PROCEEDINGS

Joint USA/Italy Workshop
on
Seismic Repair and Retrofit
of Existing Buildings

May 7–11, 1984
Rome, Italy

The Center for Environmental Design Research — University of California, Berkeley

Istituto di Scienza delle Costruzioni — Universita di Roma

The National Science Foundation — Washington, D.C.

INTENTIONALLY BLANK

Henry J. Lagorio
Center for Environmental Design Research
College of Environmental Design
University of California, Berkeley

Carlo Gavarini
Istituto di Scienza delle Costruzioni
Universita di Roma

John B. Scalzi
National Science Foundation
Washington, D.C.

Center for Environmental Design Research
College of Environmental Design
University of California, Berkeley

Proceedings prepared with the assistance of
Todd S. Phillips, Research Assistant
Kellie Barnes, Administrative Secretary

Center for Environmental Design Research
College of Environmental Design
University of California, Berkeley

TABLE OF CONTENTS

INTENTIONALLY BLANK

PREFACE

A Joint U.S.A./ITALY Cooperative Project was developed in 1983 to organize and convene an interdisciplinary Workshop on Experimental Research for Structures and Components in Repair and Retrofit Methods for Earthquake Hazards Mitigation. The resulting Workshop was held in Rome, Italy, during May 1984. Its fundamental goal was to provide a forum for scientific exchange between the U.S.A. and Italy involving participation by researchers and technical, professional consultants of both countries in seminar discussions and field study for the improvement of earthquake hazards-resistant design. The Workshop's immediate objective was to focus on the repair, strengthening and rehabilitation of existing buildings in order to protect them and their occupants against major, damaging earthquakes.

Eight (8) official participants from the United States (U.S.), fifteen (15) official participants from Italy, one (1) observer from the U.S. and twelve (12) observers from Italy met and worked together at the Institute of Construction Sciences of the University of Rome during two days of the Workshop in a free exchange of technical information. In addition to the formal presentation of scientific papers, open discussions of retrofit methods and research results occurred, which, in turn, were later enriched by a series of supplementary, informal group activities, including lunch and dinner work meetings.

An additional two days of the Workshop were devoted to observation and study, in the field, of current reconstruction efforts in the Irpinia area of Southern Italy, site of the damaging Campania/Basilicata earthquake which occurred on November 23, 1980. Several hilltowns, including Sant'Angelo dei Lombardi, San Gregorio Magno, and others that were severely damaged by the earthquake, were visited by the eight U.S. participants, accompanied by a team of six Italian participants. As part of this activity representative examples of local building construction types and earthquake damage characteristics, as well as field tests of on-site, existing masonry walls were examined. In addition, building repair, strengthening, and retrofit methods, and examples of urban scale planning and design reconstruction programs were analyzed and discussed.

On the U.S. side, the project was supported by the National Science Foundation (NSF) through Grant No. CEE 8303857 to the Center for Environmental Design Research (CEDR) of the University of California at Berkeley. Fiscal support by NSF and the personal attention given to the project by Dr. John B. Scalzi, NSF Program Director in Earthquake Hazards Mitigation, are cordially acknowledged and gratefully appreciated.

On the Italian side, the project was supported through the sponsorship of Professor Carlo Gavarini, Director of the Istituto di Scienza delle Costruzioni, University of Rome, and Head of the Gruppo Nationale per la Difesa dai Terremoti, Consiglio Nazionale delle Ricerche (CNR) in Rome. Professor Gavarini's personal efforts and contributions, generously and

freely given, toward the success of the Workshop and its field trips are respectfully recognized and deeply valued.

The observations, opinions, findings, assessments, summaries and conclusions presented in these Workshop Proceedings are those of the participants and individual contributors and do not necessarily reflect the views of NSF in Washington, D.C., and CNR in Rome, or any other governmental organization or institution in either the U.S. or Italy.

Henry J. Lagorio, AIA
Director, 1980-1984, CEDR,
University of California
Berkeley, California

EXECUTIVE SUMMARY

INTENTIONALLY BLANK

Workshop Summary and Recommendations

Henry J. Lagorio (I)

INTRODUCTION

Both countries, the United States and Italy, are located in high risk seismic areas, and both have a long history of experiencing major, damaging earthquakes. Since 1980, Italy alone has endured five major seismic events with severe damage to urban and rural areas in its central and southern regions, and with severe losses to a wide range of physical facilities and society. During the last 10 years, each country has developed significant research components related to earthquake hazards reduction programs. With this background information in evidence, it was clear that both countries could benefit from a program of mutual exchange of scientific data focusing on the mitigation of the effects of damaging earthquakes.

Taking this into consideration, a joint cooperative workshop was convened in Rome for four days in May, 1984, with support from the National Science Foundation and the National Research Council, as an initial step in assessing the feasibility of establishing opportunities for the exchange of technical information on specific topics dealing with earthquake engineering. As a starting point, emphasis was placed on those activities dealing with vulnerability problems of the urban environment and repair/retrofit methods of existing structures and facilities. At the conclusion of this initial meeting, a strong recommendation was made to continue exchange efforts by exploring the possibility of developing a more structured plan for a program of cooperation in several, different topics related to earthquake hazards reduction.

GENERAL COMMENTARY ON WORKSHOP PROGRAM

The four day Workshop was organized by the Institute of Construction Sciences of the University of Rome and the Center for Environmental Design Research of the University of California at Berkeley, who were co-sponsors of the event. The participants from the United States and Italy met and worked together at the Institute in Rome for two days in a program consisting of formal presentations of technical papers followed by informal discussion sessions. After an introductory briefing session led by Professor Gavarini for the U.S. delegates on Monday evening, May 7, the formal activities of the Workshop started on Tuesday morning, May 8, with opening addresses by Professors Gavarini and Lagorio. The addresses were then followed by Technical Sessions I and II, in which nine technical papers were presented as part of the official program. Two additional working sessions, Technical Sessions III and IV, were

(I) Director, Center for Environmental Design Research, University of California - Berkeley

completed Wednesday, May 9, and saw another eight presentations made. In addition to these seventeen invited technical papers presented formally to the participants, six other papers addressing specific topics were presented at various intervals during the two day Workshop. Each of the four Technical Sessions was followed by the opportunity for open discussion, and each was supplemented further by informal group meetings that included lunch and dinner sessions.

Two days of the Workshop, Thursday and Friday, May 10 and 11, were devoted to the field study of the reconstruction efforts which followed the earthquake in the Campania/Basilicata area of southern Italy on November 23, 1980. Several damaged hilltowns, including Sant' Angelo dei Lombardi, San Gregorio Magno, and others, were visited by the eight U.S. delegates and a team of six Italians. As part of this activity, examples of on-site field test methods, building repair and strengthening problems, urban scale planning and reconstruction programs were examined and reviewed. The Workshop was officially adjourned Friday evening, May 11, after the return to Rome from the field trip.

The entire program of the seventeen invited papers presented during the four day Workshop is given in detail on pages and of this publication. Note that the six additional papers presented at intervals during the Workshop, and reproduced in the proceedings, are also listed.

SUMMARY

Following the Technical Sessions of the first two days, the additional two days of site visits during the field study period, and formal discussion meetings throughout the entire four days of the Workshop, the participants met again to discuss areas of common interest and mutual concern for future cooperation and exchange. Discussions covered many aspects of risk analysis, vulnerability assessment, building design and construction, structural repair and retrofit methods, building classification systems, urban planning and design objectives, specific earthquake engineering problems, and different topic items all related to earthquake hazards reduction. It is impossible to transmit the full content and the spirited exchange in all discussions and meetings which took place without a complete record of all comments and points made by each individual during the four days of the Workshop. Consequently, the following summary is intended to reflect these ideas as completely as possible without reference to individual statements and observations.

RESOLUTIONS

At the conclusion of the discussions it was clearly indicated that research exchange between the United States and Italy should continue in the interest of providing mutual assistance in addressing earthquake engineering problems. Accordingly, it was unanimously resolved that the opportunity for a joint, cooperative plan be established for exchange between scholars, researchers, and professionals of the two countries, and that it be designated to:

1. Develop general procedures for continued international cooperation in earthquake engineering research between interested parties from the United States and Italy.

2. Provide a forum for U.S. scholars, researchers, and professionals with study interests in earthquake engineering to meet jointly with counterpart Italian engineers and academicians to exchange information on earthquake hazard reduction programs on an interdisciplinary basis.

3. Identify high priority topic items in the development of a specific research agenda on different topic items associated with earthquake engineering.

4. Evolve a plan for the exchange of technical data on several different topic items related to earthquake events and research developments in each country.

5. Seek effective methods for the dissemination of scientific information and the implementation of research results for the benefit of earthquake hazards reduction programs as practiced in the United States and Italy.

6. Issue joint reports and other publications as documents of record for exchange with others in the earthquake engineering research community and as specific evidence of results accomplished under the program.

RECOMMENDATIONS

Following passage of the resolutions by consensus, further discussion focused on the identification of technical research issues of joint interest and mutual concern. After extensive discussion, in which participants were given the opportunity to present research topics which seemed to be the most appropriate for immediate attention, a wide range of research activities were defined on an individual basis. The comprehensive list of research topics which resulted became quite global and, in the judgment of all who participated in the discussion, it was soon apparent that the recommendations made could not be easily resolved into a priority listing. The topic elements recommended for attention are summarized in the six areas of concern indicated below.

1. Methods to assess the vulnerability of existing buildings and urban centers located in seismic areas on a regional scale, including analyses of specific building performance and response under earthquake loads.

2. Advancement of appropriate risk analysis and assessment methods for the evaluation of the exposure of existing areas to seismic events on a regional scale. Evaluation and risk analyses to include the measure of probabilistic approaches.

3. Seismic repair and retrofit methods and techniques for reinforced concrete and masonry structures and facilities, including precast and prestressed systems. Experimental investigation of existing construction details and fundamental approaches for strengthening existing buildings of all classifications. Field tests of actual force-deformation characteristics as a measure of deformation/damage ratios.

4. Development and implementation of cost-effective earthquake hazard mitigation programs for existing building stock and urban-scale systems, including life-line systems.

5. Promulgation of earthquake preparedness methods and the development of suitable emergency decision-making processes, including emergency post-earthquake recovery policies.

6. Development of appropriate building classification systems for preparedness studies and post-earthquake damage surveys. Economic models for assessing costs of building damage loss.

CONCLUSIONS

As an extension of the four day Workshop held in Rome in May, all participants were invited to attend a half-day, follow-up discussion meeting led by Professors Gavarini and Lagorio, Co-Principal Investigators of the initial Workshop, in San Francisco during the Eighth World Conference on Earthquake Engineering (8WCEE) on July 25. The purpose of this second meeting was to seek added definition of areas of joint interest in scientific exchange of earthquake engineering information, and to develop final recommendations for future initiatives. Representative participants who attended the initial Workshop, and others from both countries, were joined by Dr. John B. Scalzi of the National Science Foundation to discuss further the results of the May, 1984 Workshop, and to assist in the identification of potential goals and objectives for a joint, comprehensive plan of exchange.

Extensive discussion of many topics occurred during this last meeting, which was held as an open forum, and individual opinions were freely expressed. It was agreed in the meeting that exchange could occur in three categories: a) Joint, cooperative research projects between individuals on topics of mutual interest; b) Free exchange of scientific information and technical data in publications, professional papers, research results, and other documents, including formal and informal correspondence on an individual basis; and c) Exchange of junior faculty, visiting scholars, and researchers at academic and professional levels.

At the end of the meeting major issues of joint interest and mutual concern were carefully considered in order to distill a generally stated research agenda and to bring into sharper focus a set of fundamental research topics suitable for study. The major issues identified for attention are summarized below in a final listing of four groups representing soft and hard topic areas of consideration.

1. Vulnerability Assessment Considerations: Risk analysis and the vulnerability evaluation of existing buildings, urban centers and towns. To include building classification systems, identification of damage patterns, damage probabilities, post-earthquake damage surveys, emergency preparedness policy decisions, and earthquake vulnerability prediction theories.

2. Hazards Mitigation Considerations: Experimental and analytical methods, hazard mitigation methods for buildings, urban centers and towns, repair and retrofit methods, including economic cost models, and assessment procedures and priorities. Code development, design and analysis methods for new and existing buildings and other facilities of masonry and reinforced concrete construction, including precast and prestressed systems.

3. Seismic Design and Energy Conscious Design Considerations: Studies and experiments of structural properties versus thermal properties of construction materials, thermal conductivity characteristics and seismic design approaches for brick, concrete blocks, and other typical construction methods and techniques. Building design, layout, and configuration. Planning and design of new buildings, and the repair and retrofit of existing buildings.

4. Bridge Research and Design for Transportation Facilities: Analytical and experimental methods in the design of new bridges, including methods for the repair and retrofit of existing bridges. New approaches to abutment design and improved studies of soil-structure interaction, using systematic methods and theory.

All interested researchers and professionals were encouraged to participate in addressing the preceding research topic areas. It was agreed that individual participation could proceed at formal and informal levels on a case-by-case basis depending on joint, cooperative interests. The final meeting of this extended workshop discussion was concluded on a positive note of shared interest in, and reciprocal acknowledgement and mutual respect for, the personal and technical research efforts of all participants. Finally, all participants expressed the desire to meet again in another joint workshop as a second effort in initiating cooperative research exchange between scholars and professionals of the two countries.

INTENTIONALLY BLANK

INTRODUCTORY PRESENTATIONS

INTENTIONALLY BLANK

Opening Address

C. Gavarini (I)

It is a great honor of mine, and a great pleasure too, to welcome you to Italy.

On behalf of the National Council of Research, the institution which corresponds to your National Science Foundation, I hope that your trip will be scientifically profitable and that all of you will personally ebjoy it. And it is my duty to help you, together with my Italian colleagues, to assure the full success of this workshop.

Italy is, unfortunately for us, very interesting from a seismic point of view; and in order to better point out this fact, we can mention two very recent earthquakes, one a week ago, April 29, localized in central Italy, in the region Umbria, which includes famous historical towns like Assisi, Gubbio and Perugia. By chance the quake was not too severe (Magnitude 5) and there were no victims, but the damage to the buildings, and in particular to the historic monuments, is rather heavy. The second one of yesterday evening, while we were talking together during our preliminary meeting, occurred at 19 hours and 53 minutes. In this case the epicenter is situated in a south-east direction from Rome and the news is very current and incomplete. Two or three victims at least have been identified, and the injured are estimated to be fifty. The Magnitude is 5.2.

The seismicity of our country is of course a challenge to us, and we are engaged against the effects of earthquakes under different organizations, one of which is within the framework of the National Council of Research, under the name of "Gruppo Nazionale per la Difesa dai Terremoti", i.e. National Group for the Defence against Earthquakes. Most of us present today work for this Group, trying to obtain through cooperation the best results in the battle against earthquakes, a battle which is difficult and in particular which cannot be conducted only on the scientific side, but which requires a global strategy in all the aspects of life, so that the appropriate approach must be essentially multidisciplinary. A trivial example is given by an anecdote of the Umbria earthquake of a few days ago: one of the few injured was a man who jumped out the window, breaking his two legs, while the building remained safe; and that man was a policeman, that is, one of those who should keep a cool head during major disasters and help other people. This means, and we all know it very well, that education is no less important than other aspects of the problem.

But today we are among specialists and of course we are seriously interested in our personal fields of research.

(I) Director, Institute of Construction Sciences, University of Rome

Taking into account both points of view, Professor Lagorio and I decided to propose both technological contributions on repairing of structures damaged by earthquakes and general contributions devoted to the problem of rebuilding and prior retrofitting of buildings and towns.

In particular we will try to give you an idea of the specific problems we are facing in a country which has many ancient towns, often with historical buildings and monuments, many of stone masonry, and many of poor construction in the little villages. So the problem is not only how to repair, or retrofit, a single building, but mainly how to act on the hundreds of thousands of buildings, in order to lower the seismic risk all over the country.

The last major earthquake was in Irpinia, November 23, 1980, and in Irpinia we will make our visit following the two days of our seminar in Rome. We have selected two towns for the first day: S. Angelo dei Lombardi and Lioni, because of their importance as examples of damages, placed as they are at the epicenter of the quake. And we have selected the town for the second day: S. Gregorio Magno, because of the interest of the global approach with which the reconstruction is being performed.

In the three towns, by the way, our National Group is involved in research and assistance. Some of the contributions during the seminar will be concerned with the activity in those towns or, more generally, in the Irpinia area.

But the activity of reconstruction in Irpinia is only beginning. We must confirm that there are many difficulties of various kinds, so that the possibilities for seeing broken buildings in that region are presently greater than those for seeing repaired buildings. In contrast, the situation in Friuli (May 6, 1976) is the opposite and reconstruction is nearly complete. Repairs performed in Friuli, mainly concerned with masonry buildings, are addressed by one paper in the seminar.

The Italian contributors to the seminar are by no means all of the researchers involved in Italy with the subject of repairing and retrofitting structures, but it was necessary to limit the number of persons involved in order to faciliate discussion and to maintain the seminar character of the meeting. Thus, our effort in choosing the invited researchers has been to give a representative picture of the activities which are currently underway in Italy.

The planning of the seminar, for the Italian contributions, is the following: the first day will consist of presentations of a more general character, devoted to the general approach and strategy for retrofitting before the earthquake and evaluating damages just afterwards. The second day will be devoted more specifically to the technical and technological problems of repairing and retrofitting.

The first Italian presentation, by Professor F. Braga, will be a general report for the definition of the problem, with the aim of introducing and discussing the questions: Why do we repair and retrofit? What are the dimensions of the problem in Italy? What are the strategies?

The second presentation, by Eng. A. Cherubini, is devoted to the reconstruction plan of the town S. Gregorio Magno, from a general point of view, starting from the urbanistic problems and including the human aspects of social reconstruction.

The third presentation, by Professor V. Petrini, is concerned with the problem which comes before the retrofitting process, that is, the question of assessing the vulnerability of buildings. The activity in this field is presently large in Italy, and Professor Petrini, as Director of the previously mentioned National Group, is the best man to discuss what we are doing all over Italy in this regard.

In the following presentation, Eng. P. Angeletti will treat in particular the question of assessing the vulnerability of reinforced concrete buildings.

As you probably know, a bradiseismic crisis has been present in the town of Pozzuoli, near Naples, for many months, almost a year. The continuous bulging of the soil (1,5 meter in one year) is accompanied by recurrent seismic crises. The most severe quake was in October. Professor R. Ramasco will explain the problem we are facing, i.e. to evaluate, in such circumstances, the vulnerability of buildings in Pozzuoli and its surroundings, and in particular to identify which buildings are safe or not. The investigation is based on the methodology established by our National Group, but with specific forms and criteria calibrated for the present situation. The results will be useful not only for the immediate problem of evacuation but also for the problem of repairing and retrofitting buildings.

The contribution of Eng. M. Dolce is concerned with the results obtained and the studies performed following the damage survey on thirty six thousand buildings in Irpinia after the 1980 earthquake. The results obtained, especially the damage probability matrices, are very interesting for the question of assessing vulnerability, being an important experimental field test.

The second day begins with the general survey of Friuli, which I mentioned before, presented by Professor D. Benedetti.

More technological in character will be the contribution of Professor A. Zingali, devoted to the question of the shear resistance of masonry walls, with indications how the question is treated by our building code on repairing, and with a report on the field experiments we are now starting in S. Gregorio Magno.

Professor G. Augusti will speak about reinforced concrete, treating in particular the question of the seismic upgrading of irregular buildings.

The theme of reinforced concrete will be completed by Professor A. Samuelli Ferretti, who will discuss experiments that are currently being carried out in a coordinated action in many Italian laboratories concerned with the repairing of beam column joints.

Finally, a presentation will be made by Professor G. Grandori, who certainly is well known by all of you, which logically should have been presented during the first day, as the subject involves zoning. The problem is very important in relation to the retrofitting question, because our present official zoning requires improvements and a better assessment, which could be an important factor of optimization in the process. Included, too, are heavy emphases on economical and technical considerations as part of the final objective of obtaining a seismically safe country.

This is the offical program on the Italian side.

Additional secondary presentations will be possible during the discussion periods, and in this regard I hope that discussion is lively and profitable.

In conclusion, I do hope again that the seminar will be a success. I beg your pardon for the simplicity of our organization, which sometimes may be wanting, and I finally hope that, if the result of this first experiment is encouraging, it will be the first of a long series, and that the cooperation of our two countries in this field will continue.

Overview of Masonry Research Projects in the United States

Dr. John B. Scalzi (I)

The greatest risk for loss of lives and property during an earthquake is in the large number of unreinforced masonry buildings which exist in the United States since colonial times. These buildings constitute the largest inventory of hazardous buildings in all regions of the country, including the moderate to high risk earthquake zones. As a result, there are many different types of buildings with masonry construction which signifies that the number of research projects which are required are endless. Because of the unique nature of masonry construction, very little research on the seismic performance had been conducted until the San Fernando earthquake of 1971 brought it to the attention of the earthquake community and the Federal Government because of the collapse of many masonry buildings. Practically all of the existing buildings were constructed before seismic regulations were instituted and/or enforced. Therefore, in order to strengthen these buildings, it is necessary to develop the appropriate research information which will evaluate the inherent strength of the structure, and then to verify various methods for the repair and strengthening of the several structural components such as walls, roofs, floors, and connections of all assemblies. These topics became the bases for the research projects supported by the National Science Foundation (NSF).

One of the important features of the NSF earthquake program is the dissemination of the research results. An effective method has been the support of a North American Masonry Conference periodically (at four year intervals) by which researchers present the results of their work to a broad audience and exchange data and information on their projects. The First Conference (1) was held at the University of Colorado, Boulder, Colorado, in 1978, and the second one (2) was held at the campus of the University of Maryland at College Park, Maryland.

These Conferences produce a Proceedings of the papers presented and those accepted for publication in order to contain in one volume all the research data which was developed for the time period between Conferences. By this procedure, a researcher or designer can easily refer to the Conference Proceedings to learn what is being done and what has been accomplished in the research projects of masonry analysis, design and construction.

A brief description of the current research projects is presented below with a statement of the objectives and the status of the project at this time.

(I) Dr. John B. Scalzi, National Science Foundation, Washington, D.C., USA

Methodology for Mitigation of Seismic Hazards in Existing Unreinforced Masonry Buildings (3)

This project developed a methodology for the mitigation of seismic hazards in existing unreinforced masonry buildings to be applied throughout the entire United States because of the various degrees of vulnerability to earthquake damage. The methodology will provide analysis techniques and procedures to determine: degree of seismic hazard, mechanical properties of unreinforced masonry materials and components, requirements for hazard mitigation, and cost-effective methods for repair and strengthening these types of buildings.

The research consists of a combined analytical and experimental investigation which includes: dynamic testing of full scale walls subjected to out-of-plane motions at the top and bottom of the wall typical of those walls which would occur in a building; and dynamic and static testing of full scale horizontal diaphragms subjected to in-plane loadings. The walls varied from 10 to 16 feet with height-to-thickness ratios between 14 and 25. The walls were of 3 wythes of common clay brick, grouted and ungrouted concrete block, and grouted clay block. The diaphragms were 20 x 16 feet and included wood, metal deck, and concrete filled metal deck systems.

The objective of these tests is to correlate the diaphragm response to the resistance against collapse of unreinforced masonry walls subjected to out-of-plane seismic motions. Simplified analytical methods will be developed from this research. As a matter of interest, the results from this investigation contributed significantly to the development of a city ordinance to reduce the earthquake hazard of existing buildings.

Basic Properties of Clay Unit Masonry (4)

The objective of this research is to investigate in-depth the factors which control the strength of brick masonry in compression under static and dynamic loading environments. The tasks include:

1) Triaxial tests of several mortar mixes to develop mortar strength, modulus of elasticity and poisson's ratio for use in a masonry failure analytical model.

2) An apparatus for tensile testing of whole bricks and for use in combined tensile-compressive tests of bricks has been developed. These data are also required for the failure model.

3) A method to reduce the lateral confining friction force exerted on compression specimens has been developed. This is used in compression tests of brick and brick prisms to obtain "unconfined" compressive strengths for use in the failure model. Compression tests of strain-guaged bricks have demonstrated the effectiveness of the interface friction reduction method.

Future work includes experiments of prisms made with different brick-mortar combinations to enable comparisons between theoretical predictions based on the failure model and actual performance, and also includes cyclic compression tests of prisms at rates simulating seismic loadings on the compression of full-scale masonry.

An Investigation into Nondestructive Evaluation of Masonry Structures (5)

Six nondestructive evaluation (NDE) methods were investigated to assess their potential use for strength and condition evaluation of masonry using unmodified commercially available equipment. The methods were: vibration, rebound hammer, penetration, ultrasonic pulse velocity, mechanical pulse velocity, and accoustic-mechanical pulse. The candidate methods were applied to two-wythe cantilever wall specimens. Companion small-scale specimens, specimens removed from the walls subsequent to the NDE tests, and in-the-wall specimens were tested to destruction to provide compression, shear and flexural strength data for correlation studies. Flaws, in the form of delaminated bed joints, were created in the wall specimens to determine the ability of each NDE method to detect defects in masonry.

Measurements from each method except accoustic-mechanical pulse were compared to strength properties as established by destructive tests using bivariate linear regression analyses. Results of the analyses indicate that strength properties of the masonry tested could be generally estimated by some of the NDE methods considered with the highest correlation coefficients in the 0.8 to 0.9 range. Investigation of the accoustic-mechanical pulse method was very limited, but indicated that consistent measurements could be obtained and that flaws could be detected.

It was concluded that NDE methods offer a means of relative quality assessment and flaw detection, and with present procedures and equipment, a limited quantitative evaluation of the type of masonry tested. It was also concluded that with refinements of procedure and some modifications to equipment, the efficacy of most of the methods evaluated would be enhanced. Further investigation of the accoustic-mechanical pulse method appears warranted based upon the very limited experience gained and that further studies are needed to assess the utility of the NDE for other types of masonry.

Cyclic Response of Masonry Anchor Bolts (6)

This experimental research project is nearing completion in which the strength of "J" bolts embedded in brick and concrete block masonry is determined. The bolts investigated varied in diameter from 3/8" to 1-1/4" and were tested in shear, axial, and combined shear and axial directions. The tests were repeated with monotonic and cyclic loading.

Results indicate that strengths increase with bolt diameter up to the 3/4" size, with little additional benefit from larger sizes. Current code allowables appear to have a safety factor of approximately ten. The results of this research are extremely important to the designers, engineers, and code bodies because very little reliable data were available when most of the currently used values for bolt strengths were established.

Seismic Response of Composite Masonry in New and Existing Structures (7)

This research project assesses the behavior of new and existing composite masonry buildings subjected to static and cyclic laodings using an analytical and experimental approach.

The analytical phase includes the development of a finite element model in the form of a "Superelement" which may be used in the analysis of large and complex composite walls. The "Superelement" includes both wythes and the collar joint, and considers shear failure at the collar joint as well as in-plane failure of each wythe. The emphasis is on the development of criteria for delamination of the interface due to shear stresses caused by the superimposed loads, thermal effects, expansion and shrinkage due to moisture, and creep.

The finite element model for the interface shear strength will be verified by means of static and cyclic tests. The values of constants assumed in the development of the finite element model concerning shear failure will be modified, if necessary, to achieve a better correlation between the analytical and experimental results. The results of this research project will develop criteria for the design of composite masonry walls subjected to in-plane loads.

During the first year of this project, a composite element that is capable of transferring shear under linear elastic conditions was developed. Currently, thermal effects, moisture expansion and contraction, and creep phenomenon are being incorporated into the model. Sixty composite wall specimens, each 16 inches square, constructed with 3/8" thick slushed mortar collar joints have been tested under static and cyclic loads. Variables in these tests were mortar, reinforcement, type of loading, and absorption in brick and block. Specimens with 2 inch collar joints are currently under preparation for continuation of the experimental phase.

Strengthening of Brick Masonry Walls with Shortcrete (8)

The objectives of this experimental research project on strengthening of existing unreinforced brick walls are four-fold: 1) Investigate the bond strength between shotcrete and old, molded clay brick and the effect of wetting the brick or coating it with epoxy prior to application of shotcrete; 2) determine the interface shear capacity of the brick-shotcrete composite; 3) determine the need for and influence of steel dowels which anchor two wythes of brick to a surface layer of shotcrete; 4) investigate the possible use of a thick, lightly reinforced layer of shotcrete on one or both sides of a brick wall for seismic strengthening in regions of moderate earthquake risk.

One or two wythe brick panels measuring 1m x 1m and 1.3m were reinforced with a 9cm layer of shotcrete on one side and were tested to determine the overall behavior of the brick-shotcrete system and to determine whether the bond between the brick and shotcrete is sufficient to develop full composite behavior for in-plane loads. The shotcrete (f^o = 47.9 MPa) was reinforced with a welded wire fabric, with a reinforcement ratio of 0.0013 in each direction. Some panels were dry prior to shotcreting, others were thoroughly wetted, while some were coated with epoxy. Wall panels were tested under a reversed cycle, in-plane load across the diagonals. Results indicated that similar panels with different surface conditions developed the same strengths but that the wetted and epoxy-coated brick panels exhibited greater inelastic deflection capacity and, thus, greater seismic resistance. The shotcreted panels averaged more than thirty times stronger than unstrengthened brick panels. Unreinforced wall panels could not sustain reversed loading.

Reinforcing bar dowels of size number three (10mm diameter) were epoxy bonded into two wythe panels prior to shotcreting. Panels with dowels remained intact while those without dowels delaminated. Some panels were strengthened with a 3.8cm layer of shotcrete reinforced with an expanded metal lath (0.0025 reinforcement ratio). Strengthening of the brick panels was shown to be proportional to the thickness of the shotcrete. Future tests will examine the out-of-plane behavior of 0.7m x 2m panels in order to determine better values for the interface shear resistance.

Seismic Resistance of Masonry Piers (9)

This research project has the objective of improving the seismic resistance of masonry piers. The investigation consists of experimental and analytical studies. By improving behavior, we mean increase the ability of the pier to sustain deformation and absorb energy before it falls.

The experimental arrangement can be explained briefly. The pier is first subjected to a specified vertical loading using vertical actuators. Then horizontal displacements are imposed cyclically at the top of the pier by means of horizontal actuators. The amplitude is increased monotonically until failure. Combinations of displacements and horizontal forces necessary to realize these displacements are recorded and result in hysteresis loops. The envelope of these loops acts as a measure of the pier^{o}s capacity to absorb energy. A recent improvement in the test procedure which prevents rotation at the top of a pier has been achieved by removing restraining rods and coupling the vertical actuator loads to the horizontal displacements.

The method used for increasing seismic resistance of the piers depends on whether the pier will demonstrate a flexural or shear failure. Flexural failures will occur when the vertical load is relatively light, that is, at the upper stories of a building. This failure can be improved by reducing the vertical reinforcement and using toe plates to reduce toe crushing. Shear failure (typical x-cracking) occurs when the vertical load is large. Shear failure can be improved by the addition of horizontal reinforcement. The most important aspect of this failure mode is the anchorage of the horizontal bars. Anchor plates at the ends of the bars have dramatically increased the ductile behavior of the walls.

Basic Properties and Strength of Connections for Concrete Masonry Buildings (10)

The objectives of this project are the determination of the basic properties of concrete masonry units, prisms, and walls, using various mortars, and the strength of connections between walls and floors.

The basic properties of concrete masonry have been determined and published in several journals, such as the First North American Masonry Conference. The investigation of the strength of the connections is in its final stages and publication of the results will be available later in the year.

Repair of Masonry Walls with Epoxies (11)

This project was recently awarded, therefore, the researcher is just starting his planning for the series of tests which will be performed subsequently. The objective of the investigation is to determine the feasibility of drilling a hole in a one or two story masonry wall, then inserting a reinforcing bar of pre-determined size, and grouting around the bar with standard grout, concrete mixture, and structural epoxy. A cost-benefit study will be made to determine the best method for the repair and/or strengthening of the wall to increase the seismic resistance. There are many masonry buildings of one or two stories in height in the United States which could be preserved by this technique if it is found to be effective, technically and economically.

Slenderness Ratios for Masonry Walls (12)

The objective of this project, which is completed, was to determine the response of walls with different ratios of height-to-thickness (slenderness ratio) when subjected to eccentric axial forces acting in combination with lateral forces which simulated the condition of gravity loads on the walls with wind or seismic forces in the lateral direction.

A total of thirty full size panels were tested consisting of tilt-up concrete, concrete block, clay brick, and clay blocks. Slenderness ratios of 30 to 60 were investigated by using a special test fixture constructed specially for this project.

A result for one of the walls is the following data: Concrete masonry wall 24 feet, 8 inches high, 4 feet wide, reinforced with 5 number four bars of grade 60 steel, block strength of 2600 psi, slenderness ratio of

38, subjected to a vertical load on the wall of 860 pounds per lineal foot and lateral load of 75 pounds per square foot at yield of steel, deflected 6.5 inches and attained a maximum deflection of 11.2 inches.

Similar data for the other tests have been evaluated and the results are being used to formulate building code requirements for consideration by code writing bodies.

It is interesting to note that this project was independently supported by the Structural Engineers Association of Southern California and the Southern California Chapter of the American Concrete Institute.

The National Bureau of Standards (NBS) (13) of the U.S. Department of Commerce has conducted several projects related to the strength, stability, and structural performance of masonry walls and has reported the results in the Structural Journal of the American Society of Civil Engineers and in several NBS publications of the Building Science series.

REFERENCES

(1) Proceedings of the First North American Masonry Conference, August 1978. Editor, J.L. Noland and J.E. Amrhein, University of Colorado, Boulder, CO 80309.

(2) Proceedings of the Second North American Masonry Conference, August 1982. Editor, Donald W. Vannoy and James Colville, University of Maryland, College Park, MD 20742.

(3) Methodology for Mitigation of Seismic Hazards in Existing Unreinforced Masonry Buildings. ABK-A Joint Venture, M.S. Agbabian, S.B. Barnes, John Kariotis. Project Director Robert Ewing, Agbabian Associates, 250 North Nash Street, El Segundo, CA 90245.

(4) Basic Properties of Clay Unit Masonry, Atkinson-Noland and Associates, and University of Colorado, James L. Noland and Daniel P. Abrams. Address: Atkinson - Noland, 2619 Spruce Street, Boulder, CO 80302.

(5) An Investigation into Non-Destructive Evaluation of Masonry Structures. James L. Noland. Address: see (4).

(6) Cyclic Response of Masonry Anchor Bolts, Russell H. Brown, Department of Civil Engineering, Clemson University, Clemson, South Carolina 29631.

(7) Seismic Response of Composite Masonry in New and Existing Structures. Russell H. Brown and Subhash Anand, Clemson University. Address: see (6).

(8) Strengthening of Brick Masonry Walls with Shotcrete. Lawrence F. Kahn, School of Engineering, Georgia Institute of Technology, Atlanta,

GA 30332.

(9) Seismic Resistance of Masonry Piers. Hugh D. McNiven, Director, Earthquake Engineering Research Center, 47th and Hoffman Boulevard, Richmond, CA 94804.

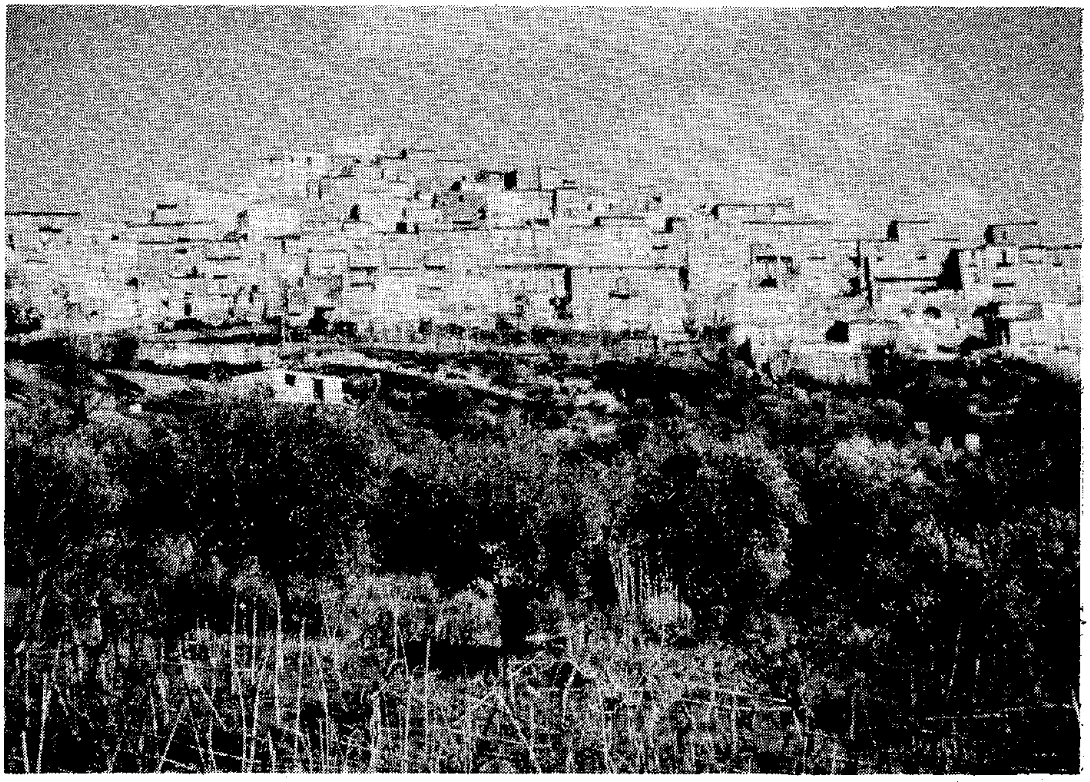
(10) Basic Properties and Strength of Connections for Concrete Masonry Buildings. Gilbert A. Hegemier, University of California, San Diego, CA 92307.

(11) Repair of Masonry Walls with Epoxies. Joseph M. Plecnik, Department of Civil Engineering, North Carolina State University, Raleigh, NC 27607.

(12) Slenderness Ratios for Masonry Walls. Structural Engineers Association of Southern California and ACI - Southern California Chapter. Contact: James E. Amrhein, Director of Engineering, Masonry Institute of America, 2550 Beverly Boulevard, Los Angeles, CA 90057.

(13) National Bureau of Standards. Contact: Felix Y. Yokel, National Bureau of Standards, U.S. Department of Commerce, Washington, D.C. 20234.

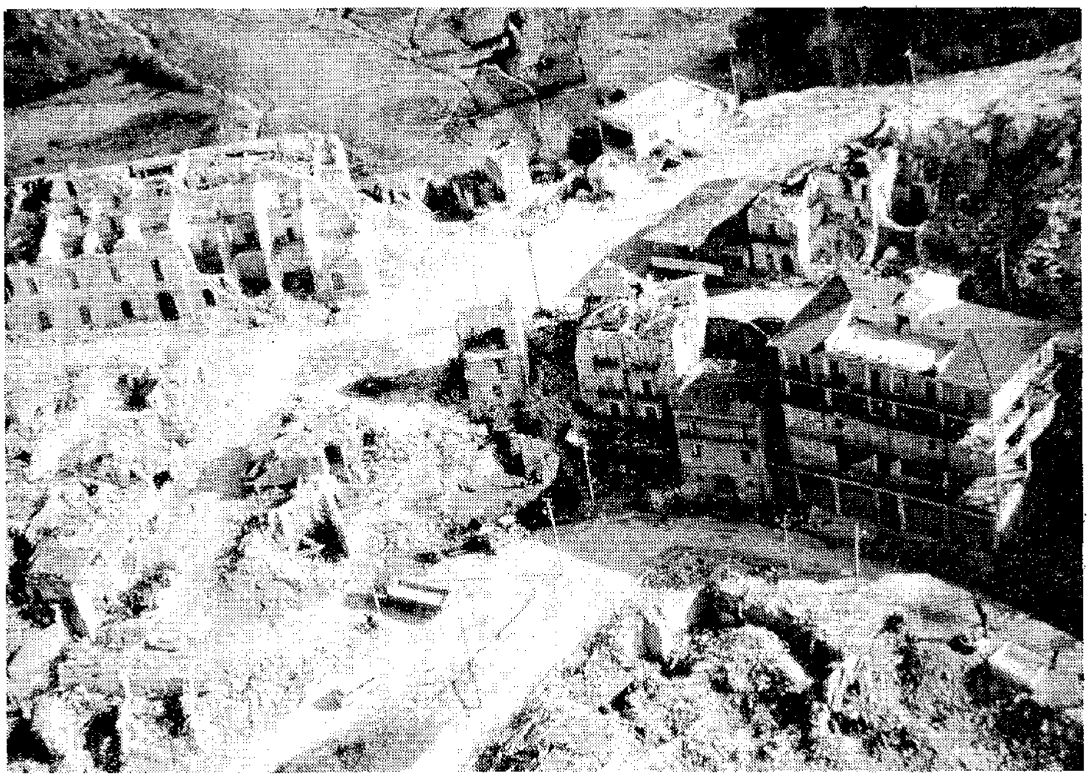
General photographs of damage to Italian hilltowns, 1980 Irpinia earthquake. (Photographs: H. Lagorio/G. Mader, 1981)



View of Italian hilltown from valley floor. Photograph: H. Lagorio/G. Mader

Earthquake damage of attached, multi-storey dwellings, 1980 Irpinia earthquake. Photograph: H. Lagorio/G. Mader

Damage from 1980 Irpinia earthquake in Conza della Campania.
Photograph: H. Lagorio/G. Mader



Damage pattern in center of hilltown, 1980 Irpinia earthquake.
Photograph: H. Lagorio/G. Mader

INTENTIONALLY BLANK

PARTICIPANT PRESENTATIONS

INTENTIONALLY BLANK

Seismic Vulnerability of Existing Reinforced Concrete Buildings. Preliminary Experimental Results for a Theoretical/Experimental Approach

P. Angeletti (I)

SUMMARY

With the aim of assessing seismic vulnerability for existing r.c. buildings, a subjective approach (already tested through a survey of the 1980 earthquake in Southern Italy) is shown. Experimental tests on 12 r.c. infilled frames (1:2 scale) are now in progress in Southern Italy, at Lioni, in order to evaluate vulnerability in terms of damages v/s interstory drift for the sub-assemblages. Preliminary results are shown for three types of specimens.

INTRODUCTION

The specific purpose of assessing seismic risks of existing buildings on medium/large areas can be met by subdividing the analysis into two phases (Reference 1):

1) Determination of seismicity;
2) determination of vulnerability (in terms of damages or victims).

Results from the first step of analysis define expected seismic events on areas examined as macroseismic intensity, spectral acceleration, on the basis of historical reports, tectonical and geological knowledge, measures from past earthquakes, and microzonation analysis. In Italy at the moment, a general seismicity map (Reference 2) and a few microzonations (f.i., Reference 3) are available.

Results from the second step of analysis should define vulnerability functions, i.e. should allow for an assessment of expected damages or victims whenever a seismic event is defined (damages or victims v/s seismic intensity).

Italian Results

In Italy several methodologies have been employed. A first type is a statistical one on the basis of References 4 and 5, and vulnerability functions are defined as damage matrices. A second type is a subjective one (References 6, 7, 8) in which vulnerability is defined as scores (Reference 6) or curves (Reference 7). The approach considering r.c. buildings (References 7, 8) has been tested through damages surveyed immediately after the 1980 earthquake in Southern Italy (Reference 9). In order to conduct other experimental trials, a theoretical/experimental

(I) Researcher, University of Rome, Italy

approach is attempted. This third type of methodology has been used by Scholl et al (Reference 10). Three steps are necessary for attaining the final goal:

1) Finding out experimental vulnerability functions for structural as well as non-structural components (damages v/s variables like interstory drift or local acceleration).

2) Performing a theoretical analysis to correlate seismic levels with chosen local parameters (drift or acceleration).

3) Summing up every partial damage in order to obtain a total damage for the building (or the class of buildings) considered.

It is quite obvious that the first step has to be performed taking the materials and techniques of the area into consideration. The following paragraphs briefly explain the subjective approach and the tests.

ASSESSMENT METHODOLOGY FOR R.C. BUILDINGS

Twenty parameters are taken into account and they are divided into 8 groups (resisting elements, foundation, diaphragms, plan configuration, types of varying elevation, safety of non-structural elements, and safety of interior lines). Each parameter can be classified on 3 levels, A, B, C, from the best to the worst one. A score is assigned to each level according to the importance of the parameter; f.i. the first one (type of main resisting system) can be classified on level A - stiff and strong construction; B - stiff, brittle infilled frames, but strong frames after masonry collapsed; C - flexible, weak constructions. The penalties are equal to 0 for the first level, -1 for the second, and -2 for the third. Summing up all scores, a distance between the (V) curve (at the extreme right in Figure 1) and the (i) curve representing the building considered can be found. Further details might be found in References 7 and 8.

In order to take into account low reliability information on some parameters (f.i. foundation, ductility and strength of critical elements), a 4-level information reliability degree (high, medium, low, no information) is considered, which scatters the obtained average curve (Reference 9). Results from theoretical evaluations have been compared

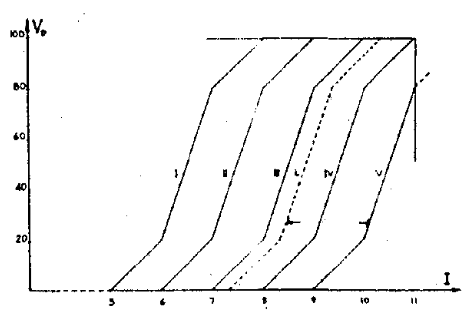


Fig.1: Uncorrected curves

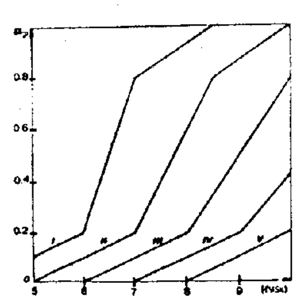
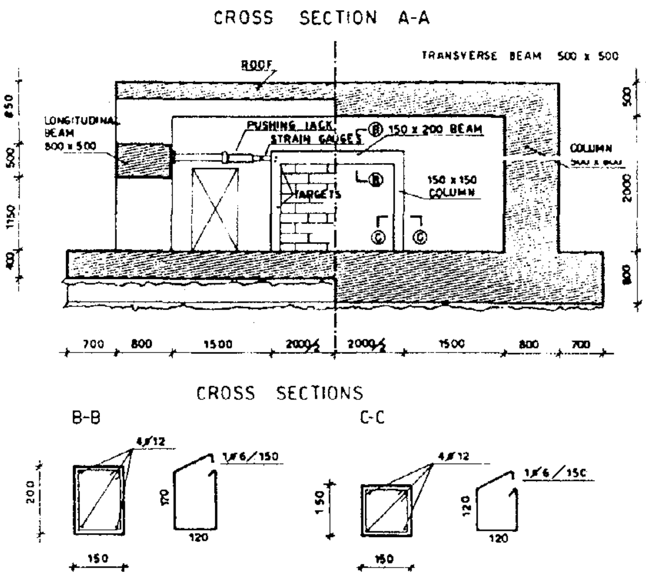


Fig.2: Corrected curves

with a sample of about 100 r.c. existing (plus about 60 collapsed) buildings struck by the 1980 earthquake. The comparison has led to corrections for vulnerability curves (Figure 2) and for penalties (Reference 9).

EXPERIMENTAL VULNERABILITY FOR R.C. INFILLED FRAMES

In order to test the methodology by the theoretical approach described above, 12 1:2 scale specimens were built at Lioni with local materials and techniques. The general plan and cross-sections of specimens are shown in Figures 3 and 4.



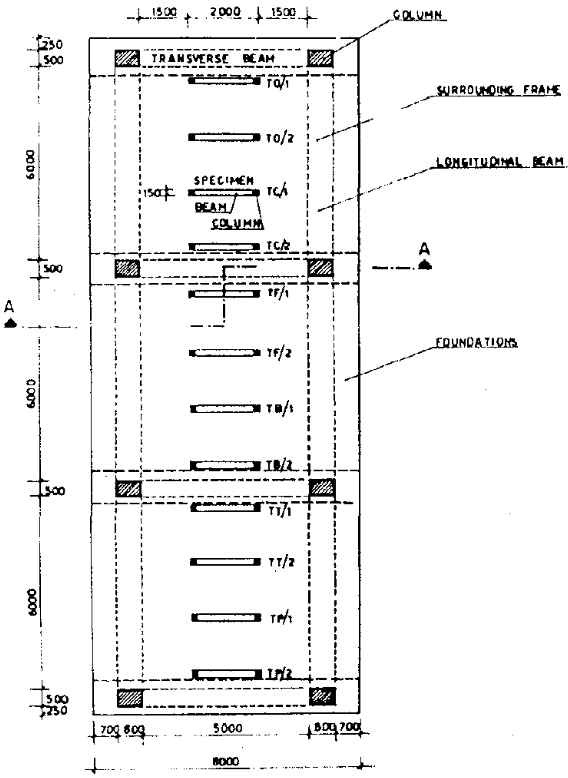


Fig.3 (left): Plan of specimens.

Fig.4 (above): Cross-sections of specimens.

The 12 r.c. frames were poured in place in a bigger r.c. structure supporting all vertical and horizontal loads.* Six types are considered, according to local techniques:

- TO: Non-infilled frames
- TC: Frames infilled with low reinforced poured in place concrete
- TF: Hollow tile infilled frames
- TT: Hewn conglomerate (sand) stone infilled frames
- TB: Concrete block infilled frames
- TP: Field stone infilled frames plastered with cement mortar.

---

* Specimens were built as final practical application of a course for building workers held by C.R.E.S.M. (Center for Social and Economic Development in Southern Italy) at Lioni during October-December, 1983.

Loads are imposed by two 20-ton pushing jacks horizontally placed and supported by small steel structures. The loading level is measured by electrical strain gauges on a steel bar in series with the jack. Loads are imposed according to the scheme shown in Figure 5.

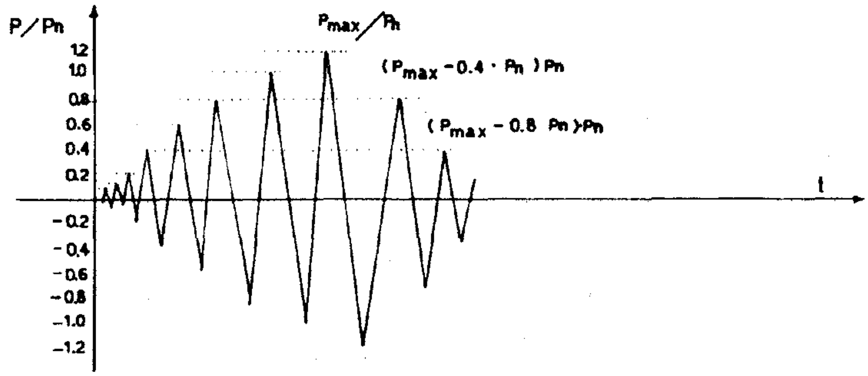


Fig.5: Loading history.

Displacements at the top and bottom of the beam, at the middle of the column (targets in Figure 4), were measured to assess interstory drift and joint rotation by means of a theodolite looking at the targets (precision reached is about 0.05 millimeters). Damages at each loading level have been recorded by a set of 4 slides (for the 4 corners) as well as by a step-by-step motion picture.

TESTS PERFORMED. PRELIMINARY RESULTS

Three specimens have been tested: TO/2, TF/1, TT/2; therefore, only preliminary results are available. For the first specimen (non-infilled frame, TO/2) only one jack was applied (one way loading history). Moreover, no slides or motion pictures were obtained, (cracks and damages were just drawn). The test stopped after heavy cracks, spalling and large displacements had occurred. In Figure 6, load v/s displacement is shown. Maximum load and corresponding displacement are 3 t and 32 mm.

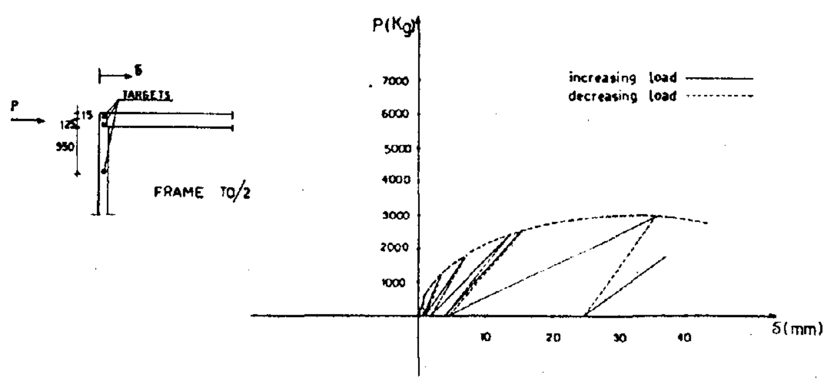


Fig.6: Results for TO/2 frame.

For the second test, (hollow tile infilled frame, TF/1), the ultimate damage state was reached when the masonry completely collapsed and heavy damage had occurred on r.c. elements. The maximum load and corresponding displacement are 7.5 t and 20 mm (Figure 7).

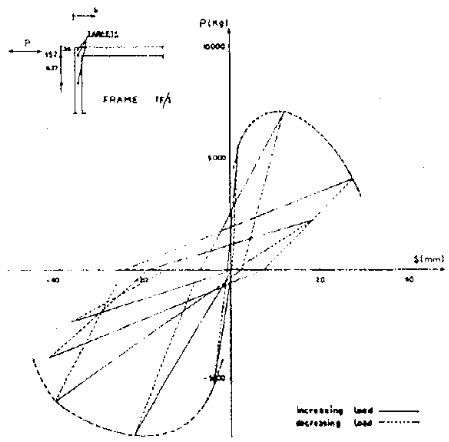


Fig.7: Results for TF/1 frame.

For the third test, (hewn conglomerate infilled frame, TT/2) the ultimate damage state was reached when the bond for beam steel reinforcement had been lost in the joint area, due to bad anchorage (straight bar with no hooks). Low damage for masonry was observed. The maximum values for load and displacement are 13 t and 17 mm (Figure 8).

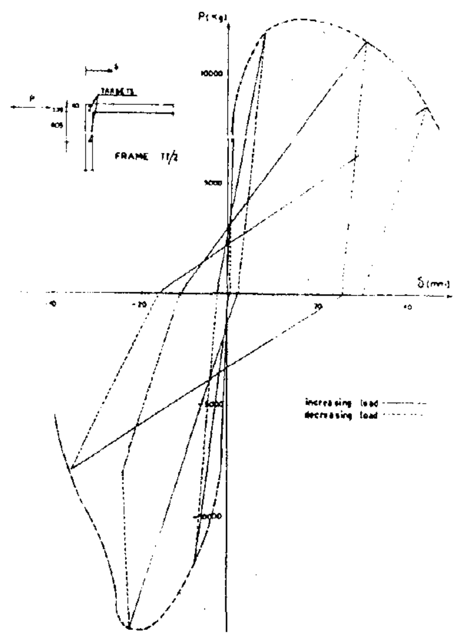


Fig.8: Results for TT/2 frame.

DAMAGE STATE

In order to obtain damage functions v/s interstory drift, a definition of damage states is necessary. According to Reference 10, damage factor DF, which represents the vulnerability for the component considered, is the ratio:

$$DF = \frac{\text{cost of repair to original conditions}}{\text{total cost for a new component}}$$

Because damages for each component are referred to two different elements (unreinforced masonry and r.c. frame), two different sets of damage states are defined:

1) R.C. frame:

DS0	: no damage	no repair
DS1	: section cracked, but steel not yielding	epoxy injection
DS2	: steel yielding, no loss of strength	epoxy injection, plus new reinforcement, plus new anchorage
DS3	: loss of strength	to be rebuilt (cost of repair greater than 100 percent of a new component)

2) Unreinforced masonry*:

DS0	: no damage	no repair
DS1	: intermediate damage state	partial reconstruction
DS2	: ultimate damage state	to be rebuilt as new

A cost of a repair can be defined only when unit costs are known and a repair technique is chosen. The techniques described above are related to damage states. Costs are defined for damage thresholds and are assumed to vary as linear functions between two following thresholds. The cost ratios data are shown in Table A.

Damage theshold ratios for r.c. frame:

	DS0	DS1	DS2	DS3
DF	0	0.14-0.17	0.42-0.51	1.10-1.15

Table A

* In Reference 10, three damage states attributed to reinforced masonry are here attributed to unreinforced masonry because it is cast in an r.c. frame.

Damage threshold ratios for masonry:

DS0	DS1	DS2
0	0.65-0.72	1.10-1.15

Table A (continued)

With these ratios an average curve damage factor v/s interstory drift for 3 specimens tested can be drawn (Figure 9). In the same Figure, curves taken from Reference 10 for r.c. and masonry elements are also shown.

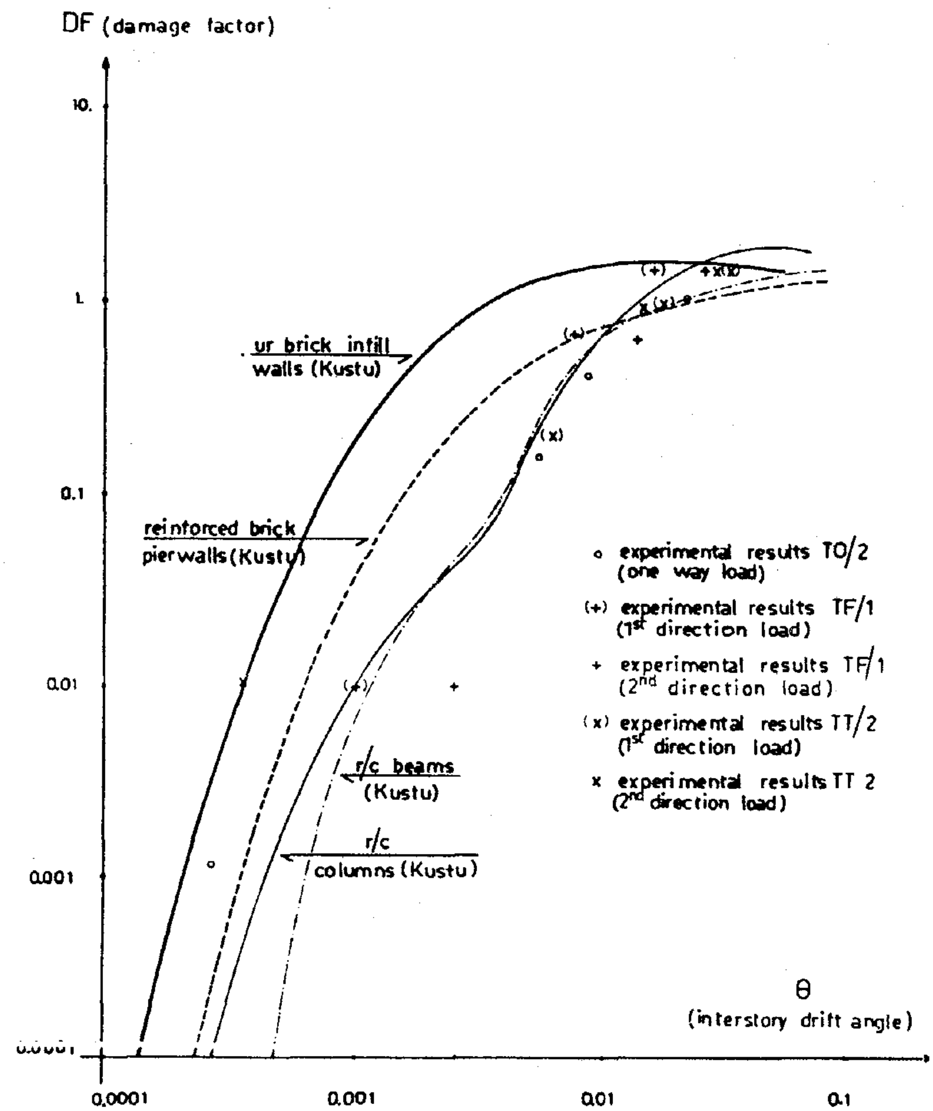


Fig.9: Experimental results for tested frames and comparison with Kustu's results

CONCLUSION

A quite good agreement with some results of Reference 10 is evident in Figure 9. Subsequent tests will show better the behavior of specimens. Specimens tested will be repaired and tested again for assessing DF/θ curves for repaired components.

ACKNOWLEDGEMENTS

A special thanks is extended to all workers of C.R.E.S.M.'s course who built specimens, and to Mssrs. L. Barbera and A. Videtta, Director and Partner of C.R.E.S.M., for their support. Mr. W. Aniballi did measurements of displacement with theodolite, Mr. A. Crocini measured loads, and Dr. A. Custodi conducted slide and motion picture photography.

REFERENCES

(1) C. Gavarini, P. Angeletti: "Vulnerabilita sismica degli edifici esistenti. Lo stato dell'arte e le prospettive di sviluppo della ricerca in Italia" - "Industria Italiana del Cemento" No.2 - Febbraio

(2) V. Petrini (coord.): "Pericolosita sismica e politica di difesa dai terremoti in Italia" - P.F. Geodinamica (C.N.R.) - Pubblicazione No. 442 - ESA Editrice - Roma 1981

(3) P.F. Geodinamica (C.N.R.): "Indagini di microzonazione sismica" - Pubblicazione No. 492 - Edizione settembre 1983 - Roma

(4) F. Braga, M. Dolce, D. Liberatore: "A statistical study on damaged buildings and an ensuing review of the M.S.K. '76 scale" - ESA Pubblicazione No. 503 - Roma

(5) M. Dolce: "Damage Statistical Matrices for Italian Low Rise Buildings" - 7th International Symposium on Earthquake Engineering - Roorkee 1982

(6) D. Benedetti, V. Petrini: "A Method for Evaluating the Seismic Vulnerability of Masonry Buildings" - L'industria delle costruzioni No. 149 - Roma, marzo 1984

(7) C. Gavarini, P. Angeletti: "Assessing Seismic Vulnerability in view of Developing Cost/Benefit Ratios for Existing R.C. Buildings in Italy" - 8th WCEE - San Francisco 1984

(8) Regione Toscana - G.N.D.T.: "Scheda per la valutazione della vulnerabilita sismica degli edifici di muratura e cemento armato (Istruzioni per la compilazione) - Firenze 1984

(9) P. Angeletti, C. Gavarini: "Un metodo di valutazione della vulnerabilita sismica per edifici esistenti di c.a. Confronti sperimentali". 2 Convegno Nazionale "L'Ingegneria Sismica in Italia". Rapallo, 1984

(10) R.E. Scholl, O. Kustu, C.L. Perry, J.M. Zanetti: "Seismic Damage Assessment for High-Rise Buildings" URS/Blume - San Francisco 1982

INTENTIONALLY BLANK

Seismic Upgrading of Irregular Reinforced Concrete Buildings

G. Augusti (I), A. Vignoli (II)

SUMMARY

A correct seismic upgrading of a building, rather than strengthening all its members, requires the analysis of its response to a seismic input and the elimination of the most dangerous aspects, among which the torsional motions play a most prominent role.

This paper summarizes two case studies, in which the torsional components of motions were eliminated from the first two modes of oscillations of two non-symmetric r.c. residential buildings, while maintaining the stresses under standard seismic actions within admissable limits. In the first case, shear walls or bracings were inserted into an existing structure; in the second case, appropriate modifications of the original design were studied (1), (2), (3), (4). Details of buildings and calculations are not included in the present paper.

FIRST BUILDING

The first building examined is a reinforced concrete 4-storey residential building of Solofra (Avellino), which was diffusedly but not heavily damaged by the Southern Italy earthquake of 23 November 1980, which in Solofra reached MM 8 intensity (1), (2).

The C-shaped plan of the building (Figure 1) more than justified the damage. In (1) the introduction of bracings such to eliminate torsional components of free motion was studied. Three types of bracings were taken into consideration, as shown in Figure 2.

The dynamic analysis of the building, with and without bracings, was performed by means of the well known TABS-77 computer program, run on the Honeywell DPS7 of the Computing Center of the University of Florence. Different soil compliances were assumed, limiting for simplicity the analysis to a Winkler sub-soil. Investigating by trial-and-error among bracing plans subject to architectural constraints related to the use of the building, the bracing plans shown in Figure 3 were obtained as typical solutions of the set problem: remarkable differences were obtained for rigid soil ($K=\infty$) and deformable soil ($K=10$ kg.cm^{-3}).

Figure 4 compares the percent decrease of bending moments caused by horizontal static forces in the columns of four structural frames when bracing of type (A) or (B) are introduced with the respective plan of

(I) Professor of Structural Engineering, University of Florence, Italy

(II) Asst. Professor of Structural Engineering, University of Florence, Italy

the first row of Figure 3, and the building is founded on a Winkler soil with compliance coefficient K-5 kg.cm^{-3} or K=15 kg.cm^{-3}, or on a rigid soil (K-oo). Inspection of Figure 4 immediately shows that the stress relief in the columns is greatly reduced by the soil compliance, and in some cases (see especially Frame No.7) the column moments are increased rather than decreased. In other words, the bracings designed for rigid foundation soil are not efficient on a deformable soil.

The plans shown in the second row of Figure 3 eliminate torsional components from the first two modes of oscillation in this case assuming K=10 kg.cm^{-3} (2), (3). Figure 5 refers to these bracing plans and compares the reductions in bending moments in the columns at each storey. In particular, Figure 5-0 compares the three bracing types, while the other diagrams (Figure 5-A, 5-B, 5-C) investigate, for each bracing type, the sensitivity to variations in the value of K, which is generally low, at least in the range of 5-15 kg. cm^{-3}.

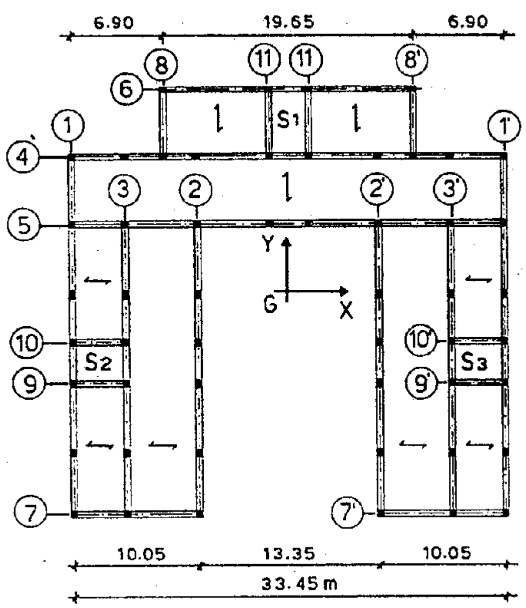


Fig.1: First case Study: plan and structural frames.

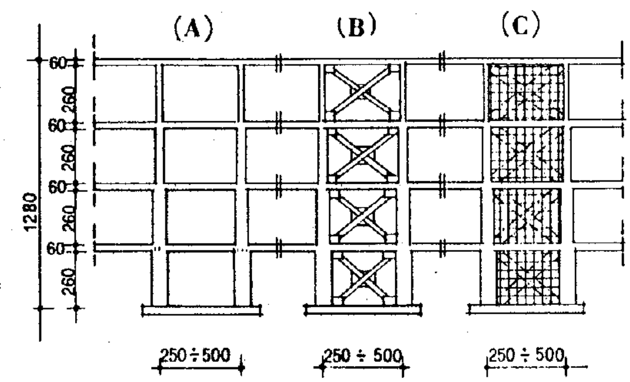


Fig.2: Bracing types investigated:
(A) r.c. shear walls;
(B) steel X-bracings;
(C) reinforced brick walls.

SECOND BUILDING

The second case study (3), (4) refers to a 6-storey residential building in the city of Arezzo, 80 Km. southeast of Florence. No seismic provisions were included in its design, because only in 1982 Arezzo was included in the 2.d category (S-9) seismic zone: in particular, following a frequent practice in Italy, the frame beams are as deep as the

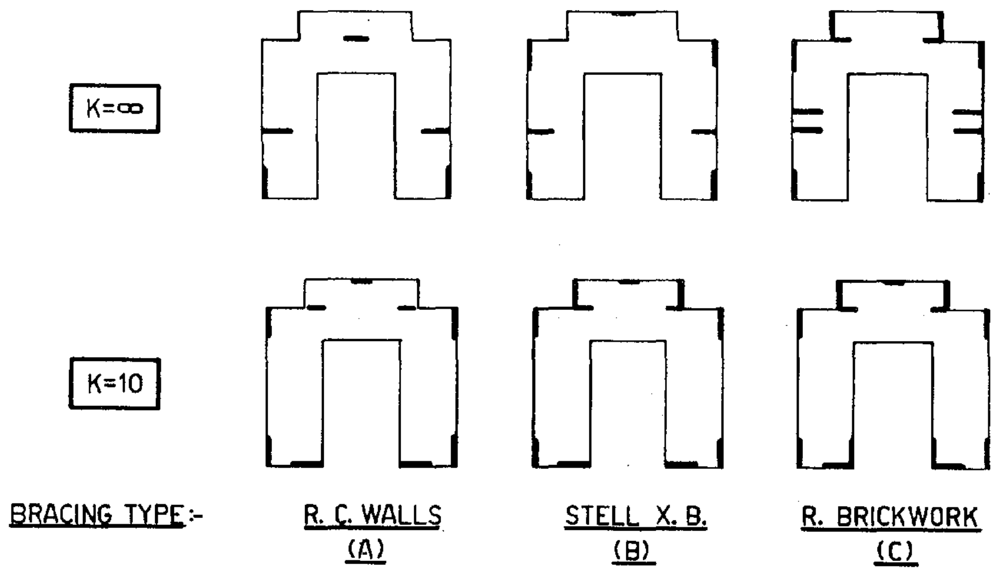


Fig.3: Bracing plans eliminating torsional components of free motion in case of rigid and deformable subsoil.

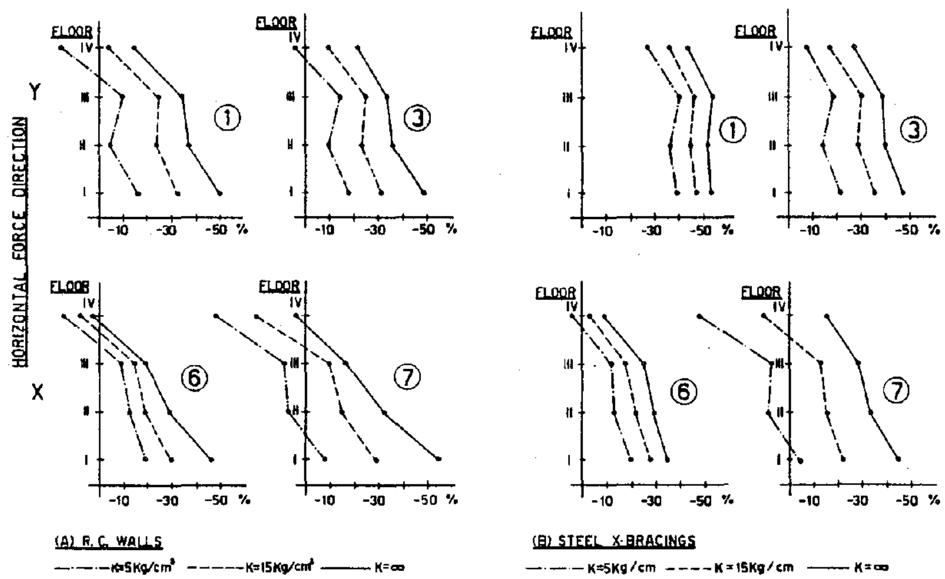


Fig.4: Percent reduction of sum of column bending moments in frames 1, 3, 6, 7 (cf. Figure 1) due to bracings with the plans of row K=oo in Figure 3. Comparison of the cases of rigid and deformable subsoil.

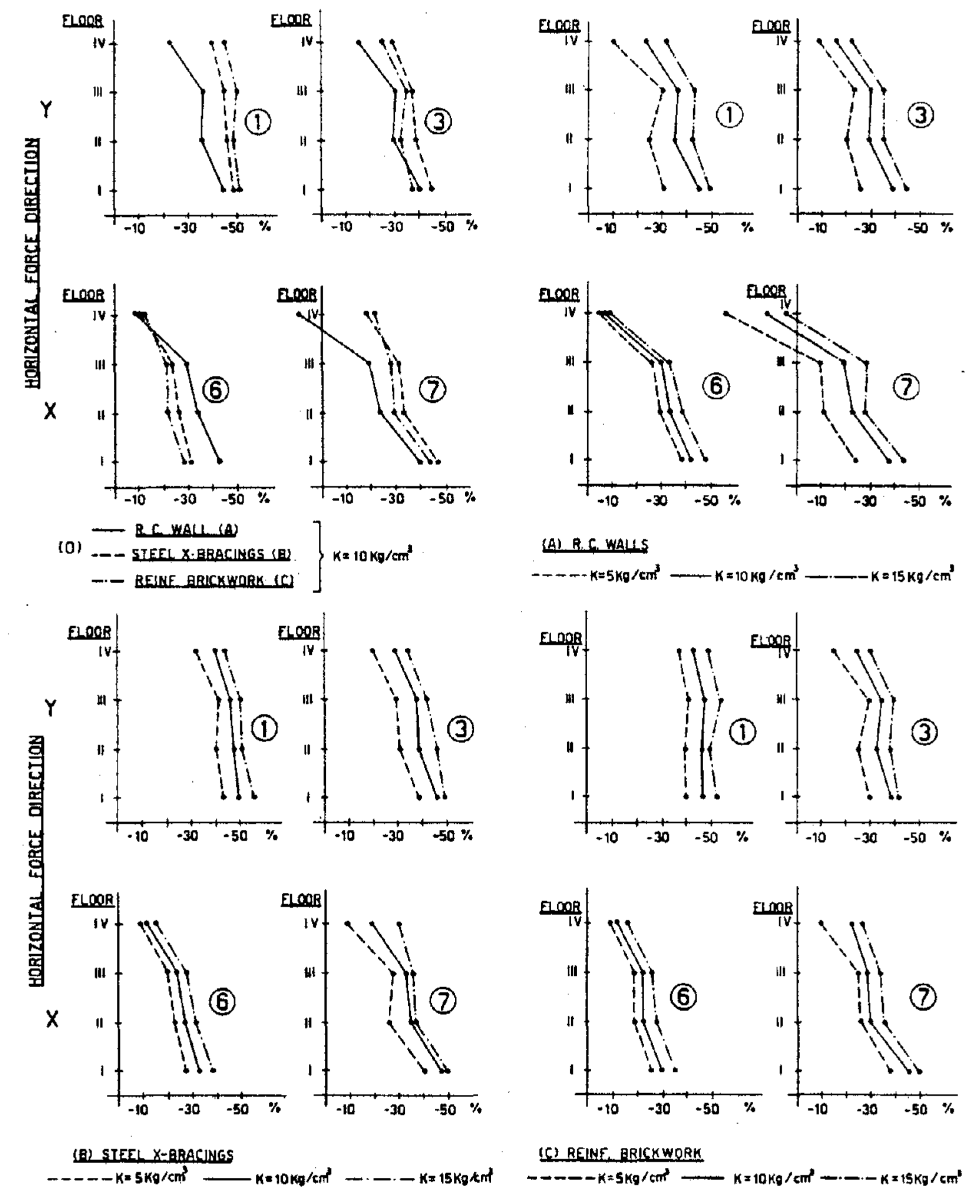


Fig.5: Percent reduction of sum of column bending moments in frames 1, 3, 6, 7 due to bracings with the plans of row K=10 in Figure 3. (0) Comparison of effects of different bracing types; (A) (B) (C) comparison of three soil compliances.

floors (22 cm structural section depth). The dynamic behavior of this building has been investigated by a specially developed computer program (5), which is more suited than TABS for small computers and, moreover, takes account of torsional and warping stiffness of vertical members. Calculations have been performed on the HP-1000 computer of the Department of Civil Engineering. Rigid constraints were assumed at the bottom of the columns, while a deformability equivalent to a soil compliance coefficient $K=10$ kg.cm^{-3} was assumed for the stair walls and the box and slab elements.

Note that the layout of this building (Figure 6) appears much better than the previous one from the viewpoint of seismic response. Nevertheless, due to the elongated shape and the eccentric staircase, very significant torsional components are present in the first two modes of oscillation (Figure 7-0).

The generally low stiffnesses caused a rather high fundamental period (T_1=1.23 sec), however below the limit for compulsory dynamic analysis set by the Italian Seismic Regulations (T_1=1.40 sec); but the inadequacy of static analysis for this building has been exhaustively proved in (4).

In this case, rather than studying the retrofitting of the existing building, it has been investigated which modifications could have been introduced in the original design, without altering the structural layout, in order to improve its seismic response and in particular to eliminate torsional components from the first two modes of oscillation, and at the same time to keep the stresses below admissable limits when the building is subjected to the horizontal static forces prescribed for the S=9 Italian seismic zone (0.07 times the gravity loads). To this aim, the original design was modified in the following ways, as diagrammatically indicated in Figure 7 (1 to 5):

Trial No.1: a) Transverse floor-incorporated beams, framing into the columns, added to the original structure; b) end columns displaced

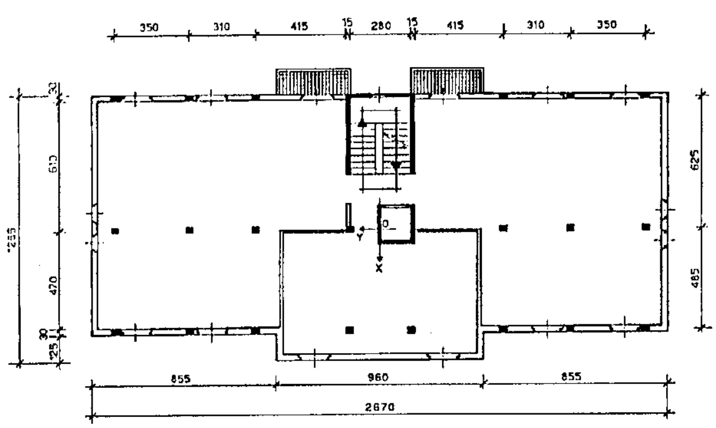


Fig.6: Second case study: overall plan and column layout.

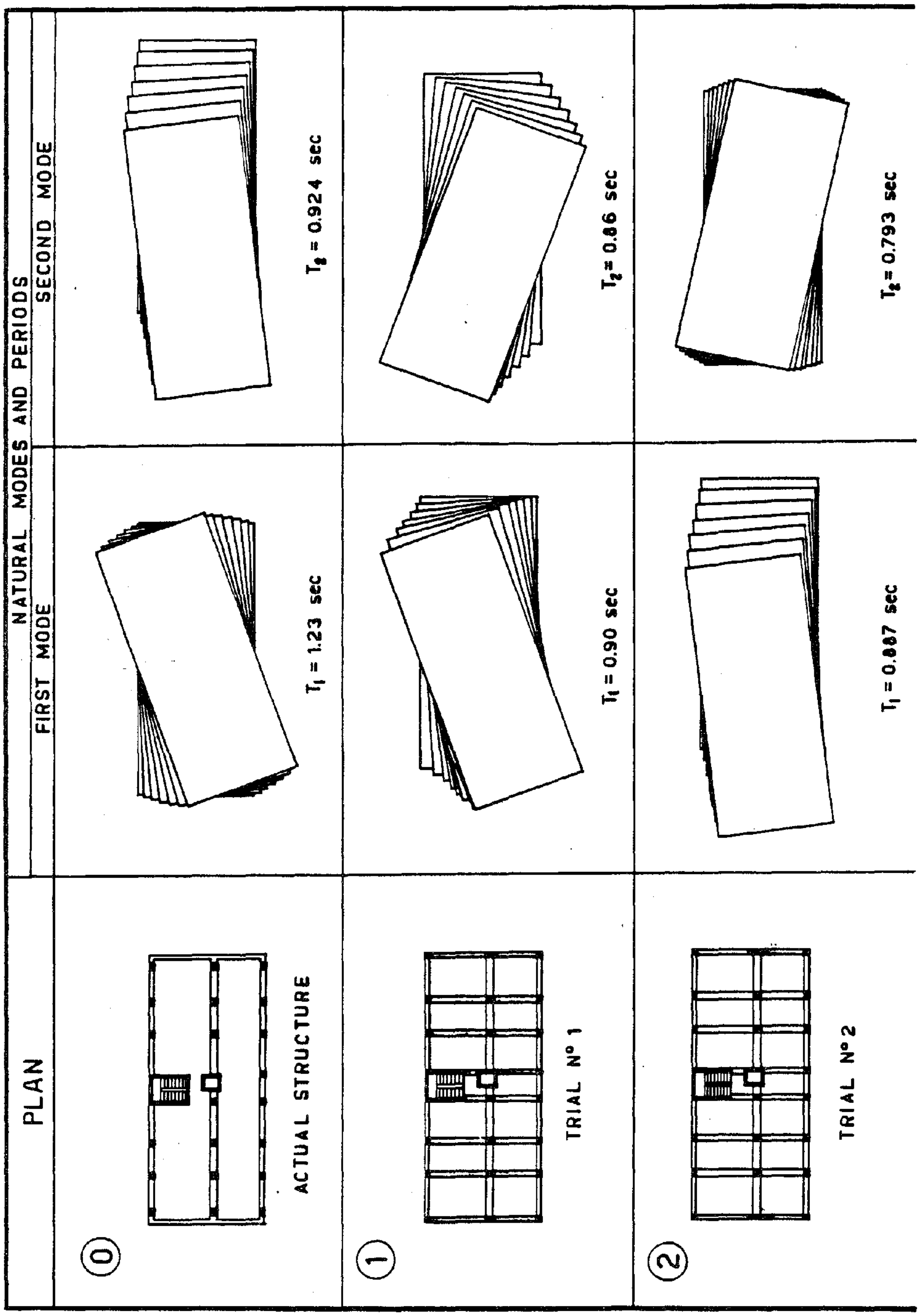
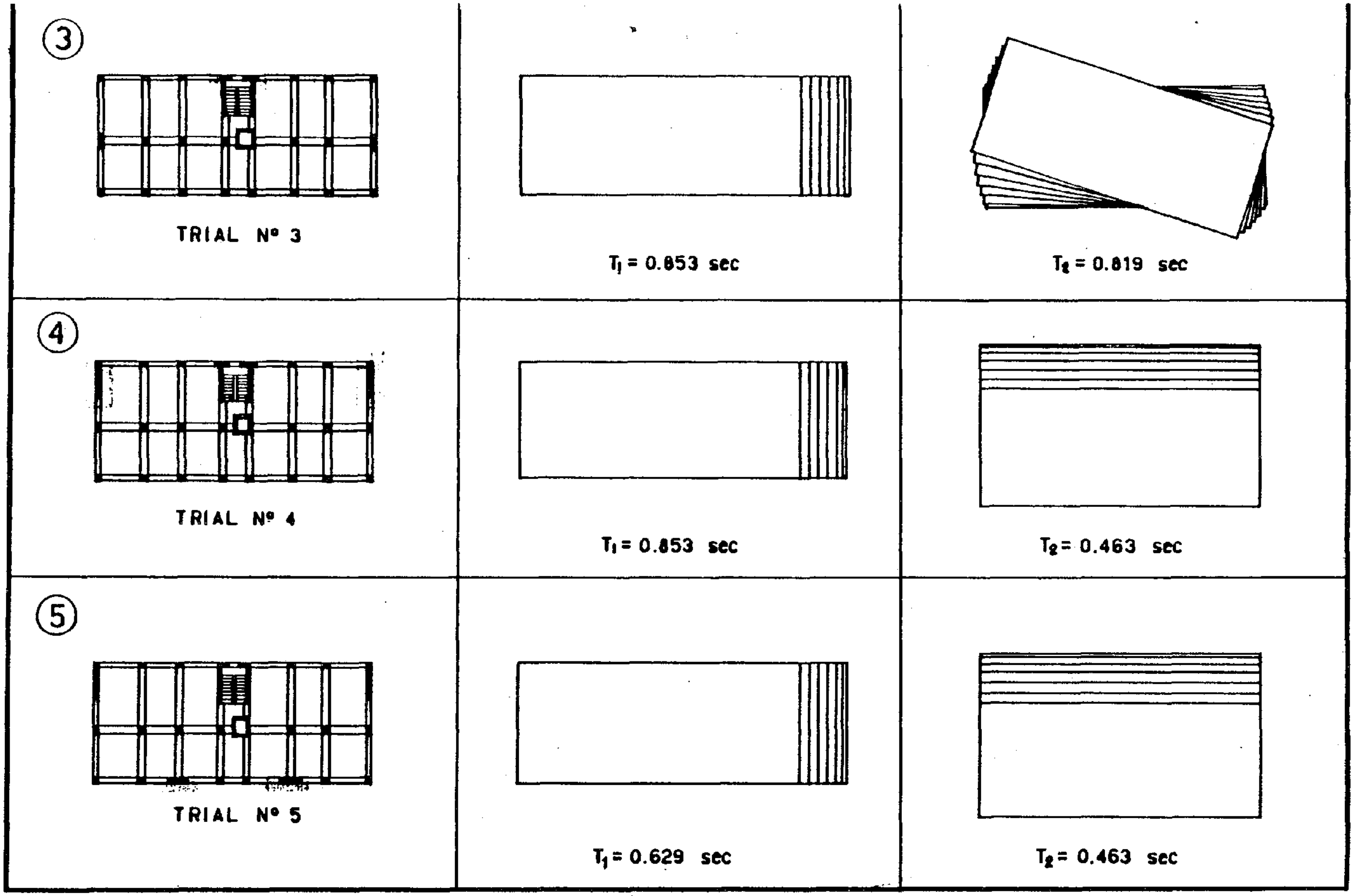


Fig. 7: Modes and periods of free oscillation: (0) original structure; (1-5) strengthened structures as described in the text.



3
TRIAL Nº 3
$T_1 = 0.853$ sec
$T_2 = 0.819$ sec
4
TRIAL Nº 4
$T_1 = 0.853$ sec
$T_2 = 0.463$ sec
5
TRIAL Nº 5
$T_1 = 0.629$ sec
$T_2 = 0.463$ sec

to the outside wall plane; c) increase to 30cm of the depth of the beam all around the building perimeter;

Trial No.2: a), b), c) as above; d) section of the central end columns increased from 30x40 (or 30x30) cm to 30x100 cm;

Trial No.3: a), b), c), d) as above; e) thickness of the staircase outside slabs increased from 15 to 30 cm;

Trial No.4: a), b), c), e) as above; no d); f) two eccentric end slabs, 30 cm thick and 300 cm deep, added.

At this stage, torsional components had been eliminated, but the building was too deformable in the longitudinal direction, and the stresses were too high. Therefore, a further solution was tried:

Trial No.5: a), b), c), e), f) as above; no d); g) two longitudinal slabs 30x200 cm added; h) the two staircases outside the slabs transformed into a single wall by deep horizontal beams (Figure 8).

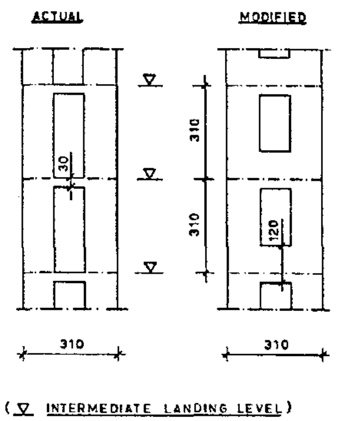


Fig.8: Staircase wall.

CONCLUSIONS

The studies briefly reported in this paper, and others still in progress, show that small modifications can be included at a small extra cost in the original building design and greatly improve its seismic response, even in the cases in which functional and/or aesthetic reasons suggest an irregular, unbalanced layout. Existing buildings can be retrofitted in a similar way by shear walls or steel bracings.

ACKNOWLEDGEMENTS

This research is part of a continuing research program of the Department of Civil Engineering, University of Florence, supported by the Italian National Group for Earthquake Effect Reduction (Gruppo Nazionale per la Difesa dai terremoti, CNR), and by the Association of Public

Housing Institutions of Tuscany (Consorzio Regionale degli IACP della Toscana).

REFERENCES

(1) A. Vignoli, G. Augusti, A. Chiarugi: "Dynamic behavior and Seismic Upgrading of Reinforced Concrete Public Housing Buildings", 7. th E.C.E.E., Athens, Sept. 1982, Vol. 4, pp. 103-110.

(2) A. Vignoli, M. Vivoli: "Comportamento di un edificio in c.a. durante il sisma del 23.11.80 e proposte per il suo adeguamento", Giornale del Genio Civile, Rome, Vol. 121, No. 7/8/9, 1983, pp. 293-308.

(3) A. Vignoli, G. Augusti, A. Chiarugi: "Seismic behavior and design of non-symmetric buildings", 8.th W.C.E.E., San Francisco, July 1984.

(4) A. Vignoli: "Considerazoni ed esempi di adeguamento sismico per edifici a telaio in c.a.", Univ. of Florence, Dept. of Civil Engineering, UFIST/13/1983.

(5) A. Vignoli, M. Dragoni: "Un algoritmo di calcolo automatico per lo studio del comportamento sismico di strutture spaziali", Univ. of Florence, Dept. of Civil Engineering, UFIST/06/1983.

INTENTIONALLY BLANK

Repair and Retrofitting Work Carried Out on Privately Owned Old Property in the Sellano (Perugia) Area Affected by the November 19, 1979 Earthquake

Carlo Bientinesi (I)

SUMMARY

In this report reference is made to a series of technical-administrative services in the repair and retrofit of older, existing buildings provided by the Chiaromondo Cooperative since 1980 in the Commune of Sellano, Perugia, which was damaged during the earthquake of November 19, 1979. Sellano is part of the Valnerina District, located in the southeastern part of the region, which also includes the Communes of Norcia and Cascia. This district is a mountainous area; it is very big and sparsely populated (15,000 inhabitants scattered in hundreds of small villages). Due to the precarious economic situation, very little has been spent over the years on the upkeep and preservation of the 30,000 rooms, over half of which were built before 1940, and which form the district's architectural heritage. The intensity and duration of the 1979 earthquake were considerable. This was the last in a long series of earthquakes which had affected this area over the preceding 20 years. It damaged buildings which were of great value as far as their homogeneous character and architectonic qualities were concerned, but they were also delapidated and of poor quality from a technological point of view and often lacking in basic sanitary facilities.

INTRODUCTION

Two new factors were introduced into the legislative provisions drawn up by the state and the Region for rehabilitation of urban areas damaged by earthquakes. These two factors are:

1) the admissibility of a capital account grant to be used not only for repairing damage but also to carry out consolidation work and provide antiseismic protection as well as hygienic-sanitary improvements;

2) possibility for the Communes to draw up an Urban Plan for the organization and coordination of repair work. Uniformity must be guaranteed also for repair work carried out by individual owners on buildings not included in the Urban Plan. In the context of the Urban Plan, the Communes themselves were required to provide the executive plans and to estimate the grant needed.

The Commune of Sellano followed the instructions issued by the Region and established two phases for setting up an urbanistic-normative framework to address the problem:

(I) Representative, CHIAROMONDO, Soc. Cooperativa di Engineering

1) Study of the Urban Zones;

2) Drawing Up of an Urban Plan.

PLANNING AND MAPPING METHODS

After a brief survey and on the basis of data collected on building and demographic concentration in the area, 15 built-up areas were singled out (Sellano has 40) whose total area is equal to about 33 hectares. During the first three months of 1981, a physical inspection was carried out using specially prepared forms organized in the following way:

Form No.1: Completed for each building and includes data on type, solidity, building age, the name of the owner, and the general state of preservation of the building.

Form No.2: Refers to each single household. It describes the characteristics, the qualitative conditions, whether it is owner-occupied or rented, the socio-economic situation of the occupants. It also establishes whether the grant has been applied for or not, whether the owner intends to carry out repair work himself or not, what improvements are needed in the house according to the owner and the surveyors.

Form No.3: Deals with the building. It analyzes its characteristics and the state of internal and external structural elements and finishings. When the type and state of both horizontal and vertical structures have been defined, a framework emerges which enables us to define, with sufficient accuracy, the typological and technological quality of the building. This is an essential premise to any type of repair work.

After these forms were examined, we were able to draw up a number of tables, with the last one showing the areas to be included in the Urban Plan. Precedence must obviously be given to the cases in which more than one negative phenomenon coexist in the same building. In order to establish this immediately, all the shaded areas which indicate each single phenomenon were superimposed on the same part. In this way, the more heavily shaded areas were those in the worst condition and, therefore, they would have to be repaired within the context of the Urban Plan.

The following determining factors were chosen from all those which emerged:

a) Property owned by a number of different persons.

b) Application for L.R. 50

c) Overcrowding in houses

d) Poor static structural conditions

e) Poor hygienic conditions

This method was applied to the 15 villages and it led to the establishment of 24 Urban Plans, equal to a total of 16 hectares which, in turn, equals less than half the area examined in the survey. Even though the use of the Urban Plan had to be limited, the need to achieve uniformity in repair work carried out on buildings which were included in the Urban Plan, and on the others which were not, could not be ignored. It had to be certified that adequate quality levels were being obtained for both types of repairs as far as static repairs and the preservation of the formal technological and typological features of existing structures were concerned. To achieve this objective, ready-to-use forms were compiled, using existing ones as examples, which could be used directly by workmen carrying out repairs on historical buildings. In this way the person who is working on the spot has not only criteria but also models available. These are often very simple and banal, but they are appropriate and they represent a minimum quality level. At this stage, it is important to reconstruct the overall picture and to supply quantitative data concerning work which has been carried out on private property whose damage had been assessed previously.

<u>Phase I</u> - <u>Study of the Urban Zones</u>

Surface area concerned:	33 hectares
Population concerned:	959 persons (66% of Commune population)
Households assessed:	590

<u>Phase 2</u> - <u>Drawing Up of the Urban Plan</u>

Number of Urban Plans:	24
Overall surface area:	16 hectares
Overall cubic volume:	400,000 m^3
Number of applications from the Commune:	about 450 (50%)
Blocks designed:	220

<u>Phase 3</u> - <u>Estimation of Damage</u>

Blocks designed:	200
Overall cost of repair work:	21 billion Lire
Average cost of retrofitting per cubic meter:	85,000 - 120,000 L/m^3

Total estimated cost assessed for both the Urban Plan zone and work outside it:	45 billion Lire

Phase 4 - State of Reconstruction Work

Work sites set up:	about 65
Cost of work let out on contract:	about 8,000,000,000 Lire
Grant awarded:	about 5,900,000,000 Lire

CONCLUSION

Completion of work on the sites, which were set up initially, has enabled us to calculate the average cost of repair work, and these data are indicated in Table 1. (It must be noted, however, that these contractors have been willing to carry out repair work at low prices in view of the present difficult economic situation in Italy.)

Names of the principals and collaborators on this project are presented in Appendix A.

APPENDIX A

"CHIAROMONDO" Soc. Coop. di Engineering
C.so del Popolo, 47 - 05100 TERNI
Tel. 0744/56849

Principals

Eng. Federico Sotgiu - Chairman (ex)
Now Vice Chairman of CICP
Coordination

Arch. Moreno Ciavattini
Coordination
Drawing Up of Urban Plans
Design of Public Works
Version of Master Urban Plan

Arch. Carlo Bientinesi - Chairman
Drawing Up of Urban Plans
Design of Public Works

Arch. Paolo Stefanini
Drawing Up of Urban Plans

Eng. Alessandro Mazzei
Design of Public Works
Design of Carrying Structures
Direction of Public Works

Eng. Alberto Custodi
Design of Carrying Structures
Computer Technical Manager

Geom. Gianni Paoli
Damage Survey
Grant Examination

Geom. Roberto Cellamare
Graphics, Rendering

M.A. Stefania Gentili
Graphics, Rendering

Collaborators

M.A. Piergiorgio Agri
M.A. Stefania Amadei
M.A. Antonietta Aniballi
Geom. Walter Aniballi
Geom. Roberto Belinci
Geom. Claudio Berretti
M.A. Cristina Biscioni
Stud. Patrizia Campili
Geom. Paola Careddu
Geom. Marco Donatelli
Ing. Vincenzo Fattorini
Geom. Antonio Fortunati
Geom. Camillo Leonardi
Geom. Maurizio Leonelli
Geom. Fabrizio Luciani
Rag. Alessandra Luzzi
M.A. Loretta Onofri
Geom. Stefano Pioli
Geom. Fabrizio Porchetti
Geom. Ermanno Procida
Geom. Giuseppina Proietti
Geom. Stefano Proietti
Stud. Daniela Ricci
Stud. Domenico Ripanti
Paola Testa

INTENTIONALLY BLANK

Repair and Retrofit of Existing Buildings: Introductory Report

F. Braga (I)

SUMMARY

Repairing and retrofitting buildings is a typical problem in countries, such as Italy, with widespread seismicity and an ancient civilization. This problem, however, is also becoming important to California, the most seismic area in the U.S., where many existing buildings were constructed without any seismic code at the beginning of this century. These considerations explain the opportuneness of this workshop and of its location in Rome.

The present report aims at separating out in a systematic way the main topics of the problem and, at the same time, at providing an hypothesis for a joint U.S. - Italy approach. In particular, the main subjects to be dealt with and the relevant researchers will be singled out, and some differences of interest will be emphasized.

INTRODUCTION

Great attention has been paid recently by earthquake engineers to the problem of repairing and retrofitting existing buildings and, in fact, an entire session was devoted to it in the latest Earthquake Engineering Conference.

This problem obviously is of paramount importance in a country, such as Italy, where nearly all the territory is subjected to seismic hazard, and where built-up areas are often quite old. Unfortunately, the size of the problem (1 million buildings to retrofit) is such that the single cases examined to date (Refs. 1, 2) are of little significance, and more extensive studies are therefore required. On the other hand, at least until some years ago, little attention has been paid to this kind of problem in countries, such as the U.S. and Japan, which are very advanced in the field of earthquake engineering. This situation makes the solution of such a problem very difficult at the present time, and much effort will be required in order to obtain really satisfactory solutions.

The present workshop, then, provides a precious opportunity to make a state-of-the-art contribution by comparing experiences relevant to such different situations as the Italian and U.S. ones, and by identifying the subjects of common interest and the fields where the research is not yet sufficiently developed.

From the Italian point of view, it could turn out to be a good occasion to stimulate international interest and attention to the problem, and to discuss a typical Italian problem with new interlocutors of wide

(I) Asst. Professor of Earthquake Engineering, University of Rome

experience. From the American point of view, it can be an occasion to understand better the Italian reality, and to treat a subject of probable future interest to the American people.

Among the various, possible aims of a workshop, two are of particular interest: the direct exchange of methodologies and know-how, and the comparison of different situations. Given the very strong differences existing between Italy and the U.S. in terms of seismic hazards, building typologies, administrative organization, monetary resources, etc., a direct exchange of methodologies does not appear practicable. Rather than searching for common methodologies, it seems more appropriate

* to attempt together a collective review of the problem, through a list of the arguments to be dealt with; and

* to favor a comparison, on the same topics, between two different situations, in order to make use of the experiences in one country by adapting them to the actual situation of the other one.

The present report attempts to address these objectives. In particular, the general problems that arise in the Italian situation will be examined, and the U.S. colleagues will complete the analysis by explaining the problems relevant to the situation in the U.S.A. The workshop will not turn out to be a mere listing of possible techniques of repair and retrofit; rather, the decision problems to be faced before operating will also be given an important place.

POSITION OF THE PROBLEM

The general organization of the problem of repair and retrofit can be schematized as shown in Figure 1. As will be seen, the problem presents three levels:

Level 1: Identification of the structural typologies (by this term is meant a class of buildings which have a statistically homogeneous and well-defined behavior under seismic conditions.

Level 2: Evaluation of the vulnerability of such typologies relative to seismic intensity, and identification of the deficiencies and typical damages of each typology.

Level 3: Evaluation of cost, priority and value which, suitably integrated, will lead to the definition of an organic policy of repairing and retrofitting.

In order to better understand the meaning of each cell, the organization scheme is repeated in Figure 2 by locating the various researchers attending this workshop in the cell which, at least to the knowledge of the writer, seems to be covered by their works. As will be seen, most of those present are at Level 2, while the cells of Levels 1 and 3 are

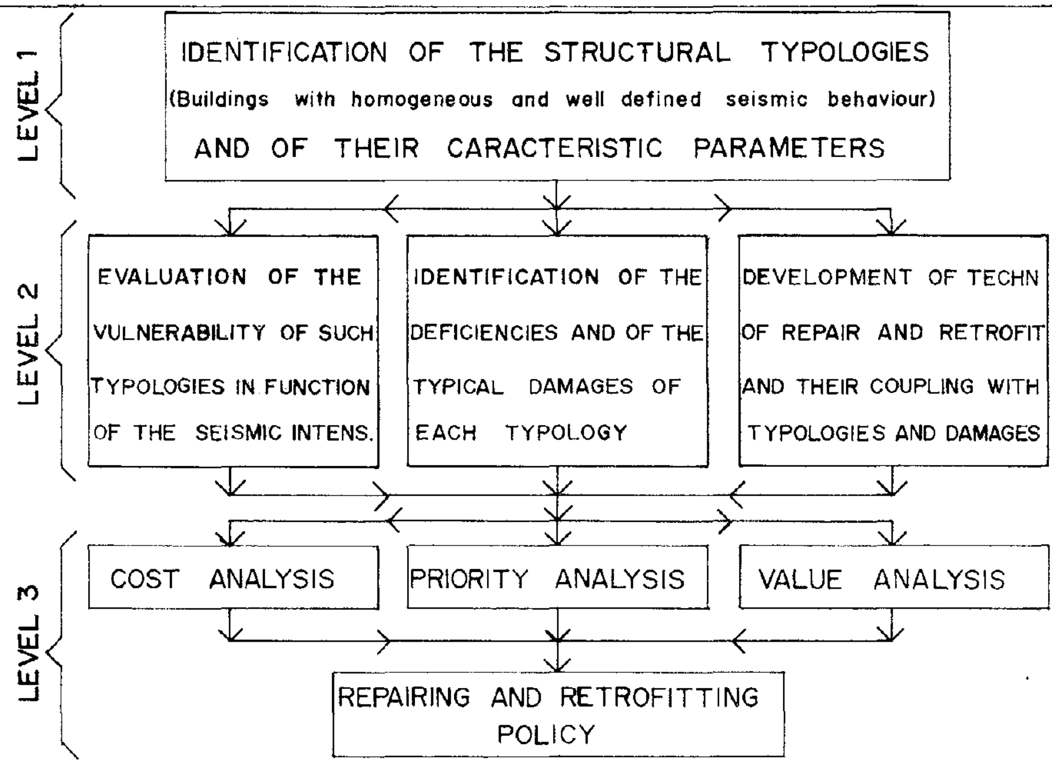


Fig. 1 — General organisation of the problem.

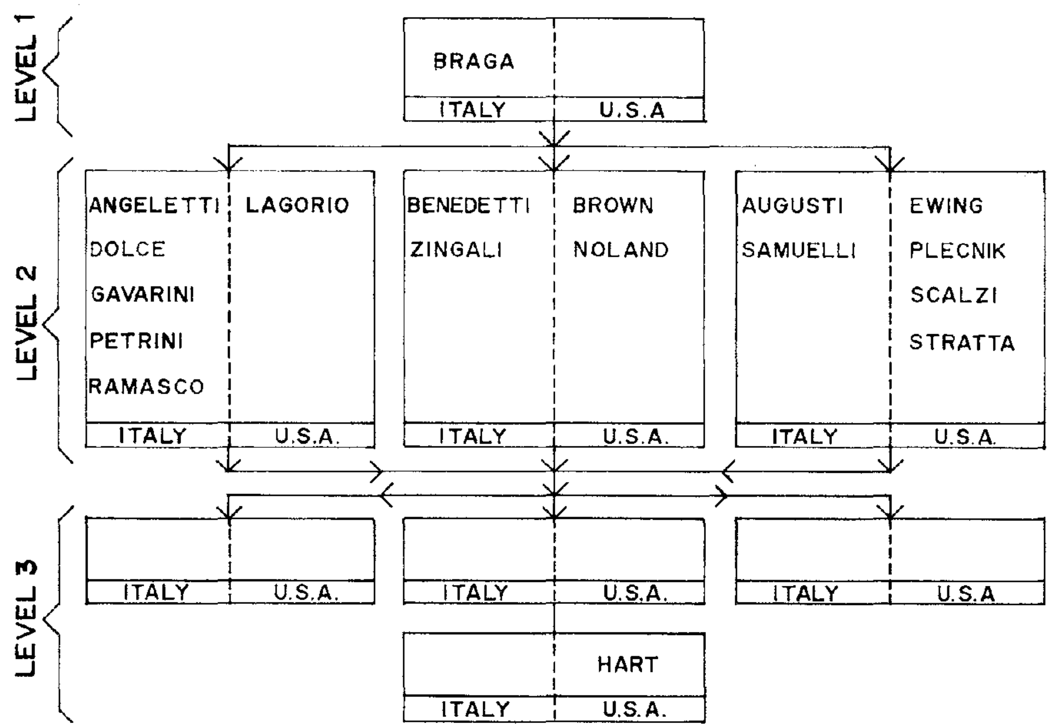


Fig. 2 — Location of the various researchers.

practically empty.

This situation is not to be attributed to errors in the organization of the workshop; rather, it reflects the international scientific situation.

Before speaking about Levels 1 and 2, it appears opportune to speak of Level 3. The latter is, if fact, the level of the "policy" of repairing and retrofitting, and it is therefore at this level that the most significant decisions are made.

Level 3

The overall quality of the repair and retrofit operations, which is reflected in the modifications and improvements of the quality of life in the earthquake-prone built-up areas, depends strongly on the quality of the choices made at Level 3.

An example can help to understand better what is meant by the three evaluations stated above. Given a certain earthquake-damaged village and a certain allocation of funds to repair, which is usually far lower than what is really needed, it is necessary to evaluate:

a) what the costs of the various techniques are for the different levels of damage (cost analysis);

b) which priorities are to be assigned to the various buildings. The necessary operations cannot be carried out at the same time on all buildings, and the available funds are never adequate to operate on all buildings (priority analysis);

c) which coefficients of importance are to be assigned to the various buildings (dwellings, office buildings, factories, museums, etc.) in order to treat them in the analyses required above as homogeneous elements (value analysis).

The evaluations described in points a, b and c are almost never made at present, owing to several reasons such as:

* the lack of an exhaustive and universally accepted data base, (an example of the minimum data base requested is given in Figure 3);

* the complexity of the necessary mathematical handling;

* the need -- especially in value analysis -- for expertise other than that relevant to the fields of Engineering and Architecture, and which is often lacking in organic mathematical bases.

The problems are large and complex. The only way they can be solved is to proceed step-by-step. It is hoped that people attending the workshop will take into consideration the previously listed problems and

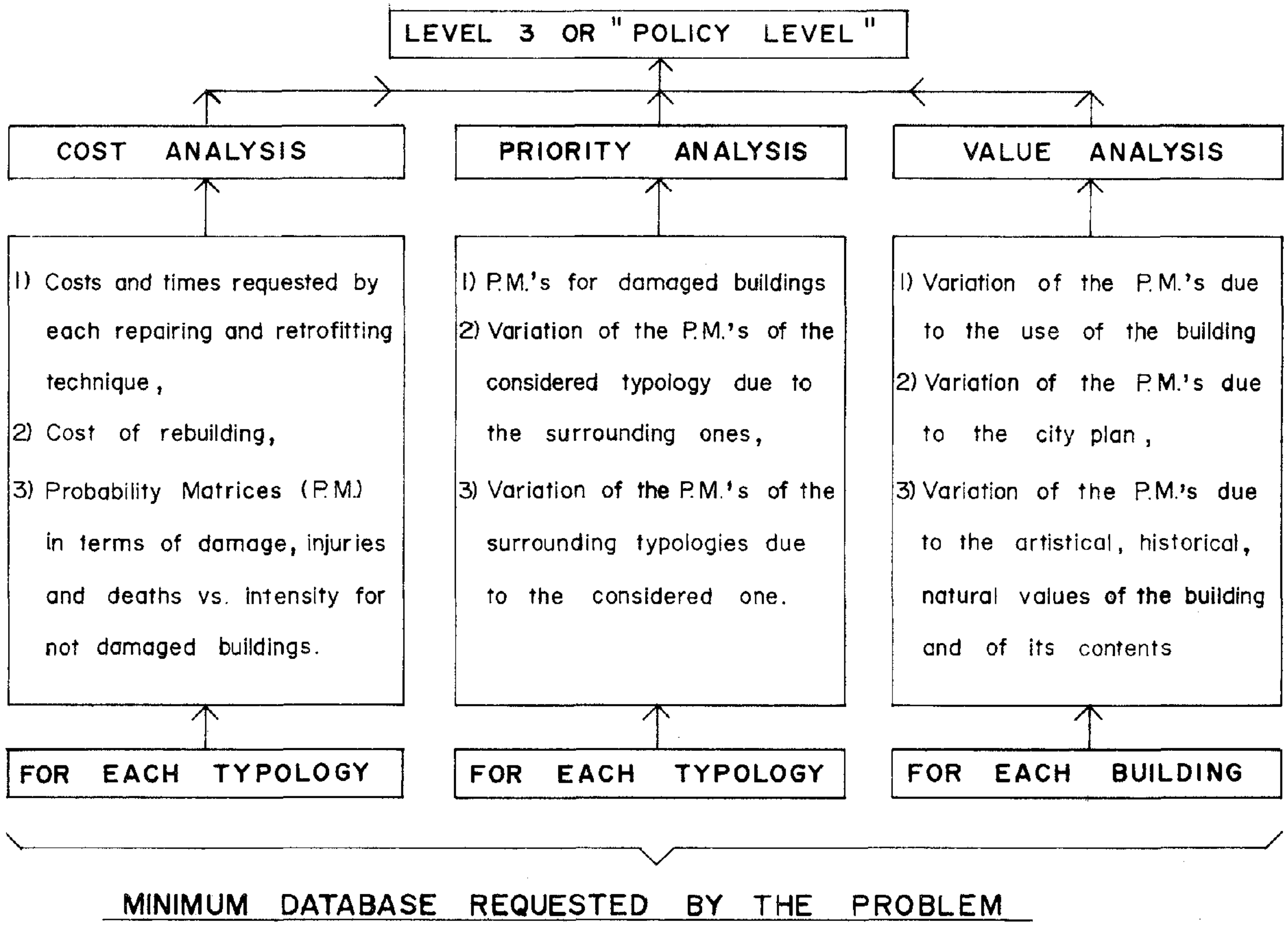


Fig. 3 — Database requested by level 3 of the problem.

TO IMPROVE THE LEVEL 3 QUALITY

SPEAKING OF	GIVE, IF POSSIBLE, DATA ON	IN SUCH A WAY TO TREAT
VULNERABILITY	a) Sensitivity, if any, of the typology considered to the aggregation with similar or different typologies; b) Influence of the urban plan on the vulnerability; c) Typical damages that buldings of the typology considered cause to nearby buildings of other typologies.	the building not only as a single object but also as an element of a complex urban organism.
REPAIR AND RETROFIT TECHNIQUES	1) Typologies to which the technique is more apt; 2) Damage which better eliminates: 3) Damage requested to make convenient its use, 4) Its unitary costs; 5) Its unitary times;	the repair and retrofit operation as remedies which vary from case to case depending on the typology and the damage level of the considered building.

Fig. 4 — Information requested to improve level 3.

provide observations able to solve some of them. The contributions required to improve knowledge at Level 3 are shown in Figure 4. For example, when speaking of vulnerability, it is appropriate to treat the single building not only as an isolated objected but also as a part of a built-up area. In this respect, the variations in vulnerability due to the presence of nearby buildings, and the variations caused to the nearby buildings, should be put in evidence along with, more generally, the variations produced by different urban plans.

For example, as far as repair techniques are concerned, it will be useful to put in evidence the peculiarities of the operation, and to specify the typology to which it can be applied, in order to understand the relation between technique and typology. It is also recommended that information be given about costs and times required for the operations, about level and kind of damage for which an operation is best employed, and about the damage produced by the operations, in order to make the decision between repairing/reinforcing and rebuilding more rational.

Finally, notices, criteria and methodologies which lead to the evaluation of the values to be assigned to the various buildings, especially historical and artistic buildings, would be of particular interest to Italian researchers.

Levels 1 and 2

As stated above, Levels 1 and 2 are sufficiently consolidated and dealt with both in Italy and the U.S. Rather than examine the problem, as in the case of Level 3, it is more opportune to treat the ongoing Italian research presented at this workshop.

Regarding Level 1 -- the identification of the structural typologies with the same vulnerability -- the actual trend in Italy is to make further subdivisions inside the three traditional typologies (Masonry, R/C, Steel). This trend is particularly marked for masonry structures which are by far the most numerous and vulnerable in Italy. To better define the typologies, the effects of the various materials and local techniques should be studied in detail. A large body of experimental work is needed, and it is only at a beginning stage in Italy at present.

An attempt to define typologies of masonry buildings has been made by myself in collaboration with Drs. Dolce and Liberatore. This work put in evidence the greater influence of the type of horizontal structure, with respect to the type of vertical structure, on the seismic behavior and, therefore, on the definition of masonry typologies (Refs. 3, 4).

Many researchers deal at present with Level 2. This is to be attributed to the increasing interest in vulnerability estimations. Such an interest has been brought on by the last seismic reclassification law of the Italian territory which described as seismic many zones which were not previously identified as such. Therefore, many regions have to face the problem of evaluating the seismic risk of buildings lying in such zones. Many of the Italian researchers present here deal with this problem, especially Angeletti, Benedetti, Braga, Dolce, Gavarini, Petrini, and Ramasco. The data needed for the evaluation are collected by using suitable forms. Collections of data have been done in various regions, but the results are not yet available (Refs. 5, 6). In any case, the data will require some checks, at least at a numerical level, since the forms have been defined by experts through methodologies like the "Delphi Method". A calibration of the vulnerability estimations could be obtained by surveying the damage to buildings which were struck by recent earthquakes and by comparing such damage with the forecasted damage according to the vulnerability estimation.

A lot of work, again at Level 2, has also been done on repair and retrofit techniques. The reason for such interest is to be attributed to the considerable damage to existing buildings produced by earthquakes which have struck Italy since 1976. Among other researchers who are working on this topic, Benedetti (Refs. 7, 8) and Zingali, for masonry structures, and Augusti (Ref. 9) and Samuelli, for R/C structures, are to be mentioned.

The effectiveness of the techniques for masonry structures is confirmed by several experiences, and they were largely applied in Friuli. The effectiveness of the techniques for R/C structures is also confirmed by laboratory experiences, but their applications are still few.

The data for masonry buildings available at present are abundant (relevant to about 20,000 buildings) and are under study. When analyzed, the data on costs and times will be of great value.

Some results, which have been elaborated by firms that worked in Friuli, are at present available. Among others, there is a report of S.V.E.I. (Ref. 10) which gives costs for two different types of masonry structures.

CONCLUSIONS

From what has been said, and from what can be concluded from U.S. reports, the workshop could turn out to be an important occasion to compare experiences and to extend knowledge.

Apart from information relevant to level 3, it will be possible to compare experiences at Levels 1 and 2. Such a comparison will be useful, if made with regard both to the framework of the problem and to the single techniques of repair and retrofit. In particular, the capability of strengthening a large number of buildings in a short time and at low costs should be emphasized when comparing the various techniques. In fact, the very large number of buildings to retrofit is the main characteristic of the problem in Italy.

Finally, it is hoped that other meetings will follow this workshop and that the U.S. - Italy scientific cooperation on the problem will receive a more stable and systematic arrangement in the immediate future.

REFERENCES

(1) WAEE - Proceedings of the 7th WCEE, Session on Repair and Strengthening of Structures, Istanbul 1980.

(2) EAEE - Proceedings of the 7th ECEE, Session 9 - Repair and Strengthening of Structures and Monuments, Athens 1982.

(3) Braga F., Dolce M., Liberatore D.: "A Statistical Study on Damaged Buildings and an Ensuing Review of the M.S.K. -76 Scale " - Italian Geodynamic Project, Report N. 503, Roma 1982.

(4) Braga F., Dolce M., Liberatore D.: "Influence of Different Assumptions on the Maximum Likelihood Estimation of the Macroseismic Intensities" - Proceedings of the 4th ICASP, Firenze 1983.

(5) Gavarini C., Petrini V., Ramasco R.: "Vulnerabilita sismica del patrimonio edilizio: prime sperimentazioni" - 2 Convegno Nazionale su l'Ingegneria Sismica in Italia, Rapallo 1984.

(6) Braga F., Ramasco R.: "Valutazione su base statistica della vulnerabilita sismica degli edifici di Pozzuoli: prime elaborazioni" - 2 Convegno Nazionale su l'Ingegneria Sismica in Italia, Rapallo 1984.

(7) Benedetti D., Formis L.: "Comments on the Effectiveness and Costs of Anti-Seismic Consolidation Methods for Stone Masonry Buildings" - Costruire, N.100, Roma 1982.

(8) Benedetti D., Benzoni G.M.: "A Numerical Model for Seismic Analysis in Masonry Buildings" - to be published in Earthquake Engineering and Structural Dynamics.

(9) Augusti G., Matteuzzi M.: "Repair and Strengthening of Reinforced Concrete Structures after Seismic Damage" - Proceedings of the 7th ECEE, Athens 1982.

(10) D'Amato A.: "Antiseismic Rehabilitation of 600 Buildings in Friuli. Operative Methods Adopted and First Statistical Evaluation of Repair Costs" - S.V.E.I., Roma 1984.

INTENTIONALLY BLANK

Strength and Behavior of Retrofit Embedments in Concrete Masonry

Russell H. Brown (I)
Marshall L. Brown (II)

SUMMARY

Test results on three types of retrofit anchors are presented. Wedge, sleeve and toggle anchors were embedded in concrete masonry and tested monotonically in tension, shear, or combined tension and shear. Several anchors were found to be suitable for structural applications. Wedge and sleeve anchors worked well in grouted cores in both tension and shear. Wedge anchors worked well in shear when embedded in mortar joints. Toggle bolts were effective in ungrouted cores.

INTRODUCTION

Anchorages and embedments are frequently used in masonry as structural attachments for pipe hangers in nuclear power plants, shear wall stiffening plates, hand rails, fire escapes, etc. Brown and Whitlock (Ref. 1) reported on extensive testing of J bolts subjected to monotonic and cyclic shear and axial forces. J bolts must be installed during original wall construction or require substantial destruction of the wall for retrofit installation. The purpose of this paper is to provide test data for other forms of embedments which are suited for installation in existing concrete masonry walls.

SCOPE

The overall study of retrofit anchors of which this paper is a part includes tests on four classes of anchors: wedge, sleeve, toggle, and chemical. Anchors will be tested in both concrete masonry and brick masonry walls, however, this paper is limited to tests on concrete masonry walls. Sizes of embedments ranged from 3/8 in (10 mm) to 3/4 in (19 mm) diameter, and embedment depth was in accordance with the manufacturer's instructions. Three or more replications of each test were performed for statistical purposes. Chemical anchors are not included in the phase of the study herein reported.

(I) Professor and Head, Civil Engineering Department, Clemson University, Clemson, SC

(II) Graduate Research Assistant, Civil Engineering Department, Clemson University, Clemson, SC

The project is not yet complete, and this paper must be considered an interim report of the research in progress. It includes test results for anchors.

TEST PROGRAM

Loading Apparatus

The load apparatus was identical to that developed by Brown and Whitlock (Ref. 1) utilizing smaller load cells. Fig. 1 illustrates schematically the technique in which axial, shear, and combined axial-shear loads are applied.

The bolt being tested was installed in a 40 x 40 x 18 in (100 x 100 x 46 cm) wall specimen and was installed in a loading channel with its axis colinear with the horizontal actuator. The wall specimen was clamped by friction using a clamp jack (Fig. 1) and clamp beam which compressed the wall specimen against the reaction frame.

Load Application

In order to apply axial tension or compression to a bolt, the horizontal actuator (Fig. 1) was used. In order to achieve combined shear and axial load, only the inclined actuator was used. In order to apply direct shear, both actuators were used simultaneously. The horizontal and inclined actuators were synchronized in such a way that the horizontal actuator component was equal and opposite the horizontal component of the inclined actuator, resulting in only a vertical component of load. Using this technique, the eccentricity of the shear load was minimized or even eliminated.

Instrumentation

Load Cells - Forces in both actuators were monitored using load cells mounted in series with the actuators. In most applications, the horizontal actuator was equipped with a 10 kip (45 kN) capacity cell, the inclined actuator with a 20 kip (90 kN) cell.

Displacement Measurements - Horizontal and vertical movements of the loading channel were monitored using displacement transducers, one mounted horizontally, the other vertically.

Data Acquisition - The data acquisition system included a mini-computer with disk storage which received output from load cells and transducers. Sampling rates varied but a rate of 4.0 Hz was often used. An x-y plotter was also used to monitor load and displacement during the tests.

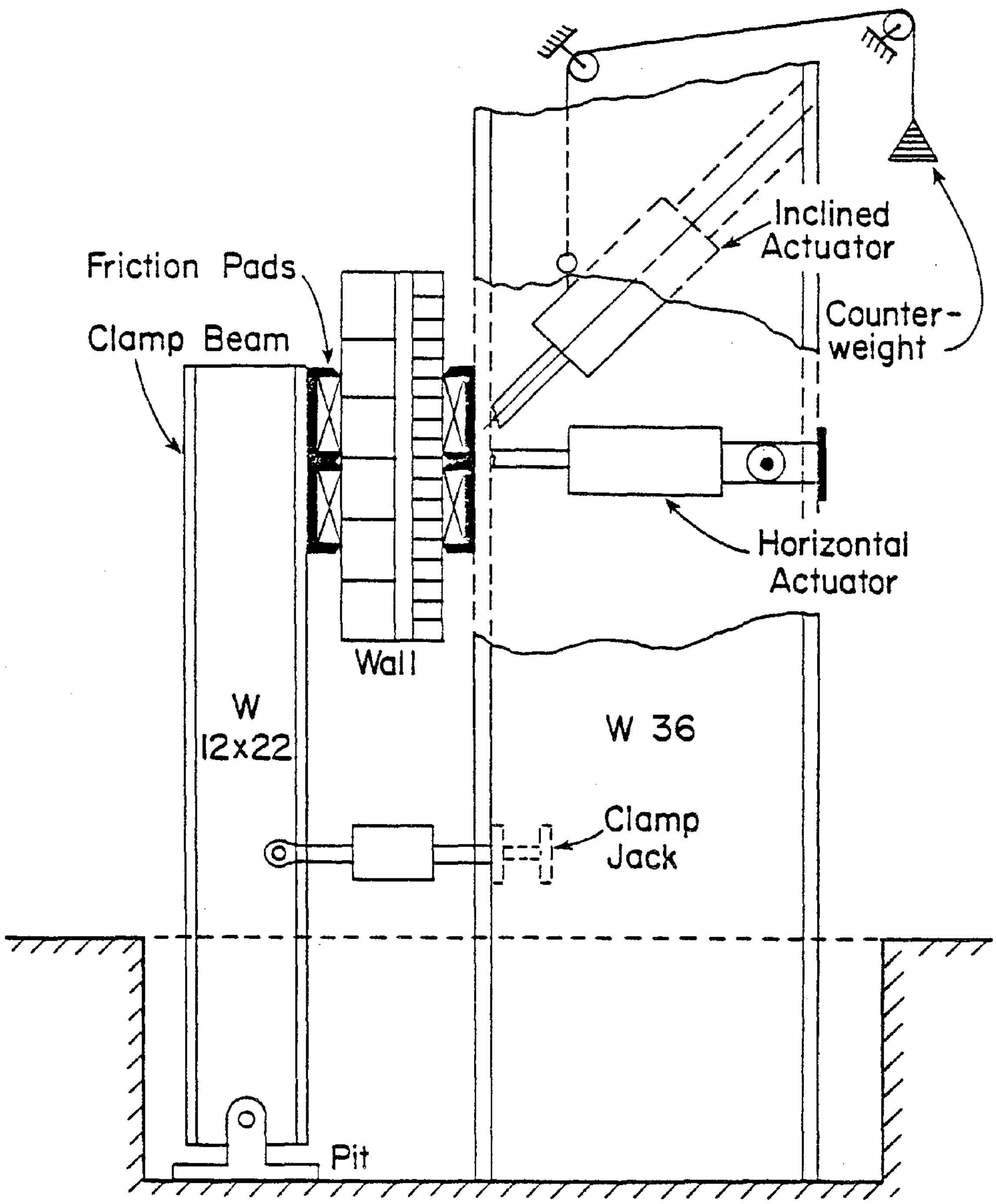


Fig. 1 Bolt Testing Machine

Wall Specimens

A total of 38 double wythe concrete masonry walls were constructed using 8 in (20 cm) lightweight concrete masonry units and a fully-grouted collar joint. In most cases all of the cells of the block were fully grouted except in some specimens where toggle bolts and sleeve anchors were to be inserted into the cells of the ungrouted cores.

The walls were constructed by journeymen masons during one construction operation. After the walls were allowed to cure for two days, a grout truck delivered grout to the construction site where the grouting operation then took place.

A vertical reinforcing bar was used in the corner cells and truss type joint reinforcement was used in the top and bottom bed joints. This steel was used to arrest cracks which may have formed in the testing of the bolts. Since the steel was not close to the bolts, it is not expected that the results were affected by its presence.

Material Properties

Concrete Masonry Units - The blocks used in the project were intended to conform to ASTM C90-75 (Ref. 2), Grade N-I. The units were tested for physical properties and the results are given in Table 1. Note that the units were slightly under strength, 970 psi (6688 kPa) compared to a 1000 psi (6895 kPa) requirement.

Mortar - Type N Portland cement-lime mortar was intended to comply with ASTM C270-80A (Ref. 3). Mortar properties are given in Table 2.

Grout - Grout was obtained from a local batch plant and was intended to comply with ASTM C476-71 (Ref. 4). Grout was tested for physical properties which are reported in Table 3.

Bolts - Bolts were obtained from Hilti® Manufacturing Company and were installed in accordance with manufacturer's recommendations for concrete application. Bolts varied in diameter from 3/8 in (10 mm) to 3/4 in (19 mm). Individual bolt diameters and embedment depths are given in the following section of this paper.

TEST RESULTS

Load Capacities

The load capacities and pretightening torques of the anchors tested are given in Tables 4, 5, and 6. The only anchor that appears to be suited for installation in ungrouted face shells is the toggle,

TABLE 1. BLOCK PROPERTIES

Number	Compressive Strength Based On Gross Area (psi)	Compressive Strength Based On Net Area (psi)	Splitting Tensile Strength (psi)
1	980	1880	105
2	950	1800	105
3	980	1820	120
AVERAGE	970	1833	110
CV	1.8%	2.3%	2.7%

Note: 1 psi = 6.89 kPa

TABLE 2. MORTAR PROPERTIES

Batch Number	Cube Compressive Strength (psi)	Cylinder Compressive Strength (psi)	Splitting Tensile Strength (psi)
1	-	570	-
2	600	650	-
3	-	590	74
4	650	690	-
5	-	660	77
6	950	760	-
7	-	630	-
8	-	730	-
9	-	630	-
10	-	750	79
11	-	720	75
AVERAGE	733	670	76
CV	25.8%	9.6%	2.9%

Note: 1 psi = 6.89 kPa

TABLE 3. GROUT PROPERTIES

Number	Cube Compressive Strength (psi) (4 in x 4 in)	Cylinder Compressive Strength (psi)
1	1690	1368
2	1480	1491
3	-	1898
AVERAGE	1585	1586
CV	9.4%	17.5%

Note: 1 psi = 6.89 kPa, 1 in. = 25.4 mm

TABLE 4. AXIAL STRENGTH TEST VALUES

ANCHOR TYPE	DIAM. (IN)	EMBED. DEPTH (IN)	EMBED. LOCATION	PRE-TIGHTENING TORQUE (FT-LBS)	n	AVG. LOAD (LBS)	COEFF. OF VAR. %	MANUF. AVG. LOAD (LB)
WEDGE	3/8	1 5/8	grouted cell	5	6	907	21.5	
	1/2	2 1/4	grouted cell	20	5	2075	36.4	
	5/8	3	grouted cell	30	5	2574	22.2	
	3/4	3 1/4	grouted cell	40	4	3263	21.4	
SLEEVE	3/8	2 1/2	grouted cell	10	4	1133	19.1	
		1 3/4	joint	<5	3	390	10.3	
		1 1/2	hollow cell	<5	4	322	21.6	1279
	1/2	2 1/4	grouted cell	30	3	1338	22.7	1674
		2 1/8	joint	10	6	623	64.1	
		1 1/2	hollow cell	10	4	498	70.6	
	5/8	3 3/4	grouted cell	30	3	2424	33.2	3444
		2	joint	10	6	1617	42.2	
	3/4	3 3/4	grouted cell	40	6	3027	54.1	8232
		2	joint	25	4	1897	49.8	
TOGGLE	3/8	-	hollow cell	10	3	955	13.4	1079
	1/2	-	hollow cell	15	3	1067	13.5	1137

Note: 1 lb. = 4.45 N, 1 in. = 25.4 mm, 12 in. = 1 ft.

TABLE 5. SHEAR STRENGTH TEST VALUES

ANCHOR TYPE	DIAM. (IN)	EMBED. DEPTH (IN)	EMBED. LOCATION	PRE-TIGHTENING TORQUE (FT-LBS)	n	AVG. LOAD (LBS)	COEFF. OF VAR. %	MANUF. AVG. LOAD (LB)
WEDGE	3/8	1 5/8	grouted cell	5	3	2327	16.1	
	1/2	2 1/4	grouted cell	20	3	5800	6.0	
	5/8	3	grouted cell	30	3	8067	1.4	
	3/4	3 1/4	grouted cell	40	-	not available		
SLEEVE	3/8	2 1/2	grouted cell	10	3	3094	4.9	
		1 3/4	joint	<5	3	1400	7.1	
		1 1/2	hollow cell	<5	3	1434	19.5	3064
	1/2	2 1/4	grouted cell	30	3	5167	4.9	5112
		2 1/8	joint	10	3	2667	11.5	
		1 1/2	hollow cell	10	3	2098	8.2	
	5/8	3 3/4	grouted cell	30	3	7133	9.3	8217
		2	joint	10	3	3933	7.8	
		1 1/2	hollow cell	10	3	1800	5.6	
	3/4	3 3/4	grouted cell	40	3	8600	6.2	13,412
		2	joint	25	3	4367	11.5	
		1 1/2	hollow cell	10	3	2167	25.4	
TOGGLE	3/8	-	hollow cell	10	3	2133	2.7	2521
	1/2		hollow cell	15	3	2950	7.4	2410

Note: 1 lb. = 4.45 N, 1 in. = 25.4 mm, 12 in. = 1 ft.

TABLE 6. COMBINED SHEAR AND TENSION TEST VALUES

ANCHOR TYPE	DIAM. (IN)	EMBED. DEPTH (IN)	EMBED. LOCATION	PRE-TIGHTENING TORQUE (FT-LBS)	n	AVG. LOAD (LBS)	COEFF. OF VAR. %	MANUF. AVG. LOAD (LB)
WEDGE	3/8	1 5/8	grouted cell	5	3	875	34.6	
	1/2	2 1/4	grouted cell	20	3	1852	17.8	
	5/8	3	grouted cell	30	3	3096	22.4	
	3/4	3 1/4	grouted cell	40	4	3306	25.2	
SLEEVE	3/8	2 1/2	grouted cell	10	3	1602	10.1	
		1 3/4	joint	<5	3	534	71	
		1 1/2	hollow cell	<5	4	276	49.9	
	1/2	2 1/4	grouted cell	30	4	2296	17.5	
		2 1/8	joint	10	2	952	13.7	
		1 1/2	hollow cell	10	4	522	30.6	
	5/8	3 3/4	grouted cell	30	4	3867	11.2	
		2	joint	10	3	1783	36.9	
	3/4	3 3/4	grouted cell	40	3	4205	12.1	
		2	joint	25	4	2014	19.1	
TOGGLE	3/8	-	hollow cell	10	3	915	10.2	
	1/2	-	hollow cell	15	3	1136	6.9	

Note: 1 lb. = 4.45 N, 1 in. = 25.4 mm, 12 in. = 1 ft.

and it did not exhibit impressive strengths. The wedge anchor is well-suited for installation in grouted cores only. Sleeve anchors are suitable for grouted face shells, but have severely inconsistent and reduced capacities for ungrouted face shells and bedjoint-headjoint intersections.

FAILURE MODES

There were three observed failure modes; stud failure, masonry failure, and holding mechanism failure.

Tension Loading

The 3/8 in (10 mm), 1/2 in (13 mm), and 5/8 in (16 mm) diameter wedge anchors tested in tension experienced a failure of the holding mechanism due to the wedges crushing into the walls of the masonry hole allowing the stud to slip from the hole. The 3/4 in (19 mm) diameter wedge anchor produced a masonry pullout cone.

The 3/8 in (10 mm), 1/2 in (13 mm), and 3/4 in (19 mm) sleeve anchors embedded in grouted cells experienced holding mechanism failures where the stud and sleeve pulled far enough out of the hole to allow the stud to free itself from the sleeve. For the 5/8 in (16 mm) diameter sleeve, however, a masonry cone failure occurred.

All of the sleeve anchors embedded at the intersections of the head and bed joints experienced masonry failures. The 3/8 in (10 mm) diameter sleeve anchors in the hollow cells experienced holding mechanism failures similar to those found with the 3/8 in (10 mm) diameter sleeve anchors embedded in grouted cells. The 1/2 in (13 mm) diameter sleeve anchors in the hollow cells experienced masonry pullout cone failures.

The toggle bolt anchors for the hollow cells experienced a ductile holding mechanism faiure of the bolt bracket which bears against the inner side of the faceshell.

Shear Loading

The 3/8 in (10 mm), 1/2 in (13 mm), and 5/8 in (16 mm) diameter wedge anchors subjected to pure shear loadings experienced ductile failure of their studs. For the 3/4 in (19 mm) diameter wedge anchors, masonry failures accompanied the stud failures.

The 3/8 in (10 mm) and 1/2 in (13 mm) diameter sleeve anchors embedded in grouted cells also experienced ductile failure of their studs. For the 5/8 in (13 mm) and 3/4 in (16 mm) diameter sleeve anchors, masonry failures accompanied the stud failures. The sleeve anchors embedded at the intersections of the head and bed joints experienced masonry failures, although the 3/8 in (10 mm) and 1/2 in (13 mm) diameter anchors had partial shearing of their studs. Sleeve anchors in the hollow cells had the same failures as the sleeve anchors embedded in the joints.

The 3/8 in (10 mm) diameter toggle bolt anchors underwent stud shear failure with partial bolt bracket failure. The 1/2 in (13 mm) diameter toggle bolt anchors had holding mechanism failures due to ductile bolt bracket failure.

Combined Shear and Tension

The 3/8 in (10 mm) and 1/2 in (13 mm) diameter wedge anchors subjected to equal magnitudes of shear and tension exhibited holding mechanism failures similar to those experienced for these diameters in pure tension. For the 5/8 in (16 mm) and 3/4 in (19 mm) diameter wedge achors, masonry failure cones developed.

The 3/8 in (10 mm) diameter sleeve anchors embedded in grouted cells experienced ductile stud failures. The 1/2 in (13 mm), 5/8 in (16 mm) and 3/4 in (19 mm) diameter sleeve anchors embedded in grouted cells had stud distortion but failed due to masonry cracking. The sleeve anchors embedded in the intersections of the head and bed joints developed masonry failures with the 3/8 in (10 mm), 1/2 in (13 mm), and 5/8 in (16 mm) diameter anchors displaying stud distortion. The sleeve anchors in hollow cells experienced masonry break-out failures with distortion to the sleeves but not to the studs.

ANALYSIS OF TEST RESULTS

Effect of Bolt Diameter

It is apparent from Tables 4, 5, and 6 that the load capacities increase with anchor diameter. The toggle bolt anchors exhibited practically the same strengths for both the 3/8 in (10 mm) and 1/2 in (13 mm) diameters.

Effect of Loading Direction

Comparison of Shear to Tension - The shear strengths developed were typically 2-4 times the tensile strengths for all anchors tested. It can be seen in Table 4 that the coefficients of variation for the wedge and sleeve anchor tension tests were very high, typically greater than 20%. For shear tests, Table 5 shows that the coefficients of variation of the wedge and sleeve anchors were reasonably low, most being less than 10%. This shows that axial strengths vary greatly for each wedge and sleeve anchor size. This can be attributed to the holding mechanism's ability to perform correctly, non-homogeneity of the masonry, slightly greater hole diameter after drilling than expected, and voids in the joint.

The toggle bolt anchors produced shear strength more than twice as large as their tensile strengths. All toggle bolt coefficients of variation were low compared to those of wedge and sleeve anchors.

Combined Shear and Tension - Comparing results of Tables 4, 5, and 6, the combined shear and tension tests more closely resemble pure tension tests than shear tests. It can also be seen that the coefficients of variation are typically greater than 15% and several are greater than 30%. This can be attributed to most of the failures being masonry failures.

COMPARISON TO MANUFACTURERS' PUBLISHED LOADS

The only published loads of the retrofit fasteners in masonry studied in this research are for the toggle bolt anchors done by Hilti® Engineering (Ref. 5) and the sleeve anchors in grouted and hollow cells by ITT-Phillips (Ref. 6). These values are presented in Tables 4 and 5. Any other testing by manufacturers has been done in concrete slabs and not masonry.

The values for sleeve anchors obtained by the manufacturers, in general, exceed those reported in this study. Since comparable masonry units and grout were used, there is no apparent explanation for the difference.

SUMMARY AND CONCLUSIONS

The results of tests on 173 masonry retrofit anchors are reported. Three types of anchors in various sizes were embedded in concrete masonry and subjected to shear, tension, and combinations of shear and tension. Monotonic tests were performed on each size and configuration. Results of the research have lead to the following conclusions:

1) There are several commercially-manufactured retrofit embedments that are suitable as structural anchors for concrete masonry.

2) Pure shear loads are typically 2-4 times those found for axial (tension) loads.

3) Combined shear and tension test loads more closely resemble axial (tension) test loads than shear test loads.

4) Wedge, sleeve, and toggle bolt anchors give very consistent values in monotonic shear loadings.

5) Wedge and sleeve anchors give severely varying values for axial (tension) loadings.

6) Toggle bolt anchors have consistent shear, tension, and combined loading strengths in monotonic loadings.

7) Sleeve anchors placed in head and bed joint intersections or in hollow cells are too inconsistent to pin-point their true ultimate loads for axial and combined loadings.

8) Wedge and sleeve anchors are well-suited for grouted cell applications.

9) Toggle bolt anchors are recommended for hollow cell applications.

10) Embedment of sleeve anchors in mortar joints is not recommended for axial (tension) applications.

ACKNOWLEDGEMENTS

The authors wish to thank the National Science Foundation for financially supporting this research. The authors are also grateful to Andy Dalrymple, Theodore Light, Jon Caime, Howe Crockett, Peter Andersen, and Ed Coggins for their assistance in conducting the tests and constructing the walls. The work of Dr. A. R. Whitlock in the development of the test apparatus is gratefully acknowledged.

REFERENCES

1. Brown, Russell H., and Whitlock, A., Rhett, "Strength of Anchor Bolts in Masonry," Clemson University, Clemson, SC, August, 1983.

2. "Hollow Load-Bearing Concrete Masonry Units," American Society for Testing and Materials, ASTM C90-75, Philadelphia, PA.

3. "Mortar for Unit Masonry," American Society for Testing and Materials, ASTM C270-73, Philadelphia, PA.

4. "Mortar and Grout for Reinforced Masonry," American Society for Testing and Materials, ASTM C476-71, Philadelphia, PA.

5. Hieronymus, Frank, "Hilti® Toggler Bolt Hollow Wall Anchor Performance Test," Hilti Engineering of North America, Tulsa, OK, March, 1982.

6. "Carbon and Stainless Steel Sleeve Anchor Test Program in Hollow and Grout Filled, Light-Weight and Medium-Weight Masonry Block Wall," ITT-Phillips "Red-Head," Chicago, IL, April-May, 1983.

INTENTIONALLY BLANK

The New San Gregorio Magno

Alberto Cherubini (I)

SUMMARY

I am going to talk about a remarkable experience that we are living in San Gregorio Magno, a little country village with a population of six thousand people in Salerno's hinterland, and about the development of its reconstruction plan.

The earthquake that hit San Gregorio on November 23, 1980, was an eighth degree one on the MSK scale, and its epicenter was about 24 kilometers from the village. You can easily get there by car on a motorway and this circumstance proved very useful to some colleagues of mine and myself when, four days after the earthquake, we decided to go there just to give assistance.

We stayed in San Gregorio for two months, facing every kind of difficulty, and what we experienced there has clearly shown us how essential it is to have an efficient structure of intervention, able to bring assistance as quickly as possible, in order to avoid human suffering and the waste of energy. A year and a half later in September, 1982, the Town Council had to issue reconstruction plans and it decided to rely upon us for the job. This paper presents an account of the development of the plan.

INTRODUCTION

We have started our difficult task with great care and energy while encountering every kind of technical and administrative problem. We have been charged with urbanistic plans, public works projects, planning and programming of repair interventions, organization of everything involved in the reconstruction and assistance to Municipal Offices -- quite a wide task, as can be seen. At the same time, many private citizens have asked for our cooperation and assistance in repairing and/or rebuilding their own houses. Obviously this was not compatible with our activity for Public Administration, and for this reason we decided to delegate this problem to another team. With our team of architects, structural engineers, town planners, and construction technicians, we organized and conducted an accurate survey of all damages. The principal aim was to draw an efficient Recovery Plan, as called for by a 1978 law.

RECOVERY PLANS

The 1978 law stipulates that relevant areas must be divided into individual small portions -- from now on let me call tham "minimal units" -- including one or more buildings, but in any case being the

(I) University of Rome

minimal size of the area where it is possible to assure a structural and/or architectural recovery. The whole village was divided into three parts, each of which has its own Recovery Plan different from the others. I will now explain these three plans (Figures 1-2).

The First Recovery Plan includes the most recent construction in the village suburbs. Damages in this area are less serious than elsewhere, and the buildings, two storey ones, are generally detached. In this plan we place more attention on urban design than on building problems; that is, intervention involving roads and public utilities design.

Survey results in this part of the village are in accordance with planning directions. The choice of "minimal units" is not fixed, and private planners can act according to their own well-grounded criteria, as well as structural or architectural unity, or by making reference to cadastral units.

The Second Recovery Plan includes the old part of the village, a Middle Ages historical center full of buildings that have been placed one on top of the other for years until the last century. Here, partial collapses and serious cracks damaged most of the area, nevertheless buildings are still standing. Once we limited this area and checked every structure in it, a map of all damages was drawn according to a standard schedule. This schedule contained the following data for each building:

1. Position
2. Utilization
3. Metric data (size)
4. Quality and condition of structural system
5. Extent of damage in vertical and horizontal structures, in ceilings, and in internal and external stairs.

Data on damages are worked out in order to give an evaluation of their degree. A scale is reported on the map with the following classes:

A. Light or insignificant damage
B. Medium damage; not very deep cracks
C. Serious damage
D. Very serious damage which caused total collapse or required demolition.

EVALUATION OF DAMAGE

In order to evaluate the significance of damages, we made reference to standard crack classifications for masonry and reinforced concrete buildings. Every class of damage is represented by a different building taken as a sample. In this way we recorded structures covering more or less 260,000 sq. m. (Figure 3).

This map of damages has shown us the extent of the earthquake's effects, and it has allowed us to point out which areas were exposed to a greater seismic risk. We have thus succeeded in making urbanistic decisions on a large scale. Widening and closing streets, creating squares, reducing maximum building heights, and increasing the distance between buildings, which means:

1. Whether to maintain preceeding utilization or not. This is of use particularly for stables and agricultural premises in the old town center.
2. Whether to maintain the original number of stories and pre-existing foundation or not, mostly relying on San Gregorio Magno's geological map.
3. To decide what kind of intervention is preferable in order to safeguard the building property of the village.

Different levels of intervention are defined as follows:

L1 Repairing
L2 Restoration and/or rebuilding of parts or elements of the structure
L3 Total rebuilding "in situ"
L4 Total rebuilding with rebuilt facades respecting urbanistic criteria
L5 Urbanistic restructuring with demolition of the pre-existing building fabric
RC Restoration

We have thus obtained a second map, the map of "minimal units" with levels of intervention. Together with it, there is a list of buildings with all data subject to regulations, as well as every aspect to safeguard or take care of during the project. To date, the Town Council has approved 250 repair or reconstruction projects, according to the above mentioned law issued with the Recovery Plan. The use of "minimal units" wholly projected by one or more planners has resulted in a unity in both architectural and structural methodologies.

The Third Recovery Plan includes six particular areas to be entirely rebuilt as they were completely razed by the earthquake where 32 people died. These areas will become the Administrative Offices of the village, which explains why they must be rebuilt as soon as possible. A very detailed project has been carried out for this part of the village (Figure 4). Also for the Third Recovery Plan, besides a huge quantity of technical data for a well-defined rebuilding intervention, all data subject to regulations and every aspect involving public safety have been considered.

It is noteworthy that in these six areas all citizens have been required to delegate to the Town Council the right to make all necessary decisions. So the Council has been allowed to form new condominiums with a distribution of flats different from the original patterns.

Where the Administrative Offices will rise, we have decided to repair old buildings and to create a new open space in order to widen the present shape of the square. A step-like arrangement will give new premises to the Commune (Figures 5-8). Newly described urbanistic means, together with plans for a public service building for homeless people, and plans for re-starting of industrial activities, have been approved by the Government. Today, there are strict regulations to be observed. We have also started with a planning of public works, as well as schools, roads, and other community service systems. We have then agreed with the Council upon a technical program of intervention according to the availability of public funds in the next several years.

For the time being, planning for future development of the village is under consideration and will be carried out in conjunction with the rebuilding objectives. All of the above will perhaps attract and bring back home emigrants, and will help to reduce the phenomenal emigration that has been a problem for years.

CONCLUSION

Talking about town planning is a unique opportunity to meet the public and to get input from citizens. With the support of the Council, it has given us the opportunity to engage in a much deeper cultural approach with people, leading them, we hope, to a new style of life through a gradual process.

Two interventions are needed. The first is an historical-architectural one; the second is technical. On the one hand, and together with town planning objectives, there needs to be research on the village's history, customs, traditions, and architectural forms. On the other hand, agreements with the University of Rome and the National Council of Scientific Research, providing the execution of applied research on buildings, with tests "in situ", training of technicians, and establishing of drill sites, are also needed. A further agreement with the "Geophysical Observatory of Trieste" will allow us to issue a map of soil stability and seismic risk. And new techniques of seismic shocks survey will be needed, using a mobile network of oscillographs.

What will the new San Gregorio be? I think it will be a village that is rebuilt in the best way, with a new consciousness of its history and resources, and a community prepared to talk about its problems. In a word, it will be an intelligent answer of society to nature's destroying power.

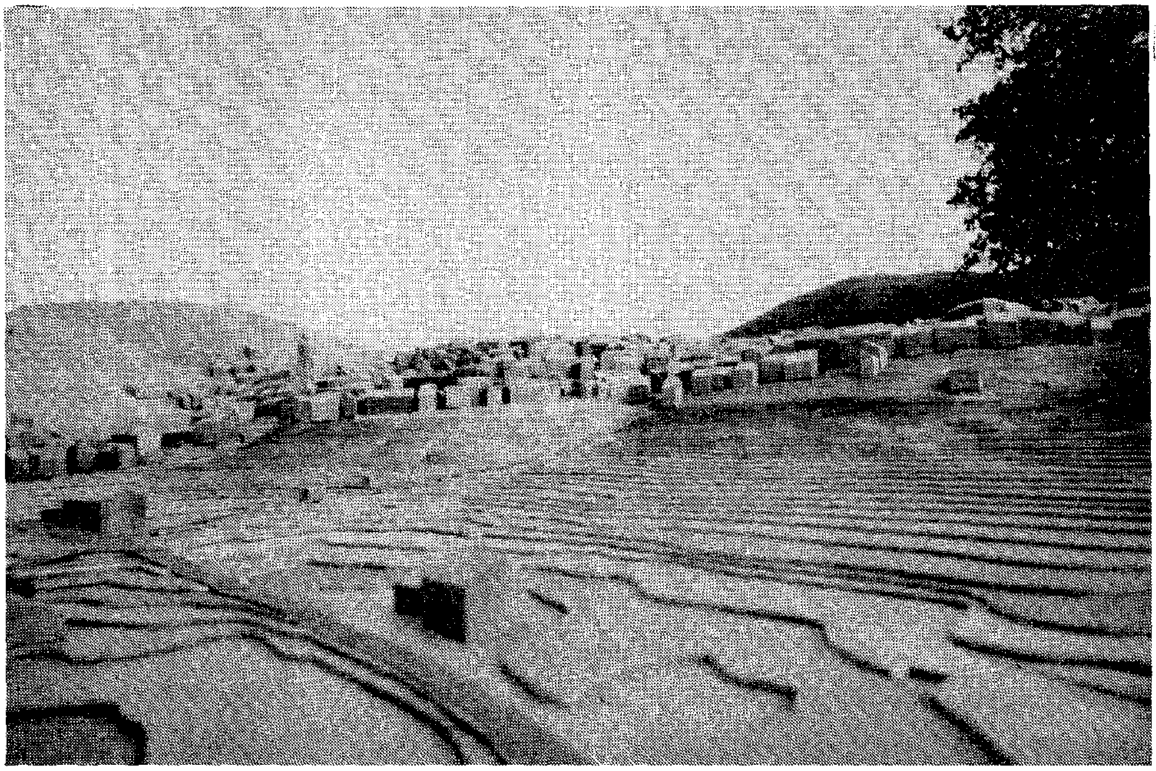


Fig. 1: San Gregorio looks onto a terrace over the so-called "Pantano." The village agricultural and forestry economy is to be safeguarded and improved. On the mountain slopes, towards the north, are the characteristic cellars, dug out in the rocks. In the 16th Century, San Gregorio was already famous for its excellent wines.

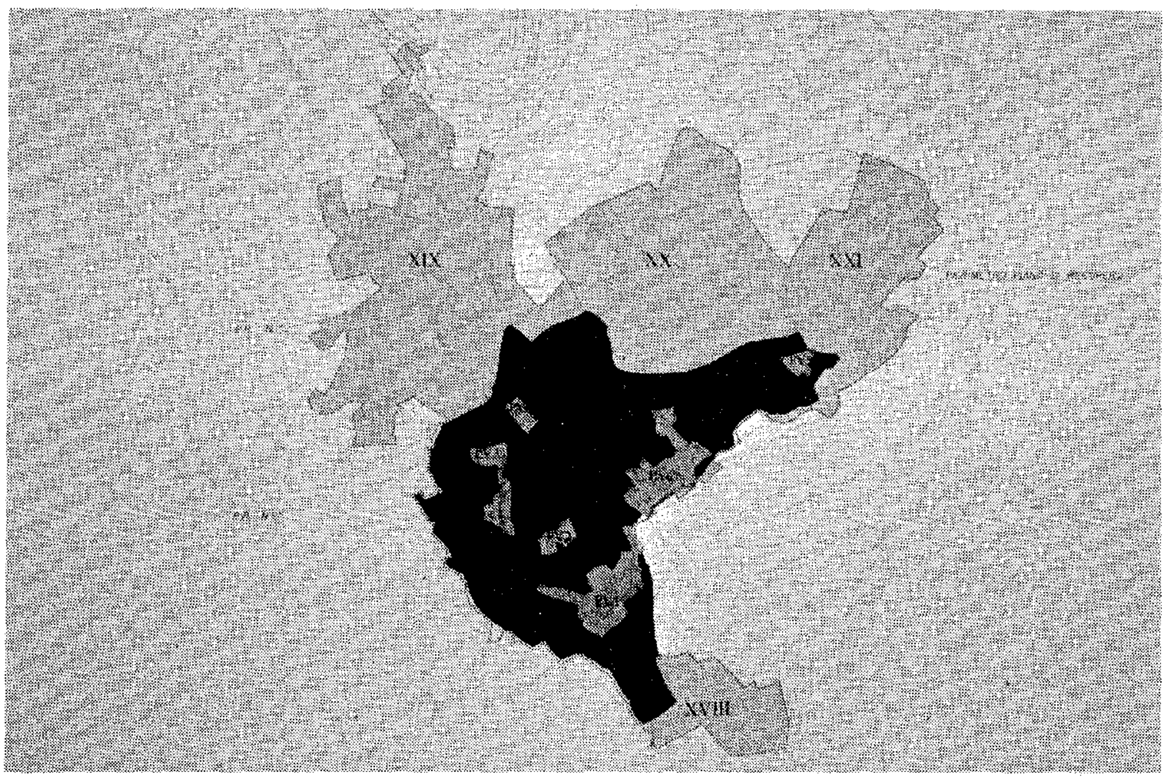


Fig. 2: Village division into three parts. The light gray refers to the First Recovery Plan, the black area to the Second, and the dark gray corresponds to the six areas to be reconstructed according to the Third Recovery Plan.

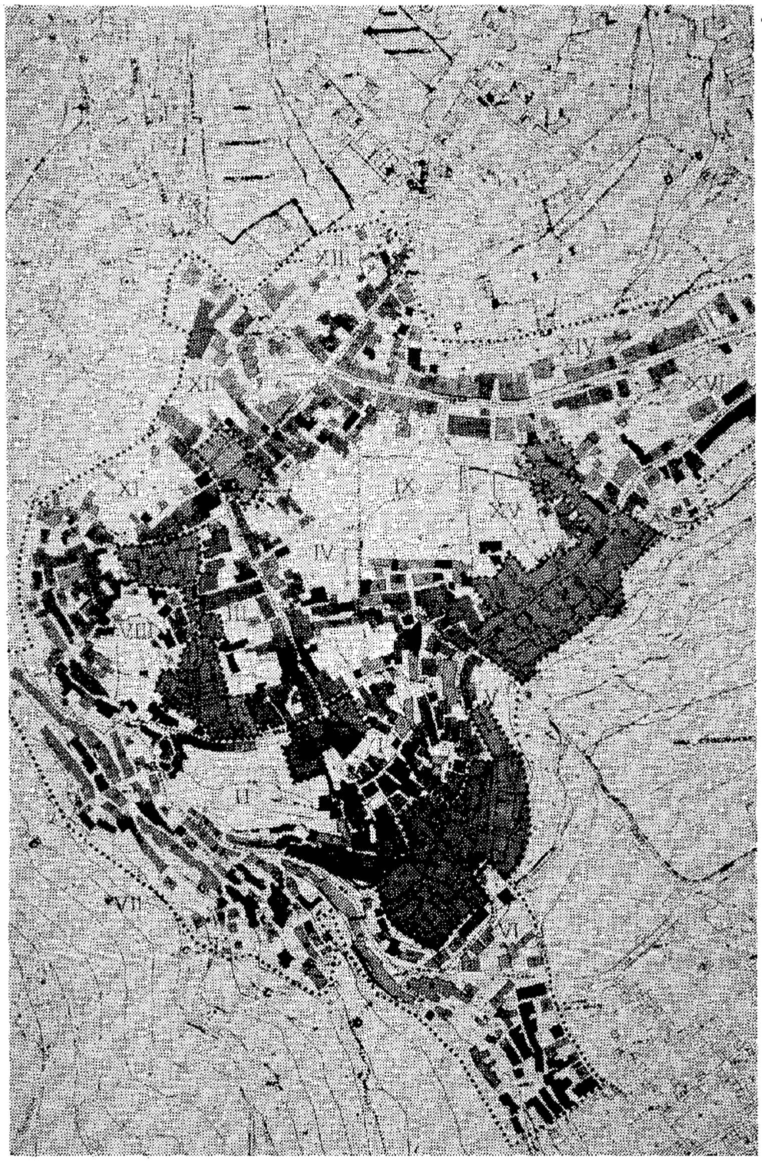
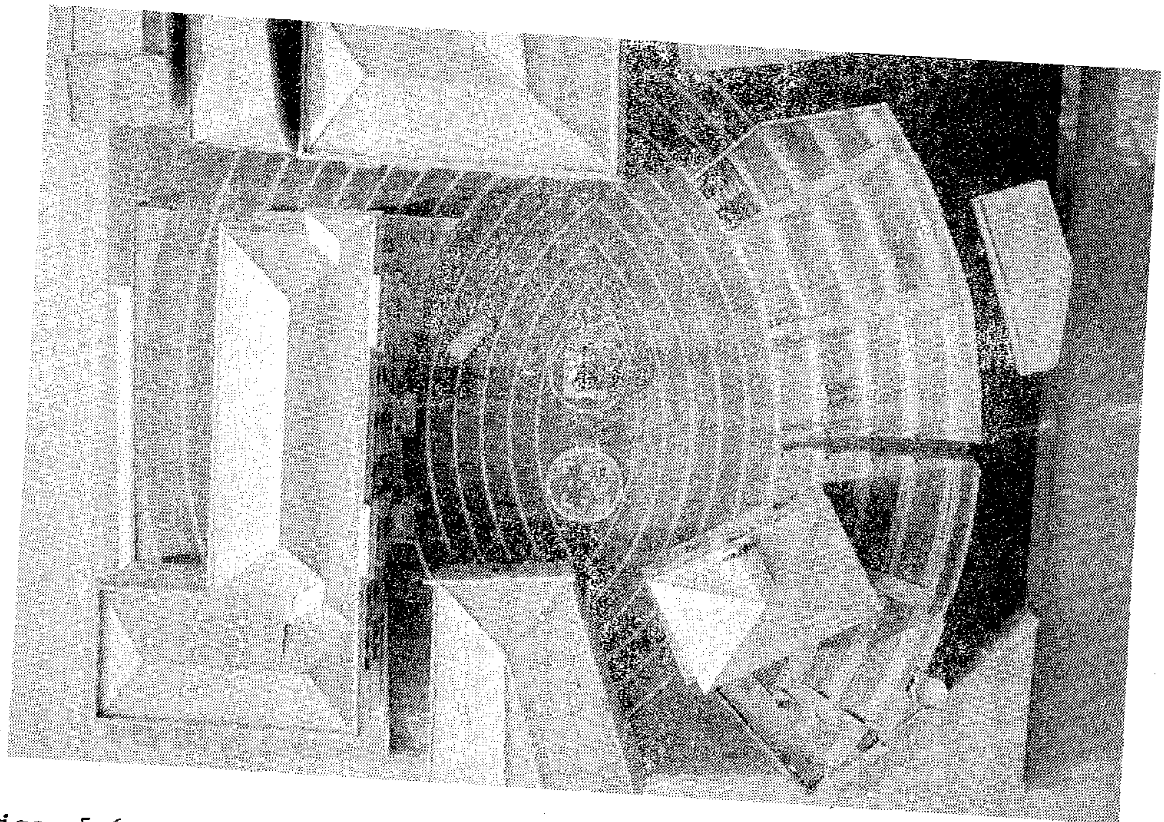
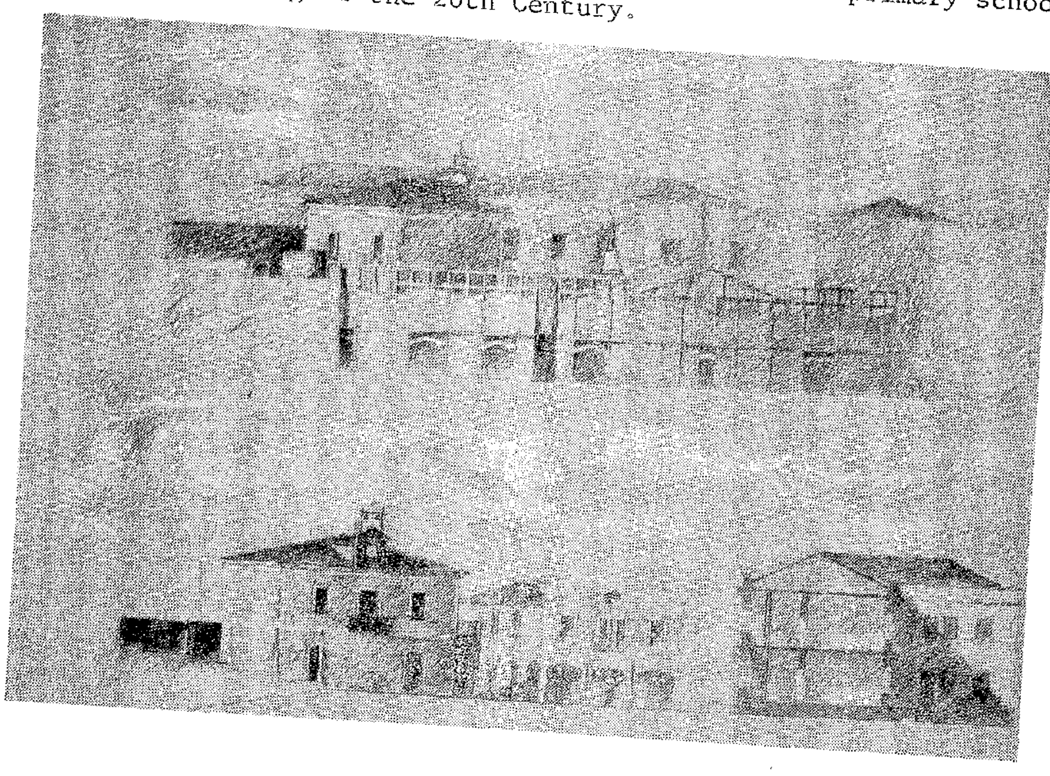


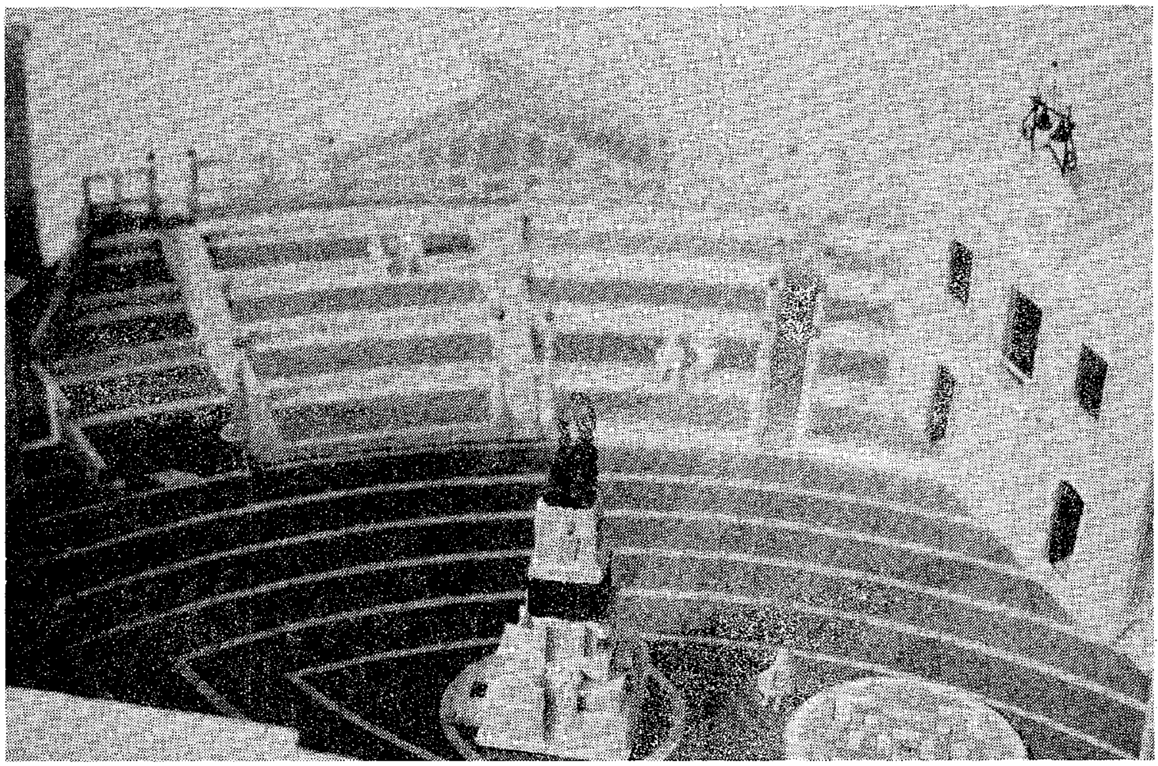
Fig. 3: Extract of the map indicating damage level by means of different colors (the darker ones refer to the most serious damage). The gray areas are those completely destroyed, where the Third Recovery Plan is applied.

Fig. 4: Third Recovery Plan. The projects are recommendations because they cannot be compulsory for the designers, whereas the choice of the typology of the outer openings among those represented is compulsory.

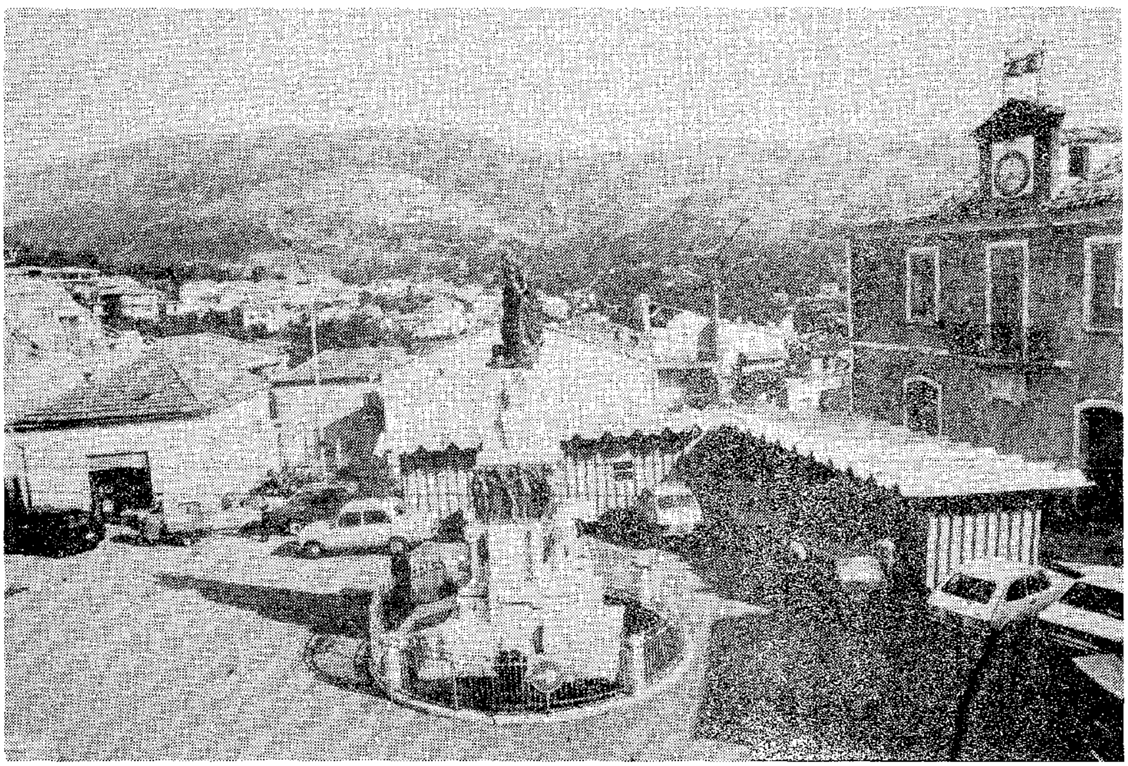


Figs. 5-6: The planning of the new Administrative Offices in Piazza Municipio. The Community Seat will be located in the former primary school, built at the beginning of the 20th Century.





Figs. 7-8: The new Administrative Offices project (above) together with the original picture (below) during the emergency period with containers used as temporary facilities. The Recovery Plan is a means to re-establish balance in the growth of a village, according to a model of development worked out by technicians, administrators, and citizens.



Antiseismic Rehabilitation of 600 Buildings in Friuli Operative Methods Adopted and First Statistical Evaluations of Repair Costs

A. D'Amato (1)

SUMMARY

This paper presents a large scale experience with building repair and retrofitting made in Friuli (Italy) after the 1976 earthquake. Antiseismic repairs for about 600 buildings have been designed in less than two years and at present work supervision is underway. The paper deals with the problems encountered and the operative methods employed during the operation. On the basis of the observation of the economic results for the first group of 180 buildings already repaired, some correlations between repair costs, volume of buildings and seismic coefficients are investigated.

FOREWORD

Data and observations herein reported stem from the experience carried out by SVEI Spa connected with the antiseismic repair and retrofit of some 600 buildings damaged by the 1976 earthquake in Friuli. Beyond the design and technical aspects developed during such an operation, we wish to outline here two particular aspects of large retrofit operations: the rational organization of the designing and supervising of works, and the evaluation of costs of antiseismic rehabilitation. This last question will be resolved in the near future, since final data are at present available for only 180 of the 600 buildings.

THE OPERATION

The matter considered is based on the overall frame entailing the reconstruction of Friuli following the May and September earthquakes in 1976, where 997 people were killed, more than 100,000 were left homeless, and an area of approximately 4.500 km2 was ravaged. Among the actions taken towards the reconstruction, SVEI was given by the Regional Administration of Friuli-Venezia Giulia the task of designing the antiseismic strengthening works for about 600 damaged buildings scattered in an area of about 1.500 km2 and located in 34 different towns and villages. SVEI was also requested to undertake the supervision, computation and acceptance of works for about 320 of the above mentioned buildings. Because of the obvious social and economic reasons which imposed the maximum possible speed, the phase of designing had to be completed within two years, at the rate of one project for each working day. The projects had to be planned and calculated in accordance with the technical directives and conditions set forth by the Administration and their costs

(1) Civil Engineer, SVEI S.p.A., Roma, Italy

had to remain within established limits. The projects had also to receive the prior agreement of the owners. For some 80 buildings having historical and architectural significance, the design had to be accompanied by historical research and by special studies and considerations. SVEI had also to prepare the documentation and specification for tenders, which were afterwards carried out by the Administration. The supervision of construction, as well as the correlated activities, had to be made in accordance with the Normative for Public Works Execution and local regulations. At present,

- 592 buildings for about 601.000 mc have been designed;
- the amount of designed works is about 40 billions lire ($ 25 million);
- the designing phase has been completed within the assigned time;
- the supervision of construction has been split into 33 separate and simultaneous contracts, engaging 15 different contractors;
- 179 buildings have been completely refitted and handed over to owners;
- the advancement percentage of the entire operation assigned to SVEI is 80%.

During the operation, some problems peculiar to large scale retrofitting activities have been encountered. Such problems are substantially different from those which can be encountered while upgrading a single building or monument. Among these, designing speed is particularly relevant when considering the large number of homeless people involved and the adverse effect due to inflation affecting amounts on the order of hundreds of billions. Other peculiar problems of large scale building retrofitting have been experienced in Friuli, as for instance:

- the buildings to be repaired were of the most different typologies and sizes and were placed in the most different geological and other conditions;
- strong presence of owners' interference during the designing process;
- necessity for subsidizing local administrations lacking organiza- or technical skill;
- changing of programs and of normatives during the execution of the job;
- the supervision of contracts being awarded to a large number of enterprises scattered in a large area and operating simultaneously.

THE DESIGN AND SUPERVISION OF CONSTRUCTION

The basic choice for the execution of the job has been to establish a special Operative Unit in Udine and dimensioned and fitted to achieve, in a completely independent way, all the aspects of the work, ranging from the relations with the Regional Administration to the technical problems, and from taking care of the legal and bureaucratic aspects to the actual editing of photos and plans. During the peak production period (30 projects per month) the number of people assigned to the Unit was 28, plus the Coordinator. In order to reach the required production speed, the design process has been thoroughly analyzed and procedural phases have been pin-pointed and separated in order for them to be carried out by different specialized groups of technicians. In this way a real assembly line was set up, and this method made it possible to carry out up to 40 projects simultaneously. (In a particular period, up to 56 projects have been underway simultaneously.) To make the control possible some key points were spotted where the section chief or the Coordinator himself could verify the quality of work. To make the transmission of information easier, conventional symbols were studied, but most of all each phase of the work was executed with the aim of easing the following one. In order to define the technical properties of existing structures and materials, a great number of tests (some 450) on structures and foundations have been conducted. Once the structural and technical characteristics were evaluated, a first attempt to define the antiseismic interventions was done by POR method. Also if POR was not sufficient to indicate all the necessary work to ensure the antiseismic strengthening of the building, it proved to be extremely useful in quickly providing information on basic behaviour of masonry structures under shear stresses. Moreover, since the data input could be easily done by the designer's assistants, POR turned out to be a great help in fast designing.

With regard to the supervision of construction and its correlated activities, it was not possible to standardize the procedures as was done for the designing phase. First of all the work has been divided into many different contracts (32) which have started at different times and under different conditions due to particular administrative problems. Secondly, to accomplish the supervision task, and to deal with proprietors and local administrations, physical presence was requested in various places scattered over a large area to a much greater extent than during the previous phase, and this was an obstacle to a centralized productive structure.

The above-mentioned reasons have led to the creation of seven different operative Sub-Units coordinated by the Main Unit. A graduate Field Engineer was in charge of every Sub-Unit, with a number of assistants for the contracts and buildings to be supervised. In periods of full production the personnel assigned to the supervision phase numbered up to 25 people, plus the Coordinator. It is worth mentioning how demanding the supervision was of work concerning dozens of buildings grouped in so many different contracts. In fact each contract could consist of a different number of buildings (contracts involving up to 58 buildings have been dealt with) and separate accounting had to be done for each building in order to determine the exact amount of financial aid to be given to citizens by the Administration. Besides, working on existing buildings calls for a great variety of operations. Therefore, inspite of massive employment of computers for office work, such activities as measuring, checking and testing had to be carried out personally and whenever needed, which excluded the possibility of previously planned and repetitive procedures. Computers were also heavily used for the control of global advancement of contracts and, to this end, special programs have been prepared.

FIRST STATISTICAL EVALUATION OF REPAIR COSTS FOR MASONRY BUILDINGS

During the design and supervision phases, data on structural composition and characteristics of materials, foundations, soils, etc., as well as observed damages relative to each building, have been gathered and recorded. Also the effective strengthening interventions and costs have been noted. One of the subjects that we wanted to investigate when starting the collection of data was that of observing eventual correlations between repair costs, building structures, and seismic actions.

The buildings considered and first observed sample

As noted above, the 600 buildings on which SVEI operated are scattered in 34 different towns and in an area of about 1.500 km2. In such a large sample, almost every type of building and structure has been encountered and this, together with different local conditions of seismic action, make rather difficult the possibility of observing significant numbers of similar buildings in similar conditions. However, since a large number of the above mentioned buildings (some 230) are situated in the same town of Gemona del Friuli, and since 179 of these buildings are isolated houses and have already been completed, we can here identify some considerations about the repair costs of this group. (Costs refer to those of 1977 in Friuli and Table A gives the main characteristics of this group of buildings.)

The most important and homogenous groups of the observed sample are those of the buildings with concrete/tile floors and vertical structures in brick, concrete blocks and mixed masonry. Those three groups of buildings have been chosen to identify eventual relations between the buildings' repair costs and their volumes when seismic actions are comparable. To represent seismic effort on the buildings, seismic coefficient K has been selected and, particularly, the product C1xC2 (where C1 is the mechanical and hydrogeological response coefficient and C2 is the morphotectonic and local structure coefficient) has been used. Excluding from the sample the buildings with wooden horizontal structures and those with stone masonry vertical structures, a group of 147 buildings is left. In Table B this group has been divided according to vertical structures and C1xC2 values.

Global repair cost trend versus building volume and seismic coefficient

In Figs. 1 through 3, global antiseismic repair costs have been plotted as function of the volume of the buildings and of products C1xC2 for the groups of buildings with vertical structures in concrete blocks and in mixed masonry. Data relating to buildings with brick masonry vertical structures have not been given here because their values are totally uncorrelated. It must be said also that in the case of the other two groups discussed here the number of observations is very limited to be significant from a statistical point of view, but at least an embryonic trend can be seen, which was not found for brick structures. It is interesting to see that for concrete block and mixed masonry buildings

DIMENSIONI - VOLUMES	N°	%
< 700 mc.	62	35
700 ÷ 1.000 mc.	56	30
> 1.000 mc.	61	35
TOTALE	179	100

N° DI PIANI - FLOORS	N°	%
Piano terra - Ground Floor	18	10
P.t. + sottotetto - G.F. + garret	47	26
P.t. + 1 piano - 2 floors	34	19
P.t.+2p.+ Sottotetto - 2 floors + garret	79	44
P.t.+ 2 o + piani - 3 floors	1	-
TOTALE	179	100

COSTITUZIONE STRUTTURALE - STRUCTURES

TIPO	STRUTTURE VERTICALI VERTICAL STRUCTURES	STRUTTURE ORIZZONTALI HORIZONTAL STRUCTURES	N°	%
1	Murature laterizi Brick masonry	Laterocemento concrete/tile	39	22
2	Murature laterizi Brick masonry	legno wood	1	-
3	Murature blocchi cls. Concrete block masonry	laterocemento concrete/tile	72	40
4	Murature blocchi cls. Concrete block masonry	legno wood	15	4
5	Murature pietrame Stone masonry	laterocemento concrete/tile	-	-
6	Murature pietrame Stone masonry	legno wood	4	2
7	Murature miste Mixed masonry	laterocemento concrete/tile	37	21
8	Murature miste Mixed masonry	legno wood	11	6
9	Strutture in c.a. Reinforced concrete etr.	laterocemento concrete/tile	-	-
TOTALE			179	100

TAB. A - INSIEME DEI 179 EDIFICI GIA' RIPARATI IN GEMONA
THE SAMPLE OF 179 REPAIRED BUILDINGS IN GEMONA

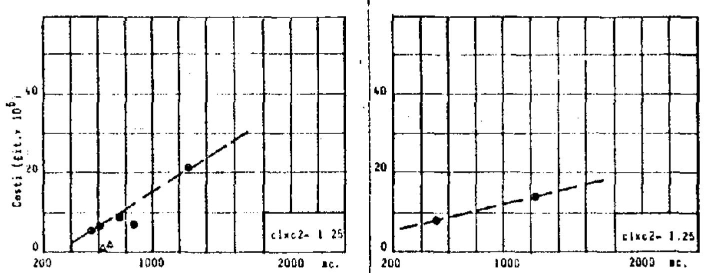
C1xC2	EDIFICI CON ORIZZONTAMENTI IN LATERO CEMENTO CONCRETE/TILE FLOOR BUILDINGS		
	Murat. in laterizi Brick Vert.struct.	Murature in blocchi cls Concrete Vert. Struct.	Murature Miste Mixed Vert.Struct.
1,25	1	7	2
1,30	5	14	7
1,35	12	6	7
1,40	16	14	10
1,45	3	17	6
1,50	-	6	2
1,55	2	7	4
TOTALE	39	71	37

TAB. B - INSIEME DEGLI EDIFICI GIA' RIPARATI CON ORIZZONTAMENTI IN L.C.
THE SAMPLE OF REPAIRED BUILDINGS WITH CONCRETE/TILE FLOORS.

COSTI TOTALI RISCONTRATI PER LE RIPARAZIONI ANTISISMICHE

TOTAL OBSERVED COSTS OF ANTISEISMIC REPAIRS

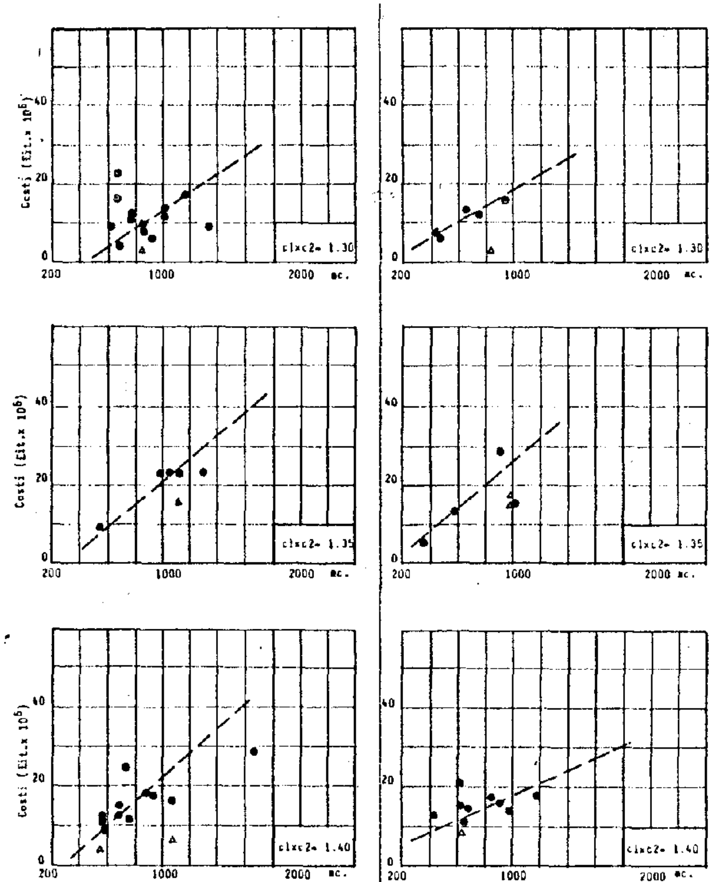
Edifici : strut.vert: muratura in blocchi cls. / " oriz: latero cemento	Edifici : strut.vert: muratura mista / " oriz: latero cemento
Buildings : vert.struct: concrete blocks masonry / horiz. " : R/c	Buildings : vert.struct: mixed masonry / horiz. " : R/c



LEGENDA

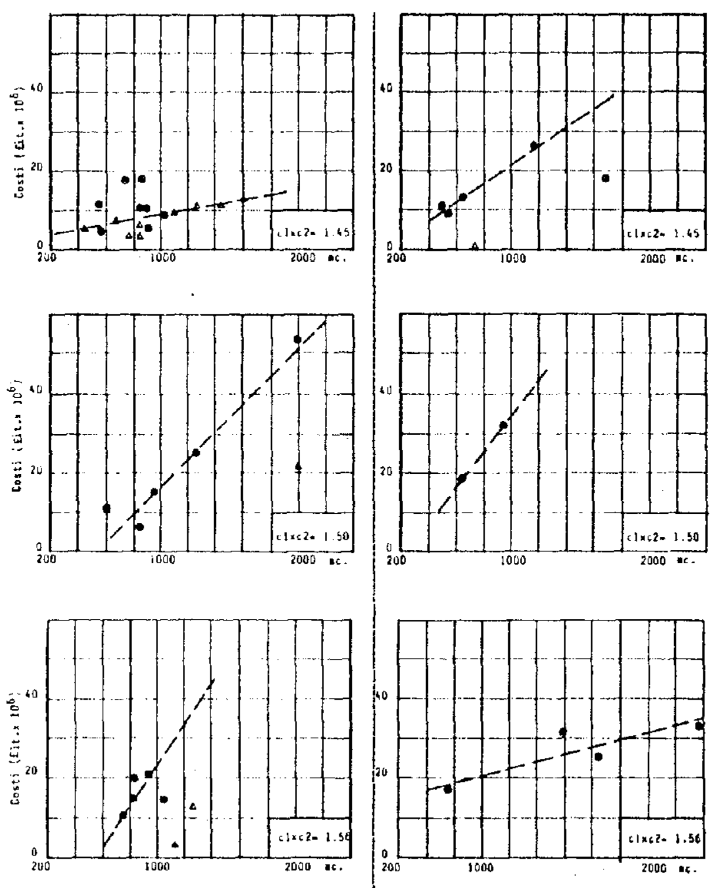
● Edifici con fondazioni normali.	● Standard founded buildings.
△ Edifici con fondazioni eccezionalmente buone.	▲ Exceptionally well founded buildings.
⊙ Edifici con fondazioni eccezionalmente scadenti.	⊙ Exceptionally poorly founded buildings.

- FIG. 1 -



Edifici : strut.vert: muratura in blocchi cls. / " oriz: latero cemento	Edifici : strut.vert: muratura mista / " oriz: latero cemento
Buildings : vert.struct: concrete blocks masonry / horiz. " : R/c	Buildings : vert.struct: mixed masonry / horiz. " : R/c

- FIG. 2 -



Edifici : strut.vert: muratura in blocchi cls.
" oriz: latero cemento

Buildings : vert.struct: concrete blocks masonry
horiz. " : R/c

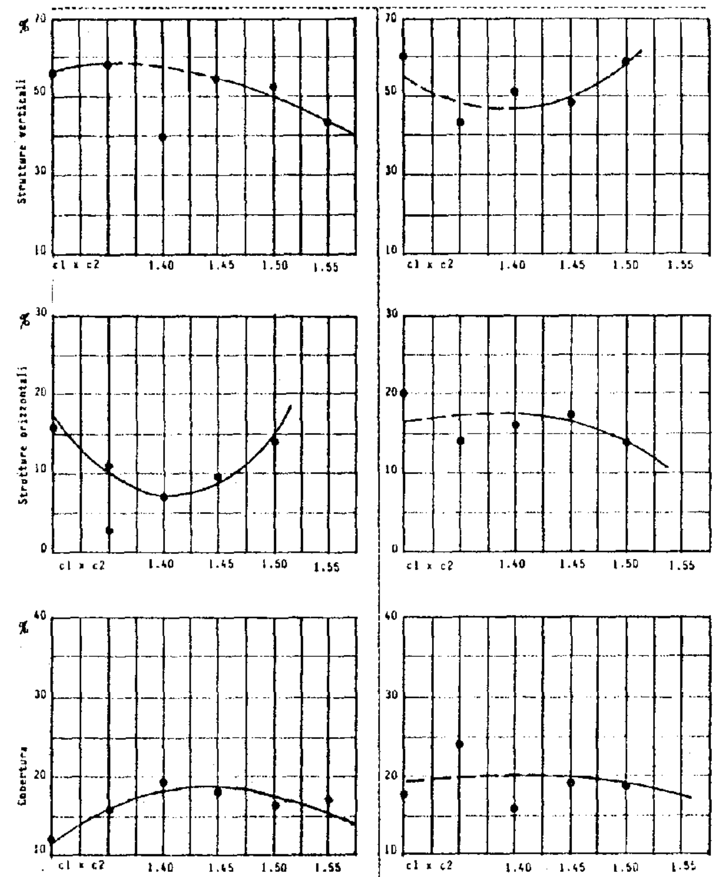
Edifici : strut.vert: muratura mista
" oriz: latero cemento

Buildings : vert.struct: mixed masonry
horiz. " : R/c

- FIG. 3 -

COSTI DI RIPARAZIONE DELLE STRUTTURE VERTICALI, ORIZZONTALI E COPERTURE: % SUL COSTO TOTALE

REPAIR COSTS OF VERTICAL STRUCTURES, HORIZONTAL STRUCTURES AND ROOFS: % OF TOTAL COSTS



Edifici : strut.vert: muratura in blocchi cls.
" oriz: latero cemento

Buildings : vert.struct: concrete blocks masonry
oriz. " : R/c

Edifici: strut.vert: muratura mista
" oriz: latero cemento

Buildings: vert.struct: mixed masonry
oriz. " : R/c

- FIG. 4 -

the relation between costs and volumes is linear and angular coefficient generally increase following the augmentation of C1xC2 product. The graphs also show very clearly how good foundations limit seismic damages. For all the buildings whose foundations were especially well made and connected, repair costs have been strongly contained. This is particularly evident in the graph of the cost of the buildings with concrete block walls and C1xC2 = 1, 45, where almost all the buildings had very good foundations.

Percentage of repair cost trends for vertical, horizontal and roof structures versus global repair costs

For the two groups of buildings for which an embryonic relation between volumes and repair costs has been seen, there has also been research to see if some trend could be found with regard to the incidence of particular repair costs on the global ones. In particular the repair costs of three groups of structures (vertical, horizontal and roof structures) have been observed and compared to the global repair costs of the buildings. The graph of Fig.4 shows the trend of average incidences of particular costs for each group of structures in function of C1xC2. The function shows some uncertainties in correspondence of some values of C1xC2. When more data about other completely repaired buildings become available, new averages will be calculated and it will be possible to deepen the observations. If a comparison is made between the corresponding functions of the groups, some differences are noted. For instance, the average repair costs of vertical structures of concrete block buildings declines for greater values of C1xC2, while for mixed masonry buildings such value increases. An opposite trend can be seen for horizontal structures, while roof structures behavior results are similar.

Evaluating Damage Probability Matrices from Survey Data

M. Dolce (I)

SUMMARY

Damage Probability Matrices (DPM's) are used to assess optimal policies of retrofit or repair and retrofit in large areas, both before and after earthquakes occur. In order to maximize the utility of DPM's, two main requirements are to be satisfied: the Content of Information should be maximized, and the Required Information should be minimized. In this paper the main problems which arise in the practical statistical evaluation of DPM's are examined and possible solutions designed to satisfy the above requirements are proposed. Finally, some DPM's, obtained by means of a statistical Maximum Likelihood algorithm applied to the data collected in Southern Italy after the 1980 earthquake, are presented.

INTRODUCTION

A Damage Probability Matrix (DPM) is a matrix whose cells provide the mass probability of a certain damage -- defined by the column index -- for a given intensity -- defined by the row index (1,2,3).

Since each matrix usually refers to a particular typology, the single row of a matrix provides

$$P[D|T,I]$$

i.e.the conditional mass probability distribution of damage for a given typology and a given intensity.

Regarding their utilization, DPM's are needed for: 1) the assessment of an optimal policy of retrofit for the reduction of the seismic risk of a region; and 2) the assessment of fund allocation and distribution for repairing after an earthquake. DPM's, in fact, permit one to evaluate the expected cost of retrofit or repair operations through the equations:

$$E[C] = \sum_{I,T} P[D|T,I] \quad P[I] V_T C_{DTu}$$

For point 1 (I is a random variable), and

$$E[C] = \sum_{T} P[D|T,I] \; V_T C_{DTu}$$

For point 2 (I is known), where

E C = Expected cost
V_T = Volume of buildings of typology T in the considered site

(I) Researcher, University of l'Aquila, Italy

C_{DTu} = Cost per volume unit of the damage D to buildings of typology T.

In order to get maximum utility from the DPM's, two contrasting prerequisites are to be satisfied:

1) the Required Information (RI) should be minimized; and
2) the Content of Information (CI) should be maximized.

RI is the information required to evaluate DPM's (e.g. statistical damage data, numerical simulations, and so on), and to use DPM's (e.g. structural characteristics of buildings, microzonation, and so on);

CI is the information on damage given by $P[D|T,I]$: the less the scatter, the more CI of $P[D|T,I]$.

According to Decision Theory, the choices relevant to RI should satisfy the equation:

$$f = U_{CI} - C_{RI} = \max$$

where: U_{CI} = Utility of the CI, i.e. global expected utility of the operations planned by means of DPM's;

C_{RI} = Cost of the RI.

Because of the difficulties in evaluating U_{CI}, it is practically impossible to evaluate f. Therefore, selecting the optimal RI is heavily a matter of judgment and of practical constraints.

Once the choices relevant to RI have been made, maximization of CI is only a matter of the algorithm adopted for the evaluation of DPM's. In particular the algorithm should be such that the dispersion of $P[D|T,I]$ be minimized.

Regarding the practical evaluation of DPM's, three different approaches can be adopted:

1) Statistical Approach: Starting from damage data after one or more past earthquakes, DPM's can be calculated by means of suitable statistical algorithms.

2) Analytical - Probabilistic Approach: Starting from numerical simulations made by using mathematical models of structures and earthquakes, DPM's can be calculated by means of suitable probabilistic algorithms which take into account uncertainties about structural behavior and earthquake characteristics.

3) Mixed Approach: This approach synthesizes the information available from approaches 1 and 2, possibly by:

a) calibrating the numerical results of the simulations to get a direct estimation of the damage;

b) simplifying the mathematical models of the structures by eliminating those parameters which, according to the statistical results, do not make important contributions to the structural behavior.

STATISTICAL APPROACH

Many problems are to be solved when evaluating DPM's. Although some of them are common to the three approaches, they will be examined in detail only with respect to the statistical approach.

A first kind of problem is relevant to the definitions adopted for Intensity, Typology and Damage (DPM $\rightarrow$ $P[D|T,I]$). These definitions condition the use of DPM's and are themselves conditioned by the type of data available.

A second kind of problem is of a merely statistical nature; it is relevant to the assumptions on the shape of the distributions $P[D|T,I]$.

Intensity (scale)

The seismic intensity can be measured through:

a) Instrumental scales;

b) Macroseismic scales.

Instrumental scales are based on ground motion parameters, such as peak ground acceleration, spectral velocity, and so on. Their use involves the following drawbacks:

- a large number of records is needed for both using and evaluating DPM's;

- there is no single quantity which gives a measure of the intensity well correlated to the damage;

- because of the lack of data on past earthquakes, it is difficult to make a reliable estimate of the seismic hazard in terms of instrumental scales.

Macroseismic scales are undoubtedly less accurate and, apparently, should lead to less sharp distributions. However, from a practical viewpoint, they present the following advantages:

- it is possible to estimate the average intensity at a site directly from the damage data;

- the intensity is univocally defined by a single quantity;

- the seismic hazard of a given site is known, at least in Italy, in terms of macroseismic scale.

All of the above considerations make macroseismic scales the only practicable measure of intensity for DPM's.

Intensity (Microzonation)

The macroseismic intensity gives an average value in a territory where, due to local amplification effects, even marked intensity variations can occur. As marked local effects lead to marked dispersion in DPM's, the importance of local effects should be, at least approximately, estimated, both when evaluating DPM's from statistical data and when using DPM's to plan retrofit or repair operations.

Typology

The typologies of DPM's should be defined through those characteristics which effect the seismic behavior of the buildings, such as kind of structure, strength of materials, sizes of structural elements, and so on.

In accordnace with the two general requirements, only those parameters should be selected which

- more markedly affect the seismic behavior,

- can be easily surveyed,

- are highly correlated to parameters that directly affect the seismic behavior, but which cannot be easily surveyed.

With particular reference to the last point, it seems opportune to define regional typologies which, for historical and natural reasons, have got well defined characteristics correlated to few parameters.

Damage

There are two possible definitions of damage:

- in terms of repair cost (usually divided by rebuilding cost);

- in terms of conventional indices of damage.

Regarding the evaluation of DPM's, the first definition requires the knowledge of the repair costs for buildings damaged by past earthquakes. These data are available, if at all, only some years after the earthquake.

Regarding the use of DPM's, the first definition brings on further uncertainties and errors due to (4):

- cost fluctuation;

- kind of strategy and techniques of repair and retrofit;

- local condition of work.

The second definition eliminates the difficulties and seems to be more suitable, although it present the following drawbacks:

- it requires a further step (cost evaluation) in the use of DPM's;

- it requires the standardization of the damage scale;

- it requires the definition of a unique damage index which describes the overall damage and which can be easily converted into monetary costs.

Shape of $P[D|T,I]$

Two kinds of distribution can be assumed:

a) Parametric distributions;

b) Nonparametric distributions.

In statistics, parametric distributions are assumed when the phenomenon can be described by a particular distribution (i.e. trials of a coin described by a binomial distribution) or when the shape of the experimental hystograms is similar to the shape of a particular distribution. Only the second criterion is suitable for PDM's.

The advantage of assuming a parametric distribution lies in the great simplification in describing $P[D|T,I]$ and in the lesser dependence of $P[D|T,I]$ on the statistical sample at disposal.

In nonparametric distributions, each mass probability p_i can assume any value $0 \leq p_i \leq 1$ satisfying the natural constraints $\Sigma_i p_i = 1$ and other external constraints.

Nonparametric distributions give better results in terms of goodness of fit, but they are strongly dependent on the sample at hand.

DPM's FROM NOVEMBER 30, 1980 EARTHQUAKE DATA

The previously stated principles have been applied to evaluate DPM's from the data collected after the earthquake of November 23, 1980 in Southern Italy (5). The data are relevant to about 38,000 buildings be longing to 41 administrative units which were surveyed by the Italian army. The buildings were surveyed by making use of a survey form and a survey manual. A statistical algorithm, based on a maximum likelihood approach, has been set up (5,6,7). It identifies the seismic intensity at each site, singles out typologies showing similar seismic behavior and groups them into classes of vulnerability, evaluates DPM's of classes of vulnerability and of typologies, and performs tests on various hypotheses (8). The following choices have been made in applying the model:

- the seismic intensity is measured through the macroseismic scale M.S.K.-76 (9);

- the seismic intensity is relevant to the territory of an entire administrative unit or, alternatively, of a single village assumed as an isoseismic unit. Because of the lack of data, there is no possibility of making a more effective microzonation;

- the typologies are defined by the combination of two factors: type of vertical structure, type of horizontal structure (see Figure 1);

- only non earthquake-resistant buildings have been considered;

- the damage to each building is quantified by an index which varies according to the damage level in the vertical structure;

- parametric (binomial) and, alternatively, nonparametric probability distributions of damage P.D.T,I. are assumed for the vulnerability classes, while only nonparametric distributions are assumed for the typologies.

The need for a statistical algorithm, such as the adopted one, comes from the difficulties in evaluating the intensity at each site by simply using the directions of M.S.K.-76 or any other macroseismic scale. The main difficulties arise because of the vagueness in the definitions of

Horiz. Structure \ Vertical Struct.	Field stones		Hewn stones		Brick masonry		Reinf. concrete	
	cl.	numb.	cl.	numb.	cl.	numb.	cl.	numb.
Vaults	A	1532	A	617	A	16	/	/
Wooden floors	A	8860	A	3294	B/C	132	/	/
Steel floors	B	5216	B	2323	C	468	/	/
R/C floors	B/C	855	C	2069	C	601	C	3383

Fig. 1 — Building typologies and classes of vulnerability.

vulnerability classes, of damage levels, of probabilistic distributions of damage, and because of inconsistencies between expected and actual correlations between damage to buildings of different vulnerability classes.

In Figures 2, 3, 4, 5, the DPM's relevant to vulnerability classes defined by the M.S.K.-76 scale are shown. The membership of the 13 typologies to the three classes, as it has been found by the statistical algorithm, is shown in Figure 1; typologies R/C floors -- field stones and wooden floors -- brick masonry have been classified in class C when considering isoseismic units coincident with administrative units, and damage distributions described by binomial distributions, in class B in all other cases. In Figures 2 and 3 are shown the DPM's obtained by assuming the isoseismic units coincident with the administrative units and with the villages, respectively, and by assuming a binomial and a nonparametric (dashed line) distribution. In Figures 4 and 5 are shown the DPM's obtained by assuming again the isoseismic units coincident with the administrative units and with the villages, respectively, and by assuming a binomial distribution only, but the macroseismic scale is partitioned into half grades. For this partitioning, only intensities from 5.5 to 7.5 have been found in the sample.

As can be seen from Figures 2 and 3, there are no strong differences between binomial and nonparametric distribution for intensities 5,6,7, while there are marked differences for intensity 8. This bad fitness can be explained from two different points of view. First, since the sample relevant to intensity 8 is composed of buildings of only two administrative units, and therefore is not sufficiently large to eliminate random fluctuation of distributions, irregular frequency damage distributions are, because of the sample, to be expected. On the other hand, high earthquake intensities emphasize differences among buildings of the same typologies more than low earthquake intensities do. For example, when the behavior of R/C buildings is quasi-elastic (intensities 5,6,7), and therefore ductility is not called for, little differences between behavior of well and badly reinforced buildings are to be expected, while the same is not true when the ductility plays an important role (intensities greater than 7). The possibility of using the binomial distribution for high intensities should therefore be investigated further on different data.

As for differences of distribution relevant to administrative units (Figures 2,4) and villages (Figures 3,5), no marked differences in dispersion are revealed by the diagrams. By taking into account the results and the observations presented in References 6 and 7, it can be said that the microzonation performed by dividing administrative units into villages is not yet sufficiently refined to single out local amplification effects when villages are big and located in mountains, as in the case of most of the surveyed villages.

In Reference 10, the DPM's relevant to the 13 typologies of Figure 1 can be found. They are relevant to the hypotheses of isoseismic units coincident with administrative units and villages, respectively. They have been calculated as relative frequencies by grouping buildings belonging to isoseismic units with the same seismic intensity. In analogy with DPM's relevant to vulnerability classes, they are smooth for intensities up to 7, while they show some irregularities, and particularly bimodality, for intensity 8. Again there is not such a difference between DPM's relevant to administrative units and DPM's relevant to villages.

REFERENCES

(1) Whitman R.V., Reed J.W., Hong S.T.: "Earthquake Damage Probability Matrices", Proceedings of the 5th World Conference on Earthquake Engineering, Rome 1973.

(2) Whitman R.V., Heger F.J., Luft R.W., Krimgold F.: "Seismic Resistance of Existing Buildings", Journal of the Structural Division, ASCE ST7 1980.

(3) Dolce M.: "Damage Statistical Matrices for Italian Low-Rise Buildings", 7th International Symposium on Earthquake Engineering, Rome 1982.

(4) Braga F., Dolce M.: "Un approccio statistico alla previsione dei danni prodotti dal sisma sugli edifici", Rend. Soc. Geol. It., 4, 1981.

(5) Braga F., Dolce M., Liberatore D.: "Southern Italy November 23, 1980 Earthquake: A Statistical Study on Damaged Buildings and an Ensuing Review of the M.S.K.-76 Scale", presented at the 7th ECEE, Athens 1982. Published by E.S.A., Rome 1982.

(6) Braga F., Dolce M., Liberatore D.: "Influence of Different Assumptions on the Maximum Likelihood Estimation of the Macroseismic Intensities", Proceedings of the 4th International Conference on Applications of Statistics on Probability in Soil and Structural Engineering, Florence 1983.

(7) Dolce M., Liberatore D.: "Modelli statistici per una scala macrosisma: analisi comparativa sui dati del terremoto del 23-11-1980", Proceedings of the 2nd Conference, 'L'ingegneria sismica in Italia', Rapallo 1984.

(8) Krishnaiah P.R., ed. "Handbook of Statistics", Vol.1, North-Holland Publishing Co., 1980.

(9) Medvedev S.V.: "Seismic Intensity Scale M.S.K.-76", Publ. Inst. Geophis. Pol. Ac. Sc., A-6 (117), Varsaw 1977.

(10) Braga F., Dolce M., Liberatore D.: "Fast and Reliable Damage Estimation for Optimal Relief Operations", Proceedings of International Symposium on Earthquake Relief in Less Industrialized Areas, Zurich 1984.

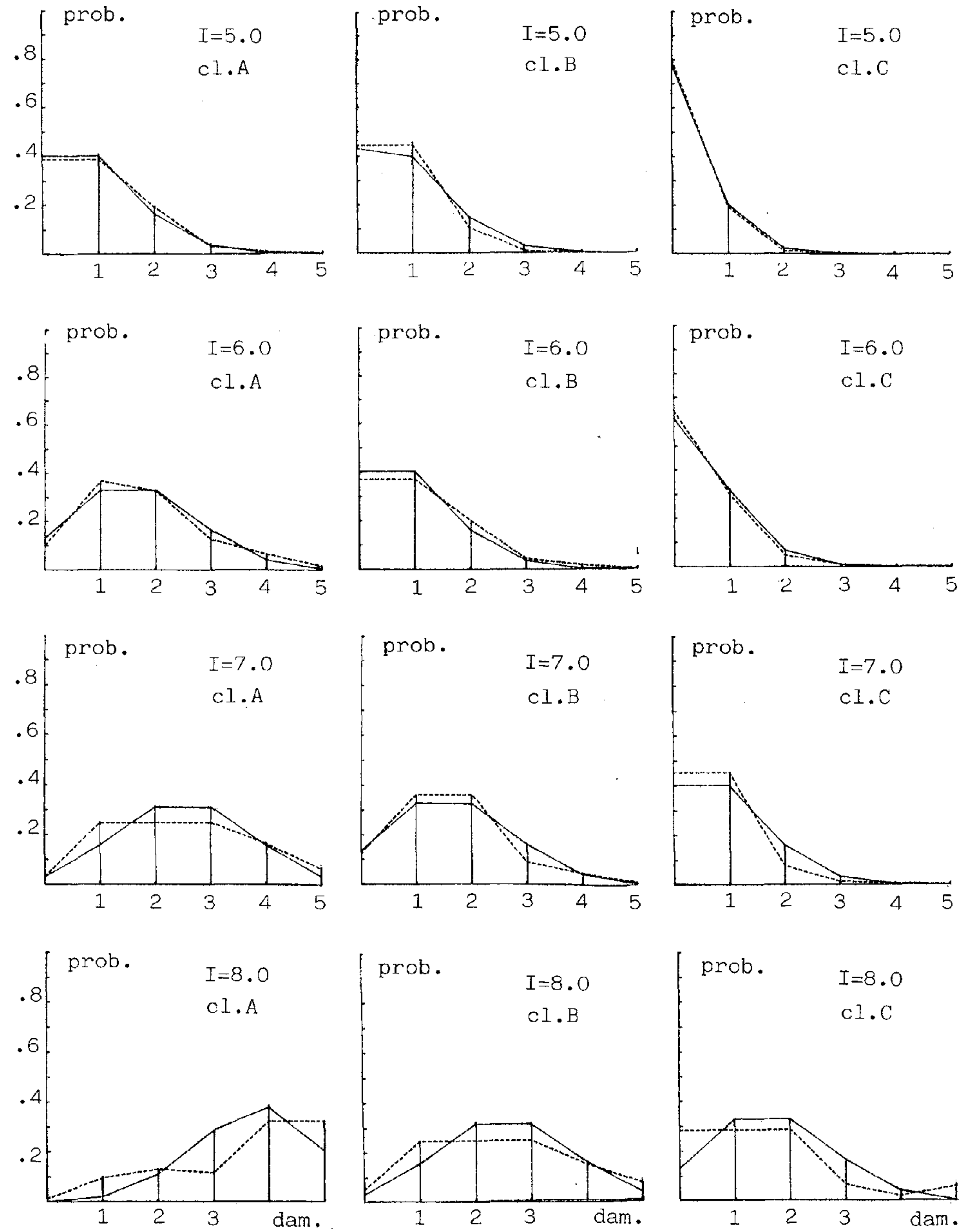


Fig. 2— DPM's for classes of vulnerability, assuming isoseismic units coincident with administrative units, and one grade intensity scale.

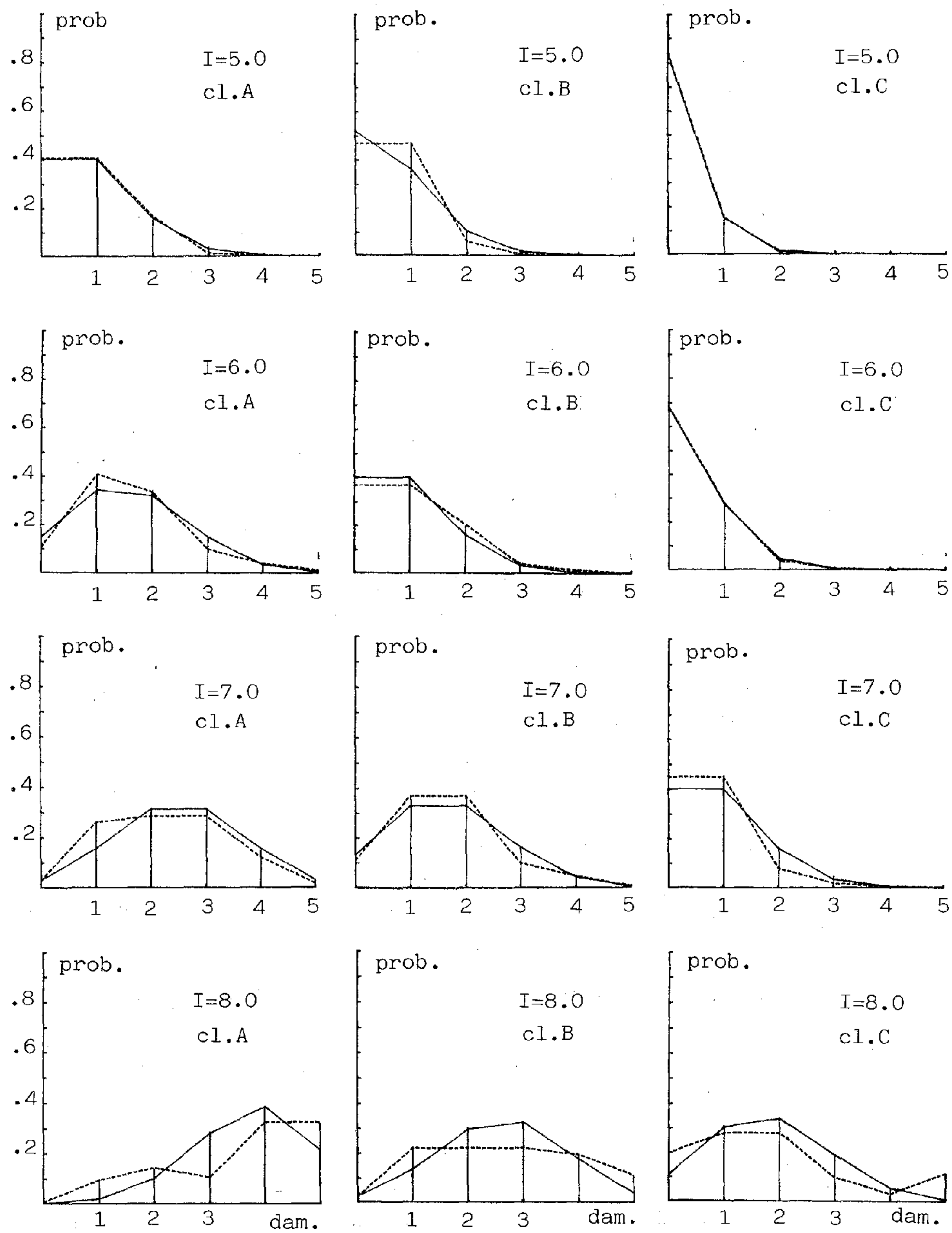


Fig. 3— DPM's for classes of vulnerability, assuming isoseismic units coincident with villages and half grade intensity scale.

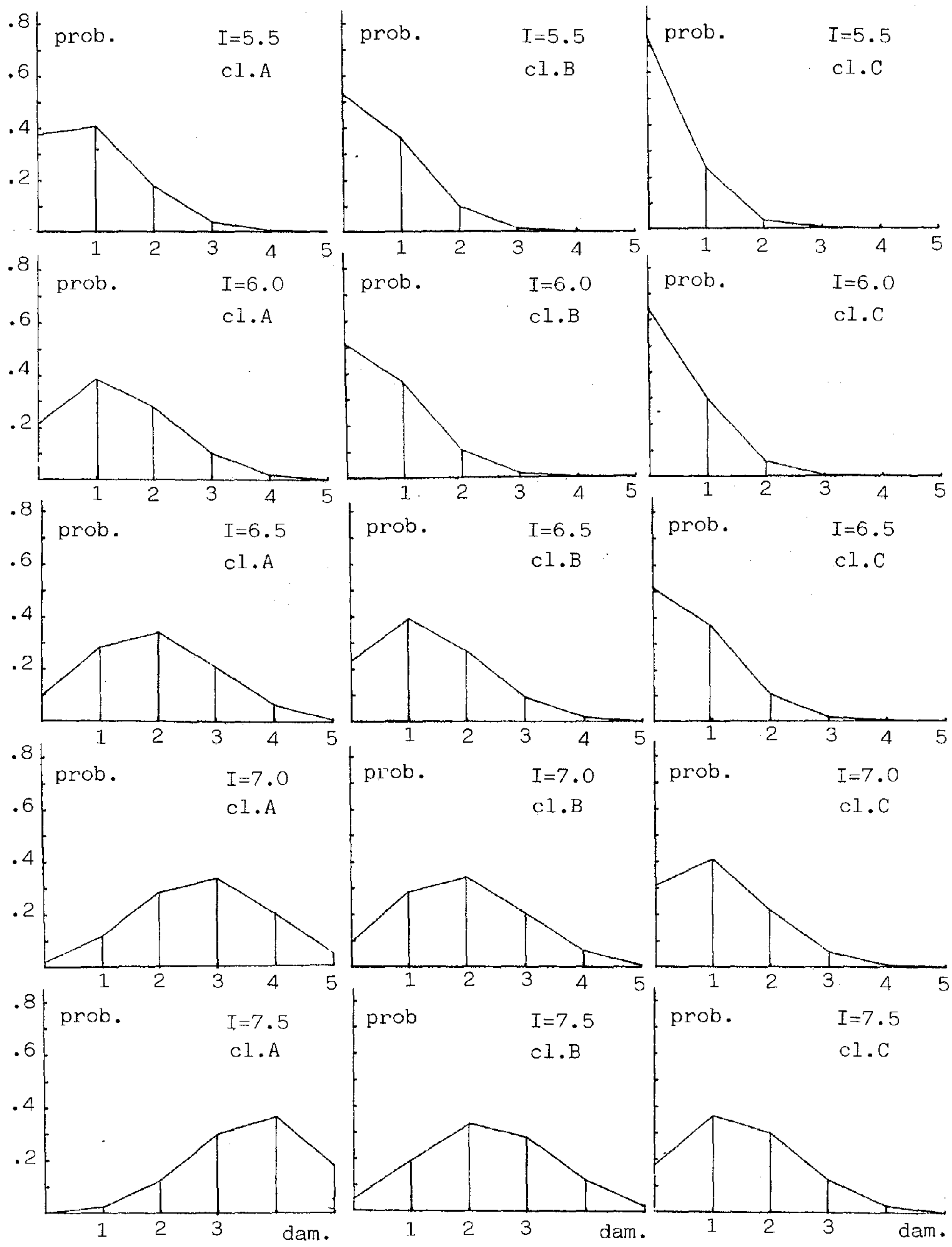


Fig. 4 – DPM's for classes of vulnerability, assuming isoseismic units coincident with administrative units and one grade intensity scale.

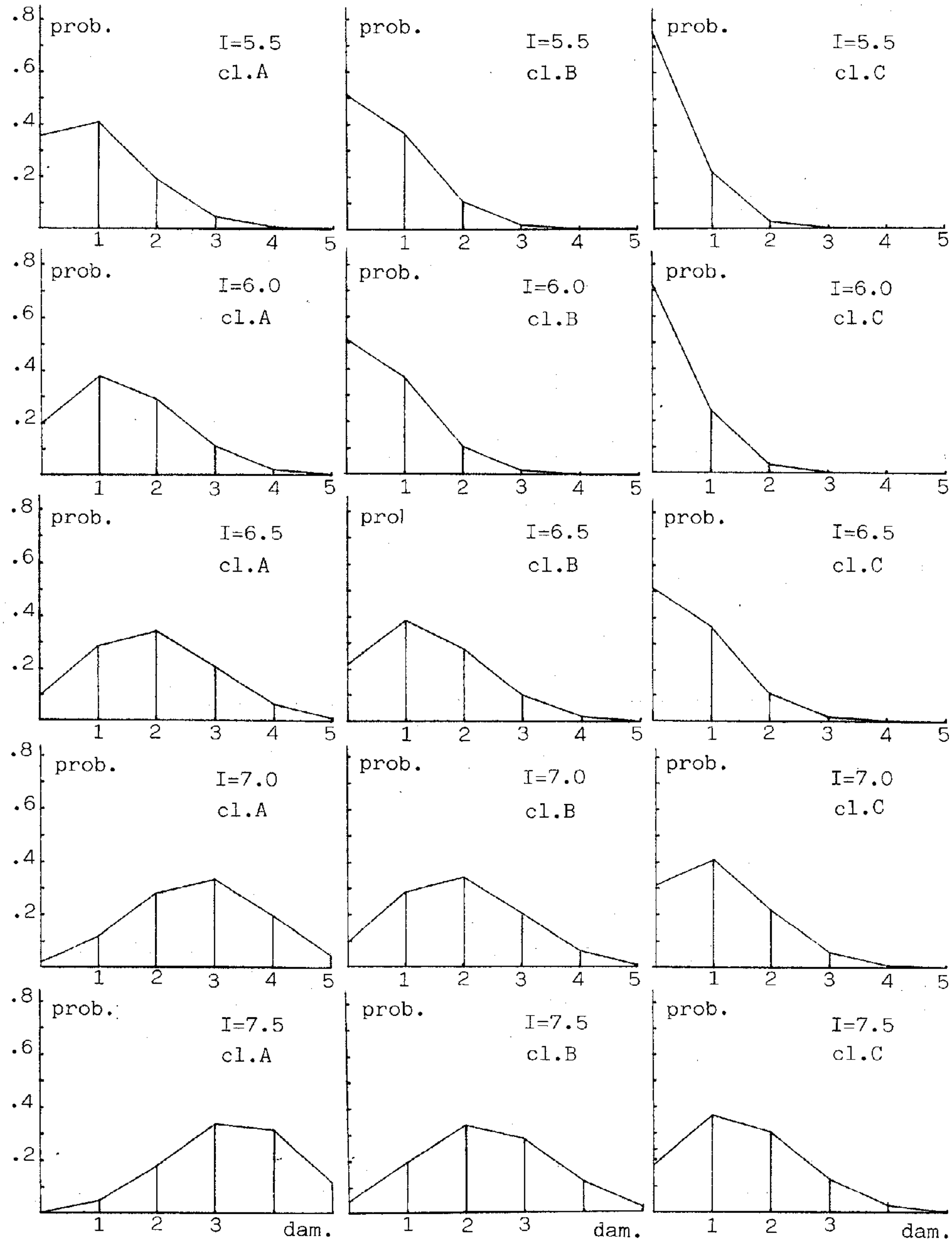


Fig. 5— DPM's for classes of vulnerability, assuming isoseismic units coincident with villages and half grade intensity scale.

Methodology for Mitigation of Seismic Hazards in Existing Unreinforced Masonry Buildings A Methodology

R.D. Ewing

SUMMARY

This paper summarizes the research that has led to the development of a methodology for mitigation of seismic hazards in existing unreinforced masonry buildings. The methodology is based on a program that combines analytical and experimental investigations and introduces several new concepts that are significant departures from existing seismic design recommendations and provisions.

INTRODUCTION

Building construction using unreinforced masonry (URM) predates the development of seismic criteria that guide the design and construction of present-day buildings. A substantial number of these URM buildings are still being used in areas considered seismically active, even though investigations of earthquake damage have confirmed that this type of building has been a major contributor to personal injury or loss of life during relatively high intensity earthquakes. Public agencies and the private sector are becoming more concerned about the potential for personal injury or death resulting from failure of these buildings. However, political jurisdictions struggling with limited budgets can rarely afford the extensive research programs required to develop rehabilitation standards. It is apparent that a system of analysis methods and procedures - a methodology - is needed for determining realistic hazard-mitigation requirements that will lead to cost-effective methods of retrofit to fill such requirements. In this way, the choice will not remain limited to either the enormous investment now required to make existing buildings conform to present standards for new construction or the economic loss resulting from the demolition of these buildings. Research can provide usable tools to meet seismic-hazard mitigation goals of cities squeezed between threats to life safety and economic constraints. The result of the research presented in this study is a methodology for the mitigation of seismic hazards in existing URM buildings.

BASIS OF THE METHODOLOGY

This paper reports on a study program that combines analytical and experimental investigations to develop a methodology for the mitigation of seismic hazards in existing unreinforced masonry buildings located in various seismic zones of the United States. The methodology is given in Reference 1a and will not be repeated here; however, the technical basis for the methodology is given in this paper.

Vice President, Agbabian Associates, El Segundo, California, USA

A review of existing research work on masonry, available at that time, showed that most of the effort had been directed toward determining the characteristics and response of reinforced masonry components to in-plane forces; and little or no effort had been devoted to typical URM building response and the dynamic interaction among the building components. Accordingly, a program plan for the development of the methodology was based on research that includes:

- Categorization of URM buildings (Ref. 1b)
- Seismic input (Ref. 1c)
- Dynamic testing of full-scale URM walls, out-of-plane (Refs. 1d and 1e)
- Static and dynamic testing of full-scale diaphragms, in-plane (Refs. 1f and 1g)
- Static and dynamic testing of URM, in-plane (Ref. 1a)
- The performance of URM buildings in past earthquakes (e.g., Coalinga, Imperial Valley, Eureka, San Fernando, etc.)
- Analysis methods that have been correlated with the tests (Refs. 1e and 1g)
- Anchorage between the walls and diaphragms (Ref. 1h)

As a result of this research, it was clear that the response of typical unreinforced masonry buildings is dynamic and highly nonlinear, and elastic or equivalent static procedures are not completely satisfactory to define their response.

NEW CONCEPTS

Based on the research conducted, the methodology incorporates several new concepts that are significant departures from existing seismic design recommendations and provisions; namely,

- Due to the nonlinear, dynamic response of URM buildings, the recommendations of the methodology are described separately for each seismic hazard zone in lieu of the use of a factored coefficient for correlation with seismic hazard zones (i.e., current code practice).
- Input ground motions for earthquake hazard zones are taken from the updated – although still tentative – seismic design provisions of the Applied Technology Council (Ref. 2). However, the ground motions are defined by mean values rather than upper bounds of motions.
- The seismic response model for the URM buildings is modeled as a rigid block on flexible soils. This basic response model is modified for URM walls with a limited story shear capacity.
- The velocity amplifications and the relative displacement response imparted to the URM walls, out-of-plane, is based on nonlinear, dynamic analyses that have been correlated with full-scale diaphragm tests in lieu of static analysis criteria.

- Dynamic stability concepts for URM wall elements subjected to out-of-plane motions are utilized in lieu of requirements for an elastic resistance capacity that is based on a prescribed static horizontal force.
- Materials resistance capacities are based on yield deformations, and inelastic behavior of materials is utilized in the recommendations.
- All existing materials and elements in the URM building that are distorted by relative horizontal or interstory displacement are considered in the response model and the structural resistance model.

Although not a new concept, the paramount consideration of the methodology is life safety. This is obtained by limiting building damage, and minimizing the probability of the separation of the URM walls and parapets from the floors and roof and the subsequent collapse of the gravity load-carrying system.

EVALUATION OF SEISMIC RESPONSE

The seismic response of URM buildings was evaluated by considering four related component responses and their interactions:

- In-plane motions of endwalls and crosswalls induced by the earthquake ground motion
- Roof and floor diaphragms subjected to in-plane motions induced by the endwalls
- Walls subjected to out-of-plane motions induced by the diaphragms and/or ground motion
- Anchorage between the walls and diaphragms

In-Plane Response of Walls

During an earthquake the ground motion is transmitted from the building/foundation interface through the endwalls (in-plane response) to the floor and/or roof diaphragms that drive the walls in the out-of-plane direction. Masonry shear walls can be considered rigid relative to the diaphragm stiffness and modeled as a rigid block resting on a soil. The soil is represented by bilinear, inelastic compression springs and impact dampers in order to provide damping if the wall separates and recontacts with the soil. Analyses performed with this model showed that, over a realistic range of building aspect ratios and soil stiffnesses, the ground motion is transmitted through the endwalls with little amplification. Additionally, the possible in-plane failure modes of the shear walls must be considered (e.g., pier rocking and/or diagonal compression failure). For diagonal compression failure, the in-plane strength can be best measured using the procedure shown in Figures 1 and 2, where the allowable shear strength corresponds to the 20 percentile values.

Roof and Floor Diaphragm Subjected to In-Plane Motions

The dynamic response of diaphragms is defined by an analysis model that has been correlated with full-scale, quasi-static, and dynamic tests (Refs. 1f and 1g). A typical quasi-static, cyclic load-deflection test of a wood diaphragm is shown in Figure 3, and has a nonlinear hysteretic behavior. The analytical model developed for this type of diaphragm is shown in Figure 4, where Figure 4a shows the overall force-deformation envelope and Figure 4b shows a typical cyclic load path. The model requires only two parameters to define the force-deformation envelope (i.e., the ultimate force capacity F_u and the initial stiffness K_i) and one parameter to define the degrading unloading stiffness. Typical URM buildings are modeled as shown in Figure 5, where the diaphragm stiffness is modeled by the nonlinear, hysteretic shear springs and the sidewall mass (the walls are assumed to crack) and tributary diaphragm mass are lumped at the nodes. Peak velocities at the top and bottom of the URM walls are obtained from the model shown in Figure 5, as well as relative deformations between the top and bottom of the walls.

It has been shown that the dynamic stability of the URM side walls subjected to out-of-plane motions is highly dependent on the peak velocities induced at the top and bottom of the wall by the diaphragm.

Walls Subjected to Out-of-Plane Motions

The dynamic stability of fully anchored walls subjected to out-of-plane motions was determined from full-scale testing (Refs. 1d and 1e). An analysis of collected data led to the formulation of dynamic stability criteria shown in Figure 6. The parameters that affect stability are:

- SRSS of the velocities imparted by the diaphragms to the ends of the URM wall.
- The ratio of weight of wall in the stories above the story under consideration, O, to the weight of the wall in the story under consideration, W.
- The height/thickness (H/t) ratio of the wall in the story under consideration.

The methodology uses the data given in Figure 6 to establish allowable H/t ratios for URM walls at various elevations in a building depending on the construction of the floor and/or roof diaphragms.

Anchorage Between Walls and Diaphragm

Adequate anchorage of the URM walls to the diaphragm is an essential part of achieving hazard mitigation in URM buildings. Anchorage forces have been developed for use in the methodology that are based on tests and nonlinear, dynamic analyses.

ACKNOWLEDGEMENTS

This research was conducted by ABK, A Joint Venture, for the National Science Foundation under Contract No. NSF-C-PFR-78-19200 and Grant No. CEE-8100532. The Joint Venture ABK consists of the three firms, Agbabian Associates (AA), S.B. Barnes & Associates (SBB&A), and Kariotis & Associates (K&A), all in the Los Angeles area. The principal investigators for the three firms are R.D. Ewing for AA, A.W. Johnson for SBB&A, and J.C. Kariotis for K&A. Dr. J.B. Scalzi served as Technical Director of this project for the National Science Foundation and maintained scientific and technical liaison with the joint venture throughout all phases of the research program. His contributions and support are greatly appreciated.

REFERENCES

1. ABK, A Joint Venture. Methodology for Mitigation of Seismic Hazards in Existing Unreinforced Masonry Buildings, El Segundo, CA: Agbabian Associates.
   (a) The Methodology, ABK-TR-08, Jan 1984.
   (b) Categorization of Buildings, ABK-TR-01, Dec 1981.
   (c) Seismic Input, ABK-TR-02, Dec 1981.
   (d) Wall Testing, Out-of-Plane, ABK-TR-04, Dec 1981.
   (e) Interpretation of Wall Tests, Out-of-Plane, ABK-TR-06, Mar 1982.
   (f) Diaphragm Testing, ABK-TR-03, Dec 1981.
   (g) Interpretation of Diaphragm Tests, ABK-TR-05, Mar 1982.
   (h) Anchorage, ABK-TR-07, Dec 1983.
2. Applied Technology Council. Tentative Provisions for the Development of Seismic Regulations for Buildings, ATC 3-06. Palo Alto, CA: ATC, 1978.

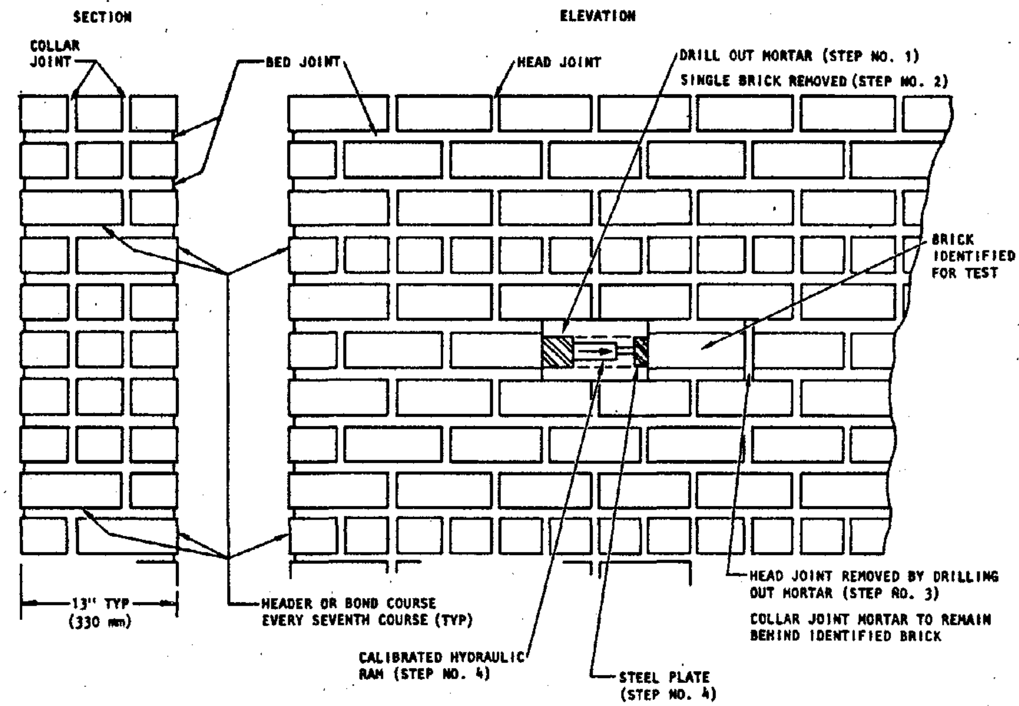


FIGURE 1. IN-PLACE SHEAR TESTS

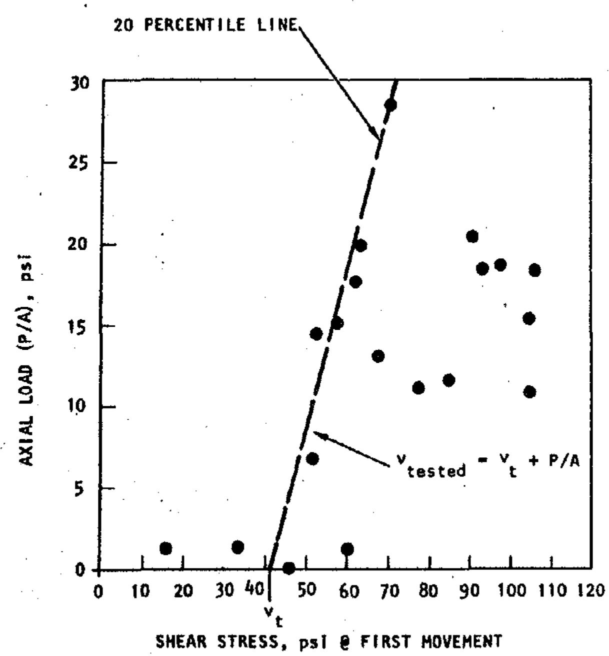


FIGURE 2. PROCEDURE FOR PLOTTING OF URM IN-PLACE SHEAR TESTS

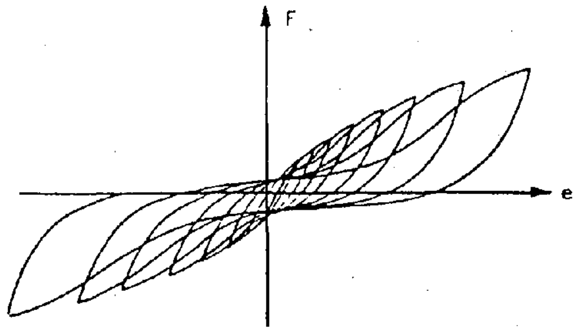
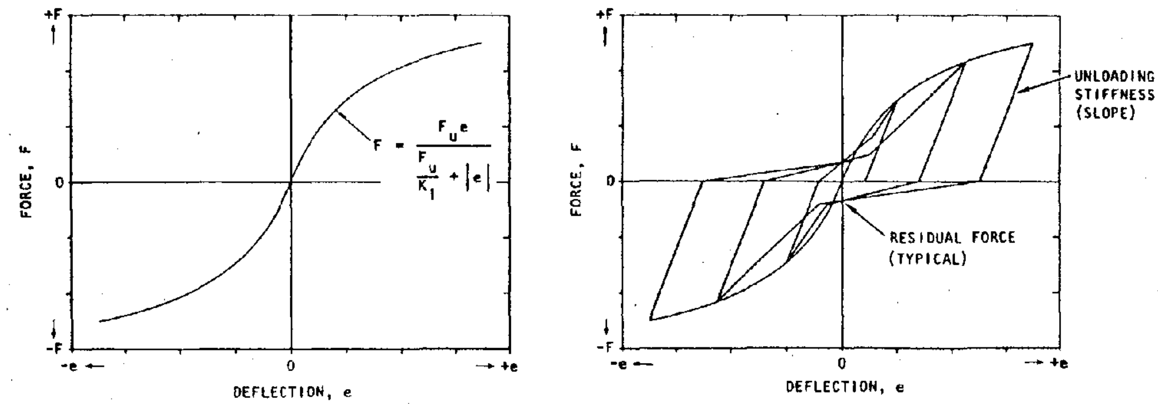


FIGURE 3. TYPICAL CYCLIC LOAD DEFLECTION DIAGRAM FOR PLYWOOD DIAPHRAGMS



(a) Force-deflection envelope of model

(b) Typical cyclic load-deflection diagram for model

FIGURE 4. LOAD DEFLECTION MODEL FOR WOOD DIAPHRAGMS

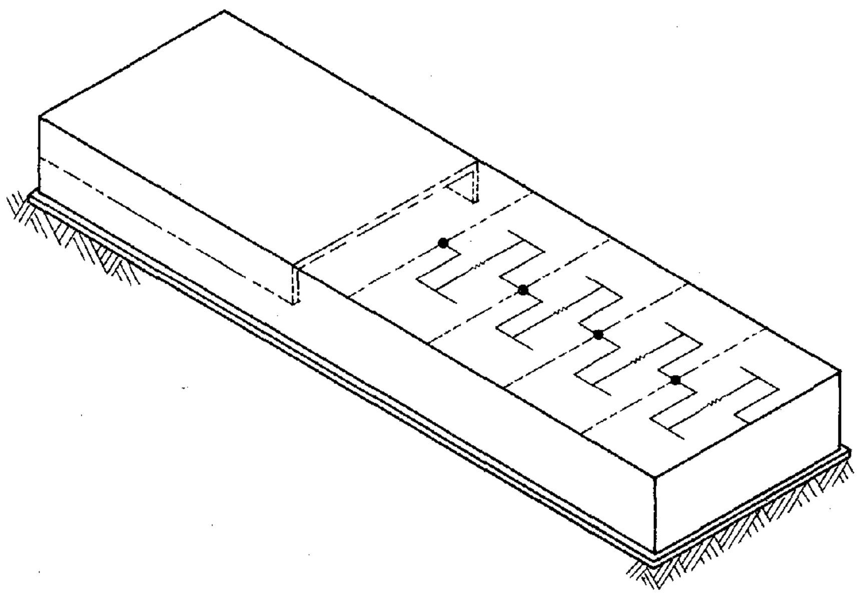


FIGURE 5. ANALYTICAL MODEL FOR URM BUILDING

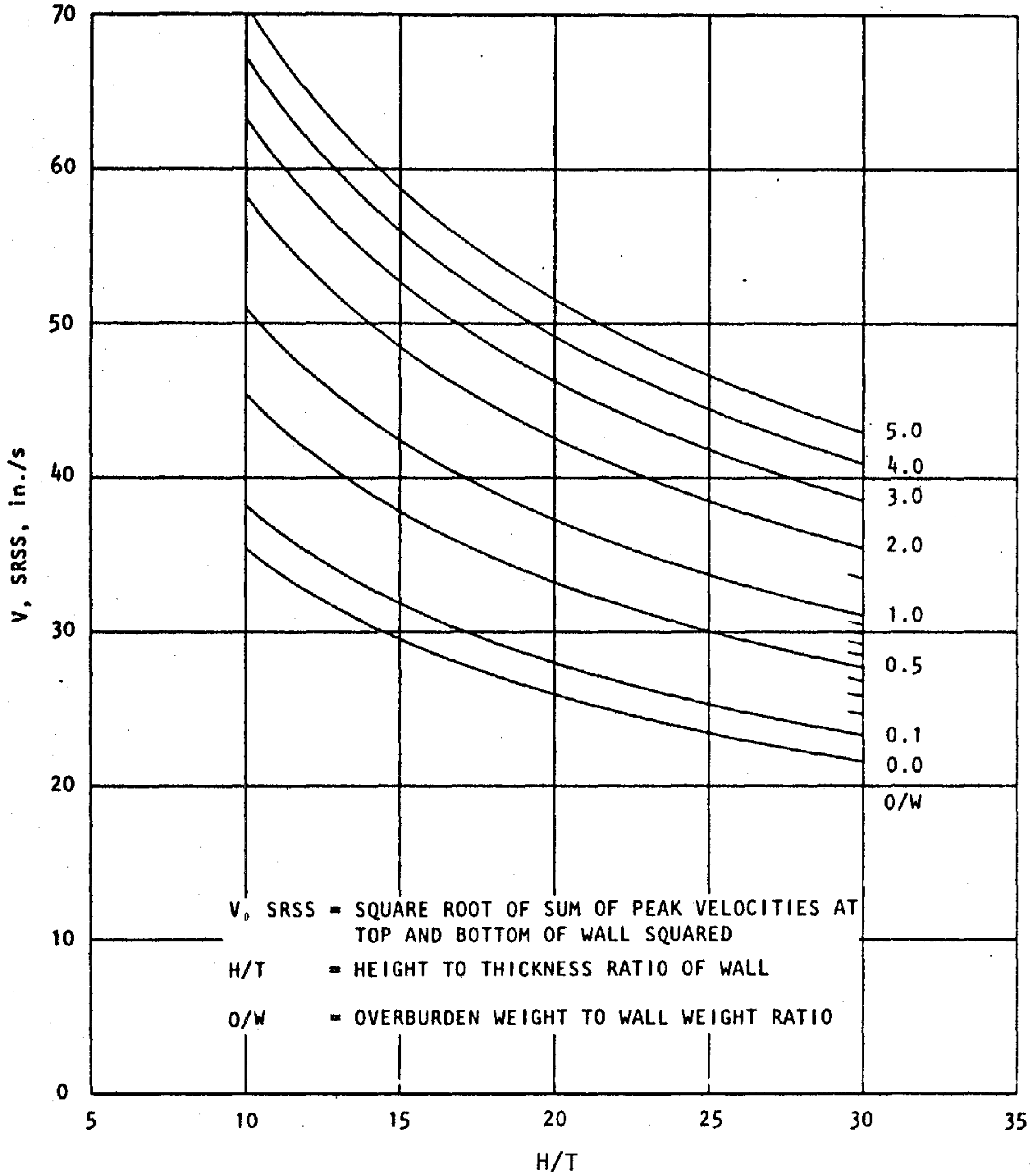
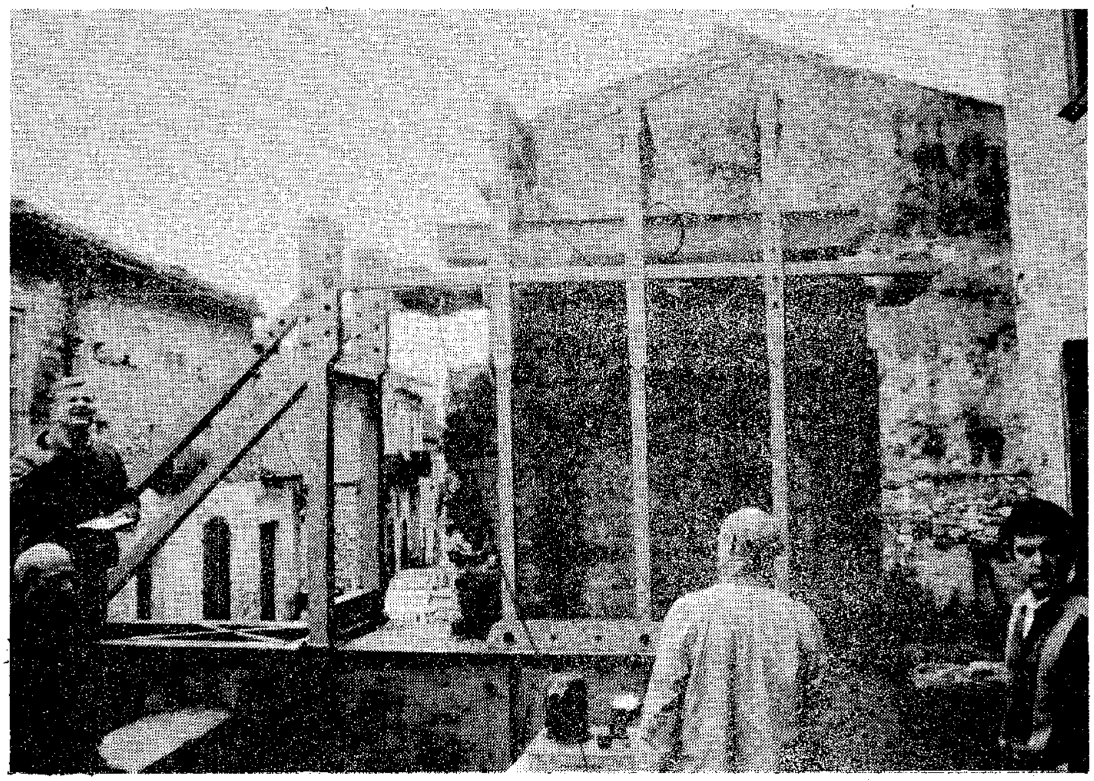


FIGURE 6. UNREINFORCED MASONRY WALL STABILITY CRITERIA, 98% PROBABILITY OF SURVIVAL

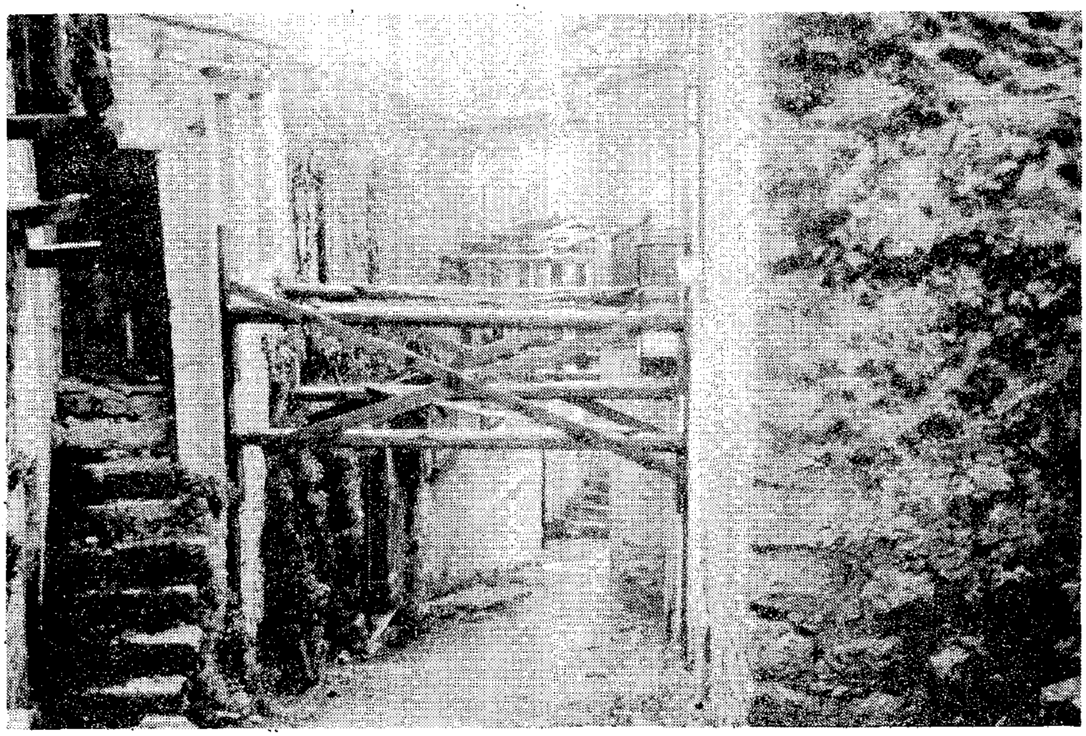
Representative photographs of site visits to Italian hilltowns taken during field study period of workshop. (Photographs: James Stratta, 1984)



Detail view of portable testing equipment for on site assessment of existing masonry wall, San Gregorio Magno. (Photograph: J. Stratta)

General view of portable testing equipment. (Photograph: J. Stratta)

Typical damage observed during site visits to hilltown areas.
(Photograph: J. Stratta)



Typical system of heavy timber bracing of adjacent buildings in a hilltown in southern Italy after the 1980 Irpinia earthquake.
(Photograph: J. Stratta)

INTENTIONALLY BLANK

Seismic Rehabilitation of Buildings Incorporating Earthquake Safety Limits

Gary C. Hart (I)
Roberto Del Tosto (II)
Thomas A. Sabol (III)

SUMMARY

The intent of this paper is to introduce important factors related to the definition and quanitification of an adequate level of safety for existing buildings, and to describe the methodology we employ to accomplish this task. As can be attested to by those involved in building rehabilitation, the problem of selecting an adequate level of safety is not one which lends itself to an easy solution. The selection process requires careful consideration of the information describing realistic maximum earthquake loads and structural response. The paper examines the uncertainty surrounding descriptions of earthquake loads and structural strength and describes the levels of safety implied by current code-based design. Two sections identify important characteristics of earthquake load estimation and structural response within the context of establishing an acceptable level of structural reliability. The Conclusion describes how we help to select and quantify an adequate level of structural safety for buildings.

INTRODUCTION

Rehabilitating structures in areas subjected to strong seismic forces is a design problem which places special structural, architectural and economic constraints on the feasibility of these projects. Those involved in rehabilitation have a responsibility to protect the historic fabric of the buildings while bringing the physical structure up to a level of strength sufficient to resist environmental loadings. Nowhere is this problem better characterized than in the effort to protect buildings from destruction wrought by earthquakes.

The traditional approach to safety in structural design has relied essentially on judgment and common sense to derive the basic provisions in contemporary building codes and material specifications. Design was

---

(I) Professor of Engineering, Department of Civil Engineering, University of California, Los Angeles, and Partner, Englekirk and Hart Consulting Engineers, Inc., Los Angeles, CA U.S.A.

(II) President, Englekirk, Hart and Del Tosto Consulting Engineers, Inc., Rome, Italy

(III) Project Engineer, Englekirk and Hart Consulting Engineers, Inc., Los Angeles, CA U.S.A.

seen as an essentially deterministic process in that the uncertainty known to exist in the real world was not explicitly incorporated into the development of the code-specified design methods. We believe that deterministic approaches to structural engineering, particularly in those projects involving seismic rehabilitation of structures, are not appropriate or desirable because the nature of structural resistance and loading is probabilistic.

Experience suggests that there is a significant contribution to be made by the application of risk analysis during the seismic rehabilitation of structures versus the application of traditional design approaches. Mandated design standards often have incorporated into their provisions several layers of conservatism. Loading estimates are increased to account for extreme value loadings while material and structural member strength estimates are decreased to account for low quality construction. These provisions often result in structures with what some regard as excessive levels of safety which can cause significant problems in projects involving seismic rehabilitation because of the budgetary and programmatic constraints present. Often the conflict between excessive seismic strengthening and the budget or program results in the cancellation of the project.

It is to no one's advantage to design a project which cannot be constructed, and it is for this reason that we believe the application of risk analyses to seismic rehabilitation projects shows significant promise toward reducing unnecessary conservatism and cost. In addition, we have found that the application of these risk analysis techniques concentrates our attention on the areas of the structure which form the critical links in the seismic system allowing us to assess better the structural impact of our design recommendations. Other benefits can accrue when the information collected during the risk analysis is used to provide a rational assessment of insurance premium levels and deductibles.

Historically there has been little provision for the incorporation of earthquake force resisting systems into buildings. This is because a practical understanding of the character of building response due to seismic forces and the incorporation of appropriate structural systems to resist these forces is a recent development. It is a fact, however, that much of the world and particularly certain areas of Italy are subject to strong seismic forces and buildings in these regions must be protected from significant seismic damage if the rehabilitation effort is to be worthwhile.

The conflict between historic practice and practical reality is intensified when the designer must establish the level of seismic force for which the structure should be designed. Selecting an earthquake force level which is too low leaves the structure vulnerable to extensive seismic damage. On the other hand, use of too high a seismic force may cause the project to be abandoned because of economic or scheduling constraints. A level of seismic safety must be selected which is consistent with the magnitude of risk inherent to the project.

It is possible to systematically characterize an acceptable level of safety for buildings in a manner which is consistent with the state-of-the-art design principles used to establish levels of safety in new buildings. Considerable work has been performed over the past decade to quantify the level of structural reliability possessed by contemporary structures. This measure of reliability can be inferred to represent a minimum level of structural safety which is acceptable to society. Use of these benchmarks of structural safety and the techniques by which they are derived form the background for defining and quantifying an adequate level of safety for buildings.

Quantifying structural reliability requires that we define the parameters which affect the structural performance of a building. We must establish material strength and behavioral properties, define relevant structural limit states and analytical models to be used and estimate the nature and magnitude of loading to which the structure will be subjected. Related to these factors is the length of time, or design life, we presume these buildings are going to remain functional.

INFLUENCES ON STRUCTURAL RELIABILITY

An essential goal in structural design is to produce a structural system with sufficient capacity to resist the effects of anticipated maximum loads imposed upon it during the life of the structure. Although this is a very straightforward concept, problems occur when we attempt to establish the magnitude and frequency of the "anticipated loads" for which the structure must be provided with "sufficient capacity" to resist. It appears, then, that there are two fundamental questions which must be answered so that buildings can be seismically retrofitted at economical cost: What are the anticipated loads and how should the capacity of the structural member be established.

To answer the first question, we need to know how to describe and quantify the nature and magnitude of the load. If we are addressing the question of earthquake loads, an exact answer to our question requires that we know the level of force to which the building will be subjected during its design life. If we knew in advance that the building would never experience significant earthquake forces, we could confidently eliminate nearly all of the lateral force resisting system from the design. If, on the other hand, we knew with certainty the maximum level of seismic force the building would experience during its lifetime, we could design an earthquake system with a capacity just equal to this level of seismic shaking secure in the knowledge that the building was safe. The advantages of this perfect information are as obviously useful as they are unlikely of ever being realized. Unfortunately, we know none of these things. Yet, we must design buildings in the face of this uncertainty and must do so with a final maximum level of risk which is acceptable to society. This suggests the use of probability theory to quantify this level of uncertainty.

The traditional approach to safety in structural design has relied essentially on judgment and common sense to derive the basic provisions in contemporary building codes and materials specifications. Design was seen as an essentially deterministic process in that the uncertainty known to exist in the real world was not explicitly incorporated into the development of the code-specified design methods. We believe that deterministic approaches to structural engineering, particularly in those projects involving the rehabilitation of structures, are not appropriate or desirable because the nature of structural resistance and loading is probabilistic. It then follows that the application of probabilistic concepts to structural engineering problems should result in a model which more closely approaches the true behavior present in the real world.

The design strength given by any of the design equations contained in contemporary building codes is considered to be a deterministic quantity in that the equation gives the designer but one value of resistance (e.g. moment capacity, shear strength, etc.) for a given set of material properties (e.g. material strengths, area of reinforcing steel, etc.). This number does not represent the value of actual structural resistance and until the member is loaded to failure only speculative statements can be made regarding the member's true strength. Similarly, only probabilistic statements may be made regarding actual loads that might be imposed on the shear wall during its design life.

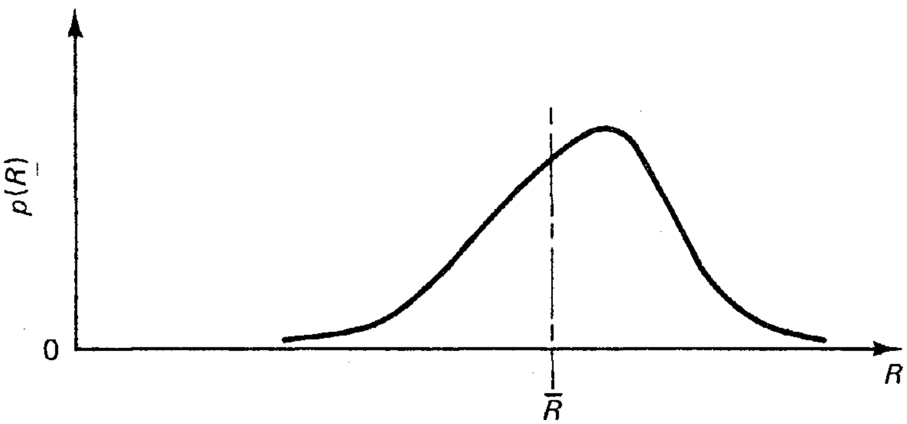
The structural engineer must, however, make predictions regarding the maximum anticipated loads which the structure will experience as well as attempt to establish the capacity of the member at a given structural limit state. Probability is essential in making these predictions in that probabilistic methods explicitly recognize that all predictions of the future have some level of uncertainty associated with them. These methods model reality by recognizing the observed scatter, randomness and uncertainty present in actual designs and quantify it using probability theory.

We can graphically represent the randomness in the resistance (R) and the load effect (U) described above and shown in Figure 1 using a probability density function (PDF). The average or mean value of the of the resistance and load effect are given by $\bar{R}$ and $\bar{U}$, respectively. In Figure 1a, the horizontal axis represents the values of R while the vertical axis provides the ordinates to calculate the probability of a value of R falling between two limits, R_i and R_j. The probability of R having a value between R_i and R_j is equal to the area under the PDF between R_i and R_j. Similar comments apply to Figure 1b.

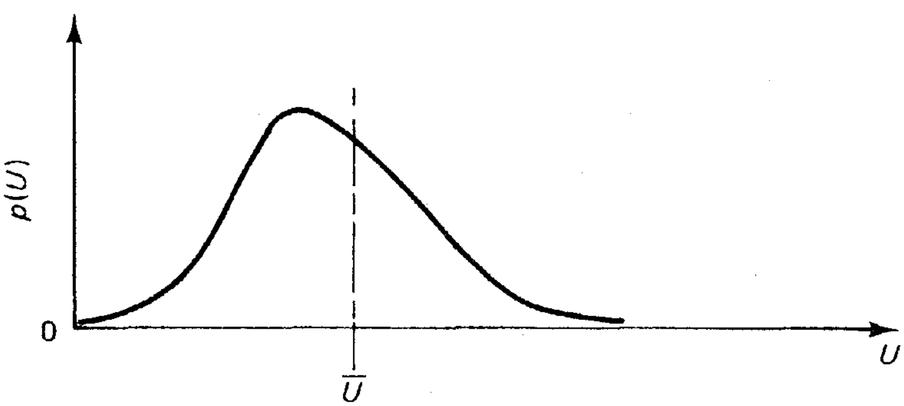
If failure is described as the condition where the capacity of the member, R, is equal to or exceeded by the specified load effect, U, then failure occurs when R minus U is less than or equal to zero; that is

$$F = R - U \tag{1}$$

where F is the safety margin.

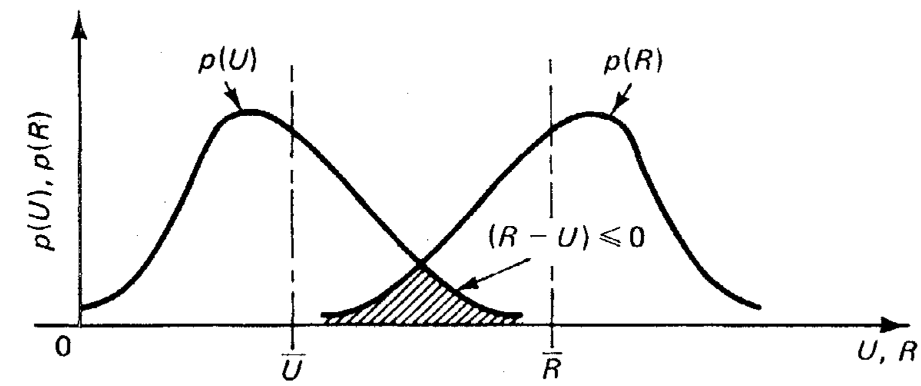


(a) Probability Density Function of Resistance, *R*

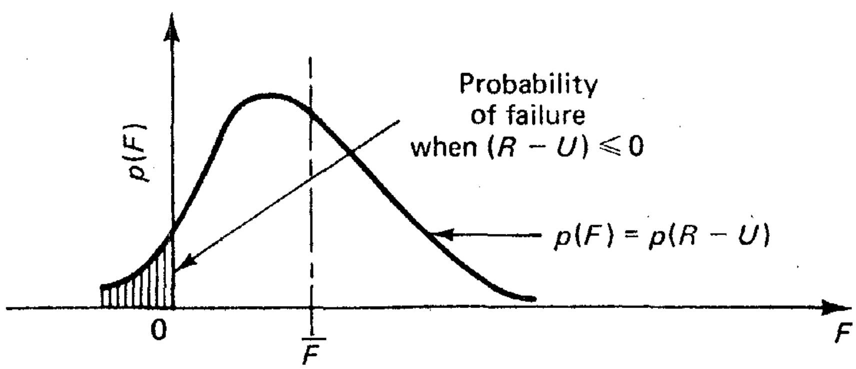


(b) Probability Density Function of Load Effect, *U*

Figure 1
Probability Density Functions (PDF) of R and U



(a) PDFs of U and R



(b) PDF of the Safety Margin, F

Figure 2
Probability of Failure

Thus, failure occurs when $F \leq 0$. This situation may be represented graphically by considering the two PDFs shown in Figure 1. If the two are superimposed on the same set of axes, as shown in Figure 2a, the shaded area represents the condition where $(R-U) \leq 0$. If the two PDFs representing R - U in Figure 2a are then expressed as one PDF in terms of U, the probability of failure is the shaded area to the left of zero in Figure 2b.

It may be seen from Figure 2a that the failure condition requires two separate events to occur before the member is judged to have failed because failure is a function of both capacity and load. Consequently, failure occurs when a member of moderately low strength is loaded with a very high level of load or a very low strength member is loaded with a moderately high load. As a result, it can be seen that the occurrence of an extremely high load does not necessarily represent a failure condition unless combined with a member of sufficiently low capacity.

The variability of the values about the mean value of the safety margin, F, is quantified by the standard deviation, sf. The standard deviation represents a measure of the spread of the data. A given value of F may be described by how many standard deviations it is away from the mean. For example, the mean of F is zero standard deviations from the mean while an extreme value of F might be at least three or four standard deviations above, or below, the mean. It is assumed that for a given value of F, the greater the number of standard deviations it is above or below the mean, the lower the probability that such a value of F will occur. Consequently, the more unlikely it is that a value of F will be less than or equal to zero, the more unlikely it is that the member under consideration will fail as it thus possesses greater reliability.

If the values of F and s are known, it is possible to define another term which gives an indication of the reliability of a particular element or structural system. The <u>reliability index</u>, β, is defined as

$$\beta = \overline{F}/s \qquad (2)$$

The reliability index has two fundamental advantages over conventional methods of reliability analysis. It respects the probabilistic nature of the structural design problem and it enables one to address safety and reliability without directly quantifying the probability of failure.

The advantage of the last observation may be more clearly understood if one considers that the load and resistance effects leading to structural failure occur at the extreme ends of the PDF describing R and U. The probability of failure is very sensitive to the PDF used to describe the distribution of the values of resistance and load effects because of the influence of the values at the extremes. The selection of different PDFs may result in changes in the probability of failure by several orders of magnitude. By avoiding the explicit specification of

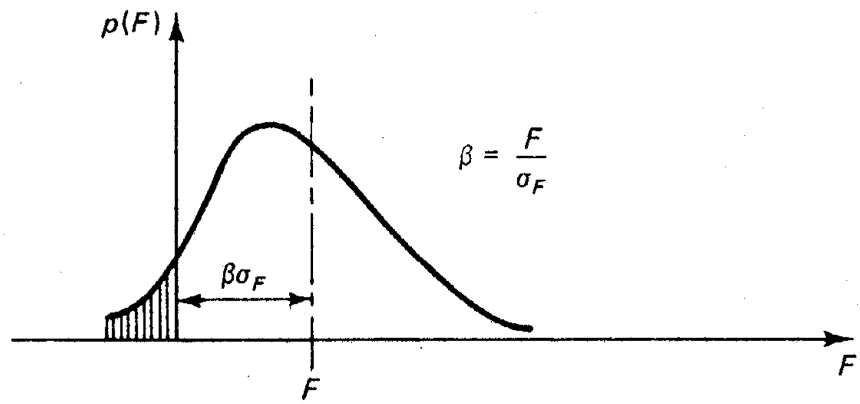


Figure 3
Reliability Index

the probability of failure and relying on the reliability index, a more robust estimate of structural reliability may be obtained.

It has been shown that most designs do not change significantly as the probability of failure is modified over a fairly wide range and measures of reliability not heavily dependent on the extreme tails of the PDFs describing the structural system should be used [1]. The reliability index is such a measure.

The reliability index is a measure of structural safety and with only the mean and standard deviation of the safety margin, the value of β may be determined. The reliability index indicates how many standard deviations below the mean of F are required before combinations of load and resistance effects will lead to structural failure, as shown in Figure 3. In other words, the greater the value of β, the greater the structural reliability and the smaller the probability of failure.

Work conducted over the past decade indicates that there are levels of structural reliability that are consistently attained by contemporary code-mandated designs. Typical values of β present in members designed to current masonry, concrete and steel design codes are shown in Table 1.

Another result of the analysis of the data collected during this research was the identification of consistent levels of structural reliability for major loading conditions. For example, it has been found that a value of $\beta = 3.0$ is consistent with current practice for load combinations involving dead and live or dead plus snow loads while $\beta = 2.5$ and 1.75 were representative for combinations describing wind and earthquake loads, respectively.

These benchmark values of structural reliability are based on contemporary practice. Knowing these values, particularly the reliability index for seismic construction, permits a rational comparison between the reliability provided by the proposed design and contemporary practice. If the designer feels that a higher level of reliability is desirable, the effect of strengthening the design on increasing β can be established. The ability to examine this relationship is an important advantage to this approach to quantifying adequate levels of safety when compared to existing methods.

It should be noted that the safety margin is dependent on the values of the load and resistance effects and it is possible to express the value of the β-index in terms of R and U if R and U are uncorrelated random variables. If the mean and the standard deviation of the load effects are given by U and s_U, the values of F and s_F are

$$\overline{F} = \overline{R} - \overline{U}$$

and (3)

$$s_F = (s^2{}_R + s^2{}_U)^{0.5}$$

TABLE 1

TYPICAL VALUES OF THE RELIABILITY INDEX
β, FROM CURRENT DESIGN CODES

Loading Combinations		
Dead plus Live (or Snow)	3.0	
Wind	2.5	
Seismic	1.75	
Materials	**Columns**	**Beams**
Reinforced Brick Masonry	6.0 - 8.5	7.5 - 8.5
Reinforced Concrete	2.6 - 4.3	2.6 - 3.8
Steel (Ultimate)	1.9 - 3.0	2.7 - 4.6

It can be shown that substituting Equation (3) into Equation (2) we obtain an expression for β in terms of R and U directly

$$\beta = \frac{\overline{F}}{s_F} \frac{\overline{R} - \overline{U}}{(s^2_R + s_U)^{0.5}} \tag{4}$$

Practical applications of this technique include the recent revision of load factors and loading combinations sponsored by the American National Standards Institute in their ANSI A58.1 (L982) 2 . These load factors are under active consideration for adoption by the American Concrete Institute 3 as well as the American Institute of Steel Construction 4 . A strength design code for concrete masonry 5,6 utilizing these load factors has already been recognized by the International Conference of Building Officials, publishers of the Uniform Building Code 7 .

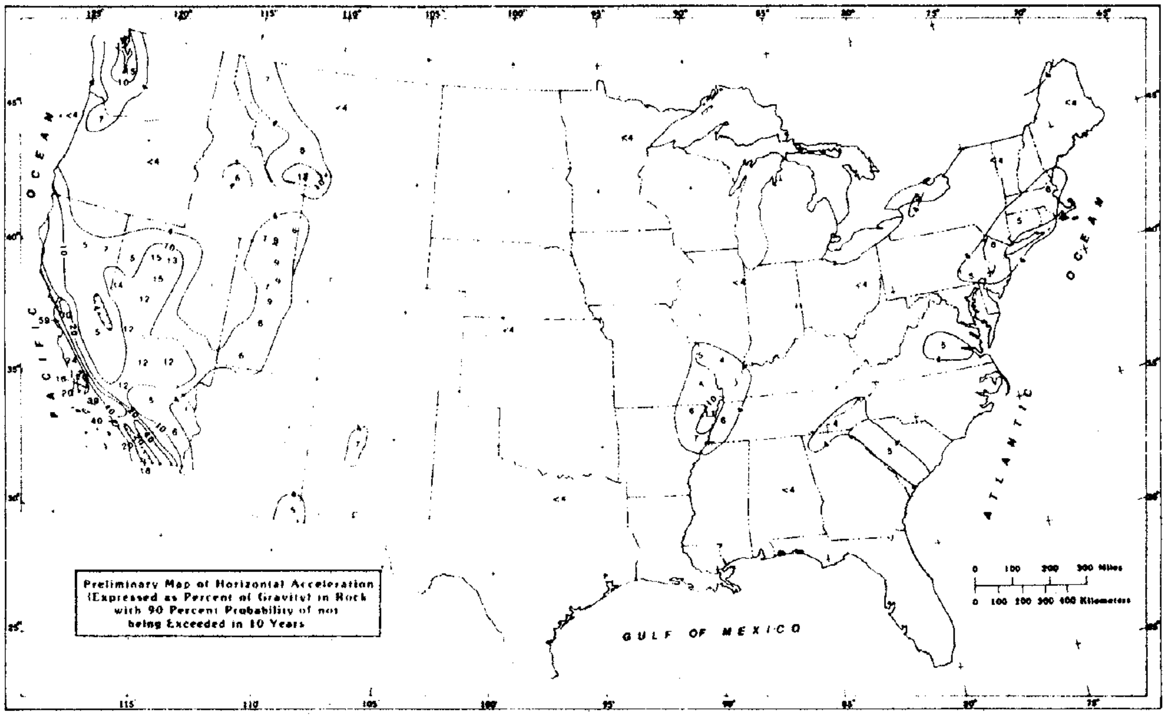
In the present case of defining an adequate level of safety for seismic rehabilitation of buildings, what remains is the quantification of the values for the loading, U, and the resistance, R, by considering the uncertainty present in these two random variables. These two topics are discussed in the following sections.

QUANTIFICATION OF EARTHQUAKE LOADS

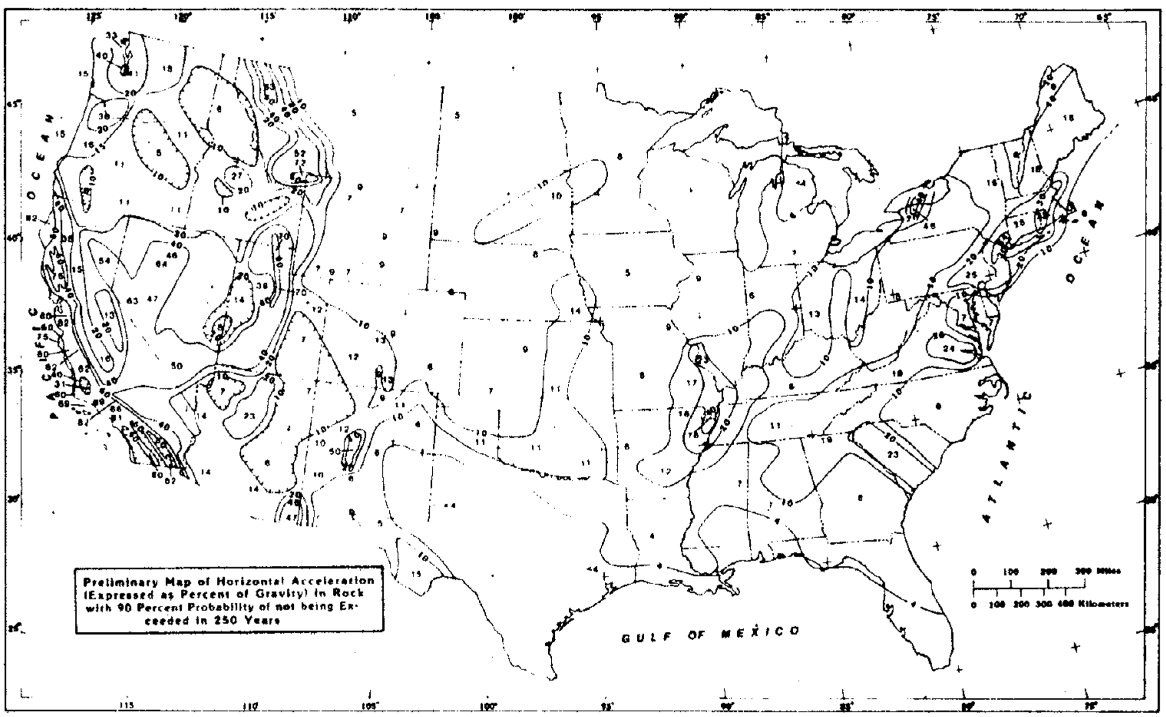
Establishing the magnitude and nature of earthquake loads is, at best, an inexact science. Yet, in recent years, considerable progress has been made in the prediction of seismic motion. Of prime importance in most earthquake studies is the estimation of effective peak ground acceleration because most engineering models use this information to derive forces experienced by structural members.

It is not the purpose of this paper to discuss all of the important considerations which affect the quantification of the maximum earthquake force. That activity is best left to geotechnical engineers and seismologists. The reader is referred elsewhere for this information. This section does discuss the criteria which should be provided to the geotechnical engineer to permit a realistic quantification of the seismic load in a manner consistent with the reliability analysis.

Just as the strength possessed by a particular structural member is not known with certainty, so, too, is the uncertainty concerning the earthquake forces one might expect during a given period of time. Earthquake ground motion and the resulting forces are therefore considered random variables. In light of this information, we select a maximum earthquake force for use in designing the seismic system with the knowledge that it may be exceeded sometime during the design life. It is neither economically nor functionally practical to select an earthquake force with such an extreme magnitude that there is effectively no chance of it ever being exceeded. But we can quantify the probability that it might be exceeded and use this information as part



a) Zoning map based on horizontal acceleration with 10% probability of being exceeded in 10 years. The higher the number, the greater the hazard.



b) Same as Figure 4 above, except for 250 years.

Figure 4
Horizontal accelerations with 10% probability of being exceeded during 10 and 250 years (9)

of our reliability analysis.

In order to obtain an estimate of design ground acceleration, the time of exposure must be established. Clearly, the longer a structure is exposed to the seismic environment, the higher the intensity of earthquake force on the structure one can reasonably expect. For the purposes of comparing one earthquake force estimate with another, a uniform period of exposure is desirable. The study on which the ANSI load factors are based assumes a 50-year design life. Using this previously agreed upon time period, earthquake forces can be specified that correspond to certain probabilities of being exceeded during the design life.

For example, the collapse level earthquake, sometimes called maximum credible earthquake, used in much commercial work is an acceleration that has a probability of 10 percent of being exceeded during a 50-year period. Approached from another angle, this corresponds to an earthquake which we expect to occur once every 475 years. Some call this the "475-year earthquake," because the 475 year time span represents the "return period" of an earthquake with this acceleration. It is important to point out that no one guarantees that this level of earthquake will occur only once, or even at all, during a 475-year period. It is only a simple way of describing the relative strength of an earthquake. Further, it is assumed that a 100-year earthquake is less intense than a 1000-year earthquake. Figure 4 shows two maps of the United States with accelerations corresponding to a 90 percent chance of not being exceeded during a 10 and 250-year period [8]. It can be seen that the accelerations associated with the longer design life are significantly greater than those of the shorter design life.

Selection of a design life is a major factor which directly influences the level of seismic shaking one would expect a building to experience. While a 50-year design life is inappropriate almost by definition, by the time a structure is considered historic, it has already exceeded its anticipated useful life. Considerations beyond those typically used in evaluating commercial buildings should be examined when selecting a design life for older or historic structures. These include the social or historical significance of the building, its uniqueness, location, estimated life of physical structure and other similar attributes. We believe that the determination of an appropriate design life should not be left solely to the structural engineer because of the many consequences implicit in the selection. Rather, it should be selected in cooperation with those with a vested interest in the project and in the case of the commercial development of an older or historic structure, this certainly includes those with more than a financial interest in the project. For example, in the absence of other information, we believe that a design life for an important historic building should be on the order of 200 years.

If a design life other than 50 years is selected, the appropriate conversion back to a 50-year design life should be made so that calculated reliabilities can be compared on a consistent basis. Such a conversion can be performed by those familiar with a probabilistic approach to ground motion estimation.

Using the uncertainty associated with the earthquake force, we can calculate the mean and standard deviation of the demand, U, as part of our effort to establish the mean and standard deviation of the safety margin, F. With the selection of an appropriate level of earthquake loading completed, attention can be turned to estimating the strength of the structure.

ESTIMATION OF STRUCTURAL STRENGTH

Estimates of seismic strength are influenced by a number of important considerations. These can include type and material of construction, date of construction and the analytical model used to evaluate the response of the structure to seismic loads. Just as there was uncertainty associated with the loading estimation, it is also encountered when estimating building strength. Thus, structural resistance is also a random variable.

Much work has been done in the area of estimating the strength of archaic building materials. This information provides the background for planning testing programs and other approaches to establishing material strength. The scatter of material strengths reported in the test results gives some indication of the uncertainty associated with estimating the strength of construction materials. Even repètitive tests frequently report varying results. This can occur because of the vagaries in the testing equipment or from genuine variations in material strength from one component or location to another. Another frequently forgotten source of uncertainty associated with physical testing is the difficulty of constructing a test which actually measures the physical quantity being tested.

However obtained, we can use estimates of the material strengths in the form of their means and standard deviations to account for the uncertainty associated with the material strengths. This information is included in our determination of the mean and standard deviation of the resistance of the structure, R.

In addition to the uncertainty associated with physical testing, there is uncertainty introduced into the analysis by the very analytical models we use. Models are, by their nature, simplifications of the physical world. These simplifications cause inaccuracies in the results obtained by our analyses. These uncertainties can be handled by introducing a random variable which modifies our calculated result to reflect the modeling inaccuracy.

Once an estimate of the member strength is completed and we have obtained the mean and standard deviation of the resistance, R, it is possible to find the safety margin, F, using Equation (1). The techniques for calculating these values are beyond the scope of this paper.

However, readers interested in investigating this part of the methodology are referred to Reference [9].

Having obtained a value of $\overline{F}$ and s_F, we can use Equation (2) to establish the reliability index, β. The calculated value of β can then be compared to a target value selected for the project. The value of β can be modified by changing the value of either the load or the resistance. Since the loading criteria is generally assumed to be fixed, for practical purposes only the resistance side of the equation is changed. If the value obtained for β is considered to be too low compared with the target value, the resistance of the building can be increased. Another method to increase β would be to reduce the value of s_F implying a reduction in the uncertainty surrounding estimates of the resistance. However, this is not usually practical.

The resistance of the building to earthquake loads can be augmented by increasing the strength of the lateral force resisting system. This might be accomplished by increasing material strengths or adding additional structural elements. The revised value of β could then be compared with the selected target value and further adjustments made as required.

CONCLUSION

The methodology presented in this paper provides a systematic method of establishing and quantifying an acceptable level of safety for the seismic rehabilitation of buildings. The essence of the methodology is summarized as follows:

1. Establish earthquake loading criteria based on the design life of the structure. Obtain an estimate of the mean and standard deviation of the earthquake load, U and s_U, respectively, for the structure.

2. Determine the strength of the existing structure to obtain an estimate of the mean and standard deviation of the resistance, R and s_R, respectively.

3. Select a target reliability index, β, which is appropriate for the project. Refer to Table 1 for ranges of β encountered in current practice. Adjust β, as required, to account for the historic nature of the structure.

4. Calculate the reliability index using Equation (4) and compare it to the value of β selected in (3). If the level of β is greater than the target value, the structure probably does not need to be strengthened. If the value of β is less than the target value, the structure must be strengthened.

5. Determine the strength of the rehabilitated structure using an appropriate design methodology (e.g. ATC-3 [10] or UBC [11] and ob-

tain estimates of the mean and standard deviations of the resistance.

6. Calculate the reliability index using Equation (4) and compare it to the value of β selected in (3). If the level of β is greater than the target value, the structure possesses sufficient strength. If the value of β is less than the target value, the strength of the structure must be increased. If desired, adjustments in the strength can be made to bring the reliability index of the rehabilitated structure as close to the target reliability index as is practically feasible.

It is important to remember that the selection of an appropriate level of seismic safety cannot be done capriciously. The value selected has serious economic and programmatic consequences which must always consider the realities surrounding historical preservation. Projects which incorporate excessive amounts of seismic retrofitting face the risk of not being rehabilitated at all. We believe that this method provides a realistic approach for assessing the benefits associated with seismic rehabilitation.

REFERENCES

(1) B. Ellingwood, T.V. Galambos, J.G. MacGregor, and C.A. Cornell, "Development of a Probability Based Load Criterion for American National Standard A58," NBS Special Publication 577, Washington, D.C.: National Bureau of Standards, 1980.

(2) "Building Code Requirements for Minimum Design Loads in Buildings and Other Structures," ANSI A58.1-1982. New York: American National Standards Institute, 1982.

(3) J.G. MacGregor, "Load and Reduction Factors for Concrete Design," ACI Journal, July-August, 1983.

(4) American Iron and Steel Institute, "Proposed Criteria for Load and Resistance Factor Design of Steel Building Structures," Bulletin No. 27, American Iron and Steel Institute, January 1978.

(5) "Strength Design of One-to-Four Story Concrete Masonry Buildings and Commentary," Reports EHI 8312 and 8313, Los Angeles, California: Englekirk and Hart Consulting Engineers, Inc., December 1983.

(6) G.C. Hart, S.C. Huang, and T.A. Sabol, "Calibration of Strength Reduction Factors for Concrete Masonry," Civil Engineering Systems, 1:1, September 1983.

(7) "Strength Design of One-to-Four Story Concrete Masonry Buildings," Research Report 4115, Whittier, California: International Conference of Building Officials, February 1984.

(8) S.T. Algermissen, D.M. Perkins, P.C. Thenhaus, S.L. Hanson, and B.L. Bender, "A Probabilistic Estimate of Maximum Acceleration and Velocity in Rock in the Contiguous United States," U.S. Geological Survey Open File Report 82-1033, Washington, D.C.: U.S.G.S.

(9) G.C. Hart, Uncertainty Analysis, Loads, and Safety in Structural Design, Englewood Cliffs, N.J.: Prentice Hall, 1982.

(10) Applied Technology Council, "Tentative Provisions for the Development of Seismic Regulations for Buildings, ATC 3-06," Washington, D.C.: National Bureau of Standards, 1978.

(11) Uniform Building Code, 1982 Edition, Whittier, CA: International Conference of Building Officials, 1982.

INTENTIONALLY BLANK

The Behavior of Brick Masonry Prisms in Compression

James L. Noland (I)

SUMMARY

An experimental and analytical research project to determine the influence of brick units and mortar upon the compressive behavior of stack-bond, solid-unit prisms is reviewed. Splitting failure was shown to be initiated in the brick units. A computer program implementing the theory closely predicts initial prism vertical splitting.

INTRODUCTION

Compressive tests of stack-bond masonry prisms are used in the U.S. to evaluate the performance of masonry and as a basis for design allowable stresses. Because masonry is an assemblege of discrete units bonded together with mortar, behavior under load is determined by interactive behavior of the two components. This paper presents the results of an experimental and analytical study to analyze the interactive behavior of brick and mortar in stack-bond prisms.

THEORY

A deformation failure theory (Ref. 1) was developed for the case of stack-bond, solid-unit prisms under compression. It is based on nonlinear, dilatent behavior of mortar and linear-elastic behavior of the units. Equation (1) is an expression of the theory in which an increment of lateral unit stress is expressed as a function of an increment of prism compressive stress, and the elastic and geometric properties of the brick units and mortar bed joints.

$$\Delta_{xb} = \frac{\Delta\sigma\left[\nu_b - \dfrac{E_b}{E_m(\sigma_1,\sigma_3)}\,\nu_m(\sigma_1,\sigma_3)\right]}{1 + \dfrac{E_b}{E_m(\sigma_1,\sigma_3)}\,\dfrac{t_b}{t_m}}$$

where Δ_{xb} = an increment of lateral stress in the clay unit

$\Delta\sigma$ = an increment of prism compressive stress

ν_b = Poisson's ratio of the unit (brick)

E_b = modulus of elasticity of the unit

(I) Consulting Engineer and Principal, Atkinson-Noland & Associates, Inc., Boulder, Colorado - U.S.A.

ν_m = Poisson's ratio of mortar as a function of principal stresses

E_m = modulus of elasticity of mortar as a function of principal stresses σ_1 and σ_3

t_b = thickness (height) of masonry unit

t_m = thickness of mortar bed joint

The theory predicts initial tensile splitting of stack-bond, solid unit prisms which occurs whenever the stress state in the mortar or the unit exceeds the applicable failure envelope. Stresses are assumed uniformly distributed over the thickness of brick and mortar.

BRICK PROPERTIES

Biaxial tension-compression tests of solid bricks were conducted to simulate unit state of stress in a prism under compression and to develop a brick biaxial failure envelope. An apparatus (Fig. 1) was developed to apply tensile force to bricks under compression perpendicular to the direction of tensile force (Fig. 2). Greased teflon sheets were used to minimize brick-platen interface friction.

The results of brick biaxial tests are presented in Figure 3 as well as those obtained by Khoo using one-third scale bricks (Ref. 2). The best-fit curve is concave which suggests that tensile stress has a greater influence in compressive strength than indicated by the straight-line Coulomb criteria.

MORTAR PROPERTIES

Mortar properties were established by triaxial tests on 4 in. (102 mm) x 2 in. (51 mm) diameter cylinder specimens. Mortar mixes used were 1:¼:3, 1:½:4½, 1:1:6, and 1:2:9* and lateral pressures applied ranged from 30 psi (207 kPa) to 1500 psi (10342 kPa).

Typical stress-strain behavior for 1:¼:3 mortar is shown in Figure 4. Increasingly nonlinear behavior was exhibited by all four types of mortar with increasing confining pressure. Strength and strain at ultimate increased with confining pressure. Elastic modulus and Poisson's ratio were dependent on the amount of axial compressive stress, confining pressure, and the mortar type. Data from these tests were used to represent mortar behavior under all stress states predicted in the computer implementation of equation (1).

PRISM PROPERTIES

Prism behavior was determined by compressive tests of five-unit stack-bond prisms. Prisms were carefully constructed using a jig to

*Denotes parts by volume of portland cement, lime, and masonry sand.

control geometry and mortar bed thickness (Fig. 5). Tests were conducted using prisms built with all combinations of two different unit strengths and four mortar mixes. Prisms were capped and loaded using greased teflon sheets to minimize prism-platen interface friction (Fig. 6). LVDT's were used to measure axial deformations.

DISCUSSION

Calculated unit and mortar lateral stresses are shown in Figure 7 as a function of prism compressive stress. The nonlinear brick failure envelope and the linear failure envelopes for two types of mortar developed in the research are shown as well as the straight-line (Coulomb) failure envelope for brick.

The nonlinear, dilatant behavior of the bed joint mortar is also illustrated in Figure 7. The brick provides sufficient confining (lateral) stress to the mortar to cause prism failure not to be mortar initiated.

The nonlinear mortar response causes brick response to also be nonlinear as shown in Figure 7. The nonlinear brick response curves in Figure 7 may be seen to intersect the brick failure envelope at a stress below that which would be predicted by linear brick stress response.

Prism stress at initial vertical cracking as calculated by the theory was consistently lower than measured ultimate prism stress as shown in Table 1. Observations of prism tests indicate that initial splitting occurs at approximately 80% to 85% of the prism ultimate stress.

Calculated and measured prism stress-strain curves are shown in Figure 8(a,b). The measured curves are the upper and lower bound of the experimental data and in close agreement with curves calculated by the theory.

CONCLUSIONS

The research has shown that the strength and deformation of masonry in compression may be determined from a careful evaluation of both the mortar and brick properties. Using the theory presented in this paper, a lower bound on the compressive strength of stack-bond prisms associated with initial lateral cracking may be calculated. Deformational characteristics of the test prisms could also be revealed by the computational model.

Tests of full-size brick units subjected to biaxial compression-tension stresses corraborated the failure envelope proposed by previous investigators (Ref. 2). The straight-line relationship between uniaxial tension and compressive strengths as prescribed by Coulomb was found to be unconservative.

Tests of mortar cylinders subjected to triaxial states of stress indicated significant nonlinear deformational characteristics which were dependent on the mortar type and the lateral confining pressure.

Deformation and strength of masonry prisms in compression were dependent on the nonlinear characteristics of the mortar, particularly for loading states near ultimate.

The theory and experimentation described in this paper should form the basis for a future study of the behavior of masonry subjected to repeated loadings.

ACKNOWLEDGEMENTS

Experimental work discussed herein was done at the University of Colorado by Scott McNary and Peter Waugh under the direction of Dr. Daniel Abrams. The theory was developed by Dr. R. H. Atkinson. The project was initiated and under the direction of the author. The research was supported by the National Science Foundation of the United States government as an Industry/University Cooperative Effort (Grants CEE 8115002 and CEE 8115003) between the University of Colorado and Atkinson-Noland and Associates, Inc., Boulder, Colorado. Dr. John B. Scalzi is the NSF program director.

REFERENCES

1. Atkinson, R.H. and J.L. Noland, "A Proposed Failure Theory for Brick Masonry in Compression," Proceedings of Third Canadian Masonry Symposium, Edmonton, Canada, June 1983, pp. 5-1 to 5-17.

2. Khoo, C.L. and A.W. Hendry, "A failure Criteria for Brickwork in Axial Compression," Proceedings of Third International Brick Conference, Essen, England, 1973, pp. 141-145.

Table 1. Measured and Calculated Prism Strengths

Brick Type	Mortar Type	Prism Strength (psi)		
		Measured	Calculated	
			(1)	(2)
1	1:¼:3	6989	4965 (71%)*	5345 (76%)
	1:½:4½	5931	4175 (70%)	4460 (75%)
	1:1:6	4713	3735 (79%)	4042 (86%)
	1:2:9	4334	2620 (60%)	2711 (63%)
2	1:¼:3	5461	3455 (63%)	4209 (77%)
	1:½:4½	5025	3180 (63%)	3689 (73%)
	1:1:6	3919	2560 (65%)	3042 (77%)
	1:2:9	2863	1980 (69%)	2275 (79%)

(1) Measured Brick failure envelope.

(2) Coulomb straight line failure envelope.

*Figures in brackets are percentage of measured strength.

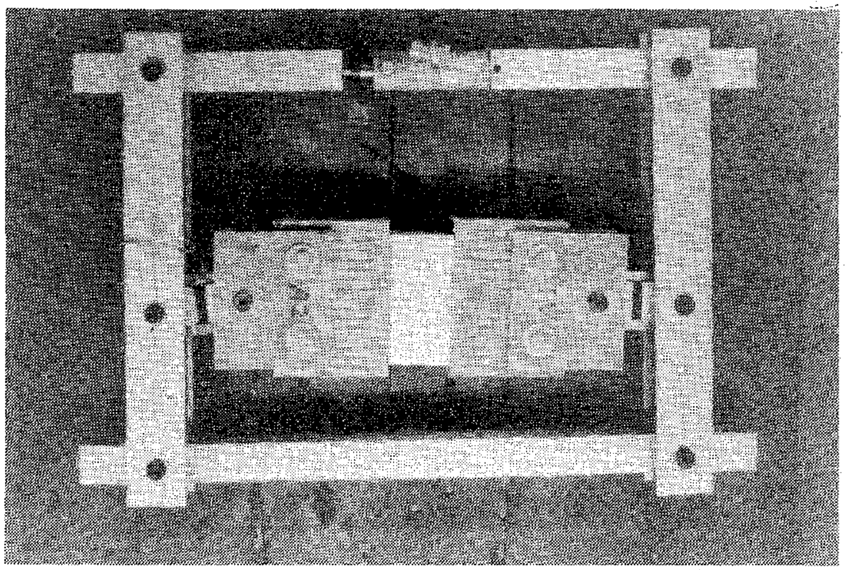


Figure 1 - Biaxial Brick Tension-Compression
Test Apparatus

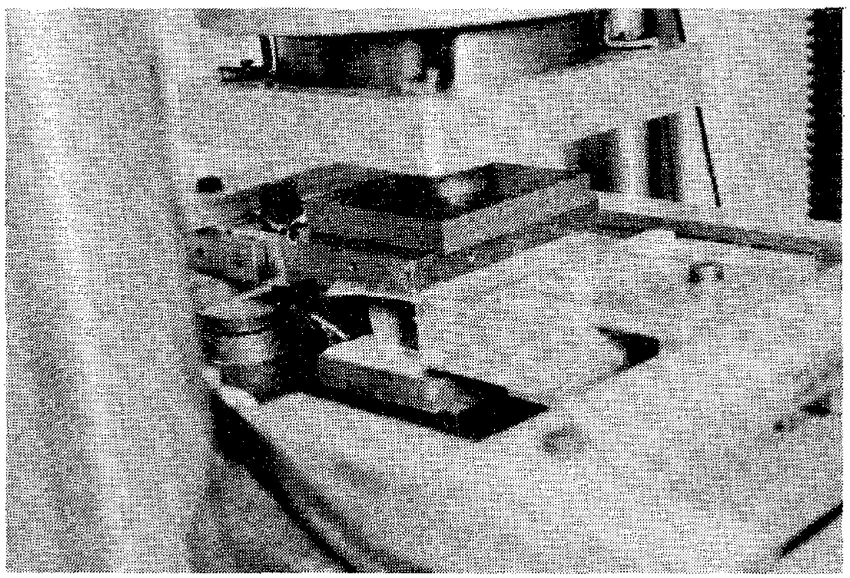


Figure 2 - Brick Biaxial Test

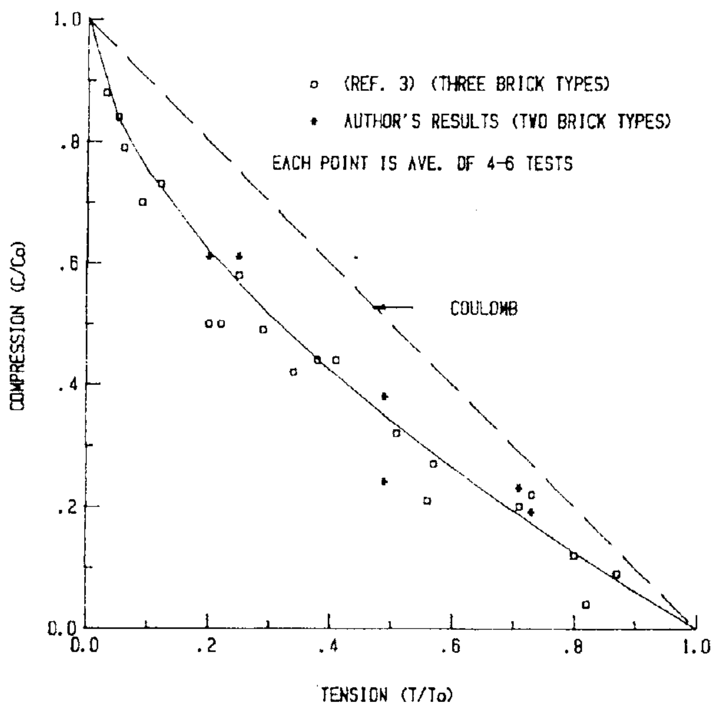


Figure 3 - Nondimensionalized Plot of Biaxial Tention-Compression Brick Tests.

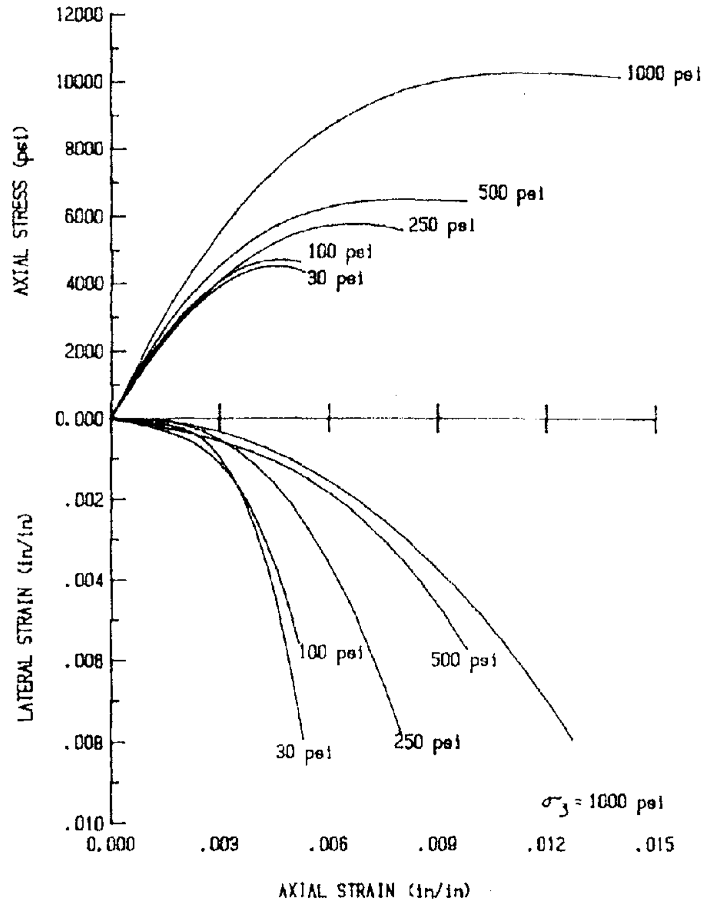


Figure 4 - Stress-Strain Plots for 1:¼:3 Mortar.

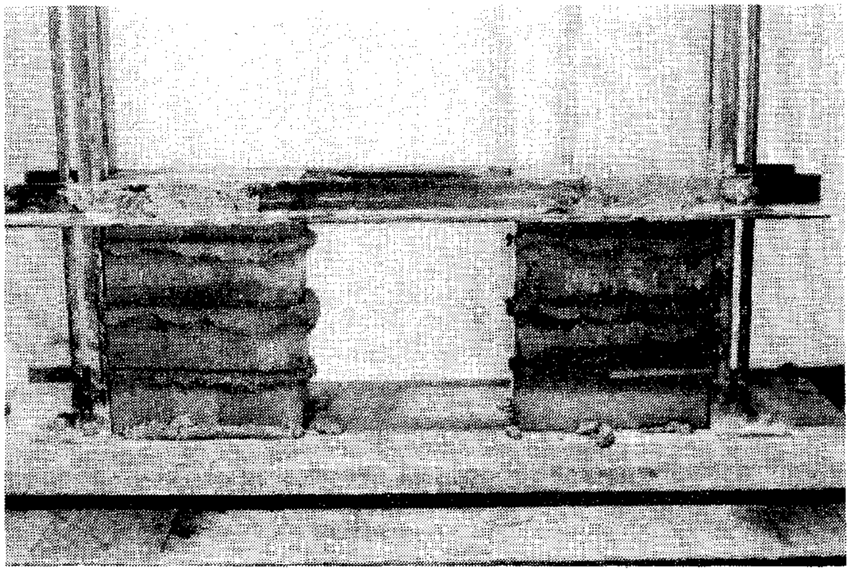


Figure 5 - Prism Construction

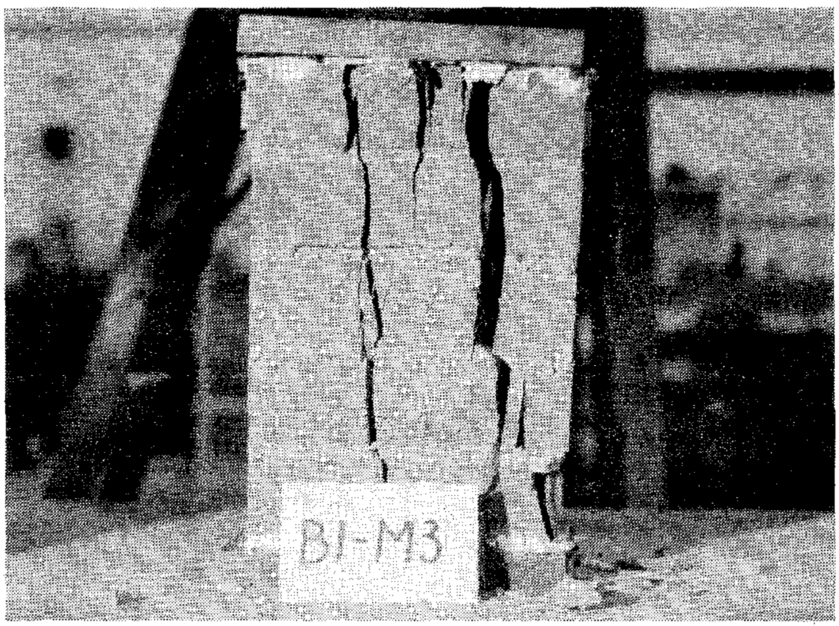


Figure 6 - Prism Testing

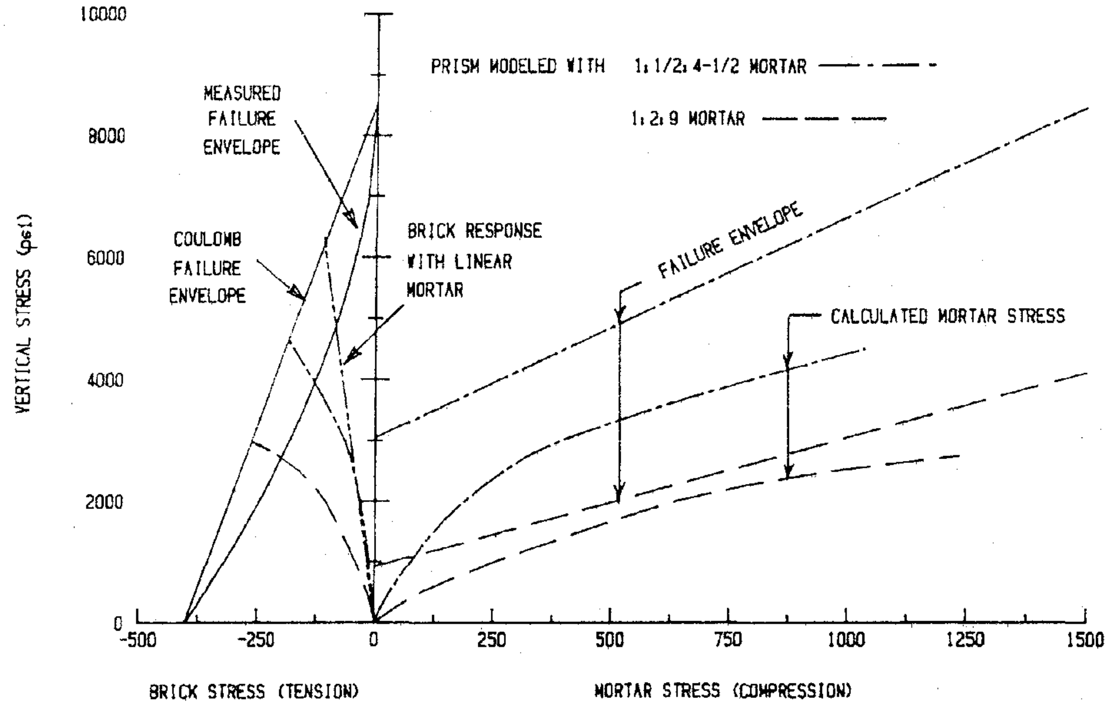
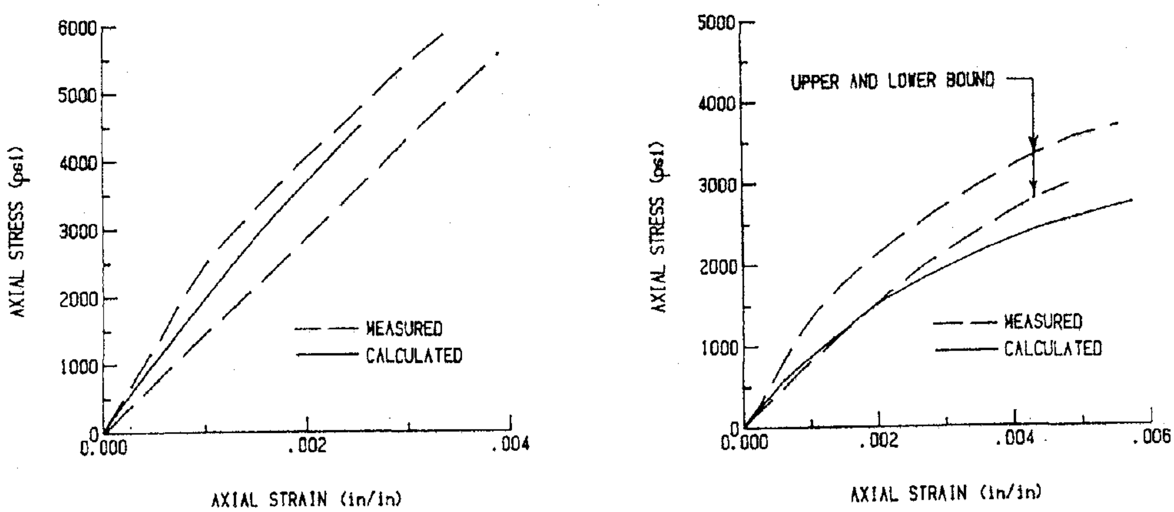


Figure 7 - Calculated Stress Paths of the Brick and Mortar for Prisms Modeled with Brick Type 1.



a) 1:½:4½ Mortar

b) 1:2:9 Mortar

Figure 8 - Stress-Strain Plots for Brick Type 1 Prisms Made with 2 Mortar Types.

Local Repairs of Reinforced Concrete Frames

S. Perno (I)
A. Samuelli-Ferretti (II)

SUMMARY

A program of tests on beam-column subassemblages currently being carried out in five Italian laboratories, coodinated by Professor A. Migliacci of Milan, is described. Specimens are damaged under simulated earthquake quasi-static strain histories, repaired with various techniques, and re-tested up to destruction.

In the laboratory of the Faculty of Engineering of Rome, specimens are repaired by means of welded steel profiles and battens.

Experimental techniques as well as the first results are described. The necessity of numerical modelling of repairs is pointed out, and examples of such an analysis are given.

INTRODUCTION
NEED FOR A METHODICAL TEST PROGRAM

Need for Guidelines in Designing Repairs

Upgrading of structures which were not designed, or are found to be inadequately designed, for resisting seismic actions aften calls for a radical modification of the overall behavior under lateral forces, such as follows from the introduction of shear walls, winged columns, and steel bracings. For this kind of upgrading, guidelines are available in Japanese codes (6). Even when this is the case (strength resistant structures), the analysis of the strengthened space frame can show weak points, mainly at joints and at member ends, which must be locally strengthened.

In the case of ductility resistant structures, and always in repairing damaged ones, local repair and/or strengthening is the rule. Researchers are actively at work in this field all around the world, as can be seen from the technical literature on the subject. (See, for example, Refs. 1, 2, 3, 5, 7, 9.)

For this kind of repair, however, guidelines are lacking (as pointed out by J. Warner in Ref. 5) and the various examples that can be found in technical papers do not offer much help to the engineer because they describe single techniques without any comparison element, so the choice is difficult. Data for dimensioning are nowhere to be found.

(I) Engineer, Rome
(II) Assistant Professor, Faculty of Engineering, "LA SAPIENZA", Rome.

As a consequence, the applications on the field, as can be observed, for instance, in the damaged areas of Irpinia, Italy, following the 1980 earthquake, show a remarkable scatter of solutions because they are based exclusively on the common sense (or the lack of it) of the designers.

The aim of this work, which has been underway since 1980 in five Italian laboratories (Polytechnic of Turin, P. Napoli; University of Ancona, R. Antonucci; University of Florence, Faculty of Architecture, N. Avramidou Maio; University of Rome, Faculty of Architecture, G. Via; Faculty of Engineering, A. Samuelli-Ferretti, Coordinator, A. Migliacci, Milan) is to produce guidelines for the design and the choice of local repairs under different situations of original strengths and damage levels.

Special Aspects of Earthquake Repair and/or Strengthening

When confronted with repairing local damages of seismic origin, the technician has two circumstances in his favor:

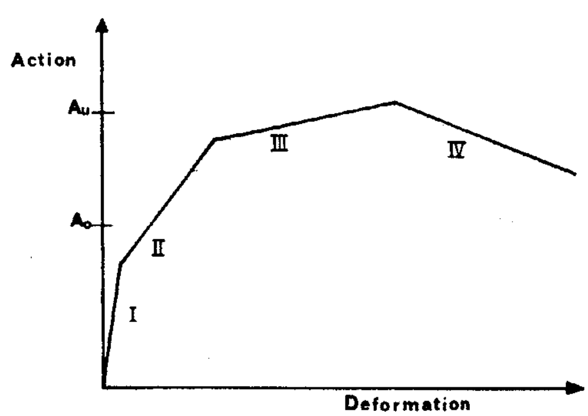
1) the origin of the damage is well known (this is often not the case for foundation settlements), and

2) if the aftershock period can be considered as having expired, the main cause of damages is not active during re-fitting work.

The latter usually can greatly reduce the need for costly, prestressed shoring.

Assessment of damages (see Refs. 2, 8) must take into account those facts, and it is the frequent experience of the authors that in many instances technicians have a tendency to overestimate the dangerousness of a given damage situation -- the "pathology image", to use the colorful term of T. Tassios.

This assessment is, of course, the first step in the repair procedure and, to avoid incorrect judgements, a good knowledge of r.c. member behavior up to ultimate conditions is necessary. This is crudely illustrated in the diagram and the table below.



Stage		beams prevalent bending	beams shear walls prevalent shear	joints composite stress	columns axial f. and bending
I. uncracked	a	0	0	0	0
	b	(possib. thin shrink cracks)	(possib. thin shrink cracks)	–	–
	c	–	–	–	–
II cracked	a	1 – 2	0,5 – 1	0,5 – 1	1 – 2
	b	possible initial crushing of conc. cover	initial slip	initial slip	initial crushing of concrete cover
	c	A	A – B	B	A – B
III yield	a	2 – 4	1 – 2	1 – 2	2 – 3
	b	crushing and partial spalling of concrete cover	appreciable slip, shear grinding of crack lips	appreciable slip, shear grinding of crack lips	crushing and spalling of concrete cover
	c	B	C	C	C
IV strain softening	a	> 4	> 2	> 2	> 3
	b	crushing of concrete core	large slip, kinking of transverse reinforcement	severe disintegration	crushing of concrete cover, buckling of re. bars
	c	C	D	D	D

a) is the residual width of cracks, in mm;
b) is a sketchy description of the damage;
c) is the evaluation of damage level according to C.E.B. proposals. (See Ref. 2, annex 1.)

The table must be regarded merely as an indication because it does not take into account:

1) the number of cycles suffered. This parameter greatly affects residual crack width (see Jimenez, Gergely and White as quoted in Ref. 3 for shear, and Ref. 4 for bending cycles);

2) the mechanical properties of concrete and steel;

3) the interaction between resultant components, such as axial force and bending, shear and bending when the first components are exceptionally large.

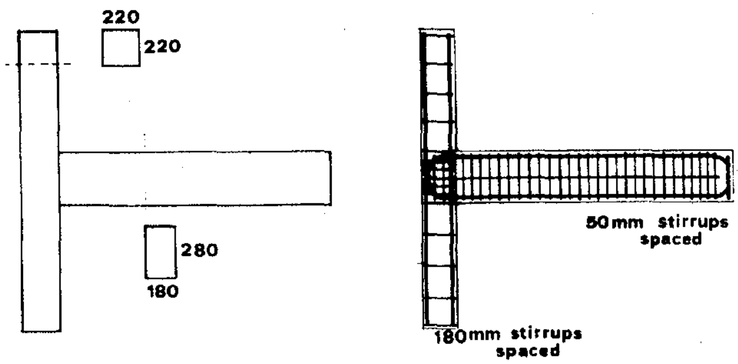
Having evaluated in this way the "pathology image" of the frame, the next step can be the assessment of its residual capacities, if there is some hope of avoiding repairs and in order to design shoring. Then the engineer must proceed to the design of overall strengthening and/or local repairs. The final analysis, according to the importance of the building, can either be dynamic or static. In any case, knowledge of stiffness and strength alteration due to repairs, as well as a careful assessment of the partial degrees of safety to be applied, is necessary.

In the following we shall try to evidence as much as possible the contributions that the results of this coordinated research can provide in solving the aforementioned problems.

Choice of the Specimens

It is practically impossible to reproduce the almost infinite variety of situations that can be encountered in actual frames: number of members concurring in a single joint, relative strengths and stiffnesses, reinforcement details (good and not-so-good), and so on. It was therefore decided to concentrate the first effort of the group on "external" subassemblages consisting of two sections of column and one section of beam, with or without transversal stubs simulating front beams.

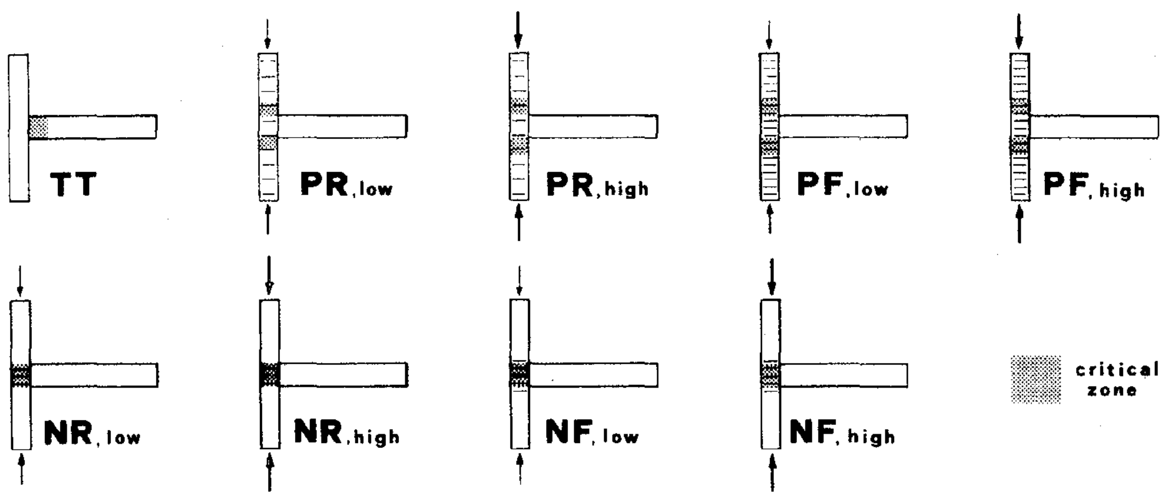
Scale effect is always a problem when strength testing r.c. elements, and a compromise solution was reached in scaling down to 2/3 of assumed typical dimensions. Overall measurements of specimens as well as an example of reinforcement are given in the figure.



The next step of the research will be the testing of internal joints.

Damage Localisation. Parameters Variation

In order to cover in depth a wide range of situations, given the aforementioned impossibility of reproducing the large variety of actual configurations, it was decided to define the different specimens on the basis of damage localisation. In other words, reinforcement has been



defined so as to reach ultimate conditions either in the columns or in the beam or in the joint. Influence of transversal reinforcement was also investigated.

A total of six specimens was obtained; further difference is caused by the imposed value of axial stress, "Low" ($4.2 N/mm^2$) or "High" ($8.4 N/mm^2$).

Segments others than the one designed to collapse are protected by means of an increase in reinforcement. The following table gives the ratios between the strengths of the critical regions and the ones of the most stressed remaining ones.

Specimen	Axial Load	end of beam, bending	beam, shear	ends of column, bending	column shear	joint
TT	—	1	1.1	1.4	1.3	1.3
PR	Low	1.2	1.3	1	1.2	1.2
PR	High	1.2	1.3	1	1.3	1.2
PF	Low	1.2	1.3	1	1.2	1.1
PF	High	1.2	1.3	1	1.3	1.1
NR	Low	1.3	1.2	1.1	1.3	1
NR	High	1.3	1.2	1.1	1.2	1
NF	Low	1.3	1.2	1.1	1.3	1
NF	High	1.3	1.2	1.1	1.3	1

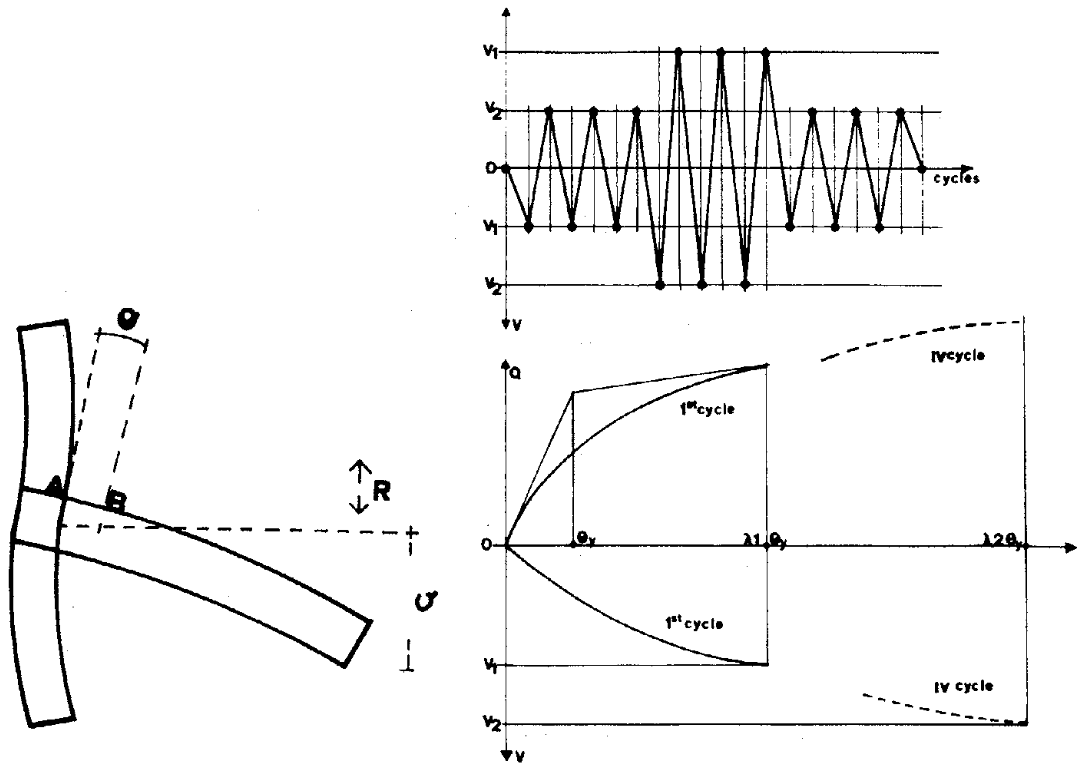
R stands for weak transversal reinforcement, F for strong

Values in excess of 1 give the degree of protection against improper localisation of collapse.

Evaluation of strength was computed following C.E.B. methods, without applying any partial safety factor to mechanical properties of steel and concrete. A check has been performed following A.C.I. code. After preliminary testing of a few specimens, it was found that the TT, PR, PF subassemblages were still in danger of collapsing in the joint, so that two transversal stubs were added, giving additional containment to the joint itself. With such an adjustment, the damage localisation followed design prediction with satisfactory accuracy.

Damage Level. Straining History

The choice among different repair techniques (R.T.) depends primarily upon the need for strength and ductility, and damage level. As pointed out before, it was of utmost importance that the test results from different laboratories regarding different collapse patterns be as comparable as possible, so it has been decided that the controlling factor of the imposed straining history should be the ductility ratio of local strains. This procedure is one of the main features of the program, so it deserves some detailed explanation. Taking as an example the TT specimens, measurements of the relative rotation ν between A and B cross sections (spaced 100 mm, that is, about one third of beam depth) as well as beam tip displacement v are taken, together with the response force Q at beam tip.



During first loading, after having recorded ν_y as the yield value (when collapse takes place in highly stressed columns or in joints, the yield point can be not very marked, but a definite knee in the relevant diagram can always be detected), straining is pushed up to a selected multiplier λ_1 (about 2 or 3 for light damages, twice as much for heavy ones). The value of $\pm$ v_1 tip displacement corresponding to first loading $\lambda_1\nu_y$ rotation is then applied alternately for three cycles. Due to curvature redistribution, ν_{max} changes during subsequent cycles.

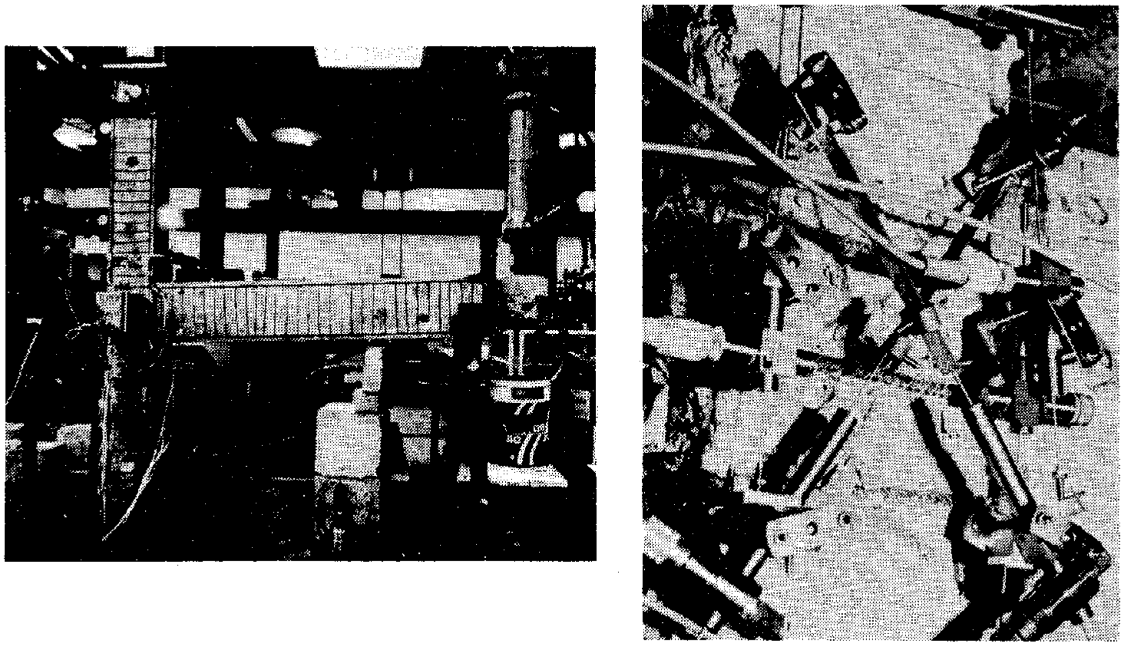
Amplitude of the fourth cycle is governed, again, by peak curvature, up to an imposed value $\lambda_2\nu_y$, to which corresponds a larger value v_2 of tip displacement. Three such larger cycles are applied, followed by three more, limited between $\pm$ v_1, that close the sequence.

After repairs, the same nine-cycle sequence is applied, limited between the same values $\pm$ v_1, $\pm$ v_2, $\pm$ v_1; if the specimen's strength is still considerable, a number of cycles up to $v_m \geq v_2$ are applied, until strength drops to about 20% of the initial value.

Experimental Set-up

Due to different facilities available, set-up varied slightly from one laboratory to another. Here the arrangement employed at the Faculty of Engineering of Rome is briefly described.

The specimens are placed in an upright position within a contrasting steel frame which supplies the horizontal constraint at the top of the column as well as the supports for two vertical jacks: one gives axial load to the columns, the second is a two-way actuator of $\pm$ 150 KN peak load and a total stroke of 500 mm, and moves up and down the tip of the beam.



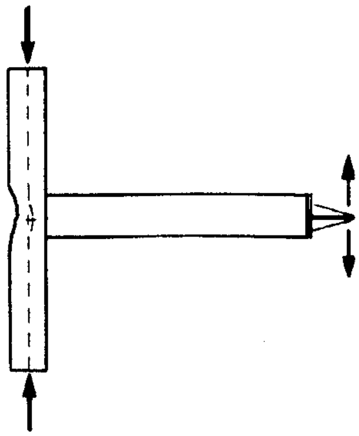
experimental setup - joint instrumentation

Its dynamometric response is transmitted to the x axes of four analogic recorders.

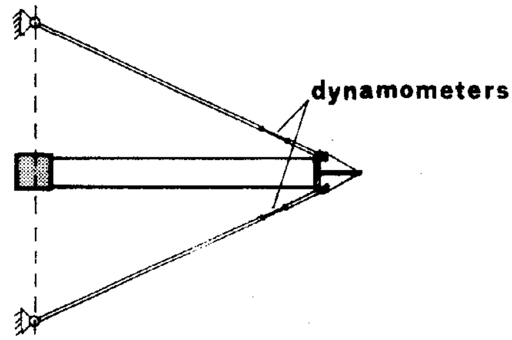
Displacement v and local strains are measured by means of inductive displacement meters; female threaded bars embedded in concrete at 100 mm spacings close to opposite faces of beam and column are the bases for the transducers, and they behaved quite regularly up to remarkable levels of crushing of concrete.

Preliminary tests indicated that, due to asymmetrical spalling of concrete cover in the joint region, a considerable amount of eccentricity took place at high levels of damage, giving way to extra bending moments which altered test interpretation. An additional restraint was thus added, to prevent axial displacement of the beam, consisting of a couple of hinged braces, dynamometer equipped, pivoting around a horizontal axis passing through the center of the joint.

Axial load thus applied to the beam reached values up to 40 KN, not affecting in a noticeable way the beam strength.



secondary excentricity



additional constraint

REPAIR AND STRENGTHENING TECHNIQUES

General Subdivision of Repair Techniques (R.T.)

Classification and choice of R.T. was part of the initial programming of the whole research, but it has to be pointed out that this is the matter that underwent the most remarkable modification during the progress of the work. Direct observation of specimen behavior during initial tests suggested quite a lot of variations to the originally planned R.T.

A major subdivision considers:

A) repairs without addition of new longitudinal steel;
B) repairs with addition of new longitudinal steel.

Obviously, class A is to be preferred, generally speaking, when the original structure is adequate and damage not heavy. On the other hand, class B R.T. are the rule when the original structure is inadequate, (if damaged, one speaks of repair and strengthening; if not, of upgrading) and/or in most cases of very heavy damage.

In the following, a classification of tested R.T. is provided.

Class A Repairs. (Without addition of longitudinal steel)

Resin Injections

Low viscosity epoxy was directly injected into cracks (R11) or the whole segment was impregnated (R12). Another technique was vacuum impregnation of polyester resin (R13), by means of a vacuum pump and foil jacket, thus minimizing the danger of air pockets.

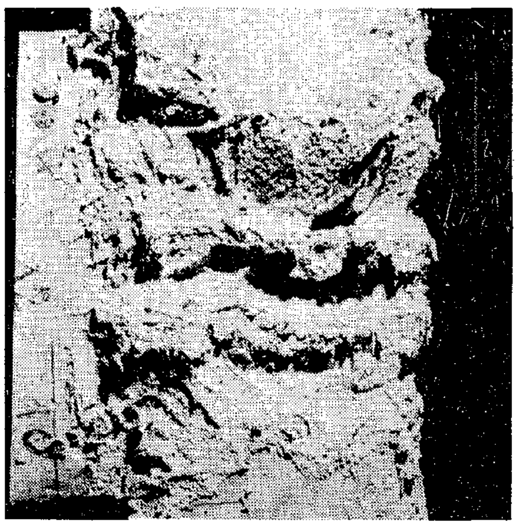
Results, in terms of strength recovery and energy dissipation, are usually good; migration of strain concentration and plastic hinges formation to the adjacent, not repaired, portions was observed, because of stiffness increase due to injections.

Cement Injections

Cement grout, plain (C11) or with the addition of non-shrink plastifier admixtures (C12), have not yet been tested. It appears that they are less suitable to penetrate thin cracks; on the other hand, the material is cheaper and less sensitive to human error in mixing.

Mortar or Concrete Restoration

When concrete destruction is present (from cover crushing and spalling to core crushing or disintegration), resin (RC) or cement (CC) mortar or concrete has to be employed.



Layer by layer application of resin mortar.

Resin mortar is a 5 to 1 or 7 to 1 mix of graded quartz sand and epoxy, mechanically stirred and applied in layers 5 to 7 mm thick. Hardened morar has 70 N/mm^2 compression strength, and ~ 30.000 N/mm^2 elasticity modulus; very good adherence with old concrete is easily obtained. Cement mortar with plasticizer non-shrink admixtures comes in pre-mixed bags; only water adding and stirring are needed. It is very fluid so that sealed false-works are needed, but cavities are easily filled. Strength is in the order of 80 N/mm^2, and E modulus is about 30.000 N/mm^2. When employing cement mortars or microconcretes, care has to be taken to ensure good adherence with old concrete; epoxy priming gives good results.

Mortar or Concrete Restoration with Steel Fabric or Ties

Restoration with or without enveloping jackets of resin or of cement concrete reinforced with welded fabrics (RCR or CCR) or stirrups (RCS or CCS) give very good local strengthening because of transversal containment. A sharp increase of stiffness is unavoidable, and care must be taken in designing that the strengthened portion extends safely to low stress sections; on the other hand, this extension gives way to noticeable moment redistribution.

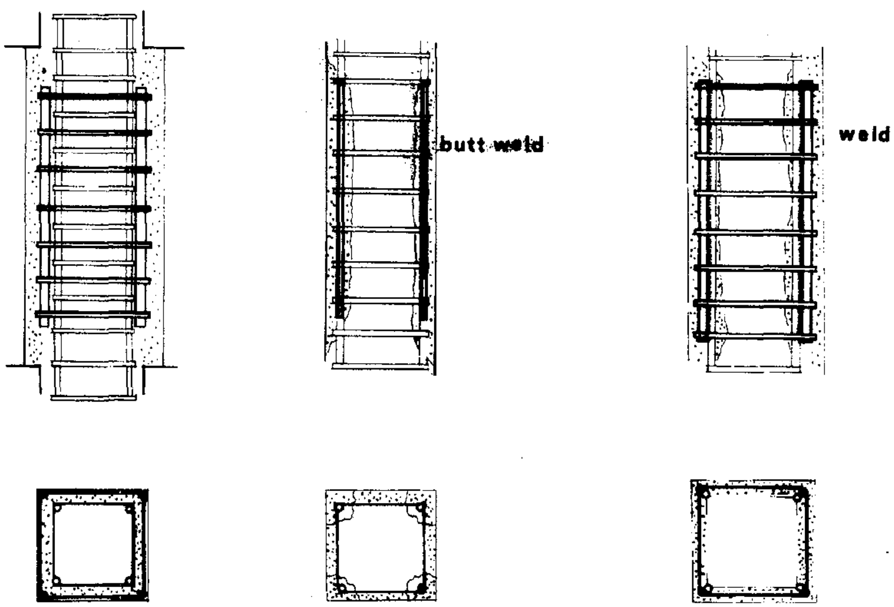
Class B Repairs. (With addition of longitudinal steel)

Addition of Reinforcement Bars

Reinforcement bars can be added by simple overlapping (BA) or by butt welding (BST) or overlap welding (BSS).

The first solution has the advantage of being applicable to steels of any grade; disadvantages are bulk and possible difficulties of arrangement in joint regions. The second solution, to be applied mainly when large spalling and buckling are present, does not increase stiffness but it requires proper grade weldable steels (old and new) and skilled workmanship to ensure strength of butt welds, to be made often in difficult environments.

The third solution is easier, but more or less cumulates the disadvantages of the two previous ones.



BA BST BSS

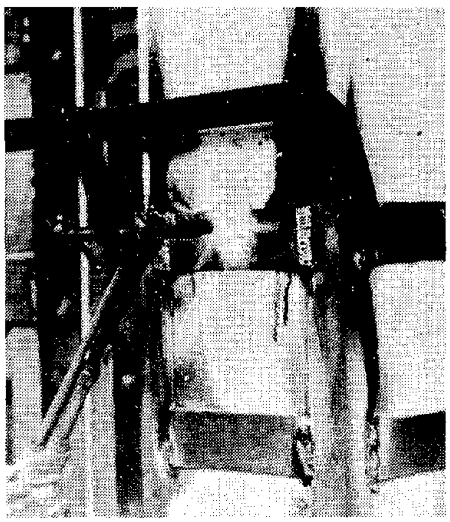
Encasement with Steel Sections

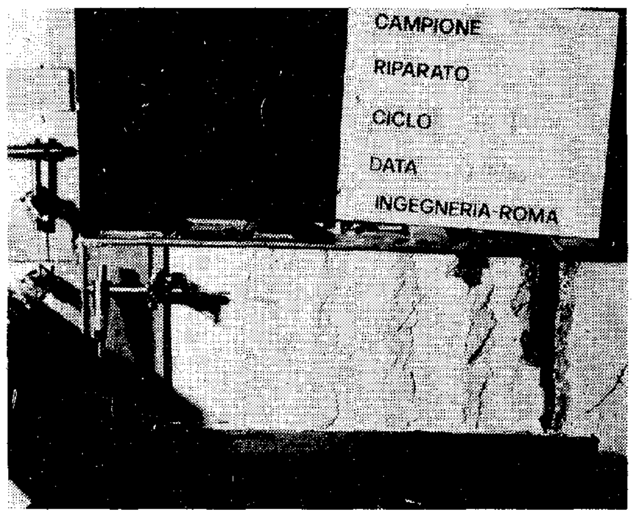
A steel encasement is formed around the damaged r.c. members. A usual, efficient array consists of corners laced by means of battens, forming square bay frames. Triangulated patterns are not advisable because of unnecessary excessive stiffness.

Connections can be either bolted or welded. Usually welding results in less bulky solutions, so it is especially advisable when complex patterns are unavoidable around joints, but it calls for skilled workmanship.

Slip between steel sections and concrete can be retarded by means of glueing and/or prestressing of battens, which can be obtained by means of bolt tensioning in bolted connections, or heat prestressing in welded ones.

By flame heating the central quarter of battens to bright red (850^{o} C) and cooling, residual stresses of more than 120 N/mm^2 have been measured by means of stress relieving techniques.

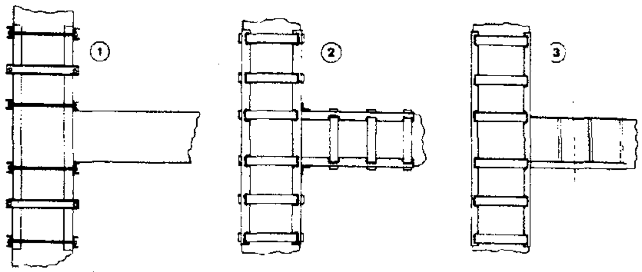




When previously glued (by means of epoxy adhesive applied to clean concrete, to use with sand-blasted corners), welding of battens causes partial burning out of adhesive. It has been found, however, that the burnt-out region does not extend in excess of 15-20mm around the welded area. An alternative means of glueing is by impregnation after construction of encasing; this technique calls for a little more skill and equipment. In both cases, corrosion protection of the inside face of corners is obtained as a by-product.

Placing of corners around the upper face of slab-supporting beams can be unadvisable because of excessive demolition required. Upper plates connected by means of rods (welded to plates through holes) embedded in lateral concrete cover have been successfully tested. The slenderness ratio of plate between rods was about 70/80.

Another way of connecting corners or plates to concrete, to be used when original transverse reinforcement is adequate, is by means of expansion bolts in drilled holes.



Classification and symbols for the described solutions are given in the following table:

Class B Repairs

Addition of reinforcement bars				
simply overlapped	BA			
butt welded	BST			
overlap welded	BSS			
Addition of steel profiles and battens				
	plain	glued	prestressed	prestressed and glued
Bolted connections	PB1	PB2	PB3	PB4
Welded connections	PS1	PS2	PS3	PS4
Expansion bolts	PE1	PE2	–	–
Glued plating (béton plaqué)	–	BP	–	–

In the following, the main results obtained from the 7 specimens tested so far at the Faculty of Engineering in Rome are illustrated briefly.

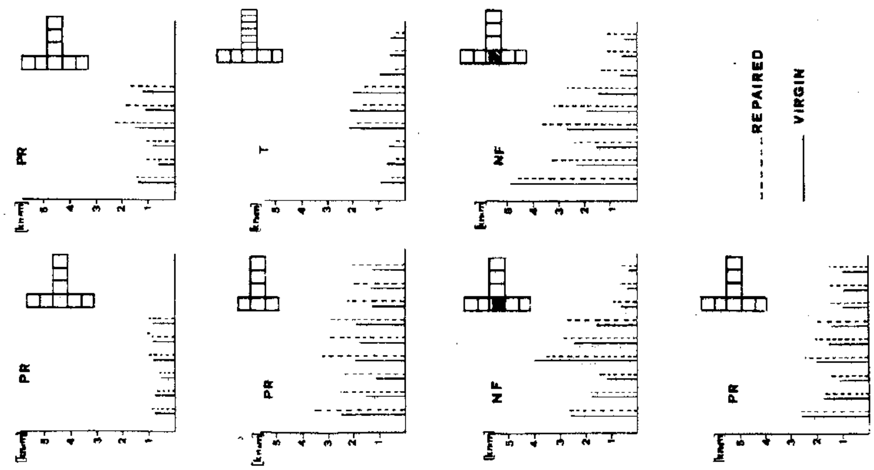
Table

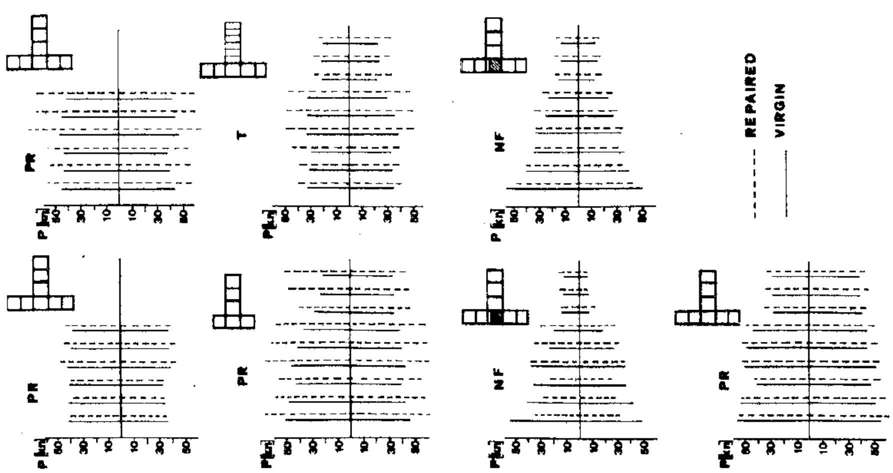
Specimen N°	Specimen type	Axial load	Repair type	Straining history: v(mm)	Damage level	Strength indexes $\frac{\Sigma\|Q_r^{(i)}\|}{\Sigma\|Q_0^{(i)}\|}$	Strength indexes $\frac{Q_r^{(9)}/Q_R^{(1)}}{Q_0^{(0)}/Q_0^{(1)}}$	Energy indexes $\frac{\Sigma E_r^{(i)}}{\Sigma E_0^{(i)}}$	Energy indexes $\frac{E_r^{(0)}/E_r^{(1)}}{E_0^{(9)}/E_0^{(1)}}$
1	PR	L	PS1	48; 62; –	C	1.03	–	1.06	–
2	PR	L	PS4	47; 67; –	C	1.38	–	1.41	–
3	PR	L	PS4	75; 87; 75	C	1.44	1.29	1.58	1.07
4	PR	H	PS1	77; 86; 77	C	1.09	1.11	1.26	1.52
13	NF	L	PS1	75; 117; 75	D	0.91	2.49	1.16	1.86
14	NF	L	PS4	83; 106; 83	D	1.03	1.50	1.41	2.30
9	TT	(H)	PS4	38; 61; 38	C	1.35	1.13	0.70	1.11

Of these, 4 are of the PF type (collapse in columns having poor transversal reinforcement), 2 of the NF type (collapse in the joint with good transversal reinforcement), 1 of the TT type (collapse in the beam).

As can be seen in the above Table, the initial damaging cycles have been rather severe, leading to heavy damages. Prior to applying the steel encasements, some perfunctory repairs were made, restoring the missing concrete portions by means of plain cement mortar (28 days strength: 25 N/mm^2), and, in a few cases, injecting with epoxy the widest cracks.

All of the specimens were repaired by means of corners and battens both in the columns and in the beam. Some care was taken in order to ensure (applying triangular brackets or, more efficiently, inclined plates) stress transmission from the steel corners in the beam to the frame of the column. It has to be pointed out that a 40 mm difference in depth exists between the two elements (beam: 180 mm, column: 220 mm).





Repairs of NF specimens were of the same type, with the addition of lateral plates that covered partially the joint. The beam of the TT specimens has upper plates, lower corners and rod battens, as previously described.

The strengthened zone of No. 3 PR specimen was noticeably shorter than in the three others of the same type. Nevertheless, overall resistance was just as good, and energy dissipation was better, probably owing to the fact that the collapse mechanism was governed by distributed shear cracks more than by localized plastic hinges.

The same observation applies to the two NF specimens which showed very good dissipating capacities. Loading history of the first two specimens was limited to 6 cycles, instead of the regular 9 programmed, because of testing problems.

The diagrams show peak values of force response at the tip of the beam during cycles; efficiency of repairs (dotted lines) can be easily compared with virgin behavior (solid lines).

Energy dissipation is showed by the same representation.

In the last four columns of the previous Table, efficiency of repairs is synthetically represented by means of four indices: The first is the ratio of the sum of responses of repaired specimens to the analogous sum of the virgin ones; the second gives a value for deterioration by comparing the ratio of last to first cycle response of the repaired specimen to the same ratio computed for the virgin one.

The last two columns have the same meaning, referring to energy dissipation. Values of indices in excess of 1 mean that the repaired specimen has better behavior than the virgin one.

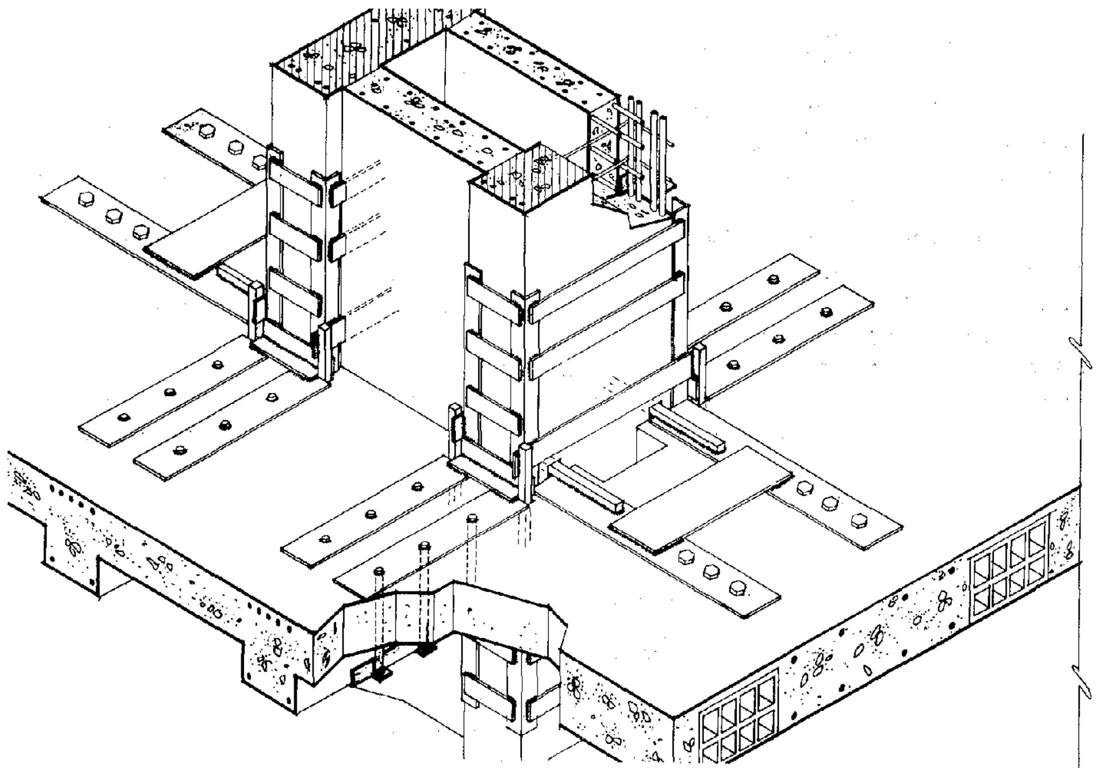
It can be seen that, with the exception of the No. 13 NF case, everywhere an improvement of resistance has been obtained. Diminution of decay is especially good (second column), as can be expected, since it is a steel structure which comes more and more into play with the degrading of concrete. This fact is quite marked in the NF specimens, whose response shows a rapid decay in the virgin ones.

Prestressed and glued specimens behave in a better way than plain ones, rows 2 and 3, but in the long run (second and fourth column) the advantage has a trend to diminishing. The TT specimen shows a decrease in energy dissipation, because the rods embedded in concrete prevented slipping and friction between encasement and concrete.

DESIGN AND ANALYTICAL MODELLING OF STEEL ENCASING REPAIRS

Need for Analytical Modelling

Specimens designed for laboratory tests are necessarily simpler than actual frame joints as they are found in buildings. The Figure below emphasizes this statement; it represents the simplest configuration devised for strengthening a joint composed of a double column, two flat and two ordinary beams.



It is easily understood that the engineer cannot rely directly on laboratory results because they can be too far from actual situations. Analytical modelling is therefore a must. To check the reliability of such modelling, as well as to calibrate the values of the relevant parameters, numerical modelling of the tested specimens has been systematiccally performed. In the following, two examples are described.

Dimensioning Criteria

Strength and stiffness of encasement have been evaluated as the ones of a beam composed of the four steel corners, thus assuming:

Stiffness: $$E_S J_S = E_S.4.A.\frac{d^2}{4},$$

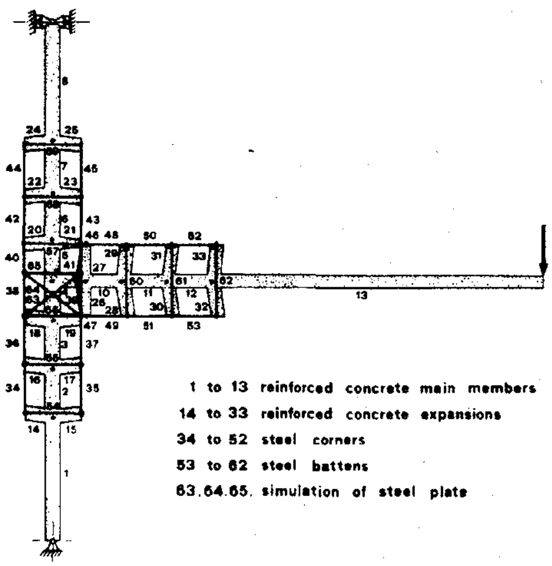
Strength: $$M_u = 2.A.d.f_y.$$

In the examples, ratios between stiffness of encasement and of original r.c. members is around 0,8, while ultimate strengths are equal. To evaluate the meaning of such ratios, it must be remembered that we are dealing with the case of severely decayed specimens. Overall stiffness of the subassemblage was, in the ninth cycle, down to only 30% of the virgin one.

Analytical Modelling

The two examples concern specimens of the PR type (collapse in the column) repaired following either the PB2 technique (corners glued to concrete) or PB4 (glued corners and flame heat-prestressed battens).

The model scheme is represented in the Figure: R.C. members are connected by means of rigid stubs to the joints of the steel encasement, which is a "frame type" structure. Analysis has been carried out by means of a very simple step-by-step procedure, suitable for a desk-top micro computer. Each step is linear, and stiffness of the composite model is changed by introducing suitable releases when a pre-established threshold of connection strength or ultimate bending moment is reached.



This adherence has been evaluated, on the basis of appropriate tests, at a $\tau_u = 3\ N/mm^2$, which corresponds, more or less, to shear resistance of the concrete employed. As a matter of fact, when specimens were disassembled at the end of testing, it was found in every case that separation took place inside concrete, since parts of it remained firmly glued to the steel.

In modelling the plain glued PS1 specimen (Fig. A), the restraint between concrete and steel members was completely released once shear strength was reached. This fact caused the sudden drops of response in the diagram, which show a sawtooth aspect not visible in the experimental one.

Figure B represents the behavior of a prestressed and glued specimen. In this case, once shear strength was reached between steel and concrete, a friction force assumed at 0.1 value of batten prestressing was maintained. As a consequence, the sawtooth drops are less important.

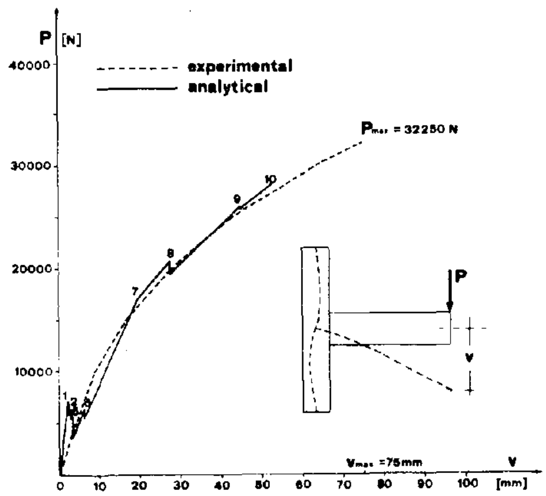


Figure A

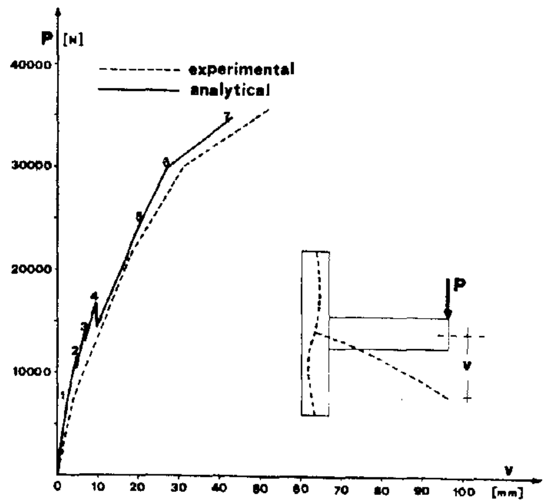


Figure B

Agreement between experimental and analytical results is not so good as it is in the previous example. This fact probably follows from imperfect modelling of behavior of r.c. members, whose stiffness was kept constant in the whole field analyzed. Better results could be obtained by introducing a progressive loss of stiffness in the most stressed r.c. members.

Of course the experimental values of stiffness in the damaged structure are not available to the designer, but analytical methods for their evaluation (at least for the first three stages of r.c. behavior, following data given in the C.E.B. or A.C.I. codes) are reliable.

The same thing applies when designing a steel encasement for strengthening an integral but weak member. In this case, the analytical procedure will involve changes of stiffness in r.c. members as well.

More sophisticated automatic non-linear procedures are also being employed, but the results obtained so far show that this elementary method illustrated is sufficient for the task at hand.

ACKNOWLEDGEMENT

The research has been financed by the C.N.R. (National Research Council).

BIBLIOGRAPHY

(1) Strengthening of Building Structures. Diagnosis and Therapy - IABSE Symposium - Venezia 1983.

(2) Assessment of Concrete Structures and Design Procedures for Upgrading (Redesign) - CEB bull. No. 162. 1983.

(3) Response of Structural Concrete Critical Sections Under High Level Reversed Actions - CEB bull. No. 161. 1983.

(4) A. Parducci, A. Samuelli-Ferretti - Flessione alternata su elementi di cemento armato sottoposti a carico assiale. Indagine sperimentale. Ist. di Scienza delle Costruzioni, No. II-149, Rome 1974.

(5) J. Warner - Repair and Retrofit of Buildings. Overview of U.S. Experience. Current Practice and Weaknesses. I Seminar of US/Japan Cooperative Research Program in Earthquake Engineering on Repair and Retrofit of Structures. University of Michigan, Ann Arbor, 1981.

(6) S. Sugano - Guidelines for Retrofitting (Strengthening, Toughening and/or Stiffening) Design of Existing R.C. Buildings. II Seminar, ditto, 1982.

(7) S. Nakata, T. Kawashima, H. Sekiguchi - A Repair Effect of R.C. Joint Assemblies Subjected to Seismic Loads. III Seminar, ditto, 1983.

(8) V. Bertero - Studies Regarding Repair and Retrofitting. III Seminar, ditto, 1983.

(9) Douane Lee - Original and Repaired R.C. Beam-Column Subassemblages Subjected to Earthquake-Type Loading. Ph.D. Thesis, University of Michigan, Ann Arbor, 1976.

(10) A Samuelli-Ferretti - Tests on R.C. Members Repaired Following Different Techniques. AICAP-CEB Symposium, Rome 1979.

(11) A. Migliacci, G. Mancini, P. Napoli - Valutazione critica dell' efficienza di interventi di riparazione su elementi strutturali soggetti a danni di natura sismica. 2° Convegno Nazionale "L'Ingegneria Sismica in Italia" - Rapallo (Italy), 1984.

(12) G. Via, M. Ciampoli, V. LaMesa - Seismic Behavior of R.C. Beam-Column Subassemblages: First Results of a Coordinated Experimental Research on Repair Techniques. 7th ECEE, Athens, Greece, 1982.

(13) N. Avramidou Maio - Coordinated Experimental Research - General Outline. 7th ECEE, Athens, Greece, 1982.

Strengthening of Unreinforced Masonry Buildings

Joseph Plecnik (I), Thomas Cousins (II), and Edward O'Conner (III)

SUMMARY

The object of this research project was to determine the factors influencing the increased strength of a multi-wythe, unreinforced brick masonry wall after being repaired with the proposed technique. The research included testing of wall panels and prisms as well as developing a design technique.

INTRODUCTION

In the early 1900's many buildings were erected using load-bearing, multi-wythe, unreinforced brick masonry. These buildings were designed using empirical methods with little or no attention given to seismic considerations. This type of unreinforced brick masonry has been able to withstand dead, live, and wind loads, but has proven to be inadequate in resisting seismic loads. The proposed method involved strengthening multi-wythe, unreinforced brick masonry walls for seismic loads. Multi-wythe unreinforced brick masonry consists of 3 or more wythes with the collar joint partially slushed with mortar and often every sixth course a header course. These walls are typically from 1 to 3 stories high. The proposed method would involve coring a 5.08 cm to 12.7 cm diameter hole vertically through a wall. A reinforcing bar would be placed in the core hole with filler material poured into the hole. The filler material could be unfilled or filled epoxy, sand-filled polyester, or grout. The distance between vertical cores, the size of the reinforcing steel, and the size of the core would depend on the seismic design requirements of the wall. In the testing program, three buildings located in the Raleigh, NC. area were chosen as typical of above described brick masonry construction, and were designated as Buildings #3, #4, and #5. After determining the compressive strength of the brick and performing "shove" tests, portions of the walls in these buildings were strengthened using the previously described method. After the walls were strengthened, panels and prisms were cut out of these walls, and transported to the laboratory for testing. The panels were loaded cyclically

(I) Associate Professor, North Carolina State University, Civil Engineering Department, Raleigh, N. C.

(II) Graduate Research Assistant, North Carolina State University, Civil Engineering Department, Raleigh, N. C.

(III) Retired Chief Building Official, City of Long Beach, Long Beach, California

for resistance to in-plane shear and out-of-plane moment. The prisms were used to determine the compressive strength of the masonry.

TEST PROGRAM AND RESULTS

Materials

Reinforcing Steel. All reinforcement used in the program consisted of No. 5 deformed bars, grade 60 steel, and met ASTM specifications A 615.

Brick. The bricks were solid and varied in color from light orange to dark purple. The 5.08 cm and 10.16 cm diameter cored pieces of brick were capped and tested according to ASTM C 67 to obtain the compressive strength of the brick. The average compressive strength of brick samples from Buildings #3, #4, and #5 was 16.0 MPa, 28.6 MPa, 19.4 MPa respectively.

Mortar. The wall specimens were removed from three brick buildings in Raleigh, North Carolina. Building #3 contained brick masonry that was constructed about 1890 using lime mortar. Building #4 was a small, one-story garage constructed about 1910 using masonry containing poor quality mortar, but higher quality than Building #3. In Building #5, a two-story brick masonry garage built about 1915, cement mortar similar to Type N mortar was used. Full mortar bedding was used, however, the collar joints between wythes were only partially full. "Shove" tests were performed on Buildings #3 and #5. The location of the "shove" tests ranged from the top to the bottom of each wall. The average failure shear stress (from "shove" test) for Building #3 was .19 MPa and for Building #5 was .34 MPa (Ref. 1).

Core Filler. In a related research project, 59 small scale specimens were built using #5 reinforcing and 3 types of filler material in the core. The 3 types of filler material were grout, a sand/polyester mix, and a sand/epoxy mix. The specimens were 2 bricks high and 20.32 cm square in cross section and were subjected to a static shear load. The results of the testing showed that a sand/polyester filler material was about 30% stronger than grout, and its strength approached that of a sand/epoxy mix. It also has the flow characteristics of a sand/epoxy mix. Due to the similarity in strength and flow characteristics of a sand/polyester mix to a sand/epoxy mix, grout and sand/epoxy mix were the filler materials used in the testing described herein. The grout was prepared following ASTM C 476. The epoxy sand filler was an epoxy adhesive with masonry grade sand added at a 1 to 1 volume ratio. 100% of the sand passed a #8 sieve and 5% passed a #100 sieve. The epoxy adhesive was a two component system with room temperature properties of 82.7 MPa tensile strength, 20.7 MPa shear strength, 46.3 MPa compressive strength, and a 1000 cps viscosity. The sand and epoxy were thoroughly mixed before pouring, with a pot life of about 30 minutes.

Core Drilling and Filling Method

All cored holes were either 5.04 cm or 10.16 cm in diameter. A Target electric drill rig was used with a diamond tipped bit. The holes were cored dry, with coring proceeding at a rate of 30 cm. per 5 minutes. The cored pieces of brick were tested to obtain the compressive strength of the brick. The core filler material was poured into the cored hole after the reinforcing steel was set in place.

Test Specimens and Test Methods

Prisms. The prisms contained a minimum of three bed joints, were one wythe thick and approximately two bricks long. Four prisms were tested, all obtained from Building #5. With the exception of specimen size, prisms were capped and tested according to ASTM E 447. The deflection across two bed joints was measured using a L.V.D.T.

Out-of-Plane Test Panels. All panels were approximately three wythes thick (30.5 cm) seven courses high (53.3 cm), and 2 1/2 bricks long (50.8 cm) with varying core size and filler material. Both the top and bottom of the brick panels were inbedded in reinforced concrete caps. The caps were used to hold the specimen in place during testing; with all loads applied to the cap instead of the brick panel. During testing, the concrete base of the panels was clamped to the floor, and load was applied to the top cap normal to the plane of the panel as shown in Fig. 2. Static cyclic load of six or seven cycles was applied to each panel with the odd cycles applied to one face and the even cycles applied to the other. For each cycle the maximum applied load corresponded to the beginning of rapid inelastic degradation of the panel. Bearing loads were not applied to these test panels, and only net deflections are reported herein.

In-Plane Panels. The size and capping procedure was the same for in-plane and out-of-plane tests. A lateral cyclic load was applied in the plane of the panel to the concrete cap. For each cycle, the maximum applied load corresponded to the beginning of rapid inelastic degradation of the panel. The panels were subjected to six or seven cycles with odd cycles applied to one end of the panel and the even cycles applied to the other. To offset the overturning moment in the panels and to simulate any compression forces in the panel due to dead load, a vertical bearing load was applied to the cap equal to approximately 75% of the lateral load. A sketch of the testing set up is shown in Fig. 3. Net deflections were measured in the direction of loading and are reported herein.

Results of Prism Tests

Four brick masonry prisms, all from Building #5, were tested in compression, and the average compression strength of the four panels was 3.69 MPa. The brick failed in compression and caused failure of the specimen because the confined strength of the mortar was greater than the ultimate strength of the prism.

Results of Out-of-Plane Tests

The results of the out-of-plane tests are shown in Table 1. A total of 5 specimens were tested, one from Building #3 and 2 each from Buildings #4 and #5. In three specimens, 4D, 5M, and 5D, the maximum load occurred in the first cycle as planned, but, in specimens 3E and 4A, the maximum load occurred in cycles 4 and 6 respectively. Due to the difficulty in locating the beginning of inelastic behavior, specimens 3E and 4A were not loaded to the limit of their elastic range during the first and subsequent cycles. Therefore, the maximum load capacity resisted by specimens 3E and 4A did not occur in cycle 1. Typically, initial failure consisted of horizontal cracks forming in the bed joints on the tension face of the specimens during the first cycle. The tension cracks were followed by crushing of the mortar on the compression face during the first cycle. During the further load cycling, the tension cracks increased in size and number while the mortar continued crushing. Compression failure of the bricks was not observed. After testing was completed, the specimens were taken apart for observation. Flow of the filler material into the collar joint is necessary to insure shear transfer between the reinforcing steel and the exterior wythe in compression. The specimen with 10.16 cm diameter cores were generally observed to have the best flow of core filler material into the collar joint of the wall specimens. Due to greater flowability, the epoxy/sand mixture resulted in greater collar joint penetration than grout. A typical load vs. net deflection plot is shown in Fig. 4. Even though the specimen is loaded to ultimate during Cycle 1, the specimen retains enough strength to resist sizable loads thru Cycle 6 and 7.

Results of In-Plane Tests

The results of the in-plane tests are presented in Table 2. Seven specimens were tested, 3 from Building #3 and 2 each from Buildings #4 and #5. In six of seven specimens, the maximum load occurred during the first cycle, but, in specimen 4H the maximum load occurred during cycle 5. The maximum shear stress listed in Table 2 is the maximum load divided by the gross cross-sectional area. The first signs of failure of the specimens was cracking in the head and bed joints on either face of the specimens with some cracks going through bricks. As the load and number of cycles increased, the number and size of cracks increased. After testing was completed, the specimens were dissassembled for observation of the flow characteristics of the filler materials with the same results as observed for the out-of-plane specimens. A typical load vs. net deflection plot is shown in Fig. 4. The specimen was loaded beyond its elastic limit during cycle 1, but was able to resist sizable loads thru cycle 6.

ANALYSIS OF TEST RESULTS

Out-of-Plane and Prism Tests

An ultimate strength analysis of the test specimens will be compared to the results of an analysis based on general flexure theory. Presently, research for ultimate strength analysis of reinforced brick masonry is being conducted, and preliminary findings show that ultimate strength theory reasonably predicts the behavior of reinforced brick masonry (Ref. 2). Following is an ultimate strength analysis based on a rectangular stress block and an analysis based on general flexure theory.

Assuming the total compression force is $.85\, f'_m [ba]$ and the moment arm is $d - a/2$ (see Fig. 1), the Ultimate Moment equation is:

$$M_u = .85\, f'_m\, ba(d - a/2).$$

All quantities are known in this equation except "a", the depth of the assumed rectangular stress distribution. "M_u" will be taken as the average failure moment of panels from Building #5 as shown in Table 1, b and d as shown in Fig. 6, and f'_m is the average compressive strength of the prisms. Using all these quantities yields a = 3.33 cm. It follows that the total compressive force is 63.38 kN, and, therefore, the stress in the steel (1 #5 bar) is 317.2 MPa. It is significant to notice that the steel did not yield (yield stress equals 413.7 MPa) and that the compression block was in the exterior wythe of brick. This confirms conclusions from observing testing of the out-of-plane specimen.

In General Flexure Theory (see Fig. 1) the Ultimate Moment equation is:

$$M_u = B_1 k_3\, f'_m\, bkd(d - k_2 kd)$$

or

$$M_u = f_m\, bd^2 w(fs/fy)(1-(wk_2/\beta_1 k_3\ (f_s/f_y))$$

where

$$w = A_s f_y / bd\, f'_m \text{ and } k = (w/\beta_1 k_3)(f_s/f_y).$$

All terms in the second equation for M_u are known except f_s, k_2, β_1, and k_3; and $k_2/\beta_1 k_3 = .44$ (Ref. 2) and $w = .2414$. Now one equation with one unknown "f_s", remains and yields $f_s = 306.8$ MPa. Again "f_s" is less than yield stress as expected.

In-Plane Tests

The "shove" test is a field test for measuring the shear strength (in the plane of the bed joint) of unreinforced brick masonry. It was used here to determine the pre-repair shear strength of Buildings #3 and #5. The average failure shear stress (from the "shove" test) for

Building #3 was .19 MPa while the average failure shear stress from the in-plane test was .58 MPa (see Table 1) over the gross cross section. This is a 300% increase in the shear resisting capability of specimen from Building #3. All these specimens were repaired with 10.16 cm diameter cores. The average failure shear stress from the "shove" test for Building #5 was .34 MPa while the average failure shear stress from in-plane test after repair was .52 MPa (see Table 2) over the gross cross section. This is a 54% increase in the shear resisting capability of specimen from Building #5. These specimen were repaired with 5.08 cm dia. cores.

Conclusions

Large diameter cores generally result in greater flow of the filler material into collar joints; therefore, providing an adequate shear transfer mechanism to attain ultimate out-of-plane moment capacity, and resulting in a greater effective area to resist in-plane shear forces. Sand filled epoxy is superior to grout as a filler material because of its strength and flow characteristics, but epoxy's cost limits its use. A polyester/sand mixture is a possible alternative due to its lower cost and strengths approaching those of an epoxy/sand mixture. Ultimate strength analysis similar to that for concrete yields satisfactory results when predicting the increased flexural strength of strengthened brick masonry walls.

References

1. Quaddoumi, Lutof F., "Masonry Joints", Project in Partial Fullfillment of the Requirements for the Degree of Master of Science Degree, North Carolina State University (SLR-84-3-5).

2. Personal Communication from Ongoing Research, National Bureau of Standards, Washington, D.C., and the University of Texas at Arlington.

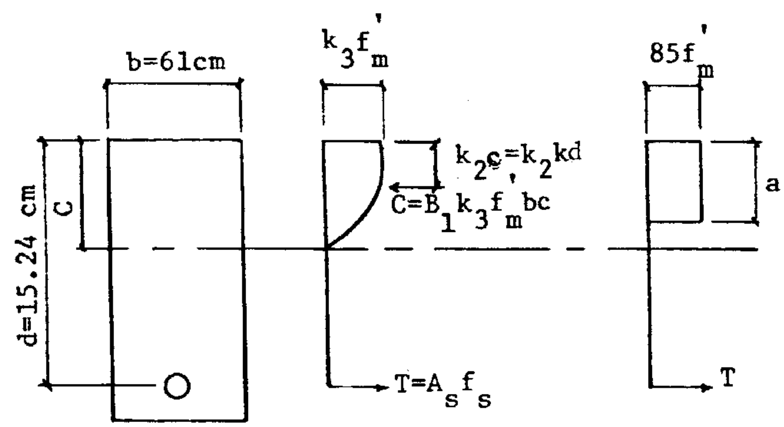


Fig. 1 General Flexure Theory vs. Ultimate Strength Analysis

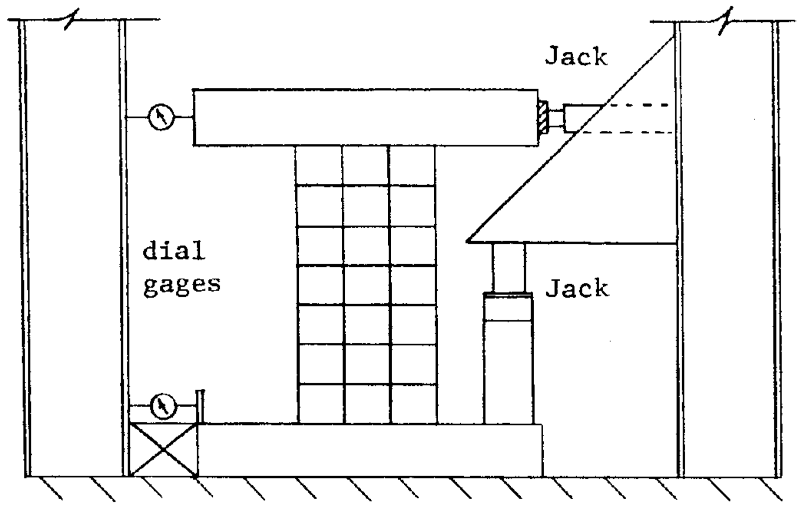


Fig. 2 Out-of-Plane Test Set-up

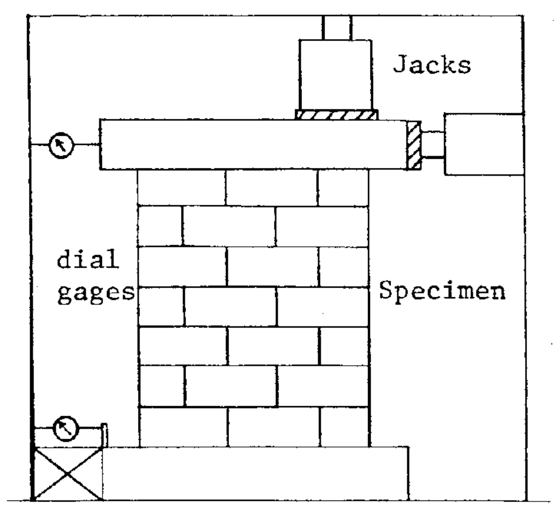


Fig. 3 In-Plane Test Set-up

Table 1 Out-of-Plane Tests

Specimen #	Core Size & Type	Gross Cross Section Area (cm^2)	Max. Load (kN)	Max. Moment (kN m)	Cycle of Max. Load
4D	5.08 cm Grout	1935	10.1	7.92	1
5M	5.08 cm Grout	1761	12.5	9.65	1
5D	5.08 cm Grout	1677	10.7	7.73	1
3E	10.16 cm Grout	1761	10.9	7.48	4
4A	5.08 cm Grout	1677	20.0	13.99	6

Table 2 In-Plane Tests

Speci-men #	Core Size & Type	Gross Cross Section Area	Max. Load (kN)	Max. Shear Stress (MPa)	Cycle of Max. Load
4B	5.08 cm Grout	1987	35.1	.18	1
3E	10.16 cm Epoxy	1626	122.8	.76	1
5C	5.08 cm Grout	1587	11.2	.70	1
5L	5.08 cm Epoxy	1626	55.6	.34	1
3C	10.16 cm Grout	1548	90.7	.59	1
3M	10.16 cm Grout	1626	67.2	.41	1
4H	5.08 cm Epoxy	1807	58.7	.32	5

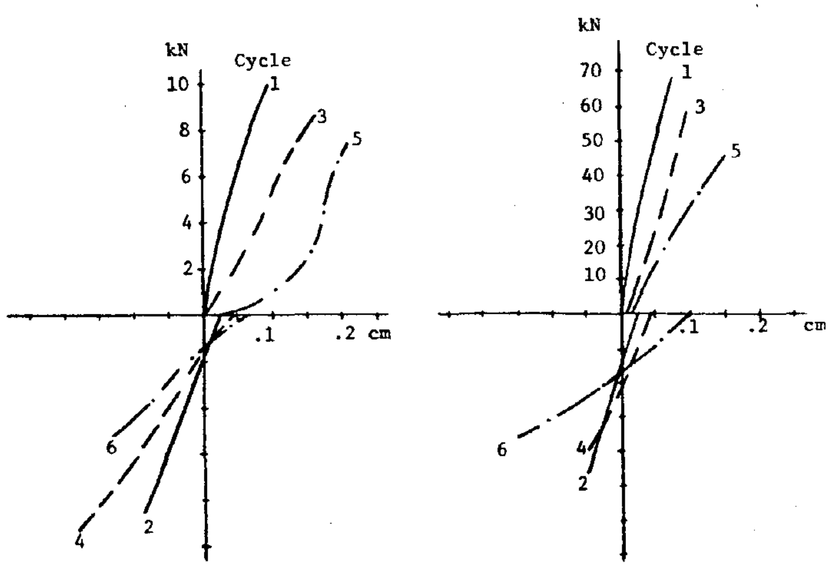


Fig. 4 Out-of-Plane (left) & In-Plane Load vs. Net Deflection Plots

Seismic Vulnerability of Pozzuoli Buildings

Roberto Ramasco (I)

SUMMARY

In this report I will try to outline the criteria adopted for the investigation on the vulnerability of Pozzuoli buildings. Such a study is still in progress, hence the results must be considered as provisional, while relative elaborations have just begun.

Since bradyseism phenomeno increased in the Phlegraean area and shocks became more frequent in October, 1983, a survey on multistorey buildings in Pozzuoli was held to be necessary. The alarm was given by the shock that occurred on October 4, more violent than previous ones, that stressed the fact that the "shaking" action could have a prevailing influence on building behavior vis-a-vis recurrent slow-working deformations effected by bradyseism.

INTRODUCTION

The Department for Coordination of Civil Protection set up a special purpose Scientific Technical Committee (S.T.C.) entrusted with the task of organizing "usability" checks on damaged buildings and infrastructures in the Pozzuoli area.

New shocks, and consequent changes in damage degree that emerged very clearly in some cases, emphasized the inadequacy of "usability" checks and required a survey that, taking into account the damage degree of buildings (which is, however, a significant finding as far a usability is concerned), allows the intrinsic "vulnerability" of buildings in view of possible intensification of the phenomenon to be determined.

So it has been suggested to make use of the competence and experience of the National Unit for Earthquake Protection along the new research line concerning the "Census and Vulnerability" of old buildings, to which many researchers from different universities contribute.

On the basis of research carried out initially, that has been discussed in order to elaborate a well-focused approach to the problem, and taking into account previously elaborated proposals concerning operative tools for vulnerability tests on masonry 1 and r.c. 2 buildings, different formats for these two typologies were prepared just to take into consideration morphological and typological characteristics which are peculiar to buildings in this area. In order to complete this introductory note, we should recall that before being used in the area covered by the survey, such forms have been tested in the field, in order to check their effectiveness, calculate weighted coefficients for dif-

(I) Professor, University of Naples, Italy

ferent "indicators" and, not the least, figure out correctly the timing of operation.

SURVEY TEAMS AND REPORT

Eight surveying teams made up of expert researchers and assistants were formed and asked to survey a lot of about 50 buildings, both in the core and in the suburbs, including reinforced concrete, masonry and mixed buildings in proportion to the real composition of the built heritage in Pozzuoli. This calibration phase lasted for three days and, in the course of it, punctual tests aimed at checking whether the forms could be easily and correctly used were carried out just to overcome any problem or malfunction that should arise. In the final report on the calibration phase issued by the "experts", stressing again the limit and the areal, rather than punctual, character of vulnerability tests carried out by investigation "at sight", the feasibility of the form as a useful tool has been stated, recognizing it as a survey means for technicians operating on Pozzuoli buildings.

In the report itslef, on the basis of the hypothesis that has been repeatedly put forward by seismologists, according to which seismic events may possibly occur in the area with an intensity commensurable to that of the October 4, 1983 earthquake, vulnerability limits for three significant groups have been set for both masonry and reinforced concrete buildings. A first group was characterized by a low score in vulnerability and no worries about stability of buildings. A third group was characterized by a high score in which buildings showed a precarious overall stability. Finally, an intermediate group in which static conditions cannot be identified clearly has been envisaged.

The limits of the vulnerability form for the "Pozzuoli problem", through which technicians are asked to express their opinion on building fitness in the event of a shock commensurable to the October 4 earthquake, have been well-defined by the final sentence in the above-mentioned report which we quote in full:

> ...nevertheless it is indespensable to specify that such separate groups cannot superintend an automatic criterion for building structures fitness (or unfitness). In fact, in case of uncertainty as to previously mentioned characteristics, or, as has been highlighted above, for any local or general situations that have not been envisaged by the form, static fitness must be ascertained and described only by a punctual analysis and a subsequent responsible evaluation which cannot be made without the advice given by the technician in charge of the survey. Thus the filling out of the form provides technicians with objective guidelines for survey of a masonry or reinforced concrete building and for analysis or parameters contributing to seismic vulnerability, while the classification groups can suggest useful formulations of the fitness opinion that is required.

DAMAGE AND VULNERABILITY SURVEY FORM

The form employed for surveys consists of the two pages reproduced in transparencies and has been obviously prepared as a coded input for the computer and further processing. It is divided into the following different sectors:

Date

Building location

Material data

Final use

Structural characteristics

Previous interventions

Damage

Vulnerability

Structural fitness

Remarks

In order to obtain a highly uniform behavior in filling out the form, many briefings have been organized for the teams so as to discuss and analyze, even by examples, both the special rules elaborated for filling out the forms and those related to the "Damage", and finally those specifically referring to "Vulnerability".

Except for the sectors "Vulnerability" and "Structural Fitness", the form is actually identical to the form that has been previously employed for estimating damages caused by the November 23, 1980 earthquake 3 . In fact, the main parts of it have not been modified not only because of their proven effectiveness, but also to prepare future data processing, as far as vulnerability is concerned, so as to be able to identify any implicitly recognizable behavior, although with a lower degree of resolution, thanks to data on structural characteristics which are at any rate more synthetic and simple.

Thus the presence of the sector "Structural Characteristics" can be explained, and its data are qualitatively similar, although more synthetic, to those of "Vulnerability", which will be analyzed in the following paragraph. I should point out that great attention has been focused on "joint" buildings, by which I mean those constructions that have been presumably built in different epochs, possibly with different materials which, being more or less connected with each other, do not possess perfect autonomy in absorbing horizontal seismic effects. Only

Il Ministro Segretario di Stato per il coordinamento della Protezione Civile

COMITATO TECNICO SCIENTIFICO PER IL BRADISISMO DI POZZUOLI

SCHEDA DI RILEVAMENTO DEI DANNI E DELLA VULNERABILITÀ DI EDIFICI PER USO ABITAZIONE O MISTO

COMUNE DI POZZUOLI

DATE
DATA 1 (G M A) — 7 SQUADRA — 10 Nº ORDINE (Numero, Lettera)

LOCALIZZAZIONE EDIFICIO — BUILDING LOCATION

VIA / PIAZZA / LOCALITÀ 14 — Nº CIVICO 39, 43 (Lettera, Isolato)

RIFERIMENTO CARTOGRAFICO 48

EDIFICIO IN CENTRO 47 [1] NUCLEO 47 [2] CASA SPARSA 47 [3] RIFERIMENTO CATASTALE FOGLIO 53

MAPPALE 57

IN CENTRO STORICO SI 60 [1] NO 60 [2] ED. URBANO 61 [1] RURALE 61 [2]

DATI METRICI — METRICAL DATA

SUPERFICIE (mq) 62

ALTEZZA (m) 66

VOLUME (mc) 68

FRONTI COMUNI AD ALTRI EDIFICI Nº 74

PIANI FUORI TERRA Nº 75

PIANI CANTINATI Nº 78

SOTTOTETTI PRATICABILI Nº 80

DESTINAZIONE D'USO — FINAL USE

ABITAZIONE SI 81 [1] NO 81 [2]
Nº ALLOGGI 82
Nº SCALE 84
Nº VANI COMPLESSIVI 86

ATTIVITÀ PRODUTTIVE SI 89 [1] NO 89 [2]
ARTIGIANATO Nº 90
COMMERCIO Nº 92
TURISMO Nº 94
ALTRO Nº 96

ANNESSI SI 98 [1] NO 98 [2]
OPIFICI Nº 99
MAGAZZINI Nº 101
STALLE Nº 103
ALTRO Nº 105

CARATTERISTICHE STRUTTURALI — STRUCTURAL CHARACTERISTICS

ANNO DI COSTRUZIONE 110

STRUTTURA PORTANTE 107
- MURATURA PIETRAME [1]
- MURATURA TUFO [2]
- MURATURA MATTONI [3]
- CEMENTO ARMATO [4]
- STRUTTURA MISTA [5]

STRUTTURE ORIZZONTALI 108
- VOLTA [1]
- SOLAIO IN LEGNO [2]
- SOLAIO IN FERRO [3]
- SOLAIO IN C.A. [4]

COPERTURA 109
- LEGNO E TEGOLE [1]
- C.A. [2]
- ALTRO [3]
- DIVERSE [4]

OPPURE 114
- PRIMA - 1900 [1]
- 1901 - 1943 [2]
- 1944 - 1962 [3]
- 1963 - 1971 [4]
- DOPO 1971 [5]
- IGNOTO [6]

PRECEDENTI INTERVENTI / INTERVENTI SI 115 [1] NO 115 [2]
- EX LEGGE 475/1981 SI 116 [1] NO 116 [2]
- EX ORDINANZA N. 381 SI 117 [1] NO 117 [2]
- ALTRI SI 118 [1] NO 118 [2]

DATA 1 (G M A) — 7 SQUADRA — 10 Nº ORDINE (Numero, Lettera)

DANNO — DAMAGE

	STRUTTURA PORTANTE	SOLAI	TETTI	TAMPONATURE ESTERNE	PARETI INTERNE	SCALE
NESSUN DANNO	1	1	1	1	1	1
IRRILEVANTE	2	2	2	2	2	2
LIEVE	3	3	3	3	3	3
NOTEVOLE	4	4	4	4	4	4
GRAVE	5	5	5	5	5	5
GRAVISSIMO	6	6	6	6	6	6
CROLLATO PARZIALMENTE	7	7	7	7	7	7
DISTRUTTO	119 [8]	120 [8]	121 [8]	122 [8]	123 [8]	124 [8]

FONTE DI INFORMAZIONE SU DANNI DISTRUTTI
125 [1] INFORMAZIONE DIRETTA [2] FOTO [3] STUDI PRECEDENTI

VULNERABILITÀ — VULNERABILITY

MURATURA 126 [1]

DATI GEOMETRICI d (m) 127 h (m) 130

RESISTENZA CONVENZIONALE

Aox (mq) 133 Aoy (mq) 136

A (mq) 139 H (m) 144

q 147

	CLASSE A	B	C	
DISTANZA MAX MURI	151 [0]	[3]	[8]	__
ALTEZZA MAX INTERP.	152 [0]	[3]	[4]	__
QUALITÀ COLLEGAMENTI	153 [0]	[4]	[20]	__
ORIZZONTAMENTI	154 [0]	[10]	[14]	__
COPERTURA DI TETTO	155 [0]	[5]	[12]	__
RESISTENZA CONVENZIONALE	156 [0]	[10]	[20]	__
STRUTTURE VERTICALI	157			
COMPATTEZZA PIANTA	158 [0]	[2]	[4]	__
SNELLEZZA COSTRUZIONE	159 [0]	[2]	[4]	__
MORF. SUP. TERRENO	160 [0]	[2]	[4]	__
STATO DI FATTO	161 [0]	[6]	[12]	__

INDICE DI VULNERABILITÀ 162

PERICOLO DI CROLLI INDOTTI CADUTA MASSI ECC. SI 165 [1] NO 165 [2]

CEMENTO ARMATO 126 [2]

RESISTENZA CONVENZIONALE σ_c (kg/cmq) 127

QUALITÀ DELLA COSTRUZIONE Rm (kg/cmq) 130

PIANO SOFFICE per 1,5 SI 133 [1] NO 133 [2]

	CLASSE A	B	C	
STRUTTURA PRINCIPALE	151 [0]	[6]	[10]	__
ORIZZONTAMENTI	152 [0]	[3]	[6]	__
RESISTENZA	153 [0]	[11]	[22]	__
TAMPONATURE	154 [0]	[4]	[10]	__
PIANO SOFFICE	155 [0]	[6]	[12]	__
COMPATTEZZA PIANTA	156 [0]	[3]	[6]	__
QUALITÀ COSTRUZIONE	157			
MORF. SUP. TERRENO	158 [0]	[2]	[4]	__
STATO DI FATTO	159 [0]	[10]	[20]	__

INDICE DI VULNERABILITÀ 162

TERRAPIENI SPINGENTI SI 166 [1] NO 166 [2]

IDONEITÀ STRUTTURALE — STRUCTURAL FITNESS

IDONEO SI 167 [1] NO 167 [2] SUB COND. 167 [3] NESSUN PARERE 167 [4]

168 [1] 169 [1] 170 [1] 171 [1] 172 [1] 173 [1]

OSSERVAZIONI — REMARKS

RILEVATORI

one team was entrusted with surveying "joint" buildings, by using one form for each individual building, thus ensuring an immediate "linkage" of the forms. In this way, after having examined each individual situation, a complete picture of behavior could be obtained so as to express a well-motivated opinion on fitness.

As to the sector "Damage", as has been previously said, we have used the codification already tested in the Irpinia earthquake with eight levels of damage with reference to most important structural and non-structural parts, namely: "carrying structure", "floors", "roofs", "external curtain walls", and "stairs".

Let us leave out for the moment the sector "Vulnerability", to which we will return later, just to make some further remarks on "Structural Fitness", which represents the conclusive datum in our survey and is referred, as has been said above, to an earthquake intensity commensurable to the October 4 shock.

Even though the intensity of the "reference" earthquake has not been determined in an objective and unambiguous way by this formulation, nevertheless such a specific objective was necessary to the survey since as new constructions built in the Pozzuoli municipality were required to comply with the special law for Possuoli, which provided that all recommendations for second category seismic areas be applied, all existing buildings would result to be unfit according to the law. On the other hand, making reference to an earthquake already endured by buildings or slightly stronger could provide useful guidelines to technicians in their survey.

Going back to fitness, by it we meant to take account of the general seismic behavior of buildings (fit YES + fit NO), whereas any localized situations of danger do not concern the overall behavior (partioning in precarious equilibrium, unsafe cornices, almost breaking coverings, etc.) and determine a sub-condition fitness in the sense that removal of damage (obtainable by works that do not effect the structure's resistance to seismic effects) implies that fitness be covered.

This criterion, which leaves out non-structural parts in terms of vulnerability, thus referring to the structure only, differentiates the form conceived for Pozzuoli from those which are still being tested. As a matter of fact, a particular emphasis has been placed on those cases in which recovery appears to be relatively easy by localized interventions on non-structural parts.

Finally, a fourth case has been envisaged for structural fitness "No Opinion", when, in the presence of works in progress that have been authorized, no survey of the building was considered to be necessary with the aid of a technician responsible for the project and/or the supervision of construction.

INTRINSIC VULNERABILITY

Intrinsic vulnerability of buildings, as it has been defined in other emerging proposals in the framework of National Unit for Earthquake Protection that are being tested at the moment, is assessed by some "indicators" which are considered significant. They have been assigned a certain score that contributes, together with the score of other indicators, to the vulnerability index of the building, thus allowing one to classify the built heritage according to a relative scale of vulnerability. It is obvious that the validity of the indication depends very much upon the significance of indicators that have been chosen, and on the relative weight given to them, whereas there are open questions, such as the determination of the allowable vulnerability threshold in different seismic areas and, first of all, the establishing of a correspondence for the vulnerability of different typologies, such as, for example, for masonry buildings and reinforced concrete buildings or, within each typology, among constructions typical of different regions.

There is no easy solution to this problem and even though many researchers are devoting their attention to this field, we should not expect concrete results in the short run. Surveys in the field, supported by specific analyses of building samples, are the tool through which such research can be continued, but they require an amount of energy and resources that are certainly remarkable, so we should not miss the chance given by the Pozzuoli events to carry on research in this field and to respond at the same time to precise requests by the Department for Coordination of Civil Protection with an operative instrument.

The vulnerability form for Pozzuoli buildings has been drawn in such a way that some requirements be met, namely:

- to be able to end the surveys in reasonably short time periods (2 or 3 buildings a day for each team);

- not to perform tests on foundations, limiting very much those on the elevation structure;

- not to ask for analytical tests on frames except in oversimplified form;

- to ensure the greatest clarity in the classification of different indicators;

- to keep a certain homogeneity in surveys of masonry buildings and reinforced concrete buildings.

In substance, such surveys can be carried out at sight by tools that are available and easy to use.

In order to reduce uncertainties in classification, only three classes have been envisaged, namely:

Class A: Favorable situation;

Class B: Intermediate situation;

Class C: Unfavorable situation.

To avoid overly specific calculations among vulnerability indicators for both masonry buildings and reinforced concrete buildings, no eccentricity parameter has been set between rigidities distribution and masses distribution.

Given the above-mentioned limitations and the uncertainties implied by any more accurate survey, the foundation ground has been introduced into the forms in a very limited way, taking into consideration its superficial morphological aspect only, which has been given, however, a low vulnerability score.

Let us examine now very briefly what are the indicators adopted both for masonry buildings and for reinforced concrete buildings.

Masonry Buildings

Adopted parameters and the vulnerability scores assigned to them are reported in the following table:

	CLASS		
	A	B	C
Maximum distance between walls	0	3	6
Interfloor maximum height	0	2	4
Quality of connections	0	6	20
Floors	0	10	14
Covering or roofs	0	5	12
Conventional resistance	0	10	20
Vertical structures			
Plan compactedness	0	2	4
Building slenderness	0	2	4
Superficial morphology of the ground	0	2	4
State of things	0	6	12

Some evaluation, as is specified in the text, refers to the local situation; some other evaluation refers to overall or average situations in the building.

Reinforced Concrete Buildings

It has to be pointed out immediately that the vulnerability test on reinforced concrete buildings, vis-a-vis masonry buildings, is far more complex and characterized by relevant uncertainties even when sophisticated tools are available. As is well-known, in fact, the correct execution of construction details (overlapped bars, correct positioning of reinforcement, efficiency of anchorage) is essential to seismic vulnerability to such an extent that survey at sight appears to neglect an undoubtedly relevant aspect.

Indicators and corresponding vulnerability scores assigned to different classes are reported in the following table:

	CLASS		
	A	B	C
Main structure	0	6	18
Floors	0	3	8
Resistance	0	11	22
Curtain walls	0	4	10
Soft storey	0	6	12
Plan compactedness	0	3	6
Construction quality			
Superficial morphology of the ground	0	2	4
State of things	0	10	20

CONCLUSIONS

Vulnerability tests based on the standards that we have briefly described above have been started by the end of November, 1983, overcoming organizational difficulties both in setting up and in training of the teams, and in practical execution of on-the-spot surveys which were carried out by summons of the City Hall just to contact the owners and tenants of uninhabited dwellings.

Weekly meetings have been and are being held even now with the technicians of the 38 teams working in the field, in order to ensure constant assistance to surveys and to solve the kind of problems they were faced with.

So far, 1762 buildings, totaling to 5.295.000m^3 and 9,400 dwellings, have been analyzed. 1,326 of them (equal to 3.242.000m^3 and 5,900 dwellings) are masonry buildings, and 436 (equal to 2.053.000m^3 and 3,500 dwellings) are reinforced concrete buildings.

145 "joint" constructions made up of 443 buildings have been detected. So they are made up of an average number of 3 buildings per each "joint" construction, and the maximum number has been reported for a "block" made up of 16 buildings on Napoli Street.

A synthetic picture of vulnerability for masonry and reinforced concrete buildings is given by the following figures in which both relative frequencies distribution and cumulative frequencies are reported.

We may point out for example that an almost even distribution of vulnerability index has been obtained for masonry buildings so that the cumulative frequency curve has an almost constant slope.

Instead, distribution of relative frequencies for reinforced concrete buildings is bell-shaped with a maximum peak corresponding to the 30-40 range of the vulnerability index.

This brief account of the vulnerability test on Pozzuoli buildings, which is still in progress, is aimed at explaining the methodology adopted for this certainly complex problem that has not been solved at all so far. The data that are being collected are a very valuable source that can be used for further research, but their meaning has not been completely spelled out yet.

The items chosen for the vulnerability indicators must be examined in detail in order to remove any redundant information, to perfect the values of vulnerability indices for different classes, and to detect any important omission.

REFERENCES

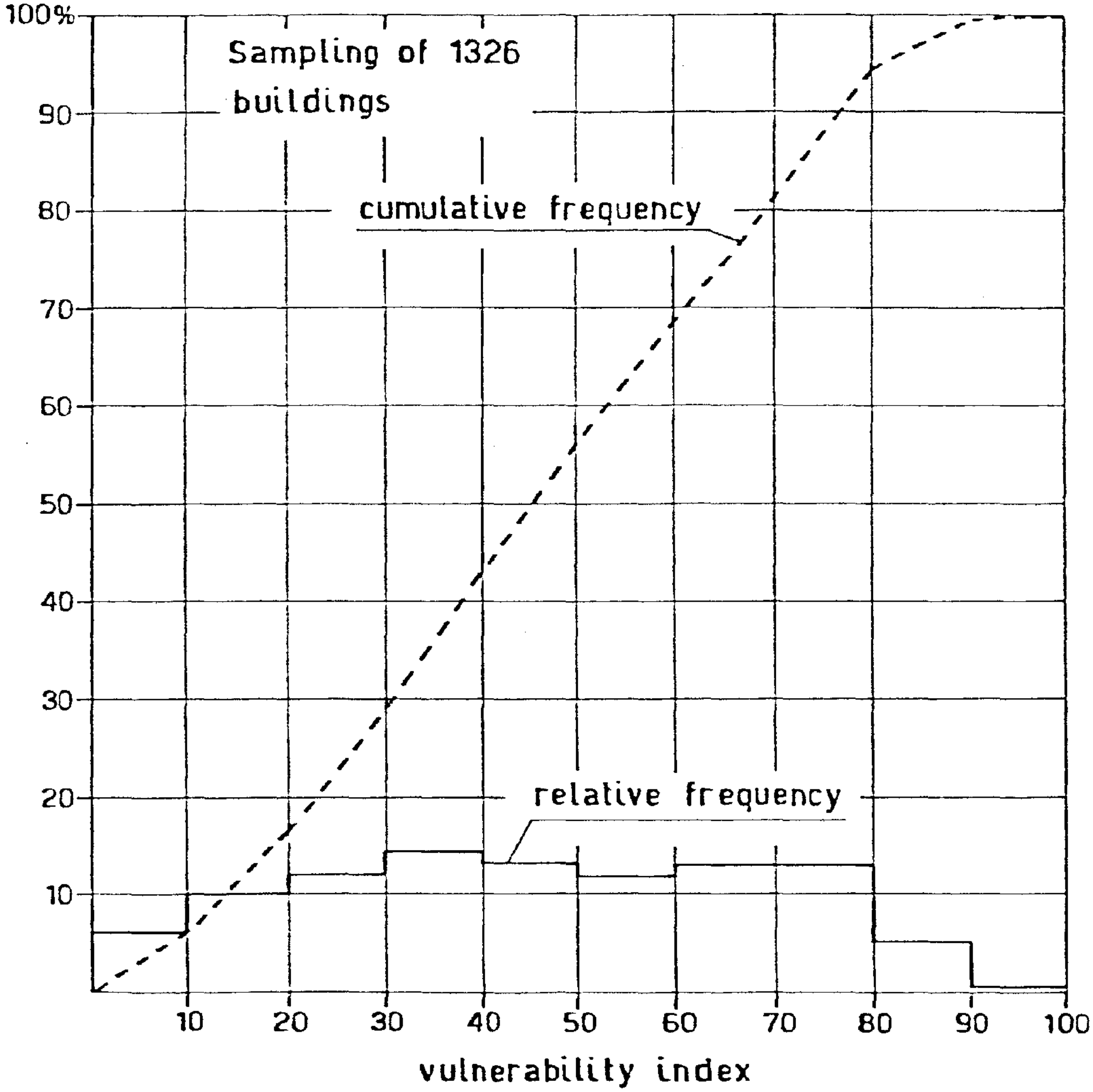
(1) D. Benedetti, V. Petrini: "Sulla vulnerabilita sismica edifici in muratura: proposta di un metodo di valutazione"

(2) P. Angeletti, C. Gavarini: "Assessing Seismic Vulnerability in view of Developing Cost/Benefit Ratios for Existing R.C. Buildings in Italy", 8th World Conference on Earthquake Engineering, San Francisco, 1984

(3) F. Braga, M. Dolce, D. Liberatore: "A Statistical Study on Damaged Buildings and on Ensuing Review of the M.S.K. - 76 Scale", Southern Italy, November 23, 1980 earthquake, 7 ECEE, Atene, 1982

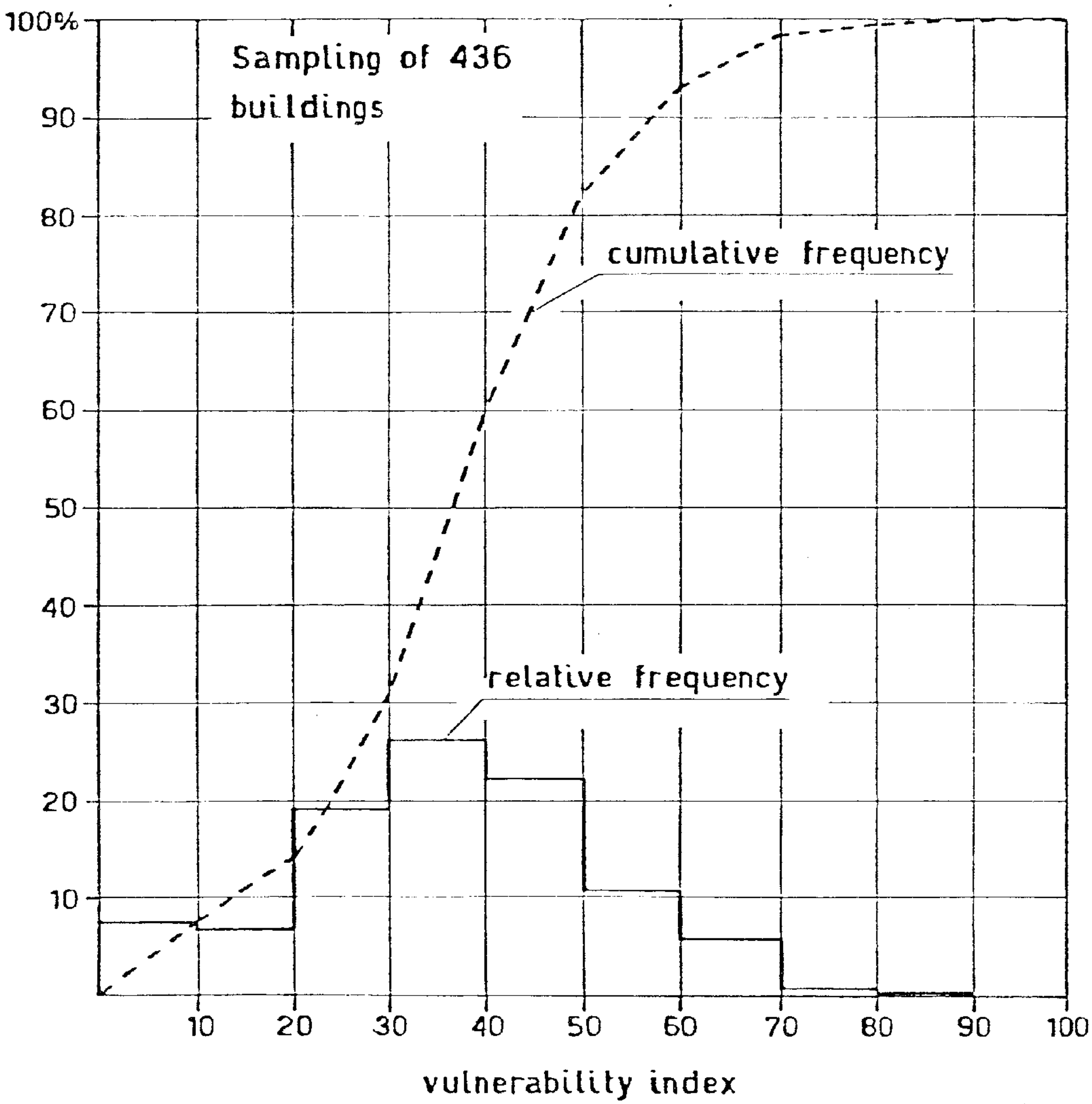
POZZUOLI - MASONRY BUILDINGS

Relative frequency and cumulative frequency of the vulnerability index



POZZUOLI - REINFORCED CONCRETE BUILDINGS

Relative frequency and cumulative frequency of the vulnerability index



INTENTIONALLY BLANK

Ductility Requirement for Reinforced Concrete Frame Design in Seismic Areas

James L. Stratta (1)

SUMMARY

This paper presents a brief background on why ductility is necessary for concrete members resisting cyclic loading due to seismic activity. It reviews many of the parameters that go into the design of Ductile Concrete Moment Resisting Frames and points out what literature should be reviewed by Engineers desiring to design these types of frames. The synthesizing effort of accumulating data from over fifty research papers provides the background information.

During the last few decades the necessity for ductility in concrete members subjected to seismic loading has been brought to the attention of the design professional. Let us review some aspects.

One of the earlier documents describing this concept is noted in the book "Design of Multistory Reinforced Concrete Buildings for Earthquake Motion" by Blume, Newmark, and Corning. This was published in 1961. Webster defines ductility as "capable of being fashioned into a new form." In engineering we think of it as "toughness" or an ability to distort without collapsing.

Ductility Factor is defined as a ratio between the maximum displacement and the yield displacement.

Cumulative Ductility Factor is the product of the Ductility Factor times the number of cycles to which the member is subjected.

In order for concrete to be classed as ductile, it must be well confined by usage of ties or stirrups spaced very closely together.

Let us now review some of the damage in recent earthquakes:

In the Manila, Philippines earthquake of 1968, the Philippine Bar Association (PBA) building suffered disastrous damage. Figure #1 (Figures are shown in Appendix "A") shows the overall building.

Figure #2 shows a badly distorted spirally reinforced column. Yet note the obvious toughness and capability to take "some" load.

---

(1) Consulting Structural Engineer, Menlo Park, California, USA

Note the close spacing of the spiral reinforcement. Figure #3 shows a tied column at the front. Note that the large spacing of ties has allowed the concrete to "flow out" of the column because it was not confined.

In the San Fernando, California earthquake of 1971, the Olive View Hospital suffered catastrophic damage. Figure #4 shows an elevation. Figure #5 shows the end of one of the wings. The obvious difference between a column with closely spaced ties versus one with ties spaced farther apart is readily evident.

In 1974, the Peru earthquake provided a most interesting example. Two one-story classrooms were very badly damaged as shown in Figure #6. While investigating the exterior, Figure #7 was taken showing failure of an exterior column. It seemed as though a failure of this type should occur either at the top or bottom of the column - so the interior was investigated. The reason for the location of the failure was immediately apparent, as shown in Figure #8. The failure had occurred where the close spacing of ties was discontinued and a larger spacing begun.

In the Friuli, Italy earthquake of 1976, the Elite Condominium, shown in Figure #9, was badly damaged. Figure #10 shows a close-up of the damaged column. The lack of ductility in this joint is apparent. This entire complex collapsed in an aftershock four months later.

The last example shows the El Centro County Services building after the earthquake of El Centro, California, in 1979. Figure #11 shows an elevation of the building which had to be totally demolished. The four failed columns are shown in Figure #12. Again the widely spaced ties allowed concrete to "flow out" of its core allowing the end of the building to drop approximately ten inches. Immediately below the failed portion of the column, closely spaced ties prevented damage in that area.

So much for background. It is quite apparent that ductility in concrete members subjected to siesmic loading is a necessity. The Applied Technology Council (ATC), located at 2471 East Bayshore Road, Suite 512, Palo Alto, California 94303, asked a team of three practicing engineers to synthesize over fifty research reports made throughout the world on the subject of concrete beam-column joints subjected to cyclical loading. These results have been published under Report ATC-11 "Seismic Resistance of Reinforced Concrete Shear Walls and Frame Joints: Implications of Recent Research for Design Engineers." The report also contains an excellent discussion of shear walls. This paper, however, will address itself only to Ductile Concrete Moment Resisting Frames. The decision to use a ductile concrete frame as opposed to a concrete shear wall design should be based on esthetics, flexibility of floor space, and economics which will be based on labor costs versus material costs.

The designer who wishes to use a ductile concrete frame should read at least the following publications:

A. ATC-03 "Tentative Provisions for the Development of Seismic regulations for Buildings." This makes for an excellent background for the determination of forces to be resisted and also includes a commentary which is very enlightening regarding seismic design.

B. ATC-11 "Seismic Resistance of Reinforced Concrete Shear Walls and Frame Joints: Implications of Recent Research for Design Engineers," the report referred to above. It also includes a copy of the ACI-ASCE Committee Report #352, a very helpful and descriptive document giving examples of the design of joints.

C. The 1982 New Zealand Code NZS 3101 which gives a clear description of requirements for the design of ductile concrete frames. This Code does not always agree with ATC-03 above, and the elightened designer may wish to investigate separate designs. New Zealand researchers have done much in this field and have contributed greatly to the present "State of the Art."

It should be pointed out that the "State of the Art" is by no means a precise one. In fact, some discrepancies between United States researchers and New Zealand researchers result in some rather wide gaps.

From ATC-11, we have: "the most important concerns for the design and detailing of reinforced concrete beam-column joints are:

1. To preserve the integrity of the joint so that the ultimate strength and deformation capacities of the connecting beams and columns can be developed.

2. To prevent excessive degradation of the joint stiffness under seismic loading by minimizing cracking of the joint concrete and the loss of bond between the concrete and longitudinal beam and column reinforcement.

3. To prevent brittle shear failure of the joint."

ATC-11 also includes tables showing the various results reached by using different Codes and differing researchers' criteria. A matrix of researchers and differing conditions shows a list of twenty-eight topics that were discussed. These topics are listed below:

1. Magnitude of column axial load.
2. Horizontal joint shear reinforcement.
3. Vertical joint shear reinforcement.
4. Joint confinement reinforcement.
5. Confinement by transverse beams.

6. Joint aspect ratio.
7. Relative absolute quantities of beam and bottom flexural reinforcement.
8. Anchorage, bond transfer and yield penetration.
9. Amount and distribution of column reinforcement.
10. Beam and column reinforcing bar diameters.
11. Aggregate interlock.
12. Dowel action.
13. Special joint devices.
14. Biaxial loading.
15. Cyclic loading and loading history.
16. Plastic hinge location.
17. Strut mechanism.
18. Strut splitting.
19. Truss mechanisms.
20. Repaired joints.
21. Prestressed joints.
22. Ratio of column to beam capacities.
23. Code requirements.
24. Concrete strength.
25. Joint stiffness.
26. Column size.
27. Lightweight concrete.
28. Joint shear strength.

Biaxial loading which is certain to occur was only discussed in one paper. (It is more difficult to prepare such a model.) Effect of time and creep which could transfer concrete load to reinforcing steel was not mentioned once! A few of the more important topics are discussed.

The forces acting on a joint are shown in Figure #13. The current popular methods for resisting these forces are:

1. Beam shear mechanism.
2. Joint truss mechanism.
3. Compression strut mechanism.

These concepts are shown in Figure #14. The beam shear mechanism is dependent upon Code allowable stresses. In 1983, Professors Park and Milburn of New Zealand compared the U.S.A. approach with the New Zealand approach. The paper pointed out a discrepancy of 5.6 times the shear allowed in a confined joint with plastic hinges at the face. This discrepancy was reduced to 2.4 times when the plastic hinge was moved away from the column face. In discussing this large discrepancy with some U.S. researchers, it was pointed out to me that New Zealand researchers use both higher ductility factors and cumulative ductility factors than U.S. researchers seem to think are warranted. The cumulative ductility factor generally utilized by New Zealand researchers is thirty-two (32) which, for example, would be a test model going through eight (8) complete cycles with a ductility factor of four (4) without the joint losing more than twenty percent (20%) of its load carrying capability.

The truss mechanism depends on vertical reinforcing to take vertical components of forces.

Confinement by Transverse Beams: In order to use the higher recommended shear stresses, it is required that beams frame into the column in both directions in order to confine the highly stressed area. The beam width must be at least three-fourths of the column dimension into which it frames and should be concentric. Eccentric framing should be carefully investigated.

Prestressed Joints: Although prestressing is not addressed in ATC-11, New Zealand researchers made some study along these lines. They prestressed the beams that framed into the beam-column assemblies they tested. Prestress wires were usually located at the neutral axis of the beam. The results showed that this prestress increased the capability of the joints to carry load. This result is in line with effect of magnitude of column load. In other words, the more confined the joint the better it performs.

Anchorage, Bond Transfer and Yield Penetration: This is one of the more important categories. As yielding begins, there is a tendency for an intrusion of the rebar yielding into the column area. This reduces the length available for bond transfer within the column depth. Thus arises the suggestion of moving the plastic hinge away from the column face. Bond transfer through the column and proper anchorage at exterior columns are an absolute necessity in order for the joint to work properly.

Column Size: It is entirely possible and quite probable that the column size may be dictated by reinforcing bar size rather than by stress. In order to develop bond, some U.S. researchers are suggesting that the column depth be at least twenty (20) bar diameters while New Zealand researchers are suggesting thirty-five (35). This requirement would suggest using small bars. However, due to the congestion of steel at these joints it is not possible. In fact, I strongly recommend that designers using ductile frames make full size sketches of the beam column joint in their offices to make certain that concrete can be placed in the joint without excessive difficulty. Smaller aggregate and higher slumps are suggested in these areas. Remember that higher cement ratios (the result of the suggestion) will cause increased shrinkage. Care must be taken that a balance is reached.

Plastic Hinge: Generally speaking, the plastic hinge is located at the face of the column. However, the problems created by bond slippage and yielding of bars within the column core area suggest moving the plastic hinge. The suggested location for the plastic hinge is a distance of about twenty inches from the face of the column. The elimination of the yielding of the reinforcing steel within the column area plus the lengthening of the anchorage area makes this a more logical location. The location of the hinge is determined by the manner in which the reinforcing steel is detailed.

Ratio of Column Capacity to Beam Capacity: In order to prevent collapse of the structure, the location of hinging must be located within the beam rather than the column. Research has determined that the optimum ratio of column capacity to beam capacity should be approximately 1.4.

Horizontal Joint Shear Reinforcement: Reinforcing for the horizontal shear in the beam column area is basically the same as that followed in beam shear areas. Research of the motion involved and the rapid deterioration of this area dictates that a high strength steel be utilized for this reinforcement. This reduces the motion and prevents excessive deformation. ATC-11 states "the steel should not have a flat yield plateau."

Magnitude of Column Axial Load: Research showed that the greater the load, the better was the performance of the joint. Yet ACI-ASCE Report 352 assumes no vertical loads on the columns. Therefore, some amount of conservatism is built into their design due to this fact.

Vertical Joint Shear Reinforcement: The joint truss mechanism requires vertical reinforcement to resist the vertical component. U.S. researchers feel that the column reinforcement suffices for this requirement and the "Amount and Distribution of Column Reinforcement" is important. Column reinforcement should not be consolidated into the four corners but should be evenly distributed throughout the four faces. A minimum of four bars per face is suggested. (See also anchorage requirements.) New Zealand researchers feel that additional vertical reinforcement may be necessary.

Some interesting comments are worth mentioning: An odd number of horizontal ties in the joint seems to provide a maximum amount of resistance since one tie is located at the center of the joint. Although the allowable shear stress (some codes) in confined joints is $20\sqrt{F'c}$ one researcher believes it should not exceed $13\sqrt{F'c}$. Interesting variances in Codes relating to the joint design are pointed out in ATC-11. The depth of the column is alternately referred to as the gross depth or the core depth. Because of the large amount of spalling that occurs during cyclic loading, it is suggested that the core depth be used as the column dimension.

Having completed the design of the ductile concrete moment resisting frame, the designer must still be aware of some architectural features. Are infilled walls to be used? Will interior partitions be constructed of weak masonry type of materials? In some countries interior partitions and exterior infilled walls are constructed of weak hollow clay units, weak concrete block units, or bricks. These weak materials have no in-plane strength but even worse when used as an infill wall within the concrete frame will attract the seismic forces because of their stiffness, which will not allow the frame to deflect as assumed in the design. Keep in mind forces will be resisted by elements of the structure in proportion to their rigidities.

When these infilled walls fail, they usually cause the boundary column to fail also. In addition, it can be demonstrated that in this type of failure each column tends to resist the entire shear rather than a portion of the shear depending on its configuration.

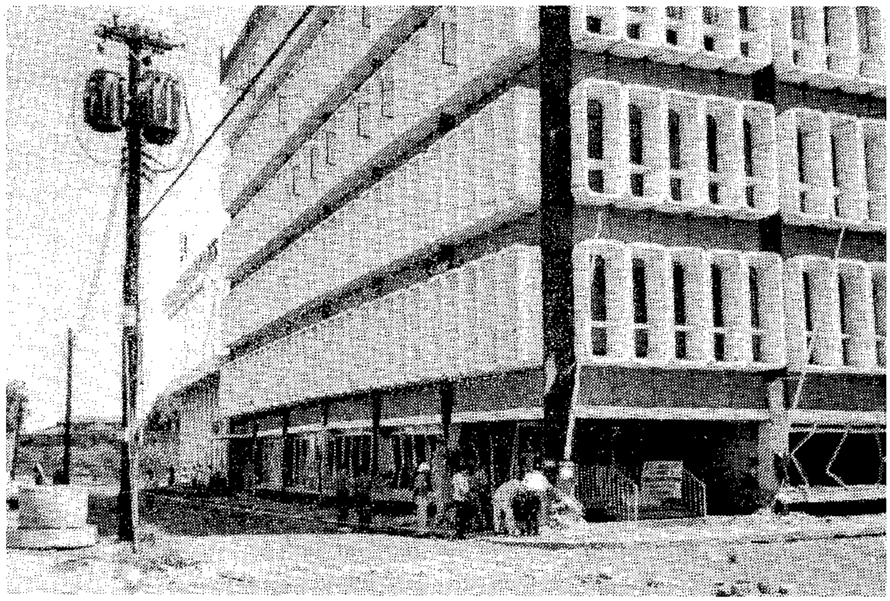
In Lioni, during the Camapnia-Basilicata, Italy earthquake of November 23, 1980, two structures were under construction as shown in Figure #15. Note that one unit is slightly ahead of the other, and that some infill walls have been completed. Figure #16 shows a close-up of the walls indicating that an infill portion has started to create a damaging effect on the structure. Note that the pure frame has had no problems.

The exterior walls of buildings utilizing the concrete frame should be reasonably flexible types, such as: Aluminum or steel cladding with windows, windows framed within the concrete frame and flexible panels, solid wall panels constructed of metal siding, sheet metal studs with exterior plastering or metal siding, etc. With some of these architectural materials, damage during seismic events will result and some repair work will be necessary. The structural system itself should be able to withstand moderate shakes with no damage, and severe shocks with repairable damage.

Incidentally, laboratory experiments have shown that repair of test specimens using epoxy as the restorative substance has resulted in the model being able to absorb almost as much energy as the original specimen.

Research work is still needed on this entire subject, and I know that some continuing work is presently being planned.

APPENDIX "A"



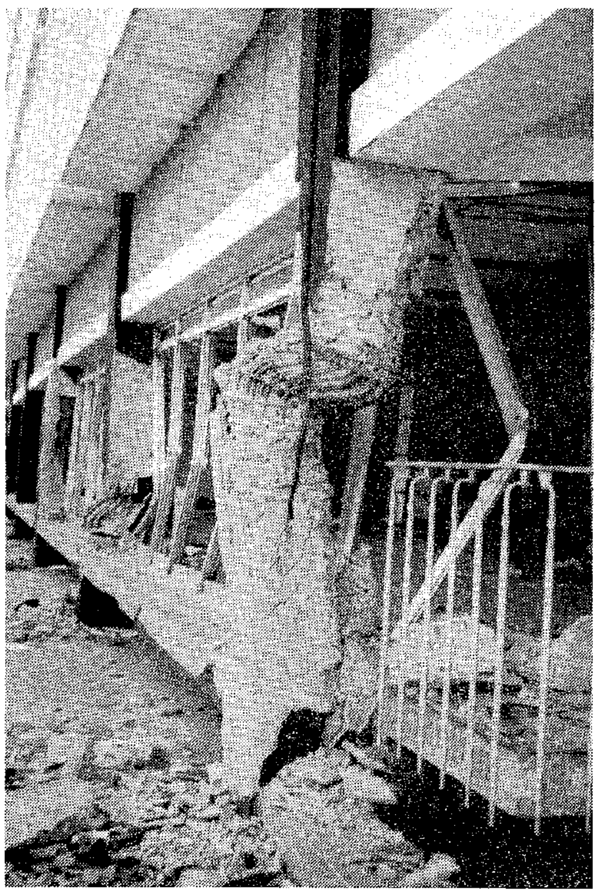


Figure #1

Figure #2

Figure #3

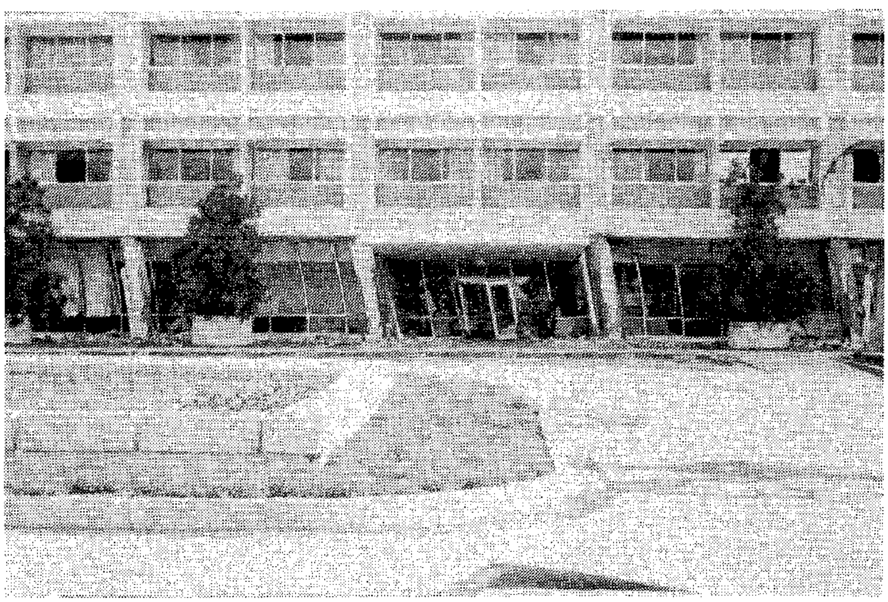


Figure #4

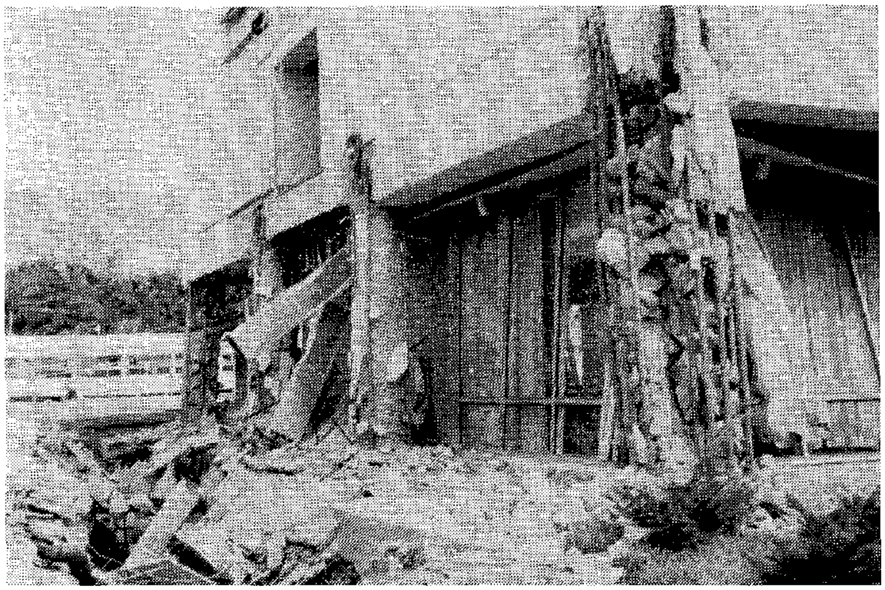


Figure #5

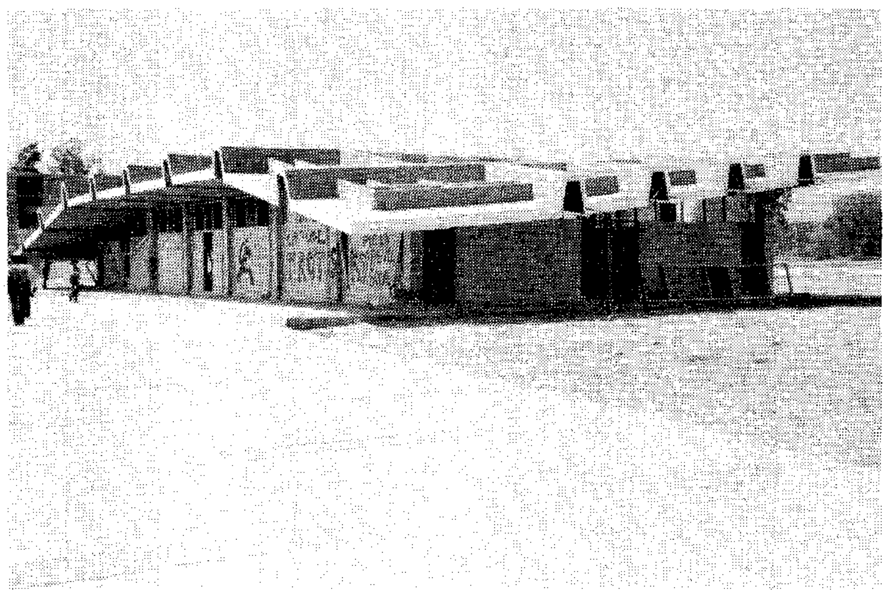


Figure #6

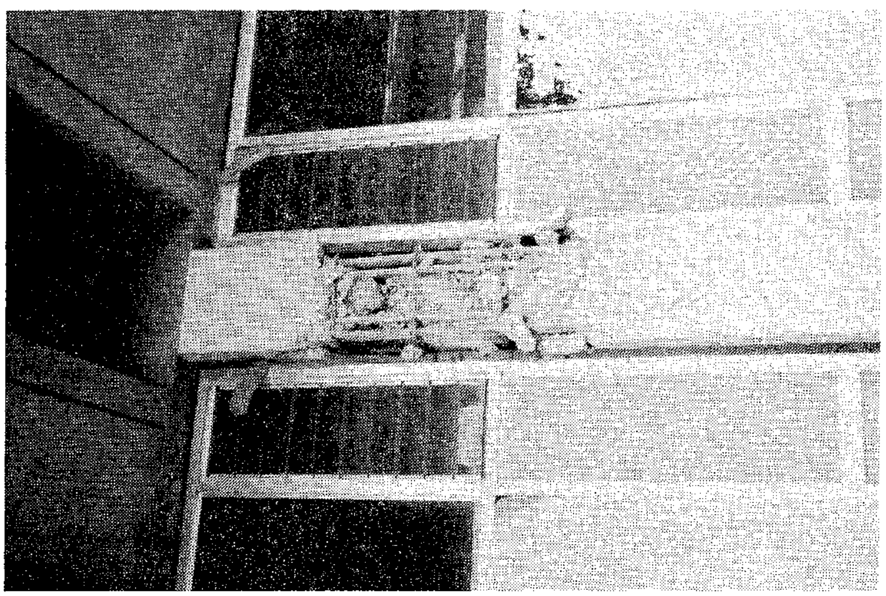


Figure #7

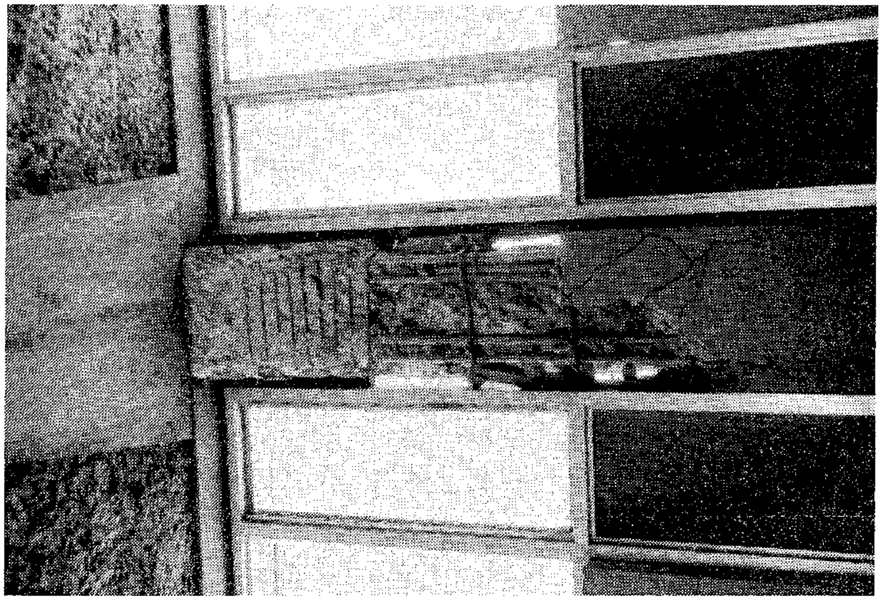


Figure #8

Figure #9

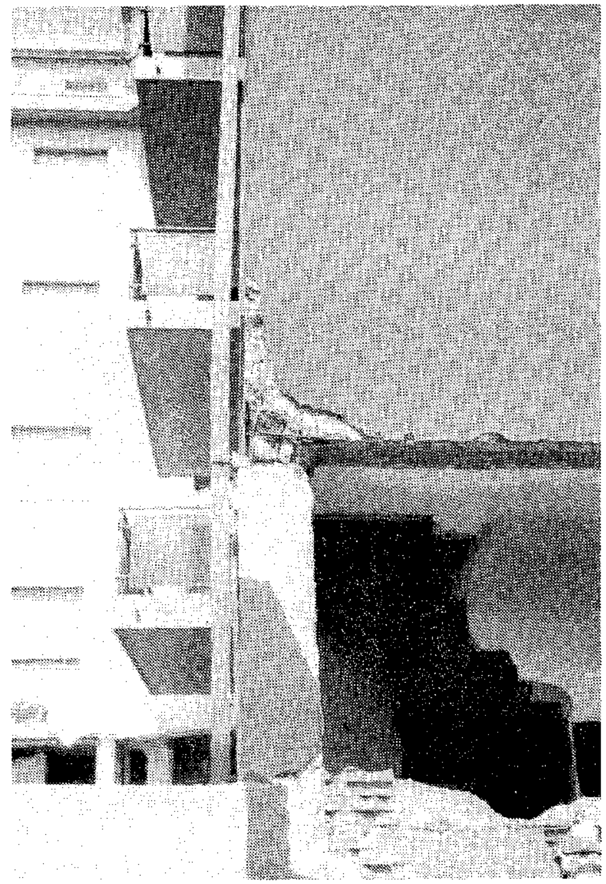


Figure #10

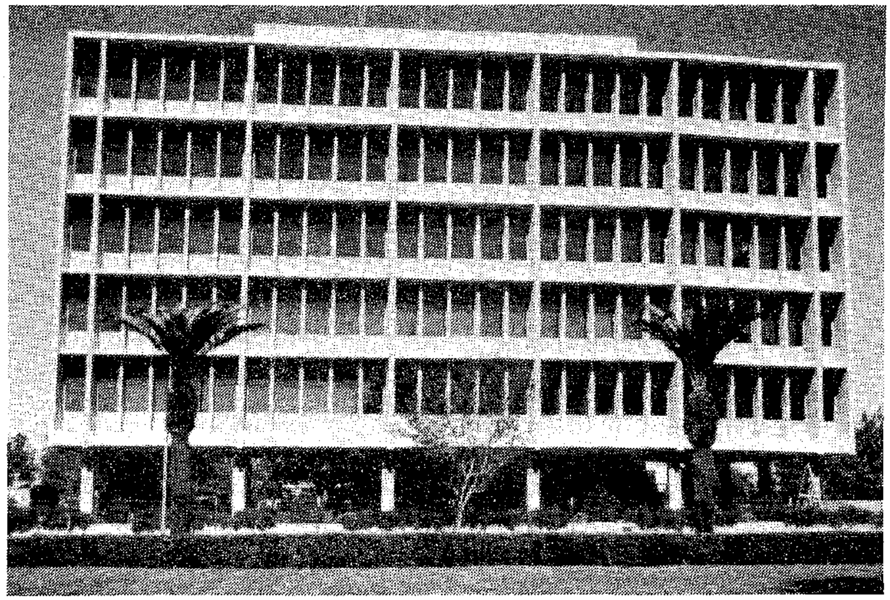


Figure #11

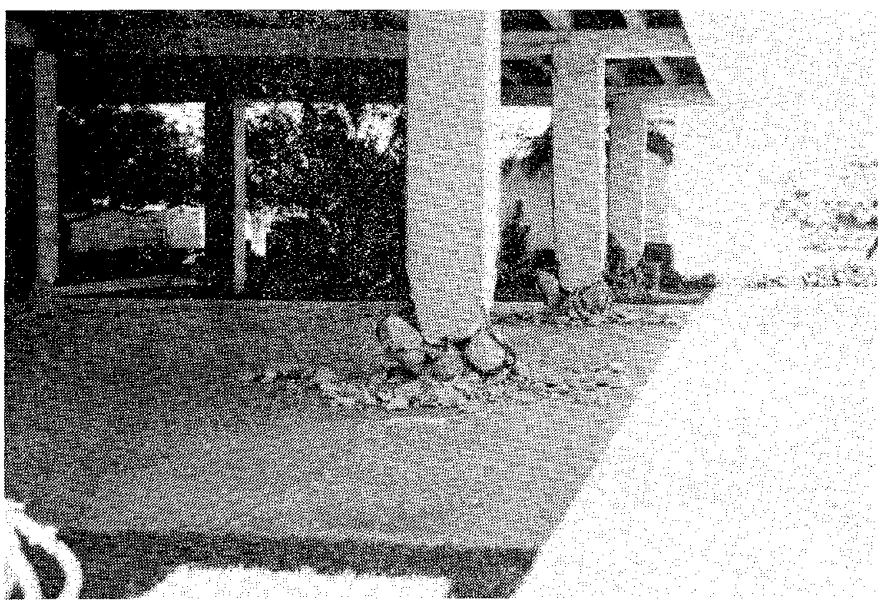


Figure #12

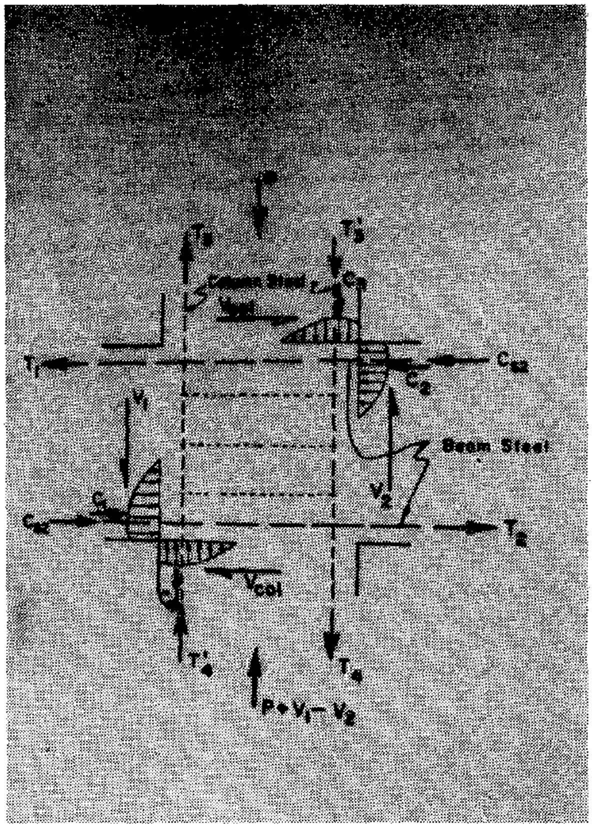


Figure #13

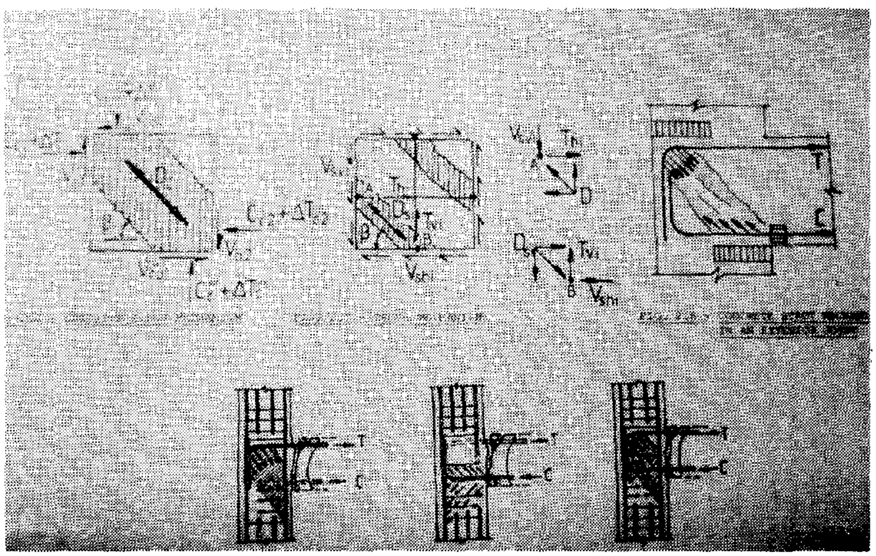


Figure #14

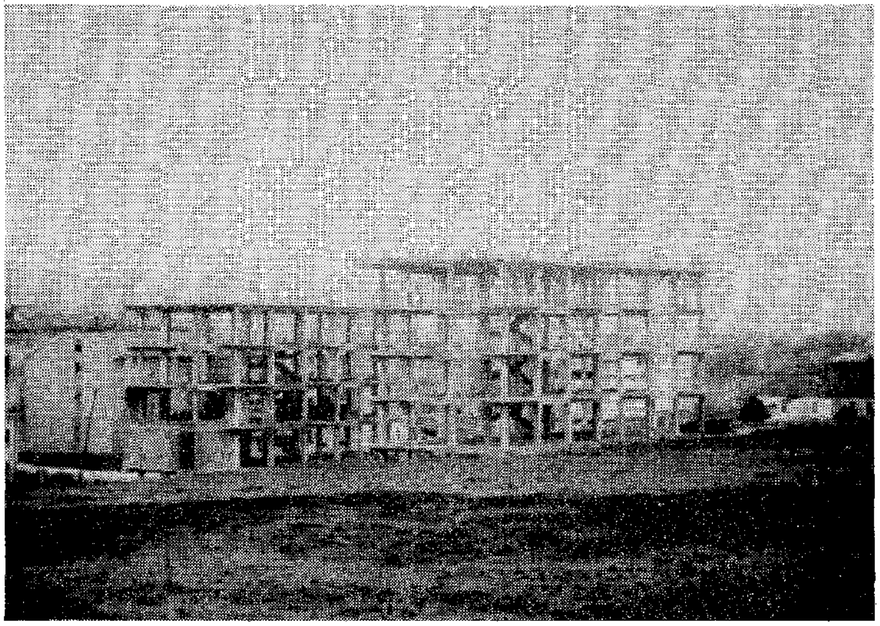


Figure #15

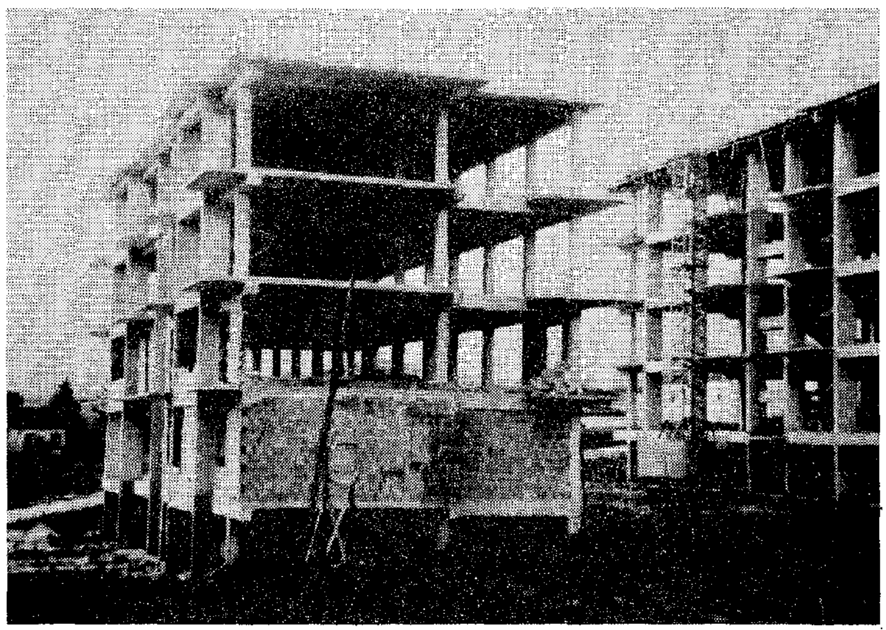


Figure #16

Tests on Specimen Walls in S. Gregorio Magno

A.E. Zingali (I)

SUMMARY

This paper presents a program of experimental research in progress in one town in the south of Italy which was strongly affected by the earthquake on November 23, 1980. This research has the objective of obtaining real data on the strength of the masonry used in that area, and on the effectiveness of low cost methods of strengthening. The tests are carried out on real masonry that is slated to be demolished, by means of movable testing equipment. The paper reports on the program of tests, the method used in order to conduct them, and the equipment designed specifically for this purpose.

INTRODUCTION

The problem of rebuilding areas struck by a strong earthquake is closely linked to that of repairing damaged buildings and of strenghening all existing ones. The built up areas in Italy, especially the smaller towns, consist mainly of old stone masonry buildings of small size. These, though they do not represent important economic or artistic values when considered individually, constitute because of their large number an important part of the Italian building patrimony and, moreover, are the urban tissue of the historic centers of the towns.

For this reason, considering that these buildings are part of Italian culture, one attempts to avoid any wrenching of their architectural character, preferring instead to strengthen or re-build them, when possible, using the same materials as before. Much research is therefore being conducted in Italy which has the purpose of perfecting low-cost strengthening methods, as well as determining the values of the parameters peculiar to the strength of stone masonry, reinforced or not.

PURPOSE OF THE RESEARCH

After the earthquake in November, 1980, the Italian rules appeared giving the values, for want of direct tests, of the masonry strength to be taken into account. These values are rather safe owing to their generality.(1)

The program of the experimental tests planned in the town of S. Gregorio Magno, a little town hit strongly by the earthquake in 1980, has the main objective of obtaining information about the strength of masonry and about those parameters, like longitudinal or transversal elasticity modulus and ductility factor, which have to be taken into

(I) Assoc. Professor of Structural Engineering, University of Rome

account in numerical analyses of masonry structures. Also the decreasing of the strength due to alternate repeated loadings is checked.

The values obtained for the original masonries are compared with the ones for the same masonries strengthened by means of the following methods:

- injections with cement mortar
- concrete plaster with steel wires
- diagonal steel bars.

The total planned number of tests is approximately 30.

The technical bibliography is rather lacking for such information on stone masonry because the main part of the research to date is devoted to brick masonry. Tests analogous to those presented herein were made by Benedetti and Casella (2), who built in the laboratory some stone masonry walls with stones of the Valnerina (a place struck by the earthquake in 1979). The walls were stressed until collapse, then repaired, and then stressed again until collapse.

The main distinctive feature of the present tests is that they are carried out on existing masonry obtained from damaged buildings which are destined to be demolished. This is made possible by designing movable equipment, described below, which enables one to stress the specimen with horizontal forces of opposite directions, as well as with vertical loads. Because the tests are just beginning at this time, and there are therefore no results yet to report, we will limit ourself to explaining the general problem, the program of tests, and the equipment.

GENERAL CONSIDERATIONS

A masonry building of small size (one or two floors), when subjected to an earthquake, is mainly stressed by shear forces acting in the plane of the walls. In most cases, if the structure is well linked by the floors, and the perpendicular walls are well connected each to the other, the collapse happens in the masonry panels included between the openings (windows or doors), except when the masonry above the openings is very weak.

This is well known, so that this kind of behavior was included in the Italian rules, and the instructions of the Public Works Ministry refer to it.(3)

The basic value for defining the ultimate strength of a masonry panel (shear collapse) is indicated by τ_K and represents the mean shear stress on the cross section through the center point of the panel when no vertical load is applied. If we suppose that the shear stress distribution is a parabolic one, by indicating with σ_n the normal tensile strength of the masonry, we have at shear collapse:

$$\sigma_n = 1.5\ \tau_K$$

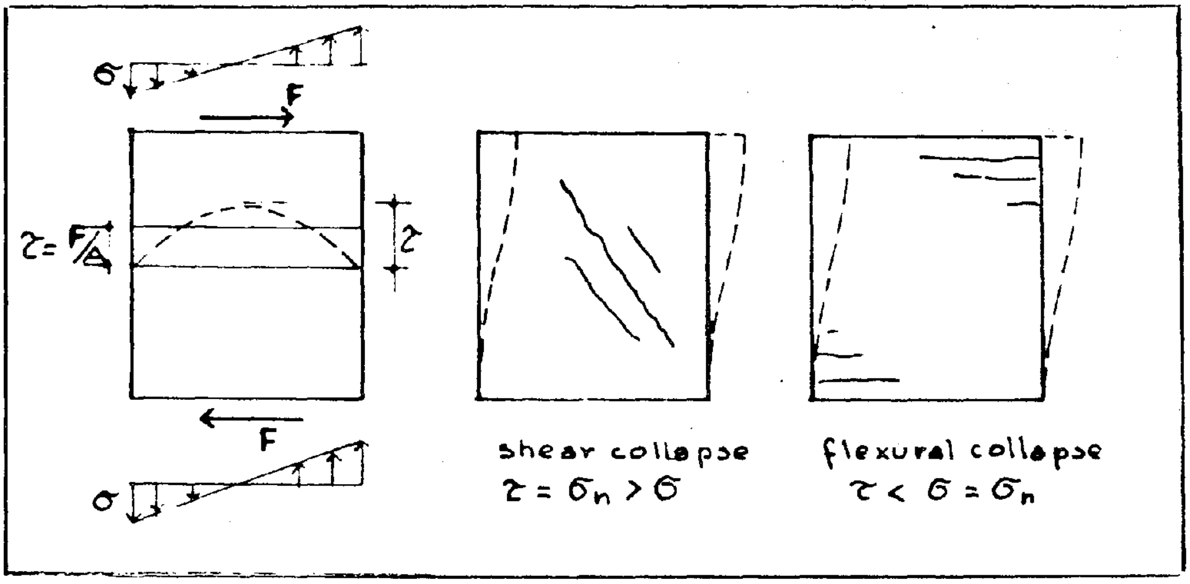


Fig.1

But is has to be noticed that, in order to obtain the diagonal crack, the maximum normal traction stress σ on the bases, due to the moment, has to be less than σ_n, otherwise the collapse happens in a flexural way. Supposing a linear distribution of the normal stress on the bases, the diagonal crack would be possible only if the ratio height/width is less or equal to 0.5, which is very small. As a matter of fact, the case of total absence of vertical load is of no practical interest because of the great weight of the stone masonry.

If we now consider the case of the panel loaded by vertical load, the principal tensile stress in the center point of the panel is

$$\sigma_1 = \frac{\sigma_0}{2} + \sqrt{\left(\frac{\sigma_0}{2}\right)^2 + 1.5\tau_m}$$

where σ_0 and τ_m are the mean normal and tangential stresses on the cross section through the center point of the panel. By equating this value with the value of the tensile strength, we obtain the expression of the collapse value of τ_m, indicated by τ_u:

$$\tau_u = \tau_K \sqrt{1 + \frac{\sigma_0}{1.5\tau_K}}$$

which is the formula given by Turnsek and Cacovic in (4).

The ultimate horizontal load is then:

$$F_u = \tau_u A$$

where A is the area of the cross section. The term with the square root in the expression of τ_u represents the increasing of the shear strength due to the vertical load effect.

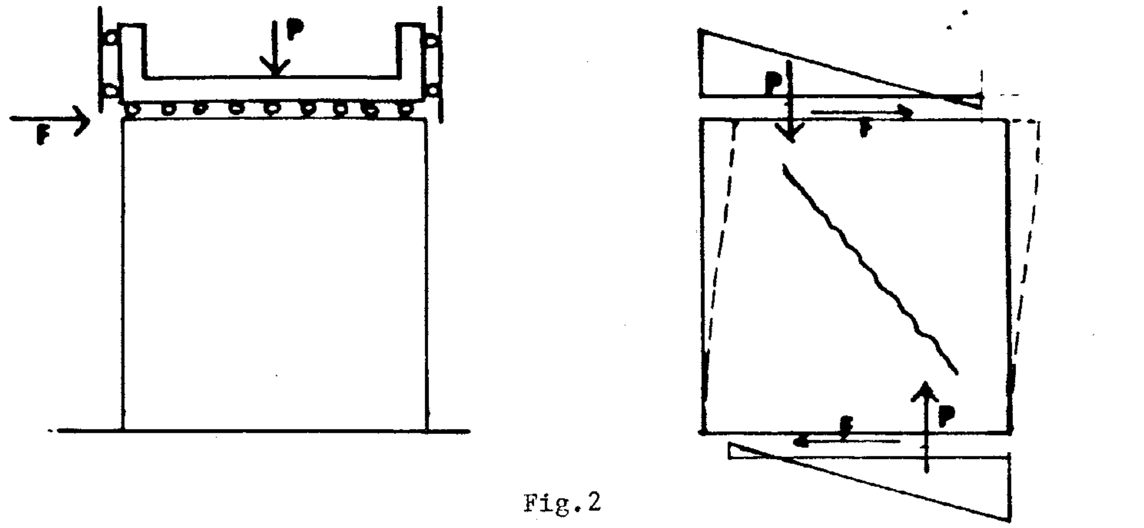


Fig.2

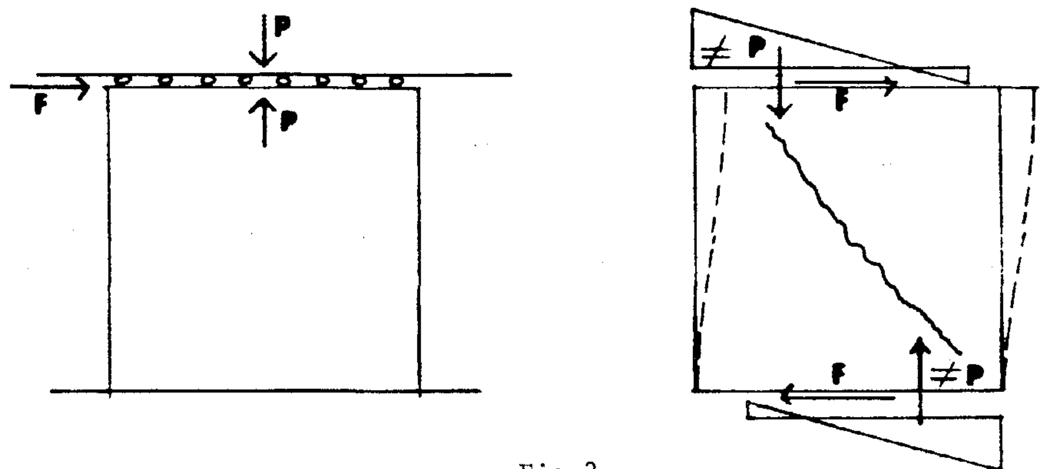


Fig.3

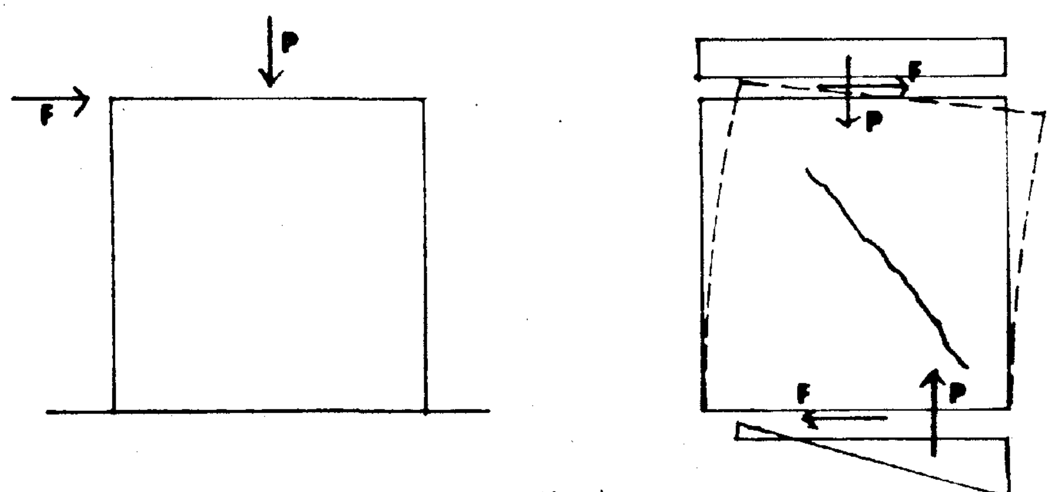


Fig.4

An important role in the behavior of the panel is played by the restraints at both bases. Referring to Figure 2, the vertical imposed load is constant until collapse. The vertical resultants of external forces move towards the ends of the bases remaining of the same value, and the collapse happens when the principal tension stress in the center of the panel reaches the tensile strength, or when the compression stress at the end of the bases reaches the crushing value. Because the tensile strength of stone masonry is much less than the crushing one, the collapse generally happens with the diagonal crack. If the vertical load is put equal to zero, and also the tensile strength is equal to zero, no horizontal force F can be applied.

In the schema of Figure 3, the vertical load is obtained by means of a previous constraint of the panel. In this case, the vertical forces can change during the load process because the system is a hyperstatic one and any vertical internal force is in equilibrium with the reactions on the bases. If no vertical force is applied and the tensile strength is equal to zero, the collapse force F is no longer equal to zero because eccentric axial forces rise on the bases to equilibrate the moment due to F.

So far only the case in which both bases of the panel are restrained has been examined. Another schema which can be considered is the panel built up on the lower base only (Figure 4.). In such a case the behavior is of a short cantilever, and the ratio height/width, in order to obtain the shear collapse, is smaller than in the case of upper base restraint.

Let us examine now the real boundary conditions of the panel when it is inside the wall of the building. If we refer to Figure 5a, it can be said that the behavior is similar to that of the panel with restraint bases and constant vertical load. But it has to be noticed that some change of the vertical load on the panels is due to shear forces of the

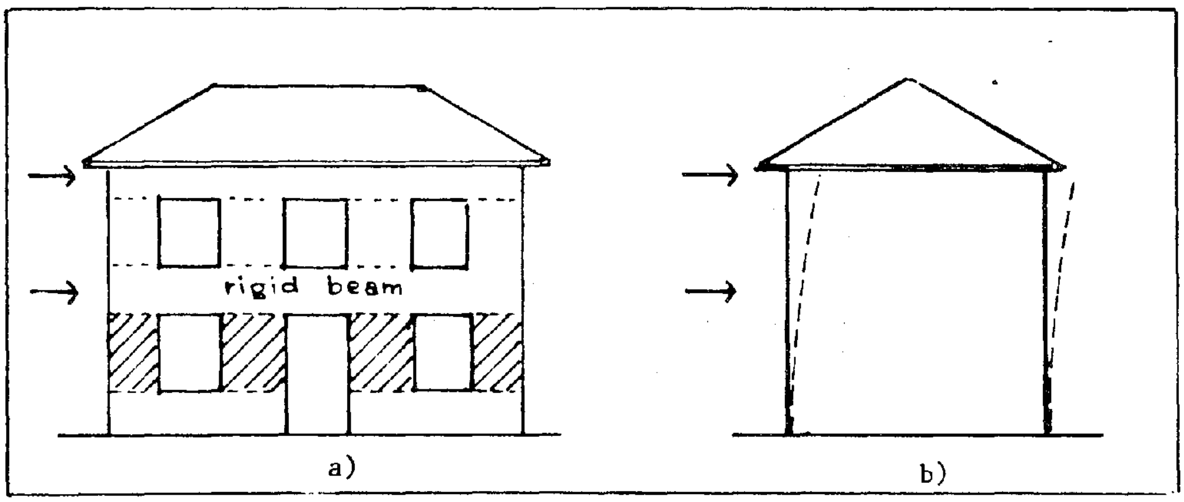
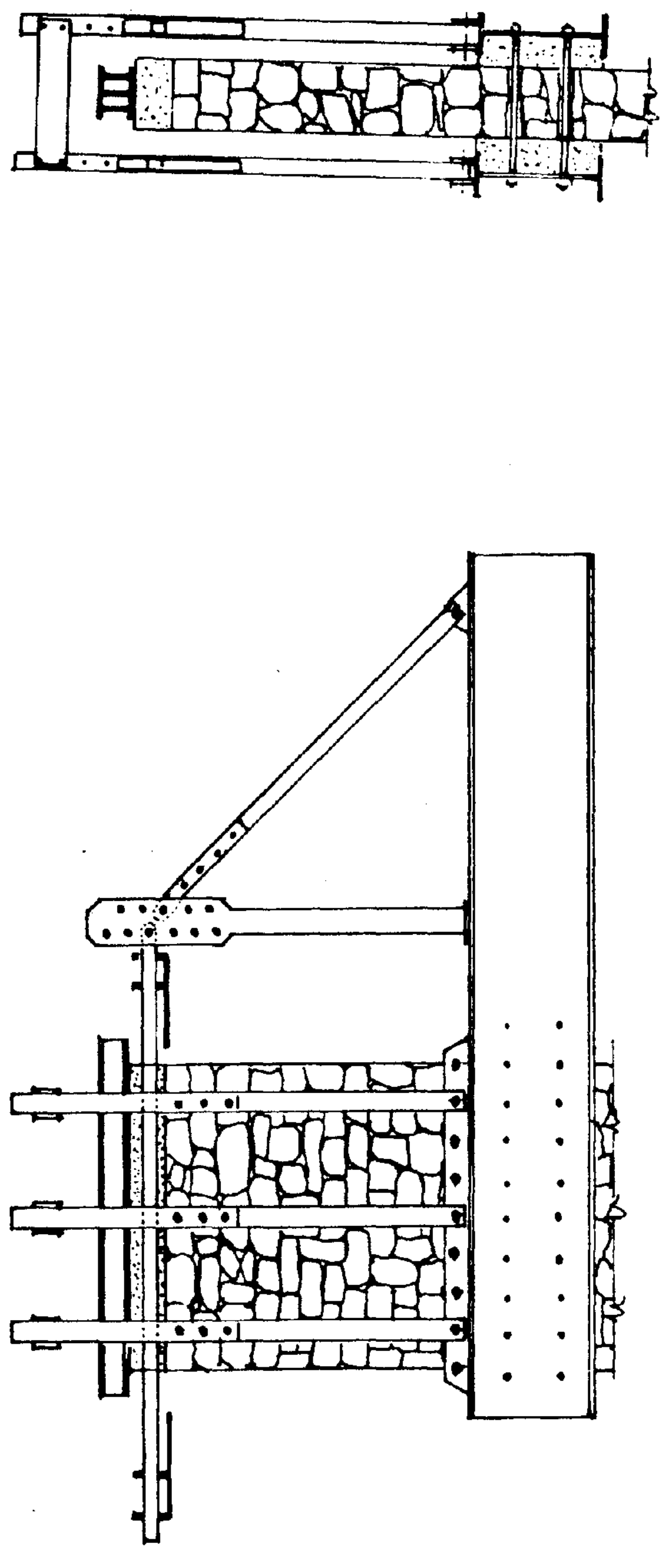


Fig.5



TESTING EQUIPMENT

Fig.6

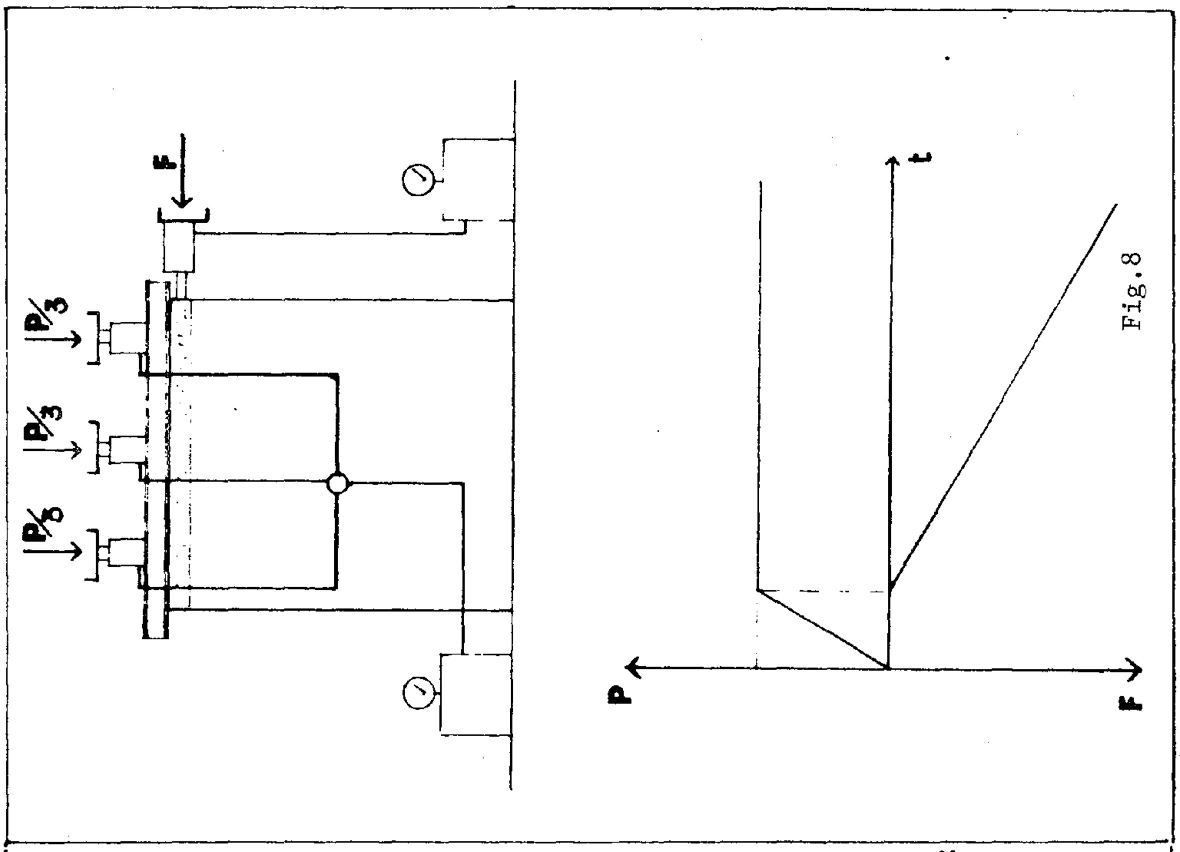


Fig.8

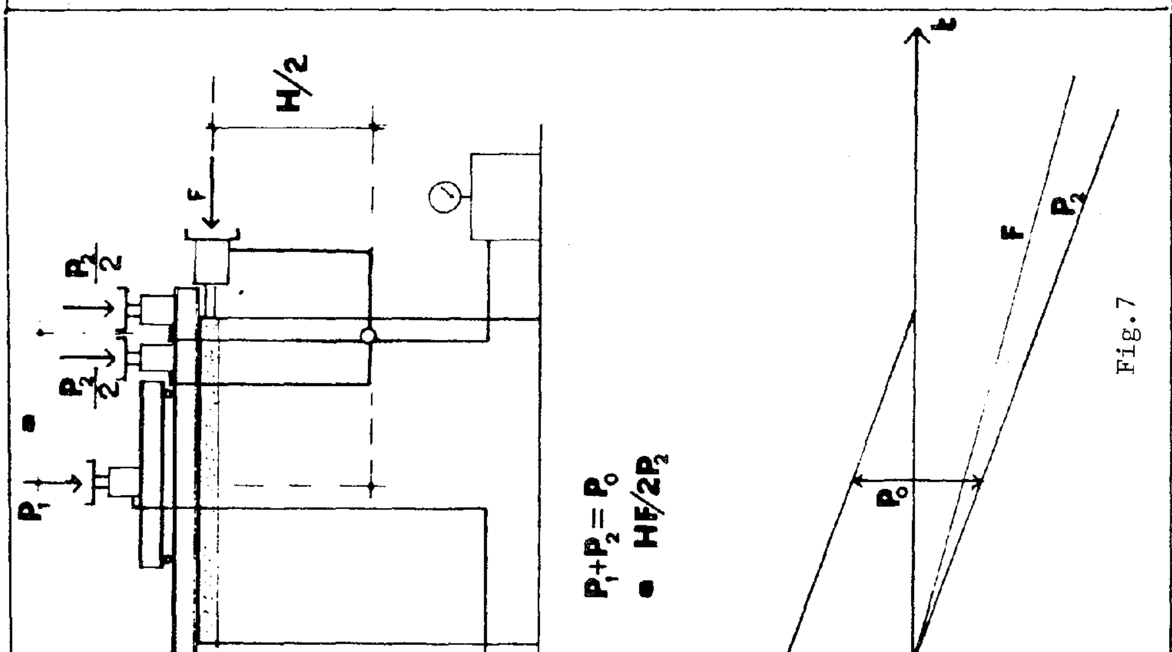


Fig.7

rigid beam above the openings, so that those on the right side of the figure are increased while those on the left side are decreased.

In the case of Figure 5b, which represents the lateral wall, without openings, of the same building, the panel is constituted by the whole wall and the behavior is that of a cantilever.

Besides the strength values, another parameter of interest is the ductility factor. At present, this is indicated in Italian rules as being equal to 1.5. As is well known, the ductility factor is important in order to distribute the seismic actions among the walls when the ultimate strength is reached in one or several of them.

DESCRIPTION OF THE EQUIPMENT

Because one of the main purposes of the testing program is to examine the existing masonry, an essential need is testing equipment that is easily assembled and disassembled. Another factor to be taken into account is the need to employ this equipment in many different conditions in the area surrounding each specimen. Accordingly, a self-equilibrated system was chosen; that is, the contrast equipment is rigidly joined to the panel. In this way it is possible to eliminate any external anchorage which would require heavy counterbalances or traction resistant foundations.

The structure is represented in Figure 6. Two steel beams, 80cm high, are placed on the base, each one for each side of the wall to be tested, which is cut beforehand to the proper size. These two beams are linked, each to the other, by means of bolted bars through holes made in the masonry. They do not stick to the wall, and concrete is cast through the free space between the beams and the wall. Before casting the internal surfaces of the beams are greased in order to disassemble the structure easily after the test is executed.

The upper sides of the beams are provided with two series of anchorages for three couples of vertical tie-rods, so that it is possible to insert three vertical jacks in any place on the upper side of the panel.

The horizontal force is applied by inserting one horizontal jack between the concrete beam on the upper side of the panel and a steel truss fixed on the steel beams at the lower base. The jack can act directly on the truss (right side of Figure 6) or by means of steel braces (left side of Figure 6). This means that the rods of the truss are stressed both in traction and compression, and therefore tubular rods, with square cross section, were employed. This solution has the advantage, furthermore, of allowing an easy adjustment of the length of the bars by means of a telescope-type device. This is in order to adjust the equipment to the size of the panel, which can vary from one case to the next.

The vertical jacks are 30 tons each, and the horizontal one is 50 tons. In Figure 7, one can see how the four jacks are connected to the two pumps in order to simulate the case of the panel with the upper base restraint while the vertical load is constant. In this case, one of the jacks gives the vertical load P_0 at the beginning of the test. The two other vertical jacks are connected in parallel with the horizontal one, which means that the vertical force, P_2, and the horizontal one, F, are in a constant ratio. The distance, a, is then chosen in such a way that the resultant of P_2 and F passes through the center point of the panel. At each step of the loading process, both F and P_2 are increased while P_1 is decreased so that the sum $P_1+P=P_0$ remains constant. In this way the vertical force P_0 moves from the center of the upper base towards the right side, while the F force increases and the resultant passes always through the center point of the panel.

The second schema of Figure 8 shows how the jacks are connected when no restraint is on the upper side of the panel and the vertical load is constant. The three vertical jacks are connected in parallel so that a uniform stress is applied.

One other problem to resolve was the cutting of the walls in order to obtain specimens of suitable size. The use of a stone chisel was excluded because the masonry of the panel would have been damaged; excluded as well was the use of a disk-saw because of the thickness of the walls.

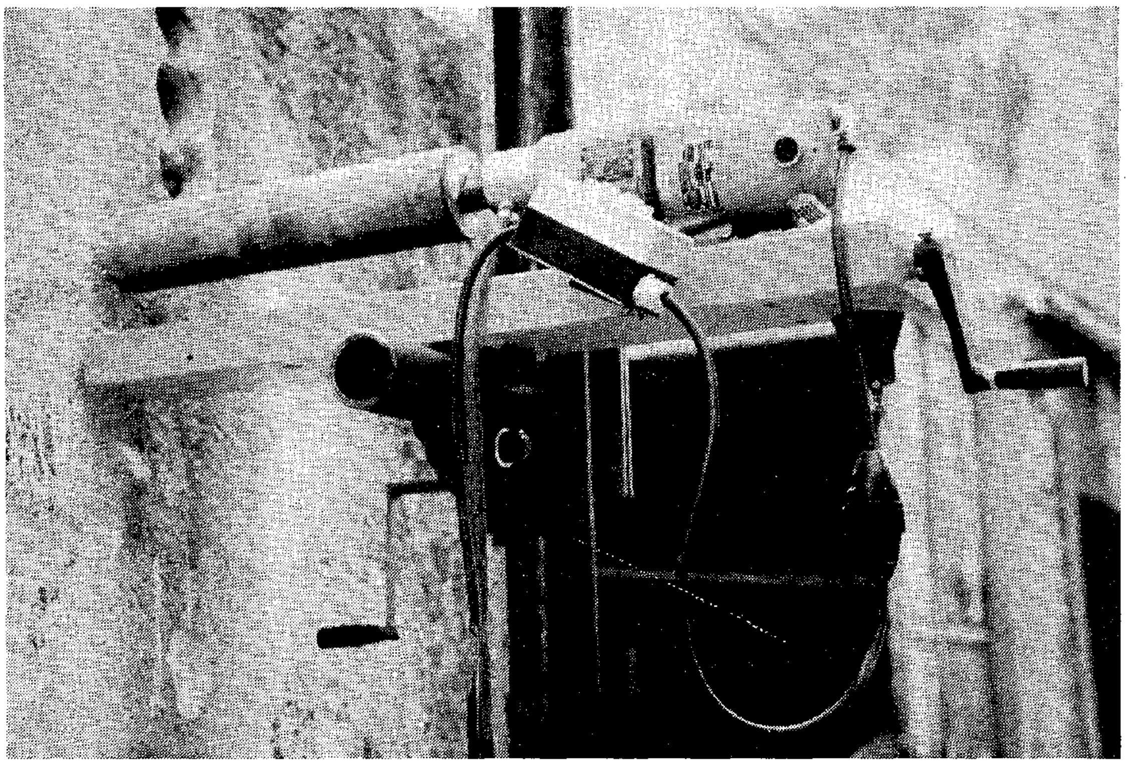


Fig.9

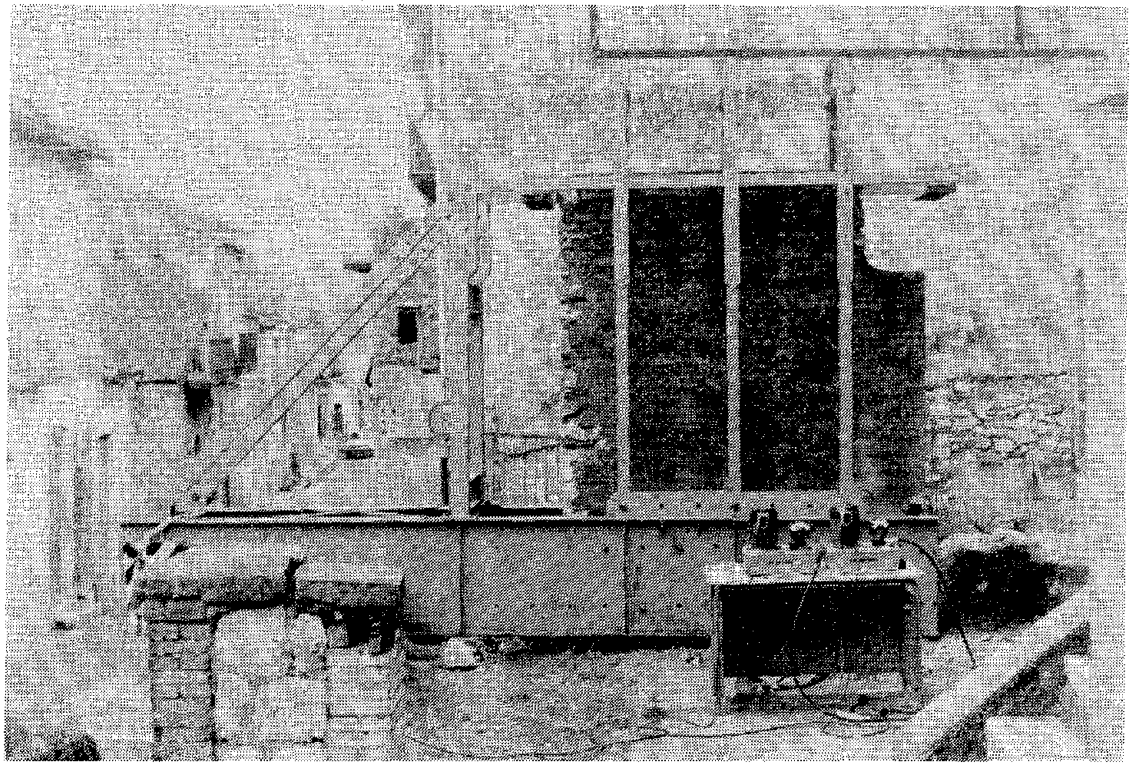


Fig.10

The solution found consisted of using a horizontal tubular drill, 10cm in diameter. By making a series of holes, one next to the other, the wall is easily cut. This cutting equipment is shown in Figure 9, and Figure 10 shows the testing equipment assembled on a specimen before

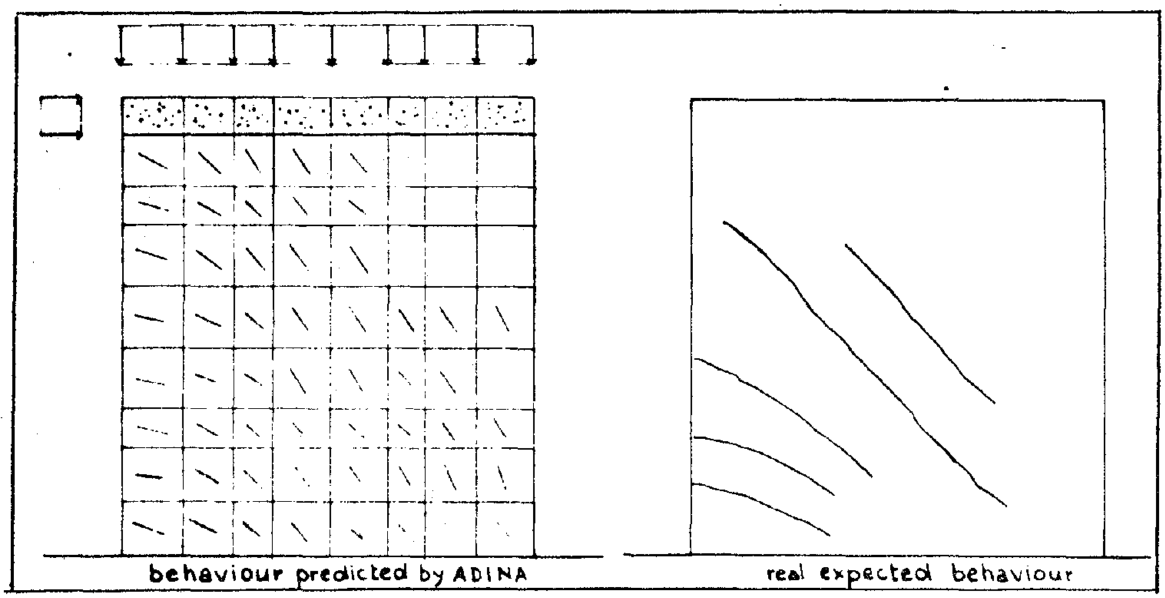


Fig.11

testing.

The behavior of the panel is compared with that resulting from a theoretical analysis performed using the finite element method. In Figure 11, an example of the results of such an analysis is shown. The non-linear program used is the ADINA (4). The mesh is formed by 72 rectangular eight joints elements. The distribution of cracks is represented here. The purpose of this comparison is to check the validity of the method for predicting the behavior of the masonry.

REFERENCES

(1) Istruzioni per l'applicazione della "Normativa tecnica per la riparazione e il rafforzamento degli edifici danneggiati dal sisma nelle regioni Basilicata, Campania a Puglie (Decreto Ministeriale 2 luglio 1981)

(2) Benedetti D. - Casella M.L. "Il comportamento di murature a sacco soggette a forze taglianti-Confronto tra diversi metodi di consolidamento" - L'Industria Italiana delle Costruzioni - Jan. 1982

(3) Turnsek V. - Cacovic F. "Some Experimental Results on the Strength of Brick Masonry Walls", Proc. of 2nd I.B.M.A.C., Stoke-on-Trent - 1960

(4) Bathe K.J. "Static and Dynamic Geometric and Material Non Linear Analysis Using ADINA", Report 82448, M.I.T. - 1976

Roma May 1984

This research is performed with the collaboration of M. Aquilino, F. Piccarreta and A. Samuelli-Ferreti, all of them of the University of Rome.

INTENTIONALLY BLANK

APPENDICES

INTENTIONALLY BLANK

SCHEDULE

JOINT USA/ITALY WORKSHOP
ON
SEISMIC REPAIR AND RETROFIT OF EXISTING BUILDINGS

<u>MONDAY, MAY 7th.</u>

5:30 - 6:30 PM	<u>Briefing Session, U.S. Participants</u>

<u>TUESDAY, MAY 8th.</u>

8:00 - 9:00 AM	<u>Registration</u>
9:00 AM	<u>Opening Addresses and Organization of Workshop</u> C. Gavarini, Istituto di Scienza delle Costruzioni, University of Rome H.J. Lagorio, Center for Environmental Design Research, University of California, Berkeley J.B. Scalzi, National Science Foundation, Washington, D.C.
10:00 - 1:00 PM	<u>Technical Session I</u> Presentation of Technical Papers Italian Participants
2:00 - 5:00 PM	<u>Technical Session II</u> Presentation of Technical Papers U.S. Participants
5:00 PM	<u>Discussion Session</u>

<u>WEDNESDAY, MAY 9th.</u>

9:00 - 11:30 AM	<u>Technical Session III</u> Presentation of Technical Papers Italian Participants
11:30 AM	<u>Discussion Session</u>
2:00 - 4:30 PM	<u>Technical Session IV</u>

	Presentation of Technical Papers U.S. Participants
4:30 PM	<u>Discussion Session</u>
5:30	<u>Adjournment of Technical Sessions</u>

TECHNICAL FIELD STUDY AND SITE WORK

1980 IRPINIA EARTHQUAKE AREA

<u>THURSDAY, MAY 10th.</u>

7:00 AM	Leave Rome by car to Lioni.
12:00 PM	Arrival in Lioni (C.R.E.S.M.) Visit r/c specimens partially tested. Site visit in Lioni.
2:00 PM	Welcoming by Vice-Mayor of Lioni. Lunch.
3.30 PM	Technical information on the reconstruction plan of S. Angelo dei Lombardi (Arch. Nora Scire and Partners - Sovraintendenza ai Beni Artistici e Ambientali di Avellino e Salerno).
5:00 PM	Site visit in S. Angelo dei Lombardi.
6:30 PM	Leave S. Angelo dei Lombardi.
8:30 PM	Arrival in S. Gregorio Magno. Dinner.

<u>FRIDAY, MAY 11th.</u>

8:30 AM	Welcoming by Mayor of S. Gregorio Magno. Site visit in S. Gregorio Magno. Field test on masonry wall, performed by Profs. F. Piccarreta, A. Zingali, A. Samuelli-Ferretti.
1:00 PM	Lunch.
3:00 PM	Technical information on repair/retrofitting/analysis of masonry buildings by Prof. Sparacio and partners (University of Naples).

4:30 PM	Final discussion. Leave S. Gregorio Magno.
9:00 PM	Arrival in Rome.

INTENTIONALLY BLANK

WORKSHOP PARTICIPANTS

Dr. Paolo Angeletti	Institute of Construction Sciences, University of Rome
Prof. Giuliano Augusti	Department of Civil Engineering, University of Firenze
Prof. Duilio Benedetti	Politecnico di Milano
Dr. Carlo Bientinesi	Chiaromondo Engineering, Terni
Prof. Franco Braga	Institute of Construction Sciences, University of Rome
Prof. Russell Brown	Chair, Department of Civil Engineering, Clemson University, Clemson, S.C.
Prof. Remo Calzona	Institute of Construction Sciences, University of Rome
Dr. Giuseppe Capponi	Studio di Progettazione Cherubini, Rome
Dr. Alberto Cherubini	Institute of Construction Sciences, University of Rome
Dr. Marcello Ciampoli	Institute of Construction Sciences, University of Rome
Mr. Gianni Clemente	Institute of Construction Sciences, University of Rome
Prof. Giorgio Croci	Institute of Construction Sciences, University of Rome
Dr. A. D'Amato	SVEI S.p.A., Rome
Dr. Renato De Angelis	ENEA, Rome
Dr. Roberto Del Tosto	President, Englekirk, Hart and Del Tosto Consulting Engineers, Inc., Rome
Dr. Angela Di Benedetto	Studio di Progettazione Cherubini, Rome
Dr. Mauro Dolce	Institute of Construction Sciences, University of Rome

Preceding page blank

Prof. R.D. Ewing	Vice President, Agbabian Associates, El Segundo, CA
Dr. A. Favilli	Institute of Construction Sciences, University of Pisa
Prof. Carlo Gavarini	Director, Institute of Construction Sciences, University of Rome
Prof. Carlo Gravina	Institute of Construction Sciences, University of Rome
Prof. Giuseppe Grandori	Politecnico di Milano
Prof. Gary C. Hart	Department of Civil Engineering, University of California, Los Angeles, and Partner, Englekirk and Hart Consulting Engineers, Inc., Los Angeles, CA
Prof. Henry J. Lagorio	Director, Center for Environmental Design Research, College of Environmental Design, University of California, Berkeley
Dr. Giorgio Lilli	Studio di Progettazione Cherubini, Rome
Prof. James L. Noland	Consulting Engineer and Principal, Atkinson-Noland & Assocs., Inc., Boulder, CO
Dr. Salvatore Perno	Institute of Construction Sciences, University of Rome
Prof. Vincenzo Petrini	Politecnico di Milano
Prof. Joseph Plecnik	Department of Civil Engineering, North Carolina State University, Raleigh, NC
Prof. Roberto Ramasco	Faculty of Engineering, University of Naples
Prof. Alessandro Samuelli-Ferretti	Institute of Construction Sciences, University of Rome
Dr. John B. Scalzi	Program Manager, Earthquake Hazards Mitigation Program, National Science Foundation, Washington, D.C.

Dr. James L. Stratta	Consulting Structural Engineer, Menlo Park, CA
Dr. Franco Versaschi	Studio di Progettazione Cherubini, Rome
Prof. Antonio Zingali	Institute of Construction Sciences, University of Rome