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PROCEEDINGS OF THE U.S.-ITALY WORKSHOP ON GUIDELINES FOR SEISMIC EVALUATION AND REHABILITATION OF UNREINFORCED MASONRY BUILDINGS. HELD IN PAVIA, ITALY ON JUNE 22-24, 1994

ILLINOIS UNIV. AT URBANA-CHAMPAIGN. DEPT. OF CIVIL ENGINEERING

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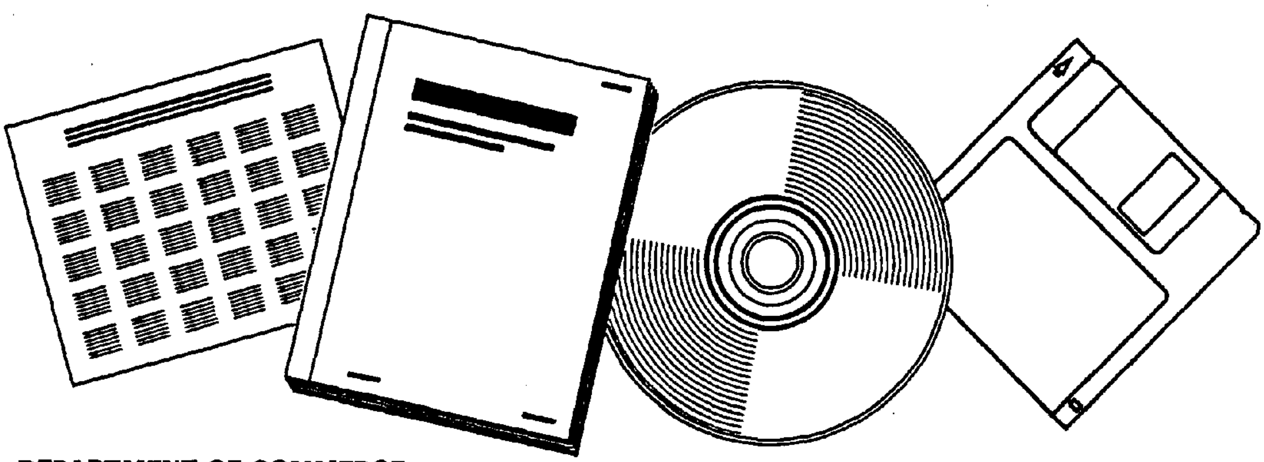

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NATIONAL CENTER FOR EARTHQUAKE ENGINEERING RESEARCH

State University of New York at Buffalo

Proceedings of the U.S.-Italy Workshop on Guidelines for Seismic Evaluation and Rehabilitation of Unreinforced Masonry Buildings

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Technical Report NCEER-94-0021

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In an effort to assist in the development of engineering procedures for the seismic evaluation and rehabilitation of unreinforced masonry buildings, designed, if at all, for gravity loads only, this workshop volume compares the Italian and American experience in these areas. The intent of the workshop was to review hazard mitigation for URM buildings as a whole. Thus, workshop topics included architectural issues in building preservation and rehabilitation, development of Eurocode 8 and FEMA-BSSC-ATC-33, case studies of retrofitting projects and in situ testing methods for masonry construction. The workshop program called for introduction of each general topic, followed by the presentation of a paper on that topic from one Italian and one American participant, and by group discussion. Accordingly, this volume offers 16 paired papers, as well as the complete text of the workshop resolutions concerning future research and cooperation in this field.

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of the
U.S.-Italy Workshop on
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of Unreinforced Masonry Buildings

held at the
Department of Structural Mechanics
University of Pavia, Italy
June 22-24, 1994

Edited by D.P. Abrams[1] and G.M. Calvi[2]
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per la Difesa dai Terremoti (GNDT)

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NATIONAL CENTER FOR EARTHQUAKE ENGINEERING RESEARCH
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PREFACE

Past earthquakes in both Italy and the United States have demonstrated the vulnerability of unreinforced masonry construction to moderate or strong ground shaking. In recent times, substantial masonry damage has been reported from earthquakes in Friuli (1976), Perugia (1979), Campania/Basilicata (1980), Whittier (1987), Loma Prieta (1989) and Northridge (1994). Because of the fears that future earthquakes will occur in the eastern United States, or in Italy where unreinforced masonry buildings are predominant, there is an immediate need to develop engineering procedures for seismic evaluation and rehabilitation of this type of structural system. Like older Italian masonry construction that has been designed only for gravity forces, and not lateral forces, seismic evaluation and rehabilitation of unreinforced masonry buildings in the eastern United States poses a challenging engineering problem. Much can be learned from the Italian experience with unreinforced masonry because of the preponderance of buildings in Italy of this type. Correspondingly, Italians can learn from American practices in rehabilitation strategies, earthquake engineering research, and code development.

On May 26, 1993 a Memorandum of Understanding was signed by Professors George Lee and Vincenzo Petrini for cooperative research and collaboration between the Gruppo Nazionale per la Difesa dai Terremoti (GNDT) of the Consiglio Nazionale delle Ricerche (National Group for the Protection from Earthquakes of the National Research Council of Italy) and the National Center for Earthquake Engineering Research of the United States. The object of the agreement is to promote and enhance cooperation and collaboration among and between research institutions, academic organizations, colleagues and others in Italy and the National Center for Earthquake Engineering Research at the State University of New York at Buffalo; to improve the body of knowledge available and thereby reduce future losses of life and property damage from earthquakes; to develop a planned program of coöperative and collaborative research in the area of earthquake hazards mitigation; and to allow for the widest possible publication and discussion of the results of that collaboration. The area of initial cooperative research was defined as seismic behavior and protection of existing masonry buildings.

The first formal activity under the Memorandum of Understanding is this workshop. The topic of guidelines for seismic evaluation and rehabilitation was chosen so that the workshop focus could be sufficiently broad to apply to the general problem of hazards mitigation. Because efforts are underway in both Italy and the United States on development of seismic guidelines for existing buildings, the workshop papers and resolutions were felt to be appropriate to augment these ongoing activities.

Rather than concentrate on the narrow subjects of computational modeling or laboratory research, workshop topics included architectural issues in building preservation and rehabilitation, development of Eurocode 8 and the FEMA/BSSC/ATC-33 Guidelines, case studies of building retrofit projects, and insitu testing methods for masonry construction. Thus, the papers contained in these proceedings provide a snapshot of the overall hazard mitigation problem rather than an in-depth collection of technical papers on a single topic.

The workshop program was arranged by first selecting the general topics, and then seeking one Italian and one American speaker. Ample time was allotted for each paper presentation which was followed by discussion periods on each topic. At the conclusion of the workshop, twenty-six resolutions were formulated based on the discussions with each set of papers.

The workshop was preceded by a half-day *Italian Seminar on Numerical Modeling of Unreinforced Masonry Buildings.* Analytical modeling efforts by various investigators throughout Italy were summarized and compared at the seminar. True analytical predictions were first presented for lateral-force behavior of a two-story research building that had been tested within the structural engineering laboratory at the University of Pavia. Then, previously concealed test data on behavior of the test structure was revealed and used to judge the accuracy of each analytical prediction.

Following the workshop, a field trip was held in the historical center of Pavia. Sites included an ancient unreinforced masonry tower structure that has been recently strengthened, and the Pavia Duomo whose structural system is continuously being monitored for movements.

The workshop and related activities were well attended with over seventy participants (see Appendix for address listings). Because of the success of this initial activity under the NCEER-GNDT Memorandum of Understanding, subsequent collaborations are expected to develop between Italian and American researchers in the area of earthquake hazards mitigation. Possible avenues for future cooperation are additional workshops, the exchange of scholars and the transfer of technical documents between the two countries.

The workshop would not have been possible without the financial support of the National Center for Earthquake Engineering Research (Project Number 93-3703) which is funded by the U.S. National Science Foundation and the New York State Science and Technology Foundation, and the Italian National Group for the Defense against Earthquakes (GNDT) which is funded by the Italian National Research Council (CNR). Appreciation is extended to the Department of Structural Mechanics at the University of Pavia for hosting the workshop, and to the Italian brick supplier, Gruppo RDB, for sponsoring the workshop lunches.

Observations, opinions, findings, and conclusions presented in these workshop proceedings are those of the individual participants, and do not necessarily reflect those of the NCEER, NSF, NYSSTF, GNDT or CNR.

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RESOLUTIONS

US-Italian Workshop on:

Guidelines for Seismic Evaluation and Rehabilitation of Unreinforced Masonry Buildings

University of Pavia, Italy
June 22-24, 1994

Over the three days of the workshop, a number of themes tended to emerge from the paper presentations and discussions which followed. Many of the comments, criticisms and views were common for Italian and American participants. At the end of the workshop, these opinions were consolidated into the following set of resolutions. They are classified by subject and are not necessarily in order of priority. These resolutions represent a consensus of the workshop participants, and do not necessarily reflect opinions of NCEER or GNDT.

General

1. The GNDT-NCEER Memorandum of Understanding signed on May 26, 1993 has initiated a fruitful cooperation between Italian and American researchers in the area of seismic behavior and protection of existing masonry buildings. Continued cooperation between the GNDT and NCEER in the future is encouraged.
2. The exchange of researchers between the United States and Italy should continue with long-term visits of junior researchers, or short-term visits of senior researchers.
3. A process should be developed for the efficient exchange of technical documents between the United States and Italy.
4. A similar US-Italian Workshop should be held in two years in the United States.

Development of Guidelines for Seismic Rehabilitation

1. Development of seismic evaluation or rehabilitation codes/guidelines should be shared between Italian and Americans.
2. Intervention strategies for seismic rehabilitation of unreinforced masonry buildings should continue to be discussed between Italian and American engineers and architects.
3. Research should be pursued to investigate relative feasibilities of various rehabilitation strategies for improving seismic resistance of unreinforced masonry buildings.
4. Relations between various seismic performance limits, and limit states for unreinforced masonry structures need to be defined.

RESOLUTIONS

Experimental Research

1. Experimental research should be continued as a means for identifying basic resistance mechanisms and calibrating computational models.

2. Further laboratory tests should be done on different structural configurations of URM building systems.

3. Ultimate load testing of actual buildings should be pursued to help understand the relations between idealized analytical or physical models and typical construction.

4. The seismic performance of retrofitted unreinforced masonry buildings during earthquakes should be examined.

5. Development of technical procedures for real-time monitoring of historic structures should be shared between the two countries.

6. Research on brick and block construction should be extended to include stone masonry.

7. Further research should be done to examine characteristics of existing and older mortars, and their influence on response of masonry construction.

8. Further development of insitu test procedures for measuring mechanical properties of masonry should be pursued.

Analytical Research

1. Analytical modeling of unreinforced masonry buildings should be pursued as a means for extrapolating laboratory test data.

2. Sensitivity studies should be done using analytical models to identify critical modeling parameters. Once identified, more experimental research should be done to better quantify those critical parameters.

3. Analytical procedures prescribed in future guidelines or codes for general seismic rehabilitation of buildings should be examined for their applicability to unreinforced masonry construction.

4. Research on analytical modeling should be focused towards development of engineering guidelines for seismic rehabilitation of unreinforced masonry buildings.

5. Both analytical and experimental research should strive to develop simple analytical models for seismic evaluation of rehabilitated buildings.

6. Seismic response of building clusters should be studied.

RESOLUTIONS

Condition Assessments

1. Methods need to be developed for relating results of condition assessments to structural analyses procedures.
2. Research should be done to develop more accurate condition assessment procedures for multiple-wythe masonry walls and rubble walls.

Social and Economic Issues

1. Cost differences for various seismic rehabilitation schemes should be studied.
2. Fundamental philosophies of architectural preservation should be shared between the two countries.
3. Limits of professional liability should be defined as they relate to seismic rehabilitation.

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Section I

Issues in Building Rehabilitation and Preservation

ARCHITECTURAL ISSUES IN THE SEISMIC REHABILITATION OF MASONRY BUILDINGS

Randolph Langenbach *

ABSTRACT

This paper explores the architectural and historic preservation issues raised by seismic repair and retrofit work on masonry buildings. The first part examines the sources of meaning in historical buildings as cultural artifacts, using examples to illustrate how buildings can have significance beyond their visual image or architectural style. For this reason, if preservation is to be successful, the actual material fabric of an artifact must be preserved. This fact must be recognized by seismic design engineers so that seismic retrofit work can be carried out with the least possible irrevocable alteration to the historic structural system as well as the historic architectural finishes. The paper goes on to explore some of the opposing differences which have existed between contemporary conservation technology and seismic retrofit practice, analyzing how seismic retrofit work may be able to benefit from knowledge and research developed in the conservation field. In conclusion, the paper suggests four areas where further work is needed to improve seismic retrofit practice with historic masonry buildings: research on mortars and post-elastic behavior of the masonry, the development of existing building-type specific building codes, finding ways to limit liability for design professionals dealing with existing buildings, and further analysis on what is an acceptable life safety goal for historic buildings.

INTRODUCTION

Our approach to the structure of buildings has gone through a transformation in modern times. Traditionally, most major buildings were solid walled structures with the walls bearing directly on the ground. With the current predominance of steel and reinforced concrete as the materials of choice for larger buildings, we are now used to the erection of frames, onto which the enclosure cladding system is attached.

With the "Postmodern" fascination with historical forms and details, the contrast between the old and new systems has only recently become particularly noticeable. This style shift has brought back a desire to design buildings which have the solid walls of their historic counterparts, but which, unlike them, have to be constructed as a series of light, jointed panels attached to the underlying frame. Often the results simply fail to capture the kind of texture and meaning which is found in the older buildings. Architects continue to struggle for solutions, only to find that the source of the feeling they are trying to capture is simply not accessible in Dryvit, GFRC, Fiberglas, or panelized veneer brick, with their frequent need for expansion joints cutting across the architectural details. As engineers work hard to convert the highly indeterminate, ambiguous and non-linear behavior of historic masonry construction into something which can be understood with mathematical certainty, architects struggle to wrest control of the seemingly rigid and unyielding materials of modern day conventional building systems, trying to breathe the kind of subtle life into them that they find at the root of the aesthetic quality of historic structures.

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This transformation in construction technology parallels a similar change in engineering practice which now relies to a great extent on frame analysis for the design of building structures. Traditional heavy wall masonry buildings tend to defy analysis by the usual present day methods, forcing many practicing professionals to do what they do not like to do – designing in part by guess work. Research in the area of unreinforced masonry is so important because without it, historical buildings will be lost simply because engineers and architects will be loath to touch them because they cannot be made to fit their mathematical design models. This may be true even though the same structures have withstood major past earthquakes, and the damage records is known. For example, a number of historic buildings in California which survived the 1906 San Francisco earthquake are threatened now more by hazard mitigation legislation than by future earthquakes.

THE CULTURAL SIGNIFICANCE OF HISTORIC BUILDING FABRIC

Modern Engineering science, new materials, and current codes have gone a long ways towards reducing the fear of catastrophe and death from earthquakes. This has been true despite the spectacular failures which each major earthquake seems to leave in its wake. Earthquake design is an evolving and constantly changing practice largely because the actual events are so rare, and when they do occur, the earthquake forces can be so large that some structural damage is expected even in new structures. As a result, the line between acceptable and unacceptable risk and performance is vague and fluid.

In the field of historic preservation, the problem of seismic risk can not be solved by stricter design codes, better new materials, or a more stringent engineering design. It is exactly these things which heighten the dilemma with older structures, threatening the very historical qualities which we seek to save.

It has become a familiar sight in many parts of the world to see the stone exterior walls of gutted buildings held up by shoring while they await the construction of new interior floors and roof. Fine old masonry buildings are often stripped of their interior finishes, with the steel reinforcing rods being erected against the inside of the exterior walls in preparation for a sheet of concrete. Roofs of ancient tiles or slate are torn off to be replaced by new tiles and slate after the obligatory concrete or plywood diaphragm is installed.

One might ask: *"what's the fuss - the exterior walls have been preserved, have they not? The interior will be rebuilt and the new work will be hidden - the view will be just the same when it is all completed."* Many architects, not just engineers, fail to understand the meaning of what is lost along the way when this kind of work is carried out. Donald Appleyard observed in 1978:

> *The professional and scientific view of the environment usually suppresses its meaning.... Environmental professionals have not been aware of the symbolic content of the environment, or of the symbolic nature of their own plans and projects.... Professionals see the environment as a physical entity, a functional container,...a setting for social action or programs, a pattern of land uses, a sensuous experience–but seldom as a social or political symbol.*[1]

Historic buildings do not just carry their cultural significance as relics by image alone. While understanding the architectural style and decorative form of historic structures is important, the cultural meaning of many of the most significant buildings is resident within the reality of the artifact itself. An historic structure is important because it is exactly that - it is old, and thus has been a part of human lives. As the English critic John Ruskin eloquently stated:

> *Indeed the greatest glory of a building is not in its stones, or in its gold. Its glory is in its Age, and in that deep sense of voicefulness, of stern watching, of mysterious sympathy, nay, even of approval or condemnation, which we feel in walls that have long been washed by the passing waves of humanity....*
>
> *It is in that golden stain of time that we are to look for the real light, and color, and preciousness of architecture; and it is not until a building has assumed this character, till it has been entrusted with the fame, and hallowed by the deeds of men, till its walls have been witnesses of suffering, and its pillars rise out of the shadows of death, that its existence, more lasting as it is than that of the natural objects of the world around it, can be gifted with even so much as these possess of language and of life.*[2]

Seismic protection and strengthening forces us to confront one of the central dilemmas of historic preservation – the fact that preservation is forced to encompass change and renewal. Unlike maintenance and rehabilitation from decay, a seismic project may tear apart a building which was otherwise in good repair and make it almost entirely new. In such an instance, only the image, rather than the substance, of much of the historic fabric is preserved. Masonry buildings are particularly vulnerable to this approach.

Sometimes seismic projects are promoted as opportunities to “restore” the original appearance of a buildings, stripping away the later alterations in order to return them to their original appearance. In Sacramento, California, the State Capitol is such an example. The Capitol was completely gutted in 1976, leaving only the exterior walls and the central drum and dome. All of the interior floors and walls were removed and replaced in reinforced concrete. The remaining masonry was covered with an internal skin of shotcrete and the floors were replaced in reinforced concrete. As a result, while the interior of this building is now genuinely spectacular, with impressive museum rooms, excellent craftsmanship, rich materials, stunning colors and textures, none of it is genuine. A "heart transplant" was authorized when an "ace bandage" may have been all that was needed. The Capitol needed to be strengthened and repaired, but one should ask whether the risk identified in 1971 could not have been satisfactorily alleviated by less drastic, destructive, and expensive measures.

> *The quest for authenticity, and the search for "real" meaning through "honesty" of form, often leads to the destruction of that which it seeks by inducing fakery....Authenticity is not a property of environmental form, but of process and relationship....Authentic meaning cannot be created through the manipulation or purification of form, since authenticity is the very source from which form gains meaning.*[3]

This gutting of structures for seismic strengthening is not limited to the United States. For example, following the 1979 earthquake in Montenegro, Yugoslavia, many structures in the historic city of Kotor have been reconstructed with reinforced concrete floors, replacing the original heavy timber. In some of these structures, reinforced concrete columns have been cut into the masonry, forming completely new reinforced concrete structures, with the historic masonry reduced to a veneer.

Another example, in Portugal, is the little mountain village of Piódau. The Portuguese government recently listed this picturesque mountain village of stone buildings as an historical site. Located in earthquake-prone country, many of the stone houses are being strengthened. The typical seismic strengthening consists of replacing their timber floor and roof structures with reinforced concrete. Some of the walls, which had been laid with very little mortar, are being re-laid in strong cement mortar. While undoubtedly safer, the visual effect of this work is the loss of the texture and feel of the traditional surfaces. The patina and sense of the country masons' and plasterers' handiwork is erased. If the approach had been to repair and augment the timber interior structures and tie them to the existing walls, rather than replace them, the historical quality of the buildings would have survived the life safety improvements.

The debate over such alternatives always turns to the question of how much life safety protection is enough. When existing archaic construction remains in use, even if improved, can it be relied on to perform adequately? However, at the core of this issue is the fact that, unless the architects, planners and engineers identify and understand the importance of the original structural and interior fabric of the historical buildings, and bring this understanding into their designs, such destruction will continue because they will do what they are used to doing with new structures. This consideration must include the evidence of the original handiwork, rather than just the appearance of a building from a distance.

Another striking example is South Hall at the University of California, Berkeley. Constructed of brick with timber floors in the 1870's, South Hall is the oldest surviving building on the campus. In the 1980's, it was gutted to undergo seismic strengthening under the University's campus wide program. The retrofit plans included the reinforced concrete "shotcrete" jacketing of the inside surface of many of the exterior walls, and the demolition and replacing of the timber floors with steel and concrete. In the process of carving channels into the walls, it was discovered that the original builders had installed bond-iron in the masonry – continuous bars of wrought iron which extended from corner to corner above and below the windows in all of the building's walls. Dog anchors, which secured the floors to the walls, were also discovered hidden in the walls. At the corners, the bond iron bars were secured by large bosses on gigantic cast iron plates which formed part of the architecture of the building.

Because the designers had never thought to investigate the structural history of the building, including whether these great cast iron ornamental plates on the corners of the building served a structural purpose, the existence of the bond iron was not known until the demolition for the retrofit. All of these bond-iron ars were cut as a result. In addition, as historically significant and advanced as this original system was, no recordation of its design was ever conducted. The irony was that one of the engineers said that, had they known of the existence of the bond iron and the dog anchors, their designs may have been different and less extensive. When it was discovered, however, it was too late to change the designs, and the early seismic technology was destroyed.

One may ask, "why is it important to preserve what had been hidden in the historic walls – nobody could see it anyway?" Perhaps documenting it, which was not done, would have been sufficient, but this example also illustrates one of the important points about seismic design – that is that many engineers and architects have the false belief that the today's engineering design is, not only better than anything which has been done in the past, but is the ultimate solution which will require no further interventions. They believe that their work will make the building strong and complete, and that no one will have to do anything other than maintenance and superficial remodeling ever again. Here, at South Hall, the designers failed to know what had been put into the walls to resist earthquakes a mere 100 years ago, despite the fact that great cast iron plates to which the bond iron straps were attached, were fully exposed on the outside of the building. What is there to make certain that our successors will be any better informed about the work done today?

In addition, with the irreversible conversion of the masonry walls of South Hall into a veneer of masonry on reinforced concrete, the integrity of those walls as masonry walls was destroyed. One of the principle advantages of masonry is that it can be repaired by being dismantled and re-laid. Now it has been fused together into one solid mass of unyielding concrete. Years later, should it be necessary to repair the brickwork or replace the concrete jacket because of rusting of the re-bars or another reason, it will not be possible. The present day seismic work will indeed last the life of the building simply because the building's life is now forced to be limited to that of the new work.

This point may seem far-fetched, but historical buildings are worthy of such long term consideration. It should be remembered that the Nineteenth Century restorers of the Parthenon introduced iron cramps which, when they rusted in the Twentieth Century, destroyed some of the original marbles. Should anyone wonder whether the state-of-the-art at the time of the 19th century restoration represented progress from earlier times, they should consider the fact that the ancient builders had used a less rust-prone iron, which, when protected by a lead jacket, survived over 2,000 years to this day without distress.

LEARNING FROM THE PAST

Many people make the mistake of thinking that it is only our generation which has discovered ways of resisting the threat of earthquakes in structural design. They come to believe that older forms of construction practice must be more dangerous simply because they were designed before current seismic codes were promulgated, or before current engineering knowledge about earthquakes had been developed. Certainly, the introduction of steel provides ductility where masonry could not, and yet the recent discoveries of the failures of the welds in over 100 of the 400 steel buildings affected by the Northridge Earthquake should provide some humility in the face of this awesome force. While many masonry buildings have tumbled in earthquakes, they have not always tumbled. As was witnessed in Armenia recently, it was the modern reinforced concrete buildings which collapsed killing thousands, while the older masonry buildings nearby remained mostly intact, providing refuge for the displaced occupants of the newer buildings.

In places as diverse as Turkey, Yugoslavia, Kashmir, and Nicaragua, indigenous forms of construction were developed or adapted to respond to the earthquake threat where available resources demanded that masonry continue to be used. In Kashmir, an elaborate system of interlocking horizontal timber runner beams was used, without vertical wood columns, to hold the rubble masonry and soft mud mortar buildings together on the silty soil. Historical reports confirm that these buildings withstood earthquakes better than the nearby unreinforced brick palace and British built government buildings. Today, many of these vernacular structures are falling in favor of reinforced concrete structures, which, because of poor local construction practices, may actually prove to be less resistant than their low tech, unengineered historic predecessors.

Restoration professionals sometimes fail to understand the subtleties of seismic resistance in older structures. Believing that strength and stiffness is necessary, they destroy original construction systems to gain shear strength at the expense of earlier solutions which may still be valid. In Dubrovnick, before the recent civil war, restorers of the historic palace uncovered an interior wall they had thought was solid masonry to find a basket wave of small timber studs, with brick or stone masonry loosely fitted together between the studs. The restoration engineer stated at a conference that this *"poorly constructed wall was immediately removed and replaced during the restoration of the building."* Instead of being *"poorly constructed,"* this wall deliberately may have been constructed in this fashion to resist earthquakes. The wall, which was similar to Bahareque construction found in Central America, may have represented a far greater understanding of seismic engineering than pre-modern builders are given credit for today.

BUILDING CONSERVATION PRACTICE VS. SEISMIC STRENGTHENING

While it is impossible to ignore present day advances and advocate a return to traditional construction practice, the narrow assumption that new is always better can blind us to the potential gains which an understanding of the earlier forms of construction may provide us in the present. This is particularly true for the advancement of building conservation and seismic rehabilitation practice. For years, these have been seen as separate and opposing fields of practice, with solutions which seem in basic conflict with each other. For example, for years, conservation professionals have specified that restoration mortar consist of a high lime mix which is weaker than the masonry units. Code requirements have established that mortar must consist of a high cement mixture and meet high strength standards which have proven to be anathema to proper conservation of older masonry walls. The discovery of the importance of reducing or eliminating Portland cement from masonry mortars in restoration is one of the cornerstones of recent conservation practice:

> *The use of lime-sand mortar ...furnishes a plastic cushion that allows bricks or stones some movement relative to each other. The entire structural system depends upon some flexibility in the masonry components of a building. A cushion of soft mortar furnishes sufficient flexibility to compensate for uneven settlement of foundations, walls, piers and arches: gradual adjustment over a period of months or years is possible. In a structure that lacks flexibility, stones and bricks break, mortar joints open and serious damage results. 4*

This was not meant to refer to masonry in earthquakes, but in light of the Kashmiri experience it is intriguing to ask, whether the notion of a "plastic cushion" might be an appropriate concept for walls subjected to earthquake forces. It is worth noting the conflict between the Historic Preservation documents which recommend using the weakest and most lime rich ASTM formula (K) (1 unit cement) to (2 1/4 to 4 units lime) for restoration work, and the Uniform Building Code, which prohibits the use of mortar weaker than the three strongest categories, known as ASTM types M, S & N: (1 unit cement) to (1/4 to 1 1/4 lime) for any mortar used in structural masonry (which includes, of course, most historic masonry walls.)

One reason for this conflict is that while the Code is founded upon the performance of the wall under load at its design strength at the point of construction, the preservation documents are aimed towards maximizing the long-term durability of walls with relatively weak masonry units in response to all environmental conditions. One only needs to compare the long-term performance of ancient masonry and modern masonry to see the merits in the softer, high lime mortars, and yet, the codes now make beneficial use of this knowledge difficult. Other examples abound where modern uses of masonry has proven short-lived because of environmental degradation of the system. Seismic design must fit into a larger performance picture, where other environmental assaults are considered as well as the occasional earthquake.

A CRISIS OF COST

Concerns over the impact of seismic strengthening policies is more than just one of potential loss of original fabric. It is also one of economics. As long as politicians and the public believe that historic masonry buildings are enormously risky unless great sums of money are spent to convert their structural systems into steel or concrete, vast numbers of important cultural monuments are at risk. This issue has expanded recently in the United States to include large scale 20th century masonry buildings constructed with steel or concrete frames. It is exactly the current crisis with these types of buildings which confirm the importance of engineering research and the development of specialized codes for masonry buildings and historic buildings in general.

The crisis can be illustrated by an example in Oakland, California, where one brick and terra cotta clad steel frame historical building, the City Hall, is being repaired from Loma Priéta Earthquake damage and seismically upgraded at the extraordinary cost of $530 per square foot ($5,700/square meter), which is more than 3 times the cost of a new building of comparable quality. Six blocks away, another office building, the Oakland Medical Building, was just repaired and seismically upgraded to the same codes for a cost of only $11 per square foot ($118/square meter). The City Hall design uses the now popular newly developed base isolation technology, while the Medical Building is a fixed base design, but both schemes were promoted as "cost-effective" designs to meet the requirements of the building code. (See figure 1)

With a difference between two projects, both promoted as necessary and expedient, of over 35 times, it is evident that there is little consensus in this particular field over what is required and beneficial to meet the seismic threat. While certainly the expected performance of the base isolated design is greater than the fixed base design, and even though part of the difference is for interior remodeling of the City Hall, it is questionable whether this justifies 35 times the cost. While many celebrated the repair and upgrade solution for City Hall because it preserved the building, historic preservation suffers in the long run from such gargantuan projects as that of the Oakland City Hall because the public begins to believe that such costly solutions are the only way to make such buildings safe.

The situation with bearing wall masonry buildings in California is no longer as distorted. The reason for this is that recent research has resulted in the development of a new code specific to this building type. While public perception on the safety of masonry buildings is still unduly negative, and price spreads between different engineers' designs can still be large, the existence of this new code has helped to narrow the spread, and make economical solutions possible.

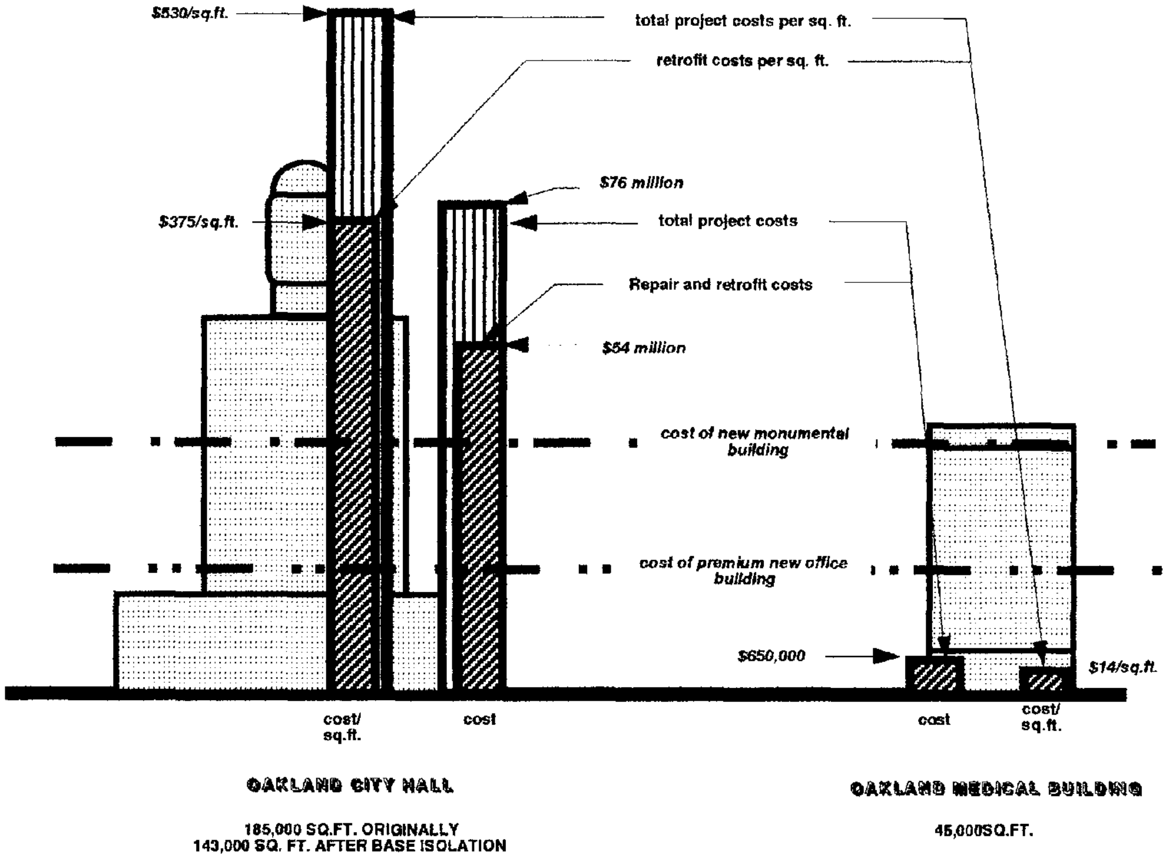

FIGURE 1: COST COMPARISON BETWEEN TWO RETROFIT PROJECTS

The code for masonry buildings, which has now been adopted as a model code in California is Appendix, Chapter 1 of the _Uniform Code for Building Conservation._ This appendix contains the engineering provisions for bearing wall masonry structures. These provisions were derived from the "ABK Methodology," an engineering design methodology for unreinforced masonry bearing wall buildings developed by a team of engineers in Los Angeles under a research grant from the National Science Foundation.

One of the principal features of this methodology is the provisions which anticipate and exploit the post-elastic behavior of the wood and plaster interior partitions and floor diaphragms, thus computing a consequential reduction in the forces on the masonry walls. Another result of the ABK research is the finding that masonry buildings actually respond differently than the way the traditional codes and engineering approaches assumed. Rather than amplifying the forces of the earthquake, the heavy masonry walled building has the effect of dampening the shaking by acting as a "*rigid rocking block on a soft soil base.*" This is to be compared with the common code analysis of seismic force on a building which models the building as a "*single degree of freedom, 5% damped elastic oscillator with a fixed base.*[5]"

Using the ABK method of analysis, the computed force levels in an unreinforced masonry building are lower than found under conventional code analysis. The results of this methodology on the design of retrofit strategies for individual masonry buildings is that the amount of strengthening work which is computed to be required is less than that shown as needed when conventional strength based linear elastic analysis is used. This approach thus reduces the retrofit intervention and costs.

An even more significant step for historical buildings in general has been taken in California with the adoption of the *State Historical Building Code*. This Code, which applies to all historical buildings, including even those which are only on local lists, allows much greater design and engineering flexibility than is possible under the conventional prevailing code which is primarily meant for new buildings. Instead of proscriptive requirements, the *State Historical Building Code* describes general performance objectives which must be met. The specific solutions are left up to the designers. The code also encourages the use of archaic materials and systems as part of the structural system, providing some minimum values for these systems where they are available.

AREAS FOR FURTHER RESEARCH AND DEVELOPMENT

1) Engineering research

Three topics for further research which would benefit masonry building preservation particularly come to mind. One is the study of the effects of mortars of varying strength and constituents, a second is further study to develop code values for stone masonry, particularly with a random ashler or rubble wall bedding pattern, and a third is further study on the post-elastic in-plane strength and behavior of masonry bearing and infill walls.

Mortar: Most masonry wall studies have not introduced mortar strength and mortar ingredients as the principal variable in laboratory experiments. The potential benefits of high lime mortar in construction are well known in the field of building conservation technology, but not adequately explored in terms of its effects on seismic performance. Historic walls are often treated only as dead load in lateral capacity analysis because of low mortar strength, but when combined with certain bedding plane and window frame reinforcing techniques, the performance of such walls may be made satisfactory by restraining deformations and avoiding collapse potential, while allowing for cracking and energy dissipation. The objectives of such research would be to establish a sound basis for the preservation of the integrity of historic stone and brick masonry by avoiding the need for destructive concrete coatings.

The most important attribute of mortar strengths below that of the masonry units is that, when the wall does crack, it does so along the mortar joints. This results in greater overall stability than if the units themselves were to fracture. At the 1988 International Brick/Block Masonry Conference in Dublin, a paper by Dr. W. Mann, Univ. of Darmstadt, generated criticism around the assertion that masonry bedded in mortar with "low cohesion [is] favorable" because it contributes to "a type of "ductile" behavior.

Such a statement is diametrically opposed to the conventional wisdom that mortar must be strong to resist earthquakes. However, in the traditional examples described above, where the weak mortar is combined with the overall flexibility of the building structure, the restraint provided by the timber beams, and the pre-compression provided by the weight of the overburden, weak mortar may be more resistant to catastrophic fracture and collapse by allowing the cracks to be distributed throughout the wall. The flexibility and internal damping of the structure can serve to change the building's response, reducing the out-of-plane forces in the masonry walls while the timber serves to keep the weaker masonry in place. While research has shown that weak mortar can cause problems with unreinforced masonry walls, particularly for out-of-plane shaking, perhaps some mechanical ties within the walls can fill the role that the timbers did in traditional construction by holding the masonry units in place while the wall deforms.

There has been some significant progress in this direction in Europe, and even in New Zealand. In Greece and New Zealand, several projects have achieved seismic strengthening by simply wrapping cables around the masonry structure, which are hidden by the stucco, or left visible on the surface. Utilizing the strengthening effect caused by tying the masonry together to create horizontal bands similar in their purpose to the timber runners of the Dhajji-Dwari system, these buildings continue bear their own weight on the unaltered existing masonry. Such systems have the advantage of causing little disruption to the historic masonry surface or the integrity of the wall. The cables are also accessible for inspection and can easily be replaced. What is radical about this, and other surface mounted strengthening systems that the retrofit work is left visible as a frank statement of this part of the building's historical evolution. Sometimes it is important to recognize that, as was the case with South Hall, greater damage may be incurred by making changes hidden behind walls which have been radically altered or rebuilt, than by exposing the changes in front of walls which are thus left intact.

Stone masonry code values: The need for more information on capacity values for rubble and random ashler masonry which can be introduced into the building codes. Engineers are loath to apply the values which have been developed for brick masonry, but the recommended test techniques, such as the push test, are only remotely applicable to stone masonry situations. Lacking even minimum code capacity values, the conservative approach is to impart little or no capacity to the existing masonry. In the United States, this has resulted in costly and invasive designs for many stone buildings and the unnecessary demolition of a number of important historical buildings because retrofit schemes proposed proved to be too expensive.

Post-elastic behavior: More research is also needed on the post elastic behavior of masonry of all types. Even the recent unreinforced masonry building codes developed in California stop short of including values derived from the behavior of masonry when it is cracking and yielding in an earthquake. The codes for present day construction such as steel and reinforced concrete are based on linear elastic calculations using reduced forces to approximate post elastic actual behavior, but designers often give very low values to masonry because of its lack of material ductility. However, as a system, there is substantial remaining capacity in a wall which has begun to crack before it becomes unstable. If buildings fell down the moment masonry walls exceed their elastic strength, there would be far greater death and destruction in past earthquakes. Practicing engineers are often loath to depend on masonry for part of the load resisting mechanism because of the lack of realistic code values on which to base their design, and thus protect their liability in the event that an earthquake exceeds the strength of the wall.

2) Building Codes:

The adoption of the *Uniform Code for Building Conservation* as a State of California's model code for existing buildings, and the enactment of the *State of California Historical Building Code* have both gone a long ways to allowing for sensitive and cost-effective improvements to historical buildings in California. A code specific to the masonry infill frame building type is under development by a team of California engineers. The absence of such a code has been made conspicuous by the breadth of costs between different projects, and the sometimes acrimonious disagreements over what strengthening is necessary.

In Europe, as the EEC has moved towards unified building codes, the problem of making existing buildings, particularly historical buildings in different countries, fit into the a single universal code must be dealt with. It is strongly recommended that a separate code for historical buildings be developed. Like the *State Historical Building Code* in California, this code should be based on performance objectives, rather than proscriptive construction procedures or systems. An Internationally standard codes which applies to new and old buildings alike, will fail to cover the specific needs of historical building types which vary from region to region. What may be sound practice in one area, may be destructive of cultural value in another. Provisions for existing buildings with archaic construction systems, and earlier interior layouts, must be included into alternate codes or many buildings will be lost.

3) Engineer's liability:

A discussion of codes inevitably leads to a discussion of the problems surrounding professional liability. In the United States, many preservation problems result from the fact that engineers and architects are afraid of malpractice claims if they undertake solutions which are different from the code, and damage occurs in an earthquake. This has often forced them to be more conservative with existing masonry buildings than they would have to be with new buildings. This is true because the code for new buildings, although expecting structural damage to occur in a major earthquake, is very specific in the construction requirements. With old buildings with archaic pre-code structural systems which cannot be made to meet the letter of the current code, designers feel vulnerable to lawsuits regardless of the level of damage.

In a sense, present day professionals thus feel forced to take responsibility for the performance of the existing building structures designed by others before their time, when all they have been hired to do is to provide some improvement to them. As a result, the owner's desire for the most minimal upgrade often balloons into a major expensive project, with hundreds or thousands of pages of engineering analysis and justification. For every project of this kind which is constructed, hundreds of buildings remain without any improvements because of the severe cost and liability implications if they are touched at all.

4) Life Safety:

Finally, a discussion of appropriate codes, professional liability, and even topics for scientific research must include also a discussion and resolution of what level of seismic protection is necessary. Codes serve to establish a lower bound of performance, but they are not designed to provide guidance as to what should be the upper bound. Economic forces are expected to provide this, but in the field of seismic upgrading, particularly for large public projects involving government funds, confusion over how much is enough has prevented people from reaching consensus on this issue. This has been true largely because the issue of life-safety is so unclear. It is as laudable as a goal as it is vague as a benchmark. For example, while modern building codes assume structural damage may occur to a code conforming new building in the event of an earthquake, many engineers and architects are loath to define what is acceptable damage for historic masonry buildings, resulting in vast expenses for new supporting structural systems. What had been acceptable only 50 years ago, is now suddenly unacceptable. In the case of frame and infill masonry buildings, provisions are sometimes even made to resist the potential of collapse in building types which have not had a history of collapsing in earthquakes in the past.

CONCLUSION

Historic structures have something to tell us which transcends their formal architectural language. This gift from the past can be erased if the integrity of the original structure is destroyed to meet the demands of hazard mitigation. Understanding both the positive and the negative attributes of masonry construction can guide us towards methods which may be less destructive of original fabric. Some of these methods may even be more effective over the long term, not only because they build on strengths which already exist, but also because they are more closely derived from local social, and economic conditions. The purpose of historic preservation is not limited to the static freezing of artifacts. It also has to do with preserving continuity within the slow evolution of building traditions - a continuity which may in the end provide the most effective and lasting defense against earthquakes.

Regardless of whether a masonry building is modeled by an engineer as a "rigid block on soil springs," or as a "non-ductile, rigid mass on a fixed base," in truth it has life. It moves, it changes color, it ages, and it responds to our own images and dreams of what buildings should be. By "moves", this is not intended to mean falling down in an earthquake, but rather the slow and subtle movement over time – by the heat of the day – by the gradual settlement of the foundations – or by the slow erosion of the mortar bed or of the bricks or stones themselves. This almost organic quality is essential to the aesthetic quality of historic masonry. If we could arrest the effects of time, traditional masonry might lose its magic. Even in earthquake country, it is this essential quality of building which must be preserved.

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[1]Donald Appleyard, *The Environment as a Social Symbol: Towards a Theory of Environmental Action and Perception*, Berkeley, Institute of Urban and Regional Development, U. C., Berkeley, (unpublished paper), 1978, p.2.

[2]John Ruskin, *The Seven Lamps of Architecture*, p.177.

[3]David Seamon and Robert Mugerauer, *Dwelling, Place and Environment: Towards a Phenomenology of Person and World,* Martin Nijhoff, Dordrecht, 1985, p.33.

[4]Harley McKee, Masonry, National Trust/Columbia Univ. Series, Washington DC, 1980, p61. (tense changed for clarity)

[5]John Kariotis, interview, 3/6/89, & Kariotis, J*et al*, ABK Methodology for the Mitigation of Seismic Hazards in URM (unreinforced masonry) Buildings, ABK, A Joint Venture, National Science Foundation Topical Report 08, January, 1984, p 2-4.

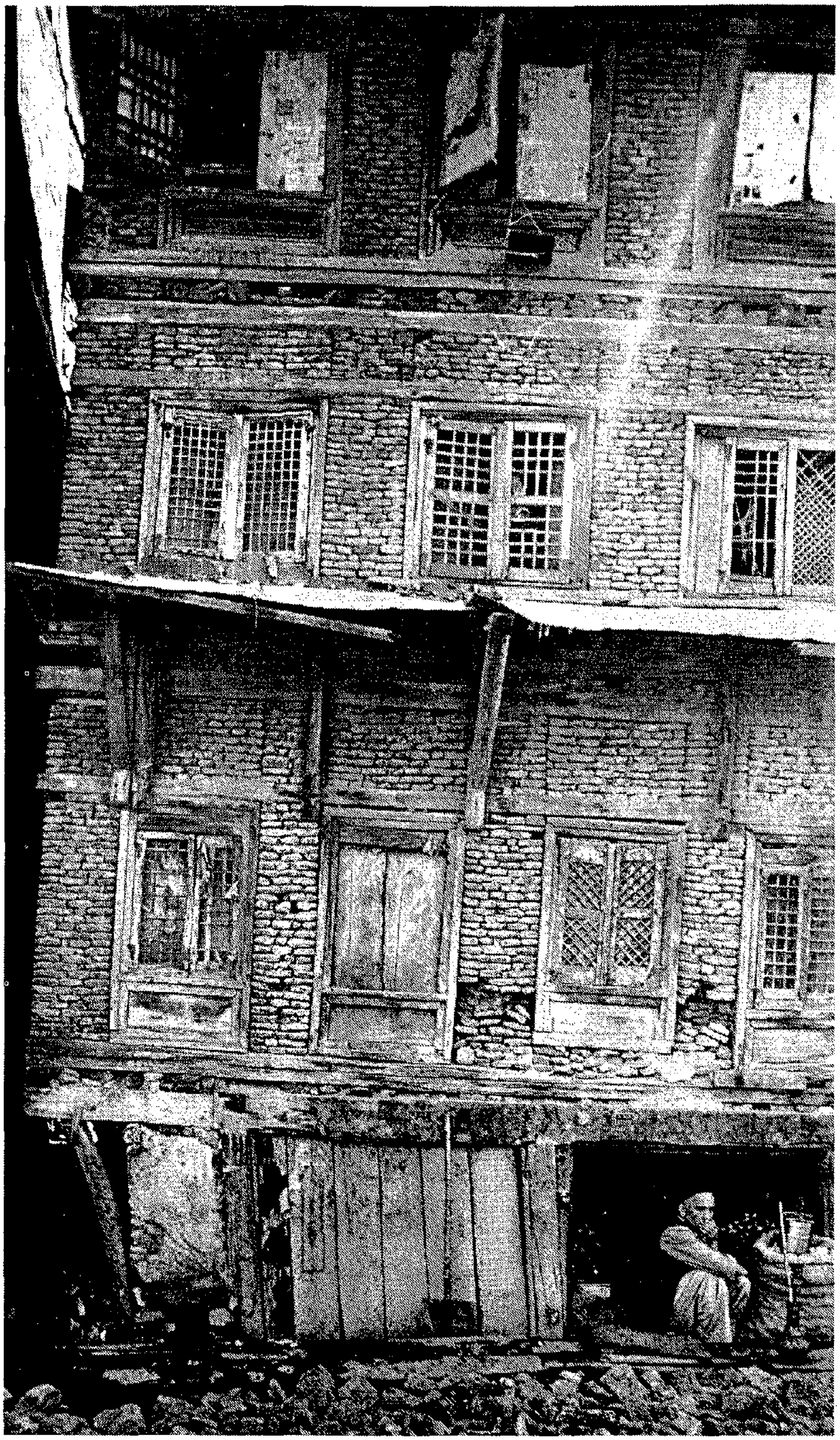

FIGURE 2: A BUILDING IN SRINAGAR, KASMIR WHICH HAS SURVIVED MANY EARTHQUAKES BECAUSE OF ITS TIMBER RUNNER BEAMS. The ends of the floor joists can be seen penetrating the wall between the horizontal beams. Tere are no vertical supports in the masonry wall.

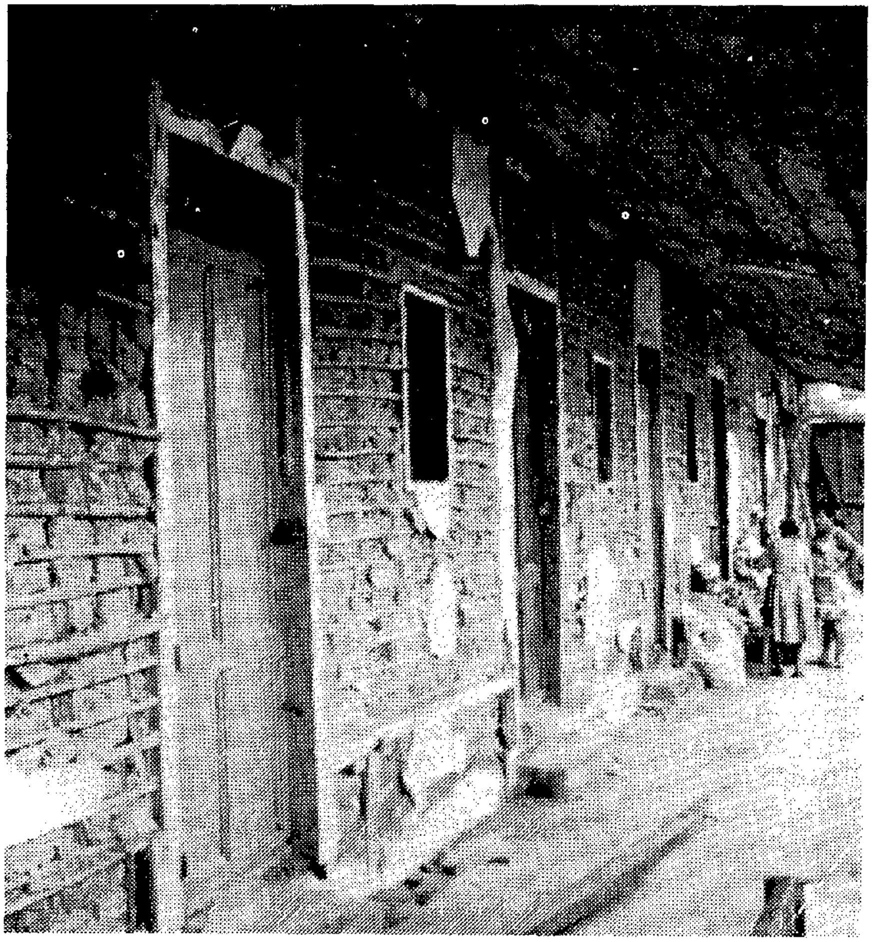

FIGURE 3: BAHAREQUE CONSTRUCTION IN EL SALVADOR AFTER THE 1986 EARTHQUAKE. The racking from the earthquake caused all of the stucco to fall off, but left the building standing.

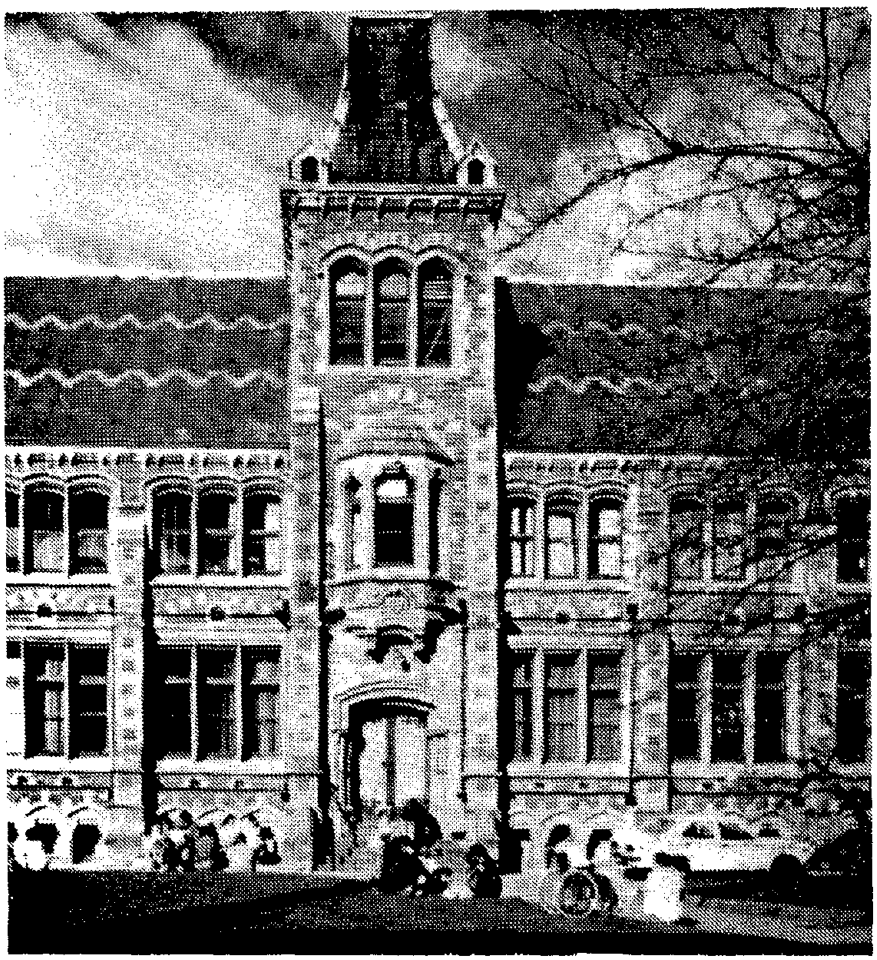

FIGURE 4: A UNIVERSITY BUILDING IN CHRISTCHURCH, NEW ZEALAND. Steel tension cables have been wrapped around the structure to reinforce it.

ACTUALITY AND MODELING OF HISTORICAL MASONRY

Antonino Giuffrè(1), Caterina Carocci(2), Gianmarco de Felice(2), Cesare Tocci(2)

ABSTRACT

The following presentation deals with the masonry hauses in historical towns, regarding them with the aim to understand their constructive actuality and to point out the most suitable way of mechanical modeling.

The research is not at its conclusion and only the main steps of its development can be illustrated toghether with some partial results, nevertheless both the approach to the constructive analysis and the modeling proposal are worth, in our opinion, to be presented and discussed. The first of them, as it will be illustrated, shows the peculiar feature of the masonry work present in historical towns in Italy and in large part of western area. From such peculiarity the second item derives: a consequent way to model, from a mechanical point of view, such masonry walls.

STATISTICAL ANALYSIS AND LIST OF THE MOST FREQUENT TYPES OF HISTORICAL MASONRY STRUCTURES IN THE ITALIAN SEISMIC AREAS

This item of the resarch is the duty of Caterina Carocci. It is going on on the basis of the seismic zoning of the territory and a direct inspection of the buildings in historical towns. Of course so extensive investigation requires a precise methodology in order to be carried out without ambiguity and in reasonable times; it is as much difficult and onerous as the mechanical informations to be accounted for the schedule of the structural types are detailed.

The first step only account for number of stories, which is indicative of the mean stress at the base of the walls; a second, non parametric, information regards the urban texture: the assembling of the buildings in relation to streets and squares, which is indicative of the quantities of external walls. But the presence of voults in the structure of the houses, the quality of the masonry and the interaxes between the walls, the structure of the roofs, the stone frame of the openings, and so on, are other important items of the schedule.

How to implement the definition of a limited number of types, representative of the mechanically relevant reality of the urban fabrics is not yet clearly stated. In the present stage of the research we go on compearing the structural features of the buildings examined in

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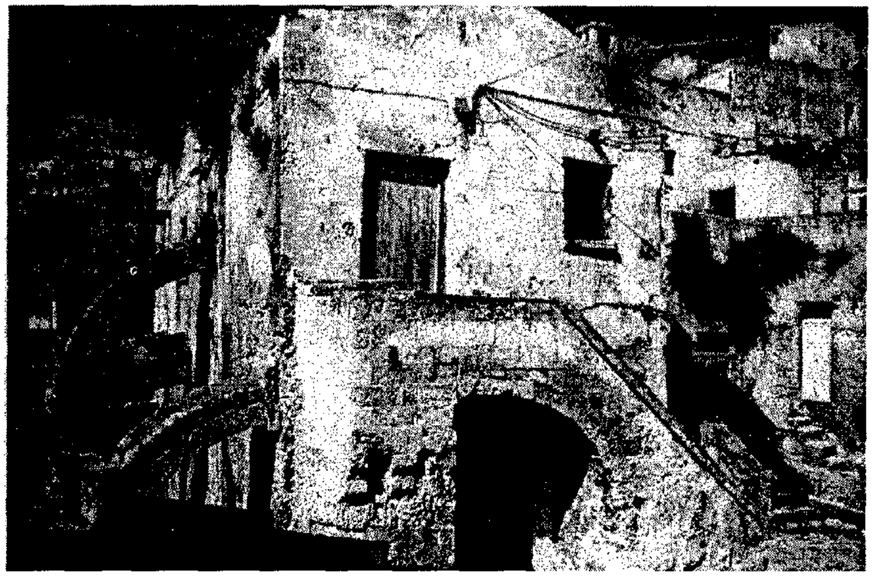
several towns. It is evident how the hauses of different towns can be similar and how dissimilar.
The following plates show some exemples.



1

2

1-2 Houses in Matera (South Italy) are built with two vaults laid upon, and the same are houses in Santorini (a greec iland), But the first ones have walls and voult made with squared stones and a weak mortar while the second have been built with raw stones and an extremely strong mortar.

3

3 houses in Barbarano (Central Italy) are completely different from houses in Ortigia (Sicily), but the number of stories is the same.

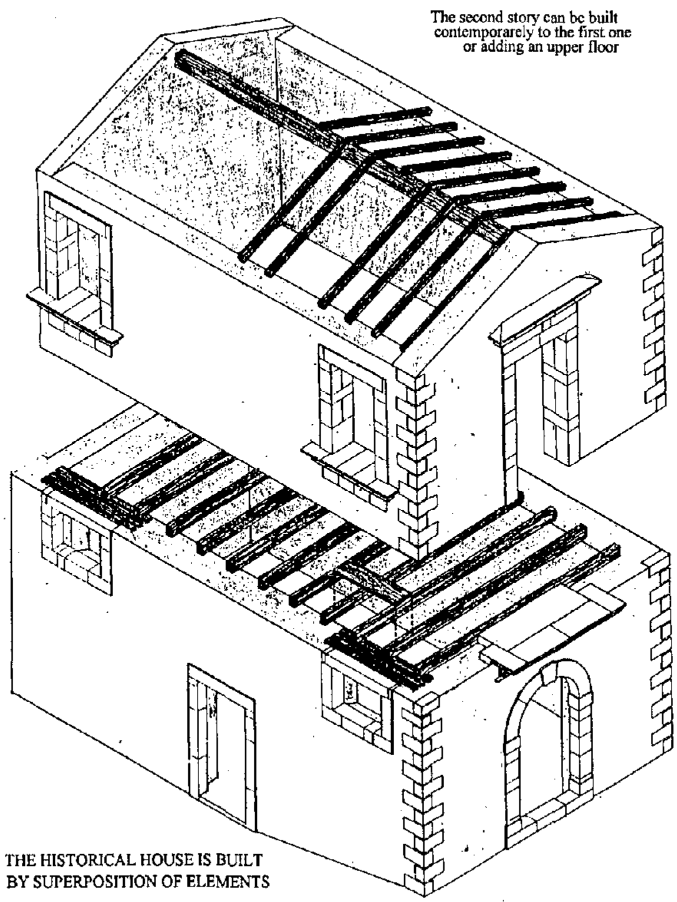
4 a

ORTIGIA

CITTA' DI CASTELLO

4 b

4 It can be useful to observe the plan of different towns: a district in Città di Castello (Central Italy) and one in Ortigia. Both the towns have been built putting near buildings one after the other, in agreement with local rules, but the different aggregation mechanisms at the base of the urban fabric produce different conditions in the reciprocal support of the buildings.



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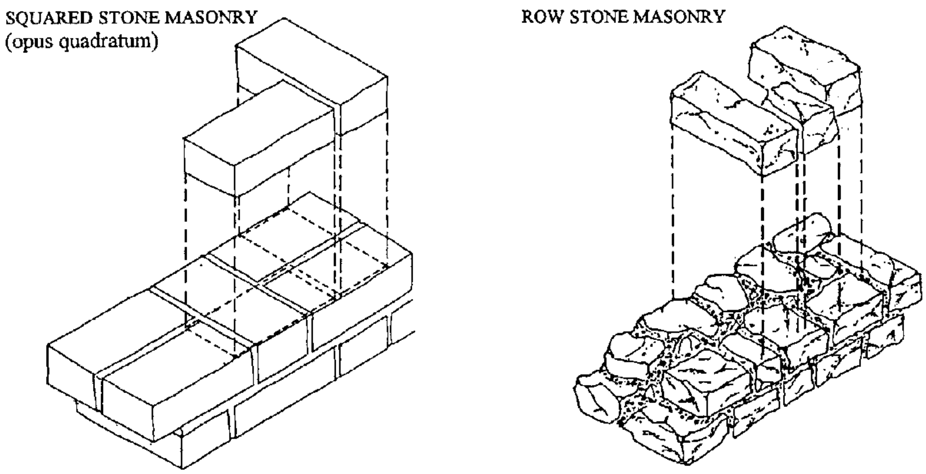
Such process of aggregation, which produces the historical town, can be observed in the single building:

5 the whole building is made by laying every element on the previous one: the beems of the floor are placed on the walls, and even the upper stories are placed on the lower ones without other continuity that the unilateral support. The handbooks of XIX century illustrate with numerous details such technics of superimposition.



The masonry wall is made of superimposed stones: their interlocking conditions the possibility of an "*monolitic*"behaviour.

6

7

8

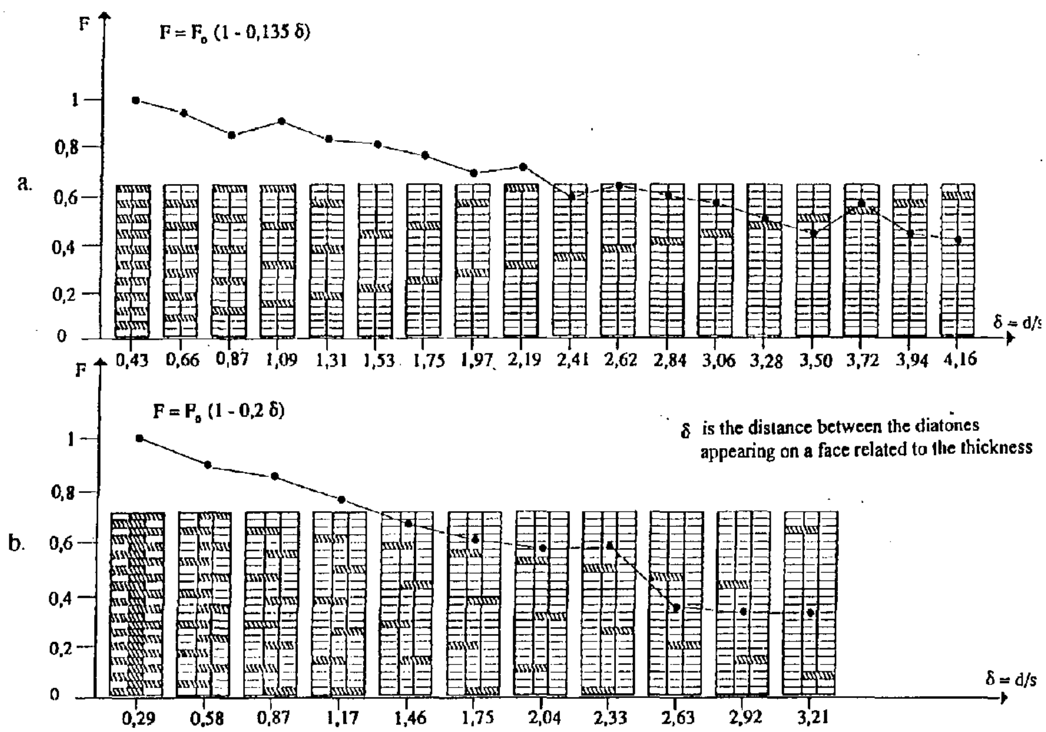
6 As the house is built laying down piece upon piece so the wall is made of superimposed stones; that is evident in walls constructed with roughly squared stones (masonry wall in Viterbo) or in raw stone masonry (Ortigia); even in Santorini, where the strong mortar achieve a significant cohesion, the dimensions of the stones suggest their setting one after the other.

It is of extreme importance to notice how the way to arrange the stone conditions the mechanical behaviour of the wall.

7-8 The first consequence of the arrangement of the stones through the wall is their resistence against the turn over. Analyses of masonry walls in different place allowed to represent some tipical transversal section, pointing out dimension and arrangement of the stones. Two opposite cases are represented by a sectioned wall in Città di Castello and an other in Castelvetere. (It should be interesting to point out how the evident better quality of the first masonry in comparison with the second, is correlate to the better constructive culture in the important town Città di Castello as regards to the peasant culture in the little village of Castelvetere).

Typical cross sections of walls observed in different towns have been collected and compared.

9 An experimental research allowed to evaluate the dacay of resistence to overturning actions as function of the quantity of *diatones*, transversal stones.

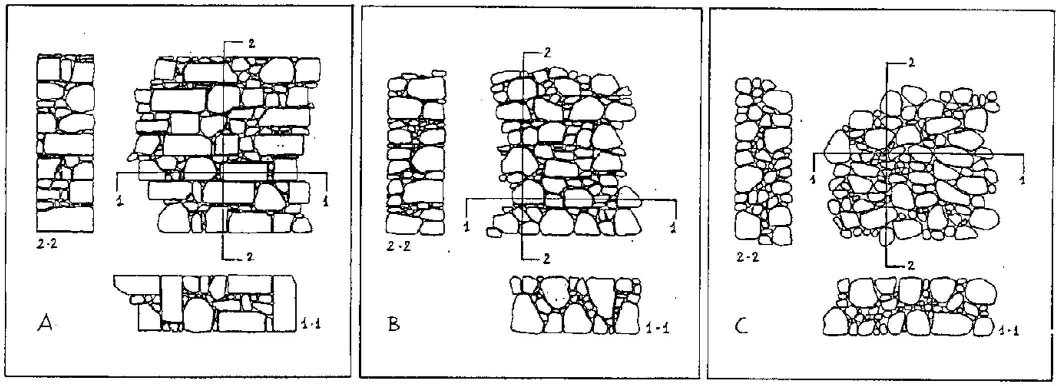


OVERTURNING STRENGTH RELATED TO NUMBER OF DIATONES

9

10 The "in plane" behaviour is affected too by the dimension of the stones: in Ortigia we found three types of masonry walls different for the dimensions of the stones, and we know that they depends from the economic possibility of the builder (little stones were less expensive then big ones).

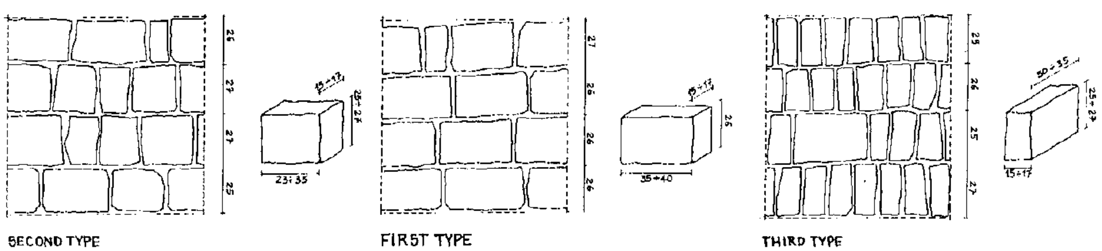
Masonry walls in Siracusa: front, horizontal and vertical sections.



The relative percentage of big and small stones conditions the "*in plane*" behaviour

10

11 In Viterbo the usage of different shaped stones is characteristic of different historical ages, as archeologists affirm.
How the different sites of the stones conditions the "in plane" resistence will be illustrated a bit later, but, if the importance of the dimensions and the arrangement of the stones is agreed it must be accepted that the first investigation on a masonry wall regards its composition.

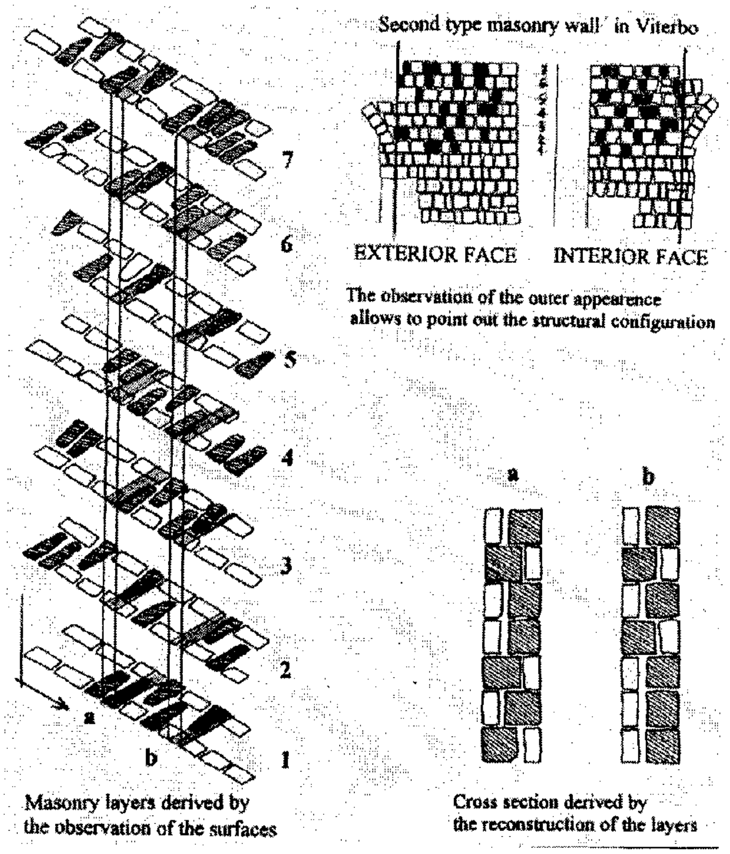


MASONRY WALLS IN VITERBO 11

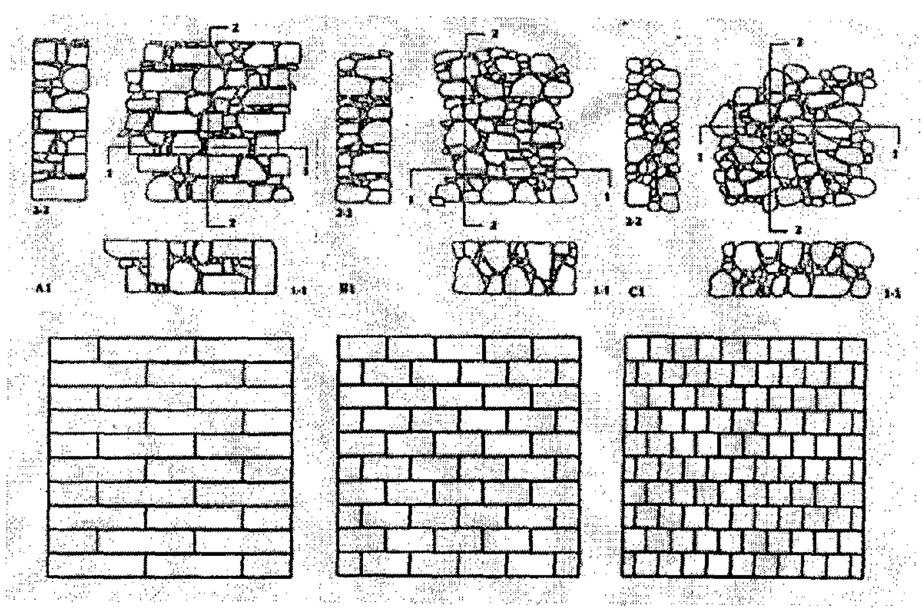
12 As an example it is shown how a masonry wall in Viterbo has been surveyed: the two faces have been examined pointing out the stones placed across the section, and the whole structure has been recognized.
Only after such analysis the mechanical quality of the wall can be examined.
13 The "in plane" behaviour should be examined accounting for the relative dimensions of the stones. The three quality of walls pointed out in Ortigia suggest the comparison between as many simplified models different for the shape of the units.
Walls made with the ratio of base over height of briks 4.6 - 2.3 - 1.0 have been examined. Little models made with squared stone units superimposed without mortar were experimentally tested at first by Lorena Sguerri.
Different load conditions were examined:

a) self weight only;
b) distributed vertical load on the upper base of the wall, applied with 5 separate loads;
c) the same loads applied on a beam connecting the top of the wall;
d) only one load applied off center keeping the connecting beam.

Significant differences have been pointed out for different brick ratios, and that proves the importance of the size of units in the mechanical behaviour of the wall.



ANALYSIS OF THE STRUCTURAL CONFIGURATION 12

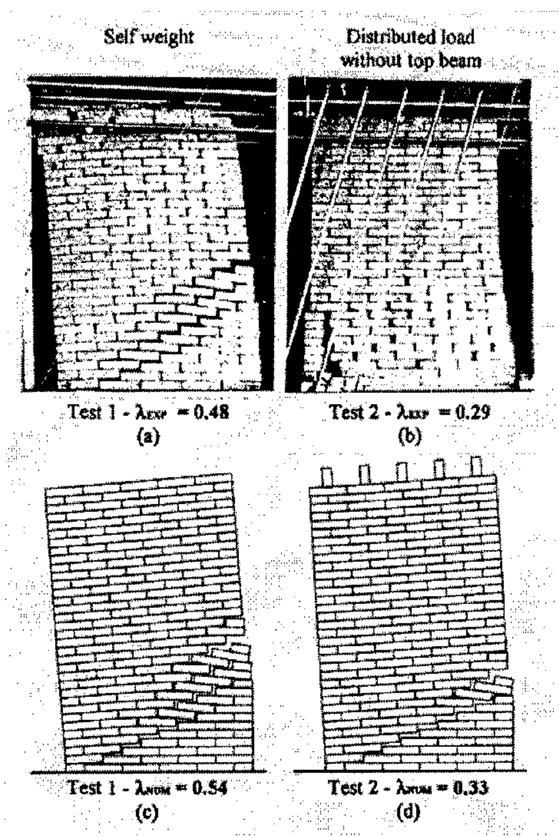


13

NUMERICAL APPROACH

Two numerical approachs have been implemented.
The first one has been applied by Cesare Tocci together with Tommaso Pagnoni. It is based on the Discrete Element Method applied tp brick walls where contact with friction is the only transfer mechanism.
Rigid elements have been selected as units, and the joints were modeled with a no tension normal spring and an elastoplastic tangenzial spring simulating a Coulomb's friction mechanism.

14 It can be seen how the numerical results are quite similar to the experimental ones. In this case (the brick ratio is 4.6) the failor of the wall follows an almost rigid overturning mechanism with or without upper load, and without connecting beam.



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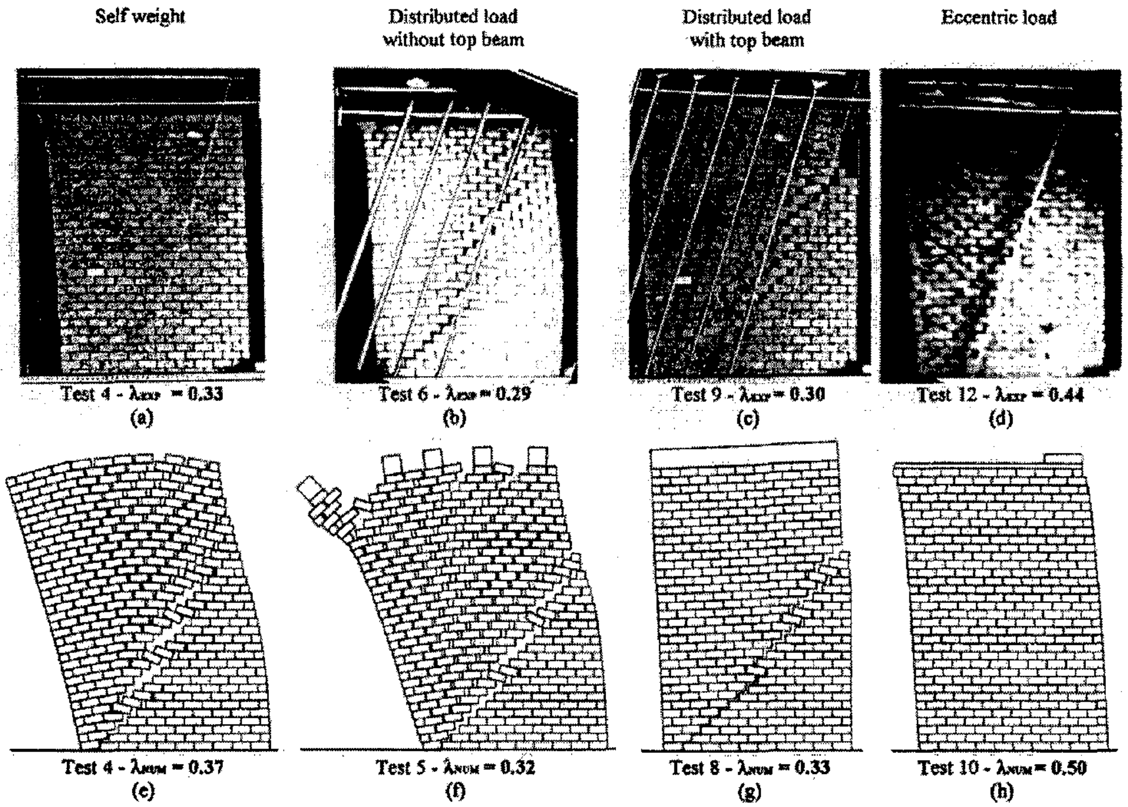
15 The brick ratio 2.3 leads to more various results: if the walls is not loaded (first case) it fails by disgregation. In this case the wall cannot achieve the rigid body failure since an "internal failure " occurs earlier.
By applying a distributed vertical load (secon case) the experimental collapse mechanism becomes closer to that of rigid body. The presence of a connecting beam at the top of the wall (third case) leads the failure mode to the rigid one, and so appen (fourth case) for a concentrate load.

16 The brick ratio 1, only numerically examined, shows how earlier the collaps is reached, due to the weak interlocking of the stones.



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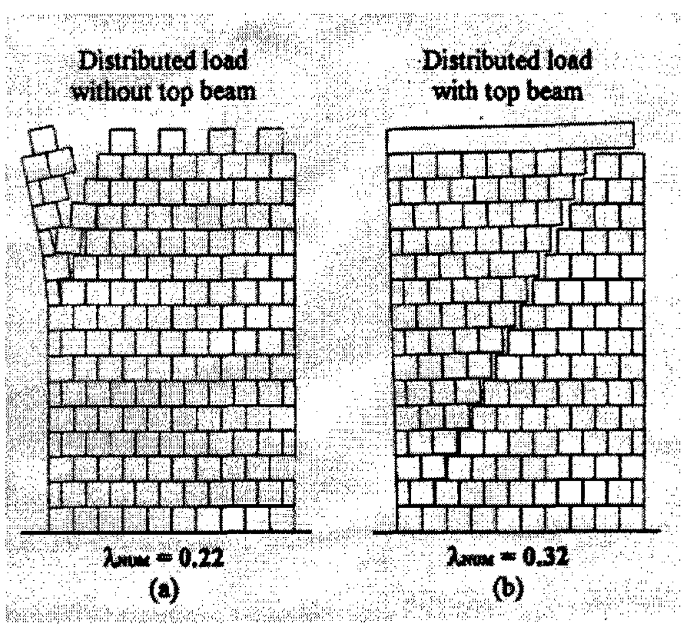
Both numerical and experimental analyses show the mechanical importance of the size and the shape of the stones, so if a continuum model has to be implemented it must account of such characteristic.

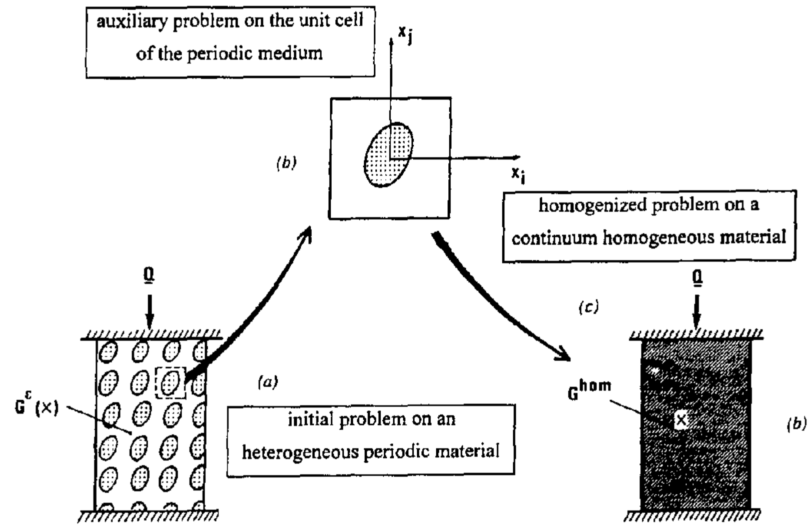


16

A proposal has been formulated by Gianmarco de Felice, following the homogenizetion method, and it is more deeply illustrated by himself in other part of this conference.

17 A lattice material made by the repetition of one elementary cell can be transformed in continuum after a carefull analysis of the cell. Such analysis explores the local failure mechanism, dependent from the shape and the arrangement of the units and from their strength too, and points out a failure domain which can be used in a Finite-Elements non linear approach. In such way the internal structure of the masonry is correctly accounted for.

OUTLINE OF HOMOGENIZATION METHOD



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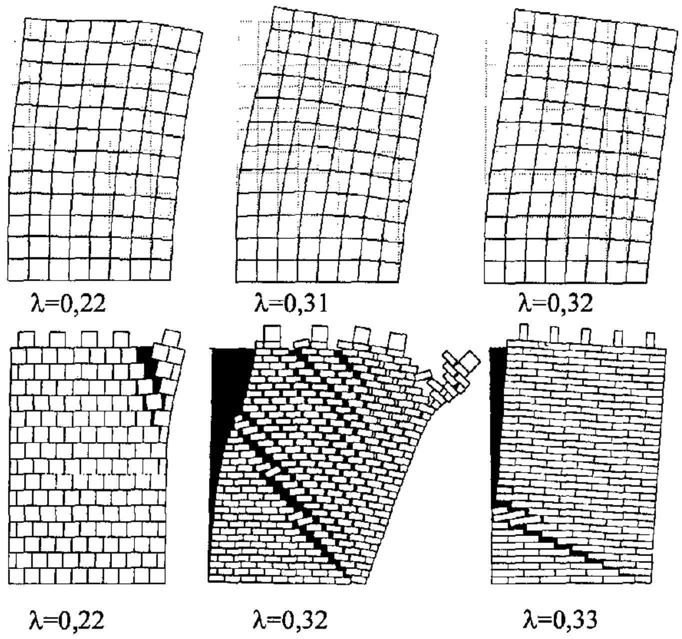
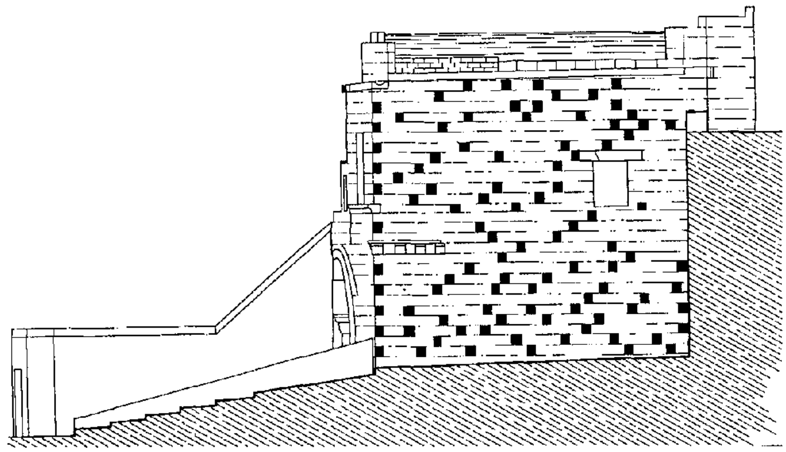
18 The same cases previously analyzed through the Discrete Elements Method have been examined with the new approach, and the results are very close to the first ones.



18

More extensive computations are going on regarding more complex structures. Such numerical approach should be improved in order to study tridimensional problems. Then the methodology for the analysis of historical buildings (limited to the walls) could be carried out as following:

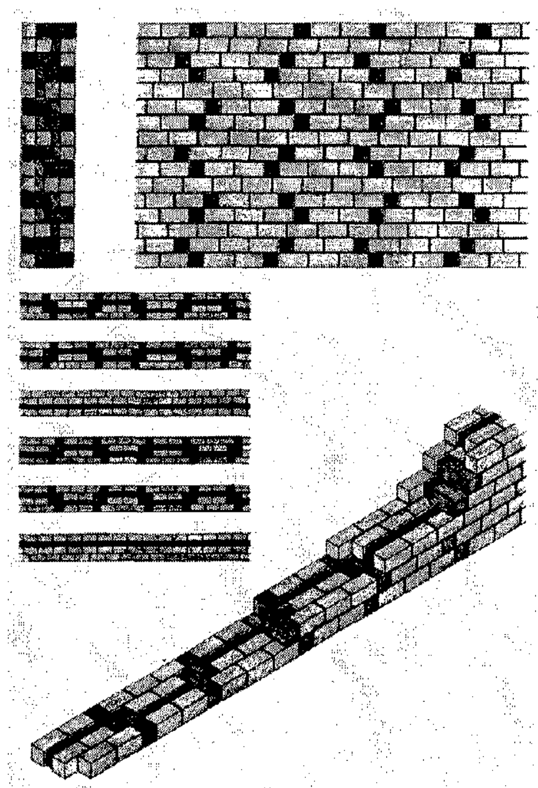
19 a) the masonry work shall be examined as said before, looking to all its stones.



19

MASONRY WALLS IN MATERA: SURVEY ANALYSIS OF THE STRUCTURAL CONFIGURATION

20 b) a model of the masonry shall be performed as a regular structure statistically derived by the observation. The idealized structure will be made by the repetition of elementary cells.

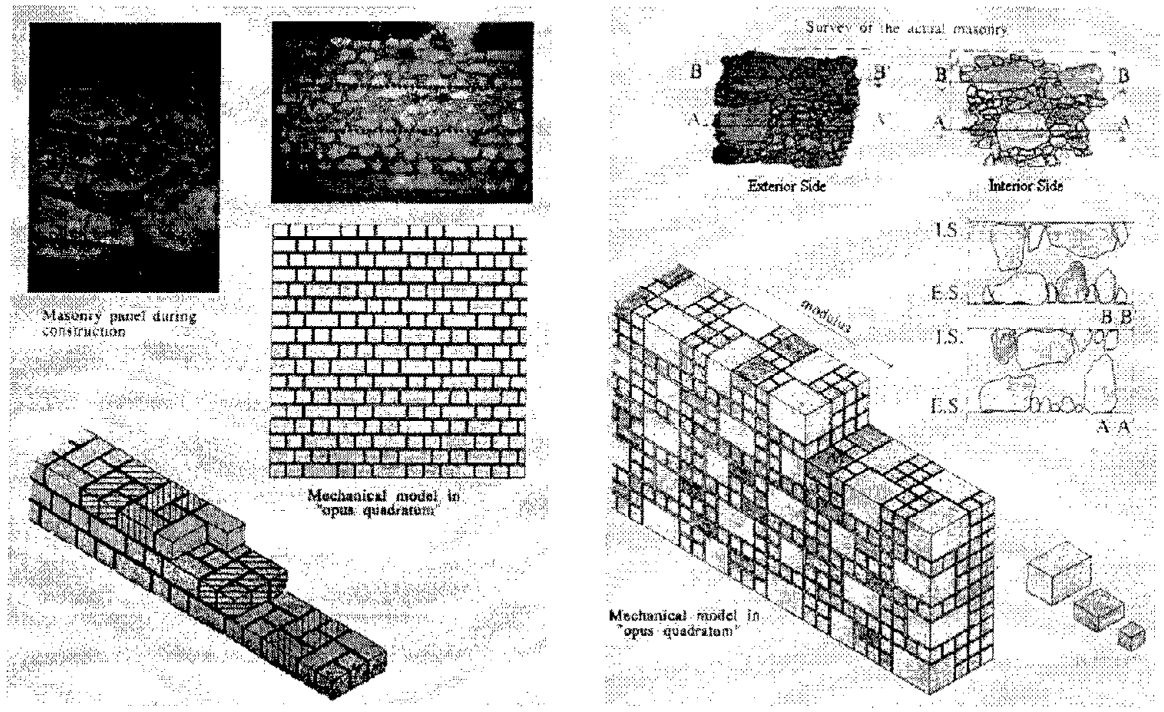


20

MASONRY WALLS IN MATERA: MODELING OF STRUCTURAL CONFIGURATION BY STATISTICAL ANALYSIS

The omogenizetion method will be able to derives by the cell the correct failure domain for a Non Linear Finite Element analysis.

21 Several cases have been examined and the elementary cells have been poined out.



MODELING ACTUAL MASONRY AS "OPUS QUADRATUM" 21

FUTURE PROGRAM

The research here briefly illustrated has the proposal to point out the actual kind of masonry buildings present in the historical towns subjected to seismic risk in Italy, and then to prepare suitable failure domains for each of the masonry types.

Such results make able the engeneer to carry out Finite Element analysis, in non linear domain, to evaluate the statical strength of the masonry wall and to verify the seismic safety of the buildings.

Section II

Development of Guidelines for Seismic Rehabilitation

AN EUROPEAN CODE FOR REHABILITATION AND STRENGTHENING

Giorgio Macchi[1]

ABSTRACT

The seismic Eurocode (EC8) includes a section called "Repair and Strengthening," which will be submitted to a final vote at the end of this year. The paper exposes the criteria of the draft code for the evaluation, the decision for structural intervention, and the redesign of buildings. Specific measures for masonry buildings are included.

STRENGTHENING AND REPAIR WITHIN THE SEISMIC EUROCODE

The Commission of the European Communities is establishing a set of harmonized codes (Eurocodes) for the design of buildings and civil engineering works, which would initially be an alternative to the different rules presently in force in the member states, and will ultimately replace them.

Several Eurocode parts have been already approved as Prestandards (ENV) with an initial life of three years; they are namely:

Eurocode 1-1	Basis of Design (a material independent code)
Eurocode 2-1	Concrete Structures
Eurocode 3-1	Steel Structures
Eurocode 4-1	Composite Structures
Eurocode 5-1	Timber Structures
Eurocode 6-1	Masonry Structures

Eurocode 8-1, Design Provisions for Earthquake Resistance of Structures is a further material independent code, but includes additional "specific rules" for buildings of different materials (Part 1.3). In this frame, additional rules are included for masonry structures (Part 1.3.6).

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The code "Repair and Strengthening" is Part 1.4 of the Seismic Eurocode 8-1. It could appear more appropriate to produce clauses to be applied in a general case, and then add special clauses for a seismic situation; however, redesign after earthquake damages and retrofitting against expected earthquakes are essential parts of the entire process for seismic protection, so that such clauses were felt to be immediately needed in seismic areas, and the work began in this field, in spite of the objective difficulties of the topics.

Therefore, such a part of EC8 is a quite general framework dealing with assessment, decision making and redesign, which as such may also be applied in different situations. The code has been drafted by a project team of CEN TC 250/SC8[2] . Informative annexes are included dealing with different materials which provide concise descriptions of intervention techniques.

INFORMATION

A minimum amount of information is needed for the assessment of the existing structure; the examination of the existing files (if any) shall be supplemented by field investigations and by tests. Such data are needed to identify the structure and allocate it in one of the ductility classes and in one of the regularity classes envisaged by the code. The identification of the subsoil class is needed in order to define the response spectrum to be used.

A preliminary survey and tests shall be sufficient to define the geometrical dimensions and evaluate the strength of the materials so that the strength verifications may be performed. Site investigations should discover possible gross errors of the structural concept and deficiencies of detailing or materials.

EVALUATION

The existing structure shall be evaluated in its actual state taking into account all of the information received; however, the methods of assessment are somewhat different from the method of design of new structures.

Modified Seismic Action

The peak ground acceleration prescribed in the specific seismic zone may be reduced for some existing buildings, if a reduced life of the building may be envisaged; the reduction may be envisaged also in consideration of socio-economic optimizations, when,, e.g. in the case of retrofitting entire urban areas, the cost of a complete adequacy is too high. It may apply also to historical buildings, in order to avoid unacceptable alterations.

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[2] Members of the Project Team: T.P. Tassios (Convenor), E.C. Carvalho, M.Fardis, C. Gavarini, A. Giuffre', and G. Macchi

Modified Material Strengths and Safety Factors

Original data on the strength of the materials are very seldom available. "Characteristic values" of the strength, based on the concept of a 5% fractile, shall be evaluated, on the basis of field tests, taking into account the additional variability inherent to the experimental method (destructive or non destructive) which is used.

For old masonry buildings, nominal values of strength may be issued by the competent authorities, and possibly be better defined by test results.

In damaged regions of the structure, the partial safety factors for material strength shall be increased, in order to take account of inadequacy of the resisting models, which are normally conceived for undamaged regions.

Structural Data

When assessing an existing structure before intervention, the residual stiffness of the structural elements shall be evaluated, in order to get the best estimation of the natural vibration period, taking account of the previous damages. An evaluation of the regularity level and of the ductility class is also needed; they shall take into account the previous damages. The efficiency of the floors as stiff diaphragms shall be evaluated.

Analysis before the Intervention

For the structural analysis, the rule is the "reference method", i.e. a linear elastic model of the structure and the modal response spectrum analysis. For regular masonry buildings of usual height it may be used in its "simplified" formulation, where only the fundamental mode is considered. A model uncertainty factor shall be introduced.

Only for some reinforced concrete or steel structures a sophisticated time-history, nonlinear analysis may be envisaged, with artificial accelerograms and hysteretic modelling of the critical regions.

For plain masonry buildings, approximate static nonlinear methods are allowed. They may be based on nonlinear force-displacement relationships of the structural elements (walls). A step-by-step procedure is followed, gradually introducing decreased stiffnesses of the walls as the lateral load increases. For the laterally loaded walls an elasto-plastic behaviour may be assumed.

Verification

Both "computational verification" and "vulnerability evaluation" methods are allowed. The conventional "computational verification" shall check that

$$\gamma_{Sd} S_{new,d} \ngtr \frac{1}{\gamma_{Rd}} R_{new,d}$$

It means that the design value of the action effect, S, shall not be more than the corresponding design resistance, R, of the structural element; the verification is therefore based on the verification of the individual cross sections. However, the action is increased (by the model uncertainty factor) in comparison with the case of new structures; similarly, the resistance is decreased (by application of the additional uncertainty factor of the resisting model) in comparison with the case of new structures.

In special cases, global "vulnerability evaluation methods" may be applied instead of the above described computational verification. Such procedures, based on a semi-empirical classification of the structural features of the buildings, have limited fields of application stated by the national authorities. They may be particularly useful in the assessment of entire urban areas and consequent cost estimation of their retrofitting; the methods are specially suggested for old masonry buildings.

DECISIONS FOR STRUCTURAL INTERVENTION

Criteria and Type of Intervention

Depending on the results of the verification, the decision can range from "no intervention at all" to "total demolition". The extent of the strengthening intervention and the type of works depend on the results of a decision procedure seeking to optimize social interests.

Criteria are given in the code for the choice of the intervention, which may basically be found among the following ones:

1. Restriction or change of building use.
2. Mass reduction.
3. Modification of the structural system, by repair or strengthening of damaged or undamaged elements, by transformation of non-structural elements into structural, by modification into a more regular or more ductile arrangement, by a beneficial change of natural period.
4. Addition of new structural elements, of a new structural system to take the seismic action, of damping or isolation devices, etc.

It is reminded that the ductility may be effected by the intervention, and that a mere increase in strength may result in a decrease in ductility.

Priority of Interventions

When the "vulnerability" indexation mentioned above is used, it directly gives a priority scale, to which only the consideration of the importance of the building should be added.

When, on the contrary, the "computational verification" is used, it is suggested to combine a "local seismic resistance" of each critical region into a "global seismic resistance" of the structure in the following way.

First, a local seismic resistance index is computed for each critical section:

$$L_{Ri} = \frac{R_{di} - S_{di(N.S.A.)}}{S_{Ei}}$$

where R is the design resistance of the section, S (N.S.A.) is the action effect of all non-seismic actions, and S is the action effect of the design seismic action.

Then, the global seismic resistance index of the entire structure is:

$$G_R = \frac{\sum_i W(L_R)^{\lambda}}{\sum_i W(\frac{1}{L_R})^{\lambda}}$$

where L are the local indices limited between 0 and 1.3, W are weighting factors accounting for the significance of the local effects and λ is equal to 0.5.

Finally, the priority index is:

$$I_p = (1 - G_R)I_o$$

where I_o is a reference priority index taking into account the importance of the building (from 1 to 2.5).

REDESIGN

Redesign Verification

As mentioned before, the analysis and the verification of the strengthened structure shall concern the ultimate limit state and are applied to:

1. The connections of the additional parts to the existing parts.
2. The critical sections of the entire structure.

The strength of the materials shall be reduced by additional safety factors allowing for the possible alterations in the reconstruction operations, and for model uncertainty. Stiffness, ductility (and consequent behaviour factors) shall be evaluated for the strengthened structure. The resistance of the new materials and of the connections shall be evaluated.

Basic Data for Force Transfer

The structural characteristics of the repaired or strengthened building shall take account of the force transfer mechanisms along the interfaces between existing to additional material. Special consideration is given to the following mechanisms:

1. compression against precracked interfaces
2. adhesion between non-metallic materials
3. friction between non-metallic materials
4. load transfer through resin layers
5. clamping effect of steel across interfaces
6. dowel action
7. anchoring of reinforcement
8. welding of steel elements
9. connection of timber-to-timber elements.

Local and Global Ductility: Behaviour Factor

Repaired or strengthened regions should be provided with the appropriate ductility in relation to the selected ductility class. Possible local brittleness due to the intervention require an appropriate reduction of the behaviour factor used in the re-analysis.

Structural regularity shall also be assumed, and differences in overstrength or interstory drift of consecutive floors should be limited.

Analysis for Redesign

The re-analysis shall take into account the fact that the intervention is applied to an already stressed structure. Effects of temporary shoring and possible redistributions of action-effects should be considered.

Stiffnesses and resistances of the structure after intervention shall be estimated in one of the following ways.

1. By an analytical estimation based on conservative constitutive laws of force-transfer mechanisms along the interfaces, taking account of cyclic degradation.
2. By a simplified estimation, in which the initial structural characteristic of a critical region is reduced by appropriate empirical "model correction factors" taking account of the behavior at the interfaces.

QUALITY ASSURANCE OF INTERVENTIONS

Design, execution, use and maintenance shall be subject to a complete quality assurance plan, developed in function of the type of intervention.

FURTHER PARTS AND ANNEXES OF THE CODE

The material independent part of the code is followed by:

Section II-
- Concrete Structures
- Steel Structures
- Masonry Structures
- Timber Structures

Annex A	Post-earthquake measures
Annex B	Information for structural assessment
Annex C	Conceptual bases for reduced pga values
Annex D	Vulnerability methods
Annex E	Quality assurance of interventions
Annex F	Particular considerations for historical buildings and monuments.

SUMMARY OF THE ATC-33 PROJECT ON GUIDELINES FOR SEISMIC REHABILITATION OF BUILDINGS

Daniel P. Abrams[1]

ABSTRACT

An overview is presented on the current state of development for a national effort to write guidelines for seismic rehabilitation of buildings. Much of the introductory material from the 25% draft of a new document under development is presented to help illustrate the approach of the overall effort. This is followed by brief summaries of some of the issues encountered by the document writers in its preparation.

INTRODUCTION

The reoccurrence of devastating earthquakes in the United States has alerted many building owners and municipalities to question the seismic performance of their buildings. Instead of simply strengthening buildings for protection of life safety, owners are now becoming concerned about the expected performance of their buildings during a future earthquake. They are now recognizing that rehabilitation costs can vary with different levels of building serviceability and anticipated amounts of damage. However, other than a compilation of suggested details (4), there are no national standards for rehabilitating existing buildings in America, let alone a consistent set of methods for evaluating a rehabilitated building to various performance limits.

In an effort to provide the tools for reducing earthquake hazards in existing buildings, the Federal Emergency Management Agency (FEMA), as part of the National Earthquake Hazards Reduction Program (NEHRP), has planned and is sponsoring development of a set of *Guidelines for Seismic Rehabilitation of Buildings* and related *Commentary* (7). The Applied Technology Council (ATC) is developing the document under contract to the Building Seismic Safety Council (BSSC). Preparation of the *Guidelines* and *Commentary* is known as the ATC-33 project.

In preparation for development of the *Guidelines* a project was done to identify and resolve general issues related to formation of a document on seismic rehabilitation. This project is known as the ATC-28 project and is summarized in (6). The subsequent and current ATC-33 project has a term of three-years, and is now nearing 50% completion. In January of 1994 a 25% draft (7) was submitted to BSSC for review. The final draft is expected in early 1996.

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[1]Professor of Civil Engineering, University of Illinois at Urbana-Champaign, USA

This paper provides an introduction to the purpose and scope of the ATC-33 project, and what is generally contained in the *Guidelines*. Because the document is still in the development stage, this paper should be regarded as a work-in-progress report towards development of a set of *Guidelines* that will change before completion. However, the information contained in this paper will be of use to communicate what state-of-the-art developments are underway in the United States on the topic of seismic rehabilitation of buildings.

OBJECTIVES AND SCOPE OF ATC-33 GUIDELINES

Objectives of Guidelines

The intended use, purpose and limitations of the *Guidelines* are explained in the first chapter of the draft (7). A few excerpts from the document are provided here to give a general idea of objectives.

> *The* Guidelines *and* Commentary *are intended to be useful to architects, engineers, building officials, building owners, government agencies, legislators and building code authorities in all parts of the United States. Most of the provisions are written presuming that the reader has the expertise of a structural engineer.*
>
> *The purpose of the document is to provide a set of technically sound, nationally applicable guidelines for the seismic rehabilitation of buildings.*
>
> *The document is neither a code nor a standard. It is a resource document that is intended to be suitable for adaptation and adoption into model codes and standards. The purpose of this document for existing buildings is thus parallel with that of guidelines for the seismic design of new buildings (1). The document is not intended to provide a ready-to-adopt complete set of building code-type regulations.*
>
> *Unlike the provisions for new buildings, this document contains options for selecting among ground motion criteria and performance objectives to achieve the desired intent of the user. The* Guidelines *may be used as a reference document by an engineer in assisting a building owner in the selection of seismic protection criteria and the conception of a seismic rehabilitation design in cases where the owner's hazard reduction efforts are purely voluntary.*

Scope of Guidelines

The scope of the *Guidelines* is clearly identified with the following text taken from Chapter 1 of the first draft (7).

> *The* Guidelines *apply to the overall structural system, its lateral-force resisting elements, and the constituent components of the elements as well as selected nonstructural components. Methods of reducing seismic risk that do not physically change the building, such as reducing the number of occupants, are not within the scope of the* Guidelines. *Rehabilitation techniques include methods for reducing seismic demand, such as the introduction of isolation or damping devices, in addition to increasing strength and ductility.*

The scope includes existing buildings and excludes new buildings, except that new components added to an existing building are included within the scope.

Nothing in the Guidelines *relates to the decision to rehabilitate. The* Guidelines *provide detailed engineering guidance on how to conduct seismic rehabilitation analysis once the decision to apply these provisions has been made. The question of when the* Guidelines *are mandatory is beyond the scope of the document. Also beyond the scope of the specific engineering requirements is the determination of which performance objective should be achieved for a particular building and what level and probability of ground motion will be used in connection with that objective. Once the intended performance objective and the ground motion criteria are known, the* Guidelines *provide consistent procedures that specify how to meet a selected goal.*

The document can be applied to all buildings regardless of importance, occupancy, historic features, size or other characteristics, which for any reason are deficient in their ability to resist the effects of earthquake shaking. Excepted are farm buildings not intended for human habitation and non-building structures such as elevated tanks and billboards unless they are part of a building. Also excepted are buildings or structures covered by special codes or standards such as bridges and nuclear power plants. The repair or stabilization of earthquake-damaged buildings is also beyond the document's scope.

THE SEISMIC REHABILITATION PROCESS

The document assumes the seismic rehabilitation process begins after an engineering evaluation study has concluded that the building needs to be seismically improved and had preliminary identified the weak links or critical types of deficiencies. The *Guidelines* do not prescribe what type of analysis should be used for this evaluation, but the provisions are based on the assumption that the building has been evaluated using FEMA 178 (5), or other procedures that achieve an equivalent level of accuracy.

Another assumption is that the design professional has already developed one or more preliminary seismic rehabilitation approaches before consulting the analytical procedures in the *Guidelines*, or that the designer will iteratively produce schematic solutions, test them against the acceptance criteria of the *Guidelines* and progressively refine the design.

The overall flow of procedures contained in the document as they relate to the broader scope of the overall seismic rehabilitation process are illustrated in Fig. 1. After a decision has been made to rehabilitate a building, a hazard reduction strategy or preliminary design should be produced to meet specific performance objectives which must be defined. Then, based on desired ground motion criteria, design forces are selected for given construction types. The building system is then analyzed for its response to seismic forces, and results are compared with acceptability criteria. If the estimated response is unacceptable, then the re-construction scheme, the ground motion criteria and/or the performance objectives must be altered. The global performance analysis is done through analytical modeling of the system comprised of vertical, horizontal and foundation elements. Once a rehabilitation scheme is chosen, construction documents and quality control procedures are established.

EVALUATION vs. REHABILITATION

The *evaluation* process is one where the seismic response of an existing building is examined using some engineering assessment procedure. The *rehabilitation* process is one where portions of an existing building are modified, or new elements are added on the basis of some engineering design procedure. Costs for a seismic evaluation project are primarily for engineering time whereas costs for a seismic rehabilitation project are primarily for construction materials and labor.

The two processes have a similarity in that some analysis of the building system must be done whether it is for the existing structure or the rehabilitated one. In fact, the analysis procedure can be the same for each process. However, the implications of the analysis may be much different depending on how it is being used. If results of an evaluation study show that some strengthening must be done, then a commitment to rehabilitate must be made which implies a major financial commitment for reconstruction. If results of a rehabilitation study show that certain members may need to be increased in size, then the financial margin will be small since a commitment to rehabilitate has already been established. Thus the consequences of the same analysis method for two different processes may be much different. Because of this, analyses done for evaluation can be associated with a smaller margin of safety than if the same analyses done for a rehabilitation.

One cannot think of the evaluation process separately from the rehabilitation process. When new elements are added to an existing building as a rehabilitation measure, the entire system of new and old structural components must be analyzed together. To insure equivalence for an existing non-rehabilitated building and a building with a very minor enhancement, the same analytical methods must be used for both. Thus, much of the product of developing analysis procedures for rehabilitation guidelines will also be applicable to seismic evaluation of existing buildings. However, as stated previously, the *Guidelines* are not intended to preclude earlier criteria development on seismic evaluation procedures.

SAFETY AND OTHER PERFORMANCE OBJECTIVES

Performance limits are defined as specific levels of serviceability to which an owner, architect or building official may expect their building to respond. *Performance objectives* are reaching particular performance limits for specific levels of earthquake intensities. A performance-based design, as included in the *Guidelines*, is one where a building is rehabilitated to perform to some pre-defined limit for a given earthquake.

Performance-based design is a new concept for development of structural engineering criteria. In the past, the only codes and specifications were for design of newly constructed buildings. For these provisions, the sole criteria was that buildings remain safe under prescribed loadings. Safety factors greater than one were applied to loads while capacity reduction factors less than one were applied to strengths. Designs were also constrained by serviceability criteria that were imposed by checking deflections (determined without safety factors) with permissible values.

With a performance based design, only one performance limit is associated with safety, that being the limit of *substantial life safety*, or the assurance that lives will not be lost because of the building collapsing, or its elements or components falling to the ground or inhibiting egress. The other performance limits are based on immediate use of the building following the earthquake which is termed *immediate occupancy*, or protection against complete collapse of the system which is termed *global stability*. In addition to these discrete performance limits, there are also two ranges of performance that may be considered. The first is *damage control* where the rehabilitation design may be tuned towards a specific level of damage somewhere between that for immediate occupancy and that for substantial life safety. The second range is *risk reduction* which varies between substantial life safety and collapse. This range is intended for those building owners who cannot afford to rehabilitate their buildings up to the necessary level to insure life safety, but wish to spend what available funds they do have in the most effective way to reduce the seismic risk as much as possible. Despite the favorable connotation, the words "risk reduction" under this definition, imply that the structure does not meet code.

Limit states are specific occurrences of damage to a particular structural element. An unreinforced masonry wall, for example, has limit states of initial cracking, extensive cracking, crushing, rocking or dislodgment of masonry units. Limit states are used to define performance limits when a structural element is used in a building with a known occupancy and importance. However, the relation between what limit states are acceptable for various performance limits goes beyond structural engineering. Social and economic factors enter in as well.

For the wall example, what extent of masonry cracking might be acceptable so that a building can be immediately re-occupied following an earthquake? Even if there were no loss of strength as a result of the cracking, would occupants still refrain from using the building with cracked walls? Would their decision be different if the building were a hospital, or a parking garage? Or, would the local building department upon seeing the cracked wall, immediately place a "red tag" on the building (a warning so that no one can enter the building until a structural engineer makes an assessment of structural integrity), and thus limit the immediate occupancy of the building. Conversely, immediate occupancy may be encumbered even if the masonry wall is uncracked if non-structural components such as windows, partitions, or utilities are damaged.

Thus, it is difficult to associate the limit state of a structural element with the performance limit for a building system without taking into consideration the occupancy and importance of the building as well as the vitality of its operation to the damage of nonstructural components. To associate limit states for structural elements with performance limits for building systems can be impossible to generalize for all buildings. Despite this, the *Guidelines* writers shall make a first attempt at performance-based design by limiting the document to typical buildings with typical occupancies.

SIMPLE AND COMPLEX ANALYTICAL METHODS

The *Guidelines* suggest the use of various analytical methods, ranging from static equivalent lateral force methods to dynamic response spectrum and time-step integration methods for estimating anticipated seismic response of rehabilitated buildings. Methods similar to those used for new construction are specified with modifications to model a mixed system of existing and new elements constructed of various materials. In addition, there are also new methods prescribed in the *Guidelines* for estimating nonlinear dynamic response of a mixed system.

Force Methods

Equivalent Lateral Force Method

The equivalent lateral force method (ELF) proposed in the *Guidelines* is similar to that contained in codes for new construction (1). A system base shear demand is calculating using the following equation.

$$V = C_s R_d W \qquad \text{Equation 1}$$

where C_S is the seismic design coefficient obtained from an equation based on the fundamental period and the ground accelerations, R_d is a demand modification factor to account for damping and site effects for buildings with different periods and W is the weight of the building and anticipated live loads.

The ELF method is based on reducing the elastic forces by a reduction factor to account for inelastic action. The reduction factor R_μ is introduced when checking the acceptability of a particular component (see discussion with Equation 2) rather than with the base shear equation so that systems with several different types of components can be analyzed.

Response Spectrum Analysis

A linear elastic dynamic analysis of the rehabilitated building may be done using common analysis procedures. Peak modal responses are determined from calculated modal periods and spectral response curves. Maximum modal contributions are combined statistically to estimate total structural response. If the value of base shear from the response spectrum analysis is less than the unreduced base shear given by Equation 1, then element forces and displacements are factored by the ratio of the equivalent base shear to the response spectrum base shear.

Determination of Component Acceptability

The *Guidelines* distinguish between structural components that are likely to undergo large inelastic displacements (frames and walls) and those that are sensitive to forces (such as connections, columns subjected to large axial forces or beams subjected to large shear force). For the case of displacement controlled elements Equation 2 is used to determine component acceptability.

$$Q = \frac{Q_D + Q_L + Q_S \pm Q_E}{R_S R_\mu} \le C \qquad \text{Equation 2}$$

Where Q_D Q_L and Q_S are the gravity dead, live and snow loads, and Q_E is the unreduced earthquake demand force. R_s is the structural quality coefficient for the entire building and is a weighted average based on relative shear forces for each component of the lateral-force resisting system. R_s varies between 0.5 for soft-story systems to 1.5 for moment resisting frames. R_μ is a demand reduction factor to account for the ductility in a component. It is based on the inelastic deformation capacity of an element and may range from 1 for columns with large axial force to 7 for a steel frame. C is the capacity of the component

For the case of force-controlled elements Equation 3 is used to determine component acceptability.

$$Q = Q_D + Q_L + Q_S \pm \frac{Q_E}{R_f} \leq C \qquad \text{Equation 3}$$

Where R_f is a demand reduction factor to account for the probable maximum strength demand on a component. It ranges from 1.5 to 3 for various force sensitive components.

In addition, story drifts must be less than permissible values.

Simplified Nonlinear Analysis Method

The *push-over analysis* is a piece-wise linear method that represents the nonlinear static behavior of a structural system. Lateral forces are increased to the system as individual members deform past their proportional limits. A nonlinear relation between base shear and top-level deflection is obtained from a series of incremental linear analyses.

The global force-deflection curve of system capacity is then superimposed on a spectral response curve for a given damping ratio. Instead of the common horizontal period axis, the spectral curve is plotted versus an equivalent spectral displacement axis that can be compared directly with the system capacity curve. The intersection of the capacity and spectrum curves is then defined as the expected nonlinear displacements for the system which must then be compared with acceptability criteria for particular limit states. The name for this approach is thus the *capacity-spectrum method.*

The capacity spectrum technique is useful for estimating peak response of a rehabilitated building because the inelastic behavior of components constructed of different materials can be modeled. Furthermore, the method solves for the peak lateral deflections of a system which then can be compared with limit states for specific performance limits.

Nonlinear Response History Computation

If a user wished to go further with the analysis, the *Guidelines* permit the use of nonlinear response history computations. However, there are no recommendations given as to what hysteresis rules, or sample earthquake ground motions to use. Results of the response history computations must show however an equivalent level of acceptability with the other more simple methods of analysis.

THE USE OF CAPACITY REDUCTION FACTORS

Capacity reduction factors contained in standards for new construction are used to provide safe estimates of structural strengths. Lower bound estimates of strength are prescribed based on the certainty of obtaining expected material and component strengths. For design of new construction, knowledge of material or component strengths can only be anticipated since the building is in the conceptual stage at the time of engineering design. Alternately, for evaluation and rehabilitation of existing buildings, material and component strengths can be measured insitu or in the laboratory, and information is available on the as-built construction. Separation of loads and resistances using capacity reduction factors, as is done for codes for new construction, is not applicable for existing buildings if the strength of the existing structure can be measured rather than anticipated.

Capacity reduction factors can be applied to nominal strengths in a rationale manner if load effects are to be compared directly with strengths as is done with the LRFD method or the strength design method of ACI 318. If the analysis method uses a surrogate method of representing displacements with equivalent lateral forces (as is done with the ELF procedure), then lower-bound values of capacity should be replaced with nominal expected values of capacity. To estimate yield in a reinforced wall for example, the analysis should attempt to model the most accurate depiction of the yield force level rather than a lower bound value which may not always be conservative. The level of conservatism should be applied to the acceptability criteria for a particular performance limit rather than to the analytically derived deflection.

Former building codes for new construction have had but one performance objective, that being life safety. The overall factor of safety against collapse has been often referred to as the ratio of the load factors to the capacity reduction factors. When the object of the analysis is to assess the performance of an existing structural system at various performance limits other than life safety, the use of capacity reduction factors is inappropriate since safety is not the concern.

Capacity reduction factors are proposed when the life safety performance of a rehabilitated structure is to be checked. Design of additional new components to an existing building should be consistent with that done for newly constructed buildings. However, when immediate occupancy, risk reduction or damage control is to be considered for a particular structure, best estimates of capacity are recommended with no capacity reduction factors. In such case, capacity reduction factors expressed in codes for new construction should be neglected.

The level of knowledge regarding a particular structural system should be represented by applying a knowledge-based factor, κ, to the acceptable displacements for a given performance objective. If as-built drawings are available, the level of quality control during construction is known, and a thorough condition assessment is made, then κ should be equal to 1.0. To the contrary, if there is no knowledge of as-built condition or quality control procedures and no condition assessment is made, then κ should be equal to 0.4.

ELEMENT ATTRIBUTES: STEEL, CONCRETE, TIMBER AND MASONRY

The *Guidelines* include separate chapters on each material typically used in existing buildings: steel, concrete, timber and masonry. Considerations are given in each chapter regarding structural attributes of vertical and horizontal lateral-force resisting elements of each material. Existing elements, enhanced elements and new elements are considered in each chapter. Reference is made to what codes for new construction may be applicable, with modifications, for design of new elements added to an existing building. Systems comprised of elements of various materials can be analyzed by referring to each of the various material chapters. Thus, if a mixed system of existing unreinforced masonry walls and new steel frames are to be considered, then the user would refer to separate masonry and steel chapters to determine the attributes for each material to use in the general analyses procedures.

Stiffness Assumptions

The *Guidelines* and the *Commentary* provide information on modeling the elastic and inelastic stiffnesses of existing, enhanced and new structural elements. Although previous codes for new construction have avoided specific recommendations for stiffness assumptions (e.g. the ACI 318 code for new concrete construction states "any reasonable assumption made be made to model the stiffness of a reinforced concrete member") the *Guidelines* will include such directives. This is again because the document includes performance-based design encompassing several performance objectives.

As an example of stiffness assumptions for an unreinforced masonry shear wall, the following text will appear in the 50% draft of the *Guidelines*.

> *Stiffness of masonry walls shall be determined based on the minimum net sections of mortared and grouted masonry in accordance with the guidelines of this subsection.*
>
> *The stiffness of an unreinforced masonry wall or pier resisting lateral forces within its plane shall be estimated based on linear elastic behavior if it is determined that the wall shall not crack when subjected to lateral forces. The masonry assemblage of units, mortar and grout shall be considered to be a homogeneous medium for stiffness computations.*
>
> *Uncracked sections based on the net mortared/grouted area shall be considered for determination of geometrical properties provided that net flexural tensile stress does not exceed tensile strengths. Cracked sections shall be considered for walls or piers when net flexural tensile stress exceeds tensile strength provided that vertical stress of at least 25 psi is present. The length of flexural cracks shall be determined based on the location of the resultant vertical compressive force, and used to determine reduced section properties.*
>
> *The stiffness of solid masonry walls subjected to lateral forces shall be determined considering both flexural and shear deformations.*
>
> *The lateral stiffness of a perforated shear wall shall be determined considering the flexibility of piers between openings. If the combination of the spandrel and the floor or roof diaphragm is sufficiently stiff in flexure, a pier may be considered to be fully restrained against rotation at its top and bottom for the determination of distributing story shears and assessing stress levels. Pier stiffness shall be based on both flexural and shear deformations.*

Strength Acceptability Criteria

Codes and standards for new construction primarily consist of criteria for required strengths of structural elements and components. These documents are referred to in the *Guidelines* where applicable. In general, there is no difference made between the principles of structural mechanics for new and old construction. Material properties may be different depending on age and condition, but the analytical methods for determination of flexural and shear strengths are assumed to be the same.

For evaluating strength of existing unreinforced masonry walls, methods are adapted from the UCBC (3) and FEMA 178 (5). These methods have emerged over the past decade from the Los Angeles Division 88 procedures which are based on research done with the ABK project done in the late seventies. Wall shear strength is estimated by multiplying reduced results from the in-place shear test by the gross area of a wall. For strength of a new unreinforced masonry wall that is added to an existing building, the *Guidelines* refer the user to the *NEHRP Recommended Provisions for New Buildings* (1) which in turn refers the user to a strength modified version of the Masonry Standards Joint Committee code, formerly the ACI 530 code (2). Here, permissible values of maximum wall shear stresses are given that are multiplied by section properties to give wall strength. A paradox arises when lateral strength of a system of existing and new walls must be estimated, and these two radically different codes must be merged. Development work is underway with the *Guidelines* to insure the consistency of such procedures.

Strength of reinforced masonry walls will follow the newly developed Limit States Design procedure for new construction. Although this procedure is still under development, it will be completed before the *Guidelines* and should replace the present 1992 MSJC code (2), and be endorsed through the NEHRP provisions for new buildings (1). Existing reinforced masonry walls will need to meet the standards for new construction if these provisions are to be applied.

Acceptable Deformations for Particular Limit States

In addition to stiffness assumptions and strength criteria, acceptable deformations for structural elements and components need to be specified. Although the concept of permissible deflections is new with performance based design, it is equally as relevant to the overall problem as the modeling procedures and assumptions of structural properties. Results from the analytical models, usually expressed in terms of a lateral drift of the structure, are compared against deflections that are acceptable to meet a particular limit state such as initial cracking, extensive cracking, yield of reinforcement, crushing of masonry or overturning for a masonry wall. If computed results exceed acceptable deformations, then the rehabilitated scheme must be revised, or the performance standards relieved.

The accuracy of the modeling should not exceed that of the acceptability criteria. However, since the criteria are often based on arbitrarily assumed estimates, this situation often exists. As mentioned previously, there is also the problem of associating a particular performance limit with a specific limit state for a structural element. As the concept of performance based design is introduced to the engineering community through the *Guidelines* and other documents, quantitative definitions of performance limits may become more clearly defined, and the precision of modeling procedures may become commensurate with the inaccuracies inherent in defining acceptable deflections.

ROLE OF INSITU TESTING

The *Guidelines* encourage the use of insitu tests of materials. If no test data is available, a penalty must be paid by using lower bound values of material strengths. These may be several times less than actual strengths because they must represent the minimum strengths possible for a wide range and age of materials.

Lower bound values of assumed material strengths are given in the *Guidelines* for masonry compressive strength, flexural tension strength, shear strength, and elastic and shear moduli. Conservative values of each property are given for masonry in good, fair and poor condition. In the absence of test data, the structural condition is subject to the qualitative judgment of the engineer which augments the degree of conservatism inherent in the lower bound estimates of strength. Because a penalty is included with each lower bound value, there is no need to assign an additional penalty with a variable capacity reduction factor for condition.

Insitu test data will reflect the general condition of a particular structural component. High test values will imply good condition where low values will suggest poor condition. Insitu measurement of material properties eliminates the need to make assumptions regarding condition.

For masonry structures, the *Guidelines* refer to standard methods for measuring strength and condition. Visual methods are described along with wave velocity tests for condition assessments. Methods are recommended for in-place destructive tests for measuring shear strength, flexural bond strength, vertical compressive stress and elastic modulus. In addition methods are discussed for laboratory destructive tests of samples extracted from existing buildings, and large-scale destructive load tests of actual building elements.

CONCLUDING REMARKS

This paper has provided a summary of present development towards constructing a set of engineering guidelines for seismic rehabilitation of buildings that will have the potential of being adopted throughout the United States as well as other parts of the world. Although the descriptions of the *Guidelines* as provided herein will change before the final draft of the document in the next two years, this preliminary view should help both Italians and United States participants understand what directions are being taken towards seismic rehabilitation of existing buildings.

ACKNOWLEDGMENTS

The author wishes to acknowledge the financial support of the Federal Emergency Management Agency for supporting the ATC-33 effort, and to the National Center for Earthquake Engineering Research and the University of Pavia for sponsoring the workshop. Views and opinions expressed in this paper are those of the author, and do not necessarily reflect those of the ATC-33 *Guidelines* authors, FEMA or BSSC.

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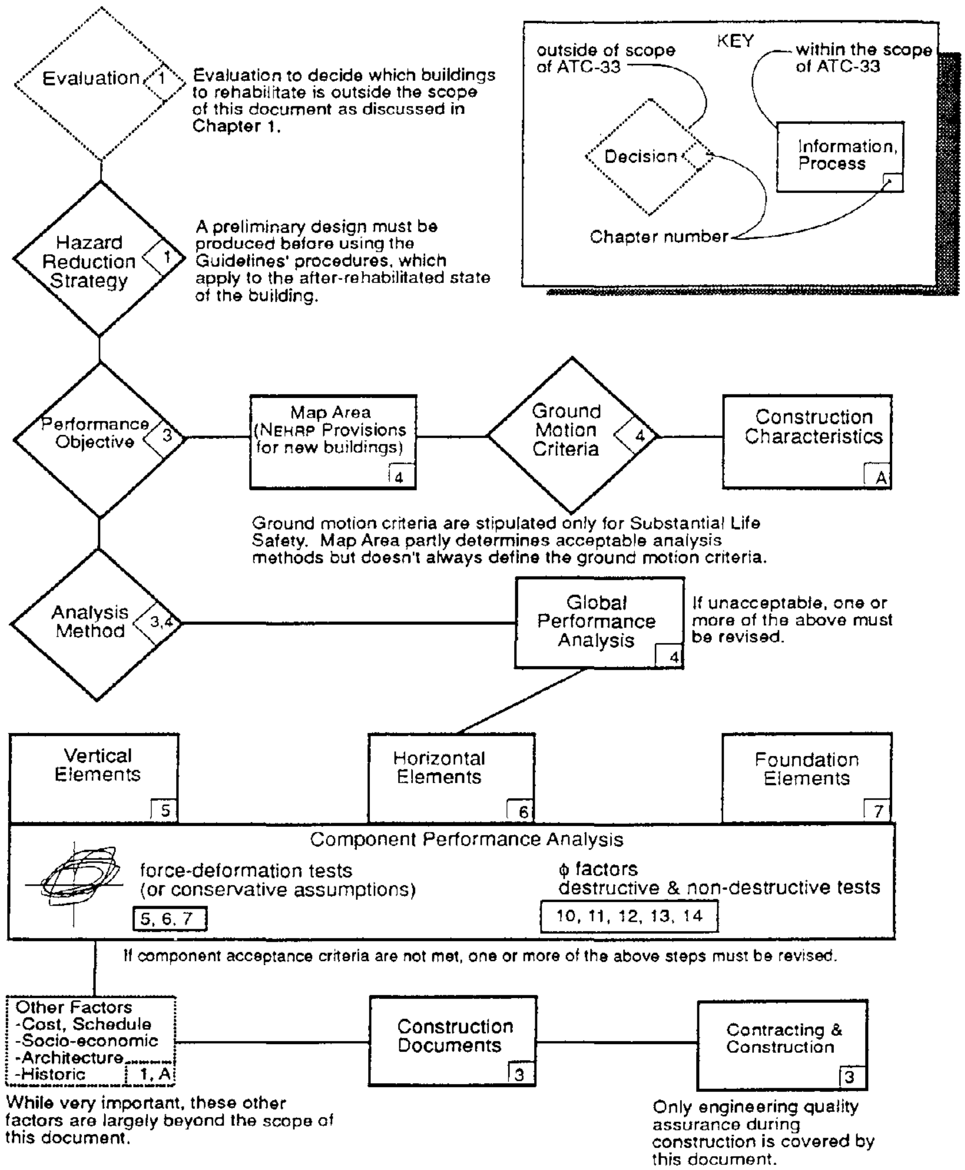

Fig. 1 Flowchart for Building Rehabilitation (from Ref. 7)

Section III

Research on Performance and Response of URM Building Systems

RESEARCH ON UNREINFORCED MASONRY AT THE JOINT RESEARCH CENTER OF THE EUROPEAN COMMISSION

Armelle Anthoine[1]

ABSTRACT

The experimental and numerical research activities of the Joint Research Centre of the European Commission in the field of unreinforced masonry are presented. The experimental activities are centred around the ELSA reaction-wall facility, where masonry panels and masonry infilled frames are subjected to static and/or pseudo-dynamic testing. All numerical activities are performed within a unique computer code CASTEM 2000, where different levels of modelling may coexist: at a micro level, brick and mortar are modelled separately; at a meso level, masonry may be considered as a plane homogeneous (anisotropic) media; at a macro level, each masonry infill/wall may be represented by one global element.

INTRODUCTION

The present paper is intended to give an overview of the research on unreinforced masonry currently carried on at the Joint Research Centre of the European Commission.

The paper is divided into two parts. The first part is devoted to experimental activities which are performed in the ELSA reaction-wall laboratory: testing of masonry panels and reinforced concrete frames infilled with masonry. In the second part, are presented the numerical activities, which are performed in a common computer code (CASTEM 2000): homogenization of masonry, modelling of masonry panels and infilled frames.

EXPERIMENTAL RESEARCH

Presentation of ELSA, the European Laboratory for Structural Assessment

The Safety Technology Institute (STI) of the Joint Research Centre (JRC) of the European Commission has built a structural assessment laboratory, based on a 16m high, 21m wide reaction-wall. Designed to resist the forces, typically several hundred tonnes, which are necessary to deform and seriously damage full-scale test models of structures, the ELSA reaction-wall is one of the largest facilities of its type in the world, only exceeded in Japan.

In addition to static and cyclic tests on large structures and components, the facility is equipped to

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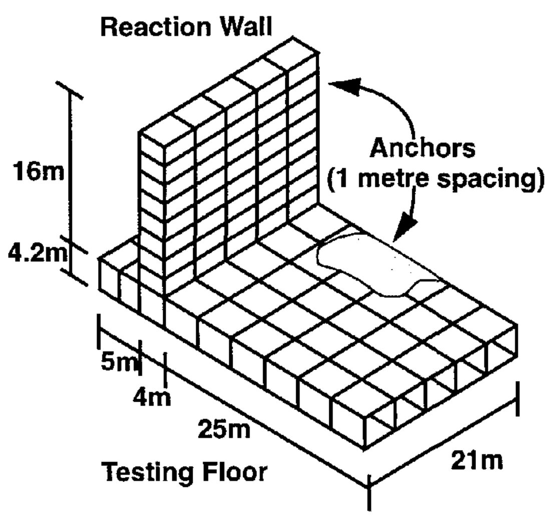
perform tests utilizing the so-called pseudo-dynamic (PSD) test technique which enables, for example, the simulation of earthquake loading of full-scale buildings.

The facility, which cost about 7 Million ECU, is used within the framework of Community-wide integrated research programmes, so that the existing expertise and facilities within the Member-states are fully used. To this end, the European Association of Structural Mechanics Laboratories was set up, at the instigation of the Applied Mechanics Unit of the STI. This association, which has more than twenty-five members, is helping to define the detailed programmes - the first of which concerns the response of civil engineering structures to severe earthquake loading.

In addition to the above programme of scientific research involving both the study of the non-linear dynamic behaviour of structures and the development of appropriate testing methods, a specific programme (HCM-PREC8 network) is currently giving support to the European Commission for the EUROCODES - the harmonized European standards for construction.

The facility is also available to external customers, for performing demonstration and qualification tests on large-scale prototypes and/or validating innovative constructions. Negotiations are in progress with several potential customers from industry and from authorities responsible for national research programmes.

The details of the reaction-wall/ strong-floor system are given in Fig. 1. The dimensions allow the testing of full-scale buildings up to five-storeys high, either quasi-statically or pseudo-dynamically. Real-time dynamic tests can also be performed on lighter models such as, for example, piping systems.



Load capacity characteristics

REACTION WALL	Bending Moment ....200 MNm
	Base Shear ....20 MN
REACTION FLOOR	Bending Moment ....240 MNm
ANCHOR LOAD CAPACITY	Axial Force .... 500 kN

Hydraulic characteristics

FLOW	1500 l/min.
PRESSURE	210 bar
ACTUATORS	Load (MN)............(0.25, 0.5, or 1.)
	Stroke (m) ....((+/- 0.25) -- (+/- 0.5))

Figure 1- Characteristics of the ELSA-Reaction Wall Laboratory.

Each actuator is driven via a digital controller which is connected in a local area network (LAN), through an optical fibre, to the main computers running the pseudo-dynamic test algorithm (capable of using explicit or implicit methods of time integration). The deformation of the tested structure is monitored primarily by displacement transducers (both digital and analogue transducers are available, covering ranges from 1mm to 1m). The laboratory has also a long experience in strain-gauge systems and related instrumentation. The data acquisition system is able to record up to 200 channels (distributed on 6 PC's) having a 12 bit resolution and a global sampling frequency of 10KHz.

The principle of the pseudo-dynamic method is schematically illustrated on Fig. 2. The kinematics of the building is supposed to be well represented by a small number of degrees of freedom, e.g. the horizontal displacements at each floor. A record of an actual (or artificially generated) earthquake ground acceleration history is given to the computer. At the first (small) time step, the horizontal displacements **d** of the floors are calculated through (explicit or implicit) numerical integration of the equation of motion, where inertial and viscous forces are modelled analytically (matrices **M** and C). These displacements are then applied to the structure by servo-controlled hydraulic actuators attached to the reaction wall. Load-cells on the actuators measure the forces **F** necessary to achieve the required deformation (structural restoring forces) and these are then used at the next time step for determining the next displacements **d** to be applied to the building.

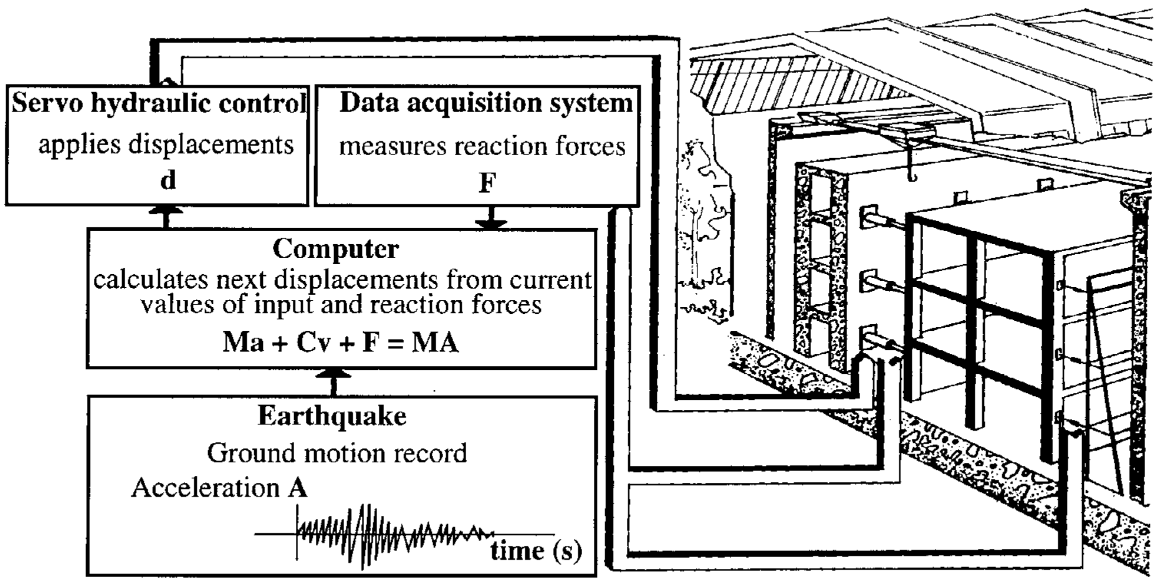


Figure 2- Pseudo-dynamic test (schematic)

Since the inertial forces are represented analytically, there is no need to perform the test in the real time-scale and, typically, an earthquake of ten seconds is simulated pseudo-dynamically in about one hour. Here is one of the major advantages of the pseudo-dynamic method, when compared to the conventional test methods (shaking table): it is possible to test very large models, with a modest hydraulic power. The second major advantage, over shaking table testing, is the possibility of monitoring very closely the progression of damage in the structure: a pseudo-dynamic test may be stopped at any moment for a detailed examination or for preventing a complete collapse.

The potential of the PSD test method has not yet been fully exploited and new fields of application can be expected. The JRC installation is the first to use fully digital servo control of the applied displacements, allowing a highly accurate test procedure and a much more versatile use of the various mathematical algorithms for numerical time integration of the equations of motion.

By using a mathematical technique known as sub-structuring, significant further developments are possible. With this procedure, only the most interesting part of a structure is tested experimentally, whilst the rest is modelled analytically. The computer takes into account the interactions between the two parts of the structure, when calculating the displacements of the tested part. Thus, structures much larger than the laboratory itself, such as bridges, can be tested: the physical testing can be limited to the piers alone, if the bridge deck is assumed to remain elastic and thus is easily modelled numerically. Fig. 3 shows the general arrangement of such a PSD testing.

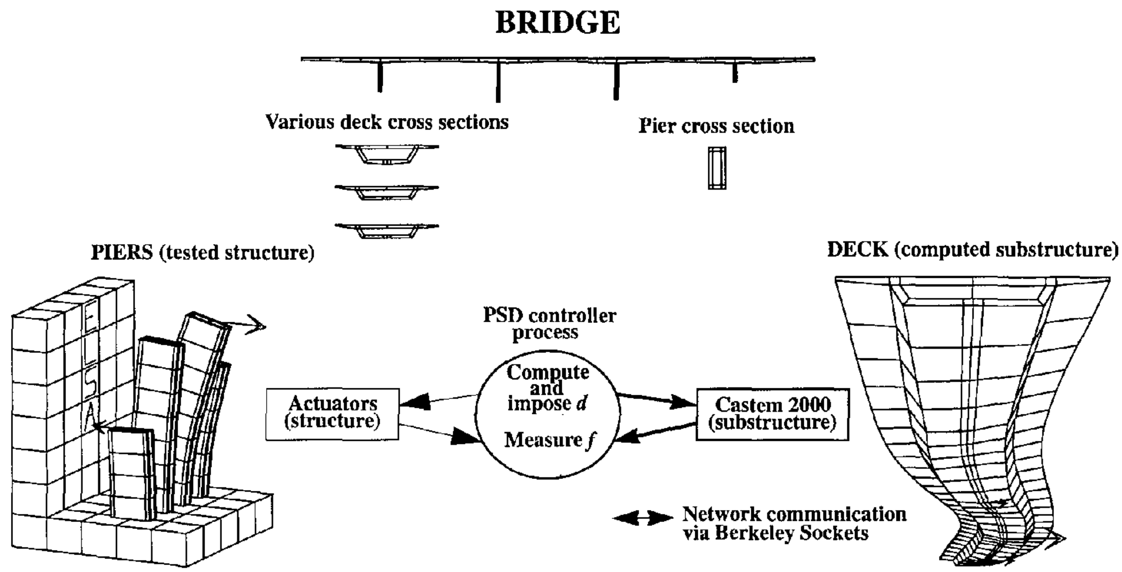


Figure 3- Pseudo-dynamic testing with substructuring: a four piers bridge case.

A further advantage, which is again well suited for bridges, is the possibility of dealing with varying/asynchronous seismic excitations at the base of the piers. Alternatively, when the structural material exhibits a strongly rate-dependent behaviour, much faster testing speeds can be achieved by reducing the physical model to those few components which are behave non-linearly, whilst the rest is simulated in the computer.

Shear-compression testing of brick masonry walls

The objective of this experimental research was to investigate the seismic behaviour of brick masonry walls subjected to complex loading conditions. Two types of walls have been chosen for being the key-elements of the building prototype to be tested at the Pavia University within the framework of a cooperative research programme ("Experimental evaluation for the seismic behaviour of structures") promoted by the Consiglio Nazionale delle Ricerche (CNR - Italy).

Five specimens having the same width d = 1m, the same thickness t = 0.25m (english bond) but different heights h = 1.35m or 2m, have been built. Three of them have been tested quasi-statically whereas the two remaining ones are planned to be tested pseudo-dynamically. The experimental set-up is schematically represented on Fig. 4.

The loading conditions have been chosen so as to reproduce as well as possible the real conditions undergone by the panels during a seismic event (constant vertical load, double bending moment). A

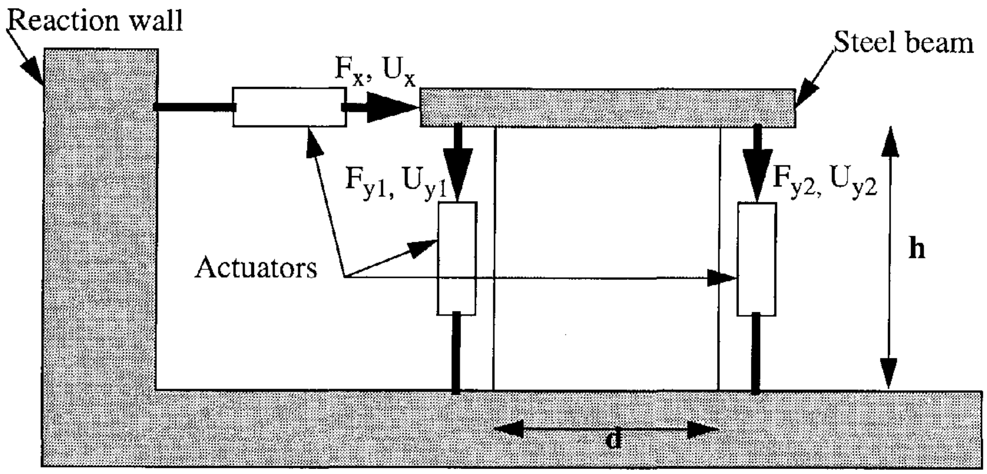


Figure 4- Schematic view of the test set-up.

uniform vertical compression stress was applied first by means of the two vertical actuators, the resulting vertical load being $F_y = F_{y1} + F_{y2}$. Next, the lateral displacement U_x was imposed by means of the horizontal actuator. At each step, the forces F_{y1} and F_{y2} in the two vertical actuators were adjusted so as to maintain the same value of the vertical load as well as the horizontality of the steel beam; thus, with the notations of Fig. 4, the forces and the displacements in the two vertical actuators were such that $F_{y1} + F_{y2} = F_y$ and $U_{y1} = U_{y2} = U_y$.

For the quasi-static test, the vertical load was F_y = 150kN and alternated lateral displacements U_x of increasing amplitude have been imposed. The walls were fully instrumented in order to get a large amount of data susceptible to be compared with finite element results (not only forces and displacements at the top, but also deformed shape of the wall).

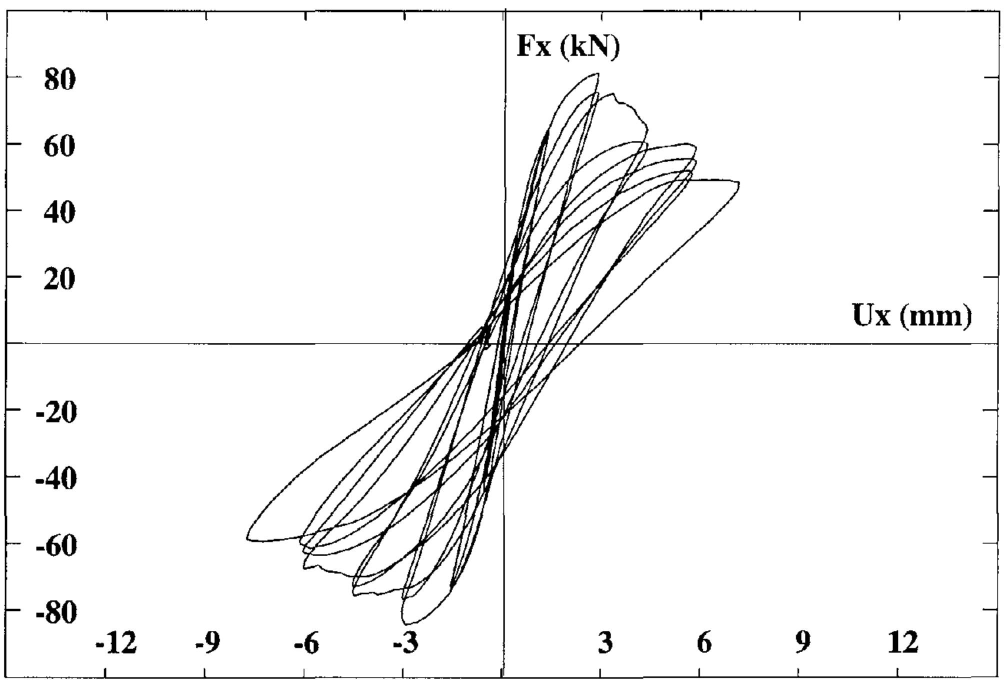


Figure 5- Horizontal force F_x versus lateral displacement U_x for the low wall.

The results of the quasi-static loading put in evidence the effect of the height/width ratio on the behaviour of the walls (Fig. 5 and Fig. 6): the ultimate load and displacement, the ductility, the degradation, the deformed shape and the failure mode are strongly influenced.

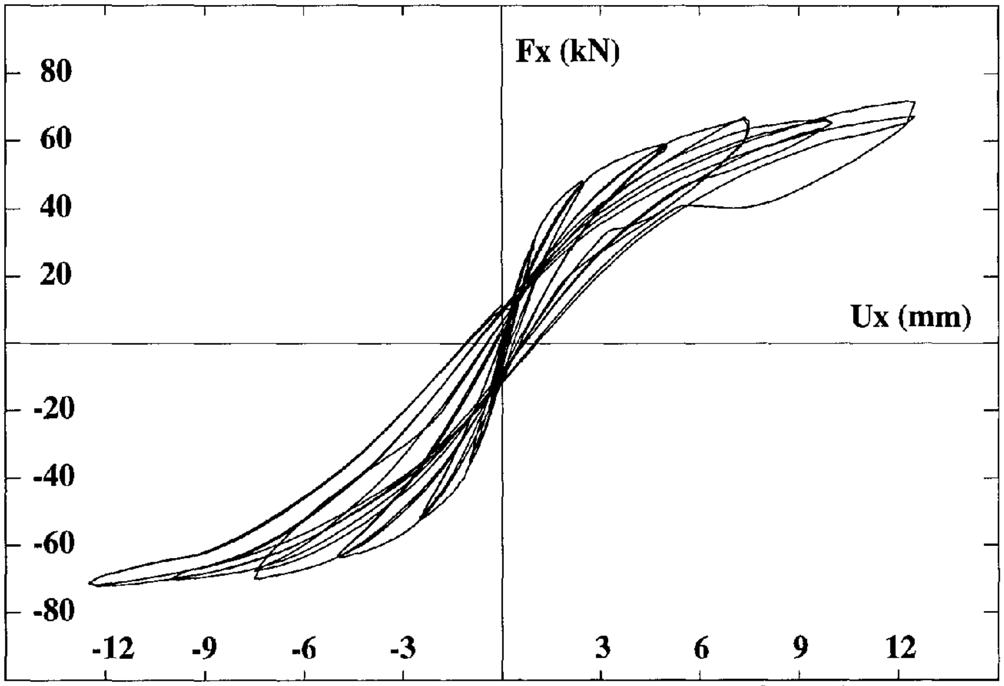


Figure 6- Horizontal force F_x versus lateral displacement U_x for the high wall.

The pseudo-dynamic testing of the low wall has also been performed. The loading conditions were the same as in the quasi-static test. The base acceleration, the vertical load and the value and position of the mass have been chosen so as to reproduce the shaking table tests performed on identical walls at the Laboratorio Nacional de Engenharia Civil (LNEC-Portugal): six accelerograms of increasing intensity (measured on the shaking table) have been successively applied (Fig. 7), the vertical load was 270 kN and a mass of 5 tons was considered at the top of the wall.

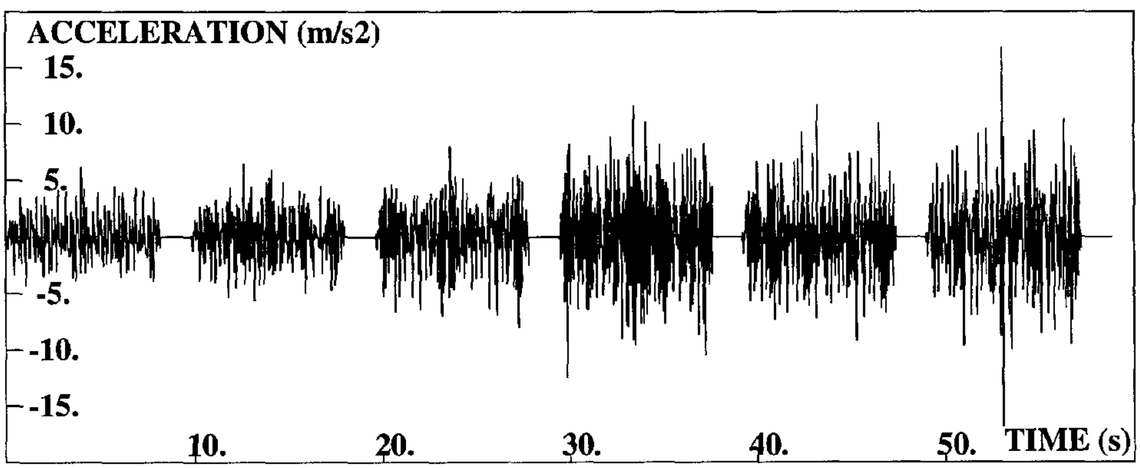


Figure 7- The six successive accelerograms.

Despite the great stiffness of the tested structure, the pseudo-dynamic test was almost perfectly controlled: in particular, the vertical load remained constant, even if each of the two vertical forces were subjected to noticeable variations (Fig. 8).

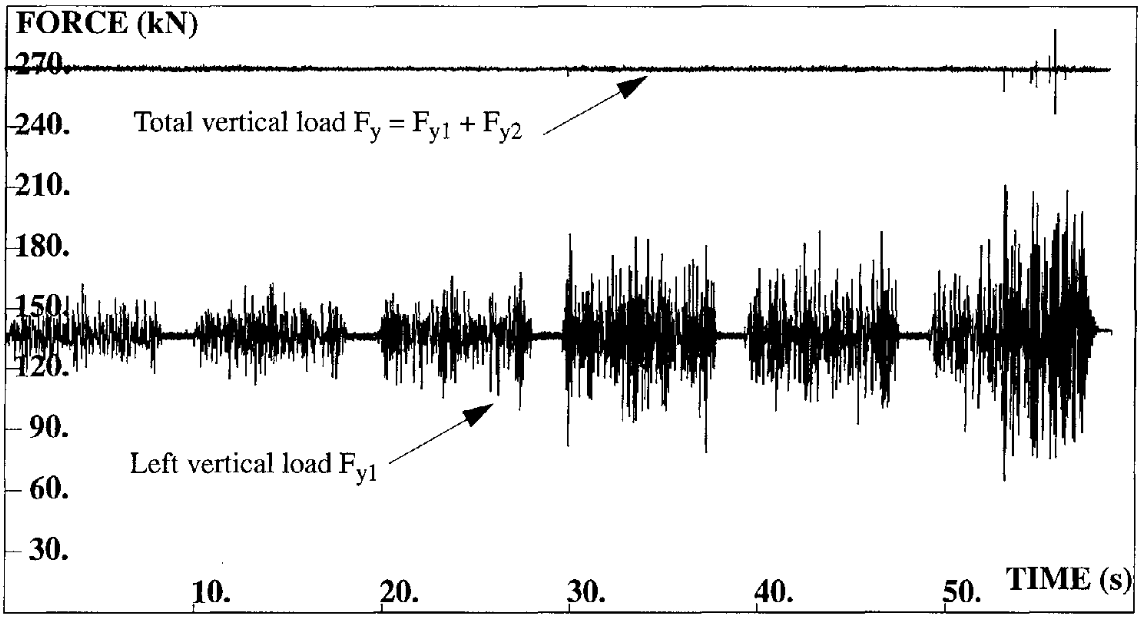


Figure 8- Total and left vertical loads during the pseudo dynamic test.

The response of the wall remained elastic until the last accelerogram where failure occurred by diagonal cracking. The hysteretic loops (horizontal force versus horizontal displacement) are displayed on Fig. 9.

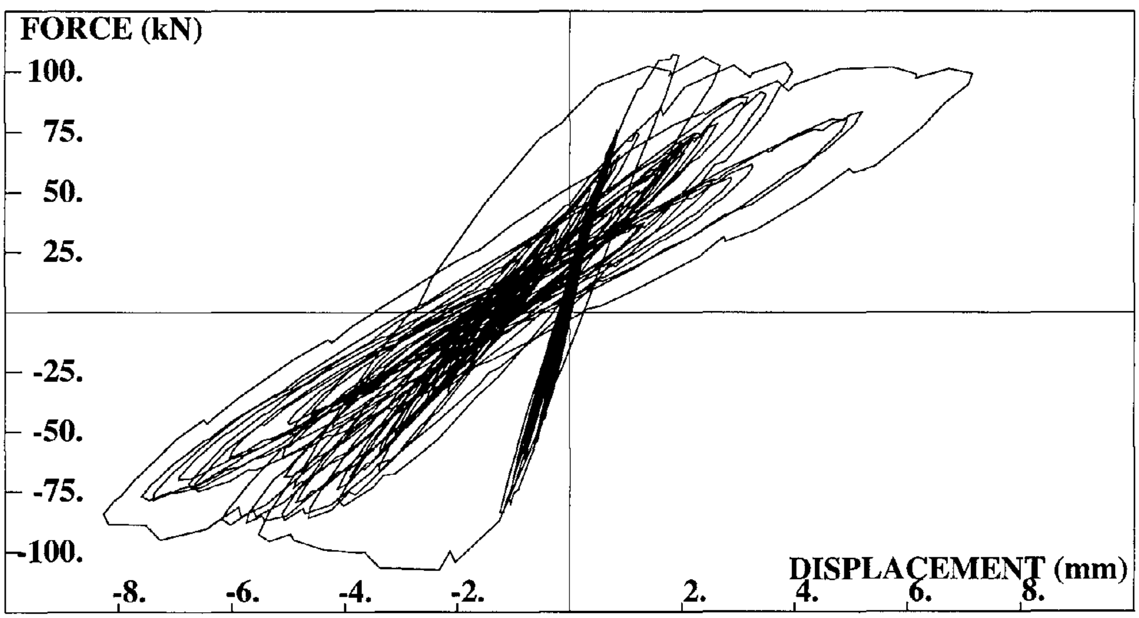


Figure 9- Horizontal force versus horizontal displacement at the top of the wall.

Testing of infilled frames

Within the framework of the HCM-PREC8 network (topic 2: effect on infills on the seismic response of frames), a 4-storey infilled reinforced concrete frame is planned to be tested pseudo-dynamically. The two lateral frames of this structure (parallel to the direction of loading) will be completely filled with masonry. The frame is composed of two bays, one 4m wide, the other 6m wide. The columns are 400 x 400 mm, the beams are 300 x 450 mm. The bricks used for the masonry infills are 245 mm long, 190 mm high and 112 mm thick and are hollowed vertically.

NUMERICAL RESEARCH

The modelling strategy

One of the key feature that differentiate modelling formulations for masonry structures is the level of discretization. For instance, a masonry wall may be considered as an assemblage of three-dimensional periodic cells, bricks and mortar being represented separately (micro level), as a homogeneous (anisotropic) two-dimensional media under plane stress (meso level), or even, as a single global element characterized by generalized stresses (moment, shear) and corresponding displacements (macro level). These formulations are complementary: refined but time-consuming models may be used to define, validate and/or calibrate less refined but more economic models. Such a deductive strategy is easy to follow when the different levels of modelling are available within the same computational environment: this is the case of the finite element code CASTEM 2000 used at the Applied Mechanics Unit, thanks to the generality of its data structure [2]. Different levels of modelling may thus coexist, even during the same computation.

Three examples, belonging to the micro and meso levels of modelling, are presented below.

Homogenization of masonry

Through the homogenization theory for periodic media [8], the macroscopic behaviour of masonry may be derived from the behaviour of its constitutive materials (brick and mortar). Such a procedure has been used by many authors but always in an approximate manner. In particular, the homogenization procedure has always been performed in several successive steps, head joints and bed joints being introduced successively [3][6][7]; moreover, masonry has always been considered either as infinitely thin (two-dimensional media under plane stress [3]), or as infinitely thick (two-dimensional media under generalized plane strain [6][7]), so that its finite thickness was never taken into account.

In order to determine the range of validity of either assumption, the homogenization theory has been implemented in a rigorous way, i.e. in one step and on the real geometry of masonry (finite thickness and actual bond pattern). Both brick and mortar being assumed as subjected to isotropic damage [4], numerical computations show that, in the linear range (elasticity), both assumptions give satisfactory results (Table 1), even if the plane stress hypothesis is scarcely verified by the three-dimensional stress fields [1]. However, in the non-linear range, i.e. when damage occurs, the plane stress assumption may lead to erroneous results quantitatively (value of the ultimate load on Fig. 10), as well as qualitatively (mode of failure on Fig. 11) in particular, under the plane stress assumption, failure occurs by cracking (vertical joint) and crushing (horizontal joint) of the mortar. The failure mode under the generalized plain strain assumption agrees well with the three-dimensional one (cracking of the vertical mortar joint and of the brick).:

Table 1: Homogenized elastic constants of running bond masonry (brick dimensions: 120 x 55 x 90 mm, joint thickness: 10mm, E, ν : 11000MPa and 0.20 for brick, 2200Mpa and 0.25 for mortar).

	Plane stress	Generalized plane strain	Three-dimensional
E_1 (MPa)	7,700	7,820	7,800
E_2 (MPa)	6,590	6,810	6,780
ν_{12}	0.195	0.201	0.200
G_{12} (MPa)	2,480	2,490	2,490

Since the generalized plane strain assumption gives satisfactory results even in the non-linear range, it should be preferred to the plane stress assumption, whenever three-dimensional calculations can not be afforded. Note however that three-dimensional calculations remain necessary as soon as the characteristics of the masonry vary through the thickness of the wall (hollow bricks, composite walls, complex bonds).

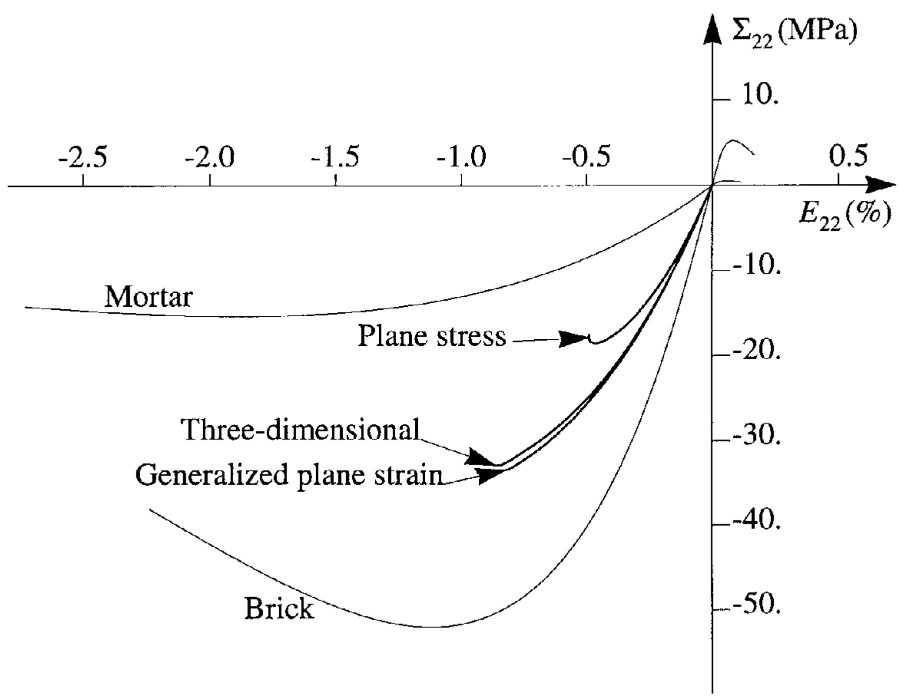


Figure 10- Uniaxial vertical compression curves.

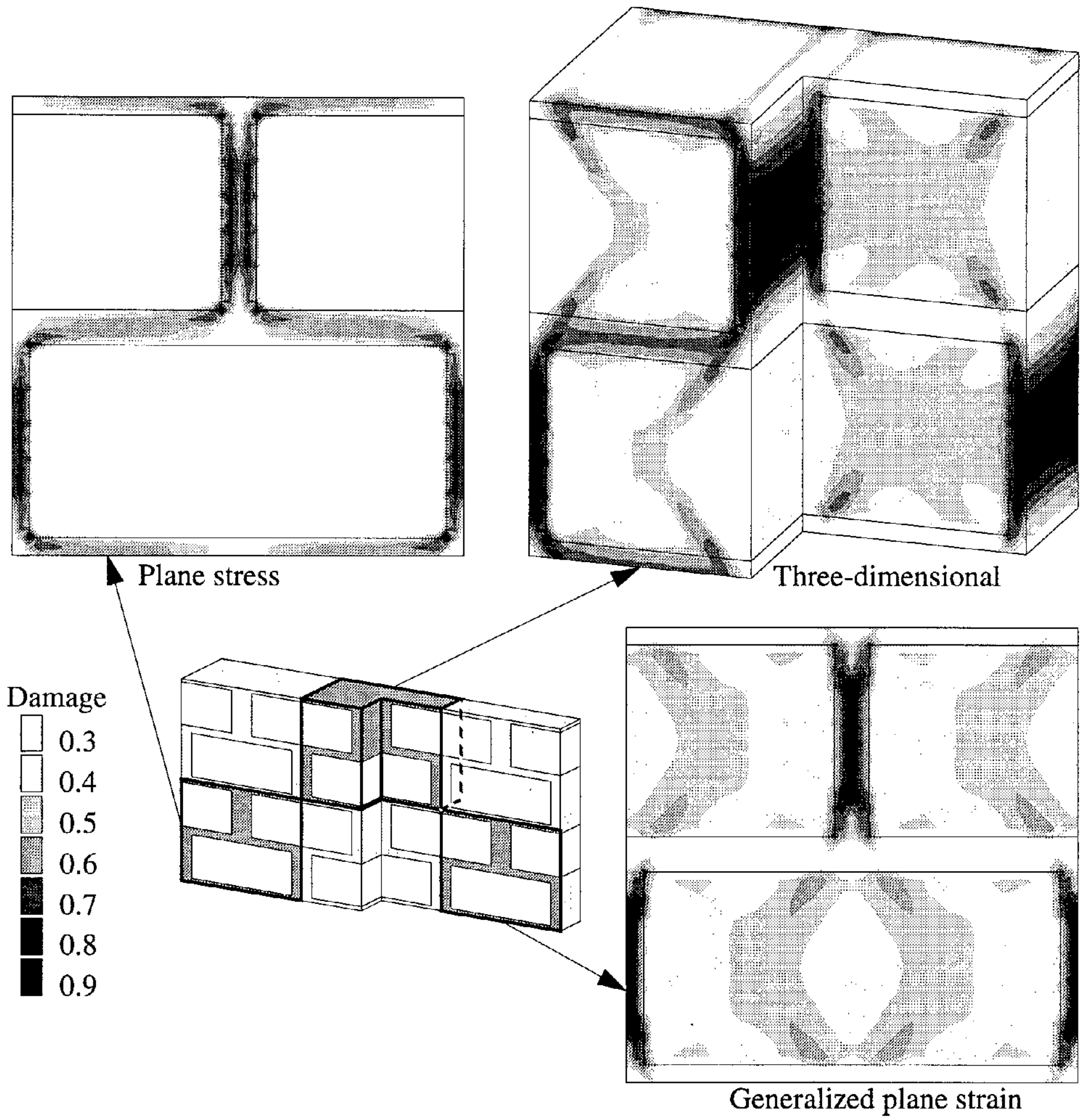


Figure 11- Damage at maximum compression stress.

Numerical modelling of brick masonry walls

Numerical simulations of the small masonry panels mentioned in the first part, have been carried out, always in the same finite element code (CASTEM 2000): masonry has been modelled as a two-dimensional continuum subjected to isotropic damage [4]. The parameters of the model have been chosen according to preliminary tests performed on small scale specimens (masonry wallettes). The loading conditions of the test were respected but the lateral displacement was simply monotonously increased. The numerical results are qualitatively in accordance with the experimental observations. In particular, the failure mode and the cracking pattern are well predicted (Fig. 12),

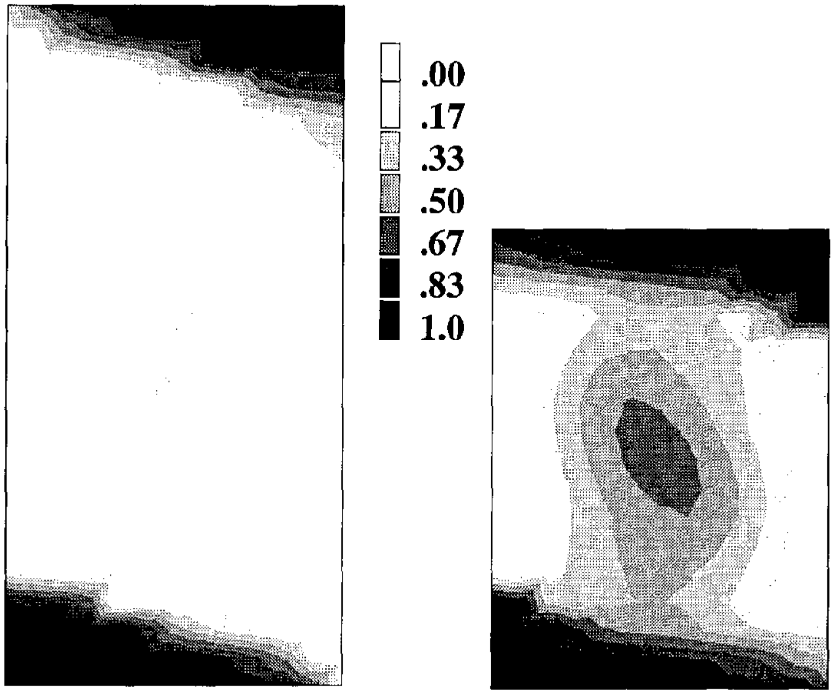


Figure 12- Computed damage index at maximum lateral displacement: the high wall fails by rocking whereas the low wall fails by diagonal cracking.

independently of little variations of the constitutive parameters. However, further refinement of the model (cyclic loading, anisotropy) is needed in order to achieve a better accuracy.

Numerical modelling of infilled frames

The purpose of this study was: to assess the possibility of identifying/calibrating simple macro-models for infilled frames (equivalent diagonal struts) through a more refined modelling of the structure; to evaluate the degree of interaction between the adjacent infilled frames of a building; to get a rough idea of the lateral forces required for testing the 4-storey infilled frame mentioned in this first part.

Different sub-structures of the lateral infilled frame are considered: each one of the two bays of the first storey, the first storey, the two first storeys and, of course, the whole frame. In each case, the behaviour of the (unfilled) bare frame is also computed in order to quantify the effect of the masonry infills.

The complete building being composed of three frames, it is assumed that the internal frame supports half of the dead load, whereas each lateral frames supports a quarter of it. The total dead load being 850 kN per storey, a load S = 850/4 = 212.5 kN is distributed on each beam (except the upper one) of the external frame. The own weight is omitted. In the case of a sub-structure, the dead load corresponding to the missing storeys (S for two missing storeys, 2S for three) is applied on the top of the three columns in the following proportions: 1/5, 1/2 and 3/10. The lateral load consists in increasing concentrated forces, proportional to the height and applied at each storey. In order to distribute these forces along the beams, it is assumed that all points lying on the mean fibre of the same horizontal beam undergo the same horizontal displacement.

Computations are performed under the plane stress assumption. The reinforced concrete frame is considered as a two-dimensional composite media, steel and concrete being modelled separately: the steel bars are represented by one-dimensional finite elements embedded in the two-dimensional concrete elements. Masonry is considered as a homogeneous material (homogenized media) so that bricks and mortar joints are not represented. As far as behaviour law are concerned, the steel is assumed elastic perfectly plastic whereas both concrete and masonry are described by an isotropic model [5], combining plasticity in compression (Drucker-Prager criterion with softening) and smeared cracking along two orthogonal fixed directions in tension (maximum tensile stress criterion with softening). Unilateral contact without friction is assumed at the infill/frame interface.

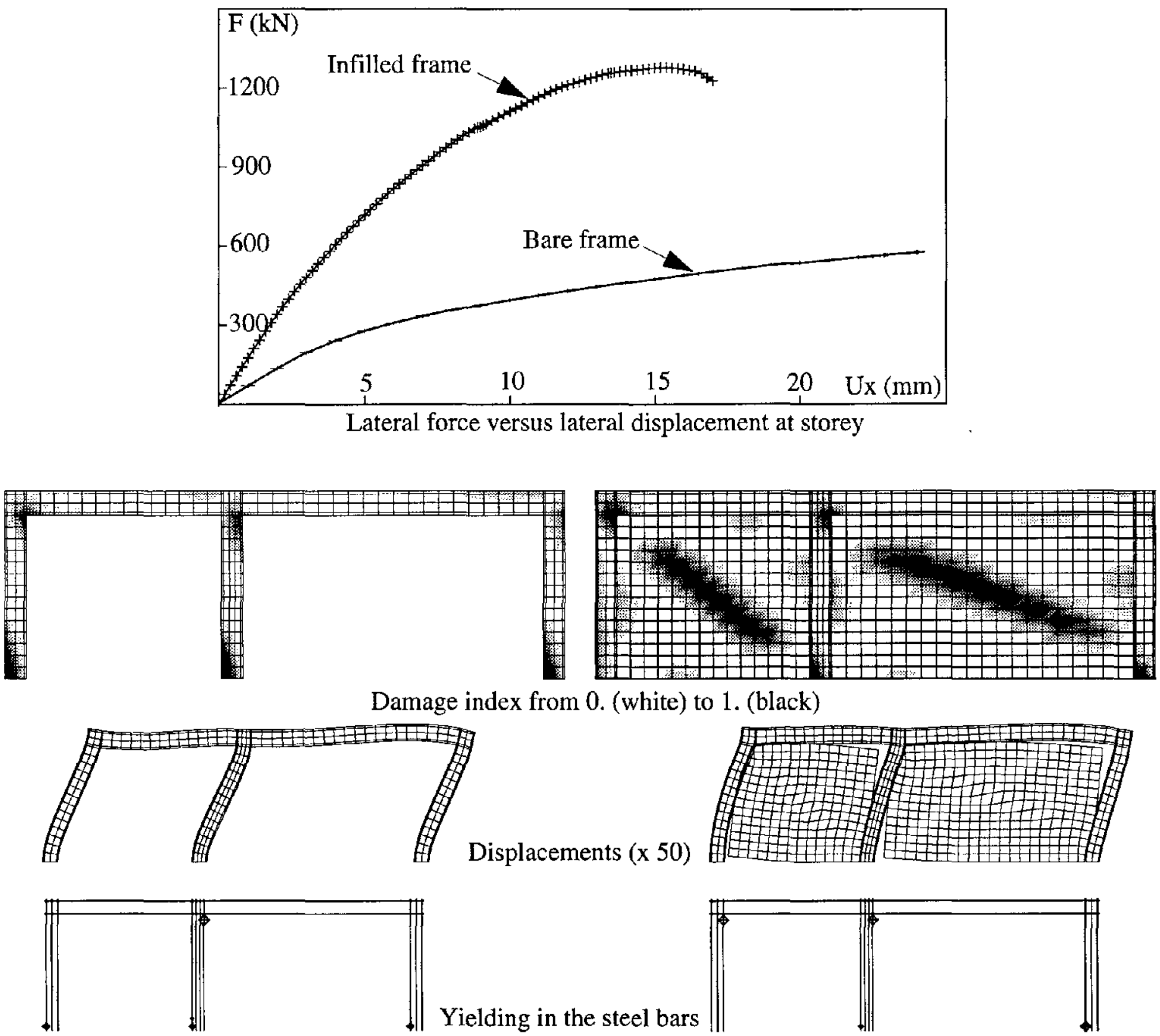


Figure 13- Results for the first storey with and without infill.

The results obtained for the first storey are presented on Fig. 13. When the masonry infills are added, the strength and stiffness of the frame are substantially increased: for the same level of lateral displacement, the lateral force is 2.5 times higher. Furthermore, the yielding in the steel bars does not occur exactly at the same locations, and the damage is spread along the left column of the bare frame. Failure of the infills occurs by diagonal cracking, as observed experimentally.

CONCLUSION

The current experimental and numerical research activities of the JRC-EC in the field of unreinforced masonry, have been briefly reviewed. The two aspects (experimental and numerical) are developed in parallel: experimental results allow to calibrate/validate numerical models; conversely, numerical analyses help in designing/interpreting experimental tests.

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RESEARCH ON THE SEISMIC PERFORMANCE OF REPAIRED URM WALLS

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ABSTRACT

Older unreinforced brick masonry buildings that were originally designed with little or no provision for lateral loading occur in large numbers in most areas of the world. The risk represented by this old building stock has been recognized as one of the major problems facing the Earthquake Engineer today. The research described in this paper involves the development of an alternate type of upgrading procedure whereby a thin coating of a ferrocement like material is attached to both sides of the masonry. The method is shown to be very effective in the seismic upgrading of such walls for strength, displacement demand and energy dissipation. Enhancement in strength of up to three times is not uncommon. All of the other factors involved in the dynamic behavior of the walls are similarly improved. The result of the testing program shows quite clearly the value of the method of renovation. A new hysteresis model is introduced which allows a nonlinear time history analysis to be made of the wall.

INTRODUCTION

The large numbers of older masonry structures, that were originally designed with little or no provision for lateral loading, represent one of the major problems facing the Earthquake Engineer today. Such buildings can be found in most areas of the world and probably account for upwards of 70% of the existing building stock. Strengthening these buildings against earthquakes has become a critically important activity.

The research described in this paper involves the development of a new upgrading procedure where by a thin coating of a ferrocement like material is attached to both sides of old unreinforced brick walls. Early preliminary research (5) indicated quite clearly that ferrocement had a high potential as a material for increasing the earthquake resistance of unreinforced brick masonry. Subsequent work was directed to determining the bonding and mechanical connector requirements and static shear behavior through a series of standard diagonal split tests (1). It was found that ferrocement sheets, used in the manner described, had a great deal of potential as a material for increasing the earthquake resistance of unreinforced masonry structures.

THE TEST PROGRAM

Four series of test have been conducted to determine the general behavior of the system and to establish its practicality. The first was to determine the bonding between the masonry and the coating and the required connect or size and spacing by performing standard diagonal split tests. The other three were to study the

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hysteretic and dynamic behavior of the masonry walls upgraded with ferrocement, using both slow cyclic loading tests and the shake table.

In the first phase of the study, ten pairs of thin ferrocement sheets each 42 inches x 42 inches x 1/2 inch (1.07M, 1.27cm) were subjected to shear or diagonal split tests in a universal testing machine to establish the buckling behavior and the size and spacing of connectors required. Four masonry specimens of the same plan size were constructed from old reclaimed brick and were also subjected to diagonal split.

In the second group of tests, eight brick walls each two wythes thick and six feet wide and eight feet high (1.83M x 2.44M) were tested. Half of the walls were coated with a 1/2 inch (1.27cm) thick ferrocement overlay on both sides and were subjected to a series of psudo-static, cyclic loading tests.

The third phase of the study, involved earthquake simulation tests, in which eight additional walls were tested on the shake table under simulated earthquake ground motion. These tests were necessary to investigate the upper strength limits, ductility requirements, energy dissipation, and strength and stiffness degradation characteristics for the coated unreinforced masonry wall system. In addition, the data from the shake table tests was used to verify predictions of behavior from a hysteric model developed for the system from the data gathered in the cyclic testing.

The fourth phase, currently underway, involves the testing under cyclic loading of a series of low aspect ratio walls in shear. Consideration in this case is given to the effect of various reinforcement configurations within the ferrocement coating.

Figure 1 shows a wall specimen in the process of being coated. The mesh layers are attached to the unreinforced brick wall specimen by thru-bolts as shown. The bolts will later be covered by the portland cement matrix material, which is forced through the mesh so that it bonds with the wet brick, as shown in the firgure. A coated specimen ready for in-plane shake table testing is shown in Fig. 2. Note that the coating is placed on both sides of the brick wall forming a composite sandwich that can be seen to be closely related to the well-known Jacketing procedures used for slender members. Future testing will involve the in-plane and out-of-plane behavior of medium aspect ratio unreinforced walls and the response of tall walls bent out-of-plane by simulated earthquake loading.

The Diagonal Split Tests

After preliminary testing (5) which illustrated the feasibility of the procedure, the first series of diagonal split tests were to determine the connector characteristics, size and spacing needed to prevent delimitation by buckling of the layer away from the wall. After a series of trials, it was determined that 1/4 inch (.64cm) thru-bolts were needed spaced at about 12 to 18 inches (.3M - .46M) depending on the thickness of the coating layer.

The second phases (1) of the diagonal split tests were performed on four 42 inches x 8 inches (1.07M x .2M) specimens constructed from old reclaimed brick using ASTM Type M mortar. Three of them were coated on both sides with the ferrocement overlay, each with a different type of mesh reinforcement. One specimen was left uncoated as a control. All of the specimens were tested under static load after proper curing in the moist room. After coating and further curing, they were tested in a high capacity universal testing machine. At first, the results were quite unsatisfactory with the diagonal crack that developed in the brick core transferring itself directly to the coating and causing a large single split in the coating. Brittle behavior followed. It was clear that the connecting bolts were too rigid and forcing the coating to follow the brick core.

To correct this problem, the holes in the coating through which the bolts passed were enlarged which allowed for movement between the coating and the brick. When this was done, instead of a single crack in the coating, a whole series of closely spaced hair-line cracks developed due to the load redistribution as illustrated in Fig. 3. The ductility of the system was greatly enhanced. In all cases, behavior was linear up to the load at which the brick core cracked. This point was indicated by noises from within the specimen and a small drop in the load carrying capacity. The ultimate load reached was on the order of 2.5 to 3 times that of the uncoated specimen. This can be attributed to the confining effect of the coating and connecting bolts. The ultimate strength of the composite specimen was largely independent of the coating thickness and the type of reinforcement used. Visible cracks in the coating did not appear until after the load reached a value close to the brick cracking load. These small cracks formed in a fairly wide band between the loaded corners and were more numerous with the smaller mesh sizes.

Pseudostatic Tests

On the basis of the preliminary diagonal split tests, a 1/2 inch x 1/2 inch (1.27cm) with 19 gage galvanized wire mesh was chosen as the standard for the ferrocement overlay which was attached to the unreinforced brick walls. In this testing program a total of 16 walls each six feet wide, eight feet high and eight inches thick were (2.44M, .2M) constructed from reclaimed old bricks. Half of the walls were coated with a 1/2 inch (1.27cm) layer of ferrocement containing two layers of mesh. As was defined from earlier tests, 1/4 inches (.64cm) bolts spaced at 12 inches (.3M) were used to prevent delimitation of the wall. The mortar strength averaged about 2200psi (15.17Mpa). The prism strength was about 2400psi (16.55Mpa) while the coating cement had a strength of about 4000psi (27.58).

The test setup for the pseudostatic cyclic loading tests is shown in Fig. 4. Two wall specimens were mounted on a relatively rigid test base that was anchored to the laboratory strong floor. A ten ton ballast block was used to represent the overburden loads at the first floor level of a typical two-story masonry building. The block was attached in such a way that a freely rotating condition existed at the top. The base of the wall was fixed. A servo hydraulic actuator was connected between the ballast block and a reaction wall, as shown. The ballast block acted as a rigid diaphragm connecting the tops of the pair of walls. This set up using a pair of walls was chosen for reasons of stability.

Calibrated sonic displacement transducers were used to measure displacements. All data was automatically recorded. Displacement measurements were at intervals along each wall, as shown, on the ballast block to sense any twisting, at the fixed base to detect any rotation or uplifting of the wall base. Tests using the same test set up were carried out for both in-plane and out-of-plane loading and for both coated and uncoated specimens.

Each pair of walls was chosen based upon the results of free vibration tests and were subjected to a series of incremental lateral loadings. For each level of load, three cycles of a saw tooth load at a frequency of .02Hz were applied to the test specimens. After each load set, a check on the ambient frequency of the system was made in order to detect internal damage. The amplitude of the controlled displacement was gradually increased until the test specimens reached failure that was defined as the load level that produced no further loss in frequency with increasing amplitude.

All of the wall specimens displayed a flexural mode of failure. During the early loading stages, flexural cracks were initiated several bricks above the base of the uncoated walls, while the ferrocement coated specimen's damage usually took place somewhat above the fixed end. The hair-line cracks then spread and allowed for a rocking and uplifting type of motion about the interior crack in the brick. The difference in the cracking behavior for the coated wall and the plain wall specimens was that no major cracks developed in the coating and the whole system behaved much like a sandwich beam.

Details of the results of these tests are given in Refs. (2, 3). Briefly, the in-plane strength was increased about three times where the brick wall specimen was coated by the ferrocement overlay. Similar improvements were noted in the flexural strength of the coated walls. Not only was the initial hysteretic stiffness as defined by the hysteretic envelope increased, the degradation in stiffness was reduced during each incremental loading step. The initial stiffness under out-of-plane loading was increased by about two times. In addition, all other important dynamic characteristics such as energy dissipation and damping were considerable enhanced by the coating with the energy dissipation increased by three to six times.

Hysteric Model

On the basis of the results of the series of cyclic load tests, a new hysteretic model was developed to enable the prediction of nonlinear cyclic response under any disturbance. The model was based on continuous algebraic function with parameters to control the height, width, slope and area of the loops and backbone curve. For details relating to the model, see Reference 4. The ability of the model to reproduce the cyclic test hysteretic loop was very good.

Simulated Earthquake Tests

The test set-up that was used for the simulation test series was basically the same as that used for the cyclic load tests, Fig. 4. For this test, the specimens were driven by an earthquake type loading generated by a high capacity shaking table located in the NCEER/Buffalo Labs.

After pairs of walls showing the nearest frequency response, as defined by a free vibration test, were selected, a banded white noise excitation having a frequency range of 0-20Hz and a time duration of 40 seconds was applied. From this preliminary test, the model frequency mode shape and modal damping factor could be estimated.

Each pair of walls, joined at the top by the 10 ton diaphragm-ballast block, were subjected to a simulated earthquake loading by the shake table. The N-S component of the 1940 El Centro Earthquake was used to drive the table. The earthquake record was attenuated so that increasing load intensities could be applied to the pair of walls. The intensity was gradually increased until failure was noted in the walls. In order to monitor the extent of damage in the walls, a white noise test was made between each test run. As for the pseudostatic tests, was defined as that intensity for which the frequency of the walls did not change from one run to the next. A typical wall response time history is shown in Fig. 5.

In order to test the hysteresis model described earlier, the measured out-of-plane response was compared to that predicted by the model. The results of this comparison are shown in Fig. 6. The ability of the model to predict the wall response under earthquake loading is seen to be good.

In addition to the above, a new series of in-plane shear tests has been completed. A series ten walls, each four foot (1.22M) high and six feet (1.83M) long, two of which were uncoated, were tested under a slow reversing cyclic load. The brick walls were all the same. The coatings however were all different. For these tests, each wall was loaded separately using a test facility as shown in Fig. 7. No shake table tests or out-of-plane tests are planned for this size wall. As for the taller walls, the uncoated walls responded in a brittle fashion while the coated walls usually acted ductility by developing an extensive pattern of hair-line cracks along the wall diameter see Fig. 8. In all cases, the resistance of the coated wall was higher than that of the uncoated walls. In-as-much as the data for these tests is still being reduced, no precise figures can be given here.

A new hysteresis model that can be used to define the loops for these low aspect ratio walls has also been developed. This model has basically the same form as the earlier model. Differences include the way in which the calibration constraints are represented. These changes lead to a simpler computation to define the constants. This work is also currently underway. Results so far look quite promising.

In addition to the tall walls in the first test series, and the low walls in the second, a third series is anticipated involving six feet (1.83M) by six feet (1.83M) brick wall specimens. Sufficient walls have been built for in-plane and out-of-plane tests, both coated and uncoated. The in-plane tests will use the same test fixture that was used for the low walls. The out-of-plane tests will be conducted on the shake table using a sine input. When the data for these tests becomes available, a hysteresis model of the type described previously will be developed. It is anticipated that the model will be suitable for both directions of loading. As with the tall wall model, only the calibration constants will change.

DISCUSSION OF THE TEST RESULTS

The general characteristics discussed in this section consist of the hysteretic behavior including the ultimate strength and strength deterioration, stiffness degradation, energy dissipation, ductility, and general dynamic behavior such as frequency response and the variation of the clamping factor for both plane and coated walls. The only parameter considered was the effect of the ferrocement overlay while other parameters such as overburden load and mortar strengths were held constant. The discussion here relates to the diagonal split tests and the tall wall tests. While the details have not yet been completely defined for the low wall tests, observations indicate behavior that is similar to that of the tall walls.

The in-plane strength is improved by about three times when the brick wall specimens are coated by the ferrocement overlay. Considerable improvement in the flexural strength of the coated walls was also indicated. Figures 9 and 10 show the hysteresis envelopes for the cyclic loading tests of the plain and the coated walls when subjected to both in-plane and out-of-plane loading. Each point shown in the hysteretic envelopes was obtained from the hysteresis loops by averaging the absolute extreme values of the six peaks of the load cycles and the corresponding absolute values of the relative lateral displacement for each test stage. It can be seen that the ultimate strength of the masonry wall was upgraded by three to four times by the overlay.

The hysteretic stiffness was defined as the slope of the hysteretic envelope. It is clear that not only was the initial stiffness of the coated wall specimens improved, but the degradation of stiffness was reduced during each incremental loading step. The improvement in the initial stiffness of the coated wall specimen loaded out-of-plane under simulated earthquake loading was about two times.

Ductility, an important indicator of earthquake resistant ability, was obtained from the hysteretic envelopes and defined as the ratio of maximum displacement at the failure point to the displacement corresponding to the same load level as indicated by the envelope. Using this definition, the ductility for the walls in the out-of-plane test was found to be 6.70 and 9.50 for the plain and coated walls respectively, while for the in-plane test it was 3.50 and 4.00. It is evident that the ductility of the masonry walls was improved by the application of the ferrocement coatings.

The energy dissipation capacity is taken as the average area contained in the hysteresis loops for three cycles of repeated loading for each stage of the test. It can be seen in the hysteresis loops that in general, in the early loading stages, the energy disappointed in both the plain and the coated walls are very small since the response was in the elastic range. In later loading stages, significant dissipation of energy was found in the coated wall specimen, particularly for the out-of-plane case. The energy dissipation capacity of the unreinforced brick masonry wall was improved three to six times by the ferrocement overlay.

A banded white noise test with very small amplitude was performed after each level of simulated earthquake loading. The frequency was then taken directly from a spectrum analyzer. The frequency degradation curve for each specimen developed by plotting the frequency versus the load level. The natural frequency was found to decrease with an increase in loading intensity. The degradation rate of frequency for the coated wall specimens appeared to be slower than for the uncoated walls.

The damping factor for both the coated and plain walls was calculated (3) and plotted against the corresponding load intensity. Generally, the damping factor is increased corresponding to the increase in loading intensity. Higher damping was found in the coated walls, especially during the later load stages.

CONCLUSIONS

Brief conclusions that can be drawn from the results of these tests are as follows: (1) The mode of failure for both the coated and plain wall specimens subjected to either in-plane or out-of-plane loading was flexural. (2) The original stiffness of coated masonry walls was increased up to two times as much as that for the uncoated walls. (3) The shear strength is increased about 1.5-2 times, and the flexural strength of the masonry walls in terms of moment capacity is increased about three times by the ferrocement reinforcement. (4) The energy dissipation capacity and the ductility are improved when coatings are applied. (5) The coated wall specimens become more stiff and have a shorter period. (6) The damping factor increased with load intensity. In the in-plane test, the coated walls had higher damping than the plain walls at the same load intensity. (7) The tests indicate that coatings of the type studied have good potential as a retrofit material for certain unreinforced brick walls.

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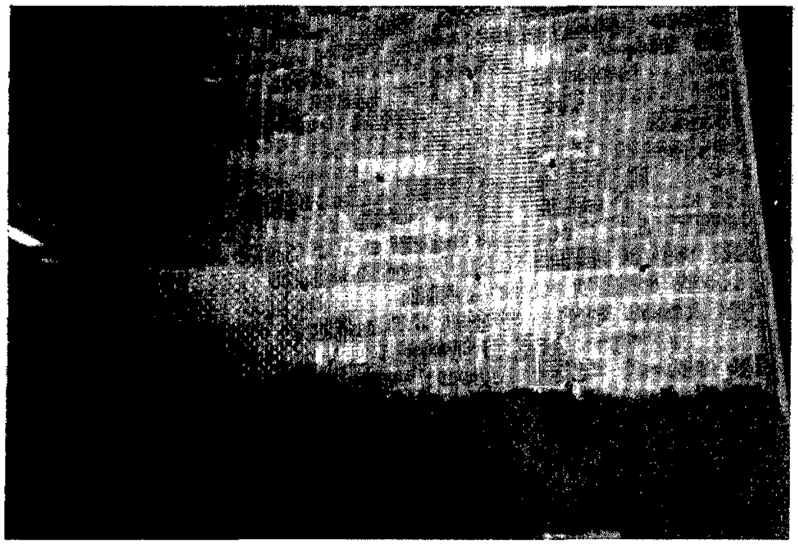


Fig. 1 - Coating Being Applied To Brick Wall.

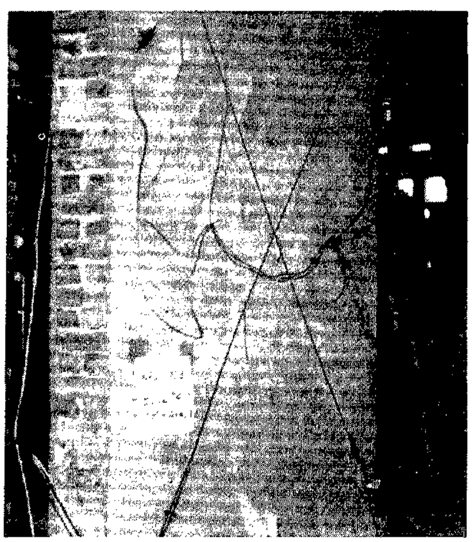


Fig. 2 - Coated Wall.

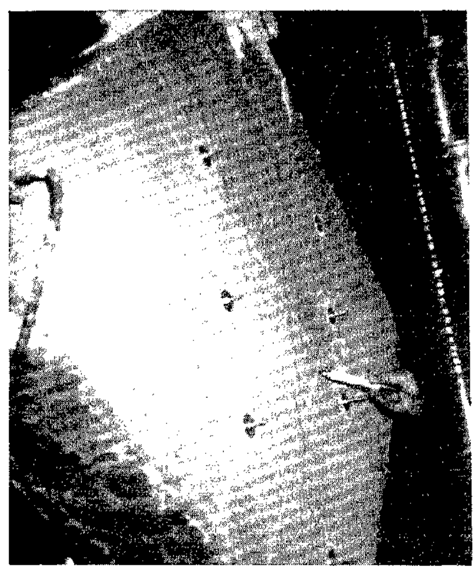


Fig. 3 - Hairline Cracks.

Fig. 4 - Test Frame For Tall Walls.

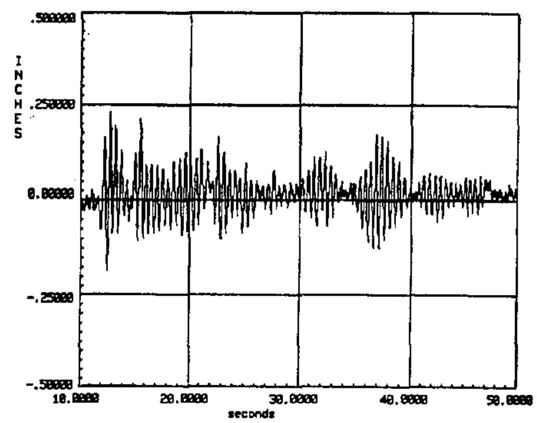


Figure (a) Experimental Results for OPW Walls-Test Phase 1

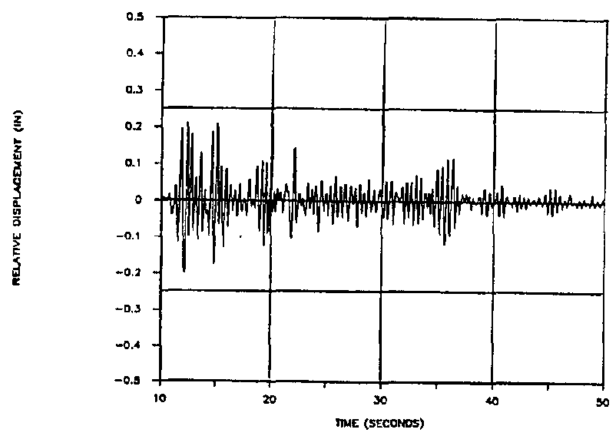
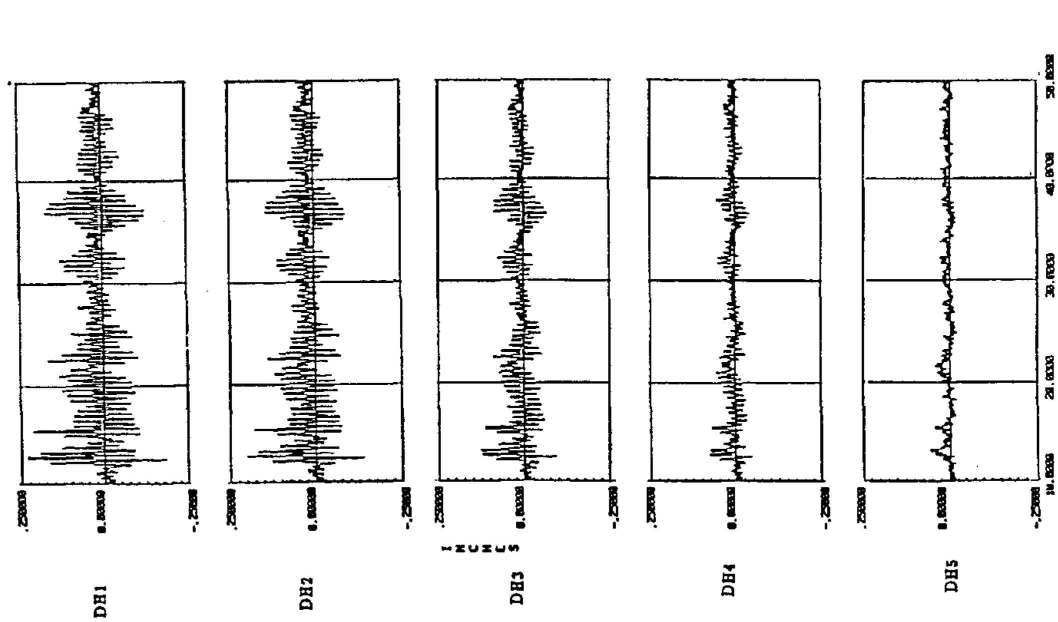


Figure (b) Analytical Results for OPW Walls-Test Phase 1

Fig. 6



Time Histories of Displacement Response for Plain Wall in Shake Table Test

Fig. 5

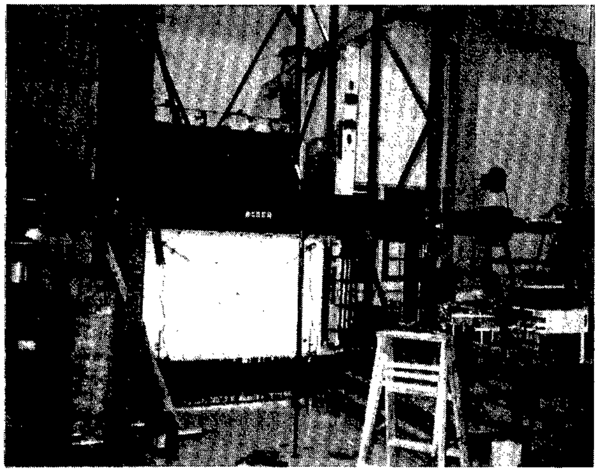
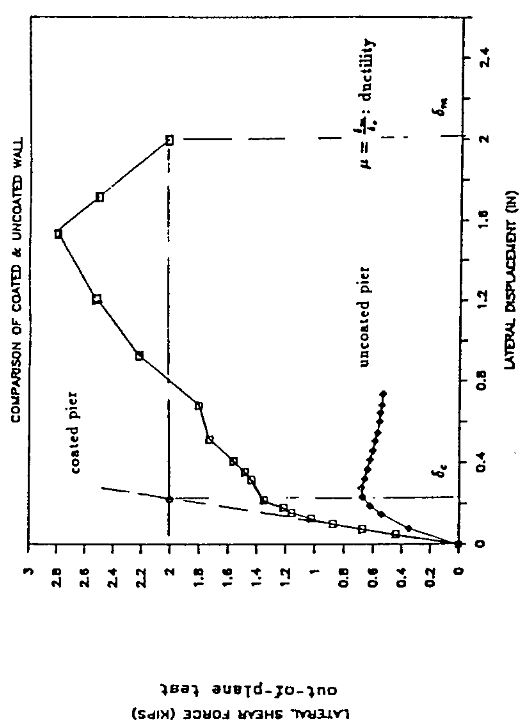


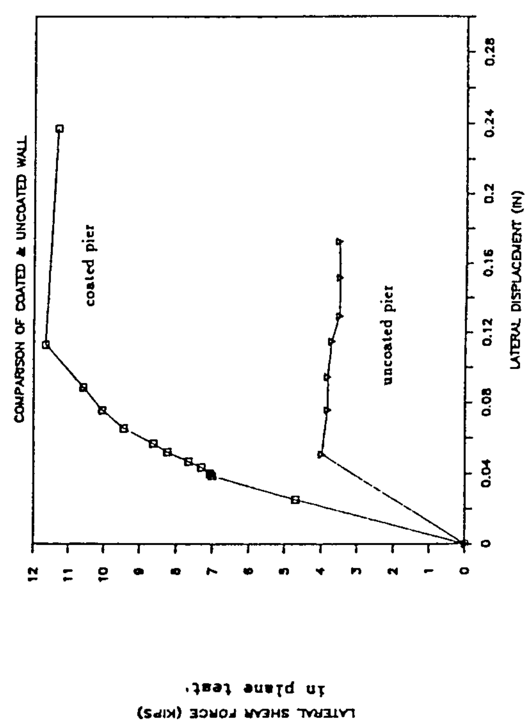
Fig. 7 - Low Wall Test Frame.

Fig. 8



Hysteresis Envelope for out-of plane test

Fig. 9



Hysteresis Envelope for in plane test

Fig. 10

DYNAMIC RESPONSE MEASUREMENTS FOR URM BUILDING SYSTEMS

Daniel P. Abrams[1] and Andrew C. Costley[2]

ABSTRACT

Results from an instrumented building and a laboratory study are presented that demonstrate the dynamic performance of typical unreinforced masonry bearing wall building systems with flexible floor diaphragms. The overall focus of the paper is to suggest ways that present engineering methods for seismic evaluation and rehabilitation can be improved through results of research on dynamic response of masonry building systems.

INTRODUCTION

Seismic performance of unreinforced masonry construction is generally considered to be poor. Most unreinforced brick masonry buildings in the eastern and midwestern United States were constructed at a time when the sole criterion was gravity loads. With a growing concern for earthquake safety across the United States, the seismic performance of many of these unreinforced masonry buildings is now being questioned. However, technologies used for the basis of evaluation methodologies are often borrowed from previous methods that have evolved for frame systems with rigid diaphragms. In lieu of such approaches, overly simplistic prescriptive approaches are used which can be conservative for some aspects of building performance while neglecting other aspects in total. Exceptions are the Appendix C of FEMA Report 178 on *Evaluation of Unreinforced Masonry Buildings* (7) and Appendix 1 of the 1991 UCBC on *Seismic Strengthening Provisions for Unreinforced Masonry Bearing Wall Buildings* (8). However, procedures set forth in these documents have not been substantiated with experimental data from tests of complete building systems. Evaluation and rehabilitation technologies can be improved if knowledge of nonlinear dynamic response mechanisms for the overall wall-diaphragm system can be refined.

The research presented in this paper helps to improve the understanding of how unreinforced masonry building systems with flexible diaphragms respond to light, moderate and intense earthquake motions.

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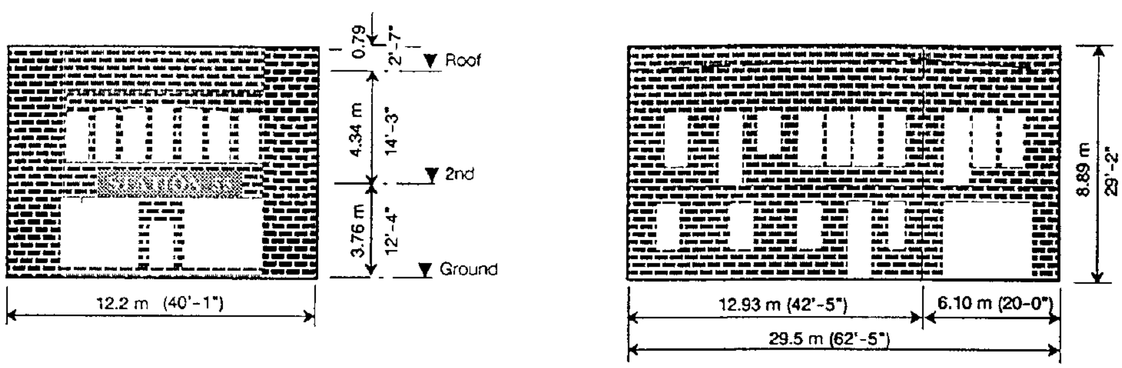
INSTRUMENTED BUILDING IN LOMA PRIETA EARTHQUAKE

A first look at true dynamic response for an unreinforced masonry building with flexible floor diaphragms was seen soon after the 1989 Loma Prieta Earthquake. A historic brick firehouse (Fig. 1) built before the turn of the century had been instrumented with three accelerometers at its base and three instruments at the roof level. The two-story structure withstood the 1906 San Francisco earthquake with little or no damage. An extensive analytical study was done to understand why the building behaved as it did, and to study the accuracy of various analytical models for computing



south elevation

east elevation

Fig. 1 Instrumented Firehouse at Gilroy, California

dynamic response of URM bearing wall buildings.

The roof and floor of the firehouse were constructed of timber joists and sheathing which were tied to the brick walls with steel anchors. The maximum diaphragm span was 42 feet (12.8 m) in the north-south direction. The building system was comprised of perforated walls on the south and east elevations, and nearly solid walls on the west and north elevations. In addition, a nearly solid interior wall ran in the east-west direction. The ratio of masonry wall area to floor area was approximately 10%.

The peak ground acceleration in the east-west direction was measured at 0.29g. The measured data suggest that the in-plane walls and the soil-foundation system had some flexibility since wall accelerations at the top of the wall were 1.41 that of the ground. Also, the span of the roof diaphragm could be considered flexible since its acceleration was 2.72 times that of the ground, and 1.93 times that at the top of the wall.

A linear elastic finite-element model was used to determine the mode shapes and frequencies. The first mode shape (Fig. 2) illustrates the flexibility of the floor and roof diaphragms in the east-west direction, and helps explain why the diaphragms amplified the wall accelerations so much. Response estimates based on equivalent static forces, spectral response, and time-step integration were compared to investigate their relative accuracies for determining normal and shear stresses. All three methods in addition to a simple hand calculation showed that peak shear stresses were less than permissible values based on the in-place shear test. This finding was consistent with the observation that very little cracking was seen.

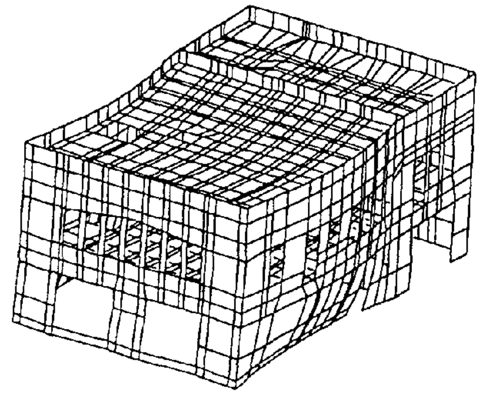


Fig. 2 Computed First-Mode Shape

More information on the firehouse, and on analytical studies to study its seismic response can be found in (5).

DESCRIPTIONS OF SHAKING TABLE EXPERIMENTS

Whereas there is nothing more representative than measured response of an actual building during an actual earthquake, the Gilroy study was limited because the amplitude of shaking did not result in cracking of the masonry. Therefore, a subsequent laboratory study was done to investigate the interaction between flexible diaphragm response and progressive damage to the masonry. The test structure was not intended to replicate the firehouse for that would have been overly detailed for the scope of the project. Instead, an attempt was made to model the essential response characteristics for a flexible diaphragm system with stiff in-plane, perforated unreinforced brick masonry walls, and flexible out-of-plane walls. In particular, it was of interest to examine dynamic response of a two-story building system which had floor and roof diaphragms with fundamental frequencies approximately equal to one third of that for a similar building system with rigid diaphragms.

Specimen Design

A photograph of the test structure is shown in Fig. 3. Dimensions of the story heights, wall lengths, pier heights and pier widths (Fig. 4) were equal to 3/8ths of those for a full-scale test structure tested statically at the University of Pavia (3). One in-plane shear wall had three window openings per story and was connected to transverse walls at both ends. The height-to-length aspect ratio was 1.34 for the two interior piers and 1.89 for the two exterior piers. The other in-plane shear wall had two door openings at the first story and two window openings at the second story of the same width. A vertical expansion joint was placed at each end of this wall to eliminate any flange effects. The height-to-length aspect ratio was 1.18 for the interior pier and 1.85 for the two exterior piers at the first story.

Model bricks were cut from full-size molded pavers, and laid in running bond with headers at every sixth course. Nominal brick compressive strength was 7200 psi (50 MPa). The mortar was designed to have a low strength so that masonry shear strength would be low. A Type O mortar was used (1:2:9 parts of cement, lime and sand). The ratio of mortar joint thickness to brick height was modeled according to typical American construction. Prism compressive strength determined from pre-construction test samples was 1900 psi (13 MPa). Diagonal compression tests of square sample panels revealed a low value of shear strength equal to 46 psi (0.32 MPa) which was comparable to the masonry shear strength for the Italian counterpart test structure which was 50 psi (0.35 MPa).

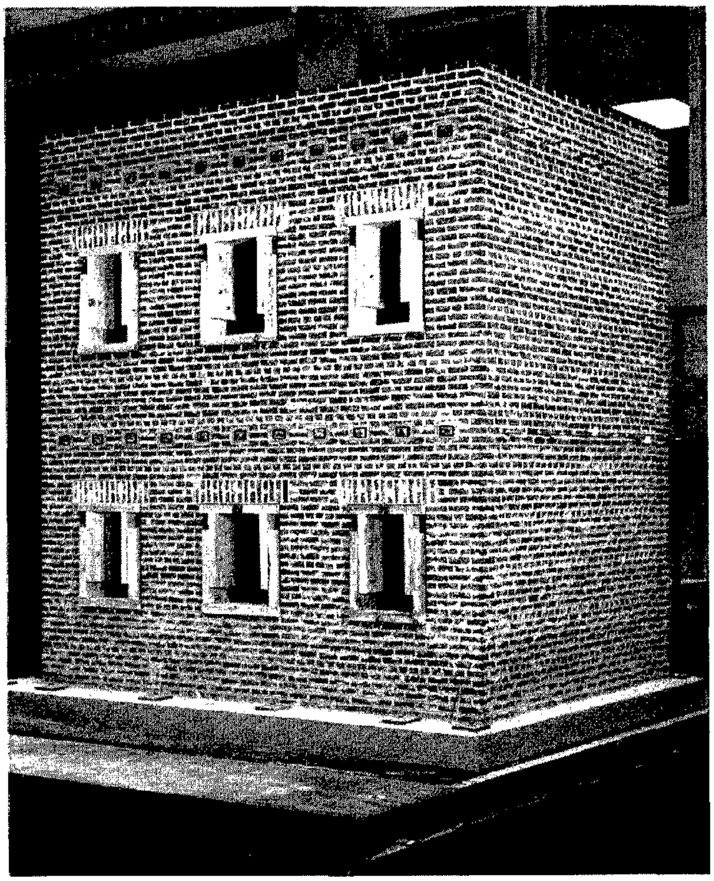
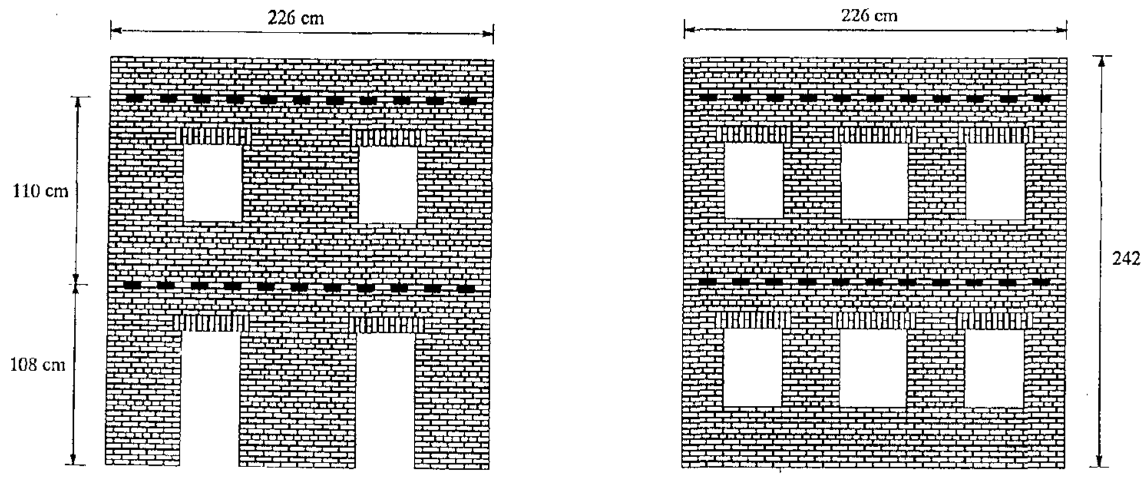


Fig. 3 Test Structure on Shaking Table

Steel plates with a total weight of 5000 lbs. (22kN) were added to each story level so that inertial forces would be large enough to cause damage at base accelerations within the operable range of the earthquake simulator. With the added masses at each story, 40% of the total mass was in the walls, and 60% was placed across the diaphragms.

The objective of the diaphragm design was to support both gravity and seismic loads with a floor system whose horizontal frequency was in the range of one-third that of the rigid-diaphragm system. A finite-element analysis showed that the fundamental translational frequency of the building system with rigid diaphragms was approximately 30 Hertz. Thus, the target frequency for the diaphragm system was 10 Hertz. The measured frequency of the floor system when vibrating in the horizontal plane was equal to 9 Hertz.

The floor and roof diaphragms were constructed using identical arrangements of steel beams that spanned from shear wall to shear wall. A rectangular beam section was aligned so that major axis bending would be congruent with bending within the horizontal plane. Horizontal inertial forces at the ends of the beams were reacted through a specially designed connection that permitted unrestrained rotation in the horizontal plane. More information on the specimen design can be found in (4).



north elevation (door wall) south elevation (window wall)

Fig. 4 Dimensions of Shaking Table Structure

Base Motions

The model earthquake which was input to the earthquake simulator was the motion measured at the Battlement Creek site in Nahanni of the Northwest Territories of Canada on December 23, 1985. The record was selected because its large magnitude, shallow depth, high frequency content and intraplate epicenter were similar to characteristics for earthquakes in the eastern United States. The duration of the recorded earthquake motions was compressed by a factor of 1.6 so that the frequencies of the test structure would lie within similar ranges of spectral response curves as for actual buildings subjected to actual earthquakes. The scaling factor was also equal to the square root of the length scale factor so that accelerations of the reduced scale test structure would be consistent with those of the full-scale prototype.

The structure was subjected to a total of 12 test runs. Amplitudes of the base motions were progressively increased so that response of the test structure could be observed (a) in the pre-cracking range, (b) in the post-cracking range and (c) at the ultimate limit state. No cracking consequential to overall behavior was observed for the first ten runs.

Instrumentation

Accelerometers were placed at the base of the structure, on the diaphragm mass and on each shear wall at each story level, and on the transverse walls. Displacement transducers recorded the lateral deflection of the building system with respect to its base. Strain gages on the steel floor beams provided a redundant measurement of inertial force.

Histories of horizontal inertial force applied to each shear wall were determined by mulitplying one-half of the floor mass times the recorded diaphragm acceleration, and adding it with the product of the tributary wall mass times the wall acceleration. Forces were summed about the base to give histories of base shear and overturning moment.

RESULTS FROM SHAKING TABLE EXPERIMENTS

Sample test results are presented below to substantiate conclusions deduced from the shaking table experiments.

Amplification of Base Accelerations

One object of the shaking table test was to confirm if the large diaphragm amplification ratios observed with the Gilroy firehouse could be replicated with another flexible diaphragm structure with a similar ratio of diaphragm-to-system frequency (in this case one to three). The ratios of peak diaphragm acceleration to peak ground and wall accelerations are shown in Fig. 5 for the last five test runs of the shaking table structure.

For the test structure, the roof diaphragm accelerated close to a maximum of 3.5 times the peak base acceleration. A slight increase in amplification factor was observed as the earthquake motion grew more intense from Run 8 to Run 10. Once the structure started to crack (during Run 11), the overall amplification factor dropped to a value as low as 1.5. This decrease was attributable to an increase in damping and a drop in frequency.

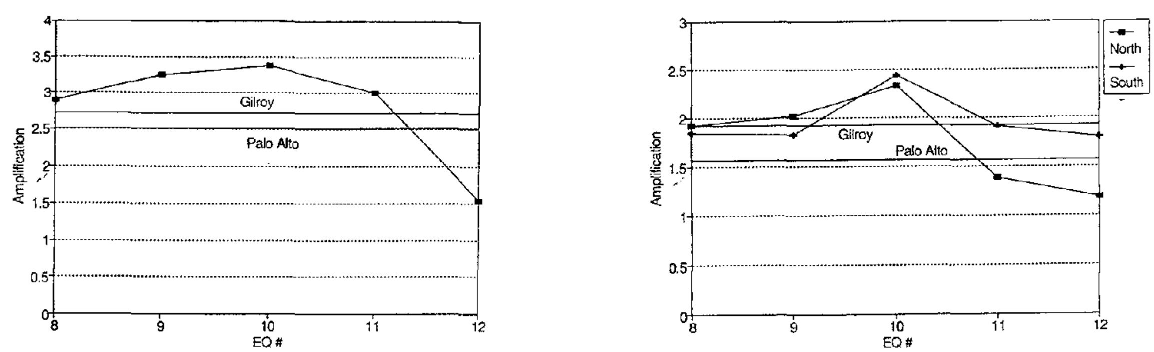


Fig. 5 Diaphragm Amplification of Base and Wall Motions

The amplification of wall accelerations was more constant across the test runs than the amplifiaction of base motions. This was because the steel diaphragm beams did not go beyond the elastic range. When the north wall (with the door openings) cracked, the wall frequency dropped and caused a slight reduction in the wall-to-diaphragm amplification ratio.

Superimposed on the plots are the amplification factors measured from the Gilroy firehouse as well as another building in Palo Alto, California which had also been instrumented during the Loma Prieta Earthquake, and had a flexible diaphragm and masonry shear walls (6). The close correspondence in amplification ratios particularly for the early low-amplitude test runs demonstrates that the design parameters for the shaking table structure were appropriate.

Seismic Strength and Drift Levels

The overall force-deflection relation for the sequence of test runs is shown with the envelope relation of diaphragm acceleration vs. top-level wall deflection (Fig. 6). This is shown for both the north (with door openings) and the south (with window openings) shear walls. The deflection is divided by the height to the second level of 86 inches (2.18 meters) to give the lateral drift percentage.

Cracking was first observed during Run 11. Before this run, the forces and deflections were linear. The drift level associated with first cracking can be inferred from Fig. 6 to be slightly less than 0.1%. After cracking, the structure continued to resist additional force until a drift level of approximately 0.3%. During the last test run (Run 12) the structure still continued to resist 80% of its peak load at drift levels in the range of 0.6%.

Two aspects of the measured response that were not conceived prior to the experiment were the large lateral strength of the system and its capacity for inelastic deformation.

Preliminary estimates of strength were three times less than the measured cracking strength. Two reasons were found for the apparent overstrength of the test structure. One, the masonry tensile

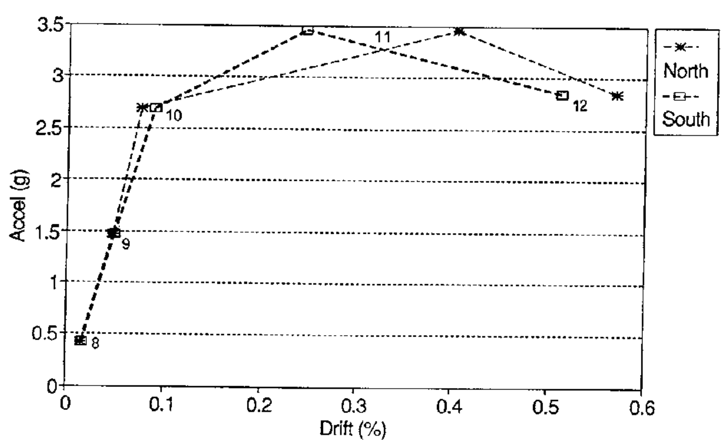


Fig. 6 Measured Top-Level Diaphragm Acceleration vs. Drift

strength was stronger than anticipated. A low value was assumed for the calculations, but the walls were able to resist flexural tensile stresses as high as 145 psi (1.0 MPa). Secondly, lateral forces were estimated by entering the spectral response curve with a frequency corresponding to the fundamental mode of the system (as is commonly done for rigid diaphragm systems). In hindsight, lateral forces applied to the walls should have been estimated by reading spectral response curves for the wall motions corresponding to the lower diaphragm frequency. This would have resulted in a smaller wall force for a given intensity of shaking.

Measured Force-Deflection Behavior

One common opinion among structural engineers in the United States is that unreinforced masonry walls loose their strength after cracking. This myth has been disputed by the first author on the basis of static tests of URM walls subjected to vertical compressive stress (1,2). The test data shown in Fig.6 provide additional evidence that an unreinforced masonry system can resist loads near its capacity at many times its cracking deflection. This is an important distinction in terms of the equivalent lateral forces for which a building system must be designed or evaluated.

Ductility for the unreinforced masonry system can be seen by the measured moment-deflection curves for each shear wall (Fig. 7). Moment has been deduced from the two diaphragm records and the two wall acceleration records. Displacement is measured at the top level (at a height of 86 in or 2.18 m) relative to the base. Data is shown for Run 11 which caused initial cracking. The acceleration and deflection records were filtered to include frequency components less than 15 Hertz (with a transition range between 15 and 20 Hertz) to eliminate wall vibrations and spurious high frequency measurements.

Again, the deformation capacity of the masonry walls was striking. The north wall (with door openings) was pushed to 0.4% drift with a 30% loss of peak strength. The maximum drift for the south wall (with window openings) was approximately one-half of this amount. Furthermore, the hysteresis loops were not pinched. Because of the vertical compressive stress, cracks closed before the lateral force was reversed, and thus no sudden change in the rate of deflection was observed. This implies that substantial energy was being dissipated through hysteresis.

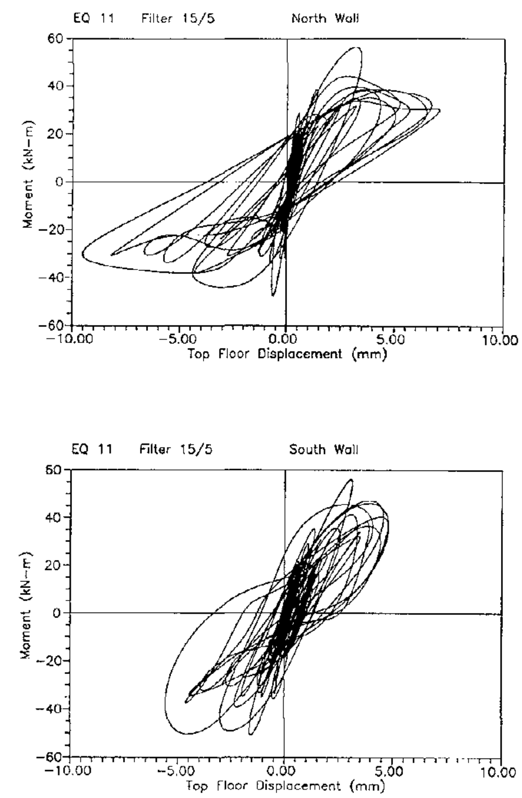


Fig. 7 Measured Moment-Deflection Relations

Pier Rocking Mechanisms

During Run 11, horizontal bed-joint cracks were observed at the top and bottom of the three piers adjacent to the door openings at the first story of the north wall. Rocking of these piers about their toes was clearly observed. The shape of the hysteresis envelope for the north wall in Fig. 7 is characteristic of rocking behavior for unreinforced masonry piers. After cracking, the ultimate capacity agrees well with a simple hand calculation of rocking strength. Pier rocking capacity was assumed as the simple product of pier gravity loads times the width of each pier divided by the height of each pier. Lateral strength of the base story was assumed to be equal to the summation of pier rocking strengths. The gravity load was assumed to act at the edge of the pier where contact is made. Axial forces due to overturning moments were neglected when determining rocking capacity of the story because of symmetry.

Lateral Deflected Shapes

Measured deflected shapes are shown in Fig. 8 for the north wall at times of peak top-level deflection for Runs 10, 11 and 12. Data has been taken from deflection data that was filtered of frequency components larger than 15 Hertz (with a transition range between 15 and 20 Hertz). Similar deflected shapes were observed for the south wall. Rocking behavior at the base-story was prevalent on the displaced shapes for Runs 11 and 12 where a very large percentage of the deflection was observed to occur.

The measured displaced shape can be quantified in terms of the ratio of the first to second level deflections. A sample plot of this ratio versus time is shown in Fig. 9 for the north wall during Run 11. Low and high values should be disregarded because they occur at low displacement amplitudes. Still, the variation in the deflected shape during the shaking is significant.

For response of an elastic shear beam with equal story stiffnesses, the ratio of first to second level deflections ranges from 0.50 for a concentrated force at the top level to 0.67 if the lateral force is distributed equally between the first and second levels. The addition of global flexure with the story shear mechanism would lower these values since the displacement ratio for a flexure beam would be less than 0.50. For a first-story rocking mechanism and a rigid second story, the displacement ratio would be 1.0.

The predominant displacement ratios shown in Fig. 9 were in the range of 0.50 to 1.00 suggesting that the wall was behaving as an elastic system at low amplitudes and as a rocking system for larger amplitudes.

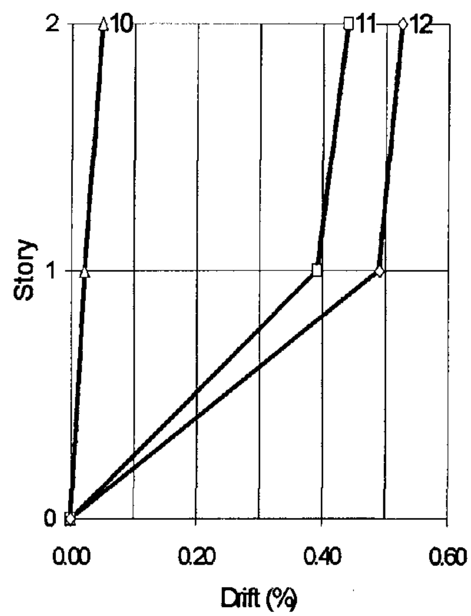


Fig. 8 Deflected Shapes for North Wall

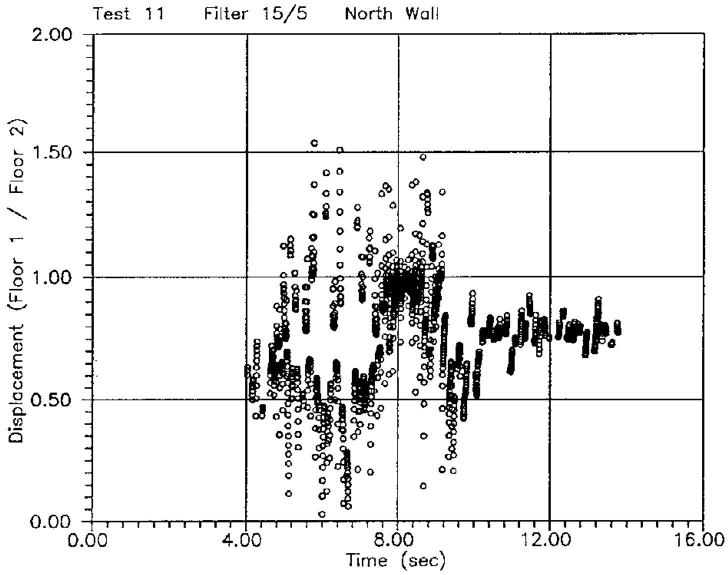


Fig. 9 Ratio of First to Second Level Deflection vs. Time - Run 11

Lateral Force Distributions

The distribution of horizontal inertial forces along the height did not remain constant throughout the duration of an earthquake simulation, nor did it approach the common inverted triangular distribution that is often assumed for rigid diaphragm systems. As viewed on a computer animation, lateral forces did approach a uniform distribution, particularly near times of peak response. This was because the two diaphragms had almost identical stiffness and mass, and the two shear walls were much stiffer than the diaphragms. The two diaphragms were excited at the same time and remained in phase for most of the duration. An alternate verification is shown in Fig. 10 where the height of the resultant force is plotted versus time for the north wall during Run 11. If forces at the first and second levels are equal, the resultant is at the midheight of the second level, or at 75% of the total height of the structure. The measured data shown in Fig. 10 was close to this value.

Unlike the deflected shape which varied with the amplitude of motion (Fig. 9), the height of lateral force resultant was clustered about a narrow range that was nearly constant for all test runs. Inertial forces at the two levels from the elastic diaphragms remained similar despite the level of damage to the masonry walls.

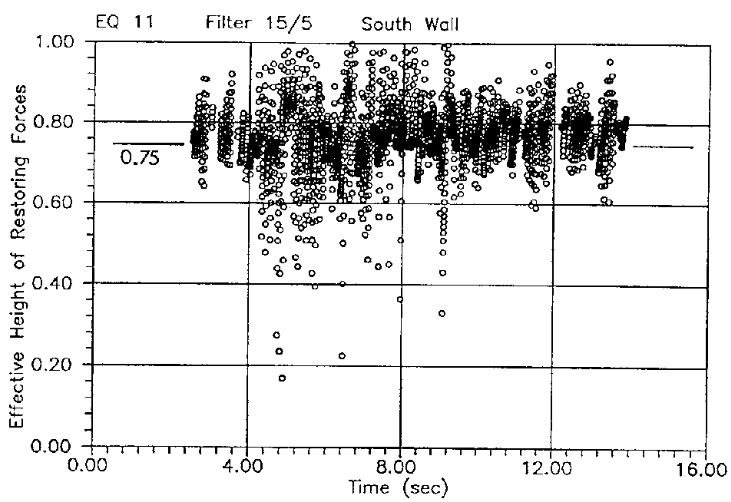


Fig. 10 Height of Lateral Force Centroid vs. Time - Run 11

If the masses or floor systems had not been identical at the two levels, the frequencies of the two diaphragms would have been different, and the lateral forces would not have been in synchronization. Thus, the lateral force distribution would have varied for every instant in time. However, the worst case scenario for base shear or base moment would be when the two forces would reach their peaks simultaneously. If each diaphragm had the same amplification of ground motions, then forces could be assigned to each level in proportion to the relative amounts of floor mass to the total mass.

Flexible Diaphragm Effects

The floor and roof diaphragms were designed so that the two shear walls would be truly decoupled. As a result, there was little or no twisting of the diaphragms in plan which implied no torsional forces. Each wall vibrated independently within its plane. Deflections, frequencies and phase angles were different for the two walls.

One common convention for flexible diaphragm systems is to assign seismic forces to individual shear walls on the basis of tributary mass. Because the stiffness of the diaphragm is assumed to be small, there will be no load sharing with the stiffer walls. However, the experiment revealed that inelastic action in one wall can limit the force attracted to the other wall. This was because the diaphragm acted as a simply supported beam between the two walls. When the base-story piers of the north wall started to rock, the north wall could not react any further lateral force. With the lost reaction of the north wall, no additional inertial loads could be developed and applied to the south wall. By definition, the peak lateral force applied to the north wall was the same as that applied to the south wall (note that the force axes in Fig. 7 are the same for both walls). Thus, because the strength of the south wall was slightly larger than that for the north wall, the south wall did not develop its full strength.

This observation implies a new method for seismic rehabilitation of existing buildings. Floor-to-wall connections can be retrofitted so that diaphragms span as simply supported beams between adjacent parallel walls. To limit the strength demands on one wall, the strength of the adjacent wall can be decreased. Weakening can be done by enlarging the height of window openings so that the height-to-width aspect ratio of piers is increased. The pier aspect ratio should be controlled so that the rocking strength will be less than the pier shear strength. If the piers carry gravity forces, they will rock in a ductile mode. Following an earthquake, horizontal bed joint cracks caused by rocking will close under gravity stress and little or no damage will be detected. In this case, seismic rehabilitation does not infer strengthening but weakening.

SUMMARY AND CONCLUSIONS

The shaking table study reported herein is still in progress. However, the following general conclusions have emerged upon an initial study of the measured data.

a. Amplification ratios for the shaking table structure were similar to those observed with an actual building in a seismic event. In-plane masonry shear walls can amplify ground motions by as much as 2.0. Flexible roof diaphragms can amplify ground motions by as much as 3.5.

b. The building system for the shaking-table test was much stronger than was anticipated.

c. Lateral forces attracted to shear walls are a direct result of the frequencies of the diaphragm and not the overall frequencies of the system. Spectral response of the diaphragm should be determined to estimate lateral forces to be resisted by in-plane shear walls.

d. An URM building system can be ductile if shear walls support significant vertical compressive force.

e. Pier rocking was the prevalent mechanism for one in-plane wall with door openings. The maximum story shear was easily estimated by summing static rocking capacities of each pier.

f. Lateral deflection of the perforated in-plane shear walls was primarily a result of pier distortion. Measured deflected shapes could be estimated with a simple story-shear model that neglected global flexural behavior.

g. Because the mass and flexibility of the roof and floor diaphragms were nearly equal, and the stiffness of the shear walls was large relative to the diaphragm stiffness, maximum amplitudes of lateral force at the first and second levels were nearly the same, and many times in synchronization with each other.

h. No torsion or transverse coupling between the two parallel shear walls was observed.

i. Because the diaphragm forces were shared equally by the two parallel walls, the lateral force attracted to the stronger wall was limited to the strength of the weaker wall.

Further research is being done to correlate measured response with estimates based on numerical models.

ACKNOWLEDGMENTS

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EXPERIMENTAL RESEARCH ON RESPONSE OF URM BUILDING SYSTEMS

G. M. Calvi[1] and G. Magenes[2]

ABSTRACT

Results from a laboratory study on the seismic response of urm structures are presented. Shear-compression tests on walls were performed first, followed by tests on the same walls damaged and repaired. Finally a test on a full scale building prototype has been performed, that will then be repaired, strengthened and re-tested. Some preliminary comments on the comparison with the dynamic response of a similar model (scaled to 3/8) are also presented. The paper tries to correlate several different experimental and numerical experiences to give a overall picture of the impact of laboratory studies on the understanding of seismic response of urm buildings.

INTRODUCTION

The evaluation of the seismic vulnerability of existing buildings has been recognized as a major problem either because of the large number of buildings constructed before the development of rational seismic codes and because of the lack of knowledge about the properties of the materials used and about the seismic response of single elements and whole complex structural systems. This is particularly emphasized for the Italian historical centers, where the great majority of the buildings were constructed with heterogeneous masonry materials whose properties are almost totally unknown apart from the considerable mass density.

The interactions between different structural elements are in general also unknown and complex: consider as significant examples the interaction between perpendicular walls whose connections are not clear, or the interaction between masonry walls and timber floors.

Effective models for the numerical simulations of such buildings should therefore be more refined in the constitutive relations of the structural elements and of their connections rather than be able to represent the exact geometrical configuration.

Also, it has to be noted that these types of buildings usually show a rather low initial fundamental period of vibration and a highly non linear response even at low level of excitation, dissipating energy both by friction and by hysteresis. It is therefore fundamental to investigate not only the strength of the structural elements, but also the energy dissipation capacity associated with all the possible damage or failure mechanisms, in order to be able to define rational force reduction factors

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to be used in simplified evaluation processes.

The research presented in this paper summarizes the research carried out in the recent years at the University of Pavia on urm systems and contributes to understand the seismic response of urm buildings with flexible floor diaphragms, discussing the kind of knowledge that can be learned from different kinds of tests and numerical analyses.

SHEAR-COMPRESSION TESTS ON URM WALLS

Materials, geometry and testing procedures

Fired clay solid bricks and lime mortar were used to prepare specimens that should have been able to match the properties of ancient materials.
A number of tests were performed on bricks, mortar and small masonry assemblies. The mortar was prepared with hydraulic lime and sand, in volumetric ratio 1:3. The main results of these tests are presented in table 1. The wallettes tested in compression were 250 mm (thickness, two unit breadth) x 765 mm x 680 mm (height).
Particular care was used in testing brick triplets to evaluate the apparent cohesion and friction coefficient between bricks and mortar. The best estimate for the relation between shear strength and normal stress is given by equation (1), derived from a linear regression on the experimental data (5):

$$\tau = 0.206 + 0.813\,\sigma \qquad (1)$$

These results confirmed that the mechanical properties of the masonry were similar to those of a very good quality old masonry.
A total of five full scale specimens were then prepared, all being 1500 mm wide and 380 mm thick (three wythes), with a height of 2 m (three walls) and 3 m (two walls). The purpose was to perform one preliminary monotonic test, and to explore two aspect ratio values and two nominal vertical compression stress values (0.4 and 1.2 MPa). The reasons for these choices were discussed by the same authors in ref. (3). The shear-compression tests were performed in a double bending condition (rotations at the top and at the bottom were restrained) (5). The vertical force was applied first and the horizontal cyclic load was then applied keeping the valves of the vertical jacks closed. As a consequence of the testing procedure the vertical load increased together with the horizontal load, depending upon the stiffness degradation of the wall. The axial load increment was percentually more significant for a lower nominal value, but the forces in the jacks were obviously monitored and acquired with continuity.

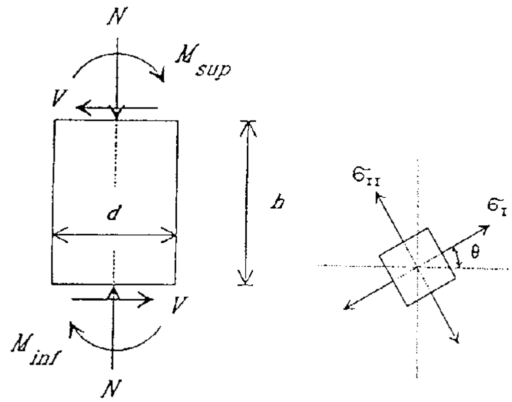
Experimental results

The results of the tests are summarized in table 2 and in figure 1, some comments are presented in what follows.

Table 1. Tests on the materials used for the shear-compression tests.

test type	n. of specimens	mean (MPa)	c.o.v.
compression on brick f_b	28	19.72	8.82%
splitting on brick f_{mt}	29	1.26	20.28%
compression on mortar f_m	15	4.33	1.84%
splitting on mortar f_{mt}	14	0.66	8.44%
mod. of rupt. on mortar f'_{mt}	15	1.59	3.47%
direct tension on mortar joint f_{jt}	13	0.073	10.53%
compression on masonry f_u	5	7.92	20.2%
E	4	2991	15.1%

Table 2. Actions at first shear cracking and stresses at the center of the panels according to f.e. analyses (5,6)



	MI1m ; MI1	MI2	MI3	MI4
h (m)	2.0	2.0	3.0	3.0
h/d	1.33	1.33	2.0	2.0
N (kN)	580 ; 639	386	705	393
V_t (kN)	259 ; 259	227	185	153
M_t^{inf} (kNm)	185 ; 228	223	228	219
M_t^{sup} (kNm)	335 ; 290	231	327	240
σ_x (MPa)	-1.023 ; -1.123	-.668	-1.245	.691
σ_z (Mpa)	-.066 ; -.05	-.111	-.002	-.017
σ_{xz} (MPa)	-.695 ; -.679	-.602	-.486	-.402
$\sigma_I = f_{tu}$ (MPa)	.287 ; .280	.274	.165	.163
σ_{II} (MPa)	-1.377 ; -1.452	-1.052	-1.413	-0.881
θ (gradi)	-27.7 ; -25.8	-32.6	-19.0	-25.0
$X = \sigma_z/\sigma_x$	.065 ; .045	.165	.002	.024

Wall MI1m and MI1, h = 2 m, σ_m = 1.2 MPa.

Wall MI1m was tested monotonically, wall MI1 cyclically. The maximum horizontal load corresponded to the first diagonal crack, and decreased rapidly to a lower value function of the friction in the mortar beds. The failure mode concerned mainly the mortar beds, with slight damage in the bricks.

Wall MI2, h = 2 m, σ_m = 0.4 MPa.

The first failure was due to a shear sliding mechanism located at the top mortar layer, with an apparent friction coefficient between 0.57 and 0.65. Thanks to the axial load increment the horizontal load also increased up to the formation of diagonal cracks. The post peak behaviour was similar to case MI1.

Wall MI3, h = 3 m, σ_m = 1.2 MPa.

The failure mode involved sub-vertical cracks started in the central area of the panel, with extensive brick damage. The cracks extended slowly, cycle after cycle, with a correspondent strength deterioration.

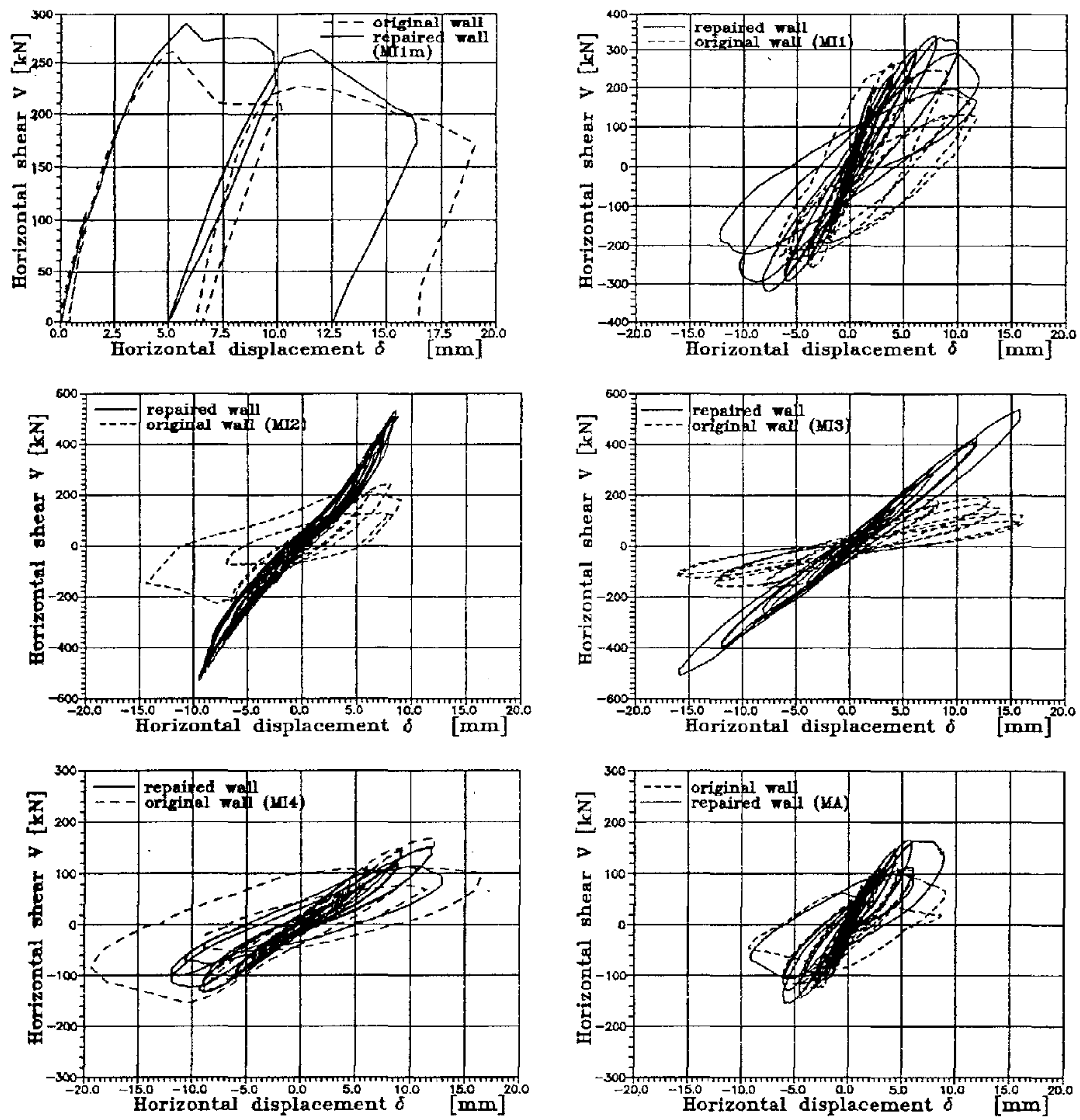


Fig. 1. Results of the shear-compression tests on brick masonry walls. Dashed line: original walls; solid line: repaired walls.

Wall MI4, h = 3 m, σ_m = 0.4 MPa.
The mortar joints collapsed, allowing the formation of two wide diagonal cracks. The increment of the vertical load was is this case critical to avoid a flexural failure.

In table 2 it can be seen how the apparent tensile strength f_{tu} of the different masonry panel is affected by a considerable variability, and seems to assume two roughly constant values, associated with the two different aspect ratios. It has to be noticed that this seems to be true even for cases in which the orientation of the principal stress is approximately the same. This result suggests that the models based on the tensile strength of masonry assumed as a constant parameter should be used with care when the masonry is strongly orthotropic as in this case.

Further experimental tests on full scale masonry walls were then performed using a PSD technique at the JRC of the CEC in Ispra and on a shaking table at the LNEC in Lisbon, and the results are still to be published.
The geometry of the walls was similar to those tested previously, except for the fact that all dimensions were scaled to 2/3, having therefore a thickness of 250 mm (two unit breadth instead of three).

SHEAR-COMPRESSION TESTS ON URM WALLS DAMAGED AND REPAIRED

Repairing techniques

Two commercially available materials were selected to repair the walls described above, to be used as a function of crack width and distribution (4). A cementitious grout specifically oriented to applications in historical masonry was injected into walls MI1m, MI1, MI4 and MA (an additional wall built with common lime mortar and tested with the same procedure as the other walls), where the main cracks were generally larger than 10 mm, while a two-component epoxy resin was injected into walls MI2 and MI3, where the majority of the cracks were thinner than 1 mm.
In addition to an appropriate choice of the material properties, a major element determining the effectiveness of the repairs through injections is the procedure followed during injection. Skilled labour is therefore required. For the walls repaired with cementitious grout the following procedure was followed:

- core-drilling (bit 20 mm dia.) of the holes for injection; in the case of diagonal cracks the holes were drilled only in proximity of the main cracks;
- washing of cracks and holes with water, to remove the debris and to give a preliminary imbibition to the masonry;
- positioning of small plastic tubes (internal dia. 10 mm) in the holes;
- application of fine plastic nets with a gypsum plaster to the surface of the walls to avoid grout spills;
- injection of water (soak of the bricks), from the bottom to the top of the walls, followed by the injection of a "substrate conditioner" fluid, aiming to restrain the water absorption from the grout to the bricks;
- injection of grout, starting from the bottom to the top of the wall, with injection pressure lower than 0.5 atm;
- after curing of the grout, removal of the gypsum plaster.

The procedure for the injection of epoxy resin was the following:

- core-drilling (bit 10 mm dia.) of the holes for injection;
- washing of cracks and holes with water;

- positioning of small plastic tubes (internal dia. 6 mm) in the holes;
- after drying of the masonry, coating of the wall with plastic nets and gypsum plaster;
- injection of resin, from the bottom to the top of the walls, with the aid of a mixer with pressure regulator (pressure lower than 0.5 atm);
- after curing of the resin, removal of the gypsum plaster.

The quantity of injected material was between 1.6 and 4.1 % of the volume of the walls in the case of cementitious material, and between 5.5 and 7.7 % of the volume of the wall in the case of resin . The repaired walls, after curing of the injected materials, were subjected to a shear and compression test, according to the procedures followed in the original undamaged condition.

Experimental results

In all the tests on walls repaired with cementitious grout (MI1m, MI1, MI4, MA) the cracks due to shear developed at a different location than the repaired cracks; in wall MI4 however the failure was initiated by the propagation of a thin existing crack, which was not filled by the grout. The crack was not visible before the application of any load. The experimental plots of fig. 1 and table 3 show how the original shear strength (expressed in terms of f_{tu}) has been restored practically in all walls: all specimens showed a slight increase in strength, with the exception of wall MI4, where the slightly lower strength can be attributed to the mentioned pre-existing crack.

The walls repaired with epoxy resin (MI2, MI3) increased the strength to such a level that the tests had to be stopped because the limits of the reaction system for the horizontal force had been reached, without any sign of cracking in the repaired zone at the center of the panels. In both walls MI2 and MI3 the maximum horizontal load reached in the test was higher than 500 kN. The numbers shown in table 4 are therefore based on the maximum shear loads V_t (and corresponding N) reached in the tests. It should be reported that, when the tests had to be stopped, some signs of crushing in compression of the masonry were detected at the top and bottom corners of the walls.
The increase in shear strength was much higher than expected: it is known in fact that the shear strength in masonry is generally limited by the strength of the joints and the tensile strength of the bricks. The dramatic increase in strength can be justified by considering that the high injectability of the resin has allowed its penetration in extremely small cracks and voids, creating an internal "lattice" structure of high strength. This structure was able to absorb the tensile stresses induced by the horizontal shear. This hypothesis is supported by the high amount of resin which had to be injected, and will be verified by cutting and inspecting the interior of the walls.

The apparent stiffness of the repaired walls was not significantly different from that of the virgin walls, varying between 80 and 120 % of the initial value. Where resin was used the repaired walls tended to behave linearly, therefore showing a significantly higher stiffness at high load levels.

Table 3. Walls repaired with cementitious grout: calculated strength of the original walls (f) and of the repaired walls (f'). Subscripts *1,2* correspond to the strength calculated in opposite directions (cyclic tests), subscript *m* to the mean of the two values.

wall	h [m]	$f_{tu,1}$ [MPa]	$f_{tu,2}$ [MPa]	$f_{tu,m}$ [MPa]	$f'_{tu,1}$ [MPa]	$f'_{tu,2}$ [MPa]	$f'_{tu,m}$ [MPa]	$f'_{tu,m}/f_{tu,m}$
MI1m	2	.284	-	.284	.317	-	.317	1.12
MI1	2	.269	.259	.264	.358	.292	.325	1.23
MI4	3	.208	.185	.196	.178	.140	.159	0.81
MA	2	.140	.129	.134	.198	.171	.184	1.37

Table 4. Walls repaired with epoxy: calculated strength of the original walls (f) and of the repaired walls (f'). Subscripts *1,2* correspond to the strength calculated in opposite directions (cyclic tests), subscript *m* to the mean of the two values.

wall	h [m]	$f_{tu,1}$ [MPa]	$f_{tu,2}$ [MPa]	$f_{tu,m}$ [MPa]	$f'_{tu,1}$ [MPa]	$f'_{tu,2}$ [MPa]	$f'_{tu,m}$ [MPa]	$f'_{tu,m}/f_{tu,m}$
MI2	2	.323	.293	.308	.670	.652	.661	2.15
MI3	3	.169	.146	.157	.631	.598	.614	3.91

LABORATORY TEST ON A FULL SCALE URM BUILDING PROTOTYPE

A test on a full scale two storey building was designed and is in progress at present. In this test the seismic forces are simulated by statically applied horizontal forces. The criterion followed for the application of the seismic actions was derived from a shake table test performed at the University of Illinois at Urbana-Champaign, U.S.A., described in (1), and will be discussed in the following sections.

Description of the building prototype

The building to be tested has been built with two-wythe solid brick walls, arranged in English bond, with a total wall thickness of 250 mm. The plan dimensions are 6 x 4.4 m and the height is 6.4 m, with non-symmetric openings as shown in figure 2; a disconnection between one of the longitudinal walls (wall D or "door wall", parallel to the direction of loading) and the two transverse walls (walls A and C) are also shown. All the walls are glued to the laboratory strong floor, so that any crack or slip will take place in the structural walls and not at the interface between walls and foundation. Fourteen flat jacks have been embedded at the base of the walls to allow a measure of the variation in vertical stresses due to the application of the horizontal displacements simulating the seismic action.

The floors consist of a series of isolated steel beams (I section, h = 140 mm), which ensure a sufficient strength for vertical loads and a light coupling between longitudinal walls, as typical of flexible floors. The beams are directly embedded into the masonry walls, without any concrete tie-beam.

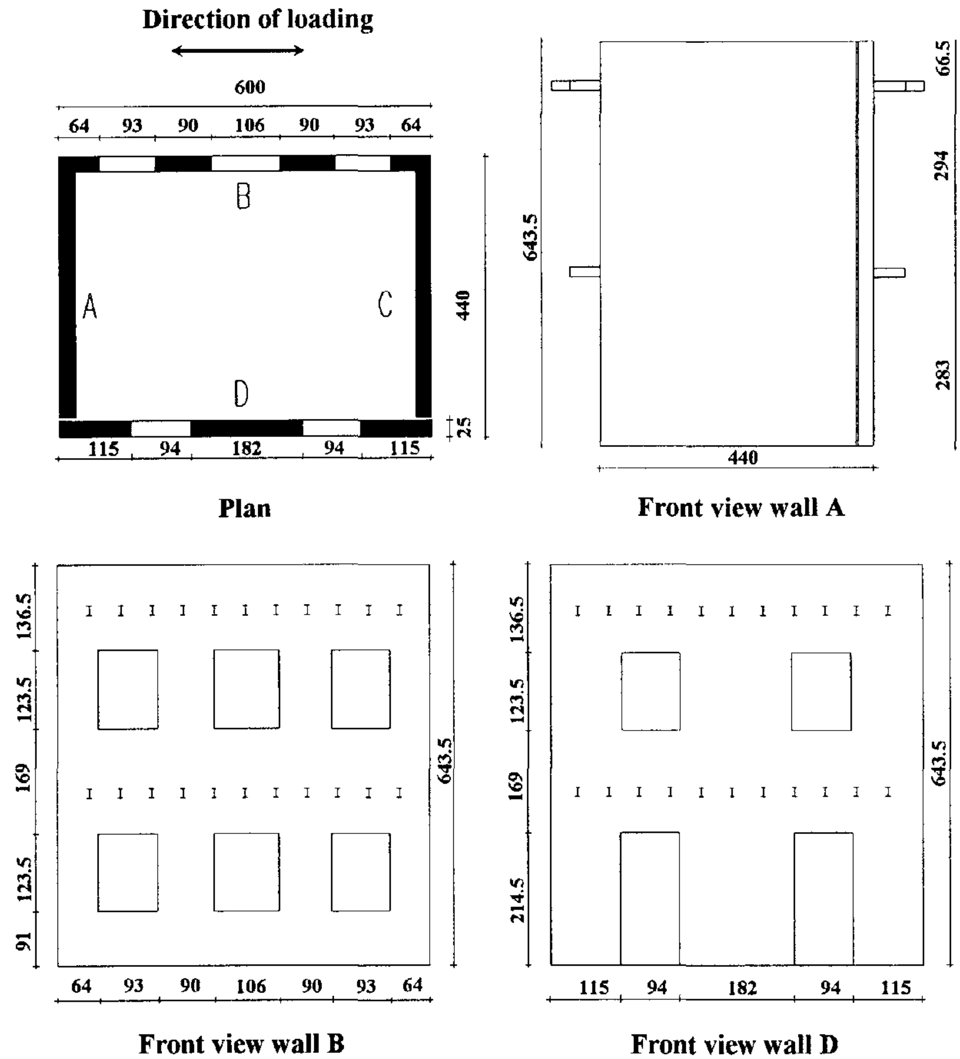


Fig.2. Plan, front, lateral views and dimensions of the prototype, dimensions in centimeters.

After the first destructive test, which is being completed at the time of writing, some retrofitting measure will be taken, in addition to the repair of the cracked masonry. A concrete slab will be cast to connect the floor beams. Tie-beams will be introduced to connect floors and walls, and the intersecting disconnected walls will be tied together.

Load application system

Both vertical and horizontal loads were applied through the floor beams.
Concrete blocks were used to simulate gravity loads, for a total added vertical load of 10 kN/m^2 per floor. Each block is simply supported on two adjacent beams with interposition of teflon sheets to avoid the formation of a rigid floor through the load system. The state of stress resulting from dead

weight and added load results in vertical stresses ranging up to 0.4÷0.5 MPa at the reduced section between the openings level at the ground floor. Such stress can be considered as corresponding to floors extending on both sides of a wall, in more complex buildings, or to a higher number of storeys.

The seismic forces were simulated by the application of four concentrated horizontal forces applied at the two longitudinal walls at the floor levels (fig. 3). The horizontal actions were introduced into the floors at the intersections between floors and beams, by means of four displacement-controlled screw jacks. The displacement history to be applied at each jack was derived from the response recorded from the shake table test on the scaled building, in order to allow an appropriate comparison of the static and dynamic response. The alternative of a pseudodinamic test based on the same accelerograms has been excluded because of the difficulties in a proper simulation of the distributed masses and of the recognized dependence of masonry behaviour from the rate of loading due to crack propagation (8).

The four screw jacks have a stroke of 400 mm and a maximum static force of 1000 kN, are all equipped with swivels at both ends and with load cells. The end plate of each actuator is connected to four steel rods, shown in figure 4, which in turn distribute the loads to the floor beams; the rods are instrumented with strain gauges at each wall opening, in order to allow an estimate of the horizontal force transmitted to each masonry pier during the loading history, which should vary as a

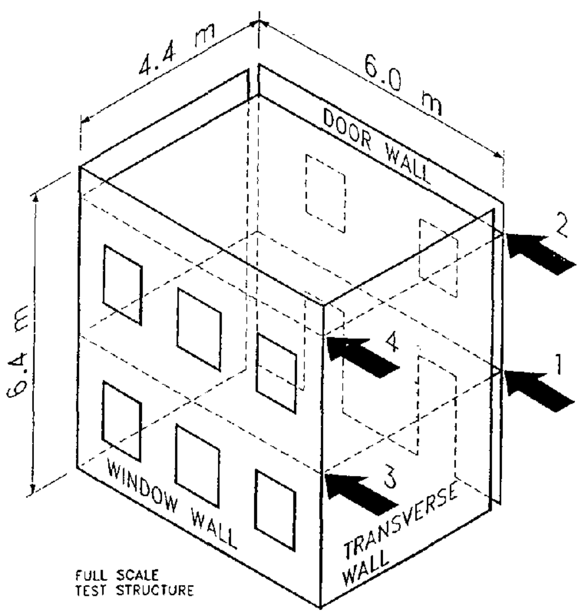


Fig. 3. The seismic forces in the full scale static test are simulated with four concentrated forces.

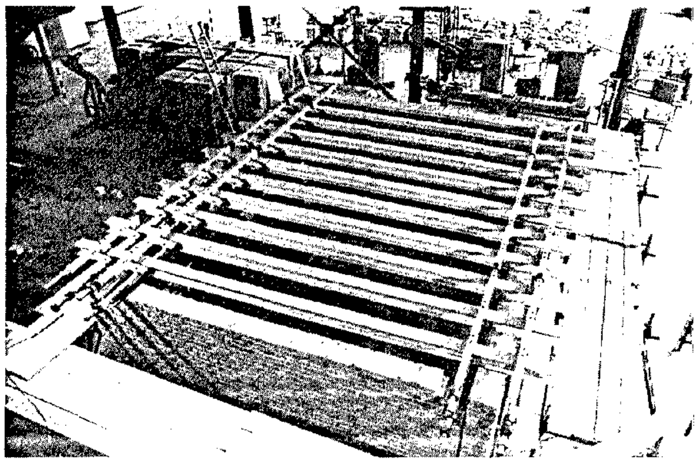


Fig. 4. View of the floor beams during construction. The longitudinal bars used to distribute the horizontal forces to the floor beams are visible

function of local damage.
The jacks are driven by electric motors with gear reducer and an impulse modulation frequency converter, allowing a variation of the linear velocity of the screw jack from 0.025 to 0.25 mm/s. The screw shaft position is read by digital displacement transducers (Sony Magnescale) at intervals of 100 ms, and fed back to a computer which adjusts the velocity in function of the distance from the target displacement and stops the jack when it is reached within the desired tolerance (the minimum usable tolerance is 5 μm).
The use of screw jacks, coupled with the high stiffness of the reaction system, assures the possibility of applying true displacement histories and of following the brittle phenomena and softening branches typical of the response of masonry structures.

Data acquisition system

The data acquisition system consists of an analog to digital converter which assures the possibility of scanning 231 channels at a rate of 2÷3 samples per second per channel.
Three types of transducers are used during the quasi-static tests: linear potentiometers, strain-gauges and pressure transducers. The instrumentation includes transducers for the following measurements:

- horizontal absolute displacements (hybrid-track linear potentiometers);
- vertical absolute displacements and possible uplifts or slips at the base (hybrid-track linear potentiometers);
- deformation of each masonry pier (linear potentiometers, 6 for each pier);
- force at each jack (4 load cells);
- forces at different locations along the load distribution rods (strain gauges);
- vertical stresses at the base of the longitudinal walls (flat jacks with pressure transducers).

Dynamic and static identification

The destructive test on the prototype was preceded by a series of dynamic identification tests, using a hydraulic excitator to input inertia forces into the structures, and reading the generated acceleration signals at 8 different locations of the structure by means of servo-accelerometers.
Tests on the undamaged structure were completed using a sinusoidal excitation (sine sweep) in the range 3-18 Hz. Other tests on the damaged structure are foreseen, using also other types of excitation (e.g. pulse excitation).
The identification algorithms used were previously developed for the case of brick masonry towers (7).
The first two significant (non-local) modes of vibration were detected at frequencies of 5.90 and 10.00 Hz, the first one being a flexural mode and the second a torsional mode which involved flexural deformation of the two longitudinal walls in opposite directions.
A four-by-four stiffness matrix was also experimentally determined using either flexibility and stiffness methods, i.e. loading one screw jack at a time and reading four displacements or applying one displacement and reading four reaction forces.
The experimental stiffness matrix in the undamaged state is reported in table 5.

Table 5. Four-by four stiffness matrix of the prototype.
Experimental (static test), undamaged state [kN/mm]

238.69	-97.05	-0.62	-2.23
-97.05	100.60	-3.90	-5.34
-0.62	-3.90	207.47	-77.31
-2.23	-5.34	-77.31	87.44

Numerical analysis

Linear dynamic and simplified nonlinear static analyses were performed to have a first estimate of the behaviour of the prototype.
The dynamic modal analyses were performed to determine the frequency intervals to investigate in the dynamic identification tests. Several linear finite element models with different levels of detail were used. The most accurate was made with 3-d solid elements using the f.e. code ADINA. The masonry was modeled as orthotropic, and the values of the elastic moduli (*E*= 1900 MPa, G= 570 MPa, ν = 0.1) were derived from the results of the static in-plane shear tests. The influence of the type of connection between floors and transverse walls was also investigated. As expected, a significant number of modes are associated with out of plane bending of the walls. In particular, the role of out-of plane bending of the transverse walls A and C (fig. 1) is strongly affected by the presence or absence of links at the floor level with the floor beams or with the longitudinal walls. Depending on the presence or absence of these links, the first global translational mode in the longitudinal direction varies of approximately 15 %, and the mass participation factor ranges from 62 % to 79 %.

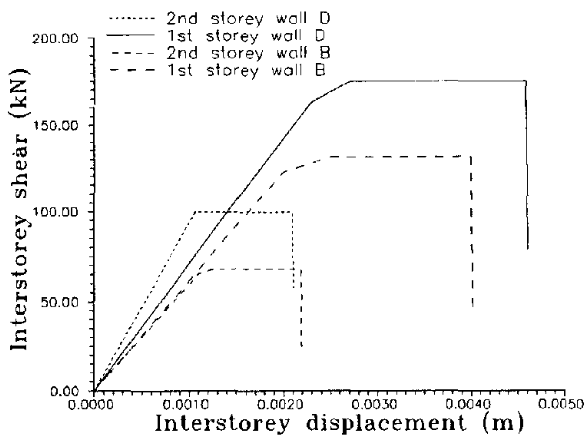


Figure 5. Calculated interstorey shear-drift envelopes for walls B (window wall) and D (door wall).

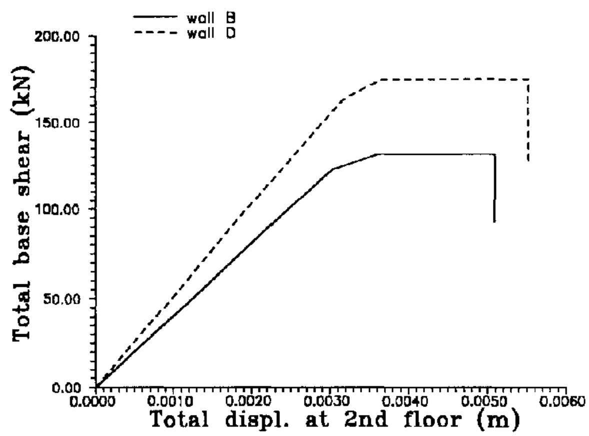


Figure 6. Example of calculated shear-displacement envelopes for walls B and D, assuming a 1/1 ratio for the seismic forces at 1st and 2nd storey.

Nonlinear static analyses were performed to estimate the ultimate horizontal load carrying capacity of the walls. A simplified "storey mechanism" approach (9) was followed, where the failure mechanism of each storey is modeled as a "pier- failure" mechanism. The behaviour of each pier is expressed by means of a horizontal load-horizontal displacement envelope, in the form of a bi-linear (elastic-perfectly plastic behaviour), which is determined by the maximum or ultimate strength, the elastic stiffness, and the ultimate displacement, beyond which the reacting force is assumed to drop to zero. The ultimate strength of each pier is defined as the lowest shear associated to a) diagonal cracking (V_t), b) flexural failure (toe crushing) (V_f), c) shear sliding (V_s). The initial elastic stiffness, as well as the strength parameters, were derived from the interpretation of the static cyclic tests on masonry piers.

With such an approach, the interstorey shear-drift envelope for each wall of each storey was calculated by imposing horizontal displacement compatibility. Figure 5 shows the interstorey shear-drift envelopes for the longitudinal walls D (door wall) and B (window wall), where the collaboration of the transverse walls is neglected. From the interstorey envelopes it is possible to estimate the total shear carrying capacity of the whole wall, assuming a ratio between the forces (or the interstorey drifts) at the first and second storey. As an example, in fig. 6 the total base shear vs. the absolute displacement at the top of the building is reported, in the case where the seismic forces at the first and second storey are assumed to have a 1/1 ratio. Such approach, though simplified, can also give an estimate of the sequence of failures and the associated failure mode in the piers.

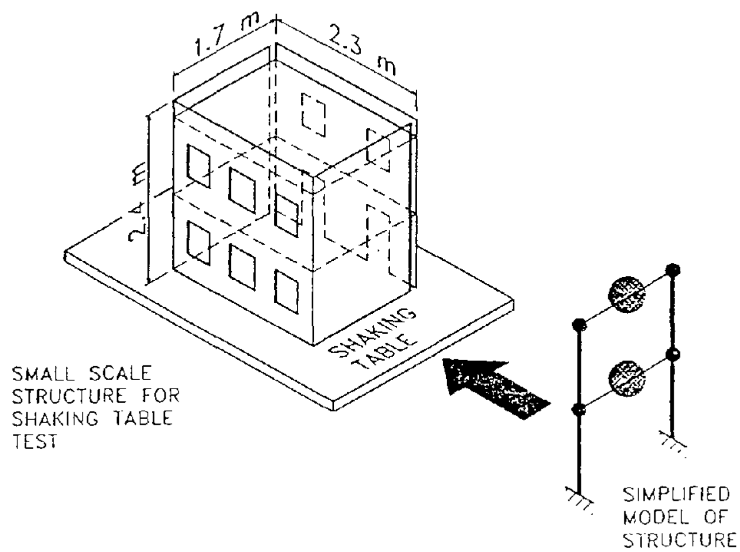


Fig. 7. Idealization of the response of the shake table model tested in Urbana.

Suggestions from the dynamic response observed in Urbana

It happens sometimes that after an experimental test it is realized that some aspects of the response should have been obvious even before testing, on the base of simple physical observations. In the case of the shake table test on the scaled prototype the relative stiffness of floors and walls was such that the response was dominated by the behaviour of the flexible floor diaphragms, which were supporting approximately 60 % of the total mass of the building (fig. 7). Since the masses at the two floors were equal, the peak acceleration at the floors tended to be equal, and, as a consequence, the four forces resulting at the intersection of floors and walls were essentially equal, particularly at peak displacement. This fact is reported and explained in (1), where it is shown that if the height of the resultant force in the several test runs is considered, the centroid of the data is close to 0.75, a value which corresponds to equal force distribution between the first and second floor. A triangular force distribution would have been therefore farther from reality.

The same kind of observation does not apply to displacements, since the relative displacement at the upper and lower storey was dominated by the response of the walls, and therefore by the damage distribution and by a significant rocking response. Therefore no constant ratio between displacements was noticed.

The low coupling of the two longitudinal walls was confirmed, but the fact that equal forces were forced in the two walls by equilibrium, resulted in a force limitation according to the strength of the weaker wall, the stronger wall being therefore unable to develop his full strength capacity.

The implication of these observations on the static test with imposed displacements are simple in principle and partially difficult to implement:

- the low coupling and the similar stiffness of the walls suggest to apply equal displacements at the top, with the advantage of having zero force transmitted across the upper floor diaphragm;

- the equal force distribution and the lower sensitivity of the walls to changes in the force distribution than to changes in the displacement distribution suggest to control the lower jacks displacement in order to have the same load as at the upper storey, and this implied the development of a special control algorithm to be used during the test.

The displacement history applied at the top of the walls is shown in figure 8. In each run the peak target displacement was corresponding to a total drift (top displacement/height of the building at 2nd floor) of 0.025 %, 0.05 %, 0.075 %, 0.1 %, 0.2 % and 0.3 %. The history refers to the tests completed at present. Other runs at higher drifts will be performed soon.

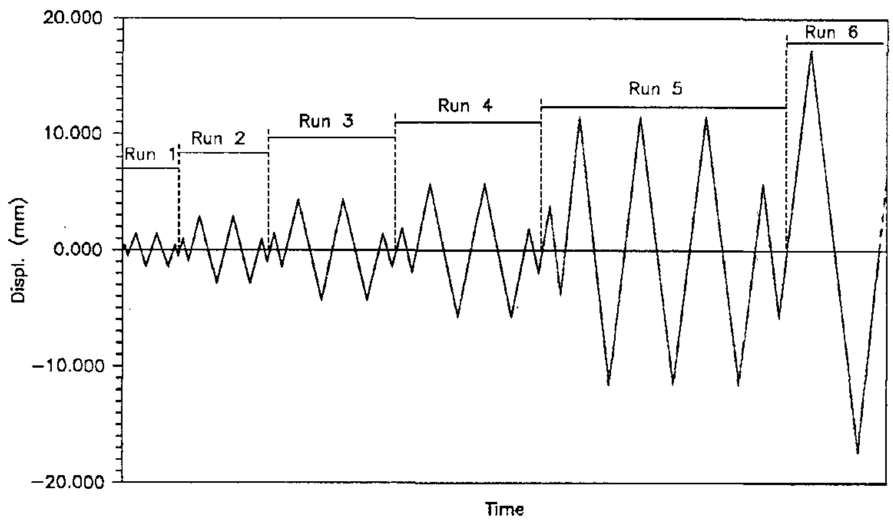


Fig. 8. Displacement history applied at the top floor in the runs already completed.

Preliminary test results

Since testing has not yet been completed at the time of writing, and data interpretation is at an initial stage, only preliminary impressions can be presented here.

In figure 9 and 10 the overall response is shown for each wall, in terms of interstorey shear versus interstorey drift and in terms of base shear versus top displacement. The maximum horizontal force in wall B (window wall) is attained at a total drift of approximately 0.2 % after which a decrease in shear is noticed at 0.3% drift. The maximum horizontal force in wall D (door wall) is reached at 0.2 % drift, but no significant decrease in the first cycle at 0.3 % can be noticed.
The progression of damage propagation was very interesting. Clearly the damage started in the spandrels, reducing the coupling between different masonry piers, but the ultimate strength has been governed by shear cracking and failure of the central masonry piers.

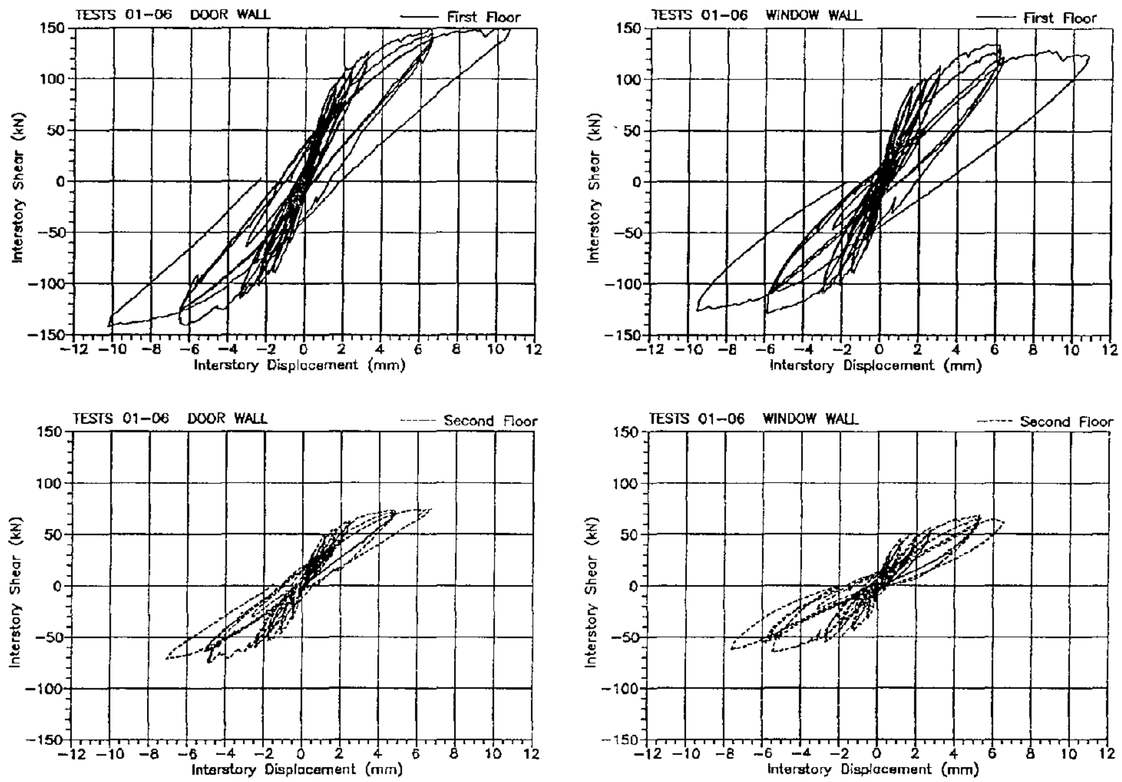


Fig. 9. Experimental interstorey shear vs. interstorey displacement curves

From the simplified progressive displacement profiles, shown in figure 11, the different response of the two walls can be noted: in the case of the door wall the contribution of the lower storey to the total displacement at the top was more significant than in the case of the window wall, where the deflected shape was essentially linear even after significant damage. The measured maximum upward vertical displacement at the top of the building was also significantly different in the two cases, being in the range of 25 % of the horizontal displacement in the door wall, and in the range of 10 % in the case of the window wall.

CONCLUSIONS

The study reported herein is still in progress, and the authors believe that the data interpretation will allow important conclusion on a number of topics, such as:

- validation and improvement of numerical models;
- effectiveness of preliminary tests on materials and sub-assemblages in giving information for the prediction of the behaviour of complex structural systems;
- effectiveness of non-destructive tests and structural identification methods;
- comparison of static and dynamic response (damage distribution, stiffness, hysteresis)
- interaction of wall sub-elements with the adjacent parts of the buildings;

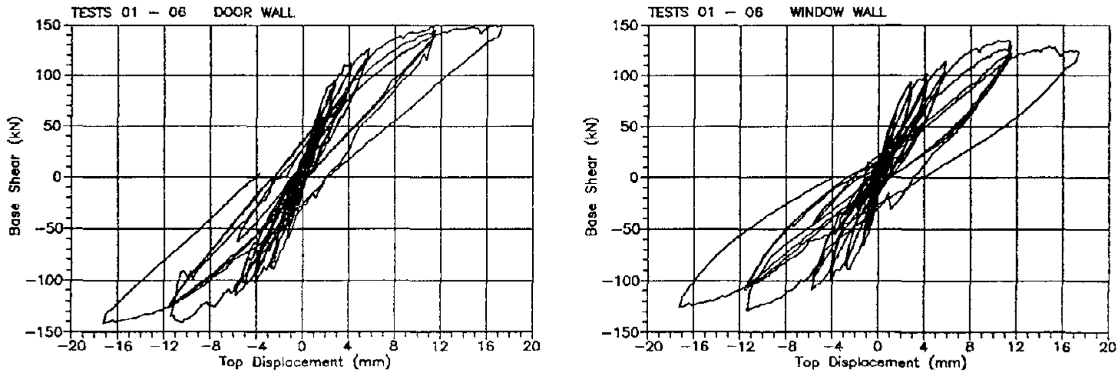


Fig. 10. Top displacement vs. base shear for walls D (door wall) and wall B (window wall).

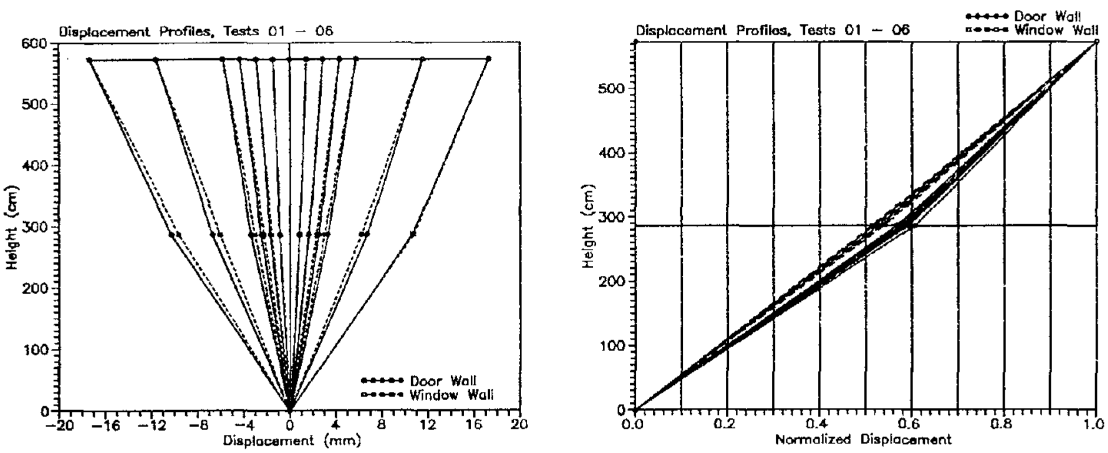


Fig. 11. Experimental displacement profiles.

- effects of openings and of damage localization;
- load redistribution between different walls;
- effect of flexible floors;
- effect of different connections between intersecting walls;
- effectiveness of repair and strengthening intervention;
- implications on evaluation and rehabilitation methods.

It can be already anticipated that unreinforced masonry structures subjected to horizontal loads do not necessarily show brittle response, allowing significant increase of the horizontal displacement without loss of strength. The conditions for which this happens will be discussed after an in-depth examination of experimental and numerical data.

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Section IV

Analysis Methods for Evaluation of Rehabilitated Buildings

SIMPLIFIED METHODS FOR EVALUATION OF REHABILITATED BUILDINGS

Peter Gergely[1] and Ronald O. Hamburger[2]

ABSTRACT

Several approximate and simple methods of analysis are described which consider the nonlinear response of structures. These are being considered for guidelines for the retrofit of existing buildings but they might also be appropriate for codes for the design of new buildings. These methods are based on the static nonlinear analysis of the building. Several examples are given.

INTRODUCTION

Current seismic codes account for the reduction of response as a result of nonlinear action in a very approximate way. Response reduction factors (R in the USA, Q in Europe) are used to reduce the design force levels for various structural systems based on nonlinearity, excess capacity, and observed performance in past earthquakes. Although this approach is simple and works reasonable well for regular and uniform structures, it cannot account for irregularities which change the force distribution as nonlinearities develop.

Increased attention is being paid worldwide to the rehabilitation (retrofit) of existing structures because most buildings have not been properly designed for earthquake resistance and most of the risk to society comes from the deficiencies in the existing building inventory. The retrofit of existing buildings is much more complicated and more expensive than the seismic design of new buildings. Therefore, evolving guidelines for the rehabilitation of existing buildings, such as the FEMA project (1), are considering more realistic analysis methods. The additional engineering work required by these methods is justified by the much higher cost and greater complexity of the problem. However, it is expected that these methods will eventually be incorporated also into codes for the design of new buildings.

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Another reason for the need for more realistic analysis methods is a growing trend toward performance-based design. Current codes are based largely on the prevention of collapse for the design earthquake. However, owners, society, and municipalities may expect better performance in many cases. For example, for certain important facilities we may want to limit damage, avoid loss of life even if there is no collapse, or even try to assure continued operation of the facility. The design for these performance levels may depend on the seismic hazard. Current analysis methods are insufficient because they cannot realistically predict the forces and deformations in irregular structures, especially in systems where nonlinearities do not develop uniformly throughout the structure. Improved analysis methods should reflect the change in inertial force and element deformation distributions as nonlinearities occur.

IMPROVED ELASTIC ANALYSIS (IDR) METHOD

A significant improvement over the use of a single R (or Q) factor is afforded by the Inelastic Demand Ratio (IDR) approach. This procedure continues to rely on linear analysis and, therefore, does not account for the inelastic redistribution of forces and deformations. A regular lateral force analysis (or modal analysis) is performed, as in current codes, without the R factor. The elastic member forces are divided by the member capacities at the same member deformation (end rotation) levels and the resulting Inelastic Demand Ratios are compared to limiting values for various member types. Typical values are (11): Ductile steel and concrete beams 3.0, steel beams in braced frames 2.0, K-braces and connections 1.25, flexure of concrete walls 2.0 (single steel layer) or 3.0 (two layers). If the limiting values are exceeded, redesign is necessary or a nonlinear analysis approach must be used to more carefully examine demands.

STATIC NONLINEAR ANALYSIS METHODS

The various simple nonlinear analysis methods being considered for code use rely on static nonlinear analyses because nonlinear time-history analysis is too complex and unreliable for use in design offices, except by a few experienced engineers. On the other hand, the static nonlinear analysis of structures is relatively simple.

The Pushover Curve

The plot of the total lateral force (base shear) versus the deformation (usually the roof displacement) is called the "pushover curve". This analysis is relatively fool-proof since most engineers can easily estimate the story shear capacities and thus get an idea of the peak load level. It is likely that computer codes will be improved to include such static "pushover" analysis automatically. In the meantime, an easy way to perform pushover analysis is to reduce the stiffnesses of members whenever the elastic limit is reached and additional load is applied to the structure with the new stiffnesses. Often it is not necessary to modify the stiffness for every hinge formation, but only after several new hinges have formed. The forces and displacements are calculated incrementally. A typical pushover analysis result is shown in Fig. 1. The pushover analysis curves tend to be nearly elasto-plastic (bilinear) for frame structures because a story mechanism develops soon after the first flexural hinge forms. On the other hand, the force increases to much higher levels for structural systems with walls.

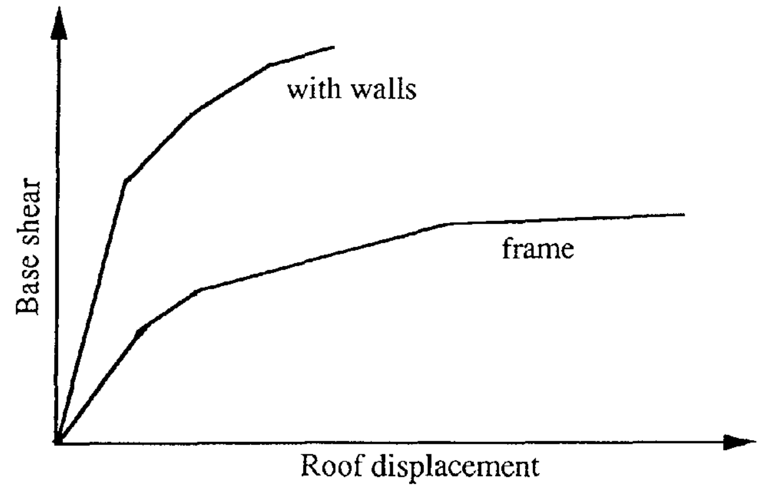

Fig. 1 Typical Pushover Curves

The pushover curve reveals significant information about the characteristics of the structure and is, therefore, important even if no further use is made of it. It shows the progress of damage in the structure, the concentration of hinges, and indicates whether there is a soft story (story mechanism developing before significant hinging occurs elsewhere in the structure). It also helps trace the load transfer among the various load-resisting structural systems in the nonlinear range. This is important because many failures have occurred in earthquakes as a result of poor load transfer.

In the simplest approach, the pushover analysis is performed for an elastic force distribution, for example for the code forces which are functions of the floor masses and the heights. Alternatively, the lateral force distribution is obtained from a modal analysis, though the combination of modal forces at each floor level cannot be done in a rational manner. The forces by either approach are still elastic and do not reflect the stiffness changes. For example, the force distribution for a building with a soft story is significantly different from that of a regular structure.

The reduced (tangent) member stiffness may be too low since the actual member response is cyclic. To account for the higher effective member stiffness, one may assume a secant stiffness for members with nonlinearities, corresponding to an assumed member deformation level (Fig. 2), and perform the next step on the stepwise force analysis. If the resulting member deformation is different from the assumed one, another secant stiffness is assumed. This iterative analysis converges rapidly.

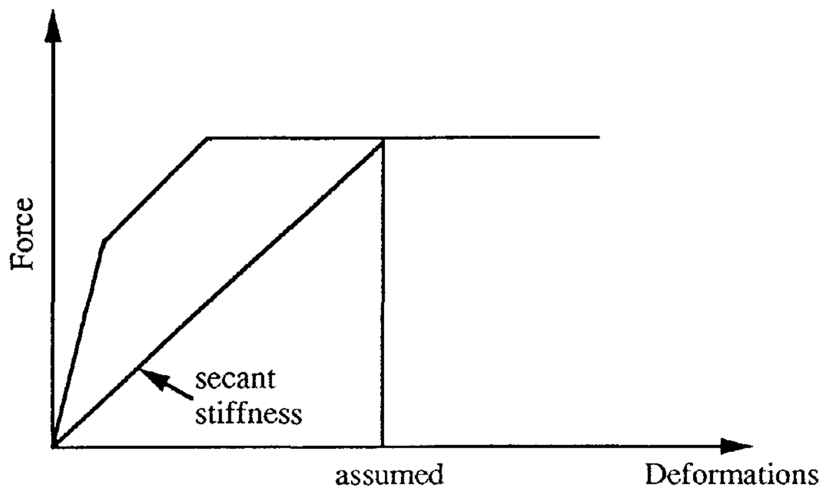

Fig. 2 Secant Member Stiffness for Iterative Approach

For irregular structures it may be necessary to improve the force distribution in the pushover analysis. One way is to generate a load vector as the product of the current displacements and the masses. The new displacements from this load application is an improved inertia force vector shape. If used iteratively, it would converge to the first mode shape for the current stiffness matrix and we could also get the natural frequency from the Rayleigh quotient. Again, this analysis can easily be performed by an ordinary static analysis program.

The pushover curve is a property of the structure; thus we need to estimate the seismic demand. Several approximate ways have been proposed for code use.

The Capacity Spectrum Method

An appealing way of estimating the demand on a system described by its pushover curve is the Capacity Spectrum (10). The base shear and the roof displacement values at each point on the pushover curve are transformed into spectral acceleration and spectral displacement values, usually for the first mode, using the basic equations of modal analysis. The resulting curve is the Capacity Spectrum, which is superposed on the response spectrum curve of the design earthquake, as shown in Fig. 3 for a ten-story steel frame with five bays, one of which has infills. The intersection of the capacity spectrum curve and the response spectrum curve for a selected value of damping gives an estimated response from which the damage state and displacements can be readily calculated.

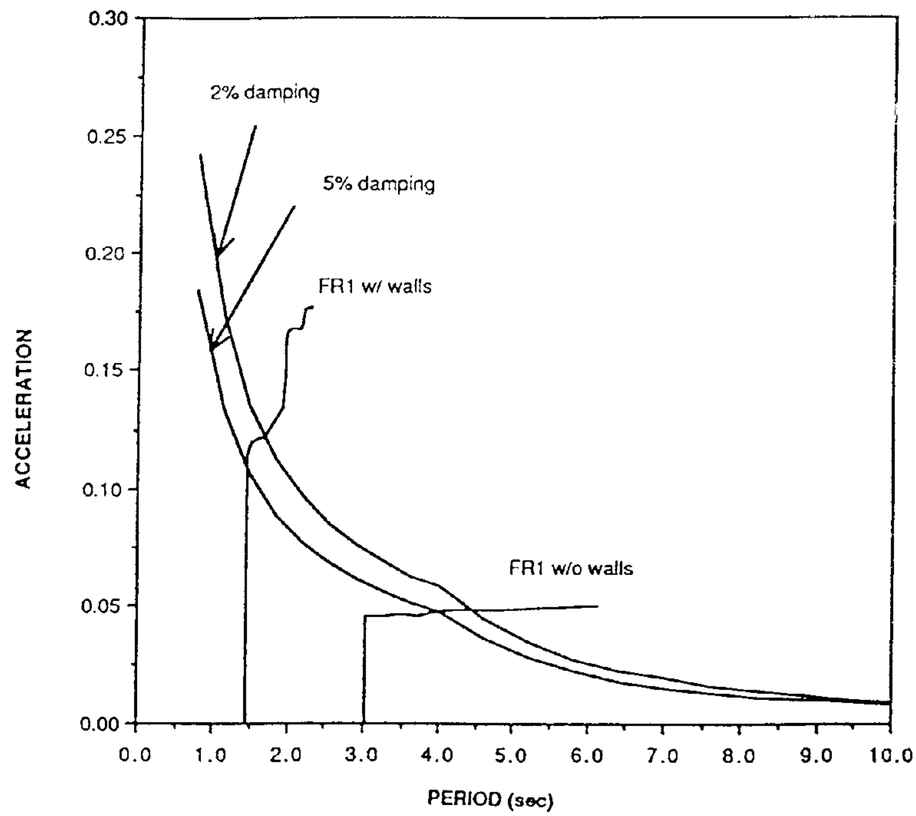

Fig. 3 The Capacity Spectrum Method for a Ten-story Infilled Steel Frame

It is necessary to employ an interpolation scheme between the spectral curves for various amounts of damping. The reduction in response caused by period lengthening is included in the method. However, the hysteretic energy absorption is not. Although it is not possible to replace ductility effects by equivalent viscous damping for the entire frequency range, using an increased damping approximately accounts for the hysteretic effects; this may be inaccurate for short-period structures. As the intersection is further to the right on the capacity spectrum curve, higher damping is appropriate. Relationships have been suggested by Iwan (8) and others for estimating the effective viscous damping as a function of global ductility. On the other hand, the lower spectral curves are for higher damping. Thus the interpolation needs to balance this opposing trends on the curve.

An interesting and more revealing way to plot response spectra is to plot the spectral acceleration S_a versus the spectral displacement S_d (rather than versus the period). Such a plot is indicated in Fig. 4. Constant period lines are rays from the origin. This plot shows the relationship of acceleration and displacement and indicates the effects of changing period. A hypothetical capacity spectrum curve is shown in Fig. 5, together with an interpolation method (4) in which the elastic response (initial period and low level of damping) is identified as point A on a code-type design spectrum. The collapse stage (point B) defines the corresponding period line, which intersects the spectrum with high damping at point C. The line connecting points A and C intersects the capacity spectrum at point D, which is defines the actual response.

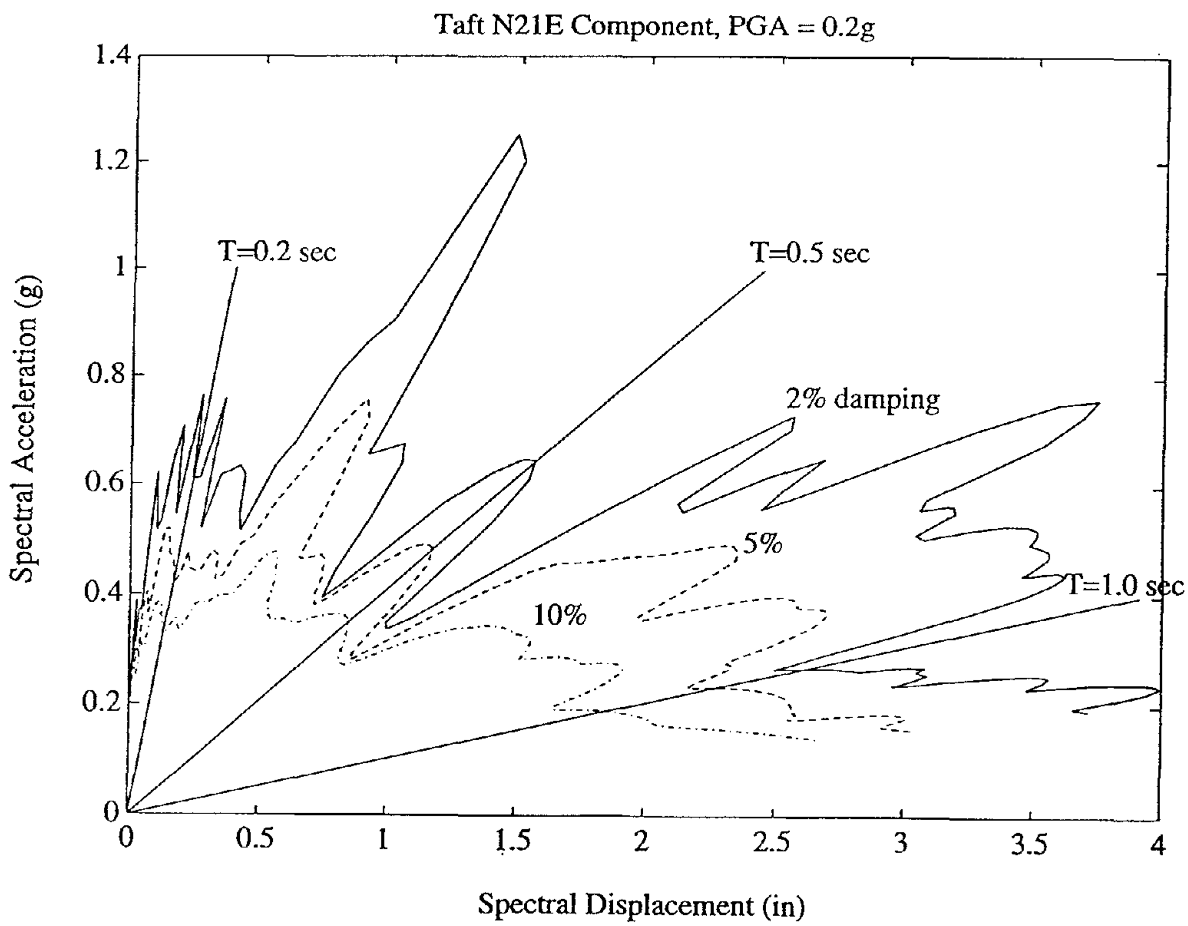

Fig. 4 Response Spectrum Curve Plotted in a New Way

INFILLED FRAMES

A relatively simple and a sophisticated analysis method has been tried to predict the static and dynamic response of infilled frames. Many buildings worldwide consist of light steel or concrete frames that were designed only for gravity loads and unreinforced concrete infills. The response of such structures is being studied analytically and experimentally at Cornell University and elsewhere.

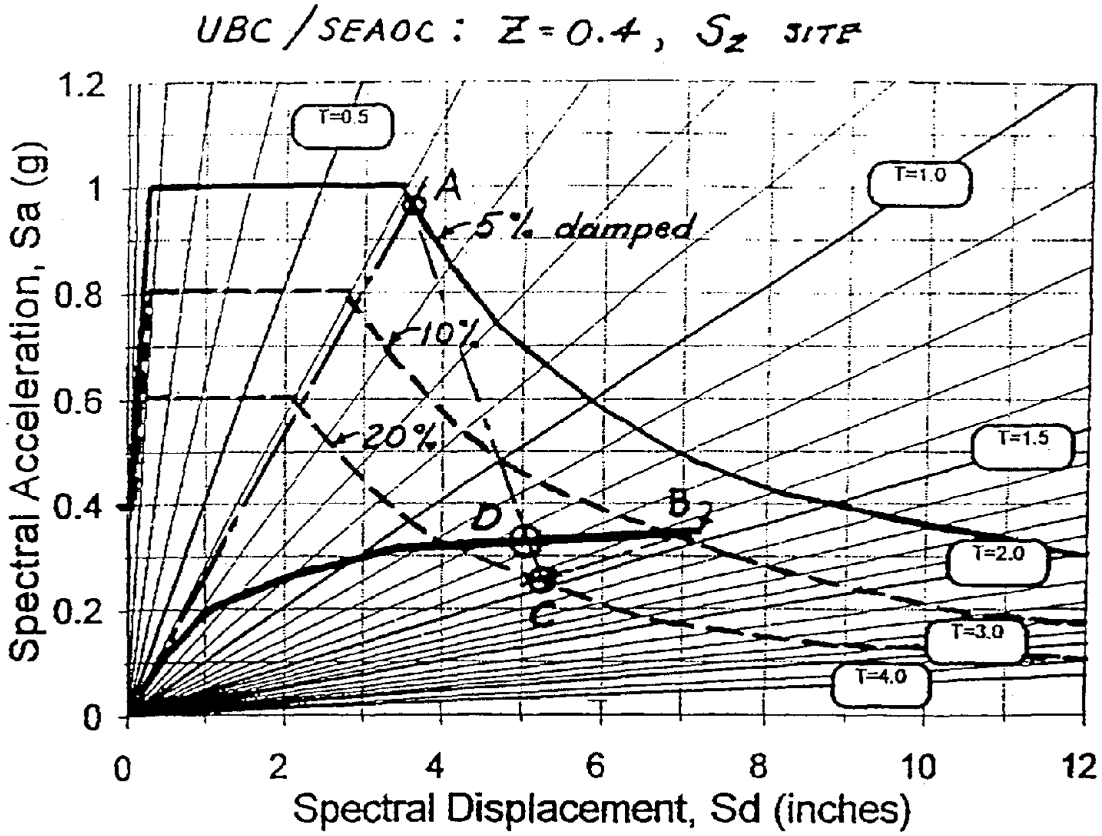

Fig. 5 Capacity Spectrum Method Interpolation

In the simpler model the infill is modelled by three parallel struts in each direction (2): the central strut is diagonal, the other two intersect the beams or columns at a distance from the joint. The three-strut model is able to account for hinges forming in the beams or columns after the infill crushes at the corners and thus the central strut loses its stiffness. The model allows efficient dynamic analysis of multistory infilled frames and gives a reasonably accurate picture of the overall response. The three-strut model was used in calculating the pushover curve in Fig. 3.

The detailed analysis (5) used the finite element program DIANA (3) which can model cracking, plastification, initial gaps, and sliding. Bilinear relationships were used for normal and tangential forces at the wall-frame interface. Good agreement with experiments was obtained for the load-displacement relationship (Fig. 6). Current research concentrates on infills with window and door openings.

EXAMPLE APPLICATIONS

The use of these simple inelastic evaluations methods can permit rapid, and more importantly, realistic evaluations of the demands and capacities of complex structures. To illustrate this, an example analysis of a ten-story steel frame building with infill unreinforced brick masonry is presented. The building, located in San Francisco, California, has a rectangular footprint, with side dimensions 37 by 45 meters. Infill wall panels are typically 43 cm thick, and have regularly distributed penetrations for windows on all sides. Fig. 7 shows a partial elevation of a typical wall. Stories are 3.8 meters tall and bays 4.1 meters wide. Bays are penetrated by pairs of centrally located window openings, 2.1 meters high by 1.1 meters wide.

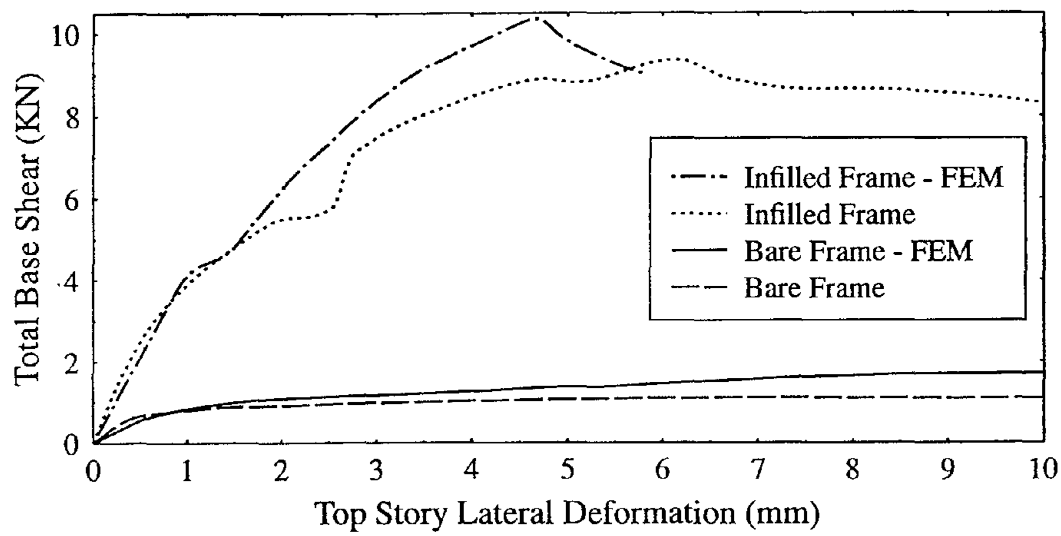

Fig. 6 Comparison of DIANA Analysis of an Infilled Frame with Experiment

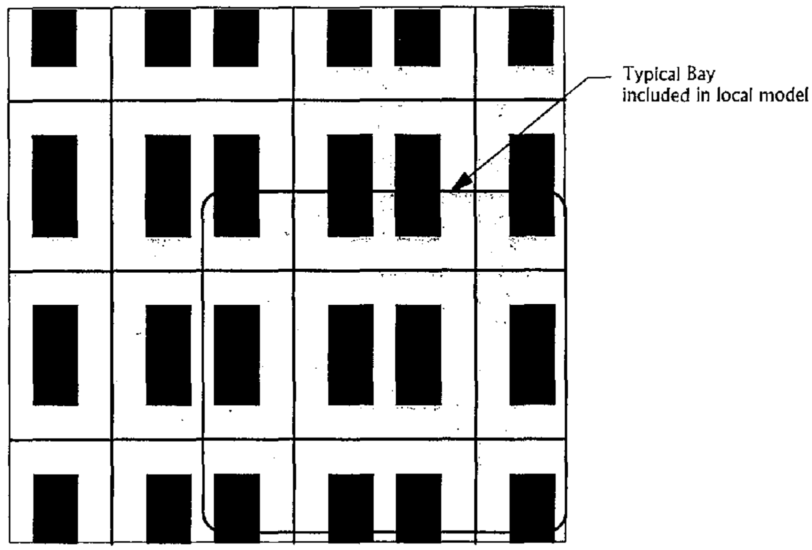

Fig. 7 Partial Elevation of Infill Masonry Wall

At low levels of lateral loading, masonry walls infilled within frame structures behave as shear diaphragms. However, at relatively moderate load levels, principal tensile stresses induced in the masonry by shearing forces result in cracking of the masonry. Following such behavior, the masonry tends to behave primarily as a compressive material to resist lateral deformations induced in the frame. A number of models have been proposed by researchers for physical representation of these phenomena. Solid infilled panels are commonly represented by a series of diagonal struts which span between opposing corners of the frame panels. Following initiation of cracking in such panels, secondary failure modes are typified by compressive crushing of the end bearing zones of masonry against the frame, followed by either plastic hinging or shear failure of frame elements. A three-strut model (2) has recently been demonstrated to be capable of capturing these progressive failure modes.

Alternative models have been proposed for infill panels with large penetrations, such as those in the subject building. Finite element modeling (6) indicates that such infill panels tend to from a system of skewed fields, tangent to the openings in the panels, similar to those indicated in Fig. 8. For masonry of average strength, failure sequences for these compressive fields, classically described as diagonal shear cracking of the piers and spandrels. Following initiation of such cracking, a series of plastic hinges form in the framing elements, until a complete mechanism can form. The ultimate strength of such panels tends to substantially exceed the first formation of shear cracking and occurs at much larger lateral deformation levels.

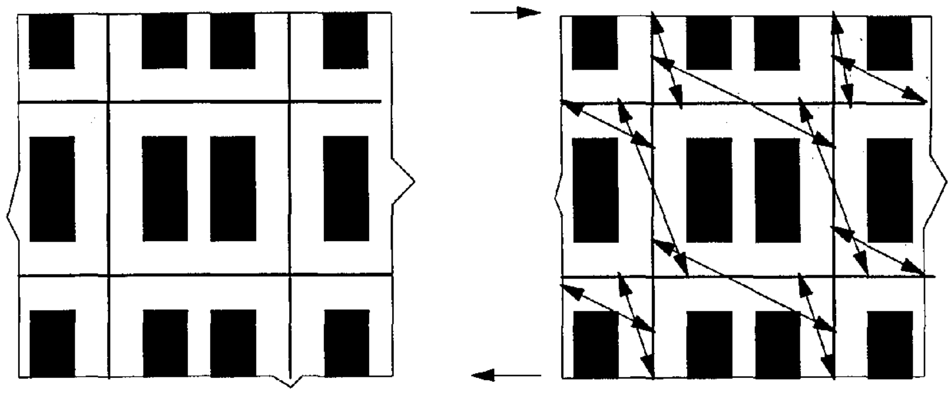

Fig. 8 Typical Compressive Fields in Laterally Loaded Penetrated Wall Panel

Such behavior for a typical bay can be modeled fairly simply, using pairs of inclined braces, located on either side of the central pressure lines indicated in Fig. 8b. Such a model was developed for the typical bay of the example building, and is shown in Fig. 9. Based on previous studies, struts in this model were assigned axial areas equal to a rectangular section with thickness matching that of the wall and width equal to twice that of the wall. Steel column elements were modeled conventionally. Each beam was broken into a series of segments, this allows the relatively low plastic moment capacity of the semi-rigid framing connections to be modeled separately from the larger plastic moment capacity of the steel section.

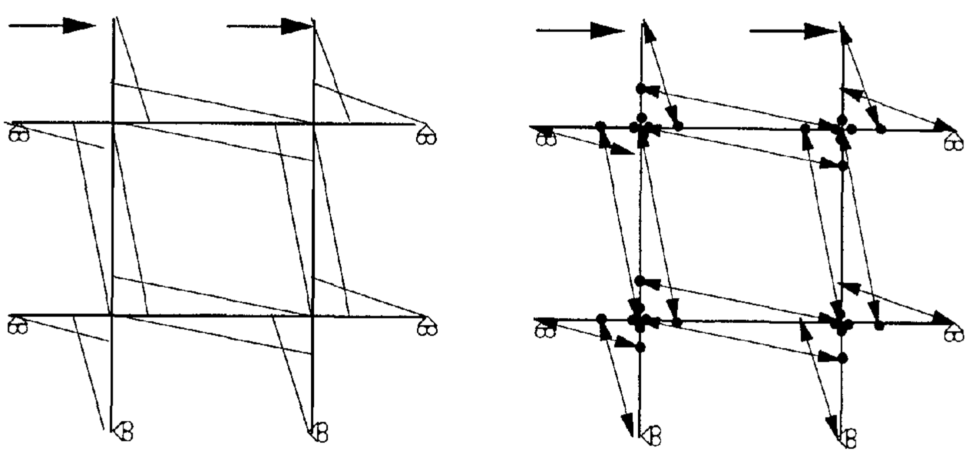

Fig. 9. Simple Model of Infill Strut Behavior

DRAIN 2D (12) software was used to perform a static pushover analysis of the model representing the typical bay. This curve is presented in Fig. 10. At any lateral deformation level, the total story stiffness may be taken as the stiffness of the representative panel, factored by N/2, where N is the total number of columns in the frame at the particular story. An iterative solution technique was then utilized to determine the demands induced on the structure by the design earthquake, following an approach previously suggested in (9). In this approach, a three dimensional analytical model of the structure was created. The steel frame was modeled using the elastic properties of the beams and columns. The stiffening effect of the masonry infill was modeled by a pair of diagonal struts across each bay of the frame. The stiffness of the struts is selected to match the secant stiffness of the typical bay model presented above, at a given deformation level.

Initially, a demand deformation resulting from the design earthquake is estimated using judgment and an appropriate secant modulus determined for the infill masonry. Then, the global diagonally braced frame model is subjected to a response spectrum analysis, using the stiffness obtained from the first estimate, and a response spectrum adjusted for an appropriate amount of effective damping, based on the expected extent of hysteretic response. The analysis will typically indicate a somewhat different pattern of story deformations than originally estimated. Using these new estimates of the deformation demand on the structure, a new series of secant modulus stiffnesses is selected from the local pushover model, and the global three dimensional model is revised using these properties. The response spectrum analysis is repeated and a new pattern of deformations derived. Typically, three or four iterations will result in convergence of the predicted deformation pattern. When this convergence occurs, the displacement demand on the structure will have been determined and the stress state on masonry and framing elements can be obtained directly from the nonlinear model and DRAIN analysis. For the example structure, interstory drifts of approximately 6.3 cm, uniformly distributed throughout the structure, were determined as the demand produced by the design response spectrum. This corresponded to achievement of nearly a full mechanism in the frame, as indicated by the location of the final secant in Fig. 10. Evaluation of compressive demands in the various masonry struts for this demand level were acceptable. However, axial loads in the end columns of frames resulting from structure overturning were excessive, indicating a potentially hazardous condition. Strengthening of the structure, through local reinforcement of the corner columns, could permit the building to withstand the expected demands.

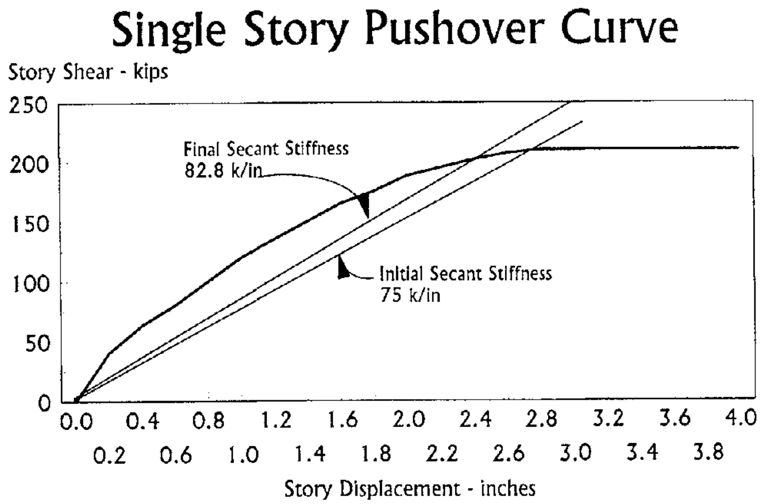

Fig. 10 Pushover Curve for Story, showing iterative deflection estimation approach

A capacity spectrum method analysis of the building was also performed, as a check on the iterative approach. To perform this analysis, a stick model was constructed consisting of a series of shear beams, each representing the stiffness of an individual story, stacked one upon the other. The stick model was subjected to an incrementally increased pattern of lateral loads, with a vertical distribution equal to that specified by the Uniform Building Code (7) for the static analysis procedure. At a given level of lateral loading, the stiffness of each story was taken as that predicted by the individual story pushover analysis previously described. The total pushover curve for the structure, consisting of a plot of the lateral deflection at the center of mass against the total applied base shear was then plotted against the design response spectrum, as shown in Fig. 11. The intersection of the two curves represents the total demand base shear and lateral deflection of the structure.

The intersection of the two curves occurs at a lateral deflection of the center of mass of the structure of approximately 51 cm. Note that this corresponds to a roof deflection of approximately 64 cm, which corresponds well with the 6.3 cm average interstory drift predicted by the iterative approach. Therefore, the two approaches to estimating the earthquake induced demand on the structure produced similar results.

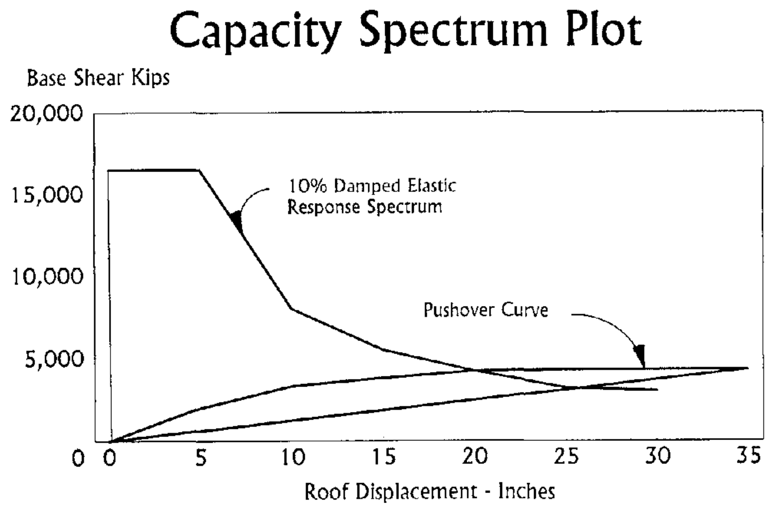

Fig. 11 Capacity Spectrum Plot for Example Structure

CONCLUSIONS

Current elastic seismic analysis methods are inadequate for the estimation of the internal force and displacement distributions. Codes and guidelines under development are introducing relatively simple nonlinear approaches. Several variations of this approach are described, among them the Capacity Spectrum Methods, which relies on a nonlinear static pushover analysis of the structure. Examples are given for this approach, including infilled frames.

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MODELING UNREINFORCED BRICK MASONRY WALLS

Luigi Gambarotta[1] and Sergio Lagomarsino[2]

ABSTRACT

The response of in-plane loaded brick masonry walls is analysed by applying constitutive models which take into account the mechanical behavior of each component and the relative interfaces, i.e. decohesion and slipping in the mortar joints and failure in bricks. To this end, a damage model for mortar joints is proposed and then applied in two different models for brick masonry walls. The former is a composite model based on a description of mortar joints and brick units by means of finite elements, while the latter is a continuum model in which the mechanisms of inelastic deformation in the horizontal mortar joints and brick units are smeared. The composite model is thus assumed as a reference model to check the capabilities and the validity limits of the continuum model, which turns out to be very effective in the finite element simulation of large scale masonry walls. The capabilities of the proposed models are backed-up by two examples referring to simple rectangular walls and large scale perforated walls subjected to cyclic horizontal actions superimposed on vertical actions.

INTRODUCTION

In spite of the ancient and wide use of brick masonry in buildings, difficulties still arise in modeling such "material", to the extent that the need for a better understanding of the structural behavior of masonry buildings becomes even more relevant, mostly when lateral load-carrying capabilities are required of structures located in areas of high seismic risk. Since brick masonry is a composite material, preliminary formulations of constitutive models for the individual components have to be put forward, which may be more or less detailed according to the degree of accuracy of the masonry model to be proposed [1]. This task must not only consider the bricks and mortar, but must also take into account the brick-mortar interface which plays a central role, in that this is where decohesion and frictional slipping take place.

An interesting approach considers the mortar joints, the weakest components of the brickwork, represented by an interface model: a discontinuum element connecting the bricks [2,3]. This assumption implies that the damage and failure of bricks need to be properly modeled. In fact, the brick model must also include the compressive failure of the masonry, which actually involves

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both bricks and mortar, in that the interface ignores the Poisson effect and then only partially describes the brick-mortar interaction. On these bases, the masonry may be analysed by a "composite" scheme, allowing the wall response to horizontal actions superimposed on vertical forces to be accurately simulated through finite element models. However, the description of each brick by means of several finite elements turns out to be computationally intensive, so precluding the analysis of shear walls in real cases. These cases need to be analyzed by means of continuum models, whose constitutive equations have to be obtained through the homogenization of the contributions to the mean strain given by each masonry component. In this context, the composite model may be assumed as a reference for the validation of the continuum model.

In this paper, a continuum model for brick masonry is formulated, based on a mortar joint model capable of taking into account the effects of the compressive stress on the joint damage and the hysteretic response to cyclic shearing strains. At first, this joint model is applied in a composite model for masonry, in which the failure of biaxially stressed bricks is also taken into account, and whose validity has been verified by the simulation of experimental tests on shear walls. Once this reference model has been defined, the continuum model is then considered, based on the homogenization of a stratified medium representative of the brick masonry. In this approach, which seems to be interesting due to its simplicity, two typical layers are considered: the mortar bed joint layer and the layer representing the brick rows together with the head mortar joints. However, the mechanisms of inelastic deformation in the head joints, related to horizontal extension and dilation, are neglected, since from the analyses carried out it results that their influence on the global wall response is weak. The constitutive equations obtained have been applied in the finite element analysis of shear walls under horizontal forces superimposed on vertical forces.

A DAMAGE MODEL FOR MORTAR JOINTS

Let us consider a mortar joint between two bricks, reproducing the test configuration used by Atkinson et al. [4] in testing masonry bed joints under direct shear. The mean strain $\varepsilon_m = \{0 \ \varepsilon_2 \ \gamma\}^t$ in the mortar joint, which takes into account the damage in the mortar and the decohesion in the mortar-brick interface, is assumed as depending on the mean stress $\sigma_m = \{\sigma_1 \ \sigma_2 \ \tau\}^t$, σ_2 and τ being the components of the resolved stress on the joint plane. Considering stress states which only induce inelastic deformation in mortar, the mean strain may be split into the elastic and inelastic strain components:

$$\varepsilon_m = \mathbf{K}_m \sigma_m + \varepsilon_m^* \quad , \tag{1}$$

where $\mathbf{K}_m$ is the elastic compliance matrix of the mortar and $\varepsilon_m^* = \{0 \ \varepsilon_m^* \ \gamma_m^*\}^t$ represents the inelastic extension and sliding of the mortar joint. Denoting the joint damage variable as $\alpha_m \geq 0$, the extension and sliding are assumed as follows:

$$\varepsilon_m^* = h(\alpha_m) \, \mathcal{H}(\sigma_2) \, \sigma_2 \quad , \tag{2.1}$$

$$\gamma_m^* = k(\alpha_m) \, (\tau - f) \quad , \tag{2.2}$$

where $h(\alpha_m)$ and $k(\alpha_m)$ represent the opening and sliding compliance of the mortar joint, which increase with the damage variable from the initial state $h(0)=k(0)=0$. In equation (2.1), the

Heaviside function $\mathcal{H}(\sigma_2)$ takes into account the unilateral response of the joint. Finally, f is representative of the friction in the mortar-brick interface [5]; when tensile stress acts on the mortar joint ($\sigma_2>0$) the variable f vanishes, while under compressive stresses it limits the sliding activated by the shear stress.

Equations (1) and (2) imply that the internal variables α_m and f must be known at any step of the loading history through evolution equations. To this end, it is worth noting that the variables γ^*_m and α_m are associated to f and $\mathcal{Y}_m$, respectively, the latter being the strain energy release rate defined as follows:

$$\mathcal{Y}_m = \frac{1}{2} k'(\alpha_m)\, \mathcal{H}(\sigma_2)\, \sigma_2^2 + \frac{1}{2} k'(\alpha_m)\, (\tau - f)^2 \tag{3}$$

The evolution equations of the internal variables are formulated on the basis of two conditions to be satisfied at any time in the loading process. The variable f has to satisfy the friction limit condition

$$\phi_s = |f| + \mu\sigma_2 \quad , \tag{4}$$

involving the friction coefficient μ, to which the simple flow rule

$$\dot{\gamma}^*_m = v\dot{\lambda} \quad , \quad \dot{\lambda} \geq 0 \quad , \tag{5}$$

is associated ($v=f/|f|=\pm 1$). The damage evolution is also ruled by imposing the strain energy release rate to be less than or equal to the mortar joint toughness $\mathcal{R}_m$ (assumed as depending on α_m), that is

$$\phi_{dm} = \mathcal{Y}_m - \mathcal{R}_m \leq 0 \quad ; \tag{6}$$

when the limit condition is attained ($\phi_{dm} = \dot{\phi}_{dm} = 0$) the joint damage rate ($\dot{\alpha}_m \geq 0$) is assumed to take place in the infinitesimal load step.

Due to the complexities of the damaging mechanisms in the mortar joint, the compliance functions $k(\alpha_m)$ and $k(\alpha_m)$ cannot be deduced on a mechanical basis. Consequently, the following simple assumption, implying $\mathcal{Y}_m$ to be independent of α_m, is put forward:

$$k(\alpha_m) = c_{mn}\, \alpha_m \quad , \quad k(\alpha_m) = c_{mt}\, \alpha_m \tag{7}$$

When tensile stress is active on the mortar joint ($\sigma_2 \geq 0$), condition (6) may be rewritten as follows:

$$\phi^+_{dm} = \frac{1}{2} c_{mn}(\sigma_2^2 + \rho_m \tau^2) - \mathcal{R}_m(\alpha_m) \leq 0 \quad , \tag{8}$$

where $\rho_m = c_{mt}/c_{mn}$.

Once the stress state and the function $\mathcal{R}_m(\alpha_m)$ are given, the damage variable α_m can be evaluated by solving equation $\phi^+_{dm} = 0$. From this, it follows that the existence of a limit state $(\sigma_2, \tau)_m$, representative of failure, is subordinate to the existence of a maximum for the toughness function $\mathcal{R}_m(\alpha_m)$. While the phase preceding this limit condition ($\mathcal{R}'_m = d\mathcal{R}_m/d\alpha_m > 0$) is characterized by a stable response, the subsequent phase $\mathcal{R}'_m < 0$. is stable only if strain is controlled. Thus, it is assumed that the toughness function reaches a maximum $\mathcal{R}_{mc}$ for α_m=1; the failure condition of the mortar joint under tensile stresses is then obtained from condition (8):

$$\sigma_2^2 + \rho_m \tau^2 = 2\mathcal{R}_{mc}/c_{mn} = \sigma_m^2 = \rho_m \tau_m^2 \quad , \tag{9}$$

where σ_m and τ_m represent the tensile and shearing strength of the mortar joints.

When the compressive stress acts on the joint (σ_2<0), the sliding $\dot{\gamma}_m^*$ and damage $\dot{\alpha}_m$ rates must be evaluated considering conditions (4) and (6) which, through assumptions (2) and (7), become

$$\phi_s = \left|\tau - \gamma_m^*/c_{mt}\,\alpha_m\right| + \mu\sigma_2 \le 0 \quad , \tag{10}$$

$$\phi_{dm}^- = \frac{1}{2}\gamma_m^{*2}/c_{mt}\,\alpha_m^2 - \mathcal{R}_m(\alpha_m) \le 0 \quad . \tag{11}$$

If the friction limit condition ϕ_s=0 is reached, two different evolutions are possible in the infinitesimal load step. When $\phi_{dm}^- < 0$, only sliding rate $\dot{\gamma}_m^*$ can take place, which may be evaluated by solving a linear complementarity problem. On the other hand, if $\phi_{dm}^- = \dot{\phi}_{dm}^- = 0$, sliding $\dot{\gamma}_m^*$ and damage α_m rates may take place; a linear complementarity problem must also be solved in this case, giving a unique or double solution if $\mathcal{R}_m'$>0 or $\mathcal{R}_m'$<0. Finally, the failure condition is obtained by imposing $\phi_{dm}^-(\alpha_m$=1)=0 and $\phi_s(\alpha_m$=1)=0, and by virtue of equations (2), (7) and (4), turns out to be expressed as:

$$|\tau| + \mu\sigma_2 = \tau_m \quad . \tag{12}$$

Once σ_m, τ_m and μ are given, both the limiting resistance domains, delimited by the curves of equations (9) and (12), and the limiting sliding domain, obtained from equation (10) assuming $\dot{\gamma}_m^* = 0$, are defined in the plane (σ_2, $|\tau|$) as shown by Figure 1a.

The response to normal tensile stresses by the proposed mortar joint model is shown in Figure 1b, which exhibits both linear unloading, stiffness deterioration and strain-softening. These diagrams are obtained by assuming the following toughness function:

$$\mathcal{R}_m(\alpha_m) = \begin{cases} \mathcal{R}_{mc}\,\alpha_m & 0 < \alpha_m < 1 \quad , \\ \mathcal{R}_{mc}\,\alpha_m^{-\beta} & \alpha_m > 1 \quad , \end{cases} \tag{13}$$

which attains its maximum $\mathcal{R}_{mc}$ when α_m=1 and asymptotically tends to vanish for α_m increasing (β>0).

Figure 1c shows the model response to cyclic shearing strains superimposed on normal compressive stress. During the loading phase the response is elastic until $\tau \le -\mu\sigma_2$ ($G^*=1+G_m c_{mt}(1+\mu\sigma_2/\tau_m)$); once this stress level is reached, sliding and damage take place. The strain-softening phase is characterized by the asymptotic tendency of the shearing stress, for increasing shearing strains, towards the value $-\mu\sigma_2$. In addition, unloading is at first characterized by an elastic response with no sliding until the friction limit condition is reached, after which sliding takes place with reduced stiffness, according to the damage level previously attained. This response is characteristic of mortar bed joints as shown in Figure 1d, where the theoretical results from the present model fit in well with the experimental results obtained by Atkinson et al. [6].

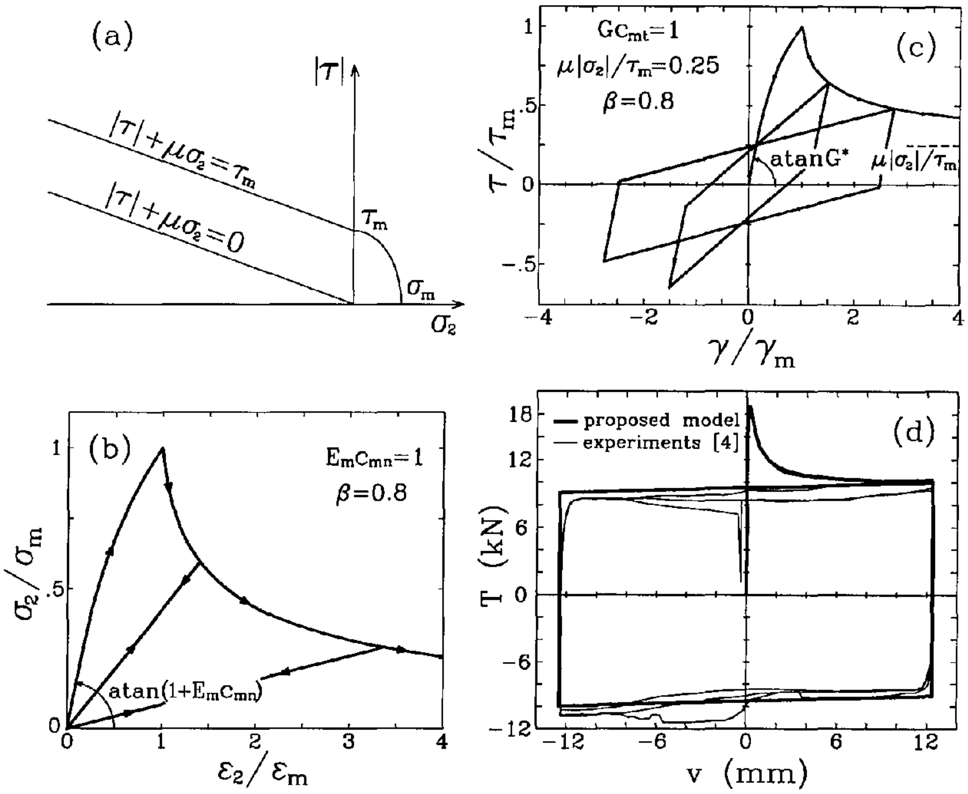

Figure 1. Damage model for mortar joint: (a) sliding and failure curves in the (σ_2, $|\tau|$) plane; (b) model response to tensile stresses; (c) model response to cyclic shearing strains; (d) simulation of direct shear tests by Atkinson et al. [4].

A COMPOSITE MODEL FOR BRICK MASONRY

Let us now consider brick masonry walls subjected to constant vertical forces and monotonically or cyclically increasing horizontal forces. In order to determine the effect of the damage in the mortar joints and bricks on the structural response of the wall, a composite model is considered. The model assumes the brick masonry wall as consisting of rectangular continuum elements, representing the bricks, connected by interface elements representing the mortar joints.

Failure in bricks leads to the activation of further mechanisms of inelasticity in the mortar, not taken into consideration in the joint model described in the previous Section. In order to simplify matters, these effects are incorporated in the continuum elements. To this end, the properties of the continuum elements are determined considering, as a reference, the compressive failure in the masonry, which starts in the bricks due to the Poisson effect, and the tensile failure in the bricks. Moreover, this approach requires the joint to be reduced to a one-dimensional interface which only considers the normal and tangential stresses on the joint plane.

The continuum elements are assumed as being a homogeneous material characterized by: elasto-plastic constitutive equations, Drucker-Prager failure criterion and strain-softening in the post-critical response. These assumptions may be acceptable if monotonic load histories are considered, so precluding inversions in the strain rates. In particular, the second assumption is acceptable in that the ratio between the resistance to compression of the masonry σ_{Mc} and the resistance to tensile stress of the bricks σ_{bt} is not very high ($\sigma_{Mc}/\sigma_{bt} \approx 2 \div 3$).

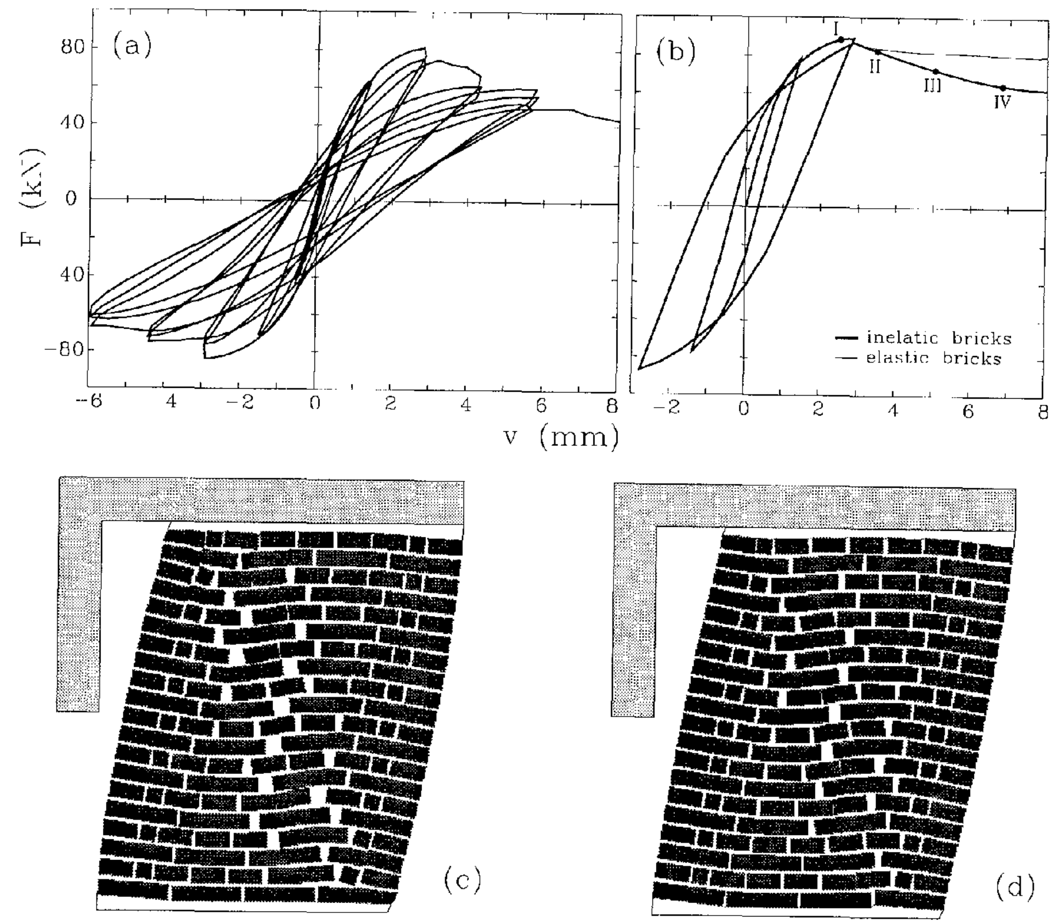

Figure 2. Rectangular masonry panel: (a) experimental response to horizontal actions [6]; (b) theoretical response by the composite model; (c) wall configuration (v=10 mm); (d) wall configuration assuming elastic bricks (v=10 mm).

Various rectangular walls have been examined using a finite element procedure which considers the interfaces and bricks as being divided into finite elements. The walls, initially compressed, are subjected to imposed horizontal deflections with rotation inhibited at the top edge, as illustrated in Figure 2c. Particular attention was given to walls with different geometric ratios and fabric. In slender walls (height/width≈2) an overturning mechanism is activated; damage is due to tensile stresses in the mortar bed joints at two corners and the response is stable until the compressive failure of the bricks at the other corners is reached. Vice versa, in squat walls (height/width≈1) the damage takes place in the bed joints on the diagonal, inducing the limit state of resistance; the corresponding stress states are characterized by the formation of two inclined compressed bands. In the subsequent post-peak response, the bricks on the diagonal break under tensile stresses while crushing takes place at the corners.

As an example, the results of tests carried out on a wall [6] of average slenderness have been considered and reported in Figure 2a in terms of applied lateral deflection v and corresponding horizontal force F. The wall is 100 cm wide and 135 cm high, the brick size is 24 cm × 12 cm × 5.5 cm and the thickness of the mortar joints is s=1 cm. The simulation by the proposed composite model is shown in Figure 2b, from which it can be seen, in addition to a good agreement with the experimental results, that the failure mechanisms in the bricks only influence the post-peak behaviour. The theoretical response was obtained by using parameter values obtained from tests on the materials [7]: E_M=1800 MPa, ν_M=0.2, μ=0.5, σ_m=0.1 MPa, τ_m=0.25 MPa, β=0.9, sc_t=0.5 mm/MPa, σ_{Mc}=4.8 MPa, σ_{bt}=2 MPa. Figure 2b also shows some characteristic states: I) failure of the mortar bed joints on the diagonal; II) failure of the mortar bed joints at the base; III) tensile failure of the bricks and failure of the mortar head joints on the diagonal; IV) compression failure of the bricks at the corner.

Figure 2c represents the configuration of the wall corresponding to the top deflection v=10 mm. In the diagonal band, a considerable dilation of the head joints, sliding of the horizontal joints and the tensile failure of some bricks can be seen, while the compressed bricks at the corners exhibit notable inelastic deformation. Moreover, Figure 2d shows the configuration obtained by the assumption of elastic bricks, which turns out to be similar except for less dilation in the central zone of the wall.

Therefore, the results obtained represent a reference for the evaluation of simpler constitutive models, as illustrated in the next section.

A CONTINUUM MODEL FOR BRICK MASONRY

In order to formulate a continuum model, brick masonry is assumed as being a stratified medium consisting of bed joint layers and layers representing bricks and mortar head joints. This simplification, analogous to that by Pietruszczak and Niu [8], only considers the inelastic deformation in the mortar bed joints and in the bricks; however, it neglects the mechanisms involving both bed and head mortar joints, which cause vertical extension and dilation (Figure 2c,d). The constitutive equations can be deduced by homogenizing mortar bed joints and brick layers and considering the effective thickness of the mortar bed joint [9]. The mean strain is then given by:

$$\varepsilon = \mathbf{K}\,\sigma + \eta_m \varepsilon_m^* + \eta_b \varepsilon_b^* \quad , \tag{14}$$

where σ is the mean stress in the masonry, η_m and η_b are the volume fractions of the mortar joints and brick units respectively (η_m+η_b=1), and $\mathbf{K}$ is the elastic orthotropic compliance of the masonry, which depends on the elastic moduli of mortar and brick and on the volume fraction η_m. The inelastic strain in the mortar joint ε_m^* is obtained according to the mortar joint model previously proposed. Moreover, the inelastic strain ε_b^* due to damage and failure in bricks, only considers the main contributions of brick damage, induced by compressive and shear stresses, so neglecting the effect of the horizontal component σ_1 of the stress.

Then the components of the inelastic strain in bricks $\varepsilon_b^*=\{0\ \varepsilon_b\ \gamma_b\}^t$ are assumed, similarly to the mortar joint model, as follows:

$$\varepsilon_b = c_{bn}\,\alpha_b\,\mathcal{H}(-\sigma_2)\,\sigma_2 \quad , \tag{15.1}$$

$$\gamma_b = c_{bt}\,\alpha_b\,\tau \quad . \tag{15.2}$$

Here, the normal strain ε_b^* only takes into account the compressive stress effect by means of the Heaviside function. Moreover, both strain components linearly depend on the damage variable α_b of bricks and on the brick compliance parameters c_{bn} and c_{bt}. The strain energy release rate in bricks, associated to the damage variable α_b, is then expressed by

$$\mathcal{Y}_b = \frac{1}{2} c_{bn}\,(\mathcal{H}(-\sigma_2)\sigma_2^2 + \rho_b\tau^2) \quad , \tag{16}$$

where $\rho_b=c_{bt}/c_{bn}$. Thus, as with the mortar joint model, the damage evolution is assumed to be ruled by the condition

$$\phi_{db} = \mathcal{Y}_b - \mathcal{R}_b \leq 0 \quad , \tag{17}$$

so that when the limit condition ($\phi_{db} = \dot{\phi}_{db} = 0$) is attained, the brick damage ($\dot{\alpha}_b \geq 0$) is assumed to take place in the infinitesimal load step. In condition (17) the toughness function $\mathcal{R}_b(\alpha_b)$ of bricks is introduced and in the following is assumed as expressed in the form given in equation (13) (the parameter $\mathcal{R}_{bc}$ being considered at this point).

This formulation gives the failure condition for monotonic compressive stresses ($\sigma_2<0$) as follows:

$$\sigma_2^2 + \rho_b\tau^2 = 2\mathcal{R}_{bc}/c_{bn} = \sigma_b^2 = \rho_b\tau_b^2 \quad , \tag{18}$$

where σ_b represents the compressive strength of the masonry, while τ_b is the shear strength of the bricks. In the (σ_2, $|\tau|$) plane the sliding and failure curves relating to monotonic stress paths are shown in Figure 3, exhibiting both the joint failure and the cap model due to the bricks failure.

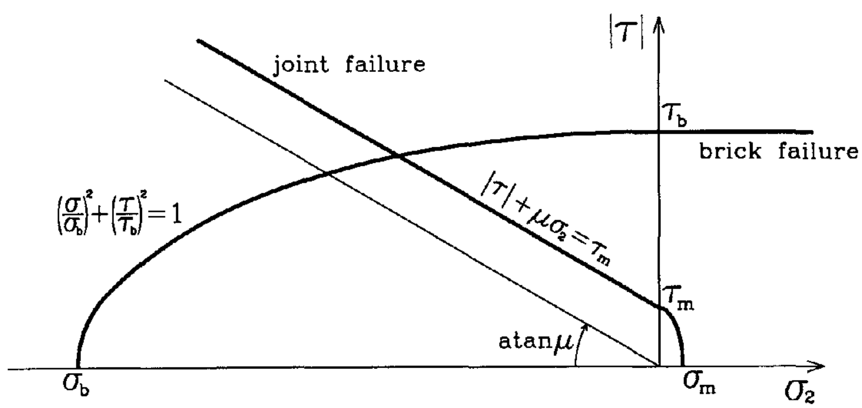

Figure 3. Joint and brick failure domains in the (σ_2, $|\tau|$) plane.

These simple assumptions for the brick model imply a brittle response of the masonry in compression, similar to that shown in Figure 1b for the mortar joint. On the other hand, a complex description of the inelastic deformation in bricks implies notable complexities in the application of the model.

The continuum model proposed has been applied in an incremental finite element procedure for the in-plane analysis of brick masonry walls based on isoparametric elements. The incremental solution is obtained by means of an iterative "initial stress" method whose fundamental peculiarity concerns the algorithm for the finite load step integration of the constitutive equations. In fact, this task turns out to be very complex due to the interaction of the three mechanisms of inelastic deformation involved in the present model. The finite increments of the stress and internal variables corresponding to finite strain increments are obtained by means of a predictor-corrector procedure, similar to that proposed by Simo et al. [11], in which the correction phase is based on a Newton-Raphson algorithm involving the limit functions of the active mechanisms at each iteration.

This procedure has been applied to analyze both rectangular shear walls, having the same geometry as that described in the previous section, and a large scale perforated wall; in both cases values of the model parameters directly correlated to those applied in the composite model previously described have been considered (Table I). Furthermore, in order to take into account size effects, rectangular finite elements were chosen with dimensions such that, for each point of integration, a representative volume element of the brick masonry is considered as having width equal to the mean distance between the head joints, and height equal to one course of bricks and mortar.

Table I. Model parameters.

	E_m (MPa)	ν_m	E_b (MPa)	ν_b	η_m
elastic moduli	539	0.18	2200	0.28	0.154
	σ_m (MPa)	τ_m (MPa)	μ	$1/c_{mt}$ (MPa)	β_m
mortar joint	0.05	0.23	0.577	520	0.8
	σ_b (MPa)	τ_b (MPa)	$1/c_{bn}$ (MPa)	β_m	
bricks	5	2	2500	0.4	

In the case of slender walls both the response and the collapse mode obtained by the continuum model are almost coincident with that obtained by the composite model. Small differences in the overall response of squat walls come out of the two models, even if the composite model supplies extension and dilation in its central zone that the continuum does not. The diagram in Figure 4 shows the response of the squat wall, which turns out to be very close to the experimental response [6]. In particular, at the state A (corresponding to the top deflection v=7 mm) the damage is spread over two diagonal bands and in the upper corners, as shown in Figure 5b; the corresponding stress state is characterized by two compressed parallel bands and an unloaded band between them (Figure 5a).

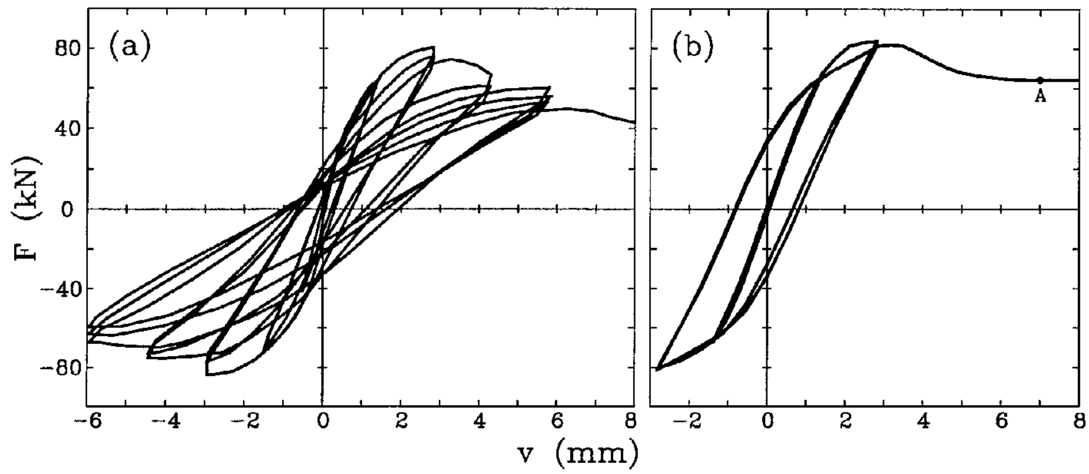

Figure 4. Lateral hysteretic response of a squat wall: (a) experimental, (b) continuum model.

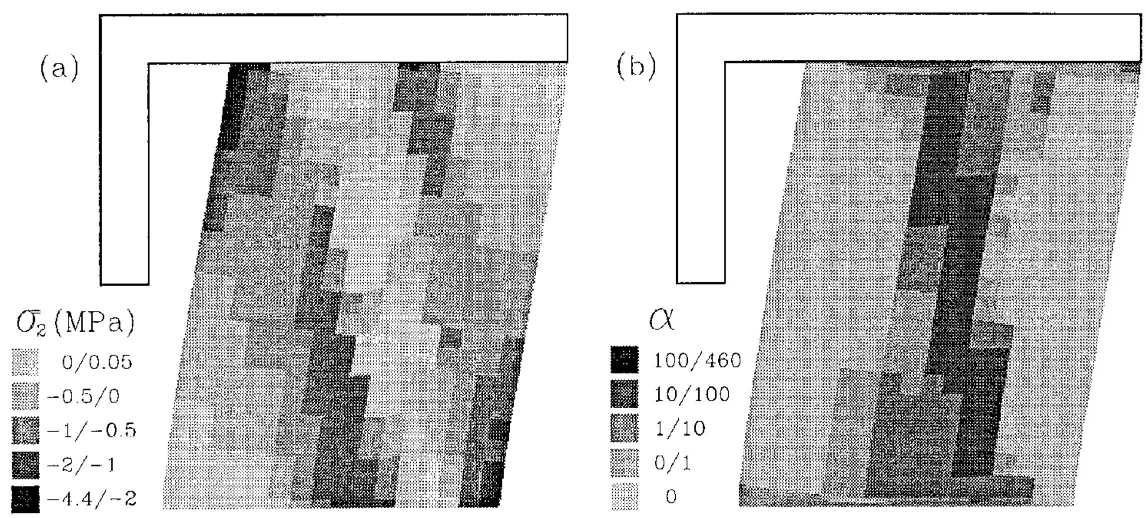

Figure 5. Rectangular wall: vertical stress (a) and joint damage α_m (b) distributions at the state A.

The second example concerns the simulation of the in-plane behavior of the perforated shear wall of the prototype brick masonry building experimented by Calvi et al. [12] in Pavia, the finite element model of which is shown in Figure 6. The experimetal hysteretic response to cyclic horizontal displacements applied to each floor level, imposing to the two story forces to be equal, is shown in the diagram of Figure 7a. The model response (Figure 7b) is able to catch both the horizontal strength of the wall (150 kN) and its dissipative behaviour; the small differences that appear in the first cycles may be attributed to a non uniform distribution of the joint strength in the real wall. Figure 8 shows the vertical stress and the mortar joints damage distributions corresponding to 0.2% drift. The damage evolution is characterized by a first degradation of the mortar joints over the four lintels, which appears at 0.1 % drift, followed by the damage at the base of the wall sections, and then of the central panel. Increasing the lateral displacement the damage propagates in the central panel in the form of a "Y" till the collapse. As shown in Figure 8a, the vertical stress is not very high, so the effects of bricks failure are limited; moreover, it can be observed that one lateral pier is unloaded, while in the central one there is an unloaded band in which extensions and slidings take place in the mortar joints. Finally, it is also worth noting that a considerable rotation at the top of the wall apperas, due to the non uniform compression in the piers.

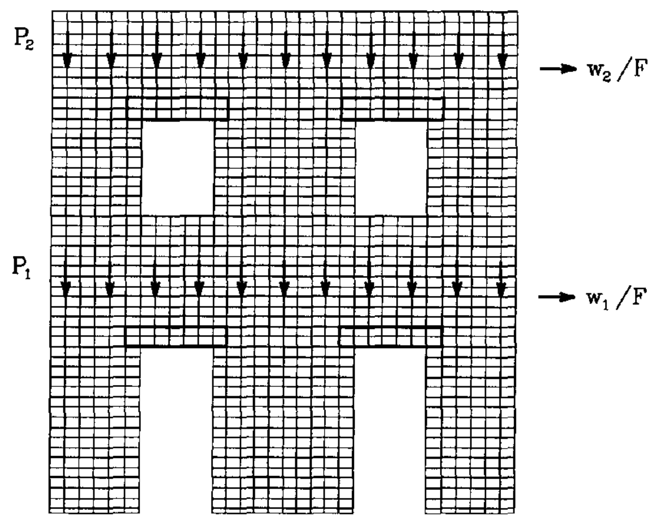

Figure 6: Finite element model of the large scale perforated wall [12].

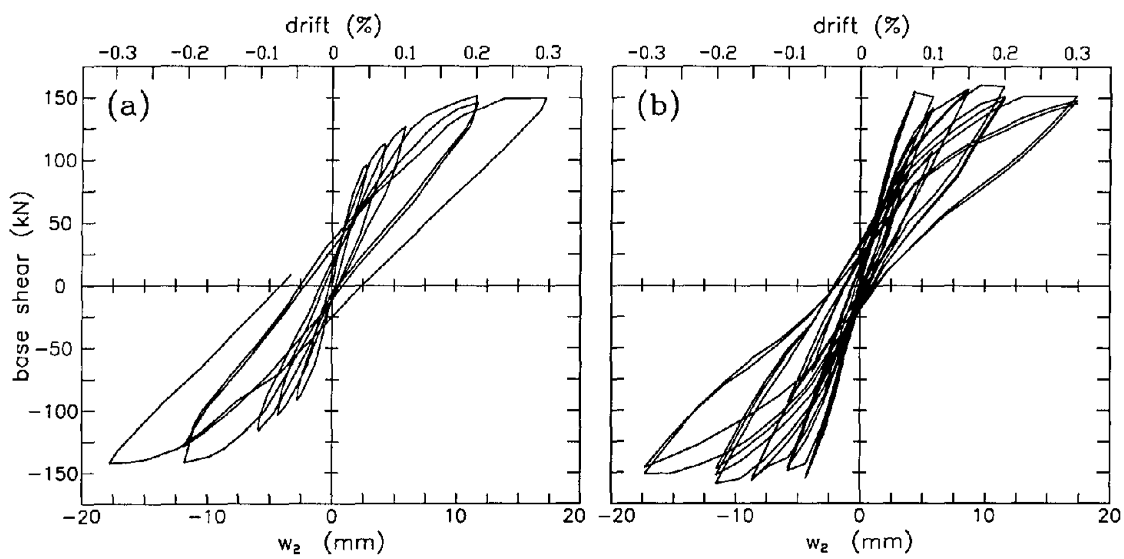

Figure 7. Experimental (a) and theoretical (b) lateral hysteretic response of the large scale perforated wall [12].

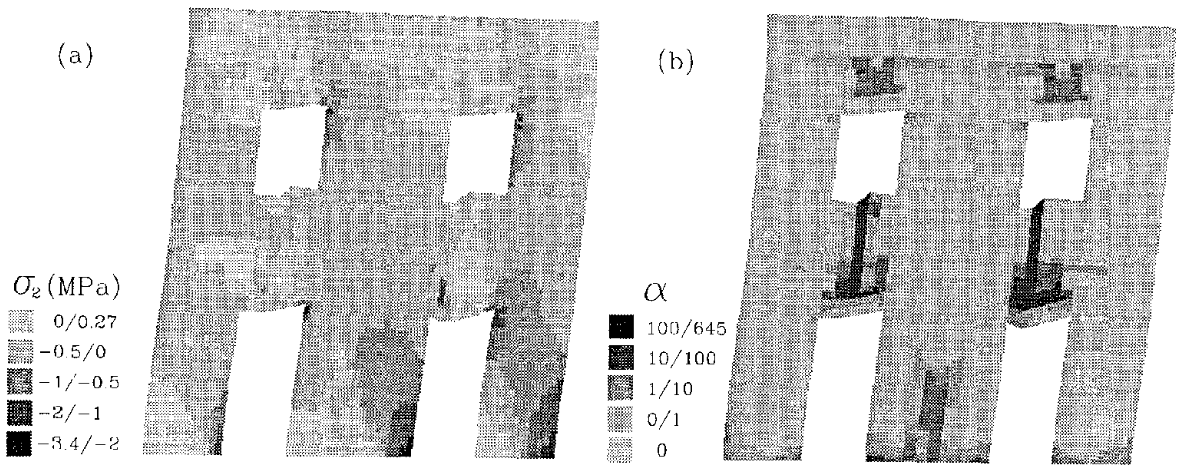

Figure 8. Large scale perforated wall: vertical stress (a) and mortar joint damage α_m (b) distributions for 0.2 % drift.

CONCLUSIONS

The proposed continuum model, which considers damage mechanisms both in mortar bed joints and in bricks, allows the response of masonry walls subjected to horizontal forces to be obtained with some accuracy. Moreover, several characteristics of the brick masonry behavior can be taken into account, such as brittleness under tensile stress, hysteretic response to cyclic loads, increasing of the lateral strength and ductility with compressive vertical stress, and influence of the slenderness of the walls on the collapse mechanism.

The simulation of some experimental results obtained from rectangular shear walls [6] and large scale perforated walls [12] have shown the good capabilities of the continuum model to describe the hysteretic response, the failure strength, the damage evolution and the post-peak behavior. On the other hand, the composite model presented here is a useful tool for stating the validity limits of the continuum model and thus improving it.

The proposed models could also be useful tools for setting-up experiments, for evaluating the influence of the material parameters and the geometry and, finally, for developing simplified procedures for the analysis of masonry buildings.

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FAILURE CRITERION FOR BRICK MASONRY UNDER IN PLANE LOAD: A MICROMECHANICAL APPROACH

Gianmarco de Felice [1]

ABSTRACT

An homogenization procedure is outlined to identify the overall mechanical properties of masonry by a micromechanical approach. Starting from the sole knowledge of the strength properties of their constituents (bricks and mortar) and of their disposition within the wall, a failure condition is analitically obtained for single lined brickwork. All the parameters in the equation have a clear mechanical meaning and, as expected from experimental results, strength properties explicitly depend on the geometry of bricks. The above failure condition is finally applied to compute the collapse load of walls subjected to in plane increasing loads and its effectiveness is cheked comparing the results with experimental ones and with other numerical approachs.

INTRODUCTION

As pointed out by several tests (12, 15) a correct modeling of historical masonry has to be related to the geometry of brick units and to their texture within the wall. This is required especially when the mechanical properties of masonry are highly eterogeneous (mortar is much weaker than brick). To better undersand the importance of the inner structure of masonry, small scale brick walls without mortar have been tested by placing the wall on a table which was slowly inclined until failure occurs (2). The results (*Figure 1*) clearly show how the shape of the bricks influences the overall resistence of the wall.

With this in mind, the main task of this study is to bring to the level of the overall response of masonry the informations regarding its inner structure. This is the first step towards a general procedure for identifying the macroscopic properties of masonry, from those of bricks and mortar, by means of a micromechanical approach.

The basic assumption of such approach is that masonry can be regarded as a periodic medium obtained by the repetition of a volume element. While in a brickwork the presence of such representative volume is obvious, in an irregular stone masonry, the representative volume is hidden behind the complexity of masonry work but it still exists and can be identified. As for brick masonry, also for row stones masonry, geometrical rules exist regarding size and disposition

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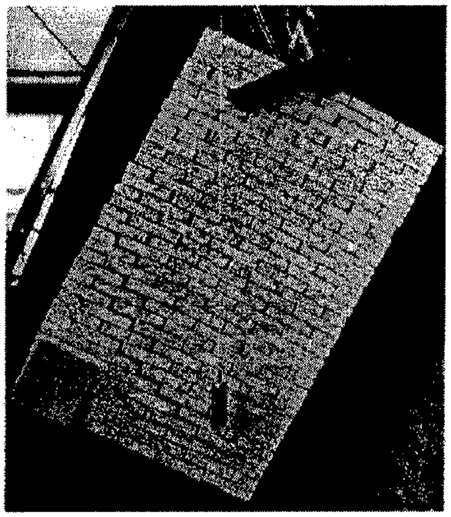
of the units in masonry texture; those rules booth depend on the availability of materials and on the construction techniques of the region. They can be deduced by a survey of masonry and a representative volume made by stones and mortar in a particular arrangement can be identified (6) Althoug the attention will be focused on brick masonry, the micromechanical approach wich is presented in the following can be applied (not trivially) to stone masonry too.



$\lambda = 0{,}48$

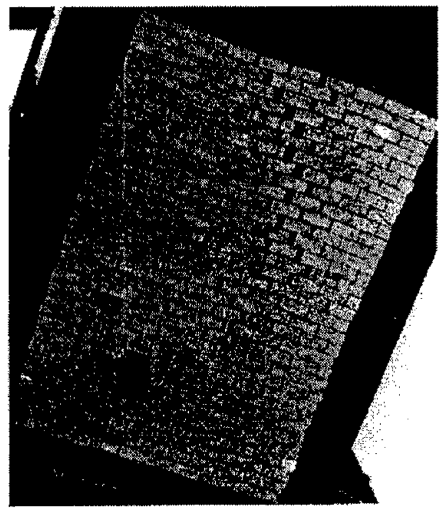


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Figure 1 Experimental failure configuration and corresponding horizontal load (normalized with respect to vertical load) of two walls with different brick-ratio.

BETWEEN MICRO AND MACRO MODELING OF MASONRY

Statement of the problem

We assume to know, for each constituent (brick and mortar) of masonry, the set G(**x**) of all the stress states admissibles in the material:

$$\tau(\mathbf{x}) \in G(\mathbf{x}) \qquad\qquad \mathbf{x} \in V \tag{1}$$

According to the heterogeneity, the set G, wich define the strength properties of the medium, is a periodic funtion of the place, while the behaviour of the material is no further specified.

A direct approach to analyse such a structure requires the exact discretization of the wall according to the heterogeneity. The main difficulty of such a model depends on the fact that heterogeneity dimension is small when compared to the dimension of the wall and the computational effort, related to a separated modelling of bricks and joints, becomes quickly too high. Several approach have been performed for a micromodelling of masonry: contact elements are employed in the framework of non linear finite element analysis (1, 10); a direct limit analysis

formulation for block structures have appeard in litterature (9, 1); recently brick walls subjected to quasistatic lateral loading have been modeled in the contest of discrete element method (Giuffrè, Pagnoni e Tocci, 1994). Anyway only simple panels have been analysed, while for application to engineering problems a macroscopic description of masonry is required.
To search for homogeneous (or homogenized) material, having overall mechanical properties analogous to that of the heterogeneous medium, experimental or phenomenological methods can be used. Several attempts to calibrate macroscopic models by experimental results have been developed for masonry brickwork, but generally a large number of parameters, without a clear mechanical meaning, are involved in the descritpion of the overall behaviour; moreover the validity of the model is restricted to specifics of the sample.
In alternative, a micromechanical approach can be used by means of a homogenization technique.

THE HOMOGENIZATION APPROACH

The assumed periodicity of the geometry and of the mechanical behaviour of masonry allow to apply the homogenization method in the framework of periodic media. This is very convenient in order to obtain, in a precise form, the passage from a microscopic description of the problem to a macroscopic one.
The macroscopic or homogenized model can be derived by means of an homogenization technique, regarding the asimptotic behaviour as a scale factor μ tends to 0; where μ represents the small ratio ℓ/L between the dimension of representative volume element ℓ and of the overall structure L (*Figure* 2).

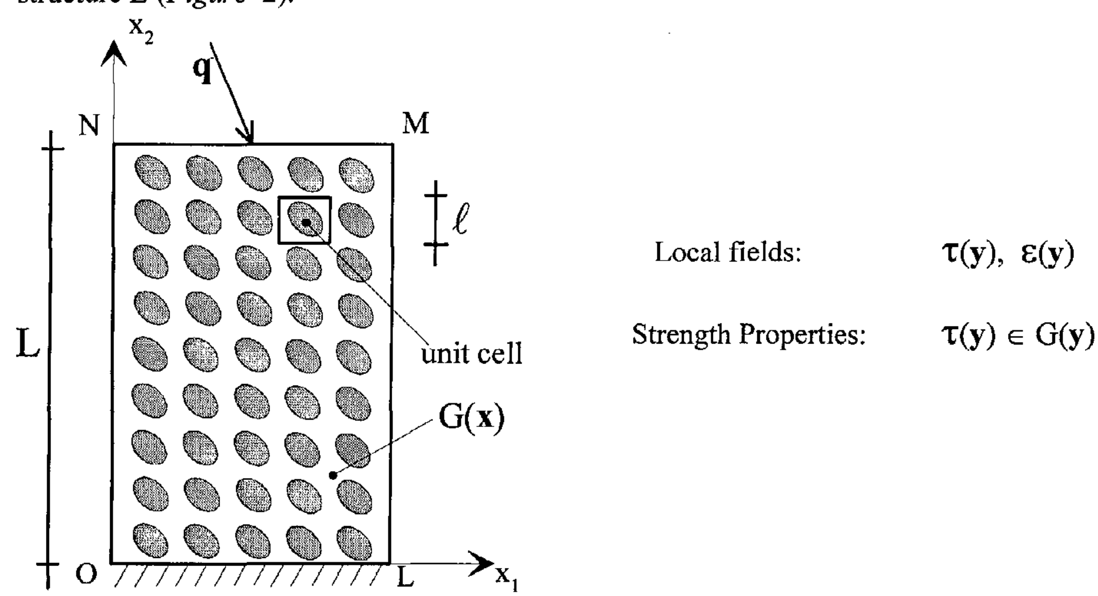


Figure 2 Initial Problem on a heterogeneous periodic material

It results, from arguments on oscillating functions, that the macroscopic stress **T** and strain **E** tensors must be the averages of the corresponding microscopic fields

$$\mathbf{T} = \lim_{\mu \to 0} \tau(\mathbf{x}) = <\tau(\mathbf{x})>,$$
$$\mathbf{E} = \lim_{\mu \to 0} \varepsilon(\mathbf{x}) = <\varepsilon(\mathbf{x})>; \tag{2}$$

where <.> stands for the averaging operator on the period of the microstructure

$$<.> = \frac{1}{\Omega} \int_{\Omega} . \, dV.$$

Since the local stress field τ is constrained by the constitutive equation (1), its average, the macroscopic stress $\mathbf{T}$, has to be constrained too:

$$\mathbf{T}(\mathbf{x}) \in G^{hom} \qquad \mathbf{x} \in V \tag{3}$$

where, since the structure has assumed to be strictly periodic, the macroscopic properties of the homogenized structure, named G^{hom}, are constant. While the initial problem regards an heterogeneous material, where strength properties are defined by the set G(x) depending on the position in accord to the hererogeneity, the homogenized problem (*Figure* 3) regards an homogeneous fictitious material, with constant strength properties defined by G^{hom}.

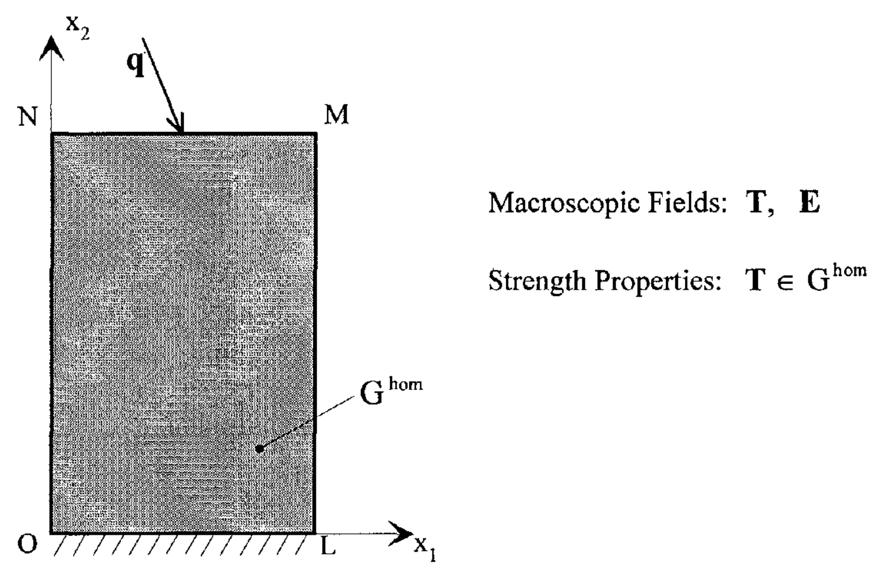


Figure 3 Homogenized Problem on a continuous homogeneous material

Clearly we still have to construct the macroscopic set G^{hom} and to demonstrate that, as the scale factor μ tends to 0, the limit load of the initial heterogeneous periodic material tends to the limit load of homogenized material. As usually in homogenization methods, the macroscopic properties may be obtained by a solution of a boundary value problem wihtin the basic period of the structure. A suitable definition of the macroscopic strength condition (17) is the following:

G^{hom} is the set of macroscopic states of stress $\mathbf{T}$ fulfilling the following requirements:

$$\begin{array}{ll} \text{mean condition:} & \mathbf{T} = \langle \tau(\mathbf{y}) \rangle \\ \text{equilibrium:} & \operatorname{div} \tau(\mathbf{y}) = \mathbf{0}, \quad \mathbf{y} \in \Omega \\ \text{constitutive equation:} & \tau(\mathbf{y}) \in G(\mathbf{y}), \quad \mathbf{y} \in \Omega \\ \text{boundary conditions:} & \tau(\mathbf{y})\, \mathbf{n}(\mathbf{y}) \ \text{antiperiodic} \end{array} \tag{4}$$

and the proof of convergence is given except "edge effects" occuring at the boundary of the structure.
The macroscopic strength condition relies on the solution of a yield design problem (*Figure* 4) where the load consists in the average value of the field of stress within the period of the structure.

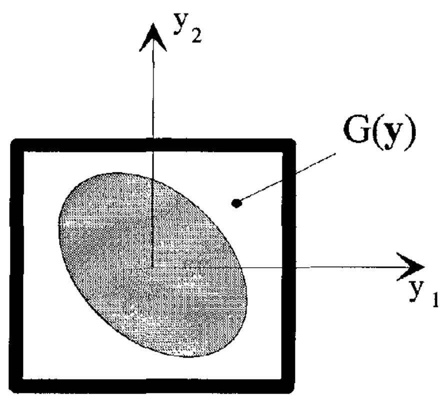
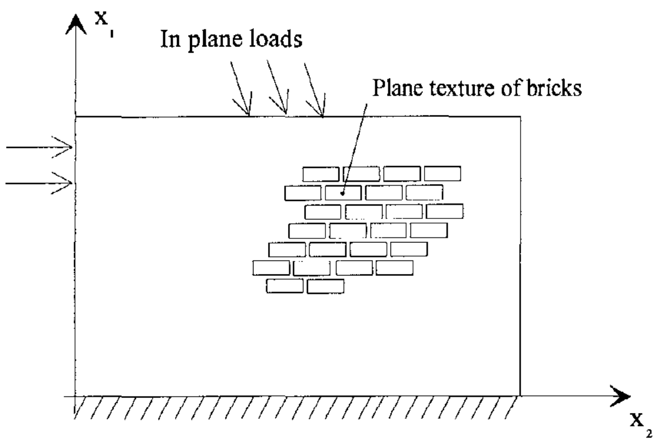


Figure 4 Auxiliary yield design boundary value problem on the basic period of the structure.

THE BRICK MASONRY MODEL

Recently, some attempts to analyse brick masonry as a composite material have appeared in litterature; in order to apply a simplyfied homogenization procedure some kind of assumptions are made. For example, masonry is regarded as a continuum made by a mortar matrix with bricks as reinforcing inclusions (11) or alternatively, as a brick matrix intercepted by mortar weak inclusions (4). In other cases (14, 15, 5) the presence of vertical joints is neglected and brickwork is regarded as a multilayred material. This is not completely satisfactory with respect to failure condition, because, as shown by experimental tests (*Figure* 1), the interaxe of vertical joints strongly affects the strength of masonry.



i) masonry has a plane texture

ii) masonry is subjected to in plane loads

iii) mortar joints are simply interfaces between bricks

Figure 5 Assumptions

In this approach the effect of brick geometry on masonry strength is explicitely considered in the model, while some other simplifications will be assumed:
i) only on line texture of bricks is regarded, even if usually masonry work has a more complex inner structure, because bricks are arranged both in the front and in the section of the wall.
ii) only in plane loads are considered and a plane stress calculation is performed, disregarding the possibility of a splitting failure in the plane of the wall. This kind of failure is expected in the case of uniform compression, as outlined by experimental tests (13).
iii) finally, mortar joints are modeled simply as interfaces between bricks. This last assumption leads to an overextimation of compressive strength which can be explained by referring to the case of axial load: because of the different deformabilities of brick and mortar, as pointed out by Hilsdorf (8), lateral compressive stress appear in the mortar, while the bricks are subjected to lateral tensil stress. Therefore the compressive strength of brickwork is lower then the strength of brick unit.

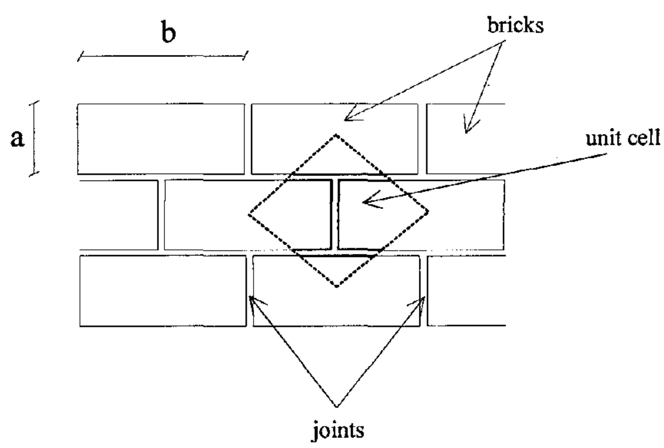


Figure 6 Unit cell of on line brickwork

The above assumptions allow to define the geometrical properties of brickwork only by a parameter wich mesure the form ratio, high to wide, of a single brick unit:

$$m = \frac{a}{b}$$

Mechanical properties of components of masonry are assumed as follow:
i) for bricks the set g^{bricks} of admissible states of stress is defined by means of a Mohr-Coulomb criterion in plane stress; two parameters ft and fm, respectively the tensile and compressive strength, are involved in the yield condition (*Figure 7*).

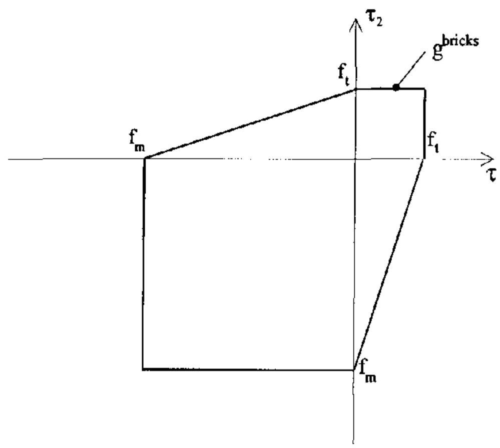


Figure 7 Strength condition for bricks

ii) for joints for bricks the set g^{joints} of admissible states of stress is defined by means of a Coulomb criterion with cohesion (c) and coulomb friction (φ)

THE MACROSCOPIC STRENGTH CONDITION

We take in account separately, in two steps, the contribute of joints and bricks to the macroscopic strength condition for masonry.

Influence of joints

We first consider only the joint effect (that is equal of considering bricks infinitely resistent): the resolution of the auxiliary yield design problem, on the basic period of the structure, leads to an analitical expression for the macroscopic strength condition (3). Clearly the macroscopic domain is open in the direction of compression and the shape of the failure surface is anisotropic. As shown in *Figure 8*, the shape of the domain in the plane of principal stress depends on the angle θ between principal stress directions and joints directions.

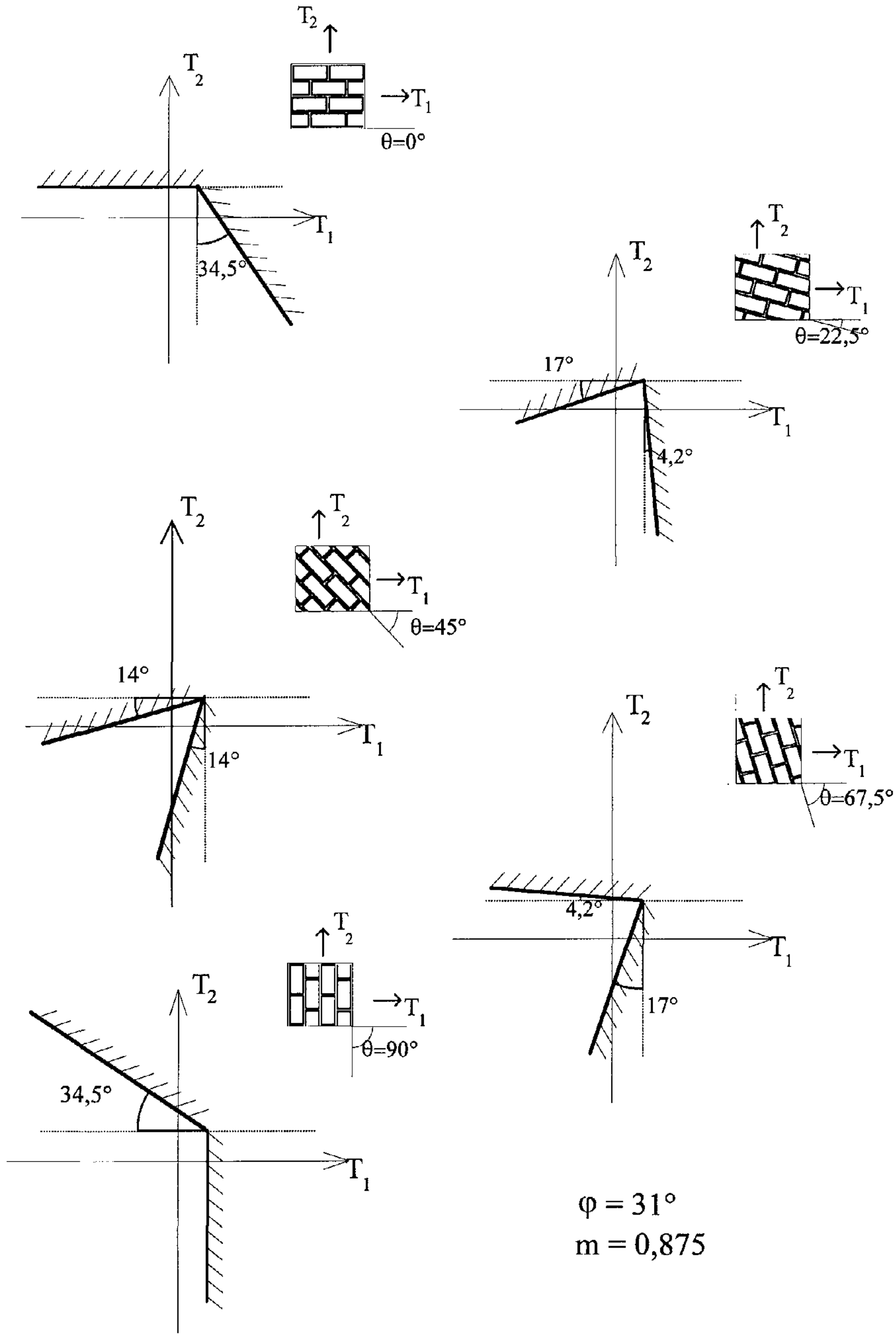


Figure 8 Macroscopic strength condition whit respect to joints failure in the plane of principal stress

The projection of the domain in the stress space, on the refernce of joint directions, leads to the following equations (*Figure 9*):

$$\mathbf{T} \in G^{\text{joints}} \Leftrightarrow \begin{cases} \Phi_{1,2}(\mathbf{T}) = \mp T_{12} + f\ T_{22} - c \le 0 \\ \Phi_{3,4}(\mathbf{T}) = +2mT_{11} \mp (2mf+1)T_{12} - fT_{22} - c\,(2m/f+1) \le 0 \end{cases} \tag{5}$$

where both, mechanical properties of joints (cohesion c and friction coefficient $f = \tan\varphi$) and geometry of bricks (*form ratio* $m = \frac{a}{b}$) explicitely appear in the formulation.

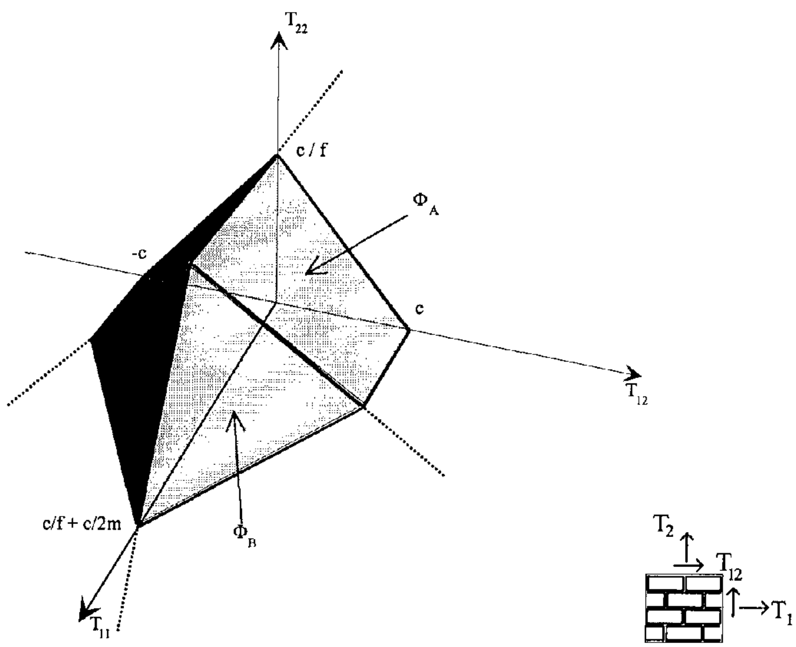


Figure 9 Macroscopic strength condition whit respect to joint failure in the stress space on the refernce of joint directions

The expressions obtained define the macroscopic strength condition of masonry in the tension range, while in the compression range, the resistence of bricks has to be taken in account. The criterion, analitically obtained by a micromechanical approach, is quite similar to the one obtained by Page (12), interpolating numerical results performed by finite elements method (*Figure 10*).

Influence of joints and bricks

For taking in account the strength condition in the bricks, the auxiliary boundary value problem on the basic period of the structure cannot be solved analitically, but a numerical procedure has to be followed.

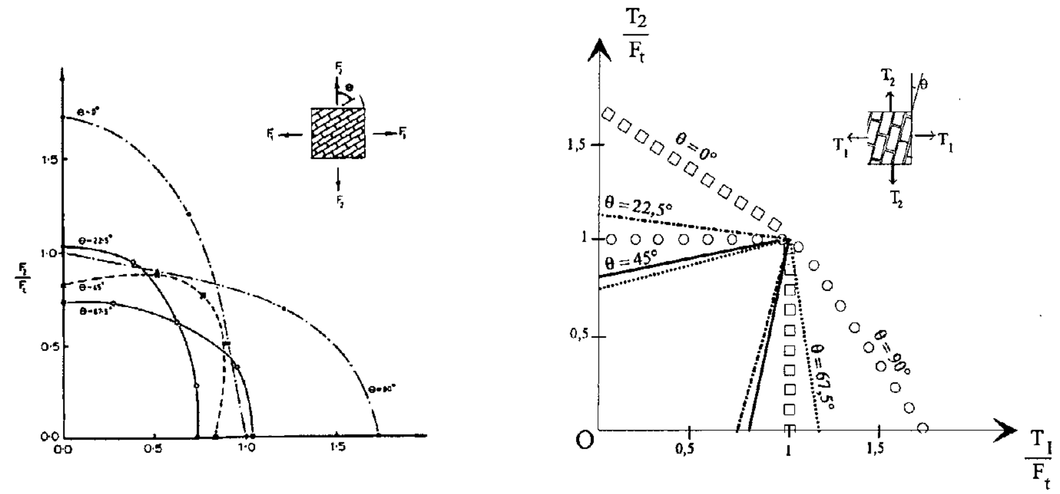


Figure 10 Comparison with the failure condition numerically obtained by Page (12) in the tension range

Anyway, the macroscopic domain can be bounded in a very simple way. It can be demonstrated (3) that such a domain named G^{hom} is bounded, from above, by the intersection between the domain G^{joints}, previously obtained by considering only the joints effect, and the domain G^{bricks} of the admissible states of stress in the bricks; similary it can be bounded, from below, by the intrsection of the domain G^{bricks} and the domain G^{static}, wich is the lower bound domain of joints effects, obtained by a static approach with uniform stress fields:

$$G^{static} \cap G^{bricks} \subset G^{hom} \subset G^{joints} \cap G^{bricks} \tag{6}$$

where:

$$\begin{gathered} G^{hom} = \{\mathbf{T} \mid \exists\, \boldsymbol{\tau} \text{ S.A.}\mathbf{T};\ \boldsymbol{\tau} \in g^{joints} \cap g^{bricks}\} \ ^{(2)} \\ G^{bricks} = \{\mathbf{T} \mid \exists\, \boldsymbol{\tau} \text{ S.A.}\mathbf{T};\ \boldsymbol{\tau} \in g^{bricks}\} = g^{bricks} \\ G^{joints} = \{\mathbf{T} \mid \exists\, \boldsymbol{\tau} \text{ S.A.}\mathbf{T};\ \boldsymbol{\tau} \in g^{joints}\} \\ G^{static} = \{\mathbf{T} \mid \mathbf{T} = \boldsymbol{\tau} \in g^{joints}\} \end{gathered}$$

The two bounds are shown in the following Figures for different value of the angle θ between joints and principal stress directions.

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(2) S.A. stands for statically admissible and means that equilibrium, mean condition and boundary conditions are satisfied.

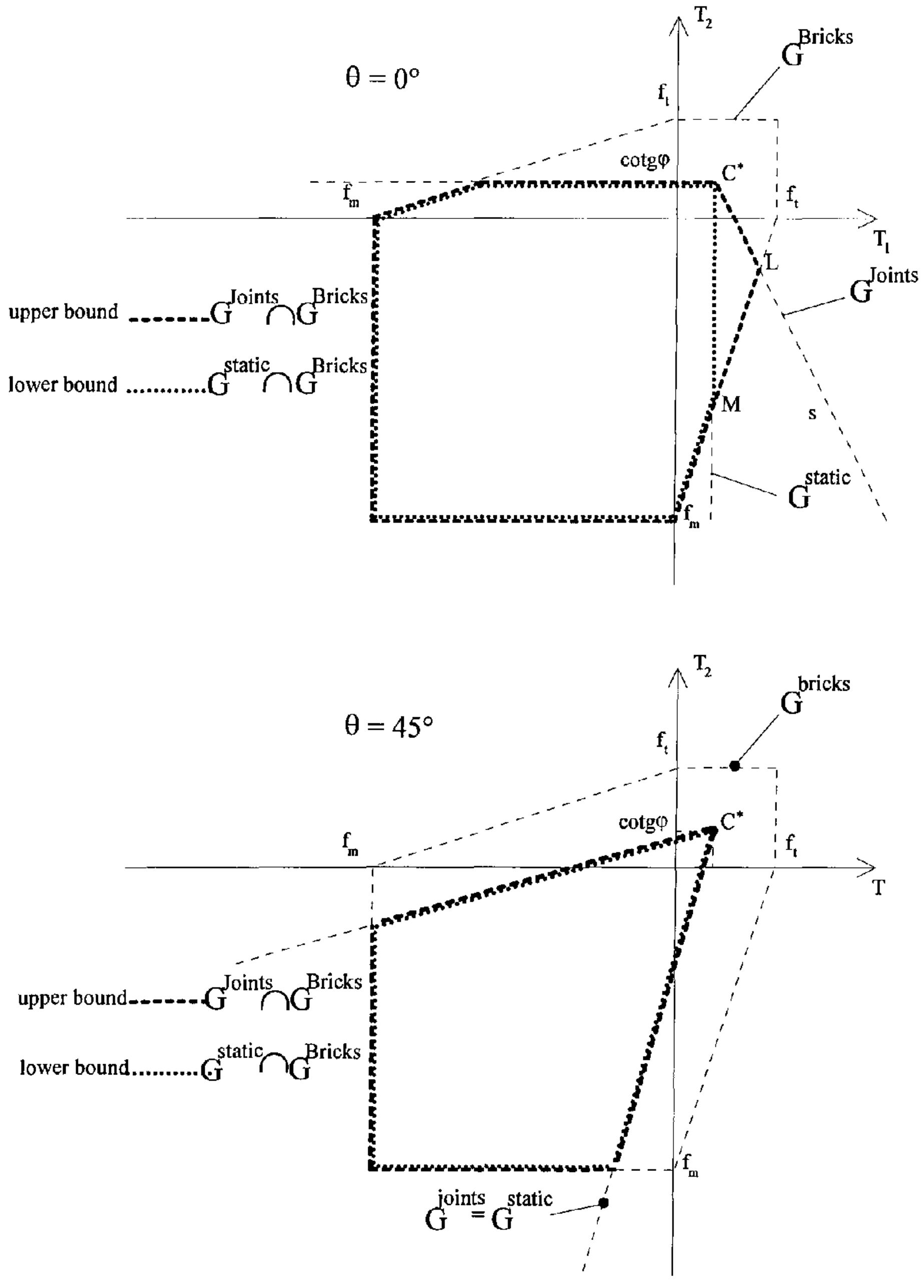

Figure 11 Macroscopic strength condition with respect to joint and brick failure

LIMIT ANALYSIS OF IN PLANE LOADED WALLS

Once the auxiliary problem has been solved and the macroscopic strength condition has been obtained, the homogenized problem can be threated by a traditional finite element analisis. Regarding only the limit behaviour of the structure, an elastic-perfectly plastic model can be used,

where:
i) the elastic properties can be arbitrary;
ii) the limit surface of elastic domain is given by the resolution of the auxiliary problem of homogenization procedure;
iii) the flow rule can be assumed as associated or not to the criterion (in the first case only an overestimate of the limit load will be reached).

In order to validate the previously described procedure, three walls with different brick shape, in absence of mortar, have been tested on the inclined table in the laboratory and corresponding discrete element computation has been performed (7). The results of homogenized model (*Figure 12*), requiring only few computational effort, apper to be, both in the collapse load value and in the failure configuration, very similar to those of the discrete element method.

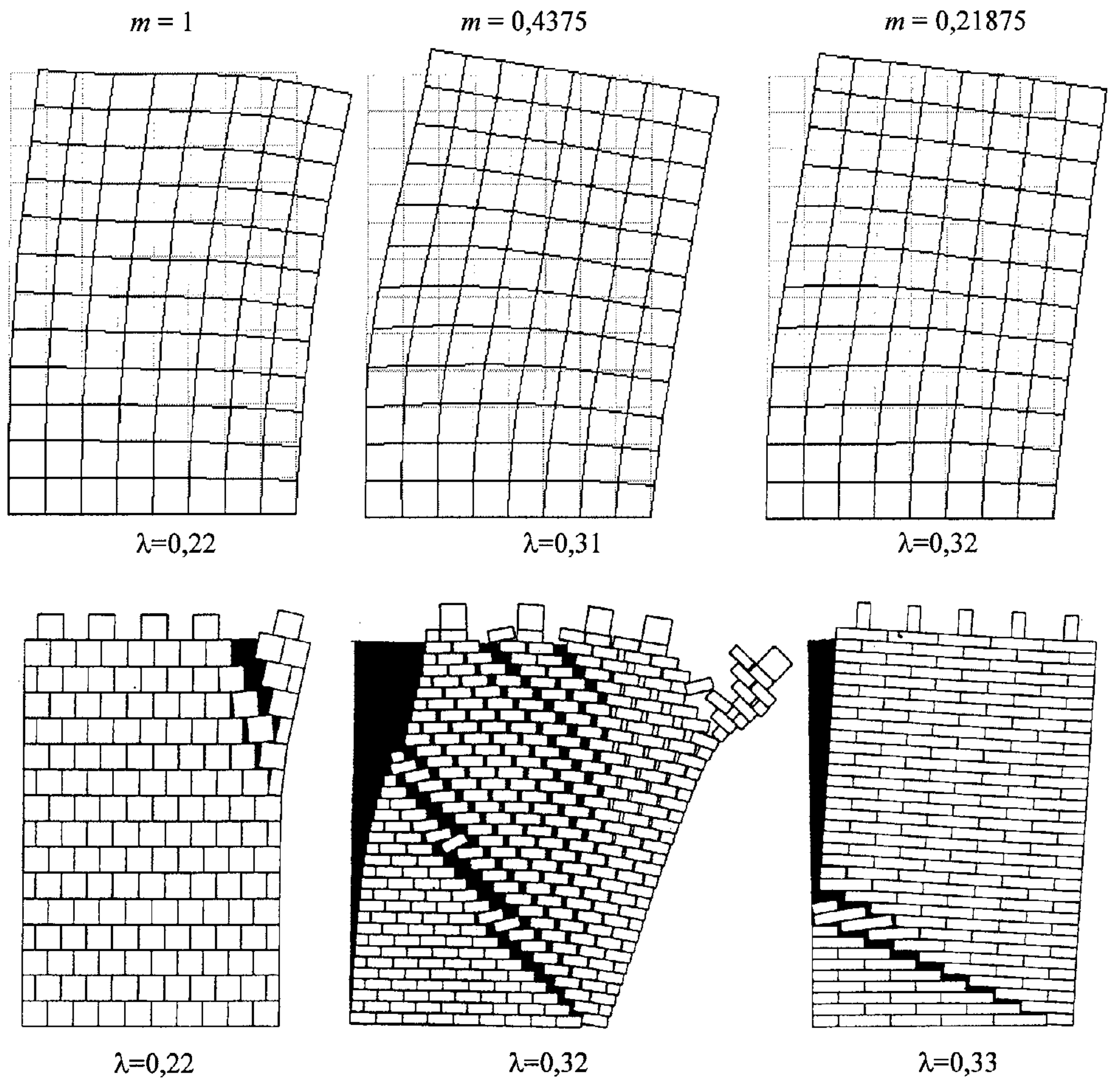

Figure 12 Comparison between homogenized and discrete structure limit behaviour

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Section V

Case Studies of Building Preservation Projects

REHABILITATION OF URM BUILDINGS IN THE EASTERN UNITED STATES

John C. Theiss, P.E.[1]

ABSTRACT

This paper describes the methodology used to evaluate existing unreinforced masonry buildings subject to strong ground motion and the methods and assumptions used to strengthen the buildings. The technical criteria and the owner performance objectives are presented along with the retrofit strategies that were used to strengthen the buildings.

INTRODUCTION

The earthquake history of the Central United States is dominated by the series of great earthquake which ruptured the New Madrid Seismic Zone in the winter of 1811-1812. On December 16, 1811, three earthquakes ruptured the entire southern segment of the New Madrid Seismic Zone, a length of about 90 miles. These earthquakes had magnitudes of 8.6 (2:30 a.m.), 8.0 (8:15 a.m.), and 8.0 (noon). On January 23, 1812, another great earthquake having a magnitude of 8.4 ruptured the central segment of the fault, a distance of about 45 miles. On February 7, 1812, the last and largest earthquake in this series occurred near the town of New Madrid. This earthquake had a magnitude of 8.8 and ruptured the entire northern branch of the fault zone. Between the occurrence of the first earthquake on December 16, 1811 and March 15, 1812, the aftershock sequence included:

- 5 earthquakes of magnitude 7.7
- 10 earthquakes of magnitude 6.7
- 35 earthquakes of magnitude 5.9
- 65 earthquakes of magnitude 5.3
- 89 earthquakes of magnitude 4.3

Eighteen of the 1811-1812 New Madrid earthquakes were felt as far away as Washington, D.C. A total of about 1600 discernible aftershocks occurred in this three month period. About as many earthquakes occurred in the Mississippi Valley in this three month period as occurred in Southern California in the 40 year period between 1932-1972.

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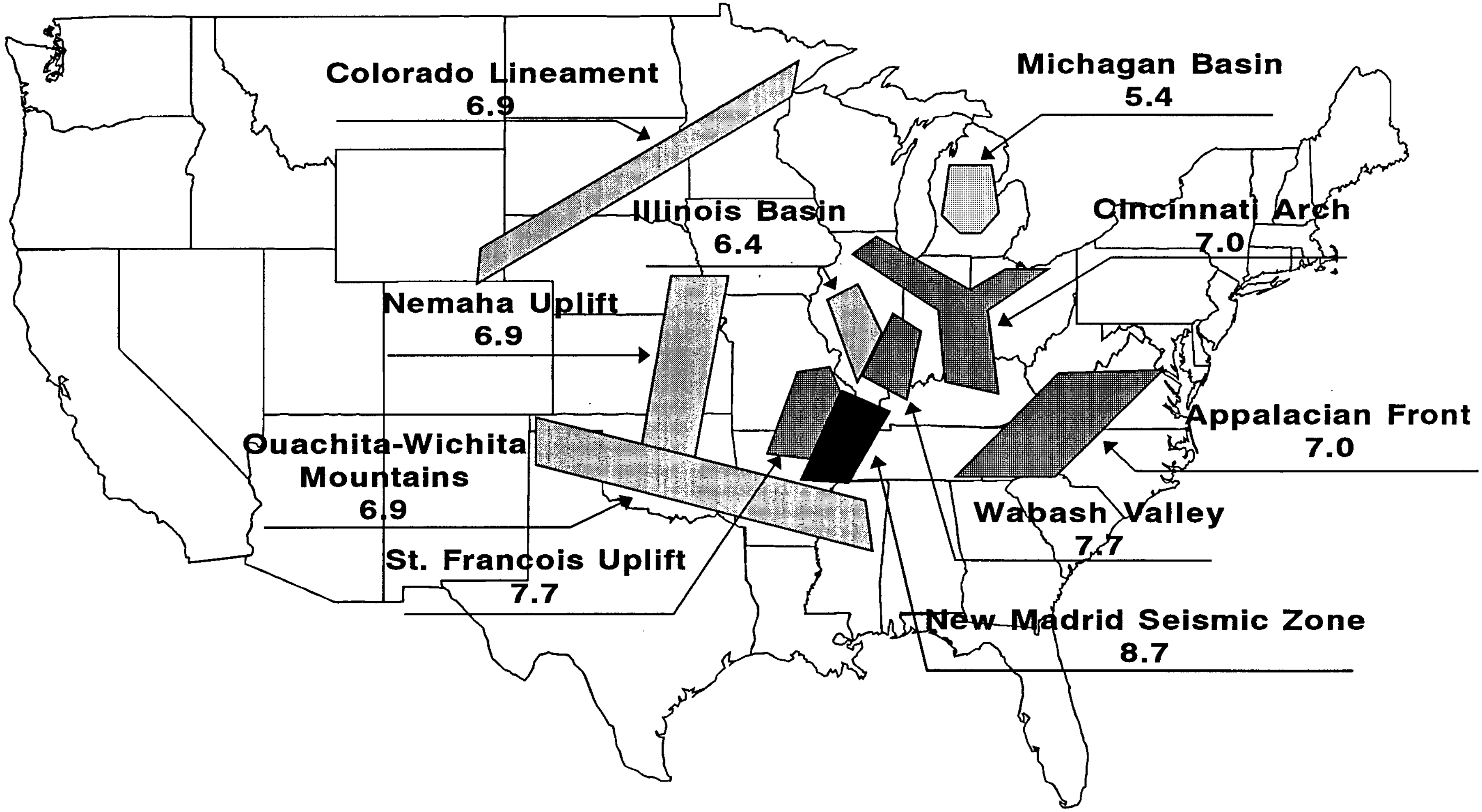
[1]President, Theiss Engineers, St. Louis, Missouri, USA

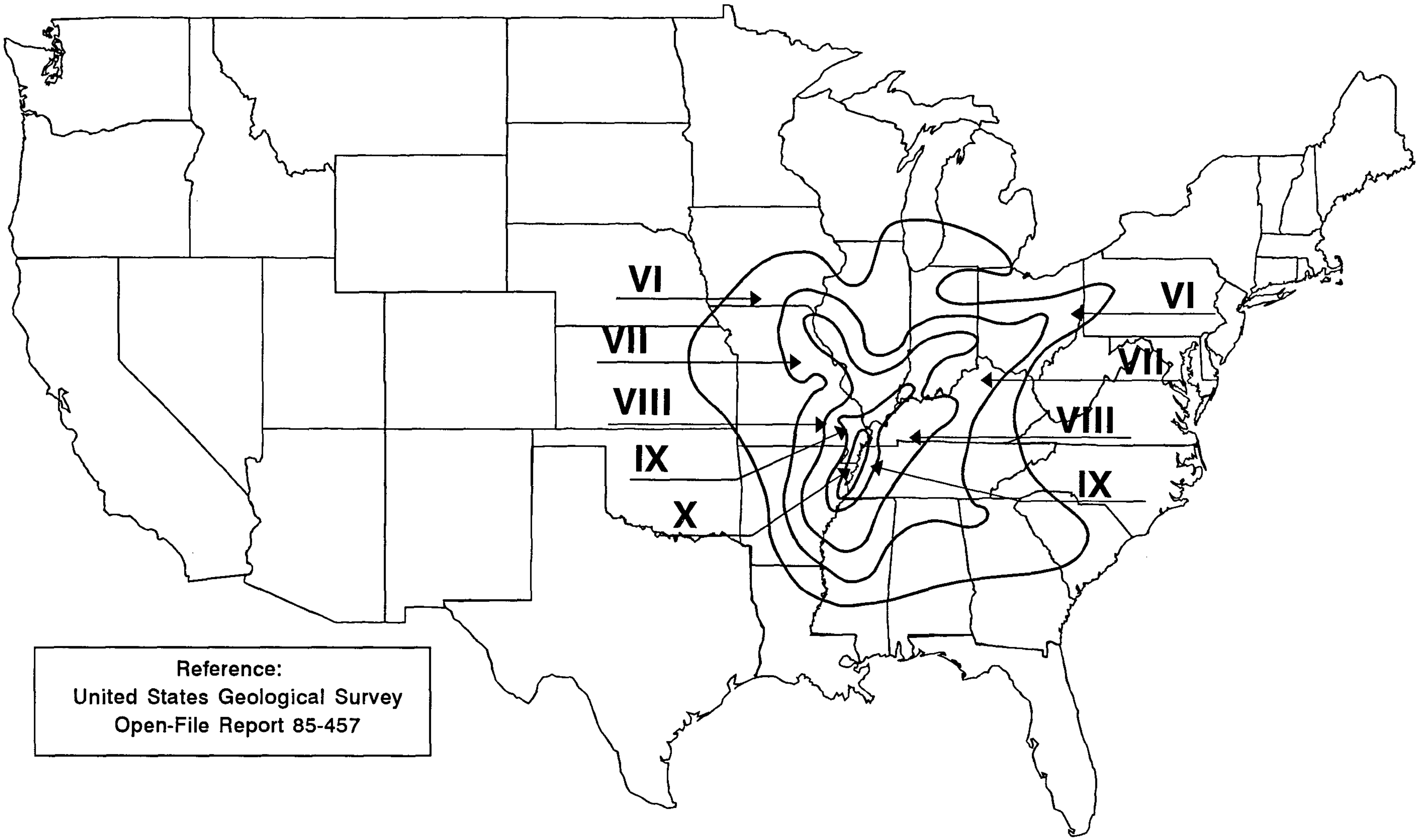
Central United States Earthquake Source Zones

Surface Wave Magnitudes With 1000 Year Recurrence Interval

Reference: Nuttli, 1985



MMI Contours for an 7.6 Magnitude Earthquake Epicenter in the New Madrid Seismic Zone



Reference:
United States Geological Survey
Open-File Report 85-457

The New Madrid Seismic Zone is the most active seismic zone in the Mississippi Valley. The "average behavior" of this seismic zone includes 350 measurable seismic events per year.

SEISMIC RISKS

A probabilistic study of the earthquake risk in the New Madrid Seismic Zone has been published by Arch C. Johnston and Susan J. Nava of Memphis State University. This study modeled the behavior of the New Madrid fault system by using four different probability functions. After a thorough investigation of the data, they presented the following estimates of earthquake risk from the New Madrid Seismic Zone:

MAGNITUDE	AVERAGE REPEAT TIME (YEARS)	PROBABILITY IN 15 YEARS	PROBABILITY IN 50 YEARS
6.3	70 (+- 15)	40 - 63	86 - 97
7.6	254 (+- 60)	5.4 - 8.7	19 - 29
8.3	550 (+- 125)	0.3 - 1.0	2.7 - 4.0

PERFORMANCE OBJECTIVES

The evaluation of an existing building is based upon the criteria and limitations established by the Owner, the Building Code and the National Earthquake Hazard Reduction Program (NEHRP), Handbook for the Seismic Evaluation of Existing Buildings.

The performance objectives were established with the Owner at the beginning of the project. These objectives fall into two categories:

a) Life Safety
b) Continual Operation

Those buildings designed for life safety considerations only are based upon the minimum requirements as specified in the Building Codes and NEHRP Provisions for ordinary buildings.

ACCEPTANCE CRITERIA

The acceptance criteria for out-of-plane bending was based upon the allowable H/T ratios. Diaphragms were checked based upon demand capacity ratios.

SEISMIC DESIGN CRITERIA

The seismic design criteria given in the model building codes used in the United States are generally based on a ground motion which has a 10% probability of occurring in a 50 year exposure period. There are two seismic design approaches currently in use in the United States. The first is working stress design; the second is strength design. The following is a summary of each method:

Working Stress Design:

Guidelines: SEAOC Blue Book

Building Code: 1987 BOCA

Base Shear Formula:

$V = (Z\,I\,K\,C\,S) * W$

Z = Seismic Zone Coefficient

I = Importance Factor

$I = 1.5$ for essential facilities

K = Ductility Factor for Framing System

$K = 4.00$ for unreinforced masonry

S = Site Factor

C = Adjustment for Fundamental Period

$C = 1 / (15*T**1/2)$

An upper limit is placed on V by:

$C \leq 0.12$ & $C * S \leq 0.14$

Strength Design:

Guidelines: 1991 NEHRP Recommended Provisions

Building Codes: 1993 BOCA

Base Shear Formula:

$$V = \frac{1.2 A_v S}{R T^{2/3}} \leq \frac{2.5 A_a}{R}$$

A_a = Effective Peak Acceleration (EPA)

A_v = Effective Peak Velocity-Related Acceleration (EPV)

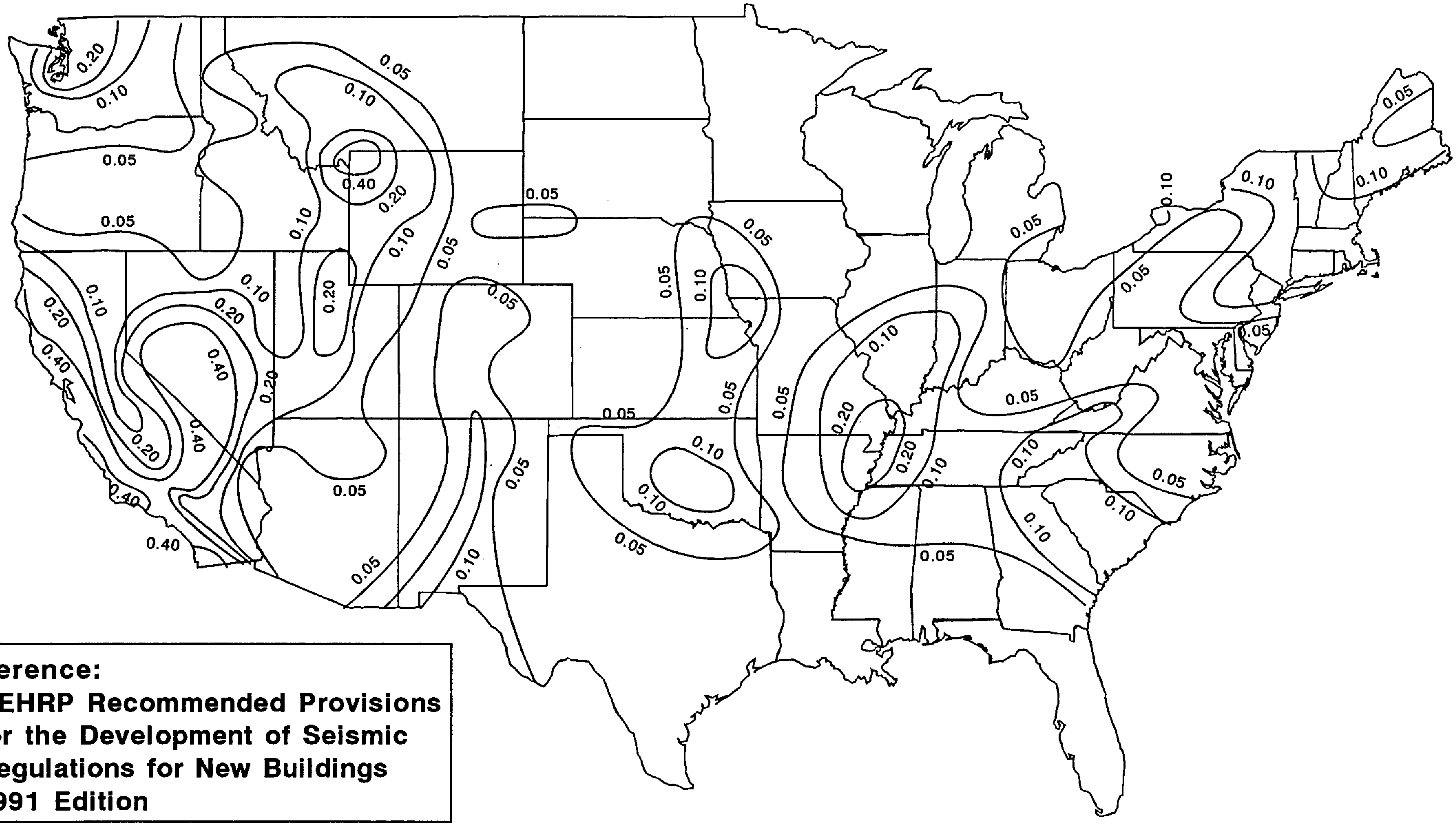
R = System Response Modification Factor

$R = 1.25$ for Unreinforced Masonry

Maps giving values for the Effective Peak Acceleration coefficient "Aa" and the Effective Peak Velocity-related acceleration coefficient "Av" are given in the commentary to the NEHRP Recommended Provisions.

Contour Map for Coefficient A_a

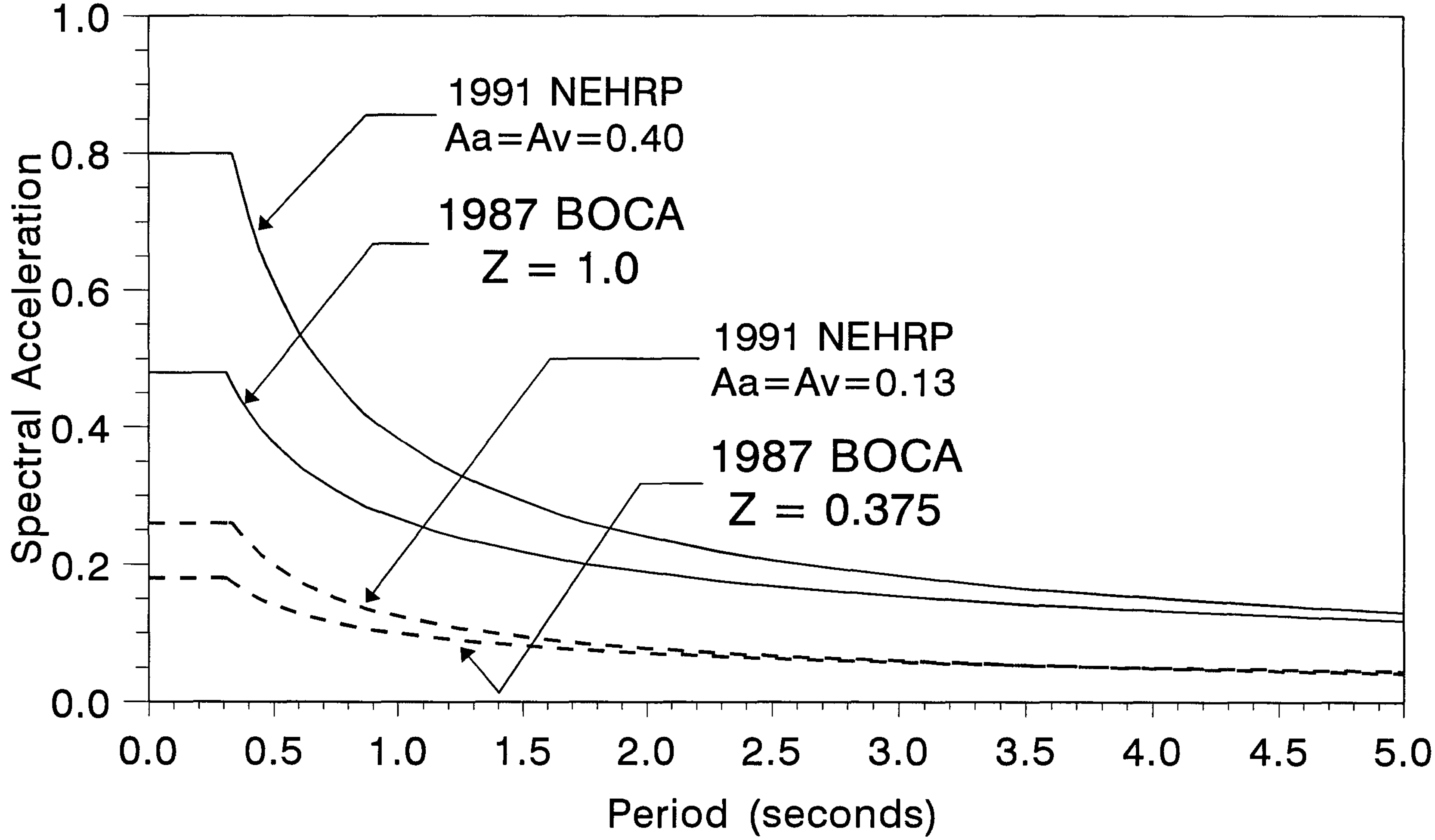
10% Probability of Exceedence in 50 Years



Reference:
NEHRP Recommended Provisions for the Development of Seismic Regulations for New Buildings 1991 Edition

Response Spectrum Comparison

R = 1.25   K = 4.00   S = 1



EVALUATION

Guidelines for the evaluation of the seismic resistance of existing buildings are given in the document NEHRP Handbook for the Seismic Evaluation of Existing Buildings (FEMA 178), published by the Building Seismic Safety Council. This evaluation process consists of identifying weak links in the building's lateral load resisting system, and methods for determining the strength of existing structural elements. The following general procedure is given:

- Data Collection
- Building Classification
- Analysis
- Design Checks
- Evaluation Statements
- Follow-Up Field Work
- Materials Testing
- Final Evaluation

DATA COLLECTION

Data collection consists of examining available drawings, specifications, and soils reports from the original construction. Site visits are imperative to verify that the existing construction matches that on the drawings. It is not unusual for a building to have been substantially altered by subsequent work.

BUILDING CLASSIFICATION

The FEMA 178 document classifies buildings by construction and structural system. For each class of buildings there is a discussion of the lateral load path and the typical deficiencies.

ANALYSIS

The FEMA 178 document allows design loads which are reduced from levels given in the NEHRP Recommended Provisions. For a given level of ground motion, the effect of this force reduction can be explained as follows:

- For a given exposure period, the probability of the ground motion is increased;
- For a given probability, the exposure period is reduced.

DESIGN CHECKS

The FEMA 178 document gives presumptive values for component strength and stiffness which can be used if test data are not available. The condition of the materials must be verified before these values are used. These presumptive values are based on a life-safety criteria, and assume that the building will be damaged during the design level earthquake. Examples of presumptive values in FEMA 178 are given in Appendix A.

EVALUATION STATEMENTS

FEMA 178 contains detailed evaluation procedures for the lateral load resisting systems of different structural systems. These procedures are in the form of check lists with commentaries.

FOLLOW-UP FIELD WORK
Additional field visit are usually required during both the evaluation phase and the design phase.

MATERIALS TESTING
The presumptive strength values given in FEMA 178 are lower than the values used for new construction. Higher values may be used if qualified testing is performed. For unreinforced masonry buildings, the most applicable test is the shove test for in-plane shear values. Since the presumptive values for in-plane shear strength are very low, ranging from 6 - 12.5 psi, the in-plane shear test can have a large benefit-cost ratio. Examples of values from in-plane shear tests are given in Appendix B.

FINAL EVALUATION
The end result of the evaluation process is a report on the capacity of the existing lateral load resisting system for the required seismic risk and the desired performance objective.

TYPICAL UNREINFORCED MASONRY BUILDINGS

CONSTRUCTION

Unreinforced masonry bearing wall buildings in the Central United States are typically one to five stories. The following outline summarizes the construction of these buildings:

- WALLS:
  - Single and Multiple Wythe Construction of concrete masonry clay, brick, clay tile:
    - Hollow Clay Tile
- ROOF DIAPHRAGMS:
  - Plywood or Wood Boards on Wood Framing
  - Steel Deck on Steel Joists
  - Concrete Slab With Metal Deck on Steel Joists
  - Wood Boards on Steel Joists
  - Panels (typically 2 ft. x 4 ft.) on Steel Joists
  - Poured Gypsum Over Fiberboard on Steel Joists
- FLOOR DIAPHRAGMS:
  - Straight or Diagonal Wood Boards on Wood Framing
  - Plywood on Wood Framing
  - Concrete Slab With Metal Deck on Steel Joists
- FOUNDATIONS:
  - Typically Narrow Strip Footings Under Walls

TYPICAL DEFICIENCIES

The typical deficiencies found in unreinforced masonry buildings in the Central United States can be classified as follows:

- Lack of continuity in the lateral load resisting system;
- Inadequate component strength;
- Inadequate component stiffness;
- Inadequate connections between components.

IN-PLANE SHEAR TESTS

The in-plane shear tests were performed in order to provide a quantitative determination of the strength of the existing masonry walls, and to establish confidence levels for the analysis of these existing buildings, as well as for the design of the seismic retrofits.

The maximum shear values measured were approximately twice the average value, and the minimum were approximately 40 percent of the average. The range of the values underscore the variability of masonry construction, yet the minimum values still provide for a lower bound with the factor safety of four, which justifies the use of a higher allowable stress. This higher stress is no higher than that used for new masonry construction.

CASE HISTORIES

Southeast Missouri Telecommunications

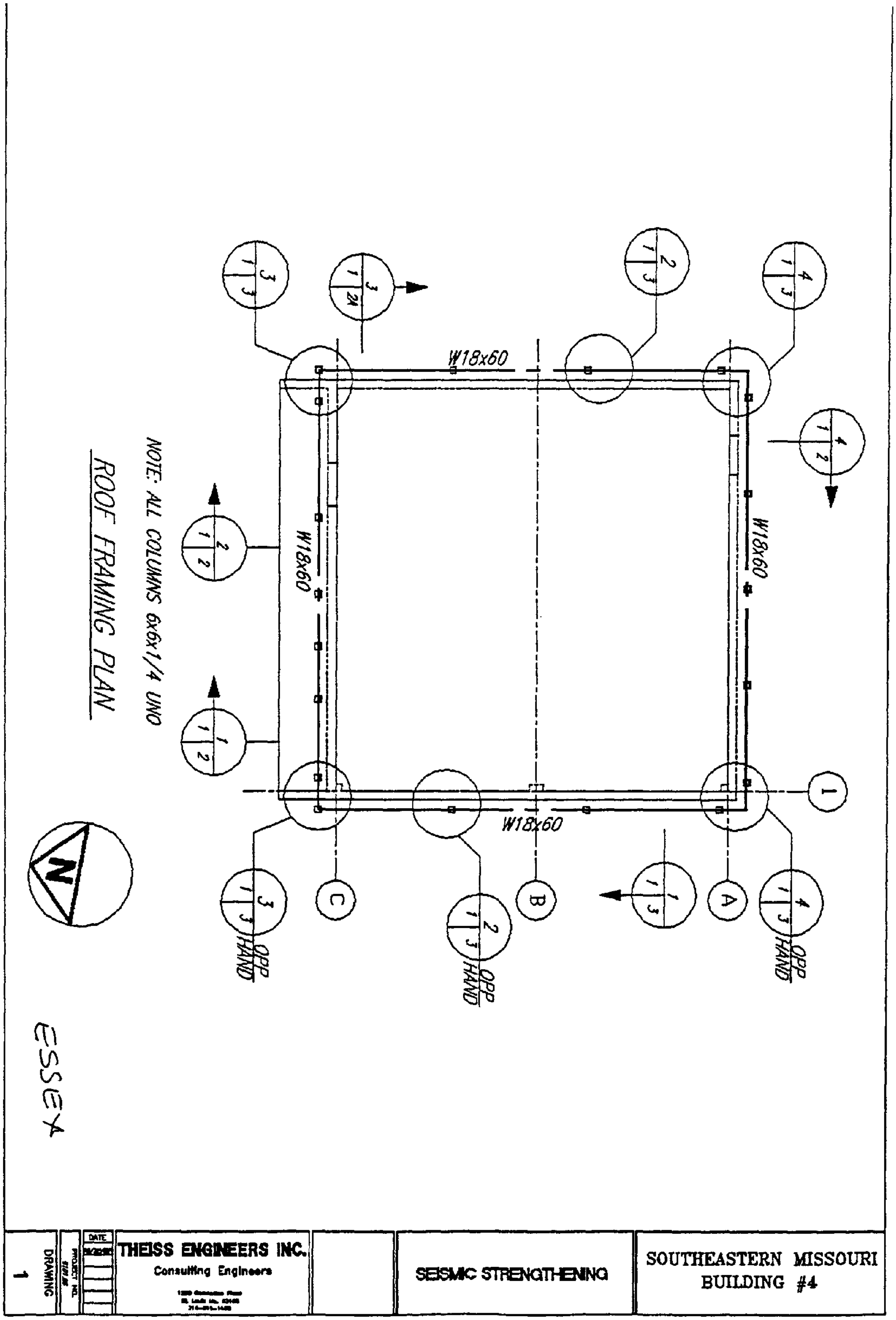
- 24 One-Story URM Buildings
- Aa = Av = 0.30
- Performance Objective:
  - Continued Operation
- Special Considerations:
  - No Work Inside Building
  - No Work on Roof

Southeast Missouri Telecommunications
Typical Deficiencies

- Diaphragms Inadequate
  - Panel Roof Systems
  - Wood Boards on Steel Joists
- Diaphragm-Wall Connections Inadequate
- Walls Inadequate Out-Of-Plane
- Short Walls Inadequate In-Plane

Southeast Missouri Telecommunications
Retrofit Strategies

- All Work Done Outside Building
- Horizontal Perimeter Truss or Beam Around Roof
- Braced Frames at Inadequate Walls
- New Foundation Elements


ROOF FRAMING PLAN
NOTE: ALL COLUMNS 6x6x1/4 UNO
W18x60
W18x60
W18x60
W18x60
OPP HAND
OPP HAND
OPP HAND
A
B
C
1
N
ESSEX
THEISS ENGINEERS INC.
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DATE
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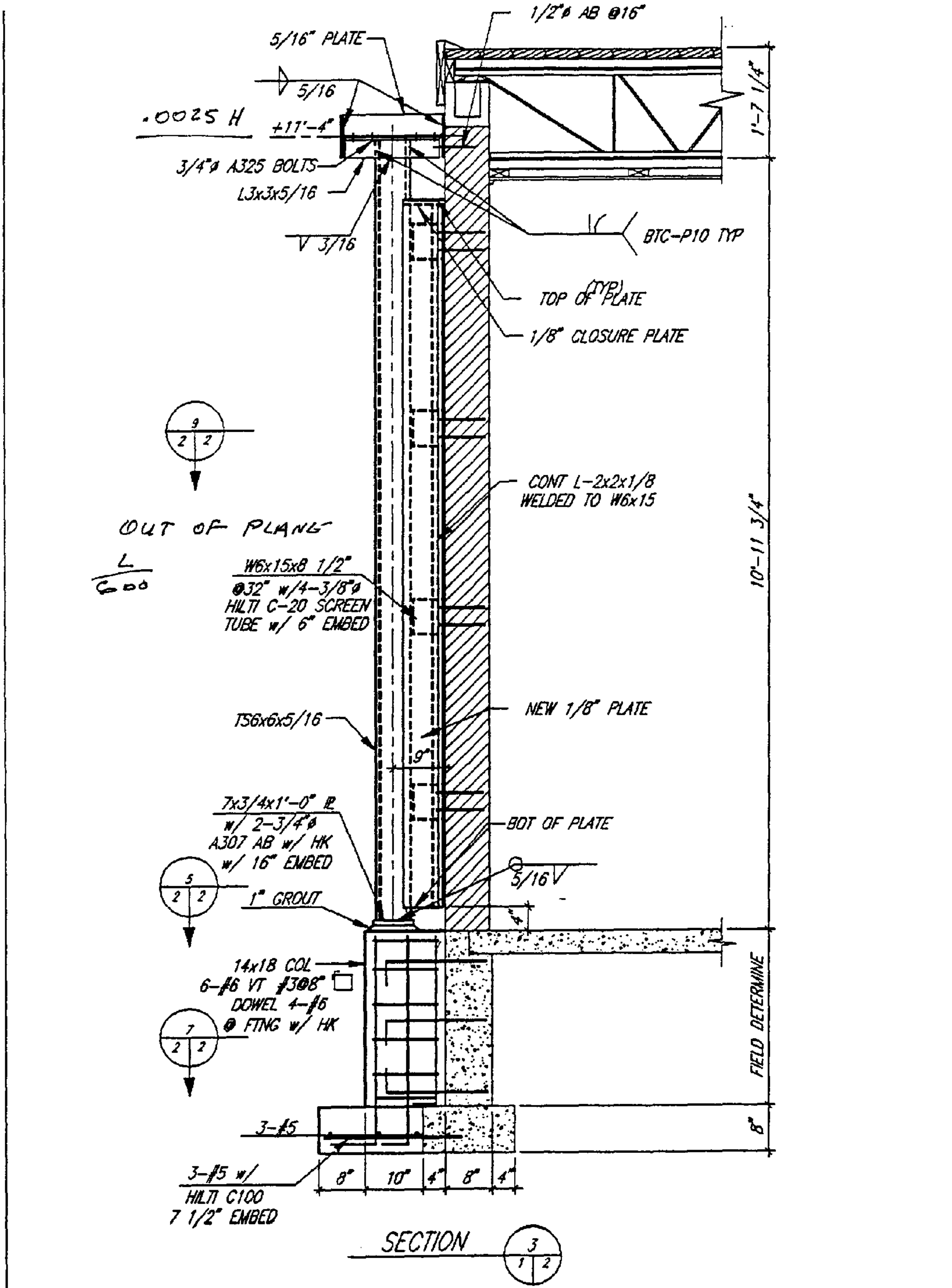
1
SEISMIC STRENGTHENING
SOUTHEASTERN MISSOURI
BUILDING #4



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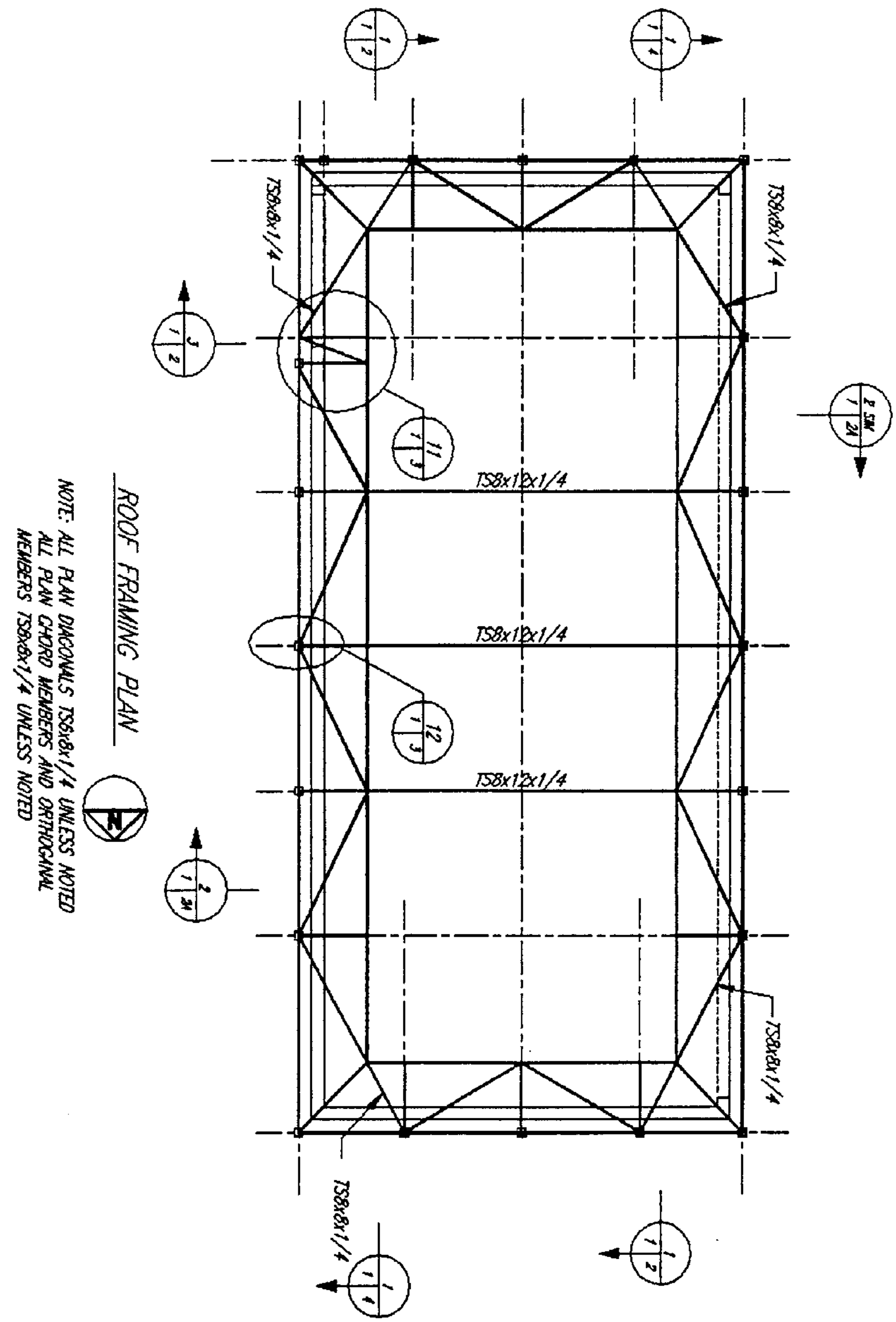
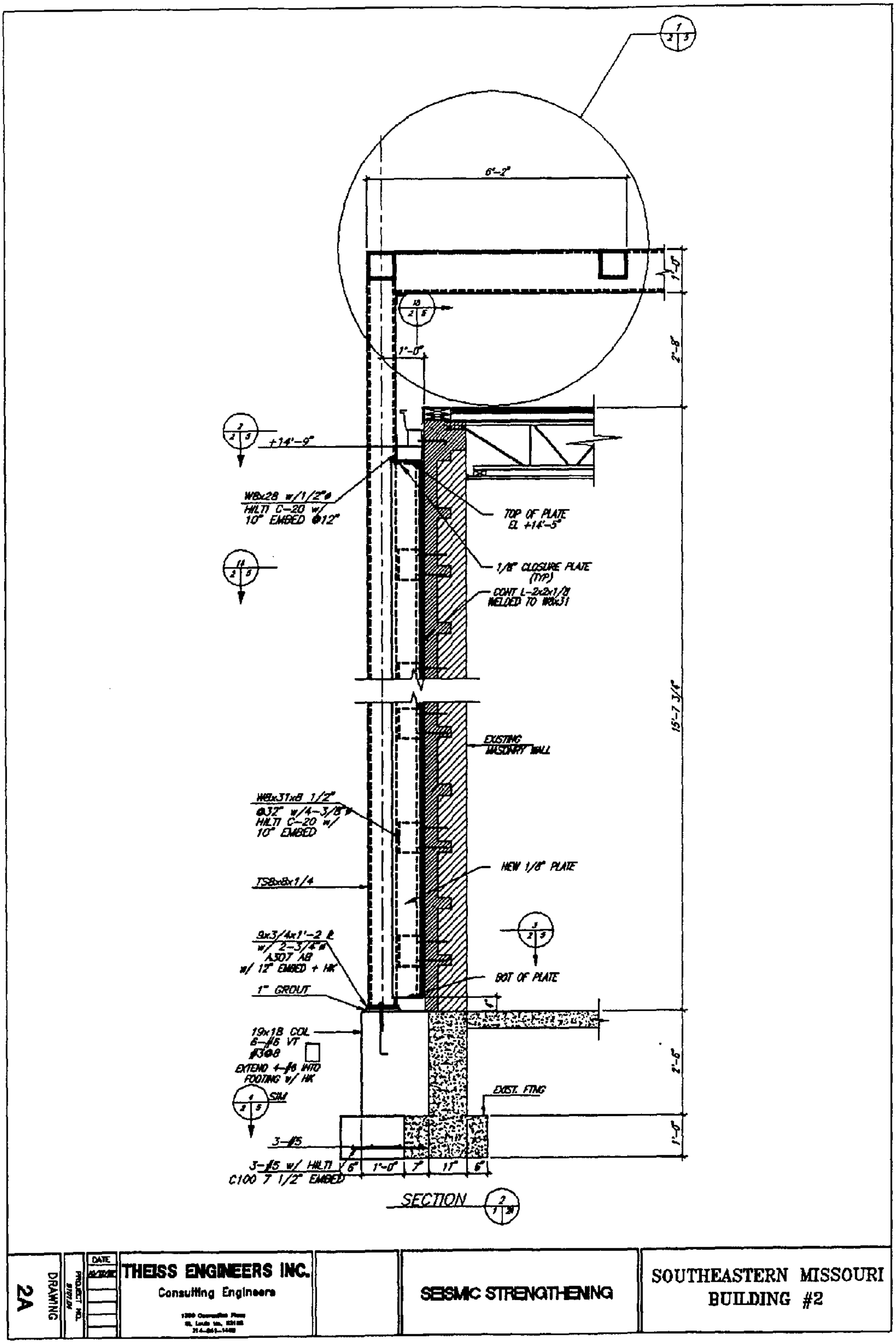
2						



NEW MAP

DRAWING 1	PROJECT NO.	DATE	THEISS ENGINEERS INC. Consulting Engineers		SEISMIC STRENGTHENING	SOUTHEASTERN MISSOURI BUILDING #2


6'-2"
1'-0"
2'-8"
15'-7 3/4"
2'-6"
+14'-9"
W8x28 w/1/2"ø HILTI C-20 w/ 10" EMBED @12"
TOP OF PLATE EL +14'-5"
1/8" CLOSURE PLATE (TYP)
CONT L-2x2x1/8 WELDED TO W8x31
EXISTING MASONRY WALL
W8x31x8 1/2" @32" w/4-3/8"ø HILTI C-20 w/ 10" EMBED
NEW 1/8" PLATE
TS8x8x1/4
9x3/4x1'-2 ℞ w/ 2-3/4"ø A307 AB w/ 12" EMBED + HK
BOT OF PLATE
1" GROUT
19x18 COL 6-#6 VT #3@8
EXTEND 4-#6 INTO FOOTING w/ HK
SIM
EXIST. FTNG
3-#5
3-#5 w/ HILTI C100 7 1/2" EMBED
6" 1'-0" 7" 11" 6"
SECTION
DRAWING 2A
PROJECT NO.
DATE
THEISS ENGINEERS INC.
Consulting Engineers
SEISMIC STRENGTHENING
SOUTHEASTERN MISSOURI BUILDING #2

CASE HISTORIES

South Indiana Telecommunications

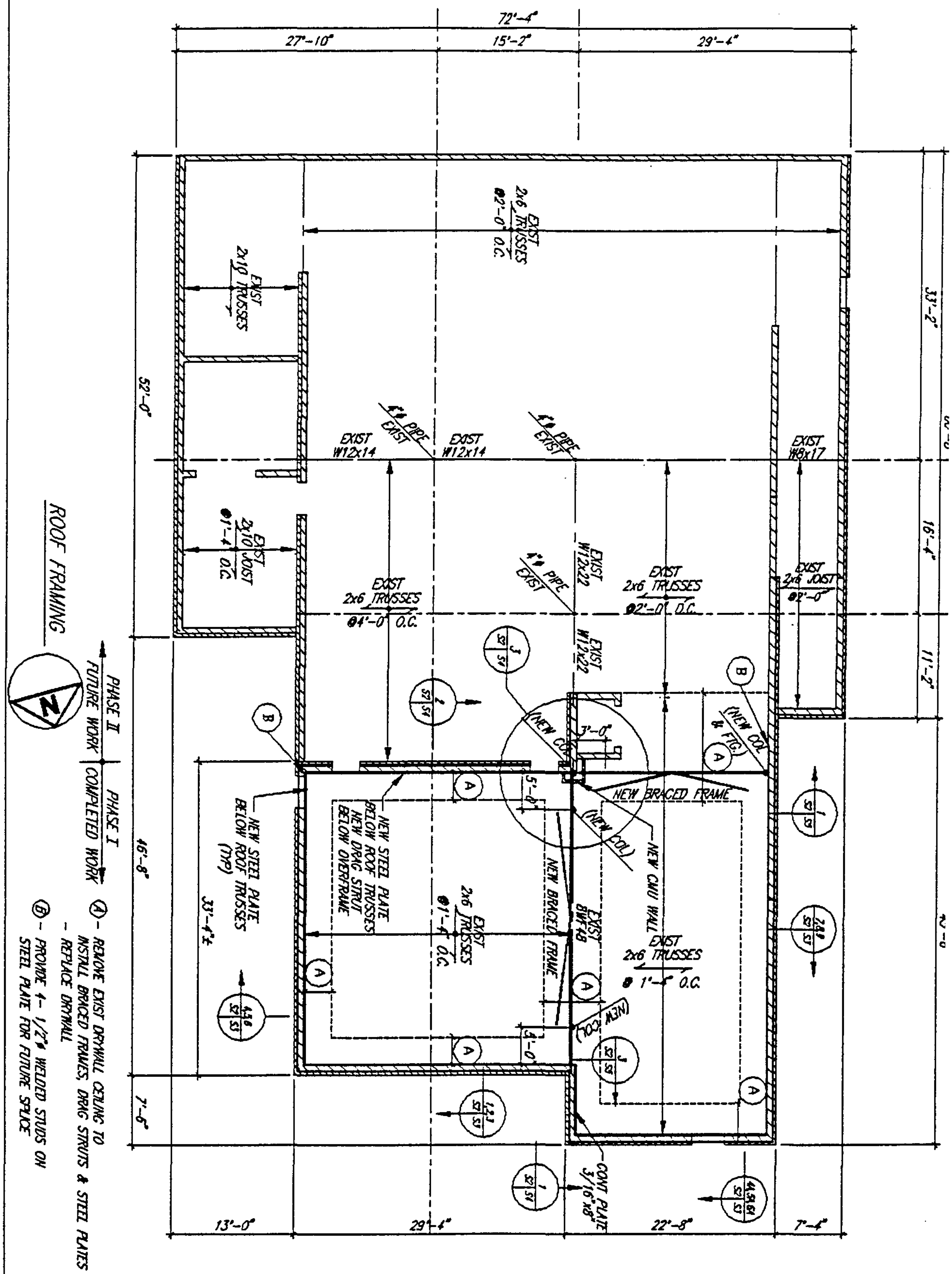
- 9 One-Story URM Buildings
- Aa = Av = 0.20
- Performance Objective
  - Continued Operation
- Special Considerations
  - Dust and Vibration Control
  - Work Inside Building Allowed
  - Roofing Replaced

South Indiana Telecommunications
Typical Deficiencies

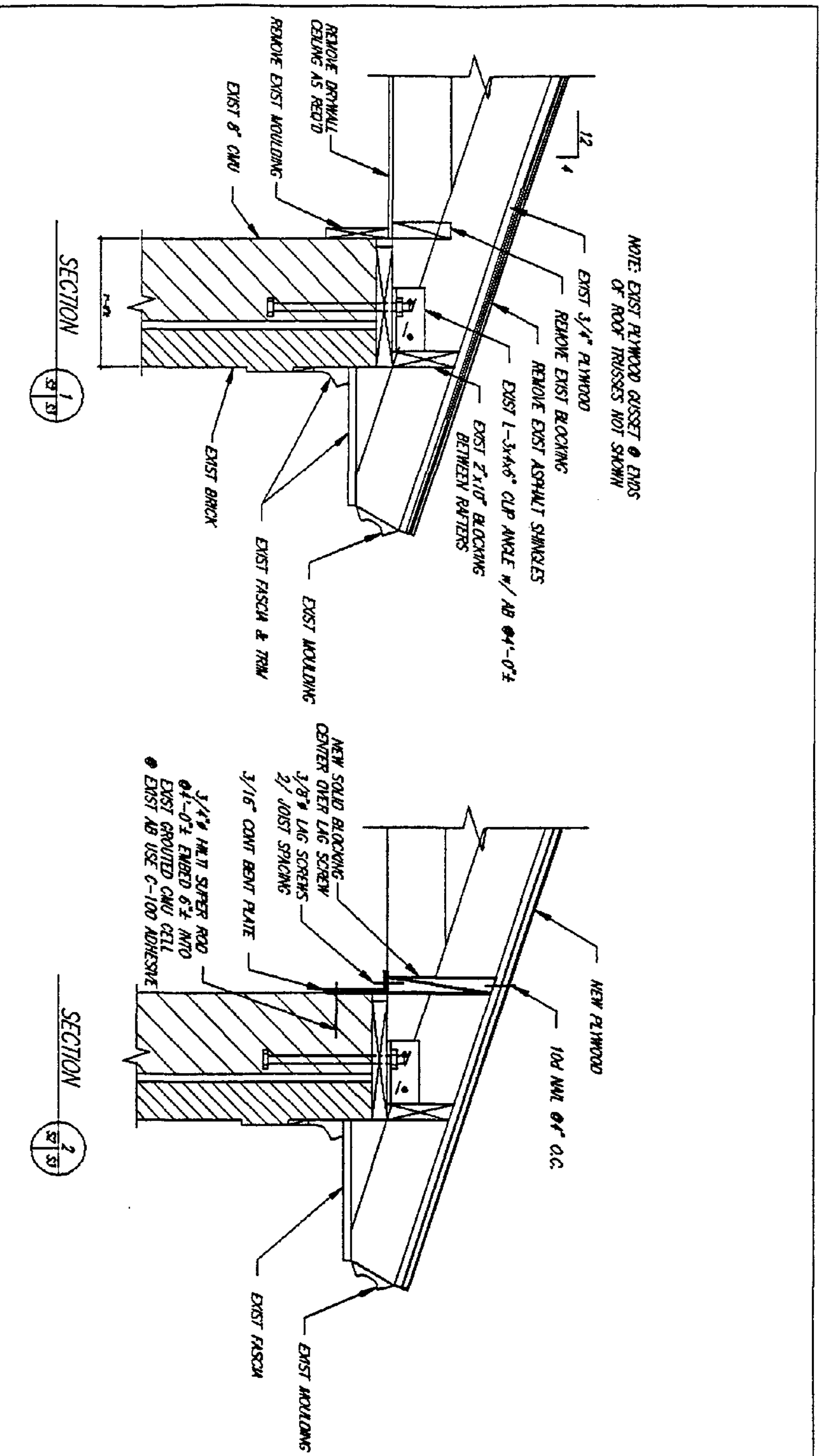
- Diaphragms Inadequate
  - Panel Roof Systems
  - Straight Sheathing Boards
- Walls Removed During Expansion
- Diaphragm-Wall Connections Inadequate
- Short Walls Inadequate for In-Plane Shear

South Indiana Telecommunications
Retrofit Strategies

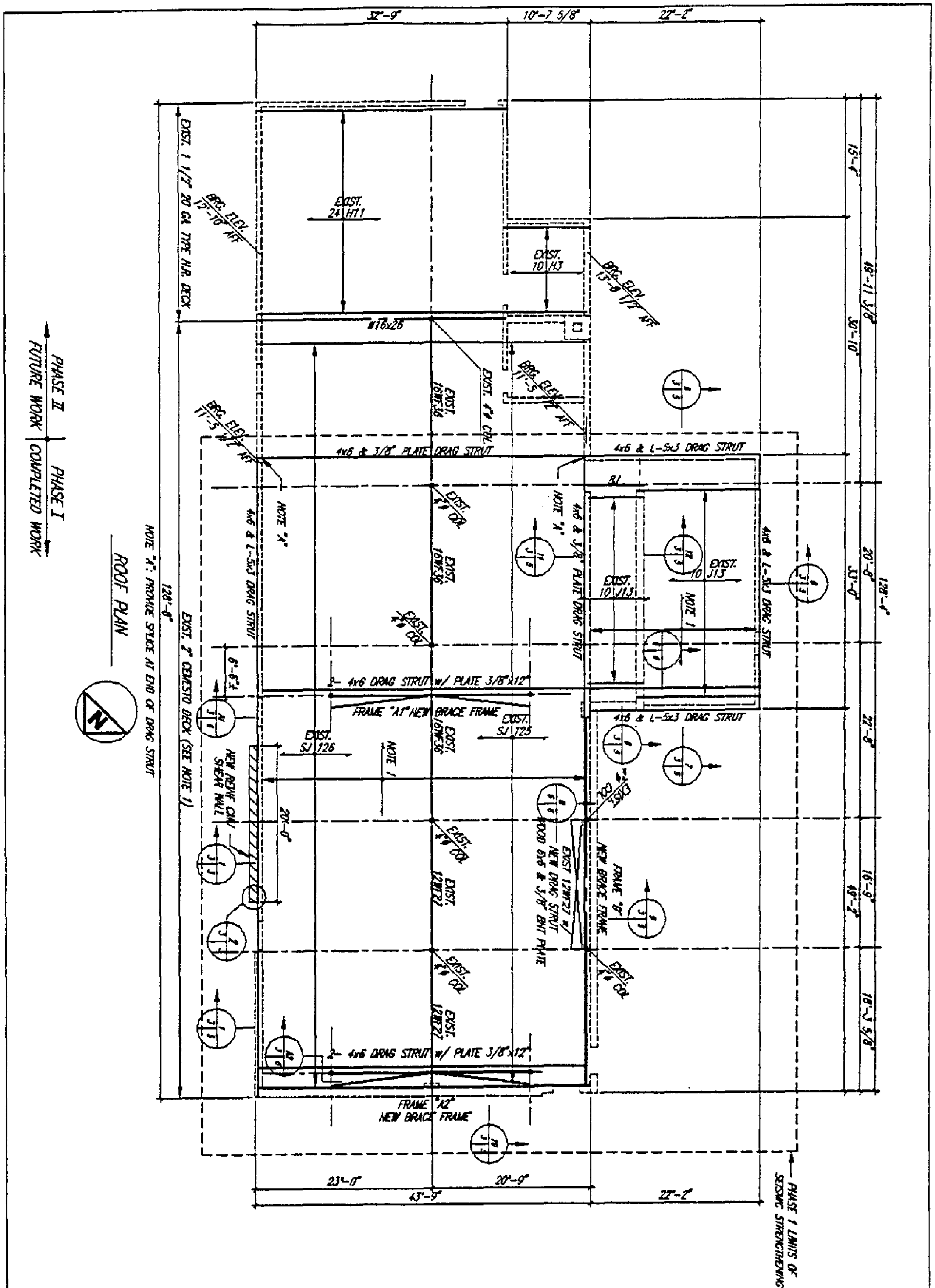
- Work Inside Building and on Roof Allowed
- New Plywood Roof Diaphragms
- New Diaphragm-Wall Connections
- Steel Braced Frames Inside
- Reinforced Masonry Shear Walls Outside
- Use Existing URM Shear Walls if Possible



DRAWING 1	DATE	THEISS ENGINEERS INC. Consulting Engineers	SEISMIC STRENGTHENING	SOUTHWESTERN INDIANA BUILDING #1



DRAWING S3	PROJECT NO.	DATE	THEISS ENGINEERS INC. Consulting Engineers		SEISMIC STRENGTHENING	SOUTHWESTERN INDIANA BUILDING #1



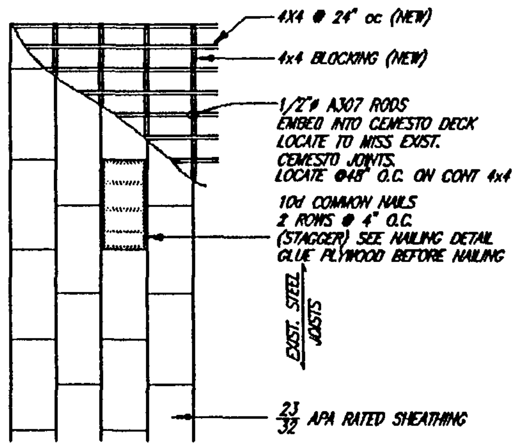
THEISS ENGINEERS INC.
Consulting Engineers

DATE 4/9/93

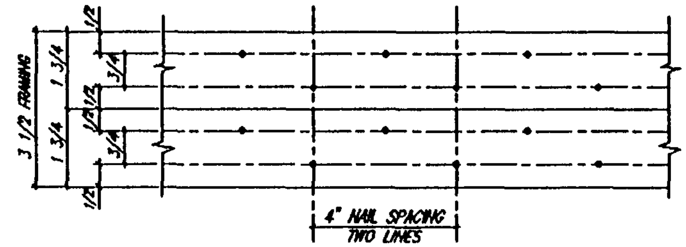
DRAWING S3

SEISMIC STRENGTHENING
SOUTHWESTERN INDIANA
BUILDING #2

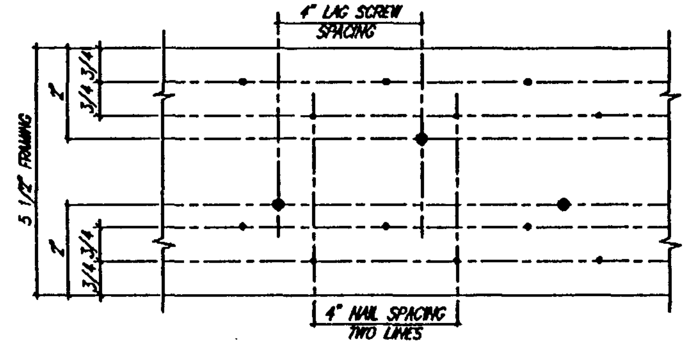
NOTES:
1. INSTALL NEW TREATED 4x4 SLEEPERS @ 24" oc DIRECTLY OVER EXIST. 2" CEMESTO DECK.
2. SEE DETAIL THIS SHEET FOR CONNECTION BETWEEN 4x4 CEMESTO DECK.
3. SEE SPECIFICATIONS FOR DRILLING PROCEDURE.
4. INSTALL NEW APA RATED SHEATHING OVER 4x4 SLEEPERS. BLOCK ALL PANEL EDGES WITH 4x4.
5. SEE DETAILS THIS SHEET FOR PLYWOOD INSTALLATION AND NAILING.



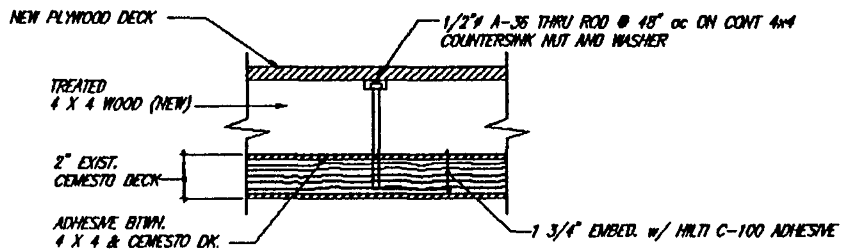
PLYWOOD INSTALLATION DIAGRAM



3 1/2" FRAMING – NAILING DETAIL
NO SCALE



5 1/2" FRAMING – NAILING DETAIL
NO SCALE



CONNECTION DTL. OF WOOD SLEEPER
TO EXISTING CEMESTO DECK
NO SCALE

SEISMIC STRENGTHENING
SOUTHWESTERN INDIANA
BUILDING #2

THEISS ENGINEERS INC.
Consulting Engineers

DATE
PROJECT NO.
DRAWING
S4

CASE HISTORIES

Southeast Missouri Utility

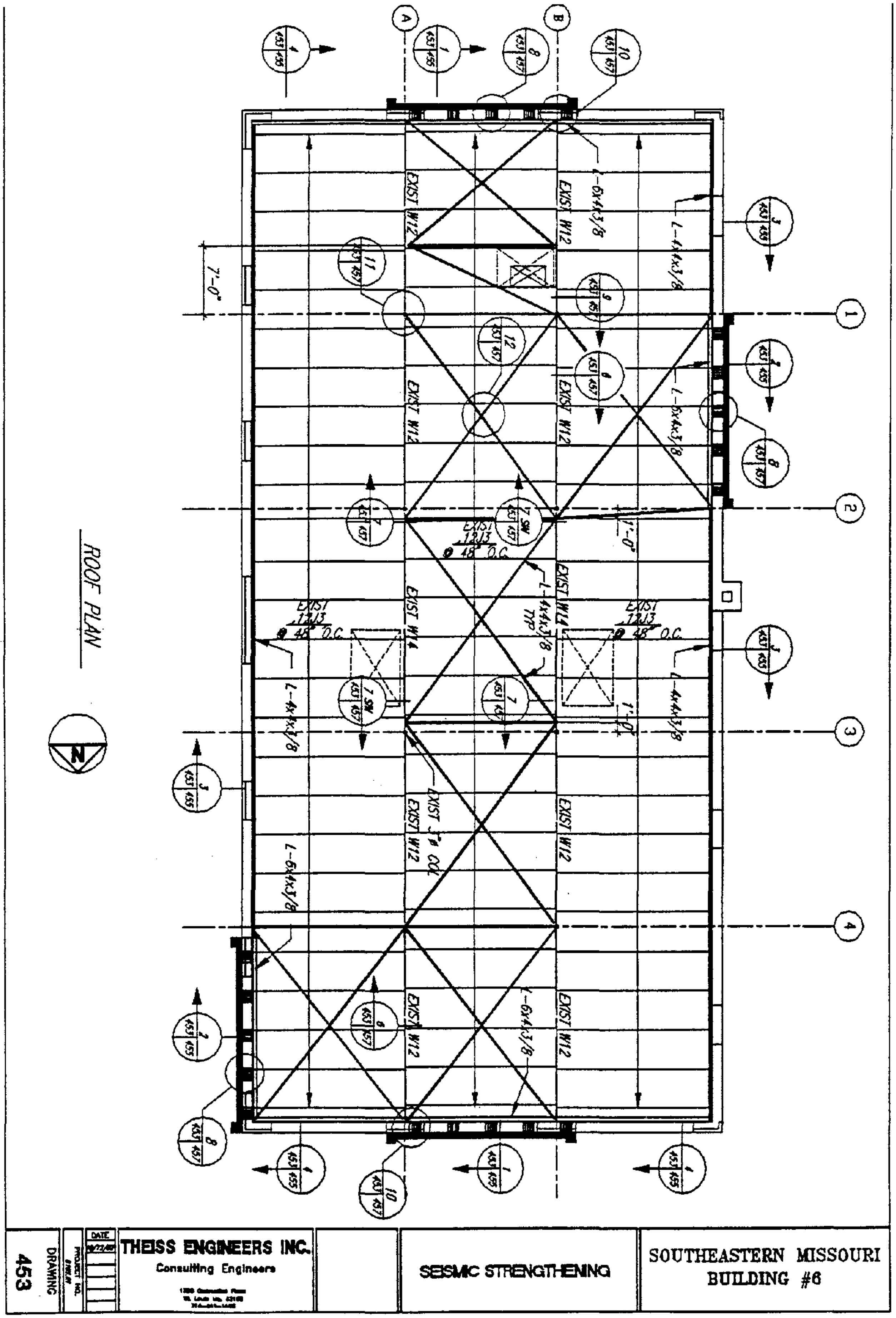
- 1 Two-Story Building
- Aa = Av = 0.30
- Performance Objective:
  Life Safety
- Special Conditions:
  Dust and Vibration Control
  New Shear Walls Outside Building

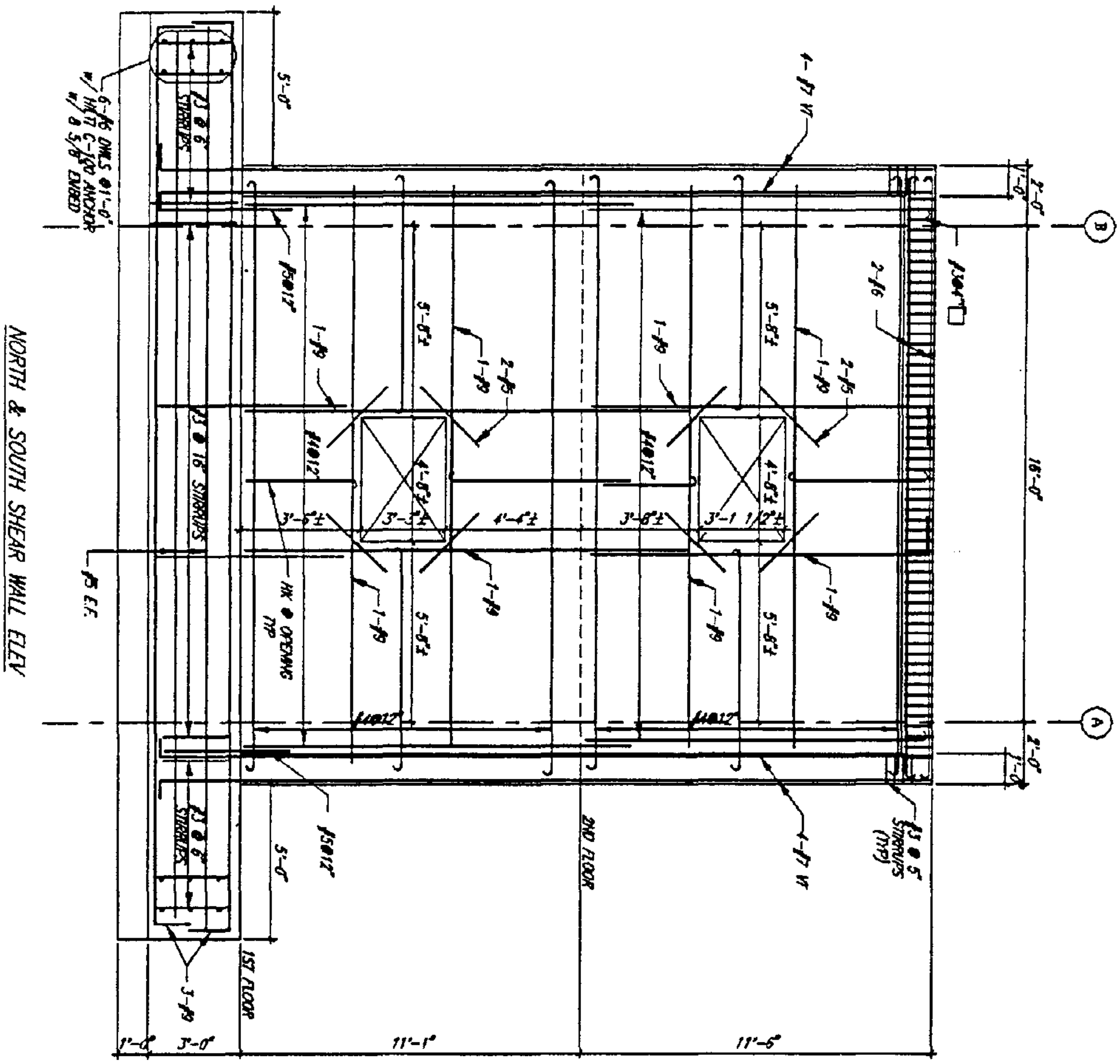
Southeast Missouri Utility
Typical Deficiencies

- Perforated URM Shear Walls Inadequate
- Panel Roof Diaphragm Inadequate
- Diaphragm-Wall Connections Inadequate

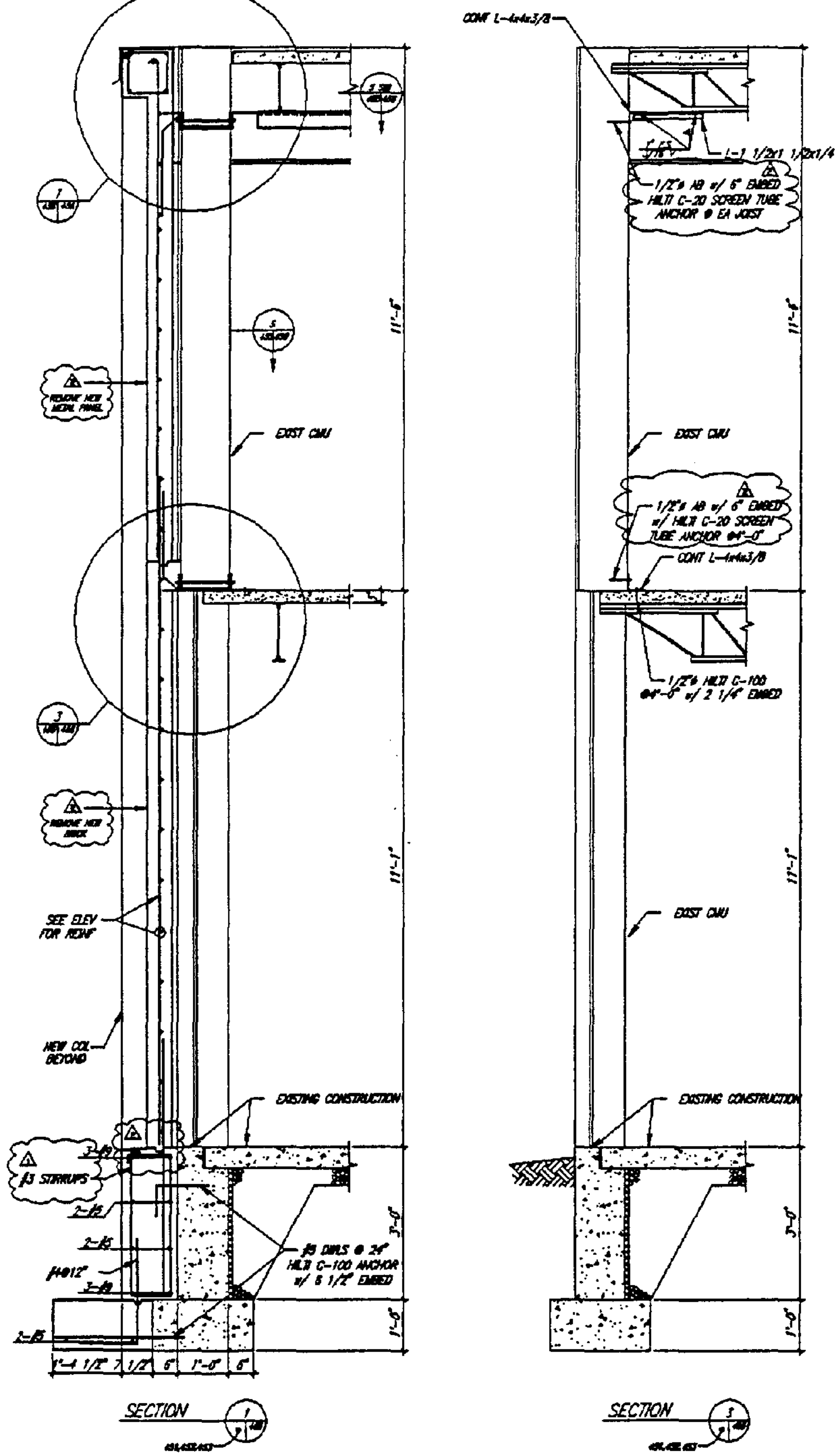
Southeast Missouri Utility
Retrofit Strategies

- New Reinforced Concrete Shear Walls Outside Building
- New Horizontal Steel Bracing Below Roof
- New Chord Around Floor Diaphragm


ROOF PLAN
EXIST W12
EXIST W14
EXIST 12J3 @ 48" O.C.
L-4x4x3/8
L-4x4x3/8 TYP
L-6x4x3/8
EXIST 3" ø COL
7'-0"
1'-0"
THEISS ENGINEERS INC.
Consulting Engineers
SEISMIC STRENGTHENING
SOUTHEASTERN MISSOURI
BUILDING #6
DRAWING
453



DRAWING 454	PROJECT NO.	DATE	THEISS ENGINEERS INC. Consulting Engineers		SEISMIC STRENGTHENING	SOUTHEASTERN MISSOURI BUILDING #6



DRAWING 455	PROJECT NO.	DATE	THEISS ENGINEERS INC. Consulting Engineers		SEISMIC STRENGTHENING	SOUTHEASTERN MISSOURI BUILDING #6

CASE HISTORIES

Southeast Missouri School

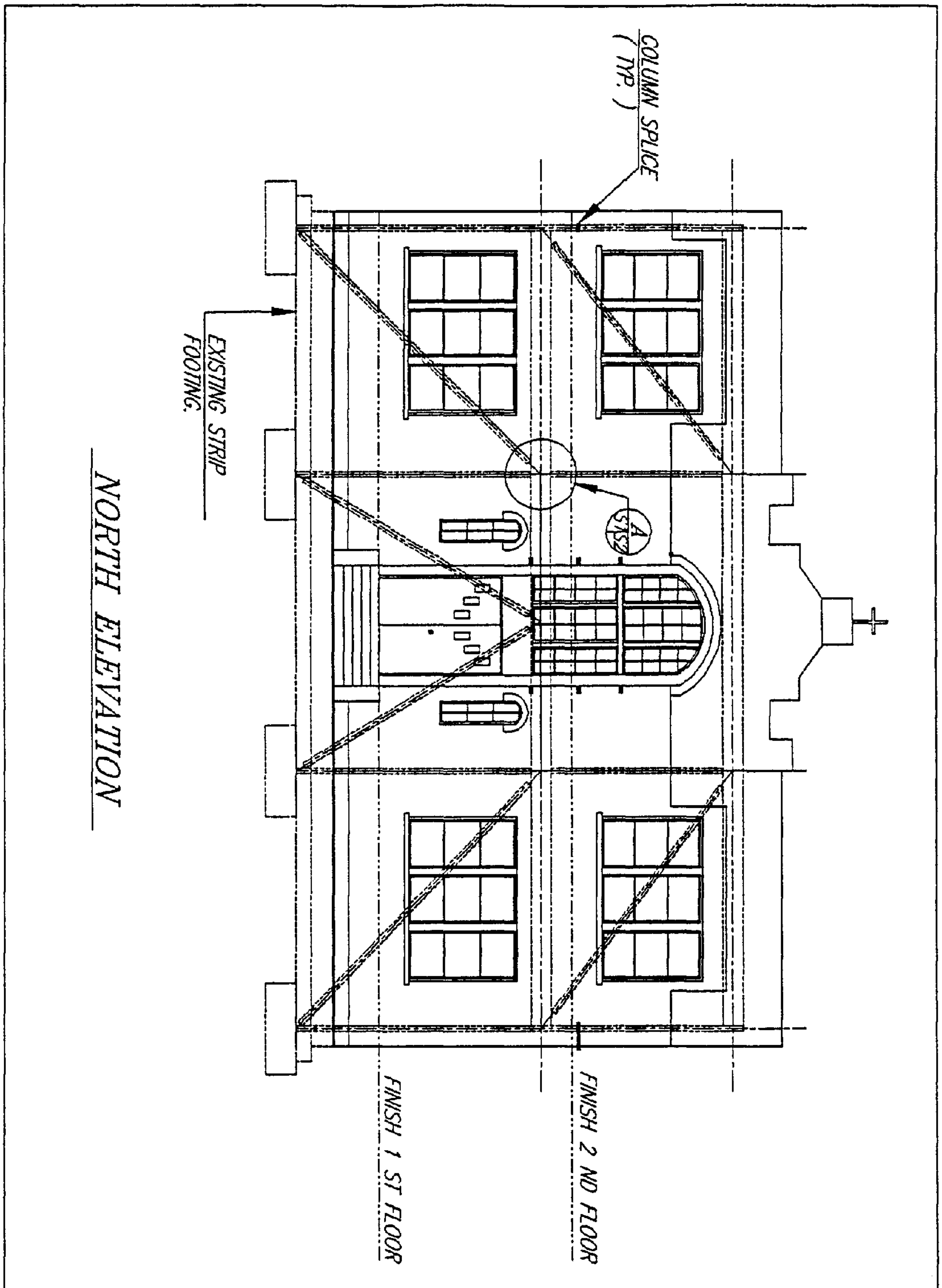
- Two-Story Building
- Aa = Av = 0.30
- Performance Objective:
  Life Safety
- Special Conditions:
  Retrofit Materials Donated
  New Bracing to Remain Exposed

Southeast Missouri School
Typical Deficiencies

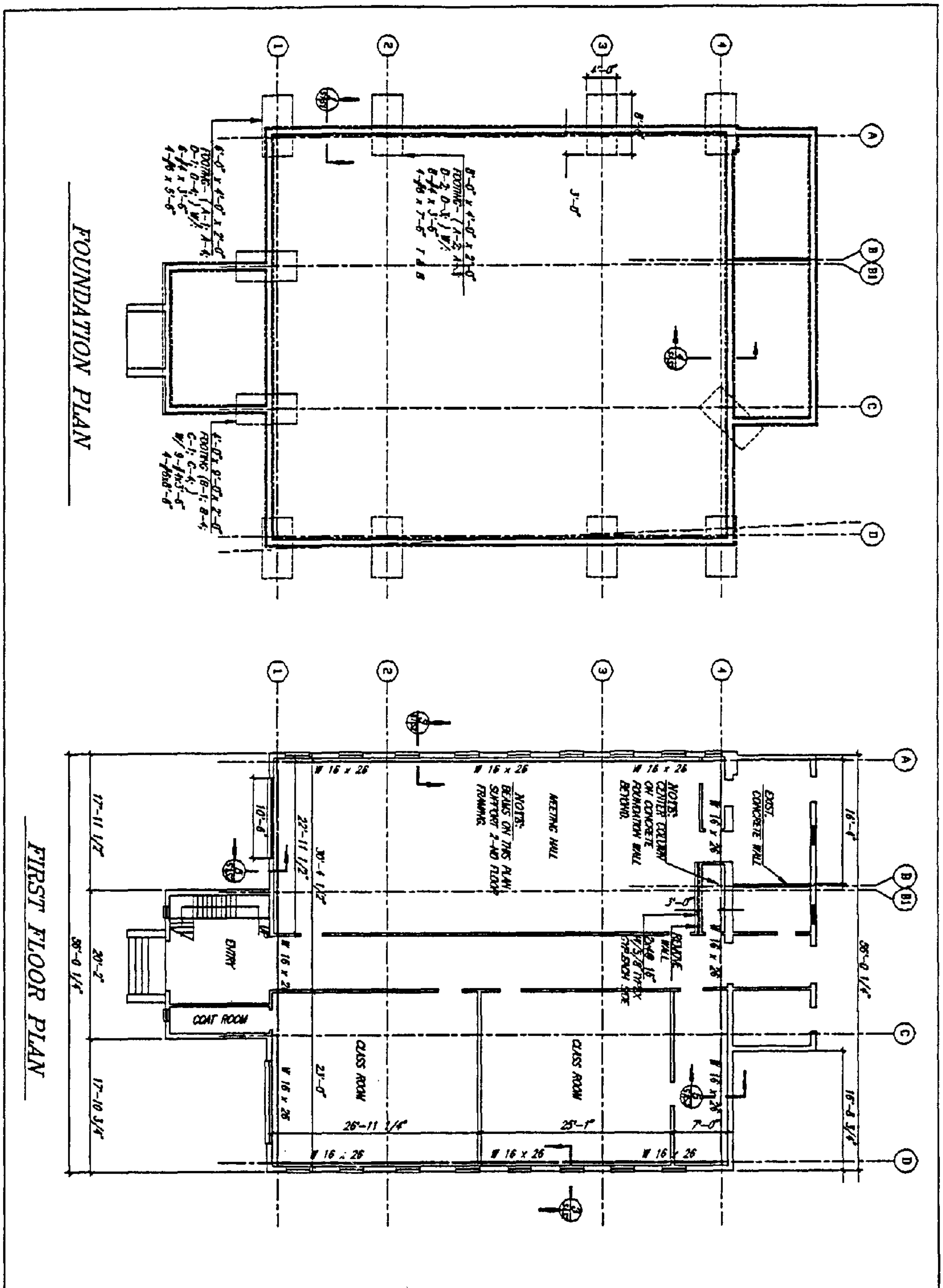
- Perforated URM Shear Walls Inadequate
- Diaphragm-Wall Connections Inadequate

Southeast Missouri School
Retrofit Strategies

- New Braced Frames Inside Building
- Additional Wall-Diaphragm Connections



DRAWING S2	THEISS ENGINEERS INC. Consulting Engineers	SEISMIC STRENGTHENING SOUTHEASTERN MISSOURI BUILDING #5



S1
DRAWING

PROJECT NO. 9067

DATE

THEISS ENGINEERS INC.
Consulting Engineers

SEISMIC STRENGTHENING
SOUTHEASTERN MISSOURI
BUILDING #5

CASE HISTORIES

St. Louis Missouri Office Building

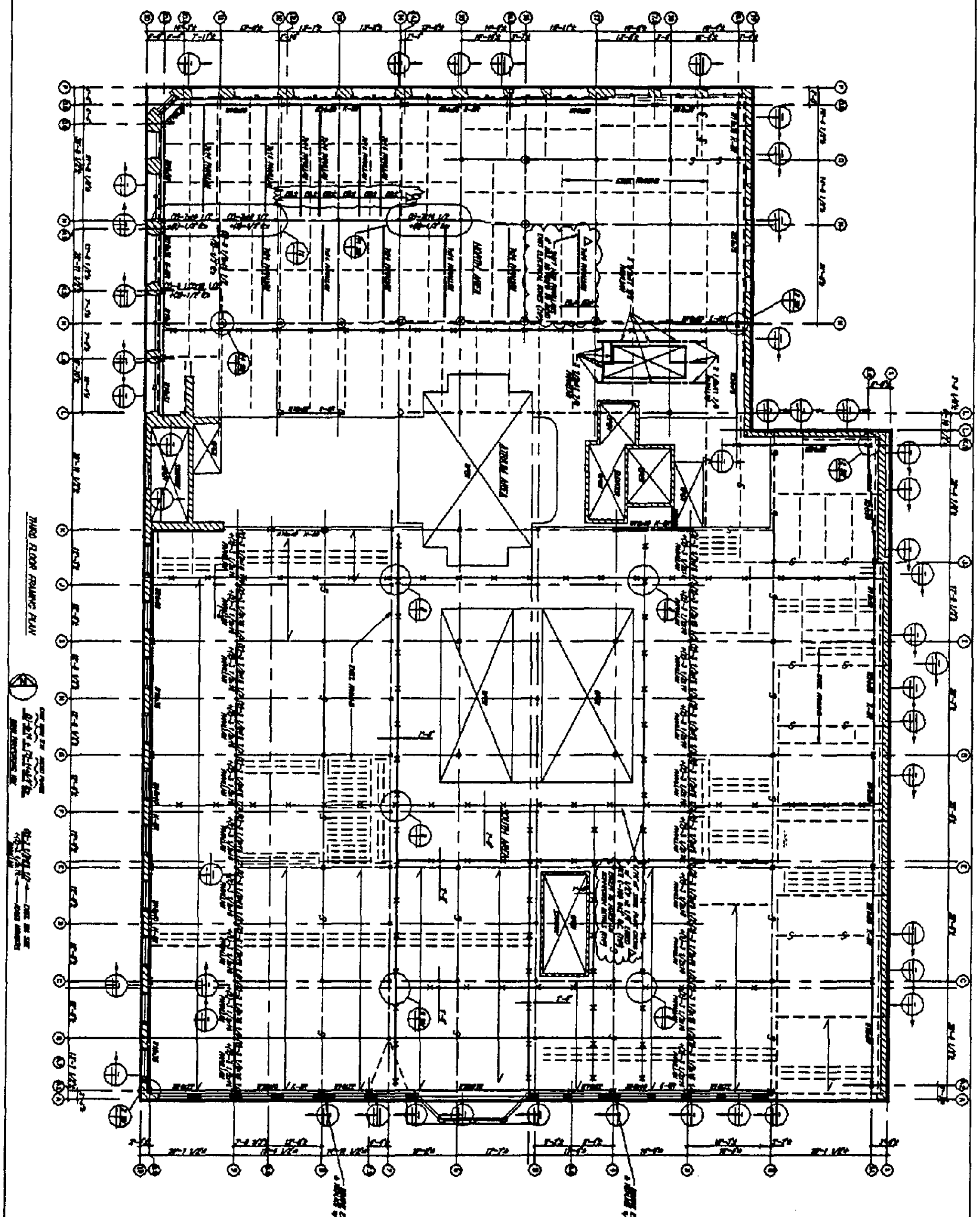
- 1 Six-Story Office Building
- Aa = Av = 0.13
- Performance Objective:
  - Life Safety
- Special Considerations:
  - Adjacent Subway Station
  - Adjacent Lower Building

St. Louis Missouri Office Building
Typical Deficiencies

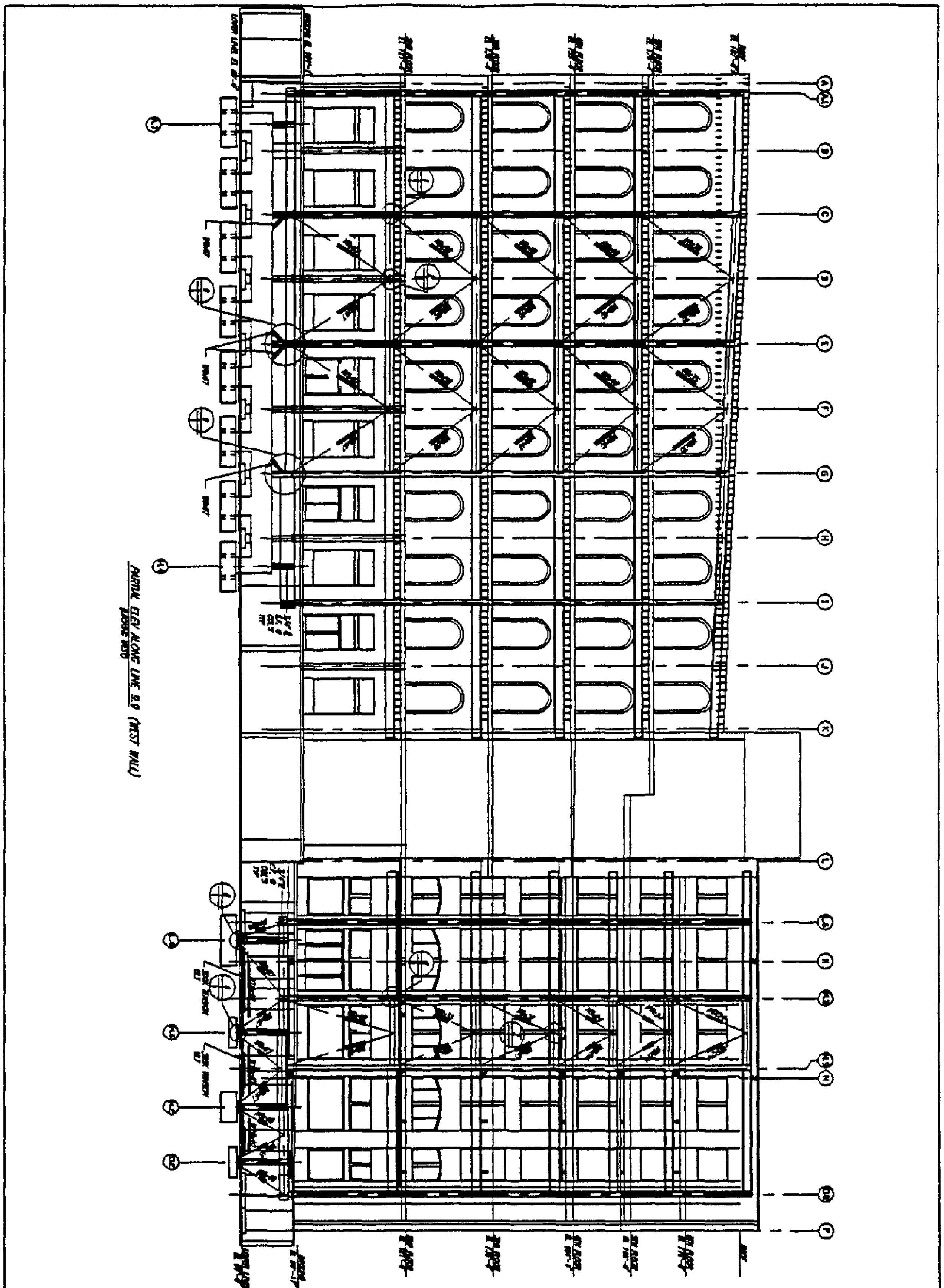
- Perforated URM Shear Walls Inadequate
- Diaphragms With Large Openings
- Dead Load Added During Renovation
- Unsymmetrical Arrangement of Existing URM Walls

St. Louis Missouri Office Building
Retrofit Strategies

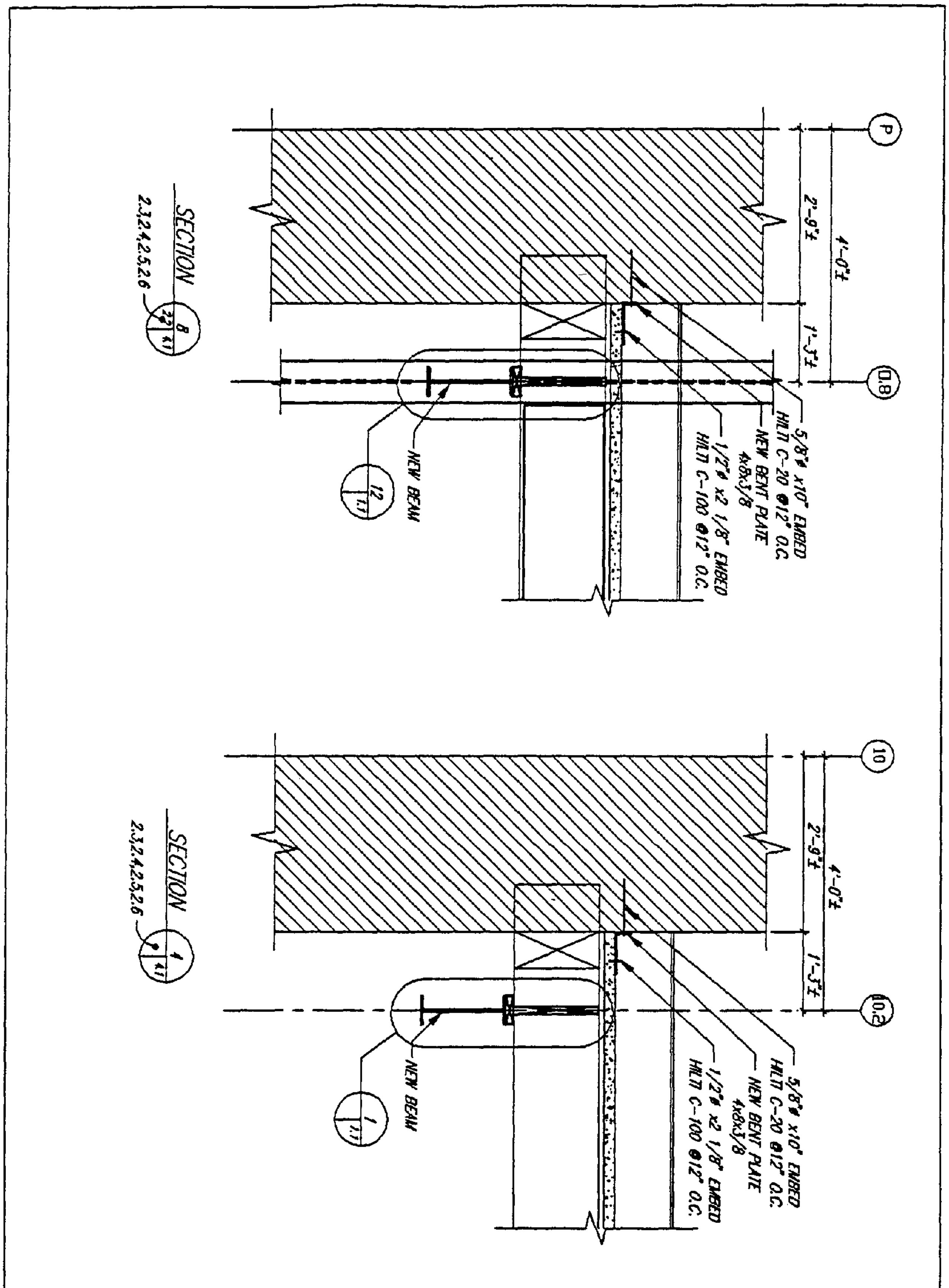
- Steel Braced Frames Inside Building
- Reinforce Edge of Diaphragm Openings
- Non-Structural Retrofit
  - Parapets
  - Skylights
  - Mechanical/Electrical
  - Raised Access Floor



DRAWING S2.3	PROJECT NO.	DATE	**THEISS ENGINEERS INC.** Consulting Engineers	**SEISMIC STRENGTHENING**	ST.LOUIS MISSOURI BUILDING #1



DRAWING S5.1	THEISS ENGINEERS INC. Consulting Engineers	SEISMIC STRENGTHENING	ST.LOUIS MISSOURI BUILDING #1



DRAWING S4.1	THEISS ENGINEERS INC. Consulting Engineers	SEISMIC STRENGTHENING	ST.LOUIS MISSOURI BUILDING #1

APPENDIX A
PRESUMPTIVE VALUES FOR EXISTING CONSTRUCTION
Reference: FEMA 178

Presumptive Values for Diaphragms
Strength Values for Seismic Shear

Existing Construction	Shear (plf)
Roofs with Straight Sheathing Roofing Directly on Sheathing	300
Roofs with Diagonal Sheathing Roofing Directly on Sheathing	750
Floors with Straight Tongue-and Groove Sheathing	300
Floors with Diagonal Sheathing and Finish Wood Flooring	1800
Metal Deck with Minimal Welding	1800

Presumptive Values for In-Plane Walls
Strength Values for Shear
Reference FEMA 178

Construction	Allowable Shear Stress
Solid Brick Masonry	10 psi
Hollow Unit Masonry	6 psi
Grouted Concrete Masonry	12.5 psi

APPENDIX B
RESULTS OF IN-PLANE SHEAR TEST

Results From 15 Buildings

	CMU	Brick	Composite
Number	55	26	5
Average	411	370	413
High	605	724	605
Low	138	167	164

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REHABILITATION OF URM BUILDINGS IN ITALY

C. Gavarini*

ABSTRACT

The problem concerning the rehabilitation of Unreinforced Masonry Buildings in Italy is presented in its general dimension, a problem which deals with the definition of seismic hazard, the vision of seismic risk for the whole population of buildings present on the territory, the related concept of seismic vulnerability, the basic problem of how to reduce the risk on the whole population and on the single buildings, the particular questions which arise for historical and monumental buildings, and for strategic buildings, and finally the problems regarding applications: technical codes, specific laws and spreading of knowledge.

INTRODUCTION

The seismic protection of masonry buildings is considered in Italy as a serious problem with many aspects to study and numerous fields of applications. We may mention at least the following sectors:
- the definition of the seismic hazard in every part of the country;
- the assessment of the seismic risk concerning the huge population of existing masonry buildings present on the territory;
- related with the previous one, the problem of assessing the (seismic) vulnerability of the buildings;
- the problem of how to reduce the seismic risk;
- related with the previous one, the problem of how to reinforce the single buildings, i.e. to reduce the vulnerability by means of appropriated techniques;
- the problems related to the systematic application of the previous techniques, in terms of general policy, technical Codes, diffusion of the knowledge, and so on;
- the particular problems related to the special parts of the population of buildings which present the specific aspect to be historical buildings and/or historical areas;
- idem for the even more particular problems related to single monumental buildings;
- idem for the existing masonry buildings which present, possibly together with the previous historical/monumental/artistical character, the property of being "strategic" from a functional point of view.

All the problems mentioned above have been and are presently treated by the Italian National Group for the Earthquake Loss Reduction. The experimental results which are being presented here in Pavia are an example of such an activity. Now I will give some information on the other sectors.

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Seismic Hazard

Officially and practically, Seismic Hazard is presently defined in Italy by the Seismic Zonation, i.e. a map of the Country where every city or village, with all its territory, is assigned to a seismic zone, within 4 possible categories: I, II, III and no seismicity. A Design horizontal acceleration is assigned to every category: it is a conventional value which corresponds to an elastic design, and the four values are 0.10 g, 0.07 g, 0.04 g, zero. To give an idea on an international basis, we may say that such values correspond roughly to the PGA values indicated within the EC8 activity, i.e. 0.35 g, 0.25 g, 0.15 g.

Such a zonation is the result of a very long process, and it presents defects and incoherences. Now, the results obtained on this subject by our National Group, after 14 years of activity, should allow a radical revision, based on a series of maps which give the expected ground maximum seismic acceleration for given return periods. In particular one of the maps gives the PGA/475 years, as suggested by the EC8. Within the National Group, the discussion is now active on the meaning of such an indication from EC8; an indication which appears to concern essentially new ordinary constructions, while the problems related with existing buildings, historical areas and monumental buildings, and with strategic buildings, plants and lifelines may suggest to consider more maps.

Seismic Risk

As it is well known, Seismic Risk depends mainly on three factors: hazard, vulnerability, exposure. Models may be proposed for the definition of each of these factors and theories for the definition of the Risk may be subsequently obtained. Such a task has been performed by our Group: there is a model for ordinary masonry or reinforced concrete buildings proposed by V. Petrini, and this model has been very recently applied in particular to the whole population of public buildings of Emilia Romagna (2676 buildings) with many results, the first being a list giving the priority obtained for intervention, based on the expected damage (01, 02).

Seismic Vulnerability

As mentioned in the previous chapter, seismic risk is based, besides hazard, on vulnerability and on exposure. The question of how to obtain such information on a population of buildings has been considered within the National Group since 1984, starting with masonry ordinary buildings (03) and reinforced concrete buildings (04) and dealing then with historical buildings (05), monumental buildings (06, 07), industrial buildings (Alessi and others: 01).

Vulnerability may be defined as the "weakness", versus assigned external actions, of structures or systems. In the field we are concerned with, the actions are of mechanical nature, in particular they are seismic actions.

The evaluation of vulnerability may be undertaken at different levels: we can consider as their upper limit the full structural analysis of single constructions, as typically defined by the Codes: but this kind of approach is not regarded vulnerability assessment. Another limit may be fond in a very simple and quick approach, able to give an evaluation in a few minutes, typically on the basis of an expert judgment.

A vulnerability survey is based on the collection of information on a number of parameters, which are considered important. This collection may be performed filling one or more forms or adopting an expert system implemented on a portable PC. The collected information may be summarized in a vulnerability index to be utilized in a model of risk or it may be considered as a global input for a number of analyses: assessment of priorities, hypotheses of retrofitting, costs of retrofitting, costs-benefits analyses, optimal strategies for intervention on large scale, ... Such analyses may be performed defining suitable models; typically they deal with uncertainties and often they include the evaluation of data of very different nature: numbers in the common engineering sense but also fuzzy sets, synthetic judgments, cultural values, drawings, photos, texts, historical data, social implications, ... As far as numbers only are considered, possibly affected by uncertainty defined on a probabilistic basis, the models may be numerical; but when other kind of information must be introduced, other approaches have to be considered and suitable informatic tools able to help in evaluation and decision making are to be developed and/or used, in particular expert systems i.e. systems able to deal with knowledge of general nature, not only numerical.

The actual situation within the National Group is the following (01, 08, 09, 10, 11, 12, 13, 14, 15):
- there is a form, with two levels, for ordinary masonry buildings; there are models for the definition of a vulnerability index and the evaluation of retrofitting costs; there are data concerning thousands of buildings: some are both on vulnerability and on damage from recent earthquakes so that calibration studies are possible (and have been performed);
- there is a form for reinforced concrete buildings, with an old version and a recent one, and there are models for the vulnerability evaluation, both at level I and II;
- there is a form and a model for industrial buildings;
- there is a form for churches at level II (without model) and a form with a model at level I for monuments (churches and other);
- there is a methodology for the assessment of vulnerability of bridges;
- there are studies on the vulnerability assessment of buildings of special importance for their use ("hazardous buildings");
- there are studies in progress on models, new approaches, calibration, data bases, graphical restitution, fuzzy sets;
- there are studies on lifelines, in particular electrical nets; - there is a form and an expert system (so called AMADEUS) for the evaluation of the usability of buildings after earthquakes.

Reduction of Seismic Risk

The reduction of seismic risk may be considered at a global level, i.e. looking at whole populations, or it may be considered at the level of single buildings. The first case involves a general policy aiming to reduce the global risk. In the second case the approach based on the reduction of the vulnerability seems generally the most appropriate. Of course the two approaches are related, in the sense that the results obtained for the second one are useful for the first, but not exclusively.

Seismic Reinforcement of Buildings

To reduce vulnerability of a building means to reinforce its structure: such a problem has technical aspects and then economical ones. The technical question is at the center of the studies presented here in Pavia. But also the economical aspects are of paramount importance, if we consider that in a seismic zone there can be thousands and more buildings to be improved from the point of view of vulnerability.

Historical and Monumental Buildings

If the buildings concerned present historical interest and importance, and in particular if they are monuments, it isn't sufficient to think about the mere technical and economical aspects related to vulnerability and its reduction. The criteria based on optimization in the use of money, may represent one aspect of the problem, but not always the only one. The whole approach has to be reconsidered.

And the problem must be discussed distinguishing between the two cases of historical buildings and single monuments, because in the first case a population is to be considered while in the second case single objects are concerned.

More precisely appropriated definitions must be given to the terms Historical Building and Monument. Proposals have been given within EC8 in the following way:

- a MONUMENT is a building which has an important "cultural value", so high that it is necessary to guarantee the PRESERVATION of the building, generally with its architectural, typological, material characters;

- an HISTORICAL BUILDING is a building of an URBAN AREA which, as a whole, has a "cultural value" (an Historical urban area), while the single building in question is not a monument. That means that the PRESERVATION concerns the general urbanistic character of the whole area and that History may give meaningful informations.

The principles concerning vulnerability and its reduction are still valid; but the new concept of PRESERVATION may give rise to some problems both in the evaluation of the situations and in the decisions on what to do to obtain a correct SEISMIC PROTECTION.

As to the definition of risk itself, we may mention another important question: the Seismic Protection of a Monument or an Historical Urban Area is something different from the seismic protection of an ordinary building; generally seismic protection of a building means the protection of people involved, mainly human lives, and (sometimes) the protection of some material value, a value which can be expressed in monetary terms. In the case of a monument there is the "cultural value" to protect, besides safeguarding people involved in the earthquake. So the demand of protection goes in two different directions that must be considered separately. As we see, dealing with monuments and historical buildings under seismic risk new concepts, definitions, methods must be necessarily introduced. An approach that takes in consideration such questions has been proposed as an Annex within the activity of EC8 Project Team 6, as a contribution to Part 1.4 of EC8: "REPAIR AND STRENGTHENING". The proposal is presented in (16).

The mentioned EC8 proposal is only a very general approach, with few principles. To proceed further on we should establish for single situations, and in particular for single countries, specific documents related to the particular cases of buildings considered; the proposal is to prepare specific CODES OF PRACTICE. The task has received some answers in Italy: for Historical Buildings from A. Giuffre` (05,17), and for Monuments from another National Body, the National Committee for the Seismic Protection of Monumental Buildings and Italian Cultural Heritage (18, 19).

Strategic Buildings

When STRATEGIC buildings in seismic zones are concerned, the question is rather simple if the buildings are "ordinary", but it becomes difficult if the buildings are both strategic and monumental, or at least historical; and such a situation is not rare in Italy. The argument is also treated in the EC8 proposal, which indicates the rather simple concept that the requirements of a "Strategic Seismic Safety" (to ensure the efficiency of the building after the earthquake) could possibly not agree with the "Monument Seismic Safety and Preservation": in such cases the solution is often trivial: the building may remain or strategic or monumental and the official Bodies in charge for the decision must choose! If some possibilities of an "ecumenical" solution seem to be at hand, a serious effort may be made by the Bodies, after a cost/benefit analysis where the benefit is an increase of safety and the cost is a loss of integrity.

Practical Problems: Codes, Knowledge, Applications

If one wants to solve the problem of reducing seismic risk for existing populations of buildings, both ordinary and monumental, according to the general concepts mentioned above, a very important consideration has to be made: the problem is very difficult, and may become impossible, if the technical Codes do not "help". Indeed the technical Codes are generally related to the questions concerning "new" buildings and not "existing" ones, and in such conditions there are many difficulties. Appropriated principles, as the ones mentioned before, have to be established by Law and implemented in technical Codes, and then in Codes of Practice. The proposal within EC8 and the documents proposed in Italy are examples in the correct direction, as well as the international proposal Skopje 88 (20) and the Californian text State Historical Building Code (21).

Another question related to the real possibility to perform systematically the reduction of seismic risk is the problem of spreading knowledge among Technicians, a task which is far from being trivial. In such a task, besides the usual tools like books and Courses, the more modern informatic devices seem to be useful, like Softwares giving help in Design, Data Bases with interesting Case Histories, presentation of Codes in form of Hypertexts and so on.

Finally, help to the task of retrofitting existing buildings in seismic zones, must come directly in the field of applications, in form of Laws (think on Los Angeles, Ordinance 154807, Jan 7 1981), Insurances' contribution, organization of works on entire urban sectors. The Civilization of working to correct the existing situation must take the place of creating everything new.

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Section VI

Testing Methods for Evaluation of Insitu Material Properties

MEASURING MASONRY MATERIAL PROPERTIES

Luigia Binda [1], Giulio Mirabella Roberti [2], Claudia Tiraboschi [3] and Silvia Abbaneo [4]

ABSTRACT

Results from comparative experimental studies on bricks and mortars used to build the full-scale model and on assemblages and small prisms of the same materials are presented. Compressive and flexural tests, shear and tensile bond strength tests are carried on with the aims of: (i) measuring the most important parameters to be used for modeling the material behaviour, (ii) correlating specimen size and dimension to the experimental results, (iii) comparing test methods proposed by international codes on testing masonry materials. During the research several problems were focused, mainly concerning the difficulty of measuring the displacements on small specimens of weak materials as the bricks and the mortars used for the full-scale model. Some of the problems have already been solved by the careful calibration of the measurement methods and equipments.

INTRODUCTION

The materials to be tested were carefully controlled to be the same as the ones used for the full-scale model. The bricks were sent directly from Pavia to the D.I.S., Politecnico and mortars and prisms were prepared by the same masons who had built the two floor model in Pavia. Every time a new quantity of mortar was prepared, specimens were also sampled for laboratory tests.

Codified tests on the hydraulic lime and aggregates were performed so as tests on fresh mortars. On bricks and hardened mortars chemical, optical and physical analyses were carried on, in order to gain better knowledge on possible mutual influence of the materials on the masonry behaviour. The chemical analysis of the lime confirmed that it is an hydraulic lime. Typical characteristic of the mortar prepared by the masons was a high water/binder and a low binder/aggregate ratio, while the grain size and distribution of the siliceous aggregate was varying within a quite narrow range, these properties indicating that the mortar would be a weak strength mortar. In fact this mortar was intended to simulate, in the full-scale model, the weak joints usually found in existing buildings. The soft mud made bricks of the kind frequently used in practice for restoration, also simulate the solid bricks found in old and ancient masonries. These bricks are characterized by high water absorption and capillary rise, rather low strength. Due to the mentioned properties of the materials a successful bond between bricks and mortars is seldom realized and also the compressive and shear strength of the walls can be fairly low.

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Compression tests were performed on whole bricks, cubes and small prisms cut out of bricks, cubes and prisms of mortars at different time curing. It was clear after the first tests were carried on, the great difficulty of obtaining reliable values for elasticity modulus and Poisson ratio due to the influence of the specimen dimension and of the low strength and stiffness of the materials. Therefore different types of machine constraints and measurement techniques for the specimen displacements were adopted and comparison were made between the results in order to achieve the possible best knowledge and give the right information to the researchers working on analytical models.

Shear bond tests (triplet tests) were also performed trying to measure at the best vertical and horizontal absolute and differential displacements within the specimens, in order to control the real meaning of the test for detecting shear bond and to calibrate some inelastic models.

Finally the compressive tests on masonry prisms carried on under displacement control with an MTS servocontrolled machine (250 tons), allowed for the detection of post peak behaviour of the masonry and for a better understanding of the stress distribution in the specimens. Others, like couplet tensile bond strength tests also gave some interesting information.

OUTLINE OF THE EXPERIMENTAL WORK AND RESULTS

The results of the experimental work developed until now are reported in the following and commented in order to give a first evaluation of the data to be used for mathematical models. The tests were carried on according to several codes which are listed in Annex 1 of [1].

Specimen and test design

Whole bricks were submitted after surface grinding to compressive tests in the three principal direction following the request of the Italian code [2]. Flexural tests were also performed on whole bricks; biaxial and uniaxial compressive tests will also be later carried on as shown in Fig. 1. Cubes, 40 mm side and prisms (40x40x120 mm) were cut out of whole bricks; mortar prisms (40x40x160 mm and 40x40x120 mm) were also prepared for the flexural and compressive tests at different times of curing (Fig. 2). In Fig. 3 are shown the designed specimens for tensile and shear bond tests and for tests on masonry.

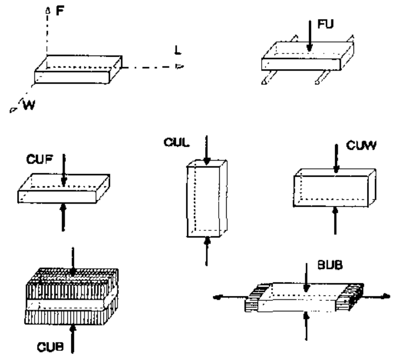
Chemical and optical tests on bricks and mortars

Mortar properties. The binder was defined by the producer (Legoplast) as hydraulic lime; in Table 1 the results of the chemical analysis [3] is reported confirming a high content in soluble silica. The sand is coming from the Ticino river near Pavia; in Fig. 4 its grain size and distribution is represented. This aggregate contains quartz, feldspar and flints (Annex 2 of [1]).

Brick properties. The bricks have a soluble salt content of .36 % and a sulfate content of .015 % largely within the range allowed by the Italian code [4]. Three slightly different colours were detected for the bricks and the results of petrographical-mineralogical analyses on all three types (Annex 2 of [1]) are given in the following. They all are characterized by two distinct grain sizes, the first one with an average diameter below 0.1 mm, the second one between 0.1 and 1.0 mm. The lithological composition is mainly given by quartz mono- and polycrystalline and minor quantities of feldspar and mica (Fig. 5).

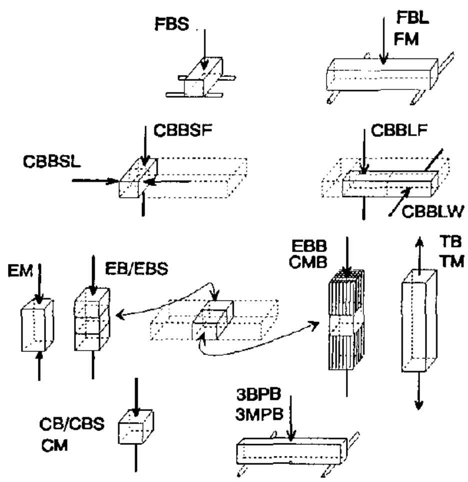
Physical properties

As previously mentioned mortar specimens were prepared by the masons while working at the construction of masonry specimens (MIX1); these mortars were cured at 20°C and 90% R.H. for more than 28 days, then cured at 20°C and 50% R.H. until tested in order to simulate the air curing



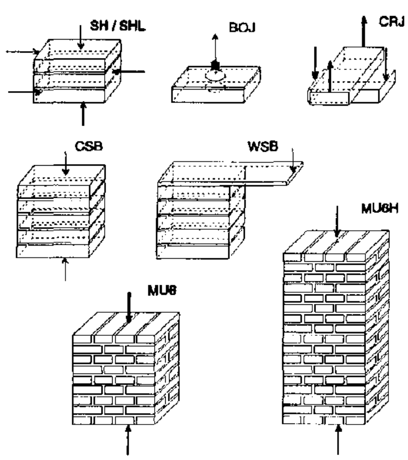
*CUF	COMPR. STR., UNIT, FLATWISE
*CUL	COMPR. STR., UNIT, LENGTHWISE
*CUW	COMPR. STR., UNIT, WIDTHWISE
*FU	FLEX. STR., UNIT
CUB	COMPR. STR., UNIT, BRUSH
BUB	BIAXIAL, UNIT, BRUSH

Fig. 1 Test design for whole units.



*FBS	FLEX. STR., BRICK, SHORT (4x4x12 cm)
*FBL / *FM	FLEX., STR., BRICK, LONG (4x4x16 cm)/ MORTAR
CBBSL	COMPR. STR., BRICK, BRUSH, SHORT, LENGTHWISE
CBBLF	COMPR. STR., BRICK, BRUSH, LONG, FLATWISE
CBBLW	COMPR. STR., BRICK, BRUSH, LONG, WIDTHWISE
*EB / *EM	ELASTIC MOD., BRICK (4x4x12 cm) ELASTIC MOD., MORTAR (4x4x10 cm)
*EBS	ELASTIC MOD., BRICK, STRAIN GAUGES
*EBB / *CMB	ELASTIC MOD., BRICK, BRUSH COMPR. STR., MORTAR, BRUSH
*CB / *CM	COMPR. STR., BRICK / MORTAR
*CBS	COMPR. STR., BRICK, STRAIN GAUGES
TB / TM	TENS. STR., BRICK - TENS. STR., MORTAR
3BPB/3MPB	3 (BRICK) / (MORTAR) POINT BENDING

Fig. 2 Test design for brick and mortar cubes and prisms.



*SHL	SHEAR BOND TEST LONG (LENGTH 23 cm)
*SH	SHEAR BOND TEST (LENGTH 17 cm)
BOJ	BOND, JOINT
*CRJ	CROSS, JOINT
CSB	COMPR., STACK, BOND
WSB	WRENCH, STACK, BOND
MU6	MASONRY UNIT, 6, (HEIGHT 60 cm)
*MU6H	MASONRY UNIT, 6, (HEIGHT 110 cm)

Fig. 3 Test design for small masonry assemblages and prisms.

COMPONENT	%
SiO_2	20.80
Al_2O_3	2.67
Fe_2O_3	2.81
CaO	44.36
MgO	1.85
Na_2O	0.74
K_2O	1.72
SO_3	1.23
Loss on ignition	23.68
CO_2	21.03
Sol.Sil.	5.70
S	0.024
Insoluble residue	15.77

Tab. 1 - Chemical analysis of the hydraulic lime

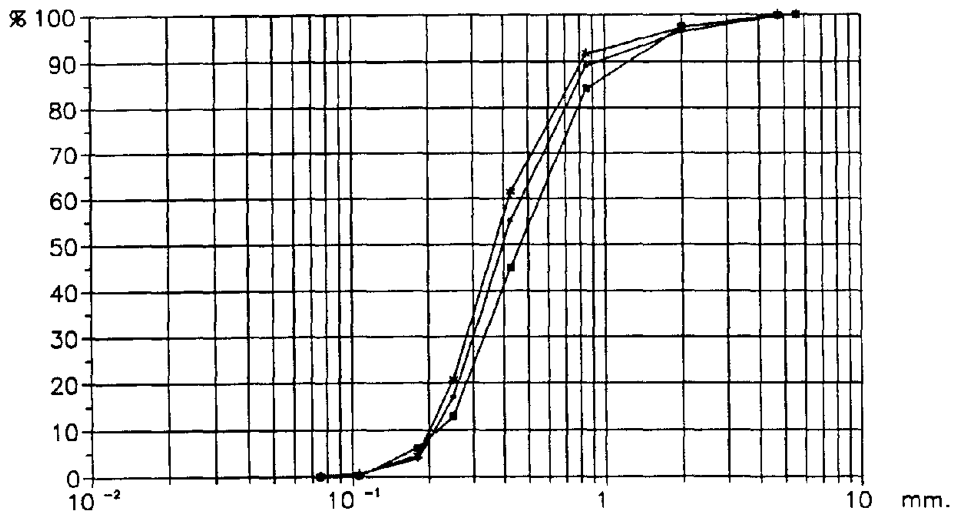


Fig. 4 - Grain size and distribution of the sand

of the full-scale model. In order to refer better to the usual preparation of laboratory specimens, another mix (MIX2) was prepared later on, having a lower ratio binder/aggregate (1/3) and also a lower ratio water/binder (0.69 instead of 1.1). This second type of specimens is still being cured at 20°C and 90% R.H. for the 180 days tests; therefore only results of tests after 28, 60 and 90 days are here reported. The most important physical properties influencing the mechanical behaviour of the materials and the adhesion between mortars and bricks are the ones connected with the workability, expansion and water retention of fresh mortars and the initial rate of suction of bricks.

Mortar properties. The physical tests on fresh mortar were performed following the Italian Code [3], [5], [6] or CEN prEN [7] as reported in the Annex 1 and the results on MIX1 and MIX2 are reported in Table 2. The physical tests on hardened mortars [7], are presented in Table 3. For MIX1 the mean values for several preparations during the masonry specimen construction are reported. In fact there was practically a coincidence between the different preparations as it can be seen in Annex 2. A reduction of the water absorption coefficient and a tendency to increase of the bulk density with the time of curing can be remarked for MIX1. MIX2, as expected due to its composition and to laboratory curing, has a lower bulk density, water absorption coefficient and water absorption percentage with respect to MIX1.

Brick properties. The physical tests on bricks were performed following prEN draft proposals [8] and RILEM LUM 76 Recommendations [9]. The results are presented in Table 4, were the data are given separately for two different colours, defined red and braun found in the batches from which the specimens were sampled. Apparently the two types of bricks have the same properties, with slight differences, apart from the total absorption which seems to be the 11% lower for the braun bricks. In the following the two types of bricks will not be separated any more. Both types of bricks show a weak tendency to efflorescence according to the Italian Code [4]. The two types of bricks have a rather low bulk density compared to other facing bricks and a high I.R.S.

If the physical properties of mortars and bricks are compared the following comments can be done: (i) the total absorption by immersion is practically the same for the two materials, therefore if salt or frost-defrost actions affect their prformance, they will decay at the same time, (ii) the high initial rate of suction of the bricks can cause serious problems to the bond between brick and mortar joints, unless the bricks are not wetted prior to their use in the wall construction, (iii) the high porosity and low bulk density of the bricks indicate that they will be low strength bricks. In Fig. 6 the average capillary rise diagram is presented for bricks and the two mortars,showing the much higher capillary rise of bricks and the difference between MIX1 and MIX2.

Mechanical tests

The aims of these tests were to detect as much as possible mechanical parameters useful for the analitical modelization purpouses and furthermore to compare different test methods and procedures required or suggested by national and international codes of standard and recommendations. Therefore the specimen shape, dimension and preparation and the testing procedure were very carefully studied for every test. Tests on single materials (mortars and bricks), small assemblages and wall prisms are being carried on, and special attention is paid to the equipment and procedure adopted for measuring loads and displacements.

Compression tests on bricks and mortars. The first problem to be solved concerned the specimen dimension and preparation. In the case of solid bricks, testing of the whole unit can be chosen, as for other masonry units; nevertheless the height of the bricks is so small (no more than 40mm after grinding), that the inevitable friction between the steel platens of the machine and the brick contact surfaces can heavely influence the value of the strength of the unit. Therefore cubes of 40mm side

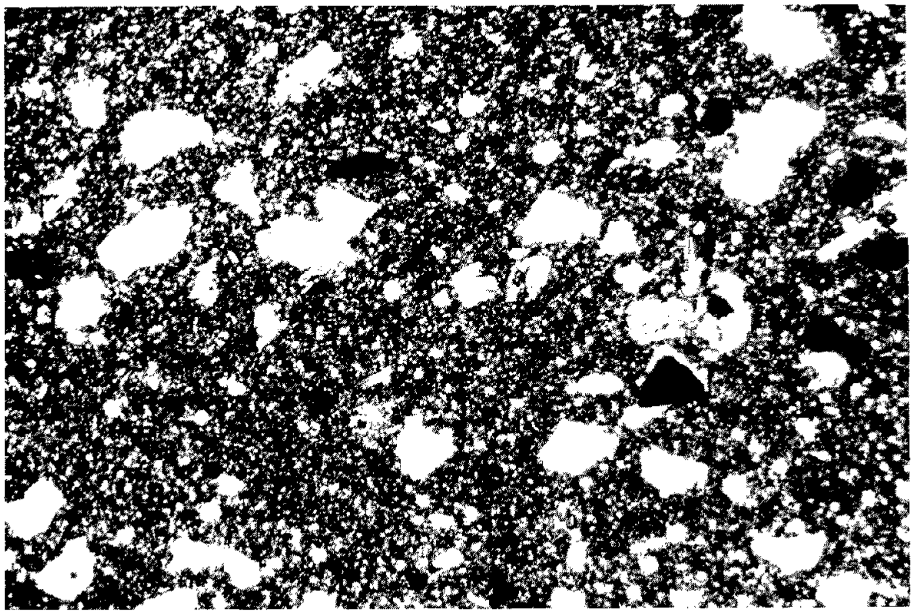


Fig. 5 - Thin section of the brick; the two distinct grain size of the quartz are clearly visible

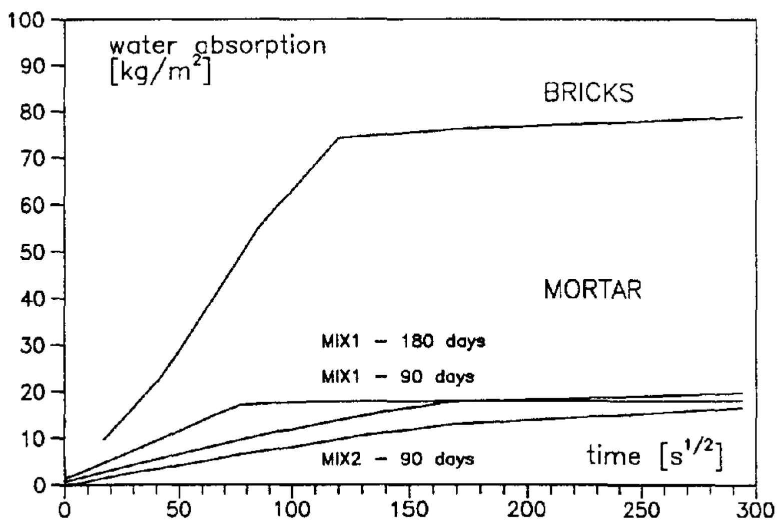


Fig. 6 - Capillary rise average plots for mortars and bricks

N.	MORTAR						LIME PASTE		
	COMPOSITION (by Weight)			FLOW VALUE	EXPANSION	WATER RETENTIVITY	INDEFOR-MABILITY (Le Chatelier)	SETTING TIME (Vicat)	
	B.	AG.	w/b	mm	%	%	mm	Start	End
MIX1	1	4.4	1.1	192	-1.7	71.5	0.0	3h50	7h20
MIX2	1	3	0.6	170	-0.7	77.9	0.0	3h50	7h20

Tab. 2 - Test results on fresh mortar and lime

N.	BULK DENSITY			WATER ABSORPTION COEFFICIENT (capillary suction)			WATER ABSORPTION		
	kg/m³			$kg/(m^2h^{1/2})$			%		
	90 days	180 days	360 days	90 days	180 days	360 days	90 days	180 days	360 days
MIX1	1724	1755	1747	6.54	12.86	13.66	18.39	15	15.23
MIX2	1688	-	-	4.79	-	-	17.43	-	-

Tab. 3 - Test results on hardened mortars

BRICK COLOUR	SOLUBLE SALT	ALKALINE SULPHATE	EFFLORE-SCENCE ATTITUDE	BULK DENSITY	I.R.S.	WATER ABSORPTION COEFFICIENT (capillary suction)	WATER ABSORPTION
	%	%		kg/m³	$(kg/m^2)/min$	$kg/(m^2 min^{1/2})$	%
BROWN	0.36	0.015	very slight	1677	4.31	5.16	18.88
RED	-	-	very slight	1687	4.49	-	16.29

Tab. 4 - Physical tests on bricks

are cut out of the whole brick and subsequently tested; the cutting scheme is given in Fig. 7, taking into account the expected strength and stiffness distribution within the brick itself, due to the production process .

In order to detect the elastic modulus and the Poisson ratio of the materials, prisms rather than cubes should be tested, but due to the low height of the solid bricks (55 mm) it is impossible to cut prisms normally to the flatwise direction. Therefore tests on cubes simply grinded, with brushes between the platens and on three alligned cubes as shown in Fig. 2 (EB/EBS) are being compared. When testing cubes the measure of the displacements in the vertical and horizontal direction becomes very difficult due to the reduced space for applying instruments. LVDTs cannot be in fact applied directly on the specimen, but only between the two plates so that the measure of the displacement is influenced for low loads by the presence of the plates. Strain gauges were applied to the specimens, but it could be detected that the stiffness of the glue used for attaching the gauge to the specimen can influence the results. Estensometers are also being used when dealing with small specimens.

Compressive strength and deformability of whole units. The tests were carried on according to Fig. 1; 20 specimens were tested flatwise, 10 lengthwise and 10 widthwise and the average results together with the standard deviation and the variation coefficient are given in Table 5 (Annex 3 of [1]). As expected the highest strength s given in the case of CUF specimens due to the low ratio between height and length and width; the lowest value is given by CULs due to instability phenomena. On the contrary the maximum deformability seems to be obtained in the case of CUF. In Fig. 8 three stress- strain curves are presented as an example; it is clear the different behaviour of the bricks, particularly the strength normally to the flatwise direction.

Compressive strength of cubes and prisms. Specimens CB, EB, EBB, CM, EM, EMB were tested in order to compare and possibly correlate the results obtained on cubes and prisms for the strength and deformability. This research is being carried on, as already mentioned, due to the fact that only cubes can be tested in the case of bricks used flatwise; a similar problem concerns the mortars, since the code requires the use of 40x40x160 mm specimens. As mentioned above the difficulty concerns more the calculation of elasticity modulus and Poisson ratio, which should be obtained from prisms. In the case of bricks the specimens EB were built with three cubes obtained as in Fig. 3 and their data were compared with CB and EBB, the second ones tested with steel brushes between two plates [10]. In Table 6 the mean values and variation coefficients of the peak stress f_u, ultimate strain ε_u and secant elasticity modulus E_s calculated between the 30 and 60 % of the peak stress are given for CB, EB. As expected the highest value of f_u is achieved by the CB specimens. In Table 7 the mean values and variation coefficient are given of compressive strength, ultimate strain and elasticity modulus for mortar specimens (MIX1 and MIX2) at different times of curing. In Fig. 9 the same values are plotted as a function of time for the two mixes. The strength at 28 days of curing is given for MIX1 prepared in Pavia. The unexpected low strength of MIX1 can be explained as dependent on the high ratio w/b and b/s used for that mix, while the reduction of strength with time is probably due to the type of curing adopted for MIX1. Curing dry can slightly reduce the strength in the case of hydraulic mortars as shown in Fig. 10 [11].

As already mentioned, one of the difficulties to be faced was the right choice of measurement technique of horizontal and vertical displacements in the case of small specimens (EB, EM, CB, CM, EBB, EMB) when it was impossible to apply LVDTs directly on the specimen itself. In Fig. 11a is shown the device used for the measurment of the displacements. Fig. 11b,c gives as an example the plot of stress versus strain measured by vertical and horizontal strain gauges applied on two EBS and CBS specimens and vertical and horizontal displacements measured by LVDTs

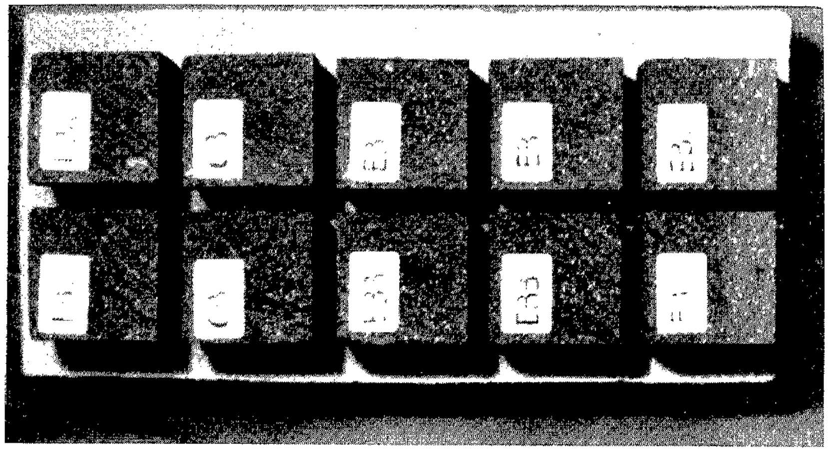


Fig. 7 - Cutting scheme for bricks

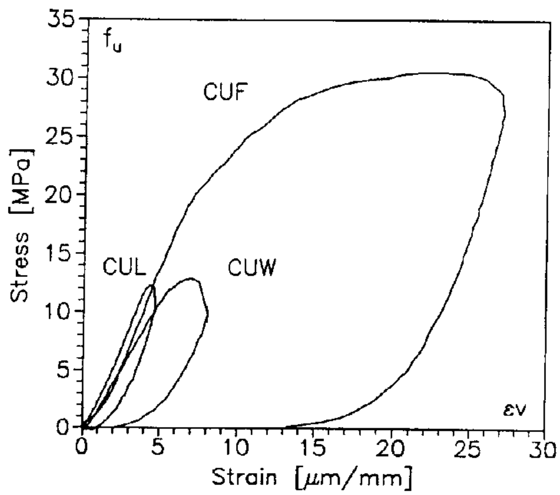


Fig. 8 - Stress-strain curves for whole units

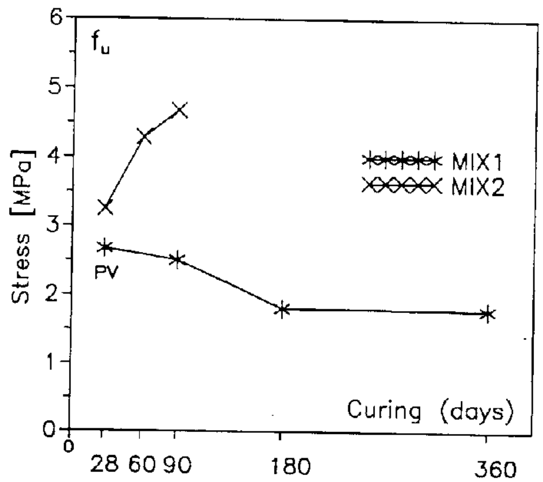


Fig. 9 - Strength versus curing time for mortars (MIX1, MIX2)

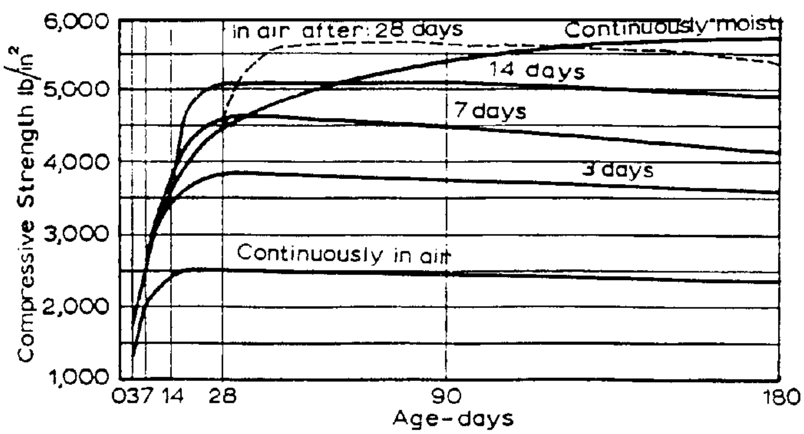


Fig. 10 - Influence of moist and air curing on concrete strength

	CUF	CUW	CUL
N. of tests	20	10	10
f_u[MPa]	26.50	11.05	10.70
$\underline{v}$	10.52%	11.60%	14.92%
N. of tests	-	10	10
$\varepsilon_u 10^{-3}$	-	6.60	4.32
$\underline{v}$	-	9.42%	6.88%
N. of tests	20	10	10
$E_{s(30\text{-}50\%)}$[MPa]	2758	2020	2766
$\underline{v}$	16.96%	8.76%	11.58%

where:
f_u = peak stress
ε_u = peak strain
E_s = secant modulus between 30 and 50% of f_u
$\underline{v}$ = coefficient of variation

Tab. 5 - Compressive test results for whole units

	CB	EB
N. of tests	25	23
f_u[MPa]	14.25	10.52
$\underline{v}$	12.93%	20.28%
N. of tests	17	13
$\varepsilon_u 10^{-3}$	9.01	5.66
$\underline{v}$	9.32%	10.77%
N. of tests	18	23
$E_{s(30\text{-}60\%)}$[MPa]	2171	2156
$\underline{v}$	13.71%	20.15%

Tab. 6 - Compressive test results for cubes and prisms from bricks

	MIX1					MIX 2		
	CM				EM	CM		
Curing	28 days	90 days	180 days	360 days	360 days	28 days	60 days	90 days
N. of tests	12	8	8	10	10	20	20	20
f_u[MPa]	2.61	2.46	1.79	1.78	1.07	3.25	4.28	4.69
$\underline{v}$	8.25%	11.88%	5.54%	6.09%	5.71%	6.05%	3.35%	3.94%
$\varepsilon_u 10^{-3}$	-	-	-	-	3.63	-	-	-
$\underline{v}$	-	-	-	-	18.22%	-	-	-
$E_{s(30\text{-}60\%)}$[MPa	-	-	-	-	533	-	-	-
$\underline{v}$	-	-	-	-	26.11%	-	-	-

Tab. 7 - Compressive test results for cubes and prisms from mortar

	FU	FBL	FBS	FM (MIX 1)		FM (MIX 2)		
Curing				28 days	90 days	28 days	60 days	90 days
N. of tests	30	10	10	6	13	10	10	10
f_{tf} [MPa]	2.44	3.28	3.04	1.08	0.61	1.39	1.80	2.02
$\underline{v}$	25.90%	6.90%	8.58%	5%	19.9%	6.26%	3.33%	4.95%

Tab. 8 - Flexural strength test results for whole units, brick and mortar prisms

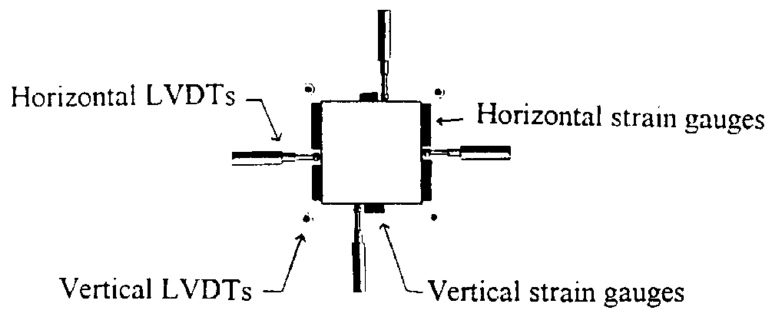


Fig. 11a - Device for the measurement of the displacements

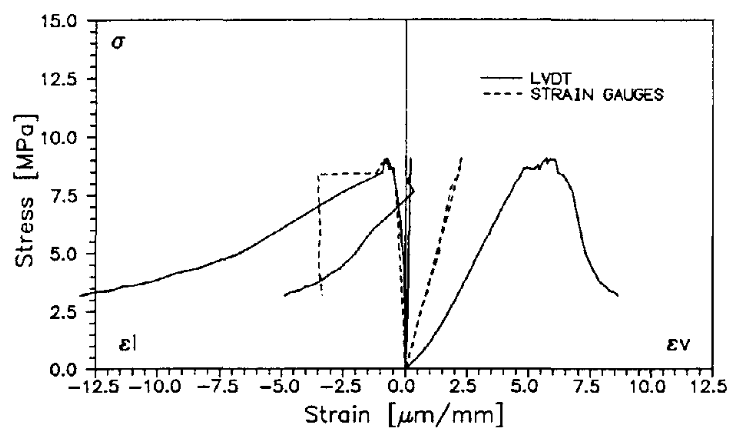


Fig. 11b - Stress-strain curves for specimen EBS

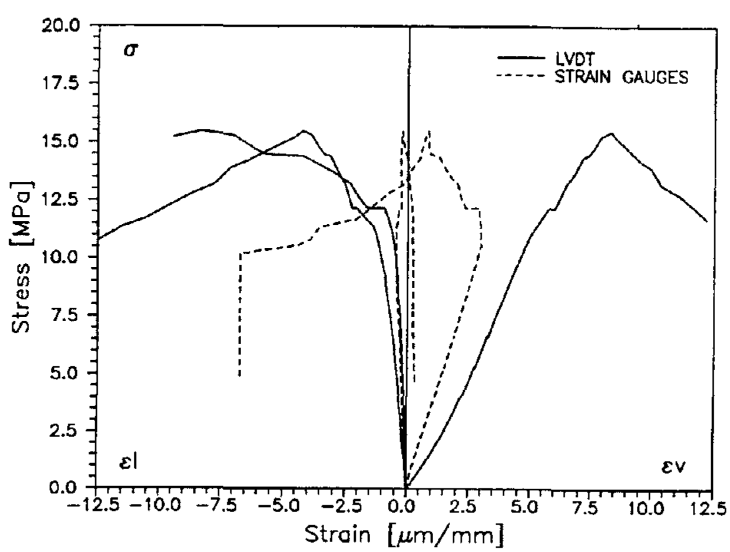


Fig. 11c - Stress-strain curves for specimen CBS applied between the plates and in the middle of the specimen. While there is coincidence on horizontal displacements, the vertical displacements measured with LVDTs are two times the ones measured with strain gauges. This difference cannot be accepted, therefore it is important to understand which measure of the two is really representing the specimen deformation. Fig. 12 gives the stress-strain plot for the glue used to attach the strain gauges. The strength and stiffness of this glue (which was already chosen as one with the lowest stiffness among the available glues for laboratory purpouses), are much higher than those of the brick. In fact the vertical stiffness measured on EBS and CBS by strain gauges is practically coincident with the one of the glue.

In order to detect the reliability of this conclusion, ESPI (Electronic Speckle Pattern Interferometry) was used to measure the vertical and horizontal displacements on a brick specimen on which strain gauges were applied; the work was done in collaboration with M.Facchini a Phd student working at Euratom-Ispra in Varese (Italy). As shown in Fig. 13a the stiffness of the glue is so much influent that interferometry fringes detect clearly the position of the gauge; Fig. 13b shows the same differences, as found in laboratory between strain gauges and LVDTs.

Finally the estensometers CPDT (Clamp in Point Displacement Transducer) shown in Fig. 14a, with a base of 20 mm, positioned in the center of the specimen, were used to measure vertical displacements and the results obtained were compared to the ones of LVDTs. In Fig. 14b is shown the whole device for the measurement of the displacements. The curves plotted in Fig. 15 show that the LVDT curve in its linear part is parallel to the extensometer curve; therefore the secant modulus obtained by LVDTs between the 30 and 60 % of the peak stress can be accepted as a reliable value for the elasticity modulus.

Concerning Poisson ratio, the deformation in the horizontal direction during the elastic phase is so low that the difference between strain gauge and LVDT is minimum (Fig. 11a, b); therefore the LVDT measurement, being the less expensive and most useful device also for detection of post-peak deformations, could be recommended also for codified tests. In order to calculate the elasticity modulus, EB specimens should be preferred to CB specimens in the case of bricks; they also give a very low coefficient of variation, even if their preparation is much more difficult than the one of CB specimens. EBB specimens should avoid this difficulty since the steel brushes could eliminate the plate influence [10]. Few tests were carried on with loading-unloading cycles in order to compare the results with the ones given by EB specimens. Fig. 16 a, b gives two stress-strain plots as an example. Nevertheless some more work must be done to adapt the stiffness of the brushes to the brick and mortar specimens; this work will be continued in the future research. When accomplished there will be an alternative to EB specimens in the case of bricks. The detailed complete results from all the tests on brick and mortar specimens are given in Annex 3 of [1].

Flexural strength of whole units and small prisms. Table 8 gives the mean value and the variation coefficient of the flexural strength of whole units FU tested according to Fig. 1 and for specimens FBL, FBS, FM (MIX1and MIX2) according to Fig. 2. High scattering in data result for FU due to the specimen dimension amd FM for MIX1 due probably to cracks developed by shrinkage. There is instead a good behaviour of MIX2 with an increasing of flexural strength with time.

Tensile bond strength tests. These tests (CRJ in Fig. 3) were carried on following the recommendations[8] and using MIX1 and MIX2. The results are given in Table 9 together with the description of the failure mechanisms. Once again a reduction of the strength with curing time can be observed when MIX1 was used, as it happened with all the tests on cubes and prisms and a higher strength when MIX2 was used; nevertheless the scattering in the results is very high for both mortars. This is unfortunately the characteristic of that type of test, which otherwise could be very interesting for detecting the tensile bond strength; when the bond between mortar and bricks is very weak, then there is a random behaviour of the specimens, due to the dependency of the results from the local situation.

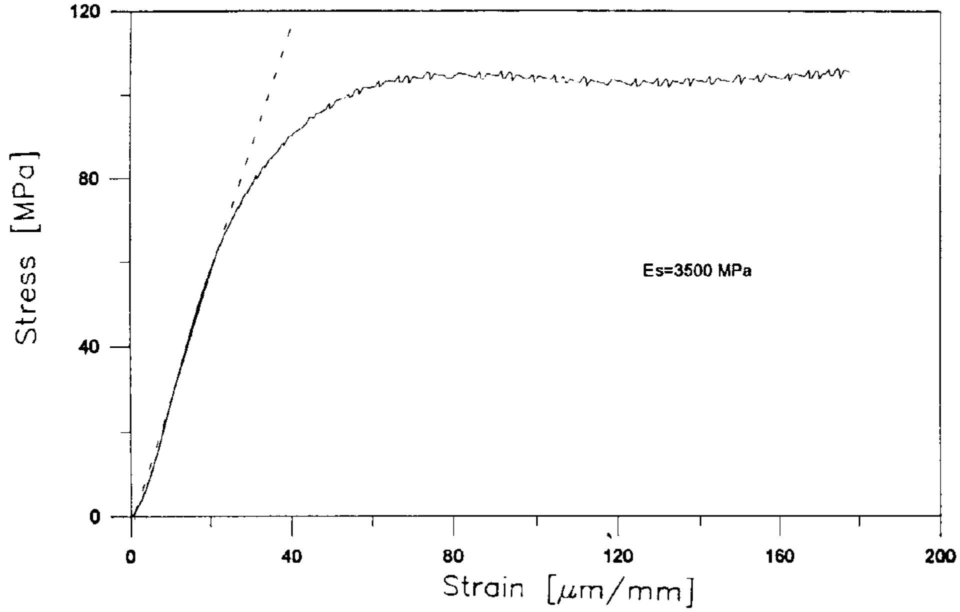


Fig. 12 - Stress-strain plot of the glue used to attach strain gauges

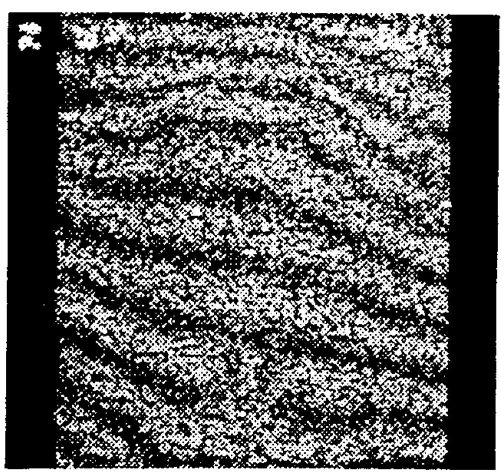


Fig. 13a - Presence of the strain gauges revealed by the intereferometry

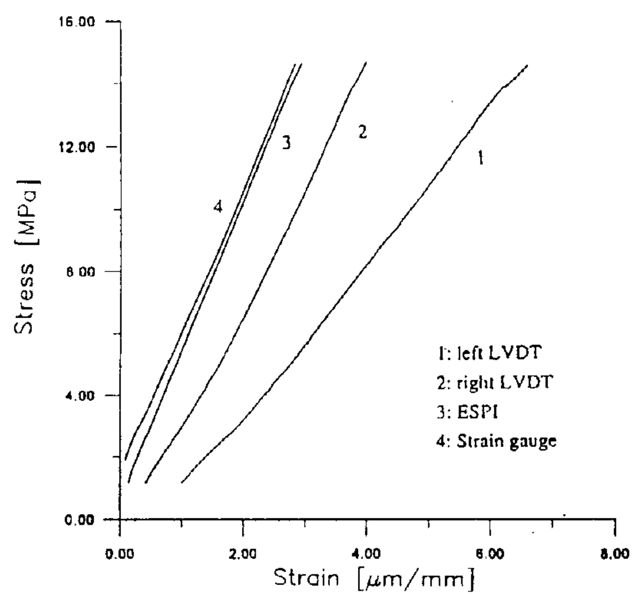


Fig. 13b - Stress-strain curves obtained with the strain gauges and by measuring the interferometric fringes

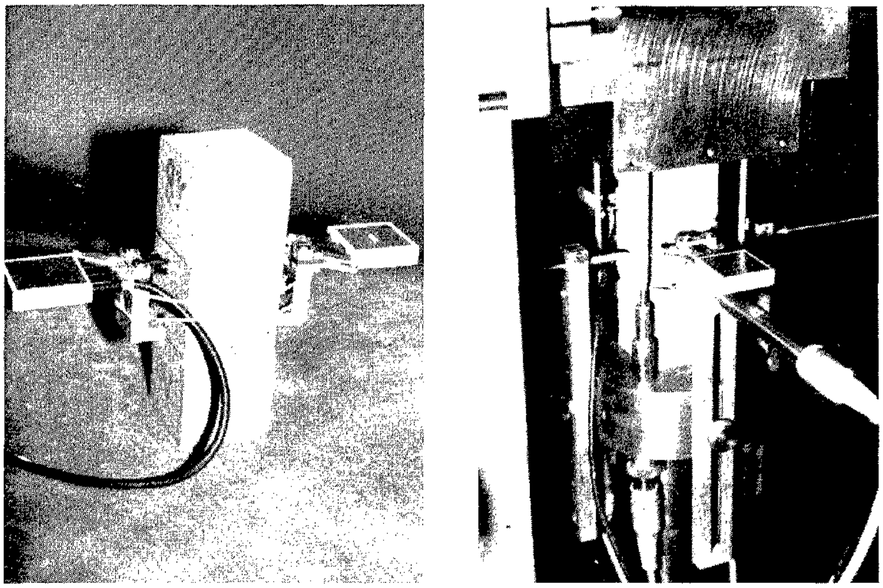


Fig. 14a,b - The extensometers (CPDT, Clamp on Point Displacement Transducer) used for measuring vertical displacements (a), together with the LVDTs (b)

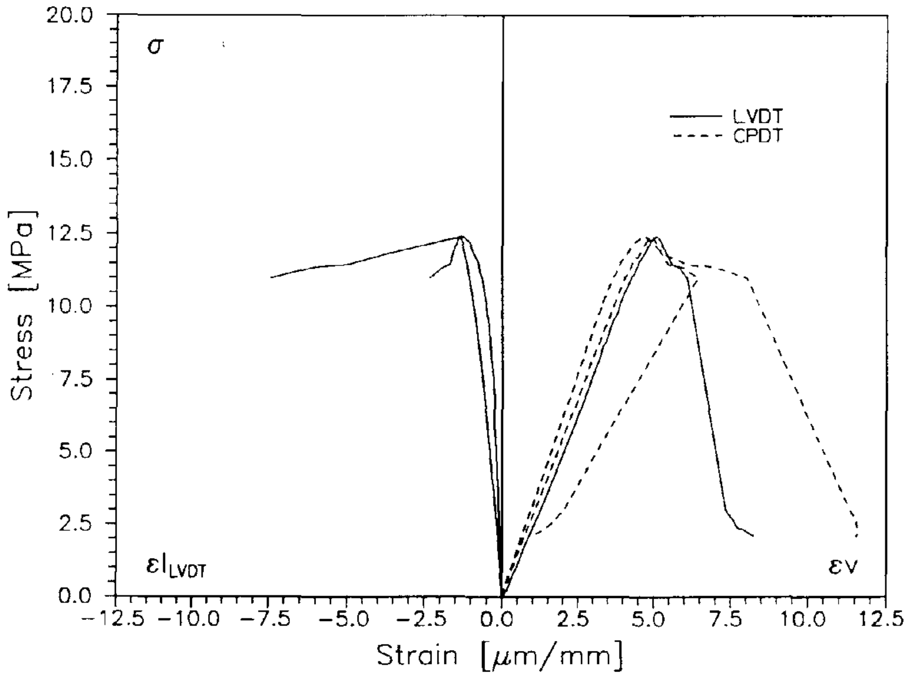


Fig. 15 - Stress-strain curves obtained from CPDT and LVDT measurements for EB

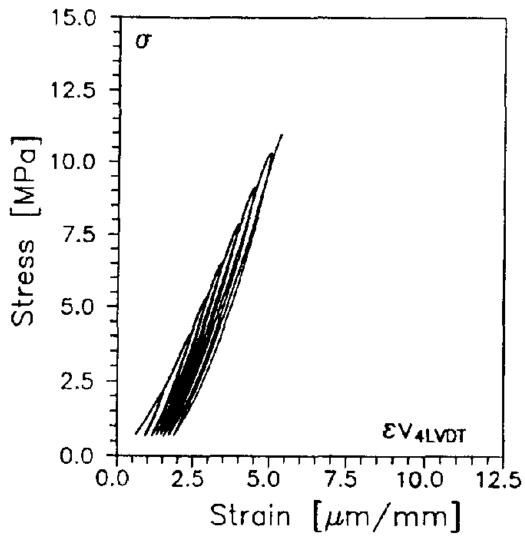


Fig. 16a - Stress-strain plot for EB under cyclic vertical load

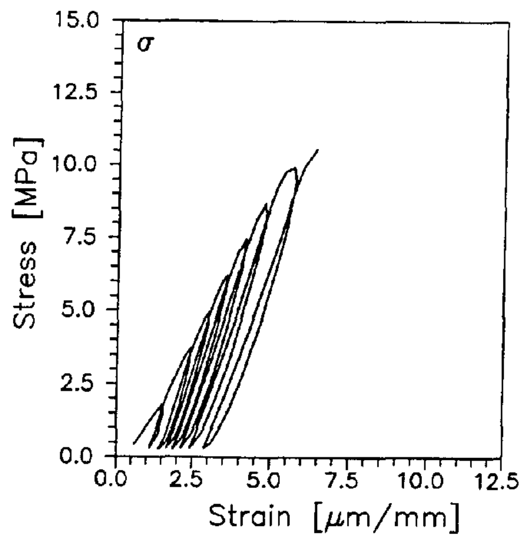


Fig. 16b - Stress-strain plot for EBB under cyclic vertical load

Sanded side (S) / (US) Unsanded side	MIX 1		MIX 2
	90 days - 20°C, 90% R.H.	180 days - 20°C, 50% R.H.	90 days - 20°C, 90% R.H.
Number of specimens	7	12	16
f_t [MPa]	0.085	0.039	0.069
$\underline{v}$	30.20%	23.90%	27.38%
Failure mechanism			
(M) Mixed detachment	1 M	0 M	0 M
(US) Detachment from the unsanded side	1 US	4 US	9 US
(S) Detachment from the sanded side	5 S	8 S	7 S

Tab. 9 - Tensile bond strength tests results

Shear bond (triplet) test. The specimens were built according to Fig. 3; the bottom and the top surfaces were carefully controlled to be plane and parallel by capping with a gypsum thin layer and the vertical faces were grinded to ensure a good contact with the testing device. Two different series of specimen were tested: the first one with a length of 170 mm, the second one with a length of 24 mm (approximately a whole unit length), in order to detect the influence of the specimen dimensions. The tests were carried on following RILEM 127 MS draft [12]; constant vertical loads were applied to the specimens, at different levels as follows: 0.12, 0.40, 0.80, 1.25 MPa, in order to calculate the friction angle at the contact mortar joint-brick. The middle brick was loaded by an horizontal jack having the contrasts on the upper and lower brick. The horizontal displacements of the middle brick were recorded as well as the differential displacements of the upper and lower brick; the total vertical displacements were recorded as well (Fig. 17). Three triplets were tested for each level of vertical load applied for the two series of specimens; the load-displacement curves are presented in Fig. 18 for the different levels of vertical load.

The load-displacement plot for the second group of specimens (240 mm) show a quite low scattering in the data compared to the first one, due to the greater length; this allows to draw a line interpolating the peak shear stress values (using the mean values of the peak shear loads) against the corresponding mean normal stress. The results are given in Table 10 and shown in Fig. 19 in a τ-σ plane. Even if these values are not representing the real local situation, the good correlation found shows that a mean shear strength can be calculated once the normal stress is evaluated. Some useful information can also be derived from the softening branch of the load-displacement curves, indicating that a residual strength is still available under large displacements, mainly due to the friction contribution, when a normal load is applied. The detailed results can be found in Annex 3 of [1].

NAME	AREA	P [KN]	T [KN]	σ_v [MPa]	τ_p [MPa]
SHL012A	283.2	3.40	17.354	0.12	0.31
SHL012B	285.6	3.43	15.636	0.12	0.27
SHL012C	284.4	3.41	13.014	0.12	0.23
SHL040A	284.4	11.38	26.173	0.40	0.46
SHL040B	285.6	11.42	26.654	0.40	0.47
SHL040C	288	11.52	30.004	0.40	0.52
SHL080A	285.6	22.85	42.082	0.80	0.74
SHL080B	285.6	22.85	38.059	0.80	0.67
SHL080C	286.8	22.94	41.714	0.80	0.73
SHL125A	283.2	35.4	52.523	1.25	0.93
SHL125B	282	35.25	53.736	1.25	0.95
SHL125C	284.4	35.55	50.954	1.25	0.9
SHL000A	285.6	0	1.818	0	0.032

Tab. 10 - Shear bond test

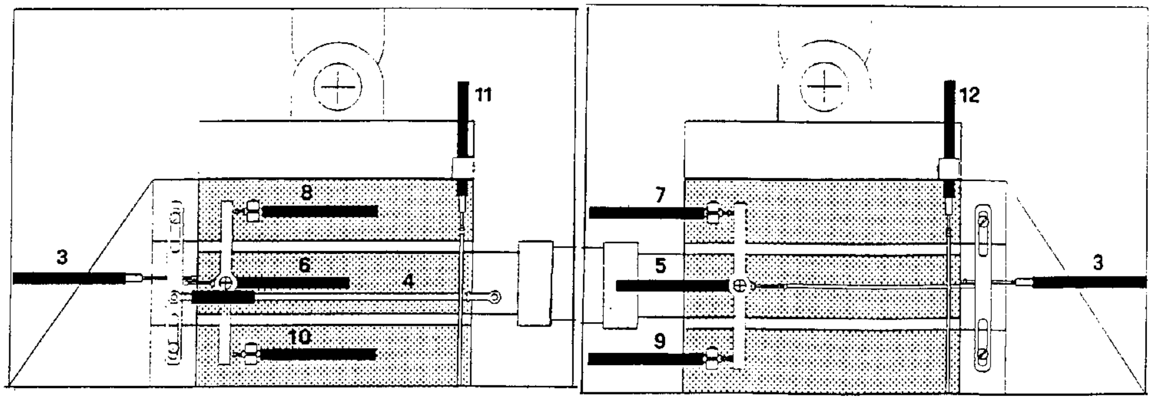


Fig. 17 - Design for displacement measurements on the triplet

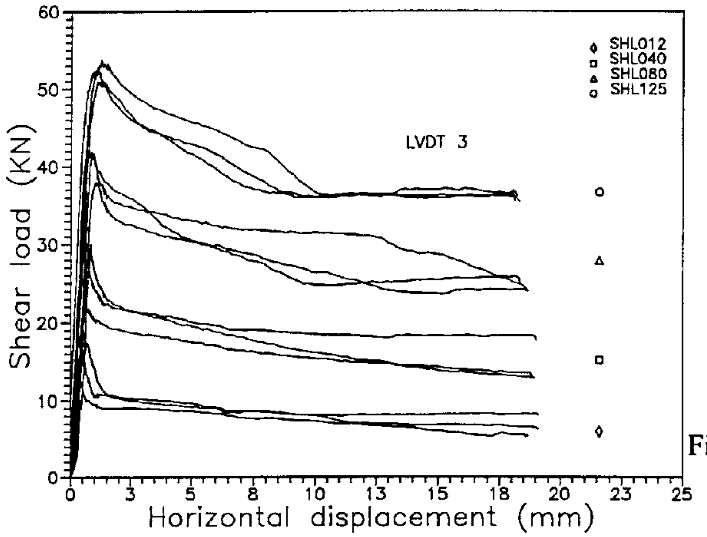


Fig. 18 - Shear load-displacement curve for the different values of horizontal load

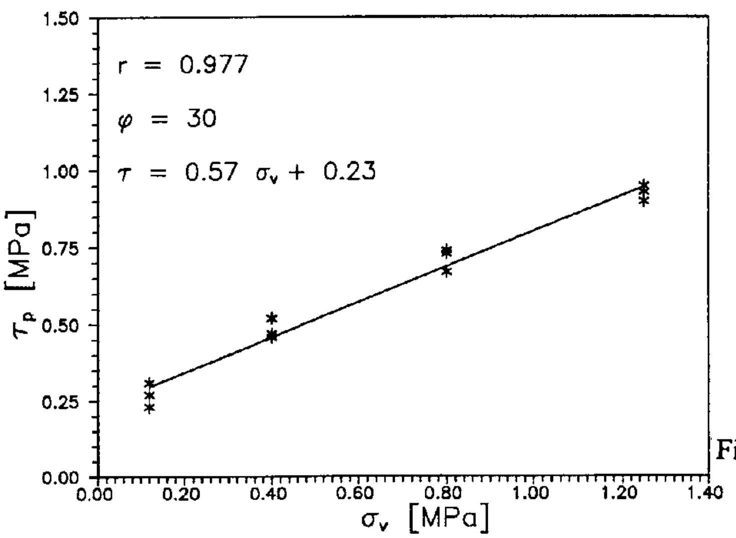


Fig. 19 - Interpolation of peak shear stress and strain

Compressive tests on masonry prisms. The tests were carried on partially following the prEN [8]. A servocontrolled MTS of 250 tons was used and the test was carried on under displacement control in order to follow the post peak behaviour of the masonry prisms; vertical and horizontal displacements were measured following the scheme of Fig. 20. In Table 11 the strength, ultimate strain and secant modulus of the tested prisms are presented. Fig. 21a and 21b show the stress-strain curves where both horizontal and vertical strains are reported. More details on these tests are given in Annex 3 of [1].

Comments on the results and future development of the research.

Some interesting results for the characterization of the full-scale model materials have been achieved, having measured some of the most important parameters for modeling.The tests will be repeated in order to reach a statistically representative sample size. The interpretation of the results should be discussed with the researchers implementing mathematical models. Some significant results can nevertheless be anticipated: Fig 22 shows the variation of the compressive strength calculated on whole units, cubes and prisms; these can be three choises for the parameter strngth for the model input. Fig. 23 shows the stress-strain plots for brick, mortar and masonry prisms, useful for eventual empirical formulae expressing the correlation between masonry and components.

Very interesting results were also achieved in the case of shear bond strength; they will be compared in the future to the data obtained from shear-box tests on the same materials, which will be carried on at Boulder University.

Several other tests have still to be carried on like biaxial tests for bricks, direct tensile tests on bricks and mortars. Bond wrench tests, three point bending test will be performed in the future to better characterize the materials, compressive tests on stack bond prisms, during which the deformations in mortar joints and in bricks will be measured. Several problems have been solved concerning the difficulty of measuring displacements in small specimens; these difficulties seem to have been overcome.

	f_u [MPa]	$\varepsilon_u 10^{-3}$	Es [MPa] (30-60%)	$\varepsilon l/\varepsilon v$(AB) (30-60%)	$\varepsilon l/\varepsilon v$(CD) (30-60%)	$\varepsilon l/\varepsilon v$(ABCD) (30-60%)
MU6H01	5.75	6.4	1281	-.26	-.24	-.25
MU6H02	5.66	5.4	(2411)	(-.14)	(-.15)	(-.14)
MU6H03	6.06	5.81	1255	-.19	-.10	-.14
MU6H04	6.01	6.22	1389	-.15	-.21	-.18
MU6H05	7.52	5.06	2040	-.14	-.14	-.14
mean value	6.2	5.79	1491	-.18	-.17	-.18
v	12%	10%	25%	29%	37%	29%

Tab. 11 - Mechanical parameters of the masonry prisms

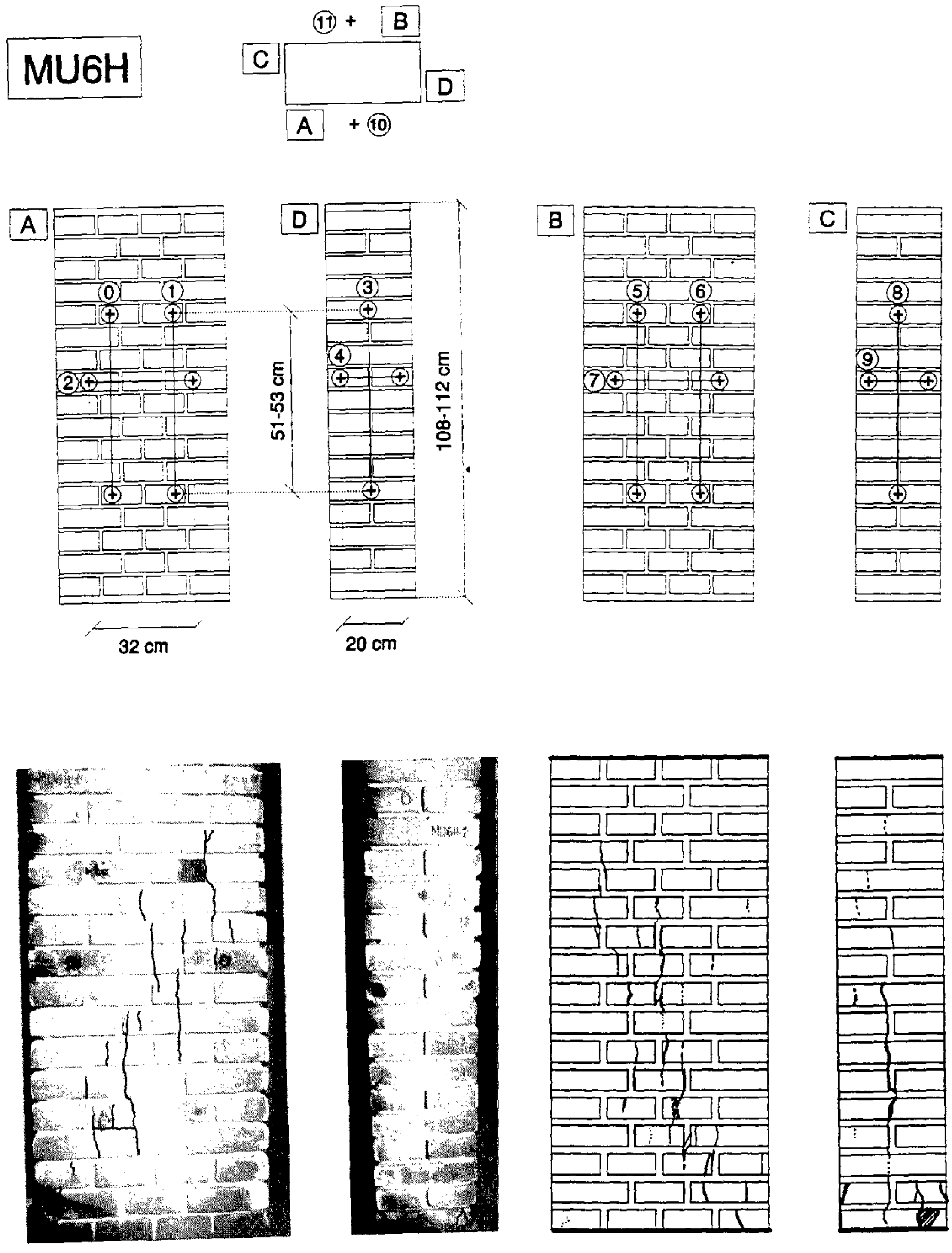
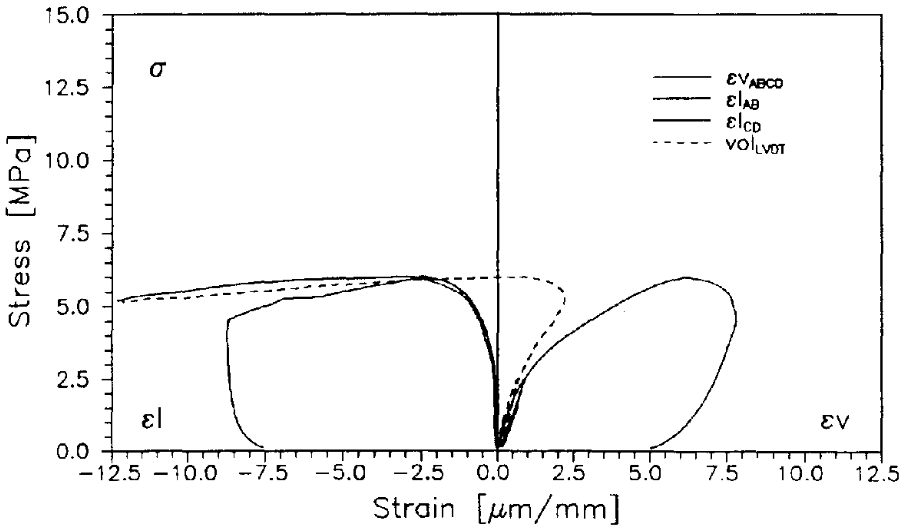


Fig. 20 - Design for displacement measurements on the prisms, and crack pattern



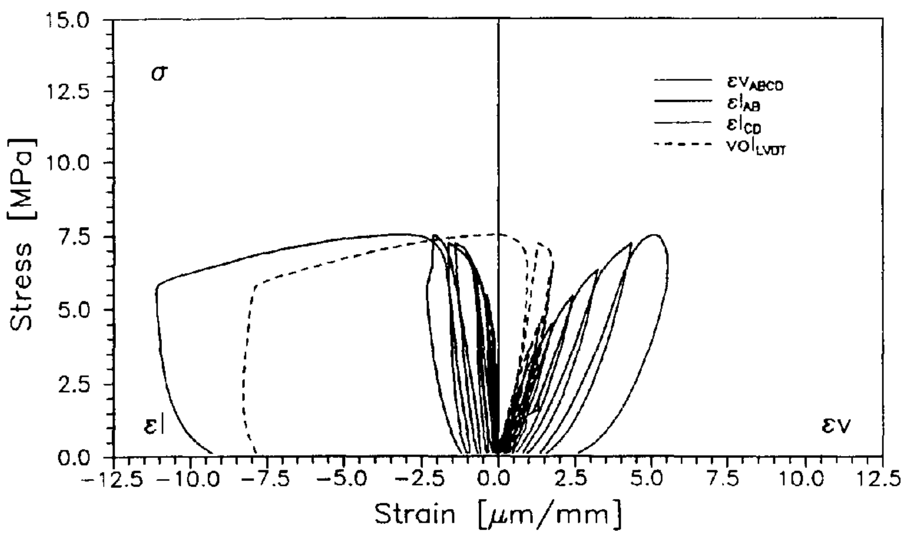


Fig. 21 a,b - Stress-strain curves for prisms MU6H.

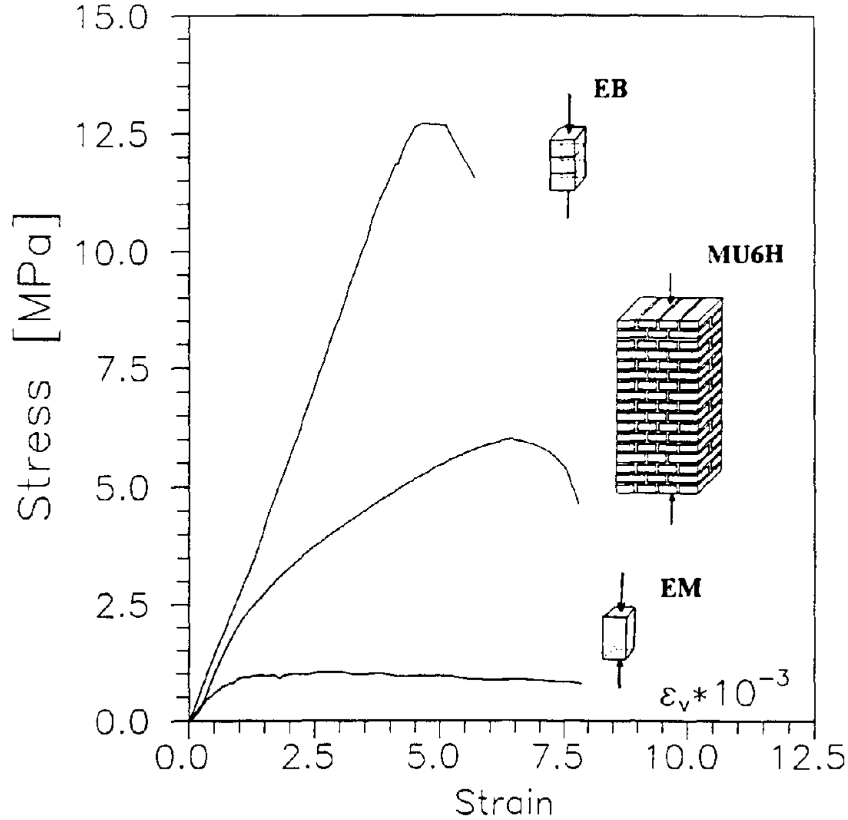


Fig. 22 - Stress-strain plots for mortar, brick and masonry prisms

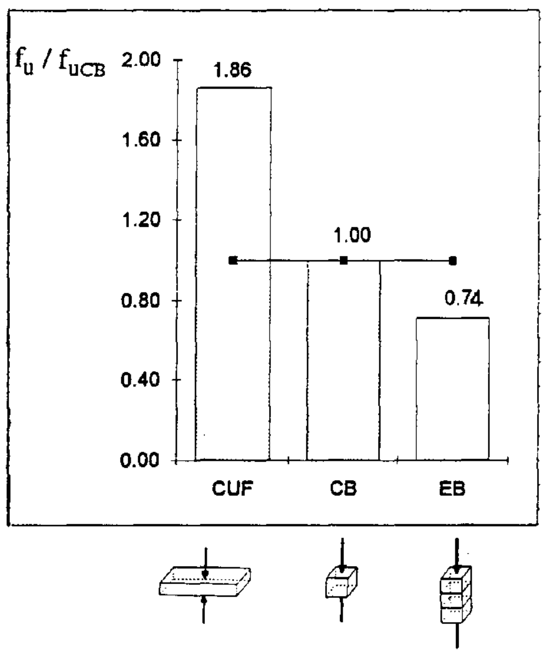


Fig. 23 - Brick specimens: strength ratio cube/whole units, cubes/prisms

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IN-PLACE EVALUATION OF MASONRY MATERIALS

R. H. Atkinson[1]

ABSTRACT

An important element in the seismic retrofit of historic or older masonry buildings is the accurate assessment of the structural condition of the building. Quantitative data is needed for engineering determination of the existing condition and for design of any necessary retrofits. Non-destructive evaluation (NDE) methods adapted from concrete and rock technology or especially developed for masonry have been employed for condition assessment of existing brick masonry. Among these methods are ultrasonic and mechanical pulse tests, single and double flatjack tests and a bed-joint shear test. These NDE methods will be described and critiqued with respect to their application to historic masonry buildings. Examples of applications to historic and older masonry buildings will be given.

INTRODUCTION

A large inventory of unreinforced masonry historic buildings exists, many of which may be structurally marginal or inadequate for their present or proposed use. Recent advances in seismic hazard mapping as well as an increased recognition of the need to provide safety to the public have resulted in more stringent performance requirements in many parts of the country.

A critical first step in any repair or seismic upgrading project is the need to assess the condition of the structure. This assessment is needed to establish the existing condition of the structure so as to first determine the need for repair or strengthening and then to determine the type and extent of structural repair or strengthening required. Means to assess the progress of the repair in real time are required especially when techniques such as grouting are being used. Finally, an assessment of the final structure is required to assure that the repairs have provided the needed upgrade of structural capacity. A quantitative assessment of the final structural condition after any repair or upgrade can also provide a set of baseline measurements which would be used after any subsequent seismic loading to assess the building's condition.

A structural engineer faced with the task of determining the condition of an existing unreinforced masonry building would ideally like to have quantitative knowledge of the strength and deformation

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stiffness properties of the masonry and knowledge of the distribution of these values throughout the building. In an existing building the masonry materials, method of construction and current condition may vary widely over the building's extent due to different stages of construction, environmental deterioration, loadings from past use, past repairs, settlements and seismic events, etc. The structural condition evaluation of an existing building requires a carefully planned program of exploration and testing similar in many respects to that taken by a geotechnical engineer surveying an unknown site.

REVIEW OF IN-PLACE EVALUATION METHODS

Visual

A general building survey should be the fist step in any structural condition assessment program. The visual survey identifies zones of different masonry quality and location and extent of structural flaws. Items to be included in the survey are:

Definition of the geometry of the structure.
Measurement of settlements, tilting and wall movements.
Examination of connections (wall to roof and floors).
Environmental deterioration.
Corrosion of any metallic elements.

The results of the visual survey can be used to identify locations to conduct in-situ tests and to obtain destructive test specimens if allowed

Stress Wave Transmission Tests

Stress wave transmission tests provide a convenient, non-destructive means to assess in a qualitative manner the condition of the masonry. Both high and low frequency tests have been used: the frequency of the transmitted wave determines the sensitivity of flaw location and the expanse of masonry which can be evaluated with one placement of sensors. Most investigators have utilized the wave transmission velocity to characterize masonry quality due to the simplicity of measurement [8,11]. While the velocity can provide a good qualitative measure of masonry quality, experience with ultrasonic wave velocity measurements in concrete as well as masonry shows a poor quantitative correlation between velocity and compressive strength [6,13]. Transformation of the received wave to the frequency domain using Fourier Transform methods has been proposed as a means to obtain more information on the masonry material [6,9]. Working in the frequency domain permits determination of the attenuation of the wave energy as a function of wave frequency which may provide better correlation to the condition (strength) of the masonry structure.

Ultrasonic Pulse Test

The application of the ultrasonic pulse velocity technique adopted from concrete has been a common NDE method applied to masonry. It can be used to identify individual crack and flaw locations as well as develop a map of wall velocities useful for comparative evaluation purposes. Figure 1 shows the rapid attenuation in the amplitude of a 54 kHz input signal in a masonry wall simulating historic construction built using molded clay units and a 1:6:18 (C:L:S) mortar giving a prism compressive strength of 705 psi [6]. Previous work [6,7,8,11] point to several conclusions on the use of ultrasonic velocity. First, a generalized relation between velocity and strength is not possible. Second, that ultrasonic waves can be useful in determining crack and flaw location, especially in modern masonry. And finally that parameters other than velocity need to be studied.

A series of ultrasonic velocity measurements can be used as the input data for a tomographic reconstruction program as discussed below.

Mechanical Pulse Test

Mechanical pulse tests are similar in concept to ultrasonic tests except that the input pulse has a much lower frequency and a higher amplitude which results in increased travel distance compared to a typical ultrasonic pulse [10,12]. The pulse is generated by a hammer blow or by use of a calibrated pendulum and is recorded some distance away by an accelerometer mounted on the structure, Figure 2.

A series of mechanical pulse velocity tests is often used to develop a contour map of pulse velocity over an area of interest. A single low reading may indicate a distinct void while a zone of low readings may indicate a region of low strength masonry or a region containing a system of distributed flaws (damaged zone). Interpretation of results is complicated if the wall is composed of good quality exterior masonry surrounding interior wythes of poor quality masonry since the wave travel path will favor the higher quality exterior wythes giving an incorrect interpretation of overall wall quality.

In-Situ Tests

In situ tests are semi-destructive tests to provide quantitative measures of engineering properties of masonry in units which are directly applicable for use in calculations necessary to evaluate the stability and strength of a structure under a given set of loads. They are termed semi-destructive because it is necessary to remove a small length of mortar bedjoint or a single unit in order to insert the loading devices. If care is taken in conducting the test so as to avoid inducing new cracks into the wall the damage associated with these tests can be easily repaired by replacing any removed units and by replacing or repointing mortar joints.

By conducting the tests in situ, problems from specimen disturbance associated with cutting and transportation to a laboratory for testing are avoided. Additionally, the severe disturbance to the appearance of the structure associated with removal of destructive specimens is also avoided.

Because in situ tests can be labor intensive a preliminary survey should be used to locate sites for these tests. Sites should be selected to be representative of the major categories of masonry quality determined by visual means or by using wave velocity methods. In addition, site selection may be influenced by accessibility and architectural considerations.

In-Situ Stress Measurement

The use of flatjacks to determine the in situ stress state was adapted from the field of rock mechanics by Rossi [14] who has applied this technique to many historic masonry structures in Italy.

Evaluation of the magnitude of in situ compressive stress in the vertical direction is a simple process of stress-relief induced by the removal of a mortar bedjoint, followed by restoration of the original geometry by pressurizing a flatjack inserted into the empty bedjoint. A flatjack is a thin steel envelope that is pressurized with a fluid to apply a uniform stress over the projected area of the flatjack. By using a Whittemore gage or other deformation measurement device a baseline distance is established on either side of the slot to be cut for the stress-relief. The pressure in the flatjack, with suitable calibration factors, at which the original dimension is reestablished is taken as the vertical stress in the wall at that point. This test is illustrated in Figure 3 and has been adopted as an ASTM Standard Test Method, C-1196 [3].

Care in the interpretation of in situ stress data is required as the method measures stress only the outer layer of masonry. Since many older masonry walls consist of good quality exterior wythes surrounding poorer quality inner wythes, assuming a uniform stress field through the wall based on stress measurements in the outer wythe may erroneous.

In Situ Deformability Measurement

The deformation properties of masonry may be directly evaluated by inserting two parallel flatjacks, one directly above the other, separated by several courses of masonry, and pressurizing them equally, thus imposing a compressive load on the intervening masonry. The deformation of the masonry between the flatjacks is then measured at several increments of flatjack pressure and used to construct the stress-strain curve of the masonry, Figure 4. This test provides a direct measure of masonry deformability which is needed as input for stress analysis or deflection calculations. This test has been adopted as an ASTM Test Standard, C-1197 [4].

If the pressure in the flatjacks is increased until the stress-strain curve begins to flatten, as illustrated in Figure 4, a lower bound estimate of masonry compressive strength can be obtained. To obtain this type of information, however, requires pressure levels which will induce considerable cracking between and around the flatjacks which may be unacceptable for some structures.

Deformability and strength information obtained from this test has the same limitation affecting the in-situ stress test since only the outer masonry wythe is loaded.

In-Place Shear (Shove) Test

The in-place shear test was designed to measure the in situ bedjoint shear strength of masonry walls [1]; a parameter required for seismic hazard analysis. The test requires the removal of a masonry unit and a head joint on one or both sides of the test unit, Figure 5. The test unit is then displaced horizontally relative to the surrounding masonry using a hydraulic jack. The horizontal force required to cause first movement of the test unit is recorded and used in a procedure to estimate the bedjoint shear resistance of the masonry [17].

A modification of the original procedure employs a small flatjack inserted into the headjoint space next to the test unit to apply the horizontal force to the unit, Figure 6. This procedure eliminates stress concentration effects on the test unit resulting from the removal of the adjacent unit to accomodate a hydraulic jack.

Bond Wrench Test

Out-of-plane flexural loads can result from seismic loading, from wind loads or from differential building movements. The flexural tensile bond strength can be determined in-situ using a modification of the laboratory bond wrench test method contained in ASTM C 1072 [2]. The test involves removing two adjacent units from the wall at the test location, Figure 7. The head joints of the two units thus exposed are removed as well as any mortar in the collar joint of a multiple wythe wall. A clamp is then attached to the test unit and a calibrated recording torque wrench is attached to the clamp, Figure 8 . Manual pressure is applied to the torque until failure occurs on the bedjoint under test. A simple calculation, accounting for the direction of the torque, the dead weight of the testing apparatus and the geometry of the test, is made to determine the flexural tensile bond strength at failure.

STATISTICS

Adequate characterization of masonry material properties requires that a suitable number of tests be conducted such that the aggregate of the test data collected provides an acceptable determination of the property in question. The number of tests conducted will be influenced by the size of the masonry structure under evaluation, the expected variation of properties based on visual examination, the desired accuracy with which the property value should be known and the inherent variability of the test being used. Other factors such as access to the building, allowable disturbance to the building appearance and cost can also be important factors.

The estimated coefficient of variation (COV) for a number of in-situ or NDE tests are given in Table X [16]. Also given for comparison is the COV value for prism test both on laboratory built and field cut prisms.

Table X
Estimated Coefficient of Variation of Masonry Test Methods

Technique	Lab Testing	Field(Testing and Materials)
Prism Test	10%	20%
Flat Jack Test	10%	18%
In-Place Shear Test (Shove Test)	23%	30%
Bond Wrench	30%	36%
Ultrasonic Pulse Velocity		
Direct	8%	20%
Indirect	12%	30%

TOMOGRAPHY

Ultrasonic pulse travel time data may be used in tomographic reconstruction programs to define a map of pulse velocities on a two-dimensional slice. The internal distribution of pulse velocities is taken as giving an indication of the masonry quality. The slice may be taken through the wall or may lie with-in the plane of the wall. The technique requires that a large number of travel time readings be obtained. Ideally the readings should be made on all four sides of the area to be evaluated. In practical applications often only three or sometimes two sides of the area are available for making ultrasonic measurements. In these cases zones near a boundary not having any measurement points will have poor definition.

Medical applications of tomography using X-rays as the information source are wide spread. Acoustic tomography was originally developed as a geophysical tool to characterize rock conditions over a large area. In contrast to X-rays, which travel in straight lines through a body, acoustic waves will bend, reflect and refract depending on the value of the acoustic impedance's of adjacent materials. As a result the reconstruction algorithms required for acoustic tomography for applications to civil structures of masonry or concrete are complex [5].

Acoustic tomography has been applied for the evaluation of injection grouting efficiency as a repair technique for unreinforced masonry piers and walls [15]. The first ultrasonic travel time measurement survey is taken with the wall or pier in the damaged, unrepaired state. With extensively damaged walls obtaining meaningful travel time data may be difficult due to the presence of extensive cracks. The tomographic reconstruction from the initial survey serves to locate the position and magnitude of damaged zones within the masonry section .After the wall has been repaired by injection grouting a duplicate travel time survey is conducted using the same pattern of transducer locations as used on the first survey. A comparision of the two plots of velocities maps provides an indication of the extent of grout filling of voids and cracks. Figure 9 shows velocity plots from an unreinforced masonry pier before and after repair by injection grouting [15]

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list of figures

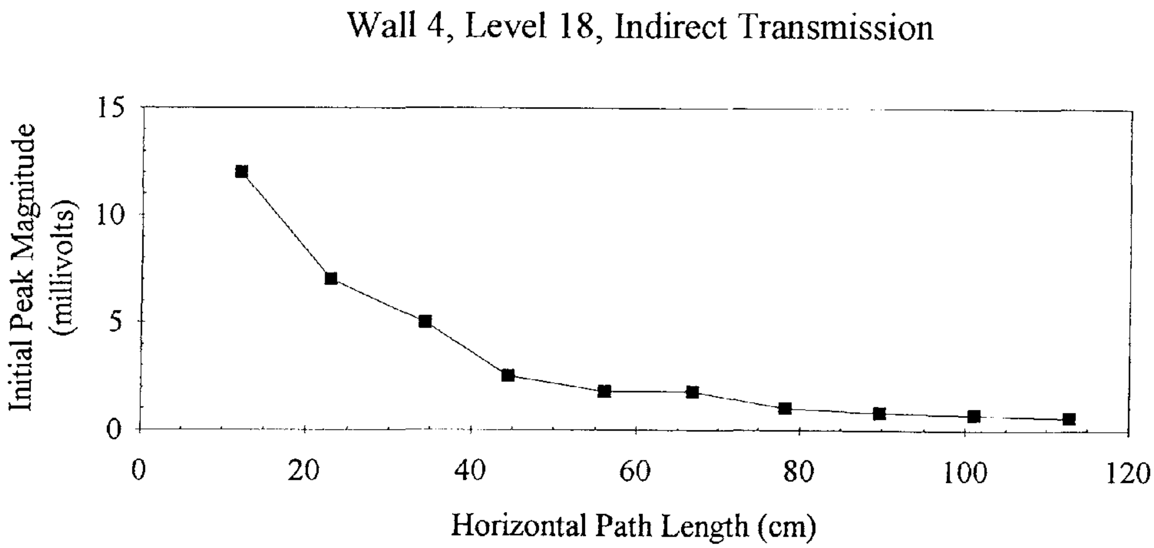

Figure 1
Attenuation of Ultrasonic Wave Amplitude

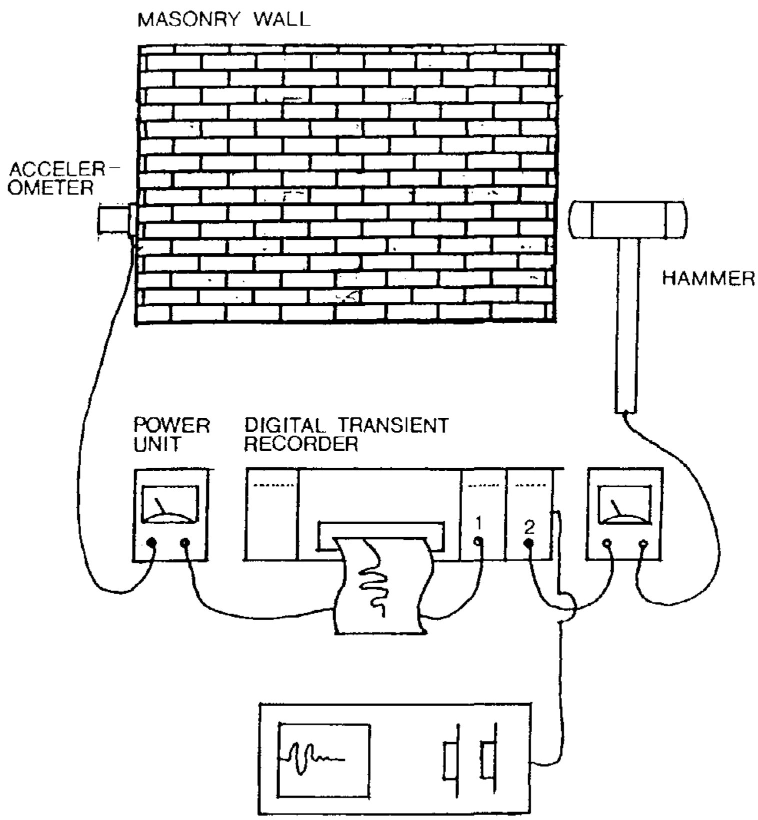

Figure 2
Mechanical Pulse Testing Equipment

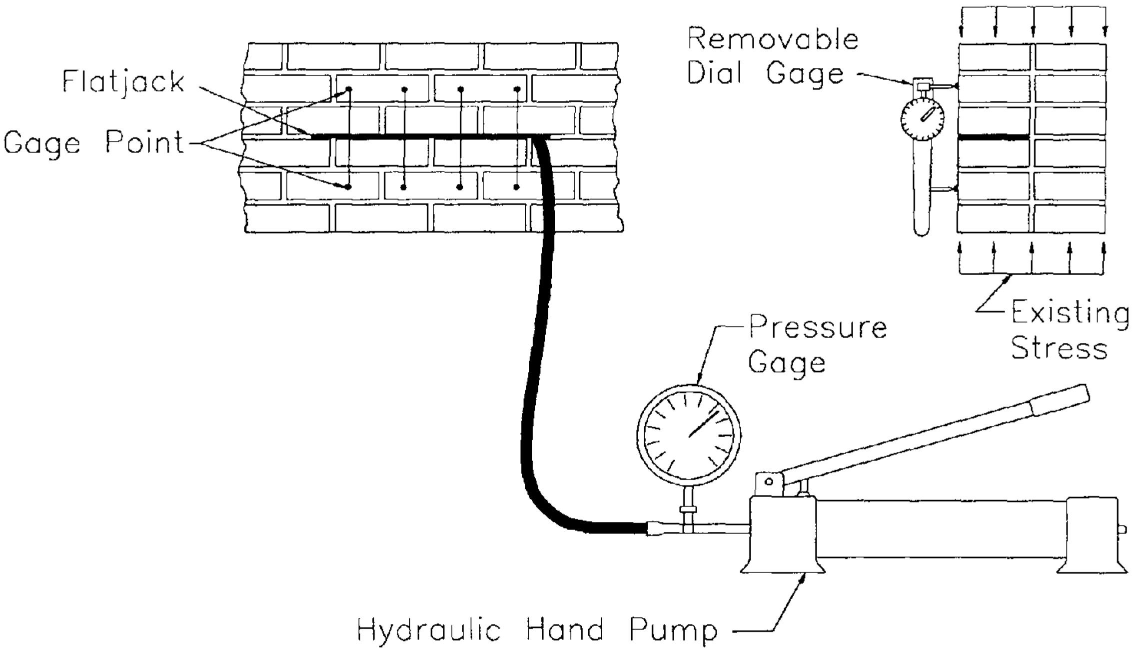

Figure 3
Determination of In-Situ Vertical Stress

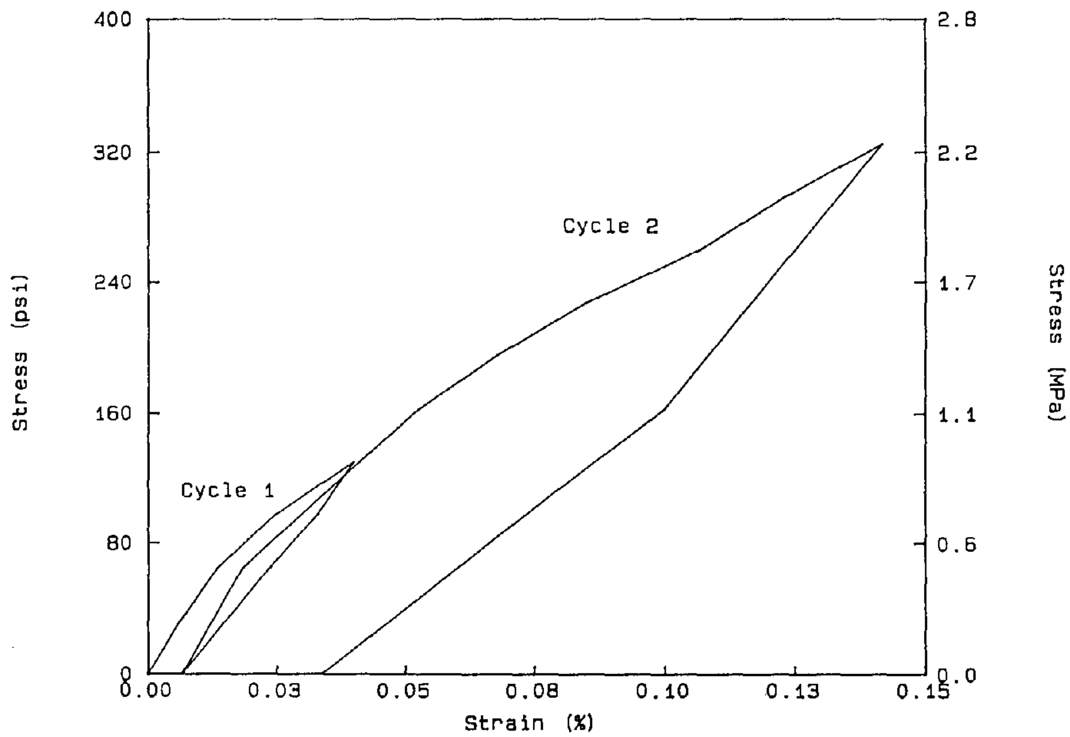

Figure 4
Stress-Strain Curve of Masonry Determined by Double Flatjack Test

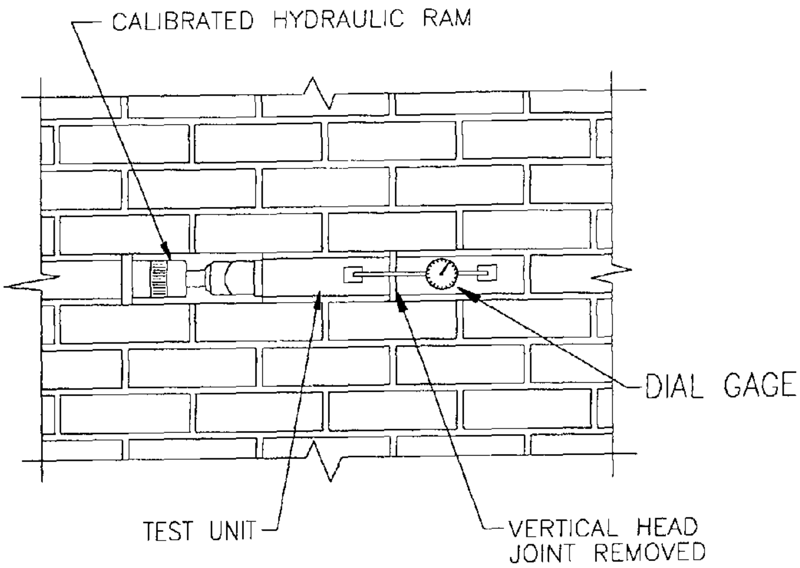

Figure 5
In-Place Bedjoint Shear Test Setup

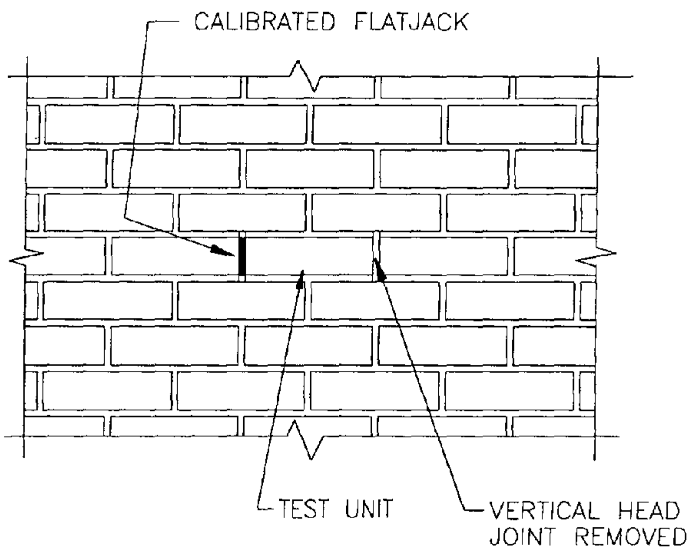

Figure 6
In-Place Bedjoint Shear Test Using Headjoint Flatjack

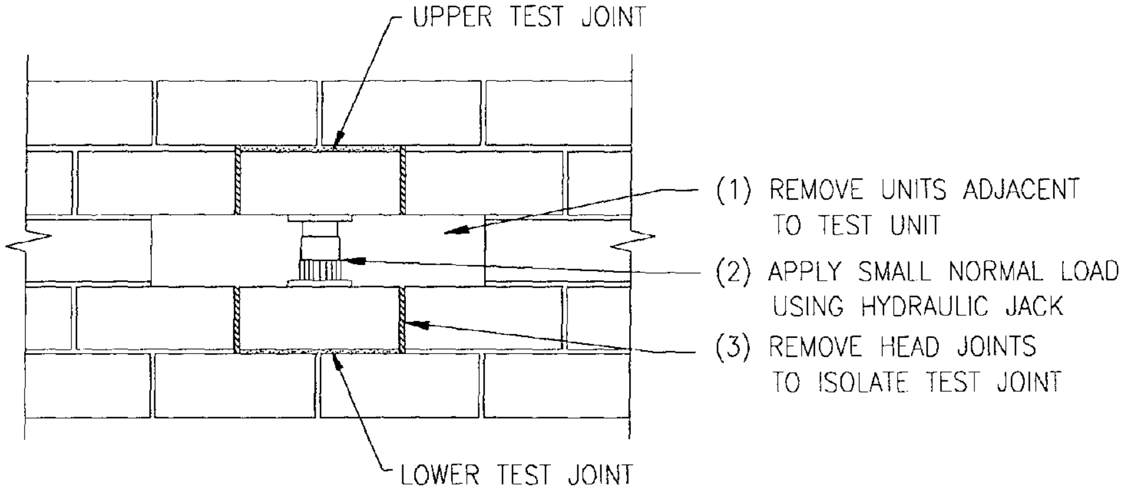

Figure 7
Test Arrangement for In-Place Bond Wrench Test

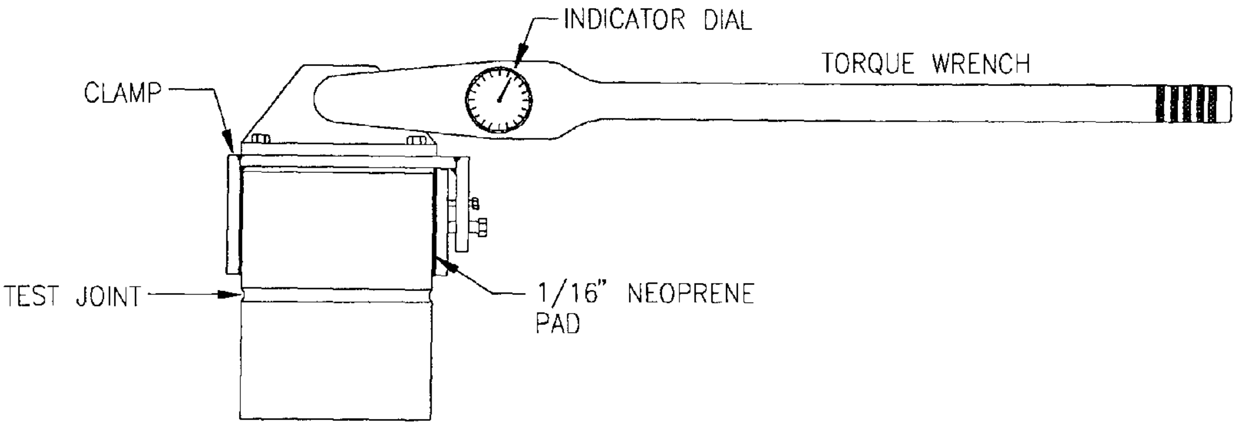

Figure 8
In-Place Bond Wrench Test Equipment

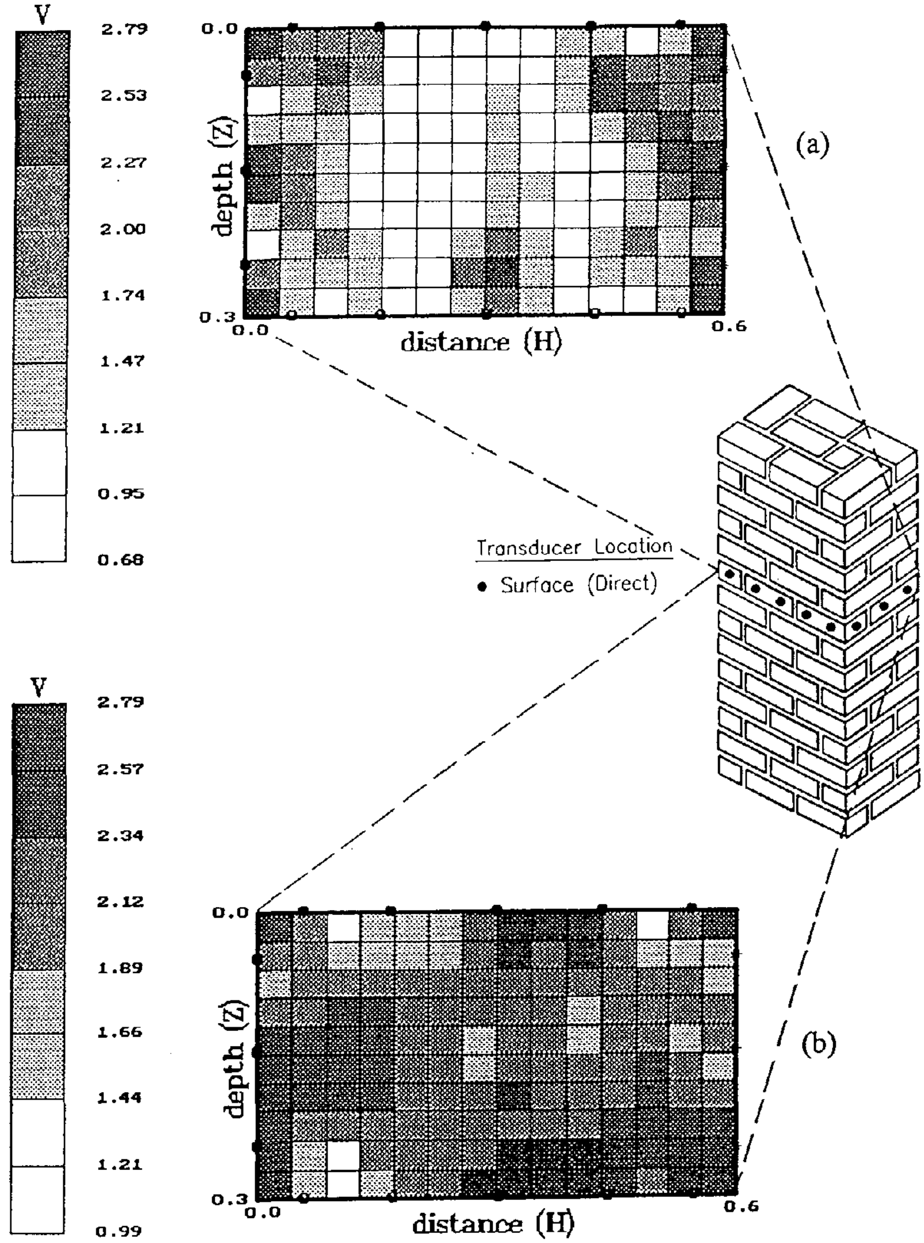

Figure 9
Tomographic Reconstruction of the Velocity Distribution in One Course of a Masonry Pier.
Dimensions are in Meters; Vertical Velocity Scale Indicates Meters/Millisecond.
(a) As-Built Condition, with Voids in the Collar Joint and Interior Wythe
(b) Following Repair by Grout Injection

DEVELOPMENT AND USE OF A MOBILE LABORATORY FOR THE ASSESSMENT OF URM BUILDINGS

Mauro Cadei[1] , Paolo Panzeri[2] , Alberto Peano[3] , Paolo Salvaneschi[4]

ABSTRACT

A mobile laboratory, provided with a knowledge based system, is presented whose objectives are to support the procedures leading to the seismic assessment of masonry buildings and the planning of precautionary operations on them. The architecture of the laboratory and its functions are described with emphasis on the main parts of it: the hardware part, consisting of a vehicle, a special container, experimental equipment and electronic devices; the software part with a knowledge-based system, in particular, implementing models of the structure and possible seismic behaviours of buildings. The testing of one model is also presented through the comparison of its results with the seismic behaviour of large scale masonry building prototypes tested on a shaking table.

INTRODUCTION: THE MOBILE LABORATORY

The importance of retrofitting existing buildings in order to obtain a uniform level of safety in case of seismic events has been widely recognised as a major problem in civil engineering. The procedures required to establish a diagnosis and to suggest a therapy for a single building or for classes of buildings are complex and heterogeneous. They require both theoretical knowledge and practical experience because a building can be examined on the basis of direct observations, *in situ* or laboratory tests and numerical analysis.

This strongly suggests the need for developing a facility to support this cognitive process in data acquisition, storage and representation, in modelling, in simulation and evaluation, in decisions about retrofitting and in planning of activities.

A mobile laboratory supporting the seismic assessment and planning of retrofitting for masonry buildings has been designed and implemented in ISMES. It is equipped with devices allowing experimental tests to be performed for the acquisition of data related to the physical properties of the materials and to the structural features of the buildings. It is also provided with software systems for data acquisition and management and, in particular, with a knowledge-based system for supporting the evaluation and planning process.

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The whole research has been initially oriented towards masonry buildings, representing a large part of the historical heritage in most European countries. It has however produced a framework suitable to be extended to other different structural types.

STRATEGIES FOR USING THE MOBILE LABORATORY

In performing surveys for seismic assessment of buildings, three deepening levels can be identified. A first level allows the large scale localisation of the higher priority areas of a town or a city. A second level increments the knowledge needed about the structures and allows a more detailed evaluation. It is applied to smaller areas, localised in general by the first level, identifying higher priority buildings. A third level allows deeper detail in the seismic assessment of single buildings and their structural components.

The requirements in term of input data increases from the first to the third level as the first and second level require essentially visual inspection while experimental tests are necessary at the third level.

The general strategy driving the evaluation process should consist of applying the evaluation at one level generating results and limiting the target for the more detailed but more expensive levels.

THE ARCHITECTURE OF THE MOBILE LABORATORY

The main requirement for the design of the mobile laboratory was the integration of a knowledge-based decision support system with the test equipment generating input data. The mobile laboratory consists therefore of a vehicle provided with a special container divided into two parts:

- a laboratory allowing the execution of experimental tests on the materials and structural components (in situ and in the laboratory) and supplied with all the related equipment;

- an office provided with electronic devices such as data acquisition systems and a workstation, running the software systems, with related peripherals.

THE DATA ACQUISITION AND MANAGEMENT SYSTEM

The data acquisition and management system supports both static tests and dynamic tests (such as vibration test on floors or analysis of chains) for the mobile laboratory. It includes a set of tools for reading, processing and presentation of signals and it also offers data base facilities.

Processing functions include statistics, analysis in the time and frequency domains, fitting, smoothing and filtering.

THE KNOWLEDGE-BASED DECISION SUPPORT SYSTEM

From a technological point of view, the IGOR decision support system is a hybrid system, mixing knowledge-based systems technology and conventional technology (procedural languages, data bases management techniques and man/machine interfaces). It is built on three main layers:

- *models* of the physical world, collecting knowledge related to a single building or to all the buildings in a village (or a region);
- *functions* on the models, implementing possible operations related to them;
- *man-machine interface*, allowing the interaction between the system and the human expert at a proper conceptual level.

The relations between the models and the functions are:

- a model of a single building and its environment and a model of a set of buildings (e.g., a village) are related by a *load/save function* which can move data between them;
- an *input function* allows the input of values for the attributes of the models and modifying them when more data are available from new observations, measurements and experimental tests;
- an *output function* generates reports and graphical representations of the models;
- a *generalisation function* supports generalisation of some attribute value of a specific building over classes of homogeneous buildings;
- a *simulation function* allows the simulation of a seismic event on a single building model generating an expected damage on it;
- an *evaluation function* produces a ranking of the expected damage;
- an *explanation function* produces an explanation of the expected damage processes with reference to the structural features of the building responsible for them;
- a *retrofitting planner*, after simulation and evaluation, supports the choice of the best intervention techniques;
- a *planner* supports decision making on the use of different models, data acquisition, evaluation and generalisation of results.

Object oriented modelling is the base technique used in designing the software system implemented in C++ language and the shell Nexpert Object on a SUN workstation.

State of the art in modelling masonry buildings

From the modelling point of view, the development of finite-element methods have allowed the analytical solution of complex civil engineering structures. Unfortunately, this is easily obtained only if a linear elastic behaviour of the structural elements is assumed. Moreover, computer programs which can deal with simple non-linear behaviours, as needed in seismic modelling, do exist, but the difficulties in getting the correct constitutive parameters are great. The results are therefore less reliable, more specialised users are required and, at the same time, the needed computer resources and the possible numerical difficulties significantly increase.

Masonry buildings are particularly affected by such situation, since the description of their geometry is less complex than the constitutive equations of the structural elements and of their interactions. The modelling of masonry buildings through finite-element analysis is therefore very complex or totally unsatisfactory.

The necessity of evaluating the seismic performance of large numbers of existing masonry buildings has therefore given a strong impulse to the development of associational/empirical expert systems. These, however, only represent a part of the knowledge needed to solve this kind of problems. In fact, engineering reasoning takes advantage of both associational/empirical knowledge and knowledge coming from domain theories, expressed using numerical procedures. Moreover, the engineer's evaluation is based on two key elements:

- *hierarchical reasoning*: reasoning with hierarchical models of systems;

- *causal mechanisms*: empirical knowledge and domain theories and both qualitative and quantitative information are used in the context of causal mechanisms modelling the system behaviour.

These different types of knowledge have to be integrated (3).

The modelling technique

The formalisation of the models for a single building required the development of a specific modelling technique.

A model of a physical system (a building) collects the knowledge related to:

- *structure*: describing its subsystems and relations (*is-connected-to* and *is-part-of* relations) between them;

- *attributes*: describing physical quantities and their values;

- *behaviours*: describing modifications of the structure and the values of attributes in time.

Each type of knowledge can be organised in a hierarchy, with different levels of abstraction, and a combination of one level from each hierarchy is a model (*simple model*) of the physical system. An *extended model* is a set of (more then one of) these simple models with dependencies among the values of attributes related to them. In it, the modelled system is simultaneously represented at different levels of abstraction allowing to deal with incomplete knowledge at one abstraction level.

In our modelling formalism, the structure is modelled by instances of subclasses derived from three classes (in the object oriented modelling sense):

- *elements*, representing subsystems (e.g., walls);

- *connections*, representing physical connections between subsystems;

- *interfaces*, representing contact parts, having their own attributes and behaviours, between subsystems (e.g., wall-floor interfaces).

Both elements and interfaces belong to the *components* super class and have attributes and behaviours. *Stimuli* (e.g., a force) are a class of attributes which, at a definite time, can be marked by a *token* (a value is present) or not (no value), while *properties* (e.g., a friction coefficient) are attributes always having a value. It is possible to express dependencies between the values of different attributes, even belonging to different components and different simple models in the same extended model, so that, changing one of them, the other is immediately modified too. That allows the description of physical laws and the representation of quantities at different levels of abstraction.

Behaviours are modelled so that each component expresses one or more *processes* (e.g., a process modelling the shear sliding of a wall) which can be activated by stimuli and influenced by properties. Each process is characterised by:

- a *precondition on the existence of input stimuli*;
- a *precondition on the values of input attributes*;
- a *body*.

When the preconditions are true (some cause is present and some threshold is overcome), the process can start, removing the tokens from input stimuli and executing the body. This is a computation, expressed, in general, by a mix of associational/empirical rules, procedures and calls to external programs, generating new stimuli and values of properties. In such a way, associational/empirical and procedural knowledge can be mixed and quantitative and qualitative computations can be mixed as well.

A process can be graphically represented as a rectangle, while stimuli and properties are circles connected to them by oriented arcs representing input and output relations. The result of linking different processes is a net embedded in the components. Starting from a set of initial stimuli, the net can be run simulating the system behaviour (from an earthquake to a damaged building).

The models of a single building

Three models of a single building, at different levels of abstraction, have been created. The first and second models respectively correspond, as concerns input data, to the first and second vulnerability assessment forms by the Italian Gruppo Nazionale per la Difesa dai Terremoti (GNDT) (1).

In *model 1*, the building is modelled as one element and the description of both the seismic action and the building attributes are totally qualitative. The seismic input is defined by the seismic zone coefficient while the properties used to characterise the building response are the date of construction (in relation with the date of enforcing a seismic design for the site of the building), the materials and the construction type, the regularity of the geometry, and the existence of damage from previous earthquakes.

The expected damage is qualitative (*low damage, medium damage, high damage*) and evaluated on the basis of associational/empirical rules.

In *model 2*, the seismic input is defined by a peak acceleration obtained from catalogue data and the building is conceived as one element: a box-like behaviour is assumed and the total shear resistance is compared with the expected horizontal force.

The ability of the building to act as a rigid body is also checked: any structural situation which is not likely to allow a rigid body motion is considered to be a possible cause of unreliability of the model.

Model 3 is an extended model so that the building is simultaneously modelled at three abstraction levels. It provides a very detailed description of the non-linear mechanical behaviour of the structural elements and the connections between them. That allows the analysis of damage processes inside single elements and the generation of a diagnosis in term of structural features and physical properties of the materials responsible for them.

At the first level, the building is modelled as one element and the associated behaviour assumes a one-degree-of-freedom model.

At the second level the building is modelled as composed by macro elements (one component for the foundation, each wall, floor and the related interfaces).

At the third level each wall is described as composed by end-walls (between floors) (see 6 for more details).

The processes at level 2 and 3 model the transmission of motion and energy through the components and phenomena as amplification or attenuation of motion and energy dissipation (from the point of view of the excitation or acting stimulus) and damage to structural elements (equivalent to energy dissipation from the point of view of the building). Examples of damage processes are, related to the in-plane behaviour of a wall, flexural cracking, shear cracking, shear sliding and rocking.

The simulation is started by putting values into the earthquake attributes of the *geological structure* and these stimuli are propagated through the first abstraction level. The resulting effects related to the *building* are inherited, through attributes dependencies, by the building components (level 2) and the simulation goes on.

When some damage process is activated, according to the time ontology (event to event search), the simulation is stopped and the resulting properties in the lower levels are summarised, through dependencies, to the upper levels. For example, when a damage process is activated in a *wall*, the stiffness property of the *building* is changed too.

The simulation is run several times (each time starting from the previously generated status) and the end is reached when a steady state motion is reached, either because of the low level of the forces developed by the input acceleration or because of convergence between the input and the dissipated energy (see 6 for more details).The *event to event* search method has been applied. In this time ontology, when a simulation on the model is started, no time interval elapses until a significant event (a damage) is reached. The *first* event is not necessarily the first to happen in time, but rather the one which requires the lowest *load multiplier* (in a sense, it is the most probable).

EXPERIMENTAL EQUIPMENT AND TESTS

The key issues in selecting the experimental tests provided by the laboratory were:

- to generate all the input data required by the seismic assessment software, particularly model 3;
- to ensure reliable and really interesting information;
- to provide quick and low cost tests and analysis;
- to limit weight and volume of related devices;
- to limit electric power required;
- to allow execution of the tests on site, particularly on a building in an old urban nucleus.

Table 1 shows the list of the experimental tests provided by the mobile laboratory and their relation to the physical parameters directly or indirectly estimated.

Some data are generated by more than one test allowing different strategies to be employed in different cases depending on the structural features of the building and the operating conditions.

Table 1. Experimental tests provided by the mobile laboratory.

TEST	PHYSICAL PARAMETERS / STRUCTURAL FEATURES
flat-jack test	state of existing stress, longitudinal elastic modulus, Poisson ratio, compressive strength
shear test along the mortar layers	shear strength of mortar-masonry joint, friction angle, masonry cohesion
centred compression on a panel	compressive strength of masonry, longitudinal elastic modulus, Poisson ratio
diagonal compression on a panel	tensile strength of masonry, shear modulus
hammer test on the mortar layers	compression strength of mortar layers
Bond-Wrench test	flexural tensile strength
compression of a brick (or core)	strength in compression of bricks (or cores)
point load test	tensile strength of bricks or mortar layers
vibration test on floors	in plane floor rigidity, floor-walls connection effectiveness
static load test on floors	floor-walls connection effectiveness
tie rods analysis	effectiveness of tie rod tension
sonic test	presence of anomalies, areas of deterioration, voids, crumbling
thermography	test of the internal structure (arches, beams, hidden openings, chimneys...)
endoscopy	internal structural characteristics, voids and fissures in the masonry
removal of panels from the site then shear-compression test in a laboratory	shear strength of masonry, tangential elastic modulus

Sometimes, nevertheless, redundant tests will be needed in order to ensure maximum reliability of data.

TESTING OF MODEL 3 OF THE IGOR SUPPORT SYSTEM

A testing of model 3 of the IGOR decision support system has been based on the comparison of its results with the experimental behaviour of large scale physical models of unreinforced masonry buildings evaluated by shaking table tests.

Three physical models of brick masonry buildings in 1:1.5 scale, differing as to the floor type, have been tested on a six-degree-of-freedom shaking table in ISMES.

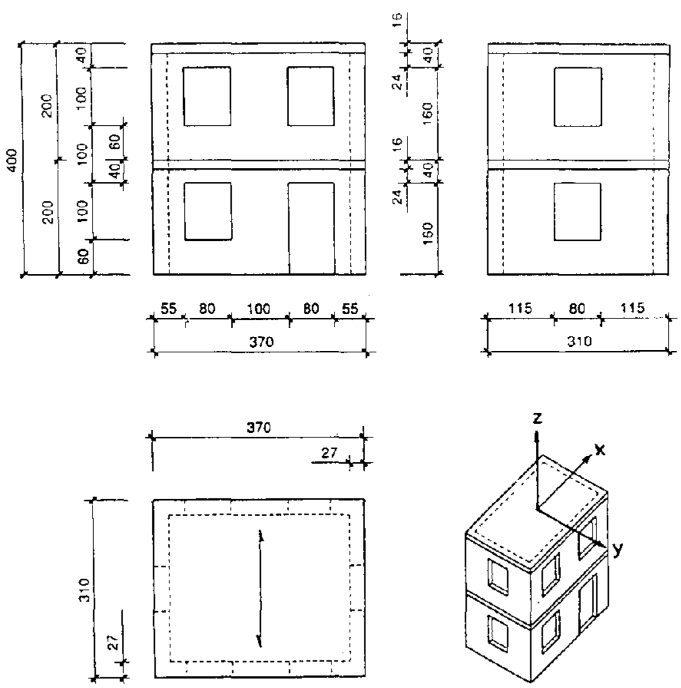

Fig. 1. Main geometrical features of the tested masonry building prototypes (lengths in cm).

Table 2. Mechanical properties of the tested building prototypes.

MECHANICAL PROPERTY	VALUE
Weight density	1.3 t/m^3
Young's modulus	21275 t/m^2
Shear modulus	8510 t/m^2
Tensile strength of the mortar-masonry joint	10 t/m^2
Compressive strength of the masonry	182 t/m^2
Tensile strength of the masonry	82 t/m^2
Shear strength of the mortar-masonry joint	0.9

The main geometrical features of the models are shown in figure 1 while their main mechanical properties, evaluated by laboratory tests, are reported in table 2.

The seismic response of the masonry buildings has been measured by means of 16 accelerometers and 14 transducers of relative displacement (fig. 2).

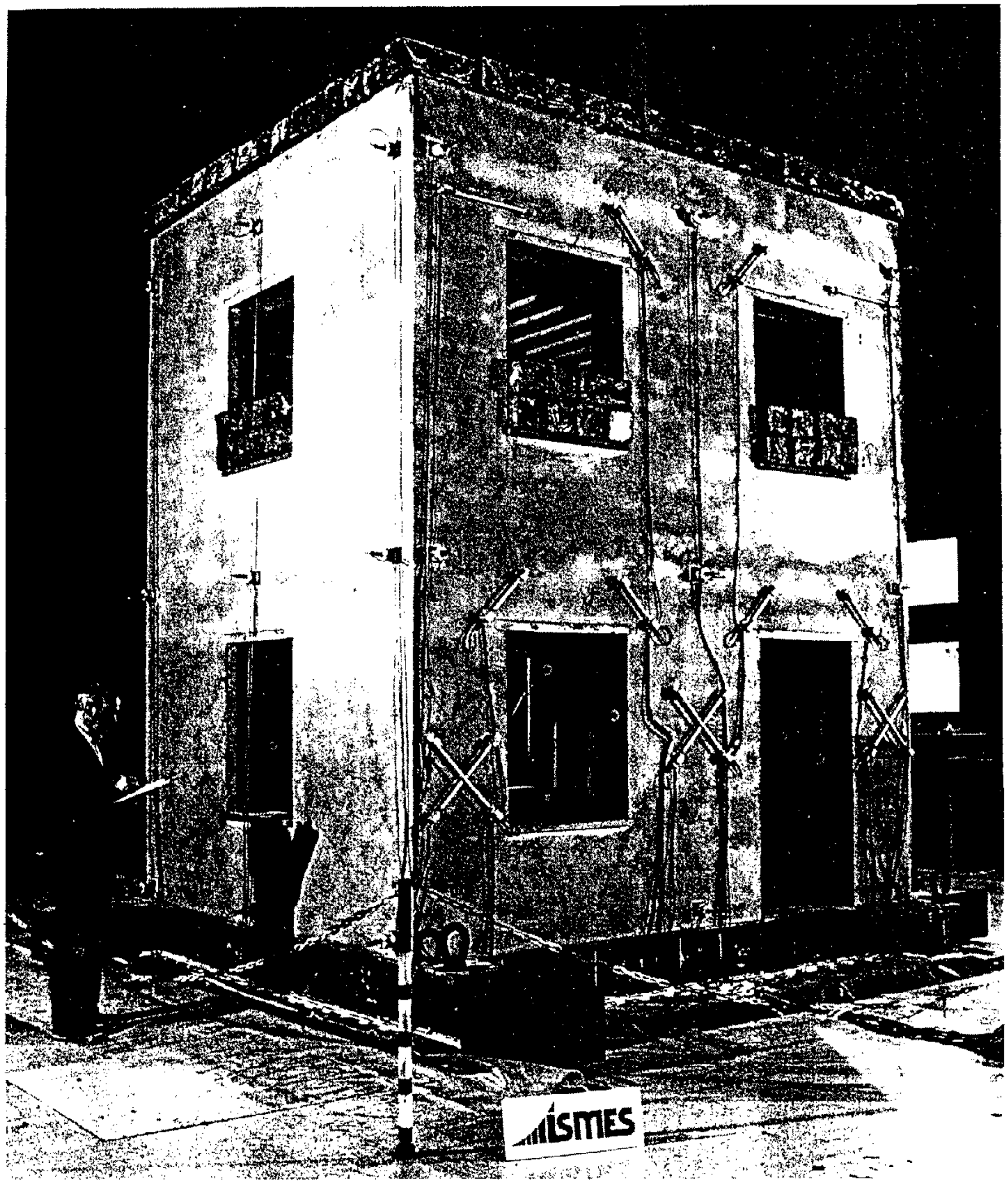

Fig. 2. An instrumented building prototype.

A test sequence was carried out applying the same time history with increasing levels of amplification (from -12 dB up to 0 dB through 3 dB steps). The response spectra and the relevant time history applied are shown in figure 3.

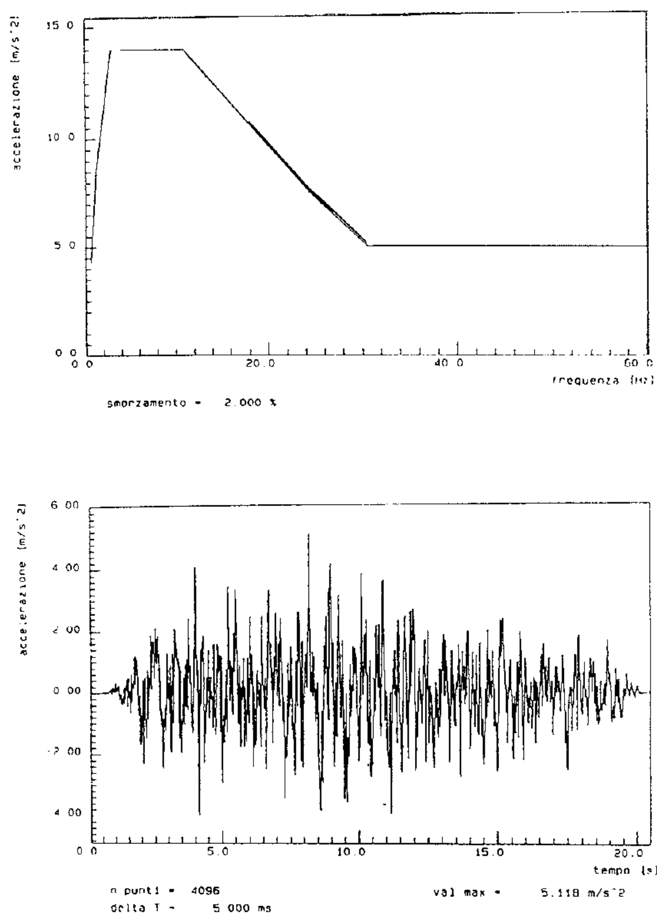

Fig. 3. Response spectra and time history applied to the building prototypes (0 dB).

Table 3 gives, for one of the tested buildings and for every step of the applied test sequence, the acceleration measured and numerically evaluated at the three reference levels of the structure:

- ground level (0 m);
- first floor level (2 m);
- second floor level (4 m).

Figure 4 shows the acceleration diagram of the final test N. 5.

Table 3. Acceleration values.

TEST SEQUENCE STEP	LEVEL [m]	EXPERIMENTAL ACC. [a/g]	SIMULATION ACC. [a/g]
1	4.0	0.21	0.25
	2.0	0.16	0.15
(-12 dB)	0.0	0.09	0.13
2	4.0	0.40	0.37
	2.0	0.20	0.23
(-9 dB)	0.0	0.18	0.18
3	4.0	0.51	0.52
	2.0	0.29	0.32
(-6 dB)	0.0	0.24	0.26
4	4.0	0.54	0.69
	2.0	0.42	0.43
(-3 dB)	0.0	0.37	0.37
5	4.0	0.75	0.97
	2.0	0.56	0.61
(0 dB)	0.0	0.51	0.52

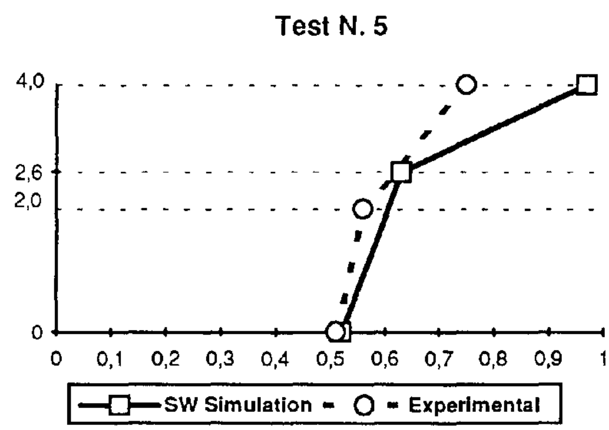

Fig. 4. Acceleration diagram for the test N. 5 (0 dB).

As to the experimentally observed damage, the tests N. 1, 2 and 3 caused only surface localised cracks, while the higher excitation levels tests (N. 4 and 5) produced significant damages consisting of deep longitudinal cracks interesting the whole extension of the walls along the main direction of the seismic load. Figure 5 shows a sketch of the experimentally observed damage related to the final test N. 5 with 0 dB level of amplification.

Consistently with the experimental results, the outcomes of the simulation through the model 3 implemented by the IGOR system did not forecast any damage for the simulation cases corresponding to the tests N. 1, 2 and 3.

The IGOR simulation of the test N. 4 gave origin to significant flexural cracks interesting the two walls along the main direction of the seismic load.

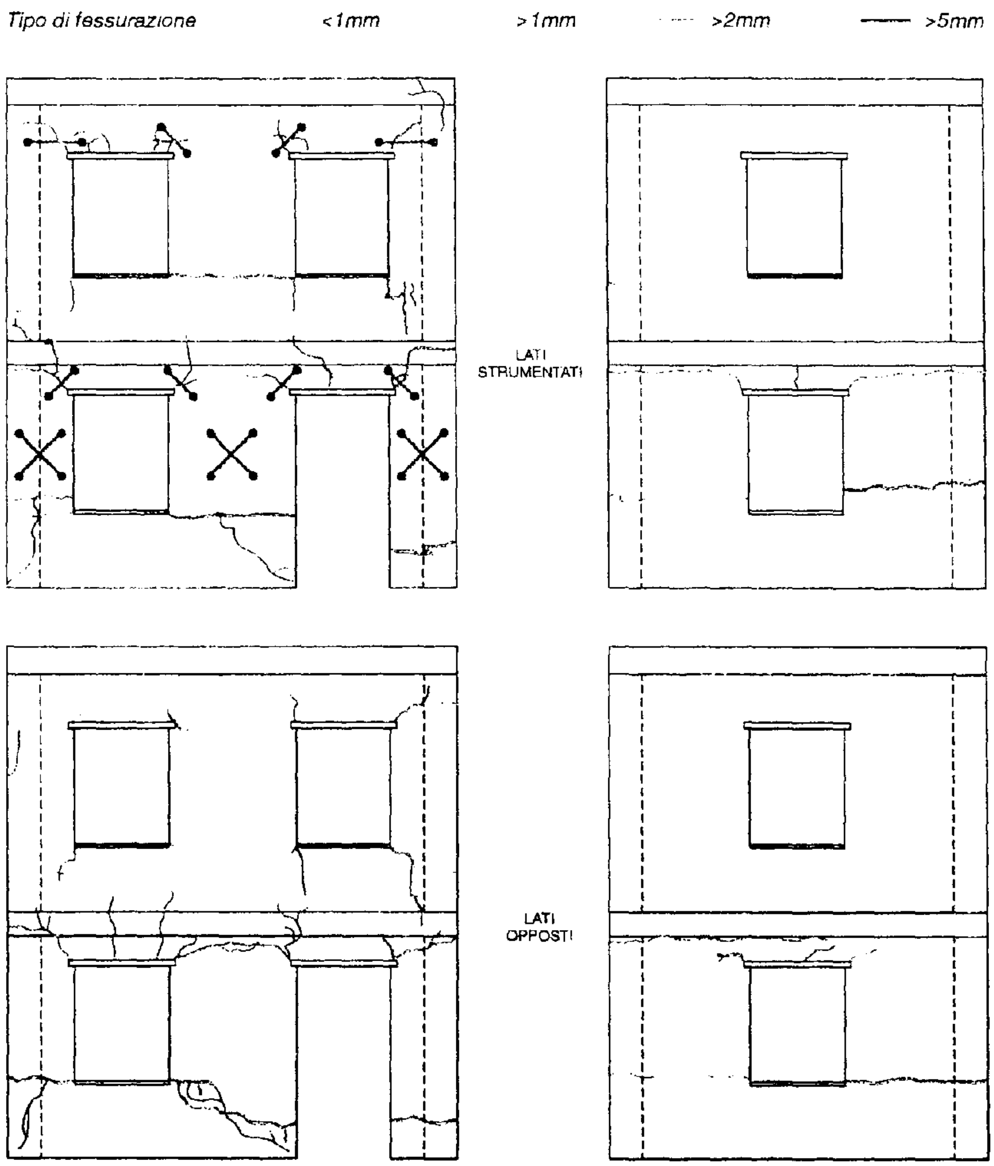

Fig. 5. Sketch of the experimentally observed damage for the test N. 5 (0 dB).

Finally, a rocking compression failure, not clearly observed during the experimental test, was generated by the simulation corresponding to the test N. 5.

The partial displacement between numerical and experimental results in the final test can be attributed to the fact that the seismic loads applied to the building in the software simulation are higher than those experimentally exerted, as it is shown by the difference between the numerical and the experimental values of acceleration related to the last two steps of the test sequence. This difference between acceleration values is clearly to be connected with the ductile behaviour of the building for strong levels of excitation, which would require to reduce the elastic reference response spectra for the software analysis through the use of a "structural coefficient" according to the indications of Eurocode 8.

CONCLUSIONS, CURRENT AND FUTURE DEVELOPMENTS

The need of support to the seismic assessment and retrofitting of buildings is dictated by the complexity of the problem and by the heterogeneity of the involved knowledge. A mobile laboratory developed for this purpose has been presented.

It integrates a knowledge-based decision support system with equipment for data acquisition about structural features and properties of the materials. The modelling technique, the simulator, the models and the user interface have been implemented, while other components of the system are under development.

A PC version of the knowledge-based decision support system is under development in the MS Windows environment. It will integrate a relational data base management system and a geographic information system. It will also implement many more models of the seismic behaviour of masonry buildings such as the GNDT model (1), Gavarini's model (4), the European Macroseismic Scale (up-dated MSK-scale) Methodology (5) and the PSI model (7).

The models 1 and 2 have been applied to about 1000 masonry buildings, comparing the results with those generated by GNDT methods.

The model 3 has been tested through comparison with the behaviour of large scale physical models of masonry buildings on shaking tables in ISMES and is being evaluated using data coming from tests performed on a real scale masonry building prototype in the University of Pavia.

A comparative assessment of the different models will be performed on data about buildings in old urban nuclei (Lisbon, Rhodes, Naples) in the framework of an EC funded project in which ISMES and The Martin Centre of the Cambridge University Department of Architecture will collaborate.

The whole mobile laboratory has been used in the seismic assessment of three buildings in an old Italian urban nucleus.

ACKNOWLEDGEMENTS

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- the European Community (Environment Program).

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Appendix A
Conference Information

Workshop Program

List of Participants

Workshop Program

US-Italian Workshop on:

Guidelines for Seismic Evaluation and Rehabilitation of Unreinforced Masonry Buildings

University of Pavia, Italy
June 22-24, 1994

sponsored jointly by
U.S. National Center for Earthquake Engineering Research and
Gruppo Nazionale per la Difesa dai Terremoti

Wednesday, June 22

2:00pm **Introduction and Welcome**
Daniel Abrams, University of Illinois and G. Michele Calvi, University of Pavia

2:30pm **Issues in Building Rehabilitation and Preservation**
Chairmen: D. Abrams and G.M. Calvi

Architectural Issues in the Seismic Rehabilitation of Masonry Buildings, Randolph Langenbach, Federal Emergency Management Agency, Washington DC

Actuality and Modeling of Historical Masonry, Antonino Giuffre', Caterina Carocci, Gianmarco de Felice, and Cesare Tocci, Terza University, Roma

Discussion

4:00pm **Break**

4:30pm **Development of Guidelines for Seismic Rehabilitation**
Chairman: C. Gavarini

An European Code for Rehabilitation and Strengthening, Giorgio Macchi, University of Pavia

Summary of the ATC-33 Project on Guidelines for Seismic Rehabilitation of Buildings, Daniel P. Abrams, University of Illinois at Urbana-Champaign

Discussion

6:00pm **Adjourn**

Thursday, June 23

9:00am	**Research on Performance and Response of URM Building Systems** Chairman: G. Ballio
	Research on Unreinforced Masonry at the Joint Research Center of the European Commision, Armelle Anthoine, Ispra, Italy
	Research on the Seismic Performance of Repaired URM Walls, Sherwood Prawel and Hsien Hua Lee, State University of New York at Buffalo
	Dynamic Response Measurements for URM Building Systems, Daniel Abrams and Andrew Costley, University of Illinois
10:30am	**Break**
11:00am	*Experimental Research on Response of URM Building Systems,* G. Michele Calvi, and Guido Magenes, University of Pavia
	Discussion
12:00noon	**Laboratory Demonstration**
	Real-Time Static Testing of Full-Scale Masonry Building Prototype, G. Michele Calvi,University of Pavia
1:00pm	**Lunch**
2:30pm	**Analysis Methods for Evaluation of Rehabilitated Buildings** Chairman: R.H. Atkinson
	Simplified Methods for Evaluation of Rehabilitated Buildings, Peter Gergely, Cornell University, Ithaca, New York, and Ronald Hamburger, EQE International Inc., San Francisco, California
	Modeling Unreinforced Brick Masonry Walls, Luigi Gambarotta, and Sergio Lagomarsino, University of Genova
	Failure Criterion for Brick Masonry Under In-plane Load: A Micromechanical Approach, Gianmarco de Felice, Third University of Roma
	Discussion
4:00pm	**Break**
4:30pm	**Case Studies of Building Preservation Projects** Chairman: P. Gergely
	Rehabilitation of URM Buildings in the Eastern United States, John Theiss, Theiss Engineers, St. Louis
	Rehabilitation of URM Buildings in Italy, Carlo Gavarini, University of Roma
	Monitoring of Ancient Masonry Towers and Domes in Pavia, Giorgio Macchi, University of Pavia
	Discussion
6:30pm	**Adjourn**

Friday, June 24

9:00am **Testing Methods for Evaluation of Insitu Material Properties**
Chairman: G. Kingsley

Measuring Masonry Material Properties, Luigia Binda, Giulio Mirabella Roberti, Claudio Tiraboschi, and Silvia Abbaneo, Milan Politecnico University

In-Place Evaluation of Masonry Materials, Richard H. Atkinson, Atkinson-Noland & Associates, Boulder, Colorado

Development and Use of a Mobile Laboratory for the Assessment of URM Buildings, Mauro Cadei, Paolo Panzeri, Alberto Peano, and Paolo Salvaneschi, ISMES, Bergamo

Discussion

11:00am **Break**

11:30am **Workshop Summary and Formulation of Resolutions:** D. Abrams

12:30pm **Close and Lunch**

1:30pm **Optional Tour of Monitoring and Rehabilitation Projects in Pavia**

Italian Seminar on Numerical Modeling of URM Buildings
Wednesday, June 22, 1994: 9:30 to 12:30pm
University of Pavia

Results of numerical models conceived by various investigators throughout Italy will be presented and discussed relative to measured behavior of the full-scale, two-story structure tested at the University of Pavia. Measured strengths, force-deflection curves, deflected shapes and crack patterns of the two-story building structure will remain undisclosed until results of each analysis is presented.

US-Italian Workshop on:

Guidelines for Seismic Evaluation and Rehabilitation of Unreinforced Masonry Buildings

Participants from United States of America

Daniel P. Abrams, *US Chairman*
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