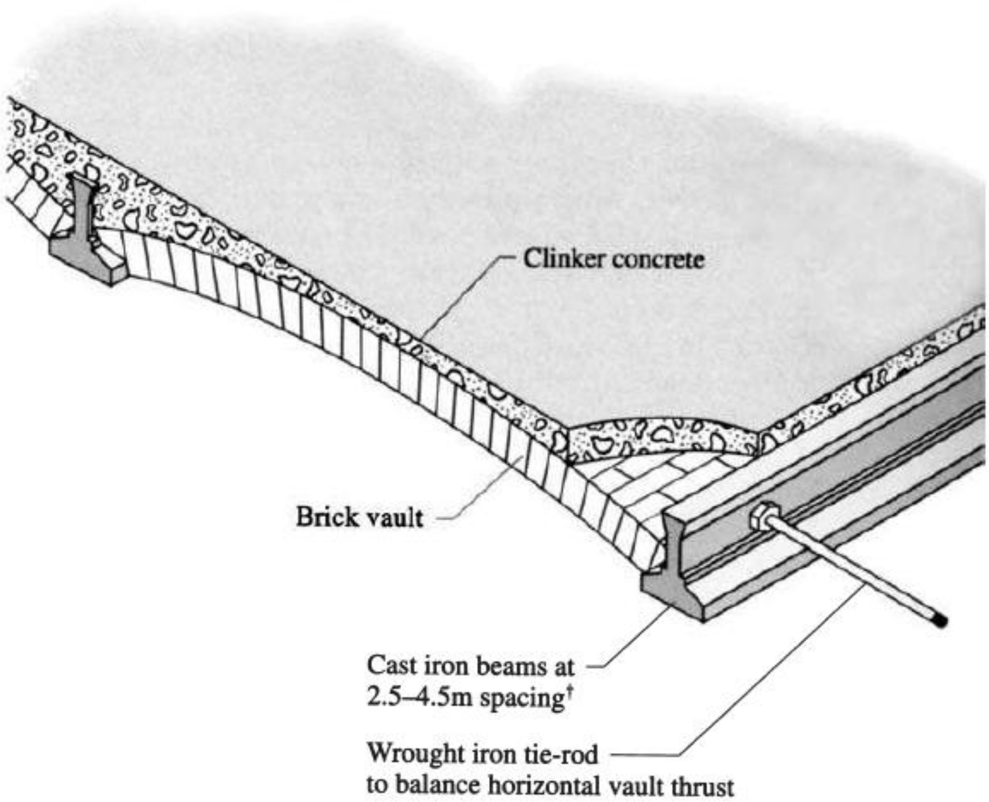


Figure 5.2.2 *Jack-arch floor, supported on cast-iron beams.*

5.2.1 Structural Assesment & Structural Appraisal

Appraisal of an existing structure requires an 'analysis' in the broadest sense of the word, in which structural calculations form only one part of the whole process. Drawings, defining the structure, are fundamental to the whole process, but there are other basic considerations to be made before launching into numerical calculations. [90]

Facts to be Taken into Account

1. The physical presence and condition of the building; it is there, it has performed

its function(s) for so many years, it may show such and such signs of 'fair wear and tear' and it may or may not show signs of past or present structural distress.

2. On closer examination, the structural system may turn out to be quite complex with, in places, several unorthodox but perfectly effective load-carrying assemblies providing multiple load paths.

3. The structural members are there and their dimensions can, in principle, all be verified (although there may be difficulties in practice) and the actual strengths of the materials, in situ, are there and can, in principle, be tested.

Fact 1 can be, and should be, used as a qualitative check on any numerical assessment. If the building stands up and the calculations show that it should have fallen down, the calculations or the assumptions on which they are based must be wrong.

Fact 2 is likely to be a challenge, but unless the obvious primary system can be shown to be adequate on its own, the challenge has to be faced by the engineer, even if it involves 'non-linear analysis' of a three-dimensional structure: to

do otherwise is tantamount to condemning a perfectly good structure out of laziness. 'Linear-elastic' analysis assumes that Hooke's straight-line stress–strain relationship applies regardless of the stress intensity; non-linear analysis takes account of the curvature of the real stress–strain diagram and of any yielding of the material prior to fracture *Fact 3* allows the elimination or reduction of some or all of the design uncertainties (described in Sec. 2.3 of [90]) and thus leads straight to Sec. 2.5. of [90])

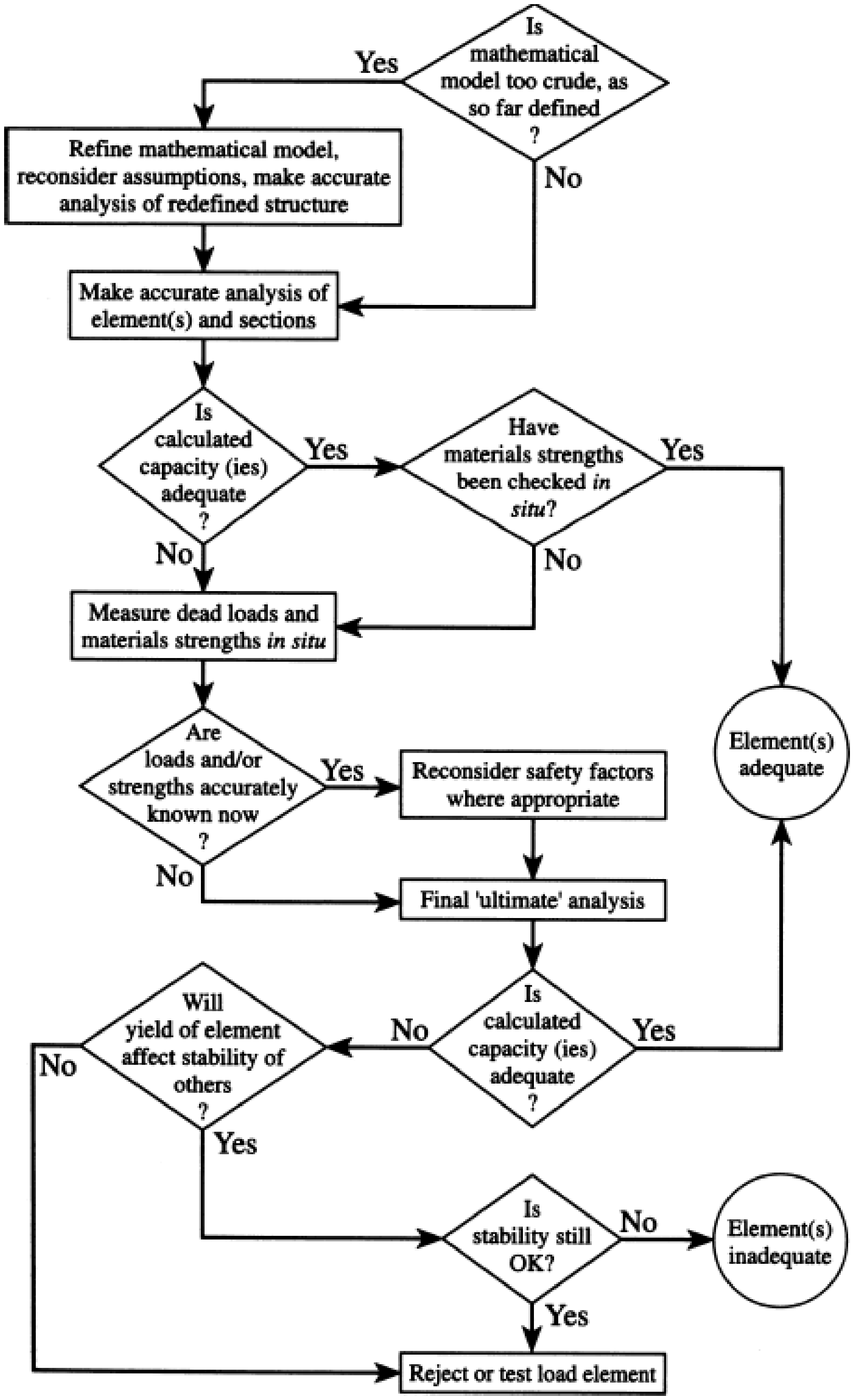


Figure 5.2.1.1 *Paths of improved appraisal (after 'Appraisal of Existing Structures', 1996). [90]*

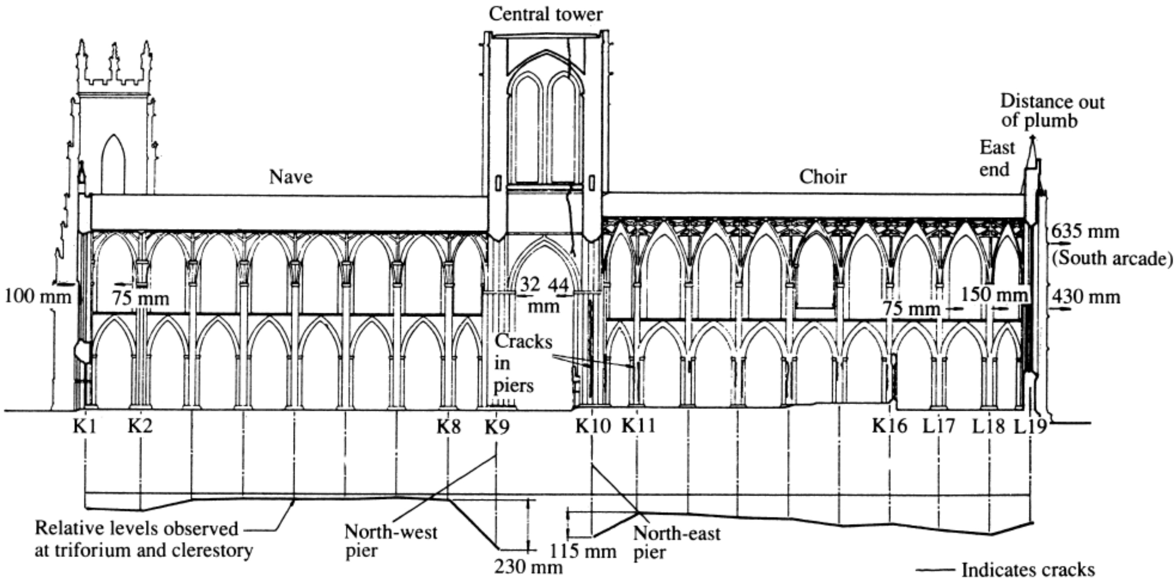


Figure 5.2.1.2 *Example of drawing, showing measured and observed defects (from Dowrick and Beckmann, 1971).Measurement of past and ongoing defects is the essential part of an assesment activity.*

Every structural appraisal must have a report as its end result covering Synopsis, List of contents, Terms of reference, Documents examined, Description of the structure, Inspections, Additional (verbal) information (if any) Sampling and testing (if any), Calculation checks (if any), Discussion of evidence, Conclusions, Recommendations.

5.2.2 Causes of Damage

Masonry is generally relatively strong in compression, has a moderate resistance to sliding along the bed-joints (shear) and is usually weak in tension. Masonry structures are therefore generally designed to work in compression and sometimes in horizontal shear. Damage to masonry structures usually takes the form of tension and/or shear cracks caused by imposed deformations, e.g. differential settlement, or by excessive lateral forces, e.g. from arch thrusts; or alternatively it may suffer from incipient bursting due to buckling of ashlar or expansion of rubble fill in structures that consist of two ashlar faces with a core of rubble and mortar.

All unintended gaps in masonry are usually referred to as ‘cracks’. For a proper assessment of causes and significance, one should however distinguish between the following categories:

- cracks through units,
- opening of bedjoints and butt joints and
- sliding’ along bed-joints.

Whilst all three may have structural implications (a) is more likely to lead to a risk to people from falling bits of masonry.

- Causes of damages may briefly be categorized as follows:
- Cracks Due to Imposed Deformations
- Cracks Due to Excessive Lateral Forces
- Cracks Due to Eccentric Loading
- Cracks Due to Embedded Ironwork
- Defects in Brickwork Cladding on Reinforced Concrete-framed Buildings
- Problems with Cavity-ties

It is obvious that repair or strengthening must be preceded by determining the real cause of the observed defects, and if the cause is still active, it must be eliminated. Otherwise the repairs will be short-lived. There is a parallel in the field of medicine: any treatment must be preceded by a correct diagnosis.

The techniques used for repair or strengthening must depend on whether thecracks are ‘live’, i.e. still moving, or ‘dead’, i.e. being due to a cause, which has been eliminated in the past. If the cracks are live, the cause should be treated, if at all possible, and the subsequent repairs to the masonry will be largely cosmetic.

5.2.3. Repair and Strengthening of walls and piers

Compatibility

For all types of construction, repair materials must be compatible with the original materials. This applies to the stone or brick and the mortar. There has been many problems in the past due to interventions with incompatible materials.

The quality of the repair mortar will influence the weathering performance of exposed masonry. If it is too weak and porous, it will be eroded too quickly. If it is too strong, it will almost invariably be too impervious (adding a small proportion of Portland cement in substitution of lime will only increase the strength of the mortar marginally, but it will reduce its permeability significantly). It may then trap water that has got into the units, e.g. from driving rain, and prevent it from draining down; such water, lodging in the units over each impervious bed-joint, is likely to exacerbate frost damage in the form of spalling-off of the face of the bricks or stones.

The other objection to the use of modern strong mortars is that they prevent the slight sliding of stones or bricks on the bed-joints which, when walls are subjected to imposed deformations such as differential settlement, usually only lead to minor cracking along the joints. With such sliding movement being prevented, cracks will form through the stones or bricks and such cracks are obviously far more disfiguring and more difficult to repair than cracked joints.

The current consensus is that repair mortars should have a composition as close to the original as possible.

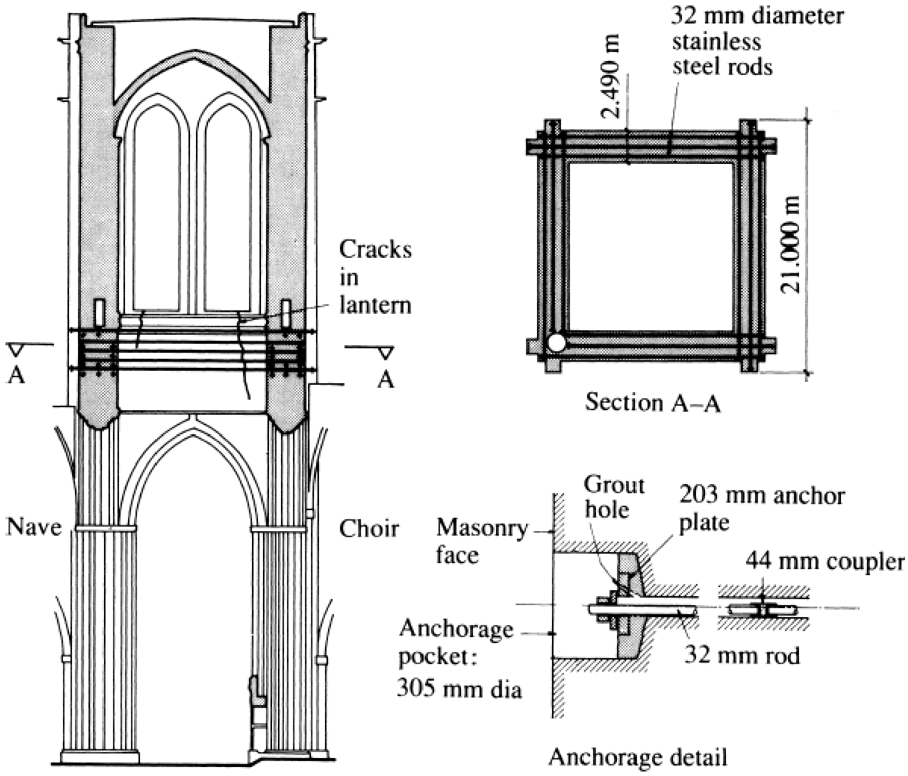


Figure 5.2.3.1 *Tensile strengthening of the spandrel walls of the central tower of York Minster (from Dowrick and Beckmann, 1971).*

Strengthening to improve Tensile Resistance

In walls which are cracking due to tension stresses, which are produced by differential settlement or by thrusts from roof structures, and if these root causes cannot practically be completely eliminated, it is beneficial to provide tensile resistance against further opening of the cracks. This can be by means of tie rods which, whilst inside the building, are external to the masonry. This is frequently seen in church towers, that have been ‘strapped up’ in the past. Alternatively, they can be placed in holes, drilled longitudinally within the thickness of the wall, as was done in the central tower of York Minster (Figure. 5.2.3.1).

Another method is to form horizontal rebates in the masonry and insert reinforcing bars, which are mortared into the rebates to provide horizontal tensile strength. A variation of this, which is appropriate to relatively thin brick walls, is to rake out every third or fourth horizontal joint and embed thin stainless steel reinforcing rods or stainless steel strand, such as used in the standing rigging of yachts, in the re-pointing mortar. Theoretical and experimental research at Bath and Karlsruhe Universities has demonstrated that this is an effective alternative to foundation underpinning, when remedying settlement cracking in brick façades (see Cook et al., 2000). For joint reinforcement, as well as for tie rods, the future forces in the 'uncracked' wall should be assessed, as far as practicable, by calculation

or otherwise, and used as the basis for the 'design' of the reinforcement. Where the cracks extend from, and are most serious at, the top of the wall, the remedy may be a complete capping beam of reinforced concrete. This device is most appropriate for thick walls where the beam can be formed in a chase, cut in the top of the core of the wall, so as to leave the face masonry intact on both sides. Such beams are particularly effective in towers, where they form complete ring beams. In a bell tower, the reaction forces, created by the swinging of the bells, can be evenly distributed, from the bell frame over the entire perimeter of the tower walls, by such a ring beam. Objections are sometimes raised against the introduction of alien materials, such as concrete and particularly steel, into masonry. The use of more 'sympathetic' forms of construction, such as the introduction of courses of tiles, is advocated as an alternative. Two or three courses of tiles with staggered butt joints will.

Anti-seismic Strengthening

Generally, the first step in seismic strengthening of buildings with masonry walls is to make provisions to ensure that the plan shape remains undisturbed: corners must be prevented from behaving as hinges, T-junctions must not pull apart and straight lengths of wall must not be free to bow in plan or move laterally. This requires that the floors are made to act in their plane as stiff diaphragms or plates

and that all walls are tied laterally to the floors. The substitution of timber floors by reinforced concrete slabs is often advocated as a means of creating rigid diaphragm floors; there may, however, be other

means of achieving this. It should be borne in mind that the introduction of concrete floors will add mass at high level in the building. This will lead to an increase of the inertia forces, imposed on the vertical structure during an earthquake, and is therefore undesirable. What is essential is the tying of walls to the floors. At an inspection of the damage after the 1979 earthquake in Kotor and Budva, Montenegro, buildings were observed where an entire wall parallel to the floor joists had fallen away, whilst the walls on which the floor joists were supported and which therefore had a modicum of lateral restraint were still standing. Some of the anchoring and tying devices, described under previous sections may be useful for the purpose of providing lateral restraint to walls. It should, however, be remembered that seismic movements may temporarily *diminish* the vertical pre-load from gravity; this may reduce the holding power of expanding anchors. It should also be borne in mind that the reciprocating nature of the seismic inertia forces may cause stone(s), in which tie rods or anchors are fixed, to be pulled out by ratchet action. Where the vertical dead load on walls is insufficient to counteract the seismic effects, e.g. in the upper storeys of buildings, vertical tensile resistance has to be provided. Reinforced concrete 'columns', which in reality are just concrete casings to reinforcing bar tie rods, are very often advocated for this. If located within the thickness of the wall, they do however require vertical chases to be cut; these impair the horizontal integrity of the wall. If placed externally to the

wall, such tie columns are visually obtrusive and difficult to connect to the top of the wall. Depending on the nature of the construction of the wall, it may be possible to drill vertical holes through the height of the wall in which tie rods can be placed, terminating in rock anchors in the foundations. Drilling such deep holes does, however, require the use of water or air flush. The former may find its way through joints to the face of the masonry and/or it may be necessary to drill relief holes to allow the water to escape in order to reduce the hydrostatic pressure. As the released water carries the drilling dust in suspension, it can cause unsightly streaking. In addition, seepage can damage wall paintings. Air flush of the drilling, on the other hand, will normally produce vast clouds of dust. In the absence of wall paintings, the best course may be to grout in relief tubes to throw the dust-laden water clear of the wall and to point, and thus seal, all joints prior to using water-flush drilling. A further development of the vertical tie bars is to tension them and thereby vertically pre-stress or pre-load the masonry. The advantages of this are that the shear resistance of the wall is enhanced by vertical pre-load and that the natural frequency of the wall is increased and hence removed from a possible resonance frequency of the earthquake motion. On the debit side, the preload will prevent the opening of joints and the extra damping that would otherwise result from this, and it will prevent the possible change in movement pattern that might otherwise increase the seismic resistance . Vertical pre-stressing was rather elegantly used on the clock tower in Dubrovnik. The pre-stressing cables were placed just clear of the masonry in the re-entrant corners; they were anchored on a reinforced concrete slab at the top and in the rock under the foundations and were surrounded by fire insulation inside tubular shields. Apart from the supporting rebate for the concrete slab, the masonry is untouched and the intervention could fairly easily be reversed, if so desired.

5.2.4 Structural Restoration of Arches ,Vaults and Domes

Structural Aspects of Vaults and Domes

Vault is a roof or ceiling in stone, concrete or some other material, constructed upon the principle of the arch. It may be semicircular or segmental in profile, or be made up of arches and segments of arches in various combinations. A vault will commonly have stones or blocks arranged concentrically in such a manner as to support each other in compression. Vaults have also had a deep symbolic significance: the emperor’s canopy in Imperial Rome, the Vault of Heaven in a Christian church, or simply as a demonstration of the financial power and status of a patron.

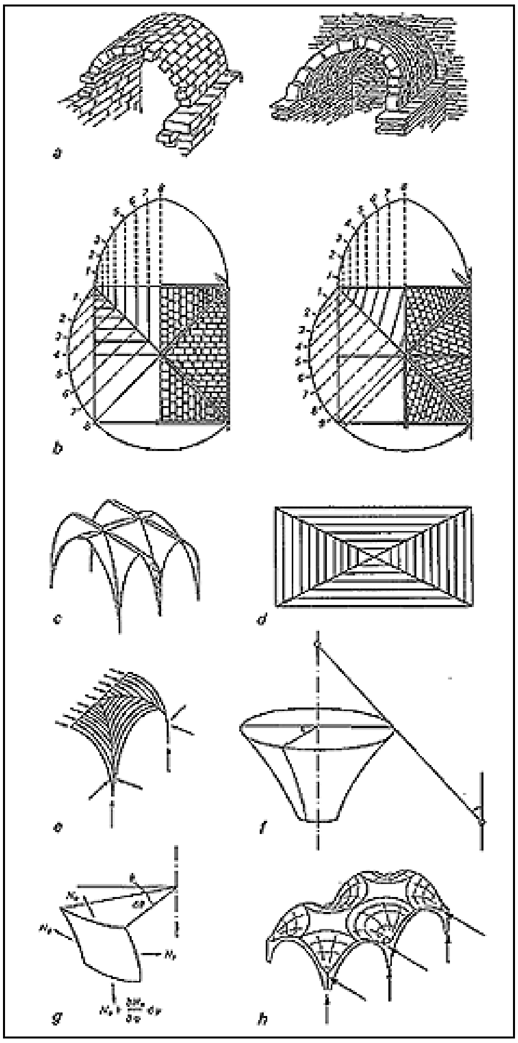


Figure 5.2.4.1.1 *Masonry vaults: (a) semicircular barrel vault: (left) built with supporting formwork; (right) built without support; (b) stonework coursing for vaults: (left) French way; (right) English way; (c) quadripartite vault seen from above; (d) rectangular vault thought of as a series of arches; (e) forces necessary to maintain the equilibrium of a quadripartite vault; (f) basic shell of revolution defining the surface of a fan vault; (g) stress resultants acting on an element of a fan vault; (h) basic assembly of a complete fan fault*

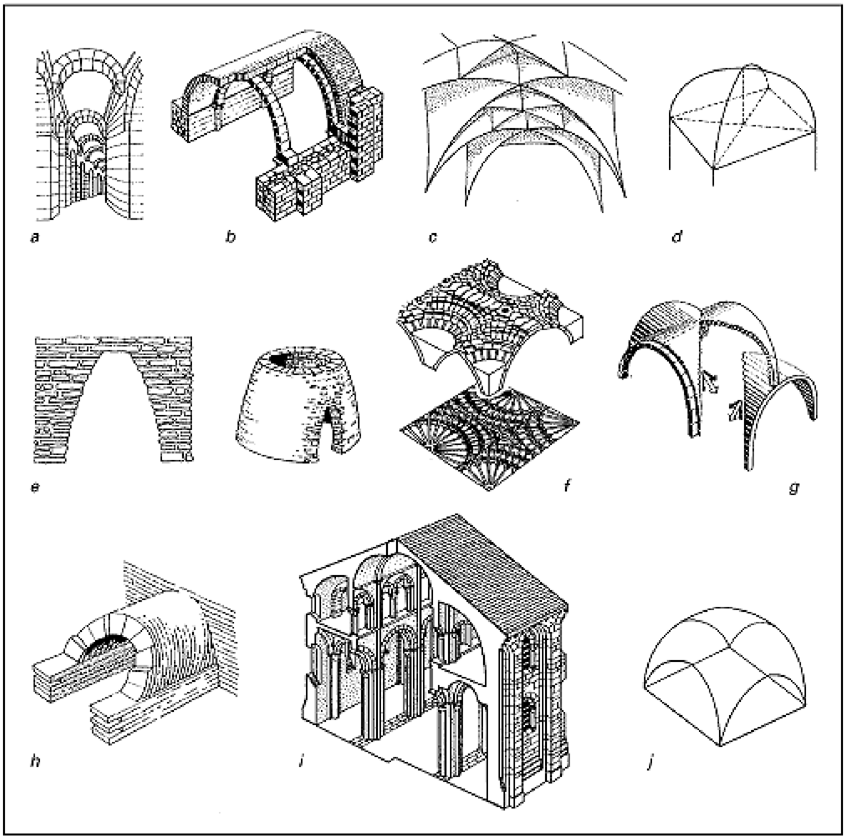


Figure 5.2.4.1.2 *Types of vault: (a) annular; (b) barrel; (c) cellular; (d) cloister; (e) corbelled; (f) fan; (g) groin; (h) pitched-brick; (i) quadrant barrels (over galleries); (j) sail*

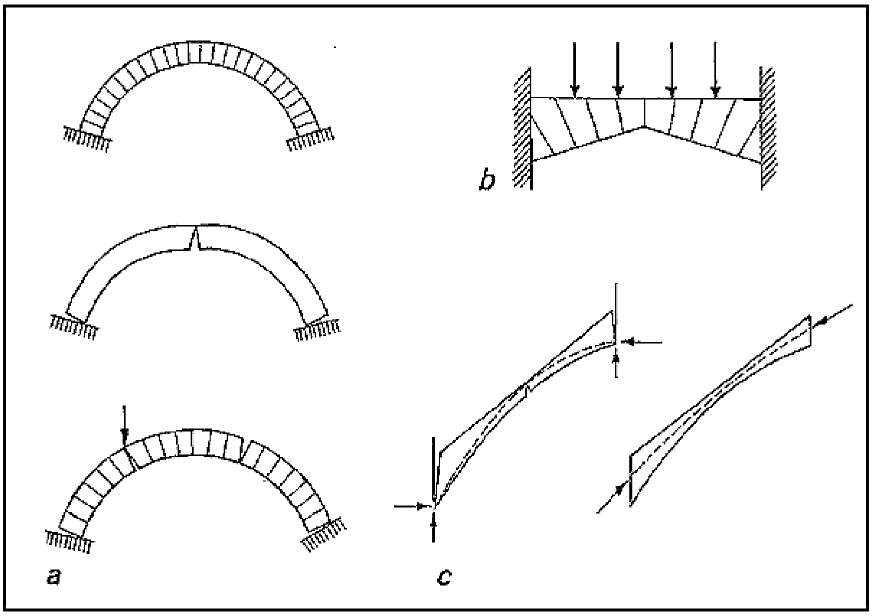


Figure 5.2.4.1.4 *Masonry arches: (a) (from top to bottom): fitting perfectly between abutments; cracking to accommodate an increased span; at collapse by the formation of four 'hinges'; (b) flat arch; (c) flying buttress: (left) passive state; (right) working state*

Causes of Failures

The master Theorem (the Safe Theorem)

Ideas of a hanging cord, corresponding to the line of thrust in an arch, give a vivid picture of the way in which such a simple structure behaves. There is, however, a master theorem of masonry, which, although it applies to any assemblage, no matter how complex, will be presented for the simple arch. The first inescapable fact is that the system of compressive forces in the structure, equilibrating the self-weight of the building and carrying wind and other live loads to the ground, must lie completely within the boundaries of the masonry. The three simplifying assumptions (no tensile strength, great compressive strength, no slip) enable an account of the behaviour of masonry to be embraced within the modern so-called plastic theory of structures. The plastic theorems are then available, of which, for masonry, the ‘safe theorem’ is the master. The simple statement of the theorem is that if a line of thrust can be found to lie within the masonry (for example the funicular arch within the real arch), then the structure can never collapse under the given loads. The statement may seem self-evident, but it has important consequences.

The power of the safe theorem lies in the fact that the satisfactory thrust line found by the analyst need not be the actual thrust line, which will necessarily shift as small, unpredictable changes occur in the environment, causing the structure to respond by a shift in its pattern of cracks. The actual state of a structure will constantly alter. All the analyst need calculate is just one possible satisfactory state, and the real structure will also be able to find such a state for itself. There are no conceivable settlements or other disturbances that can cause the thrusts to shift outside the masonry, as can be seen by considering in detail some of the consequences of settlement, together with some practical applications of the safe theorem.

Settlement and Cracking

The five-minute theorem for masonry may be stated for, say, a flying buttress: if the work stands for five minutes after the timber centering of the flying buttress is removed on completion of the stonework, then it will stand for 500 years (an upper limit depending on the decay of the material). [33] This is all that is needed to satisfy the safe theorem by confirming experimentally, on the real structure, that the shape of the flying buttress is correct. This flamboyant statement assumes that the loads on the flying buttress are static, arising from self-weight and dead thrusts from the nave vaulting. It does not necessarily apply to the upper flying buttresses in some large Gothic churches, the function of which (*see* Figure 5.2.4.1.5 below) is to resist wind forces acting on the great timber roof; and it concentrates only on the masonry, ignoring any possible interaction with the foundations, whose settlement is measured in years, even a decade or so, rather than minutes.

Foundation settlements can lead to geometry changes rather larger than those envisaged so far. Under the crossing tower of a great church there are almost always indications of gross distortions:

the once horizontal courses of masonry in the nave, choir and transepts abutting a crossing may indicate settlements of as much as *c.* 300 mm (now rather more than the thickness of a pencil line on the drawing board). This is both commonplace and straightforward; the crossing piers, themselves highly stressed (at one-tenth of the strength of the material), require high bearing stresses from the soil for their support. The whole of the plan area at the crossing typically requires such a high mean supporting stress that the modern.

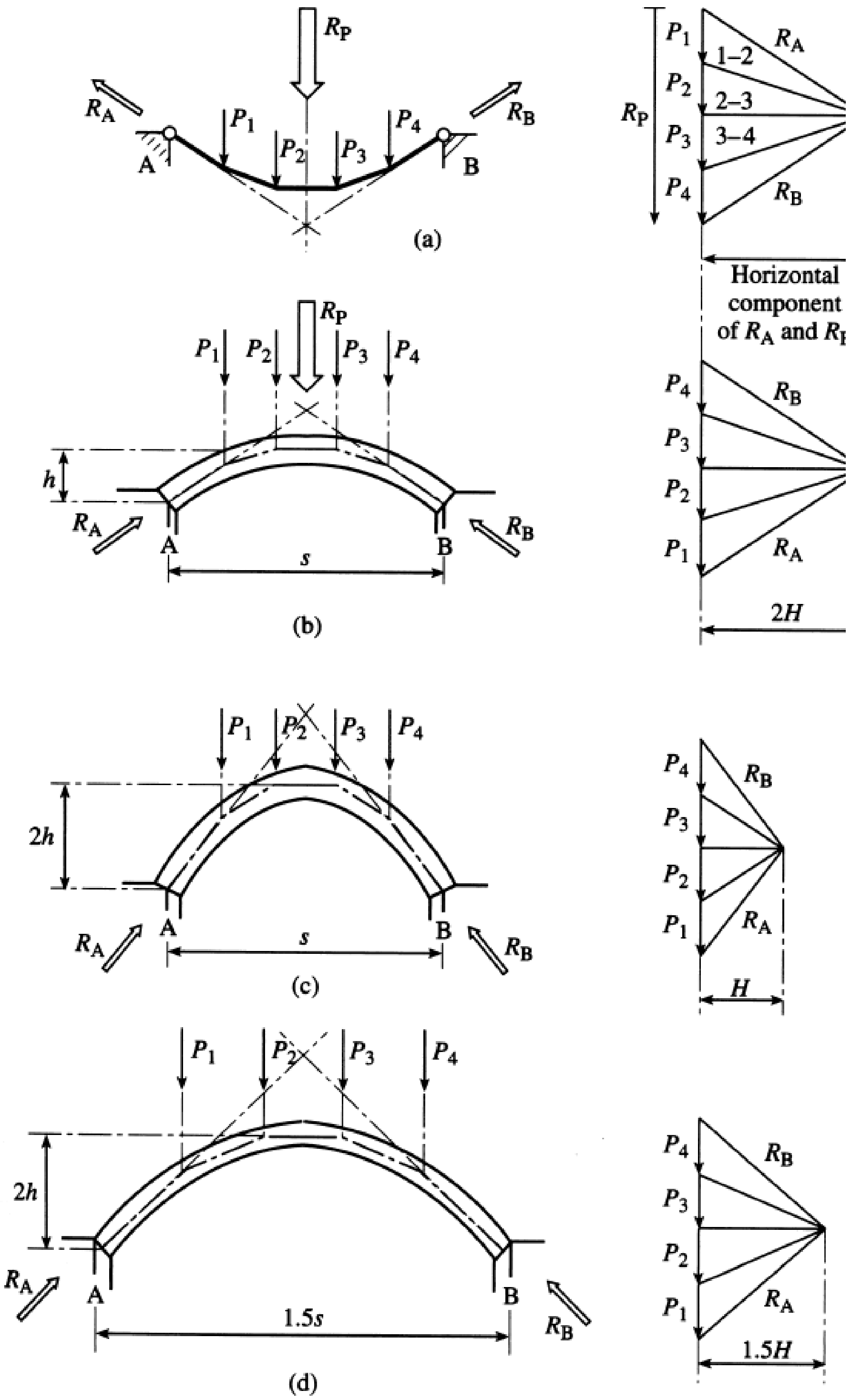


Figure 5.2.4.1 *String polygon and arch thrust-lines.*

engineer, concerned to prevent cracks developing in his design, would use piles to limit foundation movement. The high stresses under the medieval structure ensure that consolidation of the soil and resultant settlement will inevitably occur. If the settlement can be tolerated, however, the stresses are not so high as finally to distress a stiffish clay.

The ‘soil-mechanics’ timescale for consolidation of a tower in relation to the surrounding portions of a structure is one generation. There are numerous records of towers collapsing within 20 years of their completion, for example the first central tower of Winchester Cathedral. These collapses resulted from uneven settlements, not necessarily grossly upsetting the geometry of the tower itself, but throwing extremely high loads on abutting masonry. In some cases this masonry has been reinforced to restore stability, sometimes with concealed internal raking buttresses, occasionally spectacularly, as with the strainer arches of *c.* 1338 at Wells Cathedral. Once the risk period of a generation is past, and the tower has survived either untouched or with reinforcement—and provided there are no changes in the general soil condition, such as would be caused by alterations in the level of the water table (which may have led to the collapses at the cathedrals of Ely in 1322 and Chichester in 1861)—the tower may well be deemed to be structurally safe.

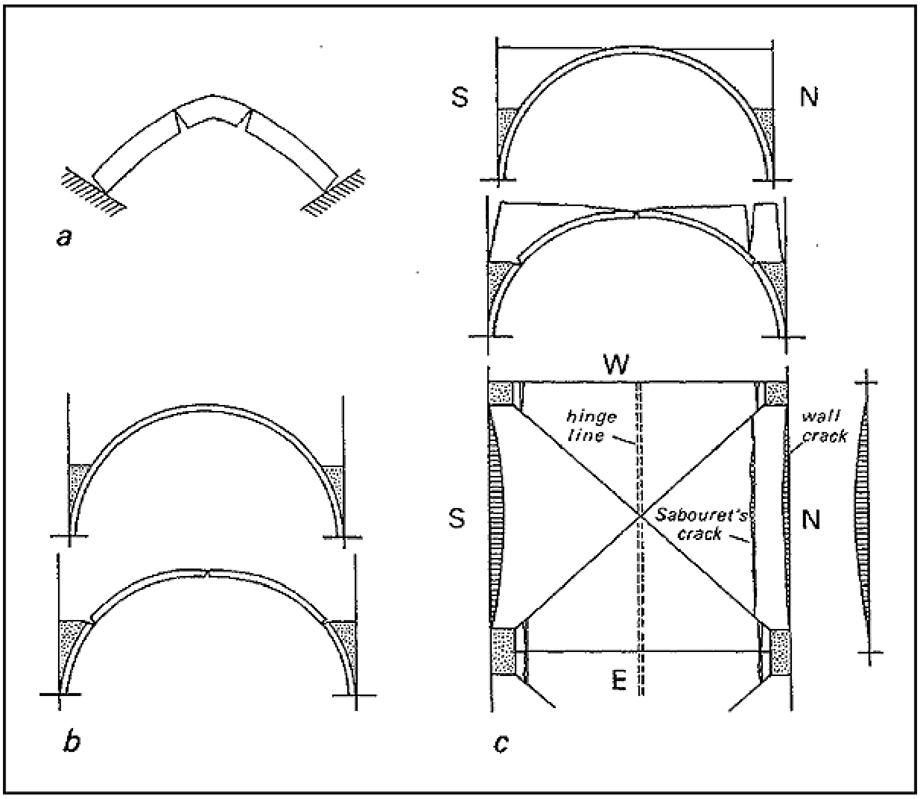


Figure 5.2.4.2 *Crack development: (a) pointed arch; (b) barrel vault; (c) quadripartite vault*

Insufficient confinement of the horizontal forces transmitted by vaults to walls is one of the main causes of the failure of the vaulted structures. Material deterioration and cracks in the vertical walls caused by local soil subside or excessive or localised loads are the other resons. The problem of insufficient horizontal reactions is the most important and its resolution is fyndamental to avoid other static problems.

The reactions should be introduced by specific structural elements such as ties, buttress or perimeter walls. Ties should be places at the arch springing lineto directly support the horizontal forces, but frequently they are not present. In some cases they can be substituted by extrados ties.This solution which has been used already in the past centurieswith steel or wooden ties were adopted more frequently since 14th century.

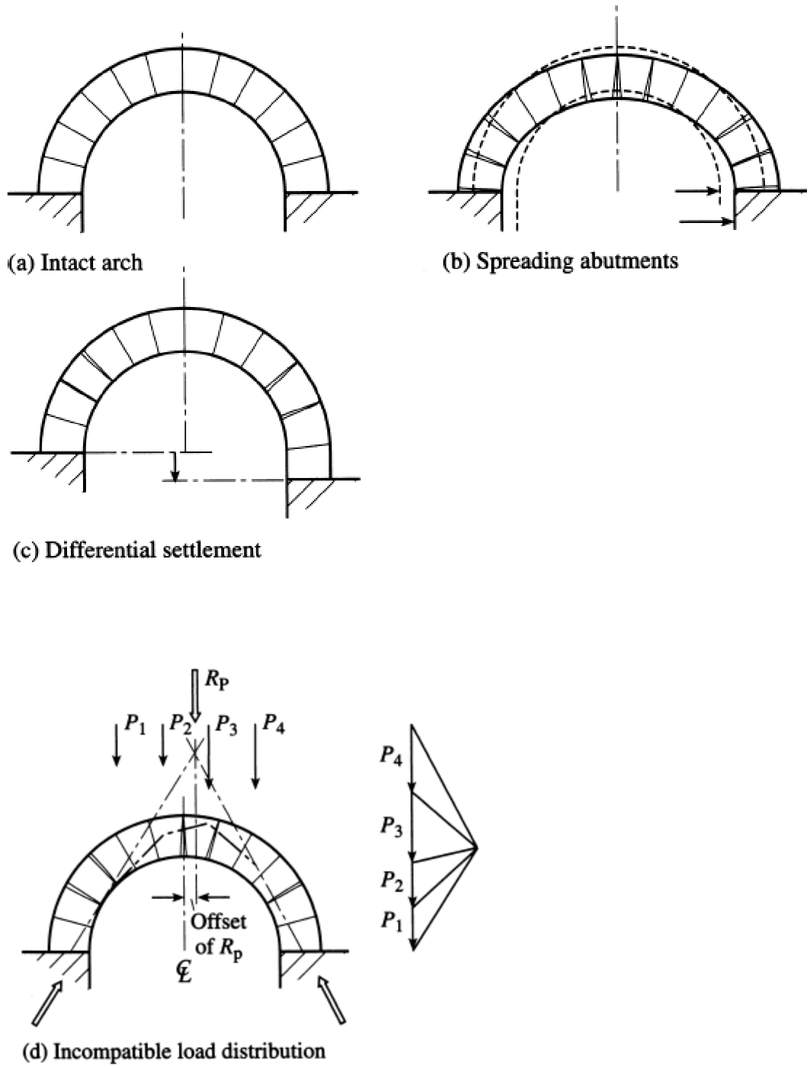


Figure 5.2.4.3 *Diagnoses of causes of arch cracking.*

All these aspects have been dealt with in great depth by Professor Jacques Heyman in his books *The Masonry Arch* (1982) and *The Stone Skeleton* (1995), in which he quotes Coulomb to the effect that, if a thrust-line can be constructed that nowhere is closer to the extrados or the intrados than one-tenth of the thickness of the arch (or vault) the arch will be structurally stable. The edge distance of onetenth of the thickness was originally stated for Gothic stone masonry arches, where the strength of the stone is hardly ever critical; for heavily loaded arches, or arches built of soft stone or bricks, the critical edge distance may have to be increased to, say, one-seventh or one-sixth, to avoid compression failure of the materials.

All the above assumes that the masonry spandrel panels, and/or the vaults, or any other element, carried by the arch, have little or no stiffness, nor strength, in the plane of the arch and simply acts as a load. If, however, the arch merely forms the top of an opening in a large expanse of solid masonry wall, constructed of large blocks of stone with thin joints of strong mortar, a different mechanism, first identified by Sir Christopher Wren, can develop.

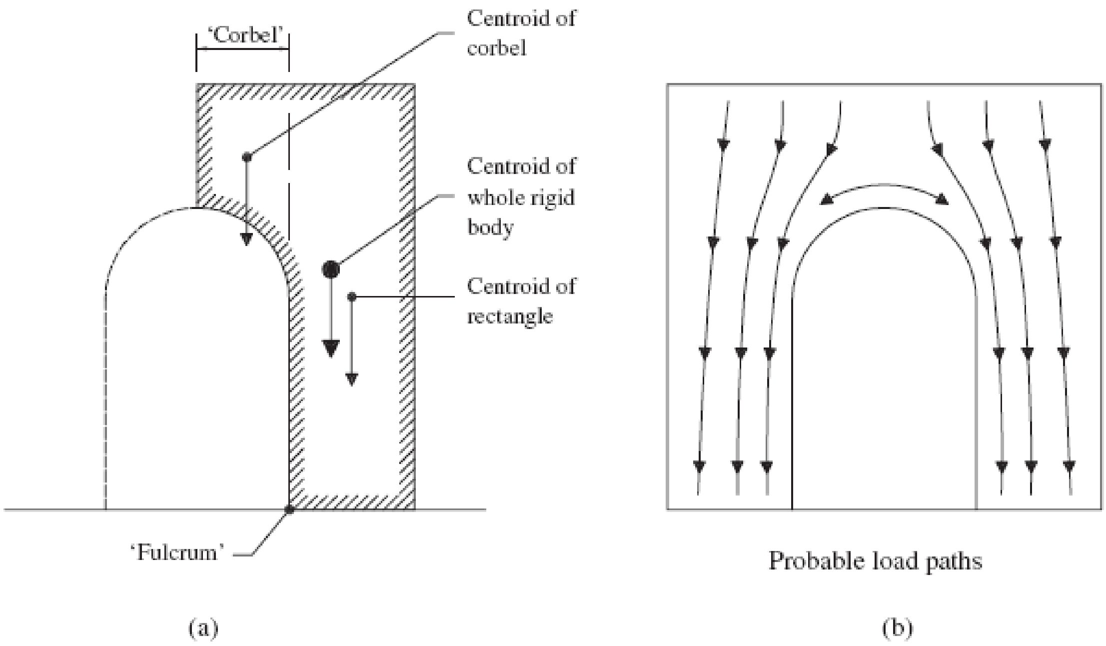
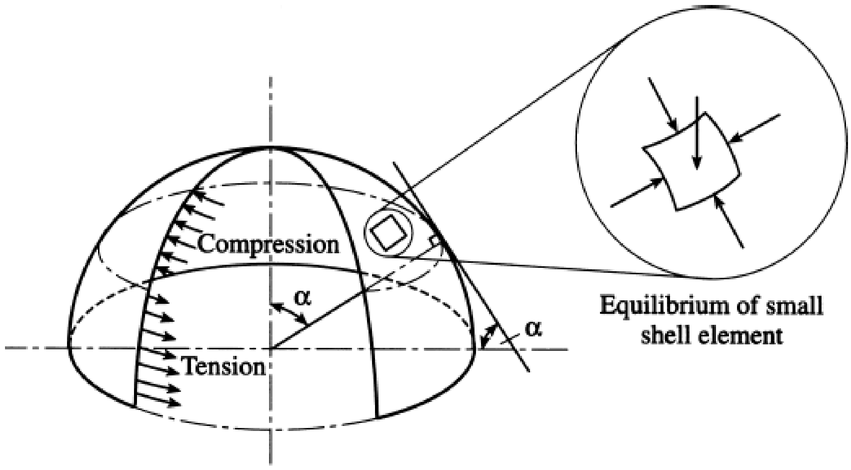


***Figure 5.2.4.4 (a**) Wren's thrust-free, rigid body arch action; **(b)** the likely resulting load paths. This mechanism involves some, largely horizontal, tensile forces in the virtual corbels over the arch and the reveals of the opening. In masonry, of the type described above, this would not be a problem, as the shear strength under the preload from the masonry above would be adequate to transfer the horizontal force at a butt joint in one course to the stones in the adjacent ones.*

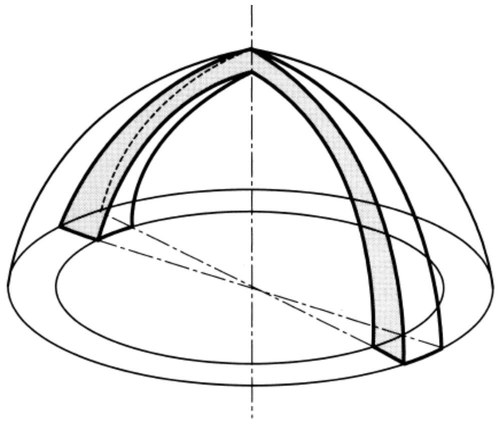
Structural Aspects of Domes

To describe the complex structural behaviour of domes , there are two simplfying assumptions two simplifying assumptions. Each of them explains a part of the overall behaviour and together they account for most of the structural strength. The first assumption is that the dome is a 'shell', with a thickness that is very small relative to its other dimensions. If the basic principles of equilibrium are applied to a small element of such a shell together with the condition that the shell has no bending strength (due to its small thickness), a set of differential equations is obtained. These equations look very off-putting to a person with only elementary mathematics, but if the geometry of the shell is known, they can be solved and will show how the shell, as a whole, can carry its load.

When such calculations are carried out, it will however be found that whilst there is compression along the whole of the meridians (the curves following the intersections between the dome surface and vertical planes through its apex), the circumferential stresses (along the circles formed by the intersections of horizontal planes with the dome) change from compression to tension when the slope from horizontal of the surface of the dome reaches a certain value that depends on the shape of the dome and the distribution of the load (see Fig. 5.2.4.4 *a*).



(a) Membrane-shell dome, showing ring compression and tension



(b) 'Orange-segment' arch—Behaviour of dome

Figure 5.2.4.4 *Membrane and arch rib action of domes.*

For a part-spherical dome with uniform load per unit area of its surface (such as selfweight), the angle is 51.8°. If a spherical dome has to act as a membrane shell, it therefore has to have a certain tensile strength in the circumferential direction unless it is rather shallow. It is however possible to shape a dome, carrying only its self-weight, so that no tension arises. Robert Hooke (cf. Sec. 1.2.1), who assisted Wren at St Paul's Cathedral, arrived (by means of experiments with weighted string polygons?) at the ideal shape which he called a 'cubico-parabolic conoid'. The inner dome of St Paul's spans a diameter of some 32 m and is only two bricks (45 cm) thick. It shows no structural cracks, despite the explosion of a Second World War bomb in the North Transept and it has no iron ties or cramps in it. A drawing from the eighteenth century (copied from Wren's original?) shows the actual shape as consisting of three spherical 'bands' with different radii, joining tangentially and 'flattening' towards the top. The absence of structural cracking indicates that the actual shape of the dome is very close to the ideal theoretical shape, as calculated by Professor Jacques Heyman.

The second, simplifying, assumption abandons the concept of the thin shell and represents the dome by a series of arch ribs. Each arch rib consists of two opposed 'orange segments' formed by cutting the dome (with its real thickness) by two vertical planes intersecting at the apex (see Fig. 5.2.4.4*b*). If it is possible to construct a thrust-line for the forces acting on one such arch, such that it does not get closer to either intrados or extrados of the arch than the value appropriate to the material (one-tenth of the thickness for stone and, say, one-sixth for brick), then each arch is stable and strong enough. If each arch is stable and strong enough, then clearly the dome is stable and strong enough, even though it may show cracks along the meridians.

In reality, dome behaviour will usually be a mixture of the two simplified mechanisms, postulated above: where the slope is less than the one at which the ring force changes from compression to tension, membrane action is likely to dominate (thus overcoming the objection to the orange-segment model, that it produces infinite stresses at the apex). Lower down the orange-segment behaviour will take over, unless the masonry has been reinforced with metal tension rings (and even these will stretch before developing their force).

Radial cracks in the lower part of a dome are therefore not necessarily a symptom of drastic structural shortcoming, however unsightly they may appear on the decorations inside, but if they continue to widen, it is a sign of inadequate horizontal restraint at the base of the dome. This is because, like arches and vaults, domes require horizontal restraint at their base; this is irrespective of their structural action being predominantly 'orange-segment arch' or 'thin shell membrane'. Like arches and vaults, domes react to uneven settlement by cracking, but the crack patterns are likely to be more complex.

Repair and Strengthening Techniques

Structural distress in arched structures will usually manifest itself by cracking or opening of joints. Before any structural intervention is considered, it is however essential to ascertain whether the cracking is progressing or whether it is static and, if so, if the cause is unlikelyto recur, in which case only 'cosmetic' repairs are required.

The continuing, increasing, leaning of walls and piers, subject to thrust from arches or vaults, may have been caused when the original builders misjudged the foundation conditions or got the geometry of the structure wrong. Alternatively it may be due to later generations having interfered with the structure or with the surrounding ground. The result is usually a slow spreading of the arches or vaults with correspondingly increasing tilt of the external walls or piers.

In the case of spreading arches and vaults buttressing is one of the possible technique; an alternative to buttressing is the provision of ties, which prevent the springings of the arches from moving apart. Physical ties at springing level are usually architecturally undesirable and it is only on rare occasions that one can transfer the restraining force to the springings from a tie located at an invisible or visually unimportant level.

Professor Schultze of Aachen University has provided an example in which tie rods, inserted to prevent the spreading of the abutments of a Gothic vault, had their mid-points raised above the crown of the vault by means of a steel roof truss, so as to make them invisible from below the vault.

There has been a number of instances when arches and vaults have been ‘strengthened’ by being suspended by grouted-in hanger bars from a concrete structure built above (and in many cases right on top of) the original masonry structure. The attraction of such an arrangement is that the concrete relieving structure can be designed and built to conform today’s codes of practice, thus circumventing the ambiguities in the original construction, e.g. if sufficient buttressing is not available, the relieving structure can be made to act as a beam.

Such interventions preserve the appearance of the underside of the arch or vault, but change the function from structural to merely decorative. They should therefore be considered as a last resort, particularly when they involve casting or spraying the concrete directly on to the masonry, as in that case the intervention becomes irreversible. If provision of such an overlay structure, after careful consideration, is found to be the only solution, a separation membrane e.g. polyethylene sheeting should be used.

The shape of horizontal sections through domes, whether polygonal or circular, can make it relatively easy to provide additional circumferential strength and/or horizontal base restraint; removal of the roof cladding and a small thickness of the masonry will enable the insertion of a tension ring. As a single steel rod or cable may cause too high bearing stresses and hence tend to ‘cut’ into the masonry (especially if this is of soft brick), a number of smaller bars or cables should be used together with spreaders along meridians. Such hardware must be adequately protected against ingress of rainwater, unless stainless steel or nonferrous alloys are used. If space permits, reinforced concrete ring beams may be used, preferably with a separation membrane to facilitate a future renewal or removal, so that reversibility of the intervention is made easier.

No attempt should be made to close up cracks by tensioning the ring ties. For one thing, dislodged particles of mortar, etc. will have wedged themselves into the cracks so as to resist closure and, secondly, where cracking is caused by insufficient buttressing of the base ring, the dome will have modified its original shape and have spread so as to 'follow' the receding buttresses. Closing the cracks, would then 'shrink' the dome and cause separation cracks at the buttresses. A moderate amount of pre-tensioning may, however, be beneficial by enabling the ring ties to restrain further movement without significant extension, as such extension would be accompanied by further cracking.

The system of extrados ties can successfully contain only if high vertical actions are present in the supporting walls, as it generally happens in historical buildings with vaults on the ground floor and two or three floors above. In case of insufficient vertical action, like in low buildings or in upper floor vaulted buildings, the equilibrium condition requires vertical ties in the walls. These can be placed in drilled holes inside the walls or externally, on surfaces.

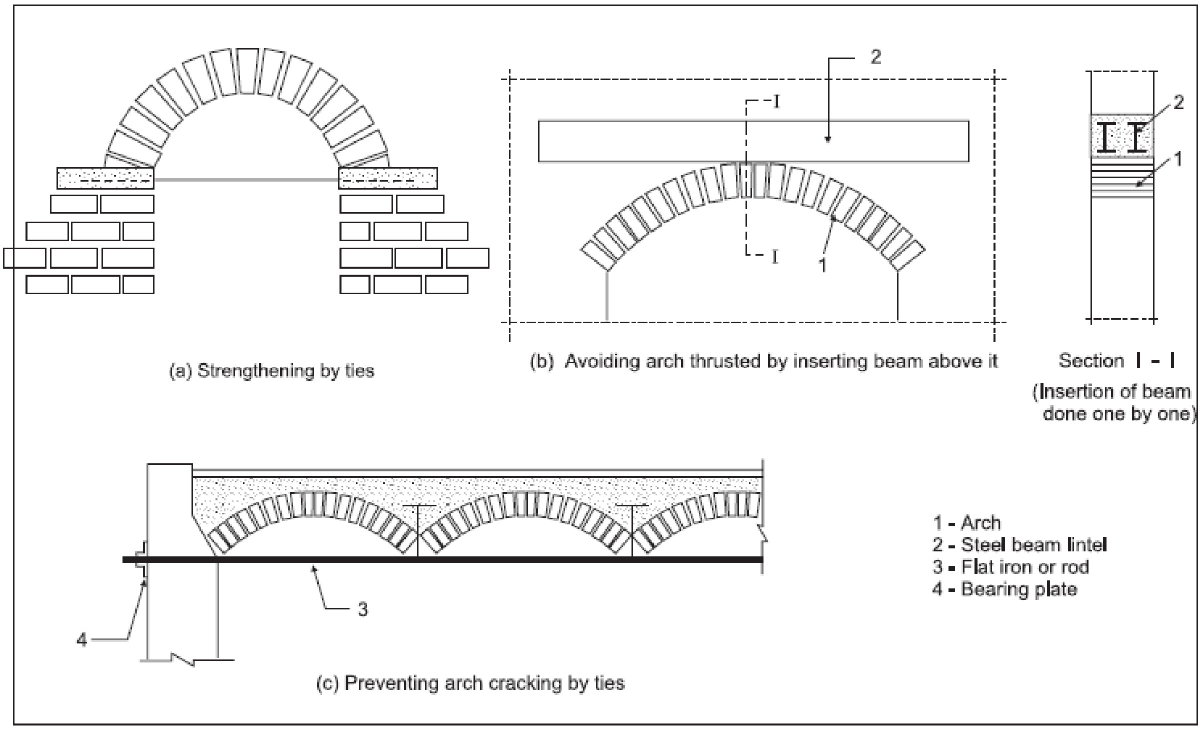


Figure 5.2.4.5 *Strengthening an arched opening in masonry wall*

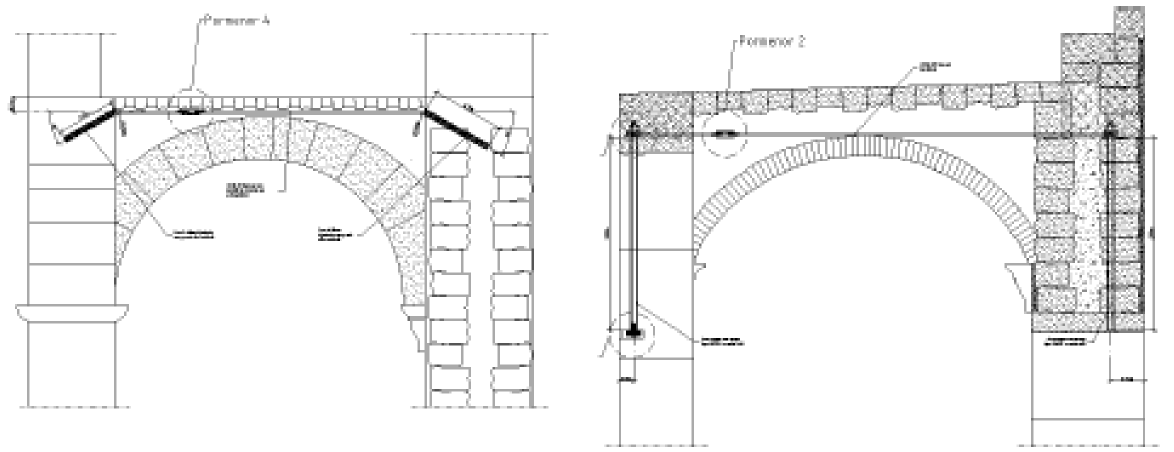


Figure 5.2.4.6 *Salzedas' cloister (a) 1st level tying; (b) 2nd level tying and stitching*

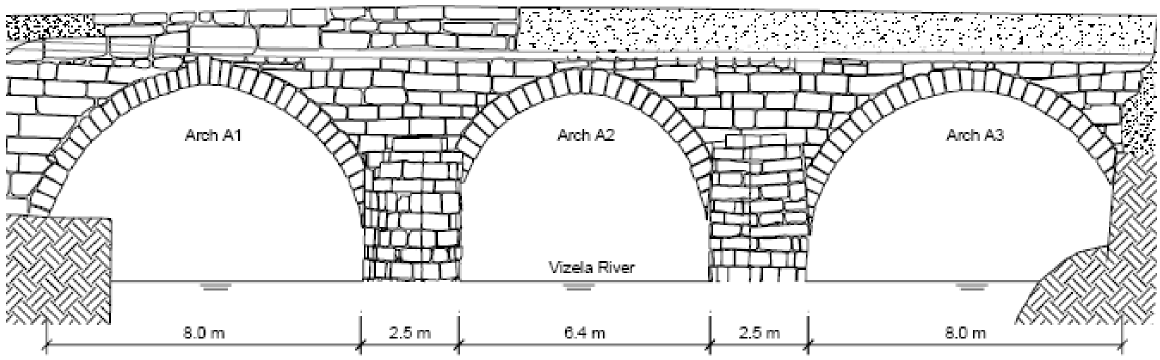


Figure 5.2.4.7 *Negrelos Bridge (upstream view), Guimarães*

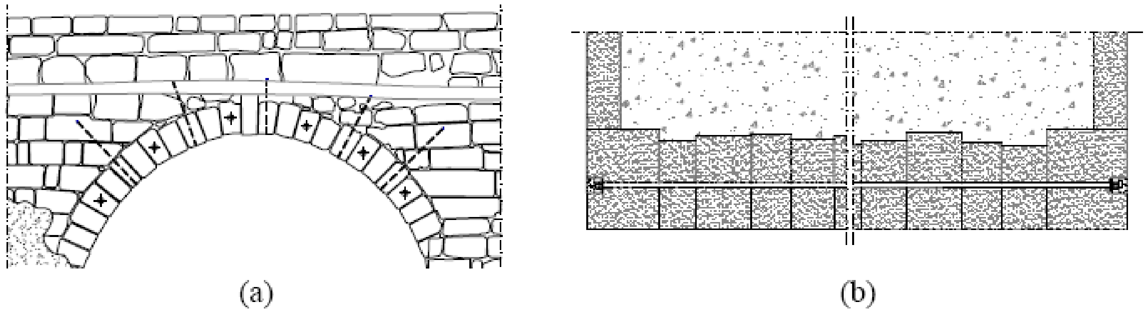


Figure 5.2.4.8 *Negrelos Bridge, Guimarães Strengthening of the central arch: (a) adopted anchor scheme; (b) full bridge width anchors.*

To prevent any additional increase of the longitudinal cracking in the intrados of the central arch as well as to assure its future stability, a set of four horizontal stainless steel anchors across the full bridge width, endowed with cylindrical steel anchorage plates at each side of the arch, were proposed, see Figure 5.2.4.8 above. Also, two additional shorter stainless steel anchors were used close to each springing. In addition, it was recommended a light injection of the arch, at the intrados. For the connection between the arch and the spandrel walls a similar solution was developed. Five stitching anchors in each side of the arch were used with the purpose of linking the spandrel walls to the external arch voussoirs, see Figure 5.2.4.8a. In order to face the out-of-plumbness of the spandrel walls above the left arch, it was decided to use two horizontal stainless steel anchors across the full bridge width provided with cross-shaped anchorage plates at the extremes. The shape of the plates was due to aesthetic reasons.

To repair the high level of damage found in the downstream cutwaters, with some stones cracked and others out of their original place and disconnected from the piers, the dismantling and subsequent rebuilding of the most deteriorated areas was proposed. On the other hand, the upstream cutwaters, less damaged but also in a poor condition, are to be injected with a lime-based grout after conclusion of the joint repointing works. All infesting vegetation is to be removed using the most adequate procedures, and all masonry joints that show degradation are to be carefully cleaned and repointed. In order to prevent the fines from being washed out of the fill material, leading to voids and thus affecting the carrying capacity of the bridge, it was recommended the execution of an adequate waterproofing and drainage of the pavement.

5.2.4 Bed joint Reinforcement Technique

General

The mechanical properties and the behaviour of masonry depend strongly on the orientation of the bed joints towards the applied load and the stress state of the joints.

The bed joint reinforcement technique is based on the insertion of steel bars in the mortar bed joints previously excavated for few centimeters and then refilled by a repointing material. It is particularly applicable to facing masonry having regular courses (e.g., brick or stone masonry rather than rubble stone walls), where it can be appropriate for several structural conditions.

High compression loads, often characterizing massive brick masonry structures such as towers, curtain walls and heavily loaded pillars, can lead to critical conditions due to the activation ofcreep (Fig. 5.2.5.1) .Typical damage—vertical and sub-vertical; thin, but very diffuse cracks—is worsened by cyclic stresses (due to thermal and hygroscopic strains) or low dynamic forces (windaction or bell ringing), and can affect bricks even in a large portion of the wall thickness [26]. This type of damage, generally disregarded in such structures, in comparison with other, more evident, critical conditions (large cracks, out-ofplumb, large-scale deformations, etc.), can induce sudden, unexpected brittle collapse, as observed in several cases (e.g., Civic Tower of Pavia, St. Mark's Bell-tower in Venice, the Cathedral of Noto in Sicily; the Bell-tower of St. Magdalen in Goch, Germany) (Figure. 20) [[26] even at stress values 40–60% lower than the strength of the masonry under short-term static loads [29].

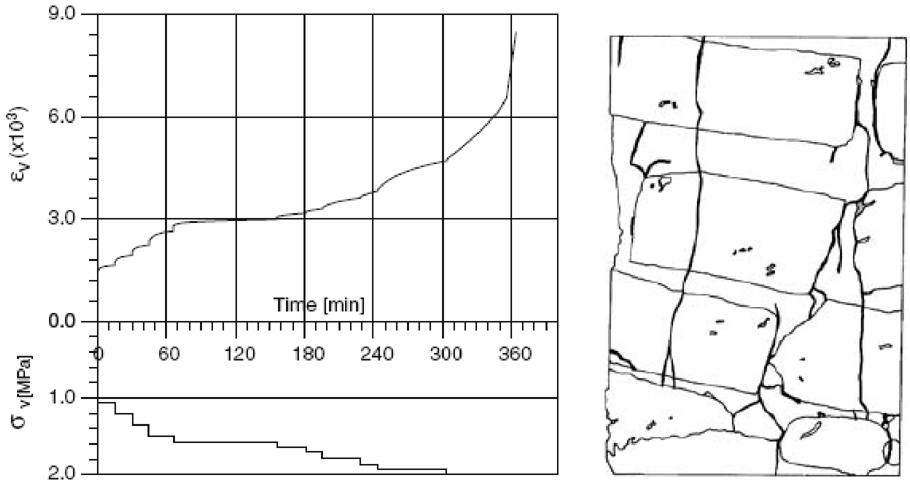


Figure 5.2.5.1 *Typical timestress/ strain behavior of overstressed masonry: phases of primary, secondary and tertiarycreep (corresponding to triggering of uncontrollable collapse), due to slow load steps reproduced in laboratory on masonry prisms (Polytechnic of Milan)[29]*

In the last few years, several experimental surveys performed in collaboration by the University of Padova and the Polytechnic of Milan, have demonstrated that creep damage can effectively be counteracted byminimal reinforcement of the superficial layer of masonry, properly inserted into bed joints. This ''bed joint reinforcement'' technique (later known also as ''structural repointing'') does not require particular skills or tools during application and can be performed quite easily and quickly [31, 32]. It involves removing a few centimeters of the mortar in the bed joint for (5–7), placing the reinforcements (small bars or plates), and filling in the cut with a suitable embedding material.

Early experimental studies dealing with applications of low-diameter (5 or 6 mm) stainless steel reinforced bars (Fig. 5.2.5.1) showed how effective this intervention was in reducing transverse dilation due to high compression stresses, as the bars bear the tensile stresses otherwise directed to the bricks. Therefore, although a significant increment in strength is not detectable, the technique can directly limit crack development, thus improving the performance and safety of the structure.

Applications to both sides of the wall of suitable transverse connections and the use of more compatible mortars (hydraulic lime-based) as embedding material give the best results. A feasibility study, supported by numerical modeling calibrated on the experimental results, also optimized intervention design (amount and distribution ofreinforcement) and at the same time minimized the obtrusiveness of the repair phase (Figs. 5.2.5.7, 5.2.5.8).

Application of Bed Joint Reinforcement Technique

The application of the technique involves many aspects, namely: the preliminary preparation of the joints, the choice of the reinforcement, both in terms of material and of spacing and amount, the type ofrepointing material, and the aesthetic of the wall face. The main operative phases for a correct execution of the intervention are described in the following:

(1) Possible removal of plaster or finishing from the surface, to check the masonry conditions.

(2) Accurate inspection of the masonry; it should be appropriate to inject some large voids or replace some bricks.

(3) Cutting of the bed mortar joints by using suitable (very common) tools; the recesses should be at least 10 mm high and 50–80 mm deep, so that the reinforcement can be inserted and the remaining mortar in the masonry can bear the applied loads.

(4) Removal of powder or rubble by air or water, depending on both the existing and the repointing materials (water can be appropriate to avoid the excessive absorption from the new to the old mortar, whereas the groove should be kept dry in the case of use of special polymeric materials).

(5) Placing of a first layer of embedding material, which should be accurately compacted. Mortars are usually made of hydraulic lime for a better compatibility (chemical, physical and mechanical) with the existent ones, and can contain special additives (e.g. having expansive properties to compensate the shrinkage during the hydration phase).

(6) Placing of the reinforcing material: steel bars or plates (stainless, in general) or FRP laminates can be used. To increase the friction between reinforcement and the existing mortar rough surfaces should be preferred, as when using steel bars may should be also previously sandblasted in order to clean their contact area. The use of more bars with smaller diameter rather then one with higher size should be preferred. However, due to the small thickness of the bed joints (usually around 10–15 mm), only reduced size reinforcements (4–6 mm in diameter) can be inserted. The placing of spacers can be useful to separate the reinforcement from the surface of the bricks.

(7) A second layer of mortar has to be applied over the first bar to cover it sufficiently; a further bar is inserted if necessary, then another layer of mortar to cover it.

(8) A final layer of repointing material should be placed in the last available 15–20 mm, to seal the horizontal joints and for appearance and homogeneity; special sands or pigments can be used to obtain required effects. The proposed technique does not show particular difficulty of application; some care is required during the operative phases (cutting of the bed joint, cleaning, repointing), particularly in the case of historic buildings, but every operation can be performed quite easily and quickly. It is clearly a surface intervention but the combination of this technique with minor reconstruction (where bricks are particularly damaged) and/or injections (especially for multi-wythe

masonry walls with internal core having a high percentage of voids), can improve the global strength of the structure.

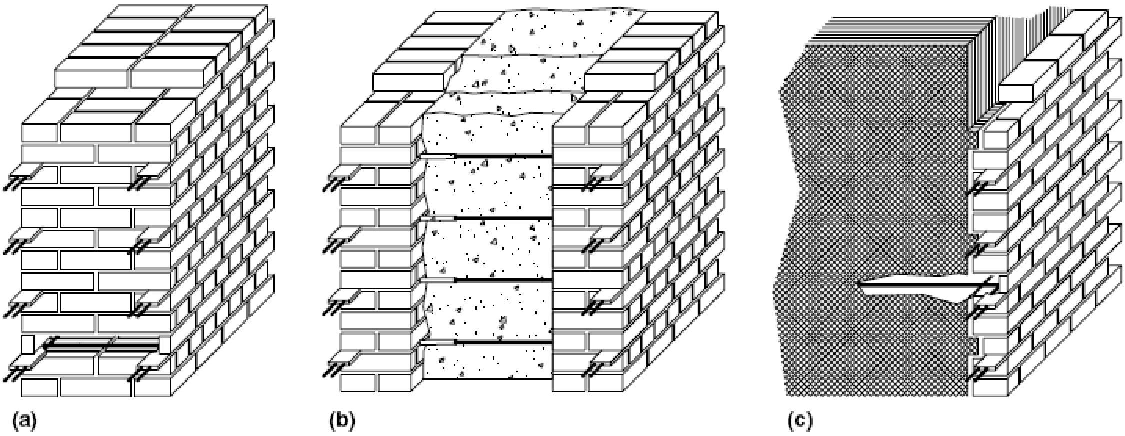


Figure 5.2.5.2 *Example of application of the technique on different types of brick masonry walls: (a) solid bearing wall; (b) multi-leaf wall; (c) multi-leaf wall with external veneer wythe.*

Further advantages can be obtained by the use of thin strips in rather than circular bars. Higher flexibility of the thin strips in case of uneven joints, and better behavior against splitting, allow more superficial, less obtrusive interventions. More research work therefore focused on the potential of applying thin strips and studying their mutual interactions with masonry, at both overall and local levels.

Lastly, correct application of the techniques requires proper solutions of compatible embedding mortars, to avoid undesirable further problems with the original masonry.

Despite the number of researches and efforts devoted to FRP materials, until now, real applications have only been performed with stainless steel bars in combination with lime-based repointing mortars, as they guarantee effectiveness and durability, even in exceptional and not easily predictable conditions.

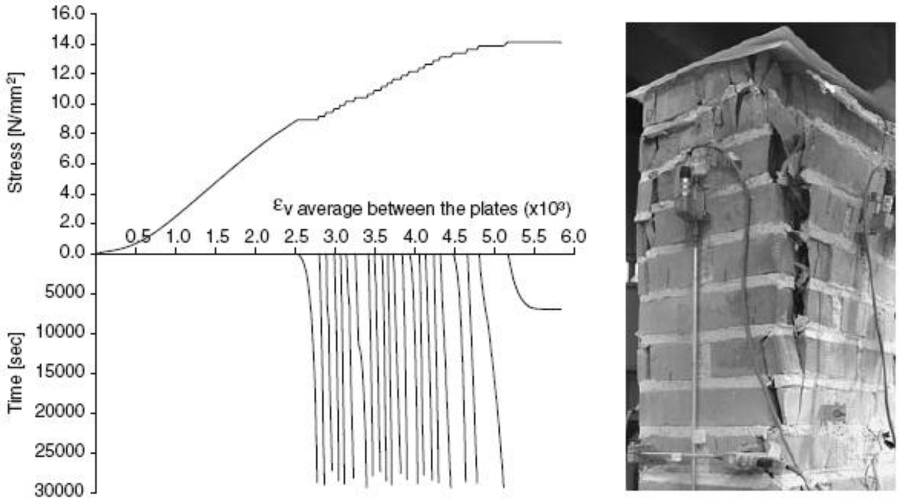


Figure 5.2.5.3 *Creep simulation on masonry prisms strengthened with FRP strips (Polytechnic of Milan) [28]*

Figure 5.2.5.4 *Collapse of Civic Tower of Pavia (1989) (a) and Cathedral of Noto (Siracusa, 1996) (b), Italy*

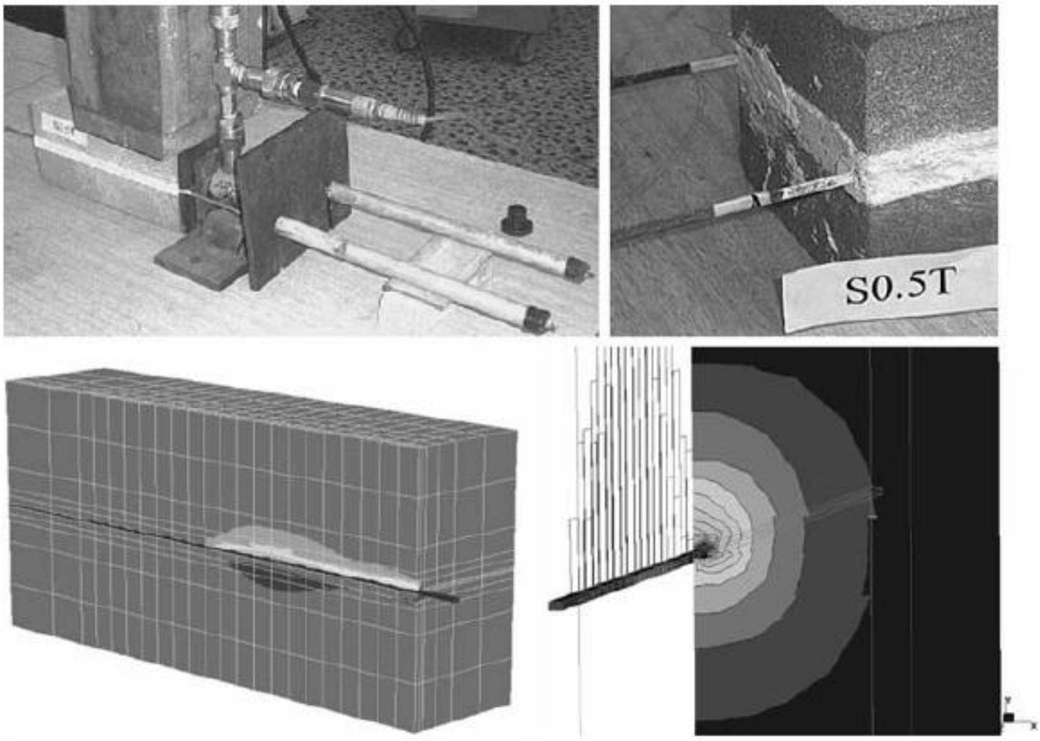


Figure* 5.2.5.5Experimental study of interface behavior of CFRP strips and numerical simulation, showing stress migration along anchoring length and influence of rectangular section in limiting mortar splitting (due to elliptical stress distribution around strip) [27]*

Experimental validation of the performance [26] of the bed reinforcement technique means that it may be proposed for the consolidation of several ancient constructions and monuments at risk , counteracting dilation under high compressive stress by improving material toughness. Note that, in real cases, several types of damage may co-exist, so that proper selection and combinations of various intervention techniques should be considered. In many cases, both injections and partial, limited rebuilding (the tradi-tional ‘‘scuci-cuci’’) are used, to reduce stress concentration and to replace the most severely damaged parts, respectively.

These techniques act locally by improving the mechanical behavior of the material, thus contributing to rehabilitation of the proper loadbearing capacity of the structure.

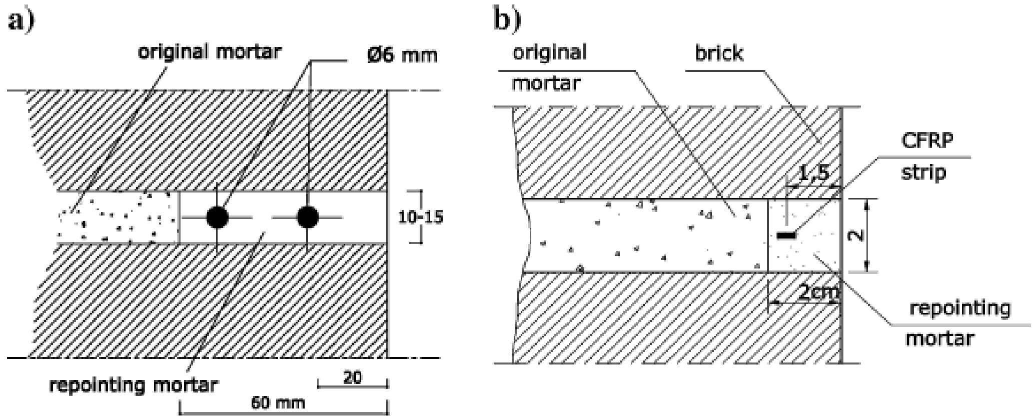


Figure 5.2.5.6 *Bed reinforcement technique with steel bars (a) and thin FRP strips (b)*

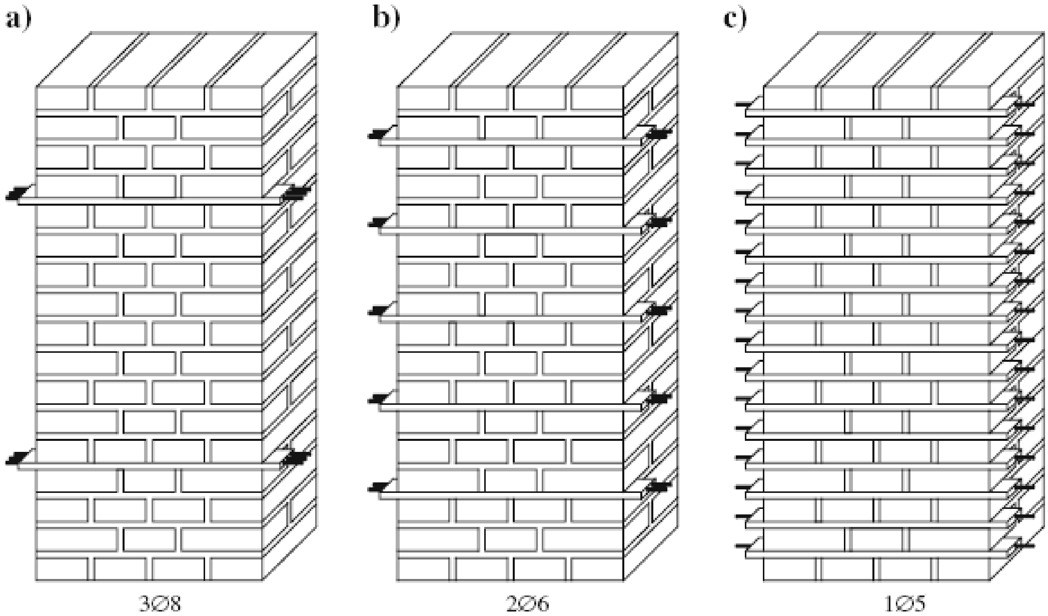
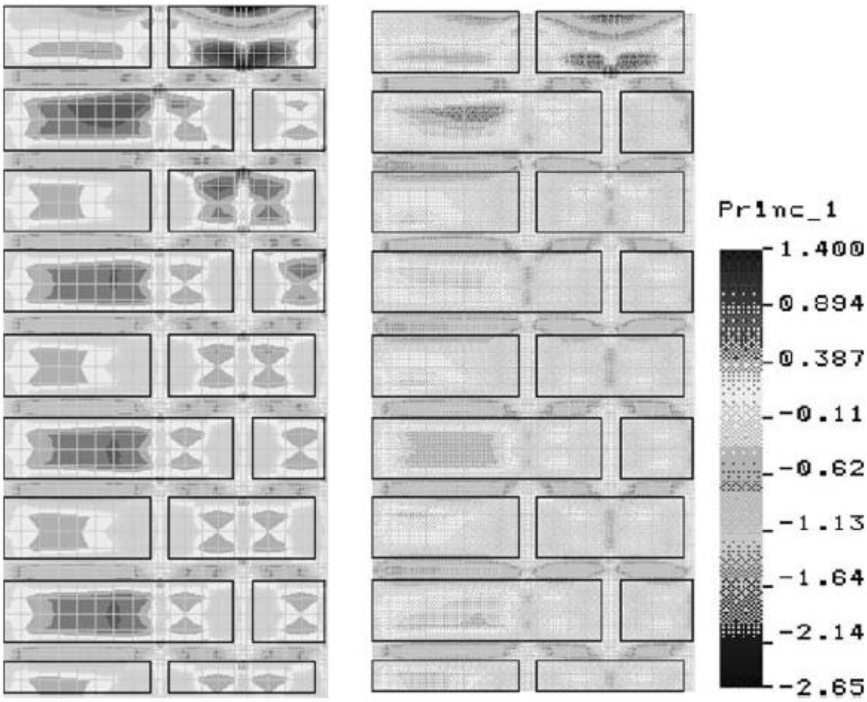


Figure 5.2.5.7 *Optimization of **a) b) c)** bed joint reinforcement in a brick masonry panel (same amount of steel is distributed): numerical analysis shows reduction of tensile stress in bricks of 20%, 40% and 50% in cases (a–c), respectively. Positioning of two bars every three joints optimizes structural performance with minimal obtrusiveness*



Figur 5.2.5.8 *Example of panels strengthened with various types of reinforcement: (a) steel bars; (b) circular CFRP bars (anchoring in short sides with metal sleeves); (c) thin CFRP strips (no overlap at corner, unless specially designed elements are available)*

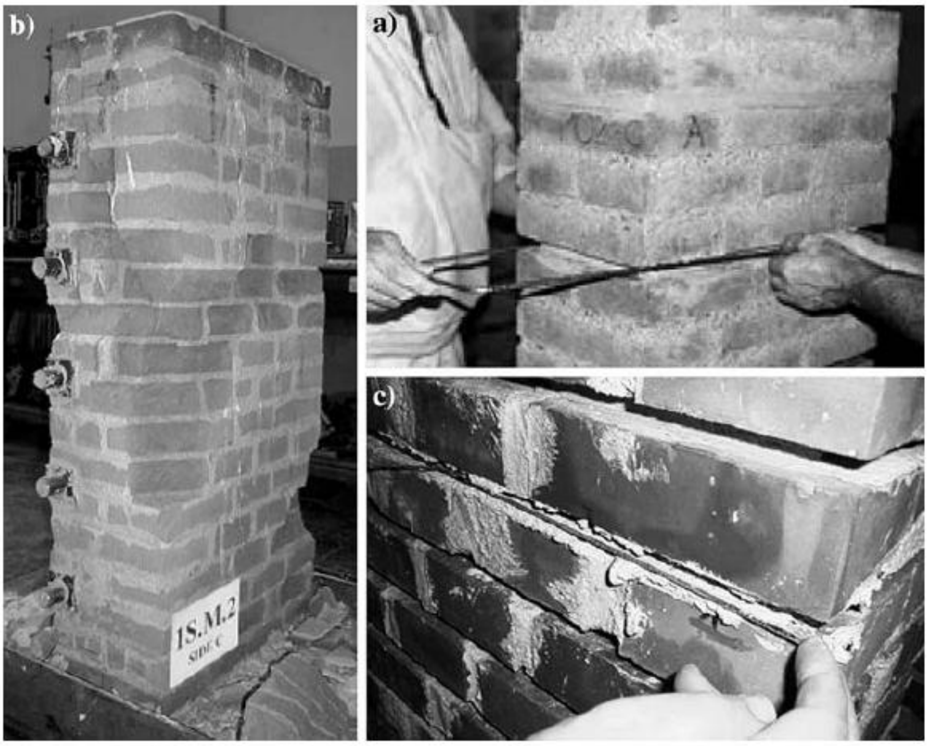


Figure 5.2.5.9 *Example of panels strengthened with various types of reinforcement: (a) steel bars; (b) circular CFRP bars (anchoring in short sides with metal sleeves); (c) thin CFRP strips (no overlap at corner, unless specially designed elements are available)*

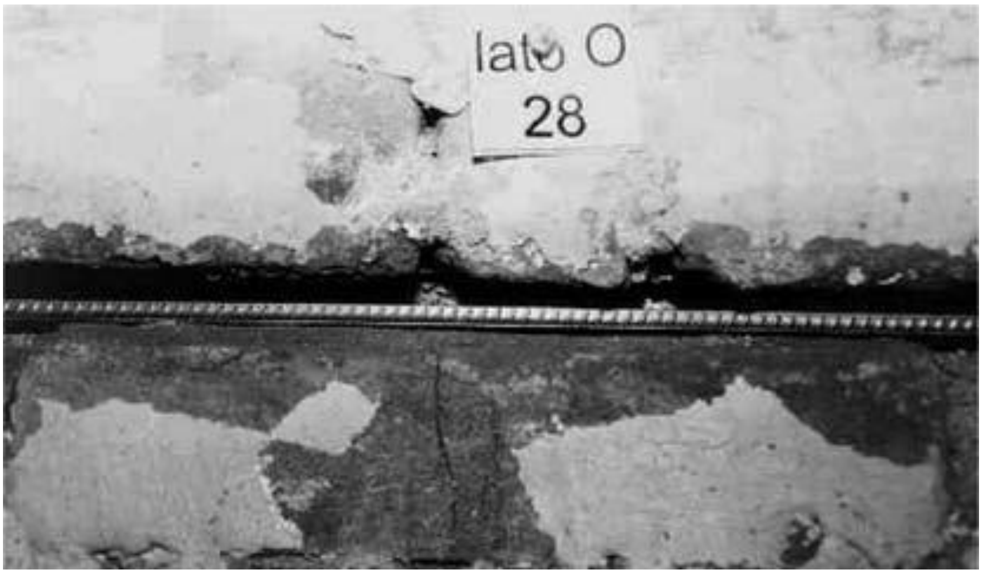


Figure 5.2.5.10 *On-site insertion of reinforced bars into joints*

Figure 5.2.5.11 and 5.12 show the intervention works carried out upon the determination of hazardous cond,t,ons found in some of the pillars of the church of S. Sofia, Padova, Italy [26]. Several cracks were detected, and steel reinforcement rings, together with injections and local rebuilding, were provisionally placed in some portions of the structure. All the sides of the pillars have been strengthened with the bed joint reinforce-

Figure 5.2.5.11 *Church of S. Sofia in Padova: provisional measures on cracked pillars*

__Figure 5.2.5.12__ Phases of excavation, positioning, pinning and repointing of a damaged pillar (reinforcing rings were then removed)

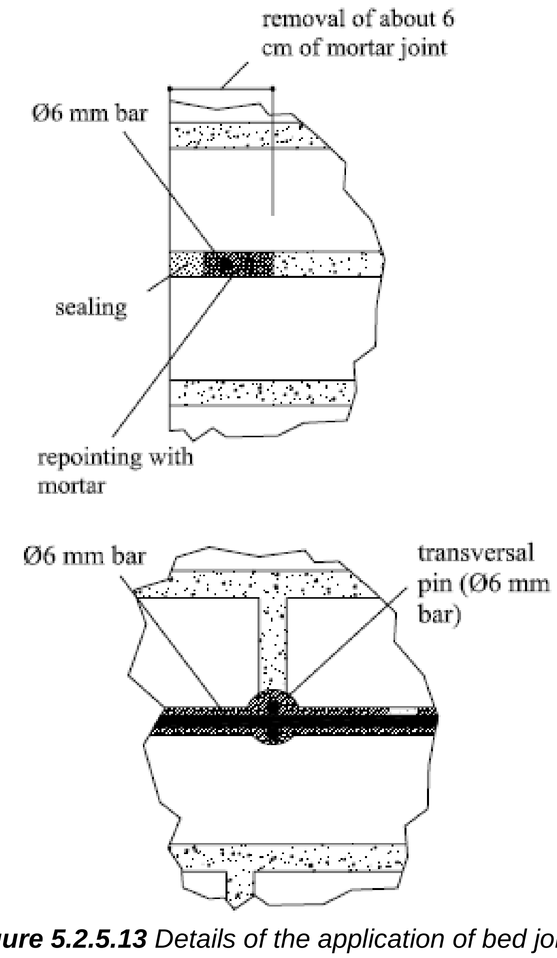


__Figure 5.2.5.13__ Details of the application of bed joint reinforcement

5.2.6 Seismic Retrofitting of Unreinforced Masonry with Metallic or Different Material Tie-Rods

Many experiences in restoration and rehabilitation of damaged masonry buildings have been carried out in Italy during more than fifty years; in many cases the interventions were performed after a long period of misuse and lack of maintenance following the second world war. During this period also major earthquakes have taken place in most of the Mediterranean countries [40] . Unreinforced masonry buildings are characterized by their vulnerability to the seismic action being . Especially in historic centers, having subjected to non homogeneous constructive phases due to extensions occurred in the past by merging parts or superimposing new ones the inadequate connection of parts through excessively slender elements can result in overturning mechanisms when subjected to seismic lateral loading. The study of the building rules and structural details, such as the wall sections, has a great importance in the mechanic behaviour of a building. As an example, historic stonemasonry buildings present a peculiar structural and durability behaviour which is different from the one of the brick-masonry buildings [Giuffrè, 1991], [Giuffrè, 1993].

Figure 5.2.6.1 *Damages after Umbria earthquake to a building repaired by introduction of concrete tie beams.*

The severe damages suffered by URM buildings which were repaired and strengthened after recent earthquakes in Italy, have proved the inadequacy of the retrofitting strategies and techniques adopted in the last thirty years. Examples of inadequate interventions have been given by the introduction of additional reinforced concrete elements in the form of rigid diaphragm, r.c. ring beams, columns inserted within the depth of the walls without specific attention to the effects of an increased level of mass and stiffness. Unexpected local or global failure mechanisms due to the enforced deformation incompatibilities with the original structures could result. These issues opened a wide cultural debate, exacerbated by a strong public opinion reaction after the Molise Earthquake event in 2002, where the partial collapse of a primary public school caused the decease of twenty seven children. As a result, a new seismic hazard zonation was approved along with the development of a new seismic code (OPCM 3431/05), issued by the Italian Civil Protection Department In particular, a special Annex 11E has been included specifically focussed to the description of alternative feasible retrofit and strengthening techniques for Unreinforced Masonry Buildings. The different proposed solutions have been subdivided based on their targeted performance objective, while highlighting limits and positive features of each solution also depending on the masonry typology.

Retrofitting Interventions

The performance of a building subject to an earthquake motions is governed by the inter-connectivity between structural components as well as the individual component's strength, stiffness and ductility.

When separate components of the structural system are strengthened care should be taken to ensure uniform distribution of lateral stiffness in both directions in plan. When constructing additional structural elements, for example new RC columns or RC shear wall between columns these should also allow for uniform distribution of stiffness in plan and in elevation. Rules for horizontal and vertical regularity should be followed when strengthening the original structure. Below are listed rules that should be followed to achieve satisfactory repair:

- The structural walls should be uniformly distributed in both directions in plan.
- The number of walls and their cross section dimensions are based on the seismic resistance design. The structural walls should be bonded together and tied to ensure common action Floors should be capable of rigid diaphragm action.
- Floors should be connected to structural walls by means of steel ties to provide restraint at top of walls against out-of-plane vibration
- The foundations must be strengthened as well to ensure adequate load transfer to the soils.
- In cases where RC shear walls are being added to the structure their foundations should be carefully detailed and constructed

The aims of the intervention to retrofit a building therefore are:

a) to favour a monolithic three-dimensional behaviour of the structure, against the horizontal seismic loads, by means of wall-to-wall and floor-to-wall connections,

b) to contain the lateral loads from vaults and roof.

The three-dimensional behaviour represents a minimum requirement in order to apply analytical methods for the seismic analysis of the whole building. As a matter of fact these methods assume the wall behaviour is dominated by in plane action, being the out-of-plane mechanisms constrained. To reach these goals viable solutions proposed are:

1) metallic or different material tie rods;

2) external circumferential tie-rods;

3) local connections;

4) ring beams.

A brief description of advantages and limits of each solution, along with implementation issues, is given below.

Metallic or different material tie-rods

*M*etallic tie-rods should be located at the floor level along the two principal directions of the structure (Figure 5.2.6.2a), and anchored by means of rosehead washers bearing plates. They favour the development of a three-dimensional behaviour of the structure providing a valuable connection between orthogonal walls and supplying restraints against the out of plan mechanisms of the walls. In addition they can significantly improve the in plane behaviour of walls with openings, by developing a strut and tie mechanism in the spandrels below and above the opening. For an efficient implementation, tie-rods must have a suitable stiffness (big diameter and limited length bars being preferred); the pre-stressed force should be controlled within appropriate values in order to avoid local damages. When adopting rosehead washers bearing anchorage, inclined bolts (Figure 5.2.6.2b) should be preferred to actual plates as they involve a wider portion of the masonry wall.

Anchorage plates (Figure 5.2.6.2c) have to be preferred when dealing with low-quality masonry (i.e. stonemasonry made of little elements) which need to be locally reinforced. It is not recommended to locate the anchorage into the wall section, particularly dealing whit disconnected rubble filled stone masonry walls.

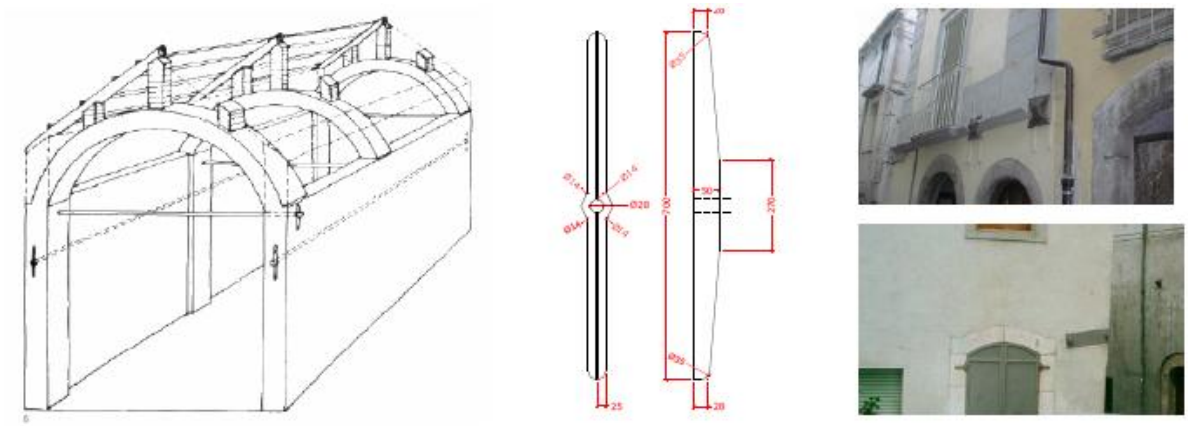


Figure 5.2.6.2 *Improvement of structural connection: a) Metallic tie-rods along the two main direction of the structure, b) details of rosehead washer bearing bolt, c) examples of rosehead washer bearing bearing plate*

External circumferential tie-rods

External circumferential tie-rods can be made of metallic elements (Figure 5.2.6.2 a), FRP (fibre reinforced polymers) or other composites (Figure 5.2.6.2b). They further enhance the three-dimensional behaviour of the structure providing a effective connection between orthogonal walls. This retrofit intervention is effective for buildings with limited wall length. Concentration of tensile stressed in the building corners should be avoided by means of additional repartition plates or by smoothing the corners when FRP strips are adopted.

Figure 5.2.6.3 *External circumferential tie-rods: a) metallic elements, b)FRP or other composites (Blasi et al. 1999)*

Local connections: disconnecting and reconnecting adjacent units, installation of steel ties, installation of metallic or other material transversal connections.

Alternative technique to improve local connections in stone or masonry buildings can be achieved by disconnecting and reconnecting adjacent units or elements (Figure 5.2.6.4a), by the installation of steel ties (Figure 5.2.6.4b) or by the installation into drilled holes of transversal elements made of conglomerate, or reinforced with metallic or FRP bars (Figure 5.2.6.4 c).

Local connections aim to improve wall to wall connection when existing wall joints are deteriorated either due to seismic or non seismic-related damages or are intrinsically of poor quality. However, they are effective in enhancing the building three-dimensional behaviour only when dealing with good quality masonry. For poor quality masonry, a meaningful improvement of the global behaviour is not guaranteed by this solution and the adoption of ties has to be preferred.The use of deep drilling and reinforcing has to be considered only when no other option can be adopted and avoided for rubble filled stone masonry walls. Attention should be given to the durability of the elements adopted as transverse tie (e.g. stainless steel, composite materials, fibre reinforced polymers or others materials) as well as to the chemical compatibility of the injected mortar.

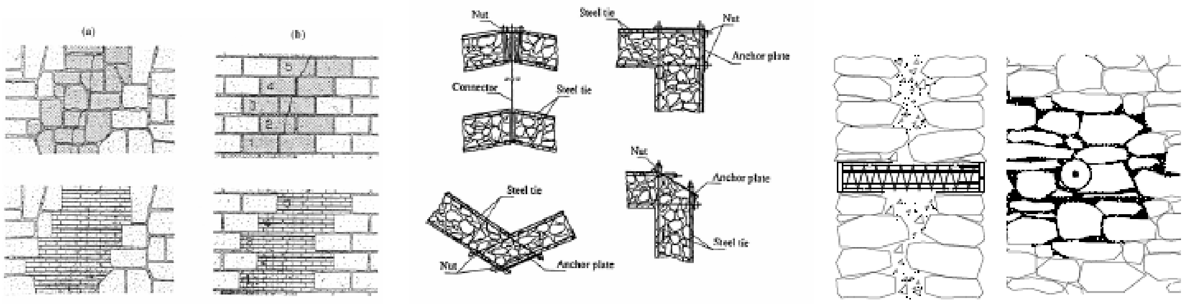


Figure 5.2.6.4. *Local connections: a) disconnecting and reconnecting adjacent units, b) installation of steel ties, c) reinforcing steel bars grouted into drilled holes.*

Ring beams

Ring beams represent an effective solution to improve the wall to wall connections (in the upper part of the building where the masonry wall is characterized by a limited cohesion because of a limited compressive level) and to improve the wall to roof connection. Different techniques and material can be employed, ranging from reinforced masonry to steel or reinforced concrete ring beams.

Reinforced masonry ring beams allow achieving a high level of preservation and compatibility with the characteristics of the existing masonry walls. A good quality masonry material has to be used along the whole wall thickness. The most natural solution is the use of bricks. Steel reinforcement is located in the masonry ring beam and bond is provided via cement mortar. The connection to the existing wall connection can be obtained, when dealing with good quality mortar, through the postinstallation of grouted reinforcing steel bars, only dealing with good quality masonry. For poor quality masonry, a local strengthening of the upper part of the wall is needed and the connection is guaranteed by simple friction.

Steel ring beams (Figure 5.2.6.5) represent a valid alternative to the previously proposed solution, thanks to their lightness, limited invasiveness and the fact that they allow proper connections with the timber elements of the roof (possibly limiting their horizontal loads). They can be realized employing a steel truss made of steel angle or plates or placing steel plates on the inner and outer sides of the wall, connected through transverse threaded bars or rods. A satisfactory connection to the wall can be

achieved in either case without the use of post-installed bars. A local strengthening of the masonry wall is still required when dealing with poor quality masonry. An effective variation to the previous proposed solution, is to locate the steel ring-beam on both sides of rather then at the top of the wall. In this way the intervention can be implemented without the need to remove the existing roof, to demolish the upper part of the wall, or to increase the roof level. Moreover, in this configuration the masonry wall is more stable with respect to the upper part, thanks to the contribution of the vertical load from the roof. The lateral ring beam can be realized by a double steel truss structure (both on the inner and outer side of the wall) welded together directly on site in order to minimize issues related to geometrical irregularities, The connection between the inner and outer lateral steel ring beams will be guaranteed through drilled and grouted bars/rods.

Reinforced concrete ring beams can be realized only if limited in height, in order to avoid excessive increase in weight and stiffness as well as in tangential stresses between the ring beam and the wall, which could lead to sliding of the ring beam and desegregation of the masonry wall. Because of the difference in stiffness between masonry and reinforced concrete element, a local strengthening of the upper part of the wall can be in any case required. The connection to the wall can be improved by using post-installed grouted bars/rods.

Figure 5.2.6.5 *Steel ring beam: a) connection between the wood roof element and the walls; b,c) partial strengthening of the plywood panel diaphragm and its connection with the steel ring beams (Regione Marche 2000).*

An effective connection between the horizontal slab and the ring beam has to be obtained in order to avoid sliding of the wooden slab beams and the consequent collapse of the slab. The connection between ring-beam and wall allows, moreover, for a better distribution of the horizontal seismic load and provides a restrain against overturning mechanisms of the wall. Local connections can be realized through drilled holes, provided excessive disturbance to the wall is avoided. The insertion of ring beams in the wall thickness has to be avoided at low floor levels. In fact, they can cause negative effects on the load distribution along the wall.

Interventions to Increase the Strength of Pier Walls

Poor quality masonry pier walls can be subjected to disaggregating collapse mechanisms of the outer side of the wall, due to widespread cracks. The installation of transversal connections within rubble filled walls is a primary need, as the analytical methods for the local or global seismic analysis assume a monolithic transversal behaviour of the wall. Anyway these interventions alone are insufficient to favour the three-dimensional behaviour of the building and to improve the whole structural integrity. The most proper intervention has to be chosen on the basis of the building typology and quality, among the possible alternatives herein presented, the most proper intervention has to be chosen on the basis of the building typology and quality. Materials with the highest compatibility both from the physical-chemical and mechanical point of view have to be employed. Non masonry-based elements, and in particular reinforced concrete elements, have to be employed with caution.

In order to restore the wall continuity along the cracks and in regions of high deterioration, adjacent units or elements can be disconnected and reconnected (Figure. 5.2.6.4a). The same technique can be employed to reduce or to infill small or large openings. Materials similar to the original ones for shape, dimension, stiffness and strength have to be used. The newly introduced elements have to be linked to the original ones through proper connections in-plane and perpendicularly to the wall, in order to achieve a homogeneous and monolithic behaviour of the wall.

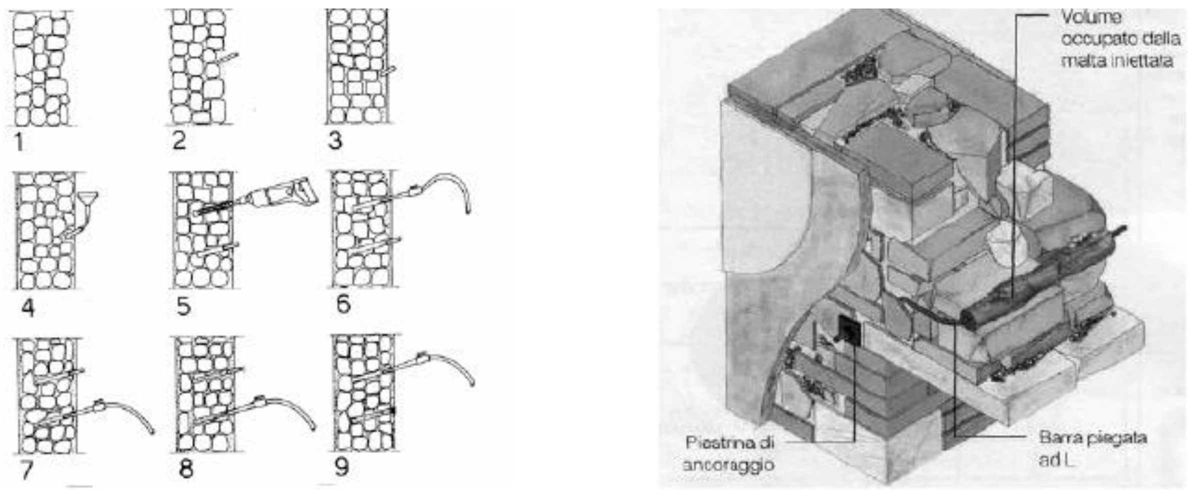


Figure 5.2.6.6 *Increase the pier strength: a) grout injections, b) reinforced injections (Blasi et al. 1999).*

Grout injections (Figure. 5.2.6.6a) can be employed to improve the mechanical characteristics of the wall by increasing the continuity of the masonry and hence its strength. This intervention can not be applied when the masonry typologies cannot be properly injected due to the low percentage of voids.

Particular care has also to be taken when choosing the injection pressure and the type of mortar/grout to be injected in order to avoid chemical and physical incompatibility with the wall. Steel reinforcing or FRP bars can be grouted or cemented into drilled holes (Figure. 5.2.6.4c, Figure. 5.2.6.6b) in order to increase the pier strength and prevent instability phenomena (Lagomarsino et al. 2001). This technique can be used also on poor quality masonry, since it does not induce damages during the execution. It is particularly suitable for rubble filled walls as it improves the effective transversal connection between the inner and the outer wythe. In the case of degraded wythes, it is advisable to firstly employ strengthening techniques such as grout injections or restoring the mortar beds. Restoring the mortar beds, when penetrating the depth of the wall from both side, can lead to significant improvements of the masonry mechanical parameters, particularly for low thickness masonry wall. For walls characterized by medium and high thickness and by non-suitably connected wythe, this intervention could be insufficient to guarantee an actual increase in strength and should thus be suggested only in combination with other interventions. Particular care is required in the selection of the mortar. The possible installation of bars, metal or FRP strips when restoring the mortar beds, can further increase the intervention efficiency (Figure. 5.2.6.7a). If the portion of the masonry wall requiring a retrofit intervention is limited, small diameter transversal bars, bolted through washer on both the sides of the wall can be adopted as a valid alternative (Figure. 5.2.6.7b) to increase the pier strength. A small pre-tensioning can be applied in presence of detached wythes and to contain out-of-plane local deformations. This technique can be used when dealing with regular texture masonry typologies, squared stones, blocks or bricks. The adoption of ties widespread along the three orthogonal directions, but particularly in the transverse direction, can increase the monolithic and mechanic behaviour of the wall, increasing the shear strength and the in-plane and out-of-plane flexural strength.

Figure 5.2.6.7 *. Strengthening solution for pier walls: a) restoring of mortar joints and installation of steel bars, b) transversal bars bolted through washer.*

In order to improve the connection of the wall and to increase the mechanical properties of the wall, traditional jacketing techniques can be adopted. A reinforcing mesh (6 to 8mm diameter) can be placed on both faces of a wall and connected by transversal steel ties. Cover is provided by a cementmortar- based layer. It is not advisable to adopt this technique for the whole building due to the major increase in stiffness and mass as well as for preservation and functional reasons. This technique is only effective if the wall jacketing is realized on both side of the wall and proper transverse connections are adopted (e.g. post-installed reinforcing steel bars). A mesh consisting of FRP strips can also be used as an alternative to the steel mesh when dealing with regular masonry typologies (Figure. 5.2.1.6.8c). The efficacy of this alternative technique is increased when the mesh is positioned on both sides of the wall.

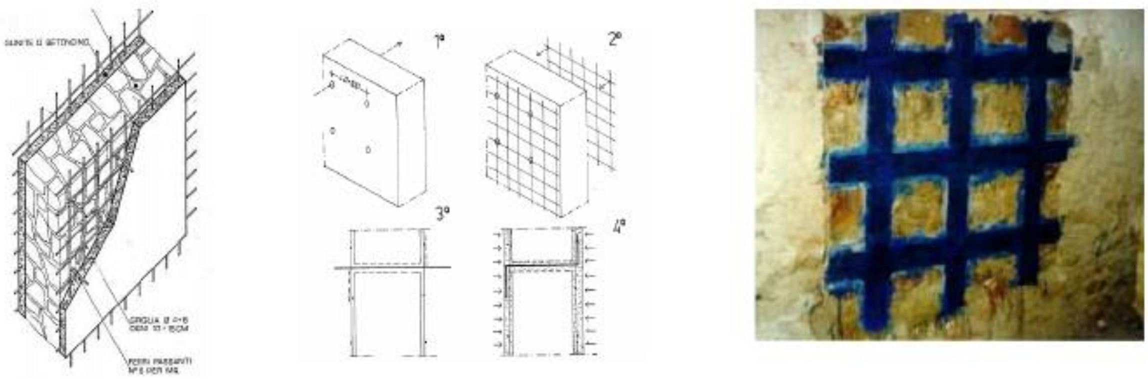


Figure 5.2.6.8. Increase the shear strength: a) b) Wall and pier jacketing, c) Wall and pier jacketing with FRP strips.

Vertical post tensioned tendons can be efficiently used provided that the masonry walls are able to sustain the increased level of compression loads at local and global level. Particular attention should be given to the tendon anchorage zones. Losses in the prestressing force, due to differential deformations in the masonry (e.g. due to creep and shrinkage) have to be taken into account.

5.2.7 Details for Repair and Strengthening of stone and brick Masonry Houses

The performance of the exisiting masonry houses during destructive earthquakes urged governments to conduct effective repair ,strengthening-retrofitting techniques.

Below, strengthening of masonry structural stone and brick/block walls is discussed according to [59]. Based on the initial state of the structure, the seismic resistance verification will indicate the required degree of improving the strength of the walls . The specific repair or upgrade method of choice is influenced from the layout of the walls and type of masonry units and mortar. The following techniques will be discussed:

- Repair of cracked walls
- Repointing the joints of brick/block masonry with cement mortar
- Construction of reinforced cement or reinforced concrete jacket on one or both sides of walls
- Partial reconstruction of brick/block masonry walls
- Construction of RC tie-columns for confining of brick/block walls

Repair of cracked walls

Depending on the size of the cracks, different techniques and repair materials may be used. For brick/block masonry cracks may be sealed with cement mortar, when crack width is between 5 and 10 mm and the thickness of the wall is small. In cases where the wall is of normal thickness and the depth of the crack doesn't allow sealing with mortar, cement grout, which contains admixtures against shrinkage should be injected. For fine cracks ie. width between 0.3 - 3.0 mm. the use of epoxy resins is recommended. When the cracks are larger than 10 mm, the cracked masonry should be reconstructed. For crack width between 1.0 - 3.0 mm cement grout can be injected instead of epoxy resins.

Repair of heavily cracked brick masonry wall with RC coating is shown on Figure 5.2.1.6.9

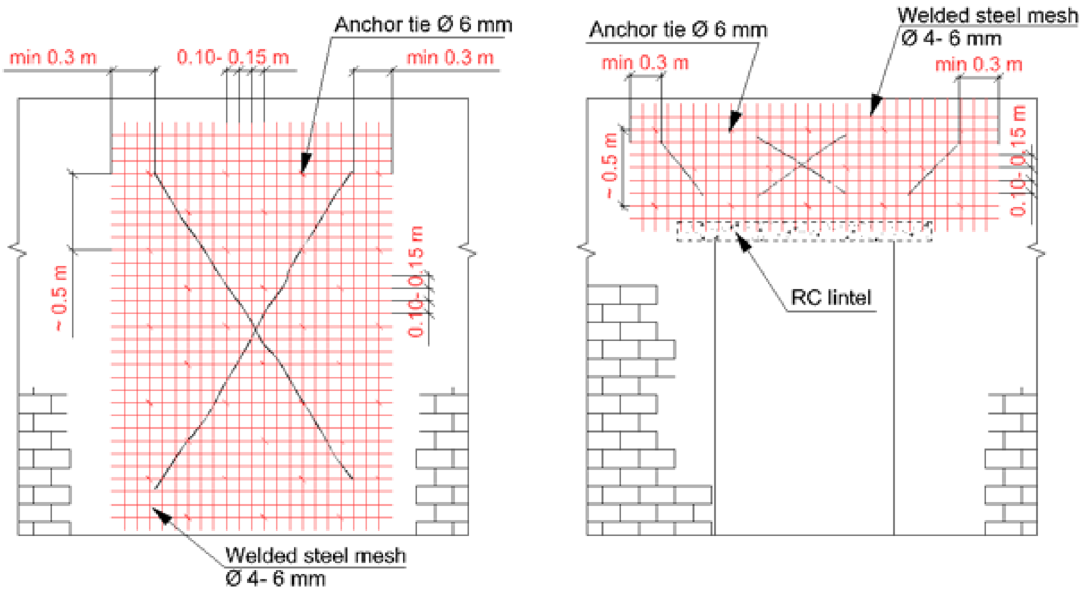


Figure 5.2.7.1 *Repair of heavily cracked brick masonry wall with RC coating [51]*

In cases of masonry walls with cracks wider than 10 mm, where reconstruction is not feasible due to lack of masonry units with required dimensions or properties, RC jacket can be applied. Before applying the jacket the cracks should be sealed with cement mortar or grouted according to the above procedure. The jacket is applied as a concrete cover reinforced with light welded mesh □ 4 - 6 mm over the cracks- see the section for coating of walls.

Repointing the joints of brick/block masonry with cement mortar In cases of brick or block masonry walls where the masonry was left unplastered the mortar can considerably deteriorate with time especially if exposed to severe weather conditions. In addition to that if the brick or block masonry walls built with poor quality mortar where the bed-joints are relatively leveled by replacing part of the original mortar the mechanical properties of the walls can be improved. This technique is termed repointing and should be carried out at only one face of the wall at a time. The existing mortar can be removed using high-pressure water jet with pressure about 40 bar. The depth of mortar removal can be max 1/3 of the wall thickness, due to the eccentricity of the compression forces in the wall, and should be verified by calculation. After mortar removal the surface should be prepared by cleaning watering. In cases where high-pressure water jet was used surface preparation for repair is not necessary. When the joints in the brickwork are about 10 mm it is possible to embed rebars in the joints to improve the dutility and strength of the masonry. The rebars should be anchored at the ends of the wall panels by bending around the corner joints or in the case of confined masonry in the tie-columns. The new cement mortar is then installed. After curing of the mortar on the repaired side the procedure is repeated on the other face of the wall. A detail for wall repointing is shown on Figure 5.2.7.2 below.

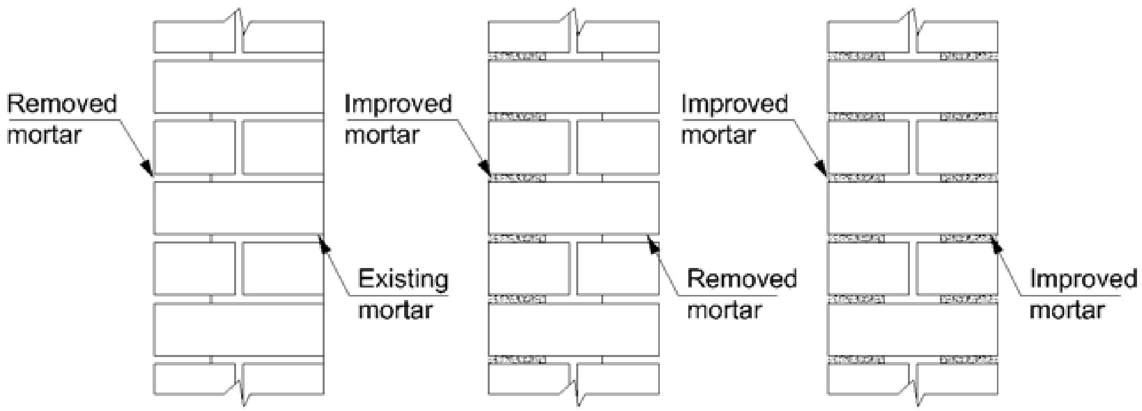


Figure 5.2.7.*2 Repair and strengthening of brick masonry by repointing*

Construction of reinforced cement or reinforced concrete jacket on one or both sides of walls

This method is mostly used in the instances where strengthening of the wall is necessary for rehabilitation purposes or when the wall was severely cracked and damaged in the event of an earthquake. This method is applicable to most types of masonry walls and consists of applying reinforced cement or concrete coatings onto one or both faces of a wall. However application of coating on both sides is strongly recommended. The reinforcing material can be steel rebars mesh, ferro-cement as well as carbon fibres or other carbon composites. The steps for reinforced cement coating, applied to brickwork or blockwork, as described in [2] are :

In cases where the masonry is plastered, plaster should be removed

Mortar is removed from the bed joints to a depth of 15 mm. The cracks in the wall are grouted acoording to the procedure outlined above

- The wall surface is cleaned, moistened with water and spattered with cement milk
- Then the first layer is applied in the form of cement mortar with thickness 10 - 15 mm. The stength of the cement mix should be around 20 MPa
- The reinforcing mesh □ 4- 6 mm at 100 - 150 mm intervals is placed
- Holes are drilled through the wall for cross ties, 4 - 6 steel bars per m^2
- The steel bars(anchors) □ 6 mm are threaded through the holes and then grouted with cement or epoxied
- The reinforcing mesh is connected to the anchors. Minimum laps of the reinforcing mesh to be 300 mm
- The second layer of cement is applied with thickness 10 - 15 mm

Detail showing application of RC coating to brick masonry wall is shown on Figure 5.2.1.6.1

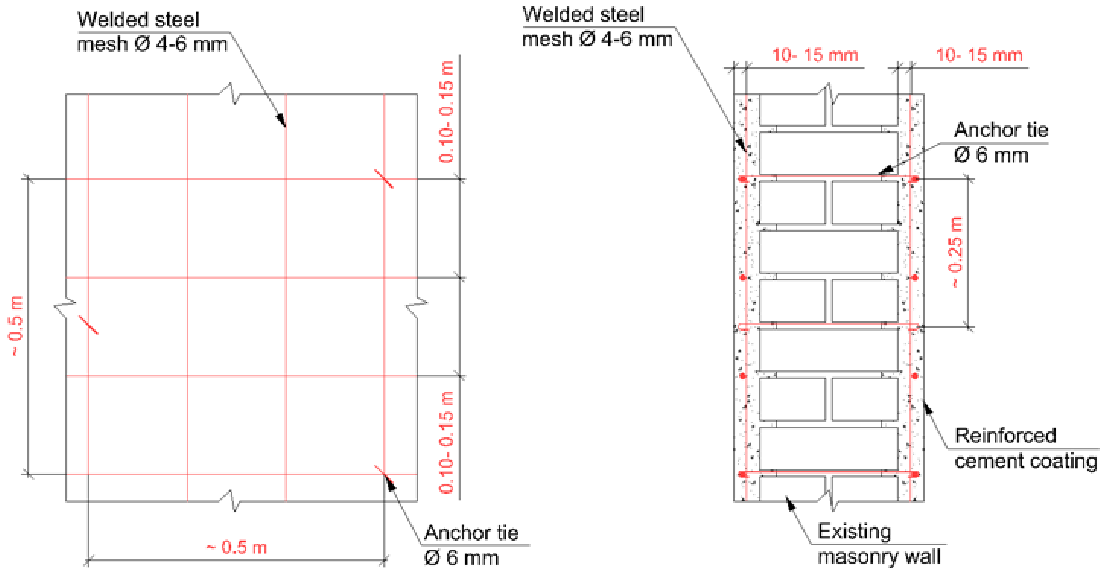


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***Figure 5.2.7.3-a** Applying RC Coating to brick masonry wall [51]*

For reinforcing the cement coating apart from steel bars, can be used ferro-cement, and carbon fibre or polyester fibre composites. According to experimental studies lateral resistance of brick walls can be approved with composites reinforcement to the same extent as with steel reinforcement. Performance of jacketed brick and block masonry walls has been tested both in laboratory and in-situ. The importance in the detail was found to be the cross-anchoring of the jackets through the wall. Table 5.2.7.1 displays test results for both brick and block masonry and different types of coating reinforcement. Due to the substantial increase of the shear strength the bending strength of the walls with height/width ratio greater than 1.0 governed the failure mechanism. The effectiveness of the jacketing technique is greater for the weakly bonded walls.

The steps for reinforced concrete jacketing, applied to rubble or cut-stone stone masonry walls, as described in [2] are :

- In the case of plastered stone masonry, plaster is removed
- Loose stones are removed and any crack or voids are sealed by injecting cement grout or mortar
- Through holes are drilled for □ 8 mm rebars anchors 9/m^2 - every 0.5 m

Alternatively in cases of thick walls or only one-sided jackets shear connectors can be formed with reinforced concrete. For this purpose stones are removed from the walls in a regular pattern and reinforcement cages are placed in the voids spliced 400 mm to the main reinforcing mesh.

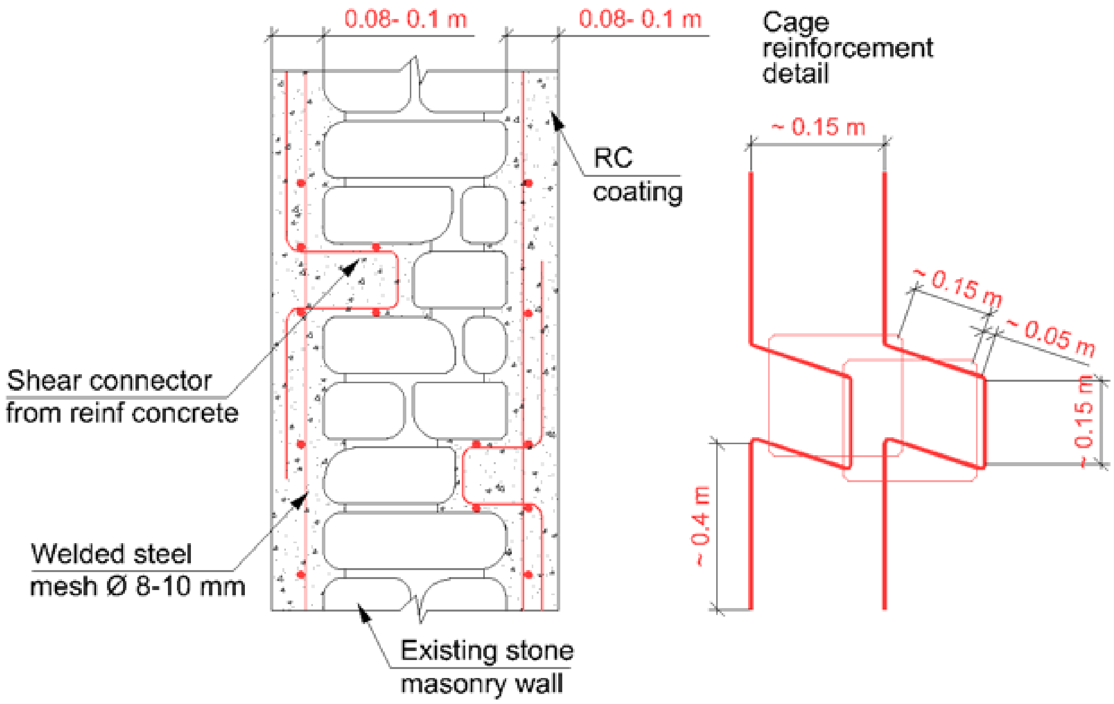


Figure 5.2.7.4-b *Detail of a connector for bonding the RC coating to stone masonry wall [51]*

- The surface of the wall is treated with water jet
- The reinforcing mesh- 250 x 250 mm, □ 10 mm bars is fixed to the cross bars in position
- The formwork is erected and the concrete is poured into

In order to complete seismic resistance verification of masonry buildings when strengthening the walls according to this technique, the following equation for determining the lateral stiffness is used:

$K_{e,eq} = K_{e,w} + K_{e,coat}$,
where the meaning of symbols is as follows:
$K_{e,eq}$ = lateral stiffness of equivalent wall ,
$K_{e,w}$ = lateral stiffness of original masonry wall ,
$K_{e,coat}$ = lateral stiffness of RC coating

Conservative estimation of the design shear resistance can be achieved using the following equation:

$H_{sd,eq} = C_{rh}*A_{rh}*f_{yk}/\square_s + C_{rv}*A_{rv}*f_{yk}/\square_s$,
where the meaning of symbols is as follows:
$H_{e,eq}$ = design shear resistance of equivalent wall ,
C_{rh} = 0.9 (horizontal reinforcement capacity reduction factor) ,
C_{rv} = 0.2 (vertical reinforcement capacity reduction factor),
A_{rh} = area of horizontal rebars cross-section ,
A_{rv} = area of vertical rebars cross-section ,
f_{yk}= yield stress of reinforcing steel,
$\square_s$ = 1.0 (partial safety factor for steel) In the above equation the design shear resistance of the jacketed wall is calculated based on the tensile capacity of the horizontal mesh rebars and the dowel capacity of vertical mesh rebars. The dowel mechanism is created from the vertical rebars resisting bending at the moment of shear failure. The mechanism of action of vertical and horizontal reinforcement of a masonry wall failing in shear is represented on Figure 5.2.1.6.12

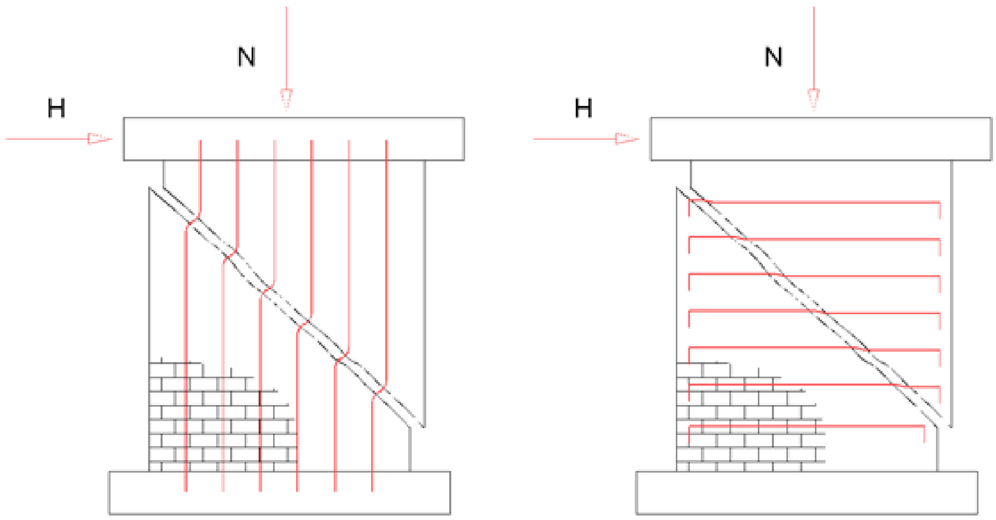


Figure 5.2.7.5 *The mechanism of action of vertical and horizontal reinforcement of a masonry wall failing in shear [65]*

The shear capacity of the masonry and the concrete of the jacket is not taken into account. The bending strength of jacketed walls can be conservatively calculated by neglecting the contribution of the masonry and analysing the reinforced concrete jackets as RC shear wall. The effect of reinforced coating on the lateral resistance of walls is shown below in Table 5.2.7.1:

Type of masonry		Type of cement reinforcement	Lateral resistance		Multiplier
Masonry unit Grade	Mortar Grade		Original [kN]	Strengthened [kN]	
Brick, B20	M 0.4	Steel	34	118	3.5
Brick, B10	M 0.3	Steel	47	167	3.6
C. block, B7.5	M 5	Steel	128	167	1.3
Brick, B20	M 7.2	Ferro-cement	276	693	2.5
Brick, B15	-	Carbon	299	426	1.4

Partial reconstruction of brick/block masonry walls For brick and block masonry walls where crack width is more than 10 mm the damaged masonry should be reconstructed as outlined in [59] The damaged part of the wall should be demolished and reconstructed. The new masonry should be from masonry units units the same type and dimensions as the existing ones. Cement mortar should be used for the reconstruction. Steel masonry connectors can be embedded in mortar to improve the connection between new and existing masonry as well as the bonding between the wall leaves. When reconstructing masonry walls around openings, possible measure is to increase the strength and stiffness of those zones. This can be achieved through construction of reinforced masonry or reinforced concrete window frames. The downside is however the subsequent unpredictable behaviour. Construction of RC tie-columns for confining of brick/block walls In the case of seismic rehabilitation and retrofit of unreinforced brick/block masonry the building's integrity can be improved by improving its structural system. In the past years, both in rural and urban areas of the world, a large number of unreinforced masonry buildings with RC slabs have been constructed. In the cases where RC bond-beams are provided under the floor slab the masonry of such houses can be effectively confined by casting tie-columns at corners, wall intersections and on both sides of large openings. Construction of new tie-column at T junction in brick masonry wall is shown on Figure 5.2.7.6

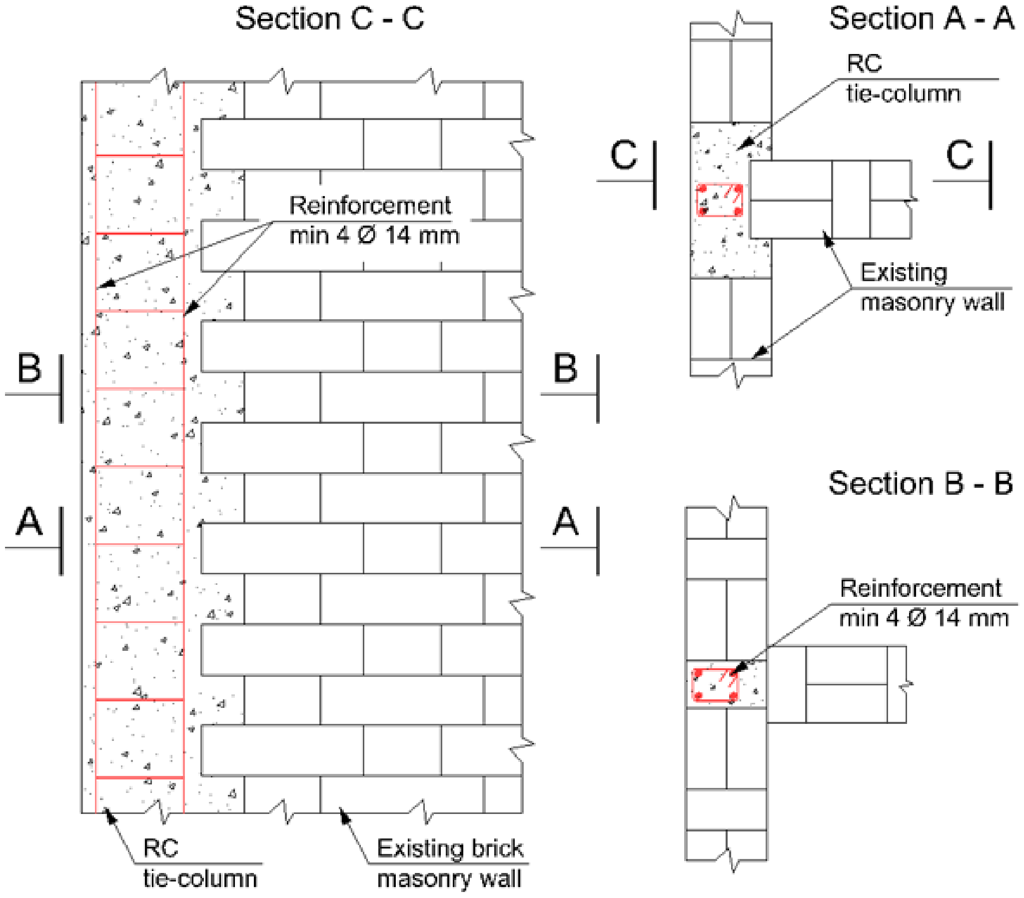


Figure 5.2.7.6 *Construction of new tie-column at T junction in brick masonry wall*

This technique lends itself quite naturally in the case of retrofit of earthquake damaged houses. The possibly cracked masonry at corners and wall intersections is deconstructed and the surfaces of the adjacent walls prepared by spraying with water. In order to ensure confinement of masonry panel, the reinforcement of the new RC column should be anchored into the bond beam or welded and the column should be footed on the already existing strip footing. If foundation doesn't exist a footing should be cast and incorporated in the foundation of the building. The size and distribution of longitudinal and shear reinforcement are shown on Figure 5.2.7.6 Normally the depth of the tie-column is determined to fit into the wall and the other dimension is similar but not less than 0.2 m. Care should be taken to distribute these elements uniformly in plan and elevation.

In cases when the brick/block masonry at corners and wall intersections zones is undamaged or deconstructing it is not possible tie-columns are replaced with installation of isolated rebars. The reinforcing bars are placed in vertical channels cut into the brickwork. This bars are anchored to the rest of the wall by means of stirrups- see Figure 5.2.7.7

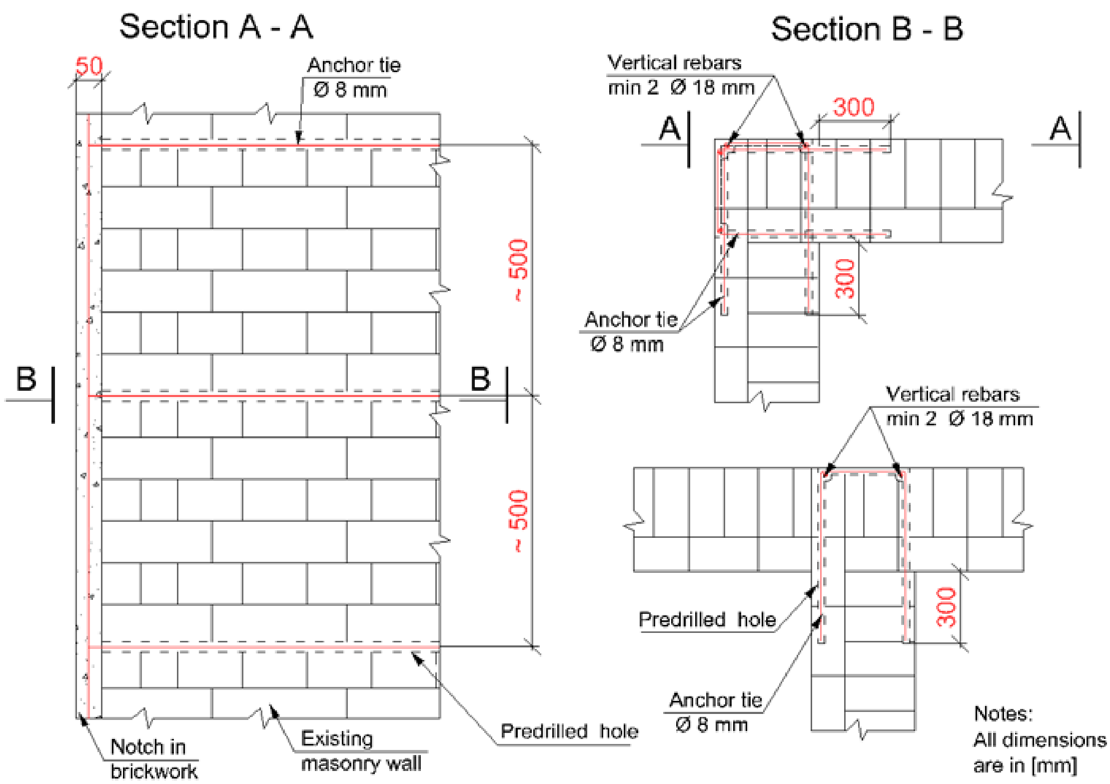


***Figure 5.2.7.7** Detail for confining of masonry walls vertical steel ties [45]*

Details for improving the structural integrity of brick/block masonry houses

Details for improving the structural integrity ie. the interconnection between structural components of brick/block masonry houses are discussed below. These techniques are applicable for most types of brick/block masonry walls. The methods appllicable are:

- Tying of walls with steel ties
- Tying of walls with RC bandage at floor/roof level
- Anchoring of floors and roofs to walls
- Stiffening of floors and roofs in their plane
- Reinforcing and/or stitching the wall intersections
- Tying of walls with steel ties

Steel ties have long been used for preventing lateral instability of perimeter walls as well as for improving the integrity of corners by installing ties in proximity to the buildings' corners.

Position and distribution of steel wall ties in plan of a stone masonry house are shown on Figure 5.2.1.6.15. On Figure 5.2.1.6.16 the vertical position and distribution of steel wall ties are displayed.

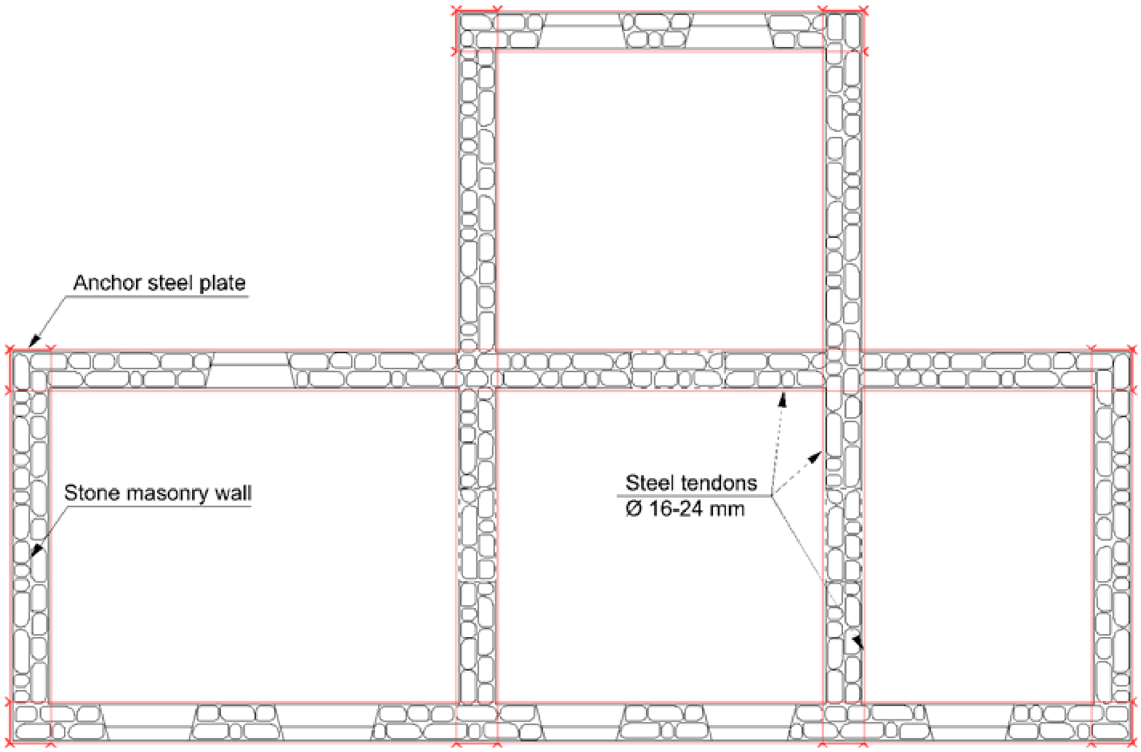


Figure 5.2.7.8 *Position and distribution of steel ties in plan of a stone masonry house*

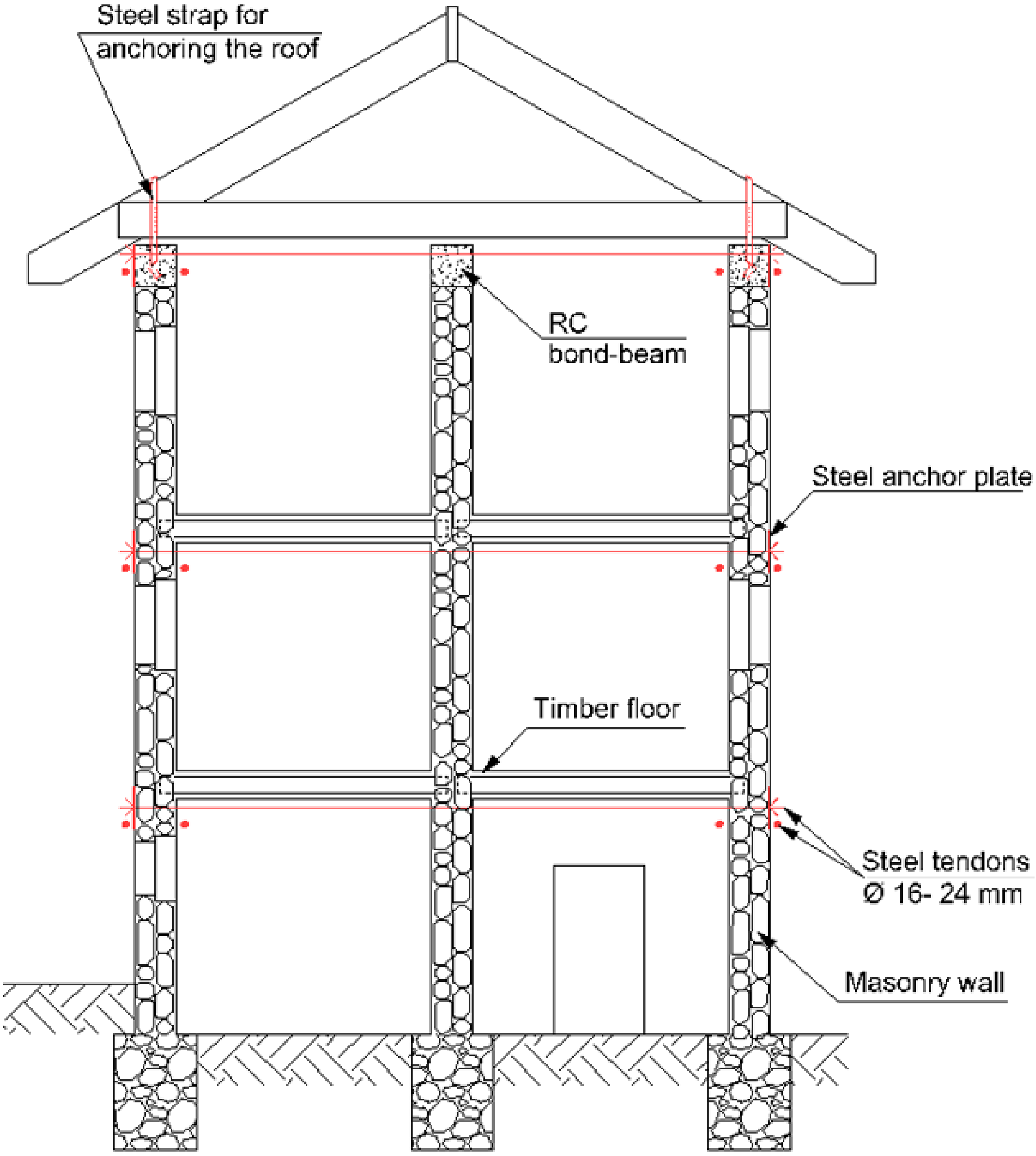


Figure 5.2.7.9 *Vertical position and distribution of steel ties for brick & stone masonry house*

The technique can be successfully applied for both rehabilitation and retrofit purposes. The steel ties can be made from ordinary reinforcing steel bars by threading them at the ends. The ties are fixed into position by being bolted to an anchor plate at each end. When installing the ties along longitudinal walls of considerable length stirrups should be placed through drilled holes in the wall to keep the ties in position. The anchor plates are made from steel and should be designed based on both structural and architectural considerations. The anchor plates can be rectangular, with an X or circular shape. For best performance ties should be installed on both sides of a wall just below the floor and bolted on a single plate. In the case of plastered masonry walls the plaster should be cut about 0.05 m deep on each side, to form a horizontal channel for placing the steel rebars. After completing the installation of the steel ties, all steel components should be protected against corrosion. Detail for tying of walls at T junction using steel ties is shown on Figure 5.2.7.10 On Figure 5.2.1.7.11, detail of anchoring of wall steel ties at a corner is displayed.

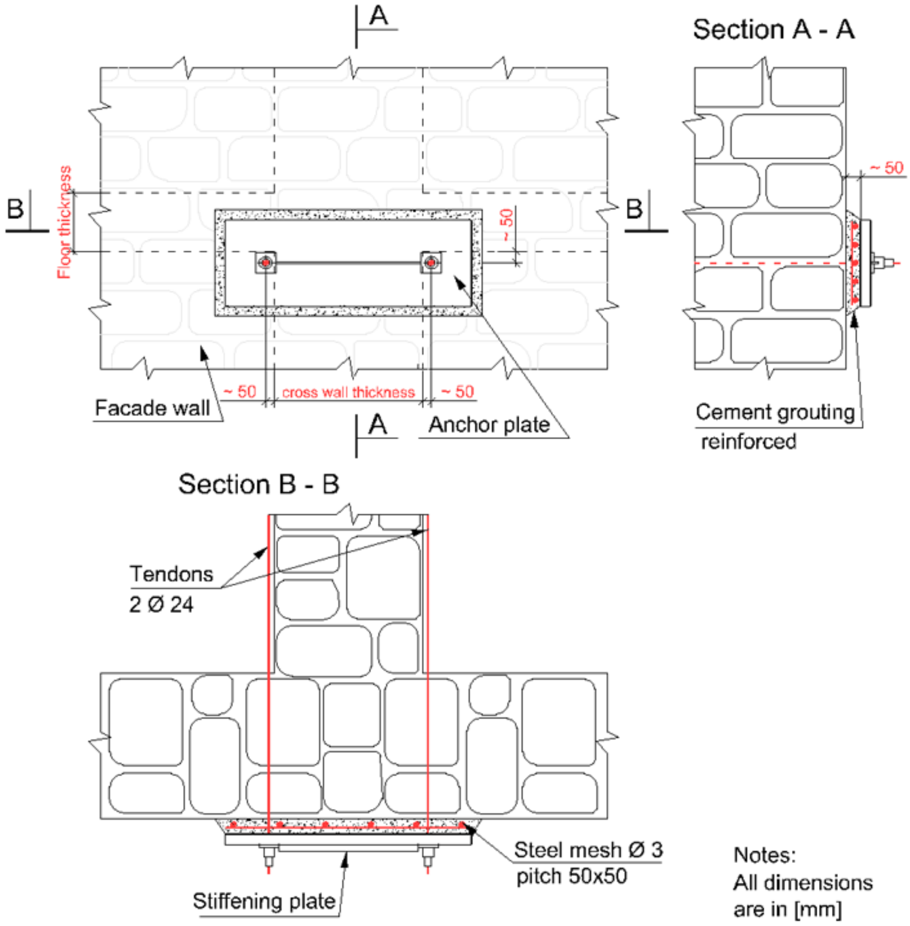


Figure 5.2.7.10 *Detail for tying of walls at T junction using steel ties [46]*

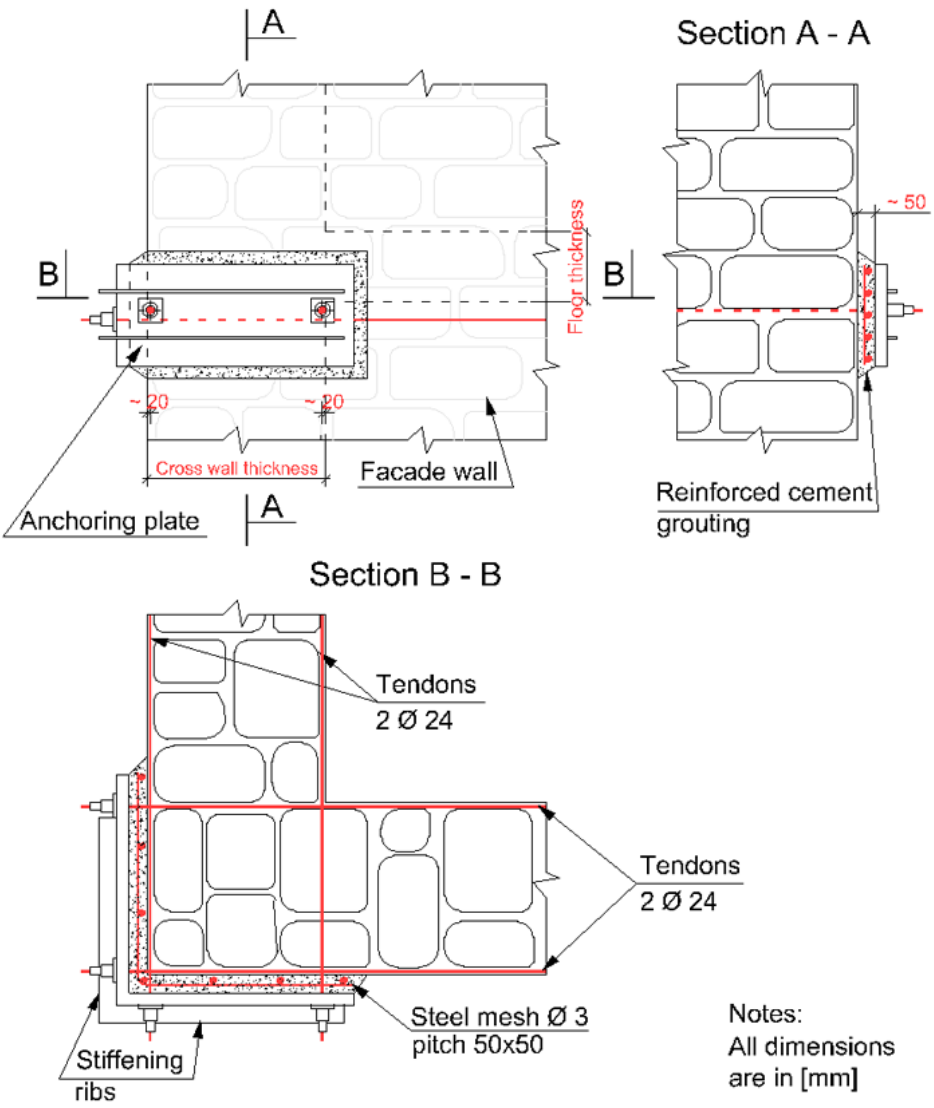


Figure 5.2.7.11 *Detail of anchoring of wall steel ties at a corner [46]*

According to [61], the performance of ties is governed by:

- In case when steel ties are placed along the walls being transverse in respect to the seismic motion, the ties together with the upper part of wall perform like a bond-beam. Consequently ties together with a strip of the upper part of wall should be designed to sustain a bending moment, due to the out-of-plane wall vibration. According to this criteria both ties should be identical and designed just like longitudinal bond-beam reinforcement.

- In case when steel ties are placed along the walls being longitudinal in respect to the seismic motion, the ties take part in the global structural response by forming horizontal tensile(no compression) members at each floor level. A global truss system develops, where the compression forces carried from compression masonry diagonals are carried over from strorey to storey, thanks to the horizontal truss members ie. the steel ties. From the experimental studies [61] , forces which develop in the longitudinal ties at ultimate state are close to the seismic shear force induced in the models.

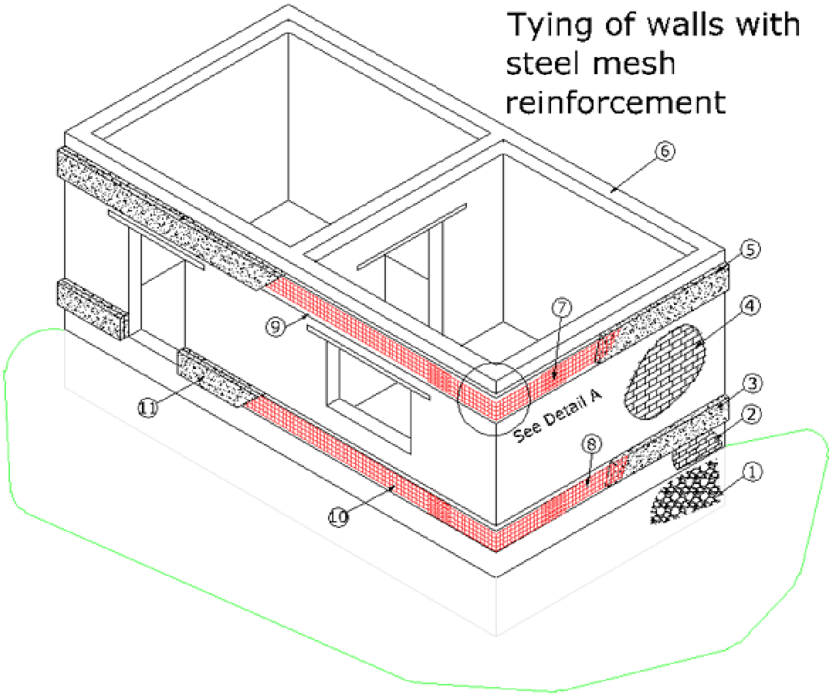
Consequently at least two design checks are required for each tie to estimate its min diameter. The biggest determined wall tie diameter from the design checks for all ties, in plan and elevation, should be used throughout the building. The minimum diameter of a single tie rebar can be determined by the formulae:

D_{min} = SQRT(4*$H_{u,seg}$/□ *n*f_y) ,
where the meaning of symbols is as follows:
D_{min} = the minimum tie diameter ,
$H_{u,seg}$ = the ultimate seismic resistance of critical portion of the building ,
f_y = the yield stress of reinforcing steel ,
n = the number of ties (rebars) ,

Tying of walls with RC bandage at floor level

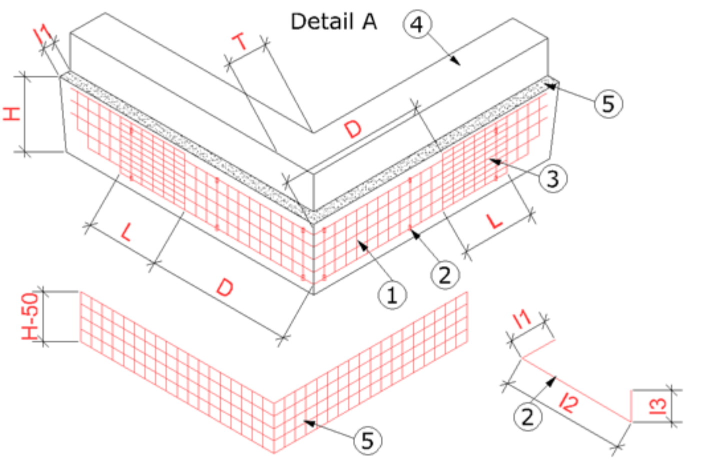
Broad horizontal bandages at lintel or roof level are also recommended .The steel mesh should be fixed to the brick/block wall by means of flat steel ties at 400mm horizontal and 400mm vertical intervals. The steel ties should be embedded in the masonry wall through purposely drilled holes. After completion of the wall a base of cement:sand mortar, mixed in 1:3 proportion and thickness 15mm, for the steel mesh should be installed. The welded steel mesh, minimum □ 5 @ 50x50 pitches, should be attached to the already cast strip of mortar by means of the embedded ties. Finally a second mortar layer of 15mm thickness is installed to complete the reinforcing bandages.

The steel ties should be embedded in the adobe masonry during construction. After completion of the wall a base of cement:sand mortar, mixed in 1:4 proportion and thickness 15mm, for the steel mesh should be installed. The welded steel mesh (diameter min 2mm @ 50x50 pitches) should be attached to the already cast strip of mortar by means of the embedded ties. Finally a second mortar layer 15mm thickness is installed to complete the reinforcing bandages. See Figure 13 and Figure 14 for



① Foundations	② Plinth masonry	③ First layer of plaster of bottom reinforcing band
④ Adobe masonry wall	⑤ First layer of plaster of top reinforcing band	⑥ Bond beam
⑦ Mesh reinforcement splice	⑧ Reinforcing mesh	⑨ Top reinforcing band
⑩ Bottom reinforcing band	⑪ Second course of plaster of bottom reinforcing band	

Figure 5.2.7.12 *Tying of walls with steel reinfrocement*



① Cold bent galvanised steel wire mesh 50x50 with diameter min Ø2
② Steel cross ties - 400 apart with diameter Ø2 - 4
③ Steel mesh splice
④ Roof bond beam
⑤ Layer of mortar to provide base for the reinforcement

Figure 5.2.7.13 *Tying of walls with steel mesh- detail*

L	>=	0.5m
l1	>=	50mm
l2	>=	T+l4
l3	=	50mm
l4	=	15mm
T	>=	0.25m
D	>=	T+0.6m
H	=	0.4m

Anchoring of floors and roofs to walls

The techniques for anchoring of floors and roofs to brick/block walls are applicable to both brick and stone masonry buildings.

In the case of masonry buildings with timber floors tying the walls together at wall intersections, with steel ties, it is not enough to ensure monolithic performance ,especially in the case where distance between transverse-cross walls is significant. To reduce uncoupled wall vibrations and prevent possible out-of-plane cracking or failure, the walls need to be anchored along their span with steel connectors to the floor joists.

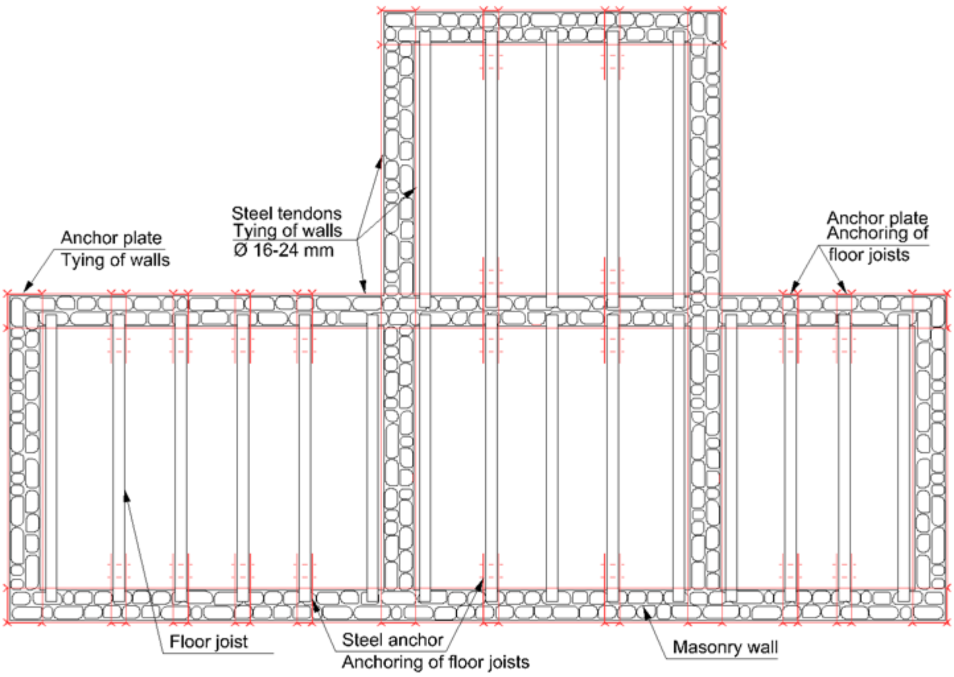


Figure 5.2.7.14 *Position and distribution of wall and floor joists steel ties in plan of a stone masonry house*

A detail showing anchorage of timber floor joists to a stone masonry wall is shown on Figure 5.2.7.15

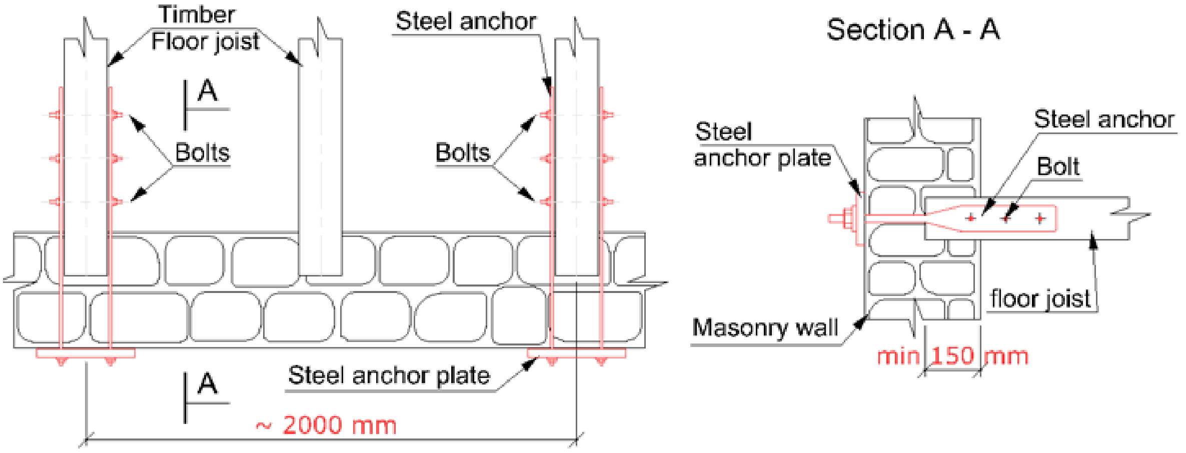


Figure 5.2.7.15 *Detail for anchoring of timber joists to stone masonry wall [45]*

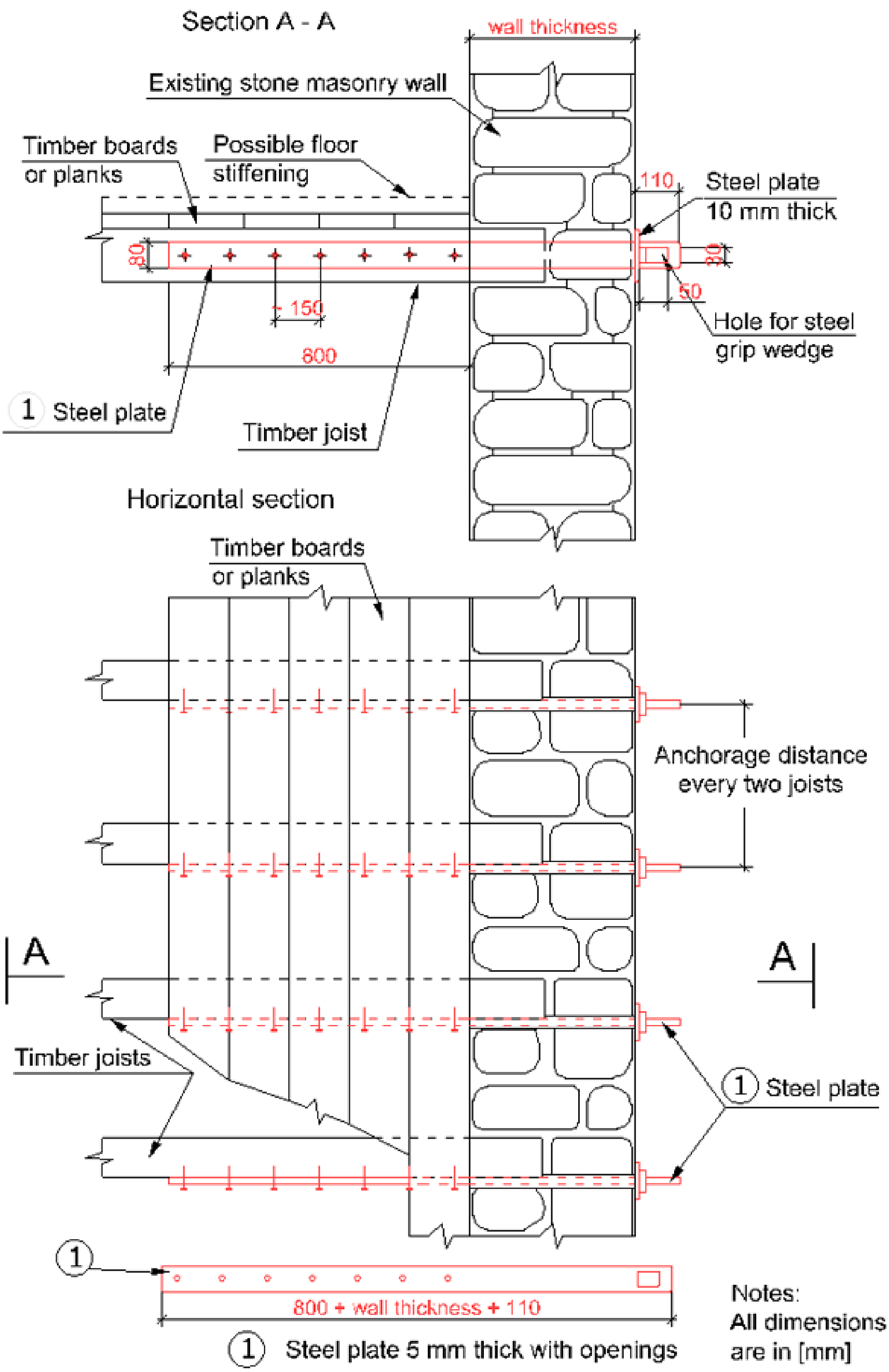


Figure 5.2.7.16 *Detail for anchoring of floor joists to masonry walls [46]*

Furthermore in cases where the floor structure is weakly connected to walls this technique will prevent sliding mechanism and collapse of the floor. Depending on the structure of the timber floor and the distance between cross walls is determined the position of the steel connectors. Apart from connecting the floor joists to walls by means of steel connectors, additional diagonal ties can be installed. In the case of considerable distance between cross walls, and complications with anchoring of existing timber components to walls, a complete metal truss can be installed on top of the timber structure below the floor finish. The steel truss is placed so that its sides are in contact with at least three adjacent walls. Using steel anchor bolts the truss is connected to the walls - Figure 5.2.7.17

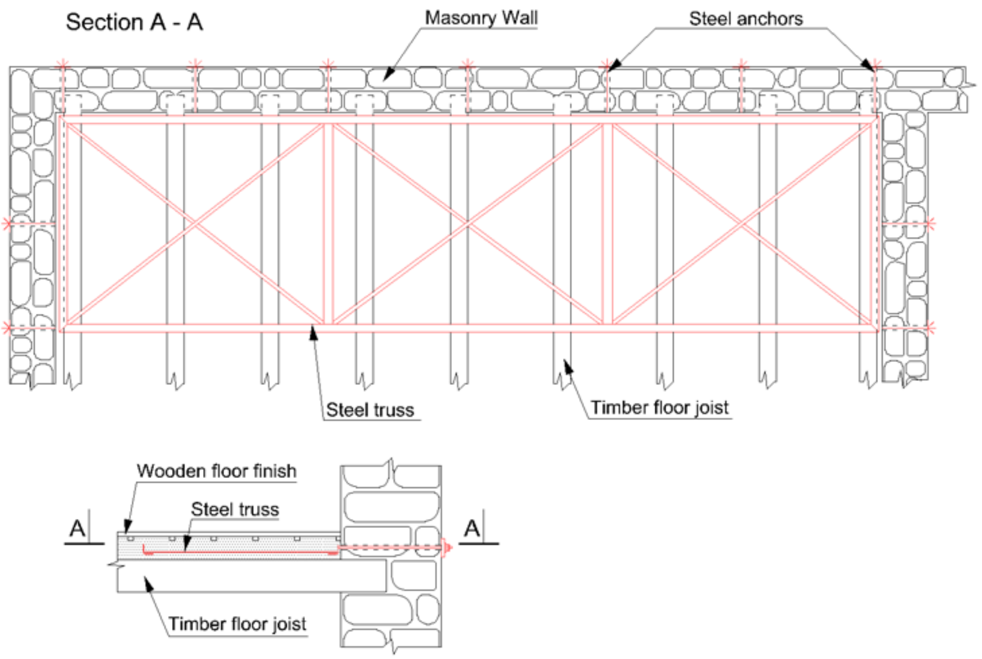
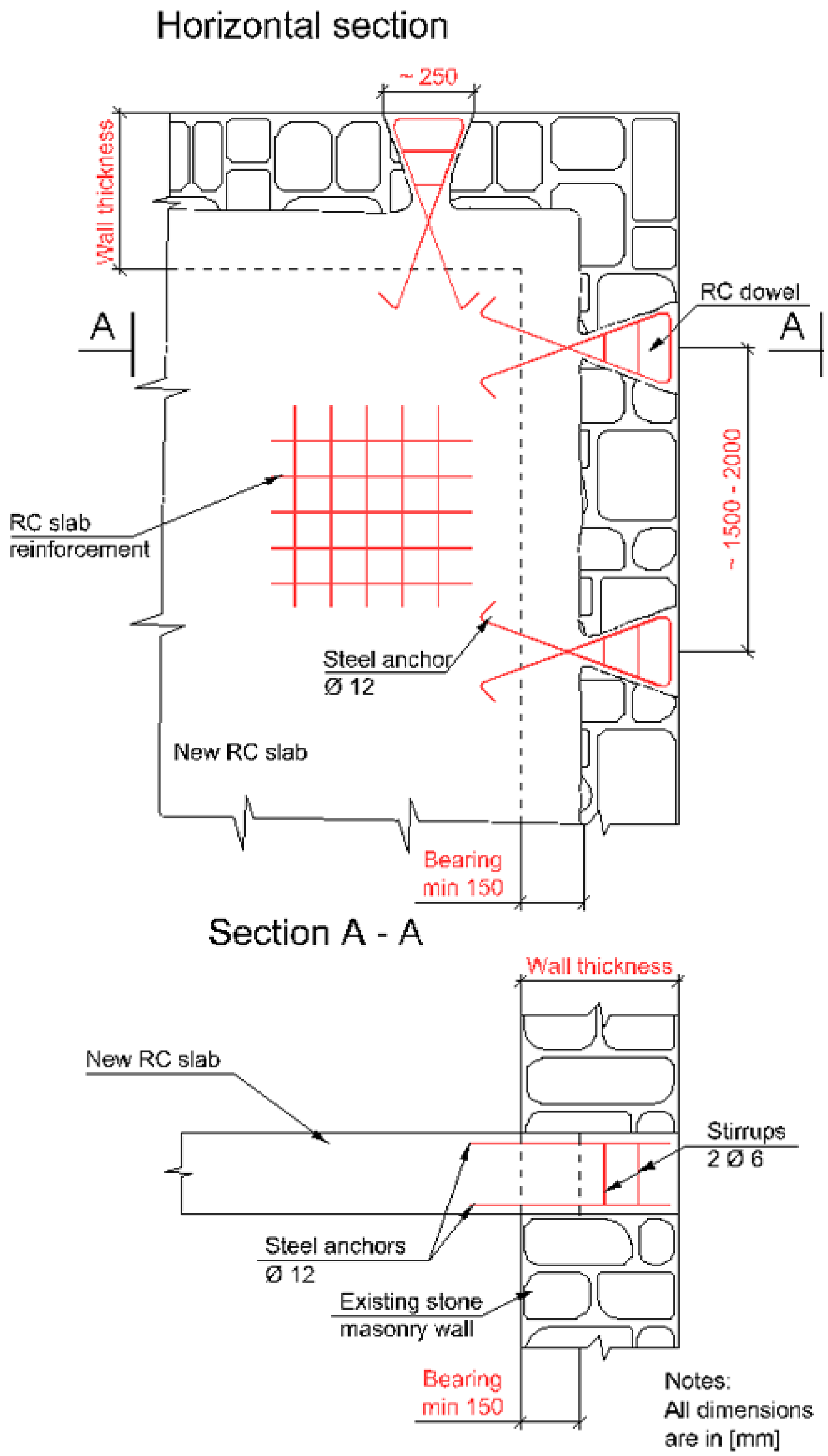


Figure 5.2.7.17 *Sstiffening of a large span timber floor by placement of steel truss [45]*

Stiffening of floors and roofs in their plane

In order to achieve distribution of seismic forces in respect to the walls stiffness the floors and/or roofs must be ideally stiff in their plane. In this case is said that the floors are acting like horizontal diaphragm. Semi-flexible or flexible floors/roofs distribute the seismic loads based on the floor to wall stiffness ratio or based on tributary wall areas. For retrofit purposes, especially when the timber floor and/or roof structure is damaged a straightforward solution is to replace it with cast in-situ reinforced concrete slab. Simultaneously with the slab are casted perimeter bond-beams on top of all walls. Installation of new RC slab is shown on Figure 5.2.1.6.25. If the perimeter bond-beams do not pass over the entire width of walls the slabs should be anchored to the walls using reinforced concr



anchors.

Figure 5.2.7.18*- New RC slab cast in situ and its anchoring into stone masonry walls [45]*

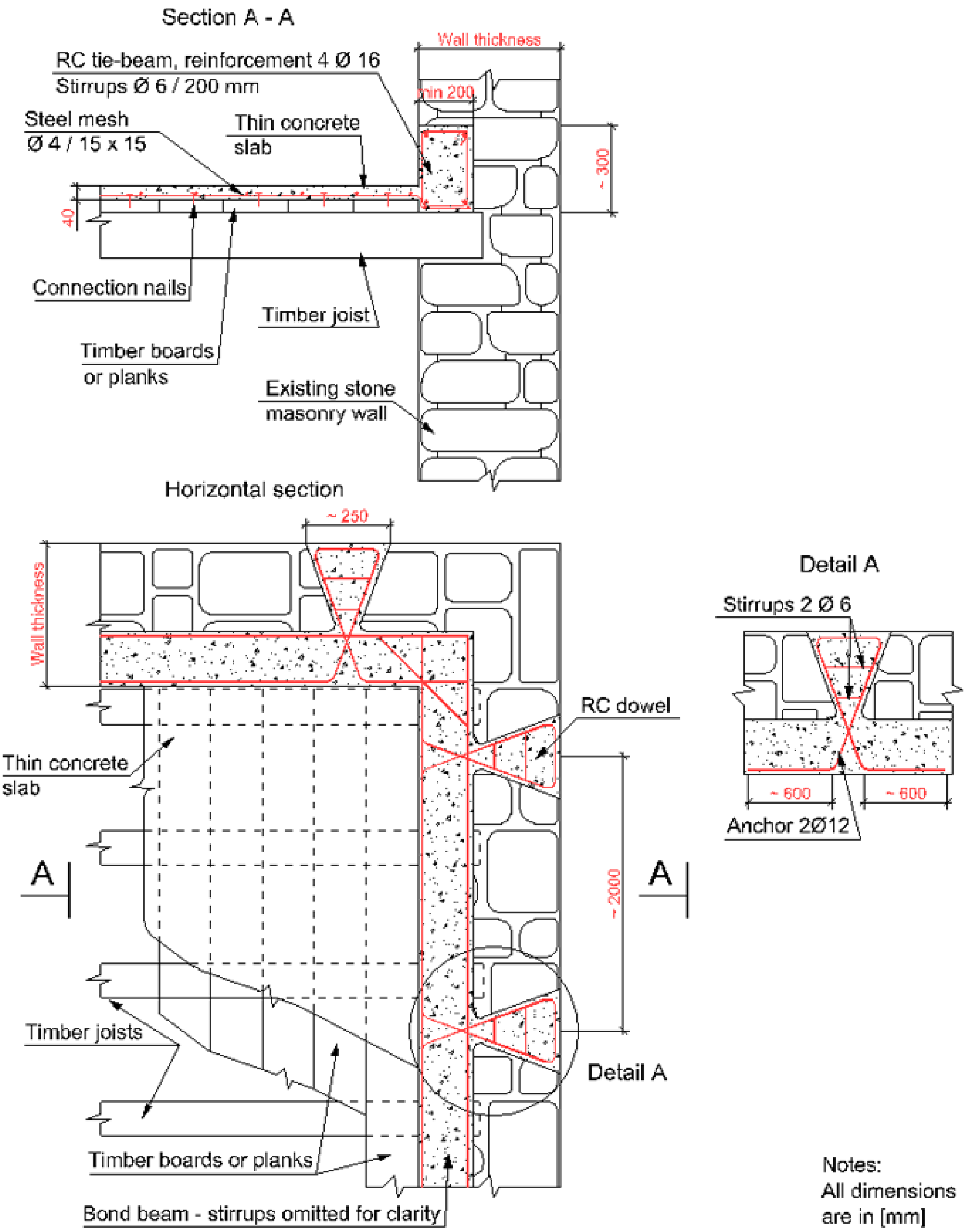


***Figure 5.2.7.19**- Construction of new thin RC slab and bond beam cast in situ, anchored to sone masonry walls [46]*

In some cases the timber floor can be retained and just stiffened using thin reinforced concrete slab connected to a bond beam. For this solution the timber floor is used as a formwork for casting a thin RC slab. The steel reinforcement of the new slab needs to be securely nailed to the floor boarding. In the stone wall, stones are removed to allow casting of a bond beam together with the slab. Since the bond beam would only partially engage the walls, a through RC dowels are needed to anchor the bond beam to the walls - see Figure 5.2.7.19, below.

For rehabilitation purposes, when the quality of the timber floors and/or roofs is good, stiffening of the floors can be achieved by nailing timber planks perpendicular to the floor joists. Depending on access this can be done both on top and bottom faces of joists. In the case of nailing on only one face two layers of sheathing is applied, see Figure 14. Plywood sheets can also be used.

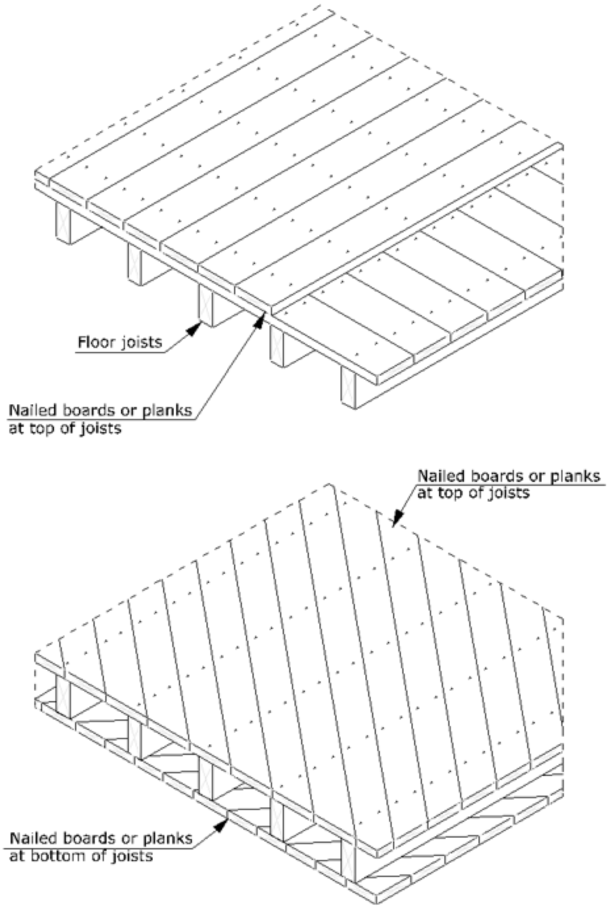


***Figure 5.2.7.20-a** Stiffening of timber floors by nailing boards or planks*

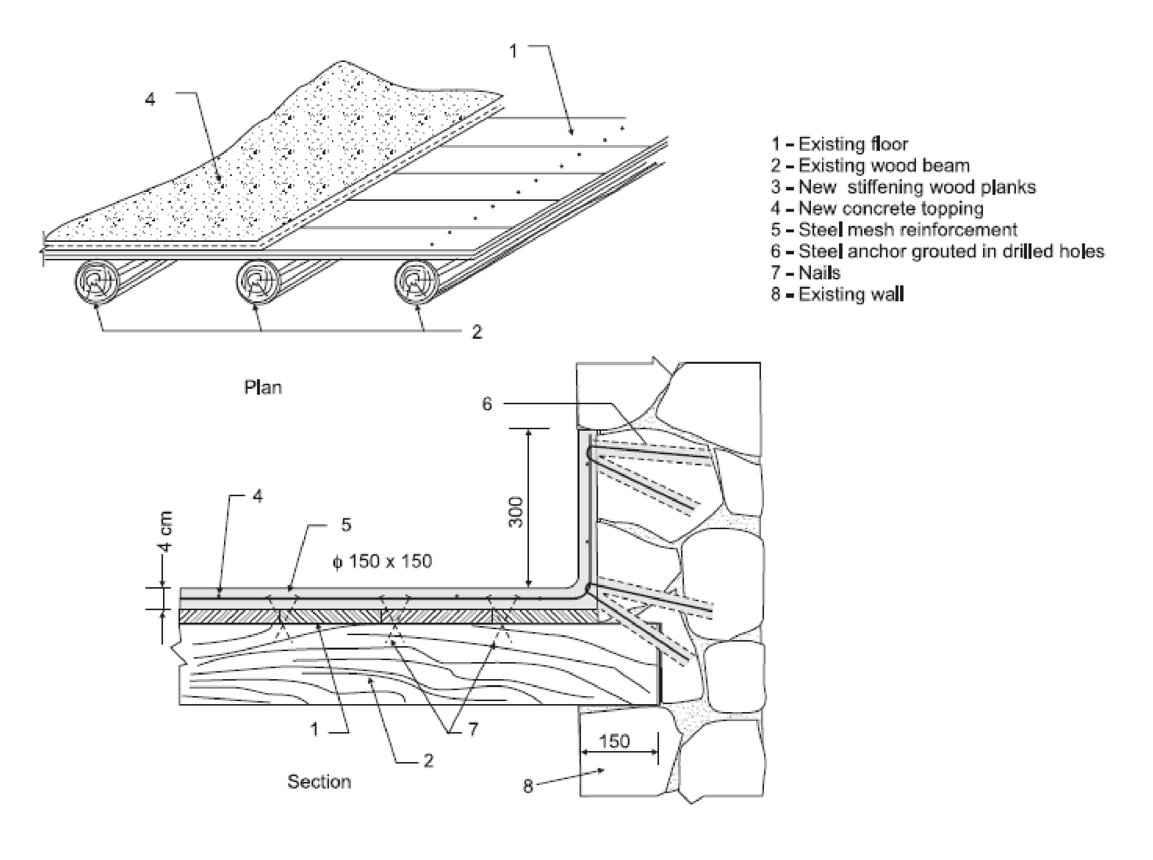


Figure 5.2.7.21-a *Stiffening wooden floor by reinforced concrete slab and connection to wall*

Reinforcing and/or stitching the wall intersections

Masonry wall intersections zones can be strengthened by constructing reinforced concrete jacket. In the case of post-earthquake repair/strengthening of masonry houses where the connection between intersecting walls has already been damaged stitching of the walls together can be done with steel elements called **stitches.** When using reinforced concrete the method consists of applying RC coating at corner and wall intersection zones. Often this method is applied together with the RC jacketing technique, discussed earlier. The same steps should be followed as in the method - construction of reinforced cement or RC jackets on one or both sides of wall. Additional rebar links should be provided, diagonally connecting the internal and external jacket across the corner. When jacketing wall intersections the rebar links are placed in X pattern across the corner. The links can be max 0.5 m appart. Details showing anchoring of RC jackets at corners and wall intersections are shown on Figure 5.2.1.6.28

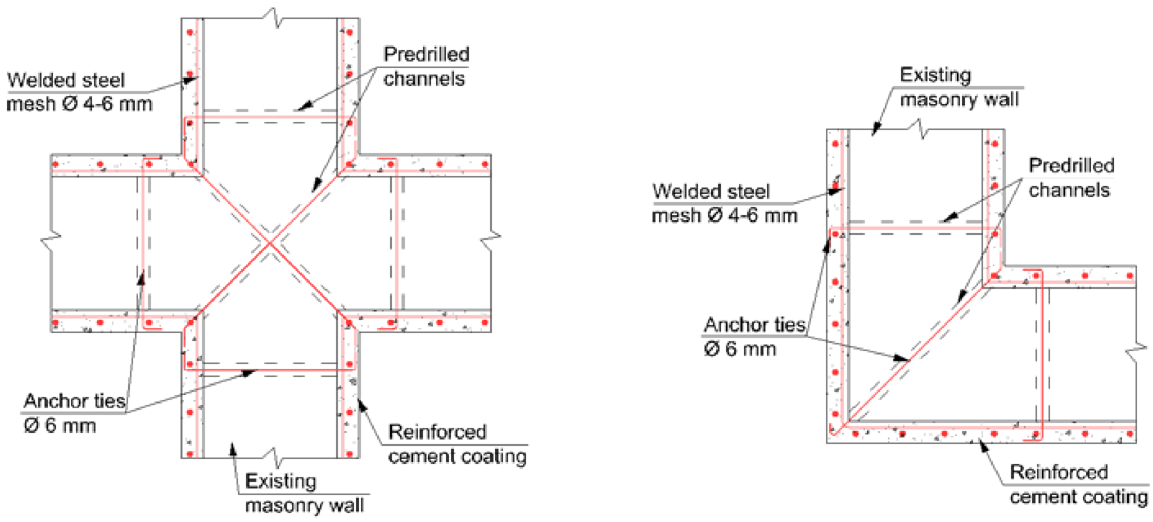


***Figure* 5.2.7.22** *Details showing anchoring of RC jackets at corners and wall intersections[51]*

If for some reason only the corners and wall intersections zones are being strengthened ensure that the lenght of the RC jacket on each side of wall is at least 3 times the wall thickness, to ensure sufficient anchorage. When using steel for stitching wall intersections are used steel strips. These should be with cross section of the order 40 x 4 mm and sufficient developing length of min 3 times the wall thickness. These anchor strips are welded or bolted to an anchor plate. To install the stitches the stones from the correspondent masonry course in the corner area are removed. Depending on the structure of the wall and its resistance stitches are placed in 0.5 - 0.75 m intervals. After the instalation of the steel strips the removed masonry units are placed back using rich cement mortar. Any cracking of masonry in the corner zone should be sealed and full grouting of the intersection zone should be completed. Strengthening of damaged corners and T junctions of masonry walls using steel stitching is shown below on Figure 5.2.1.6.29

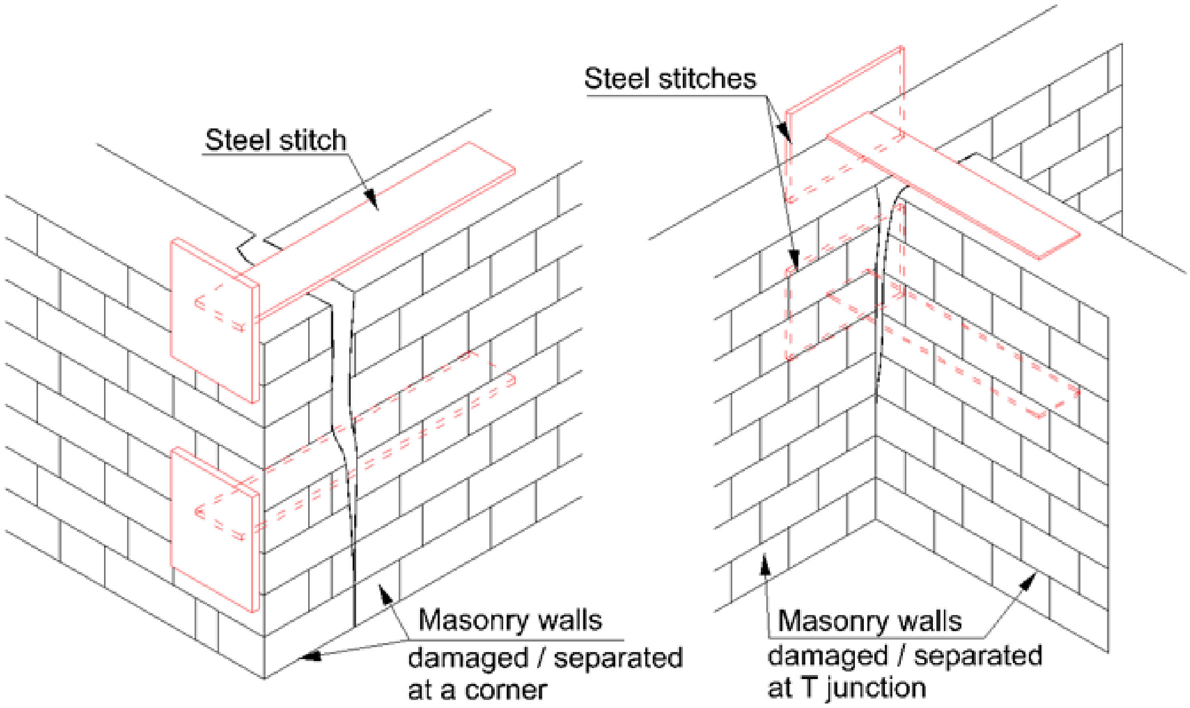


***Figure 5.2.7.23** Strengthening of corners and T junctions of masonry walls using steel[51] stitching*

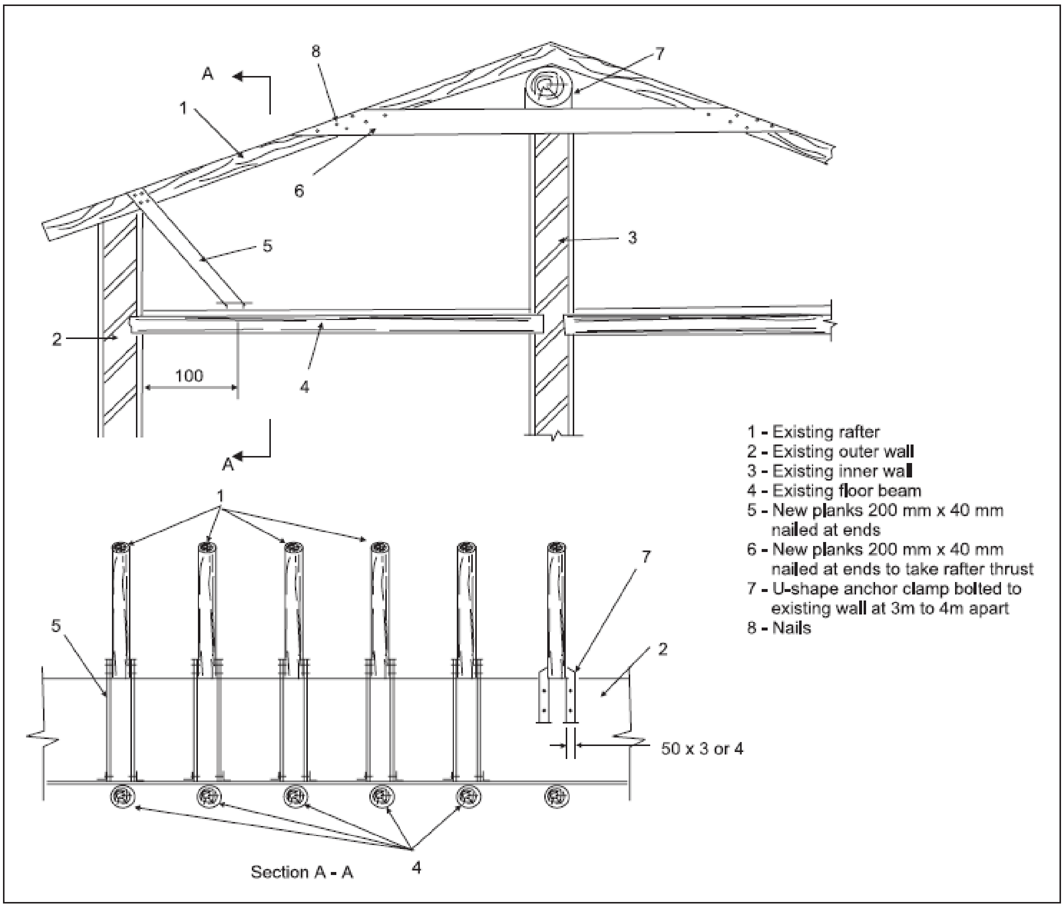


Figure 5.2.7.24 *Roof modification to reduce thrust of walls*

Roof structures should be fully braced against lateral loads. In the case of retrofit of earthquake damaged building, damaged roofs can be removed and rebuild according to specifications for new construction.

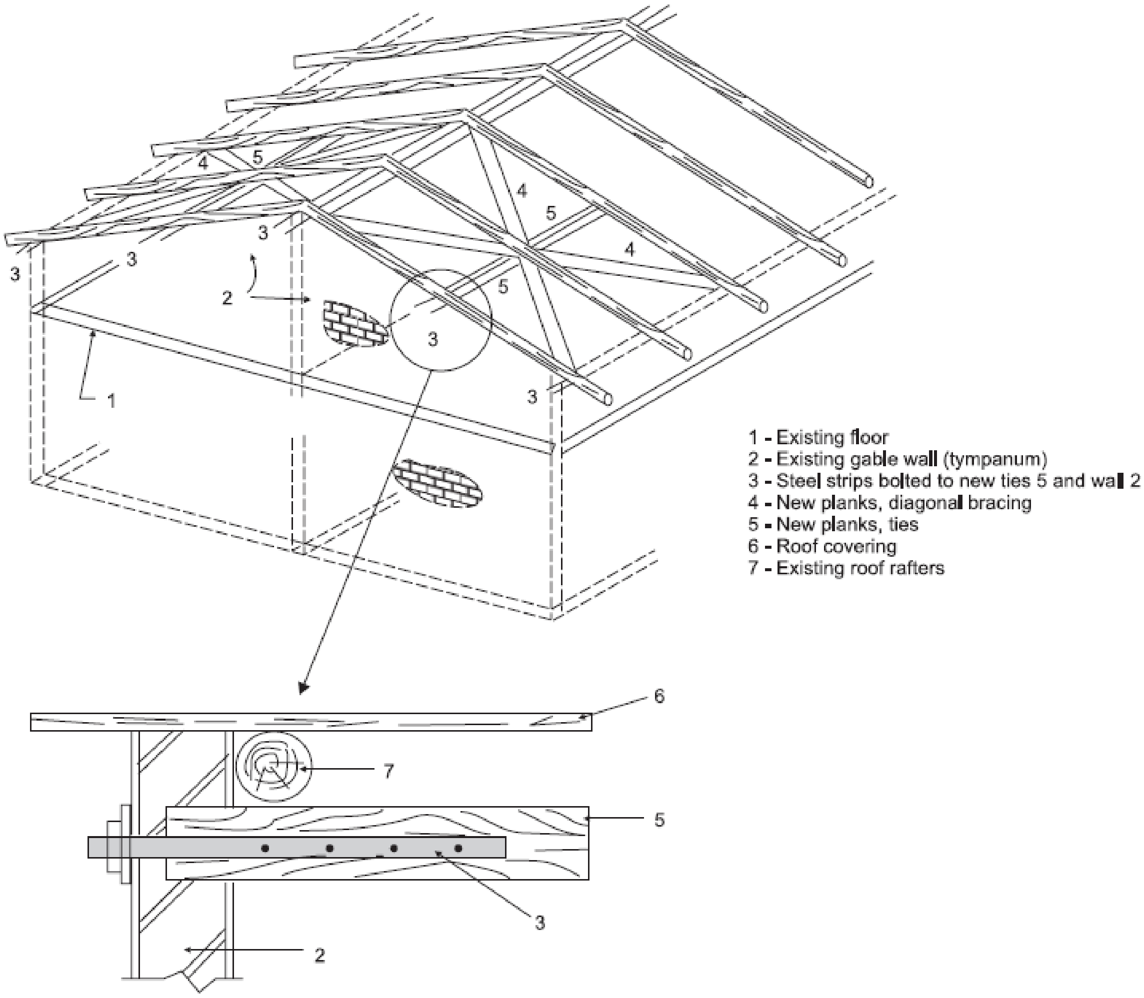


Figure 5.2.7.25 *Details of new roof bracing*

5.2.8 Strengthening of foundations

Measures regarding strengthening of foundations are usually taken as part of seismic retrofit of a building. Geothechnical advice is required and specialised solutions in cases where masonry building has been damaged due to soil failure. Soft clay sites or loose medium-densed sand which is waterlogged may potentially liquefy and should be avoided. In cases where no soil failure was observed foundations still may need to be strengthened when introducing new vertical structural members like tie-columns or shear walls. Interventions to the foundation system are also required due to deterioration of structural materials with time as well as improve the integrity of the building.

When it has been established beyond doubt that the only (or the best) way of remedying the defects of a building is to strengthen its foundations, there are a number of techniques available. Each of these has its particular field of application, its advantages and disadvantages, and all of them involve a certain amount of risk. These factors have to be carefully weighed up, before the final decision is made. Particular care should be taken *not* to accept the guarantees offered by contractors, operating proprietary techniques, without reading ‘the small print’ which may invalidate the guarantee, if certain unforeseen conditions are encountered during the work. The advice of an engineer with specialist geotechnical knowledge *and* with experience in foundation strengthening, as well as familiarity with the structural characteristics of old structures, should always be sought, regardless of the method envisaged.

Existing old masonry buildings are often without foundations. The vertical loads are transferred to the soil directly through the basement walls. In such cases construction of RC strip foundations under the basement walls can be applied. Depending on access limitations or ownership boundaries, the new strip foundations can be constructed by stitching to the sides of the existing basement walls or foundations. Before strengthening the existing foundations, the walls are first consolidated by grouting. As discused in [62] hydrophobic additives may be added to cement grout in order to prevent the propagation of humidity. Typically RC strip foundations are constructed under the walls for strengthening of basement walls or foundations.

York Minster exploratory excavation 1967: The imposition in the fifteenth century of the 16 000 tons Central Tower on the eleventh-century foundations, intended for a much less heavy crossing, caused differential settlement which distorted and cracked the foundation masonry.

Figure 5.2.8.1 *York Minster exploratory excavation*

A continuous, rigid and strong superstructure, such as a solid masonry wall with few and small openings, may be able to bridge across a local area of weak soil under the footing (Fig. 5.2.8.2). Conversely, an articulated superstructure, such as a colonnade, with individual lintels spanning between columns, will offer no resistance to the settlement of a column founded on a soft spot (Fig. 5.2.8.3). The damage in the latter case will however be less than that caused to a rigid wall thatis not strong enough to bridge or cantilever over differential settlement, or which has been weakened by subsequent alterations.

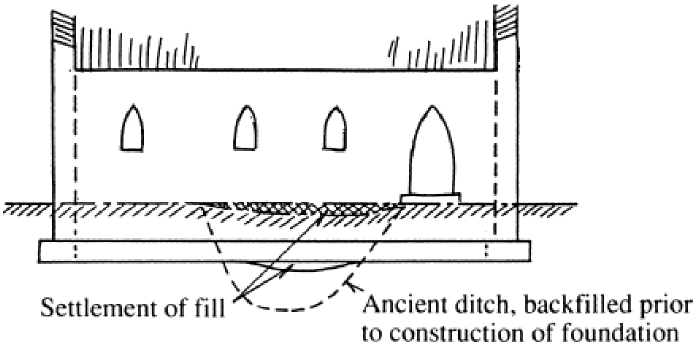


Figure 5.2.8.2 *A rigid and strong superstructure bridges over 'soft spots' and resists differential settlement.*

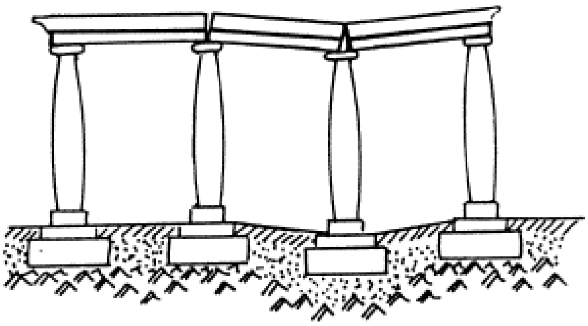


Figure 5.2.8.2 *A flexible superstructure has to follow differential settlement.*

Other Details of strengthening URM buildings

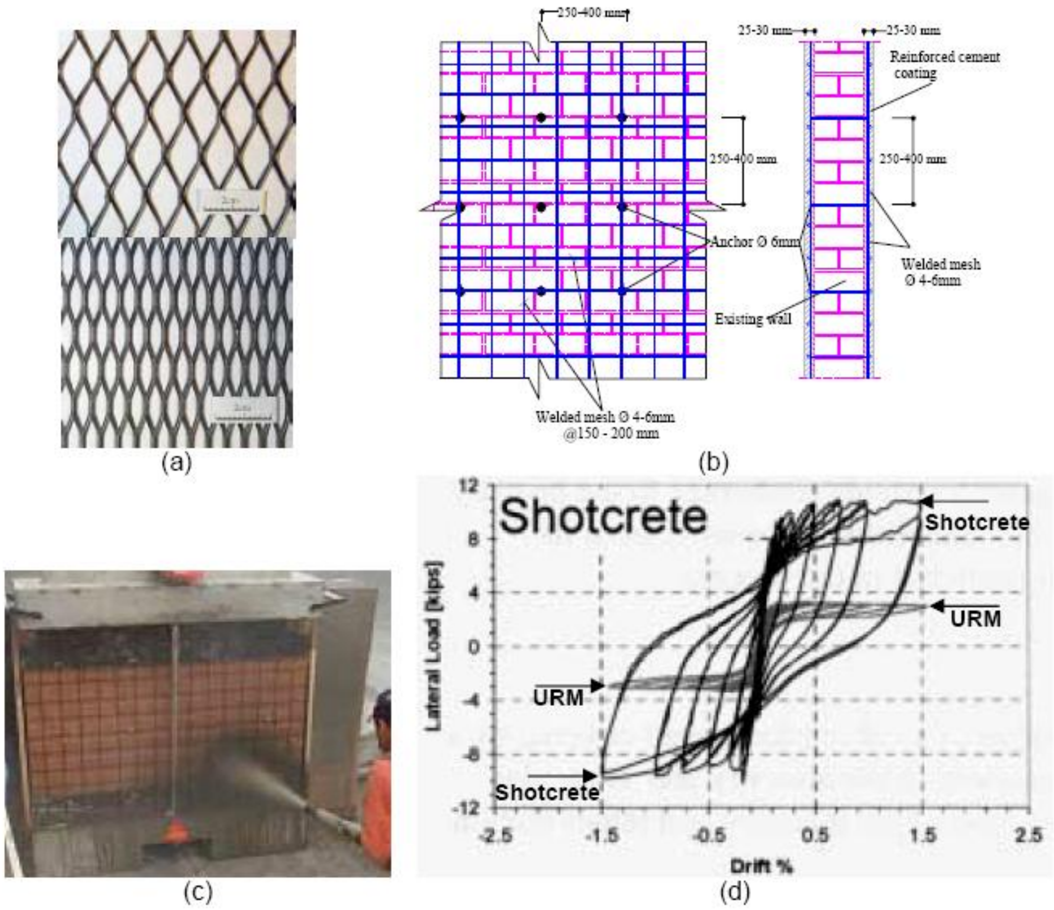


Figure 5.2.7.26 *Surface treatment:(a) hardware samples used in ferrocement (TWP Inc.), (b) reinforced plaster typical dimensions, (c) application of shotcrete for test specimen (ElGawady et al. 2004), and (d) hysteretic curves for a specimen before and after retrofit using shotcrete (Abrams and Lynch 2001)*

Surface treatment is a common method, which has largely developed through experience. Surface treatment incorporates different techniques such as ferrocement, reinforced plaster, and shotcrete. By nature this treatment covers the masonry exterior and affects the architectural or historical appearance of the structure.

Ferrocement

Ferrocement consists of closely spaced multiple layers of hardware mesh of fine rods (Figure 5.2.7.27 (a)) with reinforcement ratio of 3-8% completely embedded in a high strength (15-30 MPa) cement mortar layer (10- 50 mm thickness). The mortar is troweled on through the mesh with covering thickness of 1-5 mm. The mechanical properties of ferrocement depend on mesh properties. However, typical mortar mix consists of 1 part cement: 1.5-3 parts sand with approximately 0.4 w/c ratio (the ferrocement network, Montes and Fernandez 2001). The behavior of the mortar can be improved by adding 0.5-1% of a low-cost fiber such as polypropylene. Ferrocement is ideal for low cost housing since it is cheap and can be done with unskilled workers. It improves both in-plane and out-of-plane behavior. The mesh helps to confine the masonry units after cracking and thus improves in-plane inelastic deformation capacity. In a static cyclic test (Abrams and Lynch 2001), this retrofitting technique increased the in-plane lateral resistance by a factor of 1.5. Regarding out-ofplane behavior, ferrocement improves wall out-of-plane stability and arching action since it increases the wall height-to-thickness ratio.

Reinforced plaster

A thin layer of cement plaster applied over high strength steel reinforcement can be used for retrofitting (Sheppard and Tercelj 1980). The steel can be arranged as diagonal bars or as a vertical and horizontal mesh. A reinforced plaster can be applied as shown in Figure 1 (b). In diagonal tension test and static cyclic tests, the technique was able to improve the in-plane resistance by a factor of 1.25-3 (Jabarov et al. 1980, Sheppard and Tercelj 1980). The improvement in strength depends on the strengthening layer thickness, the cement mortar strength, the reinforcement quantity and the means of its bonding with the retrofitted wall, and the degree of masonry damage.

Shotcrete

Shotcrete overlays are sprayed onto the surface of a masonry wall over a mesh of reinforcing bars (Figure5.2.1.6.32 (c)). Shotcrete is more convenient and less costly than castin-situ jackets. The thickness of the shotcrete can be adapted to the seismic demand. In general, the overlay thickness is at least 60 mm (Abrams and Lynch 2001, Tomazevic 1999, Karantoni and Faradis 1992, Kahn 1984, Hutchison et al. 1984). The shotcrete overlay is typically reinforced with a welded wire fabric at about the minimum steel ratio for crack control (Karantoni and Fardis 1992). In order to transfer the shear stress across shotcrete-masonry interface, shear dowels (6-13 mm diameter @ 25-120 mm) are fixed using epoxy or cement grout into holes drilled into the masonry wall (Abrams and Lynch 2001, Tomazevic 1999, Karantoni and Faradis 1992, Kahn 1984). Other engineers believe that a bonding agent like epoxy is required to be painted or sprayed on the brick so that adequate brick-shotcrete bond is developed (Kahn 1984). However, there is no consensus on brick-to-shotcrete bonding and the need for dowels. Diagonal tension tests of single and double wythe URM panels (Kahn 1984) retrofitted with shotcrete showed that, dowels did not improve the composite panels response or the brick-shotcrete bonding; header bricks satisfactory joined the wythe of existing masonry panels. In addition, Tomazevic (1999) and Kahn (1984) recommended wetting the masonry surface prior to applying shotcrete. Kahn (1984) shows that such brick surface treatment does not affect significantly the cracking or ultimate load, it affects to limited extend the inelastic deformations. Retrofitting using shotcrete significantly increases the ultimate load of the retrofitted walls. Using a one-sided 90 mm thick shotcrete overlay and in diagonal tension test, Kahn (1984) increased the ultimate load of URM panels by a factor of 6-25. Abrams and Lynch (2001), in a static cyclic test, increased the ultimate load of the retrofitted specimen by a factor of 3. This retrofitting technique dissipates high-energy due to successive elongation and yield of reinforcement in tension (Figure 1(d)). Although in diagonal tension test (Kahn 1984) the improvement in the cracking load was very high, in static cyclic test (Abrams and Lynch 2001) the increment in the cracking load was

insignificant. Typically, the shotcrete overlay is assumed to resist all the lateral force applied to a retrofitted wall with the brick masonry being neglected all together (Abrams and Lynch 2001, Hutchison et al. 1984). This is reasonable assumption for strength design since the flexural and shear strength of the reinforced shotcrete overlay can be many times more than that of the URM wall. This assumption may result in some cracking of the masonry as the reinforcement in the shotcrete strains past yield. This may violate a performance objective for immediate occupancy or continued operation.

External reinforcement

A steel plates or tubes can be used as external reinforcement for existing URM buildings. Steel system is attached directly to the existing diaphragm and wall (Figure 5.2.1.7.28(a)); however, Rai and Goel (1996) show that horizontal element can be connected to

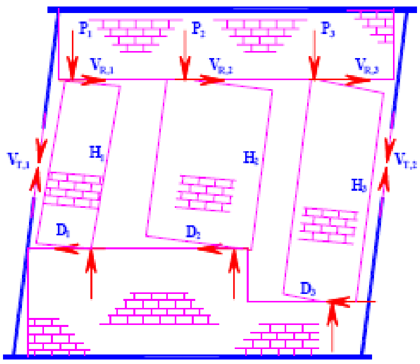


***Figure 5.2.7.28 (a)** Using vertical and diagonal bracing (Taghdi 2000)*

***(b)** Creating in-fill panel (reproduced after Rai and Goel 1996)*

The vertical and diagonal bracing improves the lateral in-plane resistance of the retrofitted wall by a factor of 4.5 (Taghdi 2000). The increment in the lateral resistance was limited by crushing of the masonry at ends (toes) followed by vertical strips global buckling. In the case of creating infill panel, the rocking motion of the pier is associated with a vertical movement of its corner butting against the support masonries and the steel verticals resist the motion by restringing this vertical movement. This mechanism put both vertical members under tension forces (Figure 5.2.1.6.33 (b)). The system increased the in-plane lateral resistance of the retrofitted wall by a factor of 10. In addition, the external steel system provides an effective energy dissipation mechanism (Taghdi 2000, Rai and Goel 1996).

5.2.9 Masonry Cladding to Iron and Steel Framed Buildings

Many city buildings from the end of the nineteenth and the first half of the twentieth century appear at first glance to be of traditional masonry construction. Closer investigation will however show that the structure is a frame with columns (‘stanchions’) and beams of wrought iron or steel.

Common Arrangements of Cladding

In the earlier buildings with this type of structure, the façade columns had brickwork built around them and between the columns; brickwork spandrel walls were supported on the beams. The brickwork may, in turn, have a bonded cladding of stone or, in the later buildings, the stone cladding is built directly against the steel stanchions, supported on the beams and sometimes tied back to the frame with metal cramps (Fig. 5.2.9.1). From the late 1930s, it became increasingly common to encase the steelwork in concrete in order to provide fire resistance and corrosion protection. Terra cotta was sometimes used for these buildings in a similar way as stone; as a cladding bonded to a brick backing and on some decorative features like domes, the hollow terra cotta blocks were directly supported by and tied to the members of a metal skeleton forming the structure of the ‘dome’.

Problems and their Diagnosis

When no concrete encasement was provided, the steelwork would usually have had some paint protection, but this would often be little more than a coat of ‘red oxide’ applied at the works and touched up on site. In the main, reliance was placed on the masonry cladding and the roof to protect the steel from the elements and thus prevent corrosion. Inadequate gutter details and erosion of masonry mortar joints, aggravated by poor maintenance, will however over the years allow water to get at the steel. Corrosion will start where the paint film, if any, has been damaged during the cladding construction and will then spread from there. Rust can propagate under a seemingly intact paint film. The severity of the corrosion is not always related to inadequacy of the painting. Water, getting in at high level, may run down the stanchions without causing much rusting, but where the vertical flow is impeded by cleats and beams, etc., the water collects and causes severe rusting. The corrosion rarely causes significant loss of strength of the frame. It can, however, have serious consequences for the masonry cladding for the following reasons. Rust, once formed, encourages the corrosion of the underlying metal; it also occupies a volume that is between 6 and 10 times as large as the volume of iron from which it is created. As a result, the rust will form a crust that will increase in thickness with time and exert an outward pressure on the masonry cladding. This pressure leads to cracking of the masonry, usually along lines parallel to and within the width of the steel or iron members. In some case, entire stones or brick courses are displaced, but usually the cracking disrupts the masonry units themselves, particularly when, as in the case of stone, the units are shaped with internal sharp re-entrant corners to fit round the flanges of the steel sections.

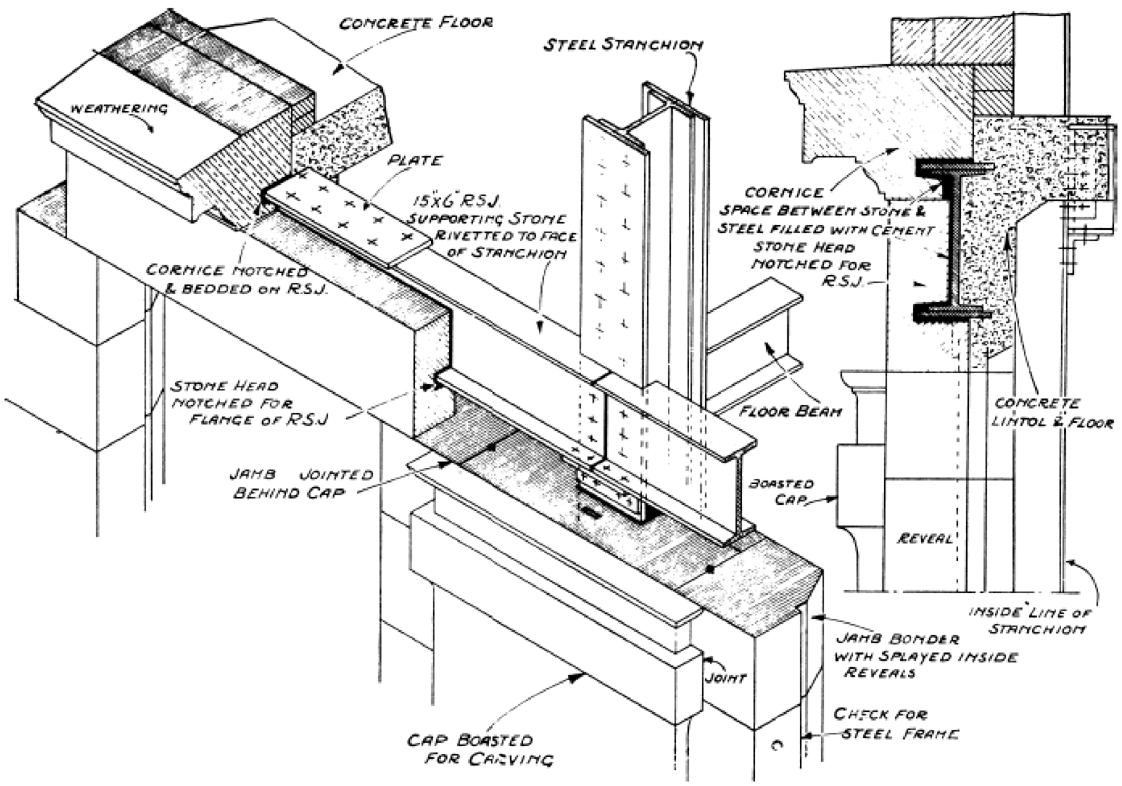


Figure 5.2.9.1 Details of masonry cladding to steel-framed building (*from Warland, 1953*)

The diagnosis is simple in principle; once it is established that the building has an iron or steel frame, cracks on the façade along the lines of the metal frame members behind the face indicate serious corrosion underneath. The extent of the trouble is, however, far more difficult to ascertain, as absence of cracks does *not* prove absence of rust. The reason for this is that due to the inaccuracies in the relative alignment of the frame relative to the cladding, the cladding will in some areas have been built hard up against the frame, whereas in other places there may be a gap, 10–20 mm wide, between the back of the cladding and the frame member.

In the first case, the rust will initially have filled any porosity and voids in the mortar bedding of the cladding and the pressure will then have built up until cracking occurred. In the second case, the rust will have been able to grow into the gap without exerting any pressure on the cladding, which will therefore, on inspection, show no cracking. If, however, at the time of inspection, the rust has just filled the gap, it may only be a few years before the pressure builds up enough for cracking to start. These two situations can exist on the same façade within a metre or two from each other. The extent of the corrosion attack on the frame, and hence the amount of remedial work necessary, can therefore *not* be assessed from an inspection of the face only, but requires removal of the cladding. It is quite common to find cracking, indicative of corrosion, on only 10–20 per cent of the facades of a building. If then the building owner insists on a guarantee that no further work will be required for the next 25 years, it means dismantling the whole of the cladding. Such dismantling inevitably causes damage. Replacement material, to match the existing, may be difficult to obtain in the quantities required, and the cost of such an operation is usually out of proportion to the benefit.

A less drastic and far more economical approach is to remove the cladding where it is cracked and from adjacent parts of the frame, as far as rust is observed, to treat the rusted metal as described below and reinstate the cladding. Financial provision should then be made to cover the possibility that isolated areas, where rust was not discovered first time, may have to be dealt with in 5 or 10 years time.

Repair Techniques

Normal repair and/or replacement techniques will be used for the masonry with special attention being paid to achieving weatherproof joints. For the exercise to be successful, it is however essential that the corroded metal surfaces are properly treated. *All* rusted surfaces must be exposed, not just those facing the cladding; this may entail extensive removal of brickwork surrounding the frame members. All rusted surfaces must then be given radical remedial treatment against corrosion. The treated surfaces must be protected with an abrasion resistant paint or by heavy-duty self-adhesive tape so as to prevent breaching of the paint system during the reinstatement of the masonry cladding.

5.3 Innovative Techniques

5.3.1 General

In additional to the traditional techniques, the repair, consolidation and restoration of historic buildings requires several other techniques which may be described as innovative techniques.

5.3.2 Prestressing & Post Tensioning

Masonry has a relatively large compressive strength and a low tensile strength.

This is the main reason why masonry has been used so far primarily as a construction material for vertical members subjected essentially to gravity loads. Apart from this principal action, however, in-plane shear and out-of-plane lateral loads as well as imposed deformations caused by deflections and volume changes of floor slabs may be applied to masonry walls. Small lateral loads and deformations may be resisted due to the weight of the walls. However, for larger lateral loads, walls with low axial loads exhibit a poor cracking behavior and a low strength. These drawbacks may be eliminated by post-tensioning the masonry. Post-tensioning introduces the desired level of axial load in a wall to enhance the strength, performance, and durability of masonry structures. The prestressing steel helps avoiding brittle tensile failure modes of masonry walls .

5.3.3 History of Prestressing & Post-Tensioning

The first application of post-tensioning is believed to have been conceived by Eugene Freyssinet in 1933 for the foundation of a marine terminal in France. The technology was introduced to the United States in 1950 in the Walnut Lane Bridge in Philladelphia, PA. Post-tensioning is now used extensively in bridges, elevated slabs (parking structures and residential or commercial buildings), residential foundations, walls, and columns and in strengthening existing structures .

The idea of post-tensioning of masonry is not new. In 1825 a post-tensioning method for tunnelling under the River Thames was utilized in England. The project involved the construction of vertical tube caissons of 15m diameter and 21 m height. The 0.75m thick brick walls were reinforced and post-tensioned with 25mm diameter wrought iron rods. Since the 1960's research on, and a number of applications of, prestressed masonry have been reported primarily in England [2,3,4]. Applications include a prestressed masonry watertank, retaining walls, large walls in buildings and even road and railway bridge abutments.

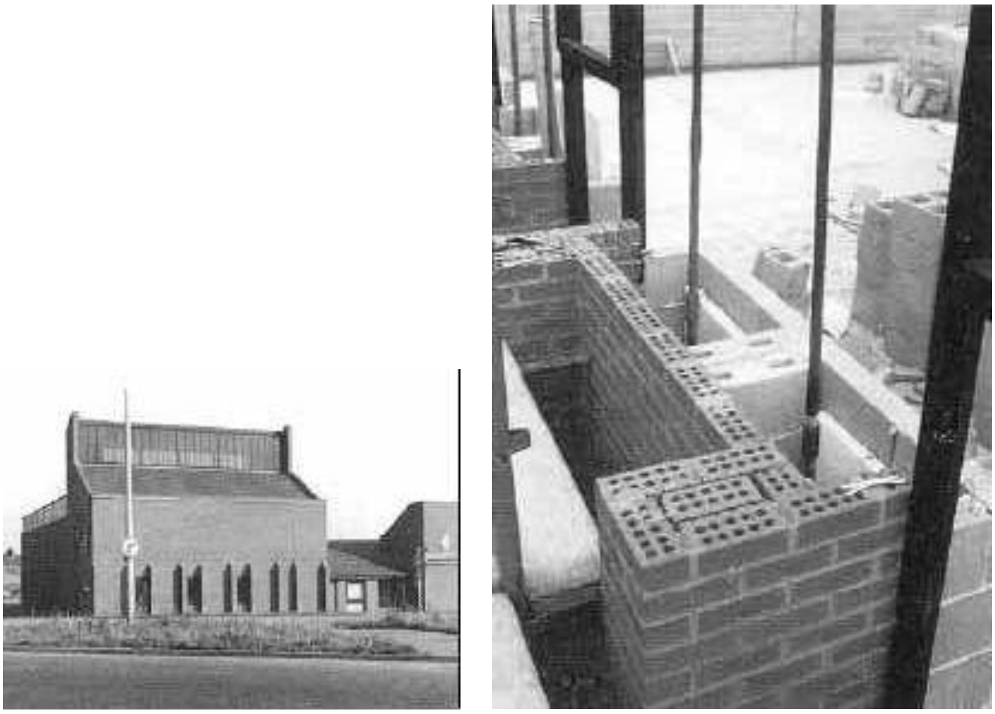


Figure 5.3.3.1 *Salvation Army Hall, [23], Photograph courtesy of Curtins Consulting Engineers* *Wall Section*

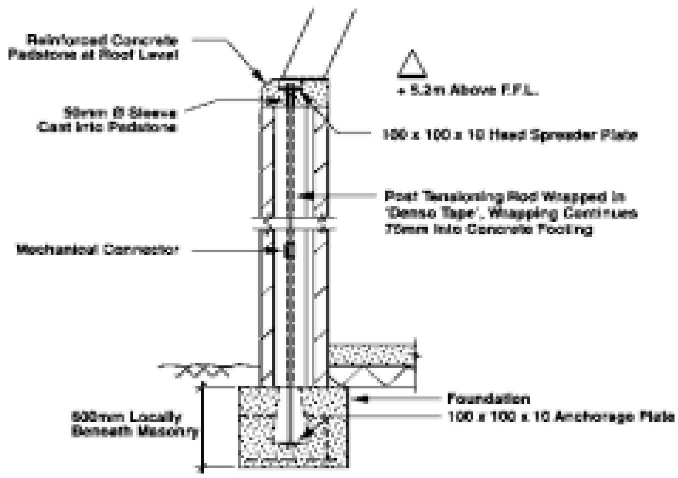


Figure 5.3.3.2 *Wall Section Orsborn Memorial Hall, [24], Courtesy of Curtins Consulting Engineers*

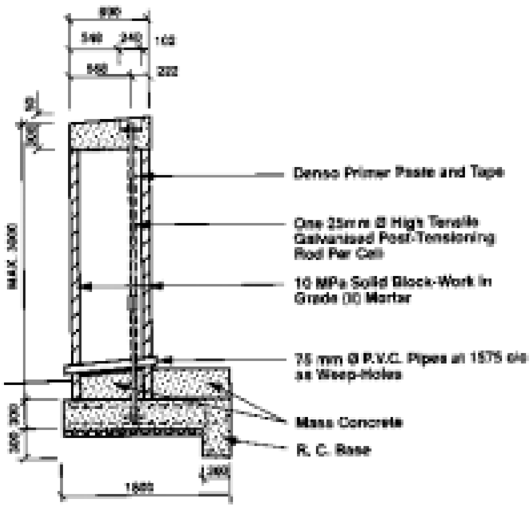
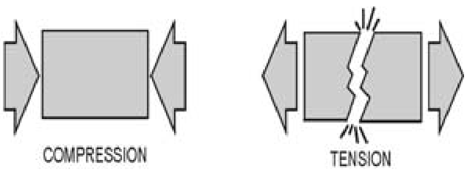


Figure 5.3.3.3 *Retaining Wall Section, [24], Courtesy of Curtins Consulting Engineers*

What is Post-Tensioning

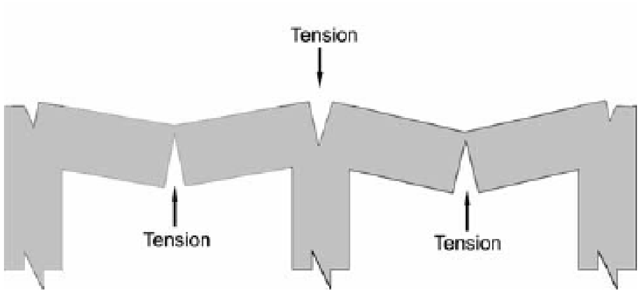
Post-tensioning is a method of producing *prestressing* into concrete, masonry and other structural elements. The term *prestressing* is used to describe the process of introducing internal forces (or stress) into a concrete or masonry element during the construction process in order to counteract the external loads that will be applied when the structure is put into use (known as *service loads*). These internal forces are applied by tensioning high strength steel, which can be done either before or after the concrete is actually placed. When the steel is tensioned before concrete placement the process is called *pre-tensioning*. When the steel is tensioned after concrete placement the process is called *posttensioning*. The advantages of utilizing prestressed concrete and masonry have long been recognized by engineers. When a designer wants to take advantage of those benefits, they must determine whether the structure is to be constructed using the pre-tension method or the post-tension method. Pre-tensioning is generally accomplished at a manufacturing facility where concrete members are constructed in special casting beds with steel bulkheads that hold the steel in place while tension is applied. Concrete is then placed around the pre-tensioned steel and allowed to harden. The steel is then cut loose from the bulkheads and the entire *precast concrete* member is transported to the project site for assembly. This process may be limited to the use of standard shapes, and sizes that can be easily transported. Post-tensioning is done at a project site and requires little to no modifications of the same forming system that would be used to construct non-prestressed concrete. The systems used to post-tension concrete and masonry consist of prestressing steel that is housed inside a duct or sheath, which allows the prestressing steel to be placed inside the typical job site formwork at the same time rebar and other reinforcing is placed. Concrete is placed in a typical manner and allowed to reach a predetermined strength before the steel is tensioned. Since the prestressing steel is housed in the sheathing or duct, it will be free to move inside the concrete during the tensioning operation, and since the steel is tensioned after concrete placement, the tensioning is done against the hardened concrete instead of relying on large steel bulkheads. Using the post-tensioning method of prestressing enables to get all the advantages of prestressed concrete or masonry while still enabling the freedom to construct the member (slab, wall, column, etc,) on the job site in almost any shape or configuration imaginable.



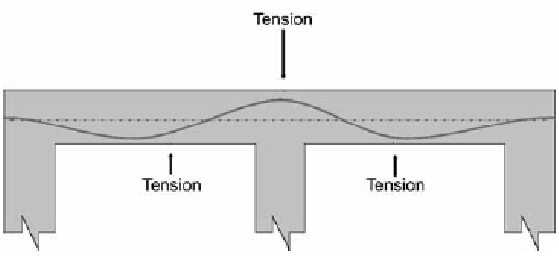
What Does Post-Tensioning do to Concrete and Masonry?

When a concrete or masonry element is prestressed, it means that the steel is being tensioned and the concrete or masonry is being compressed. *Compression* is a force that squeezes or crushes and *tension* is a force that pulls something apart. As building materials, concrete and masonry are very strong in compression, but they are relatively weak in tension. Steel, on

the other hand, is very strong in tension. Putting the concrete or masonry into compression and the steel into tension before any substantial service loads are applied puts both of these building materials into their strongest states. The result is a stiffer concrete or masonry member that is being actively compressed and has more capacity to resist tensile forces.



One of the things that happens to a concrete floor, or a masonry wall, is that they are subjected to forces that cause them to flex and bend. Examples of this include slabs on ground where the edges of the slab are forced upward by swelling soils, elevated concrete slabs where gravity and other applied loads pull down on the slab in between supports, and walls that might be subjected to lateral forces from wind or seismic activity. This bending creates high tensile forces that can cause the concrete and masonry to crack. This is where the use of reinforcing is applied. Since steel has a high capacity to resist tensile forces, it can be embedded in the concrete at the *tension zones* (the areas that tensile failures could occur) allowing the tensile forces to be handled by the reinforcing steel.



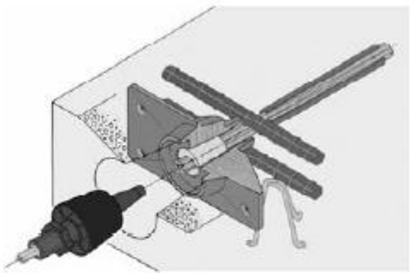
Adding post-tensioned reinforcing combines the action of reinforcing the tension zones with the advantages of compressing the concrete or masonry structure.

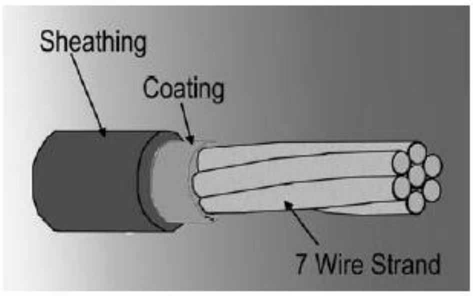
Masonry has a relatively large compressive strength but only a low tensile strength. Therefore, masonry has been used so far primarily as a construction material for vertical members subjected essentially to gravity loads. Apart from this principal action, however, in-plane shear and out-of-plane lateral loads as well as imposed deformations caused by deflections and volume changes of floor slabs may be applied to masonry walls. Small lateral loads and deformations may be resisted due to the weight of the walls. However, for larger lateral loads, walls with low axial loads exhibit a poor cracking behavior and a low strength. To overcome these disadvantages, masonry may be post-tensioned. Post-tensioning offers the possibility to actively introduce any desired level of axial load in a wall to enhance strength, performance, and durability of masonry structures. The prestressing steel helps avoiding brittle tensile failure modes of masonry walls and offers major advantages for the connection of vertical and horizontal members in precast construction. Existing structures may be strengthened by prestressing to comply with recent code requirements for lateral loading; in particular, seismic areas.

Materials and Equipment used in Post-Tensioning?

The basic element of a post-tensioning system is called a *tendon*. A post-tensioning tendon is made up of one or more pieces of prestressing steel, coated with a protective coating,

and housed inside of a duct or sheathing. A tendon will have anchors on each end to transmit the forces into the structure. Long tendons may also have intermediate anchors along their length. The prestressing steel can be a high strength steel strand (typical in horizontal applications) or a high strength steel bar (typical in vertical applications). Prestressing steel is manufactured to applicable ASTM requirements. Typical strand sizes are 0.50 in. and 0.60 in. diameters, and bar sizes can typically range from 1 in. to 2.5 in.





Tension is applied to prestressing steel by using a *hydraulic stressing jack*. The jack bears against one of the anchors that is embedded in the concrete and pulls the steel to a predetermined force. As the tensioning is occurring, the steel is being elongated, and the concrete or masonry element is being compressed. When the proper tensioning force is reached, the prestressing steel is anchored in place. The anchors are designed to provide a permanent mechanical connection, forever keeping the steel in tension, and the concrete in compression.

The duct or sheathing that houses the prestressing steel provides one layer of corrosion protection. A tendon with a duct that contains multiple pieces of prestressing steel strand is commonly called a *multistrand tendon*, and a tendon in which a single prestressing steel strand is covered in a plastic sheathing is commonly referred to as a *monostrand tendon*. Inside the duct or sheathing, the prestressing steel is covered in a protective coating that provides another level of corrosion protection. This coating can be a specially formulated type of grease, or it can be a specially designed type of grout. When grease is used, the prestressing steel is permanently free to move relative to the sheathing and the tendon is referred to as an *unbonded tendon*. When grout is used, the steel is permanently bonded to the sheathing and is referred to as a *bonded tendon.*

The hydraulic jacks used to tension the post-tensioning tendons range from compact 60 lb jacks used to stress monostrand tendons and small bar tendons, to very large jacks requiring special hoisting equipment, used to simultaneously tension all of the strands in large multistrand tendons.

5.3.4 Strengthening of Historic Buildings with Post-Tensioning

Post-tensioning of masonry structures means the combination of a relatively new technique with an old material for strengthening purposes which requires several issues to be considered. The major concern is the correct evaluation of the properties of masonry. Masonry structures have been known to last for thousands of years, yet many older structures are condemned because they do not meet current loading requirements or their durability is questioned, often because of improper maintenance practices in the past. Masonry is not a homogeneous material which lends itself to simple elastic analysis which generations of engineers have used for steel and concrete. New methods of analysis enable the full potential of masonry to be realized.

Although several methods exist for determining the real characteristic of masonry, it is not possible to use properly each of them in any situation. Another concern relates to the problem of the lack of exact rules and formulas for the design of prestressed tendons.

The codification of post-tensioning masonry has only begun recently (e.g. MSJC 1999). Much research has been conducted in the last decade on post-tensioning masonry worldwide (e.g. Lissel and Shrive 2003, Rosenboom and Kowalsky 2003, Schultz et al. 2003, Laursen et al. 2002).

The successfull application of post-tensioning rely on the proper execution of the following:

- Determination & assesment of the real properties of the masonary and the building
- Appropriate design according to the relevant codes
- Proper application of post-tensioning

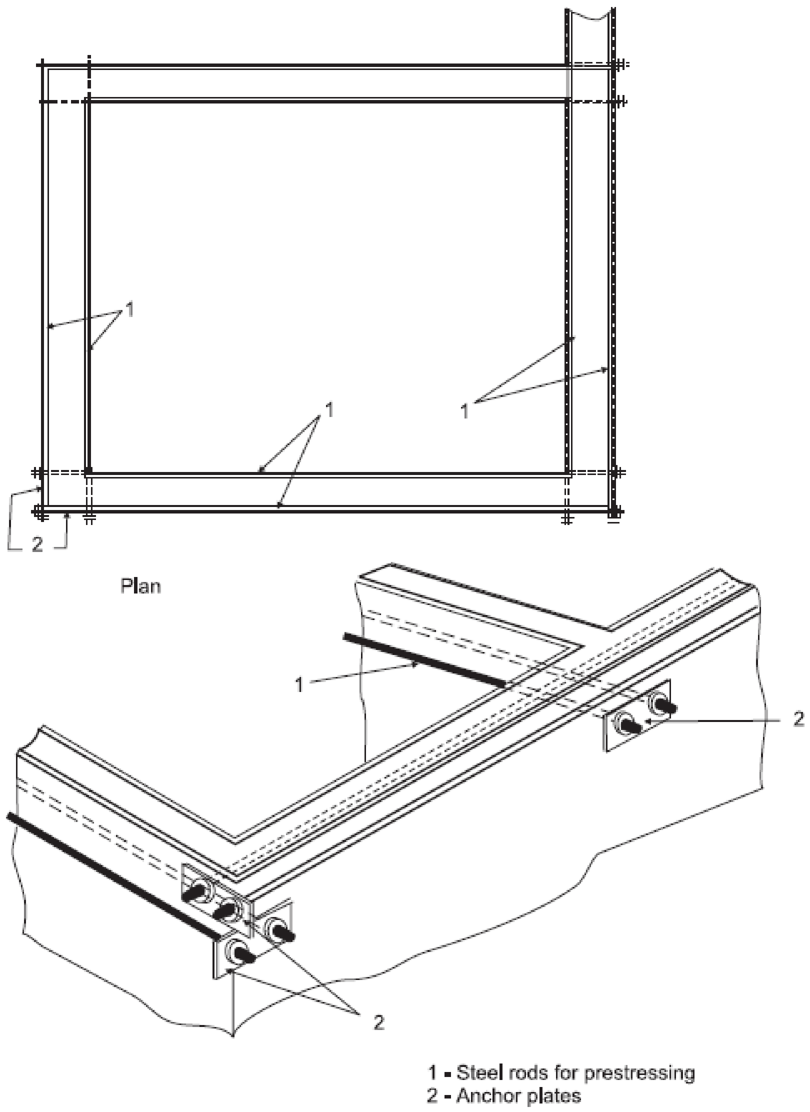


Figure 5.3.3.4 *Strengthening of walls by prestressing*

A Horizontal compression state induced by horizontal tendons can be used to increase the shear strength of walls. Moreover this will also improve considerably the connections of orthogonal walls, Figure 5.2.2.4 The easiest way of affecting the precompression is to place two steel rods on the two sides of the wall and strengthening them by turnbuckles. Note that good effects can be obtained by slight horizontal prestressing (about 0.1 MPa) on the vertical section of the wall. Prestressing is also useful to strengthen spandrel beam between two rows of openings in the case no rigid stab exists.

External binding

Opposite parallel walls can be held to internal cross walls by prestressing bars as illustrated above, the anchoring being done against horizontal steel channels instead of small steel plates. The steel channels running from one cross wall to the other will hold the walls together and improve the integral box like action of the walls. The technique of covering the wall with steel mesh and mortar or micro-concrete may be used only on the outside surface of external walls but maintaining continuity of steel at the corners. This would strengthen the walls as well as bind them together. As a variation and for economy in the use of materials, the covering may be in the form of vertical splints between openings and horizontal bandages over spandrel walls at suitable number of points

only, Figure 5.2.2.5

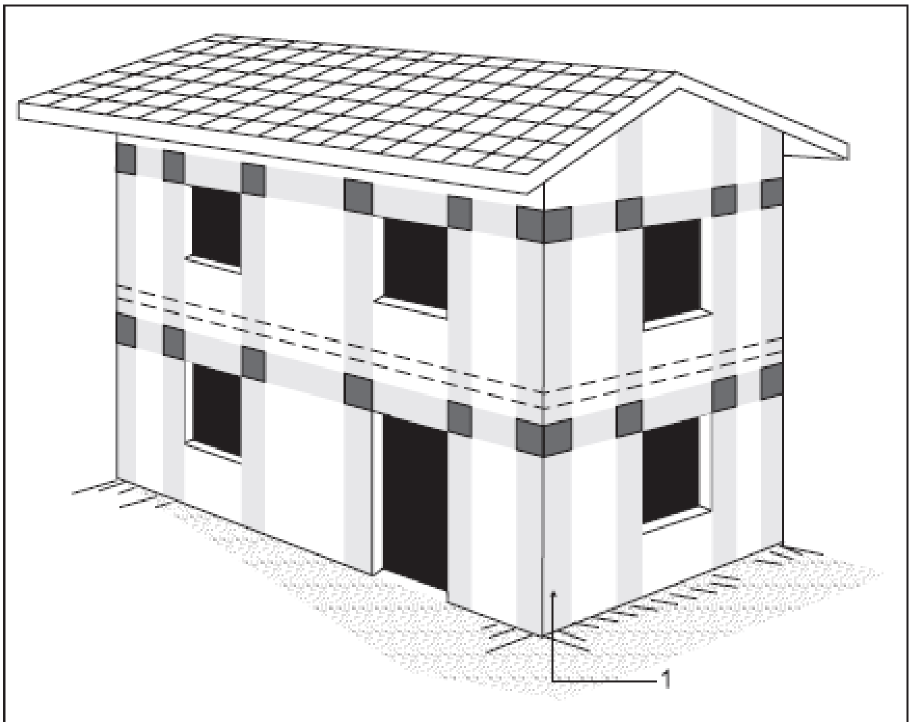


Figure 5.3.3.5 Splint and bandage strengthening technique

Anchorage of post-tensioning in masonry is more complicated than in r.c. as masonry has a relatively low compressive strength. The self-activating dead end can be encasing to continuous and heavy r.c. foundation beams, constructed on either side of the wall bottom and connected well with it. At the top, post-tensioning is anchored in the existing r.c. elements (Figure 5.3.3.5 (c)) or in a new precast r.c. special beam or specially stiffened steel plates. Anchorage devices and plates are usually placed in a

recess of the surface, and covered later on with shotcrete or cement mortar. The requirement for bottom anchorage penalizes considerably this retrofitting technique. Vertical post-tensioning resulting in substantial improvement in wall ultimate behavior for both in-plane and out-of-plane; in addition, it improves both cracking load and distribution. Rosenboom and Kowalsky (2003) show that for cavity walls, the posttension grouted specimen has lateral resistance much higher (40%) than the ungrouted one. For grouted specimens, although to bond or not the bars have insignificant effect on lateral resistance; the specimen who has un-bonded bars has higher lateral drift (70%) over the bonded specimen. The un-bonded grouted specimen has a drift up to 6.5%. However, un-bonded post-tension tendons may show low energy dissipation due to the lack of yielding of reinforcement (VSL 1990).

The effect of horizontal post-tensioning needs extensive experimental examination. Although some basic calculations of principle stresses show that the horizontal posttensioning improves the resistance, Page and Huizer (1994) experimental test did not proof these calculations. In a linear finite element model, Karantoni and Faradis (1992) show that horizontal post-tensioning of spandrels did not significantly improve the building behavior. In addition, they show that if horizontal and vertical post-tensioning is combined together the resulting positive effect is higher than the sum of the

individual effects of the two directions post-tensioning. For bonded grouted post-tensioning the ultimate tendon force may be determined assuming rigid bond and plane sections similar to design of r.c. post-tensioning. Thus, the tendon will reach their yield force. For un-bonded post-tensioning the tendon force will increase from service up to ultimate load depending on the deformations. This

increment in the tendon force may be estimated by applying rigid mechanisms. For short time behavior and under the same post-tensioning force, strand configuration and amount has insignificant effect on wall behavior (Al-Manaseer and Neis 1987).

(a) (b) (b)

Figure 5.3.3.5 (a) *post-tensioning using FRP,* ***(b)*** *flexural crack in post-tension wall,* ***(c)*** *posttensioning jacking frame (ISIS)*

Table 5.3.3.1 below summarizes the efficiency, advantage, and disadvantage of different techniques some of them involving steel & iron explained before. Where available, figures from static cyclic or dynamic tests are given to indicate the improvement in the retrofitted walls. [84]

Tech.	Efficiency		Advantage	Disadvantage
	In-plane	Out-of-plane		
Ferrocem-ent	$F_r \rightarrow 1.5\ F_{ur}$ $D_r \rightarrow 1.7\ D_{ur}$	Improves stability	Low cost Low technology Limited added mass	Space reduction Arch. Impact Requires arch. finishing Limited efficiency Limited E.D.
Reinforc-ed Plaster	$F_r \rightarrow 2\text{-}3\ F_{ur}$ Improves D_r	Improves stability	Low technology Limited added mass	Space reduction Arch. Impact Required arch. finishing
Shotcrete	$F_r \rightarrow 3\ F_{ur}$ $D_r \rightarrow D_{ur}$	Improves stability	High increment in F_{ur} Very significant improvement in E.D.	Space reduction Heavy mass Violation of perform. level Disturbance occupancy Arch. Impact Required arch. finishing
Injection	Restores initial stiffness $F_r \rightarrow 0.8\text{-}1.4\ F_{ur}$	Can restores initial stiffness	No added mass No effect on building function No space reduction No arch. Impact	Epoxy create zones with varying stiffness and strength High cost of epoxy No significant increment in F_r using cement-based grout
External Reinforc-ement	$F_r \rightarrow 4.5\text{-}10\ F_{ur}$ $E.D_r. > 1.5$ $E.D_{ur}$.	N.A.	High increment in F_{ur} Prevent disintegration Improves ductility and E.D.	Corrosion Heavy mass Violation of performance level Requires arch. Finishing Disturbance occupancy
Confinem-ent	$F_r \rightarrow 1.2\text{-}1.5\ F_{ur}$ $D_r \rightarrow D_{ur}$	Prevent disintegration	Prevent disintegration Improve ductility and E.D.	Not easy to introduce Limited effect on F_{ur} Required arch. finishing Disturbance occupancy
Post-tension	Improves F_{ur}	Improves F_{ur}	No added mass No effect on building function	High losses Anchorage system Corrosion potential
Center Core	$F_r \rightarrow 2\ F_{ur}$ $D_r \rightarrow 1.3\text{-}1.7\ D_{ur}$	Improves F_{ur}	No space reduction No arch. Impact No effect on building function	Creation of zones with varying stiffness and strength.

F_r, F_{ur}: lateral resistance for retrofitted and unretrofitted specimens respectively, D_r, D_{ur}: lateral displacement for retrofitted and unretrofitted specimens respectively, E.D.: energy dissipation

5.3.5 Use of Shape Memory Alloy Devices and Shozk Transmitters for Seismic Protection of Monument

Conventionally, the most common method that has been applied in the past to enhance the behaviour of monumental structures under service and dynamic conditions is the localised reinforcements, usually steel bars or cables. These intervention techniques increase considerably the overal capacity of the structures by providing the necessary links between the discountinous parts of the structures thus conferring increased ductility. Masonary buildings are vulnerable not only to seismic ground motions but also to ambient vibrations caused by traffic, wind, bells, etc. In many cases, these conventional intervention techniques are not sufficient to prevent collapse.

Although Santa Sofia Church is located in Padova which is not of high seismic risk, considering the fact that some monumental structures have recently suffered severely during earthquakes of lower intensity than those in the past centuries we can expect that the curch would be damaged by an earthquake of lower intensity. Historic masonary buildings are highly vulnerable due to cumulative damages from the past earthquakes and due to ageing which progressively reduces the strength of the materials. Seismic resistance has sometimes been decreased by wrong restoration interventions as well. Considering the curreent situation of Santa Sofia, which had to be strengthened to protect the building from collapse under static loads, it may be worth to consider innovative intervention techniques which proved to be very effective to protect the structures under both static and dynamic loads.

The use of devices based on superelastic shape memory alloy devices has proved very effective in improving the seismic resistance of masonary structures. The performance of such types of devices were investigated for different sructural aspects e.g. Shape Memmory Alloy Devices (SMADS) can be used to prestress the masonary while at the same time avoiding the overstressing due to the force limitation offerred by the superelastic plateau of the alloys. Other types of SMADS become active only during dynamic actions and they do not imply any load transmittion to the masonary structure. These types of SMADS can be used as horizontal restraints to enhance the out-of-plane seismic strength of outside masonary walls like the facades of the curches. One important characteristic of SMADS is that they are custom designed taking into account the specific characteristics of the monuments.

The results of EC funded ISTECH Project which aimed at developing innovative techniques for the restoration of cultural heritage structures shows through numerical models and intensive tests that SMADS can increase substantially the stability of masonary structures. The out-of plane shaking table tests on masonary mock-up displays that structure equipped with SMADs remains unharmed exposed to an earthquake of at least 50% higher intensity compared to the one reinforced with conventional steel ties. Pseudo-dynamic test on other masonary mock-ups subjeected to in-plane seismic excitations also show significant increase in the seismic resistance of the mock-ups equipped with SMADs.

These successful results led to two restoration applications by utilizing this innovative technique: The Bell Tower of the S. Giorgio of Trignano Church in S. Martino in Rio which is damaged during the October 1996 earthquake, and the transept tympana of the Basilica of St. Francesco in Assisi, which was damaged during the September 1997 earthquake.

Shape memory alloy devices (SMAD) which have been developed specifically for the use in ancient masonry structures are axial devices exploiting the superelastic properties of shape memory alloys used in the form of small diameter wires **SMADs can be substituted for conventional steel ties to connect different structural elements, e.g. façades to floors or the roof.** The advantage accrued over the use of steel ties is mainly due to their force limitation capacity. For example, when used as horizontal ties to connect façade

walls to roof, SMADs permit controlled displacements and can limit transmitted forces, according to the device constitutive law. Figure 5.3.5.1 shows an example of such a law for a device of the multi-plateau type with 3 plateaux. The design engineer can select two or more force levels and corresponding displacements; these are then realized using groups of shape memory alloy wires with different lengths, properly connected within the device.

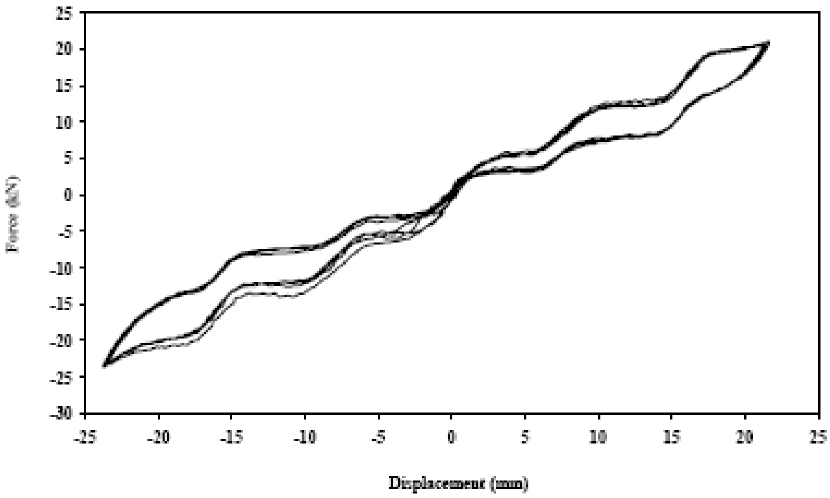


Figure 5.3.5.1 *- Force vs. displacement loops measured on a SMAD during a sinusoidal test (amplitude -+ 24 mm, frequency 2 Hz*

Shock transmission units (STU) have been widely used in the last 30 years in new structures – mainly bridges and viaducts or prefabricated buildings – as dynamic or temporary restraints. They are piston/ cylinder devices that utilize fluid flow through orifices to provide for a reaction that is the result of a pressure differential across the piston head and is function of the velocity applied to the mentioned piston. STU are designed to work as a safety belt for a car driver: at the occurrence of a dynamic movement exceeding the so called “activation velocity” (induced for example from an earthquake) they react as a very stiff -temporary - restraint, whilst for very slow movements, such those imposed by thermal expansion, they do not provide for a major reaction. During the last decade, STU have also been applied in the restoration of some historical masonry buildings. The first of these applications was effected in the church of San Giovanni Battista in Carife, Italy, which was heavily damaged by the Irpinia of the damage included the complete collapse of the old timber roof. A new steel truss roof was built and shock transmission units were used to connect the new roof to the masonry walls. This was done to keep roof thermal elongations from inducing high stresses on the walls and, at the same time, guarantee a situation where the roof can share the seismic actions with all the walls during a seismic attack (owing to the roof high in-plane stiffness and temporary stiff connections to the walls given by the STU). The most recent application of shock transmissionunits in monuments was implemented at the Basilica of San Francesco in Assisi, Italy, and is described in the chapter of “Case Studies”.

Figure5.3.5.2 *Shape memory alloy devices installed in the Basilica of San Francesco in Assisi.*

Figure5.3.5.3 *- A pair of 220 kN shock transmission units installed in the Basilica.*

5.3.6 Shoring

Shoring is a temporary work designed to resist stresses arising from subsidince, overtoppling or busting and lanslip. Although it is a temporary work and therefore the contractors’s primary responsibility the architect and engineer should check the design and be prepared to comment upon it. Judgement and experince is essential to avoid any accident which may cause loss of life and damage to the monument needing to be preserved.

There are three types of shoring: Dead shoring which takes the vertical loads while enabling walls, columns and piers to be rebuilt, or propping. Horizontal shoring or strutting prevents bursting and landslip. The third one is diagonal or raking shoring which can prevent bursting and landslip and overtopping It is highly recommended and desirable to tighten up or prestress shoring to its calculated loading to prevent cracks or other damages. The prestressing is done by jacks of suitable type, either screw or hydraulic. Overtightneing the wedging or over prestressing should be avoided not to cause any damage to the old building one is seeking protection.

Figure 5.3.6.1 Shoring, east end, York Minster, England
(Courtesy: Shepherd Building Group Ltd)
Raking shores support the wall which is leaning outwards and near collapse. Flying shores act as horizontal struts between the raking shores. The whole wall had to be kept intact in order to avoid damage to the famous early fifteenth century Alpha and Omega window which is as large as a tennis court

A refinement of jacking was devised at York Minster where flat-jacks were placed under the bases of 26 m (85 ft) long steel raking shores. These jacks were linked to a constant pressure mechanism which applied a horizontal force of 20.3 t (20 tonf). The shores were able to accommodate movement in the building and the jacks also absorbed the thermal movements in the shores themselves with atotal expansion and contraction of about 30 mm (1.2 in). When inserting shores, needles are necessary to transfer the stresses to the masonry, and as has been said it is often useful to find the originalputlog holes, if they exist. Shores should always beremoved carefully; if they have prestressing jacks,the procedure is greatly simplified as the pressurein the jacks can be lowered in stages.

5.3.7 Drilling

Drilling vertically to big depths through homogeneous materials is a comparatively simple and well established practice, but accurate drilling horizontally through masonry is more difficult. Drilling through masonry and brickwork permits the insertion of both liquid grout and reinforcingsteel in a way that is not readily visible afterwards. Such techniques have opened up new possibilities in the field of building conservation, as they enable old structures to be reinforced in an invisible manner. The technique of drilling and grouting was first developedin the 1920s by Harvey, in his work on Tintern Abbey. It enables the worker in the preservation and repair of historic buildings to comply with some of the essential principles of structural conservation work:

1. Make maximum use of the existing structure.
2. Repair the structure in a virtually invisible way.
3. Adjust the strength of the repairs to the appropriate degree.

Figure 5.3.7.1 Drilling bits, York Minster, England (Courtesy: Shepherd Building Group Ltd)

All the drill-bead components and assemblies are powered pneumatically for both rotary and rotary percussive drilling units.Advantages and disadvantages- high speed rotary percussive is the most economical means of drilling: tungsten carbide tipped bits are much cheaper than diamond crowns. This type of drilling is fast but demands a homogeneous mass, prefarably of like material, to provide for accuracy in operation where long holes are required horizontally in masonry.Diamond core drilling is rather slower and thus more costly, but under adverse conditions in bad materials is more accurate in maintaining alignment.

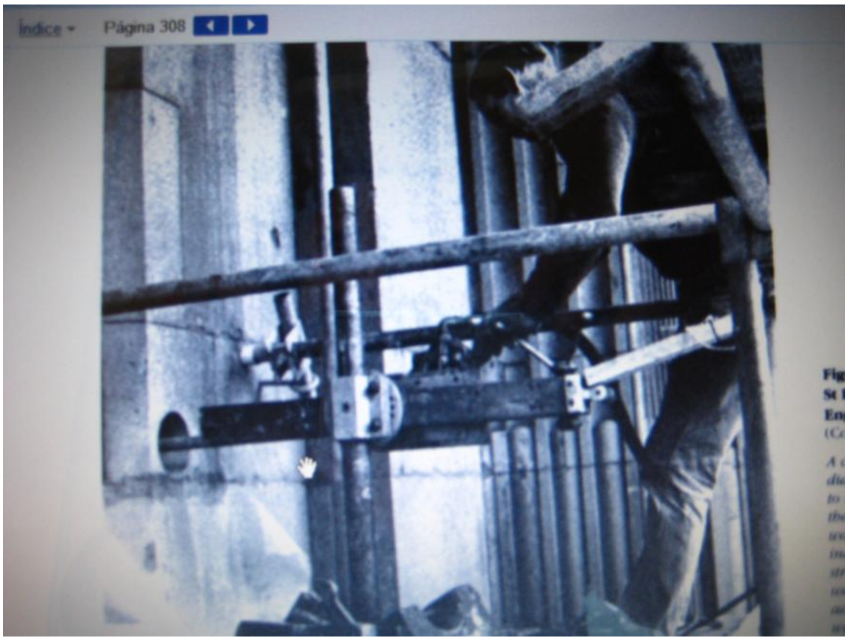


Figure 5.3.7.2. *Enlarged foundations, central tower, York Minster, England*

After drilling through 15 m of concrete, stone, mortar and wood the hole is lined with a pipe to prevent local collapse and the tensile stainless steel bars inserted. These were then streched and given a final tensioning after two months and then grouted. The probes are all ready to receive the liquid grout injection.

5.3.8 Use of Stainless Steel and Titanium in Restorarion

Over time, heat, cold and corrosion eat away at the fabric of a building. As a result, demand for systems to protect and conserve old and landmark buildings is high. However, the products used have to meet very specific requirements.

Usually stainless steel is only associated with modern architecture product services yet it can also be used to revitalize old buildings without completely changing their character. In Cologne Cathedral, for example, stainless steel beams were used to replace the old, severely corroded iron beams of a viewing platform 100 meters above the ground. Thanks to their outstanding resistance to corrosion, they can withstand all wind and weather. At the same time the material is strong enough to support the crowds of visitors without problem. The Dresden Frauenkirche church and the equestrian statue outside Bremen’s town hall are two further structures that have been stabilized with Nirosta stainless steel reinforcing elements. In these examples, the material was chosen not for its appearance but for its functionality.Thanks to the wide range of materials available, the right grade can be found for almost every conceivable mechanical load and every type of corrosion. [87] 20 years ago ThyssenKrupp Titanium supplied material for one of the most prominent Islamic buildings in the world the Hassan II Mosque in Casablanca. 30 metric tons of titanium were incorporated into the towering doors of the religious monument, which stands directly on the sea. This means that as well as being significantly lighter than steel gates, they provide outstanding protection against corrosion in the aggressive sea air. In addition, special designs were developed to create an interesting contrast in gray-gold. [87]

Figure 5.3.8.1 *The Hassan II Mosque*

The Mosque, the second largest in the world after the Masjid al-Haram in Mecca, was designed by the French architect **Michel Pinseau.** It looks out onto the Atlantic and its sheer size almost overwhelms both city and coastline.

Titanium has been performing an important behind-the-scenes task in the Acropolis in Athens for decades. The steel bars originally used for restoration work had to be replaced with titanium because only titanium has the corrosion resistance needed to protect the marble of the world-famous temple columns from decay. [87]

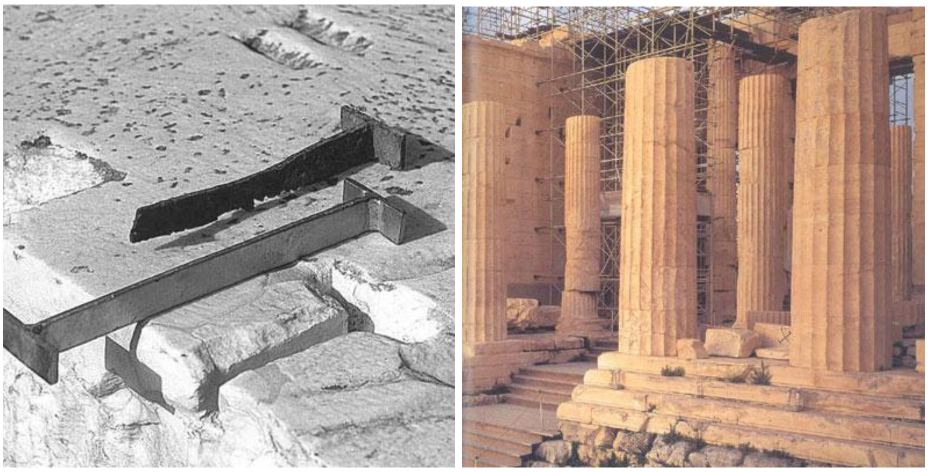


Figure 5.3.8.*2 Titanium Cramp Replacing Acropolis*

The inadequacy of earlier interventions in the Acropolis monuments is due mainly to the replacement of the ancient iron reinforcements and the use of uncoated iron elements to join together fragmented architectural members or to reinforce the resistance of others. Thanks to the strength and durability of stainless materials, monument conservation is a highly promising application for the future. Further projects are planned or already under way: For example, ThyssenKrupp Titanium supplied material for the restoration of the roof of the famous Basilica di San Francesco in Assisi, which was destroyed by an earthquake in 1997. A titanium structure is also to be used to stabilize the foundations of the Campanile di San Marco in Venice. In addition, the dome of Berlin Cathedral, known as the "lantern", is to be restored using material from ThyssenKrupp Nirosta.

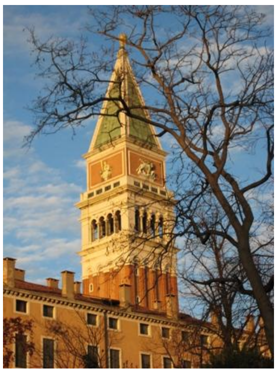


Figure 5.3.8.3 *Campanile di San Marco in Venice*

CHAPTER 6

CASE STUDIES

6.1 Problems Related to Repair and Strengthening of Historic Masonry Buildings in Historic Centers of Umbria

6.1.1 Introduction

This case study explains briefly the problems related to the strengthening of historic masonry buildings which involves also reinforcing nets. The stusy is based on the knowledge and experience gained after the 1997 Umbira-Marche earthquake which offered a unique opportunity for the observation , evaluation and verification of the results not only the 1997 earthquake but also the performance of the repaired and strengthened structures after the 1979 earthquake[6]. The actual efficiency of current techniques for repair and strengthening of historic and masonry buildings are discussed based on the extensive surveys on damaged buildings.

Relevant damages which have been surveyed in stone-masonry buildings after the Umbria-Marche earthquake (1997-98 confirmed the need of improving the knowledge of the seismic response of old masonry buildings and of the reliability of retrofitting techniques.

To this scope a detailed Data-base is being built, which contains overall geometrical data (plan, views, etc.), masonry data, representation of the structural system, retrofitting history, detaileddescription of the damage, and mechanical interpretation of the damage or collapse process [1] of a number of buildings.

6.1.2. Failure Mechanisms of Repaired and Unrepaired Buildings

Failure mechanisms are here discussed based on surveys carried out on two historic centers which have different characteristics and are representative of many historic centers of Umbria. Two typologies of buildings can be considered as representative of the two centers:

(i) stonemasonry

buildings with walls made of irregular stones (mainly calcareous with some few blocks of travertine), having the wall section made by two partially connected leaves (interested by partial reconstruction after subsequent earthquakes), and timber floors and roofs;

(ii) stonemasonry

buildings as in (i), already repaired with partial or total reconstruction of the damaged walls (also using different materials as bricks, tuff blocks, etc), and having floors and roofs remade with concrete beams and hollow clay blocks. In both cases, the internal partitions and the floor height are rather irregular. Due to the soil slope the number of floors can increase downhillup to seven.

6.1.2.1 Isolated buildings (Montesanto)

In the case of isolated buildings four main mechanisms were identified for non repaired or badly repaired structures:

1. Out of plane of loadbearing walls with local or total collapse of the facades or of the corners, or large deformation of the walls. This mechanism is due to the lack of connection between orthogonal walls and between walls and floors or roofs (e.g absence of tie rods) and to the presence of large openings .

2. Out of plane mechanisms with local or large failures of the upper part of the walls and collapses of parapets, cornices and spandrels . Large diffused cracks appear where beams are settled, and local collapses occur due to the high stresses caused by the hammering. Due to the thrust of the roof and to the absence of connection between the roof and the masonry (also due to the masonry heterogeneity) the detachment of concrete ring beams was also observed .

3. Wall disconnection and leaf separation with local or global failures. The presence of inhomogeneities in the wall, the lack of connection between the leaves of multiple leaf walls, the filling of openings without good connection between the old and the new parts or the use of different types of materials can be the causes of such mechanism.

4. In plane mechanisms due to shear stresses with diagonal cracks of piers and walls at the different floors. They are mainly due to: bad positioned openings, differential stiffness of the walls between openings, presence of weak lintels (Figure. 6.1.5).

6.1.2.2 Row houses (Roccanolfi)

An accurate study is still ongoing in the case of Roccanolfi, in order to better define and understand the typical mechanisms of large blocks of houses where the buildings are attached together forming a sort of curtain and built on steep slopes of the soil. To this aim groups of buildings were identified as building blocks. The damage of each block has been studied and surveyed, also adopting axonometric representations which can better show the different levels of the soil. This study allowed to stress out the typical mechanisms of failure of the cases when buildings are tied together along the streets and in differential levels. In all the blocks, the first and the last building are badly damaged by local collapses and large cracks . When the collapses occur in the internal part of the blocks they always interest the non repaired buildings adjacent to the repaired ones In the central part of the building curtains with the presence of decayed floors and roofs large continuous deformations and out of plumb of the walls were detected. Moreover, due to the hammering of the two blocks cracks and damages appear where vaulted passages connect two blocks of buildings . This phenomenon is more clear when only one of the blocks was repaired.

***Figure. 6.1.1**: Failures of end buildings in a row* ***Figure. 6.1.2**: Failure of center non repaired buildings in a row*

6.1.3 Structural and Material Compatibility Problems

The 1997 earthquake was not so much destructive to leave only ruins, but its intensity was such that many errors and mistakes were stressed out. In fact, most of the failures were due to lack of knowledge of the materials and of building construction details , which caused a wrong choice of the repair technique and, very frequently, the poor application of it. Therefore, it is possible to say that there are not bad techniques but only inappropriate and poor applications due to lack of knowledge and of skillness.

Some critical comments on the application of the most frequent modern techniques, based on the damages surveyed after the 1997 earthquake are given in [6]

There are three main techniques used in the area:

- Grout injection.
- Concrete ring beams and roof and floor substitution
- Wall and pier jacketing

6.1.4.Critical Comments on Wall and pier jacketing.

The technique consists in the positioningof a reinforcing net (f= 6 to 8mm) on both faces of a wall, connected by frequent transversal steel ties, and applying on the two faces a cement mortar based rendering. The aim is to improve the connection of the wall, and to increase the tensile and shear strengths and the ductility [5]. The same technique can be carried out to connect load-bearing and shear walls and to close also large cracks.

This technique was largely applied in Italy to irregular multiple leaf stone- walls and it is recommended by the Italian Code. Nevertheless, due to the inhomogeneity of the walls, to the cost and the difficulty of connecting the two faces, its execution on site is not very easy. The most diffused mistakes made on site are described in the following, together with the consequent damages:

(i) lack of connection between the nets in orthogonal walls and in correspondence with the floors, which cause discontinuities between the walls ;

(ii) lack of overlapping between two different sheets of the net (Figure 6.1.4),

(iii) absence or too spaced steel transversal connectors (Figure 6.1.5), which can cause the separation of the reinforced layers from the wall;

(iv) use of too short connectors;

(v) insufficient thickness of the steel cover with consequent steel corrosion (Figure 6.1.6);

(vi) lack of uniformity of distribution of the repaired areas in the structure, which can cause torsion stresses due to the non uniform distribution of the stiffness.

Figure 6.1.4 *Lack connection Between the nets*

Fig. 6.1.5 *Lack of connectors*

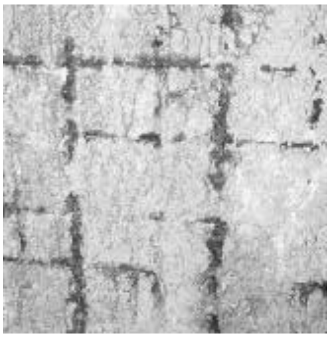


Fig. 6.1.6 *Corrosion of the nets*

6.1.5 Concluding Remarks

- Multiple leaf stone-masonry structures are peculiar; their behaviour under seismic loads and their compatibility to repair techniques still need more knowledge to be understood; they cannot be compared even to brick or regular stone-masonry structures;
- The basic attitude of the near past to fit the real structure to reference analytical models applicable to other masonry structures implied sometimes invasive and non compatible retrofitting techniques;
- When using new techniques and materials experimental research has to be carried out before,

not only on the mechanical behaviour but also on the physical and chemical compatibility with the existing structure and materials.

As regards compatibility problems, it is worth noting that repair techniques were used in the past centuries and the present ones are sometimes only a reproposal of them using modern materials, which can be incompatible with the existing ones. A better knowledge of the traditional techniques and new research to apply them in a modern way will be one of the major issues of the future research of the authors in this field. At present, in fact, very few research has been carried out on the behaviour of rubble and multiple leaf stone structures before choosing the appropriate repair techniques .

6.2. French Pantheon- The Causes of Structural Disorders of Soufflot's Pantheon in Paris

6.2.1 General

The French Panthéon (Figure 6.2.1), started by Soufflot in 1756 as the biggest church in Paris, dedicated to Sainte Genevieve, patron saint of the town, and finished by Rondelet in 1790, is, probably, the first building to be calculated with modern methods of structural engineering. The need for these calculations was the consequence of the adoption of an innovative building technique (the reinforced stone masonry) and of the reduced dimensions of wall structures, that did not follow the classical building rules. Still in the design phase, the first objections aroused from the architects linked to the old academic building tradition, particularly from Pierre Patte [6], because the structures did not respect the canonical proportions: pillars too slender, domes masonry too thin, windows too large. To answer this first campaign of polemics, Jacques-Germain Soufflot and Emiland-Marie Gauthey, director of the prestigious "École des ponts et chaussés", carried out the first systematic compression tests on stone specimens and the first calculations [3,5], demonstrating that the pillars had a cross section large enough to sustain the weight of the domes considering a centred load (Figure 6.2.2.2).

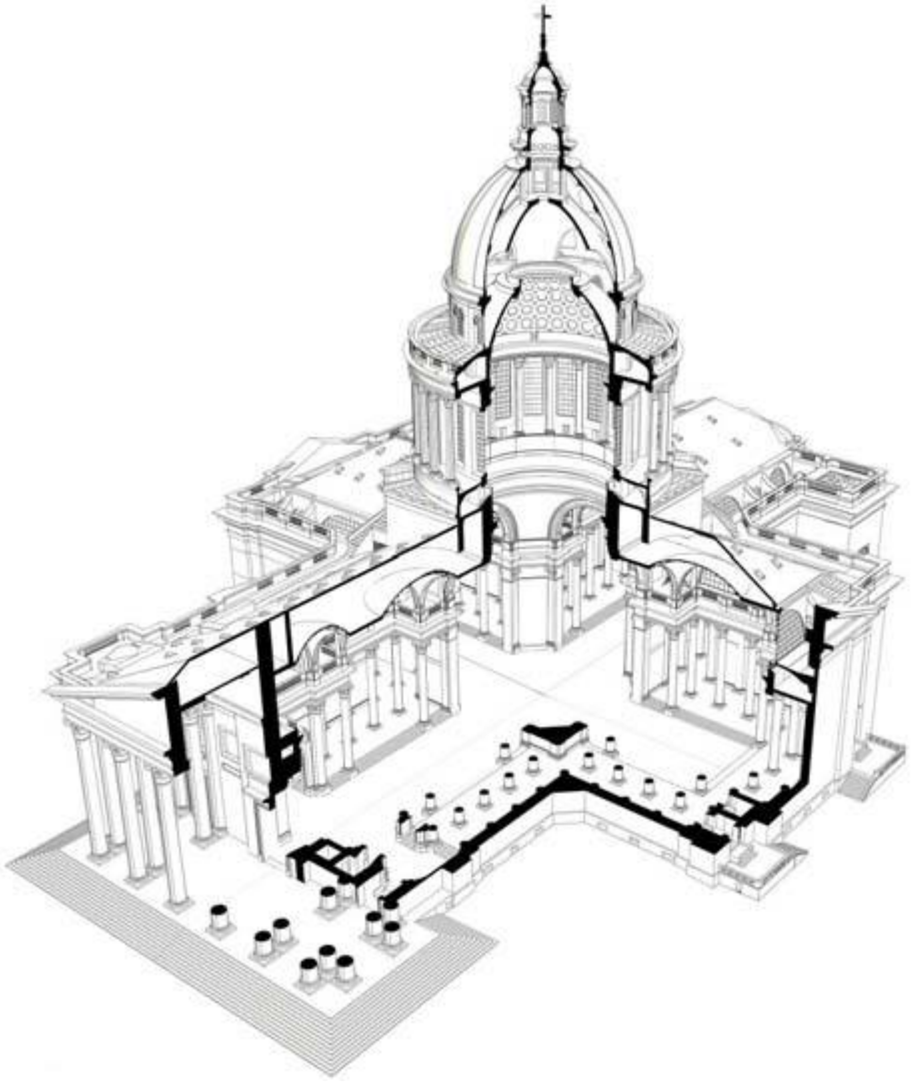


Figure 6.2.1. Axonometry of the monument with view of the inside

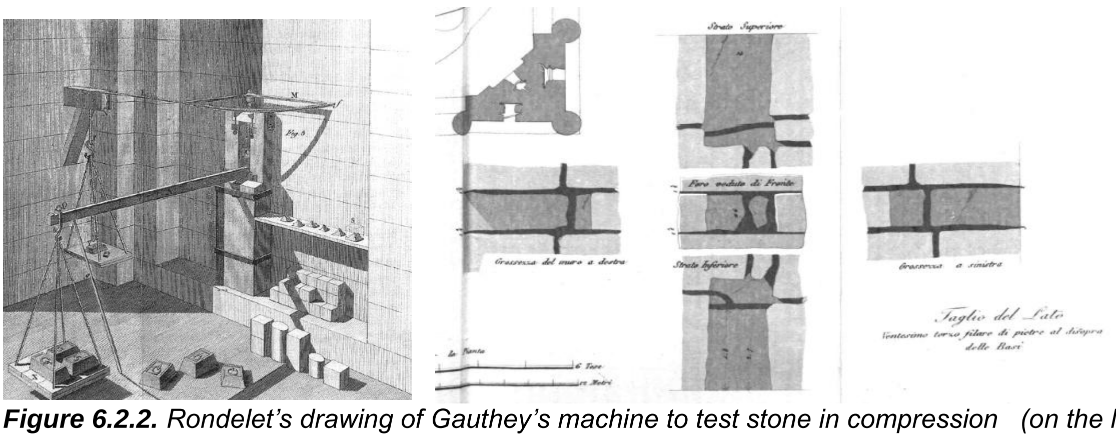


Figure 6.2.2. *Rondelet's drawing of Gauthey's machine to test stone in compression (on the left) and Rondelet's assays on the pillars (on the right).*

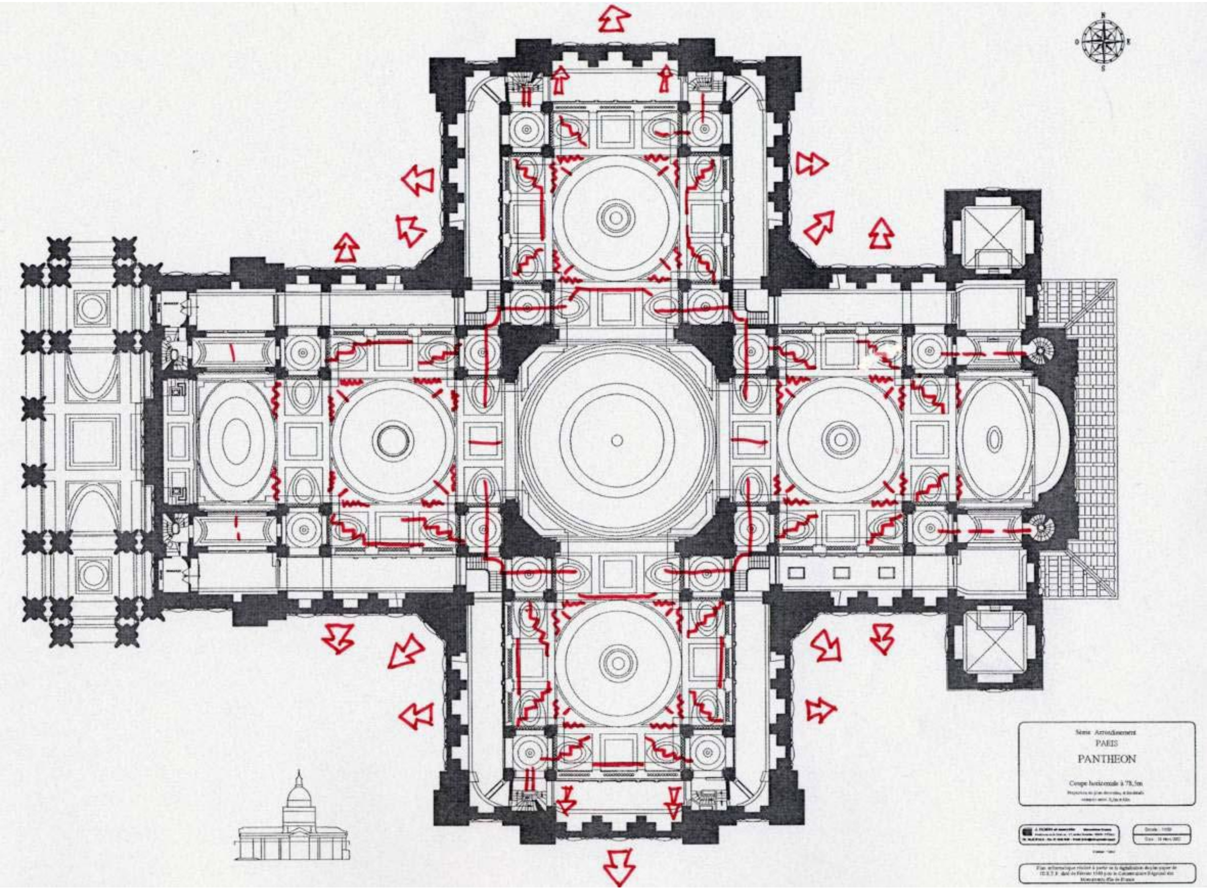


Figure 6.2.3. *Schematic survey of the main crack systems (position and global interpretation)*

6.2.2 The Causes of the Main Disorders

The reinforced stone masonry was a building technique completely new at the time of the construction of the Panthéon. It was made possible by the spreading of iron due to the technological development at the beginning of the industrial era. This new technique allowed Soufflot to realize very slender structures, comparable only with the structures that one century later will be made with reinforced concrete. Indeed, looking at the drawings depicting the iron clamps in the flat arches, it seems to look at drawings of reinforced concrete trusses (Figure 6.2.4). Soufflot has developed his design basing only upon his structural intuition and also the calculations made by his friends and co-workers Gauthey and Rondelet are little compared to the complexity of the problems posed by the new structures.

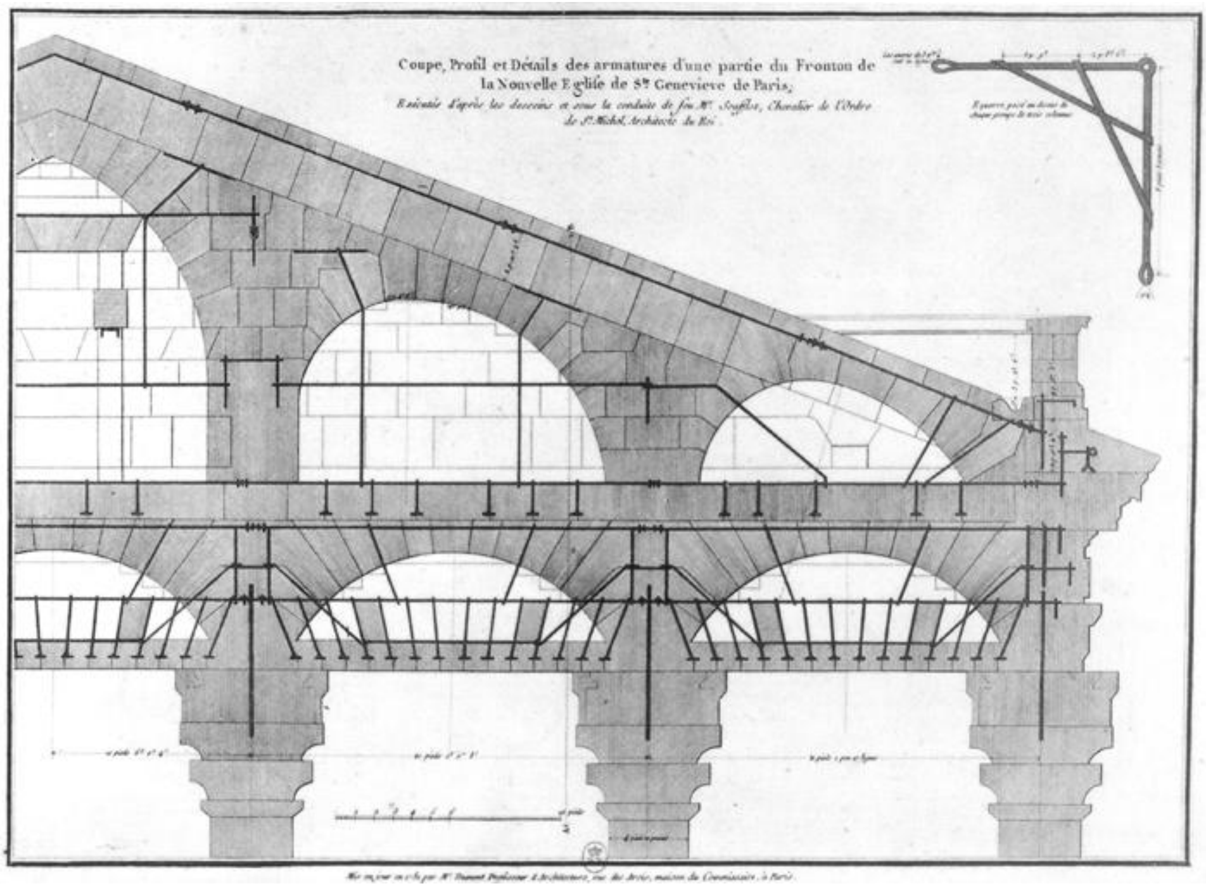


Figure 6.2.4 *The drawing depicting the iron clamps in the flat arches clearly recalls reinforced concrete trusses*

The slenderness of the structures, together with the real long time deformability of the masonry (2000 MPa) has, however, produced large deformations on structures, unpreviewed and non congruous with the fragility of reinforced stone masonry. The presence of iron clamps, indeed, produces, under long time loads, stress concentrations in the stone and consequent fractures. The characteristic shape of the cracks in the stones due to the presence of the iron clamps was exactly recreated with numerical crack models. In brief it was demonstrated as the cause of fractures, repeated in a perfectly symmetric way in the whole building, is to be traced back in the global deformability of the masonry, associated with the local fragility of connections . In particular, the structural elements that mainly show problems and deformations are the four great arches, with a span of over 30 metres, that sustain the external colonnade of the tambour. These arches thrust on the boundary walls both with high static horizontal forces (about 3300 kN) and with dynamic actions caused by the wind.

6.2.3 Concluding Remarks

As it is clearly impossible to remove all the clamps between the stones without destroying the monument, the solution to hinder new fractures must be searched in the introduction of pre-stressing systems in masonry and, overall, in the active opposition to the thrust of the four great arches.

In conclusion, despite the courage to design such building with a new approach, the designers of the French Panthéon, could not foresee creep phenomena, stress concentrations due to iron clamps and plastic deformations under long time loads that have only recently been inspected.

6.3 Minarets at Risk in Afghanistan

6.3.1 Introduction

There has been a dramatic loss of architectural heritage in Afghanistan not only because of earthquakes but alsodestructions due to war.

Among the hundreds of archeological sites, the minaret of Jam and the five minarets of Herat need urgent safeguard works. Many minarets were lost in the past and the existing ones are exposed to a serious risk of collapse. Apart from the seismic actions, dangerorus inclinations of the minarets and the poor state of the masonary increase the risk.

Figure 6.3.1 *Minaret of Djam in Afghanistan*

Figure 6.3.2 *Decorated exterior of the Minaret of Djam*

In 1961, architect Andrea Bruno began surveys and projects for the restoration of Afghan monuments together with the Italian institute ISMEO which continued in 1976 and 1979 with surveys and restorations of Jam and Herat for UNDP and UNESCO (Bruno 1981).

All the experts who had the opportunity to see it agree that the tower is on the verge of collapse. The out of plumb is 2.70 m and a large horizontal crack is present at the height of 3 m from the base. The structural analysis show that the eccentricity of the self weight is is such that the cross section is cracked for one third of the diameter, and the edge of the masonry is subject to a a very high compression, 1.2 Mpa. A moderate earthquake or a strong wind may cause the collapse of the minarete, as already happened to the four other minarets which were still standing in 1915.

Figure 6.3.3 *Minaret 5 (Musallah Complex)*

6.3.2 Emergency Securing of the Minaret Five in Herat

Minaret 5 is made of brick, 42 m high with a diameter of 5m only at the base. The surface has a kashi decoration with a geometrical pattern filled in with blue tiles.

Both the numerical assesment and the site measurements showed an extremely alarming situation. As there was not enpough time for the implementation of the structural interventions approved by UNESCO in 2003. i.e . stabilization of the foundation by menas of single bar micropiles and strengthening of the elevation by means of a vertical prestress applied through high strenght steel bars directly connected to the micropile bars. There fore emergency measureswere applied to the minaret during July , August and September 2003.

The emergency securing was realized by means of simple steel stays anchored to concrete blocks poured in the natural soil (out of any remains), thus avoiding the necessity of driving piles as fixed points.The proposal was conceived in a way to use only the means available on site for the anchoring. Steel cables were supplied from Italy together with the anchorages , hydraulic jacks and other elements required for the operation.

The concrete blocks, built by local workers, were embedded in the soil at a distance of 25 m from the basis of the tower. The centers of the blocks were placed on two axes 15 degree far from the maximum lena plane in order to secure the tower for an angle of 30 degree. In each block, 4 steel pipes were embedded in the concrete in order to accomodate 4 stays and their anchorages.

Each stay was made of sinfle 0.6" strand of galvanized high tensile steel (area=150 mm2, strenght = 265 kN) . Each stay was applied along a precisely defined path (see figure 6.3.4 and the model of Figure 6.3.5): Each starndstarts from a concrete block and reaches the tower surface with the appropriate inclination, the is winded to it along a variable angle spiral until a sub-horizontal circle allows to begin a descending path towards the other concrete block and its anchor. This arrangement of the stays was essential in order to transfer to the tower the vertical component of the stabilizing action through the steel –to masonry friction, therefore without slip and without heavy steel fastenings. In this way the small force of 100 Kn only, applied by jacks to each of the four strands, has been sufficient to close the base crack, to decrese the eccentricity of 0.65 m and therefore bring back the masonry to state of total compression , the maximum stress was reduced to 0.6 Mpa.

The application of strand was performed in two weeks by a single tcehnician from Italy who was supported by local workmanship.

The continous monitoring showed that the top of the tower moved back about 16 mm and therefore showed the favorable effect. This quick, simple , efficient intervention show s how emergency works may avoid the total loss of some precious monuments in a cheap way and without invasive structures.

6.3.3 Minaret of Jam

The minaret of Jam is far more important than the Herat minarets for its extraordinary size: 65 m. Of height, close to that of Qutub Minar in New Delhi that was built in the same period at the end of the 12th century. The tallest minaret in the world isolated in the mountains in the Left Bank of the Hari Rud River. The tower has a great architectural and historic value and is now the symbol of Afghanistan in the World Heritage. It is very important monument for the Ghurid Dynasty.

The architecture is highly sofisticated. The tower is built with fired bricks, and is formed by three cylindrical superposed shafts on an octagonal basis. The diameter of the base is 9 m only. Two helicoidal internal stairways leading up one above the other , to a balcony at the top of the first tier at the level of 40 m stand for the unusual features of the tower.

A first essential emergency intervention was implemented between 1975 and 1980 when A Bruna installed metal gabionsprotecting the foundation from scouring. However, no further action was possibleuntil 1999, when the protection was strengthened and new mission was took place in 2002.

According to the seismic assesment works carried out in 2003, there is no direct evidence of large earthquakes ocurred at the site in the literature. However, tectonics and historical seismicity of Afghanistan suggest a very careful study. The seismic hazard of Jam was assessed by means of different available procedures. Namely the probabilistic and deterministic ones. Modal analysis, sub soil investigations and the alternative assumptions for soft and stiff soil prove that the combination of the earthquake with the gravitiy loads, with stiff soil would lead to rather high tensile stresses (0.27 Mpa) and compression stresses (1.20 Mpa). As such a tension could not be resisted by masonry, the crack formation has to be considered as well as higher compression (1.60 Mpa). This state of stress shows a collaps risk which is not allowable and the compressions strength of the brick work at the base of the tower is needed to be improved urgently.

Given the large thickness of the of the shaft, the only realistic method of immproving the vertical strenght seems to be the application of a circumferential prestress by means of high strength steel wires winded around the shaft at an high tension. Such a technique is proved to be efficient and have little intrusivity on the leaning tower of Pisa. The use of 4 mm stainless steel wires would provide also a durability appropriate to a permanent interbention (Macchi 2001).

6.4. Technologies for the prestressing rings of the leaning tower of Pisa

6.4.1.General

Prestressing techniques proved to be very useful in providing structural safeguard measures to the leaning tower of Pissa. During the eleven years of stabilization works the International Safeguard Committiee used pretensioned cables, starnds and wires for the application of active circling forces at the critical level of the first Loggia and to the foundation plinth, both for temporary strengthening measures and permanent interventions.

6.4.2. Interventions required the use of post-tensioning

Figure 6.4.1*.Piazza dei Miracoli from Northwest from Europe Illustrated, about 1875*

Among the temporary and permanent structural interventions, four of them required the use of post tensioning which are described below:

The First temporary and reversible prestress was made in 1992 on the first Loggia zone and could be defined as an urgent "first aid" intervention aimed at preserving the life of the monument during definition and realization of its final stabilization. The second temporary and reversible prestressing work was made in 1993. A temporary ring of concrete segments pressed against the bottom of the Tower surface by means of unbonded internal post-tenisoning tendons.

The beam was designed to support the lead ingots necessary to counterbalance the Tower appropriately during the time necessary for the stabilizationd of the foundation of the tower.

The third work , realised in 2001, consisted in the replacement of the first temporary ring of 1992 with permanent, but reversible, circumferential prestressing belts made of a stainless steel wire.

The fourth permanent intervention, made in 2001, is the permanent construction of a concrete ring under the Catino surface that was blocked to the foundation by means of post-tensioned unbonded internal tendons.

All the strengthening works, were carried out according to the specifications of the international committee appointed by the Italian Government in 1990 and were performed with a proven reversible prestressing technology that allows the gradual and monitored tensioning of the tendons, the control and adjustment of their stress, the replacement of the tensile elements.

Furthermore, any deterioration or damage of the Tower surface had to be avoided.

Figure 6.4.3 The landmark was turned into a building site

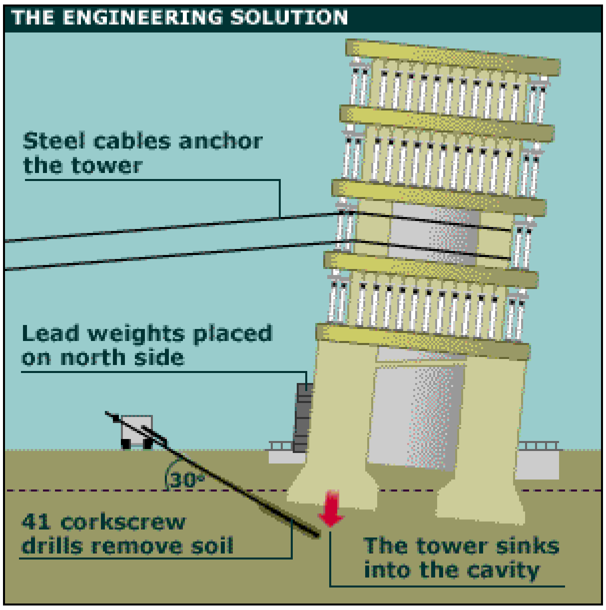


Figure 6.4.4 *The engineering solution*

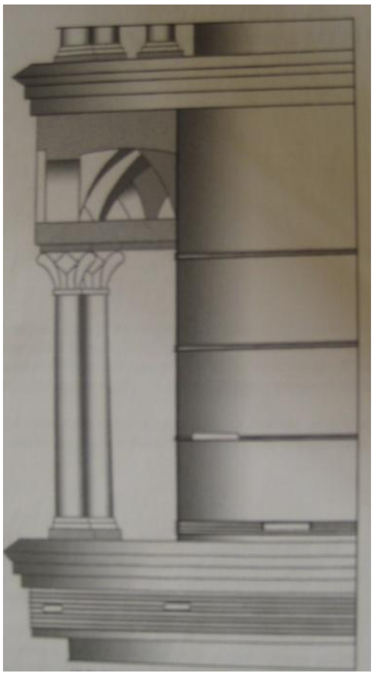


Figure 6.4.5 *Temporary circling*

Figure 6.4.6 *A steel corset was placed around the tower*

6.4.3 Temporary Circumferential Prestressing of the First Loggia

The first temporary circumferential post-tensioning intervention was based on a feasibilty study based on a feasibilty study of Professors F.Leonhardt and G. Macchi (November 1990) that made evident the urgent necessity to provide an active transversal prestress”to improve the strength of the critical zone located at the first Loggia and avoid “outside buckling of facing stones”.

The strengthening was made through the installation and jacking at 132 kN of n° 10+8 0.6”hot dip galvanised monostrand tendons encased in a clear PVDF tube inert to atmospheric pollution and to UV rays ageing.

The monostrand tendons circling the first Loggia wall (n °10 units) and the underneath Drum (8 units) were tensioned and anchored on an intermediate monostrand stressing anchorage. It was designed by VSL International and could slid on a PTFE pad to protect the marble of the Tower. The strand tails, cut after the tensioning of the rings in sufficient length to allow the control, adjustment and releasing of their force at any required stage, were encapsulated in the galvanished anchor blocks. The entire system was studied in order to assure a long term corrosion protection (30-40 years). The usual greased/waxed unbonded strand was discarded to eliminate any risk of straining the marble surfaces. The use of the stainless steel strand was set aside both fort he impossibility of producing the small quantity required and for relaxation reasons. The installation, tensioning and releasing procedures were tested, prior to perform the work on the Tower, on a full-scale ring installed on an existing concrete water tank having a diameter similar to the one of the Tower. The monostrand tendons, removed in November 2000 to allow the final circling with the 4 mm diameter stainless steel wire, were perfectly preserved as well as the anchorages and the PVD tubes.

6.4.4 The Concrete Counterweight-holder Beam

In 1992 a first check of the monitoring data of the inclination showed that the speed of the out-of-plumb movement of the Tower was increasing. The Committee felt that such a dangerous movement had to be urgently stopped in order to allow time enough for a well based safe choice of the final stabilization technique.

On this purpose, a proper concrete ring beam, provided with the necessary shortening joints, pressed against the Tower bottom like a vice through n °4+2 6T15 prestressing tendons, on which to load the counterweight, was built.

The ring beam, designed to support a load of 1000t, was gradually loaded with 600t of lead ingots. The concrete beam, shortly called lead-holder, designed by Bonifica according to the sketches of F.Leonhardt, was built in a was aimed at preventing any damage to the marble cladding. The internal unbonded post-tensioning was realised through tendons formed with six galvanised, greased, and individually sheathed 0.6 strand.Their tails were protected against corrosion with grease and encased in hot dip galvanised caps in order to assure the possibility of controlling and adjusting their tensioning when necessary.

At the end of the underexcavation in 2001, the counterweight was removed, the concrete beam was totally dismantled and proved to be a perfectly reversible intervention.

6.4.5 Circumferential Prestressing at the First Loggia with a 4mm Dia. Stainless Steel Wire

When the stabilization works (realised as known through the progressive and monitored extraction of soil cored from the ground under the Nrthern sector of the Catino), were close to get the required reduction of the Tower lean, The Committee decided to remove the temporary ringing strands installed on the first Loggia zone and to proceed, at G. Macchi proposal, to the permanent but reversible circling of that critical zone with narrow stainless steel wire made belts of the same capacity of the previous strand rings. The change aimed at less visual impact and beter durability. The guideline stated that the circumferential prestress had to be realised with a proven technology using stainless steel resistent to the stmosphere corrosion, had to be applied gradually and controlled by monitoring, should not damage the Tower surfaces pressed by the tensioned wires, should have a limited impact on the Monument, had to be reversible and, during his installation should not damage the Tower neither disturb the peculiar ambient noise coming from the tourist activities in the Piazza.

The technical design and the implementation of the prestressing rings was assigned, through the Consorzio Progetto Torre di Pisa.

The guiding idea was to build a proper winding machine that could continuously lay a stainlesss steel wire under the tension on the Tower surface using the available mechanical facilities and according to the specific characteristics of the Monument. This could be made with the technology utilised along several years in the past for the external post-tensioning of the cylindrical silos or tanks.

A cold drawn (4 mm dia) austenitic stainless steel wire (NTR 50, equivalent to ASTM A276 S20910), with an excellent resistance to (its yield point is about the double of AISI 3161, stainless steel), was chosen for the circling.

The wire dimension, was defined according to the maximum stressing force supllied by the winding machine (10 Kn) , to the mechanical characterisrics of the cold-drawn wire (resistance $\geq$ 1400 N/mm^2 , to the stressing tension (<700 N/mm^2) applicable to the wire without inducing relaxation phenomena in the stainless steel and according to the diameter of the tensioning device too.

In order to contain the maximum width of the stressing belt on the Drum in nomore than 145 mm and to make the ornamental band of black marble that decorates the cornice completely visible, the wires were winded spire against spire on four overlapped layers.of 34, 31, 28 and 25 spires respectively , that supplied a total prestressing force of approximately 910 Kn. The first three layers were painted with a waxfilm to fill the capillary voids between the spires and to prevent crevice corrosion phenomena.

Ech layer was anchored to the tower through independent stainless steel clamps while stainless steel pins were properly installed to contrast the deviation points. The wire belt es secured to the tower with five narrow equispaced stainless plates. The job was made during the first sixteen days of Jnuary 2001 under difficult weather conditions.

The circling of Loggia base was realised, in a similar way but working in a very cramped space with a belt of approximately 100 mm only formed by three layers of stainless steel wires (24+21+18 spires respectively) that supply a total prestressing force of approximately 480 Kn. The marble surface of the Tower was carefuly protected from the direct contact of the steel chain with a wooden belt warapped by a rubber band.

6.4.6 The Catino Bottom Restorign and its Connection to the Tower

The first solution designed by the Commitee for the Tower stabilization could be summerized very shortly in the construction of a concrete ring beam under the catino surface blocked to the Tower foundation through internal bonded tendons. Ten deep ground anchors, of 1000 Kn capacity each one, anchored on the North side to the ring beam allowed to apply a defined, adjustable and monitored force to the Tower on its overslope side, to reduce and to control its lean. The project is known as “ 10 anchors project” too.

When the first five segments of the foundation ring in the north edge of the tower were made, during the gorunf freezing for the construction of the first segment on the South (underslope) side, the lean of the Tower increased suddenly (September 1995). The prestressing force in the lead holder beam was immediately increased, according to the design specifications, and the counterweight increased up to 900t. That unexpected and worrying behaviour, due to an unknown accidental connection between the Catino and the Tower, caused the prompt stop of the works and successively the abonden of that project.

In the year 2001, when the stabilizing operation according to the underexcavation method was ended, the Committee decided to restore the bottom of the Catino and to connect it to the Tower foundation through a concrete ring that includes the five segments already realised, pressed against the foundation by means of post-tensioning internal replaceable unbonded tendons.

The intervention had not to remove the Catino foundations on the South side but ınly its upper layer (140mm thick) that was already cracked, had to leave visible the half height of the first step at least, had to provide the correct waterproofing and drainage of the water in the Catino area, had to utilise only stainless reinforcing bars.

The design, developed by the Author in co-operation with G. Nicolini of ALGA group and the essential contribution of G. Macchi, L. Sanpaolesi and R. Bartelletti, led to the realisation of a thin ring slab, 140 mm thick only, concreted on the South side of the Catino encasing n° 36 0.62” monostrand tendons positioned on two layers.

The connection of the 36 monostrand tendons coming from the South side to the 4 upper 12T15S tendons coming through the five concrete segments from the North side, was realised with two special trapezoidal steel anchorages 390 mm thick only resisting the 6000 kN stated ringing force, on which the tendons were anchored.

The moderate vertical lifting force (120 kN) caused by the misalignment of the centrelines of the North and South orders of tendons was balanced by the anchorages mass, by their friction against the Tower foundation, by their connection to the Catino perimetral wall.

To assure the maximum possible anticorrosion protection, galvanised, waxed and sheathed 15,7 mm super strand was used. The unbonded monostrand tendons were furthermore individually encased in additional HDPE continuous tubes, sealed at their extremity to the steel anchorages, to allow their replacemtn if necessary. The steel anchorages were protected from the corrosion with a thick (1200mm) fiml of epoxy-polymminic paint that provides their electric insulation too. The ring slab and the five concrete segments were properly anchored to the Catino and Tower foundations with stainless steel bars. The stressing operation was performed through equilibrated, symmetrical and incremental steps according to a detailed stressing program involving the constant Monument monitoring. Jacks and deviators designed on purpose were used to operate in the small available anchorages space.

After a convenient control period, the steel anchorages were inspected and filled , with a proper anti-corrosion, hydrophobic , insoluble and climatic variation resistant wax, in order to impede any water infiltration.

The realisation does not preclude the possibility to install and connect ten deep ground anchors to the five concrete sectors on the North side, if necessary.

6.4.7 Concluding Remarks

The described realisations show how a prestressing technique, conceived fort the post-tensioning of industrial buildings or structures, can be utilised fort he strengthening of Monuments of high architectural and historical value, if properly developed in details and suited to that particular use.

6.5 Use of Shock Transmission Units and Shape Memory Ally Devices for the Seismic Protection of Monuments: The case of the Upper Basilica of San Francesco at Assisi

6.5.1 General

A traditional technique to increase the strength of masonry walls and/or improve the connection between different structural elements, thus improving the local and global seismic behaviour, is the use of reinforced injections. But in many monumental structures, the presence of frescoes and/or the limited thickness of the walls, as well as the willingness to limit as much as possible not reversible interventions, does not allow use of such techniques. In these cases, additional stiffening elements are introduced, such as steel beams. This solution, however, is not without problems as it may hinder the natural breathing of very wide historical buildings, that is relative movements due to thermal effects, seasonal water table variations, *etc*.. In some cases, especially when the building dimensions are very large, it may be useful to divide the global connections in sub-elements connected by “shock transmission units” (STUs), *i.e.* elements which allow slow relative movements but prevent any instantaneous ones. At the same time, the traditional technique to improve the seismic resistance of a building guaranteeing its box behaviour, creating connections between horizontal stiff frames (roofs and floors) and peripheral walls, generally allows no important and useful energy dissipation, and can induce very high forces on the masonry elements. The linking technique may be improved using an innovative material called “shape memory alloy” (SMA) in specifically designed connection devices. Several advantages may be accrued by these devices in comparison to traditional steel ties, concrete beams, *etc.*. Particularly, by using these devices it is possible to allow “controlled” relative displacements, limiting the forces and the acceleration transmitted, and inducing energy dissipation by the masonry itself. A significant example of the application of these intervention criteria may be found in the Basilica of San Francesco at Assisi, restored after being strongly damaged by the earthquake of 1997.

6.5.2. Device Properties

Shock transmission units (STUs) are structural devices, used to connect structural elements, whose behaviour depends on the velocity of the relative movement between said elements. In effect, they allow low velocity movements, such as those due to thermal variations or other slow phenomena reacting with very low forces. Conversely, under high velocity excitations (*e.g*. earthquakes, strong winds, *etc*.), they are very stiff, preventing the structural elements they connect from significant relative movements, and transmitting them the design force. Thus, shock transmission units act as temporary restraints. Since many years they have been used in new structures, mainly in bridges and viaducts to connect deck to piers or abutments, as well as in buildings. However, their behaviour is also useful for the seismic protection of historical and monumental structures, to provide the stiffness needed to withstand the earthquake without inducing undesired forces under service conditions. The use of STUs in historical structures have been first proposed and applied in Italy about 10 years ago, for the dynamic connection of a new steel roof structure to the masonry walls of a church [1]. STUs are devices comprising a cylinder-piston system with two chambers filled with a special fluid and connected through an hydraulic circuit. When they are used in historical structures, a special care should be taken in the selection of materials . The connections of the devices should be designed to allow an easy replacement, respecting the criterion of reversibility of interventions. SMAs are metals endowed with very unusual thermo -mechanical properties, comprising the superelastic behaviour, that is, the ability to recover from large deformations (greater than 10 times that of conventional metals) during loading-unloading cycles. In the loading phase, the stress-strain curve shows a "plateau" (*i.e.:* a section where stress remains nearly constant with increasing strain), similar with the one seen by yielding but instead due to the stress-induced reversible and non damaging phase transformation from Austenite to Martensite. Loading and unloading paths generate an hysteresis loop and thus energy dissipation. The SMA superelastic behaviour makes them particularly suitable to create force limiting devices. SMA Devices (SMADs) have been developed to overcome some drawbacks of traditional very stiff steel ties, used to connect masonry walls with floors and/or roof and thus reduce the risk of out-of-plane collapse of peripheral walls. In effect, SMADs work as far more flexible ties, capable of allowing controlled displacements, and limiting forces under a pre-established value [15]. The SMADs use SMAs in the form of wires, suitably connected to work always under tension and giving the device a symmetric behaviour regardless of the direction of the displacement. They are designed to have different response for different intensity of external action:

a) For low horizontal action (wind, small intensity earthquakes) the device is stiff, as a traditional steel connection, and no significant displacements are allowed.

b) For higher horizontal actions the stiffness of the device reduces thanks to the "superelastic plateau", and "controlled displacements" are allowed. The latter should allow the masonry to dissipate part of the energy transmitted by the earthquake mainly thanks to micro-cracks in the masonry structure, taking care to avoid dangerous macro cracks. Furthermore, SMADs transmit to the structure smaller forces than steel ties do.

c) For extraordinary horizontal actions (*i.e.* higher than the design earthquake) the stiffness of the device grows up in order to prevent from excessive displacements and instability. Shaking table experimental tests carried out on mock-ups of masonry façade walls connected to the structure by different devices showed that there is significant improvement in seismic response by using SMA devices instead of steel bars to tie walls. Not only is the out-of-plane collapse of the wall partly below the tying level prevented but that of the wall partly above it (representing the tympanum of a church façade) as well [16]. The SMA with the best performance is NiTi, an alloy that also has very high resistance to corrosion. High durability of SMADs is obtained using NiTi wires and stainless steel.

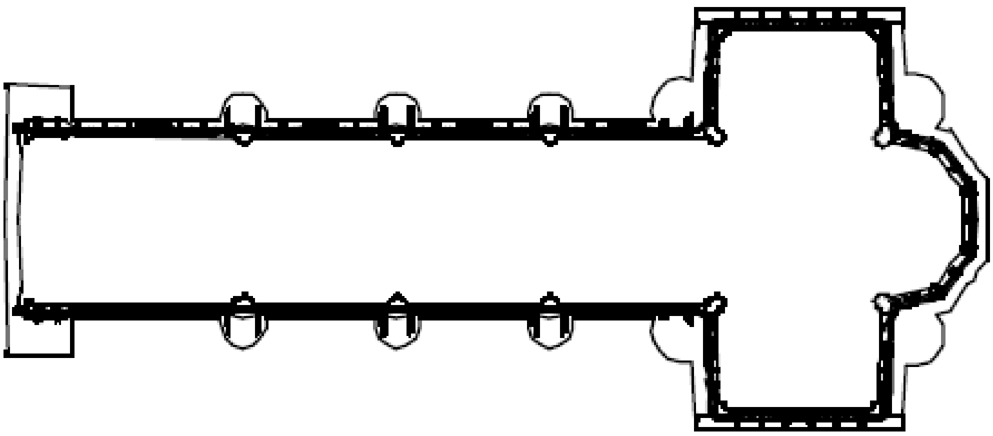


Figure 6.5.1 *The horizontal stainless steel beam all along the Upper Basilica*

Shaking table experimental tests carried out on mock-ups of masonry façade walls connected to the structure by different devices showed that there is significant improvement in seismic response by using SMA devices instead of steel bars to tie walls. Not only is the out-of-plane collapse of the wall partly below the tying level prevented but that of the wall partly above it (representing the tympanum of a church façade) as well [16]. The SMA with the best performance is NiTi, an alloy that also has very high resistance to corrosion. High durability of SMADs is obtained using NiTi wires and stainless steel.

6.5. 3 The Case of the Upper Basilica of San Francesco at Assisi

6.5.3.1 General Description

In its history many earthquakes have hit the Basilica of St Francis (Figure 26), which was built in the 13th century. Important earthquakes occurred in 1279, 1328, 1703, 1747, 1781, 1799, 1832, 1859, 1917, 1979, and yet none produced damage as great as that which hit central Italy during the night of September 26, 1997, as well as the second that struck the Basilica at 11,42 a.m. The result was the destruction of the vaults close to the façade, of

those close to the transept, of a portion of the left transept (Figure 27) and the production of large cracks and permanent deformation all over the vaults of the Basilica, leaving them in a very precarious and dangerous condition. Besides the differing impact that earthquakes of different characteristics, which followed each other during the centuries, may have produced on the Basilica, other factors have increased the vulnerability with respect to the past.

One factor is certainly the presence of a large volume of fill which was mainly broken tiles and other loose materials accumulated over centuries of roof repairs in the springer zones. During seismic activity, this fill, without any cohesion, alternatively acts only on one side, whilst on the other side the fill is detached. Furthermore the lose fill follows the movement of the vaults, opposing their recovery and facilitating increasing permanent deformations. When the quake of September 26 hit the Basilica, it is very likely that permanent deformation, reducing the curvature and therefore the bearing capacity, was already present, having been progressively produced and increased during the previous earthquakes (and this is another important factor).

Figure 6.5.2 *The Basilica of St. Francesco of Assisi*

The restoration project of St. Francis Basilica in Assisi has been made by Prof. Eng. G. Croci and Prof. Arch. P. Rocchi, with the participation of Eng. G. Carluccio and Eng. A. Viskovic, under the supervision of Dr. A. Paolucci, the artistic co-ordinator of the Ministry of Cultural Heritage, Arch. C. Centroni, Superintendent of Umbria.. The on site installation of the STUs and the SMADs was carried out by FIP Industriale S.p.A.. under the supervision of Engg. M.G. Castellano, G. Carluccio and A. Viskovic; the mathematical models and the analyses of these interventions have been done by Engg. A. Bonci and A. Viskovic. The experimental laboratory tests were carried out by FIP Industriale (test on SMADs) and by ENEA (shaking table tests on mock-ups with SMADs) under the supervision of Engg. M. Indirli, M.G. Castellano, A. Bonci and A. Viskovic, within the “ISTECH” Research Project partially funded by the European Commission (Contract No. ENV4-CT95-0106, 4th Framework Programme) . The St. Francis Basilica is built up by two distinct Basilicas, the Upper and the Lower, one on the top of the other. Vertical cracks are located near the centre of each bay of the side walls of the Upper Basilica. Those cracks, opened and enlarged many times by past earthquakes, were reopened by the last seismic event of 26 September 1997. Each crack starts from pavement level up to the level of thetall window base, often cracking the window cornice itself, and is visible both on the internaland on the external side. Considering that the top of the windows arrive near the roof level and, at the same time, in each bay of the Lower Basilica there is a large and tall opening for the access to the lateral chapels, the nave structure of the Basilica as a whole, results divided into vertical portions weakly connected each other in the middle of each bay. The cracked side walls under the windows level in the Upper Basilica are those with the presence of the Giotto’s frescos. Thus a protection is required to avoid excessive crack openings during seismic actions normal to the walls themselves, but the use of reinforcement injections is excluded. Therefore a “dry” and almost completely reversible intervention consisting of a chain of steel trussed beams placed over the internal cornice, which is located at an intermediate height just over the Giotto’s frescos and under the windows, was adopted. Each beam increases the transversal one bay of the side wall between two external buttresses and is connected to the beams in the adjacent bays through STUs, in such a way to contemporary guarantee the global horizontal connection (Figure 6.5.1). The transept south tympanum masonry wall of the Upper Basilica partially collapsed during the September 1997 earthquake [17]. Before this event the tympana walls were directly supporting the main reinforced-concrete beams of the roof structure. With such a structural solution there was no possibility to control the forces exchanged between tympanum and roof during a seismic action; furthermore, pounding of the beams against the walls was present. The SMAD characteristics seem to be particularly advantageous in this case. In fact traditional connections by steel bars between the Basilica's roof and the tympanum appear to be too stiff and dangerous for the tympanum itself.

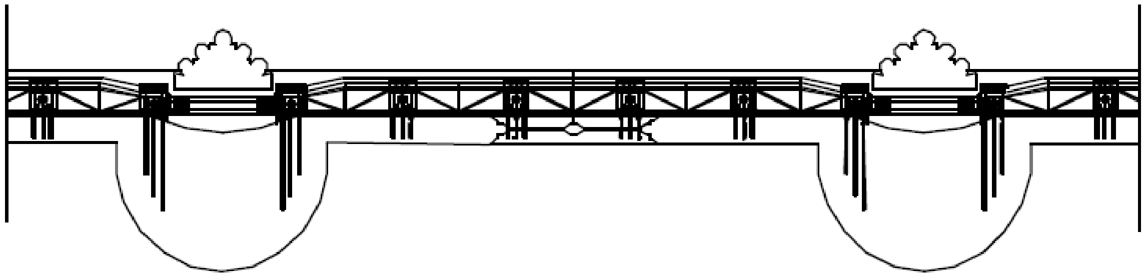


Fig. 6.5.3 *: One bay of the steel beam with couples of STU at each end*

6.5.3.2 The intervention with shock transmission units in the nave

The intervention in the nave consist in the employ of a steel girder made by a horizontal plate with an above trussed ribbing, placed on the cornice located at 7m from the pavement all along the Basilica from the façade to the apse (figure 6.5.1), and passing through the narrow chinks behind the pillars (figure 6.5.3). The section profile of this beam is shaped in order not to be seen by people standing in the nave. The beam is locally fixed in many points all along the walls, in connection with the pillars and in the transept corners. These links to the wall are carried out through suitable fixing plates on which the girder is leaned with the interposition of PTFE elements and horizontally fixed by pins. These pins do not allow any horizontal movements in the transept and in the apse area, while all along the nave, where the considerable length could cause movements due to thermal variations, such links allow longitudinal displacements. Thus in the nave the steel beam is divided in portions corresponding to the bays (figure 6.5.3) connected each other, behind the pillars, by a couple of STUs to guarantee a global stiff linking during seismic events (figure 6.5.4). Design forces of the two types of STUs used are 22 t and 30 t. Because of the particular position of the installation, stainless steel is used both for the beam and for the STU. Moreover, to avoid the danger of percolation on the Giotto's frescos as well as to reduce maintenance, the fluid used inside the STUs is not a low viscosity oil (often used for bridge applications), but a high viscosity silicon putty.

Figure 6.5.4 *The three sets of installed SMADs* *Figure 6.5.5The installed STUs.*

6.5.3.3 The intervention with shape memory alloy devices on the transept tympana

In order to modify the roof-tympanum interaction, the roof structure was disconnected from the tympanum wall and a new concrete truss was built to support the roof beams and to bring the roof vertical loads directly on the transept lateral walls. At the same time it was rebuild the collapsed portion of the left tympanum (using stones from the original quarry) and removed the Figure 6.5.4 : The installed STUs. Figure 6.5.5 : The three sets of installed SMADs deformation that both the transept tympana suffered. Than the new concrete truss was connected to the tympanum walls by SMADs spaced 50cm apart, in such a way to better

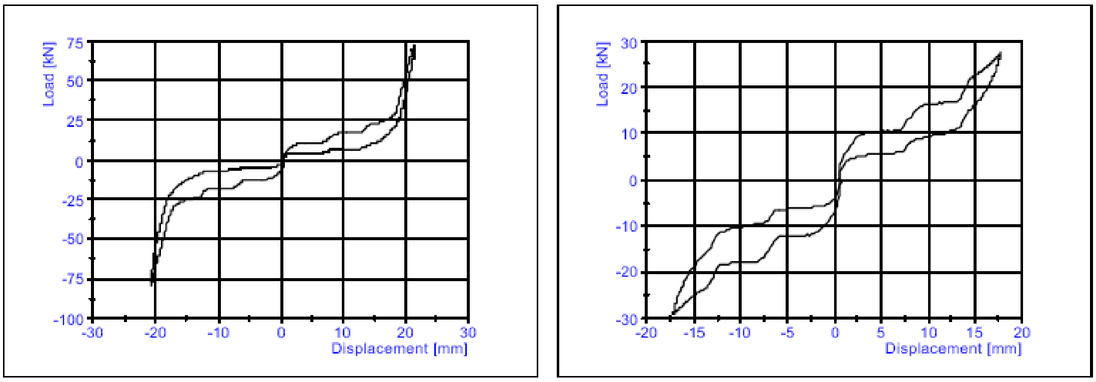


Figure 6.5.7 *: Experimental tests, force vs. displacement loops, on “multi-plateau” “self-balancing” SMAD prototypes*

distribute the exchanged forces. Three different groups of devices were designed and installed, in order to take into account the different properties required as the distance from the transept lateral walls increases up to the roof top. The different force-displacement behaviour of these groups can be perceived by their different length (Figure 6.5.5). Numerical analyses were carried out to select the

SMAD properties [18]. On the basis of the results of numerical analyses and experimental tests, it was chosen to employ mainly SMADs of the “multi-plateau” type (figure 6), obtained using three sets of SMA wires with different lengths, working in sequence for different values of the external action.

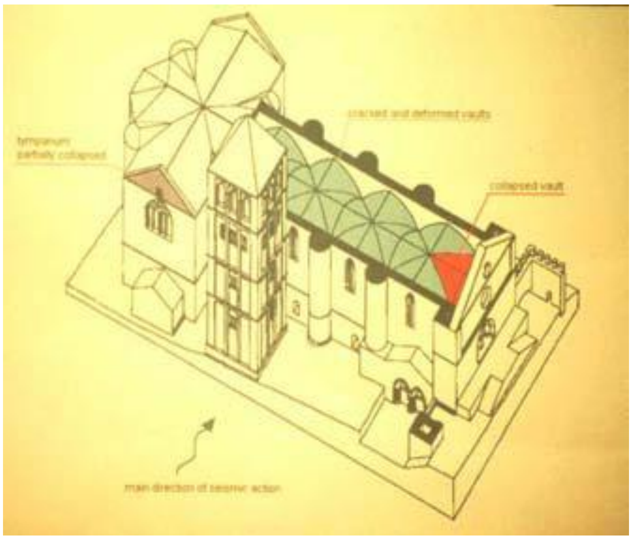


Figure 6.5.8The trussed steel beams placed over the cornice inside the Basilica

Figure 6.5.9 The Collapsed Vaults and the damaged tympanum

6.5.3.4 The reinforcement of the vaults

After a long series of tests was agreed that composite materials be used to strengthen the vaults, to create a series of thin little ribs following a pattern typical of Gothic structures on the vaults extrados (*Figure 6.5.9*) and leaving the original structure clearly visible.

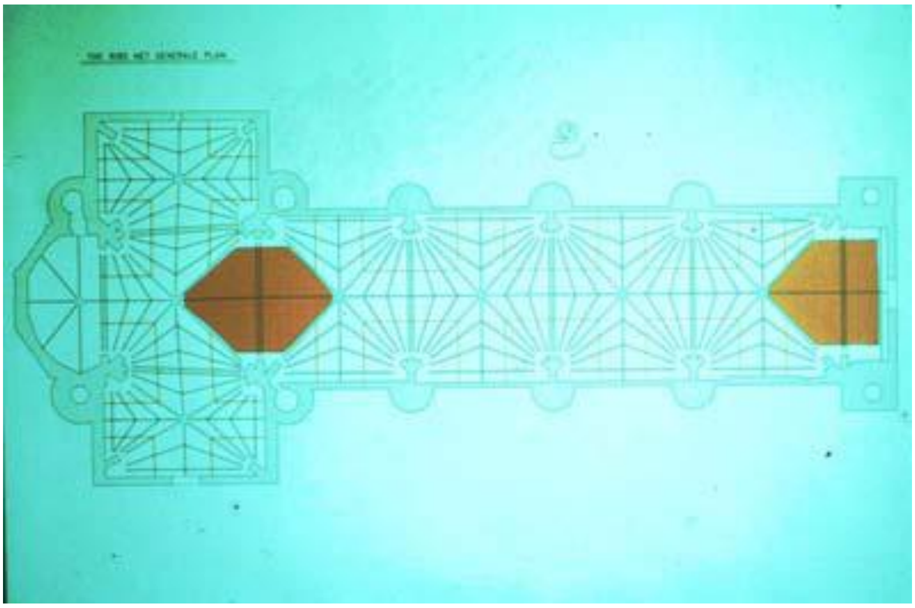


Figure 6.5.9 *The pattern followed by the new ribs placed on the extrados of the damaged vaults*

These ribs are built in situ, so that it is possible to follow the deformed shape of the vaults; whilst the width of the ribs remains constant, the height may then change in relation to the deformation of the vaults *Figure 6.5.10*).

The ribs are made of aramid fibres bedded in epoxy resins around a central timber nucleus. These fibres are light, very strong (the tensile strength of this fibre is 30.000 Kg/cm2 and that of the fibre with resin is about 14.000 Kg/cm2) and less stiff than steel, the elasticity modulus of the fibre being 1.200.000 Kg/cm2 and that of the fibres with resin 600.000 Kg/cm2.

Figure 6.5.10 *The ribs covered with aramidic fiber tissue* ***Figure 6.5.11*** *The tie bars and springs*

which connect the ribs to the roof

The ribs are connected to a system of tie bars, which are anchored to the roof. Each tie bar includes a spring, similar to the solution adopted for urgent measures (*Figure 6.5.11*). This reinforcement aims to reduce the deformability under seismic forces. The reinforcement of the vaults' cracked structure, has been completed by injections using a mortar salt-free and compatible with the frescoes, sufficiently fluid to penetrate and diffuse to all the cracks and microcracks, capable of being injected in dry masonry (no use of water is allowed) and having good strength and bond capacity so that a structural continuity through the cracks may be established.

6.6 Strengthening of San Lorenzo Cathedral with Post-Tensioning, Perugia, Italy

The San Lorenzo Cathedral was built between the 14th and 15th centuries. Important consolidation works carried out between 1633-1641 ,since, because of the weight of the roof, the columns adn the walls had diverged. Seismic movements in 1983 caused further damage which endangered the stability of the monument. A new consolidation and restoration project was decided after a thorough historical survey of the original and the lower tie beam and the central king post with two twin 0 36 mm Dywidag bars each. The column capitals showed displacements of up tp 26 cm. A new system of transverse and longitudinal ties made of Dywidag bars was introduced into the columns and walls to prevent any side displacements. The transverse tie members were prestressed and tied back to the lateral walls in order to apply to the whole structure a system of acting forces capable of counter acting the thrusts of the vaults.

6.7. Strengthening of GPO Tower with Post-Tensioning, Sydney

The general Post Office in Sydney, a more than one hundred year old sandstone masonry building, was undergone a massive restoration both inside and outside. As part of this restoration, the GPO Tower will be strengthened with four vertical post-tensioning tendons, 19 diameter 0.5" strands each, and a number of horizontal prestressing bars diameter 35mm at floor levels (Figure 6.7.1). The vertical tendons will be placed in holes diameter 100mm drilled from the top through the sandstone columns at the corner of the tower. Special steel chairs will be used to anchor the tendons and spread the anchorage forces of 1,771 kN (400 kips). The anchorages of the unbonded tendons allow for monitoring and adjustment of the tendon forces to compensate volume changes of the sandstone, if necessary. The entire restoration was expected to take five years and installation and stressing of the tendons was scheduled for 1990.

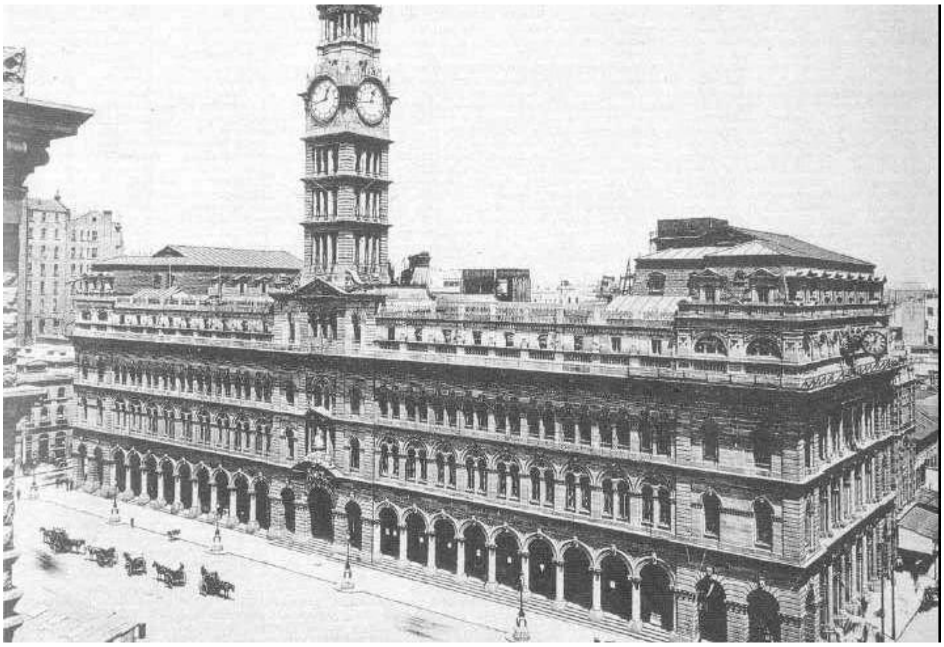


Figure. 6.7.1*: Strengthening of GPO Tower, Sydney General Post Office Building.*

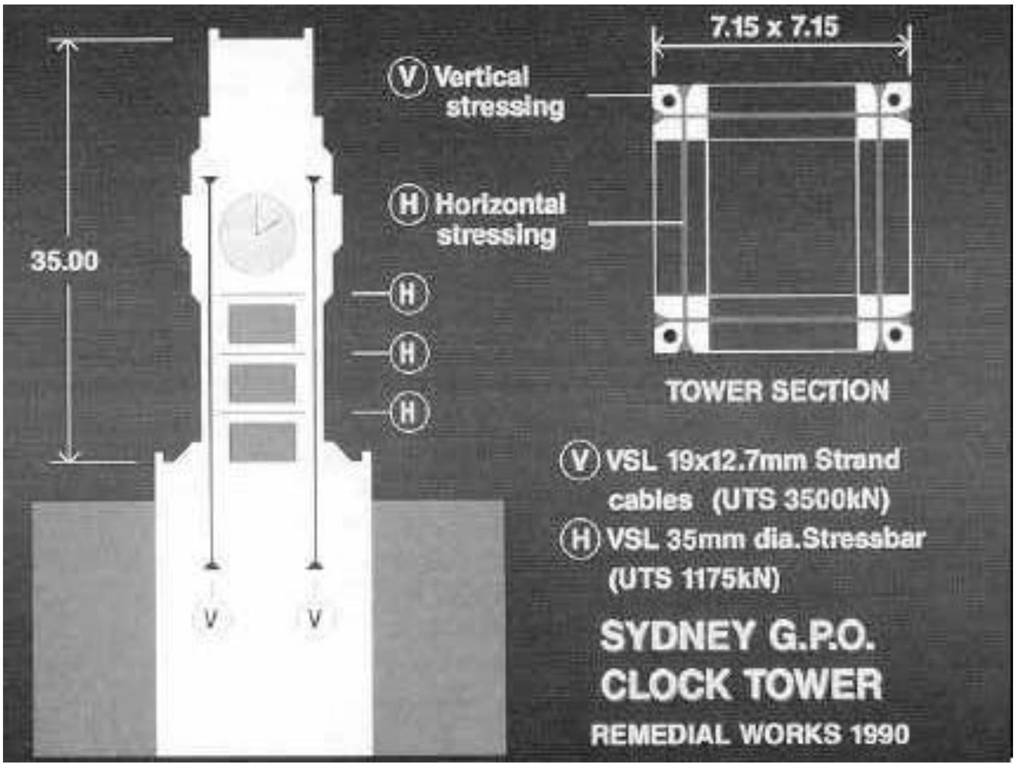


Figure 6.7.2 *Tendon layout in tower, courtesy of McBean & Crisp, Pty, Ltd.*

6.8. Chimney of Monastery of Arouca

The chimney is made of three brick masonry walls, making a tapered channel section leaning against a thick three stone granite lintels, see (Figure 6.8.1) The lintels are supported in stone columns and corbels. Hidden by the plaster, internal brick arches with 0.25 m thickness were

faiming at reducing the bending / tying load of the stone lintels by transmitting part of the load from the walls directly to the columns. In order to uncover the hidden structure of the arches, four inspection openings were made in the plaster. It could be observed that the rubble masonry fill between the arches and the lintels was not separated from the arches, as expected from good building practice. Two other defects were found, namely: (a) the main arch is asymmetrical with respect to the lintel; (b) the cross section of the left side lintel has a tapered shape, with a height reduction towards the external wall support.

The structure of the chimney is complemented by a set of iron ties, distributed along the height and inside the chimney. These ties aim at stabilizing the main wall, which is inclined about 15º with respect to the vertical position. Finally, two iron ties are also present inside the chimney, at the column corners and aligned at 45º with the lintels. These ties are part of the original fabric and it is likely that their function was to help resisting the thrust of the system of arches / lintels. It is noted that an iron cramp was added to connect externally the main lintel and the left lintel. The main damage exhibited by the chimney consists of a sudden diagonal crack, which appeared suddenly in the main lintel, close to the right support, and resulted in temporary propping of the structure. This crack intercepts the anchoring zone of the 45º iron tie that connects the main lintel and the right lintel, which is a singular and weaker part of the lintel.

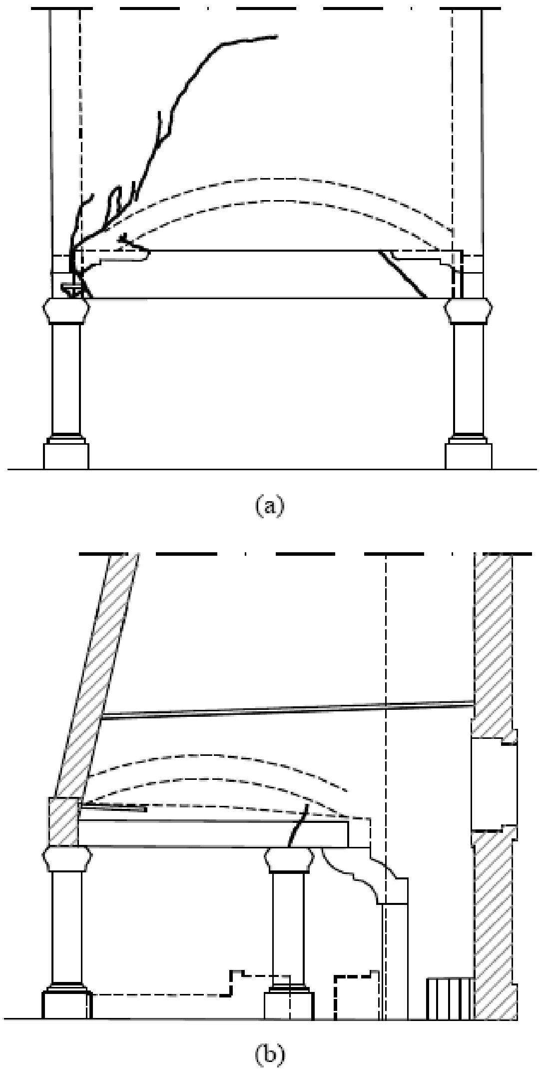


***Figure 6.8.1** Geometry and observed cracking patterns: (a) main wall; (b) left side wall.*

It is also noted that this tie is corroded close to the anchoring zone. The main lintel exhibits also several cracks close to the left support and in the vicinity of the iron cramp, which is severely corroded. In the masonry wall above the main lintel, a set of diagonal cracks is present. The inspection openings in the plaster indicate that the cracks do not intercept the hidden masonry arch, but they run through the arch extrados, see (Figure 6a). This set of cracks represents significant danger and a pre-collapse situation of the left support, with a failure mechanism involving rotation of the wall with a hinge forming at the right support. It is noted that an ancient crack is also present in the left side linter and wall, probably due to the reduction of height of the stone lintel in the corbel region, as discussed above, see (Figure 6.8).1b. As a result of this crack, two stone columns under each side lintel were added to the structure in an unknown date. In order to complete the diagnosis and safety evaluation, two three-dimensional models of the chimney were prepared aiming at simulating extreme possibilities, which take into account the fact that there is not separation between the arches and the material filling the space between the arches and the lintels. The two extreme possibilities regarding the arching effect in the walls are considered here. Therefore, the first model does not include the filling material under the arches (Model 1) and the second model considers that the walls are fully supported in the lintels (Model 2). Obviously, Model 2 is more unfavorable for the stone lintels, being the most conservative approach. The models are made of quadratic solid finite elements (bricks and wedges), with approximately 500 elements and 3805 nodes, making a total of 11415 degrees of freedom. Non-linear material properties have been considered both in tension and compression. Two loads have been considered in the analysis, one due to the self-weight of the chimney and another due to the load The linear elastic results of Model 1 and Model 2 indicated that the model without the internal arching system leads to the most unfavorable loading conditions, as expected. Therefore, this model has been selected for performing a non-linear analysis up to the collapse of the structure. Structural collapse was found for a load factor of 2.04, where the load factor represents the ratio between the applied loads and the original reference loads. For the ultimate load factor, the most damaged zones in the masonry walls occur close to the supports of the side lintels, see Figure 6.8.2a. This damage is both in tension and compression. Figure 6.8.2. b illustrates the damage (measured by the maximum principal strains) for the lintels-columns set, which clearly defines the collapse mechanism. Three plastic hinges appeared in the side lintels, one hinge at midspan with cracking at the lower face of the lintels (positive bending moments), and two hinges at the supports with cracking at the upper face of the lintels (negative bending moments).

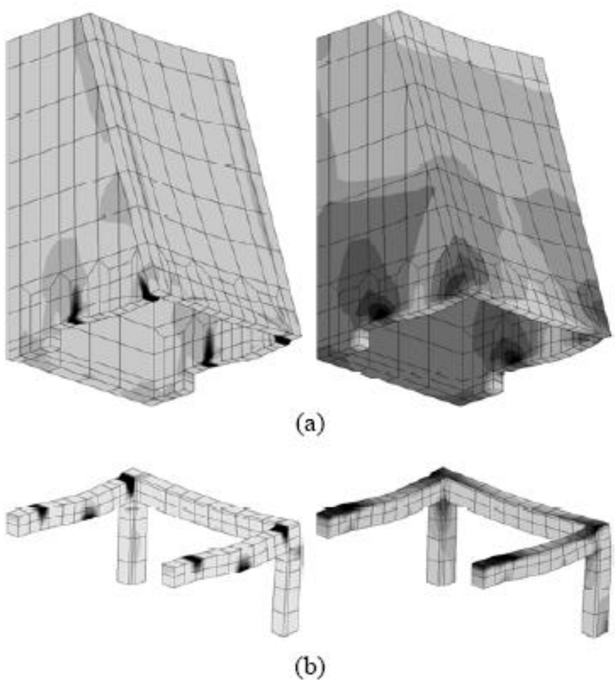


***Figure 6.8.2 .** Principal strains plotted in the deformed mesh configuration for Model 2, using non-linear analysis and a load factor of 2.0: (a) brick masonry part of the model; (b) stone columns and lintels. The maximum (tension) strains are plotted in the left and the minimum (compression) strains are plotted in the right. Results are dimensionless and darker color indicates higher damage.*

Figure 6.8.3 shows the force-displacement diagram for the mid-span of the side lintels, where the non-linear behavior is clearly visible. The global response is approximately linear until a load factor of 1.0. Afterwards, a progressive non-linear response dominates until collapse, followed by a descending branch (softening regime) captured only with a reduction of the applied load. Obviously, in a real physical situation a load reduction would be impossible and the chimney would just collapse in an uncontrolled manner.

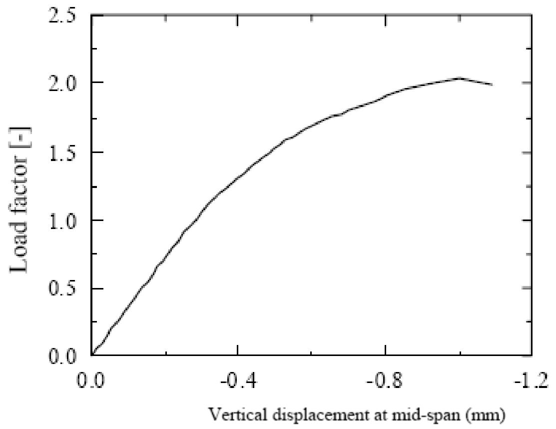


Figure 6.8.3 *Force-displacement diagram for the non-linear analysis (internal arching system ignored). Vertical displacement measured at mid-span of a side lintel.*

The results of the structural analysis clearly indicate that the sudden collapse of the main lintel is not due exclusively for structural reasons, being probably triggered by corrosion of the tie that connects the main lintel and the right side lintel (here, it is noted that the environment is aggressive due to rising damp, salt and organic materials from the old kitchen activity).

Certainly the local discontinuity associated with the hole for anchoring the iron tie also contributes to weaken the main lintel. The non-linear analysis carried out indicated that, in the most unfavorable conditions (without the internal arching action), the safety factor of the structure is 2.0 and, even then, possible collapse would occur in the side lintels and not in the main lintel.

The structural analysis results justify the ancient damage in the left side lintel. In fact, the numerical simulation does not take into account the cross section reduction in the left side lintel (the height varies linearly from 0.46 m to 0.30 m), which would reduce the safety factor significantly. This justifies the remedial measures adopted in the past by adding new stone columns close to the back supports of the side lintels. Finally, the cracks and damage observed in the main lintel close to the left support are due to the corrosion of the iron cramp and also to the collapse of the right support, as the rotation of the main lintel was responsible for the long crack along the extrados of the internal arch in the main wall. The solution for repair consists of strengthening and repairing the main lintel including:

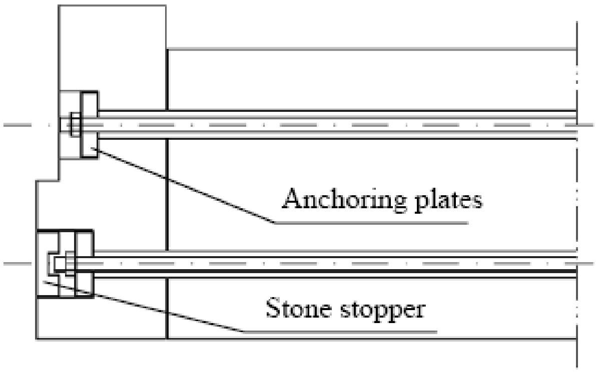
(a) reconstitution of the original stone integrity by injection of epoxy resins;

(b) hole drilling of the stone along its full length (4.70 m);

(c) insertion of bars and injection of the hole.

Figure 6.8.4 illustrates various details of the solution, which includes two stainless steel rods with a diameter of 25 mm as internal ties / reinforcement of the granite lintel.

The ties were designed after the integration of the tensile stresses of the linear elastic results for the numerical model without arching action, which is conservative. These rods are inserted in drilled holes of 50 mm and are provided with anchoring plates of 120 mm. After adjustment of the bolts, the drilled holes are injected with fluid lime mortar (Albaria Iniezione 200). Stone stoppers at both ends of the bottom tie are also included so that the anchoring plates are not visible. The stoppers are glued with epoxy resin and are made from the actual core removed from the lintel, after cutting. For the top tie, this operation is not needed because the surface finishing is plaster. It is noted that the usage of stone stoppers in both ends of the ties requires the drilling to be executed from both sides, which requires precision and qualified workers.



(a)

(b)

***Figure 6.8.4**. Aspects of the works carried out in the chimney:*
(a) detail of anchoring zone; (b) conclusion of the works.

6.9. Strengthening of Central Tower Foundations, York Minster Monastery

6.9.1 General

York Minster is the largest authentic Gothic church in Northern Europe. York's original (Anglo Saxon) Minster was severely damaged in 1069, during the aftermath of the Norman Conquest. The new Norman Archbishop (Thomas of Bayeux) tried to repair it, but a Danish raid in 1075 seems to have destroyed it completely. The exact site of this building is unknown.

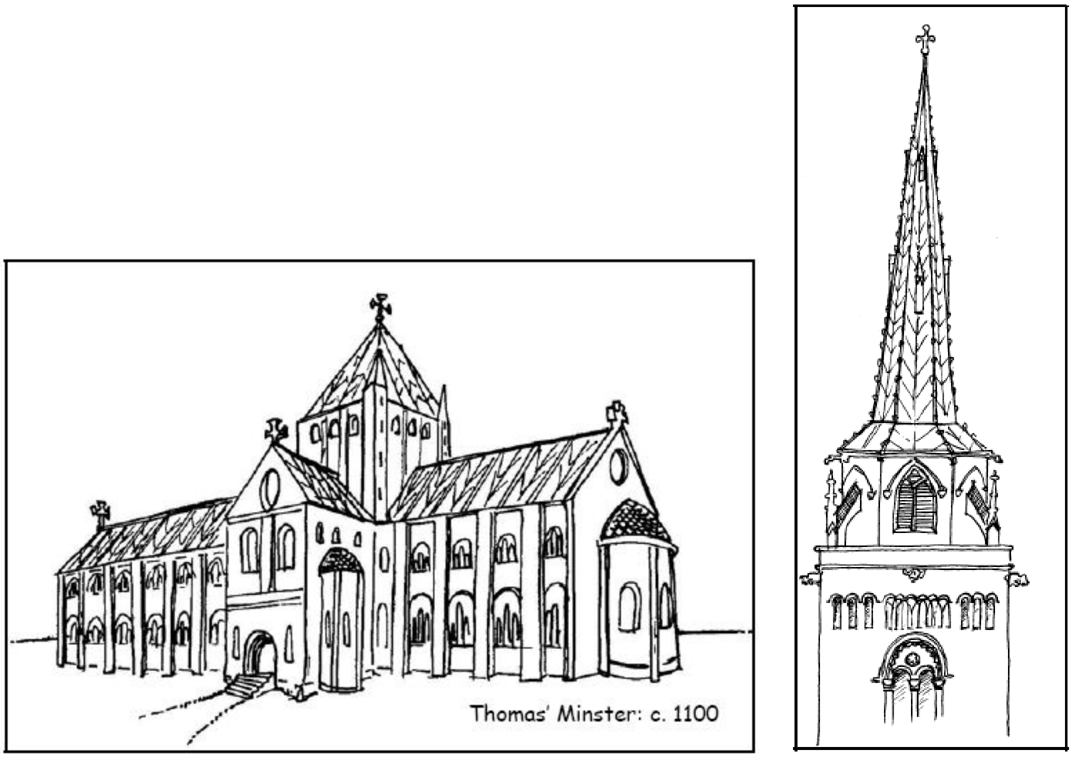


Figure 6.9.1 *York Minster*

The Central Tower

The Norman tower was altered about the same time as the transepts. We know it was given a belfry, and possibly a spire, but this version collapsed in 1407, and work on the present structure could only begin once the four massive piers had been strengthened. It was certainly intended to be taller, but it soon became clear that the foundations were unstable and it was capped at its present height of sixty metres in about 1470. Meanwhile, the two western towers were also being finished, and the whole cathedral was declared complete in July, 1472.

Below the Central Tower stands the magnificent Quire Screen, mounted with fifteen statues of Kings of England from William I to Henry VI, emphasising the link between Church and State, and the legitimacy of the Lancastrian succession. The figures are all fifteenth century, except for that of Henry VI, which is a much later replacement.

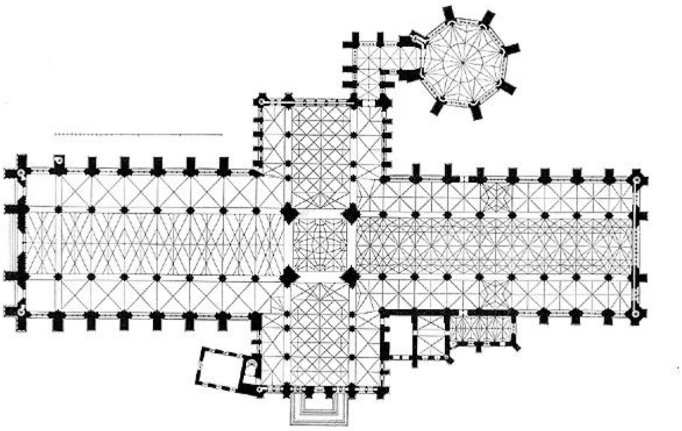


Figure 6.9.2*: The cruciform plan of York Minster*

Figure 6.9.3 *York Minster*

6.9.2 Structural Interventions to Improve the Foundations

As fresh cracks were found in the shaft of the north-east pier, this was exlored first. Severe shear cracks were exposed and glass tell-tales were fixed-those breaking within six months indicated active sttlements of the foundations, as suggested in freshness of the upper cracks. The lowest work is eleventh century followed by thirteenth century and the upper work is fifteenth century. The leventh century masons' technique of diagonal axe-cutting shows clearly.

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Figure 6.9.4 *North-east pier foundation, York Minster, England*

Similar shear cracks were found on the west side. The strenght of the horizontal strutting should be noted with the screws and plates for applying pressure to contain the masonry. The polythene pipes are for injection of liquid grout

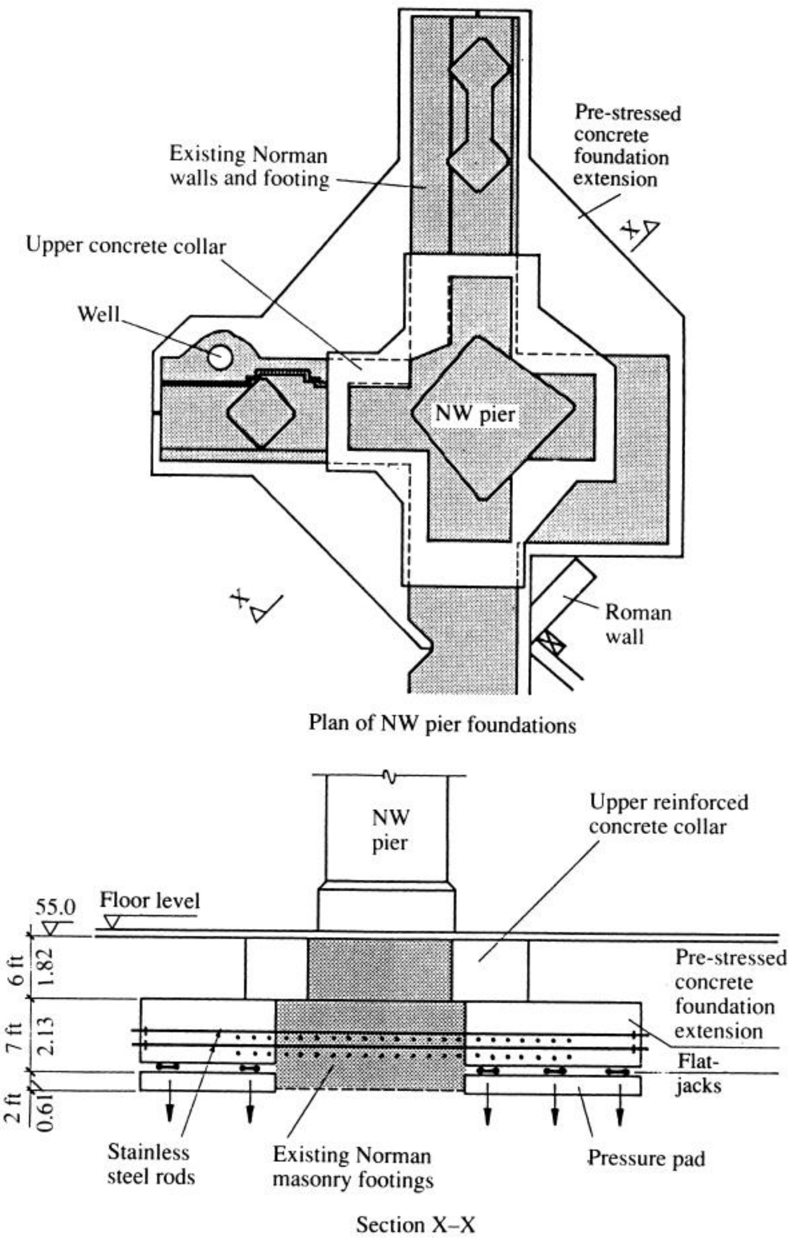

Figure 6.9.4 *Central Tower Foundation, York Minster, England (Vourtesy of Ove Arup & Partners)*

Plan of north-west pier foundation showing stainless post-tensioned layout. The project was to enlarge the area of foundations making them symmetrical about the line of thrust, so avoiding eccentric loads and dangerous pressures on the soil.

Section through north-west pier foundation. The existing masonry was contained with the new concrete. The flat-jacks were slowly inflated to prestress the soil and equlize loads carried by old adn new work.

Figure 6.9.5 *Enlarged foundations, central tower, York Minster, England (Courtesy of Shepherd Building Group, Ltd)*

A general view showing the enlarged independent foundations of the central piers which are linked to adjacent nave choir and transept aisle piers. The concrete rings to prevent bursting are above the new foundation proper. Measuring points are fixed on to each foundation movements being checked each week. Prestressed stainlee steel reinforcing rods were inserted diagonally under the main piers, 100 to each pier, in holes drilled through the original masonry.

Drilling Works

Horizontal shots of up to 20 m (66ft) length have been carried out effectievly on the central tower of York Minster through large block of hard limestone, after preliminary grouting. If a shot went off alignment, the drill was withdrawn and the hole was grouted with rapid-hardening cement. The drill used a crown-shaped drilling head with a special guiding mandrel. Horizontal shots of 15 m (49ft) length in fissured and disintegrated masonry in the Norman foundations had to be re-drilled more than once with a wide range of dirlled heads.

The 16000 ton central tower is carried by four main columns. Each column has, as its effective foundation, a cruciform of Norman strip footings. Taking into account eccentricity, the bearing stresses on the ground are much heavier than desirable. The effective area of each column is increased and the eccentricity reduced by adding slabs of concrete 2.1 m (6 ft 11in) thick in each re-entrant corner. The whole compocite foundation is reinforced by post-tensioned 32 mm (1.3 in) diameter stainless steel rods which form two layers in each elevation. This involves drilling holes up to 15 m (49ft) horizontally through the Norman foundations with a high standard of accuracy.

The centers of the rods are 380 mm (1ft 3in) apart horizontally and 450 mm (1ft 6in) apart vertically in each direction. However, since one layer in each direction bisects the two layers in the orthogonal direction, the actual vertical distance between the layers is only 225 mm (9in). Form these figures it can be seen that the target after 12 to 15 m (40 to 50 ft) of drilling, allowing workable tolerances, is a rectangle only 300x 225 mm (12x9 in) Such accuracy has proved very difficult to achieve.

For several months, before any drilling was attempted on the foundations, similar requirements had been carried out at high levels in the Central and Western towers. The walls of these towers have been protected from further cracking by reinfrocing them horizontally in certain areas with stainless steel rods. This involved drilling holes 20 m (66ft) in length but more rolerance was allowed than was later rquired for foundations. The work on the towers, by comparison went very well and success gave confidence for the drilling below-a confidence which nearly turned to despair when several months after starting on the foundations drilling, only 8 holes had been succesful out of a total of 38 attempts (400 to do). The rason why drilling in the walls of the fabric was easier than in the foundations are based on the two things essential for an accurate drilling:

- The nature of the material being drilled
- The use of the correct equipment to suit the material

The ideal material is a sound, homogeneous rock containing no fissures or voids. Mass concrete od course, is equally acceptable.

The Minster fabric, in general, is well-bonded magnesian limestone masonry and where rubble cores occur, they are well compacted and reasonably well mortared. On the other hand, the Norman mortar raft is far from ideal, for several reasons [24] .

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Figure 6.9.7 *Drilling bits, York Minster, England (Courtesy: Shepherd Building Group Ltd)*

All the drill-bead components and assemblies are powered pneumatically for both rotary and rotary percussive drilling units.Advantages and disadvantages- high speed rotary percussive is the most economical means of drilling: tungsten carbide tipped bits are much cheaper than diamond crowns. This type of drilling is fast but demands a homogeneous mass, prefarably of like material, to provide for accuracy in operation where long holes are required horizontally in masonry.Diamond core drilling is rather slower and thus more costly, but under adverse conditions in bad materials is more accurate in maintaining alignment.

Figure 6.9.8 *Enlarged foundations, central tower, York Minster, England*

After drilling through 15 m of concrete, stone, mortar and wood the hole is lined with a pipe to prevent local collapse and the tensile stainless steel bars inserted. These were then streched and given a final tensioning after two months and then grouted. The probes are all ready to receive the liquid grout injection.

CHAPTER 7

SUMMARY, DISCUSSIONS, CONCLUSIONS ....

7.1 Discussions

Metals were mainly played an auxiliary role in architecture before 1800. They were used for bonding masonry with **dowels** and **clamps**, for **tension** members with chains strengthening domes, **tie rods** across arches to reinforce the vaults, and for roofing, doors, windows, and decoration. **Cast iron**, the first metal that could be substituted for traditional structural materials, was used in bridge building as early as 1779. Its ability to **bear** loads and to be produced in an **endless** variety of forms, in addition to its resistance to fire and **corrosion**, quickly encouraged architectural adaptations, first as columns and arches and afterward in skeletal structures. Cast iron has much more **compressive** than **tensile strength** for example, it works better as a small column than as a beam. Cast iron was largely replaced in the late 19th century by **steel**, which is more uniformly strong, elastic, and workable, and its high resistance in all stresses (i.e tension, compression and bending) can be closely calculated.

7.1.1 Common Problems of exposed Steelwork

Exposed steelwork was (and is) normally given a protective coating of paint. The paint film does however break down with time under the influence of water, air and ultraviolet light, and once even the smallest area of metal is bared, corrosion sets in. Because the rust (iron oxide) occupies a higher position in the galvanic series than iron, the corrosion can, and will, spread under the remaining intact paint film. Because the rust occupies a greater volume than the steel, the paint film then gets lifted from the metal.

On surfaces that get washed by rain and dried by wind, this will happen within a certain timespan, dictated by the resistance of the paint to general atmospheric weathering. In locations on the structure which are sheltered from the wind, and from which water cannot freely drain, moisture, laden with atmospheric pollutants such as sulphates and chlorides, will be trapped and remain for prolonged periods, during which corrosion of the areas that have lost their paint will continue, for much longer than on the free-draining, wind-dried surfaces. At any one time, corrosion damage will therefore be far more severe in such moisture traps, even if no great volumes of water are present; *moisture from condensation is enough to generate corrosion*.

The detail design of many steel structures, from the first half of the twentieth century or earlier, did unfortunately not take this into account; moreover, the details that allow moisture to lodge and remain, also make cleaning and re-painting exceedingly difficult and, sometimes, impossible. An example of such details is the common truss members of two angles riveted together, back to back, with a gap, dictated by the thickness of the gusset plates, of only 10 or 12 mm. In such structures corrosion may sometimes cause significant structural weakening.

Railway station train sheds are among structures, that were subjected to particularly severe conditions in the past, being exposed to exhaust steam, mixed with smoke containing sulphur dioxide. On the credit side, they were usually subject to a regular maintenance routine, during which re-painting would be carried out, as well as circumstances permitted, even if some of the gaps and crevices could not be properly treated.

7.1.2 Common Problems of Steelwork Encased in Masonry

In buildings with an internal iron or steel structure and external loadbearing walls, the beam ends would be supported on the walls, usually through a padstone, to a depth of between half and two-thirds of the wall thickness. They would thus have a fair thickness of masonry protecting them from the weather. With the introduction of the full steel frame, the external walls could be made thinner and often, in order to reduce the projection of the steel stanchions inside the building, only a minimum thickness of masonry was provided on the outside of the stanchions and tie beams. The protection of the metal, usually given only a token coat of paint, was then heavily dependent on the masonry. Natural stone and plain brick masonry kept the rainwater away from the steelwork by repelling some of it, but temporarily absorbing a substantial proportion of it, which it would subsequently release back into the air by evaporation. In this way it acted like a heavy overcoat, as opposed to the 'plastic mac'-action of modern curtain walling, which needs to be absolutely impermeable. Glazed brick and terracotta will, unless the pointing is perfect, suffer water penetration by capillary action through the joints, with the glazing impeding later drying out by evaporation. Heavy corrosion may also have taken place, due to badly maintained gutters and downpipes. If downpipes are blocked, the gutters overflow and the water finds its way into the masonry. This also happens when lead-lined or asphalted gutters are not maintained. The water then finds its way to the (normally unpainted) metal and as the masonry is never impermeable enough to exclude air, corrosion will then progress, unseen, for some time. The time when signs of trouble appear depends on how tightly the masonry has been fitted round the steelwork: As mentioned above, rust occupies a much greater volume than the parent metal. If all gaps between brick (or stone) and steel are fully filled with mortar, rusting may not start until the lime or cement has carbonated but after that, the rust will soon fill the pores in the mortar and as more rust is formed, it will exert pressure on the masonry, which will then crack (Figure 7.1.1*a*). If, however, gaps, say 10–20 mm wide, have been left between the flanges of the frame and the back of the masonry, the rust can have expanded into the gap for a long time (50 years or more) before the gap is filled, but once this has taken place, the rust will exert pressure on the masonry and crack it (Figure 7.1.1.*b*). As on the same building there may be gaps in some locations and not in others and as the width of the gaps may vary between 20 mm and nothing, there is no way in which a visual inspection of the exterior can establish the existence or the extent of a corrosion problem on such a building. Another difficulty is that corrosion does not necessarily occur where the water gets in; it can run down a stanchion, causing little damage, until it finds a ledge on which it can collect, and this is where rusting occurs. Cracking may also be due to rusting of iron cramps forming part of the detailing of the masonry cladding, without necessarily indicating corrosion of the main steel structure. If some cracking is evident, a limited opening up may indicate how far the rusting extends beyond the cracking *in that location*, but in that location only. A possible method of ascertaining an unseen problem would be to carefully drill a core hole

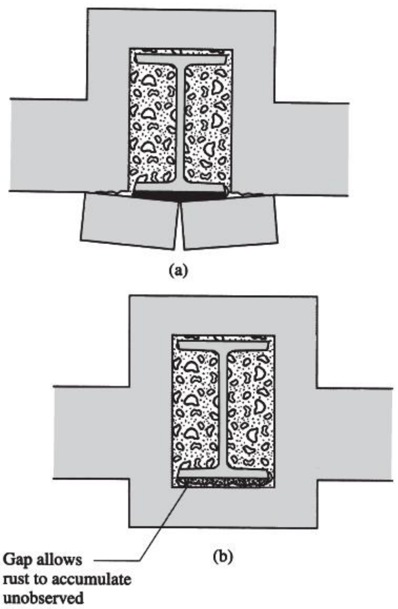

Figure 7.1.1 *Rusting steel member encased in masonry: (a) Gaps between steel and masonry fully mortared; (b) Gaps left empty between steel and masonry.*

perpendicular to the face until the core barrel touches the face of the metal. By inspection of the end of the core *and* the bottom of the hole, it should be possible to ascertain the presence and thickness of any build-up of rust. When reinstating such an inspection hole, one must not create an entry for water where there was none before. Absolute assurance of the absence of any problem does however, in principle, require complete removal of the masonry cladding with inevitable consequential damage and no guarantee that it will be put back as well as it was originally built (leaving aside the difficulty of procuring matching masonry material to replace that broken during the removal).

7.1.3 Problems with Cavity-ties

Cavity brickwork is found as cladding on multi-storey, framed buildings, as well as in load-bearing walls of dwelling houses.

On older houses, inadequate corrosion protection of the ties, together with the use of 'black ash' mortar in the outer leaf, can lead to the ties rusting. The ties, then used, were cut from 25 _ 4 mm flat steel, twisted in the middle and split at the ends; they introduced a substantial iron component in the bed-joints, and the rusting sometimes cracked the joints.

During the 1960s, ties made of galvanized wire, twisted into a butterfly shape, were introduced. Corrosion can again be a problem with these, but the main danger in this case is that they rust away almost completely, leaving the outer leaf with no connection to the backing structure. The most common defect with this type of cavity wall is, however, the failure of the bricklayer to put in the required number (usually specified as one for every 45 cm in every third course). Spacings of more than double the specified are not uncommon and have in some cases led to the outer leaf being sucked off a façade during high winds.

7.1.4 Cracks Due to Embedded Ironwork

In the past, stone masonry was sometimes built with embedded iron cramps or 'staples', which were intended to hold together two adjacent stones in areas where tension might occur, temporarily or permanently. Similarly, where arch or dome thrust needed to be restrained,

tie rods were provided within the thickness of the masonry. In both cases, the iron components were usually placed in horizontal rebates formed in the top surfaces of the stones and the ends were bent down into vertical holes.

In the case of dry-jointed masonry , rainwater is almost certain to find capillary paths to the iron and in combination with atmospheric oxygen cause rusting of the ironwork. This hazard is aggravated in exposed coastal situations, where salt-laden spray may be carried inland by wind. This is because the chlorides in the salt accelerate the corrosion (cf. the chloride effect on reinforcement corrosion; Embedded iron cramps can, if anything, cause more damage in mortar-jointed masonry than in dry-jointed. This is because the mortar, which originally was alkaline and therefore protected the iron, has been carbonated by carbon dioxide from the atmosphere and hence has lost its protective function, whilst it still confines the corrosion products. As rust takes up between 6 and 10 times the volume of the iron from which it is formed, very great expansive forces are created by the confinement resulting sometimes in spectacular displacements of large stones and/or gaping cracks.

To protect the ironwork against corrosion, it was often wrapped in thin sheet lead. Alternatively, molten lead may have been poured around the iron, fixing it in the holes and rebates at the same time as providing protection. However, the lead wrapping may not have been watertight to begin with or the mortar joints may have weathered with time and allowed lime-laden rainwater to get to the lead and corrode it and form pinholes through which water could get at the iron. In either case, the net result is the same: rust forms and spreads under the lead. The apparent effect is to make the ironwork expand (and, in the process, split the lead wrapping, if any).

Whilst this has been known to lift the entire structure above the ironwork, with the bed-joint just above the iron gaping open, it is more common to find that cracks have formed on the face of the stones (usually along straight horizontal lines) and

whole slivers of stone may have spalled off. Cracks from this cause can often be identified by the stones below being stained by rusty water leaking out from the iron.The location of embedded metalwork, even when it is not (yet) causing problems, can usually be found by the use of certain metal detectors. There are, however, some bricks and some sandstones which contain a significant amount of iron; this has been known to ‘confuse’ the instruments, so that they register metal where none is present and vice versa. They should therefore be ‘calibrated’ by a small local opening-up. Similar problems can be caused in steel-framed buildings with masonry cladding.

7.1.5 Radical remedial Treatment of Corrosion

Continuing corrosion requires a supply of oxygen and moisture and, furthermore, the presence of rust promotes corrosion, whereas the presence of certain other metals, such as zinc may provide some protection. It follows that to prevent further corrosion one must, as far as possible, exclude air and water and, to allow for imperfections in the exclusion system, remove existing rust and, preferably, arrange for some zinc to be in contact with the iron or steel. In principle this means the following treatment:

1. Identifying and eliminating, as far as possible, the source of the water that is causing the corrosion.

2. All surfaces must be cleaned down to bright metal.

3. A ‘zinc-rich’ priming paint must be applied *immediately*.

4. This priming coat must be protected with several coats of water- and airexcluding paint.

Cleaning is most effectively carried out by grit blasting, but even this cannot reach into the narrow gaps between ‘back-to-back’ angles and other crevices, resulting from unfortunate detailing, such as described above. These can only be cleaned by disassembling the whole structure. This would mean drilling out all rivets and, as riveting is no longer a ‘live’ trade, re-assembly would have to be done with some kind of bolt. This would destroy the authenticity and is therefore often not permissible on historical grounds. That apart, dismantling will almost certainly result in some damage, the repair of which will add to the already almost prohibitive cost. (An alternative may be to seal the gaps between the angles with a waterproof compound.) Radical remedial treatment is therefore rarely practicable and generally it is not justified.

7.1.6 ‘Palliative’ Treatment of Corrosion

If an exposed structure has survived 50 or a 100 years without suffering more than moderately severe corrosion and if it is not, in its future use, going to be subjected to a worse environment than in the past, cleaning with a de-scaling gun and vigorous wire-brushing will suffice as preparation for a paint system that will offer adequate protection for a good many years and which will be fairly easily maintainable (in contrast to the very special paints, referred to above).

For steelwork with ‘heavy’ masonry cladding, encasement in good quality concrete may be the answer. The masonry immediately in front of the steel would have to be removed and ‘thinned down’ to accommodate the concrete casing. The concrete would then be cast on to the cleaned steel and the masonry subsequently put back. This procedure may however entail some local damage to the masonry and consequential repair/replacement. Galvanic protection is sometimes advocated. If properly adjusted and monitored, it can be effective, but it appears not to be something that can be installed and left unattended. Whilst it may therefore be appropriate on a public monument, it may not appeal to the average building owner.

7.2 Repair & Strengthening Recommendations

7.2.1. General

Structural strengthening describes the process of upgrading the structural system of an existing building to improve performance under existing loads or to increase the strength of structural components to carry additional loads.

It is crucial to recognize that strengthening assessment and design is infinitely more complex than new construction. There are several unknown factors associated with the structural state such as continuity, load path, and material properties as well as the size and locations of previous interventions, existing reinforcement or prestressing. The degree to which the upgrade system and the existing structural elements share the loads also must be evaluated and addressed properly in the upgrade design, detailing, and implementation procedure.

Every thechnique has it's own specific advantages and specific side problems to be concerned. For example the use of threaded bars for repointing of mortar joints and transversal tying is very easy, but it may require particular aesthetical solutions [88].

***Figure 7.2.1 a**: Bending of reinforced steel bars [88]*

***b**: Tying with threaded bars[88]*

On the other hand, transversal tying reveals to be feasible, especially with regard to the short duration of the intervention.

According to [88], the which have been carried out on wall specimens show that the walls consolidated by the repointing of the mortar joints and with the transversal tying have shown compressive strengths and elastic moduli similar to those of the original ones. On the other hand, the features of the walls (in particular, the weakness of the internal core) led to consider the injection technique as the most effective intervention.

The transversal tying of walls, strongly reduced vertical and transversal strains at the peak stress. In particular, the transversal strain, thanks to the restraint effect of the ties, showed an average reduction, compared to the case of the unstrengthened walls, of about the 50% at the peak stress, and of about the 90% at the same stress level (see Table 7.2.1 and Figure 7.2.2, where the improvement in comparison with the injection cases is also shown) [88].

Table 7.3.1: Measured strain for tied and repointed walls, before and after intervention.

Wall	Vertical strain ‰			Horizontal strain ‰			Transversal strain ‰		
	Before at $f_{wc,0}$	After at $f_{wc,0}$	After at $f_{wc,s}$	Before at $f_{wc,0}$	After at $f_{wc,0}$	After at $f_{wc,s}$	Before at $f_{wc,0}$	After at $f_{wc,0}$	After at $f_{wc,s}$
2T	6.55	0.71	4.05	-0.70	-0.23	-7.09	-19.57	-2.02	-8.29
9T	4.12	0.80	3.04	-2.45	-0.11	-5.49	-10.61	-1.10	-6.34
11T	--	3.04	7.59	--	-1.43	-9.95	--	-2.57	-8.45
3R	3.19	2.78	10.45	-2.17	-2.06	-10.1	-9.84	-0.24	-13.45
7R	4.39	2.23	5.10	-5.06	-2.00	-9.27	-5.81	-2.32	-10.35
15R	--	2.36	7.90	--	-0.76	-5.57	--	-0.50	-14.01

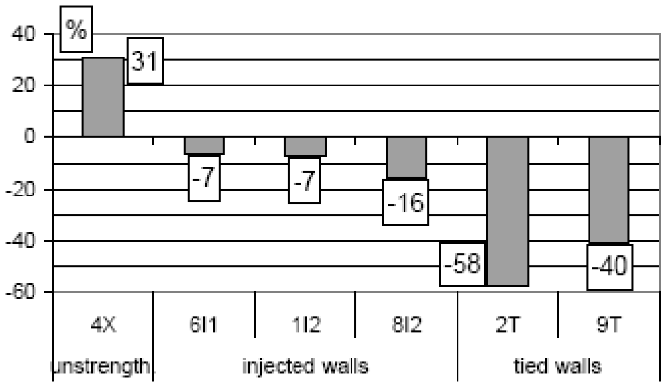

***Figure 7.3.**2: Transversal strain variation at the peak strength after injection and tying [88]*

The test results show that the best performances can be ascribed to the walls strengthened with **combined techniques**, especially when injections are involved in the intervention. The injection of grouts reveals to be the most effective technique in improving the ultimate load capacity of the walls (even more than the 50% of the original panels) and in improving the brittle mechanism of failure of the non consolidated walls. Moreover, increments of the modulus of elasticity

still compatible with the existing structures have been detected, but with significant reductions of the transversal dilation. Repointing and transversal tying reveals their efficiency mostly in terms of reduction of deformations. Nevertheless the highest performances is observed for the walls strengthened by the “integrated intervention”, that is by all the three techniques . Thus, the excavation phase during the repointing execution is effective in bettering the injection procedure, because it makes easier the hole drilling and the placement of the hoses; on the other hand, the filling of the joints can prevent the leakage of the grout. Furthermore, the repointing is effective in strengthening the outer stratum of the external leaves where, to a large extent, the influence of injections can be regarded as less strong. In the case of tying, the injection of grouts is able to connect the ties to the inner part of the wall, so that the adherence that develops can increase the confinement effects of the ties themselves. The studies and various applications prove that it is not possible to consider one simple technique for strengthening or restoring a masonry structural element or a building without employing other other techniques.

To discuss in a wider perspective , we must consider the causes of damages to the buildings because the decissions are affected directly by these causes which would be produced by increasing actions like forces, deformations, accelerations, or the change in the structural system like removal of a wall or an arch, addition of a floor, another structural element, or material deterioration thus the reduction of the strength, the stregnth (decay of the material characteristics, etc

This conveys us to the main idea underlying the philosophy of a succesfull and effective intervention with the most appropriate techniques for strengthening- restoring an existing historic building decided according to the specific needs of the building: **Proper assesment.**

Is it enough to consolidate the walls, or tying the arch or strengthening the foundations? One should consider the main causes of structural damage or risk of the buildings.

If the cause of the failure arises from the **direct actions of the static loads** which is the most common situation for historic masonry buildings, we hould consider the following actions:

- strengthening of the material by one of the techniques discussed before (i.e injection, repointing, transversal tying etc)
- stengthening of the main structural elements which a building is made of (i.e stirrups to contain lateral expansion in pillars and walls; ties or abutments to ensure the thrust in arches and vaults; etc.)
- strengthening of the buildings as a whole.

Whatever the case, it is more effective to improve the general overall structural behaviour of a building, as a whole, rather than to deal with local problems , and to to ensure good connections between the main structural elements (wall to wall, wall to floor) because eccentricities of the loads and thrusts of vaults are frequent and unavoidable. These connections can usually be achieved by means of the proper anchorage of the floor structures to the walls and the use of chains, ties, etc.

Other cases of failures may develop due to soil settlements for which the measures to be taken should be decided accordingly like:

1. Improve the soil's behaviour.

2. Modify the pressures under the foundations.
3. Improve the building's behaviour.

Modification of the pressure to the soil under foundations can be achieved by the folloeing acrions which are oftenly combined:

1. modification of the loads;
2. enlargement of the actual foundation;
3. transfer of the load to deeper strata (underpinning).

Reduction of the loads acting on the soils can be simply achieved using procedures such as lightening of the construction by the demolition of floors, the removal of heavy walls and their substitution with lighter structures in steel or concrete, or the removal of soil internally and externally along sections of the perimeter walls and replacing it with lighter materials. Where different settlements and particularly tilting phenomena have occurred, it is possible to stabilise or even reverse these changes by altering the load distribution so as to increase pressures in some zones and reduce in others. For the strengthening to counter the settlement problem sthe concept is based on the strengthening of the structure by means of:

- The formation of a series of boxes or honeycomb-type arrangements, in the basement usually in reinforced concrete, with solid or perforated walls to form a strong resistant box construction to reinforce the building.
- The insertion of braces and ties to walls or ceilings; ties are very useful for masonry buildings when there is a tendency for front walls to move out. This type of action is particularly efficient in buildings like towers where the ratio between the height and the base magnifies the effects of any small rotations in the foundations.
- A general stiffening of the construction as a whole, using additional reinforcement, inserted, for maximum efficiency, at various levels.

Another main cause of damage to buildings is the seismic actions and the measure to be ataken are simply:

- to improve the structural behaviour;
- to reduce the seismic effects.

Although, from a theoretical point of view, to strengthen the structure or to dissipate seismic energy can result in equivalent safety-level improvements, from a practical point of view, very often the former route of 'improving the structural behaviour 'is not only more reliable but alsoeasier to carry out, rather than the latter Attention must be paid to the stability of non-structural elements, such as entablatures, cornices, ridges, spires, etc. using, if need be, specialised connections, such as dowels, cramps, pins. etc.

For strengthening of of structural elements against seismic actions the following actions are recommended [89]

a- the “flexural” behaviour of the walls is characterised by cracks mainly concentrated under and over the windows the wall behaviour can be improved by ensuring good horizontal connections at floor level. This can be achieved by using “kerbs” formed with steel or synthetic bars injected in the masonry. Concrete “kerbs” can be structurally useful but often incompatible with the historical appearance and can be difficult to insert in the building; their use must therefore be limited to situations where it is not possible to ensure safety levels in more acceptable ways; it is always better, anyway, to lose the kerbs in the thickness of the masonry. Small x-frame steel braces can be inserted in the zones under the windows, which are often weak. Vertical bars or cables, often prestressed, may be placed through drilled holes ideally near the corners of the masonry to eliminate vertical tensile stresses produced in the walls by high seismic actions.

b- the “shear” behaviour of the walls is characterised by cracks mainly concentrated between the windows ; it is more favourable than the “flexural” behaviour and usually only occurs where the floor level connections have been reinforced. to improve the “shear” behaviour it is necessary to strengthen the masonry panels between the windows. This is generally a more difficult task as usually it requires major works which tend to affect the architectural value of the building. It usually involves the insertion of vertical or inclined bars or cables. In exceptional cases high strengthening may be obtained sandwiching the panel between two thin concrete slabs; this however should be regarded only as an extreme measure, to be used only when there is no other way of saving the building. Fortunately, if the masonry is of good quality, the resistance of these panels is sufficiently high by itself or if inadequate, can be improved simply by consolidating the material.

c-In the “arch effect” behaviour, the weakest zones are near the corners and under the roof, where the weight may be insufficient to ensure a vertical thrust; the insertions of bars, cables or of small kerbs may be required to provide an adequate “thrust”. Arches and vaults usually require to be strengthened generally in two situations: where there is a very flat central zone of the arch or vault and where any large movements of the springers can occur. In the first case, local shear forces and bending moments may compromise the stability. In the second case chains, ties, buttresses or similar reinforcement to stiffen the springers may be required to ensure an adequate thrust.

Besides ensuring the strength of each element it is usually necessary strengthening the building as a whole. The general policy is to ensure the proper working of all structural connections and to improve the general tensile strength in the critical zones.

7.2.2. Repair of Damage caused by embedded, rusting, Ironwork

Where there are iron cramps or tie bars within 150–200 mm from the face of exposed masonry, it is futile to hope that some ‘patent’ resin treatment will effectively stop further rusting, by excluding moisture. It will get in through weathered pointing in the joints, and the rust that has formed will attract and hold moisture. Once the corrosion has got hold, the cramps and tie-back bars will somehow have to be removed. Long bar-cramps in square or polygonal towers and spires can sometimes be partially removed by drilling out a long core of the masonry, which includes the straight part of the iron bar, its lead sheath and some of the surrounding stone. If the ends of the bar have been turned down, local ‘open surgery’is needed to remove them.

Whether the ironwork has to be replaced depends on its original intended function. Where it is clear that it was originally put in to mitigate the effects of differential settlement, which can now be considered complete, there should be no need for replacement, provided that such settlement is unlikely to resume, due to changes in subsoil and groundwater conditions. Where the iron cramps are tying back cantilevering corniches, they must be replaced. This will require temporary support of the cantilevering stonework. In church towers where there is a possibility of ringing of heavy or multiple bells, any reduction of tensile capacity is inadvisable. If any replacement cramps and/or tie bars are judged to be necessary, they should be of stainless steel and of similar tensile capacity (but not necessarily the same section) as the iron components that they replace.

7.2.3 Reversibility

There has been a number of instances when arches and vaults have been ‘strengthened’ by being suspended by grouted-in hanger bars from a concrete structure built above (and in many cases right on top of) the original masonry structure. The attraction of such an arrangement is that the concrete relieving structure can be designed and built to conform today’s codes of practice, thus circumventing the ambiguities in the original construction, e.g. if sufficient buttressing is not available, the relieving structure can be made to act as a beam.

Such interventions preserve the appearance of the underside of the arch or vault, but change the function from structural to merely decorative. They should therefore be considered as a last resort, particularly when they involve casting or spraying the concrete directly on to the masonry, as in that case the intervention becomes irreversible. If provision of such an overlay structure, after careful consideration, is found to be the only solution, a separation membrane e.g. polyethylene sheeting should be used.

7.3 Conclusions

Masonry is one of the oldest cosntruction material and tracing back the use of masonry will take us to the history of the civilization. It is not an exageration to say that masonry represents civilization. From the first ruble walls of the Stone Age to contemporary brick vener systems, it is masonry the architects and engineers turn to when performance, solidity, aesthetics are sought the evolution of which is parallel to the one of civilation.

If a structure is established through the procedures described before to be not sufficient to sustain it's life, decissions should be made on the feasibility of of repair and/or strengthening and on the special conservation restraints if any with special concern to aesthetics and style.

Viollet-le-Duc gave a clear definition of his understanding of restoration in his *Dictionnaire raisonné de l'architecture*: 'Restoration: both the word and the activity itself are modern. To restore a building is not to repair or rebuild it, but to re-establish its original state which must, at a certain moment in time, have existed.' In achieving this objective Viollet-le-Duc had the advantage of skills acquired from the new science of archaeology, and he knew that the initial step was to carry out a critical examination of the building.

This study aimed at producing a report (inventory) on stabilization / repair / strengthening materials, methods and technologies used for the structural restoration of structures including cast iron, wrought iron and steel members. A detailed literature survey was carried out by reading as many document as possible using different database including UPC library and after all the courses studied throughout this advanced master study it may be concluded that regardless of the technique decided to be applied in a restoration project, it is very important that any repair or strengthening must be based on a correct diagnosis of the problem by determining the real cause of the observed defects, and if the cause is still active by eliminating it. Otherwise the repairs will be short-lived. There is a parallel in the field of medicine: any treatment must be preceded by a correct diagnosis.

Though the study covers the stabilization/repair/strengthening techniques by using iron and steel, the quote from Carbonara, 1996 applies : Intervention should take into account the modern restoration and conservation theories which interest both monuments but also all the historic buildings .

It is known that some structural engineers in the past have considerably damaged the character of some historic buildings, by introducing strengthening devices that opposing the intrinsic mechanics of the original structure, in order to create a structural system conforming to their conventional textbook patterns and the codes of practice. There should be a compromise between the architect and the engineer realizing that unless the structure of the building is sound, its architectural and artistic glories are bound to perish which will help the architect to understand that the laws of gravity and structural mechanics apply to all structures, regardless of their antiquity or historic significance; conversely, the engineer should conceive that older buildings have unique caharacteristics, which are not often found in modern structures and for the successful operation of structural restoration techniques, it is essential that the engineer becomes familiar with these symptoms.

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Appendix I

Iron and Steel

Iron

Man's relationship with iron goes back deep into prehistoric times, and is presently believed to cover at least seven millennia. Fragments of iron and small iron objects such as beads, blades and decorative inlays have been found in archaeological sites dating to around 5000 BC, in Irak (Samarra), Iran (Tepe Sialk) and Egypt (El Gerseh). Later discoveries, corresponding to the early bronze age (3000—2000 BC) and middle bronze age (2000-1600 BC), are all situated in the east and south-east of the Mediterranean Basin, in Mesopotamia, Turkey, Egypt and Cyprus.

Written evidence of early iron-making activities exists in the form of mural hieroglyphic inscriptions and papyruses, for example in the *Book of the Dead.* However, the translation of ancient technical terms remains uncertain. Some early civilisations do not appear to have recognised iron as being distinct from copper and refer to it as black copper, in the same manner as unrefined copper. References to black metal or to metal from the sky could apply to iron or hematite ore, but also to other metals. Furthermore, the presence of objects made from iron does not necessarily imply the ability to extract the metal from its ores, since iron also exists in native and particularly meteoritic form, although the sources are by no means abundant.

The earliest evidence of iron smelting has been found at Hittite excavation sites in Asia Minor, dating from between 1700 and 1400 BC. However, this does not necessarily mean that iron-making originated in this region and then spread elsewhere. It is the aim of the present chapter to consider in more detail the dawn of iron metallurgy.

While the extraction of iron from its ores is closely related to the characteristics of the iron-carbon system, the practical exploitation of the remarkable properties of iron and steel provides a further illustration of how technical progress resulted from a combination of empirical observations and ingenuity. With rudimentary means and limited knowledge, early iron-smiths gradually developed their skills and know-how, succeeding in manufacturing a wide variety of high quality objects. This is nowhere more clearly evident than in the art of sword-making throughout the world.

The Three Sources of Iron

The earliest iron used by man was generally *meteoritic* in origin, as shown by the presence of nickel in most prehistoric objects, as well as in those from the early and middle bronze ages. The microstructure of a typical metallic meteorite is shown in Figure 1-1-1.

Figure **A1**

Polished section of a metallic meteorite, from the Henbury crater in Australia, showing a coarse Widmanstatten structure (approximate sample width 8.5 cm).
Meteoritic iron generally

Native iron is of terrestrial origin and is found in basalts and other rocks, generally in the form of small grains or nodules. It often contains considerable quantities of nickel, up to 70%. It is rarer than meteoritic iron, but has also been found in ancient precious objects.

However, most of the iron present at the Earth's surface is in the form of ores, mainly the oxides, particularly hematite (Fe2O3) and magnetite (Fe3C^), although carbonate (siderite), sulphide (pyrites) and mixed iron and titanium oxides (ilmenite) are also fairly common. *Iron extracted from ores is normally free from nickel,* and iron of this type has been found in objects dating from prehistoric times. Iron objects have been found in Egypt, in the Temple valley and Cheops' pyramid at Giza (2500 BC) and at Abydos (200 BC). However, the number of such objects is small and their authenticity is doubtful, due to their poor state of conservation (heavy rusting).

The oldest iron not of meteoritic or native origin is found as small decorative inlays in gold jewellery or tiny cult objects. It has been suggested that this iron is a by-product of the gold production process. Magnetite is frequently present in the gold-bearing sands in Nubia and could have been reduced during the smelting operation, pasty iron floating to the slag above the molten gold. Another possibility is that iron oxides were deliberately associated with other oxides used as fluxes for the manufacture of bronze.

Iron did not, however, replace bronze as the chief metal used for weapons and tools for several centuries, despite some attempts. Working iron required more fuel and significantly more labor than working bronze, and the quality of iron produced by early smiths may have been inferior to bronze as a material for tools. Then, between 1200 and 1000 BC, iron tools and weapons displaced bronze ones throughout the near east. This process appears to have begun in the Hittite Empire around 1300 BC, or in Cyprus and southern Greece, where iron artifacts dominate the archaeological record after 1050 BC. Mesopotamia was fully into the Iron Age by 900 BC, central Europe by 800 BC. Egypt, on the other hand, did not experience such a rapid transition from the bronze to iron ages: although Egyptian smiths did produce iron artifacts, bronze remained in widespread use there until after Egypt's conquest by Assyria in 663 BC.

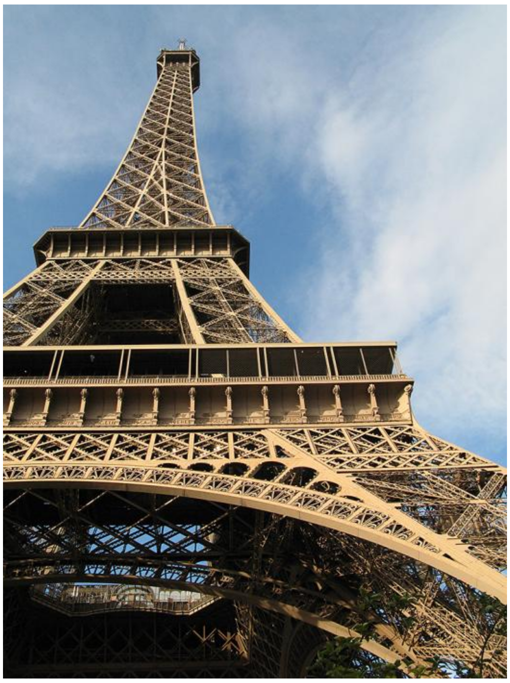
Iron smelting at this time was based on the bloomery, a furnace where bellows were used to force air through a pile of iron ore and burning charcoal. The carbon monoxide produced by the charcoal reduced the iron oxides to metallic iron, but the bloomery was not hot enough to melt the iron. The result of this time-consuming and laborious process was **wrought iron, a malleable but fairly soft alloy containing little carbon.**

Wrought Iron

The Phrase "wrought iron" is one of the most confusing terms in ornamental metal business However, the confusion is understandable since even dictionaries cannot agree on a single definition The first thing to clear up is the spelling. Many consumers spell the metal "rod iron" or "rot iron."

Wrought iron can be *carburized* into a mild steel by holding it in a charcoal fire for prolonged periods of time. By the beginning of the Iron Age, smiths had discovered that iron that was repeatedly reforged produced a higher quality of metal. Around 500 BC, however, metalworkers in the southern state of Wu developed an iron smelting technology that would not be practiced in Europe until late medieval times. In Wu, iron smelters achieved a temperature of 1130°C, hot enough to be considered a blast furnace. At this temperature, iron combines with 4.3% carbon and melts. As a liquid, iron can be cast into molds, a method far less laborious than individually forging each piece of iron from a bloom.

The term 'wrought ironwork' is often loosely used to mean decorative forged metalwork, including steelwork. Elaborate gates and railings immediately spring to mind. However, not all wrought ironwork is decorative, and steel, which was only introduced in 1856, is a very different material in both form and performance . As the oldest form of iron to be used, wrought iron is the material which gave the Iron Age its name. Architecturally its importance for fixings and fastenings was established long before the need for railings and decorative balustrades materialised, and some of the oldest examples of wrought ironwork include nails and spikes as well as agricultural implements and arms. However its decorative potential was also recognised and some of the earliest examples of the material's use include jewellery.



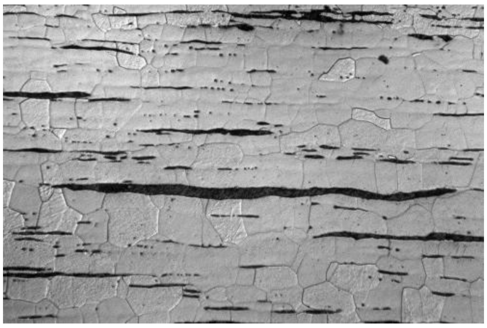


Figure I.2

The microstructure of wrought iron, showing dark slag inclusions in ferrite (iron)

Figure I. 1 The Eiffel tower is constructed from puddle iron, a form of wrought iron.

Properties of Wraught Iron

When heated during forge-welding, wrought iron does not melt but the slag fibers become molten, act as a flux, and protect the iron from oxidation and thus are very beneficial. Because the slag fibers do not have a uniform shape or size, wrought iron is very anisotropic. The greatest strength is in the direction of these threads, so it is important to have the correct orientation in order to achieve the optimum properties. This is illustrated by the fact that the strength is typically 400 MP m_2 parallel to the fibers but only 340 MP m_2 in a perpendicular direction and there is an even greater difference in the elongation of 40% and 10%. The fibers prevent internal

grain growth and consequent embrittlement and also tend to divert cracks during deformation and block corrosion pits. The other important properties of wrought iron are toughness, weldability, forgeability, and malleability. Because it can be rolled, drawn, or forged it was heated and hammered on an anvil or other base to the required shape. Blacksmiths still use it to produce horseshoes, railings, and ornamental ironwork. Because it recovers rapidly from overstrain and accommodates sudden shock without damage, it has been widely employed for engineering applications such as lifting chains, crane hooks, anchors, and railway couplings. It was extensively employed in the production of armor plates, some of which were massive: those made in Sheffield in 1856 were 400 □ □ 90 cm and 11 cm thick and weighed 3600 kg, while others in made in 1873 were 25 cm thick! One major improvement of properties was to increase the tensile strength and hardness by carburizing or “steeling” the iron. This involved the diffusion of carbon into the surface of the wrought iron. The rate of this is negligible at room temperature but is significant when the iron at about 1200 □ C is incontact with white-hot charcoal and carbon monoxide. The rate of diffusion at different temperatures can be calculated using the Arrhenius equation

$D_{coeff} = D \exp(-Q/RT)$

where D_{coeff} is the diffusion coefficient, D is the diffusion constant, Q is the activation energy, R is the gas constant, and T is the absolute temperature. If the iron is held at 950 °C for 9 h the carbon concentration at a depth of 1.5 mm is 0.5%, but at 1150 °C it is much higher, 2%. This gives a considerable increase in the tensile strength which can increase from 350–370 MN m^{-2} to about 950 MN m^{-2} with 1.2% carbon. More details of the development of carburization and examples of its use in ancient times were described by Madden et al.

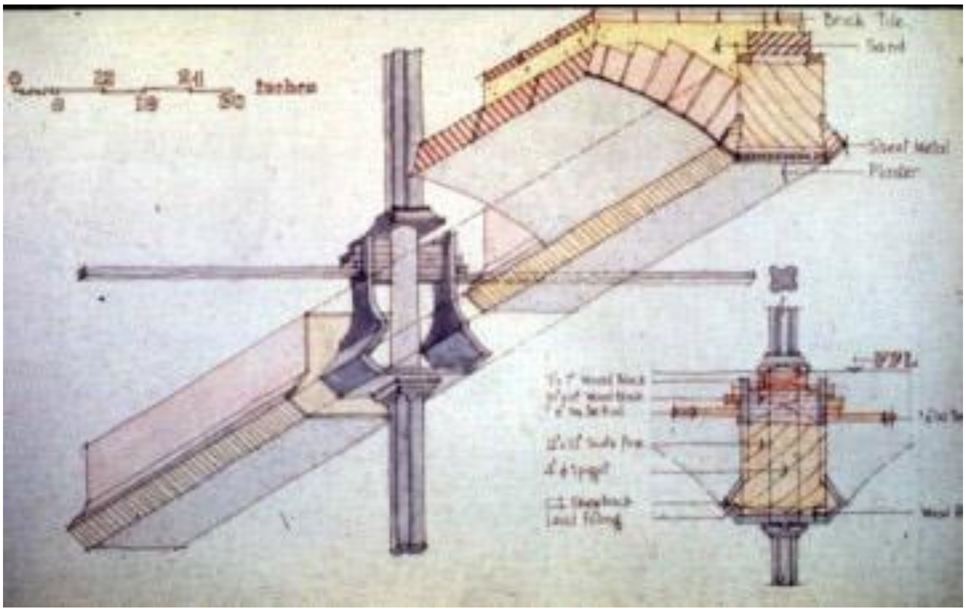


Figure I.3 The brittleness of cast iron and its low tensile strength made it unsuitable for major engineering structures.

Corrosion Resistance

Many ancient wrought iron artifacts have been recovered from the sea, indicating that the metal must have good corrosion resistance. These objects, such as guns and anchors, often have a surface appearance that is characteristic of wood, as shown in Figure 3. This is due to highly heterogeneous way Forgewelding pieces of wrought iron. Further details are given elsewhere (*12, 13*). The surface condition of the Pillar is remarkable and the original Sanskrit writing is clearly visible. One of the major influences on the rate of corrosion of iron is the presence of chloride ions. These can initiate attack on the surface to give pitting and the corrosion becomes autocatalytic. The first stage is probably adsorption onto the surface $Fe–H_2O_{ads} + Cl^- \rightarrow Fe–Cl_{ads} + H_2O + e$

Ferrous chloride is then formed

$Fe^{2+} + 2Cl^- \rightarrow FeCl_2$

and this is followed by decomposition into the more stable hydrated ferric oxide or rust

2FeCl2 + 4H2O → Fe2O3_H2O + 4HCl + H2

Hence during the formation of rust the chloride is released and the corrosion can continue .

The National Federation of Iron and Steel Manufacturers, submitting evidence in 1931 on the wrought iron trade, stated that “owing to its ability to resist all forms of corrosion resulting from atmospheres, damp and alternative wet and dry conditions, it possesses exceptionally high lasting properties.”

Iron has been available for building construction from very early times, but until the Industrial Revolution smelting from the ore was a tedious business carried out on a small scale. All the resulting products, down to the smallest of nails had to be handwrought. In consequence the material was used sparingly. However, as supplies of good quality timber ran short improved methods of smelting developed and iron in the form of straps, coach screws and the like, gradually replaced the fine timber joinery of earlier constructions. Unfortunately for us today it was sometimes used instead of more stable metals like bronze to make cramps and ties for masonry work, with consequent failures that have proved very costly to correct.

Casting iron has also been a technique known for many hundreds of years. In building, castings were not in common use until the Industrial Revolution of the late 18 th century. Methods had been developed mainly in the production of military ordnance but by the 1770s a wide variety of standard castings were available to buildings including, firegrates, chimney pots, railings, windows, sash weights, locks, latches and hinges, and many others.

The structural use of iron was developed at the same time both as wrought and cast

work. Such was the demand for structural ironwork that in the 1840s rolling mills were built to produce standard profiled sections, thus eliminating the time consuming cutting and rivetting that had previously been necessary to build up structural sections. The expansion of the railway system and the building of iron ships resulted in enormous expansion of the industry. Complete iron buildings were constructed – the great train sheds, and conservatories everywhere with of course the most famous of all being The Crystal Palace (1850). The prime example in Northern Ireland is the Palm House in the Botanic Gardens, Belfast. Standard buildings were shipped all over the world and every town of any size could boast its public lavatory, bandstand and its decorative street lamps.

Techniques of blending in other minerals to give increased strength and durability were perfected and in 1866 the first standard rolled steel joists were available. In recent years welding has taken the place of rivetting. Old wrought iron is very pure and when exposed to the weather forms a skin of black oxide which protects it against further decay.

Dowels rivets and bolts

Rivets and bolts transmit load by a combination of bearing pressure, shear and friction on the interface between the components being joined.

The structural principle of the rivet and the bolt is just the same as that of the traditional dowel or peg. In essence, whenever it is desired to transfer load from one plate to another, the method is to drill a hole in each plate and then insert a circular bar through these holes. The system relies on the pressure at the interface of the plate and the bar to transfer load and also on the shear stresses in the cross section of the bar. Iron and steel have adequate shear strength so that simply increasing the cross sectional area increases the capacity of the bar. To get adequate bearing between the bar and the plate, the ductile character of wrought iron and steel helps the action of the joint. The holes drilled in the plates have in practice to be slightly larger than the bar. If the circle of the bar and the circle of the hole remained perfect, they would only touch at a point. With wrought iron and steel connections, both bar and hole will distort significantly to give a larger area of contact. This principle is used for the dowel, the bolt and the rivet, but it was the rivet which was the predominant means of connecting wrought iron and steel from the mid-19th to the mid-20th century.

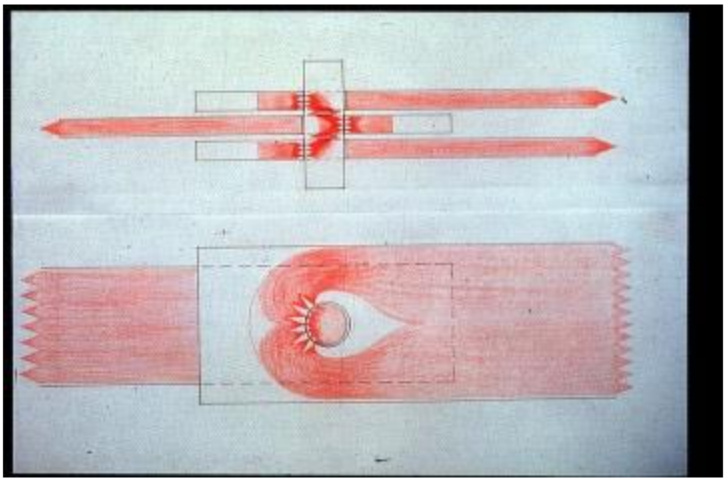


Figure I.4 Flow of forces between steel plates connected by a bar

This curious diagram is an attempt to illustrate the flow of forces between steel plates connected by a bar. The principle is the same whether the bar is a dowel, a rivet or a bolt. The lower diagram shows the forces flowing through the plate going around the hole cut in the plate and concentrating behind the bar. It should be apparent that at the cross section of the plate that goes through the hole, the stresses are most heavily concentrated. Also that for a reasonable contact area between the plate and the bar, a degree of ductile deformation is required. Were the hole in the plate and the bar to remain perfectly circular they would touch only at a point. The positive virtue of a ductile material like steel or wrought iron is that by deforming by quite small amounts it spreads the loading over a reasonable area. The upper diagram is a section through the bar showing the transfer of load between the plates and the sections of the bar that have to transfer the load by shear stresses. In order to make the diagram clearer and to give room to indicate the shear planes the plates are shown slightly apart. In practice of course, rivets and bolts clamp the plates together. Indeed the friction between the plates gave traditional joints added strength and it is utilised by modern high strength friction grip bolts.

Rivets make very effective connections between components in both wrought iron and steel.

In essence, the rivet is simply a bar with its ends hammered over to prevent it slipping out of the hole. This is easy to do with soft metals like gold, silver, or copper and has been a traditional method of jointing since prehistory. It was impossible to achieve with cast iron because cast iron cannot be squashed, it simply fractures. It is a very effective way of joining ductile metals such as wrought iron and steel. Rivets were heated in portable braziers and held in position with tongs while the protruding end was bashed over with a big hammer to form strong reliable and positive fixings. In fact, the connection had more strength than simply the shear across the cross section of the rivet since the hammering of the head gripped the plates together and load could be transferred through friction between the plates. Also, the heat and pressure would often induce some pressure welding if the conditions were favourable.

The Eiffel Tower used 2.5 million rivets. On site they were fitted by this type of four person team. The rivet was heated to red heat in a portable brazier and transferred from brazier to pre-drilled hole by the person holding it in position with the tongs. Finally while restrained at the far side it was beaten with a hammer to form the fixing head. Look at any large scale nineteenth century structure and consider the man power and organisation required.

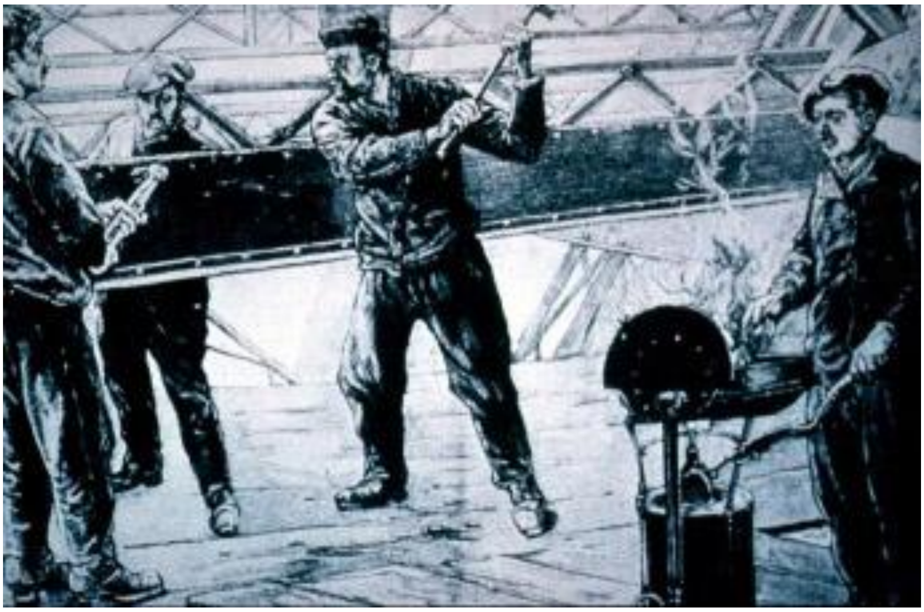


Figure I.5 The change from cast iron, with elements that fitted together, to wrought iron, joined by rivets, had a significant effect on the appearance of joints.

In terms of the visual expression of the joint, the change from cast to wrought iron was a step backwards. The relation between the parts was limited by the overlapping of flat surfaces. While this was ideal for the fabrications of steam boilers, it made the connection of columns and beams, struts and ties more difficult. Initially, sections were small in size and restricted in profile to flats, tees and angles. Larger sections would be made up from combinations of these basic profiles, and here there was scope for richer and more varied joint design. As the 19th century progressed, the introduction of steel and the development of the industry meant that the rolled sections got larger and the jointing tended to be cruder.

This column head is a particularly interesting juxtaposition of a timber roof purlin with localised increase in depth, a wrought iron edge beam made up from riveted angles and flats, cast iron decorative brackets with both opposing curves fixed by bolts and a slender cast iron column.

Figure I.6 Rivet patterns at joints contributed to the visual interest of large structures.

The most visually interesting structures of this period tended to be the very large showpiece structures built for the great exhibitions. Eiffel's great tower for the Paris exhibition of 1888 was a tour de force of joint design. As the scale of the structures got larger, it was not sensible to increase the size of the rivets, there just had to be more of them. At close quarters, the pattern of the uncountable numbers of rivets arranged in consciously designed patterns gives human scale to the vast structure. The care with which the geometry of these patterns was organised showed that engineers like Eiffel were as alive to the potential of rewarding detailed design as any architect. The Gallery of Machines for the same exhibition was not only a gigantic enclosure of space, but its structure was built up from riveted sections in a way that demonstrated a controlled richness of detail that can still be admired in the famous photograph a hundred years later.

5.2.2 Steel

Steel is composed of iron and carbon. The iron is extracted from iron ores, mined and then refined to remove the oxygen content.

Part of the refining process is to remove oxygen from these ores by heating them with coke and limestone to a temperature of about 1600° C in a blast furnace

After reducing the iron oxide the resultant material is pig or cast iron, which is brittle. Some carbon remains in addition to other unwanted elements and these must be reduced by further refinement before the material becomes steel. The carbon content of steel is crucial to its strength and is usually lower than 2.11%. A controlled system of oxidisation is used to manipulate the carbon content. Other impurities must be removed from the melt in order to achieve the correct composition. Plain carbon steels are those in which the only other element remaining is manganese.

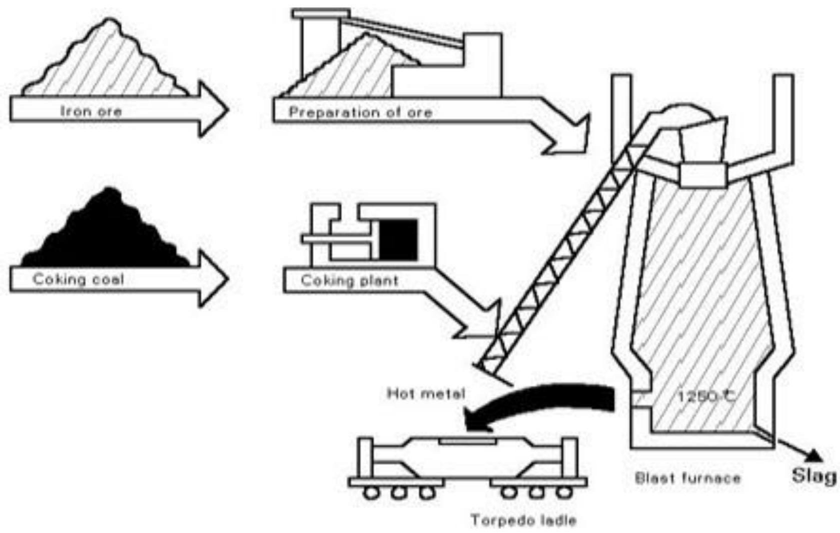


Figure I.7 After the iron has been extracted from iron oxide it is known as 'pig' or 'cast' iron.

The iron oxide is reduced either directly by the oxide combining with carbon to form iron and carbon monoxide, or indirectly by combining with this newly formed carbon monoxide to form iron and carbon dioxide. In this state the material is known as pig or cast iron.

The pig iron must be further refined to reduce the carbon content and the other unwanted elements before the material can be categorised as a particular steel.

After this process there is always some carbon left in the solid iron formed amounting to about 4% by weight. The high carbon content of cast iron results in a material which is brittle and cannot take large tensile stresses. In addition there is a total of about 6% of other unwanted elements, including silicon, manganese, sulphur and phosphorous in unmeasured quantities.

***Figure I.8 Refining iron into steel requires the re-melting of the iron in a steelmaking furnace with a large oxygen input*.**

In steelmaking the impurities in the melt have to be removed before the correct composition can be achieved. These include phosphorous and silicon (which make steel hard and induce brittleness), and sulphur (which can cause cracking in poured castings and welds).

A fundamental distinguishing property of steel is its great strength which depends partly upon the carbon content.

Most steels have carbon contents far lower than 2.11%. This figure marks the critical maximum where in the processing of the material, at 900° C, a complete phase change can occur. Above this level of carbon content, an iron-carbon alloy becomes more brittle taking on the type of performance associated with cast irons. The manipulation of the carbon content is carried out by controlled removal by oxidation.

This range of iron-carbon steels are referred to as plain carbon steels as the only other element present is manganese, present in up to 1.6% by weight and left in the metal from de-oxidation and de-sulphurisation processes. The addition of any other elements will bring the metal into the metallurgical category of alloy steels. (The general term alloy steels in the industry refers to those steels which have contributions from other elements amounting to 5% by weight and over.)

The mechanical properties of steel

The alloys and the heat treatment used in the production of steel result in it having different values and strengths. Testing must be accurate to determine the properties of the steel and to ensure adherence to standards.

Different steels have different values of strength and toughness depending on the alloys made and the heat treatments used. Testing methods are important to determine values and to ensure that standards are adhered to. Methods of testing determine the yield strength, ductility and stiffness through tensile testing, toughness through impact testing, and hardness through resistance to penetration of the surface by a hard object.

Tensile testing is a method of evaluating the structural response of steel to applied loads, with the results expressed as a relationship between stress and strain.

Specially shaped specimens of steel are subjected to a tensile force which gradually elongates the steel. The force is increased and the extension of the steel carefully measured.

The stress on the metal is evaluated by dividing the value of the force applied by the cross-sectional area of the specimen.

Stress = *LOAD* in Newtons / *CROSS-SECTIONAL AREA* of specimen in mm^2

Strain is measured by calculating the increase in length of the specimen as a proportion of the original length.

Strain = *INCREASE IN LENGTH* in mm / *ORIGINAL LENGTH* in mm

The relationship between stress and strain is a measure of the elasticity of the material.

If a load is applied to a material, and that material fully recovers, that is it returns to its original length once the load is removed, the elastic limit of the material has not been reached. However, if the material does not recover on removal of the load, it must have exceeded its elastic limit and will be said to have deformed permanently and started to behave plastically.

The elastic behaviour is characterised as the ratio of stress to strain, and is referred to as Young's modulus.

The elastic behaviour is characterised as a relationship between the stress applied and the resultant strain, and is expressed mathematically as:

STRESS / STRAIN

otherwise known as Young's modulus. This relationship allows comparisons to be made between different materials.

As the value of Young's modulus changes between materials, so then does their elastic behaviour, which is a real indicator of their stiffness. Steel has a high value of Young's modulus which is about 205 kN/mm^2 (approximately three times the value for aluminium).

The distinction between elastic and plastic behaviour can be seen from the stress-strain curve.

To explain this behaviour more fully, stress is plotted against strain in the following figure.

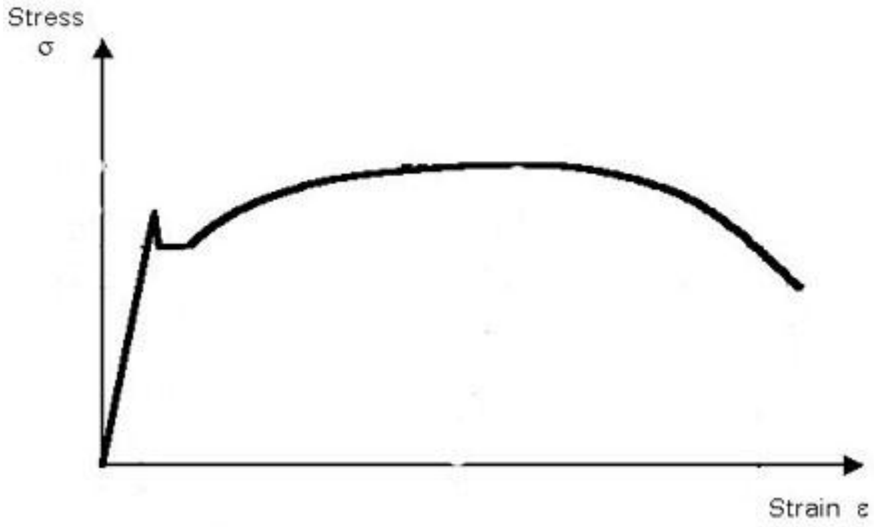


Figure A10 The initial part of the curve shows a linear relationship between stress and strain. This is the elastic region. The slope of this initial part of the curve is a measure of the material stiffness (Young's modulus); the steeper the slope, the smaller the deformation for a given load.

Beyond the initial linear part of the stress-strain curve, further increase in load produces very large increases in strain. This is the PLASTIC region.

Although there is still a considerable reserve of strength beyond this level of loading, the yield stress is normally used as the failure criterion when assessing the strength of a structural member because of the very large deformations associated with plastic behaviour. Furthermore, strains within this range are associated with permanent deformations.

Beyond the plastic region, further increases in strain are associated with more pronounced increases in stress. This is due to strain hardening.

Eventually, the specimen will reach a maximum level of stress and fail. This is the ultimate tensile strength of the steel.

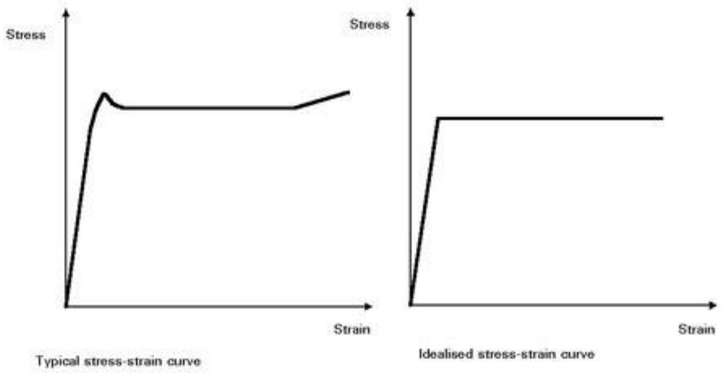


Figure A1O Typical stress-strain and Idealised stress-strain curves

Strength is a very important characteristic of steel, but other properties must also be considered when considering structural performance.

The yield strength represents the point where behaviour changes from elastic to plastic. The ultimate tensile strength is the end point of the stress-strain curve representing rupture of the material. Both yield strength and ultimate tensile strength can be increased by altering the chemical composition of the steel - particularly the carbon content.

Yield strength can also be improved by mechanically working the steel, although this reduces its ductility - the ability of the steel to sustain high strains prior to failure

At very cold temperatures some steels may lose ductility.

Under some conditions steel may exhibit a brittle, rather than ductile, mode of failure. This is not normally a problem in building structures since modern steels are manufactured so as to avoid such behaviour. However, in extremely cold conditions - for instance exposed steelwork where temperatures start to fall well below 20° C, the brittleness of steel will increase rapidly.

Steels with high carbon contents are more vulnerable under stress, and stress concentrations due to imperfections or details should be avoided.

In these low temperature conditions steel alloys are carefully specified; typically the manganese-carbon ratio is increased and the nickel content reduced, and specialist advice must always be sought.

The behaviour of steel is also notch-sensitive. Fixing details which can impose defects on the steel, for instance from screw threads, must therefore be avoided.

History of Stainless Steel

The "discovery" of stainless steel occured in the 1900 to 1915 time period. However, as with many discoveries, it was the accumulated efforts of several individuals that actually began in 1821. That year a Frenchman named Berthier found that iron when alloyed with chromium was resistant to some acids. Others studied the effects of chromium in an iron matrix, but using a low percentage of chromium. To be stainless steel, the chromium content needs to be at least 10.5%. In 1872, Messrs. Woods and Clark applied for a British patent for what they identified as an acid and weather resistant alloy containing 30 to 35% chromium and 1.5 to 2% tungsten. Then, in 1875, another Frenchman named Brustlein recognized the importance of carbon levels in addition to chromium. Stainless steels need to have a very low level of carbon at 0.15%. While many others investigated the chromium/iron composition, the difficulty in obtaining the low carbon levels persisted for many years until low carbon ferrochrome became commercially available.

In 1904, Leon Guillet published research on alloys with composition that today would be known as 410, 420, 442, 446 and 440-C. In 1906, he also published a detailed study of an iron-nickel-chromium alloy that is the basic metallurgical structure for the 300 series of stainless steel. In 1909, Giesen published in England a lengthy account on the chromium-nickel (austenitic 300 series) stainless steels. Also in England and France, Portevin published studies on an alloy that today would be 430 stainless steel. In Germany, in 1908, Monnartz & Borchers found evidence of the relationship between a minimum level of chromium (10.5%) on corrosion resistance as well as the importance of low carbon content and the role of molybdenum in increasing corrosion resistance to chlorides.

Harry Brearley, chief of the research lab run jointly by John Brown & Co. and Thomas Firth & Sons, is generally accredited as the initiator of the industrial era of stainless steel. Most of his work was on 430 (the chemical analysis was patented in 1919). The first product was table cutlery and it is still used today

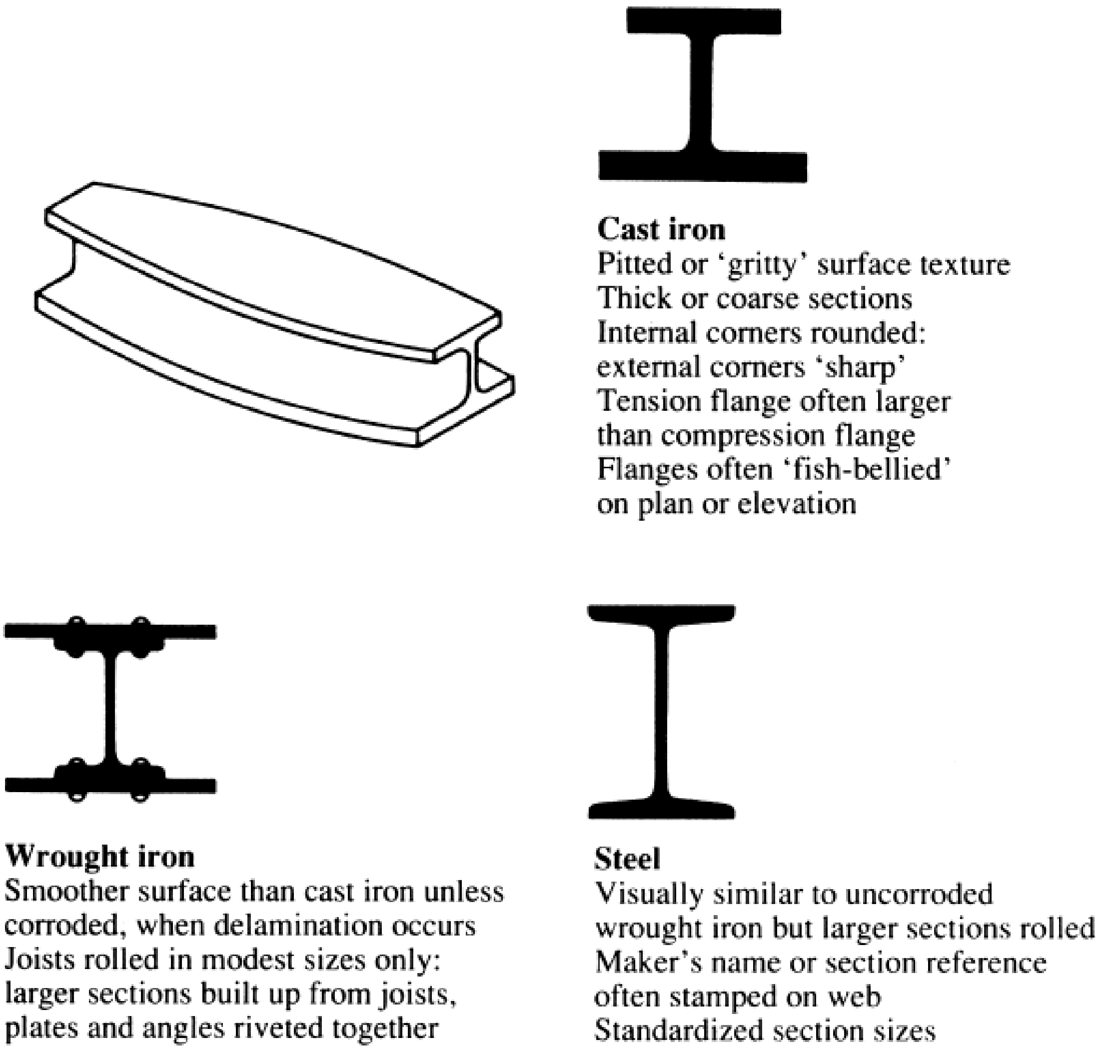


Figure I.11 Characteristics of ferrous metals, as used in beams (after Blanchard et al., 1982).

Table A1 Material properties given by Twelvetrees (*from CIRIA 'Structural Renovation of Traditional Buildings', Report 111, 1986*)

	Tons/in.^{2} (as quoted)	(N/mm^2) (approx. equivalent)
Cast iron (average values)		
Tensile strength	8	120
Compressive strength	38–50	590–780
Transverse (flexural) strength	15	230
Shearing strength	6–13	90–200
Elastic limit	1	15
Young's modulus		
Compression	5467–5879	84 500–90 500
Tension	4262–6067	66 000–93 500
Wrought iron		
Tensile strength	18–24	280–370
Compressive strength	16–20	245–310
Shearing strength	75% of tensile strength	
Elastic limit	10–13	155–200
Young's modulus	10 000–14 280	155 000–221 000
Mild steel		
Tensile strength	26–32	400–495
Shearing strength	70–75% of tensile strength	
Elastic limit	18–20	280–310
Young's modulus	13 000	201 000

Table A2 Safety factors, recommended by Twelvetrees *(after CIRIA, 1986)*

	Dead load	Live load
Cast iron		
Beams	5–6	8–9
Columns	5–7	8–10
Wrought iron and steel		
Beams	3–4	5–6
Columns	4–5	6–7

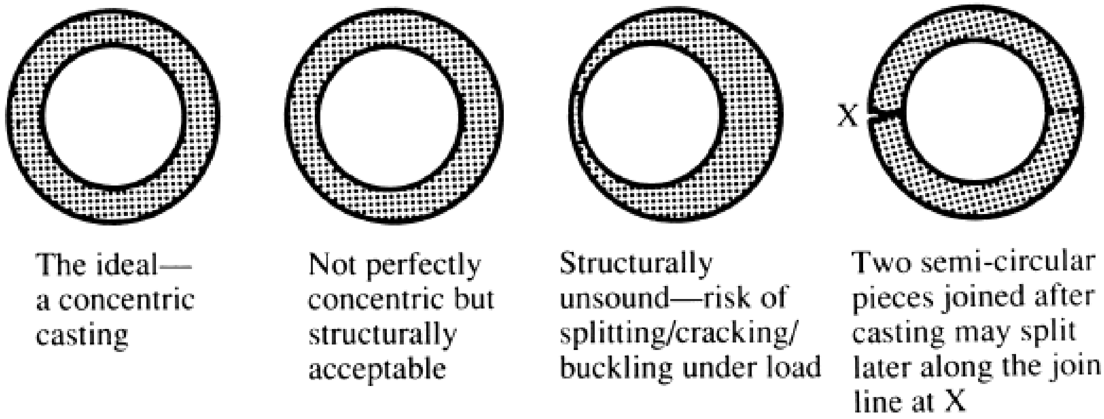


Figure I.12 Cross-sections of hollow cast iron columns, showing possible degrees of imperfection (from Blanchard et al., 1982).

Appendix II

Strengthening of Bridges Using External Post-tensioning

The use of external post-tensioning for the strengthening of existing bridges has been used in many countries since the 1950s and has been found to provide an efficient and economic solution for a wide range of bridge types and conditions. The technique is becoming more and more preferable because of the speed of construction and the minimal disruption to traffic flow.

In response to the demand for faster and more efficient transportation systems, there has been a steady increase in the weight and volume of traffic using national highway systems throughout the world. As well as increases in legal vehicle loads, the over-loading of vehicles is a common problem and this must also be considered when designing or assessing bridges. In response to increased traffic flow, bridge widening is often carried out to increase the capacity of the road network. As a result, many bridges are now required to carry loads significantly greater than their original design loads. This trend is very pronounced in developing countries such as Indonesia as a result of the accelerated regional development which has taken place over the last two decades. In many areas, the road system provides the only means of transporting goods and people and is vital for the survival and prosperity of whole regions. Minor roads which were originally constructed for light traffic are now required to provide access for modern heavy goods vehicles and roads are continuously being, up-graded to cope with the increased demands. Bridges are a critical part of this re-classification. Even bridges which have been designed using appropriate loading specifications can be under-strength for other reasons. Bad detailing, other design faults, and general wear and tear can impair structural performance. Deterioration of bridges due to steel corrosion, corrosion of reinforcement, attack by chemicals or pollution, imnpact damage from vehicles, all lead to loss of strength. Loss of prestress due to creep and relaxation also results in strength loss. Again, these problems can be accentuated in developing countries where bridge maintenance is often neglected, deterioration rates can be higher due to high temperatures, high humnidity and pollution, and the quality of site practice may be low.

Many bridges which were designed to previous loading specifications or which have suffered damage or deterioration are now inadequate for modern heavy goods vehicles. As a result, bridge owners and manager are faced with the choice of restricting traffic or carry out rehabilitation. When this is combined with the growing competition for the funding of infrastructure projects, there is great pressure to keep bridges in service and minimising capital and maintenance expenditure. Bridge strengthening as an alternative to complete replacement can provide an effective and economic solution in appropriate situations. In particular, traffic management costs can be considerably reduced especially in cases where rehabilitation can be scheduled to avoid complete closure of the carriageway.

Selection of appropriate strengthening technique The selection of an appropriate method for strengthening a particular bridge depends on a number of factors. The type of structure, the magnitude of the strength increase required and the associated costs are the main parameters to be considered. Many strengthening schemes are applicable to particular structural types and have limits on the extent to which strength can be increased. Strengthening costs would certainly be lower than bridge replacement, but the selection of a particular method of strengthening would need to be justified on economic grounds. It is important to consider, not only the initial capital costs of the strengthening project, but also the maintenance costs associated with the future in-service behaviour. The condition of the existing bridge is an important consideration. If the bridge is in bad condition, then future maintenance and safety problems might override the benefits of the reduced capital costs of strengthening and provide justification for bridge replacement. The strength and condition of the substructure must not be ignored and strengthening should not proceed without giving due consideration to the capacity of the bridge piers, abutments and foundations. The difficulties associated with traffic management and the costs arising from traffic delays should be considered in the economic justification. In some cases, this may limit the use of certain methods of strengthening.

Depending on the bridge configuration and the expected service life of the bridge after strengthening,, other factors might need to be considered before a particular scheme is adopted. The durability, inspectability and replaceability of components of the rehabilitated bridge are very important aspects. For some strengthening systemns, the ability to monitor the behaviour of the strengthened bridge mignht need to be considered, particularly where an innovative method is being used. The ability to adjust the level of strengthening in future to allow for further increases in traffic loads mnight provide useful benefits.

The appearance of the bridge after strengthening is an important consideration and should not be ignored. While bridge aesthetics have always played an important role in the design of major structures, public perception has often been ignored for short span bridges. This is now beginning to changre and emphasis is now being placed on how highway bridges look. The use of intermediate supports or props, or strengthening methods which appear unsightly, while tolerable as temporary measures, are becoming less acceptable as long termn Solutions.

Many strengthening techniques have general applicability, but some may be specific to particular bridge types and configurations. The decision to adopt a particular scheme is based on the consideration of a wide range of parameters. The remainder of this paper is concerned with external post-tensioning for bridge strengthening. The general principles, advantages and disadvantages are described in the following sections.

Post-Tensioning as Strengthening Technique The principle of external post-tensioning is the same as that of prestressing, 'ie, the application of an axial load combined with a hogging bending moment to increase the flexural capacity of a beam which improve the cracking performance. It can also have a beneficial effect on shear capacity. Precise evaluation of flexural and shear capacity of beams with unbonded tendons, either internal or external to the section, is difficult. This is because the load in the tendons is a function of the overall behaviour of the beam, rather than just depending on the strain distribution at a particular critical section. Many national and international codes present methods of determining, capacity but these are based primarily on laboratory results obtained from tests on beams with internal tendons. Post-tensioning as a means of strengthening existing bridges has been in use since the *1950s* and there are many examples of its use throughout the world. In the many situations where the technique has been applied, the prestress is applied through prestressing cables, either single or grouped strand. In some applications, the stress has been applied through high tensile bars, jacked either using hydraulic jacks or with fine screw threads. In a few cases the stress is applied using more unconventional techniques. For example, stress in a tendon can be developed by anchoring a straight the tendon in place and imposing a deflection at mid-span. The deflection is then retained by fixing the deflected point. Prestress can also be developed by applying a load to the structure to cause overall deflection prior to anchoring the tendons or bars. Specific examples of these different applications are given by Xanthakos (1996). External post-tensioning can be used to improve the serviceability behaviour of existing bridges. As in prestressed construction, the method can be used to delay or prevent the onset of cracking in concrete bridge decks. It can also be used to reduce or close preexisting cracks. This improvement in cracking behaviour also increases resistance to reinforcement corrosion. The increased stiffness provided by external post-tensioiningy can reduce in-service deflections and vibrations. The stress range can also be reduced and the fatigue performance can be improved. The presence of a deformation or sag in a bridge can be reduced or removed.

Strengthening of Beam Type Bridges

External post-tensioning as been applied most generally to beam type bridges. The tendons can be straight or draped uising deviators depending on the particular requirements. Various profiles can be adopted to suit the required combination of axial load and bendinig. Additional compressive members can be introduced if the additional compression stressed imposed on the bearn are not desirable.

External post-tensioning, can be used to improve the performance of any kind of beam bridge, be it timber, reinforced concrete, prestressed concrete, steel or composite. The main reason for its use has been to provide increased flexural strength required because of under-design, increased traffic loading, loss of structural strength due to deterioration or to correct serviceability problems. In particular, the application to prestressed beams is appropriate where losses in initial prestress due to creep, relaxation or corrosion can be restored. The technique can also be used to increased the shear strength although vertical prestress, described later, can be more appropriate in correcting shear deficiencies. It is possible to use post-tensioning, to change the structural behaviour in order to increase strength. For example, the strengthening object might be to provide continuity across a support, ie, change a series of simply supported spans to a continuous one. It can also be used to provide continuity across an unsupported joint, for example, across the joint between two cantilever spans.

Strengthening of Slab bridges

There have been no reported cases of solid or voided slab bridges being strengthened using external post-tensioning. It may be possible where the deck is narrow and the tendons could be installed on both sides of the slab. The technique could be used to replace prestress loss in prestressed slabs where, for example, corrosion has taken place. The main difficulty would be in accommodating, the anchorages and fixing them to the existing slab.

Application to truss bridges

The technique of post-tensioning has been applied to steel truss bridges since the *1950s* and various configurations have been used. Individual members can be strengthened by applying concentric prestress on highly-stressed tensile members. Groups of members can also be post-tensioned in the same way. For example, the tension chord can be stressed with a single long concentric cable. This was carried out on a number of rail bridges in the UK in the 1950s (Lee 1952). A truss bridge can be strengthened by applying a polygon tendon to the truss as a unit. The cable is fixed to the top of the truss at the supports, sloping, down to the bottom of the truss at mid-span. This method was used to strengthen a steel truss bridge in

Switzerland in 1969 (Xanthakos 1996). Because the upper chord was not strong enough to carry the induced compression, an additional strut was required along the top members. It is also possible to strengthen a truss using a draped tendon along the bottom chord. No such cases have yet been reported.

Transverse post-tensioning

External post-tensioning can also be used in the transverse direction to improve load distribution across the deck or to increase the transverse stiffness. The mnethod can be convenient for replacing conventional transverse prestress lost due to creep or corrosion. The same post-tensioning techniques are applicable.The technique can also be used to repair arch bridges which have developed cracks in the spandrel walls due to lateral pressure through the fill material (Xanthakos 1996).

Vertical post-tensioning

Longitudinal or draped tendons can be used to improve the shear capacity of beams. Where flexure is not a problemn but a significant improvement in shear strength is required, the application of vertical might provide an effective solution. Vertical prestressing tendons or bars can be applied to the web of an I-beam. For concrete box sections, they can be installed either inside or outside the box. Because of the short length required, the post-tension force can most easily be provided using high strength bars with the required prestress produced using fine threads.

Vertical prestress can also be used to provide tie-down at supports. This can be very useful in earthquake regions where lifting forces might occur, as the tie-down can be retrofitted to existing **bridges.** This technique can also be used to improve defective joints.

Advantages and disadvantages

As with all bridge strengthening methods, there are various advantages and disadvantages associated with the use of external post-tensioning. There are a number of distinct advantages which have added to the increasing popularity of this method. These are listed as follows:

- The method is economic in that it cheaper to install than methods which require major reconstruction of the bridge deck. The equipment required, while specialist in nature, is light and easy to use, particularly when single strand jacks are employed. Anchorages and deviators are easy to detail and simple to install.
- Both flexural and shear strength can be increased without the penalty of increased dead load.
- The ease of inspection increases the reliability of the bridge as any stress loss ordamage due to impact or corrosion can be determined by simple inspection procedures.
- The tendons can be re-stressed provided sufficient length of tendon is left behind the anchorage, sufficient access is made available to enable the re-fitting of the stressing jacks and a flexible corrosion protection material is used, ie, wax or grease. Restressing can be carried out with minimal disruption to traffic, provided account is taken of the reduced strength when the stress in a tendon is reduced. Thus any stress losses due to creep, relaxation, corrosion, etc, can be replaced. The stress magnitude can also be increased to allow for future additional strengthening. The use of single strand jacks greatly reduces the effort involved but the twisting of strands in a cable group should be avoided if single strands are to be re-stressed. Re-stressing can also be used to determine the residual load in the strand.
- The tendons can be made fully replaceable, as described above. The tendons can be removed to carry out a close examination. If corrosion or any other damlage is detected a new tendon can be installed.
- Where grout is used inside a duct to provide corrosion protection, individual strands cannot be re-stressed or replaced. However, the tendon can be removed by cutting the tendons and replacing the tendon-duct system. For this reason the uise of grout is not generally recommended, except in localised areas.
- Many strengthening methods cannot be applied to an in-service bridge because of the vibrations or because of access. With external post-tensioning, the strengthening of a bridge can be carried out without disruption to traffic flow~ on the bridge. No traffic restrictions are required on the bridge being strengthened other than those already applied to the unstrengthened bridge. Where the bridge is road over road, traffic restrictions would be

required on. the road under the bridge for access. For box girder bridges where access is through the abutment, no closures would be required.

- Tendons can be draped to increase eccentricity where required. Eccentricity can be increased by installing the tendons below the bottom flange.
- The friction losses associated with external tendons are considerably less than those for internal tendons. This means that the tendons can be longer and the deviation angles greater than with internal tendons.
- For box girder bridges, the tendons can be installed inside the box, so that they are not visibly intrusive.

A number of methods of corrosion protection are available. It is also possible to have multiple levels of protection, eg, inside a box, inside a ducts, wax. These methods are generally easy to install although there are detailing problems at anchorages and possibly at deviators. As with other methods of strengthening, there are disadvantages and it is important that these be understood in order to make an enlightened evaluation of this method. The main disadvantages are as follows:

- Application of the method is very dependent on the existing condition of the bridge. Concrete of poor quality should not be over-stressed and a full condition survey should be carried out to ensure that the bridge deck can take the increased stress. Where the condition of the concrete is suspect, due to corrosion, deterioration, impact damage, etc, the post-tensioning should be applied with care.
- Loss of stress due to creep and relaxation are an inherent part of post-tensioning and should be taken into account in both steel and concrete beamns. This disadvantage is negated to a certain extent because of the ability to re-stress the tendons.
- Installation of deviators and anchorages can be difficult, and careful detailing is required to account for stress concentrations in the existing deck components. Normally, drilling through or welding existing structural components to steel webs, concrete webs, flanges, etc, is required. This can be problematic, particularly since the bridge deck is already tinder-strength. However, these can normally be installed in less critical areas. Local stiffeners may be required at anchorages and deviators.
- The tendons, being external, are more susceptible to corrosion, and often need to be places in areas of run-off, eg, near joints. The tendons can be susceptible to contamination by bird and bat droppings, eg, inside boxes, along beams. Cases where tendons have been submerged during floods have been reported. Effective corrosion protection is critical to the effective performance of this method. Standard methods are available. The shear capacity of beams with external tendons is difficult to determine. Conservative methods are available but the degree of conservatism is at present unknown. Further research is required in this area.

- For shear, strengthening near supports, normally the critical area for shear, is difficult because of access difficulties. For these reasons, the method has been generally confined to flexural strengthening.
- The effects of axial load may be detrimental to the carrying capacity and needs careful consideration. Where this is the case, increasing the eccentricity of the tendons can increase the prestress without a corresponding increase in axial load.
- The ductility of concrete beams post-tensioned with external tendons is still questionable. This is because, since the tendons will never reach yield, failure will be due to concrete compression in the top flange. However, in the few laboratory tests carried out at TRL, externally post-tensioned beams failed after large deflections and with significant residual strength after failure. The behaviour of steel beamns post-tensioned with external tendons has not been investigated in detail.
- Installation of the tendons can mean working in difficult conditions and confined spaces, eg, on abutment shelves, inside boxes, on scaffolding, etc. The hardship is minimised by single strand jacks and modern access equipment.
- Where tendons need to be installed below the bottomn flange, the decreased headroom is a distinct disadvantage. This may be critical, in railway bridges for example, and may be a sufficient reason for adopting an alternative strengthening method. If the bridge is road over road, the tendons can be damaged by high vehicles passing under bridge. The mnethod should only be adopted if there is sufficient clearance to ensure that this is an unlikely event.
- The appearance of the external cabling system might discourage their uise, particularly when the tendons are installed below the soffit of thle beamns. Care in the detailing and lay-out can assist in blending the strengthening systemn in with the existing structure. If required, the tendons can be enclosed using panels: this would also help in providing some protection against corrosion or contamnination. Where the tendons are installed inside boxes or along, the webs of beamis, there is little or no visual intrusion.
- The external tendons, as in all cable stayed structures, are more susceptible to accidental damnage fromn fire, impact and acts of vandalism and public access to the tendons should be prevented. However, this type of damage will only impair the strengthening system: the resistance of the existing bridge is not altered so the likelihood of collapse is not increased.
- For proper installation of external tendons, precise fabrication, accurate installation and careful site supervision and inspection are required. The stressing sequence and schedule needs to be devised carefully to avoid damnaging, the existing deck. Because additional stress is being introduced into an under-strength structure, careful and accurate analysis is required in devising the tendon configuration. A proper understanding and accurate estimation of the stress distribution in the deck isrequired for the future safety and stability of the bridge. This

rnight-~require additional special investigations, eg, determination of the existing stress in a prestressed bridge.

The conclusion is that the method of external post-tensioning as a method of strengthening existing bridges has both advantages and disadvantages. Careful consideration is required before an effective strengthening system can be devised. The recent collapse of the Koror- Babeldaob, bridge in Palau near the Philippines shortly after it was strengthened using this method indicates the importance of such considerations (NCE 1996).

Appendix III

Manifesto for the Society for the Protection of Ancient Buildings

A society coming before the public with such a name as that above written must needs explain how, and why, it proposes to protect those ancient buildings which, to most people doubtless, seem to have so many and such excellent protectors. This, then, is the explanation we offer. No doubt within the last fifty years a new interest, almost like another sense, has arisen in these ancient monuments of art; and they have become the subject of one of the most interesting of studies, and of an enthusiasm, religious, historical, artistic, which is one of the undoubted gains of our time; yet we think that if the present treatment of them be continued, our descendants will find them useless for study and chilling to enthusiasm. We think that those last fifty years of knowledge and attention have done more for their destruction than all the foregoing centuries of revolution, violence and contempt. For Architecture, long decaying, died out, as a popular art at least, just as the knowledge of mediaeval art was born. So that the civilised world of the nineteenth century has no style of its own amidst its wide knowledge of the styles of other centuries. From this lack and this gain arose in men's minds the strange idea of the Restoration of ancient buildings; and a strange and most fatal idea, which by its very name implies that it is possible to strip from a building this, that, and the other part of its history— of its life that is— and then to stay the hand at some arbitrary point, and leave it still historical, living, and even as it once was. In early times this kind of forgery was impossible, because knowledge failed the builders, or perhaps because instinct held them back. If repairs were needed, if ambition or piety pricked on to change, that change was of necessity wrought in the unmistakable fashion of the time; a church of the eleventh century might be added to or altered in the twelfth, thirteenth, fourteenth, fifteenth, sixteenth, or even the seventeenth or eighteenth centuries; but every change, whatever history it destroyed, left history in the gap, and was alive with the spirit of the deeds done midst its fashioning. The result of all this was often a building in which the many changes, though harsh and visible enough, were, by their very contrast, interesting and instructive and could by no possibility mislead. But those who make the changes wrought in our day under the name of Restoration, while professing to bring back a building to the best time of its history, have no guide but each his own individualwhim to point out to them what is admirable and what contemptible; while the very nature of their task compels them to destroy something and to supply the gap by imagining what the earlierbuilders should or might have done. Moreover, in the course of this double process of destruction andaddition, the whole surface of the building is necessarilytampered with; so that the appearance of antiquity is taken away from such old parts of the fabric as are left, and there is no laying to rest in the spectator the suspicion of what may have been lost; and in short, a feeble and lifeless forgery is the final result of all the wasted labour.

It is sad to say, that in this manner most of the bigger Minsters, and a vast number of more humble buildings, both in England and on the Continent, have been dealt with by men of talent often, and worthy of better employment, but deaf to the claims of poetry and history in the highest sense of the words. For what is left we plead before our architects themselves, before the official guardians of buildings, and before the public generally, and we pray them to remember how much is gone of the religion, thought and manners of time past, never by almost universal consent, to be Restored; and to consider whether it be possible to Restore those buildings, the living spirit of which, it cannot be too often repeated, was an inseparable part of that religion and thought, and those past manners. For our part we assure them fearlessly, that of all the Restorations yet undertaken, the worst have meant the reckless stripping a building of some of its most interesting material features; whilst the best have their exact analogy in the Restoration of an old picture, where the partly-perished work of the ancient craftsmaster has been made neat and smooth by the tricky hand of some unoriginal and thoughtless hack of today. If, for the rest, it be asked us to specify what kind of amount of art, style, or other interest in a building makes it worth protecting, we answer, anything which can be looked on as artistic, picturesque, historical, antique, or substantial: any work, in short, over which educated, artistic people would think it worth while to argue at all. It is for all these buildings, therefore, of all times and styles, that we plead, and call upon those who have to deal with them, to put Protection in the place of Restoration, to stave off decay by daily care, to prop a perilous wall or mend a leaky roof by such means as are obviously meant for support or covering, and show no pretence of other art, and otherwise to resist all tampering with either the fabric or ornament of the building as it stands; if it has become inconvenient for its present use, to raise another building rather than alter or enlarge the old one; infine to treat our ancient buildings as monuments of a bygone art, created by bygone manners, that modern art cannot meddle with without destroying. Thus, and thus only, shall we escape the reproach of our learning being turned into a snare to us; thus, and thus only can we protect ourancient buildings, and hand them down instructive and venerable to those that come after us.

(Written by William Morris, 1877)

Appendix IV

ICOMOS Charters

The Venice Charter

Imbued with a message from the past, the historic monuments of generations of people remain to the present day as living witnesses of their age-old traditions. People are becoming more and more conscious of the unity of human values and regard ancient monuments as a common heritage. The common responsibility to safeguard them for future generations is recognized. It is our duty to hand them on in the full richness of their authenticity.

It is essential that the principles guiding the preservation and restoration of ancient buildings should be agreed and be laid down on an international basis, with each country being responsible for applying the plan within the framework of its own culture and traditions.

By defining these basic principles for the first time, the Athens Charter of 1931 contributed towards the development of an extensive international movement which has assumed concrete form in national documents, in the work of ICOM and UNESCO and in the establishment by the latter of the International Centre for the Study of the Preservation and the Restoration of Cultural Property. Increasing awareness and critical study have been brought to bear on problems which have continually become more complex and varied; now the time has come to examine the Charter afresh in order to make a through study of the principles involved and to enlarge its scope in a new document.

Accordingly, the IInd International Congress of Architects and Technicians of Historic Monuments, which met in Venice from May 25th to 31st 1964, approved the following text:

Definitions

ARTICLE 1. The concept of an historic monument embraces not only the single architectural work but also the urban or rural setting in which is found the evidence of a particular civilisation, a significant development or an historic event. This applies not only to great works of art but also to more modest works of the past which have acquired cultural significance with the passing of time.

ARTICLE 2. The conservation and restoration of monuments must have recourse to all the sciences and techniques which can contribute to the study and safeguarding of the architectural heritage.

Aim

ARTICLE 3. The intention in conserving and restoring monuments is to safeguard them no less as works of art than as historical evidence.

Conservation

ARTICLE 4. It is essential to the conservation of monuments that they be maintained on a permanent basis.

ARTICLE 5. The conservation of monuments is always facilitated by making use of them for some socially useful purpose. Such use is therefore desirable but it must not change the lay-out or decoration of the building. It is within these limits only that modifications demanded by a change of function should be envisaged and may be permitted.

ARTICLE 6. The conservation of a monument implies preserving a setting which is not out of scale. Wherever the traditional setting exists, it must be kept. No new construction, demolition or modification which would alter the relations of mass and colour must be allowed.

ARTICLE 7. A monument is inseparable from the history to which it bears witness and from the setting in which it occurs. The moving of all or part of a monument cannot be allowed except where the safeguarding of that monument demands it or where it is justified by national or international interests of paramount importance.

ARTICLE 8. Items of sculpture, painting or decoration which form an integral part of a monument may only be removed from it if this is the sole means of ensuring their preservation.

Restoration

ARTICLE 9. The process of restoration is a highly specialised operation. Its aim is to preserve and reveal the aesthetic and historic value of the monument and is based on respect for original material and authentic documents. It must stop at the point where conjecture begins, and in this case moreover any extra work which is indispensable must be distinct from the architectural composition and must bear a contemporary stamp. The restoration in any case must be preceded and followed by an archaeological and historical study of the monument.

ARTICLE 10. Where traditional techniques prove inadequate, the consolidation of a monument can be achieved by the use of any modern technique for conservation and construction, the efficacy of which has been shown by scientific data and proved by experience.

ARTICLE 11. The valid contributions of all periods to the building of a monument must be respected, since unity of style is not the aim of a restoration. When a building includes the superimposed work of different periods, the revealing of the underlying state can only be justified in exceptional circumstances and when what is removed is of little interest and the material which is brought to light is of great historical, archaeological or aesthetic value, and its state of preservation good enough to justify the action. Evaluation of the importance of the elements involved and the decision as to what may be destroyed cannot rest solely on the individual in charge of the work.

ARTICLE 12. Replacements of missing parts must integrate harmoniously with the whole, but at the same time must be distiguishable from the original so that restoration does not falsify the artistic or historic evidence.

ARTICLE 13. Additions cannot be allowed except in so far as they do not detract from the interesting parts of the building, its traditional setting, the balance of its composition and its relation with its surroundings.

Historic sites

ARTICLE 14. The sites of monuments must be the object of special care in order to safeguard their integrity and ensure that they are cleared and presented in a seemly manner. The work of conservation and restoration carried out in such places should be inspired by the principles set forth in the foregoing articles.

Excavations

ARTICLE 15. Excavations should be carried out in accordance with scientific standards and the recommendation defining international principles to be applied in the case of archaelogical excavation adopted by UNESCO in 1956.

Ruins must be maintained and measures necessary for the permanent conservation and protection of architectural features and of objects discovered must be taken. Furthermore, every means must be taken to facilitate the understanding of the monument and to reveal it without ever distorting its meaning.

All reconstruction work should however be ruled out *a priori*. Only anastylosis, that is to say, the reassembling of existing but dismembered parts can be permitted. The material used for integration should always be recognisable and its use should be the least that will ensure the conservation of a monument and the reinstatement of its form.

Publication

ARTICLE 16. In all works of preservation, restoration or excavation, there should always be precise documentation in the form of analytical and critical reports, illustrated with drawings and photographs.

Every stage of the work of clearing, consolidation, rearrangement and integration, as well as technical and formal features indentified during the course of the work, should be included. This record should be placed in the archives of a public institution and made available to research workers. It is recommended that the report should be published.

The following persons took part in the work of the Committee for drafting the International Charter for the Conservation and Restoration of Monuments:

Mr. PIERO GAZZOLA (Italy), Chairman
Mr. RAYMOND LEMAIRE (Belgium), Reporter
Mr. JOSÉ BASSEGODA-NONELL (Spain)
Mr. LUIS BENAVENTE (Portugal)
Mr. DJURDJE BOSKOVIC (Yugoslavia)
Mr. HIROSHI DAIFUKU (U.N.E.S.C.O.)
Mr. P.L. DE VRIEZE (Netherlands)
Mr. HARALD LANGBERG (Denmark)
Mr. MARIO MATTEUCCI (Italy)

Appendix V

Table 1.1 Some Natural Disasters, over a Two-Month Period (Courtesy: UN Disaster Relief Organization)

Date (in brackets if date of report)	
(1.2.80)	Cyclone Dean swept across Australia with winds reaching up to 120 m.p.h. and damaging at least 50 buildings along the north-west coast. About 100 people were evacuated from their homes in Port Hedland. Violent thunderstorms occurred on the east coast near Sydney.
12.2.80	Earth tremor, measuring 4 on the 12-point Medvedev Scale, in the Kamchatka peninsula in the far east of the Soviet Union. No damage or casualties were reported.
(12.2.80)	Floods caused by heavy rain in the southern oil-producing province of Khuzestan in Iran. The floods claimed at least 250 lives and caused heavy damage to 75% of Khuzestan's villages.
14.2.80	Earth tremors in parts of Jammu and Kashmir State and in the Punjab in north-west India. The epicentre of the quake was reported about 750 km north of the capital near the border between China and India's remote and mountainous north-western Ladaka territory. It registered 6.5 on the Richter Scale. No damage or casualties were reported.
(17.2.80)	Flood waters swept through Phoenix, Arizona, USA and forced 10 000 people to leave their homes. About 100 houses were damaged in the floods.
(19.2.80)	Severe flooding caused by heavy rain in southern California, USA, left giant mudslides and debris in the area. More than 6000 persons were forced to flee as their homes were threatened. Nearly 100 000 persons in northern California were without electricity. At least 36 deaths have been attributed to the storms. Some 110 houses have been destroyed and another 14 390 damaged by landslides. Cost of damage has been estimated at more than $350 million.
22.2.80	Strong earthquake measuring 6.4 on the Richter Scale in central Tibet, China. The epicentre was located about 160 km north of the city of Lhasa. No damage or casualties were reported.
(23.2.80)	Severe seasonal rains caused widespread flooding in seven northern and central states of Brazil, killing about 50 people and leaving as many as 270 000 homeless. Heavily affected were the States of Maranhao and Para where the major Amazon Tributary Tocantins burst banks in several places, as at the State of Goias where 100 000 people were left without shelter. Extensive damage was caused to crops, roads and communication systems. The government reported in late February that 2.5 billion cruzeiros had already been spent on road repairs alone.
(26.2.80)	Heavy rains brought fresh flooding to the southern oil-producing province of Khuzestan, Iran. At least 6 people were reported killed and hundreds of families made homeless by the renewed flooding.
27.2.80	Earthquake on Hokkaido Island in Japan. The tremor registered a maximum intensity of 3 on the Japanese Scale of 7. No damage or casualties were reported.
28.2.80	Several strong earth tremors in an area 110 km north-east of Rome, Italy. The tremors, which registered up to 7 on the 12-point Mercalli Scale, were also felt in the towns of Perugia, Rieti and Macerata as well as in north-east Rome. Slight damage to buildings was reported.
28.2.80	Earth tremor in the Greek province of Messinia in the south Peloponnisos, Greece. The tremor, which registered 3.5 points on the Richter Scale, damaged houses and schools.
7.3.80	An earthquake measuring 5 on the Richter Scale was felt on Vancouver Island off the coast of British Columbia, Canada. No damage or casualties were reported.
(9.3.80)	Heavy flooding in the southern provinces of Helmand, Kandahar and Nimroz in Afghanistan damaged or destroyed 7000 houses and rendered over 30 000 people homeless.
9.3.80	Earth tremor in eastern parts of Yugoslavia, measuring 6.5 on the Mercalli Scale. The epicentre was placed at 300 km south-east of Belgrade. There were no reports of damage or casualties.
(9.3.80)	Persistent drought in central SriLanka was reported to affect agricultural production and to ruin 150 000 acres of prime tea plantations. Water and electricity supplies were restricted by the government.
15.3.80	Heavy rains caused widespread flooding in northwestern Argentina causing the deaths of 10 people, with 20 reported missing. Nearly 4000 people were evacuated after the San Lorenzo river overran its banks.
16.3.80	A volcanic eruption occurred in the Myvatn mountain region in northern Iceland. No damage or casualties were reported.
16.3.80	Earthquake on the island of Hokkaido, Japan. The tremor registered a maximum intensity of 3 on the Japanese Scale of 7. No damage or casualties were reported.
19.3.80	Medium-strength earthquake in the central Asian Republic of Kirghizia near Naryn, USSR. No damage or casualties were reported.
23.3.80	Medium-strength earthquake near the border between Afghanistan and the USSR. Its epicentre was located about 1120 km north-west of Delhi. No damage or casualties were reported.
24.3.80	Earthquake measuring 7 on the Richter Scale in the Aleutian Islands off the Alaska peninsula, USA. Its epicentre was estimated just south of Umnak Island.
28.3.80	Torrential rain and strong winds struck central and southern areas of Anatolia, Turkey, cutting road and rail traffic. A landslide in the village of Ayvazhaci killed at least 40 people, while an additional 30 villagers were reported missing. In Adana Province 12 000 houses were affected by floods.