PB81-106106

# Proceedings

Conference on

# Research in Progress on Masonry Construction

March 1980 Marina Del Rey, CA


Editors and Conference Co-Chairmen:
James L. Noland
James E. Amrhein

# PROCEEDINGS

CONFERENCE

ON

# RESEARCH IN PROGRESS ON MASONRY CONSTRUCTION

March 1980

Marina Del Rey, CA

**Editors and Conference Co-Chairmen: James L. Noland**
**James E. Amrhein**



**SUPPORTED BY NATIONAL SCIENCE FOUNDATION GRANT NO. PFR 79-26689**

The Conference on Research in Progress in Masonry Construction was supported by the National Science Foundation under Grant No. PFR 79-26689.



Printed in the United States of America
1980

## Acknowledgements

The organizers of the Conference on Research in Progress on Masonry Construction wish to thank John B. Scalzi, Program Manager, National Science Foundation, for his interest, support and participation. His clear-sightedness and continual interest have helped make masonry research one of the most exciting areas being studied today. A large part of the developing sense of community among masonry researchers is due to his leadership and encouragement.

An important but frequently overlooked aspect of any conference is the visual one. Thanks are due to Agbabian Associates and The Masonry Institute of America for supplying audio-visual equipment and to Donald Wakefield, of Interstate Brick Company, for serving as projectionist.

The researchers whose work is represented in this volume are, of course, the sina qua non. Sharing projects which are not yet fully realized is at best risky, and the willingness of these professionals to participate in this conference is greatly appreciated. The responders also made possible the open, problem-solving atmosphere and their preparation and participation is appreciated. This conference required much more than the usual level of involvement from attendees, and the vigorous participation and commitment to the advancement of masonry research is gratefully acknowledged.

A portion of the conference was devoted to problem-solving (or proposing) working groups. The chairmen of these groups deserve a special thanks both for their leadership during the conference and for the preparation of the working group reports which close this volume.

The invaluable assistance and guidance given by Theresa Noland is gratefully acknowledged. Her expertise and ideas were extremely effective towards stimulating discussion and interaction among participants as well as towards efficient logistical arrangements.

Two reports are included herein which were not presented orally because of schedule or other conflicts. The interest and efforts of those authors is also appreciated.

# Preface

The Conference on Research in Progress on Masonry Construction was organized to
1) present an overview of masonry research in progress and recently completed;
2) allow for in-depth presentation and discussion of masonry research in progress or recently completed; and
3) help establish a sense of community among masonry researchers.

The more formal results of that conference are presented in this volume. The larger purpose, that of establishing a sense of community, seemed to be well underway at Marina Del Rey.

# Table of Contents

**Working Group Titles and Chairmen** **Report Number**

# SHAKING TABLE STUDY OF SINGLE-STORY MASONRY HOUSES

By Clough, R.W., Mayes, R.L., and Gülkan, P.

ABSTRACT: Earthquake damage to masonry construction during this century underscores the need for a better understanding of the seismic response of these structures, and the establishment of rational reinforcement requirements. An experimental investigation aimed at determining reinforcement requirements for single-story masonry dwellings in Uniform Building Code Seismic Zone 2 areas of the United States has been in progress for four years at the University of California, Berkeley. The experimental results of the investigation obtained to date have been presented in a series of three Earthquake Engineering Research Center reports. This paper contains summaries of the tests as well as tentative recommendations for reinforcement requirements for masonry houses based on realistic seismic conditions for Zone 2.

The study to date has included the testing of four masonry houses, with both unreinforced and partially reinforced wall panels, assembled to form 16 ft square models of typical masonry houses. The walls of these four houses were subjected to in-plane and out-of-plane forces separately. The masonry units utilized in the construction of all test structures were full-sized, 6-inch wide units. Each house was provided with a timber truss roof structure to which weights were attached so as to obtain realistic loads on the bearing walls.

Methods, models and test facilities utilized in the study are described and a detailed description of the measured response of each test structure is provided. Final recommendations will await the results of one more shaking table test in which the walls of the house will be subjected to both in-plane and out-of-plane forces.

# SHAKING TABLE STUDY OF SINGLE-STORY MASONRY HOUSES

by Ray W. Clough[1], Ronald L. Mayes[2] and Polat Gülkan[3]

## 1. INTRODUCTION

Seismic design requirements specified by the Department of Housing and Urban Development (HUD) are referred to "seismic risk zones" defined by the Uniform Building Code (UBC). Thus, when the UBC changed its zoning map so that Phoenix, Arizona was included in Zone 2 rather than Zone 1, HUD issued a Local Acceptable Standard No. 2 specifying that masonry housing in Phoenix must be partially reinforced, whereas no reinforcement was required for the Zone 1 designation which previously applied to Phoenix.

Significant concern over this change of construction requirements was expressed by many individuals in the local masonry housing industry because little evidence was available to demonstrate the need for increased earthquake resistance of single-story masonry houses in Phoenix. In order to determine whether such construction should be reinforced, HUD contracted with the Earthquake Engineering Research Center (EERC) of the University of California, Berkeley, to undertake a research project entitled "Laboratory Studies of the Seismic Behavior of Single-Story Residential Masonry Buildings in Seismic Zone 2 of the USA". The general objectives of this investigation, which was initiated in April 1976, were to determine the maximum earthquake intensity that could be resisted satisfactorily by an unreinforced masonry house, and to evaluate the additional resistance that would be provided in the structure by partial reinforcement.

Specific tasks included in the research program were

(1) to survey masonry construction in typical Western U.S. cities located in Zone 2, in order to identify construction components and connection systems suitable for testing;

(2) to design and test roof-to-wall connections typical of those employed in masonry construction in Zone 2;

(3) to design simple test structures consisting of full-scale masonry wall components and typical timber roof systems, and to evaluate their performance when subjected to earthquake-like base motions induced by the University of California Earthquake Simulator;

(4) to make recommendations concerning reinforcement requirements

---

[1]Professor of Civil Engineering and Assistant Director, Earthquake Engineering Research Center, University of California, Berkeley, U.S.A.

[2]Assistant Research Engineer, Earthquake Engineering Research Center, University of California, Berkeley, and Principal, Computech, Berkeley, U.S.A.

[3]Associate Professor of Civil Engineering, Middle East Technical University, Ankara, Turkey; formerly, Visiting Associate Research Engineer, Earthquake Engineering Research Center, University of California, Berkeley, U.S.A.

and connection details for masonry construction in Zone 2.

Various phases of the investigation have been described in a sequence of reports (see references (1) to (5)), which are summarized in this paper.

The purposes of this paper are to provide a brief summary of all work done during this investigation, to make general observations and draw conclusions regarding the seismic behavior of single-story masonry hosues, and to make tentative recommendations for seismic design requirements and connection details for masonry houses in Zone 2. A separate section of the paper is devoted to each of these topics.

Technical management of this research project was provided for HUD at various times by W.J. Werner, J. McCollom and R. Morony; additional HUD coordination was given by A. Gerich and L. Chang. In addition, an Advisory Panel to the project consisting of four engineers (J. Gervasio, J. Kesler, O.C. Mann and R. Sharpe) and one contractor (L. Pritchard) was established under the administration of the Applied Technology Council (ATC). This panel provided advice and suggestions at all stages of the research. ATC will prepare its own recommendations to HUD with regard to seismic design requirements for masonry house construction, based on the results obtained and observations made during this investigation. The details of ATC's recommendations will be developed by a subcontractor, Benson and Gerdin, Inc., of Phoenix with the assistance of the Advisory Panel. Final approval of the recommendations will be made by the ATC Board of Directors. R. Benson and T. Irwin of Benson and Gerdin have attended all Advisory Panel meetings.

## 2. SUMMARY OF INVESTIGATIONS

### 2.1 Field Trip

In order to familiarize the EERC project personnel with construction practices for masonry housing in Zone 2, a trip was made to HUD offices in typical Zone 2 cities of Western United States. Although the EERC research workers had been involved for more than three years under National Science Foundation sponsorship with research on seismic behavior of masonry, that study dealt with high rise building construction; hence, it was useful for them to become acquainted with the significantly different construction procedures employed in single-story residential buildings.

Three EERC researchers (Mayes, Omote and Chen) made the trip during the period April 23-30, 1976, accompanied by two HUD engineers. Visits were made to the HUD offices in Salt Lake City, Utah; Phoenix, Arizona; and San Francisco, California. Also visited were the offices of the Applied Technology Council in Palo Alto and of engineer Ralph Goers of Los Angeles.

Discussions were held at the HUD offices concerning seismic code provisions for masonry house construction. The visit to the Applied Technology Council office was made to discuss plans for appointment of the project Advisory Panel, and operational procedures for the panel. In Salt Lake City and Phoenix, extensive tours were made to see many typical masonry houses, both completed and under construction. In Salt Lake City, 80 percent of the masonry houses are of clay brick, while in Phoenix 80 percent are of concrete block, so the team was able to inspect numerous examples of both types of construction. These observations enabled the team to identify typical door and window arrangements in masonry wall panels as well as to inspect standard details used for connecting roof systems to the masonry walls. In addition, discussions with designers and contractors in Phoenix provided specific information on local seismic code requirements and the effect that code changes might have on construction practices in that area. Discussions were held with Mr. Goers in order to take advantage of his experience with seismic problems in housing construction; at the time of that visit he was completing a study on this subject for the Applied Technology Council with funding by HUD, dealing primarily with materials of construction other than masonry.

## 2.2 Study of Connection Details

### 2.2.1 Planning of Tests

At the time when this project was initiated, very little was known about the earthquake behavior of single-story masonry houses; in particular it was uncertain whether damage would occur first in the masonry walls or in the connections between the walls and the roof structure. Almost no information was available on the strength of typical roof connections, so it was not possible to predict reliably whether these might be a weak link in masonry housing construction.

Accordingly, during the early stages of planning this research program it was decided that testing of typical roof connections for masonry houses be included, and as was mentioned earlier the configuration of such connections was studied extensively during the field trip. An important requirement in planning these tests was to include connections similar to those that would be used in the structures to be tested on the shaking table, so that the performance of the shaking table specimens could be interpreted appropriately.

### 2.2.2 Connection Specimens

Five types of connections were selected for testing, three designed for typical timber roof truss systems and two for flat roof diaphragm construction. Figure 2.1 illustrates these five connection types. Connections designated C1 and C2 represented the attachment of the wall to the roof structure at the ends of the trusses; thus, these represent the load bearing wall connection. C1 was subjected to shear loads in the plane of the wall, while C2 transmitted forces normal to the plane of the wall. C3 was a typical gable end connection of the roof system; thus it was not load bearing. Figure 2.2 shows the locations in a truss roof system where connections of types C1, C2, and C3 would be found. The actual wall attachment in all of these connections is formed by bolting a timber plate to the top of the masonry wall; the trusses are then nailed and otherwise connected to the plate.

Connection types C4 and C5 were typical of the load bearing and non-load bearing edges, respectively, of a flat roof system. The actual attachment was provided by a 3x8 in. or a 2x8 in. ledger bolted to the face of the masonry wall. For C4, attachment of the roof diaphragm to the ledger was provided by metal joist hangers as well as by nailing of the plywood sheathing; for the non-load bearing connection C5, the attachment was made only by nailing the plywood sheathing. The controlled cyclic displacement loading was applied normal to the plane of the wall for both joint types C4 and C5.

A total of 19 joint specimens was tested. The masonry wall panels used in all tests were 8 ft. long and were built on 7½x16 in. concrete footings. All were made of hollow concrete block units except for two that were made of hollow clay brick, and all had #4 reinforcing bars grouted in the end cells. Anchor bolts used to attach the roof plates in type C1, C2, and C3 specimens were grouted into the appropriate cells of the top three masonry courses of the wall; ledger plate bolts for specimens C4 and C5 were placed in cells grouted only in the top two courses. For convenience, the code numbers and other pertinent information about the test specimens are summarized in Table 2.1.

It is important to note that the seismic forces which may act on the roof connections in the direction normal to the wall are quite limited. In the shaking table test structures and also in many prototype houses, only the out-of-plane inertia load of the upper portion of the wall itself will be resisted by these connections. However, if the wall of the prototype structure is laterally supported by a perpendicular partition wall, its out-of-plane stiffness can contribute significantly to resisting the seismic loads of the roof structure. Therefore, to simulate this critical load condition for test specimens subjected to loads normal to the wall, the wall was braced against out-of-plane displacement. Thus, the tests of these connections demonstrated the actual connection strength even though such large loads would not be induced in the connection in many practical situations.

### 2.2.3 Test Procedures and Instrumentation

The loading applied to the connection specimens was intended to simulate the forces developed in typical connections during an actual earthquake. Accordingly, the test specimen was anchored at its base and cyclic loads were applied by means of a hydraulic actuator. For specimen types C1 - C3, the loading varied sinusoidally, including alternating positive and negative displacements; a sequence of three sine waves at constant amplitude constituted a test "run". For each specimen, the first run was of very small amplitude; the amplitude was then increased sequentially for successful runs until failure occurred. For specimen types C4 and C5, only the positive (tensile) direction of the sine waves was applied, because the negative displacement would merely induce direct compression between the timber components and the masonry walls and, therefore, would not stress the connection.

The connections of type C1 were loaded in the plane of the masonry wall, while connection types C2, C4, and C5 were loaded normal to that plane; C3 connections were loaded both in-plane and in the normal direction. Quantities measured during each test included the actuator force and displacement, as well as displacements of various parts of the connection or relative displacements between connection components. Force or displacement quantities were measured five times per second for each gage circuit, and the results were stored in digital form on a magnetic tape. These results were subsequently processed by digital computer and presented in graphical form for evaluation and interpretation.

### 2.2.4 Results of Tests

The behavior of the joints during these cyclic tests is best described with reference to "hysteresis" loops which depict the displacement of some point or component in the joint plotted against the applied actuator force. Such figures are presented in the report on the connection tests[3] for selected deformation and displacement quantities of all test specimens. Also, included for each specimen is a graph representing the "envelope" of the actuator force-displacement hysteresis loops. These envelope curves show how the cyclic forces changed during the successively increased displacements, and thus provide a convenient overview of the connection performance.

A summary of the test results is presented in Table 2.1. In addition, data presented in Chapter 3 of this paper (Table 3.5) compare the measured strengths of the connection specimens with estimates of the maximum forces induced in the corresponding connections during the shaking table tests. The fact that no connection failures occurred during the shaking table tests indicates that the connection designs are satisfactory for use in seismic regions.

In general, it was concluded from the connection tests that specimens of types C1, C2, and C3 were limited by the strength of the nails in the joint system. The connection strengths were found to be 1.5 to 2.9 times the strength predicted from the specified UBC code strengths of the nails. The capacity of connection types C4 and C5 depends primarily on the pull-out tension strength of the bolts attaching the ledger boards to the face of the masonry wall. In the C4 connections, where the load was transmitted to the ledger through joists as well as the plywood sheathing and where the ledger dimensions were 3x8 in., the failure was primarily a consequence of bolt pull-out. It is of interest to note that this strength was found to be independent of bolt size or of gravity load effects on the joint, it depended mainly on the strength of the grout used in embedment of the bolts and the rupture strength of the face shell of the masonry unit. In the C5 connections, the capacity was found to be limited by the cross-grain bending strength of the 2x8 in. ledger board. When the washers used in the bolted attachments were sufficiently large and stiff, this type of failure was prevented and the bolt pull-out occurred; but cross-grain bending still contributed significantly to the flexibility of the connection.

## 2.3 Shaking Table Experiments

### 2.3.1 Design of the Test Structures

The basic concept of this entire project was to make use of the EERC Earthquake Simulator Facility in studying the behavior of single-story residential masonry construction when subjected to base motions similar to those observed in real earthquakes. The principal limitation encountered in planning the test program was the size of the shaking table. Its 20 ft square surface is not large enough to support a real house, and one of the first concepts of the test program was to use half-scale models. The pertinent requirements of model similitude are well established and, in principal, valid results could have been obtained by this approach. However, very extensive experimentation would have been required to obtain half-scale masonry materials and assemblages that would correctly simulate the strength of prototype components. Moreover, workmanship has an important influence on the properties of masonry components, and because it would be very difficult to model typical field workmanship in half-scale laboratory specimens it was not certain that the test results could be extrapolated reliably to field performance.

Consequently, the alternative approach was adopted — using full-size masonry components which would have strength properties representative of field construction, and assembling a system of the components arranged so that their behavior would be similar to that of an actual house. In order to simulate the true seismic behavior, it was necessary to use typical full-height wall components, to arrange them so that some were subjected to in-plane and some to out-of-plane excitations, and to apply appropriate roof loads per unit length by means of typical roof-to-wall connections. The most practical means of satisfying these requirements was to arrange the wall components in a rectangular plan, supporting a typical timber truss roof system attached by standard connection details. The roof surface at the top of the trusses was 1/2 in. plywood; gypsum wall boards were nailed on the bottom of the trusses to form the ceiling.

The most significant deviation from normal house construction that would result from this concept for the test structure was in the roof load supported per unit length of wall component. Because the total roof load varies with the square of the plan dimensions while the perimeter length depends directly on these dimensions, the average load per unit wall length of a normal house varies linearly with the plan dimensions. Thus, unless an adjustment was made, the roof load per foot of the test structure wall would have been very small. Accordingly, considering that the floor area of this test structure would be about one-ninth that of a reasonable prototype, it was decided to use a length factor of three and apply sufficient concrete blocks to the roof on the model so that its total weight would be three times greater than that resulting directly from the design roof load of 20 psf.

In planning the first test structure, it was decided that the essential behavior of the prototype walls could be simulated by a system of independent piers, and that at least one 8 ft wide pier could be assumed to exist on each side of the house. Also, it is believed that exterior corners of the house might play a special role in the performance of the masonry walls. Consequently, this first specimen was designed with an 8 ft wide panel in the middle of each of its four sides, and with a corner component 2 ft on each side located at each corner. Figure 2.3 shows the layout of this first test structure as well as a view of it on the shaking table. In order to maximize the information obtained from this test specimen, it was decided to provide partial reinforcement in four of these eight components, including one 8 ft pier parallel to the direction of test excitation and a similar pier oriented perpendicularly as well as two of the corner components.

A major aspect of the prototype behavior that was not contained in this first test structure was the influence of openings on the stiffness and strength of the wall panels. Accordingly, in planning the remaining three test structures, each wall panel was designed to include either a typical window and door opening combination or a typical large door opening. Four such wall panels formed the walls of the test structure; one of each type was oriented to resist in-plane loads, and one of each type was placed in the perpendicular direction. No direct connections were provided between the wall panels; consequently, the test condition was conservative in not including the flange effect at the corners. Figure 2.4 shows the second specimen; the third was very similar to this. Figure 2.5 shows the fourth test structure. For the second and third test structures, the in-plane and out-of-plane wall panels with the large door opening were provided with partial reinforcement, while the other walls were unreinforced. The fourth test structure was tested initially with no reinforcement in any of the walls, and then partial reinforcement was provided in all wall panels for its final stage of testing. An additional feature of the fourth test structure was that four independent 3 ft 4 in. wide piers were attached to the shaking table oriented for out-of-plane loading. These were intended to demonstrate the effect on the response behavior of increased dynamic displacements resulting from flexible supports at the top edges of the piers; they were provided with different combinations of reinforcement and dowelling details.

The masonry pier and wall components which were assembled to make the test structures were each constructed on a concrete base representing the footing that would have been provided in the field. Except for House 3, all components were made of standard two-core hollow concrete block units with nominal dimensions 6x4x16 in.; House 3 was made of two-core hollow clay brick units with 6x4x12 in. nominal dimensions. The mortar used in all components was ASTM Type S. The wall components were made at a location away from the shaking table by journeyman masons following normal construction practice, and they were considered to be representative of typical masonry quality.

### 2.3.2 Test Procedures

After curing for at least 4 weeks, the wall components were moved to the shaking table and their concrete bases were anchored in position. During the initial tests of the first house, the bases were constrained only against sliding; rocking (i.e., uplift at either end or side) was permitted because the degree of constraint to be expected with a real house foundation was not known. Early in the testing of the first house, however, it became apparent that uplifting at the ends of the in-plane walls (i.e., those parallel to the base motion) had a major influence on the response behavior, and in later tests the bases of the in-plane walls were clamped to minimize uplift.

After the wall components were attached to the table, the prefabricated roof truss system was lowered into position by the laboratory crane and attached to the wall components by bolts through the roof plate arranged at normal spacing intervals. Assembly of the test structure was a very delicate operation because of the fragility of the 8 ft 8 in. high free-standing unreinforced wall components, and some cracking of the panels occurred, especially in assembly of the first test structure. Cracks which might influence the response behavior were repaired before testing began; but little such repair was required in the later test structures.

The sequence of tests performed on House 1 is given in Fig. 2.6. During the first phase of testing of the first test structure, the roof system was oriented so that the trusses were parallel to the direction of excitation. Then after a sequence of tests was completed, the roof system was removed, rotated 90 degrees, and rebolted to the wall components. From the results obtained in testing the first structure, it was evident that the roof orientation with trusses parallel to the excitation induced more severe response effects in the walls for a given intensity of base motion, presumably because the in-plane walls (which resist the seismic loads) were not load bearing and thus did not benefit from vertical compressive stresses. For this reason, it was decided to test Houses 2 and 3 with the trusses oriented first in the transverse direction so that less damage would be incurred during the first phase of testing; then when the roof was rotated to the parallel orientation the structure was in better condition to resist the effects of the unfavorable orientation. In testing House 4, the roof trusses were oriented only in the more favorable transverse direction; it was not rotated for a second stage of testing of this structure. The sequence of tests performed on Houses 2, 3 and 4 are given in Figs. 2.7, 2.8 and 2.9, respectively.

To record the response of the test structure to the shaking table motions, a large number of electrical gages of various types was installed, ranging between 50 and 65 in different tests. Accelerometers were attached at various locations on the roof structure and on the upper portions of the wall panels. Displacement gages were arranged to measure displacements of various points on the roof and walls relative to the shaking table, and also the relative motions between various parts of the structure. During the tests the electrical signal from each gage was evaluated and recorded in digital form at the rate of 50 readings per second. After completion of each test, the digital data were transferred to magnetic tape for permanent storage and for subsequent processing and plotting.

Table motions employed in this test program were based on accelerograms recorded at El Centro, Taft, and Pacoima Dam during the earthquakes of May 1940, June 1952 and February 1971, respectively. In each case the full duration of the record was used. The El Centro and Taft motions were selected because they are considered to be typical of the Western United States earthquakes. The Pacoima Dam record was used because it has an unusual long-duration acceleration pulse which can be very damaging to stiff and brittle structures. Unfortunately it was not possible to include test motions typical of earthquakes that might occur in Eastern United States, because no strong motion seismograph records have been obtained from such earthquakes; however, it is believed that it is conservative to use the Western earthquakes instead.

In testing the first three houses, the earthquake motions were applied initially at a very low intensity (Figs. 2.6, 2.7 and 2.8), and several successive tests were made with the same motion at increasing intensities. After reaching the desired intensity with one type of motion, the process was repeated with a different earthquake motion, starting again from a relatively low intensity. As many as 19 such tests were applied with the roof in its initial orientation; a similar but shorter testing sequence was then applied with the roof rotated 90 degrees. The structure was examined carefully after each test and new damages were recorded as they were observed.

An obvious disadvantage of this sequential testing procedure is that the structure undergoes progressive damage as it is subjected to a highly unrealistic number of earthquake motions. The house was already significantly damaged by the time it was subjected to the most severe motions, thus the effect of the motion on an undamaged structure was not determined. To partially avoid this difficulty, House 4 was subjected to only six significant tests, each having a peak acceleration in excess of 0.25 g, as shown in Fig. 2.9.

### 2.3.3 Results of Experiments

The most significant results of this extensive experimental study are reported in references (4) and (5) and the reader is referred to those volumes for detailed response data. The reported results include a listing for each test structure of the "peak" positive and negative response values recorded by the more significant gages during each of a selected group of test runs. These tables quickly provide an indication of the intensity of response during the more severe simulated earthquakes, but caution is required in interpreting the response behavior from these numbers because the listed peak values are not concurrent and therefore do not define the response pattern.

The damage history undergone by each test structure is indicated in references (4) and (5) by a series of drawings of the wall panels on which are shown the cracks existing after each test; thus, each sequence of drawings illustrates the damage history of the test structure. Also included are graphs showing the variation with time of various gage readings. Correlation of such "time-history" plots, by considering relative amplitudes or phase relationships for related quantities, provides considerable insight into the behavior of the test structures. A final form of graphical display presented in these reports is an isometric view of the out-of-plane walls depicting the out-of-plane displacements plotted to a greatly exaggerated scale. Sequences of such plots drawn for selected instants of time during the most severe response phase provide a convenient visualization of the dynamic behavior. There is no need to review this entire body of response information in the present paper; however, selected items will be used to quantify the observations on response behavior presented in the next section.

# 3. OBSERVATIONS ON THE RESPONSE BEHAVIOR

## 3.1 General Description of the Response Mechanism

### 3.1.1 Seismic Response of "Box" Structures

The test specimens used in this study were typical "box" structures, which derive their primary lateral force resistance from "membrane" action of the walls and roof rather than from flexural resistance as is provided by rigid frames. The lateral force induced in the structures by earthquake excitation may be expressed by the product of base acceleration and structure mass, and it acts in the direction opposing the acceleration. For the test specimens, the greatest part of this lateral force resulted from the concrete blocks bolted to the roof; additional inertia force was associated with the mass of the timber roof structure and of the wall components.

The resistance to these inertia forces is provided in a typical box structure by the in-plane shear rigidity of the roof and the wall components; the out-of-plane shear rigidity of the wall panels and the flexural stiffness of their connections to the roof are of negligible value in resisting the roof loads. Thus, the inertia forces developed in the roof are transmitted laterally to the in-plane wall connections, and through these walls to their base connection at the shaking table. The inertia forces exerted by the out-of-plane walls are transmitted by vertical beam action to their bottom and top connections. The roof structure provides the top support for the out-of-plane walls, and these support forces add to the roof inertia forces to be resisted by the membrane action of the in-plane walls.

From this description, it is clear that the out-of-plane walls of a masonry house must have sufficient flexural strength to resist their own inertia forces when acting as vertical beams supported at top and bottom, while the in-plane walls must have the capacity to resist the inertia forces of the entire roof system plus the top half of the out-of-plane walls. In addition, the roof structure must be strong enough to transmit the roof forces and the out-of-plane wall forces to the in-plane wall connection by membrane action.

Dynamic distortions are developed in the components of the box structure corresponding to each of these load transfer mechanisms. The in-plane walls and roof structure are subjected primarily to shear distortion while the out-of-plane walls undergo flexural distortion.

During intense earthquakes, the in-plane walls may also undergo rigid-body rocking associated with cracking and uplift at the base. If the structure is symmetric with regard both to mass and stiffness, equal distortions will develop in both in-plane walls, and the roof will undergo translation without rotation. However, if the inertia loads are applied mostly to one in-plane wall or if one wall is significantly less stiff than the other, that wall will undergo greater distortions with a consequent tendency to cause rotation of the roof structure. If the roof structure has sufficient membrane rigidity, it will rotate as a rigid unit; this rotation will then induce corresponding membrane shear deformations of the out-of-plane walls. However, if the out-of-plane walls are more rigid than the roof structure in membrane action, they will resist this tendency and force the roof structure to develop "racking" (shear) distortion to accommodate the unequal displacements at the top of the in-plane walls.

### 3.1.2 Observed Behavior of Test Structures

In general, the observed behavior of the test houses was consistent with this description of the behavior of box structures subjected to lateral loads. A qualitative description of the structural response is presented in this section of the paper; the principal quantitative measures of the dynamic behavior of each test structure are summarized in a later section.

During the shaking table tests, the roof displacement amplitudes were seen to be directly related to the behavior of the in-plane walls; large displacements resulted from cracking and/or uplift of either in-plane wall. Differential displacements of the two in-plane walls were accommodated mainly by "racking" distortion of the roof structure; relatively little in-plane distortion was observed in the out-of-plane walls, so the roof structure did not rotate as a rigid unit. This observed behavior is consistent with the usual design assumption that plywood diaphragms are much more flexible in shear distortion than are masonry walls.

The flexibility of the roof system also was evidenced by its behavior during tests when the roof trusses were perpendicular to the excitation. In this orientation, the inertia forces of the concrete blocks, which were located high on the roof structure, caused the ridge line of the roof to displace longitudinally relative to the in-plane wall constraints at the lower edges of the roof diaphragm. This "racking" mechanism of the roof trusses was related to shearing distortions of the plywood roof sheathing. The large racking distortions between the roof trusses which were observed in House 1, were reduced in later tests by bracing installed between the trusses along the central plane of the structure. However, some motions of this type were observed in all tests made with the trusses perpendicular to the excitation. For the truss orientation parallel to the motion no racking distortion occurred in the roof system, and the test structure exhibited much greater rigidity.

An additional type of distortion was observed in the roof structure when the trusses were oriented perpendicular to the excitation. In this position, the gable ends of the roof provided the top support for the out-of-plane walls, and the inertia forces generated in these walls caused lateral bending of the bottom chord of the gable trusses. The roof-wall connection was designed to transfer these lateral loads into the ceiling gypsum-board of the roof structure.

Although the test structures provided a reasonable degree of flexibility at the top support of the out-of-plane walls, it was not known whether the seismic displacements of such walls in real houses might be significantly greater. As was mentioned earlier, a set of four supplementary piers was assembled on the shaking table together with House 4, to determine the effects of increased top support flexibility on the performance of out-of-plane walls. These piers were constructed on the same footing blocks as the out-of-plane house walls, and were positioned so that the table motions induced out-of-plane loading. At the top they were supported by a steel channel which provided considerable lateral flexibility. In spite of the extra deformability introduced by the channel support, the response behavior of the supplementary piers was quite similar to that observed in the out-of-plane house walls. Thus, it was concluded that the out-of-plane walls of a masonry house are not sensitive to any reasonable amount of flexibility provided in the top support, and that no special connection details need be provided to reduce the wall support flexibility.

### 3.1.3 Dynamic Response Characteristics

One of the most significant observations made from these experiments is that typical single-story masonry houses are so rigid that they do not develop very complicated dynamic response mechanisms during an earthquake. The motions of the test structures followed the shaking table motions very closely, with distortions generally being proportional to, and in-phase with the base accelerations. For this reason, the frequency characteristics of the earthquake input are not a major factor in its tendency to induce damage in a masonry house; the peak acceleration value of the ground motion is the dominant quantity controlling response to earthquakes typical of Western United States. It is believed that the same conclusion would apply to earthquake motions typical of Eastern United States, but lack of appropriate accelerograms made it impossible to study such behavior in the present test program.

Although the damage potential of a test motion depends mainly on its peak acceleration, this is not to say that the test structure responded to the base motions as a rigid body. Flexibility of any component led to increased acceleration amplitudes corresponding to the increased displacements. However, this response amplification was typical of the high frequency range of the ground motion response spectrum, well above the "dominant" frequency of the earthquake, and the response could be related to the peak base acceleration by a simple amplification factor. For this reason, it was convenient to characterize each shaking table test by the peak recorded table acceleration.

Only in some unreinforced walls subjected to intense out-of-plane excitation did the behavior differ from this stiff structure response mechanism. During their first severe tests, these walls cracked horizontally in the center two quarters of their height, and developed an out-of-plane "hinging" action. The stiffness of the cracked walls was reduced significantly during these tests as compared with their uncracked state, and the out-of-plane period of vibration increased dramatically. Also, after cracking their seismic responses were essentially harmonic in character at the reduced natural frequency, and thus differed materially from the behavior of other parts of the structure. Observation of this type of response was important in the present investigation because it led to the tentative recommendation of partial reinforcement for out-of-plane resistance in the parts of Zone 2 where earthquake motions may be strong enough to crack unreinforced walls. Concern about the response of these walls to the combined action of in-plane and out-of-plane forces led to the recommendation that one more test be performed before the final recommendations are presented. In this final test the unreinforced walls will be subjected to combined in-plane and out-of-plane forces.

## 3.2 Comments on the Behavior of Test Structures

### 3.2.1 General Comments

For the purposes of this investigation, the only significant aspect of the response to seismic excitation is the degree of damage incurred; dynamic displacements are of no importance except as they are related to structural damage. In this study, HUD defined cracking or sliding displacements in excess of 1/4 in. as unacceptable damage to structural components. It is important to remember that nonstructural items such as windows and doors might also be damaged during earthquakes acting on real houses. Such items were not included in the test structures, but damage to them is closely related to the displacements developed in their supporting structural components.

Half of the wall units included in Houses 1, 2 and 3 were partially reinforced, and all of these reinforced units behaved significantly better than unreinforced walls during all tests. No major damage was observed in any reinforced component, although some cracking was observed at the base joint connecting the walls to the footing and also above the ends of the lintels over the large door openings. These cracks resulted from rigid-body rotation of the wall panels and the performance was considered satisfactory because the residual crack widths after the tests were very small. On this basis, it is not necessary to discuss further the behavior of partially reinforced walls, and for Houses 1, 2 and 3, this discussion will deal only with the unreinforced walls. All walls of House 4 will be mentioned because all these walls were unreinforced during the first phase of testing.

During the tests some walls were oriented parallel to the table motion (in-plane) and others were at right angles to these (out-of-plane). The response behaviors of the in-plane and the out-of-plane walls were quite different, and it will be convenient here to discuss each type of wall separately for each test structure. Also, the wall behavior sometimes differed according to whether or not it supported the weight of the roof system, so the load bearing and non-load bearing conditions will be identified in the discussion. In general, for any walls the most significant indicator of performance is the peak table acceleration during the test which first caused it to crack. In addition, the behavior after cracking is important, and this will be characterized by the range of accelerations achieved during post-cracking tests when the wall continued to exhibit satisfactory behavior. The most important data characterizing the cracking and post-cracking performance of the unreinforced walls are summarized in Tables 3.1 to 3.4, for Houses 1 to 4, respectively.

### 3.2.2 Response of House 1

In-Plane Wall W3: During the first phase of testing, when this wall was not load bearing, a horizontal shear crack formed during Test 9 (T-0.21 g)*. During the next test (Test 10, T-0.27 g) rocking of the panel at this crack location was observed ($\Delta$=0.4 in.). The rocking motion was then blocked mechanically for subsequent tests. During Test 11 (T-0.29 g) the wall cracked at a level below the blocking. Test 12 (E-0.14 g) and Test 13 (E-0.28 g) used a different earthquake motion; new cracks formed during Test 13, but no permanent displacement recurred.

During the second phase of testing after repair of the structure, the roof was rotated so that W3 was load bearing. The first crack in the repaired wall was observed during Test 27 (P-0.49 g); Tests 28 (P-0.63 g) and 29 (E-0.59 g) also caused dynamic displacement at the top of the wall, but none of these tests caused permanent distortion.

Out-of-Plane Wall W4: No cracking was observed in this wall until the last test of the phase when it was load bearing: Test 13 (E-0.28 g). The crack that occurred then was at the top of the bottom block course.

During the test phase when the wall was not load bearing it cracked at the 6th course from the top during Test 19 (T-0.25 g), but displacements were very small. During the next two tests significant "hinging" developed at the crack, so that greater displacements were observed at 2/3 height than at the top of the wall. The crack was then repaired with fiberglass reinforced plaster, and it cracked again during Test 27 (P-0.49 g) at a much greater shaking intensity. During the following Test 28 (P-0.63 g) no significant hinging motion was observed at the crack, but during Test 29 (E-0.59 g) it hinged with displacements in excess of one inch at 2/3 height; the test was then terminated.

### 3.2.3 Response of House 2

In-Plane Wall A: This wall was load bearing during the first phase of testing and no cracks developed during this phase even though a peak shaking table acceleration of 0.51 g was recorded during Test 19 (P-0.51 g). After rotation of the roof system so that this wall became

---

* This notation indicates that the test motion was the Taft earthquake with a peak acceleration of 0.21 g. The letters E and P correspondingly denote the El Centro and Pacoima motions.

non-load bearing, it resisted a peak acceleration of 0.37 g (Test 30, E-0.37 g) without cracking. However, during Test 32 (P-0.52 g) a diagonal crack was formed extending downward from the reentrant corner of the window. A one inch permanent displacement was developed at this crack, and no further tests were performed because this was considered to be a major failure. Cracking also was observed above the ends of the window and door openings, demonstrating that rigid body rocking of the main wall pier accompanied its diagonal cracking.

Out-of-Plane Wall A1: The first significant crack was observed when this wall was not load bearing during Test 14 (E-0.33 g). The crack was horizontal and located at the level of the bottom of the window. During subsequent non-load bearing tests with increased intensities the deformation of the wall increased, but no hinging action occurred at the crack even with a peak acceleration of 0.51 g (Test 19).

During the second phase of testing, after rotation of the roof so that this wall was load bearing, the behavior of the crack was unchanged. Displacements increased with increasing test intensities, but permanent displacements developed only as a consequence of the in-plane wall A failure during Test 32.

The performance of this house was considered superior, but it must be remembered that the strength of the mortar used in its construction was twice as great as that in the other test structures.

### 3.2.4 Response of House 3

In-Plane Wall A: This was the clay-brick test structure. As was the case for House 2, during the first phase of testing wall A was load bearing. This wall resisted a peak acceleration of 0.45 g (Test 15, E-0.45 g) without cracking, but during Test 19 (P-0.49 g) a diagonal crack developed extending downward from the window corner and resulting in permanent displacement.

The crack was then repaired and the roof rotated so this wall did not support the roof trusses. In the non-load bearing condition the crack developed again during Test 26 (T-0.21 g). The wall then was reinforced by metal straps to permit continued testing of the other wall components. Comparison of the performance of this wall with wall A of House 2 demonstrates the significantly stronger performance that may be attributed to the stronger mortar of House 2.

Out-of-Plane Wall A1: During the first phase tests when this wall was non-load bearing, it resisted a peak acceleration of 0.45 g during Test 15 (E-0.45 g) without cracking. However, during Test 18 (P-0.39 g) a horizontal crack formed at about mid-height, and significant hinging action accompanied the cracking. During the next test (Test 19) very large hinging displacements occurred (3.39 in. at 2/3 wall height) indicating that it was potentially unstable.

After repair of the crack and rotation of the roof so that this wall was load bearing, no new cracking occurred even though a peak acceleration of 0.37 g was achieved in a test that also included a vertical excitation component.

### 3.2.5 Response of House 4

In-Plane Wall A: This wall was load bearing throughout the tests of House 4. During Test 2, a two-component excitation with peak horizontal acceleration of 0.34 g, no cracking was observed. But during the next test, a two-component Taft earthquake with peak horizontal acceleration of 0.29 g, a diagonal crack involving permanent displacement formed at the reentrant lower corner of the window and extended diagonally downward. The wall was then partially reinforced before testing was continued.

In-Plane Wall B: A crack developed at the base of this wall during Test 2, but no permanent displacement occurred along it. Motions at this crack were of the rigid body rocking type similar to those observed in the partially reinforced walls B of the other test structures. Similar rocking motions without permanent displacements were observed in this wall during Test 3.

Out-of-Plane Wall A1: This non-load bearing wall developed a horizontal crack at the level of the top of the window during the two-component Test 3, but no hinging action or permanent displacement resulted. It already had resisted a more intense two-component El Centro motion during Test 2 with no damage.

Out-of-Plane Wall B1: The behavior of this wall was very similar to that of A1, except the first cracking took place during Test 2; the crack was horizontal at the level of the top of the door opening. During Test 3 increased displacements were observed, but there was no hinging action or any suggestion of potential instability.

Pier 1: This was one of the independent piers provided with flexible support at the top. A shrinkage crack existed in this pier at the beginning of the tests, and all dynamic deformations were associated with this crack. The pier resisted strong shaking during Tests 2, 3, and 5, with moderate displacements and no tendency toward hinging action or instability; the peak acceleration during these tests was 0.34 g. During the two-component Test 6 (E-0.54 g), however, very large hinging action displacements were observed, the peak deflection at 2/3 height being over 5 in. After this potentially unstable response the pier was braced to avoid further damage.

Pier 3: This pier was similar to pier 1, but without a dowel: it did not have a pre-existing shrinkage crack. The first crack occurred during Test 2 at about 70 percent of wall height. During subsequent tests this pier's performance was very similar to that of pier 1; it also developed very large displacements during Test 6 (though the maximum displacement here was 3.5 in.) and it also was braced to avoid damage in subsequent tests.

### 3.2.6 Conclusions from the Observed Behavior

The most significant features of the observed responses of the test structures taken as a whole may be summarized as follows:

For unreinforced wall units:

(1) No cracking was observed in any major unreinforced wall unit for tests with peak accelerations less than 0.2 g. The first observed cracking in such units occurred in the in-plane walls of Houses 1 and 3 during tests with peak accelerations of 0.21 g; the units were non-load bearing during these tests. The lowest intensity shaking that caused cracking of an out-of-plane wall unit was 0.25 g in a test of House 1.

(2) Unreinforced out-of-plane walls continued to perform satisfactorily after cracking for several tests of increased intensity, but the displacements of these walls generally became excessive during tests with peak accelerations in excess of 0.4 g. These large displacements involved hinging at the horizontal crack line and exhibited potential instability, although actual collapse did not occur in any test.

(3) Cracking of unreinforced in-plane walls was of two types: a horizontal crack in piers without window openings, and a diagonal crack extending downward from the window corner in wall units having openings. Small dynamic displacements were observed in the horizontal cracks, but permanent displacements were negligible. However, the diagonal cracks extending from the window corners developed permanent displacements which became unacceptably large with continued testing or after a high intensity test. No limit of peak acceleration could be established which would ensure acceptable performance of cracked window walls.

For partially reinforced wall units:

(1) Nearly all partially reinforced wall units performed satisfactorily in all tests. None of the partially reinforced out-of-plane components developed any important cracks during any test, including several that were subjected to peak accelerations in excess of 0.5 g.

(2) In general, the partially reinforced in-plane walls also performed satisfactorily although they developed some cracks when peak accelerations exceeded 0.3 g. Cracking in the pier units without window openings was associated with rigid body rocking, and included a horizontal crack due to uplift near the base of the wall as well as cracks at the ends of door spandrels. Residual cracks after the tests were all narrow and easily repairable.

(3) The only partially reinforced in-plane wall which showed unsatisfactory behavior was the window wall of House 4. A typical diagonal crack extending from the window corner developed during the first phase testing when this house was unreinforced. The wall was not repaired, but after reinforcement was added the wall resisted a peak acceleration of 0.32 g without additional cracking. In subsequent tests with peak accelerations in the range of 0.47 g to 0.68 g additional cracks developed in spite of the reinforcing bars. The fact that no

dowels were provided with this reinforcement may have been significant because the ultimate failure was a result of rocking at the base of the wall. Such motion probably would have been constrained by vertical reinforcement fully capable of developing its tensile strength through the action of dowels.

### 3.3 Comments on the Performance of Connections

It was mentioned in Chapter 2 of this paper that no damage occurred in the roof-wall connections during the shaking table tests, and this fact was the principal evidence leading to the conclusion that the connection details of the test houses are adequate for masonry houses built in Zone 2. In an attampt to provide a quantitative basis for this conclusion, estimates of the connection forces developed during shaking table tests were correlated with the results of the cyclic loading of connection specimens.

Forces causing connection stresses during the shaking table tests were calculated from the accelerations measured at appropriate points in the test houses. For the out-of-plane walls, the roof connections must provide the top reaction to the wall inertia forces, and these inertia forces were determined from accelerations measured at the shaking table and at the top and 2/3 wall height. For the in-plane walls, the connection forces must equilibrate the inertia force of the roof system, which was calculated from the accelerations measured at the top of the roof gables, plus the reaction force applied by the out-of-plane wall connections.

The maximum connection forces of each type were calculated from the maximum accelerations measured at the appropriate locations. These forces, evaluated per unit of wall length, per connection bolt, and per roof truss, are listed in Table 3.5, together with corresponding quantities measured during the cyclic tests of the connection specimens. Comparisons of these results with UBC code allowable values are given in Table 3.6. Study of these tables reveals that the maximum forces developed in the in-plane wall connections during shaking table tests exceeded the corresponding connection strengths measured during the cyclic tests and were five times greater than UBC code allowable forces. The main reason for this discrepancy is that the specimens fabricated for the connection test program were not as strong as the connections in the actual roof structure; they were deliberately underdesigned in order to be conservative. The failure mechanism observed in these connection tests was largely associated with twisting of the 2x4 in. members representing the bottom chords of the trusses. In the actual roof structure this type of twisting was resisted by the upper members of the truss that frame into the bottom chords, but such upper members were not included in the connection test specimens.

The data in Table 3.5 suggest that damage would have been observed in the roof connections during the shaking table tests if they had not been stronger than the connection specimens used in the cyclic tests. In evaluating the connection performance, however, it is important to note that the forces listed in the Table for the shaking table tests are considerably greater than the maximum forces that can be expected in Zone 2; the peak shaking table accelerations during these tests were over 0.5 g. Hence, the conclusion stated earlier remains valid: the types of roof connections used in the shaking table tests are adequate for masonry houses built in Zone 2.

## 3.4 Extrapolation to Prototype Conditions

### 3.4.1 Seismic Input

The observed behavior of the test structures which has been summarized in the preceding section of this paper provides the basis for tentative recommendations presented later concerning seismic design criteria for single-story masonry houses. However, before these results may be applied, it is necessary to estimate the extent to which they represent the performance of real houses subjected to real earthquakes. In other words, the discrepancies between the conditions established in the shaking table tests and those existing in a prototype response to earthquakes must be identified, and their influence on the degree of damage must be evaluated.

Considering first the seismic input applied by the shaking table, the control signals which defined the input motions were derived directly from accelerograms recorded in actual earthquakes. The three earthquakes employed in the test program are representative of Western United States experience, and with appropriate scaling they are suitable examples of ground motions to be expected in Zone 2 of this region. Moreover, the response spectra of the shaking table motions are generally similar to the response spectra of the corresponding earthquake accelerograms. The normalized response spectra presented in Fig. 3.1 (taken from reference 4) show that the shaking table motions used with Houses 1 and 2 have essentially the same frequency characteristics as the actual earthquakes. Figure 3.2 (taken from reference 5), on the other hand, demonstrates that the table motions applied to Houses 3 and 4 have somewhat different frequency characteristics from the real earthquakes; the dominant frequencies of these test motions have been increased slightly for all earthquakes, and amplification effects have been increased significantly for two of the earthquakes. The reason for this change in the input for Houses 3 and 4 is not presently known because the same shaking table control tapes were used for all test structures. However, for the frequency range of the test structure (4 - 6 Hz), the accelerations induced by the table motions applied to Houses 3 and 4 are greater than would have resulted from the actual earthquakes normalized to the same peak acceleration, so the test results are conservative in this sense. For Houses 1 and 2, the shaking table motions are entirely equivalent to the actual earthquake motions.

In another respect, however, a very important difference existed between the shaking table tests and the prototype earthquakes: the shaking table could apply only one horizontal component of motion in addition to the vertical component, whereas the actual earthquakes would subject the prototype structures to shaking in two directions horizontally plus the vertical. Typically, the two horizontal components of an earthquake are of differing intensities, with the peak acceleration of one component usually being 60 to 75 percent of that in the perpendicular direction. In these tests, the shaking table motion is scaled to represent the larger component; hence, the tests are deficient in not including the simultaneous but smaller excitation in the transverse direction.

It is very difficult to estimate the influence of this discrepancy on the damage to be expected in the structure. The real earthquake subjects the masonry walls simultaneously to in-plane and out-of-plane forces, but it is not known to what extent the damage mechanisms resulting from the two types of forces are coupled. In any case, it must be assumed that the test results are not conservative with regard to this factor, i.e., that a three-component excitation would have caused more severe damages than were observed in these tests.

### 3.4.2 General Conclusions Concerning Validity of Test Results

It was concluded that the test conditions are reasonably representative of prototype conditions with respect to the general character of the earthquake motions, the foundation and roof diaphragm flexibility effects, and torsional response. With regard to geometric effects, the results probably are conservative (i.e., show greater damage than would occur in the prototype) because the walls of prototype houses generally are better tied together and provided with supplementary bracing. On the other hand, pre-existing stresses and/or cracks (due to shrinkage, temperature change or foundation settlement) might exist in the prototype, which would increase its tendency toward damage. Scaling effects with regard to the roof load per unit of wall length might favor either the model or the prototype. Considering all of these factors together, it may be concluded that the behavior observed in the shaking table tests is quite similar to the performance expected of a real house subjected to a real earthquake with the same peak acceleration.

However, the test procedure does have one major deficiency which may have a very significant effect on the structural behavior: the horizontal motions of the shaking table are only in one direction. It is not possible to determine how much more damaging the tests might have been if they had included two horizontal components of motion, but it must be assumed that a given intensity of earthquake in the field would produce greater damage than was observed in the models in response to the same intensity of single horizontal component motion. This probable increase of damage would be due to "coupling" effects, where the deformation due to one component of excitation would reduce the capability of the wall to resist the other component.

Because the unreinforced walls subjected to intense out-of-plane motion responded by significant cracking and "hinging" out-of-plane, it is probable that the capacity of such walls to resist simultaneous in-plane shear forces would be greatly diminished. For this reason, the second horizontal component of input motion would be expected to have a detrimental effect on unreinforced masonry walls. Thus, before final recommendations are presented, another test will be performed to determine the significance of the severity of this coupling effect. In order to develop the tentative recommendations presented herein it seems reasonable to assume conservatively that this effect is equivalent to an increase in the intensity of single horizontal component motion between 30 and 50 percent until the final test is performed. On the other hand, the partially reinforced masonry walls exhibited little distortion regardless of the intensity of in-plane or out-of-plane shaking; thus, there would be little tendency for coupling of the in-plane and out-of-plane effects. Probably there is no need to apply any increase to the shaking table accelerations to represent the second component effect on partially reinforced walls, but to be conservative a 20 percent increase in the single horizontal component motion will be assumed for this case.

# 4. DESIGN RECOMMENDATIONS

## 4.1 Seismic Input For Zone 2

### 4.1.1 General Comments

From the earliest stages of this experimental investigation, one of the most critical questions was the intensity of shaking table accelerations that should be used to represent the maximum earthquake motions expected in UBC Zone 2. No decision on this matter was required while the experimental studies were being performed because the test program was planned to determine the seismic capacity of masonry housing, both unreinforced as well as partially reinforced. In other words, these tests served to determine the peak shaking table accelerations at which cracking was initiated and unacceptable damage was observed. However, before design recommendations can be made for Zone 2, it is necessary to establish the intensity of shaking table motions that can be considered representative of the maximum expected field conditions.

This correlation of field excitation with shaking table motions is required, of course, to relate the damages observed in the test structures to the expected behavior of real houses in Zone 2. In addition, the test data can be used to estimate the magnitude of seismic forces induced in masonry houses by Zone 2 earthquakes. It is important to note in this regard that masonry houses are not designed by engineering analysis based on design forces; instead their construction is controlled by minimum construction standards. Therefore, it is expected that the design recommendations which ultimately result from this study will be in the form of construction standards. However, to assist in the formulation of appropriate construction standards, estimates of the seismic loads to be expected in UBC Zone 2 are presented here, together with recommendations of some minimum standards.

### 4.1.2 Effective Peak Acceleration for Zone 2

The best current estimate of expected earthquake intensities for the United States was developed by the Applied Technology Council (ATC) as part of a National Science Foundation funded project for the development of proposed seismic design regulations for buildings(6). Figure 4.1 shows the ATC map of effective peak acceleration (EPA) contours superimposed on the 1976 UBC Seismic Zoning Map. The EPA contours are intended to represent the maximum ground motions that can reasonably be expected to occur within a 50 year period; it is estimated that there is a 10 percent probability of this EPA value being exceeded during this time period. The concept of effective peak acceleration was introduced in the ATC report because it was recognized that the peak value of ground acceleration may not relate well with the damage potential of a given earthquake. Sharp spikes in the accelerogram tend to overemphasize the peak acceleration value, while the importance of long period motions due to distant earthquakes may be underestimated by the peak acceleration. The definition of EPA is presented in the next section of this paper.

It will be noted in Fig. 4.1 that UBC Zone 2 includes a wide range of EPA values from less than 0.05 to 0.2 g. It is not reasonable to impose design requirements suitable for the maximum EPA value of 0.2 g on all parts of UBC Zone 2, and accordingly the authors of this paper have defined two categories (or sub-zones) in Zone 2. Zone 2A is the area of Zone 2 indicated by the ATC map to have an EPA of 0.1 g or less, while Zone 2B is the area of Zone 2 having EPA values of 0.1 g to 0.2 g. Different tentative design recommendations are made in this report for each of these sub-zones.

4.1.3 Correlation Between Table Motions and EPA

It was noted above that the EPA is not the peak acceleration recorded during an earthquake, therefore, it is necessary to interpret the shaking table accelerations appropriately before they can be related to the EPA of Fig. 4.1. The EPA of a given ground motion is defined by the ATC in terms of the response spectrum of the motion evaluated for 5 percent damping. Specifically, a line of constant spectral acceleration ($S_a$) is drawn on the response spectrum which approximates the average spectral acceleration in the period range of 0.1 to 0.5 seconds. The EPA then is given by this average $S_a$ divided by 2.5, where the divisor is a typical response amplification factor for Western United States earthquakes. Data are not available for the Eastern United States but these values are assumed to be conservative for that area.

EPA values of the shaking table motions were determined by applying this procedure to the response spectra of the motions recorded during the testing of each model house. Because the tests were conducted with widely varying intensities, the table motions were normalized to a peak acceleration value of 1 g before the response spectra were constructed. The EPA values determined by this procedure are presented in Table 4.1. Different values are shown for the two pairs of test structures because the characteristics of the table motions changed between the tests of Houses 1 and 2 and those applied to Houses 3 and 4, as was mentioned earlier. Also, it is clear that the differing frequency characteristics of these three earthquakes led to different EPA values for their normalized table motions.

For the purposes of this discussion, the variations of EPA shown in Table 4.1 are not significant, and a single average value of 0.82 g will be adopted. This means that a table motion having a peak acceleration of 1 g is assumed to have an EPA of 0.82 g. Correspondingly, a test having a peak measured table acceleration of 0.2 g is assumed to have an EPA of 0.16 g; or inversely, it is assumed that the maximum EPA of 0.2 g indicated by the ATC for Zone 2 is represented by a peak shaking table acceleration of 0.24 g.

TABLE 4.1

EPA VALUES FOR SHAKING TABLE MOTIONS NORMALIZED TO 1 g

| Earthquake | Taft | El Centro | Pacoima |
|---|---|---|---|
| Houses 1 and 2 | 0.90 g | 0.82 g | 0.79 g |
| Houses 3 and 4 | 0.91 g | 0.72 g | 0.78 g |

Assumed Average Value = 0.82 g.

#### 4.1.4 Test Structure Response Amplification

Although masonry houses are relatively rigid structures, they do have some flexibility and therefore exhibit some vibratory response mechanisms. The significant aspect of these mechanisms is that applied seismic accelerations are amplified by the structural response so that peak accelerations recorded on the structure are greater than the peak input acceleration. This amplification effect is represented in the definition of the EPA by the factor 2.5 by which the spectral acceleration is divided. In effect, the ATC has assumed an amplification factor of 2.5 to be appropriate for typical building structures.

The experimental data obtained in this investigation provide a direct measure of the amplification obtained by seismic excitations of single-story masonry houses. During these tests accelerations were recorded at various points on the test structures, and ratios of the peak values of these local accelerations to the peak applied table accelerations indicate the local structural response amplification. Values of such ratios were determined for each of the test structures; they are presented in Figs. 6.12 to 6.14 and 6.15 to 6.17 of reference 4 for Houses 1 and 2, respectively, and in reference 5, Figs. 5.4, 5.6, 5.8 for House 3 and Figs. 5.5, 5.7 and 5.9 for House 4. These figures demonstrate that the amplification factors vary considerably from point to point on the test structures, and with differing test conditions. For example, amplification values at the roof level average about 1.8 when the in-plane walls are load bearing, but reach values over 3 for the more intense excitations when the in-plane walls carry no dead load from the roof.

Seismic amplification factors are important in the design of structures to resist earthquakes because the seismic load induced in any part of a structure is given by the mass of that part multiplied by its local acceleration. In a single-story masonry house, the principal seismic force results from the mass of the roof structure. Hence, the seismic load to which a house is subjected is given by the product of the roof mass and its acceleration; or using the amplification factor it is given by this factor multiplied by the product of the roof mass and the table acceleration.

Although it is evident from the figures presented in references 4 and 5 that no single number can represent the response amplifications for all parts of the test structure under all conditions, the ATC value of 2.5 appears to be a reasonable compromise. For most of the test data, this number is conservative; that is, the observed amplification is less than 2.5 so seismic force estimates based on this factor will exceed the actual load. The number of cases where the observed amplification exceeds 2.5 are few, and apply to localized parts of the structure or to unusual test behavior; it is not likely that they represent any significant loading condition of the complete structure. Therefore, an amplification factor of 2.5 is proposed for estimating the seismic forces induced in the test structures by any given peak table acceleration.

#### 4.1.5 Summary of Conclusions Concerning Seismic Input

(1) For the purpose of making design recommendations, the UBC Seismic Zone 2 has been divided into two sub-zones, according to ATC estimates of the expected maximum effective peak acceleration:

Zone 2A: Range of EPA = less than 0.1 g

Zone 2B: Range of EPA = 0.1 g to 0.2 g.

(2) The EPA of the shaking table motions was found to be 0.82 times the peak shaking table accelerations. Therefore, the maximum peak table accelerations that can be associated with the seismic sub-zones are:

Zone 2A: Maximum peak table acceleration = 0.12 g.

Zone 2B: Maximum peak table acceleration = 0.24 g.

(3) A response amplification factor of 2.5 is recommended for single-story masonry houses, based on these test data. Therefore, seismic loads appropriate to the two sub-zones may be estimated as follows:

Zone 2A: Seismic load = Mass x 2.5 x 0.1 g = Weight x 0.25.

Zone 2B: Seismic load = Mass x 2.5 x 0.2 g = Weight x 0.5.

### 4.2 Recommended Design Criteria

#### 4.2.1 General Comments

As was noted in the Introduction of this paper, the principal purpose of this investigation was to determine the amount and type of reinforcing that should be provided in single-story masonry houses constructed in UBC Zone 2, and to recommend design provisions that will satisfy these requirements. The tentative design recommendations are presented in this section, and are based on the observed performance of the test houses when they were subjected to Zone 2 intensity shaking table motions. However, because two sub-zones having different earthquake intensities have been identified in Zone 2, it was necessary to formulate different recommendations for each sub-zone; these are set forth in different subsections.

Before discussing the tentative recommendations, it is important to recall that during the shaking table tests performed in this study the walls of the test structures were oriented either parallel to or perpendicular to the excitation axis. Different types of behavior were observed in the in-plane and out-of-plane walls, and these different response mechanisms indicate the need for different design requirements to provide adequate resistance in the in-plane and out-of-plane directions. However, it is evident that a real earthquake shakes a structure in both horizontal directions and therefore all walls of a prototype house must have both in-plane and out-of-plane resistance. Accordingly, although the tentative design requirements have been formulated separately with regard to in-plane and out-of-plane resistance, each wall must simultaneously satisfy both types of requirements.

### 4.2.2 Criteria for Zone 2A

It was noted earlier that the maximum effective peak acceleration to be expected in Zone 2A is 0.1 g, and that this EPA is provided by shaking table tests with a peak acceleration of 0.12 g. Also, in the interim to represent the additional damaging effect that the second horizontal component of a real earthquake might have on an unreinforced wall, it was recommended that the intensity of the single horizontal component shaking table motion be increased by 30 to 50 percent. Thus, a shaking table test with a peak acceleration of 0.16 g to 0.18 g is assumed to simulate the effects of a maximum Zone 2A earthquake on an unreinforced wall.

Review of the test structure observations summarized in Tables 3.1 to 3.4 shows that no damage of any type occurred in any wall of any test structures during tests not exceeding this peak acceleration. To be more explicit, no in-plane or out-of-plane wall, either reinforced or unreinforced, was cracked significantly during any test having a peak acceleration equal to or less than 0.18 g. Moreover, unreinforced walls that become cracked during more severe tests performed satisfactorily in subsequent tests with peak accelerations of 0.18 g or less.

Based on this observation, it is apparent that reinforcement is not required in Zone 2A, leading to the following recommended code provision:

> For Zone 2A, no reinforcement is required for earthquake resistance in single-story residential buildings of standard clay brick or concrete block construction provided the length of shear wall to roof load is similar to that included in the tests.

### 4.2.3 Basis of Recommendations for Zone 2B

For Zone 2B, the maximum expected EPA of 0.2 g is provided by a shaking table test with a peak acceleration of 0.24 g. For unreinforced walls, this intensity should be increased by 30 to 50 percent to account for the damaging effect of the second horizontal ground motion component; thus, a shaking table test with a peak acceleration of 0.31 g to 0.36 g should be used to judge the performance of unreinforced walls in Zone 2B conditions.

However, review of the response observations in Tables 3.1 to 3.4 reveals that the only unreinforced walls which withstood 0.36 g intensity shaking without damage were the in-plane walls of Houses 2 and 3. The unreinforced in-plane walls of Houses 1 and 4, and the unreinforced out-of-plane walls of all test structures, were damaged by tests with peak accelerations less than 0.36 g. Also, the performance of cracked unreinforced walls was unsatisfactory during tests with peak accelerations less than 0.36 g. Based on these observations, it is concluded that partial reinforcement is required in the walls of masonry houses built in Zone 2B; however, the amount and distribution of reinforcement that is needed must be considered further.

When the walls are partially reinforced, it is believed that little coupling exists between the in-plane and out-of-plane response mechanisms in a given wall. Accordingly, the intensity of the single-component shaking table motions is increased by only 20 percent to account for perpendicular component input effects. Thus, a shaking table motion with a peak acceleration of 0.29 g is considered to represent the maximum excitation for partially reinforced walls in Zone 2B.

Study of the diagrams in references 4 and 5 depicting the cracking behavior of the test structures, and of similar data not incorporated into the reports, reveals that no cracking damage developed in any of the partially reinforced walls, either in-plane or out-of-plane, during tests with peak accelerations of 0.29 g or less. In fact, no damage to the out-of-plane partially reinforced walls occurred in any test, even including the maximum peak acceleration tests of 0.63 g, 0.52 g, 0.48 g and 0.68 g applied to Houses 1 to 4, respectively. Hence, it is clear that the partial reinforcement provided in the test structures is more than sufficient to withstand the out-of-plane effects of any Zone 2 earthquake. Also, these results show that dowels are not needed to achieve the benefits of partial reinforcement in the out-of-plane direction.

On the other hand, some cracking was observed in the partially reinforced in-plane walls of all test structures. In most cases, this cracking was at the base of the piers and above the ends of the door and window lintels. It was associated with rigid-body rocking of the piers, and does not represent a serious damage condition; in fact, the residual cracks after the test were quite unimportant, and the behavior of these piers was considered satisfactory even during severe tests. The only instance of significant damage to partially reinforced walls occurred in House 4. Tests of this house were performed first in an unreinforced condition, and cracking developed in one in-plane wall adjacent to the window-opening. After the walls were reinforced and testing was continued, additional important cracking took place in this wall during a test with a peak acceleration of 0.54 g. However, this excitation greatly exceeded that to be expected in Zone 2B, and in fact this partially reinforced wall had withstood an earlier test with a peak acceleration of 0.32 g, hence, it may be concluded that partially reinforcing the test structures provided adequate in-plane resistance to the forces expected from a Zone 2B earthquake.

The final step in formulating the design recommendations for Zone 2B is to generalize the essential factors of the partial reinforcement included in the test structures. Mainly, these recommendations are presented in the form of minimum standards which ensure adequate resistance to out-of-plane forces. These standards also pertain in part to the in-plane resistance, and it is believed that adequate in-plane resistance can be achieved by prescription of such minimum standards. Construction details and standards appropriate for HUD Minimum Property Standards (MPS) will be developed by the Applied Technology Council based on these design recommendations.

To allow ATC maximum flexibility in formulating standards, the principal recommendations concerning in-plane resistance are presented here in the form of a design procedure. This procedure involves first estimating the lateral force that would be developed in the structure due to the maximum expected earthquake in Zone 2B. The acceleration inducing this force is given by the maximum EPA for Zone 2B (0.2 g) increased by the ATC amplification factor (2.5). Thus, the acceleration acting on the roof system is 0.5 g, and the seismic force is equal to half the roof weight.

The seismic force developed at the roof level must be resisted by shear stresses in the in-plane walls, and for the purpose of this analysis it is assumed that only panels in the walls that are at least six feet wide and without window penetrations will provide significant resistance. The effective shear stress capacity of such panels was established by analysis of their maximum shear stresses during the test series. The forces inducing these stresses were given by the product of half the roof mass and the amplified peak table acceleration, assuming that each in-plane wall resists half the lateral roof load. The effective stresses were then obtained by dividing these forces by the net cross-section area for the shear panel defined in the partially reinforced wall. Maximum shear stresses calculated for wall panels that performed satisfactorily during the tests were 34, 38, 40 and 39 psi in test structures 1 to 4, respectively. Because these did not necessarily determine the limit of satisfactory performance, it is probable that the effective strength is higher than these values; but to be conservative the value of 40 psi was selected as the allowed shear stress. It should be noted that the assumption of satisfactory performance with this magnitude of shear stress is based on the premise that the shear panel has vertical reinforcement at each end, thus enabling it to accommodate rigid-body rocking displacements. To account for the ductile response of the shear wall, reinforced as stated, it is recommended that the design force of 0.5 g times the roof mass be divided by 1.5. No claim is made for the rationality of this procedure nor for its suitability for inclusion in a housing code, but it is expected to lead to partially reinforced structures with resistance equivalent to those tested during this investigation.

4.2.4 Criteria for Zone 2B

Single-story houses of clay brick or concrete block construction that are built in Zone 2B must be partially reinforced. For the purpose of providing seismic resistance, partial reinforcement must meet the following requirements.

(1) The minimum reinforcing bar size is #3.

(2) Each exterior corner of the house must be reinforced by at least one bar, with a corresponding dowel extending from the foundation.

(3) For out-of-plane resistance:

(a) At least one bar is required in each masonry element or pier extending from floor to lintel or ceiling height.

(b) In walls longer than 8 ft, bars must be spaced at an average distance of 8 ft with a maximum spacing of 12 ft.

(4) For in-plane resistance:

(a) The in-plane resistance is assumed to be provided by shear panels. These are defined as a wall or portion of a wall extending from floor to lintel or ceiling height, at least 6 ft wide and without window penetrations.

(b) A vertical #4 bar is required at each edge of a shear panel. These bars must be dowelled to the footing. The reinforcing bar does not have to be designed to resist the forces on the panel.

(c) The total height of shear panels oriented along each building axis must be sufficient to resist a horizontal force equal to half the weight of the roof system divided by 1.5, with shear stresses on the net horizontal section not exceeding 40 psi.

(d) Placement of these panels along the length of a wall must be such that they prevent excessive torsional response in the house.

(e) Structural elements other than the main walls of a house (such as the front wall of a garage) shall be designed according to the provisions of the Uniform Building Code.

## 4.3 Recommendations for Further Research

The tests conducted during this investigation have provided for the first time quantitative as well as qualitative information concerning the resistance to earthquake motions of typical single-story masonry houses. It is believed that most aspects of the behavior of real houses during real earthquakes have been simulated adequately by the shaking table tests. However, this report has emphasized that one important feature of the real earthquake situation has not been included in these tests: biaxial horizontal excitation.

No shaking table in the United States has the capability of testing structures of the size of these house models with biaxial horizontal motions, so it is not possible to study this biaxial effect with complete similitude. However, a test will be performed with the University of California Earthquake Simulator that will demonstrate the most important features of the biaxial response. Specifically, a test structure similar to those described in this paper will be positioned on the shaking table with one of its axes rotated 30 to 45 degrees relative to the excitation axis. The single horizontal earthquake component will induce both in-plane and out-of-plane forces in all walls. Thus, it will be possible to evaluate directly the extent of coupling between the response mechanisms, and its effect in producing increased damage.

Because this factor represents the major area of uncertainty remaining in the seismic behavior of single-story masonry houses, final recommendations will await the testing of one model house oriented at a skew angle on the shaking table. The major new feature involved in the conduct of this test is the design and construction of a concrete base with sufficient rigidity to support the test structure in its skewed position. In this position, the footings at the base of the walls will extend beyond the boundary of the shaking table, and adequate support must be provided although the overhang will amount to only about 2 ft. This is only a routine design problem, however, because tests have been conducted previously on the shaking table involving overhanging test systems. In any case, the relatively limited test will greatly increase the value of the work already done and summarized in this report.

# 5. ACKNOWLEDGMENTS

The investigation described in this paper is part of a research program entitled "Laboratory Studies of the Seismic Behavior of Single-Story Masonry Buildings in Seismic Zone 2 of the U.S.A.", sponsored by the Department of Housing and Urban Development under Contract No. H2387. The authors would like to acknowledge the support they have received from HUD which made the study possible. The concrete block units were donated by the California Concrete Masonry Technical Committee, and are gratefully acknowledged. D.A. Sullivan Co. fabricated the masonry walls. Planning and implementation of the tests were carried out with the suggestions of the Applied Technology Council (ATC) Advisory Panel and subcontractor. The government technical representatives on the project were the late W.J. Werner and R. Morony. Additional coordination with the Department of HUD was provided by L. Chang and A. Gerich. W.L. Dickey, consulting engineer, was present during all experiments and offered numerous helpful suggestions and background information. Student assistants P. Buscovich and T. Nearn did the laboratory work, D. Steere implemented the data acquisition setup in addition to running the earthquake simulator, and the personnel of the Structural Research Laboratory headed by I. Van Asten contributed to every phase of the experimental work. Initial phases of the study were done under the supervision of Y. Omote and R. Hendrickson.

## REFERENCES

1. Clough, R.W., R.L. Mayes, Y. Omote, and S.W. Chen, "Summary of Findings of Field Trip to the Department of Housing and Urban Development Offices in Regions VII, IX, and X", December, 1976.

2. Omote, Y., R.L. Mayes, S.W. Chen and R.W. Clough, "A Literature Survey - Transverse Strength of Masonry Walls", Earthquake Engineering Research Center Report No. UCB/EERC-77/07, March, 1977.

3. Gülkan, P., R.L. Mayes, and R.W. Clough, "Strength of Timber Roof Connections Subjected to Cyclic Loads", Earthquake Engineering Research Center Report No. UCB/EERC-78/17, September, 1978.

4. Gülkan, P., R.L. Mayes, and R.W. Clough, "Shaking Table Study of Single-Story Masonry Houses - Volume 1: Test Structures 1 and 2", Earthquake Engineering Research Center Report No. UCB/EERC-79/23, September, 1979.

5. Gülkan, P., R.L. Mayes, and R.W. Clough, "Shaking Table Study of Single-Story Masonry Houses - Volume 2: Test Structures 3 and 4", Earthquake Engineering Research Center Report No. UCB/EERC-79/24, September, 1979.

6. "Tentative Provisions for a Development of Seismic Regulations for Buildings", Applied Technology Council Publication ATC3-06, (NSF Publication 78-8, NBS Special Publication 510) U.S. Government Printing Office, June, 1978.

TABLE 2.1

SUMMARY OF CONNECTION TEST RESULTS

| Type | Reference Code | Wall Material | Construction Phase | Test Phase | Type of Loading | Rated Strength Nailing kips | Rated Strength Bolts kips | Max. Resistance kips | Max. Average Resistance kips | Max. Av. Resistance / Min. Rated Strength | Initial Stiffness kip/in. | Ratio of Stiffness at 0.5 in. Disp. to Initial Stiffness | Failure Due to |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| C1 | C1-28-5/8 | Concrete Block | 1 | 1 | In-plane | 0.80 | 1.50 | 2.42 | 2.30 | 2.88 | 9.24 | 0.52 | Nails |
| | C1-28-1/2 | Concrete Block | 1 | 1 | In-plane | 0.80 | 1.10 | 3.55 | 2.99 | 3.74 | 12.12 | 0.46 | Nails |
| C2 | SC2-28-5/8 | Concrete Block | 1 | 2 | Sym. Out-of-plane | 1.38 | 1.50 | 4.02 | 3.63 | 2.63 | 15.88 | 0.42 | Nails |
| | SC2-28-5/8(2) | Concrete Block | 1 | 2 | Sym. Out-of-plane | 1.38 | 1.50 | 4.10 | 3.54 | 2.57 | 16.56 | 0.41 | Nails |
| C3 | C3-32-1/2 | Concrete Block | 1 | 1 | Eccentric Out-of-plane | 0.48 | 1.10 | 0.90 | 0.72 | 1.50 | 3.15 | 0.41 | Nails |
| | C3-52-3/8 | Concrete Block | 1 | 1 | Eccentric Out-of-plane | 0.48 | 0.65 | 0.84 | 0.69 | 1.44 | 2.15 | 0.65 | Nails |
| | C3-28-3/8-IP | Concrete Block | 1 | 2 | In-plane | 0.64 | 0.65 | 2.42 | 1.91 | 2.98 | 11.29 | 0.34 | Nails |
| | C3-28-5/8-IP | Concrete Block | 1 | 2 | In-plane | 0.64 | 1.10 | 2.10 | 1.75 | 2.73 | 12.58 | 0.25 | Nails |
| | C3-28-5/8-3/8-SP | Concrete Block | 1 | 2 | In-plane | 1.94 | 1.75 | 6.26 | 5.04 | 2.88 | 24.85 | 0.41 | Nails |
| C4 | C4-36-5/8 | Concrete Block | 1 | 1 | Eccentric Out-of-plane | 1.43 | — | 2.95 | 2.59 | 1.81 | 18.86 | 0.24 | Bolt Pullout |
| | C4-56-1/2 | Concrete Block | 1 | 1 | Eccentric Out-of-plane | 1.43 | — | 4.29 | 3.95 | 2.76 | 15.45 | 0.60 | Wall |
| | SC4-36-5/8-1450 | Concrete Block | 1 | 2 | Sym. Out-of-plane, Roof Dead Load | 1.43 | — | 6.26 | 5.51 | 3.85 | 24.28 | 0.48 | Bolt Pullout |
| | SC4-36-5/8 | Concrete Block | 2 | 2 | Sym. Out-of-plane | 1.43 | — | 1.46 | 1.24 | 0.87 | 10.29 | 0.14 | Bolt Pullout |
| | SC4B-36-5/8-1450 | Brick | 2 | 2 | Sym. Out-of-plane, Roof Dead Load | 1.43 | — | 3.85 | 3.35 | 2.34 | 26.54 | 0.25 | Wall & Bolt Pullout |
| C5 | C5-56-1/2 | Concrete Block | 1 | 1 | Eccentric Out-of-plane | 0.61 | — | 1.40 | 1.28 | 2.10 | 4.11 | 0.56 | Ledger |
| | C5-36-5/8 | Concrete Block | 1 | 1 | Eccentric Out-of-plane | 0.61 | — | 1.70 | 1.57 | 2.57 | 4.64 | 0.55 | Ledger |
| | SC5-36-5/8 | Concrete Block | 2 | 2 | Sym. Out-of-plane | 0.61 | — | 1.81 | 1.50 | 2.46 | 7.48 | 0.26 | Bolt Pullout |
| | SC5-36-1/2 | Concrete Block | 2 | 2 | Sym. Out-of-plane | 0.61 | — | 1.92 | 1.70 | 3.79 | 11.90 | 0.25 | Bolt Pullout |
| | SC5B-32-5/8 | Brick | 2 | 2 | Sym. Out-of-plane | 0.61 | — | 1.68 | 1.56 | 2.49 | 6.45 | 0.47 | Wall |

TABLE 3.1(a)

PERFORMANCE OF UNREINFORCED IN-PLANE WALLS OF HOUSE 1

| House and Tensile Stress of Subassemblage | Panel and/or Pier | State of Cracking | Test No. | Test Input | Deflection at Top of Wall in. | Net Shear Stress on Panel psi | Tensile Stress at Crack Location psi | Description of Crack | Comments |
|---|---|---|---|---|---|---|---|---|---|
| House 1 53 psi from diagonal compression test | W3 Non-Load Bearing | Before Cracking | 8 | T-0.19 g | 0.06 | 6 | 29 2'-8" from footing 42 at the footing | Along joint 2'-8" above the footing | During Test 10 slight crushing occurred at the ends of the cracked joint but no permanent displacement occurred. Slip along this joint was mechanically prevented after Test 10 |
| | | At Cracking | 9 | T-0.21 g | 0.14 | 9 | 42 | | |
| | | After Cracking | 10 | T-0.27 g | 0.40 | 9 | N/A | | |
| | W3 Non-Load Bearing (Slip prevented at cracked joint) | At Cracking | 11 | T-0.29 g | 0.30 | 11 | 64 | Along joint 8" above the footing | No significant permanent displacement occurred during Tests 12 and 13. Additional cracks formed at the 5th and 6th joints from the footing during Test 13. The wall was repaired and the roof rotated after Test 13. |
| | | After Cracking | 12 | E-0.14 g | 0.07 | 5 | N/A | | |
| | | | 13 | E-0.28 g | 0.28 | 11 | N/A | | |
| | W3 Load Bearing and Repaired | At Cracking | 27 | P-0.49 g | 0.11 | 22 | N/A | Along joint 8" above the footing | No permanent displacement occurred during Tests 28 and 29 although crushing was evident at the ends of the walls. Strength properties were not available for the repair material. |
| | | After Cracking | 28 | P-0.63 g | 0.70 | 29 | N/A | | |
| | | | 29 | E-0.59 g | 0.37 | 25 | N/A | | |

Note: T, E and P in column five refer to the Taft, El Centro and Pacoima earthquake motions, respectively.

TABLE 3.1(b)

PERFORMANCE OF UNREINFORCED OUT-OF-PLANE WALLS OF HOUSE 1

| House and Tensile Stress of Subassemblage | Panel and/or Pier | State of Cracking | Test No. | Test Input | Deflection at Top of Cracked Wall in. | Deflection at 2/3 Height of Cracked Wall in. | Tensile Stress at Crack Location psi | Description of Crack | Comments |
|---|---|---|---|---|---|---|---|---|---|
| House 1<br><br>53 psi from diagonal compression test | W4 8' Wide Load Bearing | Before Cracking | 11 | T-0.29 g | 0.40 | 0.26 | 20 at mid-height | At first joint above footing. | This crack is of no structural significance but is consistent with the assumption that the out-of-plane walls are hinged at their bases. |
| | | At Cracking | 13 | E-0.28 g | 0.28 | 0.18 | 25 at mid-height | | |
| | W4 8' Wide Non-Load Bearing | At Cracking | 19 | T-0.25 g | 0.07 | 0.07 | 11 | At 6th course, 24" from the top. | The maximum tensile stress prior to cracking was 13 psi in Test 13. W4 was repaired over the full height of the wall after Test 21. The top deflections are less in this series of tests as the footings were bolted after Test 13. |
| | | After Cracking | 20<br>21 | T-0.21 g<br>E-0.31 g | 0.13<br>0.70 | 0.25<br>1.96 | N/A | | |
| | W4 8' Wide Non-Load Bearing Repaired | Before Cracking | 24 | E-0.46 g | 0.09 | 0.06 | N/A | At 6th course, 24" from the top. | Because the strength properties of the repair material were not determined stresses corresponding are not applicable. |
| | | At Cracking | 27 | P-0.49 g | 0.17 | 0.11 | N/A | | |
| | | After Cracking | 28<br>29 | P-0.63 g<br>E-0.59 g | 0.75<br>0.66 | 0.51<br>1.10 | N/A | | |

Note: T, E and P in column five refer to the Taft, El Centro and Pacoima earthquake motions, respectively.

TABLE 3.2(a)

PERFORMANCE OF UNREINFORCED IN-PLANE WALLS OF HOUSE 2

| House and Tensile Stress of Subassemblage | Panel and/or Pier | State of Cracking | Test No. | Test Input | Deflection at Top of Wall in. | Net Shear Stress on Panel psi | Tensile Stress at Crack Location psi | Description of Crack | Comments |
|---|---|---|---|---|---|---|---|---|---|
| House 2<br><br>147 psi from diagonal compression test | 6' Pier of Panel A Load Bearing | Before Cracking | 19 | P-0.51 g | 0.05 | 35 | 220 Cant.<br>65 F.E. | No crack formed | Roof was rotated after this test. |
| | 6' Pier of Panel A Non-Load Bearing | Before Cracking | 30 | E-0.37 g | 0.07 | 29 | Max. Stress as for Test 19 | Diagonal from reentrant corner | There was a one inch permanent displacement in panel after Test 32 and no further tests were performed |
| | | At Cracking | 32 | P-0.52 g | 1.12 | 29 | 195 Cant.<br>69 F.E. | | |
| | 6' Pier of Panel B Load Bearing | At and After Cracking | 15<br>16<br>19 | E-0.45 g<br>T-0.40 g<br>P-0.51 g | R=0.04 L=0.13<br>R=0.07 L=0.21<br>R=0.28 L=0.40 | 26<br>26<br>35 | N/A | None identified | No cracks were identified but were assumed to exist. No permanent displacements occurred for the tests given. |
| | 6' Pier of Panel B Non-Load Bearing | | 26<br>30<br>32 | T-0.26 g<br>E-0.37 g<br>P-0.52 g | R=0.16 L=0.25<br>R=0.35 L=0.31<br>R=1.07 L=1.06 | 16<br>24<br>29 | N/A | None identified | No permanent displacements occurred for the test results given. |

Notes:

(1) R in column six refers to the deflection to the right in Panel B looking at the exterior face of the panel

(2) L refers to the deflection to the left

(3) Cant. is the tensile stress calculated on the assumption that the pier is a cantilever, whereas F.E. refers to the condition of rotational fixity at the top and bottom of the pier.

(4) T, E and P in column five refer to the Taft, El Centro and Pacoima earthquake motions, respectively.

TABLE 3.2(b)

PERFORMANCE OF UNREINFORCED OUT-OF-PLANE WALLS OF HOUSE 2

| House and Tensile Stress of Subassemblage | Panel and/or Pier | State of Cracking | Test No. | Test Input | Deflection at Top of Cracked Wall in. | Deflection at 2/3 Height of Cracked Wall in. | Tensile Stress at Crack Location psi | Description of Crack | Comments |
|---|---|---|---|---|---|---|---|---|---|
| House 2<br><br>147 psi from diagonal compression test | 4'-8" Pier of A1 Non-Load Bearing | Before Cracking | 13 | E-0.29 g | 0.09 | 0.04 | 18 | At 11th course from footing at the level of the bottom of the window | There were pre-existing cracks at the top of the small pier on the left of the window. The displacements of this pier were similar to those measured for the 4'-8" pier. Crack was not repaired. |
| | | At Cracking | 14 | E-0.33 g | 0.10 | 0.05 | 20 | | |
| | | After Cracking | 15 | E-0.45 g | 0.18 | 0.08 | N/A | | |
| | | | 16 | T-0.40 g | 0.26 | 0.12 | | | |
| | | | 19 | P-0.51 g | 0.45 | 0.20 | | | |
| | 4'-8" Pier of A1 Load Bearing | After Cracking as orig-inal was not re-paired | 26 | T-0.26 g | 0.24 | 0.16 | N/A | At 11th course from footing | The large deflections that occurred during Test 32 are attributed to the in-plane failure of wall A. |
| | | | 30 | E-0.37 g | 0.35 | 0.24 | | | |
| | | | 32 | P-0.52 g | 2.80 | 1.90 | | | |

Note: T, E and P in column five refer to the Taft, El Centro and Pacoima earthquake motions, respectively.

TABLE 3.3(a)

PERFORMANCE OF UNREINFORCED IN-PLANE WALLS OF HOUSE 3

| House and Tensile Stress of Subassemblage | Panel and/or Pier | State of Cracking | Test No. | Test Input | Deflection at Top of Cracked Wall in. | Net Shear Stress on Panel psi | Tensile Stress at Crack Location psi | Description of Crack | Comments |
|---|---|---|---|---|---|---|---|---|---|
| House 3<br>36 psi from diagonal compression test | 6' Pier of Panel A Load Bearing | Before Cracking | 18 | P-0.39 g | 0.03 | 29 | 145 Cant.<br>38 F.E. | Diagonal from re-entrant corner | Wall was repaired after Test 19 and the roof rotated. |
| | | At Cracking | 19 | P-0.49 g | 0.05 | 38 | 182 Cant.<br>54 F.E. | | |
| | 6' Pier of Panel A Non-Load Bearing | At Cracking | 26 | T-0.21 g | 0.20 | 8 | N/A | Same as before repair | Wall was strapped after Test 26 to prevent permanent slip and to permit the testing of other walls to continue. |
| | 6' Pier of Panel B Load Bearing | Before Cracking | 7 | T-0.22 g | R=0.04 L=0.02 | 116 | 113 | Along joint at foundation | The crack developed the full length of the joint during Test 12. No permanent displacements were observed for all tests with this roof orientation. |
| | | At Cracking | 8 | T-0.26 g | R=0.05 L=0.03 | 20 | 143 | | |
| | | After Cracking | 15<br>16<br>19 | E-0.45 g<br>T-0.37 g<br>P-0.48 g | R=0.19 L=0.05<br>R=0.21 L=0.07<br>R=0.47 L=0.15 | 29<br>33<br>38 | | | |
| | 6' Pier of Panel B Non-Load Bearing | After Cracking | 30<br><br>34<br>35<br><br>36 | T-0.24 g H<br>-0.17 g V<br>E-0.37 g H<br>E-0.33 g H<br>-0.22 g V<br>E-0.14 g H | R=0.41 L=0.22<br><br>R=0.81 L=0.32<br>R=1.04 L=0.36<br><br>R=0.40 L=0.22 | 24<br><br>29<br>24<br><br>12 | | Existed before this test sequence began | An unusual crack pattern developed at the top of the wall during Test 35. No permanent displacements were observed for all tests with the roof orientation. |

Notes:
(1) R in column six refers to the deflection to the right in Panel B looking at the exterior face of the panel
(2) L refers to the deflection to the left
(3) Cant. is the tensile stress calculated on the assumption that the pier is a cantilever, whereas F.E. refers to the condition of rotational fixity at the top and bottom of the pier
(4) T, E and P in column five refer to the Taft, El Centro and Pacoima earthquake motions, respectively.

TABLE 3.3(b)

PERFORMANCE OF UNREINFORCED OUT-OF-PLANE WALLS OF HOUSE 3

| House and Tensile Stress of Subassemblage | Panel and/or Pier | State of Cracking | Test No. | Test Input | Deflection at Top of Cracked Wall in. | Deflection at 2/3 Height of Cracked Wall in. | Tensile Stress at Crack Location psi | Description of Crack | Comments |
|---|---|---|---|---|---|---|---|---|---|
| House 3<br>36 psi from diagonal compression Test | For left pier of A1 2' wide Non-Load Bearing | Before Cracking | 5 | T-0.16 g | 0.008 | 0.008 | 4 | At the top of the door level | The pier was permanently displaced during Test 19 due to the failure and permanent offset of in-plane wall A. The wall was repaired and the roof rotated after Test 19. |
| | | At Cracking | 6 | T-0.18 g | 0.018 | 0.012 | 6 | | |
| | | After Cracking | 15<br>16<br>18<br>19 | E-0.45 g<br>T-0.37 g<br>P-0.39 g<br>P-0.49 g | 0.08<br>0.10<br>0.18<br>1.12 | 0.06<br>0.08<br>0.18<br>2.54 | N/A | | |
| | 18" Pier adjacent to window of A2 Non-Load Bearing | Before Cracking | 9 | T-0.21 g | 0.02 | 0.01 | 8 | At the top of the window level | The 4'-8" adjacent pier cracked during Test 18 causing equal displacements at the top and 2/3 heights. The comment above is applicable for Test 19. |
| | | At Cracking | 10 | T-0.29 g | 0.13 | 0.08 | 10 | | |
| | | After Cracking | 15<br>16<br>18<br>19 | E-0.45 g<br>E-0.37 g<br>P-0.39 g<br>P-0.49 g | 0.21<br>0.25<br>0.35<br>1.39 | 0.13<br>0.18<br>0.37<br>3.12 | N/A | | |
| | 4'-8" Pier of A1 Non-Load Bearing | Before Cracking | 15 | E-0.45 g | 0.24 | 0.16 | 41 | At the middle of the window 5'-4" from the footing | The large deflections associated with Test 19 did not cause any permanent displacements in this pier. All cracks were repaired and the roof rotated. |
| | | At Cracking | 18 | P-0.39 g | 0.36 | 0.54 | 47 | | |
| | | After Cracking | 19 | P-0.49 g | 1.17 | 3.39 | N/A | | |
| | A1 Load Bearing Repaired | Before Cracking as no cracks formed with this roof orientation | 30<br><br>34<br>35<br><br>36 | T-0.24 g H<br>0.17 g V<br>E-0.37 g<br>E-0.37 g H<br>0.22 g V<br>E-0.14 g | 0.43<br><br>0.91<br>1.20<br><br>0.40 | 0.29<br><br>0.62<br>0.81<br><br>0.27 | N/A | None | Note the increased displacement due to vertical motion of Test 35 compared to Test 34. |

Note: T, E and P in column five refer to the Taft, El Centro and Pacoima earthquake motions, respectively.

TABLE 3.4(a)

PERFORMANCE OF UNREINFORCED IN-PLANE WALLS OF HOUSE 4

| House and Tensile Stress of Subassembly | Panel and/or Pier | State of Cracking | Test No. | Test Input | Deflection at Top of Cracked Wall in. | Net Shear Stress on Panel psi | Tensile Stress at Crack Location psi | Description of Crack | Comments |
|---|---|---|---|---|---|---|---|---|---|
| House 4<br><br>36 psi from rupture test and 57 psi from diagonal compression test | 6' Pier of Panel A Load Bearing | Before Cracking | 2 | E-0.34 g H<br>0.20 g H | 0.19 | 23 | 145 Cant.<br>38 F.E. | Diagonal from reentrant corner | The wall was partially reinforced after this test, however the reinforcement was not dowelled. |
| | | At Cracking | 3 | T-0.29 g H<br>0.22 g V | 0.42 | 25 | 157 Cant.<br>42 F.E. | | |
| | 6' Pier of Panel B Load Bearing | At Cracking | 2 | E-0.34 g H<br>0.20 g V | 0.05 | 23 | 184 | Along first joint from footing | No permanent displacement was observed after Test 3 and the wall was then reinforced with dowels. |
| | | After Cracking | 3 | T-0.29 g H<br>0.22 g V | 0.15 | 25 | -- | | |
| | 6' Pier of Panel A Load Bearing Reinforced with no dowels. | At and after Cracking | 5<br>6<br><br>7<br>8 | P-0.32 g<br>E-0.54 g H<br>0.28 g V<br>P-0.47 g<br>E-0.68 g H<br>0.34 g V | 0.04<br>0.19<br><br>0.39<br>0.98 | 25<br>32<br><br>39<br>40 | | | Cracks developed from the bottom right hand corner of Wall A as a result of uplifting and rocking at this location. Cracking became more severe as this sequence of tests progressed. |

Notes: (1) Cant. is the tensile stress calculated on the assumption that the pier is a canteliver, whereas F.E. refers to the condition of rotational fixity at the top and bottom of the pier

(2) T, E and P in column five refer to the Taft, El Centro and Pacoima earthquake motions, respectively.

TABLE 3.4(b)

PERFORMANCE OF UNREINFORCED OUT-OF-PLANE WALLS OF HOUSE 4

| House and Tensile Stress of Subassemblage | Panel and/or Pier | State of Cracking | Test No. | Test Input | Deflection at Top of Cracked Wall in. | Deflection at 2/3 Height of Cracked Wall in. | Tensile Stress at Crack Location psi | Description of Crack | Comments |
|---|---|---|---|---|---|---|---|---|---|
| House 4<br>36 psi from rupture test and 57 psi from diagonal compression test | Pier 1 3'-4" wide with 2' dowell only. Flexible top support | After Cracking as shrinkage crack existed before testing | 2<br>3<br>5<br>6 | E-0.34 g H 0.21 g V<br>T-0.29 g H 0.22 g V<br>P-0.32 g<br>E-0.54 g H 0.28 g V | 0.27<br>0.41<br>0.37<br>1.23 | 0.15<br>0.37<br>0.36<br>5.3 | N/A | Shrinkage at 60% wall height before testing commenced. | After Test 6, 12"x12" planks were strapped to piers 1 and 3 to prevent their collapse. Note that the pier did not collapse with the 5" displacement at wall height. |
| | Pier 3 3'-4" wide with flexible top support | At Cracking | 2 | E-0.34 g H 0.21 g V | 0.36 | 0.27 | 23 | At 70% wall height | After Test 6 pier 3 was planked as discussed above. |
| | | After Cracking | 3<br>5<br>6 | T-0.29 g H 0.22 g V<br>P-0.32 g<br>E-0.54 g H 0.28 g V | 0.50<br>0.39<br>1.23 | 0.57<br>0.58<br>3.5 | N/A | | |
| | Far left 2' wide Pier of A1 Non-Load Bearing | At Cracking | 2 | E-0.31 g H 0.21 g V | 0.08 | 0.05 | 20 | At top of the door level | The pier was reinforced after Test 3. |
| | | After Cracking | 3 | T-0.29 g H 0.22 g V | 0.15 | 0.14 | N/A | | |
| | 4'-8" pier of A1 Non-Load Bearing | At Cracking | 3 | T-0.29 g H 0.22 g V | 0.51 | 0.42 | 15 | At top of window level | The pier was reinforced after Test 3. |
| | 2' Pier of B1 Non-Load Bearing | At Cracking | 2 | E-0.34 g H 0.21 g V | 0.19 | 0.14 | 27 | At 70% wall height | The pier was reinforced after Test 3. |
| | | After Cracking | 3 | T-0.29 g H 0.22 g V | 0.45 | 0.58 | N/A | | |
| | 5'-4" Pier of B1 Non-Load Bearing | At Cracking | 2 | E-0.34 g H 0.21 g V | 0.13 | 0.07 | 15 | At top of door level | The pier was reinforced after Test 3. |
| | | After Cracking | 3 | T-0.29 g H 0.22 g V | 0.34 | 0.19 | N/A | | |

Note: T, E and P in column five refer to the Taft, El Centro and Pacoima earthquake motions, respectively.

TABLE 3.5

MAXIMUM CONNECTION FORCES (in pounds)
FROM SHAKING TABLE AND CYCLIC TESTS

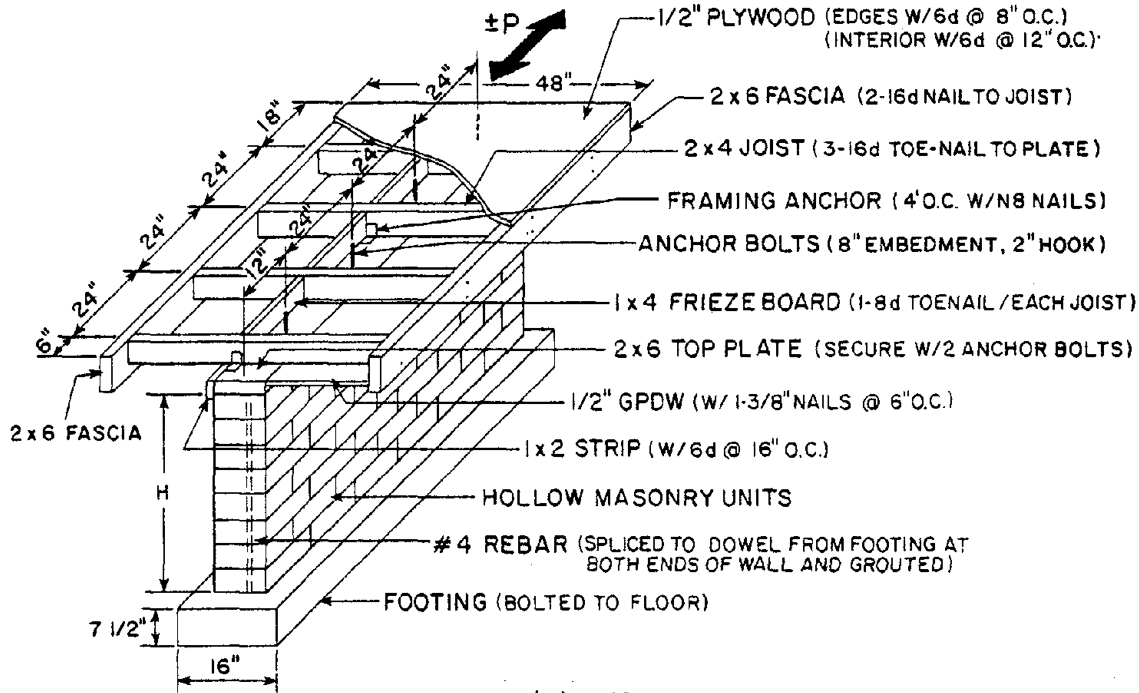
| Type of Connection | Connection Designation | Shaking Table | | | Cyclic Tests | | |
|---|---|---|---|---|---|---|---|
| | | per foot | per bolt | per truss | per foot | per bolt | per truss |
| In-Plane Load Bearing | C1 | 620 | 1985 | 1105 | 290<br>370 | 1150<br>1500 | 500<br>740 |
| In-Plane Non-Load Bearing | C3-IP | 475 | 1515 | N/A | 320 | 1260 | N/A |
| Out-of-Plane Load Bearing | C2 | 65 | 205 | 115 | 450 | 1820 | 920 |
| Out-of-Plane Non-Load Bearing | C3 | 85 | 270 | N/A | 45 | 350 | N/A |

TABLE 3.6

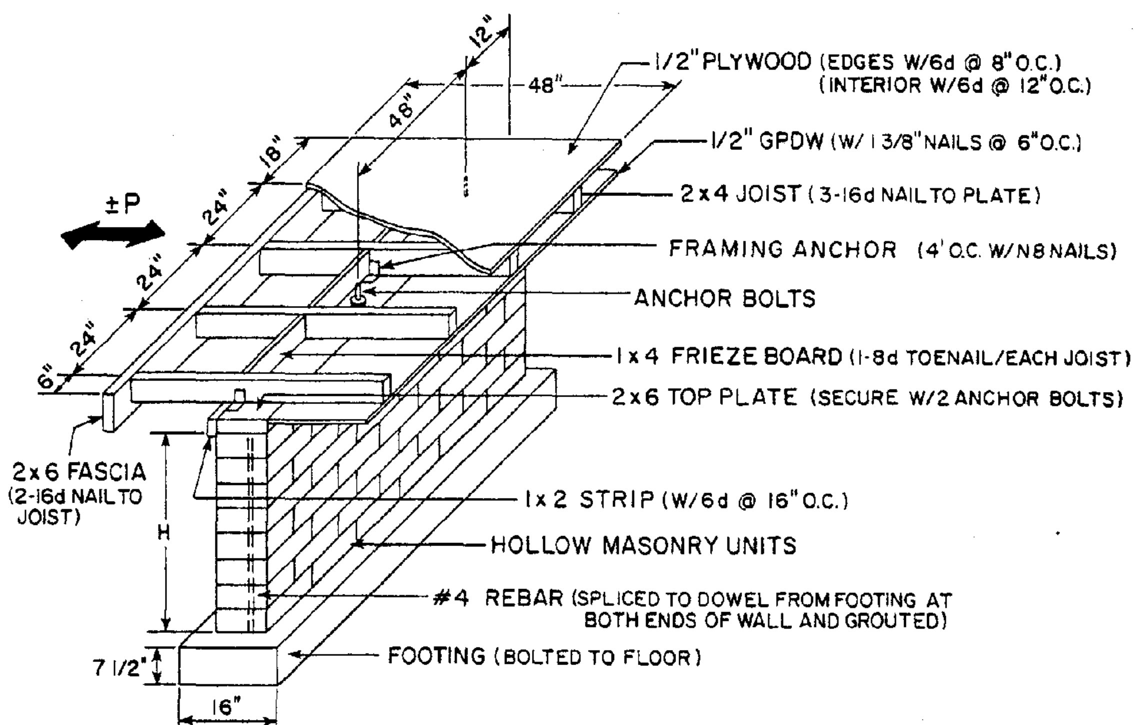
COMPARISON OF CONNECTION TEST RESULTS WITH UBC ALLOWABLE VALUES

| Type of Connection | Capacities, lb/ft | | Shaking Table / Cyclic Tests | Cyclic Tests / Code Allowable | Shaking Table / Code Allowable |
|---|---|---|---|---|---|
| | Shaking Table | Cyclic Tests | | | |
| In-Plane Load Bearing | 620 | 330 | 1.88 | 2.64 | 4.96 |
| In-Plane Non-Load Bearing | 475 | 320 | 1.48 | 2.86 | 4.23 |
| Out-of-Plane Load Bearing | 65 | 450 | 0.14 | 2.60 | 0.54 |
| Out-of-Plane Non-Load Bearing | 85 | 45 | 1.91 | 1.47 | 2.81 |

NOTE: The code allowable strength is a function of the nailing strength.

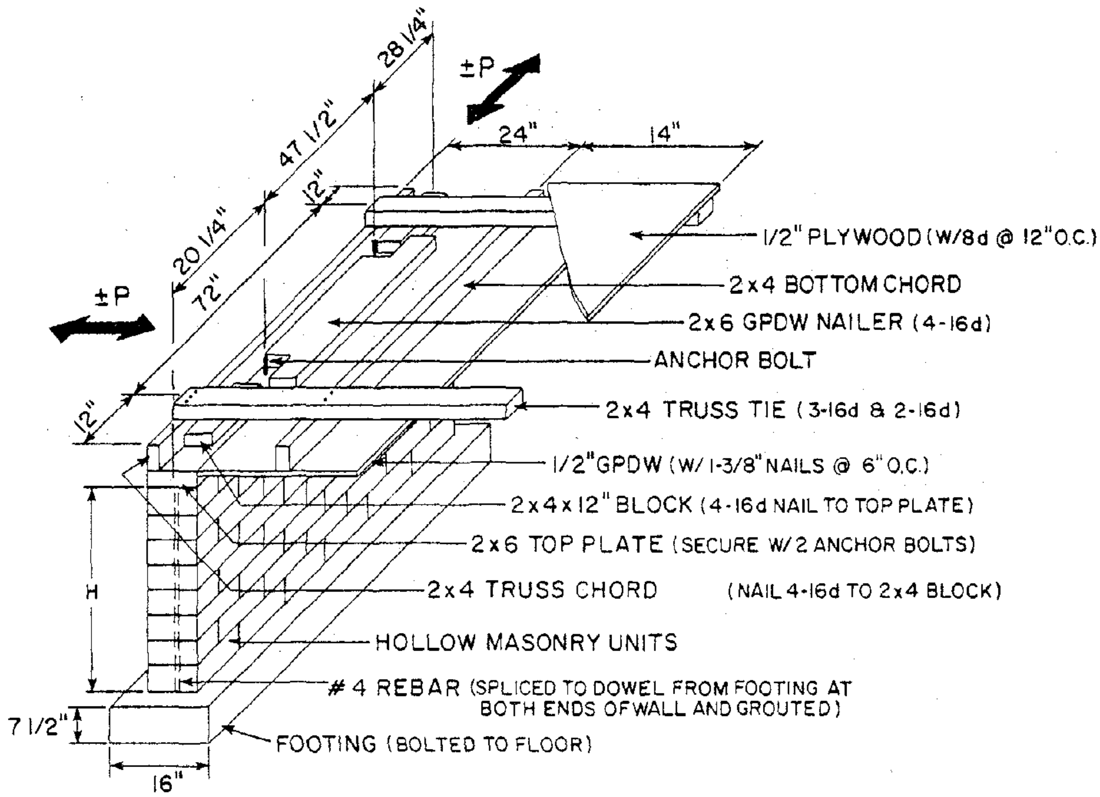



(a) C1

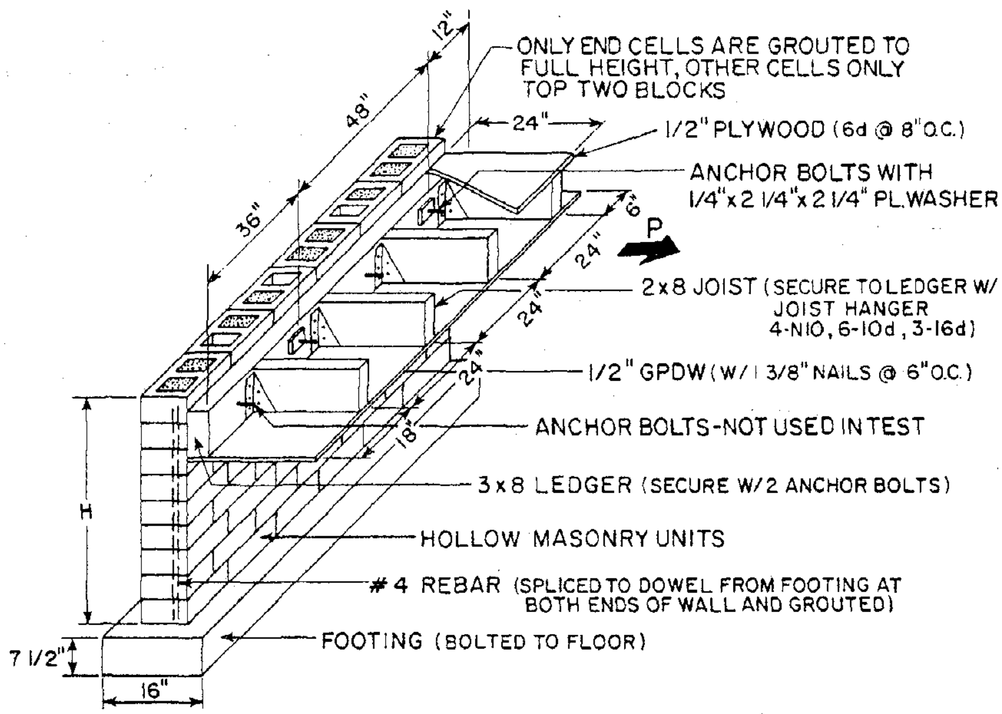



(b) C2

FIGURE 2.1 CONNECTION TEST SPECIMENS

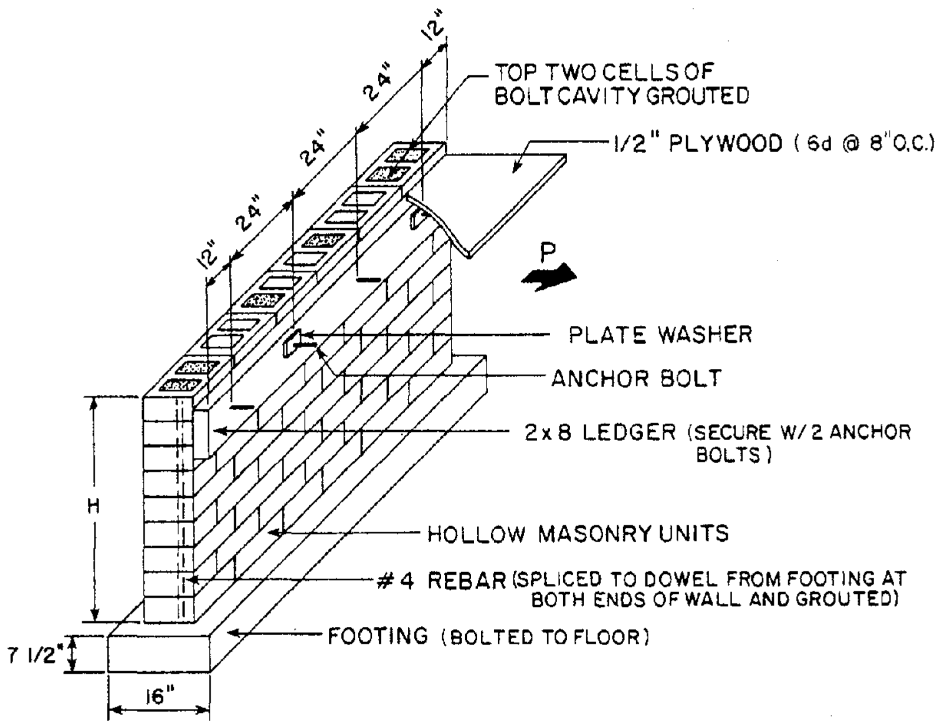



(c) C3




(d) C4

FIGURE 2.1 (CONT.) CONNECTION TEST SPECIMENS




(e) C5

FIGURE 2.1 (CONT.) CONNECTION TEST SPECIMENS

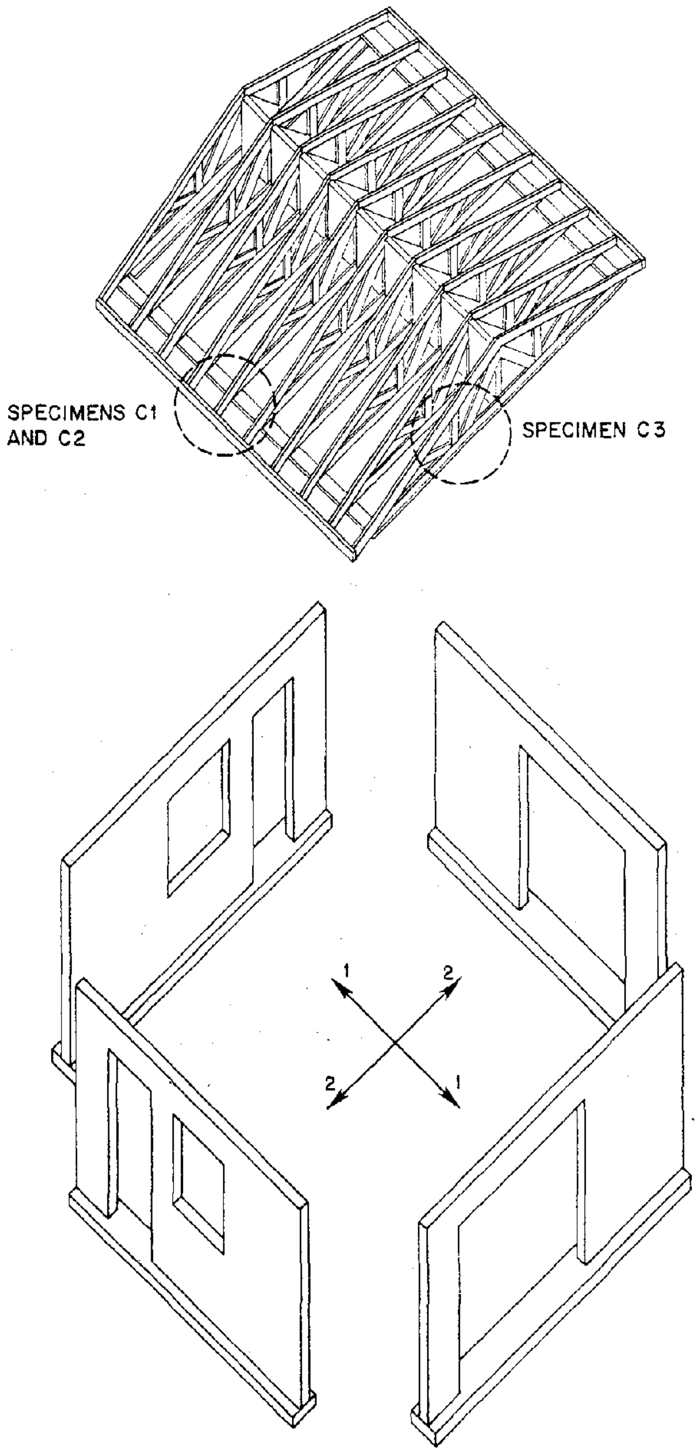
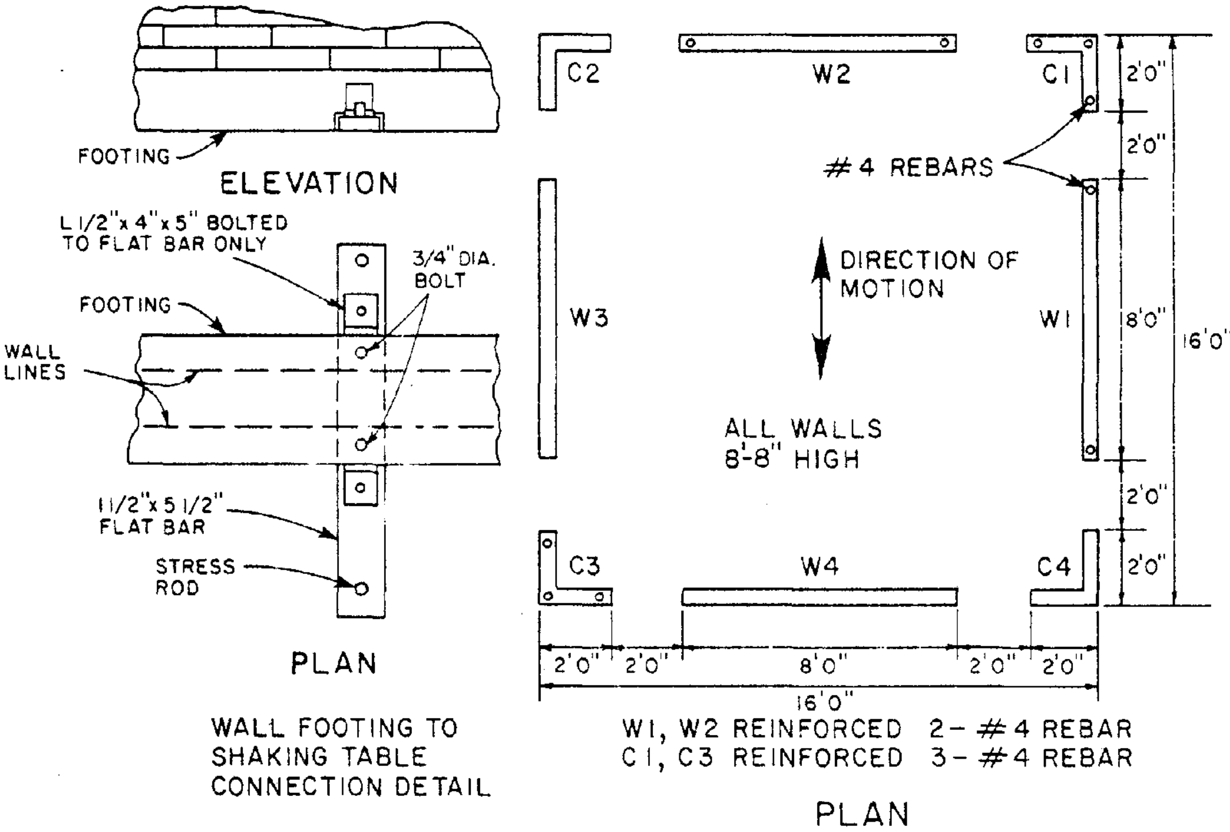



FIGURE 2.2 ROOF STRUCTURE FOR A MASONRY HOUSE



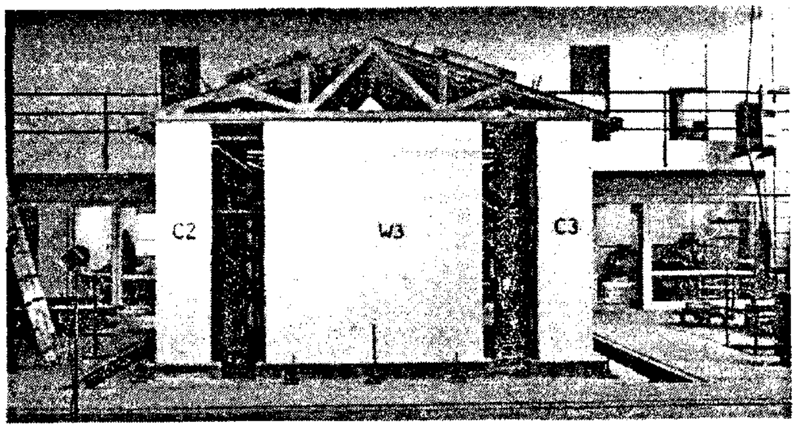




FIGURE 2.3 HOUSE 1

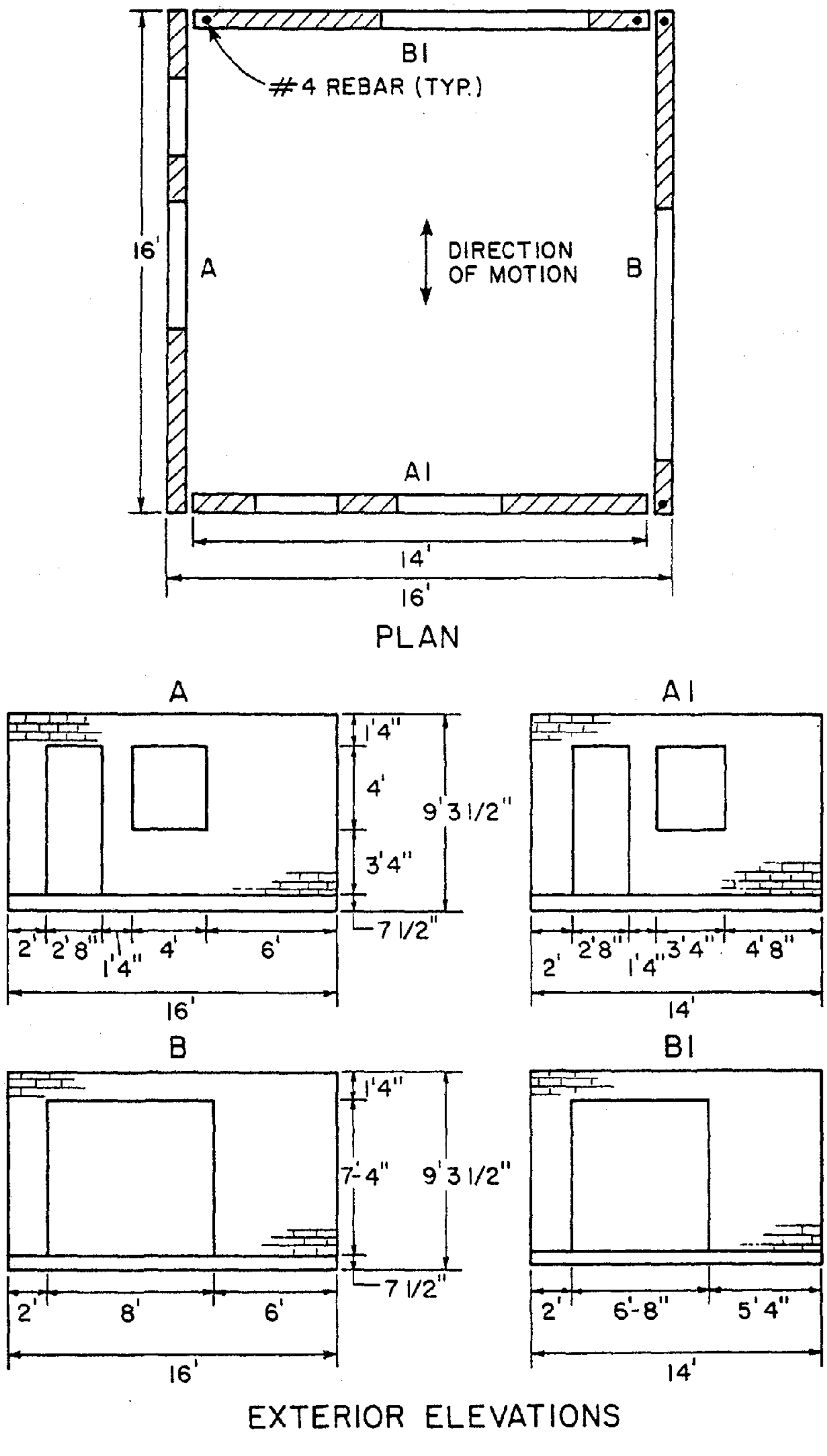



FIGURE 2.4 HOUSE 2

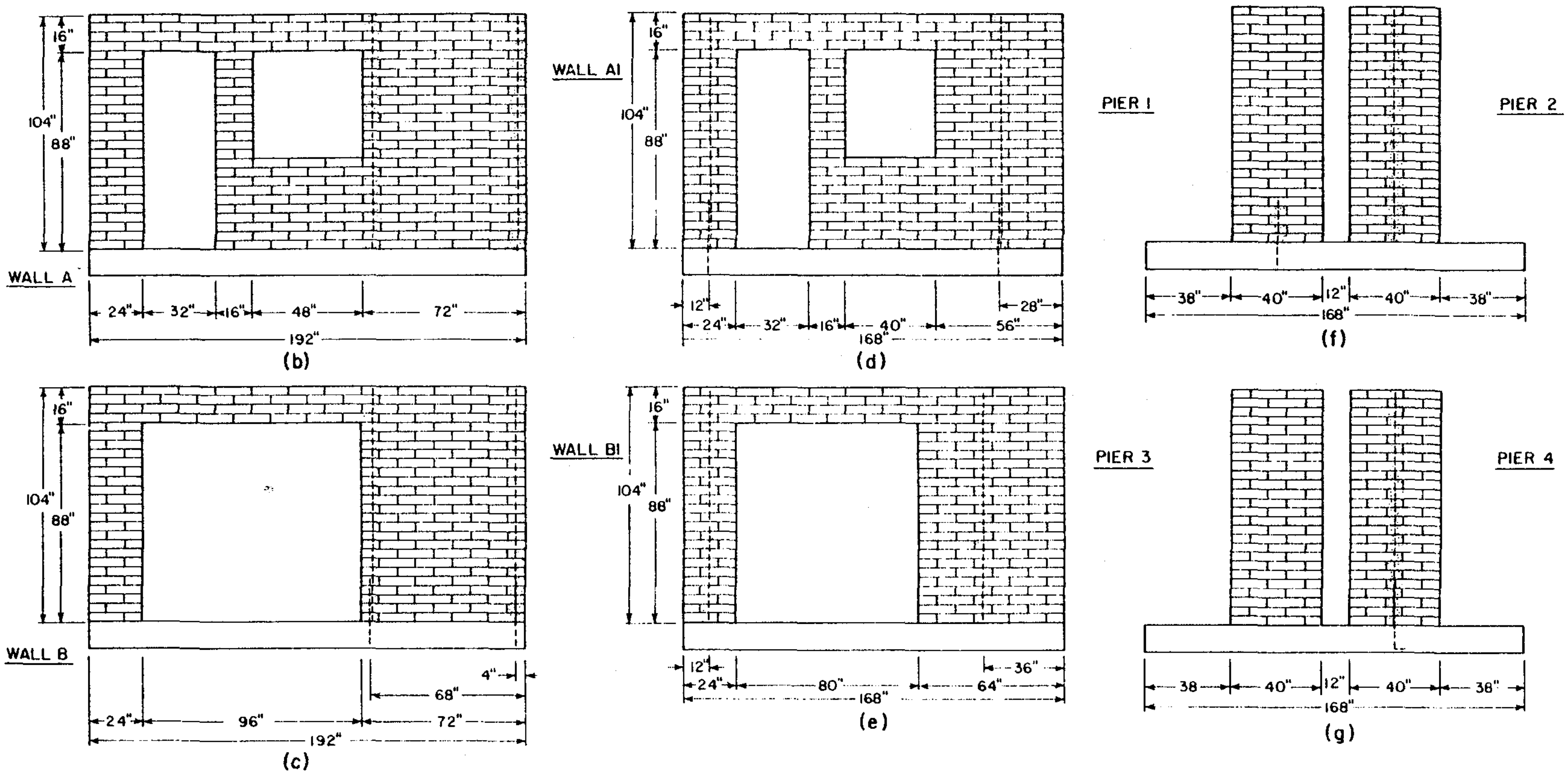



FIGURE 2.5 HOUSE 4

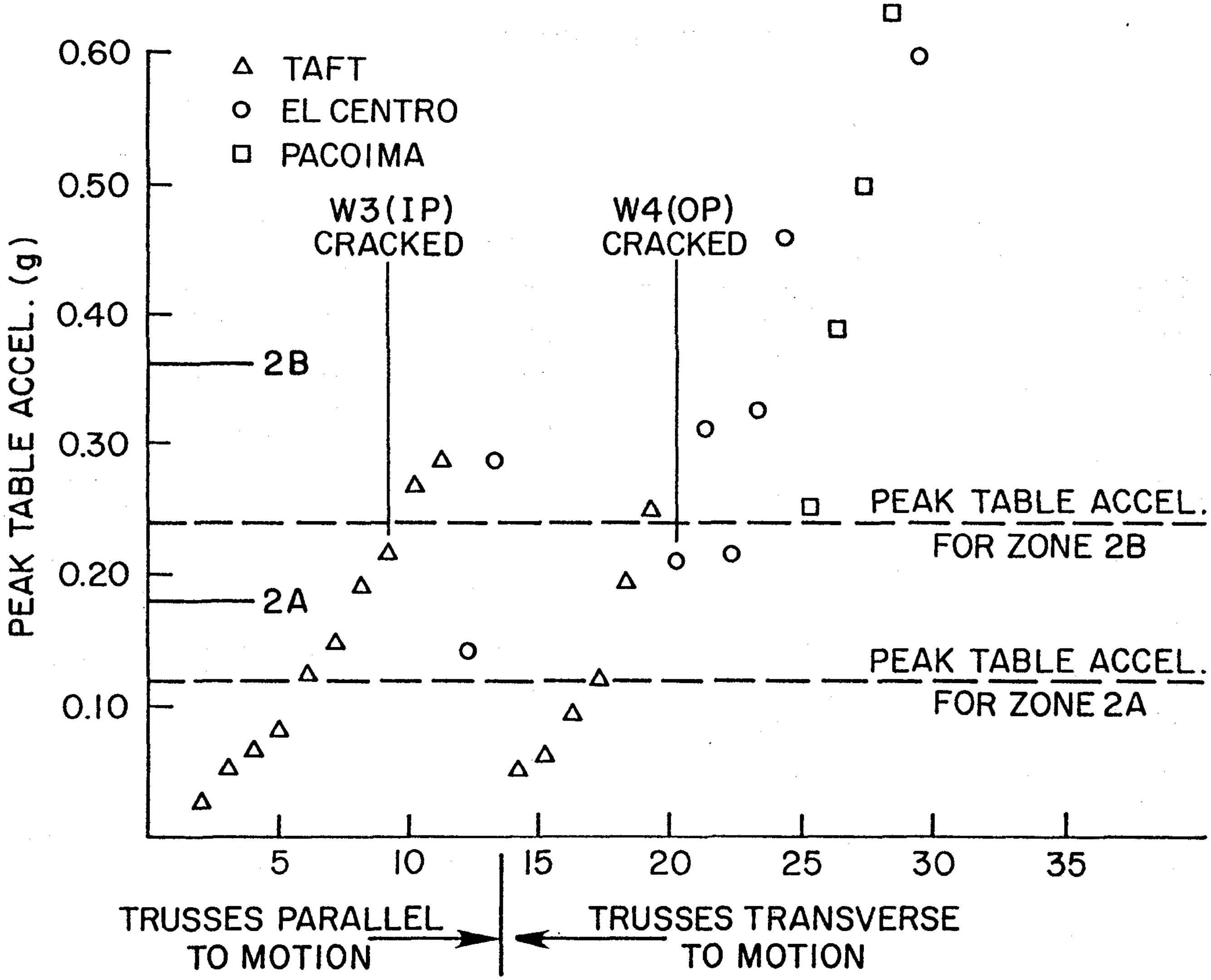



FIGURE 2.6 SEQUENCE OF TESTS FOR HOUSE 1

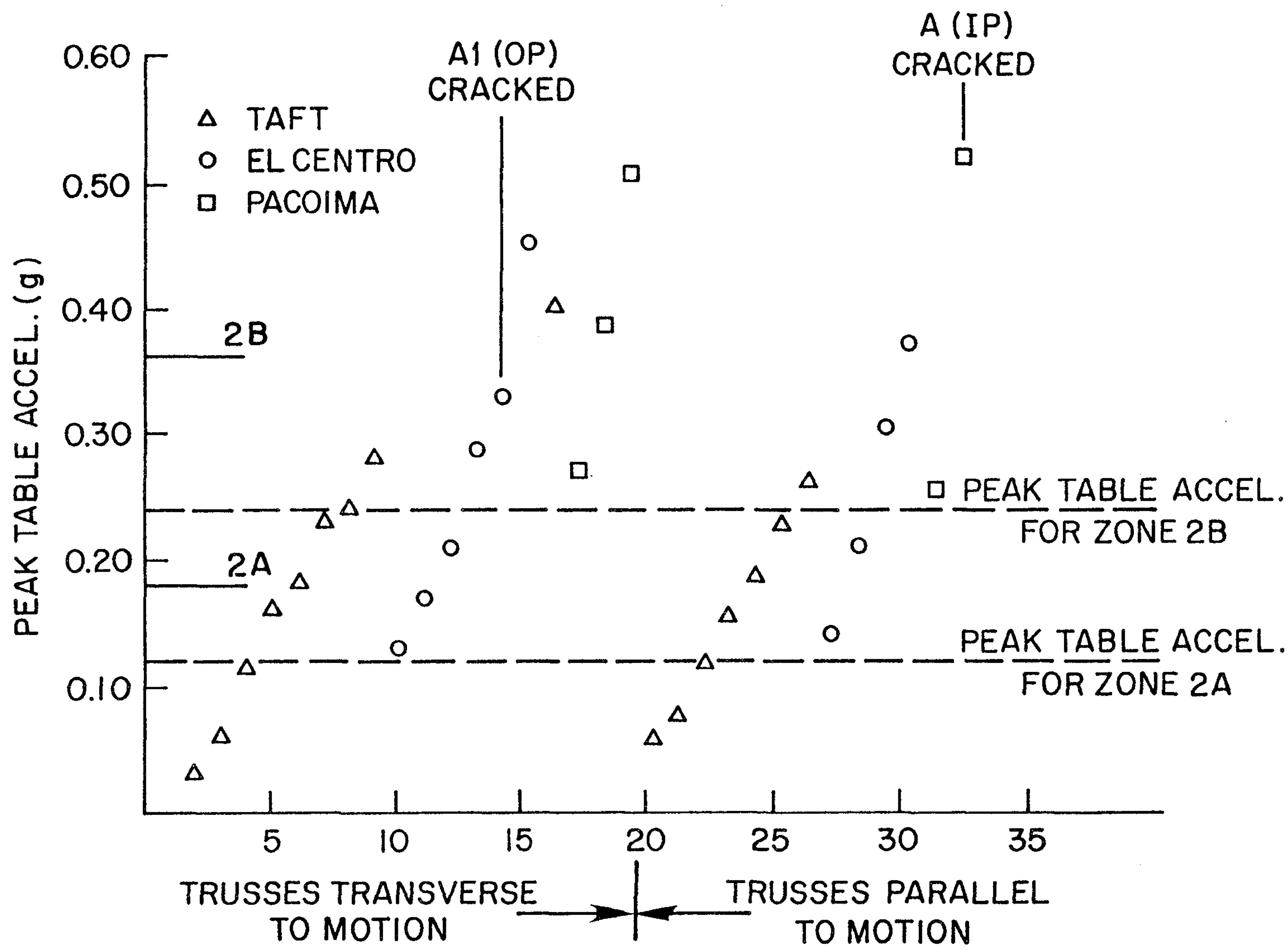



FIGURE 2.7 SEQUENCE OF TESTS FOR HOUSE 2

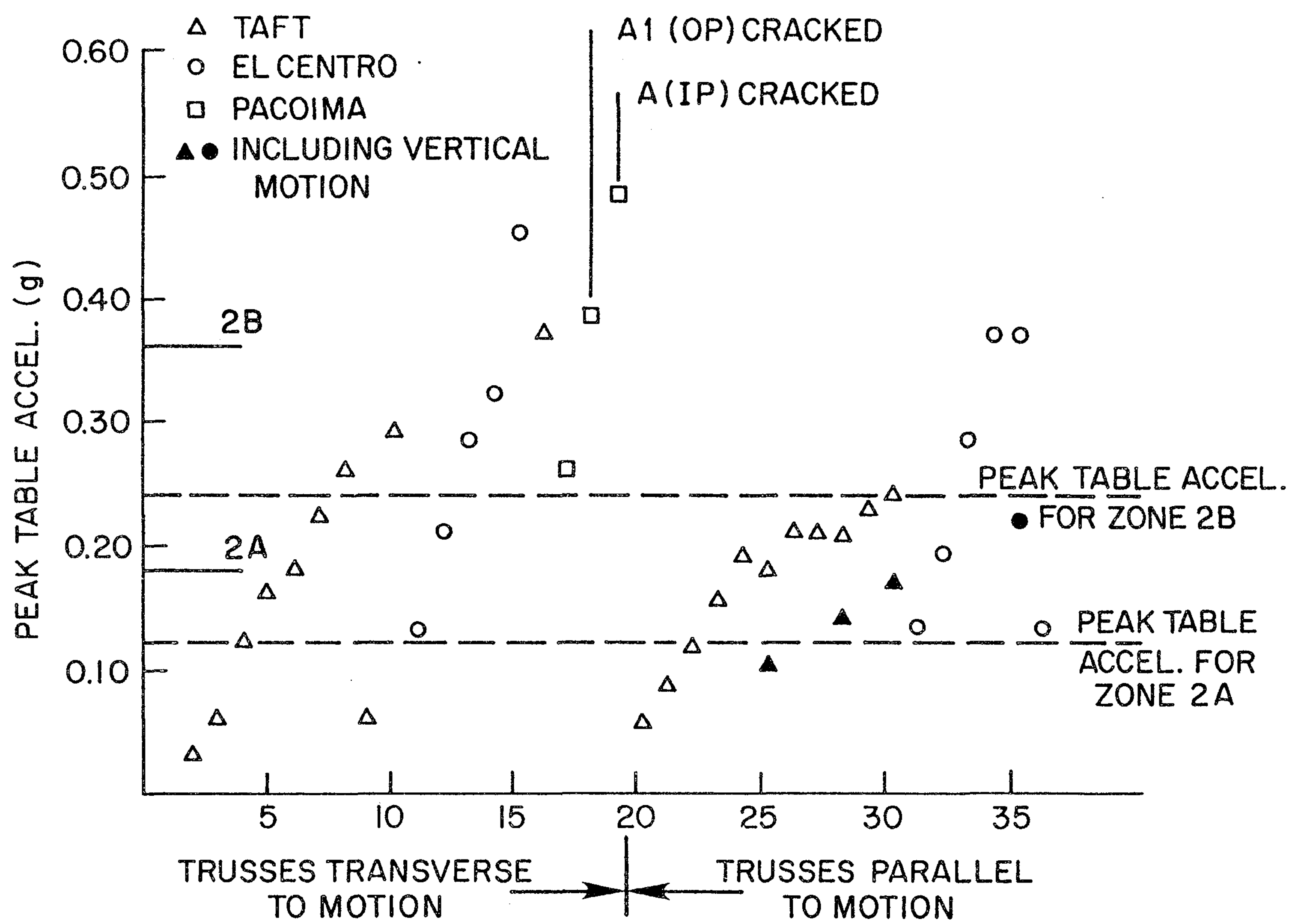



FIGURE 2.8 SEQUENCE OF TESTS FOR HOUSE 3

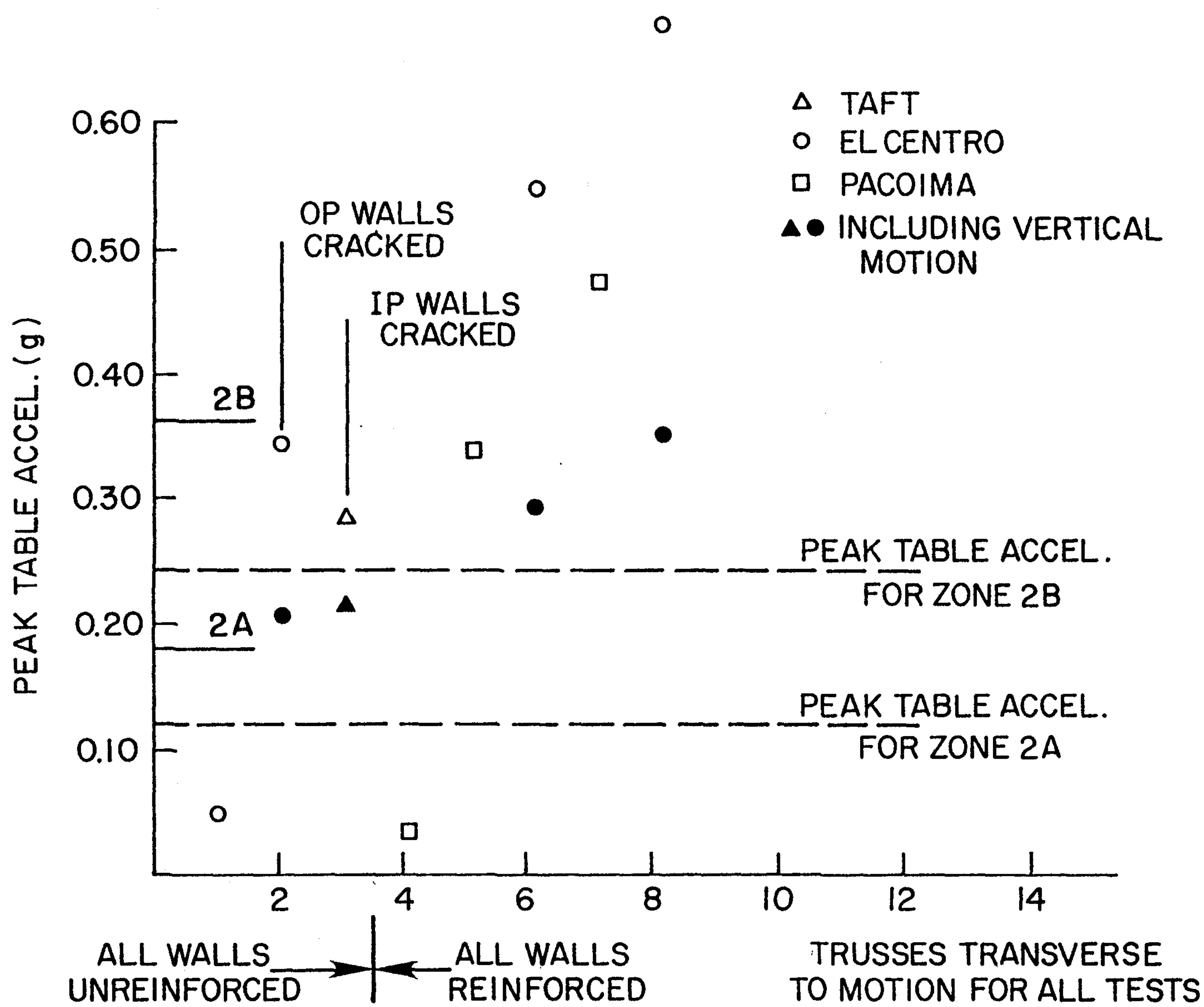



FIGURE 2.9 SEQUENCE OF TESTS FOR HOUSE 4

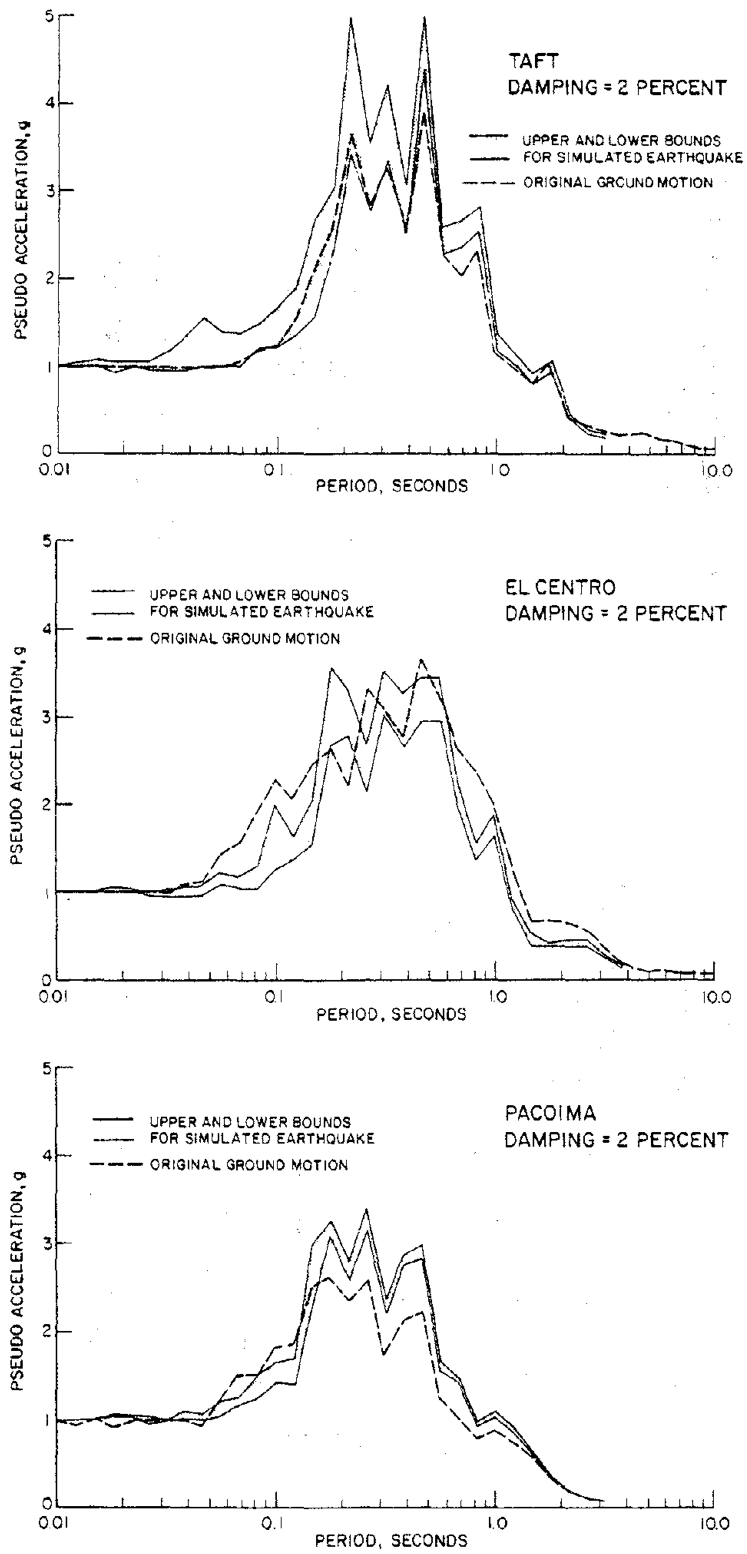



FIGURE 3.1 RESPONSE SPECTRA, TYPICAL OF MOTIONS APPLIED TO HOUSES 1 AND 2

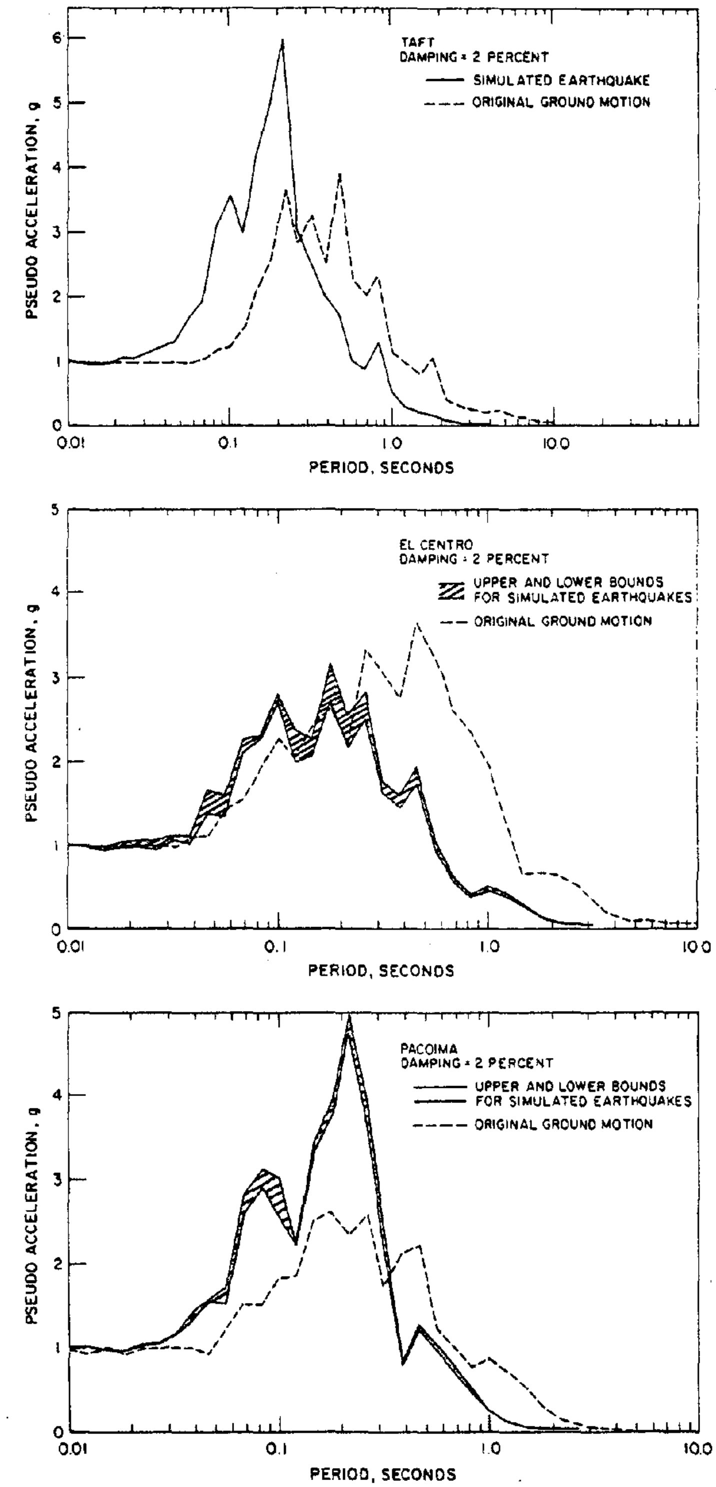



FIGURE 3.2 RESPONSE SPECTRA, TYPICAL OF MOTIONS APPLIED TO HOUSES 3 AND 4

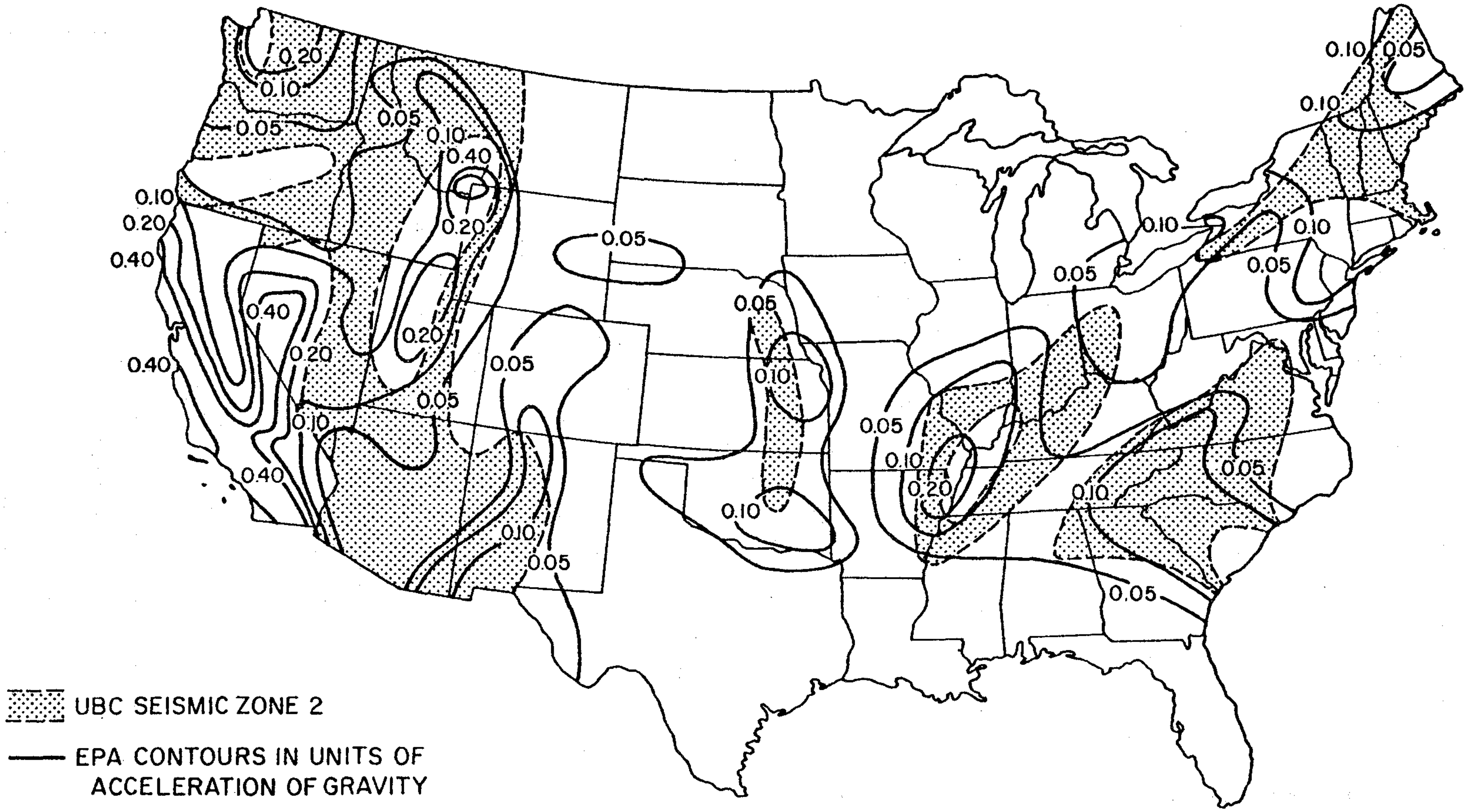



FIGURE 4.1 COMPARISON OF UBC SEISMIC ZONE 2 AND THE CONTOUR MAP OF EFFECTIVE PEAK ACCELERATIONS

# PLANAR MECHANICS OF FULLY GROUTED CONCRETE MASONRY[1]

By Nunn, R.O.[2]

ABSTRACT: This paper presents a continuum model for the planar behavior of fully grouted concrete masonry. Data was taken from 5-foot square panels cut at an angle relative to the joint system, which were tested under direct biaxial stresses, with one edge in tension, and the other in compression or stress-free.

The form of the elastic law describing linear behavior is determined, and the moduli are evaluated. These moduli show substantial anisotropy in stiffness.

The significance of prism configuration and end conditions are discussed. A law describing the dependence of uniaxial tensile strength on direction is presented, and is incorporated into a law for biaxial strength for the special case of principal stresses of opposite sign. A statistical analysis of the biaxial data gives variations in strength that can be expected.

A modified plasticity model describing the behavior of reinforced masonry following initial fracture is presented. The existence of a loading surface is illustrated, and a law is given for the change of this surface based on tensile strain. Tensile strain is also employed in a law describing stiffness degradation. Normality of the plastic strain rate is shown to hold at several points on the loading surface, and a stress rate - total strain rate matrix for plastic behavior is derived.

---

[1]Research was sponsored by the National Science Foundation under Grant #PFR 78-16581.

[2]Graduate Student, Dept. of Applied Mech. & Engr. Sci., University of California, San Diego, La Jolla, CA 92093.

# PLANAR MECHANICS OF FULLY GROUTED CONCRETE MASONRY[1]

by

R. O. Nunn[2]

## INTRODUCTION

Masonry is a form of construction that has been practiced for thousands of years. While some traditional materials, such as stone, are rarely used today, concrete block now provides a rapid and economical method of producing structures of one to ten or more stories. The wide variety of brick and block available makes possible buildings of exceptional color and texture.

Though masonry is widely used as a construction material, rather little is known of its properties. Research into its behavior has lagged behind research in other materials. Concrete, for example, has been the subject of careful study by many investigators, and the American Concrete Institute publishes a journal devoted to results of their work. No such journal has been available for the publication of masonry research results.

The main danger in this lack of knowledge lies in the response of masonry structures to seismic loading. Enough information has been accumulated through analysis and simple experiments to design structures that are quite safe under static conditions. But dynamic loading can produce markedly different stresses, including tensile stresses. While masonry is generally very strong in compression, most types are rather brittle, and can withstand much less tensile stress.

A result of this brittle behavior has been substantial damage to many masonry structures in areas that have experienced strong earthquakes [1]. But in the same area where some masonry structures have been destroyed, others have survived undamaged. It appears, therefore, that through analysis and a knowledge of material behavior, it should be possible to reduce the seismic hazard.

In an effort to provide this knowledge, an extensive research program was undertaken at the University of California, San Diego. The program consists of experimental, analytical, and numerical investigations of the behavior of masonry material and connections. The subject of this paper is an analytical representation of selected experimental results on material behavior.

---

[1]Research was sponsored by the National Science Foundation under Grant #PFR 78-16581.

[2]Graduate Student, Dept. of Applied Mech. & Engr. Sci., University of California, San Diego, La Jolla, CA 92093.

# 1. OBJECTIVES AND METHODOLOGY

## 1.1 Scope of Study

In conducting an investigation into the behavior of a material as complex and variable as masonry, it is necessary to limit the scope of the study. Some of the material variables to be considered are block type, mortar type and amount, grout type and amount, method of compaction, and amount of reinforcing. (These terms are illustrated in Fig. 1.1). Instead of looking at the effects of these variables, attention was restricted to a few combinations, which were selected on the basis of their widespread use in construction. The intent of this restriction was to allow a thorough study of the masonry types selected.

The results presented in this paper are further restricted. They are based on tests of a single combination of components, and all specimens were fully grouted. While partially grouted masonry is widely used and deserves study, fully grouted masonry is of importance in seismically active areas, and such specimens are easier to handle because of their greater strength. Both unreinforced and reinforced masonry are discussed.

In addition to selecting a material type, it is necessary to decide what properties will be investigated. In the UCSD program, only in-plane loads were considered. Though damaged structures sometimes exhibit out-of-plane failures, walls that suffer such displacements probably do so only after substantial damage has occurred. Thus, knowledge of planar behavior should enable one to predict the response of a structure until it is close to collapse.

## 1.2 Method of Investigation

In studying this planar behavior, two approaches are possible. One is to test structural elements, for example piers, then to combine these results to predict the behavior of a structure. Such a program exists at the University of California, Berkeley [2]. But the number of possible elements and their variations can necessitate a large number of tests. The other approach, employed in this program, is that of continuum mechanics. If one can determine the properties of the material, then through analysis one can predict the behavior of the structural elements, and a complete structure. The structural element tests can serve as an important check on this process.

The design of masonry buildings is presently based primarily on the uniaxial compressive strength, which is determined by prism tests (see Section 3.1). Other tests that have been conducted include beam tests and diagonal compression tests. These other tests cannot, however, determine a material behavior law relating stress to strain. While the state of stress for such loading may be known for an isotropic linear material, the solution does not hold for masonry, so the stress and strain are unknown.

In order to find such a law, one must be able to apply an arbitrary state of uniform planar stress. Such a stress state consists of two direct stresses, plus a shear stress. Direct stresses are fairly easy to apply, but the application of shear stress is quite difficult. To circumvent this difficulty, the following result of tensor analysis was taken advantage of: A general state of planar stress is equivalent to two direct stresses (the principal stresses) with zero shear stress, at some angle relative to the coordinate system of the original stresses [3]. This result is expressed quantitatively by Mohr's circle.

Since these direct stresses are to be applied to the edges of the specimen, the edges must be aligned with the principal stress directions. Thus, the masonry joints will, in general, not be aligned with the edges. Once a stress state is selected, the principal directions, and hence the specimen orientation, are determined. A disadvantage of this procedure is that for a particular specimen, the directions of the principal stresses are fixed.

The requirement that the stress be uniform applies, of course, only macroscopically. Treating a material as a continuum requires that variations in stress at the microscale be averaged out over several micro-dimensions. For masonry the micro-dimension is a block length, 16 inches. For this program, therefore, the size of the square specimen was chosen to be 64 inches.

## 1.3 Specimen Construction

The first step in producing these oblique lay-up specimens was the construction of 8-foot square walls, which were built by professional masons using conventional field practice. Grout was poured in 8-foot lifts and compacted by puddling. The materials used in construction are described in Table 1.1. After curing, a wall was faced on one side with a layer of hydrocal, then placed horizontally on the hydrocal. A dynamically balanced, high speed circular saw then cut out the specimen.

Reinforced specimens had two number five bars in each direction, for a steel to total area ratio of 0.13 percent, which is typical of masonry construction. These bars were carefully positioned to end just short of the specimen edge. After the specimen was cut, the grout was chipped away from the end of the bar, and a steel plate was welded to the bar and bonded in place with epoxy.

The walls for this program were fabricated in eight batches. Though materials and construction were the same for all batches, significant variations in strength between batches were observed. One batch that cured during a particularly wet period was considerably stronger than the others. Thus, when calculating average properties, it is necessary to restrict attention to a single batch.

## 1.4 Test Procedures

For the purpose of testing these specimens, a test frame capable of applying arbitrary biaxial stress was constructed. This frame, shown in Fig. 1.2, was designed to withstand the enormous loads required to fail a specimen, with only small deflections. The frame deflection must be small in order to control displacements when loads drop suddenly at first fracture. A smaller frame was built for testing uniaxial specimens.

The biaxial frame actually consists of two parallel frames, between which are attached hydraulic actuators that deliver the load. The actuators are arranged four to a side, can deliver up to 120,000 lbs each in either direction, and are controlled by a mini-computer. They are attached to 6-inch thick aluminum load distribution fixtures, which are bonded to the specimen (Fig. 1.3).

Two bonding materials were used. For unreinforced specimens the load fixtures were attached with a 0.25 inch layer of a polysulfide material having a low (~ 150 psi) shear modulus. This low shear modulus permitted large strains in the tensile direction with little drag. There was some concern that under compression the polysulfide might tend to extrude and thus cause tensile failure at the specimen edge, but tests at the highest level of compression showed no evidence of such an effect. In early tests the crack sometimes occurred near an edge, so a layer of epoxy 8 inches wide was added to both faces of each tensile edge, in order to force the crack into the center of the specimen (Fig. 1.3).

The bonding of reinforced specimens was somewhat more complicated. In order to transfer load to the steel, a steel plate was welded to each bar on the tensile edge, then epoxy bonded in place. The entire tensile edge was then bonded with epoxy to another steel plate, which was attached to the load distribution fixture. This procedure ensured that the steel would not debond from the much softer masonry. A polysulfide bond was again used on the compression side, in order to minimize shear drag.

In applying loads to the specimen, one tensile side was fixed in translation, and the other displaced at a prescribed rate. Both tensile sides were free to rotate. As the tensile load was applied, it was multiplied by a factor and applied to one of the compressive sides (proportional loading), which was also free to rotate. The opposite compressive side was actively prevented from either displacing or rotating. Thus, overall displacement and rotation were prevented, and uniform stress along each side was ensured.

Prism tests were also conducted as part of this investigation [4]. These tests were performed with a 300 kip Riehle machine under displacement control, and a ball and socket joint was used between the prism and the load platen in order to eliminate moments. The results were quite sensitive to prism end conditions, and these effects are discussed in Section 3.1.

## 1.5 Data Recording and Reduction

Data taken during biaxial tests consisted of the load applied by each hydraulic actuator, plus a number of displacements. The loads were measured by load cells placed in line with the actuators. The displacements were taken across the specimen at several locations, and were measured by linear variable differential transformers (LVDTs). There were eight LVDTs available. They were usually arranged with two on each face parallel with the direction of tensile loading, three in the compressive direction, and one at a 45° orientation. This arrangement constitutes a strain rosette that defines the state of strain. The rods that the LVDTs were attached to can be seen in Fig. 1.3.

The signals from these instruments were processed by a high-speed digital data acquisition system, which recorded 300 samples/sec from each channel on magnetic tape. The information on tape was converted to plots by a computer program that allowed several channels to be combined and plotted versus either time or another combination of channels.

Table 1.1 Component Descriptions

| | Block | Mortar | Grout |
|---|---|---|---|
| Description | Type N<br>Normal weight<br>ASTM C90 | Type S<br>ASTM C270 | 2000 psi coarse<br>ASTM C476<br>(6-sack grout) |
| Compressive strength* | 3300 psi | 2420 psi | 3870 psi |
| Tensile strength* | 329 psi | 215 psi | 266 psi |
| Reinforced specimens with #5 (grade 60) rebar. | | | |

*Strengths from 4 in. x 6.5 in. block coupons, and grout and mortar cylinders.

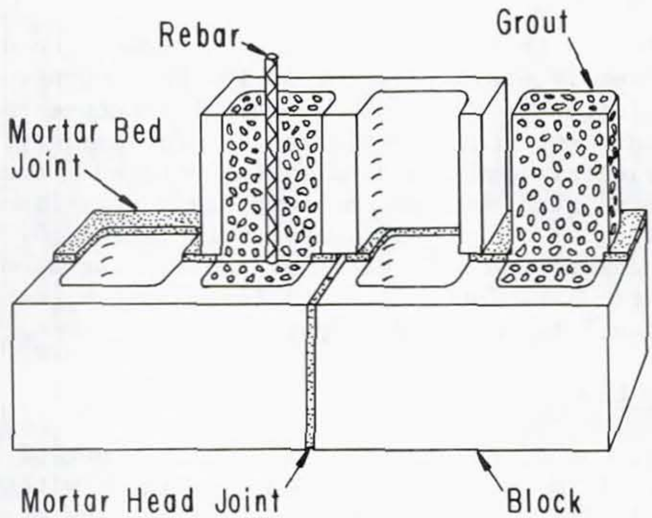



Fig. 1.1. Illustration of Masonry Terms.

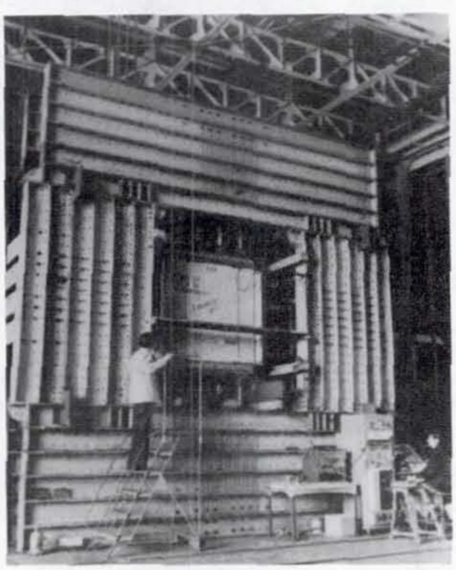


Fig. 1.2. Biaxial Test Frame.

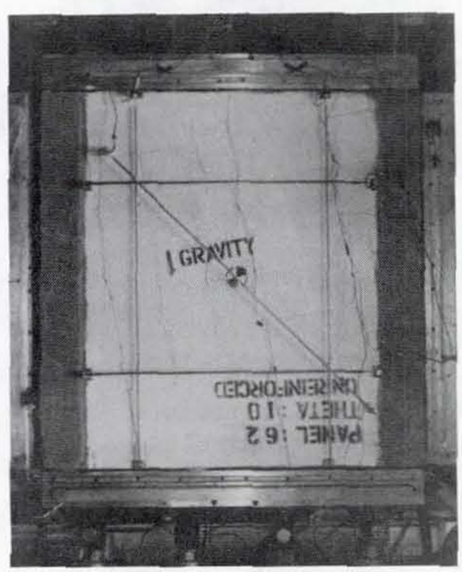



Fig. 1.3. Load Distribution Fixtures Bonded to Specimen.

# 2. ELASTIC BEHAVIOR

The first step in the analysis of a structure is to determine its elastic response. Even if one's interest is in the response once fracture has begun, prediction of the commencement of fracture requires knowledge of the behavior in the elastic range. For masonry, this behavior is quite simple for loadings usually encountered. Except, perhaps, in the high compressive stress range, masonry has a very linear response up until first cracking. This linear behavior is defined by the elastic moduli. One also needs to know the material damping for a complete description of the elastic behavior. If it is large enough, material damping can be of importance in energy dissipation.

## 2.1 Form of Elastic Law

In this treatment of the behavior of masonry, out of plane loading is assumed zero. If a coordinate system is aligned with the joint directions as shown in Fig. 2.1, the stresses and strains of interest are $(\sigma_{11}, \sigma_{22}, \sigma_{12}, \sigma_{21})$ and $(e_{11}, e_{22}, e_{12}, e_{21})$. Either set is assumed to determine the other.

This geometry and loading is one that is frequently encountered, of course. From the behavior of the material at each point in the thickness direction, the overall behavior is commonly derived by assuming a plane stress condition, then integrating stresses across the thickness. If masonry is considered solid, such a procedure is valid. But grout cores adhere poorly to the cell walls, and there are voids in the grout and at the head joints. For partially grouted masonry, in fact, a stress-strain relation at a point in the thickness direction cannot be defined, for such an element fails to be a continuum. Therefore, elastic behavior will be analyzed without considering the variation in stress through the thickness.

Since this analysis will disregard the thickness direction, it would be more correct to discuss stress resultants (force per unit length) rather than stress. But since strength is commonly expressed in terms of stress, this is not done.

Plots of the above stresses versus strain are very linear almost until fracture. Therefore, assume that within some range the following linearity condition holds:

$$\sigma_{ij} = C_{ijk\ell}\, e_{k\ell} \;,$$

where the subscripts have the range (1,2), and repeated subscripts indicate summation.

If one assumes the existence of a strain energy function W such that

$$\sigma_{ij} = \frac{\partial W}{\partial e_{ij}} ,$$

then one has symmetry of the above matrix: $C_{ijk\ell} = C_{k\ell ij}$. Such a function exists for the three-dimensional case, and the proof of its existence [5] holds for this case of stress resultants, at least for static situations.

If the above matrix is inverted, one has the symmetric matrix $c_{ijk\ell}$ such that

$$e_{ij} = c_{ijk\ell}\,\sigma_{k\ell} .$$

Because of symmetry of the stress tensor ($\sigma_{12} = \sigma_{21}$), terms can be combined to give the form

$$\begin{bmatrix} e_{11} \\ e_{22} \\ e_{12} \end{bmatrix} = \begin{bmatrix} c_{1111} & c_{1122} & c'_{1112} \\ c_{1122} & c_{2222} & c'_{2212} \\ c_{1211} & c_{1222} & c'_{1212} \end{bmatrix} \begin{bmatrix} \sigma_{11} \\ \sigma_{22} \\ \sigma_{12} \end{bmatrix} ,$$

where the symmetry condition $c_{1122} = c_{2211}$ has been employed, and the equation for $e_{21}$ ($=e_{12}$) has been eliminated.

There is one further condition that can be used to simplify the above relation. That condition is material symmetry. If the $x_1$ axis is reversed in sense, the form of the above law cannot be affected, for the $x_2$ axis is a line of symmetry. (Note that this would not be the case for a wall constructed of block open at one end and closed at the other.) Reversing the sense of the $x_1$ axis will change the signs of $e_{12}$ and $\sigma_{12}$, while leaving the other stresses and strains unchanged. Hence, one concludes

$$c'_{1112} = c'_{2212} = c_{1211} = c_{1222} = 0.$$

With new names for the four constants, the relation becomes

$$\begin{bmatrix} e_{11} \\ e_{22} \\ e_{12} \end{bmatrix} = \begin{bmatrix} 1/E_1 & -2\nu/(E_1+E_2) & 0 \\ -2\nu/(E_1+E_2) & 1/E_2 & 0 \\ 0 & 0 & 1/2G \end{bmatrix} \begin{bmatrix} \sigma_{11} \\ \sigma_{22} \\ \sigma_{12} \end{bmatrix} \quad . \tag{2.1}$$

## 2.2 Determination of Elastic Moduli

Direct determination of the values of the four elastic moduli is possible by applying loads in the following manner: First set $\sigma_{22} = 0$, while $\sigma_{11}$ is non-zero. This will determine $E_1$. Switching roles for the two stresses then determines $E_2$ and $\nu$. Any stress state having $\sigma_{12} \neq 0$ will determine G.

In this investigation, specimens were tested by applying direct (zero shear) loads to edges cut at a lay-up angle $\theta$. With the coordinate systems shown in Fig. 2.2, this implies that $\sigma'_{12} = 0$. Under this restriction, stresses transform as follows:

$$\begin{aligned} \sigma_{11} &= \sigma'_{11} \cos^2\theta + \sigma'_{22} \sin^2\theta \\ \sigma_{22} &= \sigma'_{11} \sin^2\theta + \sigma'_{22} \cos^2\theta \\ \sigma_{12} &= (-\sigma'_{11} + \sigma'_{22}) \cos\theta \sin\theta \end{aligned} \tag{2.2}$$

Two orientations have special significance in these relations. For $\theta = 45°$, one has $\sigma_{11} = \sigma_{22}$ for all values of $\sigma'_{11}$ and $\sigma'_{22}$. Because of this, the three moduli $E_1$, $E_2$, and $\nu$ cannot be determined for a 45° specimen. Clearly, for values of $\theta$ near 45°, their determination will be difficult. For $\theta = 0°$, on the other hand, one has $\sigma_{12} = 0$, so that G cannot be determined. Hence, an intermediate value of $\theta$ is necessary in order to determine all four moduli accurately. For this reason, specimens having a lay-up angle $\theta = 20°$ were chosen for the determination of elastic moduli.

From the above equations, one finds that, for direct determination of the moduli, the applied loads must satisfy these conditions:

$$\begin{aligned} \sigma'_{22} &= -\sigma'_{11} \tan^2\theta \qquad (\Rightarrow \sigma_{22} = 0) \\ \sigma'_{11} &= -\sigma'_{22} \tan^2\theta \qquad (\Rightarrow \sigma_{11} = 0) \end{aligned}$$

Note that each of these relations necessitates a tensile load. Because of the low tensile strength of masonry, care must be taken in conducting such a test not to fail the specimen prematurely. In the tests we performed, the second condition was satisfied during a test in which a failure strength of the specimen was determined. However, no test was conducted that satisfied the first condition. Hence, determination of the moduli was somewhat more complicated than in the procedure described above.

The tests that were conducted (for most specimens) were the following three: horizontal compression ($\sigma'_{22} = 0$), vertical compression ($\sigma'_{11} = 0$), and a failure test with $\sigma_{11} = 0$. From the failure test one can find $E_2$ and the term $-2\nu/(E_1 + E_2)$. Then one can determine $E_1$ from the horizontal compression test. This requires the value of $-2\nu/(E_1 + E_2)$, but since it multiplies the small quantity $\sigma_{22}$, an error in its value will have a small effect. Finally, any one of the three tests is suitable for determining G.

## 2.3 Analysis of Moduli Data

The data taken during the tests included the loads applied to the edges of the specimens, and displacements in three directions across the specimens. These are shown in Fig. 2.3. The loads were applied as shown for all specimens, but the configuration of the displacement gauges varied somewhat.

The edges of the specimen correspond to the $x'_1$ and $x'_2$ directions (see Fig. 2.2). Hence, the stresses are given by

$$\sigma'_{11} = (LD13 + LD14 + LD15 + LD16)/488 \text{ in}^2$$

$$\sigma'_{22} = (LD1 + LD2 + LD3 + LD4)/488 \text{ in}^2$$

$$\sigma'_{12} = 0$$

In the horizontal direction, the loads on each side of the specimen agreed very closely. In the vertical direction, gravity produced a difference of several psi between the loads at the top and the loads at the bottom of the specimen. For determination of elastic moduli, this offset is of no significance.

While the actual stresses were known, only changes in strains were available, because the LVDTs were not set to read zero at zero load. To find the change in strain, each displacement was divided by its gauge length, then the resulting strains were averaged. For the configuration of Fig. 2.3 this gives

$$\Delta e_{11} = \Delta \left(\frac{D1H}{GL1H} + \frac{D2H}{GL2H} + \frac{D3H}{GL3H} + \frac{D4H}{GL4H}\right) \Big/ 4$$

$$\Delta e'_{22} = \Delta \left(2\,\frac{D1V}{GL1V} + \frac{D2V}{GL2V} + \frac{D3V}{GL3V}\right) \Big/ 4$$

$$\Delta e_D = \Delta \ \frac{D1D}{GLD}$$

Examination of displacements from many specimens showed little variation in readings from the same face of the panel (e.g. D1V and D2V), but frequent significant variation, as much as 20 percent, from front to back. This is the reason that D1V is multiplied by 2. For the same reason, $e_D$ may suffer some error, since only one LVDT was available for its measurement.

The first step in analysis of the data was to determine a set of stresses and corresponding strains. These strains were found from plots of stress versus strain. For uniaxial loading, all three strains were plotted versus the applied load. For the failure test, both stresses were used, and corresponding values of the two stresses were found from plots of stress versus time.

An example of a stress-strain plot for several cycles of compression is shown in Fig. 2.4. Because the specimen was preloaded, the stress does not go to zero. To find the strain corresponding to a stress, a straight line was extrapolated to the zero-stress level along the path of increasing load, as shown by the broken line.

From the specimens tested in this investigation, two were selected that were best suited for elastic moduli determination. For many of the early specimens, determination of all four moduli was impossible, because only horizontal and vertical strains were measured. Of the remaining specimens, there were two tested at the desired 20° lay-up angle. These two specimens, nos. 79 and 84, were from the same batch, so their properties should be very similar.

The strains measured for these two specimens are given in Table 2.1. These three extensional strains ($e'_{11}$, $e'_{22}$, $e_D$) constitute a strain rosette, (0°, 90°, 45°), and from them the shear strain $e'_{12}$ can be determined. The transformation of strain components is given by

$$
\begin{aligned}
e_{11} &= e'_{11} \cos^2\theta + e'_{22} \sin^2\theta + 2e'_{12} \cos\theta \sin\theta \\
e_{22} &= e'_{11} \sin^2\theta + e'_{22} \cos^2\theta - 2e'_{12} \cos\theta \sin\theta \qquad (2.3) \\
e_{12} &= (-e'_{11} + e'_{22}) \cos\theta \sin\theta + e'_{12} (\cos^2\theta - \sin^2\theta)
\end{aligned}
$$

The strain $e_D$, which is measured by the LVDT labeled D1D in Fig. 2.3, is seen to be the strain $e_{22}$ for $\theta = 45°$. Therefore, one has

$$e_D = \frac{e'_{11} + e'_{22}}{2} - e'_{12} \ , \qquad (2.4)$$

whence

$$e'_{12} = \frac{e'_{11} + e'_{22}}{2} - e_D \quad . \tag{2.5}$$

Once the shear strain $e'_{12}$ has been determined from Eq. (2.5), Eqs. (2.3) yield the strain components in the (unprimed) lay-up coordinate system. Equations (2.2) serve to transform stress components. By applying these transformations to the stresses and strains of Table 2.1, one obtains the results listed in Table 2.2.

One can now employ the procedure described in Section 2.2 to determine the elastic moduli. Though the stress $\sigma_{11}$ is not zero, as assumed in Section 2.2, it is small enough to be neglected in order to obtain a first estimate of the values of the moduli. With those estimates one can then correct for the non-zero value of $\sigma_{11}$. The results are given in Table 2.3, where G is taken to be the average from the three tests.

With the values of the moduli known, it is now possible to predict, from the applied loads, the strains measured in the experiments. One first transforms the stress components from the primed system to the unprimed system by use of Eqs. (2.2), then finds the lay-up coordinate strains from Eqs. (2.1), then transforms to the primed coordinate strains using

$$\begin{aligned}
e'_{11} &= e_{11} \cos^2\theta + e_{22} \sin^2\theta + 2e_{12} \cos\theta \sin\theta \\
e'_{22} &= e_{11} \sin^2\theta + e_{22} \cos^2\theta - 2e_{12} \cos\theta \sin\theta \\
e'_{12} &= (-e_{11} + e_{22}) \cos\theta \sin\theta + e_{12}(\cos^2\theta - \sin^2\theta)
\end{aligned} \tag{2.6}$$

The final step is to find $e_D$ from Eq. (2.4).

The values of the moduli given in Table 2.3 were determined by using only part of the data available from the experiments. The predicted values of the remaining data will, of course, suffer some error. In order to minimize these errors, it is necessary to use all of the data in the moduli calculations.

To do this, the predicted values for the three measured strains were computed, then the moduli values were refined so as to reduce the largest errors. By calculating the coefficients of each modulus in the expressions for the strains $e'_{11}$, $e'_{22}$, $e_D$, it becomes evident how to alter the moduli. The greatest change was in the value of $\nu$. One might expect $\nu$ to be difficult to determine, since its value requires the measurement of a rather small strain. The refined values are given in Table 2.4.

Two items are noteworthy in these results. One is the small value of $\nu$ compared to its value for metals. The other is the strong anisotropy: This material is twice as stiff in the vertical direction as in the horizontal direction. This difference in stiffness may be partly due to the head joints. There is little mortar between adjacent blocks in the same course, so this joint will probably suffer substantial deformation. In contrast, the grout cores run uninterrupted in the vertical direction, and the bed joints are more completely mortared and have the benefit of compression due to gravity during curing.

Another reason may be that the grout contributes little to stiffness in the horizontal direction. Examination of the cut edges of specimens shows frequent separation of cores from the block. This separation could mean that the grout is not being loaded during horizontal compression.

The values of the strains predicted using the moduli values listed in Table 2.4 are given in Table 2.5. The measured strains are given in Table 2.1, and the predicted values follow from the measured loads (Table 2.1), and the transformation laws given above. One can see that most predicted values are within 20 percent of the measured values, and the worst percentage errors occur for the strains of small magnitude.

The error in these results probably is due principally to uncertainty in measurement of displacements. From the plots of stress versus strain (e.g. Fig. 2.4), the uncertainty in the strains used in these calculations can be estimated, and from this follows the uncertainty in moduli, listed in Table 2.6.

Table 2.1. Elastic Strains (X $10^6$) and Stresses (psi) in Principal Stress Coordinate System.

| Panel # | Test | $e'_{11}$ | $e'_{22}$ | $e_D$ | $\sigma'_{11}$ | $\sigma'_{22}$ |
|---|---|---|---|---|---|---|
| | Horizontal | -182 | 18.2 | -63.1 | -180 | 0 |
| 79 | Vertical | 43.7 | -196 | -41.7 | 0 | -350 |
| | Failure | 106 | -180 | -13.1 | 42.1 | -300 |
| | Horizontal | -176 | 20.7 | -62 | -180 | 0 |
| 84 | Vertical | 37.9 | -207 | -32.8 | 0 | -350 |
| | Failure | 94.1 | -184 | -11.5 | 40.1 | -300 |

Table 2.2. Elastic Strains (X $10^6$) and Stresses (psi) in Joint Coordinate System

| Panel # | Test | $e_{11}$ | $e_{22}$ | $e_{12}$ | $\sigma_{11}$ | $\sigma_{22}$ | $\sigma_{12}$ |
|---|---|---|---|---|---|---|---|
| 79 | Horizontal | -171 | 6.8 | 50 | -159 | -21 | 58 |
| | Vertical | -6.5 | -146 | -103 | -41 | -309 | -112 |
| | Failure | 57 | -131 | -110 | 2.1 | -260 | -110 |
| 84 | Horizontal | -163 | 7.7 | 51 | -159 | -21 | 58 |
| | Vertical | -24 | -145 | -118 | -41 | -309 | -112 |
| | Failure | 40 | -130 | -115 | 0.3 | -260 | -109 |

Table 2.3. Elastic Moduli, First Estimate

| Panel # | $E_1$ ($10^6$ psi) | $E_2$ ($10^6$psi) | G ($10^6$ psi) | $\nu$ |
|---|---|---|---|---|
| 79 | 0.907 | 1.992 | 0.541 | 0.31 |
| 84 | 0.957 | 2.001 | 0.506 | 0.23 |

Table 2.4. Elastic Moduli, Final Estimate

| Panel # | $E_1$ ($10^6$ psi) | $E_2$ ($10^6$ psi) | G ($10^6$ psi) | $\nu$ |
|---|---|---|---|---|
| 79 | 0.912 | 2.041 | 0.526 | 0.19 |
| 84 | 0.957 | 2.037 | 0.488 | 0.16 |

Table 2.5. Predicted Elastic Strains (X $10^6$)

| Panel # | Test | $e'_{11}$ | $e'_{22}$ | $e_D$ |
|---|---|---|---|---|
| 79 | Horizontal | -186 | 24.4 | -64.4 |
| | Vertical | 47.5 | -198 | -39.2 |
| | Failure | 84 | -176 | -18.6 |
| 84 | Horizontal | -182 | 25.3 | -69 |
| | Vertical | 49.2 | -205 | -34.4 |
| | Failure | 82.6 | -182 | -14.0 |

Table 2.6. Uncertainty in Moduli Values

| Modulus | $E_1$ ($10^6$ psi) | $E_2$ ($10^6$ psi) | G ($10^6$ psi) | $\nu$ |
|---|---|---|---|---|
| Uncertainty | $\pm$ 0.03 | $\pm$ 0.08 | $\pm$ 0.03 | $\pm$ 0.04 |

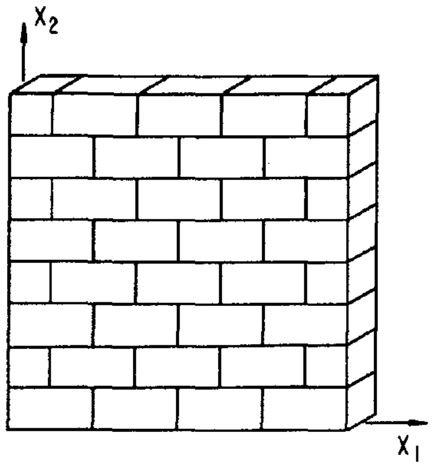



Fig. 2.1. Joint Coordinate System.

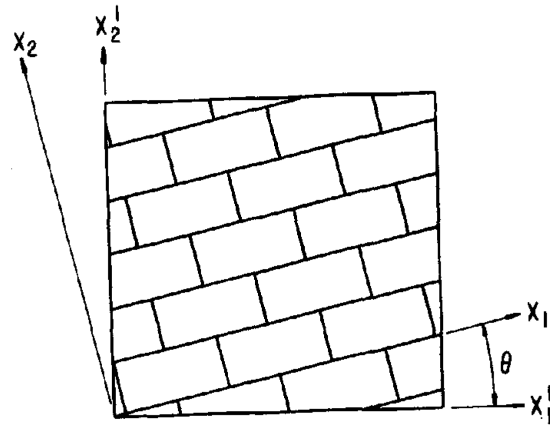



Fig. 2.2. Joint and Principal Stress Coordinate Systems.

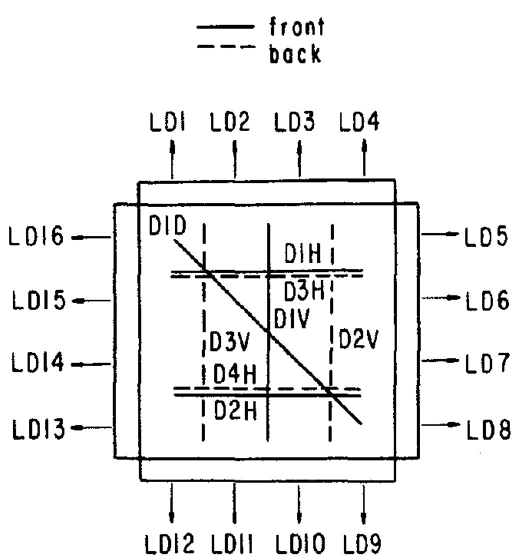



Fig. 2.3. Load and Displacement Names.

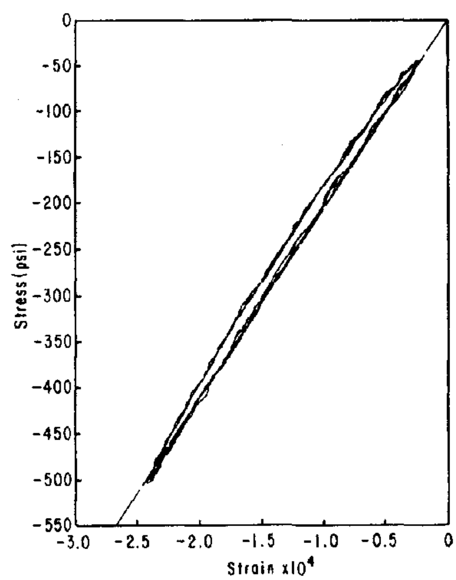



Fig. 2.4. Example of a Stress-Strain Plot.

# 3. INITIAL STRENGTH

The strength of masonry is defined by a closed surface in stress space ($\sigma_{11}$, $\sigma_{22}$, $\sigma_{12}$) at which first cracking occurs. The tests conducted in this investigation do not determine the entire surface, but concentrate on a part of the surface that should be of greatest importance in the analysis of buildings subjected to seismic loading.

Both uniaxial and biaxial strength tests were conducted, and uniaxial specimens were tested both in compression and tension. The biaxial tests were restricted to the following special case: The principal stresses and their orientation were chosen to render the normal stress on the head joint planes zero. This restriction was made because it is believed that in structures this stress is usually small compared to the shear stress or the normal stress on the bed joint planes:

$$|\sigma_{11}| << \max\ (|\sigma_{22}|\ ,\ |\sigma_{12}|)$$

## 3.1 Uniaxial Compressive Strength

Before looking at the general case of biaxial stress, consider the special case in which one of the principal stresses is zero and the other negative (compressive). For this case there remains one variable to be specified: the angle at which the stress acts relative to the bed-joint planes. Though the compressive strength and its dependence on lay-up angle should be determined by tests on large-scale panels, our test system did not have the ability to load to failure in compression. Hence, we were forced to determine compressive strength by tests on small-scale specimens, and we were unable to determine its direction dependence.

The direction of loading which is easiest to test is that direction in which masonry is normally loaded - with compression across the bed joints. The strength in that direction is related to the quantity $f'_m$, which plays a major role in the design of structures. Building codes employ $f'_m$ to limit allowable stresses. Its value is determined by tests on prisms, which are small assemblages consisting of two or more blocks laid up in a column. Specifically, $f'_m$ is determined by tests of two-course prisms capped with a high-strength sulphur fly-ash compound or a high-strength gypsum plaster [6]. $f'_m$ is taken to be the failure load in compression divided by the cross-sectional area of the prism.

As part of this investigation we examined the significance of $f'_m$ as determined by the above procedure [4]. We found that two factors significantly affect the results obtained. The first is end restraint. Friction between the specimen and the bearing plates of the testing apparatus greatly restricts lateral displacement at the ends of the specimen. This restraint results in a higher failure load than would be obtained if the restraint were absent.

Evidence of this restraint is found in the failure mode - in a two-course prism the fracture surface bends away from the bearing plate to leave a roughly conical piece attached to the bearing plate (Fig. 3.1) - and in the lower strengths obtained with prisms of more than two courses. For prisms of three courses, the end effects are smaller at the center of the prism, so that the stress state begins to approach one of true uniaxial compression. This results in proper tensile splitting in the center block (Fig. 3.2). The proper stress state is nearly achieved in prisms of four courses, so there is little variation in strength between four-course prisms and those of five courses.

This interpretation of the effects of prism height leads to the conclusion that a two-course prism should behave similarly to a four-course prism if friction between the specimen and the bearing plates were eliminated. One method of greatly reducing the friction is to use a soft capping material. We conducted tests of two-course prisms capped with a 0.25 inch thickness of a polymer material with a very low shear modulus (about 150 psi). The results were as anticipated. The failure stress was about equal to that of a four-course prism, and the failure mode was tensile splitting that extended all the way to the ends of the specimens (Fig. 3.3). That this failure stress was not less than that of a four-course prism is evidence that the polymer does not cause premature failure by extrusion.

The other factor affecting the significance of prism tests is bond configuration. The two-course prisms which are tested to determine $f'_m$ have no head joints. Two full blocks are simply laid up in stack bond. To determine if head joints influence the strength results obtained, tests were conducted on three- and five-course running bond prisms constructed from full blocks and half blocks. It was found that these prisms are significantly weaker than stack bond prisms of the same height. The reduction in strength for five-course prisms was about 16 percent.

The above results show that the value of $f'_m$ obtained from two-course prism tests is much higher than the true compressive strength of full-scale masonry. The strength of a two-course stack-bond prism with a hard cap is about 62 percent higher than that of a five-course running-bond prism, which should be close to the strength of full-scale masonry. (This does not mean that the building codes are incorrect. This artificially high strength apparently has been accounted for in the safety factors. The true full-scale masonry strength is simply never considered in building design.)

It is possible that the narrow width (one full block wide) of a running-bond prism leads to a slightly premature failure in the head joints, but the error should be small. This possibility should be investigated by testing larger specimens, for example a five-course running-bond prism two full blocks wide. For this program we did not have a machine capable of performing such a test, so our best estimate of the compressive strength of full-scale masonry is that of a five-course running-bond prism.

## 3.2 Uniaxial Tensile Strength

Next consider the case in which one principal stress is zero and the other positive (tensile). For this case we were able to load large-scale panels to failure, and hence able to determine direction dependence.

A series of tests of direction dependence was conducted for each of two batches of specimens. The results are shown in Fig. 3.4. The variable $\theta$ represents the angle between the tension direction and the normal to the bed joints, so that $\theta = 0°$ represents tension across the bed joints, and $\theta = 90°$ tension across the head joints.

For each batch there is a clear maximum strength for $\theta$ about 40°, with the strength gradually diminishing away from 40°. Thus, a second degree polynomial should provide an excellent representation of the dependence of strength on $\theta$. Further, the variation in strength with $\theta$ is remarkably similar for the two batches, with batch 6 simply shifted up from batch 5. Therefore, it appears that one should know the strength for arbitrary $\theta$ if the strength for $\theta = 0°$ is known.

For each batch, the values of the three constants for a second-degree polynomial fit of the data were determined by a least squares procedure. The results are given in Table 3.1, where

$$\sigma_t = \sigma_t^{(0)} + a\theta + b\theta^2 \ , \ \theta \text{ in degrees.}$$

Since a and b show little variation between batches, the following formula should be an excellent representation of uniaxial tensile strength:

$$\sigma_t = \sigma_t^{(0)} + 0.67\ \theta - 0.009\ \theta^2 \ , \ 0 \leq \theta \leq 90° \qquad (3.1)$$

## 3.3 Biaxial Strength

The strength of masonry under biaxial loading was studied for the special case of zero head-joint normal stress. The behavior of fully grouted masonry was expected to be similar to that of concrete, which has been carefully studied. For concrete under biaxial loading, the tensile stress at which fracture occurs decreases approximately linearly with increasing compressive stress (see Fig. 3.5, from [7]).

In this investigation the most complete results were obtained for batch 6. The original data points are listed in Table 3.2, and shown in Fig. 3.6 with tensile stress plotted versus compressive stress. The nine original data points, represented by circles, clearly show a gradual decrease in tensile strength as the compressive stress increases. For these tests, however, the tensile direction varied over a range of 80°. Hence, the results should reflect the anisotropy described in the previous section, and a more nearly linear relation might be expected if this anisotropy were somehow accounted for.

Consider what effect anisotropy should have on the data of Fig. 3.6. For panels 48 and 55, the angle between the tensile direction and the normal to the bed joints was 45°, while for panels 50 and 58 the angle was 0°. Hence, by the results shown in Fig. 3.4, panels 48 and 55 can be expected to show less of a decrease in tensile strength compared to panels 50 and 58 than they would if their tensile direction were also 0°.

This observation forms the basis of the following method of correcting the tensile stress of a biaxial test for the known anisotropy in uniaxial tensile strength. The method is illustrated in Fig. 3.7, in which tensile stress is represented by the vertical axis, and compressive stress by the horizontal axis. The circle represents the original data, and is assumed to lie on a line from A, the uniaxial compressive strength, to B, the uniaxial tensile strength for the direction of the tensile stress.

Hence, if the tensile direction is stronger by an amount $\Delta$ then the tensile strength for the direction $\theta = 0°$, the point would have fallen on the line AC if the specimen had been tested at $\theta = 0°$. Thus, the corrected point is represented by x. Anisotropy in compressive strength, if it were known, could be corrected for by the same method.

Since neither A nor B is known until a line is fitted to the data, the correction should be done by an iterative process. But fortunately, the corrections are insensitive to the final result. If one assumes the slope of the line AB, usually about 1/15, the corrected value is determined, and no iterations should be necessary.

For the biaxial tests conducted in this program, the condition of zero head joint normal stress meant that the ratio of compressive stress to tensile stress was determined by the angle at which the tensile stress was applied. Hence, this angle determines the correction $\Delta$ in uniaxial strength, and also the final correction in biaxial strength if a slope is assumed for the line AB.

The relation between the ratio of stresses and the tensile direction is

$$r = \mathrm{ctn}^2\ \theta ,$$

where r is the ratio of compressive stress to tensile stress, and $\theta$ is the angle between the tensile direction and the normal to the bed joints (Fig. 3.8).

If one known r and $\Delta$, and assumes a slope for the line AB, then the correction $\delta$ for the tensile stress is determined as follows. Let the line AB have slope 1/R. Then the lines AB and AC are given by

$$y = B\left(1 + \frac{x}{RB}\right),$$

$$y = (B-\Delta)\left(1 + \frac{x}{RB}\right).$$

The difference is

$$\delta = \Delta\left(1 + \frac{x}{RB}\right).$$

Since the original data point lies on the line AB and on the line

$$y = -x/r\ ,$$

one has

$$x = -\frac{rBR}{r + R}\ .$$

Substituting this result in the expression for $\delta$ gives

$$\delta = \Delta\left(1 - \frac{r}{r + R}\right).$$

The tests in this program were conducted for six values of the angle $\theta$. Table 3.3 gives the values of $\theta$ and the resulting corrections, for $R = 15$. Applying these corrections to the data of Table 3.2 gives the values listed in the last row. These are plotted as x's on Fig. 3.6, and are clearly closer to a straight line than the original data.

A measure of the closeness to a straight line is provided by the methods of statistics [8]. Let the corrected data be represented by the pairs of observations $\{(x_i, y_i);\ i = 1,2,\ldots,n\}$, where $x_i$ is the compressive stress ($x_i < 0$). Assume that the random variable $Y_i = Y|x_i$ is related to $x_i$ by the equation

$$Y_i = \alpha + \beta x_i + E_i\ \ ,$$

where the error term $E_i$ is a random variable with mean zero, and that each $E_i$ is normally distributed with the same variance $\sigma^2$. Then the regression line

$$\mu_{Y|x} = \alpha + \beta x$$

is estimated by the line

$$\hat{y} = a + bx\ \ ,$$

where

$$b = \frac{n\Sigma x_i y_i - (\Sigma x_i)(\Sigma y_i)}{n\Sigma x_i^{\,2} - (\Sigma x_i)^2}$$

$$a = \bar{y} - b\bar{x} \quad ,$$

and the bar over a variable indicates the arithmetic mean. Applying these results to the corrected data gives

$$b = 0.0820$$

$$a = 110.7$$

The accuracy of these estimates of the parameter $\alpha$ and $\beta$ is expressed by confidence intervals. $(1-2\gamma)$ 100% confidence intervals are given by

$$a - \frac{t_\gamma s\sqrt{\Sigma x_i^{\,2}}}{\sqrt{n\ S_{xx}}} < \alpha < a + \frac{t_\gamma s\sqrt{\Sigma x_i^{\,2}}}{\sqrt{n\ S_{xx}}}$$

$$b - \frac{t_\gamma s}{\sqrt{S_{xx}}} < \beta < b + \frac{t_\gamma s}{\sqrt{S_{xx}}} \quad ,$$

where $t_\gamma$ is a value of the t distribution with n - 2 degrees of freedom,

$$S_{xx} = \Sigma x_i^{\,2} - \frac{(\Sigma x_i)^2}{n} \quad ,$$

and $s^2$ is an unbiased estimate of $\sigma^2$ given by

$$s^2 = \frac{[\Sigma y_i^{\,2} - (\Sigma y_i)^2/n] - b\ [\Sigma x_i y_i - (\Sigma x_i)(\Sigma y_i)/n]}{n-2} \quad .$$

For the corrected data one finds

$$s = 10.4 \quad ,$$

and 90% confidence intervals are

$$101.1 < \alpha < 120.3$$

$$0.0604 < \beta < 0.1036 \quad .$$

For the purpose of predicting the strength at a particular compressive stress, one has the $(1 - 2\gamma)$ 100% confidence interval for a single response

$$\hat{y}_o - t_\gamma s\sqrt{1 + \frac{1}{n} + \frac{(x_o-\bar{x})^2}{S_{xx}}} < y_o < \hat{y}_o + t_\gamma s\sqrt{1 + \frac{1}{n} + \frac{(x_o-\bar{x})^2}{S_{xx}}} \quad .$$

The 90% confidence intervals at several values of compressive stress are given in Table 3.4.

The line determined by the above procedure is shown in Fig. 3.6. It fits the data quite well, and the confidence intervals give the variability in strength one can expect.

For use in computations, it is necessary to have an expression for the above law of biaxial strength. The statement of this law is quite simple in terms of principal stresses. Let the principal stress $\sigma'_{22}$ be positive (tensile), $\sigma'_{11}$ negative (compressive). Then fracture occurs when

$$\sigma'_{22} = \frac{\sigma_t}{\sigma_c}(\sigma'_{11} + \sigma_c) \quad , \tag{3.2}$$

where $\sigma_t$ is the uniaxial tensile strength for the $x'_2$ direction.

The quantity $\sigma_c$ requires some consideration. If the linear law held for arbitrarily large compressive stress, $\sigma_c$ (>0) would be the uniaxial compressive strength for the $x'_1$ direction. However, the tests in this investigation did not cover the high compressive stress range, and for concrete the uniaxial strength is significantly less than the quantity needed in the above formula (see Fig. 3.5). Hence, $\sigma_c$ is to be determined from the data fit for the biaxial tests, rather than from uniaxial tests. The behavior in the high compressive stress range is a subject that requires further investigation.

For some purposes it is more convenient to express the first-cracking law in terms of stress components for the joint coordinate system. Let $(x_1,x_2)$ be the joint coordinates, and $(x'_1,x'_2)$ the principal stress coordinates, as in Fig. 2.2.

The two stress invariants are then represented by the following equations:

$$\begin{aligned} \sigma_{11} + \sigma_{22} &= \sigma'_{11} + \sigma'_{22} \\ \sigma_{11}\sigma_{22} - \sigma_{12}^2 &= \sigma'_{11}\sigma'_{22} \end{aligned} \tag{3.3}$$

Substituting the first of these in Eq. (3.2) gives

$$
\begin{aligned}
\sigma'_{11} &= \frac{\sigma_c}{\sigma_t + \sigma_c}(\sigma_{11} + \sigma_{22} - \sigma_t) \\
\sigma'_{22} &= \frac{\sigma_t}{\sigma_t + \sigma_c}(\sigma_{11} + \sigma_{22} + \sigma_c)
\end{aligned}
\tag{3.4}
$$

Substituting Eqs. (3.4) in the second of (3.3) gives

$$
\sigma_{11}\sigma_{22} - \sigma_{12}^2 = \frac{\sigma_t\sigma_c}{(\sigma_t + \sigma_c)^2}(\sigma_{11} + \sigma_{22} - \sigma_t)(\sigma_{11} + \sigma_{22} + \sigma_c) \ , \tag{3.5}
$$

where $\sigma_t$ is the uniaxial tensile strength in the direction of the principal tensile stress. This strength is given by Eq. (3.1), and the angle needed in (3.1) between the $x_2$ direction and the direction of the principal tensile stress is

$$
\theta = \left| \frac{1}{2} \tan^{-1} \frac{2\sigma_{12}}{\sigma_{11} - \sigma_{22}} - 90° \ H(\sigma_{11} - \sigma_{22})[2H(\sigma_{12}) - 1] \right| \ , \tag{3.6}
$$

where

$$
H(x) = \begin{cases} 1, & x > 0 \\ 0, & x \leq 0 \end{cases} \ .
$$

Equations (3.2) and (3.5) apply only to stress states whose principal stresses are of opposite signs. And while they may hold as long as this condition is satisfied, their validity was checked only for the special case $\sigma_{11} = 0$. The test results and the curve represented by Eq. (3.5), for $\sigma_{11} = 0$, are shown in Fig. 3.9.

Table 3.1 Coefficients of Uniaxial Tensile Strength Quadratic.

| | $\sigma_t^{(0)}$ | a | b |
|---|---|---|---|
| Batch 5 | 84.3 | 0.660 | -0.00924 |
| Batch 6 | 105.5 | 0.677 | -0.00871 |

Table 3.2 Biaxial Failures Stresses (psi) for Batch 6.

| θ | 0 | 45 | 60 | 70 | 80 |
|---|---|---|---|---|---|
| Compr. stress | 0 | -128 | -305 | -446 | -1023 |
| Orig. tensile stress | 109 | 117 | 97 | 55 | 28 |
| Corrected stress | 109 | 106 | 91 | 53 | 29 |
| Compr. stress | 0 | -136 | -309 | -548 | |
| Orig. tensile stress | 101 | 120 | 99 | 70 | |
| Corrected stress | 101 | 109 | 93 | 68 | |

Table 3.3. Correction of Tensile Stress for Anisotropy

| θ | r | Δ(psi) | δ(psi) |
|---|---|---|---|
| 0 | 0 | 0 | 0 |
| 45 | 1.00 | 11.9 | 11.2 |
| 60 | 3.00 | 7.8 | 6.5 |
| 70 | 7.55 | 2.8 | 1.9 |
| 75 | 13.9 | -0.4 | -0.2 |
| 80 | 32.2 | -4.0 | -1.3 |

Table 3.4. Single Response 90% Confidence Intervals

| Compr. Stress (psi) | 0 | -200 | -400 | -600 | -800 | -1000 |
|---|---|---|---|---|---|---|
| Tensile stress interval (psi) | 133 | 115 | 99 | 83 | 68 | 54 |
| | 89 | 73 | 57 | 40 | 22 | 3 |

Fig. 3.1. End Restraint in Two-Course Prism with Hard Cap.

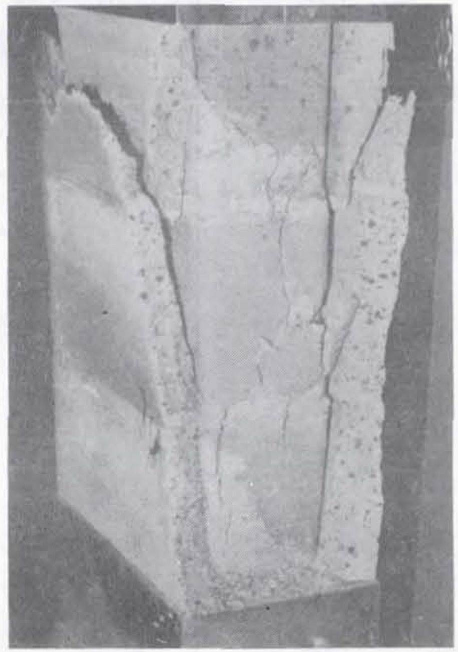


Fig. 3.2. Tensile Splitting of Three-Course Prism.

Fig. 3.3. Tensile Splitting of Two-Course Prism with Soft Cap.

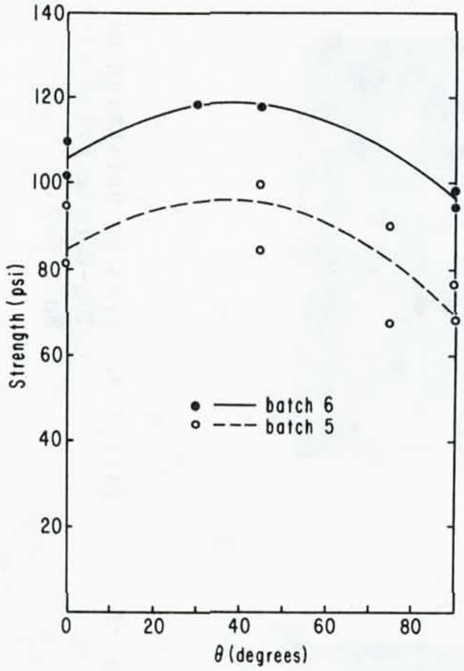



Fig. 3.4. Uniaxial Tensile Strength Direction Dependence.

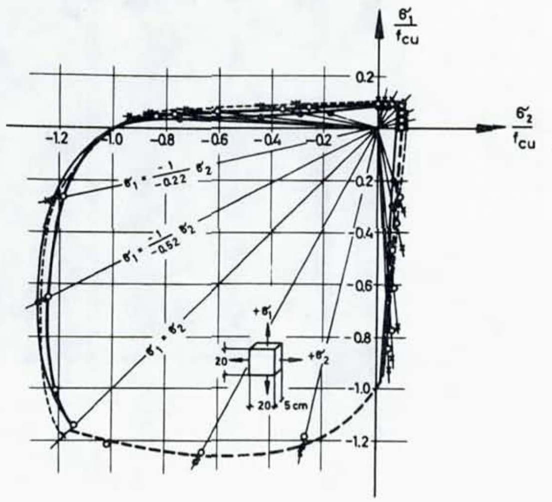


Fig. 3.5. Biaxial Strength of Concrete, from [7].

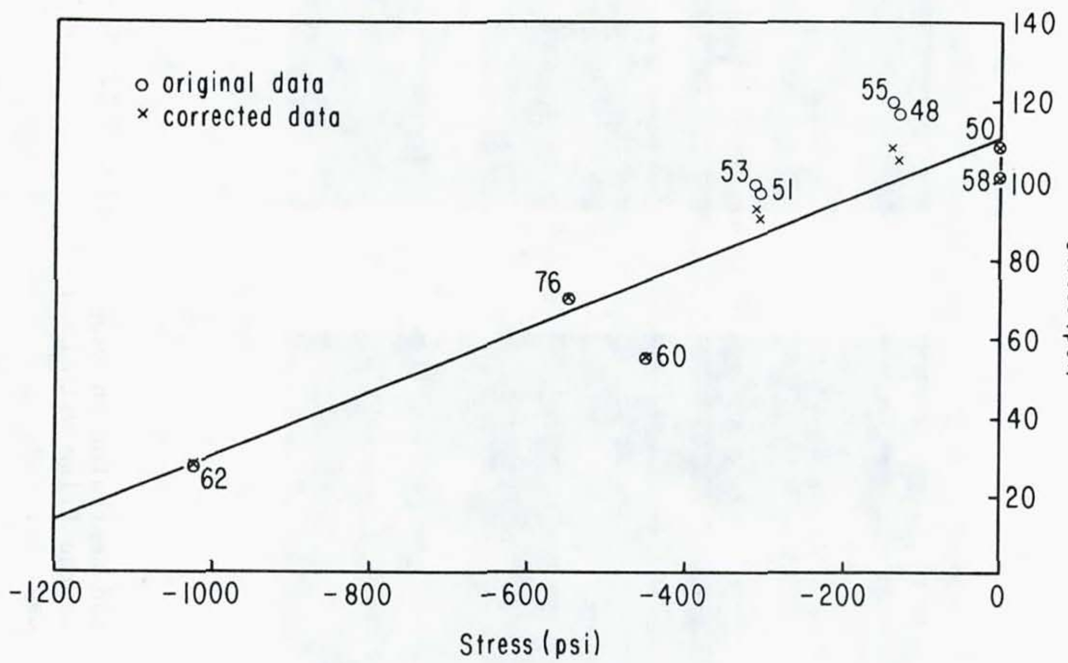



Fig. 3.6. Biaxial Strength of Batch 6.

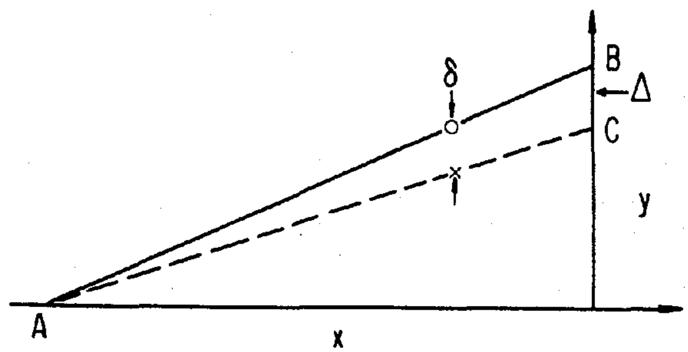



Fig. 3.7. Correction of Biaxial Stress for Anisotropy.

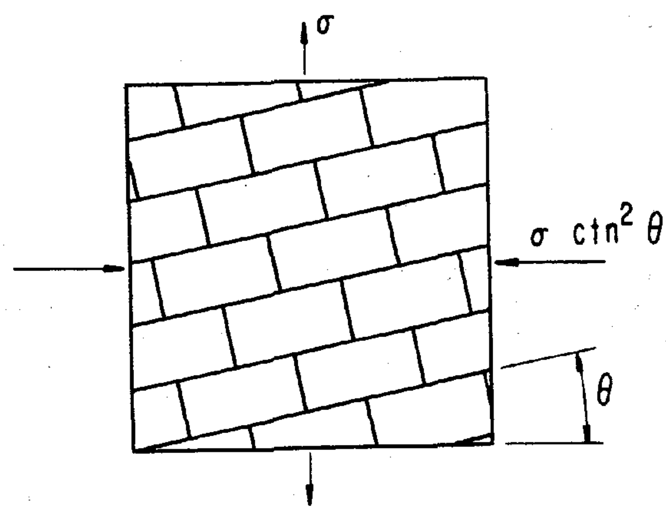



Fig. 3.8. Relation of Lay-up Angle to Stress Ratio.

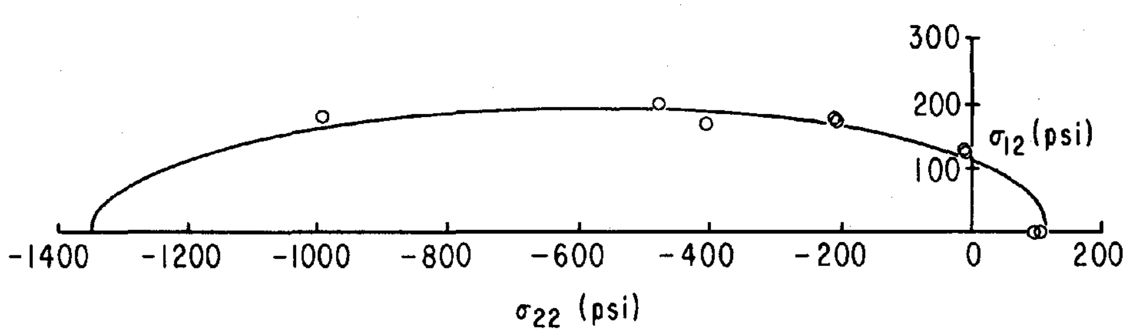



Fig. 3.9. Shear Strength versus Compressive Stress, Batch 6.

# 4. POST-FRACTURE BEHAVIOR

The behavior of masonry following initial fracture is of great importance for a structure subjected to strong seismic loading. If the material possesses some ductility and can sustain a significant part of its original load following fracture, the chances of survival of the structure will be greatly improved. Grouting and steel reinforcing will help to insure the integrity of the material after cracking begins, and the energy absorbed during ductile deformation can be an important source of damping.

The description of masonry behavior once cracking begins is, of course, very difficult, and no well developed theory exists which can accurately predict this behavior. Some of the complexities are load drop at first fracture, stiffness degradation, crack closure, and path dependence. For the case of steel-reinforced masonry, some success has been had with plasticity models. This success was hoped for, since the post-fracture behavior is determined largely by the reinforcing steel, and steel is a material for which the plasticity theory works very well.

## 4.1 Subsequent Loading Surfaces

The failure surface discussed in the previous chapter consists of the stress states at which fracture, and non-linear behavior, commence. Once the material has fractured, it is convenient to have a similar surface, called a subsequent loading surface, that represents the maximum load that can be sustained. Stresses below this surface usually cause little further cracking, and behavior below this surface is generally linear, though reinforced specimens exhibit a large increase in stiffness on transition from tensile to compressive loading.

Upon reloading, the stress-strain curve levels off suddenly as it approaches the loading surface, as shown in Fig. 4.1. The loading surface may suffer a sudden drop at the first tensile fracture, as shown with a different strain scale in Fig. 4.2, and continued deformation associated with stresses on the surface usually causes further cracking, and can cause the surface to change.

Determination of this changing surface is clearly a difficult task. The variety of possible loading paths makes a strictly experimental determination for all cases nearly impossible. The only chance of success is to restrict attention to a limited number of cases, and then to combine experimental results with some understanding of the structure of the material.

Consider first what occurs as cracking begins, for a uniaxial stress state. For uniaxial compression the only data available is that from prism tests, which are described in Section 3.1. As discussed in that section, the behavior of running-bond prisms of more than three courses is expected to correspond closely to that of full-scale masonry.

As shown in Fig. 4.3, the stress-strain curve for a prism compression test has a large linear section, then the load continues to climb a small amount before beginning to drop. The end of the linear part of the curve is believed to mark the onset of cracking. It is this stress, then, that represents a point on the initial yield surface. As deformation continues, this part of the surface expands a small amount, then begins to contract.

The effect of this cracking on other parts of the surface can only be surmised. The grout cores are known to remain intact until well after cracking begins, so the tensile strength probably drops slowly. Gradual disintegration of the material likely causes strength in the opposite direction to decrease: Once the face shells have broken off there can be little strength left across the head joints. Compressive behavior is probably very similar for reinforced and unreinforced masonry.

For the case of uniaxial tension, the behavior depends strongly on whether or not the material is reinforced. The load of an unreinforced specimen drops immediately to zero at the onset of cracking. As shown in Fig. 4.4, the behavior is linear up to the load drop. A reinforced specimen, on the other hand, is able to sustain some load following a sudden drop in load, as shown in Fig. 4.2.

This reduced load must be transmitted across the crack by the reinforcing steel. At the point where the load levels off in Fig. 4.2, the strain is about $9 \times 10^{-5}$, while the stress is 50,000 psi. If the steel, whose total area is 0.61 in$^2$, were strained uniformly, it would carry a load corresponding to a stress of only 3,000 psi. Hence, its load is being transmitted to the masonry. Since the load drops by 40 percent at first cracking, either the masonry suffers some debonding, or the wide steel spacing allows a large section of masonry to remain unloaded. If there were more reinforcing, the load would be transmitted more effectively, and the drop in load might be nearly eliminated.

As the strain continues to increase, the load gradually climbs. Because we were interested in cyclic and reloading behavior, none of the specimens tested in this investigation were loaded monotonically to large strain. However, the envelope for the cyclic curves is believed to be very close to the monotonic curve. As seen in Fig. 4.2, there is an abrupt change in slope as a reloading curve reaches this envelope. This change in slope is associated with the continuation of cracking. Hence, an unload-cycle below this envelope should have little effect on the behavior of the material, and the envelope should represent the monotonic loading curve.

As the strain increases further, the stress is seen to level off. On Fig. 4.1, the stress corresponding to yield of the 60,000 psi reinforcing steel has been indicated. It is seen that at large strain the load is just the maximum load that can be sustained by the yielded steel. For reinforced specimens this large strain produces numerous cracks, as shown in Fig. 4.5, because of the load transmitted by the steel.

Once the material has cracked, its tensile strength becomes very direction dependent. While the strength across the crack drops to a level that depends on the amount of reinforcing steel, the strength in the direction parallel to the crack is probably unaffected. Further, the strength in compression is probably affected little by tensile cracking. Figure 4.2 shows that as a cracked specimen is compressed, the load increases sharply, due to the closing of cracks.

Once these cracks have closed, the specimen is probably able to sustain a load close to its unfractured compressive strength. However, the extensive cracking associated with large strain of a reinforced specimen is likely to cause a general reduction in strength.

This completes the discussion of post-fracture strength for uniaxial stress, except for dependence on crack direction, which was not studied. So there remains the strength under biaxial loading. Because such a large variety of biaxial stress states are possible, knowledge of such behavior is rather sketchy. No tests were conducted under biaxial tension or compression, so little is known about these cases.

Since the biaxial tests that were conducted all had one principal stress tensile, the tests of unreinforced specimens ended at first cracking, for the crack meant that a tensile load could no longer be sustained. For the reinforced specimens, the loading was continued well past first cracking. The loading in these tests was proportional, with a constant ratio of compressive to tensile stress throughout the test. Thus, as cracking began and the tensile stress dropped, the compressive stress was reduced.

Reinforced biaxial tests were conducted at two lay-up angles (as defined in Section 3.3): 45°, with a compressive to tensile stress ratio of 1 to 1; and 70°, with a ratio of 7.5 to 1. Figures 4.6 and 4.7 show the tensile stress versus the corresponding strain for a specimen of each type. A comparison of these two cases with the uniaxial case shown in Fig. 4.1 reveals two items of importance.

First, while there is a load drop as cracking begins for the two biaxial cases, the drop is much less than for the uniaxial specimens. This difference is not well understood, but is likely related to the rather complex crack pattern occurring in the biaxial tests. The initial cracks, marked "1" in Fig. 4.8, fail to cross the specimen completely. Rather, they consist of several isolated short cracks, so that there remain intact segments able to carry load.

These intact segments may survive because at a non-zero lay-up angle both vertical and horizontal reinforcing steel act to prevent cracking. That is, a crack may have to cross three or four bars, instead of just two bars. But this can't be a complete explanation, because the 45° tests show a larger drop than the 70° tests. So it appears that the absence of a large drop in the tensile load must somehow be related to the compressive load in the opposite direction.

The other item to be noted is that at large strain the tensile load has climbed to about the same level (75 psi) for all three cases. For the 0° case, this level is simply the stress corresponding to the ultimate load of the two vertical bars of reinforcing steel. The load approaches this stress quickly, then remains nearly constant.

For the specimens with a lay-up angle different from zero, the behavior at large strain is more difficult to explain. It is seen that the load climbs more slowly, and appears to be still increasing at the largest strain achieved. The reason the load climbs more slowly may be that the steel is able to bend, since it crosses the crack at an angle. And since the crack must cross more bars for these cases, it may be that the maximum load will be higher. The increase would amount to a factor of $\sqrt{2}$ for the 45° case.

These tests thus provide some useful results, but rather meager information for the construction of a post-fracture loading surface. By making some assumptions, however, the task can be accomplished. It will be necessary to remember, of course, that a result of such guessing will be limited accuracy on some parts of the surface. Since there is no information on biaxial compression, this part of the surface will not be described.

It is useful to make some idealizations of the data represented by the loading curves of Figs. 4.1, 4.6, and 4.7. First, the stress will be assumed to drop with no change in strain at first fracture, to the same level, 55 psi, for all values of compressive stress. (If the compressive stress is high enough that fracture occurs at a tensile stress less than 55 psi, there will be no stress drop. It will be assumed that the material simply hardens at the same rate as in other cases.) Second, though the stress for the 45° test shows a tendency to climb, it will be assumed that the stress at large strain is the same, 75 psi, for all conditions. Finally, it will be assumed that the rate of hardening (the increase of tensile stress with tensile strain) following fracture is always $1.5 \times 10^5$ psi. Thus, if $e_t$ is the tensile strain across the crack, and $\sigma_t$ is the corresponding tensile strength, one has

$$\frac{d\sigma_t}{de_t} = 1.5 \times 10^4 \text{ psi}, \; \sigma_t < 75 \text{ psi} \quad . \tag{4.1}$$

Though there were some variations in the tensile stresses immediately following fracture, and at large strain, they were small. It therefore appears that the tensile strength after first fracture must be independent of the compressive stress, in contrast to the situation before fracture. But this is a reasonable result, since the strength is due primarily to the reinforcing steel, which should be little affected by compression perpendicular to its length.

Once the material has fractured, it is convenient to describe behavior in terms of a coordinate system aligned with the crack. Let this coordinate system be unprimed, and let primed coordinates at an angle $\theta$ represent a hypothetical principal stress orientation (Fig. 4.9). In the experiments the crack direction was always a principal stress direction, but it is necessary to be able to consider an arbitrary stress state.

Let $x_2'$ be in the tensile direction, and let $\sigma_t'$ and $\sigma_c'$ represent the tensile and compressive strengths in the $x_2'$ and $x_1'$ directions. It will be assumed that the linear relation between tensile and compressive stresses still holds. That is, for a compressive stress $\sigma_{11}'$, a point on the loading surface will have a tensile stress given by

$$\sigma_{22}' = \sigma_t' \left(1 + \frac{\sigma_{11}'}{\sigma_c'}\right) \quad . \tag{4.2}$$

As noted above, tensile strength in the $x_2$ direction should be independent of the compressive stress $\sigma_{11}$. In the above equation, this can be achieved by letting $\sigma_c'$ go to infinity when $x_1'$ coincides with $x_1$. When $\theta = 90°$, so that the compressive direction $x_1'$ is normal to the crack, the crack will be closed, so that $\sigma_t'$ and $\sigma_c'$ should have their pre-fracture values. Formulas that vary smoothly between these two cases are

$$\begin{aligned} \sigma_t' &= \sigma_t + (\sigma_t^o - \sigma_t) \sin^2 \theta \\ \sigma_c' &= \frac{\sigma_c^o}{1 - \cos \theta} \end{aligned} \quad , \tag{4.3}$$

where $\sigma_t^o$ and $\sigma_c^o$ are the pre-fracture strengths, and $\sigma_t$ is the post-fracture tensile strength described above. Since the compressive stress cannot be arbitrary, the condition $\sigma_{11}' > -\sigma_c^o$ must be added to (4.2).

If (4.2) is rewritten in terms of the unprimed stress components (as was done in Section 3.3), one obtains for the loading surface

$$\sigma_{12}^2 = \sigma_{11}\,\sigma_{22} - \frac{\sigma_t'}{\left(1 + \frac{\sigma_t'}{\sigma_c'}\right)^2} \left(1 + \frac{\sigma_{11} + \sigma_{22}}{\sigma_c'}\right) (\sigma_{11} + \sigma_{22} - \sigma_t'), \quad (4.4)$$

where $\sigma_t'$ and $\sigma_c'$ are given by (4.3), and the magnitude of $\theta$ is given by Eq. (3.6).

Equation (4.4) applies in each of the two quadrants in which the principal stresses are of opposite sign. Data from concrete suggests that for biaxial tension, the strength in each direction is independent of the other stress. Hence, each tensile strength should be given by the corresponding value of $\sigma_t'$ from (4.3). Biaxial compressive behavior is unknown.

## 4.2 Stiffness Degradation

The loading surface discussed in Section 4.1 describes the stresses that fractured masonry can support. Analysis of a structure requires in addition a knowledge of the displacements associated with stresses both on and below the loading surface. Behavior of reinforced masonry below the surface is the subject of this section.

As mentioned in Section 4.1, little further cracking occurs below the loading surface, so the behavior is generally linear. The stiffness depends, of course, on the amount of cracking, and hence can decrease whenever the surface is reached. For the uniaxial tension specimen of Fig. 4.2, the stiffness decreased to about one sixth its original value at first cracking, and Fig. 4.1 shows continued decrease at large strain.

Further, the linearity is only approximate, and holds only within certain regions. Figure 4.2 shows that crack closure associated with transition from tensile to compressive stress increases stiffness to nearly the uncracked level. Therefore, to know the stiffness, one must keep track of the strain in the direction normal to the crack.

It appears, then, the behavior below the loading surface can be treated as linear within each of two regions, which are defined by the condition of crack closure. With the cracks closed, the behavior should be close to the behavior of uncracked masonry described in Section 2. With the cracks open, the material becomes highly anisotropic, with stiffness in the direction normal to the crack dependent on the extent of cracking. In fact, the cracked specimen becomes a new material whose properties can be determined by the same sort of analysis employed in Section 2. The elastic matrix may be more complicated, however, since material symmetry may be lost (e.g., uniaxial tension applied to a specimen with cracks at 45° may produce shear strain).

The complete determination of this changing elastic matrix will clearly require more data than is available from the experiments of this program, but by combining the data that is available with some simple assumptions, one can produce a matrix that should be accurate enough to be useful.

Reinforced specimens were unloaded and reloaded at several tensile strains following initial fracture. (See, for example, Fig. 4.1.) An examination of the slopes of the tensile stress versus tensile strain paths reveals a gradual decrease in stiffness to a limiting value of about $3.8 \times 10^4$ psi, which is just the stiffness due to the two bars of reinforcing steel. (This is the value if the steel is perpendicular to the crack, but the experiments show little variation with crack direction.) This limiting value was reached at approximately the same tensile strain in each of the three tests for which this data is available, so it appears that one should be able to relate stiffness in the tensile direction to the tensile strain.

In order to complete the elastic matrix, some assumptions must be made. The first will be that the compressive stress produces about the same compressive and tensile strains as in the uncracked specimen. The next assumption concerns shear behavior, and is little more than a guess. The shear modulus is certain to decrease with cracking, but there is no data to indicate the rate of decrease. So the second assumption will be that the shear modulus decreases at the same rate as the stiffness in the tensile direction. The final assumption is that the terms relating shear stress to extensional strains are zero. (They must be zero if the crack direction coincides with the reinforcing direction.)

The elastic matrix thus takes the form (in coordinates aligned with the crack direction).

$$\begin{bmatrix} e^e_{11} \\ e^e_{22} \\ e^e_{12} \end{bmatrix} = \begin{bmatrix} 1/D_1 & -\nu/D_1 & 0 \\ -\nu/D_1 & 1/gD_0 & 0 \\ 0 & 0 & 1/2gG \end{bmatrix} \begin{bmatrix} \sigma_{11} \\ \sigma_{22} \\ \sigma_{12} \end{bmatrix}, \qquad (4.5)$$

where $\nu$ and G are the moduli of the uncracked material, $D_0$ is a constant, g is the function of strain that describes the rate of stiffness degradation, and $D_1$ is the stiffness of the uncracked material in the compression direction. The denominator of the Poisson's ratio term is taken to be $D_1$ so that the stress $\sigma_{11}$ won't produce a strain $e^e_{22}$ larger in magnitude than $e^e_{11}$.

The final step is to determine the function g. Tests were conducted on three specimens at different orientations (0°, 45°, and 70°) and load ratios. If their stiffnesses are plotted versus tensile strain, one finds curves of similar shapes, but offset in strain. Such an offset is suggested independently by the following consideration. If a specimen suffers stiffness degradation at fracture, and is immediately unloaded, the strain $e_{22}$ will become negative if the stiffness is small enough. To prevent such an occurrence, one can require the stiffness at fracture to have just that value that will yield zero tensile strain at zero stress. That stiffness then determines, through the function g, an offset in strain.

The shape of the stiffness degradation curves is about that of $1/e_{22}$. The constant $D_0$ is needed so that g will have an initial value close to one. Hence, let

$$D_o = 1.5 \times 10^6 \text{ psi} \quad ,$$

and for small $e_{22}$ let

$$g(e_{22}) = \frac{1}{2.3 \times 10^4 \; (e_{22} - \delta)} \quad , \tag{4.6}$$

where $\delta$ is the offset to be determined by the above procedure, and the coefficient $2.3 \times 10^4$ was chosen by fitting the data. For large $e_{22}$, one has $gD_o = 3.8 \times 10^4$ psi.

For the three specimens that were tested, one can find the strain $e_{22}$ at fracture from the known elastic law and the law for first fracture. This strain is given in the first row of Table 4.1. Since the tensile stress drops at fracture, one has a new value of $\sigma_{22}$. With this new value, and the calculated value of $e_{22}$, Eq. (4.5) then can be solved for $gD_0$ (given in the second row of Table 4.1). Finally, Eq. (4.6) can be solved for $\delta$, given in the last row of Table 4.1.

Table 4.2 gives the measured stiffnesses for the three specimens, and in Fig. 4.10 these are plotted versus $e_{22} - \delta$. The curve is given by Eq. (4.6) with $\delta = 0$ for small strain, with the constant value $3.8 \times 10^4$ psi for large strain, and is seen to represent the data quite well.

## 4.3 Anelastic Strain

A knowledge of the elastic strain discussed in Section 2 is needed for one to predict the commencement of fracture. But this strain, and even the elastic strain associated with the reduced stiffness of the above section, can be small compared to the anelastic strain. Therefore, a theory that relates anelastic strain to the state of stress is necessary in order to predict the response of a structure that has suffered fracture. Because of the complexity of the behavior of fractured masonry, perfect agreement with theory cannot be expected, and only a small number of cases can be checked, but some success has been achieved for reinforced masonry.

The first step in forming such a theory is to define anelastic strain. It is taken to be, simply, the strain that would remain if the stress were removed. (For definiteness, the stress path is specified as a straight line to the origin.) In the theory of plasticity, the plastic strain is the difference of the total strain, and the strain computed from the stress through the elastic law. But for fractured masonry, as discussed in the previous section, the elastic behavior can change substantially as cracking occurs. Hence, it is this modified elastic law that must be employed in calculating anelastic strain. Since our tests included several unloading paths, this information on elastic behavior is available.

With a procedure established for determining the anelastic strains, one can now look for some pattern that relates these strains to the stress state. As mentioned earlier, it was hoped that the influence of reinforcing steel would result in behavior close to that of the theory of plasticity. We have seen that the concepts of initial yield surface and subsequent loading surfaces do hold, with some modifications. So the final step is to see if the anelastic strains can be modeled as plastic strains.

In the theory of plasticity, the anelastic part of the strain is zero except on the yield or loading surfaces, and on these surfaces is determined only in direction, with the magnitude left undetermined. This direction is specified by the flow rule. Our tests have shown essentially elastic behavior below the loading surfaces, so it remains to find a flow rule.

Plasticity theories frequently employ an associated flow rule - one that is derived from the yield surface [ 9 ]. For many materials the increment of plastic strain is approximately normal to the yield or loading surface. More specifically, it is the increment of plastic strain whose inner product with the stress represents work, that is normal to the surface. Thus, if the surface is expressed in the stress space ($\sigma_{11}$, $\sigma_{22}$, $\sigma_{12}$), the vector ($de^p_{11}$, $de^p_{22}$, $2de^p_{12}$) is normal to the surface.

As explained above, though, these surfaces have not been completely determined for masonry. Only a plane section through the initial yield and loading surfaces has been checked by experiment. However, the part of the surface where tests were conducted is believed to be given by Eq. (4.4), so the flow direction was compared to its normal.

The normal direction is given by the gradient of a function that is constant on the surface represented by Eq. (4.4), for example $f(\sigma_{ij})=0$, where

$$f(\sigma_{ij}) = \sigma_{12}^2 + \frac{\sigma_t'}{\left(1 + \frac{\sigma_t'}{\sigma_c'}\right)^2} \left(1 + \frac{\sigma_{11} + \sigma_{22}}{\sigma_c'}\right) (\sigma_{11} + \sigma_{22} - \sigma_t') - \sigma_{11}\,\sigma_{22}. \tag{4.7}$$

The dependence of $\sigma_t'$ and $\sigma_c'$ on direction makes derivatives somewhat complicated. However, the normal direction is affected little by this anisotropy, so $\sigma_t'$ and $\sigma_c'$ were considered constant, giving

$$\frac{\partial f}{\partial \sigma_{11}} = \frac{\sigma_t'}{\left(1 + \frac{\sigma_t'}{\sigma_c'}\right)^2} \left(1 + \frac{2(\sigma_{11} + \sigma_{22}) - \sigma_t'}{\sigma_c'}\right) - \sigma_{22}$$

$$\frac{\partial f}{\partial \sigma_{22}} = \frac{\sigma_t'}{\left(1 + \frac{\sigma_t'}{\sigma_c'}\right)^2} \left(1 + \frac{2(\sigma_{11} + \sigma_{22}) - \sigma_t'}{\sigma_c'}\right) - \sigma_{11}. \tag{4.8}$$

$$\frac{\partial f}{\partial \sigma_{12}} = 2\,\sigma_{12}$$

Dividing by the magnitude of the gradient gives a unit vector normal to the surface.

The anelastic strain increments were determined from plots of stress versus strain. As shown in Fig. 4.1, load was applied and removed in several steps, which represent convenient strain increments. The anelastic strain is the change in strain at zero load, which is easily determined for the tests shown in Fig. 4.1.

As discussed in Section 4.2, the behavior is fairly linear below the loading surface, and since the applied stresses were proportional, the loading and unloading curves are fairly straight. Thus, for tests that did not return all the way to zero load, the strain can be determined quite accurately by extrapolation.

The flow direction was calculated at several strains for a reinforced specimen at each of the three lay-up angles tested. Because the loading system was capable of applying only direct stresses, there was no way to determine the flow direction for the important case of shear stress on the crack plane. And since loading was proportional, for each specimen only a single point on the surface was investigated. For this point, the normal direction from Eq. (4.8) is simply (0,1,0).

The angles between this normal and the increments of plastic strain are given in Table 4.3. Though agreement is not perfect, the normal is seen to give a good indication of flow direction. Thus, the loading surface given by Eq. (4.4), along with the flow rule (4.8), comprise a plasticity model that should give a fair representation of the behavior of reinforced masonry.

## 4.4 Stress Rate-Total Strain Rate Law

The results of preceding sections define a model of post-fracture behavior, but for computations it is useful to have a set of equations that relate the rate of change of stress to the rate of change of total strain. These equations will be more complicated than those of standard plasticity because of the changing elastic moduli.

For convenience, let stress and strain components be identified by a single subscript:

$$\begin{bmatrix} \sigma_1 \\ \sigma_2 \\ \sigma_3 \end{bmatrix} = \begin{bmatrix} \sigma_{11} \\ \sigma_{22} \\ \sigma_{12} \end{bmatrix} , \quad \begin{bmatrix} e_1 \\ e_2 \\ e_3 \end{bmatrix} = \begin{bmatrix} e_{11} \\ e_{22} \\ e_{12} \end{bmatrix} .$$

Let $e_2$ be the tensile strain normal to the crack, and let $C_{ij}(e_2)$ be the changing elastic moduli (from Eq. (4.5)):

$$C_{ij}\ (e_2) = \begin{bmatrix} \dfrac{D_1}{1 - g\nu^2 \dfrac{D_0}{D_1}} & \dfrac{g\nu D_1}{\dfrac{D_1}{D_0} - g\nu^2} & 0 \\ & \dfrac{g\ D_0}{1 - g\nu^2 \dfrac{D_0}{D_1}} & 0 \\ \text{sym.} & & 2gG \end{bmatrix}$$

Let f be the loading function (Eq. (4.7)), let rate of change be indicated by a dot, and let the derivative with respect $\sigma_i$ be indicated by a comma followed by i. Then the equations that describe behavior on the loading surface are

$$f(\sigma_i,\ e_2) = 0 \tag{4.9}$$

$$e_i = e_i^e + e_i^p \tag{4.10}$$

$$\sigma_i = C_{ij}(e_2)e_j^e \tag{4.11}$$

$$\dot{e}_i^p = \lambda f_{,i} \tag{4.12}$$

where $\lambda$ is a changing parameter to be determined.

Take the derivative of (4.11) and use the derivative of (4.10), and (4.12):

$$\begin{aligned}\dot{\sigma}_i &= \dot{C}_{ij}\ e_j^e + C_{ij}\ \dot{e}_j^e \\ &= \dot{C}_{ij}\ e_j^e + C_{ij}(\dot{e}_j - \dot{e}_j^p) \\ &= \dot{C}_{ij}\ e_j^e + C_{ij}\ \dot{e}_j - \lambda\ C_{ij}\ f_{,j} \quad .\end{aligned}$$

Or,

$$\dot{\sigma}_i = \frac{dC_{ij}}{de_2}\ e_j^e\ \delta_{k2}\ \dot{e}_k + C_{ij}\ \dot{e}_j - C_{ij}\ \lambda f_{,j} \quad , \tag{4.13}$$

where $\delta_{k2}$ is the Kronecker delta. From (4.9):

$$0 = \dot{f} = f_{,i}\ \dot{\sigma}_i + \frac{\partial f}{\partial e_2}\ \delta_{k2}\ \dot{e}_k \quad .$$

Multiply (4.13) by $f_{,i}$ and use the above equation:

$$-\frac{\partial f}{\partial e_2}\ \delta_{k2}\ \dot{e}_k = \frac{dC_{ij}}{de_2}\ f_{,i}\ e_j^e\ \delta_{k2}\ \dot{e}_k + C_{ij}\ f_{,i}\ \dot{e}_j - C_{ij}\ \lambda\ f_{,i}f_{,j} \quad .$$

Solve for $\lambda$:

$$\lambda = A_k\ \dot{e}_k \quad ,$$

where

$$A_k = \frac{\frac{dC_{ij}}{de_2} f_{,i} e^e_j \delta_{k2} + C_{ik} f_{,i} + \frac{\partial f}{\partial e_2} \delta_{k2}}{C_{ij} f_{,i} f_{,j}} \tag{4.14}$$

Putting this expression in (4.13) gives

$$\dot{\sigma}_i = \left[\frac{dC_{ij}}{de_2} e^e_j \delta_{k2} + C_{ik} - C_{ij} f_{,j} A_k\right] \dot{e}_k \quad , \tag{4.15}$$

which is the desired result. The partial derivatives $f_{,i}$ are given in Eq. (4.8), and

$$\frac{\partial f}{\partial e_2} = \frac{1 + \frac{\sigma_1 + \sigma_2}{\sigma'_c}}{\left(1 + \frac{\sigma'_t}{\sigma'_c}\right)^2} \left[\frac{1 - \frac{\sigma'_t}{\sigma'_c}}{1 + \frac{\sigma'_t}{\sigma'_c}} (\sigma_1 + \sigma_2 - \sigma'_t) - \sigma'_t\right] \frac{d\sigma'_t}{de_2} \quad .$$

As an example of the application of this relation, consider a loading program for which $\sigma_3 = 0$ (no shear stress on the crack plane after fracture). Thus $\theta = 0$, so $\sigma'_t = \sigma_t$, and from (4.3), $\sigma'_c = \infty$. Equation (4.8) thus gives

$$f_{,1} = 0$$

$$f_{,2} = \sigma_t - \sigma_1$$

$$f_{,3} = 0 \quad .$$

Hence, from (4.14),

$$A_2 = \frac{\frac{dC_{2j}}{de_2} f_{,2} e^e_j + C_{22} f_{,2} + \frac{\partial f}{\partial e_2}}{C_{22} f^2_{,2}} \quad ,$$

and if $\sigma_1 = 0$, $\dot{e}_1$ should be small, then from (4.15),

$$\dot{\sigma}_2 = \frac{d\sigma_t}{de_2}\,\dot{e}_2 \quad ,$$

where $d\sigma_t/de_2$ is given by Eq. (4.1). The loading path represented by this result, together with several unloading paths from (4.5), are shown in Fig. 4.11. It compares well with the experimental result of Fig. 4.1.

Table 4.1. Strain Offsets for Stiffness Curve

| $-\sigma_{11}/\sigma_{22}$ | 0 | 1 | 7.5 |
|---|---|---|---|
| $(e_{22})_{frac.} \times 10^4$ | 0.50 | 0.93 | 1.30 |
| $gD_o$ ($10^6$psi) | 1.08 | 0.65 | 0.77 |
| $\delta \times 10^4$ | -0.10 | -0.07 | 0.45 |

Table 4.2. Measured Stiffnesses

| | | | | | |
|---|---|---|---|---|---|
| | | $-\sigma_{11}/\sigma_{22} = 0$ | | | |
| $e_{22} \times 10^4$ | 1.8 | 3.5 | 9 | 19 | 28 |
| stiffness ($10^4$psi) | 20.0 | 12.6 | 7.4 | 4.9 | 4.0 |
| | | $-\sigma_{11}/\sigma_{22} = 1$ | | | |
| $e_{22} \times 10^4$ | 1.6 | 4.0 | 2.0 | | |
| stiffness ($10^4$ psi) | 64 | 16.8 | 4.0 | | |
| | | $-\sigma_{11}/\sigma_{22} = 7.5$ | | | |
| $e_{22} \times 10^4$ | 3.5 | 8.0 | 15 | 25 | |
| stiffness ($10^4$ psi) | 18.8 | 8.7 | 6.0 | 3.9 | |

Table 4.3. Angle between Plastic Strain Increment and Normal to Loading Surface.

| θ | Stress Ratio | Tensile Strain X $10^4$ | Angle |
|---|---|---|---|
| 0° | 0 | 7 | 4° |
| | | 13 | 1° |
| | | 20 | 7° |
| 45° | 1.0 | 1.6 | 11° |
| | | 4 | 40° |
| | | 20 | 28° |
| | | 70 | 4° |
| 70° | 7.5 | 3.6 | 16° |
| | | 8 | 30° |
| | | 22 | 30° |
| | | 60 | 36° |

## 5. SUMMARY

The behavior before cracking is seen to be quite linear, and its complete description is given by the elastic matrix, which shows substantial anisotropy. The conditions for initial cracking show much less anisotropy, and are known with quite good accuracy. Statistical methods give the variation in strength that can be expected. The cases of biaxial tension and compression need to be studied.

The behavior following cracking is very complex, but for reinforced masonry a plasticity model gives good results. The reduced strength and stiffness are seen to depend in a simple way on the tensile strain and amount of steel, and plastic strains show fair agreement with theory. This agreement needs to be checked at several more stress states.

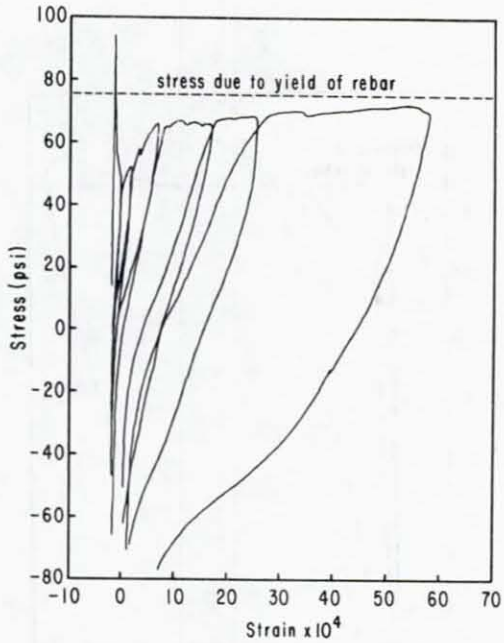



Fig. 4.1. Reinforced 0° Uniaxial Test-Large Strain.

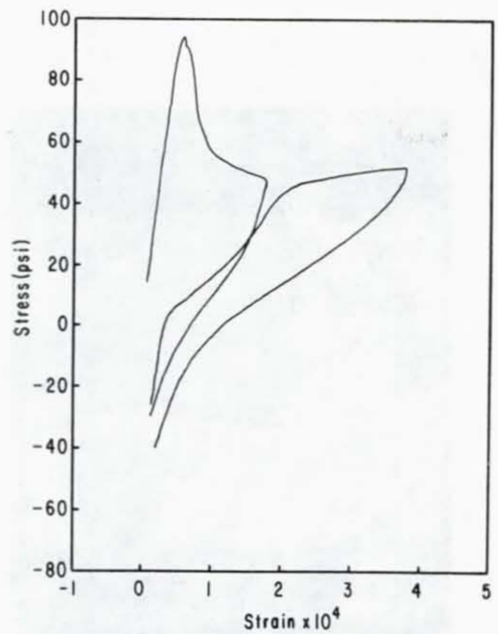



Fig. 4.2. Reinforced 0° Uniaxial Test-Small Strain.

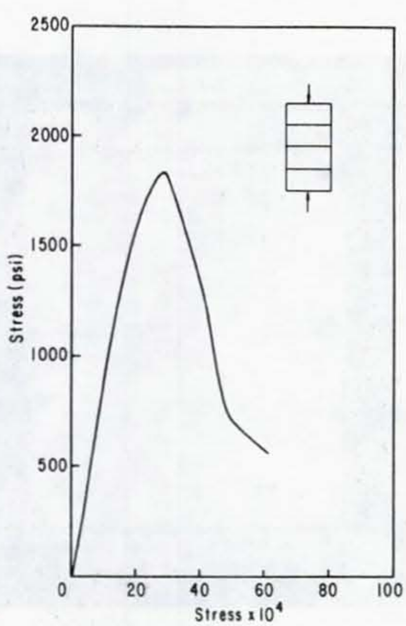



Fig. 4.3. Four-Course Stack-Bond Prism Test.

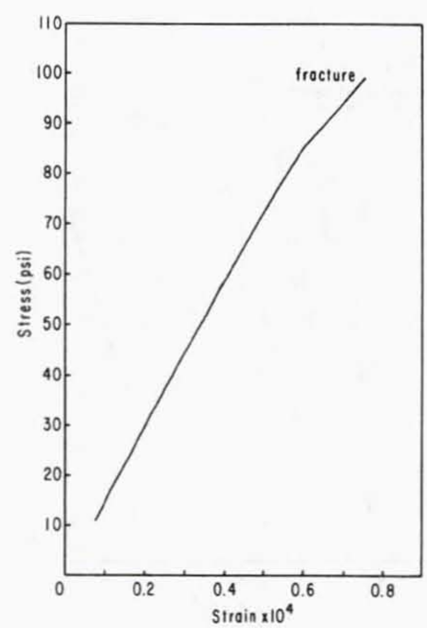



Fig. 4.4. Unreinforced 0° Uniaxial Tension Test.

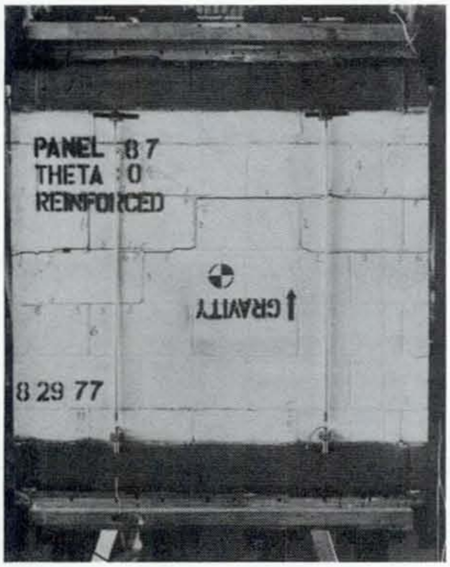



Fig. 4.5. Reinforced 0° Uniaxial Specimen.

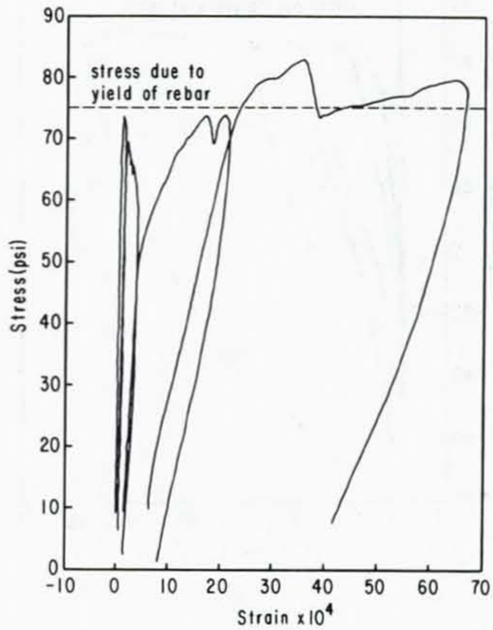



Fig. 4.6. Reinforced 45° Biaxial Test.

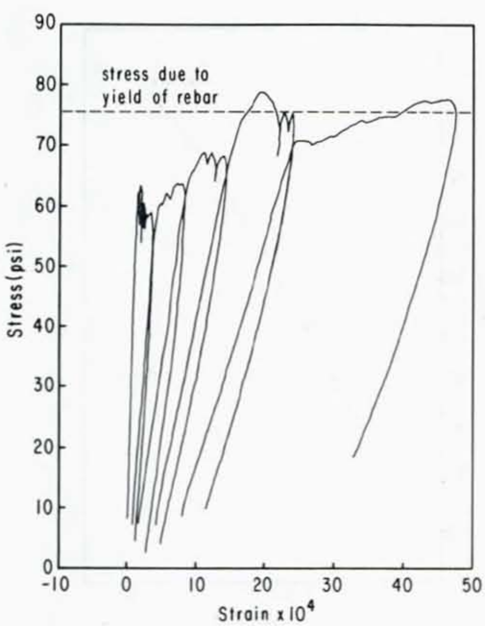



Fig. 4.7. Reinforced 70° Biaxial Test.

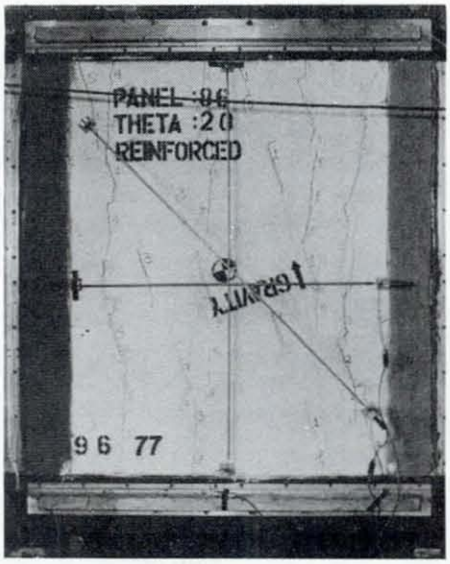



Fig. 4.8. Reinforced 70° Biaxial Specimen.

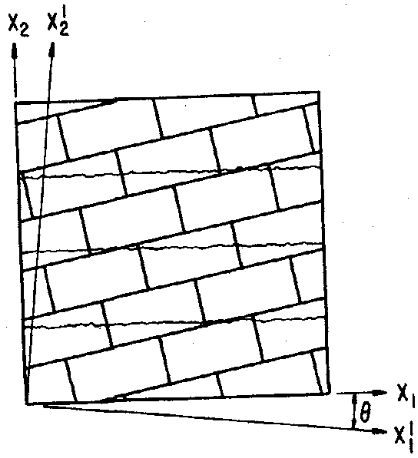



Fig. 4.9. Principal Stress and Crack Coordinate Systems.

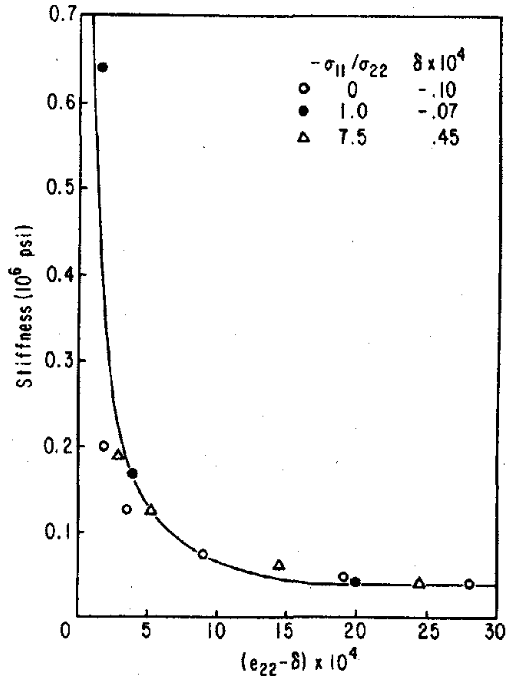



Fig. 4.10. Rate of Stiffness Degradation.

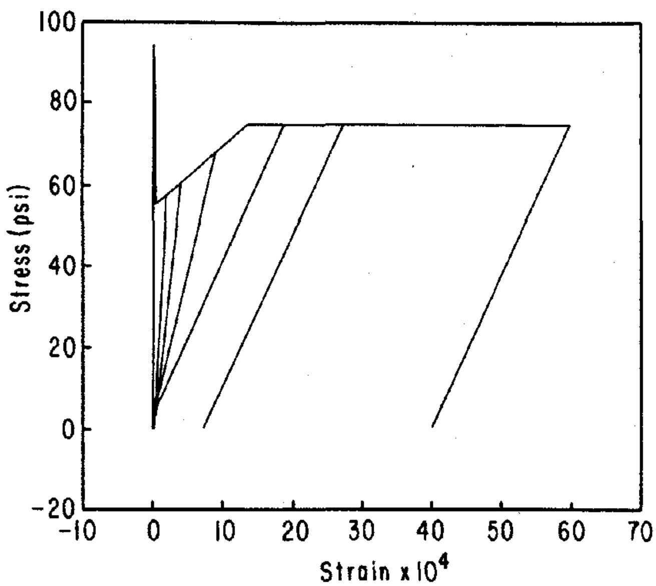



Fig. 4.11. Analytical Loading Path for $\sigma_{11} = 0$.

# APPENDIX I.--REFERENCES

1. Wiegel, Robert L., coordinating editor, Earthquake Engineering, Prentice-Hall, Englewood Cliffs, N. J., 1970, pp. 167-226.

2. Mayes, Ronald L., et al., "Cyclic Shear Tests of Masonry Piers, Volume 1 - Test Results," Report No. EERC 76-8, University of California, Berkeley, May, 1976.

3. Timoshenko, S. P., and J. N. Goodier, Theory of Elasticity, third edition, McGraw-Hill, New York, 1970, pp. 17-19.

4. Hegemier, G. A., et al., "Prism Tests for the Compressive Strength of Concrete Masonry," Report No. AMES-NSF TR-77-1, University of California, San Diego, 1977.

5. Fung, Y. C., Foundations of Solid Mechanics, Prentice-Hall, Englewood Cliffs, N. J., 1965, pp. 347-349.

6. 1977 Masonry Codes and Specifications, printed and distributed by the Masonry Industry Advancement Committee, April, 1977.

7. Kupfer, Helmut B., and Kurt H. Gerstle, "Behavior of Concrete under Biaxial Stresses," Journal of the Engineering Mechanics Division, ASCE, 1973, pp. 853-866.

8. Walpole, R. E., and R. H. Myers, Probability and Statistics for Engineers and Scientists, second edition, Macmillan, New York, 1978, pp. 282-291.

9. Goodier, J. N., and P. G. Hodge, Jr., Elasticity and Plasticity, Wiley, New York, 1958, pp. 57-61.

# SEISMIC BEHAVIOR OF MASONRY PIERS

By McNiven, H.D., and Mayes, R.L.

ABSTRACT: The program of research on the seismic behavior of masonry piers at the Earthquake Engineering Research Center. The University of California, Berkeley started in 1972 and has continued since that date. This paper is a survey and report on this total program. Almost all of the research that has been completed has already been described in detail in a number of reports issued by the Center, and so that part of the report will be covered briefly. The bulk of the report will be devoted to what we plan to do in the immediate future.

# SEISMIC BEHAVIOR OF MASONRY PIERS

By Hugh D. McNiven[1] and Ronald L. Mayes[2]

## INTRODUCTION

The masonry research program was initiated at the Earthquake Engineering Research Center, University of California, Berkeley, in September 1972. There have been no interruptions to the program since that date. The program has been devoted to a study of the behavior of masonry elements subjected to cyclic lateral loads. The program began with a study of seventeen in-plane shear tests on double-piered test specimens. These tests were followed by an extensive series on single piers. The program on single piers will continue but with a new test set up. This report coincides closely with the beginning of the new series. Eventually we plan to conduct a number of tests on spandrel girders.

Even though this report will cover the complete history of the program, the part of it that has been completed will be covered briefly. This is because the test program to date has been reported in detail in a number of reports from the Center.

We will discuss the new test set up, how it differs from the old, and how it opens up a wide range of test conditions. Considerable attention will be devoted to our future plans and what we will be attempting to accomplish.

## OBJECTIVE

The object of the program is to try to improve the behavior of masonry when it is subjected to seismic loads. By improve we mean to enhance the ability of masonry to undergo large deformation without failure and to improve its ability to absorb energy. We will discuss in the body of the paper how we measure these abilities.

The behavior of a masonry pier depends first of all on its mode of failure. The first step in the study of a particular pier is to identify this mode.

Because most failures in past earthquakes have been characterized by diagonal cracks, many research programs have concentrated on this type of failure mechanism. Test techniques used by Blume, Greenley and Cattaneo, and others, induce diagonal tension or shear mode of failure. Scrivener, Meli, Williams, and Priestly and Bridgeman recognize that there are two possible modes of failure for cantilever piers. In addition to the shear or diagonal tension mode of failure, they recognize that for certain

---

[1]Professor of Engineering Science University of California, Berkeley

[2]Assistant Research Engineer, Earthquake Engineering Research Center, University of California, Berkeley, and Principal, Computech, Berkeley, U.S.A.

piers, a flexural failure could occur. This mechanism is characterized by yielding of the tension steel of the wall, followed by a secondary compressive failure at the toe, with associated buckling of the reinforcement once confinement is lost.

When the mode of failure is identified along with its cause, improvement in behavior will be treated differently for each of the two modes. These improvements will be discussed in the body of the paper.

## TEST SET UP

After an extensive review of the literature dealing with earthquake resistance of masonry, it was concluded that exterior wall panels penetrated by numerous window openings (Fig. 1) were the components of multistory masonry buildings most frequently damaged in earthquakes, and it was decided to make an experimental study of the seismic behavior of such components. A testing fixture was designed to subject typical full-scale window piers to combined static vertical (gravity) and cyclic lateral (seismic) loads (Fig. 2). The test equipment permits lateral loads to be applied in the plane of the piers, using displacement controlled actuators with a maximum capacity of 450 kips. A vertical load may be applied to the piers through the springs and rollers shown above the spandrel beam in Fig. 2. The double pier tests were carried out with initial bearing stresses varying from zero to 500 psi.

With this load applied we next were able to impose a horizontal displacement at the top of the wall, while the bottom remained fixed. This displacement was applied by means of an actuator equipped with a load cell to record the load necessary to realize the displacement. The displacement was applied cyclically with a frequency of 0.02 $H_Z$ so that inertial effects are minimal. The amplitudes of displacement were imposed in groups of three and were increased montonically until the wall could no longer resist the horizontal load. We also followed the ability of the wall to sustain horizontal load after damage began and increased; that is after the horizontal resistance began to diminish.

The data resulting from a series of tests on a particular pier were plotted as horizontal resisting force vs horizontal displacement so that a hysteresis loop was formed for each cycle of loading. The complete data for a test would take the form of a series of hysteresis loops. A typical array of loops is shown in Figure 3. For reasons that will be shown later, a line representing the envelope of the complete array of hysteresis loops, proves to be extremely useful. A number of such envelopes is shown in Figure 4.

These double pier tests were successful because the specimen used reproduced faithfully the conditions in a highrise building. Each specimen however proved to be prohibitively expensive both in terms of time to build and test and in cost. Because of this, the decision was made to continue the program using single pier specimens. These could be tested for a fraction of the cost of the double pier specimens.

The test set up for the single piers is shown in Figure 5. The figure shows that rotation of the pier at the top is prevented by vertical

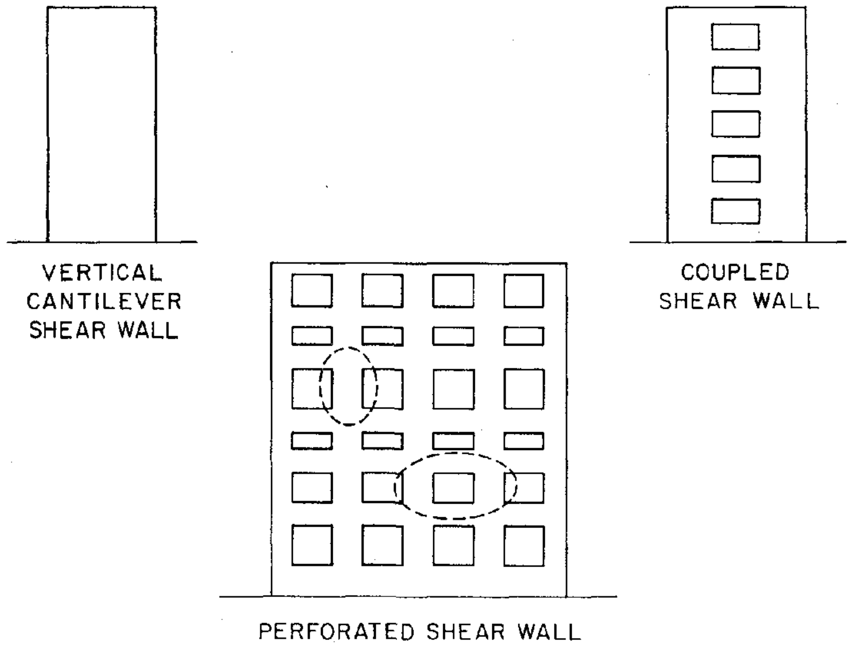



FIGURE 1: TYPICAL MASONRY BUILDING

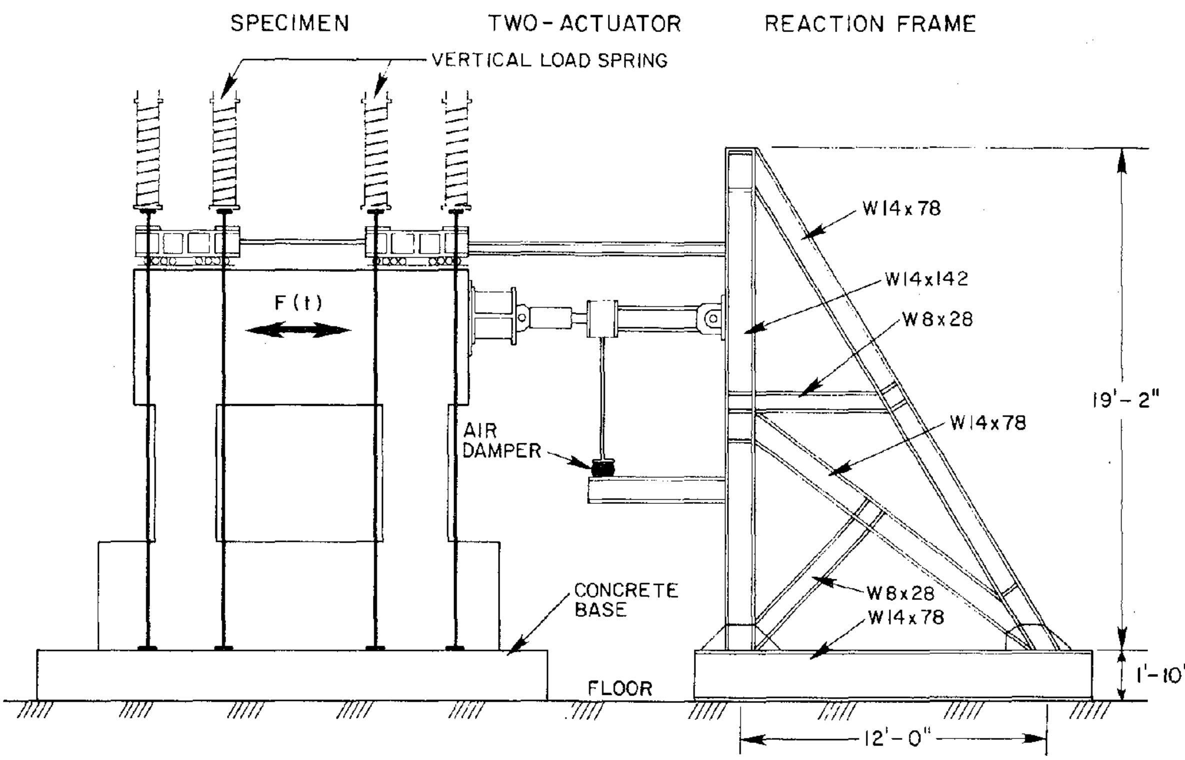
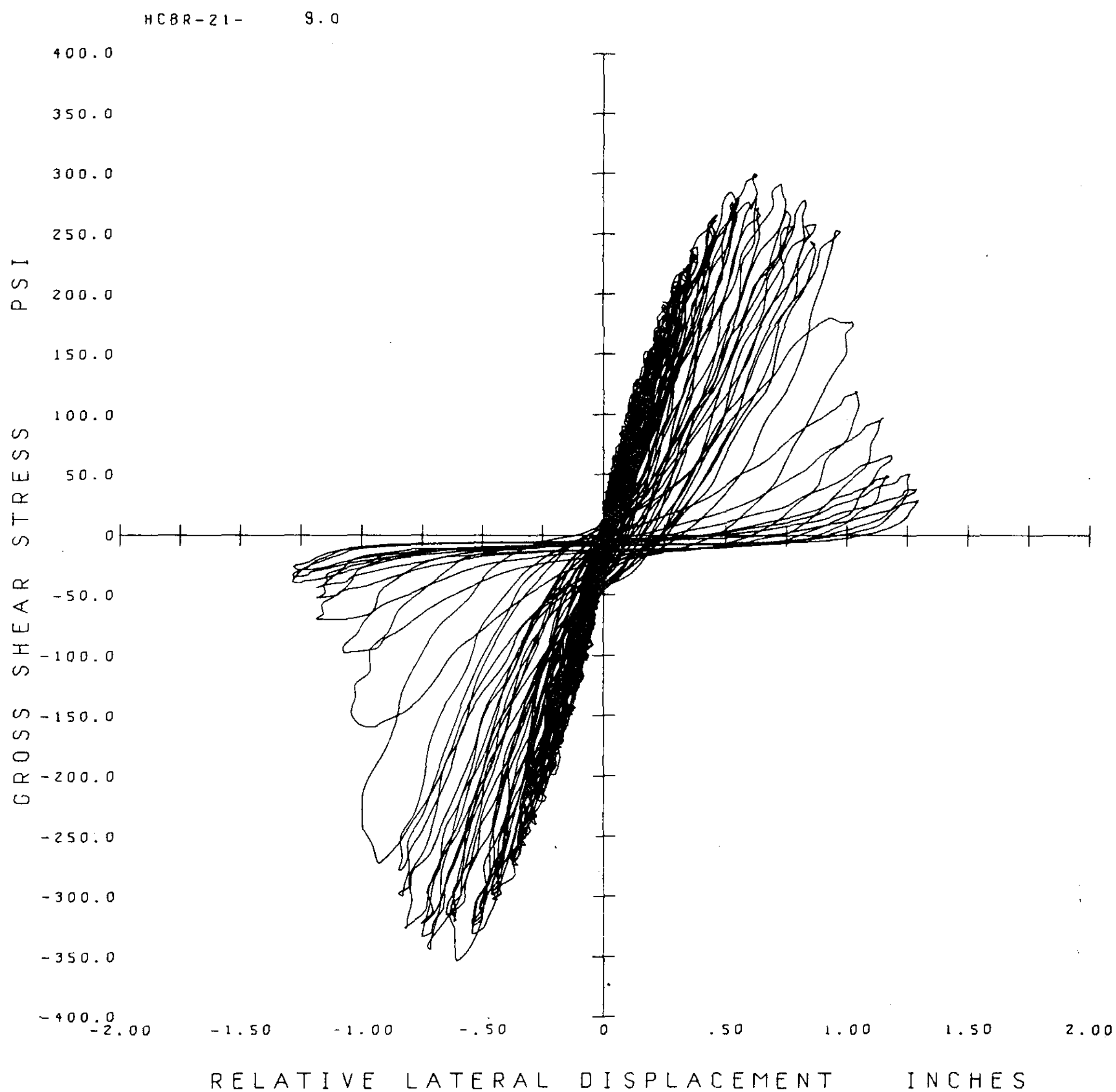



FIGURE 2: TEST SET UP FOR DOUBLE PIERS




SHEAR-STRESS-DEFLECTION DIAGRAM

FIGURE 3: ARRAY OF HYSTERESIS LOOPS

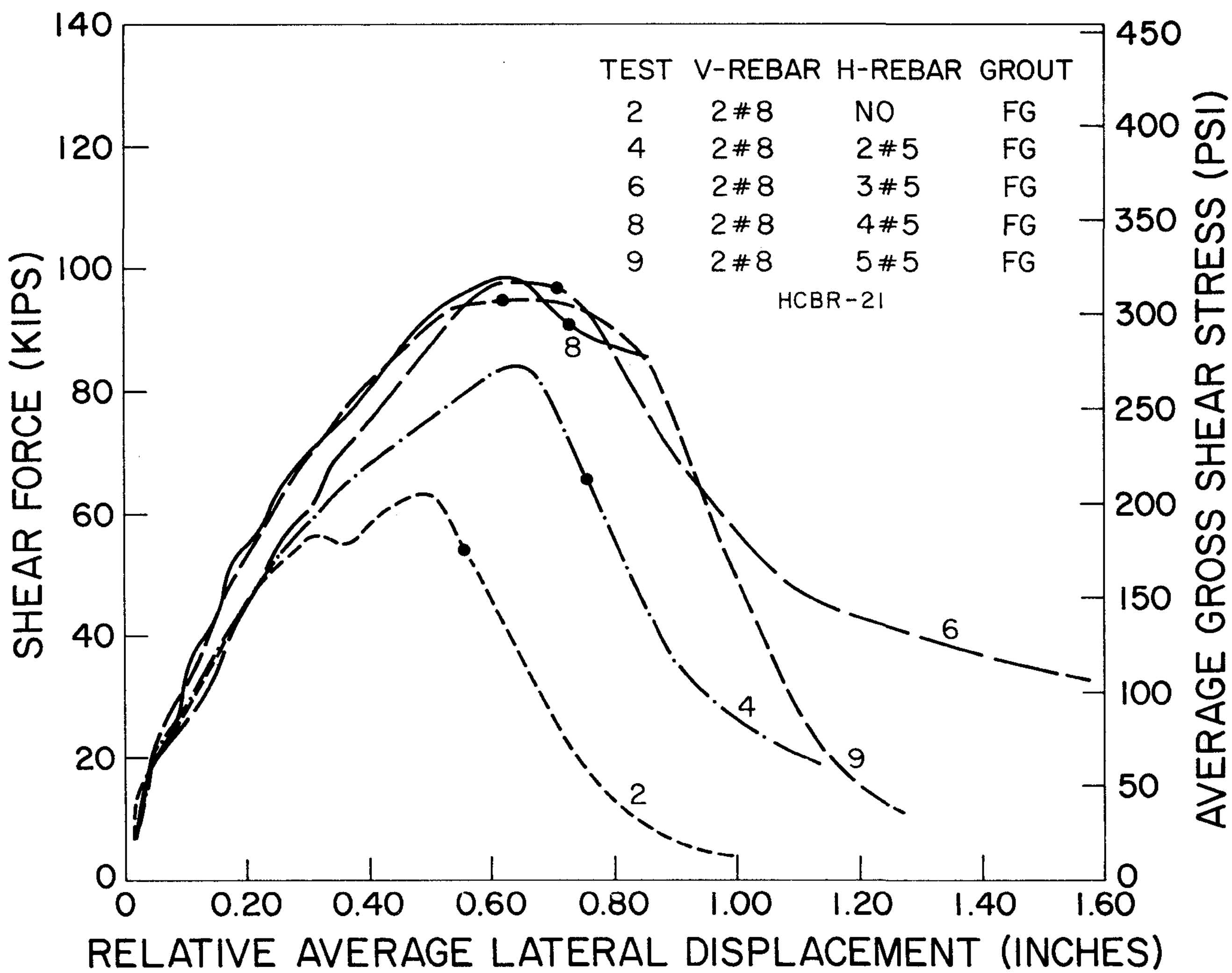



FIGURE 4: TYPICAL HYSTERESIS ENVELOPES

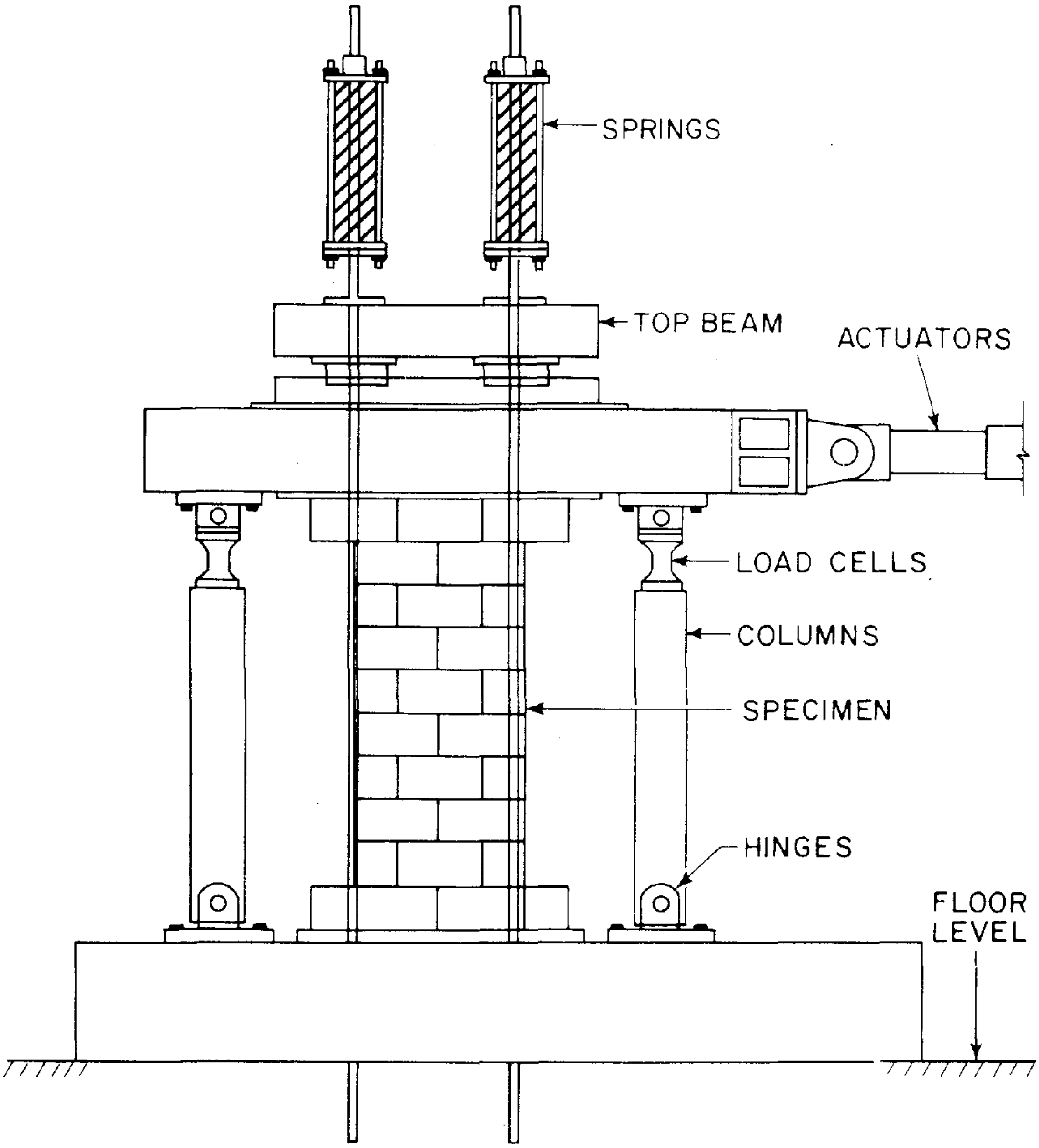



FIGURE 5: SINGLE PIER TEST SET UP

steel members connecting boundary beams at the top and bottom.

Just recently we have changed the test setup in a significant way. The vertical rotation constrainers have been replaced by actuators. These actuators can be used to impose a specified vertical load on the pier, each exerting the same downward force. Superimposed on these forces are additional equal and opposite forces which impose a moment at the top resulting in a rotation. These vertical actuators are coupled by means of a servo-mechanism to the horizontal actuators which impose the horizontal displacements at the top of the pier. As the horizontal displacement changes, the force in the horizontal actuators changes, and in turn the forces in the vertical actuators change and change by whatever amount we choose.

## TEST RESULTS

The basic product obtained from the tests was the hysteresis loops diagram, which is a plot of the lateral load against the lateral displacement of the piers. The strength and deformation properties, the stiffness degradation and the energy dissipation characteristics of the piers can be obtained from the hysteresis loops. The following sections present some of the results obtained during this investigation. In particular, the modes of failure observed, the ultimate lateral load strengths associated with them and a discussion of the inelastic behavior of the piers, as affected by the different parameters, are presented.

## MODES OF FAILURE

Two principal modes of failure were observed during the series of tests, a flexural and shear mode of failure. Sliding modes of failures associated with either flexural or shear cracks were also observed in the piers with height to width ratio of 0.5.

A flexural mode of failure was obtained in two of the HCBL-21, double pier test specimens. The mode was identified as flexural in the following way. The specimens only had horizontal cracks at the top and bottom sections and the ultimate strength of the pier was controlled by the tensile yielding strength of the vertical reinforcement. The final mechanism of failure was due to crushing at the compressive toe of the pier.

Eighty percent of the piers exhibited a shear mode of failure. This mode was characterized by early flexural cracks at the toes of the pier which were later augumented by diagonal cracks that extended through a partial zone of the pier. As the horizontal load increased, large diagonal cracks (x-cracks) formed when the diagonal tensile stress in the pier reached the tensile strength capacity of the masonry. Some of the single piers with height to width ratio of 2 or 1 exhibited yielding in the vertical reinforcement before the occurrence of the major diagonal cracks. However, as the vertical compressive load induced by the single pier test setup increased, the flexural moment capacity of the pier sections also increased while the tension vertical reinforcement continued to yield. This effect allowed the lateral load on the pier to increase until the diagonal tensile stress reached the tensile strength of the masonry and a shear failure developed.

Ten of the piers with a height to width ratio of 0.5 developed a sliding mode of failure along paths determined by previous cracks. In five of the piers sliding occurred along the bottom section of the pier and was prompted by flexural cracks developed along that section. In the other five piers, the final failure mechanism included a combination of shear cracks and sliding along a path determined by these diagonal cracks and the top section of the pier, (a bell-shape path). In most of the piers exhibiting a sliding mechanism of failure, major diagonal cracks had developed before the final failure was attained.

## ULTIMATE STRENGTH

The ultimate lateral load strength of each pier is determined by the lesser of the lateral load capacities associated with each of the modes of failure. The ultimate strength associated with the two sliding modes of failures described above proved to be quite similar to that obtained with the shear mode of failure. Therefore, the following discussion will be restricted to the flexural and the shear modes of failure.

The "flexural lateral load capacity", (lateral load capacity associated with flexural mode of failure), is a function of the tensile yield strength of the vertical reinforcement, the applied axial load and the dimensions of the pier. The methods suggested to predict the flexural lateral load capacity of a pier are similar and reasonably accurate, and are based on methods commonly used for reinforced concrete flexural elements. If all of the tension steel is assumed to be yielding, and the moment of the resultant of compressive forces around the extreme compression fiber is neglected, the moment capacity of a section under an axial compressive force N is given by

$$M = \Sigma A_{si} f_y d_i + N \frac{d}{2} \tag{1}$$

where $d_i$ is the distance between the vertical reinforcing bar with area $A_{si}$ and the extreme compressive fiber, d is the width of the pier and $f_y$ is the yield strength of the vertical reinforcement. If $M_b$ and $M_t$ denote the moment capacity of the bottom and top sections of a pier of height h, the flexural lateral load capacity of a pier fixed against rotation at both top and bottom sections is

$$P = \frac{1}{h} (M_t + M_b) \tag{2}$$

The "shear lateral load capacity", (lateral load capacity associated with the shear mode of failure), may be defined at two levels. The "shear crack strength" is defined as the lateral load required to produce the first major diagonal crack; the "ultimate shear strength" is the maximum lateral load developed by the piers. In the case of the piers with height to width ratios of 2 or 1, both quantities are the same. In the case of the squat piers, (height to width ratio of 0.5), the lateral load continued to increase after the occurrence of the first major diagonal crack because the compression toe of the pier was wide enough to carry a significant shear. Increased amounts of cracking finally produced the failure of the pier at ultimate loads that exceeded the shear crack strength by percentages varying from 5% (CBRC piers), to 11% (HCBR piers), to 67% (HCBL piers). In what follows, an effort to predict the shear crack strength of the piers, using the experimental data obtained throughout the test program, is presented.

Concurrent with the erection of the fully grouted piers, prisms and square panels were constructed using the same mortar, grout and masonry units. The prisms were one block or brick wide, had the same thickness as the piers and a height five times the thickness. The square panels had the same thickness as the piers and the panel dimension was either 32 in. (HCBL) or 36 in. (HCBR and CBRC). The prisms were tested in uniaxial compression and the panels in diagonal compression. These were performed during tests of corresponding piers. Table 1 presents the results of the prism compressive strengths $f'_m$, the panel critical tensile strengths $\sigma^o_{tcr}$ as formulated by Blume, the pier strength associated with the occurrence of the first major diagonal crack $\tau_s$, and the pier critical tensile strength $\sigma_{tcr}$. The pier critical tensile stress was computed at the neutral axis of the pier sections, following the simple beam theory for a section under combined flexure, shear and axial force; a parabolic distribution of shear stresses over the cross section was assumed.

From the ratios of Table 1 it is clear that prediction of the shear crack strength of the piers is better defined by the prism strength $\sqrt{f'_m}$ than by the critical tensile strength $\sigma^o_{tcr}$ of the square panels. This is somewhat surprising in that the square panel test is considered to be more sophisticated than the prism test since it induces a diagonal tension failure similar to that observed in the piers.

We found that the shear crack strength increases as the squatness of the piers increases and there is an increase in the shear crack strength as both the amount of horizontal reinforcement and the axial compressive stress increase. On the other hand, there appears to be no significant influence of the type of masonry construction on the ratio $\tau_s/\sqrt{f'_m}$.

## INELASTIC BEHAVIOR OF PIERS

Flexural mode of failure -- The inelastic characteristics obtained with the two double piers that displayed a flexural mode of failure, are quite desirable and similar to the behavior of an elasto-plastic material. The use of plates in the mortar joints of one of the specimens significantly improves the deformation capability of the pier.

Shear mode of failure -- The inelastic characteristics of piers exhibiting a shear mode of failure are discussed in connection with two of the parameters used in the test program: the amount of horizontal reinforcement and the type of grouting. In addition to these two variables, it is important to report that more desirable inelastic behavior was obtained with the more squat piers when compared to the behavior of the piers with height to width ratio of 2 or 1. Both the strength and deformation capacity of the piers with height to width ratio of 0.5 increased after the occurrence of the first major diagonal crack, whereas for the more slender piers significant strength degradation developed right after the formation of the diagonal cracks.

TABLE [illegible]. PREDICTION OF SHEAR CRACK STRENGTH FOR FULLY GROUTED PIERS

| Specimen | Vertical Steel Reinforcement (%) | Horizontal Steel Reinforcement (%) | Prism Compressive Strength $f'_m$ (psi) | Square Panel Crit. Tensile Strength $\sigma^o_{tcr}$ (psi) | Pier Shear Crack Strength $\tau_s$ (psi) | Pier Axial Stress at Shear Crack $\sigma_c$ (psi) | Pier Critical Tensile Strength $\sigma_{tcr}$ | $\frac{\sigma_{tcr}}{\sigma^o_{tcr}}$ | $\frac{\tau_s}{\sqrt{f'_m}}$ |
|---|---|---|---|---|---|---|---|---|---|
| HCBL-21-1 | 0.98 | — | 2432 | 320 | 144 | - 67 | 186 | 0.58 | 2.92 |
| -3 | 0.44 | — | 2256 | 337 | 152 | + 68 | 264 | 0.78 | 3.20 |
| -5 | 0.98 | — | 2592 | 280 | 114 | +145 | 258 | 0.92 | 2.24 |
| -7 | 0.98 | 0.52 | 2805 | 326 | 226 | + 37 | 359 | 1.10 | 4.27 |
| -9 | 0.98 | — | 2519 | 244 | 164 | -292 | 140 | 0.57 | 3.27 |
| HCBL-11-1 | — | — | 1330 | 124 | 135 | -120 | 151 | 1.22 | 3.70 |
| -3 | 0.17 | — | 1833 | 137 | 134 | - 69 | 170 | 1.24 | 3.13 |
| -4 | 0.17 | 0.08 | 1833 | 137 | 171 | -107 | 209 | 1.53 | 3.99 |
| -6 | 0.17 | 0.34 | 1833 | 135 | 226 | -144 | 275 | 2.04 | 5.28 |
| -7 | 0.43 | — | 1905 | 166 | 180 | - 91 | 228 | 1.37 | 4.12 |
| -9 | 0.43 | 0.17 | 1905 | 166 | 155 | -114 | 183 | 1.10 | 3.55 |
| -11 | 0.43 | 0.48 | 1330 | 133 | 240 | -139 | 297 | 2.23 | 6.58 |
| HCBL-12-1 | 0.30 | — | 2988 | 330 | 200 | - 65 | 260 | 0.79 | 3.66 |
| -2 | 0.30 | 0.05 | 2988 | 330 | 206 | - 66 | 268 | 0.81 | 3.77 |
| -3 | 0.30 | 0.10 | 2988 | 330 | 215 | - 83 | 283 | 0.86 | 3.93 |
| -4 | 0.30 | 0.15 | 2988 | 330 | 261 | -127 | 333 | 1.01 | 4.77 |
| -5 | 0.30 | 0.20 | 2988 | 330 | 226 | -106 | 290 | 0.88 | 4.13 |
| -6 | 0.30 | 0.29 | 2988 | 330 | 244 | -102 | 319 | 0.97 | 4.46 |
| HCBR-21-1 | — | — | 4502 | 375 | 267 | -580 | 204 | 0.54 | 3.98 |
| -2 | 0.51 | — | 4502 | 375 | 238 | -368 | 218 | 0.58 | 3.55 |
| -4 | 0.51 | 0.20 | 4502 | 375 | 308 | -415 | 299 | 0.80 | 4.59 |
| -6 | 0.51 | 0.30 | 4502 | 375 | 343 | -492 | 325 | 0.87 | 5.11 |
| -8 | 0.51 | 0.40 | 4502 | 375 | 346 | -485 | 331 | 0.88 | 5.16 |
| -9 | 0.51 | 0.50 | 4502 | 375 | 348 | -476 | 336 | 0.90 | 5.19 |
| HCBR-11-1 | — | — | 2535 | 282 | 278 | -328 | 284 | 1.01 | 5.52 |
| -3 | 0.18 | — | 2535 | 282 | 279 | -148 | 352 | 1.25 | 5.54 |
| -4 | 0.18 | 0.09 | 2722 | 363 | 353 | -323 | 391 | 1.08 | 6.77 |
| -6 | 0.18 | 0.34 | 2722 | 336 | 346 | -175 | 438 | 1.30 | 6.63 |
| -7 | 0.18 | 0.44 | 2535 | 282 | 280 | -241 | 317 | 1.12 | 5.56 |
| -8 | 0.45 | — | 2866 | 293 | 242 | -123 | 307 | 1.05 | 4.52 |
| -10 | 0.45 | 0.18 | 2722 | 363 | 296 | -153 | 374 | 1.03 | 5.67 |
| -12 | 0.45 | 0.62 | 2535 | 282 | 275 | -240 | 309 | 1.10 | 5.46 |
| -13 | 0.45 | 0.62 | 2722 | 363 | 329 | -312 | 361 | 0.99 | 6.31 |
| HCBR-12-2 | 0.31 | 0.08 | 2838 | — | 319 | -125 | 420 | — | 5.99 |
| -3 | 0.31 | 0.15 | 2838 | — | 351 | -150 | 457 | — | 6.59 |
| -4 | 0.31 | 0.23 | 2838 | — | 356 | -143 | 467 | — | 6.68 |
| -5 | 0.31 | 0.31 | 2838 | — | 394 | -154 | 519 | — | 7.40 |
| -6 | 0.31 | 0.52 | 2838 | — | 392 | -153 | 516 | — | 7.36 |
| CBRC-21-2 | 0.38 | — | 3315 | 284 | 295 | -477 | 264 | 0.93 | 5.12 |
| -3 | 0.38 | 0.15 | 3315 | 284 | 264 | -458 | 228 | 0.80 | 4.59 |
| -4 | 0.38 | 0.22 | 3315 | 284 | 267 | -417 | 244 | 0.86 | 4.64 |
| -5 | 0.38 | 0.37 | 3315 | 284 | 262 | -377 | 247 | 0.87 | 4.55 |
| CBRC-11-1 | — | — | 2507 | 205 | 247 | -296 | 251 | 1.22 | 4.93 |
| -2 | 0.13 | — | 2507 | 205 | 244 | -193 | 282 | 1.38 | 4.87 |
| -3 | 0.13 | 0.06 | 2507 | 205 | 239 | -186 | 276 | 1.35 | 4.77 |
| -4 | 0.13 | 0.32 | 2507 | 220 | 268 | -276 | 287 | 1.30 | 5.35 |
| -5 | 0.33 | — | 2507 | 205 | 217 | -159 | 256 | 1.25 | 4.33 |
| -6 | 0.33 | 0.13 | 2507 | 220 | 272 | -209 | 316 | 1.44 | 5.43 |
| -7 | 0.33 | 0.46 | 2507 | 220 | 257 | -169 | 310 | 1.41 | 5.13 |
| CBRC-12-1 | 0.23 | — | 2876 | 269 | 253 | -108 | 329 | 1.22 | 4.72 |
| -2 | 0.23 | 0.06 | 2876 | 269 | 250 | -127 | 316 | 1.17 | 4.66 |
| -3 | 0.23 | 0.11 | 2876 | 269 | 275 | -138 | 349 | 1.30 | 5.13 |
| -5 | 0.23 | 0.23 | 2876 | 269 | 231 | -116 | 293 | 1.09 | 4.31 |
| -6 | 0.23 | 0.38 | 2876 | 269 | 240 | - 94 | 316 | 1.17 | 4.48 |

In general, the test results show a positive influence of the horizontal reinforcement on the inelastic behavior of the piers. Increasing amounts of horizontal reinforcement improve the crack patterns, increase the ultimate strength of the piers and increase their deformation capacity. However, there is not a consistent relationship between the amount of reinforcement and amount of improvement obtained. Also, there appears to be no influence of the horizontal reinforcement on the rate of strength degradation of the piers after the ultimate strength has been attained. This favorable influence of the reinforcement on the pier behavior holds for the HCBL and HCBR piers, but is quite minimal for the double wythe, grouted core, clay brick piers (CBRC).

With respect to the influence of the type of grouting, partially grouted HCBL piers have a slightly better inelastic behavior than the corresponding fully grouted piers, if net area stresses are used in the comparison. However, in the case of the HCBR piers, the inelastic behavior of the partially grouted piers is definitively less desirable than that of the fully grouted piers; the deformation capability of the partially grouted HCBR piers is reduced, the strength degradation becomes very sharp and the ultimate strength based on net area stresses is always smaller than that of the corresponding fully grouted piers.

## FUTURE RESEARCH

Our plans for the future make sense only in the context of the whole program, particularly the immediate past. We begin by tracing out the reasons for the modification of the test setup, as much of our future depends on this change.

During the program of research on the single pier specimens we observed that the great bulk of the piers were failing in shear as the result of predominantly diagonal cracking. We had learned from the work of Priestley that these are advantages in having piers fail in flexure, but we were producing very few such failures. When we examined the response data carefully we found the reason.

The vertical load at the beginning of a test could be a chosen, nominal amount but when horizontal displacements began the vertical load increased and the greater the horizontal displacement, the greater the additional, superimposed vertical load. When failure began the vertical load was usually very large inducing a shear type of failure. We realized that the test results we had were valuable but only for the somewhat restricted case when the vertical load is large.

Further examination showed that the additional vertical load is applied to the pier by the two vertical members used to constrain the ends against rotation. During horizontal displacement, the constraining members that are originally vertical are forced to take up sloping configurations making each become the hypotenuse of a right angled triangle in which the other two sides are the original vertical length and the horizontal displacement. This forced elongation imposes large tension forces on the steel members which has to be balanced by a large compression force on the pier. As we desire much more flexibility in the variety of vertical loads, we realized that we would have to redesign our test setup so that the total vertical load, whatever, could be held constant during the full extent of the test. The actual field condtions would then be reproduced.

The solution to the problem came when we realized that the vertical "constrainers" should not impose a strain on the pier, rather each should impose a force. Each vertical member should be replaced by an actuator.

The advantages of this new setup are even greater than we had originally thought. As we have noted previously, use of actuators allows us to maintain zero rotation (or any specified) at the ends of the piers and hold the vertical load constant for the duration of a test. In the future we hope to extend the test program on single piers by exploiting the test ability. We have conducted only one series of tests with the new test setup but already we realize its importance. First, we tested a pier for which the vertical load was relatively light. The pier exhibited a flexural failure, and by allowing the vertical tension reinforcing to yield, the hysteresis envelope demonstrated that before failure the pier had undergone a large horizontal deflection.

In another phase of this first series a pier was tested when subjected to a large vertical force. This was carried out because it duplicated several tests with the old test setup, and we wanted a comparison of behaviors. The comparison is very revealing. The new test gave a hysteresis envelope that after reaching the maximum horizontal load, dropped off very sharply. You will recall from Figure 4 that under the old test setup the envelope dropped off slowly as the horizontal displacement increased. We cannot draw definitive conclusions on the basis of one test, but we can speculate on the cause of the difference. We identify the difference by tracing through what we think occurred with the old test setup. In that setup we think that when the maximum horizontal load was reached and diagonal cracking began there occurred a vertical settling of the pier which relieved part of the vertical load. This lowering of the vertical load is confirmed by the response data. Further increase in the horizontal deflection is followed by an increase in the extent of the diagonal crack, further relieving the vertical load and so on. This would account for the gradual reduction in the horizontal resistance of the pier. This would not be an inherent property of the pier but of the old test setup. This throws suspicion on the envelopes obtained with the old test setup beyond the maximum horizontal load. This by no means invalidates all of our previous findings, only that particular part.

There is another aspect of testing that we propose that is not a consequence of the test setup. This program has to do with the pattern we use for applying horizontal displacements. Instead of steadily increasing the amplitude of displacement to failure, we would increase the amplitude, then decrease it, then increase it again, etc. and so achieve a pattern that would be much closer to a realistic earthquake input. We are anxious to find what various shapes the response loops will take with this new type of input.

Earlier in the paper we described the factors that influence the response of masonry to seismic forces. Many of these: the type of masonry, the geometry of a pier, etc. may be beyond the control of the engineer. Without question the most single significant contribution to the behavior of a pier is the use of steel reinforcing. It is recognition of this that leads us to believe that we can now enter a phase where we not only observe behavior but can to a large extent control it by judicious design of the reinforcing.

To get an insight into the best way to use reinforcing we recognize the simularities and the differences between masonry and concrete. For the similarities we can benefit from design practice in reinforced concrete, for the differences we are on our own.

Masonry and concrete are similar in their weaknesses: both are brittle and are weak in tension. Masonry is different from concrete in that reinforcing can only be placed in two perpendicular directions and in itself it does not provide anchorage.

Further, the treatment that we envision is very much dependent on whether the mode of failure for the pier is flexure or shear. With the new test setup we can, by prescribing the vertical load and holding it constant, dictate whether the mode of failure for a given pier will be flexure or shear.

First we consider how we might improve the response of a pier which tends to fail in flexure. We must review what we mean by "improving the response". By this we mean increasing the ductility of the pier, which in terms of the hysteresis envelope means that we should extend the capability of the pier to maintain the maximum (or close to the maximum) horizontal load resistance through a large horizontal deflection, which would result in a hysteresis envelope rises to the maximum load and then extends horizontally through a large deflection. We plan to improve this behavior by means of a reexamination of the vertical reinforcing. If the masonry elements exhibit sizeable cavities, the vertical reinforcing can be grouted adequately so that anchorage should be no problem. Our feeling is that by decreasing the size of the reinforcing bars, rather than increasing them, we would force these bars to yield, without any cracking of the masonry, which should increase the global ductile behavior. Concurrently we may have to reinforce the compression toe against crushing.

Using reinforcing to improve a pier's behavior when the failure is due to shear is another matter. As most of the piers already tested have failed in shear we have a clear picture of the problem.

When a pier, whose failure is shear, begins to lose its capacity to resist horizontal load a diagonal crack forms, and the horizontal load could only be maintained if the integrity of the total pier could be maintained, which until now we have not been able to achieve. We have learned from reinforced concrete that the most effective wav to maintain integrity after cracking would be to use stirrups, which for our piers would be horizontal reinforcing. Even though we realize the form of the improvement, we have been unable to use horizontal reinforcing as an equivalent stirrup. The reason is that we have been unable to provide anchorage for the horizontal bars. They must be in the mortar between the coarses of masonry, and the mortar cannot by itself provide anchorage. Realizing this we have subsequently bent the horizontal bars around the vertical reinforcing. The horizontal bars inhibit the opening up of the diagonal crack very little, because again the anchorage is lost, this time because the bent bars straighten out.

There seem to us to be two possible solutions, both of which we will experiment with in future programs. The first is to apply a thread to each end of a horizontal bar and use a bearing plate and nut at each end with possible welding to provide anchorage.

The second idea is to use vertical reinforcing bars throughout the complete width of the pier. A vertical bar has the advantage of being adequately anchored but the disadvantage of being less effective as a stirrup than a horizontal bar. It is less effective as it crosses a diagonal crack at a more oblique angle than horizontal bars. Vertical bars have been used effectively by Priestley but under modest shear conditions.

We don't know how all this will turn out, none of our speculations may be correct but the experiments that we plan in order to test our ideas are bound to throw further light on how we can improve the seismic behavior of masonry piers.

# AN EVALUATION OF MASONRY ANCHOR BOLTS FOR SEISMIC APPLICATIONS

By Brown, Russell H., and Williams, Steve

ABSTRACT: A review of the applications of anchor bolts as connection devices in masonry construction is presented. The need for research concerning the cylic performance of such embedments is summarized. An experimental research project which is presently underway is described. The goals of the project will be threefold: determine the strength of anchor bolts in masonry subjected to cyclic axial forces and shear forces acting simultaneously; develop mathematical models which can be used to predict the behavior of anchor bolts in masonry using rational engineering principles; and develop design recommendations for anchor bolts in masonry structures in high wind and earthquake zones, considering simultaneous axial and shear forces. Variables to be considered are bolt size, spacing, proximity to edges, proximity to other bolts, mortar type, masonry unit type, and wall reinforcing. It is anticipated that design procedures developed will permit rational design of anchor bolts in masonry.

# AN EVALUATION OF MASONRY ANCHOR BOLTS FOR SEISMIC APPLICATIONS

By Russell H. Brown[1] and Steve Williams[2]

## INTRODUCTION

The use of masonry bearing wall structural systems for high-rise frameless buildings has increased considerably since rational design procedures were first written into specifications for both brick (1) and concrete masonry (2) construction. The resistance of such structures to lateral forces induced by wind or earthquake has been the subject of considerable attention in recent years. Because of the dependence of the structural system on the diaphragm action of both the floors and the walls, connections are of prime importance.

There are numerous floor-wall connection references in the literature detailing the connection of floors made of wood, precast concrete, & cast-in-place concrete to various types of masonry walls. Many of these recommended connections between floor systems and masonry walls require the use of anchor bolts. A thorough search of the literature reveals, however, little information relating to the strength of anchor bolts in masonry, especially those subjected to the type of forces which might be induced by earthquakes. In short, today's designer has access to rational design procedures for most masonry structural elements but virtually no information regarding the rational design of simple embedments.

A research project was therefore proposed (3) to provide information on cyclic strength and behavior of anchor bolts in masonry. The project was funded by the National Science Foundation and began in July, 1979.

---

[1]Assoc. Prof. of Civil Engrg., Clemson University, Clemson, South Carolina.

[2]Grad. student in Civil Engrg., Clemson University, Clemson, South Carolina.

[3]Numerals in parentheses refer to corresponding items in the Appendix I. -- References.

## OBJECTIVES

The objectives of the proposed research program are threefold:

1) To determine the strength of anchor bolts in masonry subjected to cyclic axial forces and shear forces acting simultaneously.

2) To develop mathematical models which can be used to predict the behavior of anchor bolts in masonry using rational engineering principles.

3) To develop design recommendations for anchor bolts in masonry structures in high wind and earthquake zones, considering simultaneous axial and shear forces.

## FUTURE OBJECTIVES

The test series on anchor bolts is viewed by the principal investigator as the first of several series of tests of masonry connections and connectors. With the equipment and expertise resulting from the current research, future testing of all types of floor-wall connections and connection devices can be done with considerably more speed and economy.

## BACKGROUND

### Types of Anchor Bolts

There are three configurations of anchor bolts which are commonly used in both masonry and concrete construction: the "J" bolt, the straight hexagonal head bolt, and the expansion bolt. The "J" bolt has been used in masonry structures for many years and can be found in recommended connection details in some very old publications. Usage of the "J" bolt seems to be on the decline, especially as an anchorage device in concrete. However, it is still used in sufficient quantity in new construction to warrant a thorough investigation into its performance. Also, since it has been used for so long, there are numerous existing structures which used the "J" bolt which may require reevaluation.

The use of straight bolts with standard hexagonal heads has become more commonplace in concrete technology. The TVA (4) considers this bolt configuration satisfactory for cast-in-place concrete. It is generally regarded as a superior bolt to the "J" configuration.

The expansion type anchor bolt has many obvious benefits, most notably that it can be embedded in existing walls. This versatility makes the expansion bolt popular in new construction and essential in retrofit construction.

All three types of anchor bolts can presently be found in numerous structures. This project will concern itself primarily with the "J" bolt configuration. It is hoped that future research will focus on the other configurations.

## Applications of Anchor Bolts in Masonry

As mentioned previously, many floor-wall connections can be found in the literature (5, 6, 7). Those connections which involve cast-in-place concrete generally use reinforcing rods and do not require anchor bolts (Fig. 1). There is a large group of recommended connections in which the primary load transfer device is the anchor bolt. Generally, bolted connections are used to connect wood or prefabricated floors or roofs to masonry walls. A shelf angle or wood ledger is attached to the wall with embedded anchor bolts (Fig. 2,3). The floor element bears on the angle or ledger and is sometimes topped with concrete giving additional continuity (Fig. 4). These anchor bolts are subjected to the action of both axial and shear forces. The floor in a loadbearing masonry structure serves as a diaphragm transferring forces due to wind or earthquake to the shear wall through horizontal shear in the anchor bolts. Gravity loads are transferred directly to walls through vertical shear in the anchor bolts.

Axial forces in the plane of the floor systems are transferred directly as axial forces in the anchor bolts. Tensile forces applied to the clip angle or ledger are transferred to the masonry wall as direct axial tensile forces in the bolts. Compressive forces are transferred in bearing between the angle or the ledger and the wall only if they are in contact.

Pyle (8) has proposed a new connection system which permits the attachment of conventional flat plate floor slabs to single wythe exterior masonry walls. The system uses concrete columns but uses masonry walls for lateral force resistance. The walls can also be used to carry gravity loads at the free edge of the slab or to control deflection. The system uses a shear key and an embedded anchorage device (Fig. 5). It eliminates the expense associated with spandrel beams and steel ledger angles.

Borchelt (9) recommends the use of anchor bolts to support connection devices in prefabricated masonry panels. In such applications, the anchor bolt resists primarily shear forces produced from either gravity or lateral loads. Anchor bolts are also commonly used as lifting devices in prefabricated brick masonry construction. In lifting applications, the anchor bolt is primarily a tension-resisting device.

## Documentation of Masonry Anchor Bolt Performance

There has been some criticism of the performance of anchor bolts during previous earthquakes. Meehan (10) notes "innumerable" examples of poor performance during earthquakes of bolts cast in masonry. He cites the need for such information as effect of bolt size, embedment, head type, and edge distance on the strength of embedments.

Amrhein (11) also cites the need to obtain information on capacity of anchor bolts and other connection devices under both static and dynamic conditions.

The Applied Technology Council (12) states: "One of the major hazards from buildings during an earthquake is the pulling away of heavy masonry or concrete walls from the floors or roofs. While requirements for the anchorage to prevent this separation have been common in highly-seismic areas, they have been minimal or non-existent in most parts of the country...observations of earthquake damage indicate that (provision of minimal anchorage) can greatly increase the earthquake resistance of buildings..." Indications seem to point toward more stringent requirements for anchor bolts in seismic areas in the future.

<u>Published Research on Masonry Anchor Bolts</u>

Only two papers could be located which reported on research of anchor bolts in masonry walls. Green and Horner (13) described a test program to evaluate the shear resistance of wooden blocks bolted to masonry and of steel angles bolted to masonry. Loads were applied slowly and without reversals. The authors somehow drew conclusions as to the suitability of such connections in earthquake construction without performing any load reversals whatsoever. They determined that the following factors affected the adequacy of such bolted connections:

1) Size, strength and spacing of anchor bolts.

2) Quality of mortar.

3) Type of masonry.

4) Movement of the anchor bolt under the design loads.

5) Quality of workmanship.

The authors pointed out in their conclusions the necessity for performing further tests to corroborate and supplement the results they obtained.

Schneider (14) conducted an extensive masonry research program, part of which considered the shear strength of bolts embedded in grout and in mortar. He embedded pairs of bolts at three different levels in a ten-course brick wall, twenty-one feet in length and eight inches in width. Each pair of bolts was subjected to a static transverse shear force. Deflection readings were taken and plotted with shear force. Three possible failure modes were observed during the testing: crushing of the brick, shearing of the bolt, or permanent distortion of the bolt. The loads were corresponded to deflections of 0.1 in. were approximately fifty percent of the ultimate load in each case.

It appears that the allowable values for shear forces on bolts published in the <u>Brick Institute of America B.I.A. Standard</u> (1) and the <u>Uniform Building Code</u> (15) are in agreement with Schneider's test results. These allowable shear forces roughly correspond to twenty percent of the force observed at a 0.1 in. deflection by Schneider's tests.

## Published Research on Concrete Anchor Bolts

The paucity of published research pertaining to the subject of anchor bolts in masonry led the authors to seek similar research results on anchor bolts in concrete. Much has been done in this area, and it should be noted that the material cited below is only a sample of the work published. Though some fundamental differences are expected to exist between the two materials, the brittle nature of masonry and concrete should produce comparable behavior.

A comprehensive research program has been undertaken by the Tennessee Valley Authority (4, 16, 17) in cooperation with the University of Tennessee (18). The TVA is concerned with anchor bolts as connection devices in nuclear power plant construction in regions of high seismic activity. Their research has led, in part, to design procedures recommended by the American Concrete Institute (19).

Richard (20) tested anchor bolts set in mortar in large holes drilled in existing concrete. Bolts of 1 in. diameter were grouted into holes of 2-1/2 in. diameter using a non-expansive mortar grout. Bolts were loaded axially and the load and failure mode observed. The required length of embedment to force failure of the bolt rather than the concrete was determined. Richard also studied group effects. Tests in which the bolts did not develop their full strength demonstrated a failure mode in which a somewhat conical portion of the concrete surrounding the bolt pulled out of the wall.

Adams (21) described pull-out tests made with exapansion bolt anchors in concrete. He found that the depth of a hole into which the anchors were embedded should be at least four times the diameter of the bolt, and that the distance from the hole to the edge or the corner of the concrete should be two or three times the depth of the hole.

Lee (22) suggested improvements in the design of foundation bolting for heavy machinery. He found that shock loads would be better resisted if bond between the shank of the bolt and surrounding concrete were destroyed and anchorage provided only at the end of the bolt. He recommended the use of reinforcing to aid the concrete in resisting bursting stresses that resulted.

## Background Summary

In summary, the literature reveals the frequent use of anchor bolts in masonry connections with very little research data to justify the practice. The research published concerning anchor bolts in concrete provides incomplete insight into the problem of anchor bolts in masonry. There appears to be a total absence of information concerning the effect of load reversals on the strength and behavior of anchor bolts in masonry. No tests of embedments in masonry considered simultaneous action of axial and shear forces. If anchor bolts are to be used as primary structural connections in masonry structures in earthquake regions, information concerning simultaneous application of shear and axial force on anchor bolts must be obtained.

# RESEARCH PLAN

## General Objectives

The experimental research program is designed to measure the strength of anchor bolts in masonry subjected to cyclic axial and transverse loading. Through the variation of many parameters, the variables which affect this strength will be established. Deformations will be measured to establish the ductility of anchor bolt connections. Failure modes will be observed for all tests to facilitate the development of accurate mathematical models and subsequent design improvements.

## Test Specimen

All test specimens will be two-wythe composite brick-concrete block grouted walls of 14 in. nominal thickness. Walls will be approximately 40 in. high and 40 in. long (Fig. 6). The minimum reinforcing required by the Uniform Building Code (14) will be provided in most walls except those where the parameter being studied is wall reinforcement. Bolts will be placed in bedjoints with the bend projecting downward into the grouted cavity (Fig. 7).

Walls will be constructed by experienced masons using inspected workmanship. Walls will cure at least 28 days in laboratory air prior to testing. Embedded bolts will be mild steel in either a "J" configuration or a straight hexagonal. The end of the bolt not embedded will be provided with threads to facilitate attachment to the steel angle used to transfer the loads (Fig. 7). Joint reinforcement will be provided in alternate concrete masonry courses. Reinforcing bars will be used to provide steel required by the Uniform Building Code (15).

A composite brick-block grouted wall was selected for the study for three reasons: 1) both brick and block can be evaluated from a single wall, 2) the wall provides sufficient thickness for at least a four-inch embedment of the bolts, and 3) a grouted wall permits the study of the effect of grout in the cavity on the strength of the bolts.

## Instrumentation

The instrumentation used throughout the experimental program will include load cells, strain gages, and displacement transducers. Load cells will be used to monitor the force applied to the bolts being tested. Cells will be arranged in series with the hydraulic actuators and will be monitored at frequent intervals.

Strain gages will be attached to the anchor bolts at the wall surface (Fig. 7). Tie wires and joint reinforcement in the vicinity of the test bolt will be equipped with strain gages to determine their role in anchor bolt behavior. Strains will be monitored at frequent intervals.

Linear Variable Differential Transformers (LVDT) will be used to measure the deformation of the free end of the bolts relative to the wall (Fig. 7). Three transducers will be used at each bolt location to monitor the three deflection components. Output from LVDT's will be monitored at frequent intervals.

An X-Y Plotter will be used to continuously monitor load and axial bolt deflection. This plot will provide the equipment operator with a general idea of the performance of the test specimen during the test.

Test Program Variables

The variables to the considered in the test program are loading, bolt size, bolt spacing, bolt position, mortar type, masonry unit type, and wall reinforcing. A brief description of these variables follows:

1) Loading - Bolts will be subjected to various combinations of shear and tension cyclic forces. A pair of independent load actuators will be attached to the anchor bolts as shown in Fig. 6. Independent control of the two actuators will permit any desired combination of shear and tension. Axial compression forces will not be applied to the bolts since, in the opinion of the authors, such forces would produce minimal effect.

   Shear forces will be applied either parallel to or perpendicular to the masonry bedjoints. This will be accomplished by orienting the wall test specimen in the frame with the bedjoints in either a horizontal or vertical position.

   The rate of loading will be much slower than the natural frequency of the system, and therefore will be considered static. Walls will be subjected to a compressive stress level of 0 to 10% of the masonry compressive prism strength. It is anticipated that axial compression will increase bolt strength.

2) Bolt Size - Bolt diameters of 3/8 in., 1/2 in., 3/4 in., and 1 in. will be tested. It is not expected that each diameter of bolt will be subjected to every possible test.

3) Bolt Spacing - In order to establish the sphere of influence of adjacent bolts, two bolts will be tested simultaneously at various spacings. Values of spacing of between 5 and 30 bolt diameters will be used initially. Preliminary test results will establish the best range of bolt spacing to permit the evaluation of sphere of influence of bolts.

4) Bolt Position - Bolts will be placed at three different levels in a test wall. This procedure will result in multiple tests from a single wall and reduce the number of walls required. The effect of proximity of bolts to wall edge can also be studied in this manner.

5) Mortar Type - Most of the tests will be performed using Type S mortar. However, additional tests will be performed using Type M mortar to determine the effect of the type of mortar on the performance of the anchor bolts.

6) Masonry Unit Type - Only one type of brick and one type of concrete block will be used in the testing program. The brick will have a compressive strength of approximately 15,000 psi and initial rate of absorption of approximately 20 grams per 30 square inches. The concrete block will be 6 in. x 8 in. x 16 in. light-weight aggregate 4 hour fire rates. All brick and concrete block will be obtained from a single plant run to eliminate, as much as possible, variations in the properties of the masonry units.

7) Wall Reinforcement - All walls will be reinforced sufficiently to comply with requirements of the Uniform Building Code (15). However, one series of tests will be performed on unreinforced walls using one bolt diameter and one mortar type to establish the effect of reinforcing.

Table 1 and 2 give an indication of the overall test program, a total of 58 bolts (or bolt pairs). Some variation in the program is expected pending results of initial series.

## Load Histories

Axial and shear loads will be applied independently using separate loading actuators. The shear force will be cycled in both directions, increasing the load magnitude (Fig. 8). Axial tension forces will be applied in a similar fashion, however, without reversal (Fig. 9).

## Loading Systems

The loading will be applied to the specimens using a closed-loop servo-controlled hydraulic system. The authors have inspected several systems and are presently writing specifications for the system to be purchased. Independent loading capability of the shear and axial forces will require two separate closed-loop systems. Presently it is anticipated that the system will operate from displacement feedback, rather than load control. Actuators will be equipped with internal displacement transducers to provide the feedback signal.

## DEVELOPMENT OF A MATHEMATICAL MODEL

The development of a mathematical model for predicting failure strengths as a function of all relevant variables is an essential goal of the research project. Prior to the completion of the experimental phase, the authors can only surmise that the mathematical modeling procedure presently used for concrete anchor bolts can be adapted to masonry. Therefore, the authors have investigated the techniques presently used in concrete technology.

The proposed addition to the ACI Code (19) assumes a conical failure mechanism and permits a stress of $4\sqrt{f'_c}$ on the projected area. Studies by TVA (17) appear to indicate that the ACI model predicts failure with reasonble accuracy. The ACI model also considers overlapping cones and proximity to edges for both shear and tension forces.

It is anticipated that the failure mechanisms of anchor bolts in masonry will be similar to those in concrete. The angle of the failure cone may be different, and shapes other than cones may result from shear failures between mortar and masonry units. Ideally, the concepts will be similar, with only minor modifications to reflect differences between concrete and masonry. A successful mathematical model for predicting the strength of anchor bolts in masonry should be capable of considering the following variables:

1) Magnitude of tension force
2) Magnitude of shear force
3) Interactive effects of combined tension and shear forces
4) Effect of cycling of applied forces
5) Direction of shear force relative to bedjoints
6) Proximity of anchor bolt to joint reinforcing
7) Strength of masonry
8) Diameter of anchor bolt
9) Embedment depth of anchor bolt
10) Proximity of anchor bolt to free edges
11) Proximity of anchor bolt to other anchor bolts
12) Type of bolt configuration ("J" vs. hexagonal head)
13) Amount of precompression in the wall

The mathematical models that will eventually evolve will be compared to the test data using statistical principles. The factors of safety will be statistically analyzed to determine the probability of over-estimating the actual strength of an anchor bolt with the mathematical model.

## CONCLUSIONS

The need for research on embedded bolts in masonry has been clearly demonstrated, and procedures for implementing this research have been set forth. The direction of the remainder of the project will focus on the actual testing of the individual specimens, and the development of mathematical models and suggested design procedures.

The ultimate purpose of projects such as these, of course, should be to help educate the design engineer. With this purpose in mind, it is intended that the results will eventually enable designers to make rational and informed decisions regarding the design of anchor bolts in masonry.

# REFERENCES


1. Building Code Requirements for Engineered Brick Masonry, Brick Institute of America (formerly Structural Clay Products Institute), 1750 Old Meadow Road, McLean, Virginia, August, 1969, 36 pp.

2. Specification for the Design and Construction of Load Bearing Concrete Masonry, National Concrete Masonry Association, McLean, Virginia, May, 1976, 34 pp.

3. Brown, R. H., "Cyclic Respone of Masonry Anchol Bolts," Proposal, National Science Foundation, November, 1977.

4. Funk, R. R., TVA Report No. CEB 75-31, October 31, 1977, pp. 4, 5.

5. Technical Notes, Brick Institute of America, McLean, Virginia.

6. Masonry Design Manual, Masonry Industry Advancement Committee, Los Angeles, California, 384 pp.

7. Elmiger, A., Architectural & Engineering Concrete Masrony Details for Building Construction, National Concrete Masonry Association, McLean, Virginia, 1976.

8. Pyle, Don T., "An Application of Thin Wall Masonry," Proceedings of the North American Masonry Conference, August 14, 15, 16, 1978, pp. 80-2 - 80-5.

9. Borchelt, J. Gregg, "Masonry Panels: Review, Present Use and Design," Proceedings of the North American Masonry Conference, August 14, 15, 16, 1978, pp. 114-10.

10. Meehan, J. F., "Suggested Researchable Items Relating to Masonry Construction," Earthquake Resistant Masonry Construction: National Workshop, September 13-16, 1976, pp. 259-268.

11. Amrhein, James E., "Some Research Needs of Earthquake-Resistant Masonry," Earthquake Resistant Masonry Construction: National Workshop, September 13-16, 1976, pp. 255-258.

12. Tentative Provisions for the Development of Seismic Regulations for Buildings, ATC publication ATC 3-06, June, 1978, pp. 350.

13. Green, N. B., and Horner, A. C., "Joist Anchors in Masonry Walls Tested for Earthquake Resistance," Brick Engineers' Digest, October, 1936.

14. Schneider, R. R., "Grouted Brick Masonry - Report of Test," Southwest Builder and Contractor, December 28, 1951.

15. Uniform Building Code, Volume I, International Conference of Building Officials, 1973 Edition.

16. Cannon, Robert W., Welded Stud Anchors: Effect of Plate Flexibility on Stud Capacity, TVA Report No. CEB79-18, July 10, 1979.

17. Burdette, Edwin G., Cannon, Robert W., and Funk, Raymond R., Anchorage to Concrete, TVA Report No. CEB75-32, December 1, 1975.

18. Bailey, John W., and Burdette, Edwin G., Edge Effects on Anchorage to Concrete, The University of Tennessee, Dept. of Civil Engineering, Research Series No. 31, August, 1977.

19. Appendix B - Steel Embedments, (Proposed Addition to); Building Code Requirements for Reinforced Concrete (ACI 318-77), 1978.

20. Richard, E. L., "Anchor Bolts Set in Drilled Holes," Journal of the American Concrete Institute, September, 1947, pp. 81-83.

21. Adams, R. F., "Some Factors Which Affect the Strength of Bolt Anchors in Concrete," Journal of the American Concrete Institute, October 1955, pp. 131-138.

22. Lee, C. A., "Design of Anchor Bolts in Foundations," Journal of the American Concrete Institute, October, 1959, pp. 339-340.

Table 1 - Basic Test Series

| Mark | Angle α** | Bolt Diameter | Bolt Spacing | Mortar Type |
|---|---|---|---|---|
| * A0-3-S0-S | 0° | 3/8"*** | 0 | S |
| A15-3-S0-S | 15 | 3/8"*** | 0 | S |
| A45-3-S0-S | 45 | 3/8"*** | 0 | S |
| A75-3-S0-S | 75 | 3/8"*** | 0 | S |
| A90-3-S0-S | 90 | 3/8"*** | 0 | S |
| A0-4-S0-S | 0 | 1/2"*** | 0 | S |
| A15-4-S0-S | 15 | 1/2"*** | 0 | S |
| A45-4-S0-S | 45 | 1/2"*** | 0 | S |
| A75-4-S0-S | 75 | 1/2"*** | 0 | S |
| A90-4-S0-S | 90 | 1/2"*** | 0 | S |
| A0-6-S0-S | 0 | 3/4" | 0 | S |
| A15-6-S0-S | 15 | 3/4" | 0 | S |
| A45-6-S0-S | 45 | 3/4" | 0 | S |
| A75-6-S0-S | 75 | 3/4" | 0 | S |
| A90-6-S0-S | 90 | 3/4" | 0 | S |
| A0-8-S0-S | 0 | 1" | 0 | S |
| A15-8-S0-S | 15 | 1" | 0 | S |
| A45-8-S0-S | 45 | 1" | 0 | S |
| A75-8-S0-S | 75 | 1" | 0 | S |
| A90-8-S0-S | 90 | 1" | 0 | S |

* Series A refers to tests performed with shear parallel to bed joints. Series B refers to tests performed with shear perpendicular to bed joints. See Table 2 for Explanation of test designations.

** $\alpha = \tan^{-1}$ (TENSION ÷ SHEAR)

*** This series will be performed on both "J" bolts and Straight Standard Hexagonal Head Bolts.

Table 2 - Additional Test Series

| Mark | Angle α** | Bolt Diameter | Bolt Spacing | Mortar Type |
|---|---|---|---|---|
| A90-3-S10-S* | 90° | 3/8" | 10 | S |
| A90-3-S5-S | 90 | 3/8" | 5 | S |
| A90-4-S10-S | 90 | 1/2" | 10 | S |
| A90-4-S5-S | 90 | 1/2" | 5 | S |
| A90-6-S10-S | 90 | 3/4" | 10 | S |
| A98-6-S5-S | 90 | 3/4" | 5 | S |
| A0-3-S0-M | 0 | 3/8" | 0 | M |
| B0-3-S0-M | 0 | 3/8" | 0 | M |
| A15-3-S0-M | 15 | 3/8" | 0 | M |
| A45-3-S0-M | 45 | 3/8" | 0 | M |
| B45-3-S0-M | 45 | 3/8" | 0 | M |
| A75-3-S0-M | 75 | 3/8" | 0 | M |
| A90-3-S0-M | 90 | 3/8" | 0 | M |

*Legend

A15 - 3 - S10 - S

- S: Mortar type S
- S10: 10 bolt diameter spacing between bolts
  S0 designation implies a single bolt
- 3: Bolt diameter = 3/8 in.
- A15: α = 15°

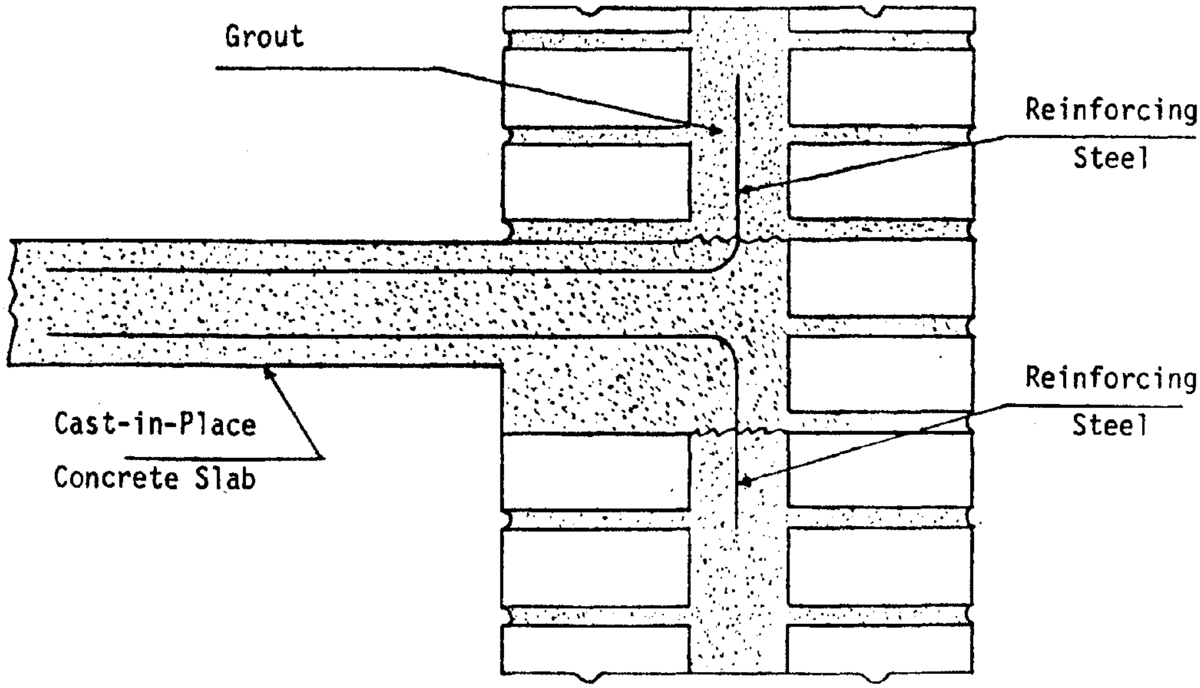



Fig. 1 Cast-in-Place Concrete Floor-Wall Connection

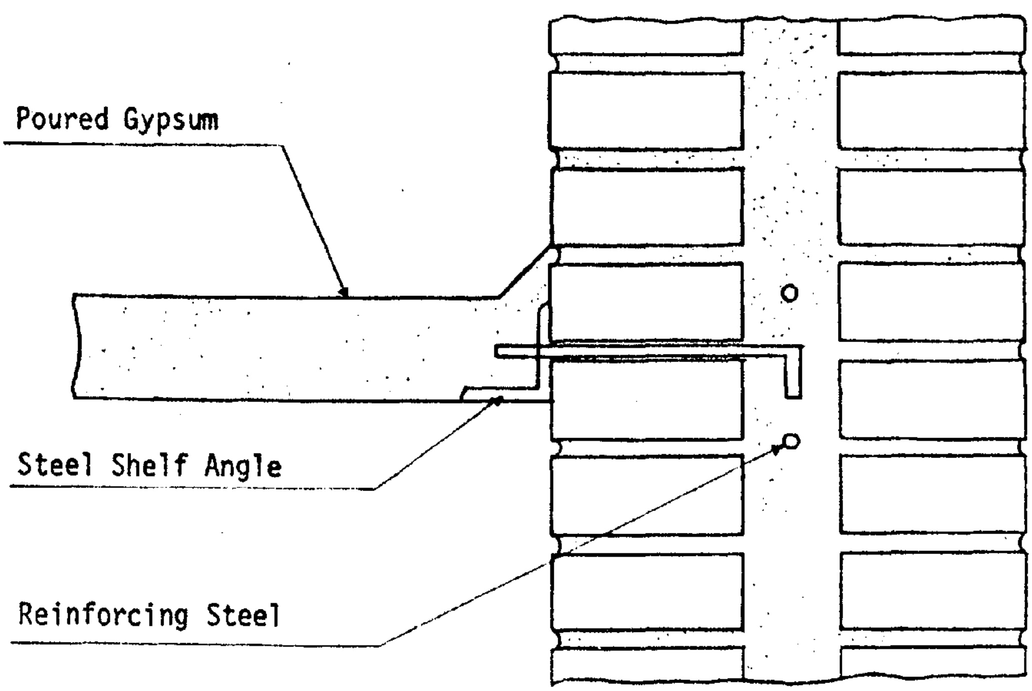



Fig. 2 Poured Gypsum Floor-Wall Connection

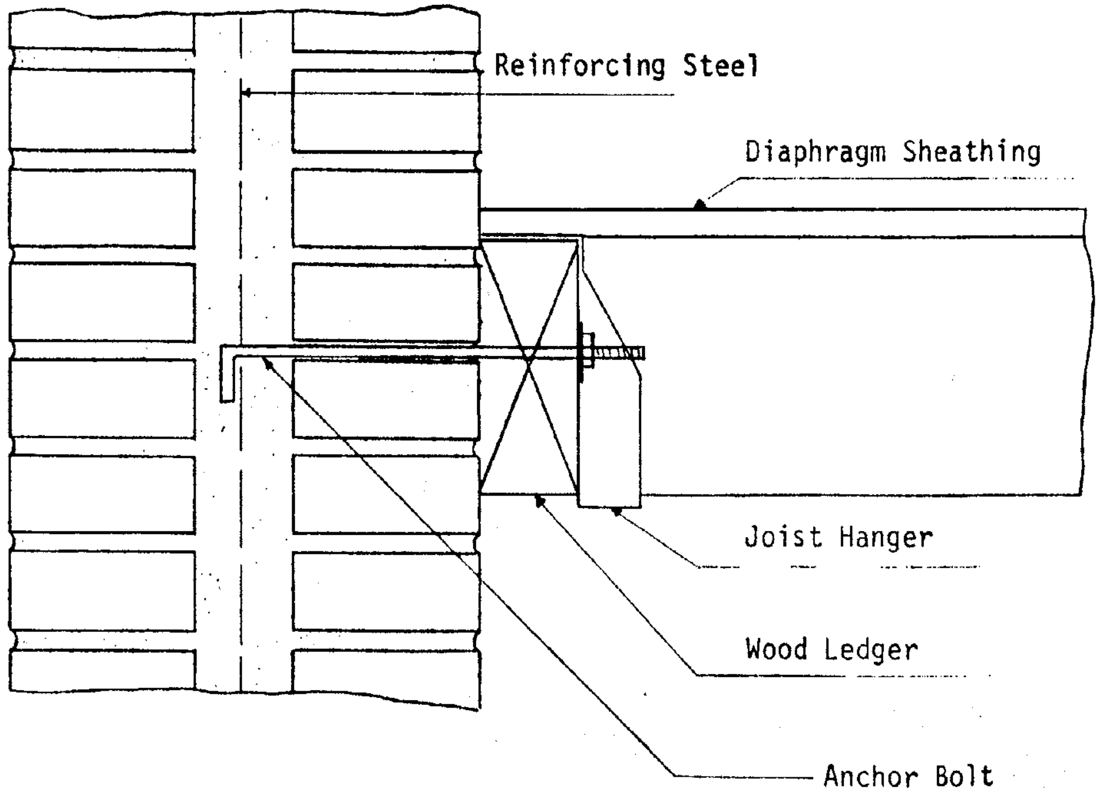
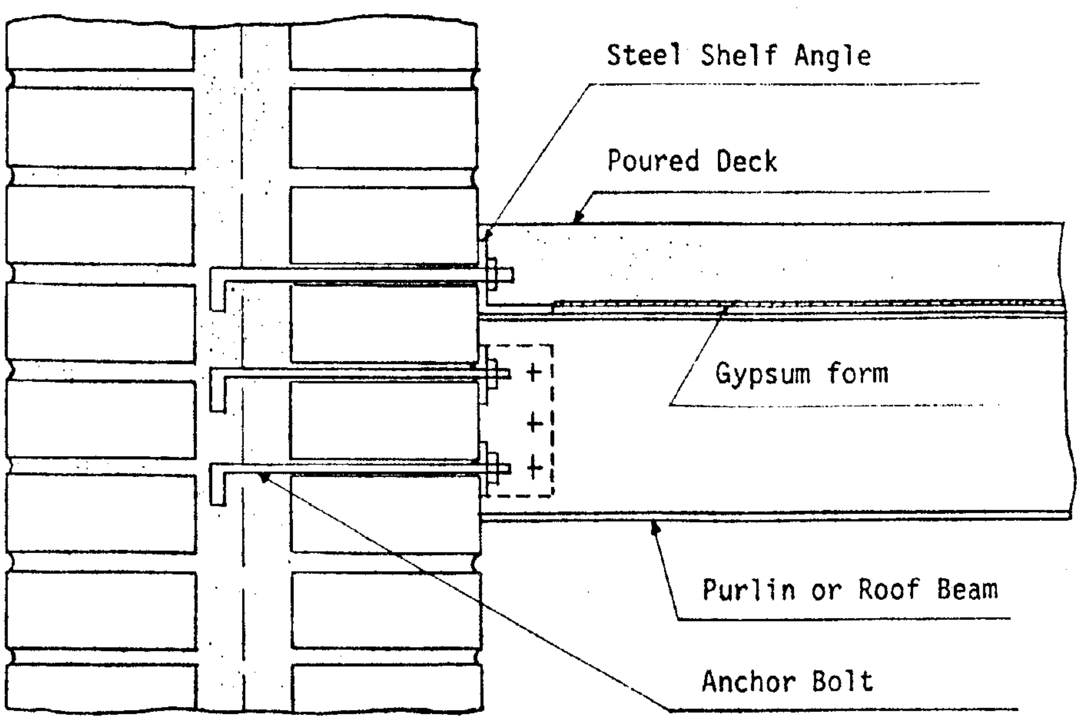



Fig. 3 Wood Diaphragm Floor-Wall Connection



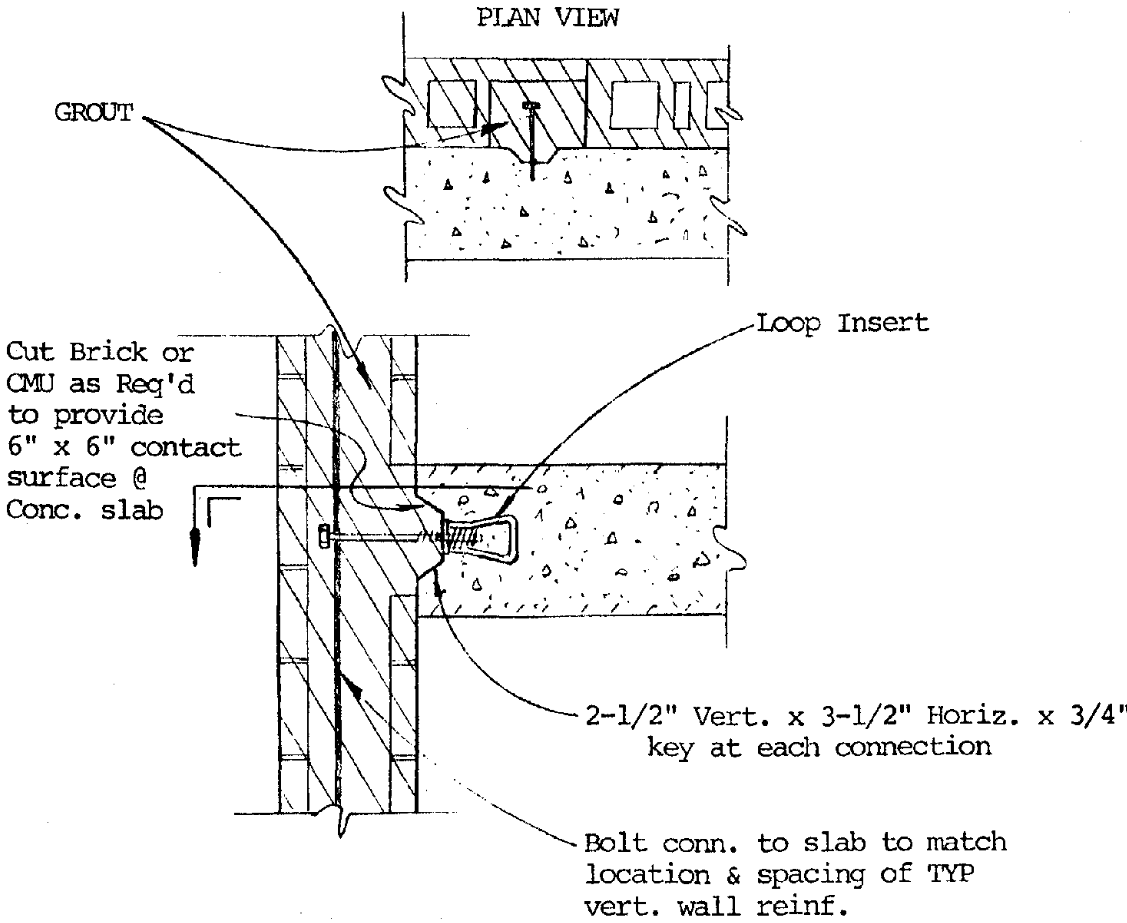



Figure 5. Floor-Wall Connection

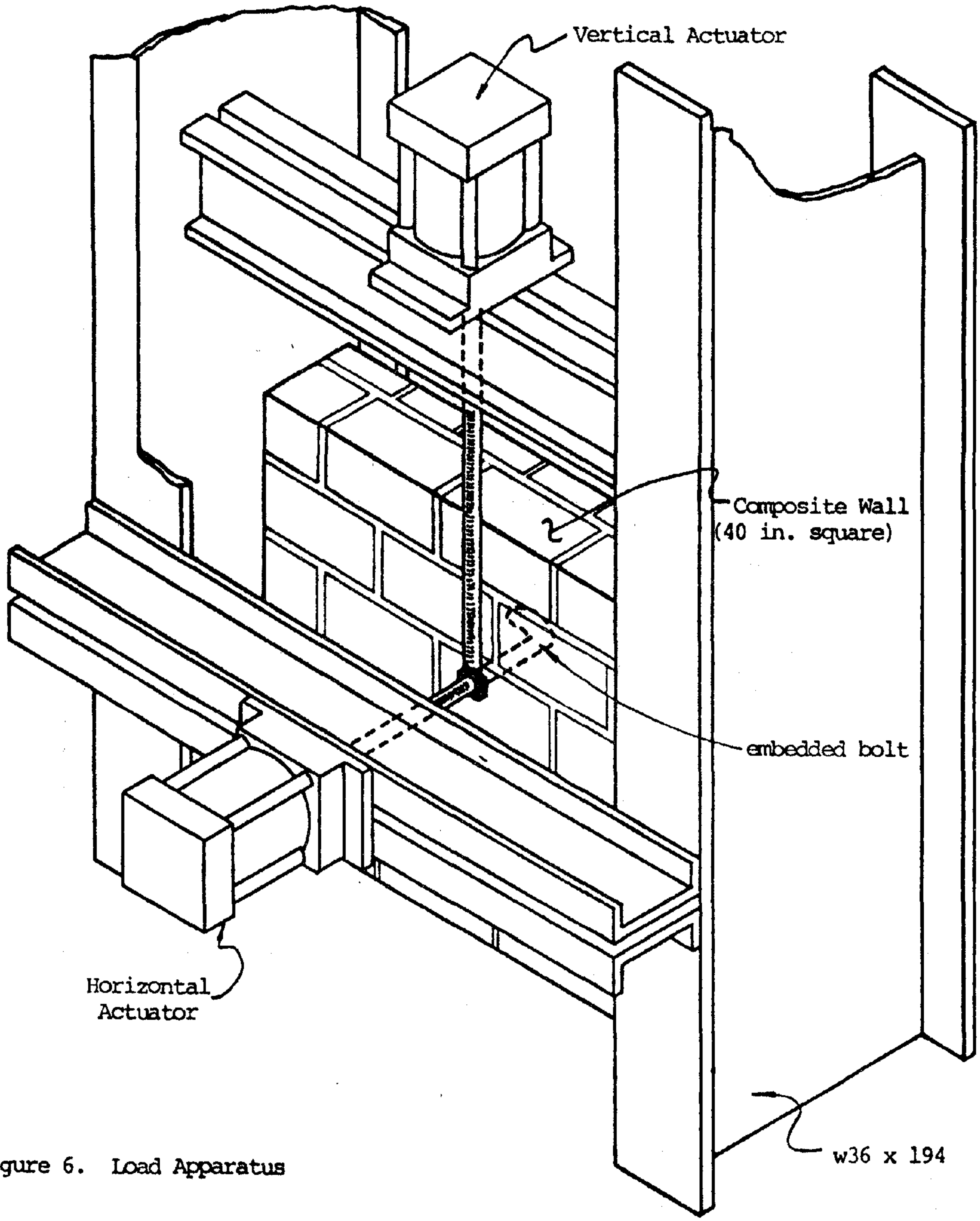



Figure 6. Load Apparatus

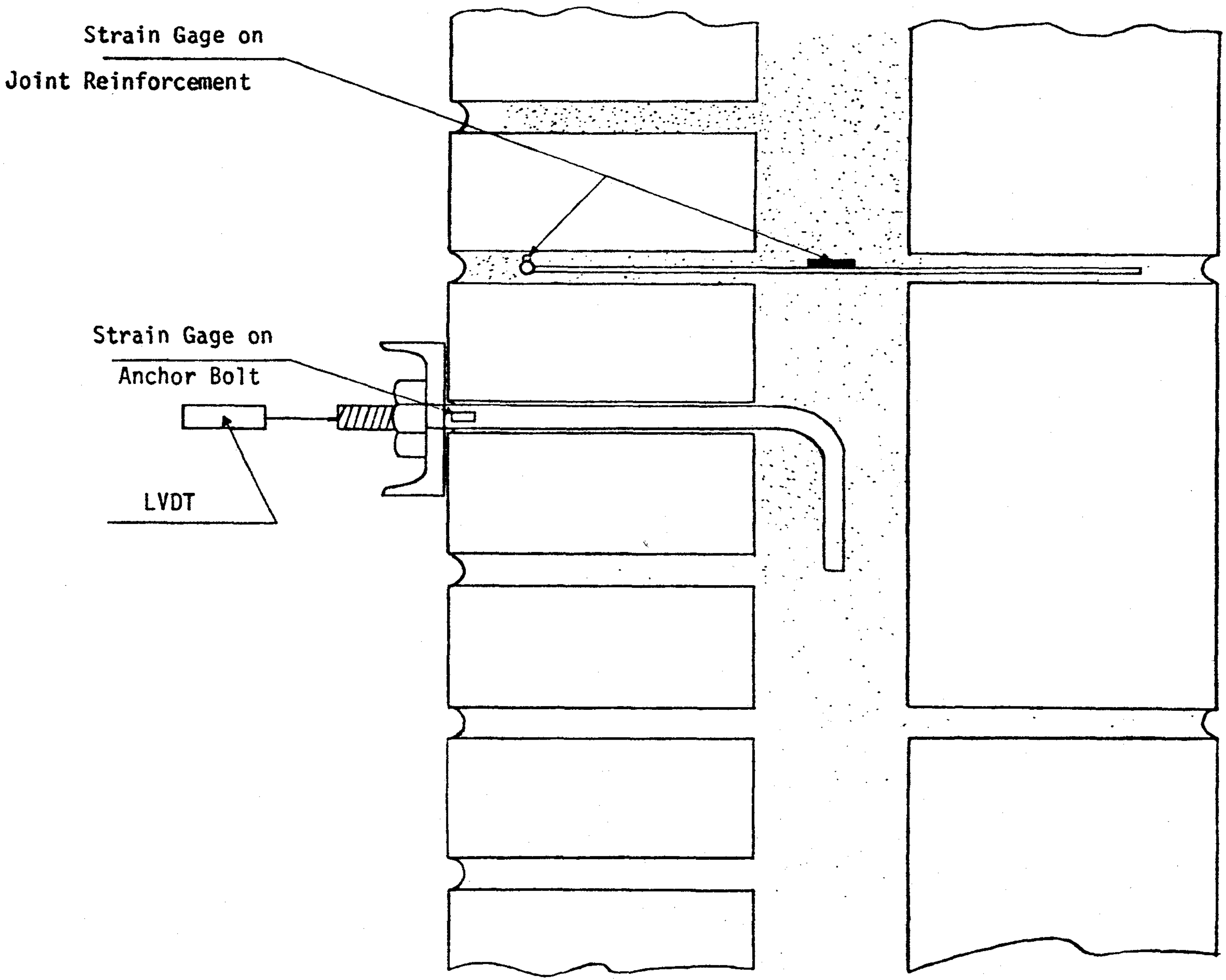



Fig. 7 Instrumentation of Bolt and Wall

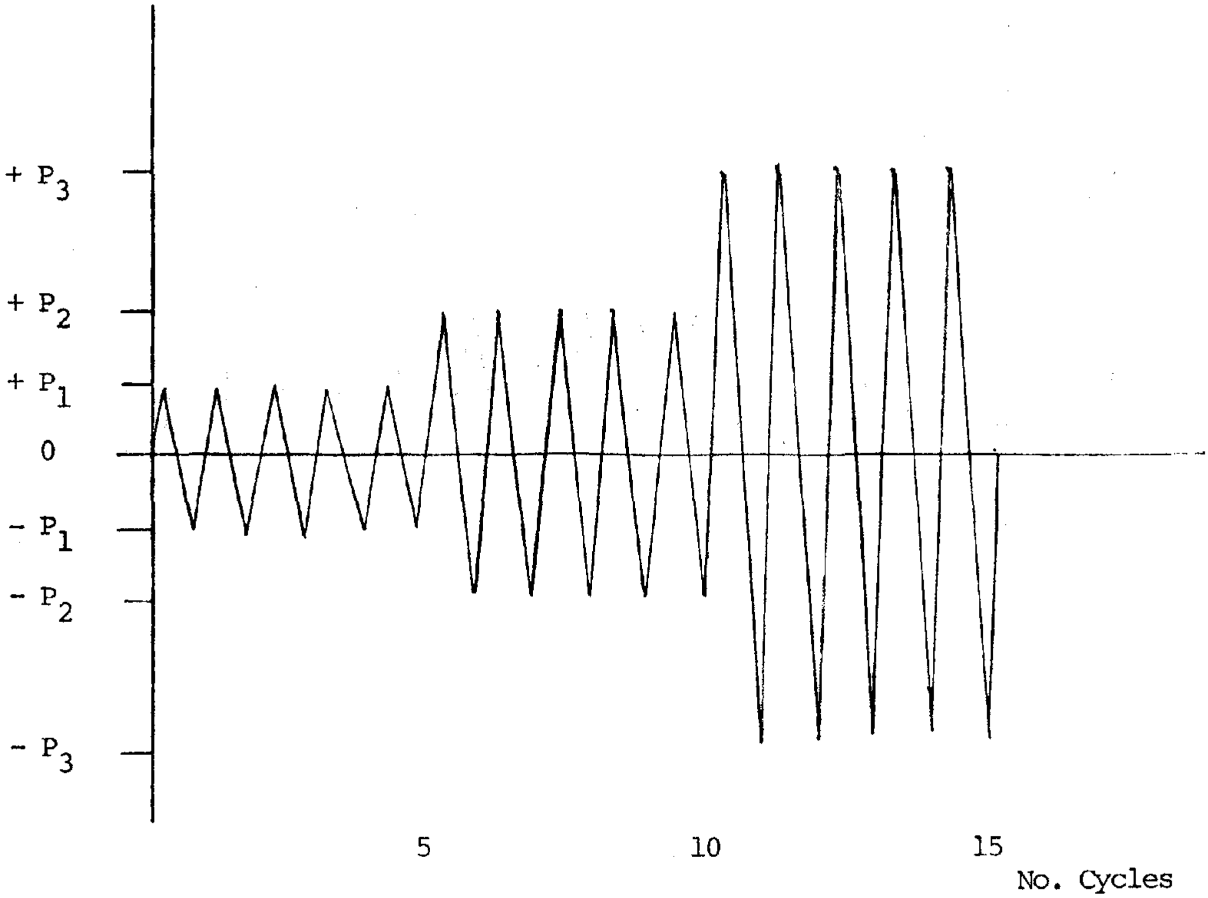



Figure 8. Loading Histories (Shear)

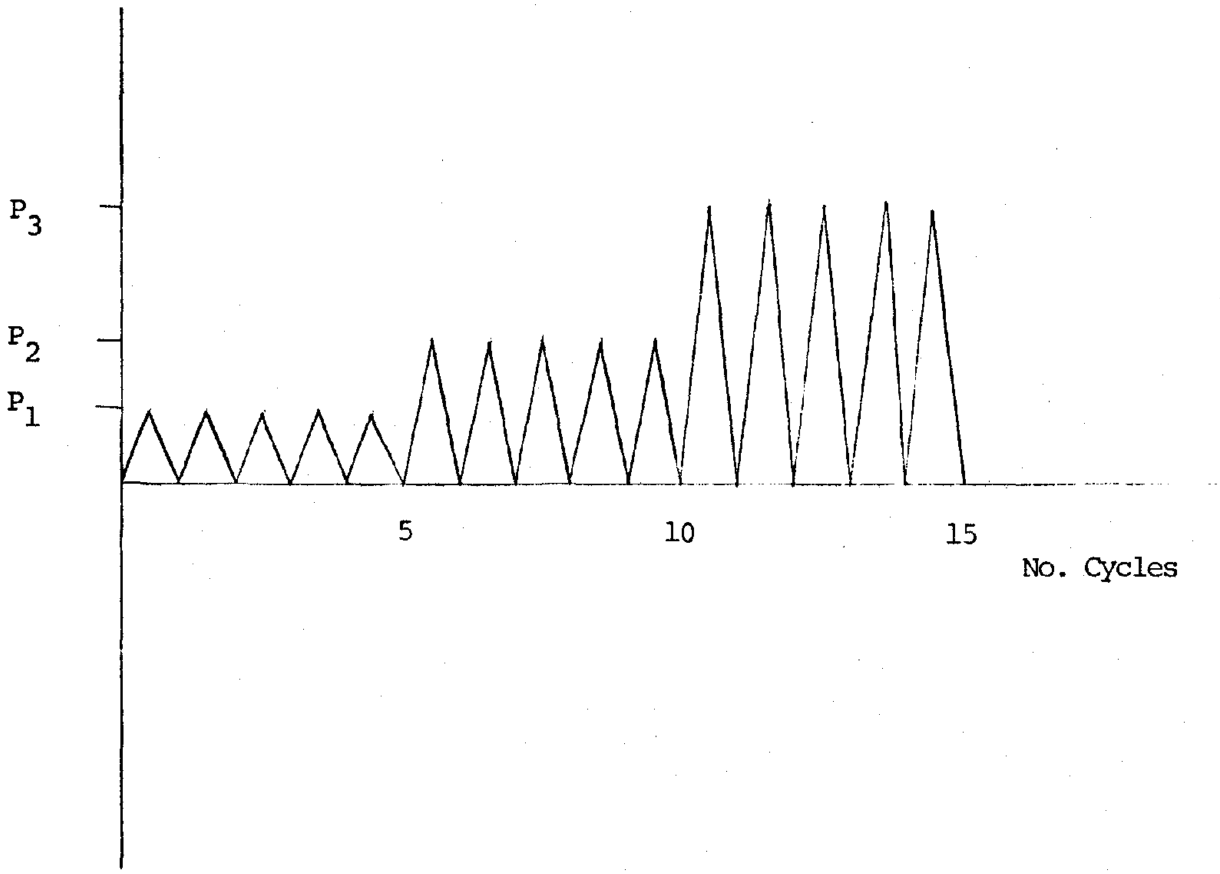



Figure 9. Loading Histories (Tension)

# A FINITE ELEMENT MODEL TO PREDICT INTER-LAMINAR SHEARING STRESSES IN COMPOSITE MASONRY

By Subhash C. Anand[1] and David T. Young[2]

## ABSTRACT

Masonry has been a part of our construction process for a long time; however, until recently its uses have not taken advantage of its inherent structural capabilities. Of particular interest are the load bearing characteristics of brick-block composite masonry walls and their behavior under certain load conditions. The small amount of experimental and analytical research performed on composite masonry has not offered insight into the understanding of interlaminar shearing stresses in the collar joint that are created by the interaction between the two wythes of masonry. Determination of these stresses is quite important for the safe and economic design of composite masonry structures.

A two dimensional composite element is developed in this study that is capable of predicting the out-of-plane interlaminar shearing stresses between the brick and block wythes without using a three-dimensional finite element model. This composite element is utilized in two-dimensional analyses of some composite masonry walls subjected to in-plane loads that are in a plane-strain condition. The results of these analyses are compared with those using the plane-strain finite element models. These comparisons indicate that the two-dimensional composite element models predict interlaminar shearing stresses in the collar joint that are in good agreement with those obtained from the plane-strain models.

Although the composite element is initially developed for only elastic conditions in an unreinforced collar joint without any consideration to fracture and failure, these criteria can be easily incorporated in the proposed model.

[1]Prof. of Civil Engrg., Clemson Univ., Clemson, S.C. 29631.

[2]Design Engineer, Piedmont Engineers and Architects, Greenville, S.C., formerly, Grad. student, Dept. of Civil Engrg., Clemson Univ. Clemson, S.C. 29631.

# A FINITE ELEMENT MODEL TO PREDICT INTER-LAMINAR SHEARING STRESSES IN COMPOSITE MASONRY

By Subhash C. Anand[1] and David T. Young[2]

## INTRODUCTION

Economical and efficient construction utilizing brick masonry and concrete masonry walls as structural components of a total structural system has prompted a rapid increase in interest in taking advantage of the structural capabilities of masonry. Coinciding with this increase in interest has been an increase in research and testing of the structural capabilities of both brick masonry and concrete masonry. This research has led to the development of more rational design procedures as opposed to the previously empirical design methods. As a result, there now exist engineered masonry design standards for the structural design and construction of brick masonry (6)[3] and concrete masonry (16), developed from research performed on masonry consisting of only brick or concrete block (6,8,17,19-21). On the other hand, there exists no thorough design standard for structural members composed of both brick and concrete masonry. Most standards (3,4,6,16) have included provisions for the design of such composite members, but the design is not based on experimental or analytical research data since very little experimentation or research with composite masonry has been performed. Since a considerable portion of all nonresidential masonry construction is composite, it seems reasonable to expect the emergence of a complete design standard for composite masonry. However, thus far the research in this area has been little more than prism and compression wall testing, and little information is available concerning the flexural or shear strength of composite masonry. Thus, for a material that is widely used in construction, further experimental and analytical research is urgently needed for the development of a design and construction standard.

Composite masonry walls are usually constructed of a single wythe of brick and a single wythe of concrete block with a collar joint (filled cavity) between the two. If the cavity remains hollow, as shown in Fig. 1 (a), the design standards developed for each material are applicable. However, when the cavity between the two wythes is parged or grouted and connected by metal ties as shown in Fig. 1 (b), the two wythes are bonded together and they react as a single unit. This composite unit maintains some of the properties that govern each

---

[1]Prof. of Civil Engrg., Clemson Univ., Clemson, S.C. 29631.

[2]Design Engineer, Piedmont Engineers and Architects, Greenville, S.C., formerly, Grad. student, Dept. of Civil Engrg., Clemson Univ. Clemson, S.C. 29631.

[3]Numerals in parentheses refer to corresponding items in the Appendix I... References.

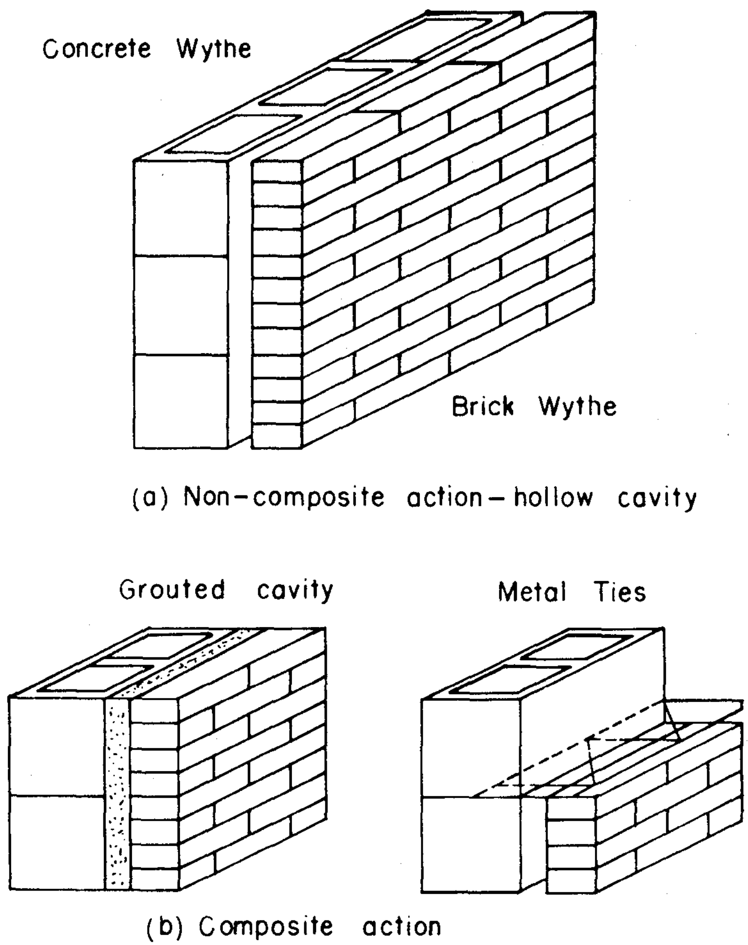



Fig. 1 Non-Composite and Composite Masonry Walls

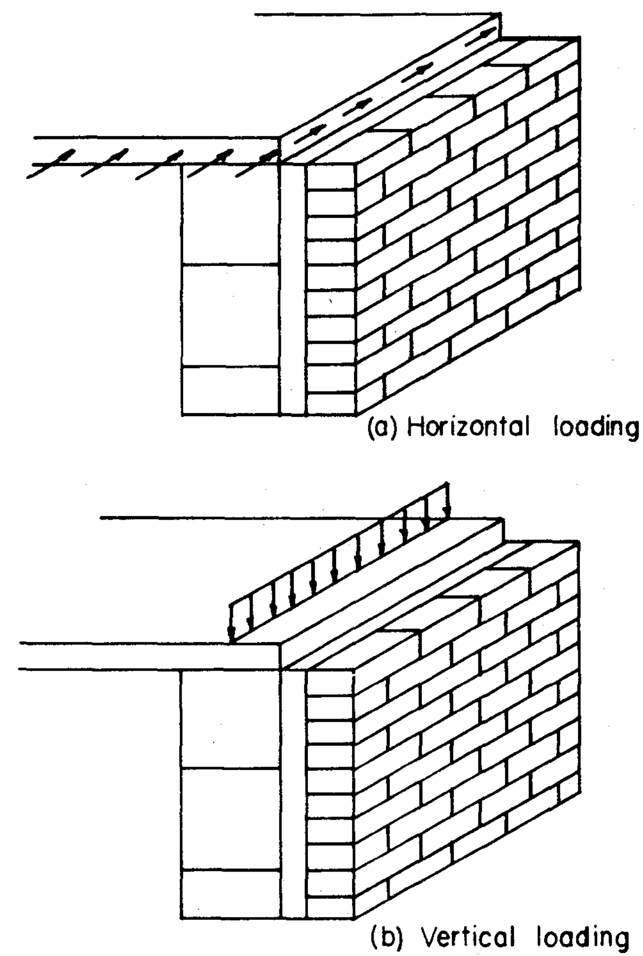



Fig. 2 Load Application in a Composite Wall

of its components and at the same time acquires its own set of characteristics which must be investigated and documented. One particular topic that needs special attention is the interaction of the two wythes under internal stresses and external loads. More specifically of concern are the shearing stresses created in the collar joint as a result of this interaction. Because these inter-laminar shearing stresses can be shown to have substantial magnitude, their effects must be understood before they can be incorporated into a design standard.

Currently available literature offers little insight into inter-laminar shearing stresses in a collar joint. As the analysis of a composite masonry wall is a complicated three dimensional problem due to non-uniform loading and possible wall openings, a numerical technique such as the finite element method can be utilized to a great advantage. In an effort to further simplyfy these analyses, a two dimensional finite element model is proposed to obtain a solution to the three dimensional problem. In this initial phase of the model development, the materials are assumed to behave linearly and the collar joint is considered unreinforced.

## Previous Research

A joint research study was conducted by the Brick Institute of America (BIA) and the National Concrete Masonry Association (NCMA) in an effort to provide sufficient technical data to develop a standard for the engineering design of composite masonry (18). The first phase involved the construction and testing of compressive prisms and the second phase compression testing of thirty long composite wall columns. Plans were made for additional testing in the areas of flexural and diagonal tension, but were never carried out. No mention was made in these studies of shearing stresses in the collar joint. Fattal and Cattaneo (7) reported research on the testing of prisms and walls of various constituents (including composite masonry) subjected to compressive and flexural loading. Examination of failures of composite walls tested under vertical compression did not reveal any separation of the interface between the brick and the block. All specimens were reinforced with truss-type reinforcing. Again, no mention was made of shear stresses in the collar joint. Grimm and Fowler (10) recognized and investigated the structural effects of differential movements in composite masonry. The long term net volume increase in brick due to moisture expansion and the shrinkage of concrete masonry upon drying can result in the development of shear stresses in a collar joint of a composite masonry wall. The substantial magnitude of these shearing stresses is magnified by the occurrence of reversible thermal expansion and contraction and creep in each wythe of highly different rates. Elementary mechanics principles were used to calculate the differential movement and resulting interface shearing stresses from the phenomenon mentioned above. It is important to note that these shearing stresses occur before the application of any loads.

All of the research mentioned above (7,10,18) dealt in total or in part with composite masonry behavior. However, only one paper (10) was concerned with shearing stresses in the collar joint and that also at an elementary level dealing with stresses due to moisture, creep, and thermal effects. In the aerospace industry, on the other hand, recognition of inter-laminar shear deformation in laminated fibrous composites has generated much interest in the development of analytical methods that can account for this phenomenon (14,15). Some of these techniques can be adopted for the study of shearing stresses in composite masonry. The research in these two references involved the application of the finite element method to the case of generalized plane stress. A composite element was developed which was composed of homogeneous membranes with orthotropic properties separated by isotropic layers of finite thickness that could develop only inter-laminar shearing stresses. The development of the stiffness matrix for the composite element was also presented. Results of the finite element solution that were presented included the distribution of normal stresses in the membranes and shearing stresses in the inter-laminar region. These results were in reasonable agreement with those predicted by an analytical technique.

## Current Research and Practice

There is no evidence of any research being conducted at present with respect to composite masonry and more specifically concerning shearing stresses in the collar joint.

The engineered brick masonry standard (6) contains the allowance for composite wall design based on the allowable stresses of the weakest constituent. Even though use of the full wall thickness is allowed, and requirements are specified for bonding with metal ties, no consideration is given to the differential movement and resulting shearing stresses at the collar joint. Similar composite masonry wall design guidelines are provided in other standards (4,16). The American Concrete Institute has issued a code for concrete masonry (3) that differs from other codes in that it recommends the use of the transformed section concept for flexural and axial design of composite masonry walls. However, there is no criteria governing shearing stresses in the collar joint.

## Related Research

Some results of research on non-composite masonry are available that may be of assistance in the analysis of composite masonry. Constitutive relationships and failure criteria for concrete masonry subjected to biaxial stresses have been developed by Hegemier *et al.* (11,12,13). Flaws at the interface of the grout and the concrete block cell walls were detected (11). This phenomenon may be similar to delamination of the collar joint in a composite masonry wall. Effects of nonlinear deformation and post-fracture behavior were also investigated. Arya and Hegemier (5) applied the finite element technique to analyze the non-linear response of concrete masonry and pre- and post-fracture behavior of joints and interfaces in a concrete masonry assemblage. The analysis also investigated masonry cracking and the effects of reinforcing steel. Page (17) dealt with brick masonry walls subjected to in-plane loading and developed a finite element model that accounted for the nonlinear behavior of masonry that was considered as a two phase material. The masonry was modelled as elastic brick elements set in an inelastic mortar matrix. Failure occurred in the joints if a prescribed tensile or shear bond stress was exceeded.

Grimm (9) recognized the importance of metal ties and anchors to the structural integrity of masonry walls, and presented a rational analysis for the design of wall anchorage and connection systems based on the physical and chemical properties of metal ties and anchors. Grimm and Fowler (10) found that differential movement between wythes of a cavity wall can result in significant bending moments in the metal ties connecting the wythes. A guide to masonry reinforcing (1) describes reinforcing and tie systems often used in masonry construction.

## PROBLEM DEFINITION

The three major sources of shearing stresses between wythes in a composite wall are: (1) differential movement of dissimilar wythes of masonry due to thermal and curing effects; (2) inplane loads applied on one wythe only or a load differential across the wythes; and (3) flexural shearing stresses caused by out-of-plane loading. The effects of differential movements in composite masonry walls due to thermal and curing effects have been investigated by Grimm and Fowler (10). The flexural shearing stresses that result from out-of-plane loading such as lateral wind loads are felt to be negligible. Shearing stresses caused by in-plane loads applied on one wythe only or by a greater load on one wythe than on the other are felt to be the most significant and are considered in this study.

The load transfer mechanism into a composite masonry wall in many types of construction is shown in Figs. 2 (a) and 2 (b). In Fig. (2) b, the floor slab rests on the interior (block) wythe of the composite wall, and the vertical in-plane dead and live loads are transferred from the floor directly into the inner concrete block wythe. Some percentage of these loads is gradually transferred to the outer brick wythe through shear stresses in the collar joint. In Fig. 2 (a), in-plane lateral loads (wind load) are transferred to the floor slab by the transverse wall. These horizontal loads are transferred from the floor slab to the inner wythe, and some of these loads are further transferred to the outer wythe through shear stresses in the collar joint. Each loading condition reiterates the importance of understanding the existence and magnitudes of these inter-laminar shearing stresses. If the resulting shearing stresses are large enough to cause a loss of bond between the collar joint and either wythe, the wall could become underdesigned. Presently, the mechanism of load transfer from one wythe to the other and the magnitudes of the accompanying shearing stresses in the collar joint are not known. The inter-laminar shearing stresses that result from the types of loading mentioned earlier are shown in Fig. 3.

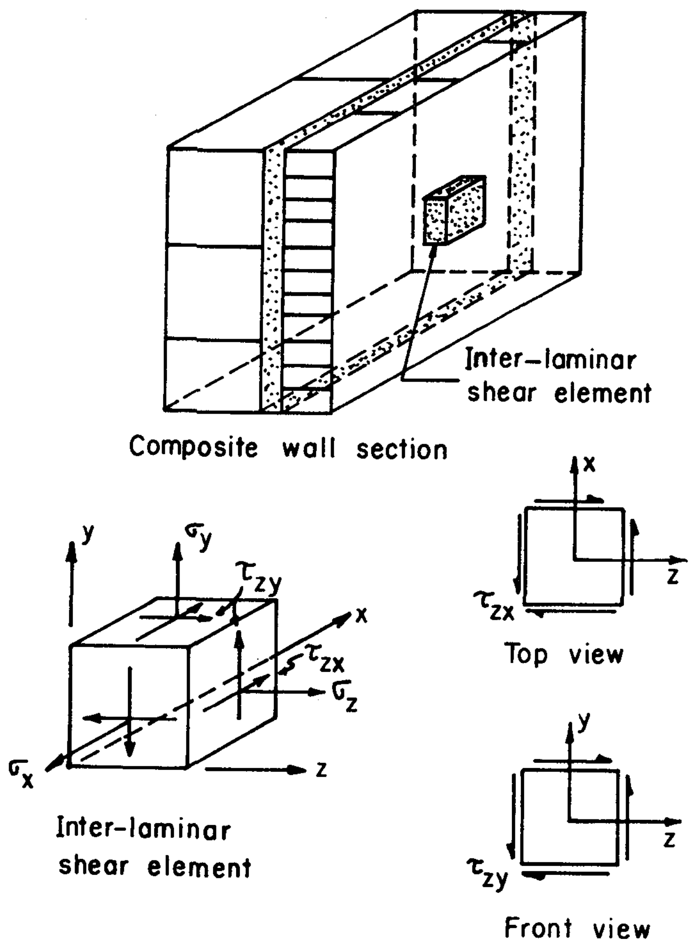



Fig. 3 Inter-Laminar Shearing Stresses Due to In-Plane Loading

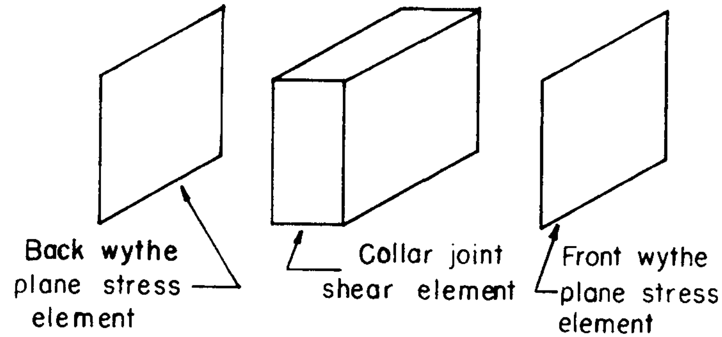



Fig. 4 Components of a Composite Element

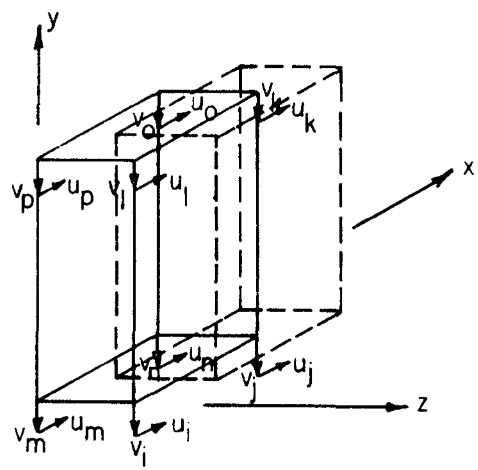



Fig. 5 Deformation of a Collar Joint Shearing Element

# SOLUTION PROCEDURE

The finite element technique for the calculation of inter-laminar shearing stresses in fibrous composites, as used in aerospace industry (14,15), will be employed in this study to understand the behavior of composite masonry walls. The finite element method, in general, is well known (23) and no attempts are made here to explain it in detail except some steps as to how this technique can be applied for the analysis of a composite masonry wall. In particular, the development of the element stiffness matrix for a composite element consisting of the two wythe faces and the collar joint is presented.

## Stiffness Matrix of a Composite Element

The analysis of a collar joint in a composite wall requires not only the in-plane stiffness for each wythe, but also the interactive or shearing stiffness of the collar joint. Thus, the element might be thought of as "three-dimensional" since the interaction between the two wythes across the collar joint is out of the plane of each wall. An example of such conditions is shown in Fig. 4 for a composite wall. Formulation of a composite element stiffness is accomplished by first developing the in-plane stiffness representing each of the masonry wythes, then developing the shearing stiffness representing the collar joint, and finally combining these stiffnesses for the total composite element stiffness. Each phase of this development is explained in detail in the following sections.

The following assumptions have been made in the development of the stiffness matrix for a composite element. (1) All materials are considered to be homogeneous. (2) Displacements are assumed to vary linearly between the nodal points in the finite element model to insure displacement continuity between the elements. (3) Out-of-plane bending effects are neglected in the development of the model. (4) The collar joint as well as the two wythes are assumed to be unreinforced.

### Stiffness Matrix of a Wythe Element

Determination of stresses and displacements in the wythes due to in-plane loads can be accomplished by the standard plane stress finite element analysis. The governing matrix equation relating forces and displacements in an element is given by

$$\{P\} = [k]\{u\} \tag{1}$$

where $\{P\}$ is a column matrix of force-components in the x and y directions at the nodes of an element, $[k]$ is the in-plane stiffness matrix for an element, and $\{u\}$ represents the corresponding displacement components at the nodes. A more detailed development of the in-plane stiffness matrix may be found in standard finite element texts (23). Quadrilateral elements, each consisting of four triangular elements, were utilized in subdividing the wythe faces into a finite element mesh. As each node has two degrees of freedom, Eq. 1 yields an 8 x 8 stiffness matrix, $[k]$, for each element.

Stiffness Matrix of a Collar Joint Element

The collar joint element stiffness provides the interaction between the two plane stress elements representing the masonry wythes. The three parts of a composite element are shown in Fig. 4 with the wythe element stiffnesses for the front wythe $[k_f]$ and the back wythe $[k_b]$ having been developed in the previous section. As in the previously reported research (14,15), the collar joint material is assumed to provide only a shearing stiffness, i.e., a resistance to shear deformation only. Shear deformation of the collar joint element is shown in Fig. 5 and is composed of displacements in the x and y directions. Since these displacements are at the nodes of the plane-stress grids and vary linearly in the plane of each grid, the stiffness properties of the inter-laminar shear elements are also based on a linear displacement field in the x - y plane. Thus, the shear strains in the collar joint shear element may be written in terms of the nodal displacements as

$$\gamma_{zx} = \frac{\partial u}{\partial z} = (u_i + u_j + u_k + u_l - u_m - u_n - u_o - u_p)/4t \tag{2}$$

$$\gamma_{zy} = \frac{\partial v}{\partial z} = (v_i + v_j + v_k + v_l - v_m - v_n - v_o - v_p)/4t \tag{3}$$

in which u and v are displacements in the x and y directions, respectively; subscripts refer to specific nodes, and t = thickness of the collar joint. From these equations, it is evident that the inter-laminar shear strain is computed from the centroidal values of displacements of the two wythes. Thus, the shear resisting medium may be regarded as a shear segment connecting the centroids of the two wythe elements facing one another. It should be noted that the finite element grids on the front and back wythes are of exactly the same size and dimension.

The strain-displacement relations can be derived from Eqs. 2 and 3 as

$$\begin{Bmatrix} \gamma_{zx} \\ \gamma_{zy} \end{Bmatrix} = [B]\{u\} \tag{4}$$

in which matrix [B], relating displacements to strains, can be defined by

$$[B] = \frac{1}{4t}\begin{bmatrix} 1 & 0 & 1 & 0 & 1 & 0 & 1 & 0 & -1 & 0 & -1 & 0 & -1 & 0 & -1 & 0 \\ 0 & 1 & 0 & 1 & 0 & 1 & 0 & 1 & 0 & -1 & 0 & -1 & 0 & -1 & 0 & -1 \end{bmatrix} \tag{5}$$

and

$$\{u\} = [u_i v_i \; u_j v_j \ldots\ldots\ldots u_p v_p]^T \quad . \tag{6}$$

Stress-strain relations in shear may be written as

$$\begin{Bmatrix} \tau_{zx} \\ \tau_{zy} \end{Bmatrix} = G_s \begin{bmatrix} 1 & 0 \\ 0 & 1 \end{bmatrix} \begin{Bmatrix} \gamma_{zx} \\ \gamma_{zy} \end{Bmatrix} \quad , \tag{7}$$

where $G_s$ is the shear modulus, which yields the material property matrix, [D], as

$$[D] = G_s \begin{bmatrix} 1 & 0 \\ 0 & 1 \end{bmatrix} \tag{8}$$

As in the case of the in-plane stiffness matrix for an element, the force-displacement relations for the collar joint shear element may be given by Eq. 1, in which the element stiffness matrix [k] is defined as

$$[k] = [B]^T [D][B]\, tA\,. \tag{9}$$

A in Eq. 9 is the area of the wythe element.

Carrying out the matrix multiplication above leads to the collar joint shearing element stiffness matrix

$$[k_{sh}] = \frac{G_s A}{16t} \left[\begin{array}{cccccccccccccccc}
1 \\
0 & 1 \\
1 & 0 & 1 \\
0 & 1 & 0 & 1 \\
1 & 0 & 1 & 0 & 1 \\
0 & 1 & 0 & 1 & 0 & 1 & & & & \text{SYMMETRIC} \\
1 & 0 & 1 & 0 & 1 & 0 & 1 \\
0 & 1 & 0 & 1 & 0 & 1 & 0 & 1 \\
-1 & 0 & -1 & 0 & -1 & 0 & -1 & 0 & -1 \\
0 & -1 & 0 & -1 & 0 & -1 & 0 & -1 & 0 & -1 \\
-1 & 0 & -1 & 0 & -1 & 0 & -1 & 0 & -1 & 0 & -1 \\
0 & -1 & 0 & -1 & 0 & -1 & 0 & -1 & 0 & -1 & 0 & -1 \\
-1 & 0 & -1 & 0 & -1 & 0 & -1 & 0 & -1 & 0 & -1 & 0 & -1 \\
0 & -1 & 0 & -1 & 0 & -1 & 0 & -1 & 0 & -1 & 0 & -1 & 0 & -1 \\
-1 & 0 & -1 & 0 & -1 & 0 & -1 & 0 & -1 & 0 & -1 & 0 & -1 & 0 & -1 \\
0 & -1 & 0 & -1 & 0 & -1 & 0 & -1 & 0 & -1 & 0 & -1 & 0 & -1 & 0 & -1
\end{array}\right] \tag{10}$$

## Total Element Stiffness Matrix

The element shown previously in Fig. 4 is the composite element representing the combination of both wythe plane stress elements and the collar joint shear element. The analysis of the total composite problem requires the superposition of the two 8 x 8 wythe element stiffness matrices and the 16 x 16 collar joint shear element stiffness matrix. The superposition of the three stiffness matrices results in a 16 x 16 composite element stiffness matrix which is given by

$$\left[ \begin{bmatrix} \underset{8\times8}{[k_f]} & \underset{8\times8}{[0]} \\ \underset{8\times8}{[0]} & \underset{8\times8}{[k_b]} \end{bmatrix} + \underset{16\times16}{[k_{sh}]} \right] \tag{11}$$

in which $[k_f]$ and $[k_b]$ are the plane stress stiffness matrices of the front and back wythes, respectively; and $[k_{sh}]$ is the shear stiffness matrix of the collar joint. It is worth noting the fact that the wythe stiffness matrices are coupled with each other through the collar joint shear stiffness matrix, and otherwise react independently.

Thus, if the collar joint were not grouted and were left hollow, as in a cavity wall, the unloaded wythe element would not receive any of the load that acts on the loaded wythe.

### Calculation of Displacements, Stresses and Strains

Using the stiffness matrix of a composite element developed in the previous section, the stiffness matrix for a finite element model of the total structure can be assembled by the standard methods leading to the equilibrium equations which are solved for the nodal point displacements. It should be noted that quadrilateral elements composed of four triangular sub-elements have been employed in this study. The strains within the plane stress elements are first calculated for the four triangular sub-elements, and the corresponding strains in the quadrilateral element are obtained as an average of those in the sub-elements. Shearing strains in each element across the collar joint are calculated by using the shearing element strain-displacement matrix [B] given in Eq. 5. In-plane stresses in the wythe elements are calculated from in-plane strains by using the plane-stress, stress-strain transformation matrix given in standard texts (23). Shearing stresses in the collar joint elements, on the other hand, are calculated from the corresponding strains by the stress-strain matrix, [D], given in Eq. 8.

## DEVELOPMENT AND VERIFICATION OF THE LONGITUDINAL MODEL

The general objective of this study is to develop a two-dimensional model that will provide accurate analysis of a composite wall subjected to in-plane loads and account for shearing stresses in the collar joint. A composite element stiffness matrix has been developed in the previous section. In order to analyze a wall, it must be represented as an assemblage of these composite elements arranged in a plane. The resulting finite element model of the wall will be denoted as the longitudinal model in the subsequent discussion.

In specific, it is desired to understand the load transfer mechanism between the two wythes through the existence of the shearing stiffness that leads to the inter-laminar shearing stresses across the collar joint. As the cross-section of the composite wall and the in-plane loading acting on it is primarily non-symmetric, certain out-of-plane deformations occur which cannot be included in the longitudinal model. To assess their importance, a wall subjected to an in-plane eccentric load along its whole length is also analyzed by using a plane-strain finite element model through the wall cross-section. This model will be denoted as the transverse model and is composed of plane-strain quadrilateral elements which have the same dimensions in the vertical direction as those in the longitudinal model.

### Material Properties, Dimensions and Loads

All analyses are conducted on walls composed of masonry and grout possessing the typical dimensions and material properties shown in Table 1. Modulus of elasticity, $E_m$, for the masonry is computed by (6)

$$E_m = 1000\ f'_m \tag{12}$$

where $f'_m$ is the typical compressive strength of masonry assemblages using type M or S mortar. Poisson's ratio, $\nu$, is determined from the shear modulus, $E_v$, which may be given as

$$E_v = 400\ f'_m\ . \tag{13}$$

Table 1. Material Properties of Composite Masonry Walls

| Masonry Type | $f'_m$ (psi) | $E_m$ (psi) | $E_v$ (psi) | $\nu$ |
|---|---|---|---|---|
| 4" Brick | 1600 | 1,600,000 | 640,000 | 0.25 |
| 8" Solid Concrete Block | 1800 | 1,800,000 | 720,000 | 0.25 |
| 8" Hollow Concrete Block | 1800 | 1,800,000 | 720,000 | 0.25 |
| Grout | 2000 | 2,550,000 | 1,062,500 | 0.20 |

Formulas for determining the modular values for grout are different from those given in Eqs. 12 and 13 for masonry units (6,16). A compressive strength, $f'_m$, of 2000 psi is representative of the range of values assumed for grout materials. Likewise, a value of 0.20 is representative of the generally accepted range of values for Poisson's ratio. A good approximation for the modulus of elasticity based on compressive strength may be given by (16)

$$E_m = 57000\ \sqrt{f'_m}\ , \tag{14}$$

from which the shear modulus, $E_v$, can be easily calculated.

In most composite masonry wall construction, the collar joint is either 2" thick and grouted or 3/8" thick and parged. Because the effects of the shearing stresses in the collar joint would be magnified in a 2" joint, the initial investigations are performed on a composite wall with a 2" joint thickness. Since composite wall action is evident only after the floor slab loads are applied, the loading utilized in this study is derived from a typical structural system having a 20 foot slab span supported on one end by a composite wall. For a live load of 100 psf and a slab thickness of 6 inches, the uniform load on the wall approximates to 2 kips per foot.

## Phase 1 - Investigation of Composite Element Behavior

### Evaluation of Collar Joint

The first phase of model development is concerned with the evaluation of the composite element behavior as assembled in a longitudinal grid. To assist in this evaluation, certain constraints are imposed on the transverse model to conform its behavior more closely to the longitudinal model.

Grids are initially chosen as shown in Fig. 6 that represent a 12 inch length of a short wall of arbitrary height. Both wythes are selected arbitrarily of 8" solid concrete block to eliminate any effects resulting from dissimilar wythes. Longitudinal constraints are imposed

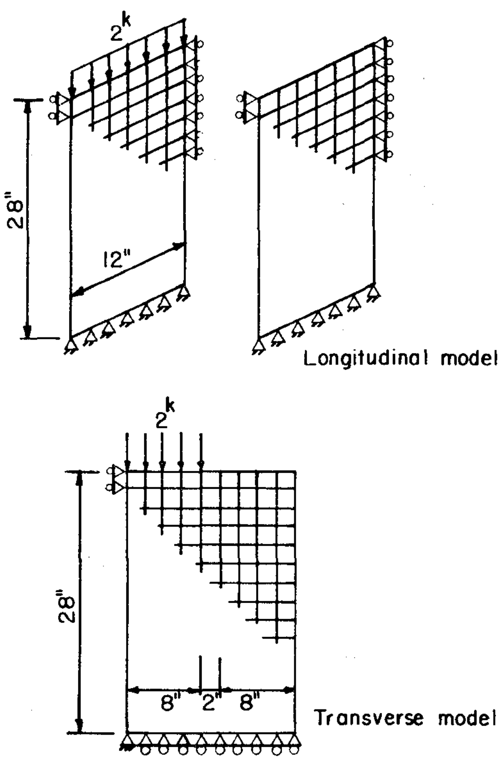



Fig. 6 Loads, and Boundary Condition for Coarse Mesh

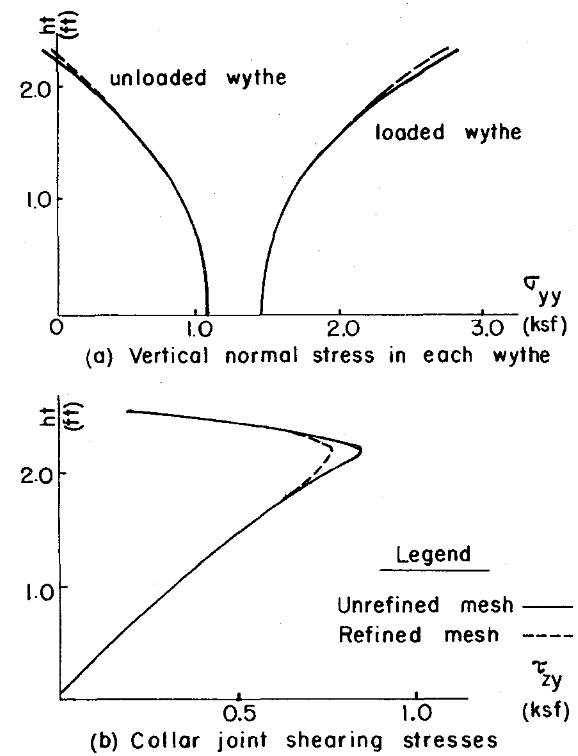



Fig. 7 Normal and Shearing Stresses in Fine and Coarse Mesh Transverse Models

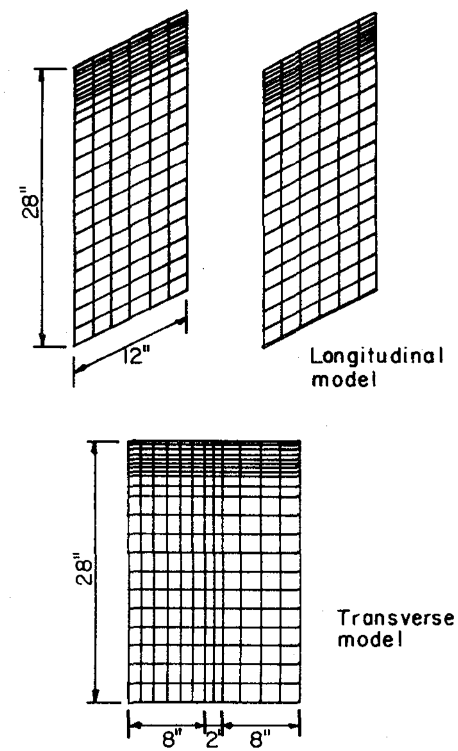



Fig. 8 Fine Mesh for Both Models

at the ends of the longitudinal model to approximate the plane strain conditions that exist in the transverse model. Boundary conditions at the base of the transverse model are represented by rollers. Lateral constraints are placed along one exterior face of the transverse model to inhibit deformation due to bending but still permitting horizontal movement due to Poisson's effects. Normal stresses in both wythes and shearing stresses in the collar joint for the coarse mesh in a transverse model are shown in Fig. 7. It is obvious from Fig. 7 (b) that most of the load transfer across the collar joint occurs near the top of the wall.

To better evaluate the shearing stresses in the collar joint created by this loading, a more refined grid as shown in Fig. 8 is utilized. Vertical normal stresses in the two wythes and shearing stresses in the collar joint for the unrefined and refined models are compared in Fig. 7 (a) and 7 (b), respectively. The results of the refined grid show a better correlation between wythe normal stresses and load intensity and also yield a smoother transition from increasing to decreasing shearing stresses.

<u>Wythe Element Deformation in the Transverse Model</u>. Utilizing the refined grids, both the longitudinal and transverse models are evaluated. The results for wythe normal stresses and collar joint shearing stresses shown in Figs. 9(a) and 9 (b), respectively, reveal significant differences between the two models. This may be explained as follows: The longitudinal model represents both masonry wythes by plane stress grids of the collar joint. Displacement at a node on either wythe represents the corresponding displacement throughout the thickness of the wythe in this model. Such is not the case in the transverse model where a wythe may deform differentially across its thickness. To enforce same conditions in the transverse model, each wythe element is made infinitely rigid in shear deformation by insuring that vertical displacements of all nodes in a wythe at a given level are kept equal. This can be easily achieved by modifying the stiffness matrix of the transverse model (22). A better agreement of the results between the two models is obtained as shown in Fig. 10, when the differential wythe deformations are eliminated by equating vertical displacements in the transverse model.

<u>Collar Joint Shearing Stiffness in the Transverse Model</u>. The stresses shown in Fig. 10, although much improved when compared to those in Fig. 9, still do not closely agree for the two models. This difference may be attributed to the stiffness of the collar joint, which has only shearing stiffness in the longitudinal model but both shearing and normal stiffnesses in the transverse model.

The stiffness matrix of the collar joint in the transverse model was modified to include only shearing stiffness. The details of this modification may be found in Ref. (22). The wall was reanalyzed using this modified stiffness in the transverse model and yielded stresses that are shown in Fig. 11. A comparison of these stresses with the corresponding results in Fig. 10 shows insignificant improvement, indicating the presence of some other effects that influence the

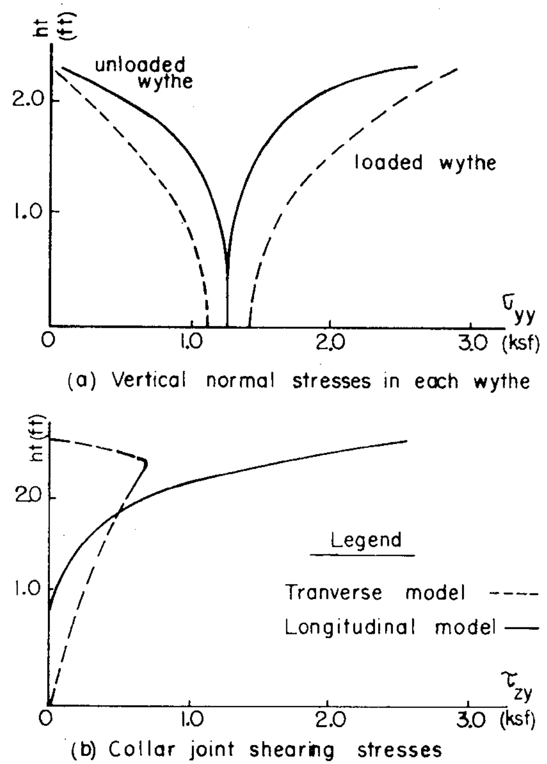



Fig. 9 Stresses in Longitudinal Model and Transverse Model With Differential Wythe Deformation

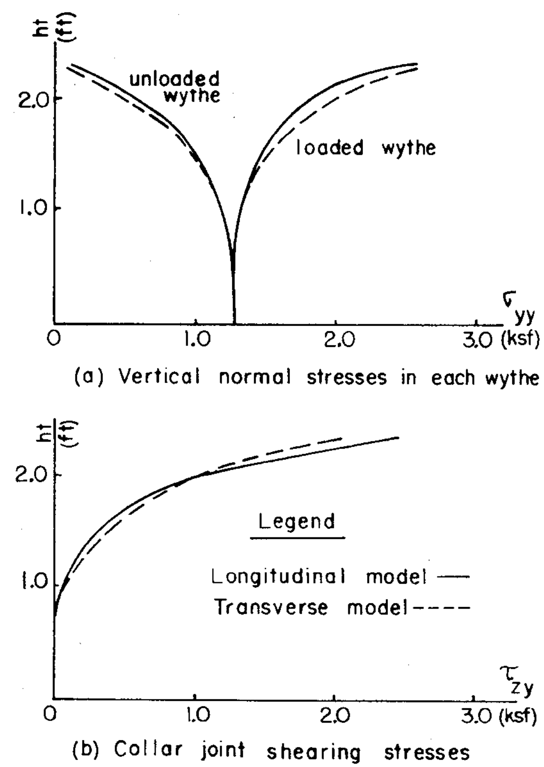



Fig. 10 Stresses in Longitudinal Model and Transverse Model With No Differential Wythe Deformation

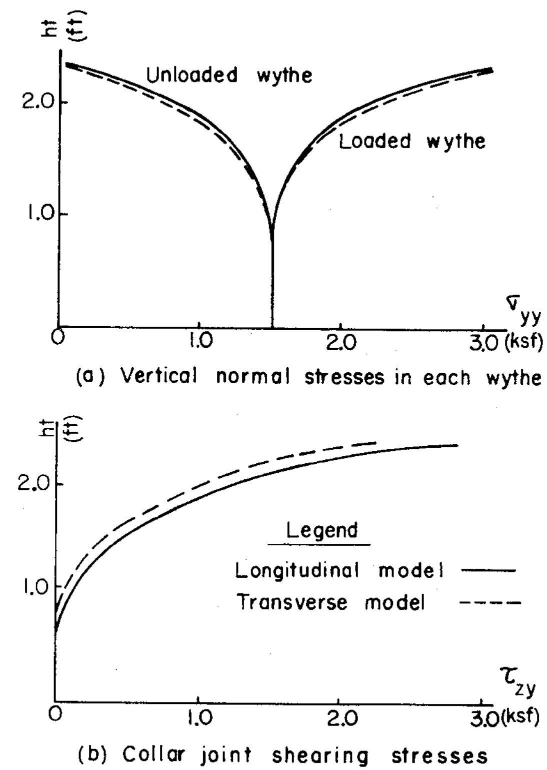
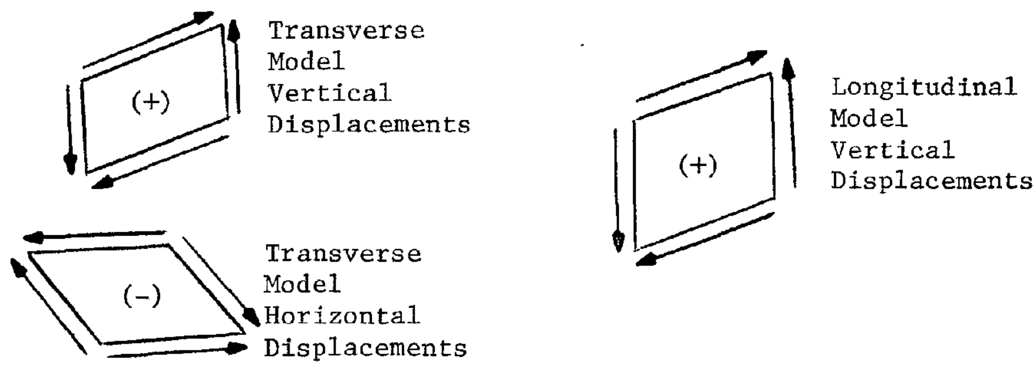



Fig. 11 Stresses in Both Models With Collar Joint Having Only Shearing Stiffness

analysis. A closer inspection of the results revealed the existence of horizontal displacements perpendicular to the wall in the transverse model. These displacements are not included in the longitudinal model. Shearing strains in the transverse model are due to the sum of the positive and negative angle changes caused by both the horizontal and vertical displacements shown in the diagrams that follow. When compared with the shearing strains in collar joint elements in the longitudinal model, the shearing strains in the transverse model are different because of the opposite effects of the horizontal displacements that are absent in the longitudinal model.




Because of the presence of interactive terms in the collar joint element shear stiffness in the transverse model relating horizontal displacements to vertical forces, the opposing horizontal displacements tend to restrict the transfer of load across the collar joint. Thus, the vertical normal stresses in both wythes in the transverse model differ from the corresponding stresses in the longitudinal model. However, if the shear strains in the collar joint are expressed in terms of only the vertical displacements of the two wythes, a modified shear stiffness matrix results for the transverse model that leads to normal stresses which are identical to those obtained from the longitudinal model. The corresponding shear stresses in the two models also coincide.

Discussion

The investigation thus far has resulted in the discovery of several shortcomings of the composite element and the corresponding longitudinal model. These shortcomings result from the incapability of the longitudinal model to incorporate certain out-of-plane effects which are present in the transverse model. First, the longitudinal model represents each wythe as a single grid of elements; i.e., elements in the wythe are incapable of any relative in-plane deformation across its width. This is not the case in a real system, and as seen in the transverse model, each wythe element does undergo transverse shear deformation. The effect of this phenomenon in the actual wall behavior is to render each wythe less stiff, thus permitting a slower load transfer across the collar joint than is indicated by the longitudinal model.

Secondly, a transverse model supported laterally to prevent out-of-plane bending, when subjected to loads in the vertical direction only, possesses a relatively high rigidity. Consequently, Poisson's effects, that yield horizontal displacements, are predominant. However, these effects cannot be included in the longitudinal model due to its inability to represent any out-of-plane displacements. These displacements in the transverse model restrict the load transfer and thus give higher normal stresses in the loaded wythe and lower normal stresses in the unloaded wythe when compared to the corresponding stresses in the longitudinal model. Likewise, these displacements in the transverse model produce shearing strains in the collar joint that are of the opposite sign than those due to vertical displacements. Therefore, the collar joint shearing strains (and stresses) in the longitudinal model are higher than those in the transverse model because of the exclusion of these horizontal displacements in the former model. Finally, since each wythe in the longitudinal model is represented by a grid at the face of a collar joint, all of the wythe load must be applied as a concentrated load at this face. In the transverse model, however, the load is represented more correctly as a uniform load across the width of the wythe. Since this uniform loading cannot be accomplished in the longitudinal model, the compressive stress in the top layer of the loaded wythe, in this case, does not give a good representation of the actual load intensity. Also, an application of the wythe load at the collar joint face results in a quicker load transfer to the unloaded wythe in the longitudinal model.

Comparisons between the two models that have been discussed in these paragraphs are based on a short, stiff composite wall. Composite walls that represent actual physical geometries and loading conditions are investigated in the following section by using both the models. Phenomena observed in the analyses of short walls are evaluated for walls under realistic conditions.

## Phase 2 - Investigation of Models Under Realistic Conditions

Whereas Phase 1 is concerned with evaluating the behavior of a composite element, this phase is involved in evaluating the results in realistic composite walls obtained by the use of the proposed composite element. Certain modifications are performed on the models used in Phase 1. A wall height of 10 feet is selected which is representative of typical floor-to-floor heights in buildings. Finite element meshes used in the longitudinal and the transverse models are shown in Fig. 12. Since most composite walls have dissimilar wythes, analyses are performed on four of the most commonly constructed masonry walls. Included are combinations of hollow block and brick as well as solid block and brick. Both of these masonry combinations are evaluated for walls with a 2 inch thick grouted or a 3/8 inch thick parged collar joint. It is assumed that floor loads are applied to a wall after the collar joint has been filled, and composite action does occur due to application of these loads.

The transverse model is used to present two limiting cases of the composite wall behavior under eccentrically applied floor loads. The

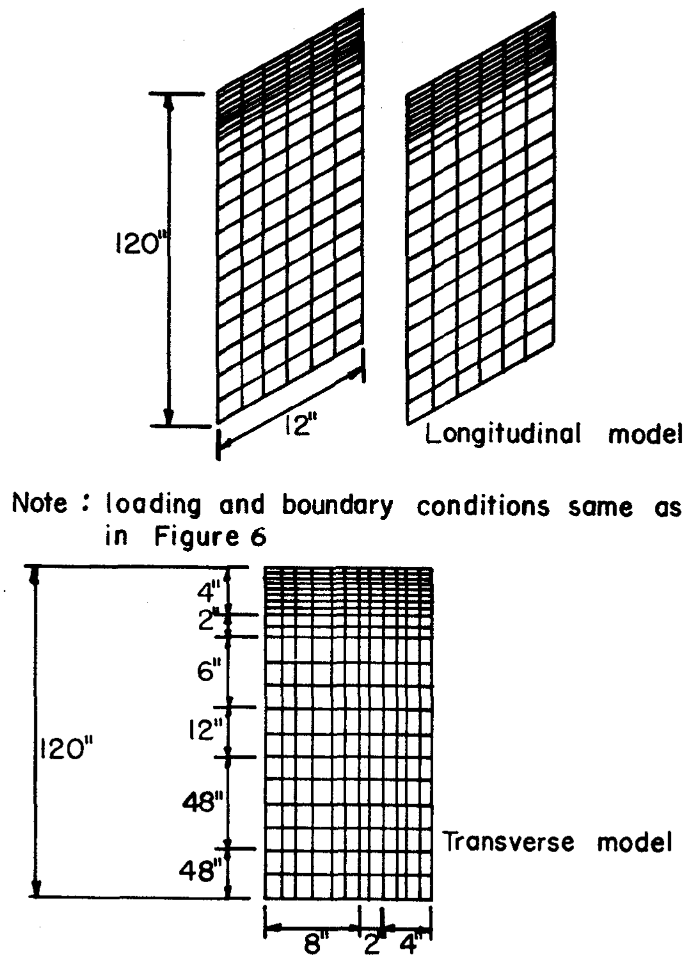



Fig. 12 Finite Element Model of a 10 Foot Wall

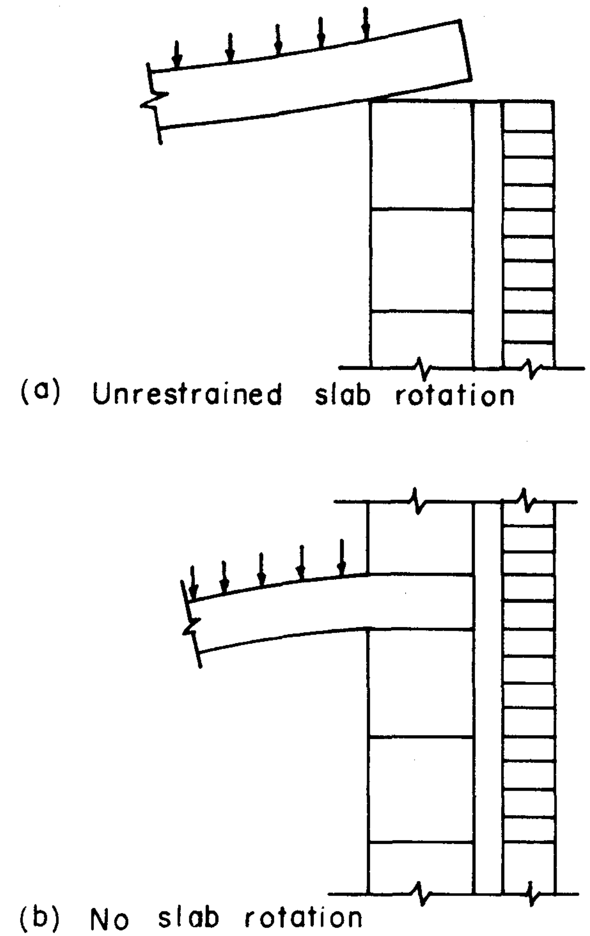



Fig. 13 Limiting Cases of Slab Rotation

resulting solutions are compared with those obtained by the use of the longitudinal model.

Limiting Case I: Wall Sections With Slab Rotation

Composite masonry wall sections in the upper floors of a building do not possess enough wall weight from stories above to prevent the floor slab from rotating under the application of floor loads. This phenomena is shown in Fig. 13 (a) as an extreme case in which the floor slab has rotated enough that all of the floor load is assumed to act at the interior corner of the block wythe in the transverse model. Because of the restraint provided by the floor slab at each floor level, the transverse model in Phase 2 is laterally supported at the top interior node of the loaded wythe. Differential vertical deformations at a given level are also permitted in order to represent a wall more realistically. In addition, analyses are performed with wythe widths modelled as single elements or as many elements across a wythe.

The vertical normal stresses in both wythes and shearing stresses in the collar joint for a hollow block-brick composite wall are shown in Fig. 14. The normal stresses in a single element model are approximately equal to the average of the corresponding stresses in all the elements of a wythe. If the collar joint is modelled with two elements, the shearing stresses in these two elements are approximately equal. They are also equal to those due to a one collar joint element solution, as shown in Fig. 14 (b). Consequently, all remaining analyses in Cases I and II are carried out with single elements in each wythe and collar joint. The results of the three remaining analyses in Case I are shown in Fig. 15. Because of the infrequent occurrence of this limiting case, results of the transverse models are shown for reference only and are not individually compared to the corresponding longitudinal models.

Limiting Case II: Wall Sections With No Slab Rotation

Composite masonry wall sections in the middle and lower floors of a building receive enough wall weight from stories above to prevent the floor slab from rotating under the application of floor loads. This phenomenon is shown in Fig. 13 (b) as an extreme case in which the vertical displacements across the top of the block wythe are considered equal. The limit of no slab rotation and equal displacements at the top of the loaded wythe is more representative of true wall behavior. Consequently, these conditions are imposed in the analyses of transverse models of composite walls.

The four walls analyzed in the previous section are reinvestigated here for the limiting case of no slab rotation. Since this case is the more predominant of the two limiting cases, results of the transverse model for each of the four walls are shown in Figs. 16 through 19 and are compared to the corresponding results of the longitudinal model. It should be noted that in order to represent the connection of the block wythe to the floor slab, the interior node at the top of the block wythe is restricted against horizontal movement.

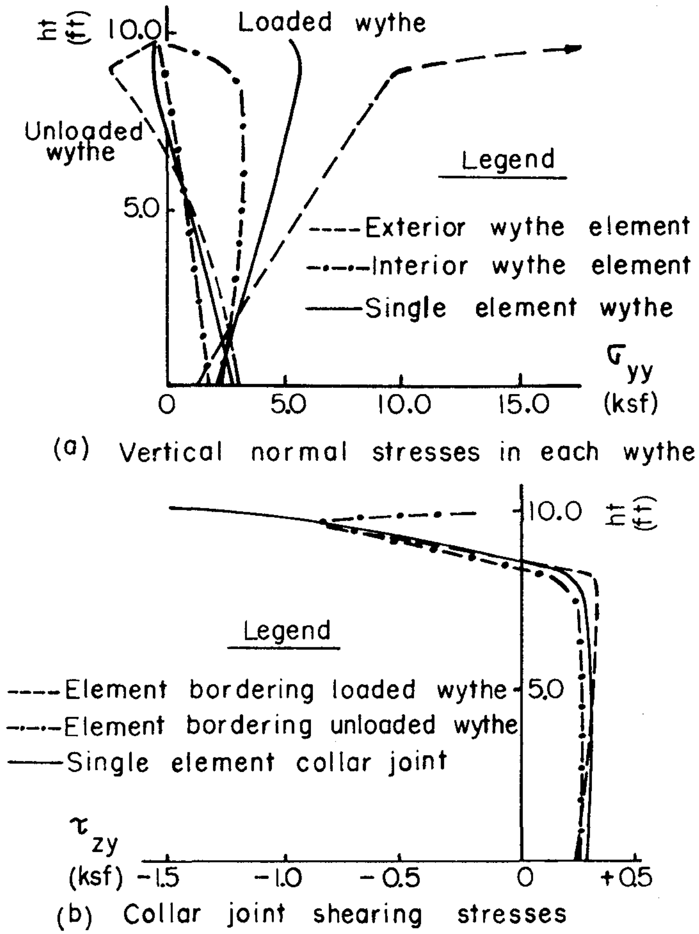



Fig. 14 Comparison of Multi- and Single Element Grids in a Transverse Model With Unrestrained Rotation

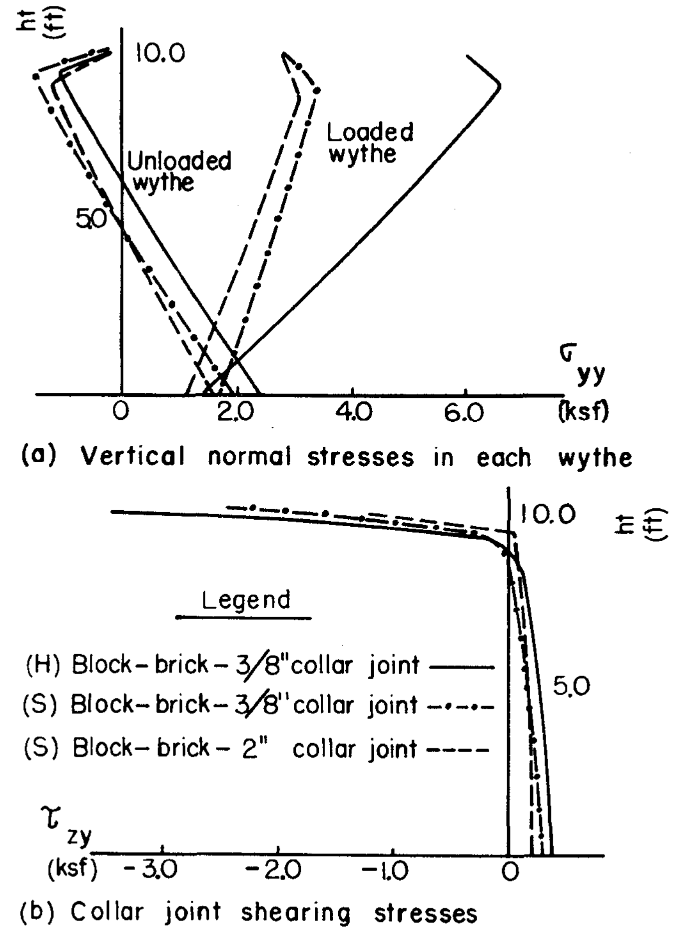



Fig. 15 Stresses in Various Composite Walls Using Transverse Model With Unrestrained Rotation

Discussion

In evaluating the results of the two limiting cases of composite wall behavior, it is important to recognize that the real wall behavior is bounded by the two cases presented earlier, results of which are considerably different from one another.

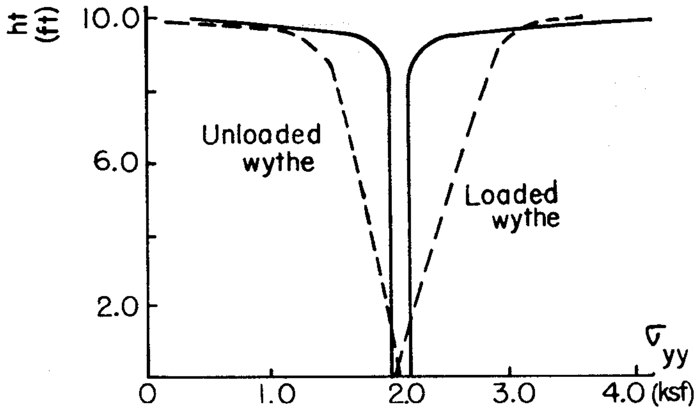
The results of the transverse models with unrestrained slab rotation shown in Figs. 14 and 15 do not compare favorably with the corresponding results of the longitudinal model given in Figs. 16 through 19. The large amount of slab rotation assumed in Case I places all of the floor load on the interior corner of the block wythe. The bending that results from such a large eccentricity produces significant horizontal displacements that are not included in the longitudinal model. One effect of these displacements is to reduce the shearing stresses in the collar joint predicted by the transverse model. As the wythes in the transverse model are less stiff than wythes in the longitudinal model due to their capability of differential nodal displacements in the vertical direction at a given level, a large load eccentricity assumed in Limiting Case I emphasizes the reduced wythe stiffness.

On the other hand, by restricting the slab rotation in the transverse model, the effects created by bending and wythe deformation are reduced, as seen in Figs. 16 through 19, where shearing stresses in the collar joint compare more favorably between the transverse and longitudinal models. By equating the displacements at the top of the loaded wythe, the effect of eccentricity of the load in the model is reduced. Consequently, horizontal displacements due to bending are not as significant. Shearing stresses in the collar joint of the transverse model are, thus, governed primarily by vertical displacements, the condition that is more comparable to that in the longitudinal model. Likewise, equating displacements in the transverse model reduces the amount of wythe deformation as the system becomes more stiff and the load transfers more quickly from one wythe to the other. This condition is evidenced in Figs. 16 through 19 in which the wythe vertical normal stresses in the transverse model are in better agreement with those in the longitudinal model than the corresponding stresses in the Limiting Case I.

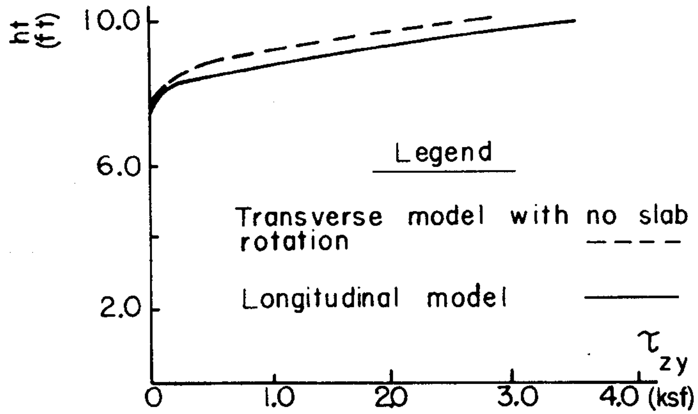
Comparison of the results for different types of walls reveals that similar trends, as discussed above, exist regardless of wall material properties and collar joint thicknesses.

## DISCUSSION AND CONCLUSIONS

The development of the longitudinal model has been accomplished in two major phases. The first phase is concerned with the conceptual behavior of a composite element to be used in the longitudinal model of a composite masonry wall. The second phase deals with an investigation of the two limiting cases of a composite wall behavior analyzed by a transverse model and its comparison with the results of the longitudinal model. The understanding of the composite wall behavior gained from Phases 1 and 2 is utilized to qualify the use of the longitudinal

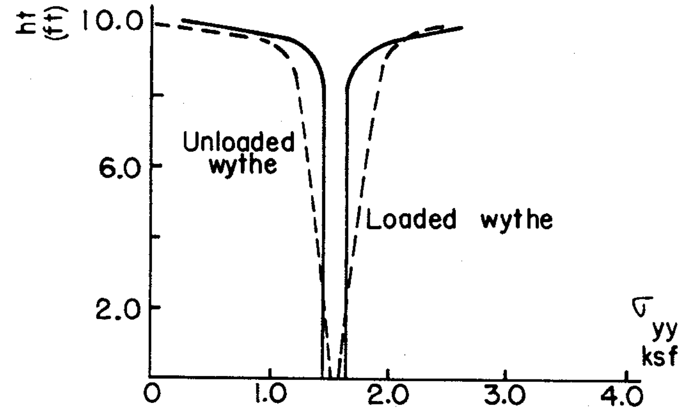



(a) Vertical normal stresses in each wythe

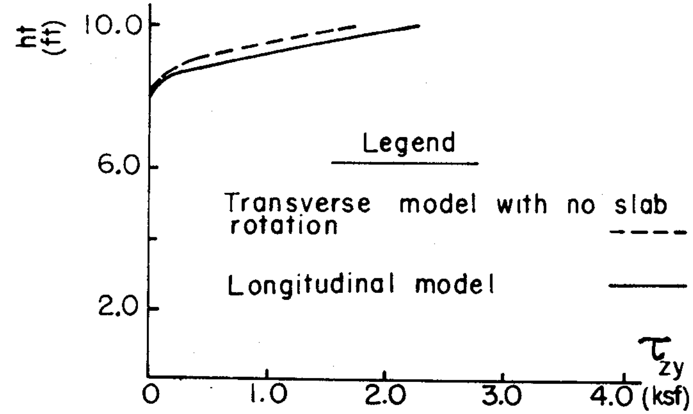



(b) Collar joint shearing stresses

Fig. 16 Stresses for Hollow Block-Brick Composite Wall With 2 Inch Collar Joint

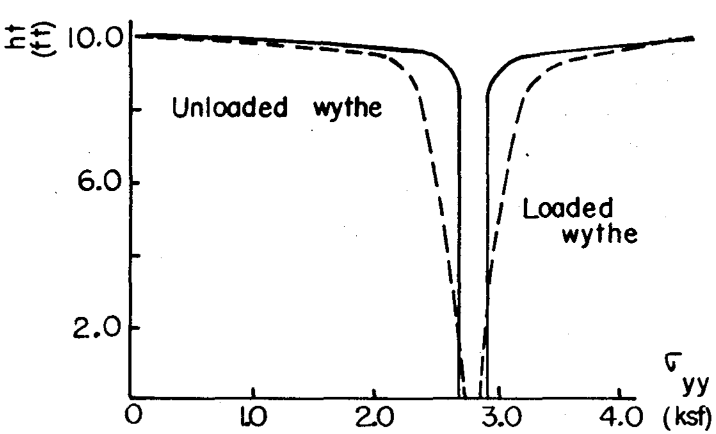



(a) Vertical normal stresses in each wythe

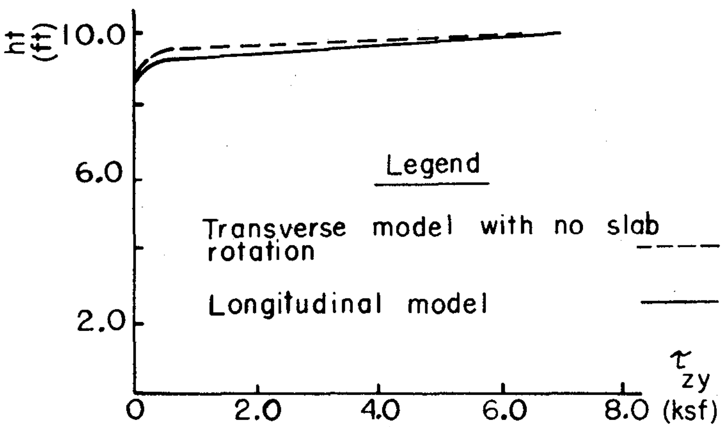



(b) Collar joint shearing stresses

Fig. 17 Stresses for Solid Brick-Brick Composite Wall With 2 Inch Collar Joint

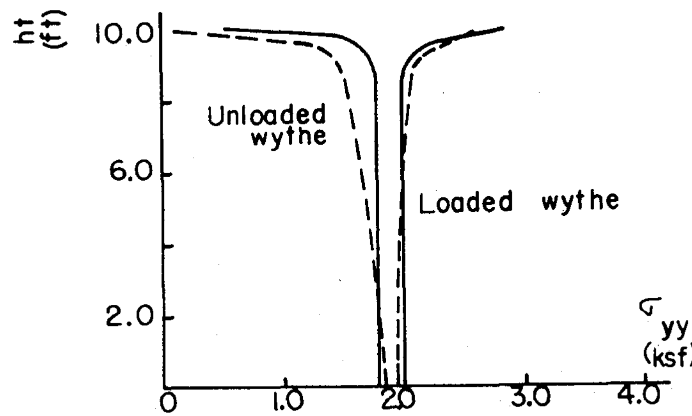



(a) Vertical normal stresses in each wythe

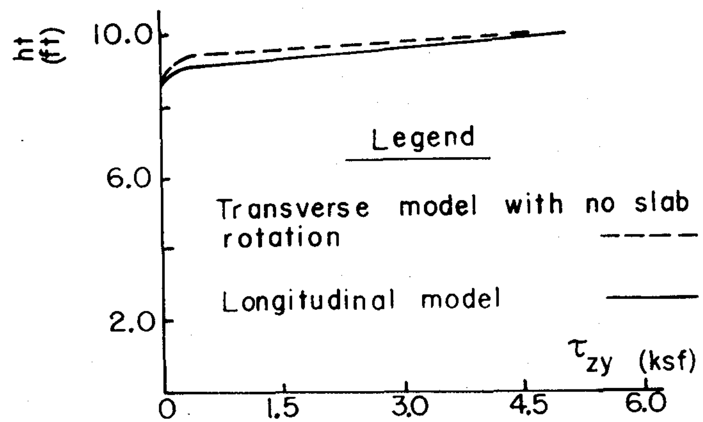



(b) Collar joint shearing stresses

Fig. 18 Stresses for Hollow Block-Brick Composite Wall With 3/8 Inch Collar Joint




(a) Vertical normal stresses in each wythe




(b) Collar joint shearing stresses

Fig. 19 Stresses for Solid Block-Brick Composite Wall With 3/8 Inch Collar Joint model for actual wall conditions as discussed in the following sections.

## Validity of the Longitudinal Model

In Phase 1 of the development, certain phenomena, such as differential wythe deformation and transverse horizontal displacements were observed in the transverse model that could not be included in the longitudinal model. In spite of these shortcomings in the longitudinal model certain realistic wall and load conditions were recognized at the conclusion of Phase 2, to which the longitudinal model could be applied successfully to obtain acceptable results for collar joint shearing stresses. The longitudinal model predicts slightly conservative results for collar joint shearing stresses when compared to those obtained by the use of the transverse model in which the rotation of the wall at the slab end is inhibited.

The corresponding curves for vertical normal stresses in both wythes from the longitudinal and transverse models, on the other hand, are not as comparable. Differences of 20 to 40 percent exist with the longitudinal model predicting a faster load transfer. However, these discrepencies in the estimation of normal stresses are considered unimportant as the primary mode of failure in composite masonry walls is anticipated to be due to excessive shearing stresses at the collar joint.

## Shortcomings of the Longitudinal Model

Several phenomena of a composite wall behavior are recognized in the transverse model that pertain to out-of-plane displacements which cannot be included in the longitudinal model. These phenomena are the effects due to bending, Poisson's ratio and transverse wythe deformation. As mentioned above, it is only to the limiting case of no slab rotation in the transverse model that the results from the longitudinal model compare reasonably well. Any deviation from this limit toward the case of extreme rotation forces the longitudinal model to be less accurate.

The reasons for the inadequacy of the longitudinal model in predicting collar joint shearing stresses are several. First, the longitudinal model is a much stiffer model since each wythe is represented by a single layer of plane stress elements. Consequently, no transverse wythe deformations are possible, and the load is transferred quickly from the loaded to the unloaded wythe. Secondly, the horizontal displacements due to Poisson's effects and bending are not included in this model and, therefore, collar joint shearing stresses are calculated from vertical displacements only. As horizontal displacements lead to shearing effects that are opposite to those due to vertical displacements, the absence of horizontal displacements in the longitudinal model results in a more conservative prediction of shearing stresses in the collar joint.

## Suggestions for Future Research

The composite element developed in this study and used in the longitudinal model has given results that are comparable to those obtained by the use of the transverse model in some loading conditions. However, much work still needs to be done before the longitudinal model can be considered sophisticated enough to predict the correct behavior of composite masonry walls.

Before any improvements in the composite element can be suggested, it should be tested further. The results obtained by its use in the longitudinal model should be compared against those from the transverse model in which the slab load is assumed to have a triangular distribution across the block wythe with the maximum acting on the inside edge. This loading condition represents an intermediate case between the Limiting Cases I and II discussed in Phase 2 of this investigation, and may be more representative of the behavior of walls in upper stories. In addition, some transverse sections in which floor slabs are also incorporated in the finite element model should be analyzed to assess the contribution of slab stiffness to the overall behavior.

If a uniform slab load distribution is assumed on the loaded wythe, the load acting on the portion of the wall that is away from the collar joint must travel some distance downwards before its effect on the collar joint is perceived. This actual behavior is automatically incorporated in the transverse model. On the other hand, the longitudinal model is incapable of representing this condition as the slab load is applied as a line load on the wythe which is represented by a single layer of plane stress elements. The above mentioned actual physical condition may be approximated in the longitudinal model by distributing the slab load over some vertical distance in the loaded wythe. This procedure may need some trial and error before the most suitable distribution is attained.

In order that the results from the longitudinal model could be compared with those of the transverse model, only vertical loads have been applied in the examples shown in this study. However, the longitudinal model can also be utilized to predict the load transfer phenomenon if the loads were applied horizontally in the plane of the wall. As the wall under this loading condition is less stiff, it is anticipated that the longitudinal model will yield more accurate results. Since the transverse model cannot be used to analyze this loading condition, three-dimensional analyses must be performed to verify the longitudinal model.

After the longitudinal model composed of the composite elements developed in this study has been tested for the conditions described in previous paragraphs, the capabilities of the composite element should be extended. These should include the incorporation of thermal strains (10), collar joint reinforcement (2,12,13), inelastic behavior (15), and fracture and failure criteria (5).

ACKNOWLEDGEMENTS

The authors are grateful to Dr. Russell H. Brown, Associate Professor of Civil Engineering, Clemson University, for his interest in this problem, and for giving many hours of his valuable time teaching us the basic principles of composite masonry. Calculations were carried out at the Clemson University Computer Center, whose cooperation is gratefully acknowledged.

APPENDIX I. - REFERENCES

1. "1979 Masonry Reinforcing Guide." AA Wire Products Co. 1978.

2. Alphonso, B. and R. H. Brown. "Shear Friction Design in Reinforced Masonry." Proceedings of the Symposium on Behavior of Building Systems and Building Components, Vanderbilt University, Nashville, Tennessee. March 8-9, 1979.

3. "Building Code Requirements for Concrete Masonry Structures." ACI 531-79. American Concrete Institute. September, 1979.

4. American Standard Building Code Requirements for Masonry. U.S. Department of Commerce, National Bureau of Standards, Miscellaneous Publication 211. July 15, 1954.

5. Arya, S. K. and G. A. Hegemier. "On Non-linear Response Predictions of Concrete Masonry Assemblies." Proceedings, North American Masonry Conference, University of Colorado, Boulder, Colorado. Paper No. 19. August, 1978.

6. "Recommended Practice for Engineered Brick Masonry." Brick Institute of America. 1969.

7. Fattal, S. G. and L. E. Cattaneo. "Structural Performance of Masonry Walls Under Compression and Flexure." NBS Building Sci. Ser. 73. Washington, D.C: National Bureau of Standards. December 1970.

8. Fattall, S. G. and L. E. Cattaneo. "Evaluation of Structural Properties of Masonry in Existing Buildings." NBS Building Sci. Ser. 62. Washington, D.C.: National Bureau of Standards. March 1977.

9. Grimm, C. T. "Metal Ties and Anchors for Brick Walls." Journal of the Structural Devision, A.S.C.E. 102: 839-858. April 1976.

10. Grimm, C. T. and D. W. Fowler. "Differential Movement in Composite Load-Bearing Masonry Walls." Journal of the Structural Division, A.S.C.E. 105:1277-1297. July 1979.

11. Hegemier, G. A., Nunn, R. O. and S. K. Arya. "Behavior of Concrete Masonry Under Biaxial Stress." Proceedings, North American Masonry Conference, University of Colorado, Boulder, Colorado. Paper No.1. August 1978.

12. Hegemier, G. A. et al. "On the Behavior of Joints in Concrete Masonry." Proceedings, North American Masonry Conference, University of Colorado, Boulder, Colorado. Paper No. 4. August 1978.

13. Hegemier, G. A. et al. "Prism Tests for the Compressive Strength of Composite Masonry." Proceedings, North North American Masonry Conference, University of Colorado, Boulder, Colorado. Paper No. 18. August, 1978.

14. Isakson, G. and A. Levy. "Finite Element Analysis of Interlaminar Shear in Fibrous Composites." Journal of Composite Materials. 5:273-276. April 1971.

15. Levy, A., Armen H. Jr., and J. Whiteside. "Elastic and Plastic Interlaminar Shear Deformation in Laminated Composites Under Generalized Plane Stress": AFFDL-TR-71-160, Dec. 1973. Proceedings, Third Conference on Matrix Methods in Structural Mechanics, Wright-Patterson AFB, Ohio. October 19-21, 1971.

16. "Specification for the Design and Construction of Loadbearing Concrete Masonry." National Concrete Masonry Association. 1970.

17. Page, A. W. "Finite Element Model for Masonry." Journal of Structural Division. A.S.C.E. 104:1267-1285. August 1978.

18. Redmond, T. B., and D. C. Patterson. "Compressive Strength of Composite Brick and Concrete Masonry Walls." Meeting Preprint 2515, A.S.C.E. National Structural Engineering Convention, New Orleans, LA. April 14-18, 1975.

19. Yokel, F. Y., Mathey, R. G., and R. D. Dickers. "Compressive Strength of Slender Concrete Masonry Walls." NBS Building Sci. Ser. 33. Washington, D.C.: National Bureau of Standards. December 1970.

20. Yokel, F. Y., Mathey, R. G., and R. D. Dickers. "Strength of Masonry Walls Under Compressive and Transverse Loads." NBS Building Sci. Ser. 34. Washington, D.C.: National Bureau of Standards. March 1971.

21. Yokel, F. Y. and S. G. Fattal. "Failure Hypothesis of Masonry Shear Walls." Journal of Structural Division, A.S.C.E. , 102:515-532. 1976.

22. Young, David T. "A Two-Dimensional Composite Masonry Finite Element Model for Predicting Inter-laminar Shearing Stresses in the Collar Joint." A Thesis presented to Clemson University in partial fulfillment of the requirements for the degree of Master of Science in Civil Engineering, December 1979.

23. Zienkiewicz, O. C. The Finite Element Method in Engineering Science. McGraw-Hill, 2nd Edition. London. 1971.

Current Research on Differential Movement Between Structural Frames and Masonry Panel Walls Conducted at the University of Texas at Austin

By Clayford T. Grimm, P.E.

ABSTRACT: Inadequate provision for differential movement between structural frames and masonry panel walls is a world wide problem and accounts for 28% of all masonry problems investigated by the author in recent years. Analysis of such problems involves consideration of the net difference between frame contraction and wall expansion, induced by moisture and thermal or freezing expansion, creep, shrinkage, elastic deformation, corrosion induced strain, eccentricities due to shelf angle deformation, and longitudinal frame displacement. A mathematical model which includes these variables is being verified by several research projects under direction by ten principal investigators.

# Current Research on Differential Movement Between Structural Frames and Masonry Panel Walls Conducted at The University of Texas at Austin

By Clayford T. Grimm, P.E.*

Expansion of masonry walls restrained by contracting structural frames has caused much damage. Over the past few years the author has investigated 22 masonry failures in 13 states and Canada, which were caused by inadequate provision for differential movement. Such failures were 28% of all masonry failures investigated in that period. The problem is recognized the world over, being described in the masonry literature of the U.S.A., Great Britain, Australia, Canada, South Africa, and Russia among others.

Analysis of such problems involves consideration of: 1. Contraction of the frame due to elastic deformation, and for concrete frames creep and shrinkage; 2. expansion of the masonry due to temperature rise or freezing and moisture expansion of brick masonry; 3. contraction of masonry due to mortar shrinkage; 4. relaxation of the masonry due to creep and shrinkage; 5. Expansion of corroded shelf angles; 6. deflection of masonry due to temperature gradient and shelf angle deformation, and 7. longitudinal frame deflection.

A mathematical model for preparing probabilistic estimates of the stress in brick masonry panel walls restrained by steel or concrete structural frames has been prepared by the author based on extensive literature search. The model requires 154 inputs, ie 28 constants and three estimates for each of 42 variables. For each of these a low, most likely, and high estimate is made. A beta distribution may be used to compute probability of the stress in the masonry being at any stated level. Alternatively a Monte Carlo approach is used.

The model is being verified by research now underway at The University of Texas at Austin. Funding for the several related projects is now at about the $100,000 level and will probably be increased. Funding is provided by the author acting on behalf of several clients. The work is being directed by several principal investigators as follows:

Dr. John E. Breen is directing research on masonry creep and shrinkage, including the physical properties of the masonry materials used in that research, ie, water absorption of the several types, moisture expansion and compressive strength of masonry units; mortar shrinkage and compressive strength; masonry prism compressive strength, elasticity, and freezing and thermal expansion; and water permeance of walls.

---

*Consulting architectural engineer and lecturer in architectural engineering, University of Texas at Austin, Austin, Texas.

Dr. Jose M. Roesset and Dr. C. Philip Johnson are directing research on crack propogation in masonry and finite element analysis of flexural stresses in reinforced and stiffened masonry plates.

Dr. Jerold W. Jones is directing research on temperature gradients through masonry sections.

Dr. Joseph A. Yura is directing research on shelf angle deformation.

Dr. John E. Breen is also directing research on buckling of masonry due to differential movement.

Mr. Tariq Cheema, a doctoral canidate, is programing the mathematical model for masonry stress induced by differential movement.

Other masonry research projects now underway at The University of Texas at Austin, not directly related to differential movement problems, include the following:

Dr. Richard L. Tucker and Dr. Richard E. Klinger are directing research of the relationship between the flexural strengths of masonry prisms and walls.

Dr. Norman K. Wagner is directing a literature search of the effects of acid rain on masonry and related materials.

Mr. Tariq Cheema is programing the author's mathematical model for the design of nonbearing panel walls. The program determins required wall thickness and mortar type for 10 types of brick and concrete masonry walls with any of 10 edge restraint conditions. Optional methods of analysis include elastic plates, elastic strips and plastic (yield line theory) design. Transformed sections are used for composite walls.

Dr. John Breen is directing an ongoing program for determining extent of facial cracks between brick and mortar and solidity of mortar joints in specimens of masonry removed from problem buildings in many locations in the United States and Canada.

The work described here will be reported in the literature in due course, if the principal investigators elect to do so.

In summary I am pleased to report that to my best knowledge, understanding and belief The University of Texas at Austin has done more research on masonry over a longer period of time than any educational institution in this hemisphere. We fully intend to continue that tradition.

Fig 1. Examination of masonry specimen from existing building to determine extent of facial cracks between brick and mortar, which can be correlated with water permeance.


Fig 2. Masonry prisms from existing buildings are split open at bed and head joints to determine percent solidity of mortar joints.

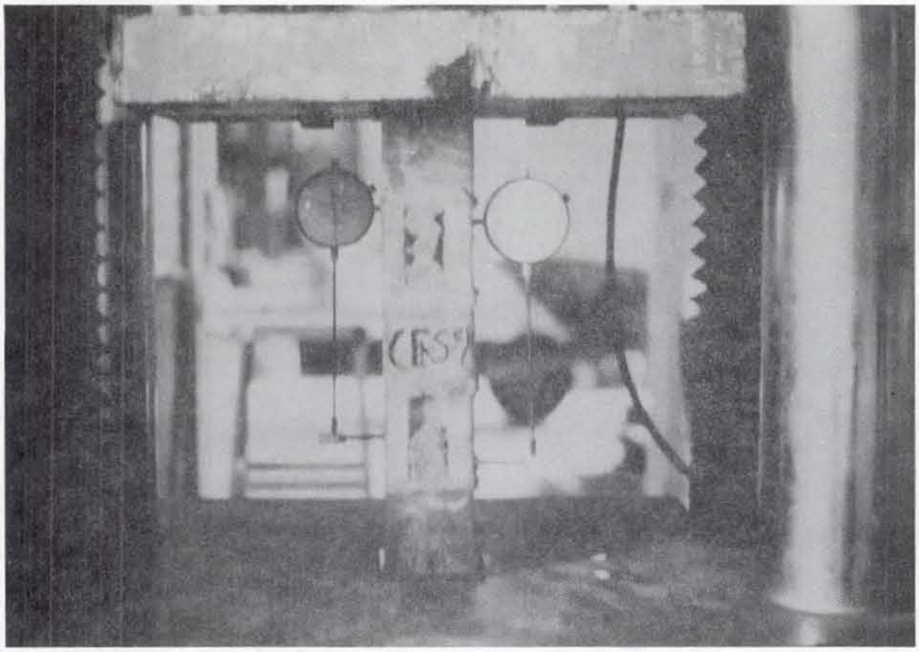


Fig 3. Test for compressive strength and modulus of elasticity in compression on masonry prism from existing building.

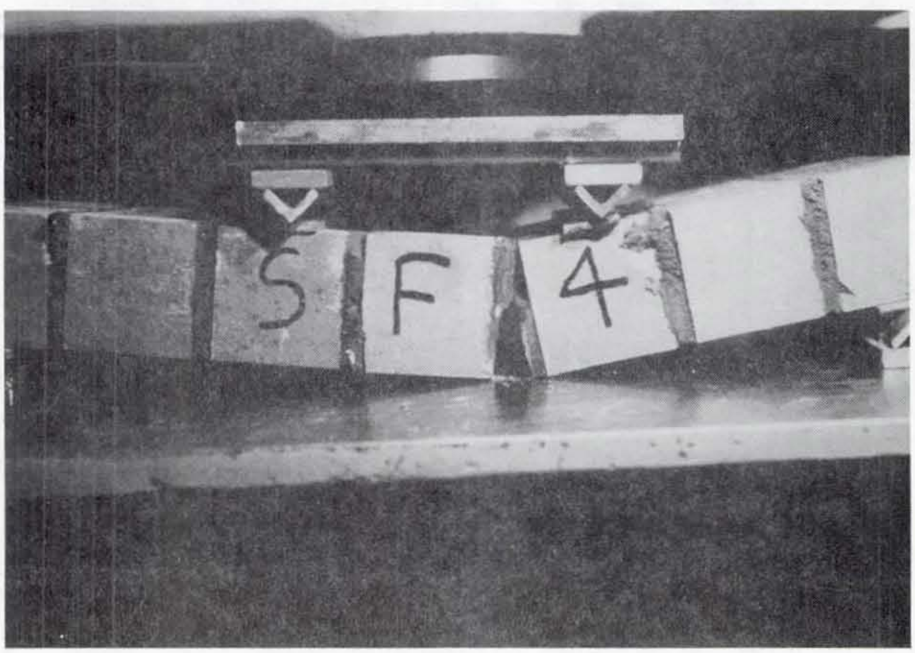



Fig 4. Test for modulus of rupture on prism taken from existing building.

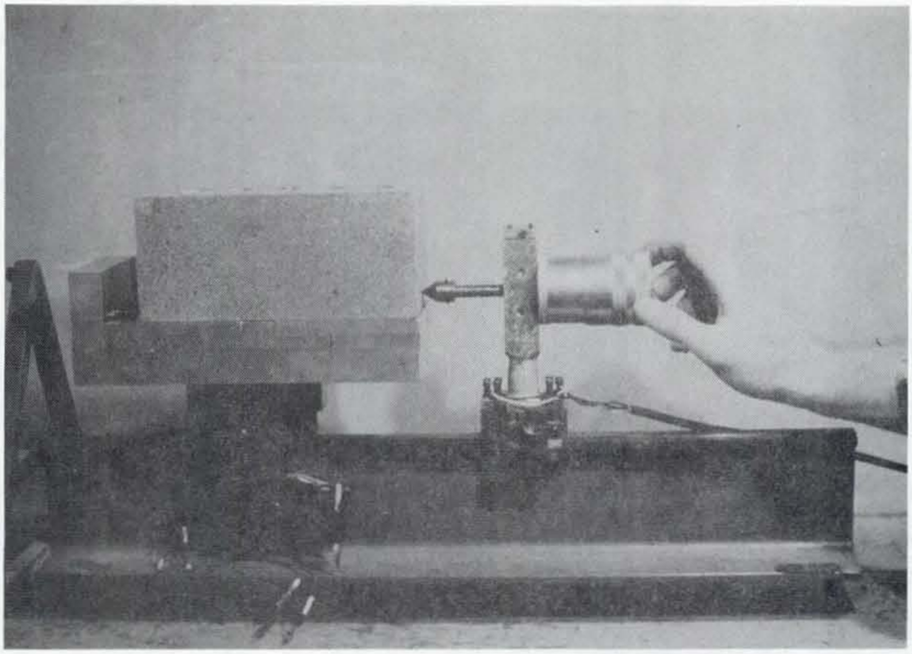


Fig 5. Brick dimension measurement in preparation for measurement of moisture expansion and/or thermal expansion characteristics of unit taken from existing building.

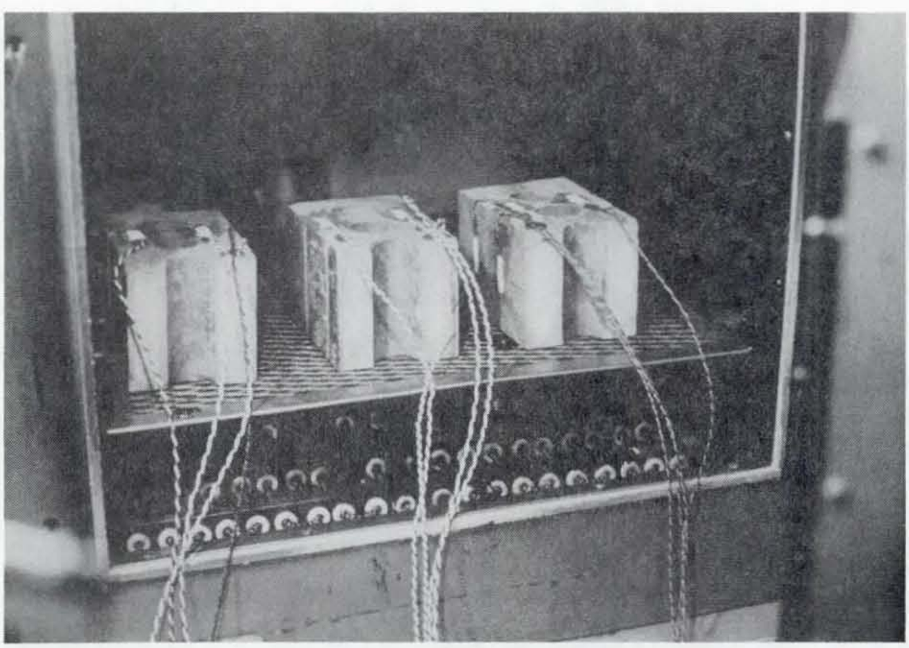


Fig 6. Instrumentation of brick in oven during test for thermal expansion characteristics.

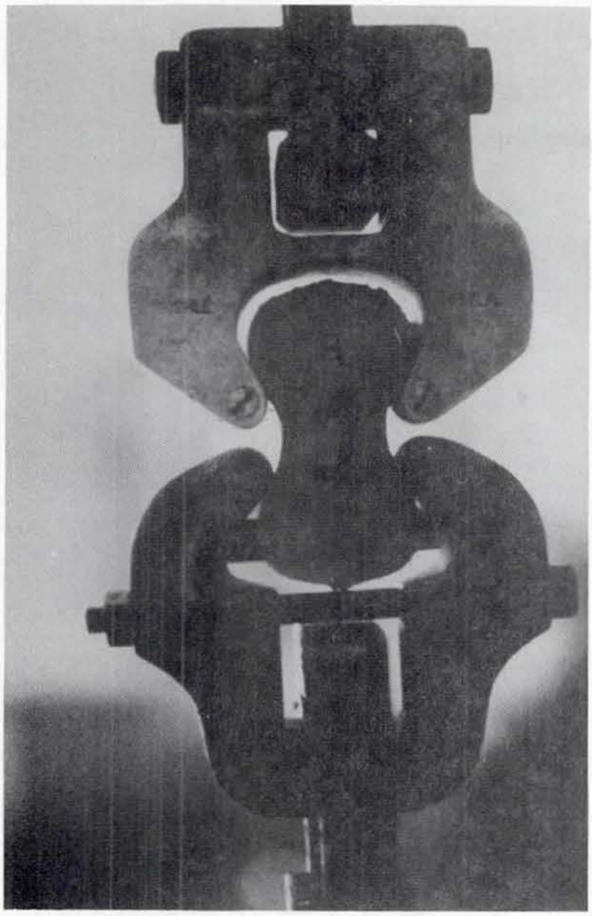


Fig 7. Mortar tensile strength test

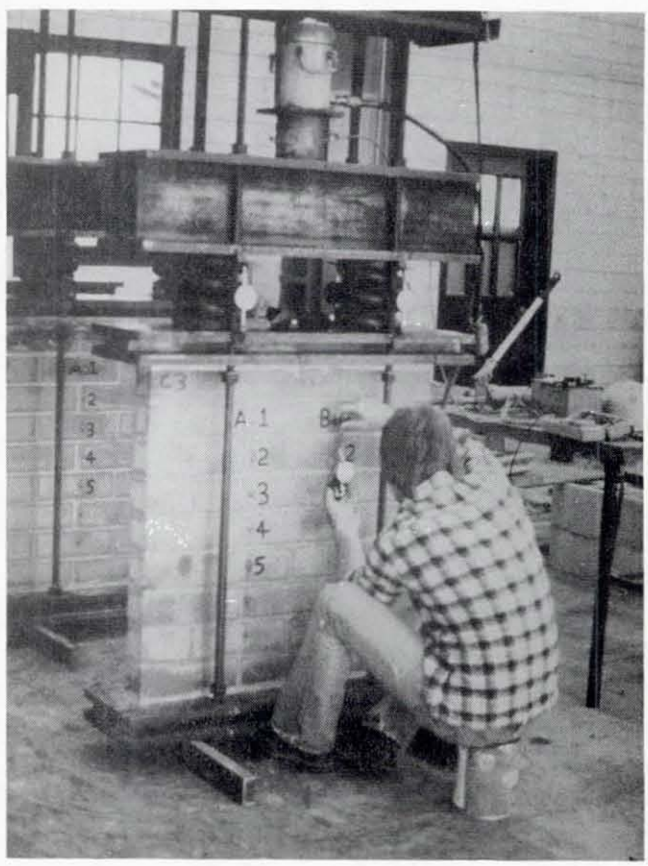



Fig 8. Brick masonry creep studies

ELIMINATION OF PROGRESSIVE COLLAPSE IN MASONRY STRUCTURES

By Russell, H. G., and Hanson, N. W.

ABSTRACT: Work is being performed to show that load bearing masonry structures can be tied together to provide structural integrity. By this means masonry structures will not progressively collapse in the event of an abnormal load. A methodology will be developed so that alternate load paths will develop. The methodology will be verified by tests on full size wall elements. Tests will also be made to investigate transfer of vertical load through the wall to floor connection. Design criteria and construction details will be developed. Design examples will be prepared.

# ELIMINATION OF PROGRESSIVE COLLAPSE IN MASONRY STRUCTURES

By Henry G. Russell and Norman W. Hanson[1]

## PROBLEM STATEMENT

In recent years, an increased awareness has evolved of the need to consider abnormal loads in design of buildings. (1 to 6)[2] Abnormal loads may lead to progressive collapse unless overall continuity and stability are maintained throughout the structure.

The importance of work described in this paper is to show that concrete masonry structures can be tied together to provide structural integrity and hence eliminate progressive collapse. The work includes tests of masonry walls subject to vertical loading to verify reinforcement requirements. Some tests are being made with the central portion of the wall omitted. Other tests are being made with the end portion of the wall omitted. Some tests are also being made to determine the transfer of load through the wall to floor joint in load bearing masonry structures. As a result of this program, it is anticipated that design criteria and construction details will be developed to prevent progressive collapse in load bearing masonry structures.

## BACKGROUND

Progressive collapse is defined as a phenomenon in which the spread of a local damage caused by some abnormal load results in the collapse of the entire building or a disproportionately large part of it. It is economically prohibitive to explicitly design load bearing masonry structures to resist all abnormal loads. This results from the absence of comprehensive information concerning the following:

1. Incidence, nature, and consequence of abnormal loadings due to phenomena not normally considered in building design.

2. Response of the structural system as a whole or its component parts when subjected to large deformation or local damage from abnormal loads.

---

[1] Respectively, Director, and Principal Structural Engineer; Structural Development Department; Construction Technology Laboratories, a Division of the Portland Cement Association; Skokie, Illinois.

[2] Numerals in parentheses refer to corresponding items in Appendix I - References.

Although it is not possible to design to resist abnormal loads, provisions should be made to ensure that the consequences of an incident are not disproportionately larger than the initiating cause. Occupant safety and property damage should be considered. Traditionally, it has been assumed that if established practice and codes were followed, the resulting building would have a satisfactory inherent degree of structural integrity. However, this assumption is only justified for those structural systems in which construction techniques result in a substantial degree of continuity.

To reduce the risk of progressive collapse in structures lacking continuity, the following solutions have been proposed:

1. Eliminate the hazards which cause local damage.

2. Design the structure for the required strength so that the hazard does not cause any local failure.

3. Allow a local failure to occur, but design the structure to bridge local damage and maintain overall structural integrity.

The first two approaches fail to consider all possible abnormal loads. Approach two is uneconomical. The third approach has received wide acceptance in Europe and in the United States because it can be made practical and economically feasible.

## OBJECTIVE

The objective of this project is to determine that general structural integrity can be achieved in load bearing concrete masonry structures. Use of horizontal bond beams and ties between bearing walls and floor systems can be used to achieve this. Design criteria for necessary ties will be developed. The bond beam which already exists in masonry structures can be tied into the surrounding elements through the use of vertical reinforcing steel dowels.

## WORK PLAN

The work plan for the project includes six separate tasks to be performed jointly by the National Concrete Masonry Association and the Construction Technology Laboratories of the Portland Cement Association. The project is sponsored by the Department of Housing and Urban Development. Work to be accomplished under each task is outlined in the following sections.

Market Survey. -- A market survey has been performed by NCMA to determine characteristics of architectural layout and structural considerations that apply to typical load bearing masonry buildings. Items addressed included size of building, slab span lengths, building heights, story heights, material and strength of masonry units, floor systems, connections, degree of reinforcement, and common construction details. Information from the market survey was used to select a prototype structure for use in subsequent tasks.

Development of Methodology. -- A rational approach to the use of alternate paths after the occurrence of abnormal loads in various parts of the structure will be developed. The task will address itself to both interior and exterior load bearing walls. Walls containing openings will also be considered. A review of specific methods and procedures to analyze overall behavior of selected load bearing masonry structures will be included. Specific methods will be evaluated to determine the degree to which the methods assure structural integrity.

Tests of Full-Scale Walls. -- In this task, tests will be performed on full-scale wall specimens to evaluate overall response to vertical loads. The tests will be used to investigate alternate methods or concepts for reinforcement and connection details. The results will verify the methodology determined in the previous task. An analytical model compatible with the experimental observations will be developed using test results.

Pullout Tests. -- The objective of this task is to determine bond development of reinforcement in various strengths and types of masonry grouts and masonry units. This work is being performed by NCMA. Pull-out tests from grouted masonry units will be made.

Vertical Load Transfer through Connections. -- Experimental testing to determine strength and interaction behavior of the horizontal floor to wall connections will be performed. The objective of these tests is to evaluate the transfer mechanism of vertical load through the connection. Axial load will be applied to selected internal connections of the floor to wall joint. In addition, external connections will be tested. The effects of moment transfer from the slab to wall are also being examined.

Development of Design Criteria. -- This final task consists of engineering analysis to quantify acceptable levels of load capacity and reserve strength. Recommended design criteria will be developed. Examples of design application and structural theory pertinent to these criteria will be developed and presented in a final report. A design methodology will be developed based on the work accomplished in the previous tasks. Design examples and a set of recommendations reflecting the results of the entire research program will be provided.

## SUMMARY

The objective of this program is to ascertain that in load bearing concrete masonry structures, general structural integrity can be achieved by the use of horizontal bond beams and ties between bearing walls and floor systems. Design criteria for these ties will be developed. Tying the structure together by means of ties may provide an alternate path for abnormal loads. Experimental tests will be performed to demonstrate the validity of the proposed design method.

## ACKNOWLEDGMENTS

The work described in this report is sponsored by the Department of Housing and Urban Development under Contract No. H2819 to the National Concrete Masonry Association. Work performed by the Construction Technology Laboratories is under a subcontract with the National Concrete Masonry Association. The description given in this report is based on the Statement of Work for the contract. The government technical representative is Ronald J. Morony. The principal investigator with the National Concrete Masonry Association is Gary D. Schoenfeld. Any opinions, findings, and conclusions given in this report are those of the authors and do not reflect the official opinions of the sponsors.

## APPENDIX I - REFERENCES

1. Burnett, E.F.P., "Abnormal Loading and Building Safety," Industrialization in Concrete Building Construction, SP-48, American Concrete Institute, Detroit, Michigan, 1975.

2. Burnett, E.F.P., "The Avoidance of Progressive Collapse: Regulatory Approaches to the Problem," NBS-BCR 75-48, National Bureau of Standards, Washington, D.C., October 1975, 173 p.

3. Fintel, M., Schultz, D.M. and Iqbal, M., "Report 2: Philosophy of Structural Response to Normal and Abnormal Loading Conditions," Design and Construction of Large Panel Concrete Structures, Office of Policy Development and Research, Department of Housing and Urban Development, Washington, D.C., August 1976.

4. Schultz, D.M. and Fintel, M., "Report 1: Loading Conditions," Design and Construction of Large Panel Concrete Structures, Office of Policy Development and Research, Department of Housing and Urban Development, Washington, D.C., April 1975.

5. Somes, N.F., Abnormal Loading on Buildings and Progressive Collapse, NTIS Com-73-11113, prepared for Office of Policy Development and Research, Department of Housing and Urban Development, Washington, D.C., May 1973.

6. Summary Report: Research Workshop on Progressive Collapse of Building Structures, (held November 18-20, 1975, the University of Texas at Austin), HUD-PDR-182, September 1976.

# TRANSFORMING RESEARCH INTO BUILDING REQUIREMENTS

by: Roland L. Sharpe

ABSTRACT: The process for transforming research results into building code requirements has become lengthy; many more groups of people are now involved than was the case a few years ago. The report presents the author's opinion on problems with the current process and how the process might be improved. An on-going project is described and work towards incorporating the results into codes is discussed as a case example.

# TRANSFORMING RESEARCH INTO BUILDING REQUIREMENTS

By Roland L. Sharpe[1]

## INTRODUCTION

Research has been performed on masonry construction throughout much of man's history. Research was mostly more or less informal, although in the past several decades the research effort has tended to become much more formalized and the results have been beneficial in many areas. Research in seismic resistance of masonry construction is fairly new -- in the last two or three decades.

Concurrent with this research effort there has been a great increase in earthquake resistant design requirements in building codes. The process for transforming research results into building code requirements has become more lengthy, and many more groups are involved than initially was the case. In many instances, considerable time is required to process the research results and have them adopted as building code requirements. The purpose of the following paper is to present the author's opinions on problems with the presently perceived process and how this process might be improved. The process is reviewed, difficulties encountered are discussed, an on-going project is described and recommendations are presented.

## PROBLEM STATEMENT

Most of the research conducted on seismic resistance of masonry is either funded by governmental agencies, by industry, or jointly by government and industry. The research is performed, the results are presented in a report, and then the results are presented to the building designers and code promulgating officials, usually with the request that certain changes or additions be made to the building code requirements. Often there is resistance to accepting the change because the research does not directly apply to the problem presented, and extrapolation or interpolation has to be made to extend the research to the problem at hand. The applicability of the results then often becomes a matter of judgment between the designers and the industry representatives presenting the results. The present paper is intended to look at this entire process and to make suggestions as to how it might be improved.

How do research needs arise? There seem to be two basic events that might occur. First, somebody has an idea to build in a more economical, safer, or efficient way. Two examples that come to mind are reinforced grouted brick masonry and reinforced grouted masonry. The other event that could occur is that something happens: a masonry structure is damaged due to stress from thermal changes, moisture change, variations in load or imposition of transient loads such as earthquake and wind, and

---

[1] Managing Director, Applied Technology Council and President, Engineering Decision Analysis Company, Inc., Palo Alto, California.

there is obviously a need that the construction should be improved to resist these conditions. The research is performed to try to improve the end product and may either be analytical or utilize experimental procedures.

The research results are needed because designers and/or building officials won't accept the proposed solution without some sort of proof because they have much of the responsibility for the end product. The building official is responsible for safeguarding public safety, property and welfare, and the designer is responsible to the owner for a safe, economical structure.

## APPROACH TO THE SOLUTION

As noted previously, most of the masonry research is sponsored by the industry or by governmental agencies. Industry and/or government groups might perform some of the research directly. Sometimes they utilize an advisory group; other times they do not. When industry does the research, there can be problems of credibility. If an appropriate advisory group is not factored in early in the project, there may be a lack of full understanding of the problem from the designer's viewpoint, and hence the results might be suspect or argued.

The other approach to research is where industry and/or government go to a university or to private consultants. Again, these researchers may or may not use advisory groups.

At the beginning a budget is established, so the research program has to be carefully thought out and organized so as to hopefully produce the results in an economical manner and within the given budget. Because of restrictions in research facilities and budget, the research experiment may not always be directly applicable to the problem. The results must be extrapolated or transformed in some manner so as to be applicable to the problem at hand.

The above approach seems to be rather straightforward, but what are some of the problems that have been involved with this approach? First, let us review who all should be involved in the process. The owner or the end user of the building is interested in a safe, economical (by economical is meant life cycle cost, including first cost and lifetime maintenance) structure. The owner is normally represented by designers (architects and engineers). Then there is the manufacturer of the materials, and closely allied with the manufacturer is the constructor or contractor, who furnishes the organization and assembles the required resources to build the structure. This includes materials and appropriate craftsmen. There is the building official who has the responsibility to the public to see that any building constructed that comes within his purview safeguards the public safety, property and welfare. And, of course, there is the researcher, who has to set up the experiment. He has the background in experimental design; he knows the physical limitations of the various types of facilities; he presumably is unbiased and independent; and he will provide a factual, straight forward research program and corresponding report setting forth a description of the research, the results obtained, and his interpretation of same.

The difficulty is to bring all of these parties together to focus on the problem and to come up with a solution considering all of the restraints such as budget, physical facilities, knowledge about the phenomena, schedule, scale factors, workmanship, variation in materials, and other factors. Further, the resulting research program must, insofar as practical, represent as-built conditions.

From a designer's viewpoint, there is a perception that the researcher and/or the funding group defines the program. They often do not obtain all the required input from the designers. They may set up an advisory group, but often such advisory groups are not fully effective for a variety of reasons. First, they are often set up after the research program has been formulated. They are not a good representative sample of the design professions and industry. Thirdly, their participatin is limited.

## DIFFICULTIES ENCOUNTERED

In transforming research needs into building code requirements, it is well to keep in mind some of the procedures that have to be followed to revise a building code. For seismic design requirements in the Uniform Building Code, the results, together with recommended provisions, must go first to the ICBO Code Changes Committee and then to the Seismology Subcommittee, which must review all provisions related to the design of structures to resist earthquakes (see Figure 1).

For seismic provisions, ICBO usually refers all proposed code changes to the Structural Engineers' Association of California. Such code provision changes are then assigned to the State Seismology Committee (see Figure 2). On some occasions, the SEAOC State Code Committee is also involved. The SEAOC Seismology Committee reviews the proposed code change in detail, often relies partly on input from local committees and/or task groups, and often sends a representative to ICBO committee meetings and other hearings. The SEAOC Seismology Committee may act independently of the Board of Directors. For changes in other geographical areas, other structural engineering associations may become involved. It is evident there are a lot of people involved and if research is to be done with the aim of developing new code requirements, a means of communication with the appropriate groups should be set up early in the program to ensure that their ideas and judgments are involved at an early stage. If this is done, then the code revision process should run much smoother.

## RESULTS TO DATE

The results to date appear to be fair to good. It does not appear for the amount of research funds expended that the end results are commensurate with the effort and funds expended. Why is this? Some of the reasons were stated previously: inadequate design of the research program, and inappropriate use of advisory groups or inappropriate use of their input. Another problem is that the research programs do not seem to be fully coordinated. Often, a research program is set up to solve a specific problem and does not consider other factors that might be interdependent with the problem at hand. One result is there are conflicts in the present code requirements for masonry and there are large variations in the code requirements for masonry throughout the United States. Some of these variations are warranted because there are different conditions in different parts of the country. However, there are others that appear could be solved in similar ways.

Difficulties are encountered in getting the research results accepted by the designers, and if the designers do not accept them it is usually very difficult to get the building official to accept. The question then arises, can this situation be improved? There has been a lot of good work done, but it seems there are a number of improvements that could be made. A project that has been ongoing for the past four years, wherein research is being done at the University of California at Berkeley (UCB) and an advisory committee was assembled by the Applied Technology Council (ATC) under contract with the Department of Housing and Urban Development (HUD), seems to have solved some of the problems that were outlined above.

The project involves the development of design guidelines for earthquake resistance of single-family masonry dwellings in Seismic Zone 2. HUD contracted with UCB to make certain tests including shaking table tests of model houses, and a variety of connection tests with the proviso that the work be done in close cooperation with an advisory group assembled by ATC. UCB also had advice from masonry industry representatives. ATC assembled an advisory panel composed of a representative of SEAOC, a structural engineer from Phoenix, a structural engineer from Memphis, a homebuilders' representative from Phoenix, and a subcontractor structural engineering firm from Phoenix. Representatives of HUD and FHA also served on the advisory panel. This advisory panel, including the author, initially met with UCB representatives, thoroughly reviewed the basic requirements for the research, what end results were expected, and how the various tests should be set up to get the maximum information, which hopefully could then be directly applied to the design guidelines. There was a good spirit of cooperation among all the parties involved. UCB developed the initial experimental design, after consultation with the advisory panel. The advisory panel then reviewed the program in detail, made certain recommendations, and after discussion most of the suggestions were accepted. A number of connections tests were developed. Most of these were based on recommendations of the advisory panel. The researchers recommended how the samples should be put together, how many tests should be made, and how the loads could be applied. Nearly concurrently, shaking table tests were run on the first model house. The advisory panel was present at some of the tests. After the results of the tests were reduced and evaluated, they were discussed in a joint meeting and the second shaking table test was planned. This process continued through four different shaking table tests. Each test was designed with full cooperation between the advisory panel and the UCB researchers.

It was realized because of the physical limitations of the shaking table that all factors could not be tested. For example, the house had to be limited to 16 ft. square. The walls of the house could not be connected at the corners, because the walls had to be constructed separately and then moved onto the shaking table; to construct them on the skaking table would hold up use of the facility for several weeks when it was badly needed for other research. There was considerable discussion on the effects of scale, workmanship, size of house, deflection of the roof diaphragm, foundation compliance, moisture conditions during construction, and many other factors. However, it is the author's opinion that the results produced from these tests, when fully evaluated by the advisory panel and the researchers, will provide meaningful input into building code requirements. The ATC subcontractor has the responsibility to take the results of the research and recommendations of UCB and to develop design guidelines for single-family masonry dwellings in Seismic Zone 2.

One might comment that the building code official was not involved. As these guidelines are to be used by HUD, the HUD representatives served this function for this project. For a project where the results would go into a building code, building code officials and others could be readily instituted in the advisory panel.

What are the advantages of the above? Many of the problems and arguments that could arise after the research was completed were raised and a solution arrived at prior to the development of the research program, or prior to the development of a new test sample. Thus, there was maximum efficiency made of the research dollar because there was concurrence on the design of each of the experiments.

## RECOMMENDATIONS

The above paper has discussed some of the problems and possible solutions to making the process of transforming research results into building code requirements more efficient. When a research problem arises and a research program is to be set up, a very clear means and procedure must be set up to get the required input from all those who are affected. This includes the designer professional (architects and engineers), building officials and building code promulgating officials, manufacturers and/or constructors (masonry industry), researchers, and the funding group. A means must be developed whereby the input from each of these is obtained and considered in a timely manner, as appropriate.

The use of an advisory committee is highly recommended. It should be pointed out that most of the members of such an advisory group or panel will be practicing design professionals. They are in business to make a living. Therefore, adequate provision should be made in the research program budget to pay the expenses and some reimbursement for time spent to each of the members of the panel. Such reimbursement should consider not only attendance at meetings, but make some provision for study and evaluation of the data prior to each meeting, so that the meeting will have meaningful results. Too often, advisory panels are set up with the idea that the expenses only would be paid for the professional with no compensation for his time. The design professional makes his living by providing consulting services, and most are from small offices; therefore, consideration should be given to reimbursement for their time.

Lastly, I would like to recommend in selecting the advisory panel, that a representative group of knowledgeable, experienced designer professionals be involved. Too often because of budget requirements, there is a feeling that one or two people on the advisory panel is adequate. Seismic design is still an art; it is not a science, and as such requires judgment of professionals. Therefore, I strongly recommend that the advisory panel be representative of current practice and have sufficient members so varying viewpoints are heard and considered in the development of the research program.

I hope the above comments have been of value. It should be noted the Applied Technology Council follows the above procedures, and for any advisory panel retained, there is provision made for reimbursement of expenses and for time of the individual involved. As a result, we find that we can meet deadlines; we can exert pressure to get comments and input from the advisory panel members; and we have been quite successful in the past seven years in doing this. Thank you.

## FLOW CHART FOR UBC CHANGE

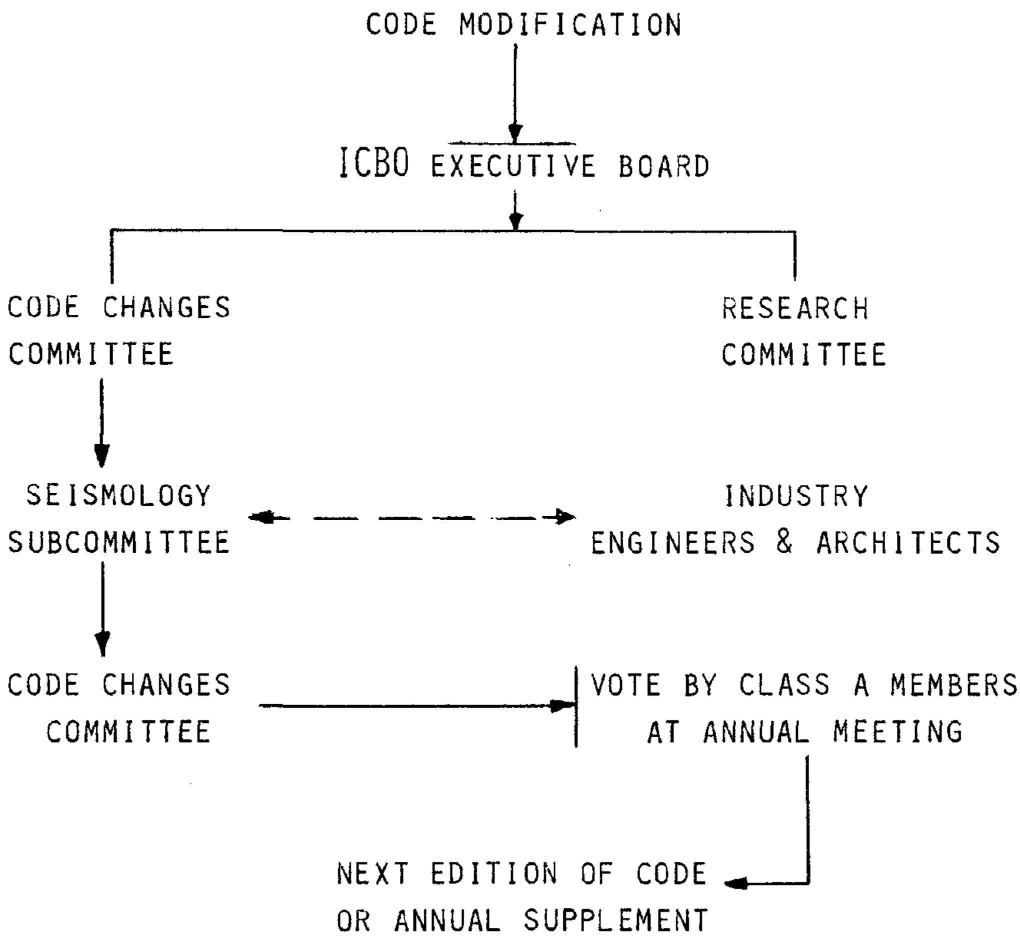



FIGURE 1

FLOW CHART - SEAOC PROCEDURE FOR SEISMIC CODE CHANGES

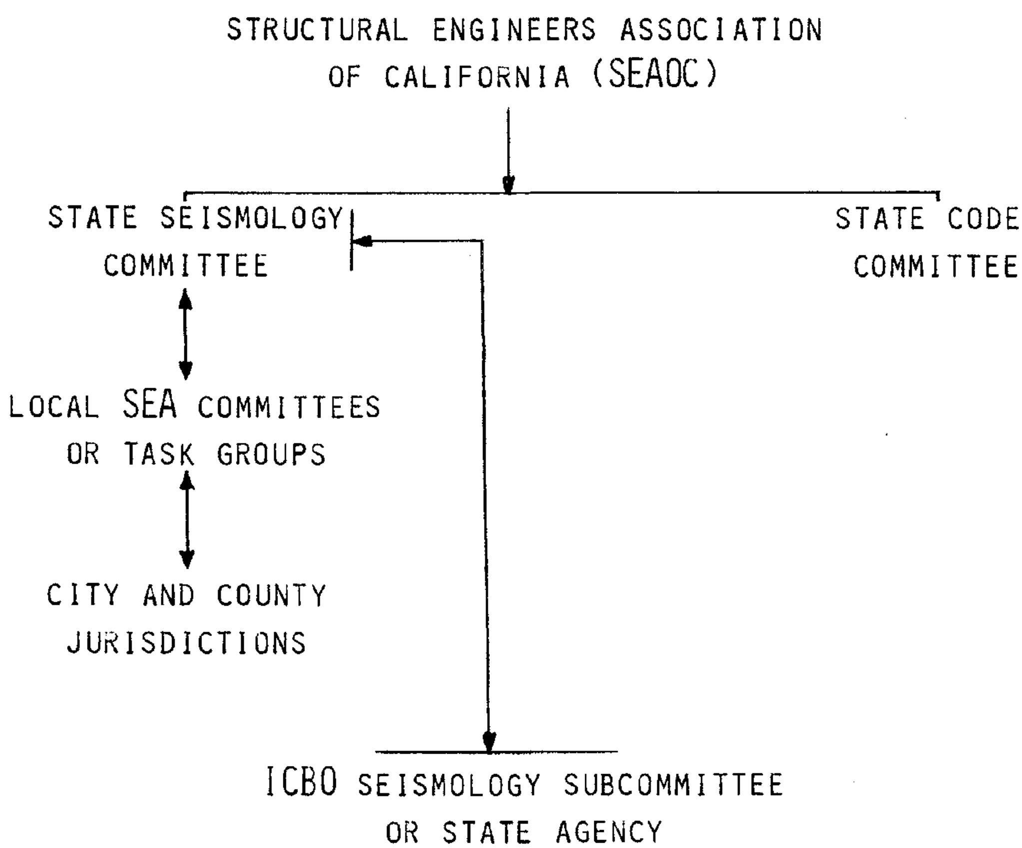



FIGURE 2

# BUILDING STANDARDS DEVELOPMENT FOR STRENGTHENING EXISTING UNREINFORCED MASONRY BUILDINGS IN SEISMIC AREAS

By Ben L. Schmid
Consulting Structural Engineer, Pasadena, California

ABSTRACT: The danger to public safety through the continued occupancy of unreinforced masonry bearing wall buildings constructed with weak mortar and without coherent stress paths from the roof to the ground in seismic areas has been of continued great concern to Structural Engineers, Engineering Geologists, Seismologists and Building Officials. A variety of ordinances and standards are described that have been implemented or proposed since 1933 to reduce the hazards inherent in such unreinforced masonry structures.

# BUILDING STANDARDS DEVELOPMENT FOR STRENGTHENING EXISTING UNREINFORCED MASONRY BUILDINGS IN SEISMIC AREAS

By Ben L. Schmid[1]

## INTRODUCTION

The danger to public safety through the continued occupancy of unreinforced masonry bearing wall buildings, constructed with weak mortar and without coherent stress paths from the roof to the ground, in seismic areas has been of continuing great concern to Structural Engineers, Engineering Geologists, Seismologists and Building Officials. This concern was evidenced in California by the extremely poor performance and collapse of many unreinforced masonry structures during the 1933 Long Beach earthquake, the 1952 Tehachapi, the 1968 Santa Rosa and the 1971 San Fernando earthquake. A variety of ordinances have since been implemented, or proposed, to reduce the hazards inherent in such unreinforced masonry structures.

This report is intended to describe some of the ordinances addressed to strengthening this category of structures.

## INITIAL SEISMIC CODES FOLLOWING THE 1933 LONG BEACH EARTHQUAKE

It was evident by the damage caused by the Long Beach earthquake in 1933 that the existing construction practices involving unreinforced masonry buildings required drastic changes. In Long Beach, approximately one-half of the City's estimated 1700 unreinforced masonry buildings received major damage which ranged from ruptured masonry walls requiring extensive repairs to damage that was so severe that repairs were impractical. More than half of the unreinforced masonry buildings in the nearby City of Compton were either demolished by the earthquake, or were so badly damaged that they had to be torn down. Local jurisdictions quickly amended their building codes to require new buildings to be earthquake resistant. The Field Act, which established earthquake design standards for public schools, was adopted by the State of California in April, 1933. The Riley Act, which initially required a minimum lateral design force of 2% G times Dead Load plus Live Load for all buildings in the State except public schools, was passed in 1933. The old lime mortar specifications were changed dramatically by significantly increasing the required amount of cement in the mortar of masonry construction and requiring the use of clean, graded sand. Reinforcing steel was specified for masonry bearing and shear walls.

[1] Consulting Structural Engineer, Pasadena, California.

Even though Building Codes developed design forces and allowable stresses, following 1933, for new construction, little was done to provide criteria for strengthening existing lime mortar buildings. The schools in California were the exception. The Office of the State Architect, through the Field Act, permitted strengthening of existing unreinforced masonry schools by the combination of gunite, cutting closely spaced vertical chases in the masonry walls to form internal concrete pilasters and providing new materials and anchorages to permit the building to resist design lateral forces in the range of 10% G for Dead Load and Live Load to 13% G for Dead Load. A number of schools throughout this state were so strengthened between late in 1933 and the current period. All unreinforced masonry public school buildings in California have now been strengthened, vacated or demolished. Some of the strengthened buildings have been subjected to subsequent earthquakes and have performed very well.

## PARAPET ORDINANCES

One of the earliest steps to minimize potential danger to occupants and pedestrians was the retroactive Parapet Ordinance adopted by the City of Los Angeles in 1949. It was the start of a successful 20 year parapet correction program whereby the seismically unstable parapets of approximately 20,000 pre-1934 buildings were strengthened or removed. The Parapet Ordinance required that the parapet be removed to within one foot of the roof or that suitable braces be attached. The walls were required to be attached to the roof framing members with rod anchors or other suitable ties to resist a minimum force of 200 pounds per foot applied normal to the wall at the roof level. A continuous concrete cap beam with at least a single one-half inch diameter reinforcing bar was typically cast on top of the masonry wall. Cast-stone and other ornamental appendages applied to the masonry walls, as well as railings, were required to be suitably anchored against lateral forces. Other Southern California communities have completed similar programs. The value of these programs were well demonstrated during the San Fernando earthquake. Instances were cited where anchored walls remained in place, while unanchored walls on the same building failed and fell to the ground. The installation of roof anchors tended to hold the building together so that complete collapse was prevented. The City of San Francisco began its implementation in 1975 of a parapet ordinance that was passed in 1969. Over 250 buildings have been corrected from a list of 1500 building owners who have been notified to date. It is estimated that 8,000 buildings will require parapet review.

## COMPREHENSIVE ORDINANCES FOR STRUCTURAL REHABILITATION OF UNREINFORCED MASONRY STRUCTURES

In Southern California, where ground shaking greater than 30% G accompanied by large ground displacements have been predicted by seismologists, the next step of strengthening is being considered. The City of

Long Beach was the first to add a subdivision to its Building Code (2) related to earthquake hazard regulations for structural rehabilitation of existing structures. The Section provided a table of lateral design force based upon number of occupants, type of occupancy, designated continued life of the building, soil factor, seismic zone and type of structure. The owner was required, through his Architect and/or Engineer, to show that the building would be made to resist the designated forces using materials and values provided in the Uniform Building Code. A number of the apparently dangerous buildings were cited and were brought into compliance or were demolished.

The City of Santa Rosa passed Resolution 9820 in 1971, following the 1969 Santa Rosa Earthquake. The Resolution has since been updated to the level of the 1979 Uniform Building Code. Design standards permit 100% increase in diaphragm stresses, 50% increase in connections and bolt values and 6 psi in masonry shear. No tension in masonry is permitted for flexural stress. Unreinforced masonry shear walls are required to be stiffened laterally by adding vertical posts or columns bolted or otherwise attached to the wall so that the wall acts as a veneer. The strengthening plans are required to be done by a Structural Engineer or Civil Engineer with a structural speciality. The standards are directed to all buildings built prior to 1958. Approximately 80% of the unreinforced buildings in the downtown area are either in the process of being strengthened, the work has been completed or the buildings were demolished.

The ATC Report (5) "Tentative Provisions for the Development of Seismic Regulations for Buildings," Chapters 13 and 14 describe the abatement of seismic hazards and guidelines for repair and strengthening. The occupancy potential and category of building type are included as a measure of the lateral design force. The length of time in years for abatement of the potential hazard is also equated.

The Uniform Building Code (7) does not presently provide code provisions for partial strengthening of hazardous buildings except to require that alterations valued between 25% to 50% of the building value shall use materials and design forces for the new work in compliance with new work requirements of the code. If the alterations exceed 50% of the value of the building, the entire building is required to be brought into compliance with code provisions for the new work. It is up to the individual Building Departments or Appeal Boards to lessen these requirements. Provisions for rehabilitation of Historic Buildings without conformance to all provisions of the code and for abatement of dangerous parapet walls and other appendages are covered in Section 104.

The State of California enacted legislation in 1973 that required each City and County to develop a General Plan for future planning. The Seismic Safety Element contained in the General Plan often listed the reduction of hazards due to the continued occupancy of unreinforced masonry buildings as a priority program (3). The City of Los Angeles has developed a proposed ordinance for the strengthening of unreinforced masonry bearing wall buildings (4). The ordinance classifies buildings into 4 risk categories based upon Occupancy and Use. Force levels are assigned for each risk category and vary from 18.6% G for Police, Fire and other essential facilities to a low of 10% G for warehouse and other low density occupancy buildings. The notification to owners is to be on a graduated time schedule with the owners of higher risk structures to be notified first. The implementation is meant to be spread over a 10 year period. The ordinance contains allowable stresses for existing materials not presently specified in the building code. Masonry unit shear tests of each existing wall are also specified with allowable minimum shear test values to permit the building to be strengthened within the ordinance. Walls of reasonable strength, with unsupported height to thickness ratios less than the limits specified, header coursing present at every sixth or seventh course, calculated shears less than permitted and with acceptable anchorage at floors and roof, can be used as shear elements without adding gunite or other surface coatings. The ordinance will not require existing electrical, plumbing, mechanical and fire systems to be altered unless they constitute a hazard to life or property. The Los Angeles Ordinance is presently under Environmental Impact Review by the Planning Department relative to social and economic evaluation.

The concept of permitting less than current Uniform Building Code lateral force values for strengthening unreinforced masonry lime mortar structures was questioned by some municipalities such as The City of Salinas. Such strengthening was to reduce the risk of death and injury, but not to prevent damage to the strengthened building. The California Seismic Safety Commission helped to initiate Senate Bill 445, which was duly passed and signed by Governor Brown last year.

The State Law permits each City and County to set reasonable levels of design forces as determined by qualified persons. The design forces may be less than those specified in current codes relative to new buildings. If the jurisdiction does not have the expertise available, it can request the Office of the State Architect to establish force values and allowable stresses for materials and or construction.

Several Cities, such as Santa Maria and Salinas are developing ordinances similar to the proposed Los Angeles Ordinance. Other Cities, such as Sacramento and San Diego have permitted strengthening of unreinforced masonry buildings on an individual basis with reasonable compliance based upon use and occupancy.

A State Historical Building Code was developed in 1977 (6), with Guidelines finalized in 1979, that permits special consideration for Historical Buildings. The buildings can be brought into reasonable compliance by the Architect and Engineer doing the rehabilitation using individual judgment within the Guidelines and with the concurrence of the Building Department. An Advisory Board is available to local jurisdictions through the State Historical Building Code Commission. Guidelines on archaic masonry and adobe reflect recent research on the state of the art in unreinforced masonry. A survey of code provisions for Historic Buildings has been reported by Green and Cooke (1).

Areas outside California that have developed standards include the State of Massachusetts, The City of Seattle and Salt Lake City. The State of Massachusetts' standard, as applied in Boston, is to require a reduction in hazard potential when there is a change in Occupancy. Seattle has required anchorage of walls in the redevelopment area of Pioneer Square. Salt Lake City requires the Architect or Structural Engineer to improve the lateral strength of the building where there is an Occupancy change or other improvements are made on the building.

## CONCLUSIONS

Development of comprehensive ordinances in seismic prone areas, especially California, for the strengthening of unreinforced lime mortar masonry buildings is progressing. Implementation of the ordinances is dependent upon the political evaluation of social and economic impact versus comparative life safety. Research is now being pursued and more will be needed to determine the minimum level of compliance and force levels to provide relative life safety for various levels of ground shaking intensity and displacement. Anticipated design intensities and displacements should be provided by Engineering Geologists for different geographic areas. Code writing agencies can then better develop ordinances that will provide reasonable strengthening at minimum expense.

A major question to be resolved relative to the development of strengthening codes is whether the code should be a prescription code that provides force levels and values of resistance for new and existing materials of construction or if the individual Architect and Engineer should develop a method of reducing the hazard following general standards to satisfy the Building Official or Appeals Board.

## APPENDIX I - REFERENCES


1. Green, M. and P. W. Cooke, "Survey of Building Code Provisions for Historic Structures", NBS Technical Note 918, September 1976.

2. Long Beach Municipal Code, Sections 8100.8000 through 8100.8014, Subdivision 80, "Earthquake Hazard Regulations for Rehabilitation of Existing Structures Within the City", Long Beach, California, 1971.

3. Los Angeles City Planning Department, "Seismic Safety Plan Draft and Final EIR", City Plan Case No. 24880, Los Angeles, California, Reprinted March 1979.

4. Preliminary Draft, Division 68, "Earthquake Hazard Reduction in Existing Buildings", Los Angeles City Building Code, Los Angeles, California, November 1978.

5. "Tentative Provision for the Development of Seismic Regulations for Buildings", prepared by the Applied Technology Council associated with the Structural Engineers Association of California. National Bureau of Standards Special Publication 510, June 1978.

6. Title 24, Building Standards, State of California, Chapter B2, State Historical Building Code, 1979.

7. Uniform Building Code, 1979 Edition, International Conference of Building Officials, Whittier, California.

# INVESTIGATION OF REINFORCED BRICK MASONRY BUILDINGS UNDAMAGED BY THE SAN FERNANDO EARTHQUAKE

By Adham, S.A.

ABSTRACT: An investigation of a group of reinforced brick masonry buildings undamaged by the 1971 San Fernando earthquake was conducted. Seismic analyses were performed in three stages. In the first stage, the six-story reinforced concrete main building was analyzed using a two-dimensional model. Relative displacements were calculated at the expansion joints and compared to the reported damage of the flexible expansion joints. This comparison was used to assess and calibrate the estimated earthquake motions. These earthquake motions were used in the second stage as input to a three-dimensional model of a reinforced brick masonry building. This building was analyzed assuming various portions to be separated by 3-in. expansion joints. The analysis of this building was repeated in the third stage using a three-dimensional model without expansion joints. The third effort included review of research on fracture theories of brick masonry. A performance criterion was developed for assessing the response of the masonry buildings during the earthquake and evaluating the effect of expansion joints on relieving internal stresses. The results of the three efforts are integrated in a set of conclusions.

# INVESTIGATION OF REINFORCED BRICK MASONRY BUILDINGS UNDAMAGED BY THE SAN FERNANDO EARTHQUAKE

By Samy A. Adham[1]

## INTRODUCTION

Numerous investigations have been made of buildings damaged in past earthquakes in an effort to understand how they responded to the ground shaking and how they might have been designed to eliminate or minimize the resulting damage. But little attention has been paid to undamaged structures in the immediate area that were also exposed to the same ground motion environment. Understanding why a structure was not damaged can contribute significantly to our knowledge of earthquake engineering and to the design and construction of earthquake-resistant structures.

This paper provides the results of a study of the behavior of a group of buildings that comprise the Sepulveda Veteran's Administration Hospital during the February 9, 1971 San Fernando earthquake. The buildings were constructed of reinforced concrete and reinforced grouted brick masonry and were subjected to strong ground motions that greatly exceeded the original design assumptions. However, these buildings did not sustain any major structural damage [1]. The facility is located in the northern part of the San Fernando Valley (Fig. 1), in close proximity to three hospitals that were badly damaged in the earthquake: the San Fernando VA Hospital, the Holy Cross Hospital, and the Indian Hills Medical Center. There was also structural damage to many other buildings in the immediate area [2].

### Background

The group of buildings of concern in this study are 22 buildings and 5 support facilities ranging in height between one and six stories (Figs. 2 and 3). At the time the hospital was constructed in 1955, it was probably the largest single project utilizing reinforced brick masonry construction ever built in the United States. A survey of the hospital buildings immediately after the 1971 San Fernando earthquake indicated that there was only minimal structural damage to the Sepulveda VA Hospital [2]. Extensive elevator and plaster repairs were required, and a number of seismic joints required replacement. However, the operation of the hospital was not interrupted.

The violence of ground shaking was evidenced by extensive damage to the 4-in. flexible expansion joints in Building 3 and window jambs on the sixth floor being jolted out of place. In addition, a permanent displacement of approximately 1 in. was reported at the sixth-floor level. Visible cracks were also reported in the reinforced concrete frames supporting the boiler house roof and the chimney of the incinerator building.

[1] Senior Staff Engineer, Agbabian Associates, El Segundo, California

Objectives

The primary objectives of this study are to assess response of reinforced grouted brick masonry structures to strong earthquake shaking and to provide information on effectiveness of expansion joints in large and/or irregular buildings.

## ESTIMATES OF EARTHQUAKE GROUND MOTIONS AT THE SITE DURING THE 1971 SAN FERNANDO EARTHQUAKE

The San Fernando earthquake had a local magnitude $M_L = 6.4$ and a focal depth $h = 8.4$ km [3]. Soils at the site consist of a surficial layer of clayey sand underlain by sand and gravel with interspersed layers of clay (Fig. 4). Ground water is at a depth of approximately 250 ft. Penetration results indicate that soil layers are mostly dense [4]. Such soil would not, in general, cause any significant sudden changes in seismic waves passing through the site.

Morrill [5] indicated that for the 1971 San Fernando earthquake, except for the anomalously high Pacoima Dam results, attenuation of maximum horizontal ground accelerations from all recording sites is, for the most part, consistent with the equation, $\log (a/g) = 3.5 - 2 \log (D + 80)$, calculated by Cloud and Perez for past earthquakes (Fig. 5).

The Orion record provides the closest record to the subject site. The peak horizontal acceleration measured at the ground level was 0.255 g. The record at Orion was considered a free-field motion record because the effects of the soil/structure interaction were assumed to be small. With the local subsurface conditions indicating no anomalies at the hospital site, local soil-modifying effects were not considered significant in the free-field study at this site. Therefore, the site was considered as one part of the total propagation path.

The strong north-south asymmetry of the radiation pattern is primarily the result of the rupture propagation from north to south. The source-station azimuths of the Orion site are south of the epicenter. The contribution of Rayleigh waves generated at source toward the sites is very large.

Regression analysis (Fig. 6) of the San Fernando earthquake records indicates accelerations of 0.30 g at Orion (Table 1). Since the peak horizontal acceleration at Orion was 0.255 g, the results of regression analysis for horizontal acceleration should be scaled by a ratio of 0.255/0.30. Therefore, the 0.50 g obtained from regression analysis at the Sepulveda site was scaled to 0.40 g to represent the peak horizontal acceleration at the Sepulveda site during the earthquake. The peak vertical acceleration recorded at the Orion site is equal to the peak vertical acceleration calculated from regression analyses. Therefore, the peak vertical acceleration of 0.29 g obtained from regression analysis for the Sepulveda site was used as the peak vertical acceleration at the site during the San Fernando earthquake.

TABLE 1. SUMMARY OF DATA NEEDED TO ESTIMATE PEAK GROUND ACCELERATION AT THE SITE DURING THE 1971 SAN FERNANDO EARTHQUAKE

| Station Name | Distance from Epicenter, km | Recorded Motions | | Maley and Cloud (1973) (Fig. 2-11) | Regression Analysis (SW/AA, 1979) | Max. Acceleration, g Based on Correlation with Earthquake Damage | Intensity of Shaking (Scott, 1971) | Maximum Acceleration, g Estimated |
|---|---|---|---|---|---|---|---|---|
| | | Component | Maximum Acceleration, g | | | | | |
| Pacoima | 8 | S16E<br>S74W<br>Down | 1.171<br>1.076<br>0.710 | 0.54 | 0.95 | | VIII-XI | |
| Olive View Hospital | 9 | | | 0.48 | 0.75 | 0.50* | VIII-XI | |
| Sylmar Converter Station | 10.7 | | | 0.43 | 0.65 | 0.50† | VIII-XI | |
| Lower Van Norman Dam | 12 | | 0.50‡ | 0.42 | 0.55 | 0.50§ | VIII-XI | |
| Sepulveda Hospital | 14 | | | 0.40 | 0.50 | | VIII-XI | 0.40 ‡‡ |
| 8244 Orion | 20 | N00W<br>S90W<br>Down | 0.255<br>0.134<br>0.171 | 0.35 | 0.30 | | VII | |

AA10362

*Structural Engineers Association of Southern California, 1971

†Agbabian-Jacobsen Associates (1971)

‡Seismoscope Record

§Seed et al. (1971)

‡‡ Using Regression Analysis: Peak Acceleration at Sepulveda = $\frac{0.50}{0.30} \times 0.255 \cong 0.40$

The Orion record, scaled to 0.40 g and 0.29 g peak horizontal and vertical accelerations, respectively, was used as input to analytical models of the hospital structures. The time history for Orion record is shown in Figure 7, while the response spectra for the scaled horizontal component is shown in Figure 8.

The response spectra for the scaled N00W component of the Orion record and ATC design earthquake for Los Angeles are compared in Figure 8. Reasonable agreement of the two spectra is shown between 2 and 5 Hz. However, for frequencies above 5 Hz, the spectrum for the scaled Orion falls below the ATC spectrum. This drop is due to the effect of deep alluvium deposits that appear to attenuate some of the high-frequency content of the earthquake motions. For frequencies below 2 Hz, the scaled Orion spectrum is higher than the ATC spectrum, due to the effect of the surface waves on the long period end of the spectrum.

## TWO-DIMENSIONAL SEISMIC ANALYSIS OF BUILDING 3

A two-dimensional analytical model of the general medical and surgical building (Bldg. 3, Fig. 3) was constructed. The six-story building has a reinforced concrete skeleton frame with a concrete panel wall faced with brick. This building was selected because it experienced considerable damage to the seismic (flexible) joints between building segments [2]. The analysis estimated the relative displacements of Segments B1 and B2 of Building 3 at the expansion joints when subjected to the developed earthquake input. These displacements were compared to the observed responses of these joints during the 1971 San Fernando earthquake. This comparison provided a tool for calibrating the earthquake input developed for the site. This input was used to evaluate the response of the reinforced brick masonry buildings at the site during the 1971 earthquake.

Figure 9 provides an elevation and typical floor plan of segments of Building 3: B1, B2, and part of Unit A. These are typical of the other two reinforced concrete structures at the hospital site. All shear walls are wall bearing, supported on continuous wall footings. Interior columns are supported on isolated footings. The reinforced concrete walls are 12 in. at the second and third floor levels and 8.5 in. at the fourth and fifth floor levels.

The walls, slabs, and columns for a typical bay in Segments B1 and B2 were modeled using a two-dimensional finite element mesh (Fig. 10). The reinforced concrete walls were modeled as plane stress quadrilaterals. The soil spring elements simulate soil/structure interaction effects [6].

To ensure an adequate computation of the response, the first six natural modes were included in the calculations. The first four significant frequencies and modes are described in Table 2.

TABLE 2. CALCULATED FREQUENCIES AND MODE SHAPES FOR BUILDING 3B

(a) Building 3B1

| Mode No. | Frequency Hz | Description |
|---|---|---|
| 1 | 5.7 | First shear mode with rocking of foundation |
| 2 | 20.0 | Second shear mode of building |
| 3 | 24.9 | Breathing mode |
| 4 | 35.6 | Combined shear and local deformations |

(b) Building 3B2

| Mode No. | Frequency Hz | Description |
|---|---|---|
| 1 | 3.2 | First shear mode with rocking of foundation |
| 2 | 10.3 | Second mode of building |
| 3 | 16.6 | Localized displacement |
| 4 | 20.9 | Vertical displacement mode |

Critical displacements were computed using the 0.40 g scaled 1971 Orion ground motions as input to the analysis. The resulting displacements are shown in Figure 11, in which the 4-in. gap at the top between the two buildings has closed by an amount of 1.5 in.

Figure 12 shows typical wall cross sections at the roof of Buildings 3B1 and 3B2. Figures 13 and 14 show damage to the flexible expansion joint at the corner of Building 3B1 adjacent to the machine room at the roof of Building 3B2. Examination of the details of this corner and other parts of Building 3 indicated that the flashing at the expansion joint appears to have been damaged by severe seismic cycles of tension and compression, and that a permanent lateral displacement of the building was reported to be 0 in. at ground level to 1 in. at the sixth floor level [2].

The results of the analysis indicate that the 0.40 g scaled 1971 Orion record provides a reasonable earthquake input for evaluating the response of the Sepulveda Hospital buildings during the 1971 San Fernando earthquake.

## THREE-DIMENSIONAL SEISMIC ANALYSIS OF BUILDING 10

The structure considered is a two-story, reinforced, grouted brick masonry building with a reinforced concrete basement (Bldg. 10, Figs. 15 and 16). All floor slabs are cast-in-place reinforced concrete. Basement walls are 12-in. reinforced concrete, while all walls constructed above basement level are of 13-1/2-in. reinforced grouted brick. All perimeter shear walls are wall-bearing, while interior columns are supported on isolated footings. Flexible expansion joints separating various portions of Segments A, B, and C are 3 in. wide, in contrast to the 4 in. used in the six-story Building 3.

Earthquake input motions, calibrated in the Building 3 analysis, are applied as input to the analyses of Building 10. The middle segment of the building was analyzed first by a three-dimensional model. Critical response of shear wall elements was evaluated. A second analysis was conducted in which the building model was expanded to include the other two segments of the building. This analysis provided an assessment of the effect of eliminating the expansion joints in the response of this building as a total unit.

Basement walls are 12-in. reinforced concrete, while all walls constructed above basement are constructed of 13-1/2-in. reinforced moderate weathering brick of mortar type M with the following properties:

- Compressive strength of brick = 2500 psi
- Compressive strength of masonry prism = 1600 psi
- Minimum steel ratio = 0.0024
- Designed according to 1951 Los Angeles city code

Shear walls and floor slabs were modeled by plate elements, while columns were modeled by beam elements. In all, the model utilized 225 plate elements and 159 beam elements and has 202 nodal points (Fig. 17).

A second model of Building 10 is shown in Figure 18. This model includes Segment A in addition to Segments B and C, which are assumed to be tied to Segment A. This model represents the condition of Building 10 without expansion joints. Segment A is represented by the same refined mesh shown in Figure 17. However, Segments B and C are represented by a smaller number of elements and nodal points. This simplification was dictated by the need to stay within storage capacity of the computer and to reduce the cost of running a three-dimensional finite-element model, while preserving the same refinement for Segment A to compare responses under both conditions. The combined model of Building 10 comprised 353 plate elements, 183 beam elements, and 314 nodal points.

The first 10 natural mode shapes and frequencies of response of Buildings 10A and 10 were included in the calculations. The first three significant frequencies and modes are described in Table 3.

The three components of the scaled 1971 Orion record were used in the analyses. Calculated critical stresses are shown in Table 4.

## EVALUATION OF PERFORMANCE OF BRICK MASONRY BUILDINGS AT THE SEPULVEDA HOSPITAL DURING THE 1971 SAN FERNANDO EARTHQUAKE

Figure 19 illustrates the fracture criterion for normal quality brick developed for this study [6]. The expected performance of Building 10 is expressed by the ratio B, which is defined as:

$$B = \frac{\text{critical shear stress}}{\text{calculated shear stress}} \tag{1}$$

A value of B greater than 1.0 indicates that the panel would perform adequately, while a value significantly less than 1.0 would postulate failure of that particular panel.

Table 4 lists elements that have critical stresses in Segment A of Building 10. The critical shear stresses are calculated from the state of stress for each element by equations given in Table 4. Performance factors for critical Elements 71, 81, and 90 are 1.8, 1.85, and 1.87, respectively. These factors are larger than 1, indicating that the building had a considerable resistance against this mode of failure during the San Fernando earthquake.

The performance of Building 10 was also evaluated using a maximum tensile cracking criteria. Fracture was assumed to initiate when tensile stresses exceeded maximum allowable tensile stresses in a biaxial state of stress. The criteria for interaction between principal tensile stress $\sigma_2$ and the principal compressive stress $\sigma_1$ are given in Figure 20. The

TABLE 3. CALCULATED FREQUENCIES AND MODE SHAPES FOR BUILDING 10

| Mode No. | Frequency, Hz | Description |
|---|---|---|
| 1 | 9.0 | First mode in X direction |
| 2 | 12.0 | Displacements in N-S and E-W directions; complex deformation pattern |
| 3 | 14.0 | First mode in Y direction |

(a) Frequency of Building 10A (with Expansion Joints — segment A only)

| Mode No. | Frequency, Hz | Description |
|---|---|---|
| 1 | 7.3 | First mode in Y direction |
| 2 | 8.3 | Twisting mode of building |
| 3 | 10.5 | First mode in X direction |

(b) Frequency of Building 10 (segments A, B and C tied together)

TABLE 4. ASSESSMENT OF CRITICAL STRESSES IN BUILDING 10A

| Element No. | $\sigma_y$ Dead Load | $\sigma_y$ Seismic Load | $\sigma_y$ Total | $\sigma_x$ | $\tau_{xy}$ (1) | Critical Shear Stress Failure Criteria: Critical Shear Stress* (2) | Critical Shear Stress Failure Criteria: Performance Factor (2)/(1) | Biaxial Failure Criteria: Principal Stresses $\sigma_1$† | Biaxial Failure Criteria: Principal Stresses $\sigma_2$† | Biaxial Failure Criteria: Performance Factor‡ |
|---|---|---|---|---|---|---|---|---|---|---|
| 71 | 31.2 | 4.9 | 36.1 | 2.6 | 84.9 | 152.0 | 1.8 | 146.7 | -110.6 | 1.3 |
| 81 | 17.5 | 19.2 | 46.7 | 4.5 | 94.0 | 173.4 | 1.85 | 166.3 | -119.6 | 1.2 |
| 90 | 30.0 | 5.9 | 35.9 | 3.8 | 81.3 | 151.8 | 1.87 | 141.2 | -105.3 | 1.35 |

Stress in psi at 12.52 seconds

(+) Sign denotes compression

* Critical shear stress

$$\tau_{cr} = 80 + 2\sigma_y \qquad 0 \le \sigma \le 50 \text{ psi}$$

$$\tau_{cr} = 120 + 0.90\,\sigma_Y \qquad 50 \le \sigma \le 400 \text{ psi}$$

† $$\sigma_{1,2} = \frac{\sigma_y}{2} \pm \sqrt{\left(\frac{\sigma_y}{2}\right)^2 + \left(1.5\,\tau_{xy}\right)^2}$$

$$x = \frac{\sigma_1}{f'_m} + \frac{\sigma_2}{0.1\,f'_m}$$

‡ Performance factor $= \frac{1}{x}$

maximum tensile stress is assumed to occur at the center of the panel and is calculated by means of Mohr's circle to be

$$\sigma_{1,2} = \frac{\sigma_y}{2} \pm \sqrt{\left(\frac{\sigma_y}{2}\right)^2 + (1.5\ \tau_{xy})^2} \qquad (2)$$

where $\sigma_2$ is the maximum tensile principal stress and $\sigma_y$ and $1.5\ \tau_{xy}$ represent the normal and shear stress on a horizontal section at the middle of the plate. The critical factor x is calculated from the interaction formula

$$x = \frac{\sigma_1}{f'_m} + \frac{\sigma_2}{0.1\ f'_m} \qquad (3)$$

The performance factor B is calculated from the relationship

$$B = \frac{1}{x} \qquad (4)$$

Table 4 gives performance factors in Segment A of Building 10, using biaxial failure criteria. These factors range from 1.2 to 1.35, indicating that the shear panels performed adequately during the 1971 San Fernando earthquake. However, the range of these factors is considerably less than the range of 1.8 to 1.87 obtained from the critical shear stress failure criteria. In addition, the lowest performance factor of 1.2 given for Element 81 indicates only a 20% safety margin over the factor of 1.0 that would postulate failure of that particular panel.

Analysis results indicate that flexural moments were very small and would not have created significant stresses.

## ASSESSMENT OF CONNECTIONS OF BUILDING 10

A qualitative assessment was made of connections and construction details by reviewing the available drawings. Some selected details are shown in Figures 21 through 24. The overall qualitative assessment indicates that connections were adequately designed and detailed.

## EFFECT OF EXPANSION JOINTS

The results of the analysis indicate that stresses were increased by 28% for Element 71, 75% for Element 81, and 42% for Element 90 when Segments A, B, and C were tied together (Table 5). However, Element 81 has a performance factor as low as 1.20 for the case when Building 10A is not tied to 10B or 10C (also shown in Table 5). Performance factors of 1.30 and 1.35 were shown in this table for the same case for Elements 71 and 90, respectively. Therefore, the analyses results indicate that if Segments A, B, and C of Building 10 had been tied together (i.e., no expansion joints), this building would have suffered numerous cracks and probably considerable damage during the 1971 San Fernando earthquake.

## CONCLUSIONS

Conclusions are summarized as follows:

1. Expansion joints were effective in reducing stresses below damaging levels and contributed to the survival of the hospital buildings.

2. Excellent connections and detailing resulted in adequate resistance to the 1971 San Fernando earthquake.

3. The use of adequate grout in shear walls between the outer and inner wythes of brick provided adequate bond for the outer wythe. This bond resulted in mobilization of the full thickness of the brick shear wall to resist earthquake forces.

4. The use of more refined shear and biaxial failure criteria provided a better tool for evaluating state of stress in the building.

## ACKNOWLEDGMENTS

This research effort has been supported by the National Science Foundation under Grant No. 79-00013. This support is gratefully acknowledged. The data on the hospital were provided by Messrs. Richard McConnell and Gerald Smith of the Veterans Administration, and Gregg Brandow and Art Navile of Brandow and Johnston in Los Angeles. Many members of Agbabian Associates contributed to the study, with computer analysis performed by Dr. Eric Lee.

TABLE 5. COMPARISON OF STRESSES IN BUILDING 10 WITH AND WITHOUT EXPANSION JOINTS

| El. No. | $\sigma_y$ | | | | | $\sigma_x$ | | $\tau_{xy}$ | | Principal Stresses | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | Dead Load | Seismic | | Total | | | | | | $\sigma_1$† | | $\sigma_2$† | |
| | | 1 | 2 | 1 | 2 | 1 | 2 | 1 | 2 | 1 | 2 | 1 | 2 |
| 71 | 31.2 | 4.9 | 17.4 | 36.1 | 48.6 | 2.6 | 7.0 | 84.9 | 109.4 | 146.7 | 190.2 | -110.6 | -141.6 |
| 81 | 27.5 | 19.2 | 1 | 46.7 | 28.5 | 4.5 | 19.9 | 94.0 | 141.6 | 166.3 | 227.6 | -119.6 | -199.1 |
| 90 | 30.0 | 5.9 | 19.8 | 35.9 | 49.8 | 3.8 | 8.4 | 81.3 | 115.4 | 141.2 | 199.8 | -105.3 | -150.0 |

(-) Sign denotes tension

* Stresses are in psi

1 Building with expansion joints (existing configuration)

2 Building without expansion joints (Parts A, B, and C tied together)

† $$\sigma_{1,2} = \frac{\sigma_y}{2} \pm \sqrt{\left(\frac{\sigma_y}{2}\right)^2 + \left(1.5\ \tau_{xy}\right)^2}$$

# REFERENCES


1. Dickey, W.L. "Now for the Good Part, or Veterans Administration's Wise Decision," *Masonry Industry,* Los Angeles, CA: Masonry Ind., Jan 1972.

2. Lew, H.S.; Leyendecker, E.V.; and Dikkers, R.D. *Engineering Aspects of the 1971 San Fernando Earthquake,* Building Science Series 40. Washington, D.C.: Building Research Division, National Bureau of Standards, Dec 1971.

3. Allen, C.R.; Hanks, T.C.; and Whitcomb, J.H. "Seismological Studies of the San Fernando Earthquake and Their Tectonic Implications," *San Fernando, California, Earthquake of February 9, 1971,* Bull. 196, ed. G.B. Oakeshott. Sacramento, CA: Calif. Div. of Mines and Geology, 1973.

4. Woodward-Lundgren & Associates (WL & A). *Geologic, Seismologic, and Earthquake Engineering Investigations and Evaluations: Veterans Administration Hospital, Sepulveda, California,* VA Project No. 04-5662. Oakland, CA: WL & A, 1975.

5. Morrill, B.J. "Seismoscope Results," *San Fernando, California, Earthquake of February 9, 1971,* Vol. III, ed. L.M. Murphy. Washington, D.C.: U.S. Dept. of Commerce, National Oceanic and Atmospheric Administration, 1973, pp 353-364.

6. Agbabian Associates (AA). *Investigation of Reinforced Brick Masonry Buildings Undamaged by the San Fernando Earthquake,* R-7933-5040. El Segundo, CA: AA, May 1980.

7. Structural Engineers Association of Southern California, *Report on Olive View Hospital,* Los Angeles, May 25, 1971, 40 pp.

8. Agbabian-Jacobsen Associates (AJA). *Assessment of Earthquake-Resistant Design of AC-DC Converter Stations, High Voltage DC Pacific Intertie,* R-7119-1984. Los Angeles, CA: AJA, Aug 1971.

9. Seed, H.B., Lee, K.L., and Idriss, I.M., "Appendix B--Preliminary Report on Lower San Fernando Dam Slide during the San Fernando Earthquake of February 9, 1971," *Effects of the San Fernando Earthquake on the Van Norman Reservoir Complex,* California Division of Safety of Dams, Sacramento, May 1971, 13 pp and 15 pl.

10. Yerkes, R.F. "Effects of San Fernando Earthquake as Related to Geology," *San Fernando, California, Earthquake of February 9, 1971,* Vol. III, ed. L.M. Murphy. Washington, D.C.: U.S. Dept. of Commerce, National Oceanic and Atmospheric Administration, 1973, pp 137-154.

11. Maley, R.P. and Cloud, W.K. "Strong-Motion Accelerograph Records," *San Fernando, California, Earthquake of February 9, 1971,* Vol. III, ed. L.M. Murphy. Washington, D.C.: U.S. Dept. of Commerce, National Oceanic and Atmospheric Administration, 1973, pp 325-348.

12. Shannon & Wilson (SW) & Agbabian Associates (AA). *Statistical Analysis of Earthquake Ground Motion Parameters,* NUREG/CR-1175. Seattle, WA: SW and El Segundo, CA: AA, 1979,

13. Adham, S; Bhaumik, A,; and Isenberg, J. *Reinforced Concrete Constitutive Relations,* AFWL-TR-74-72. El Segundo, CA: Agbabian Assoc., Feb 1975.

14. Arya, S.K. and Hegemier, G.A. "On Nonlinear Response Predictions of Concrete Masonry Assemblies," *Proc. of the North American Masonry Conf. Aug 14-16, 1978.* Boulder, CO: University of Colorado, pp 19-1 — 19-24.

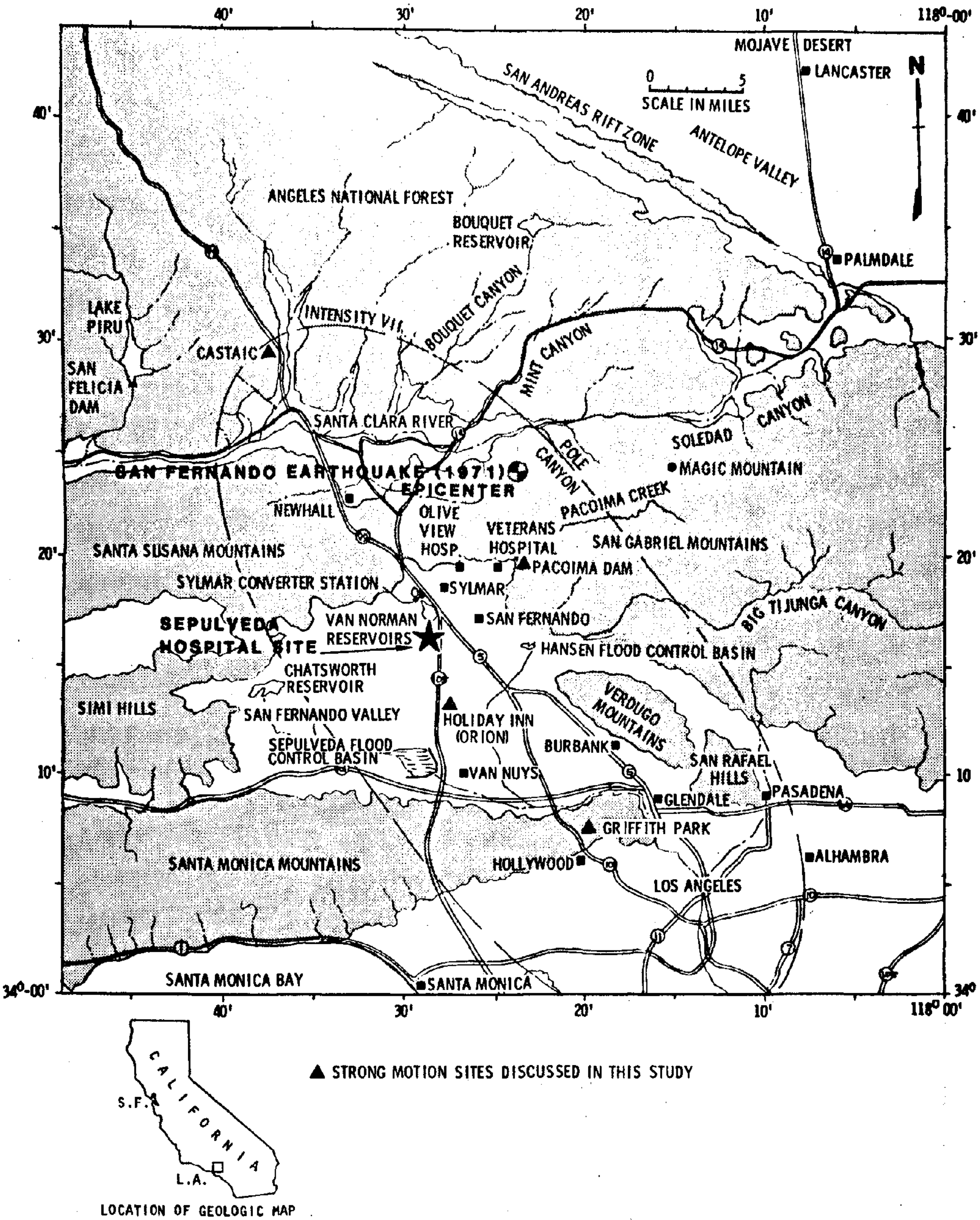



FIGURE 1. GENERAL MAP SHOWING PHYSIOGRAPHICAL FEATURES OF SAN FERNANDO VALLEY AND SURROUNDING MOUNTAIN AREAS AND LOCATION OF SEPULVEDA HOSPITAL

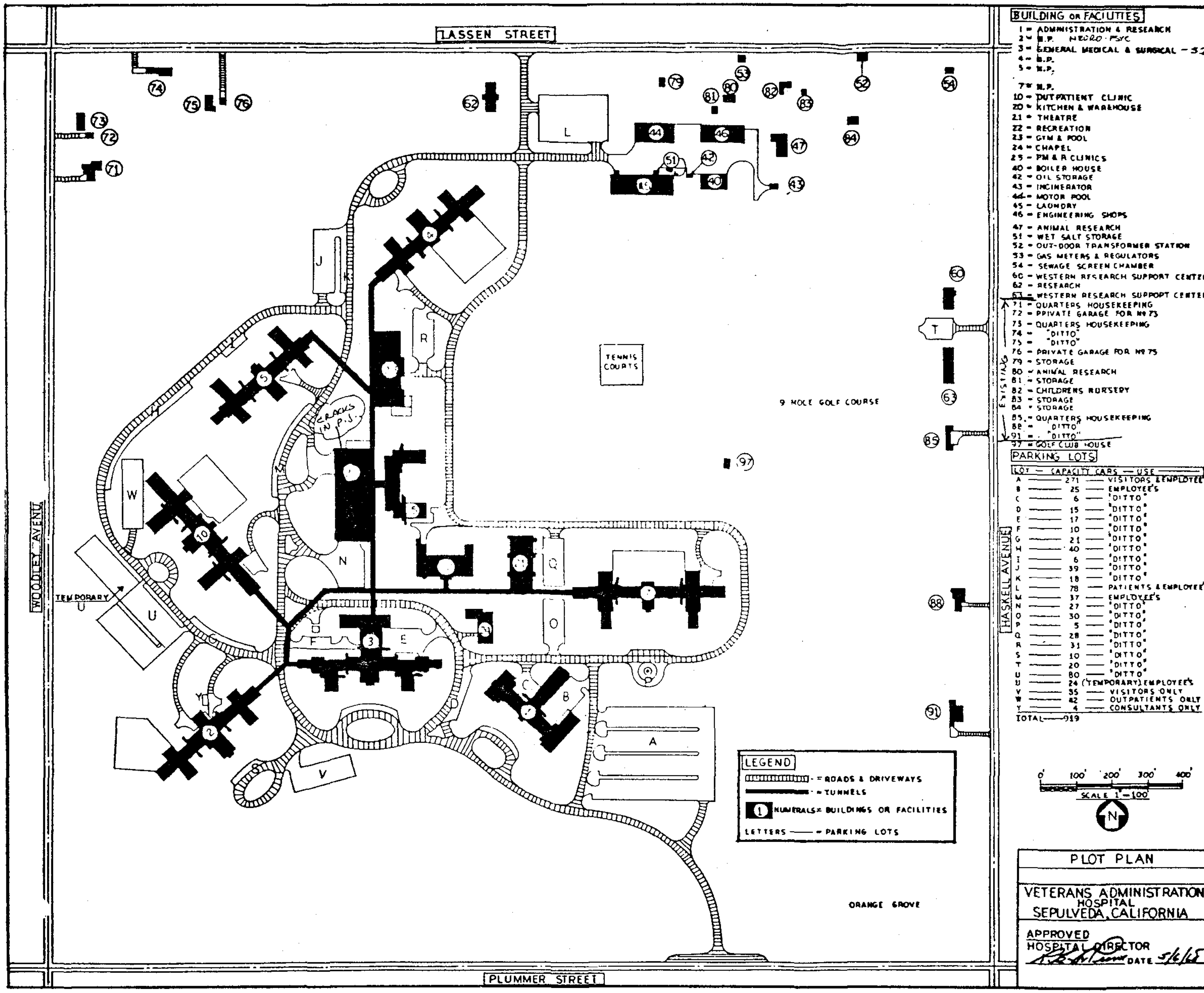
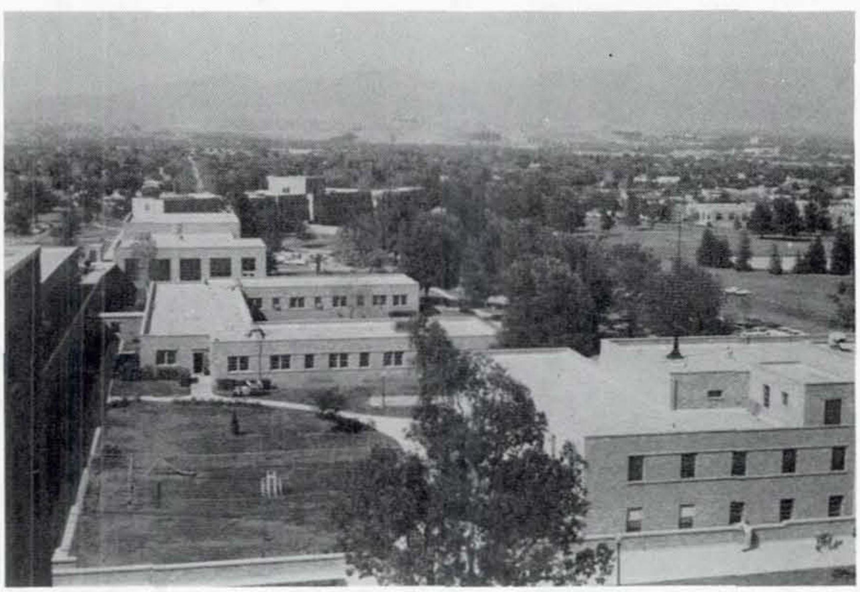



FIGURE 2. GENERAL LAYOUT SHOWING BUILDINGS 3 AND 10

(a) Building 3



(b) General view from top of Building 3

FIGURE 3. VIEW OF HOSPITAL BUILDINGS

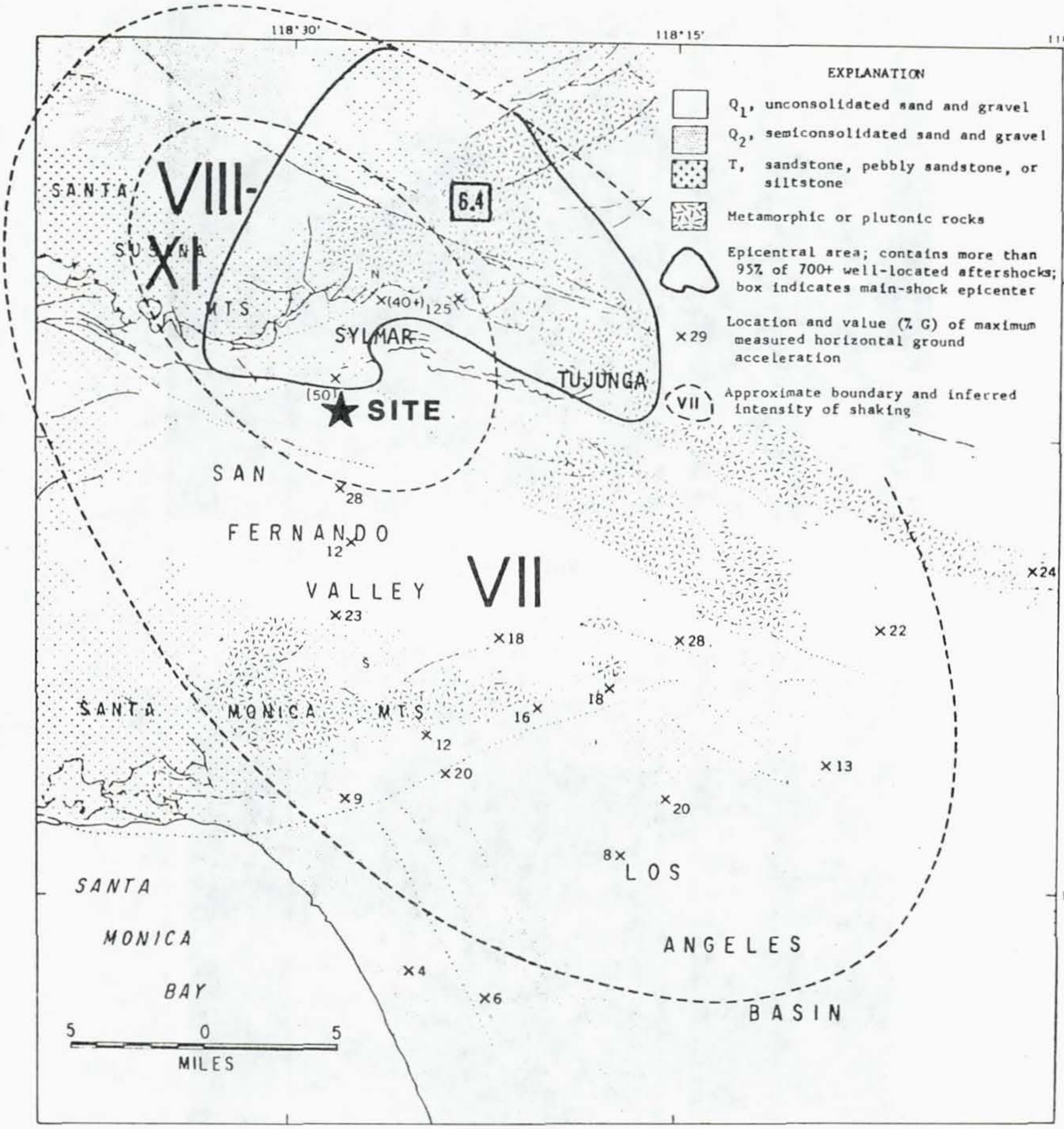



FIGURE 4. MAP OF TRANSITORY EFFECTS OF FEBRUARY 9 (1971) EARTHQUAKE (Yerkes, 1973)

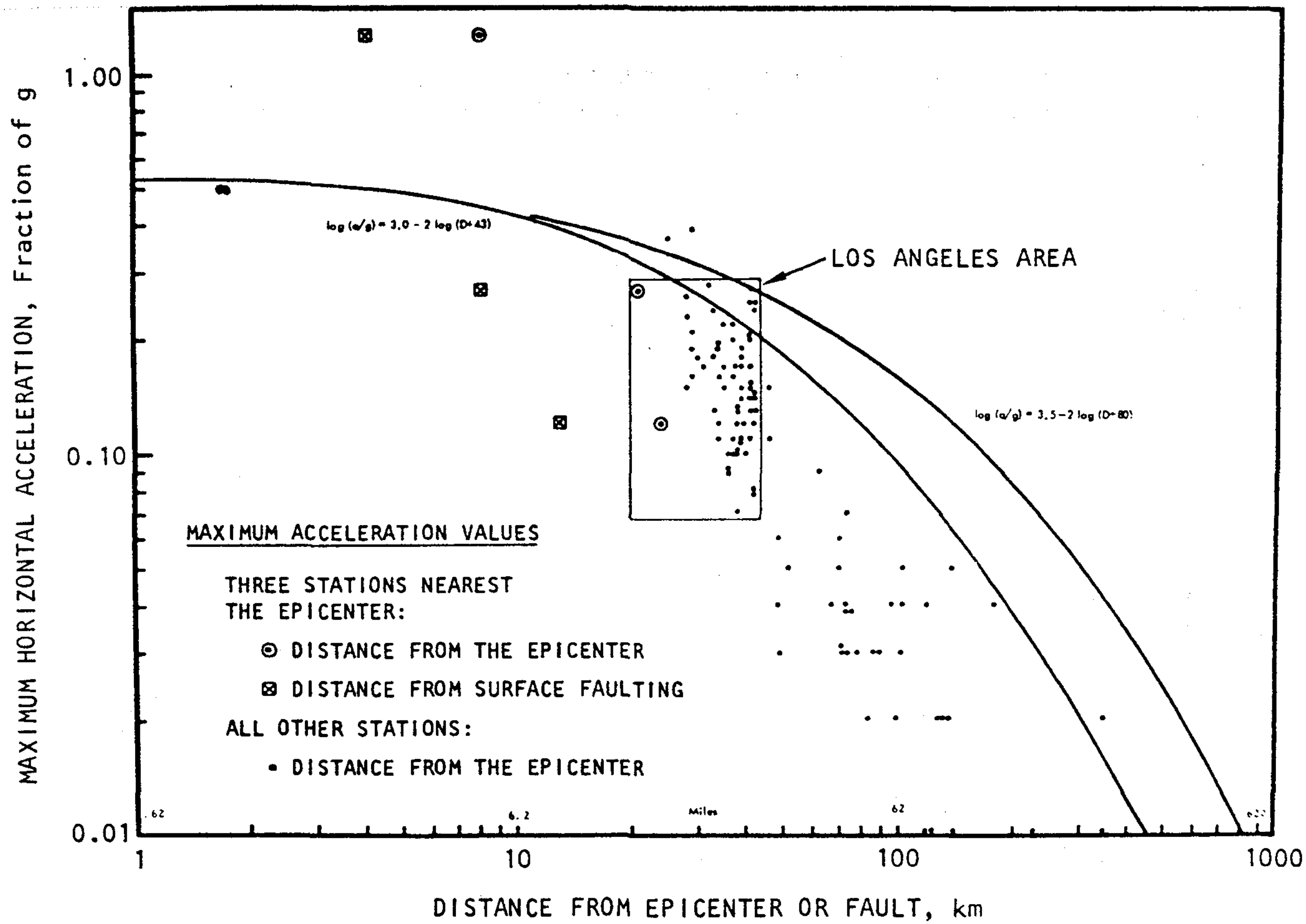



FIGURE 5. MAXIMUM HORIZONTAL GROUND ACCELERATIONS RECORDED DURING SAN FERNANDO EARTHQUAKE (VALUES $\geq$ 0.01 g) COMPARED TO ATTENUATION CURVES DEVELOPED BY CLOUD AND PEREZ (Maley and Cloud, 1973)

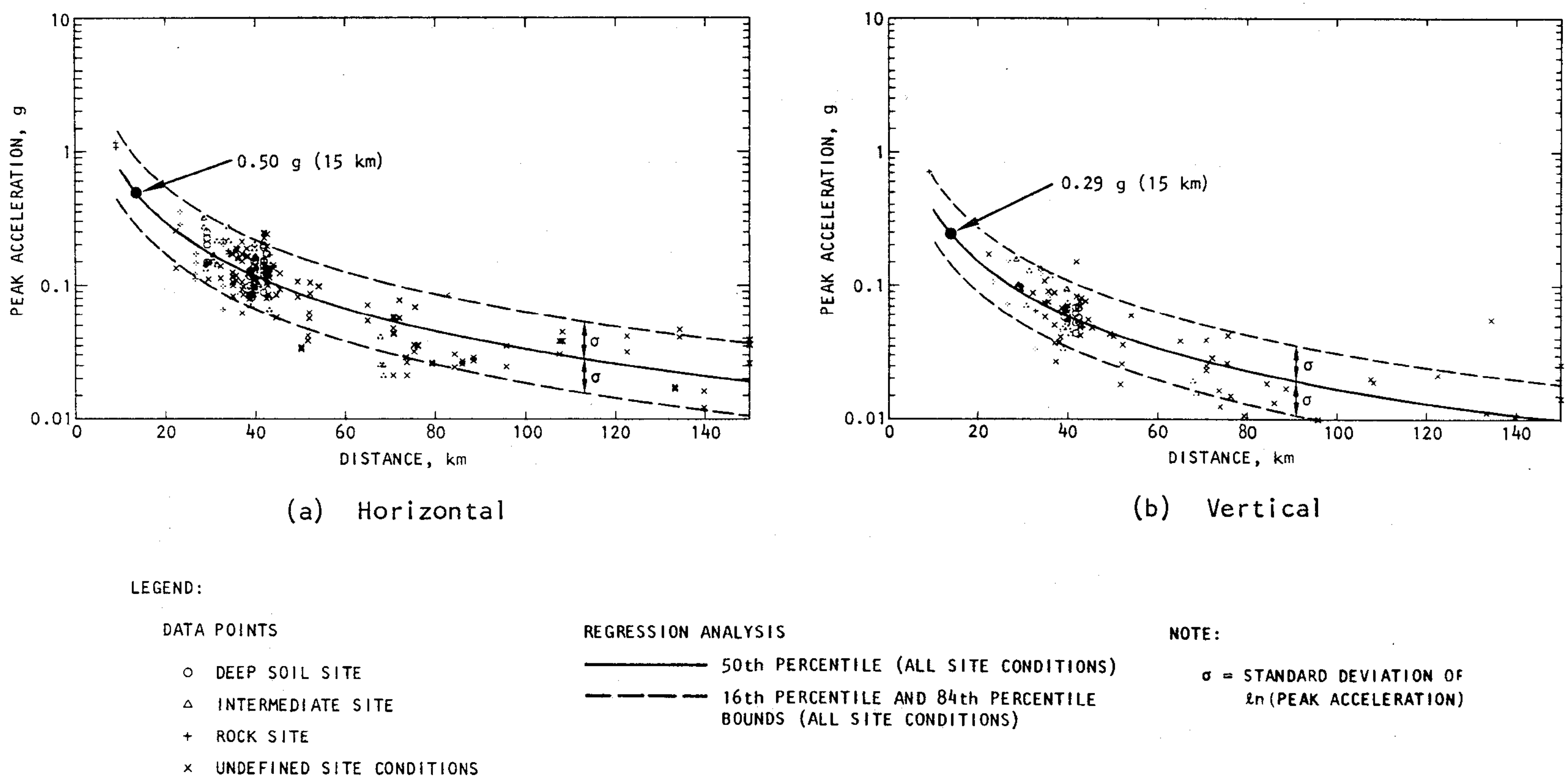
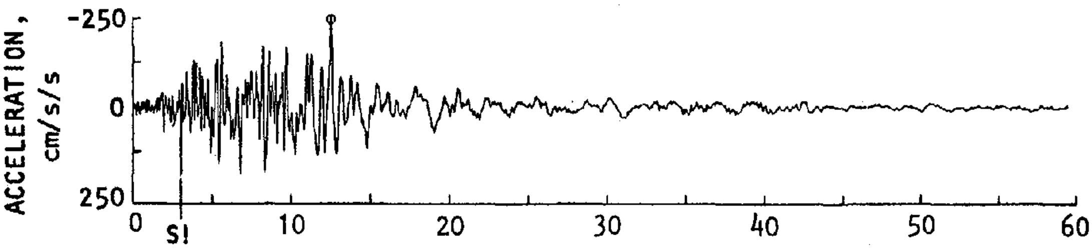
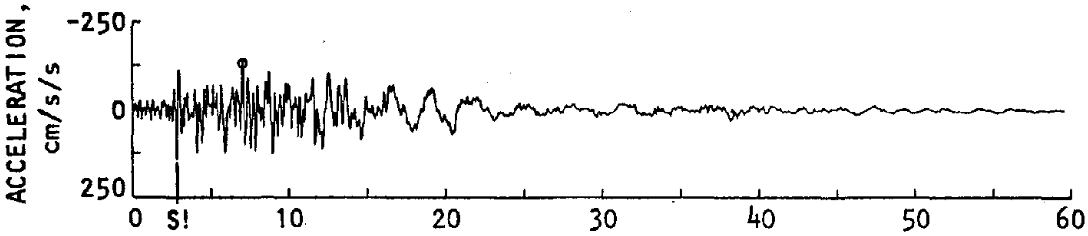


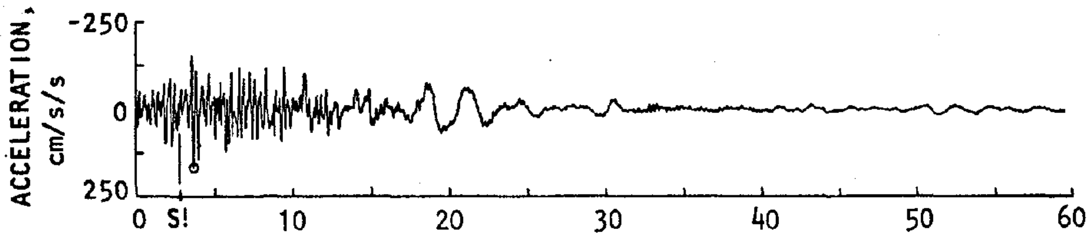

FIGURE 6. ATTENUATION OF PEAK GROUND ACCELERATIONS RECORDED DURING THE 1971 SAN FERNANDO EARTHQUAKE (SW/AA, 1979)




(a) Comp N00W




(b) Comp S90W




(c) Comp down

FIGURE 7. THREE COMPONENTS OF GROUND MOTION AT ORION SITE

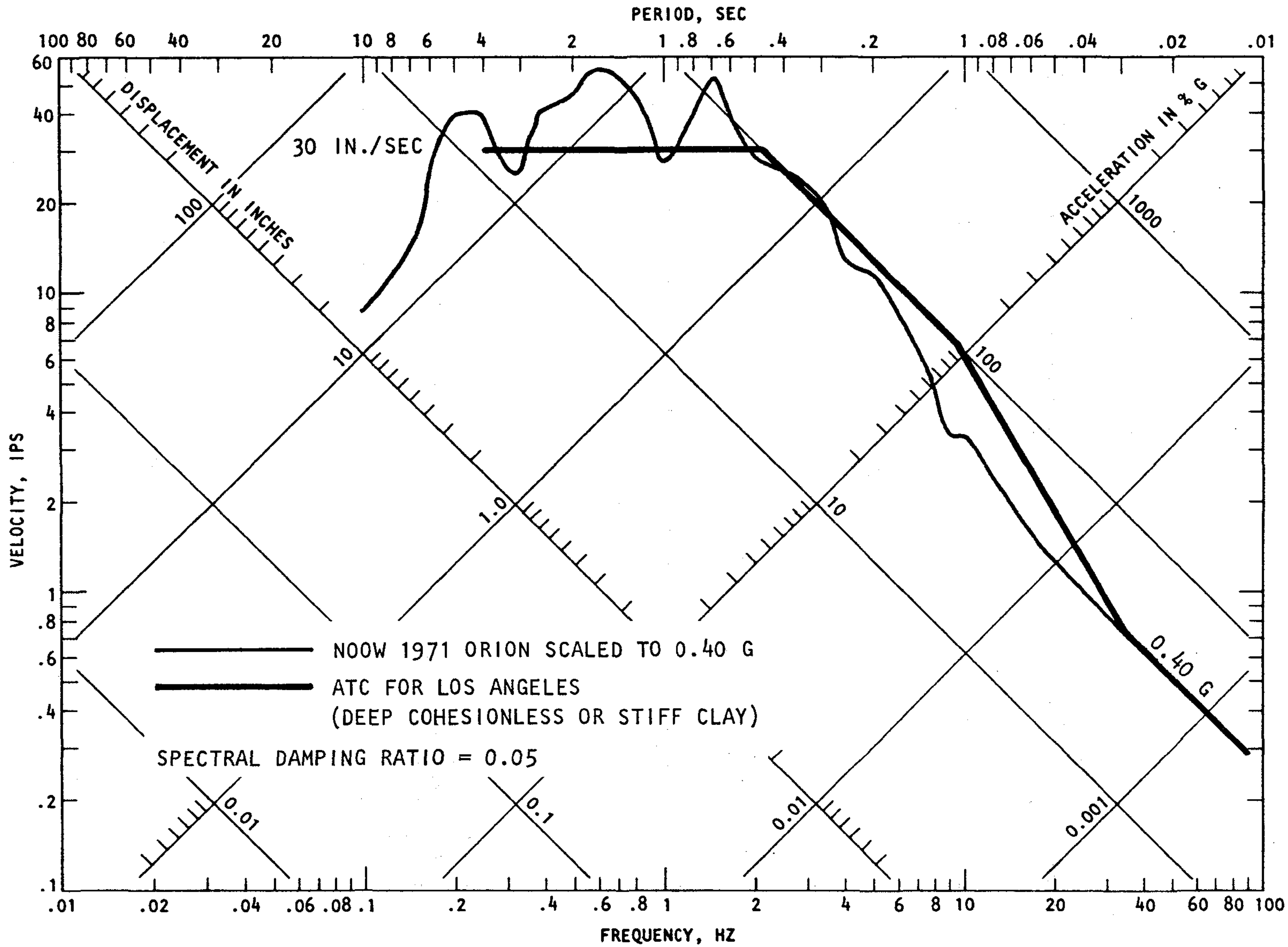
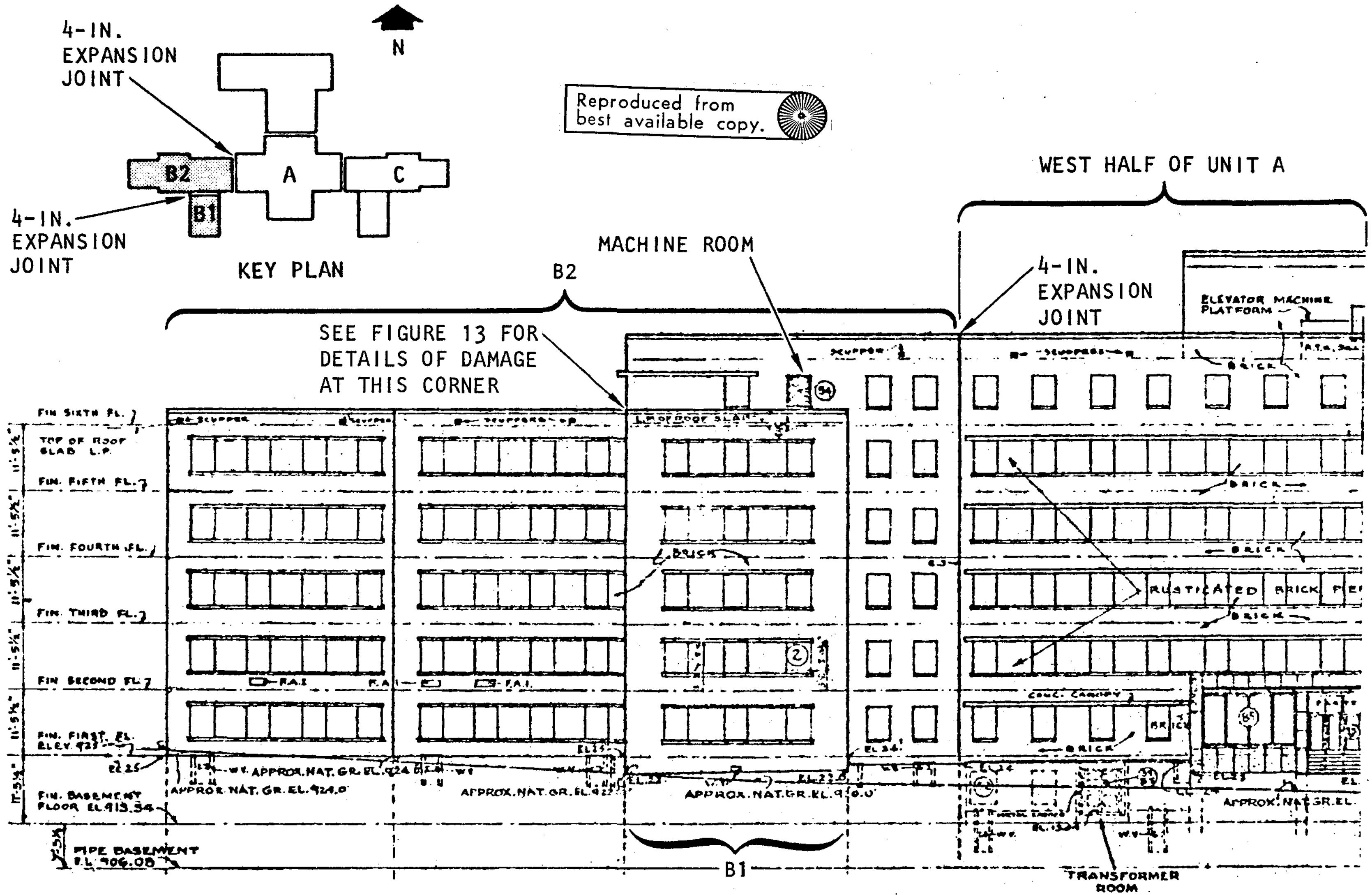
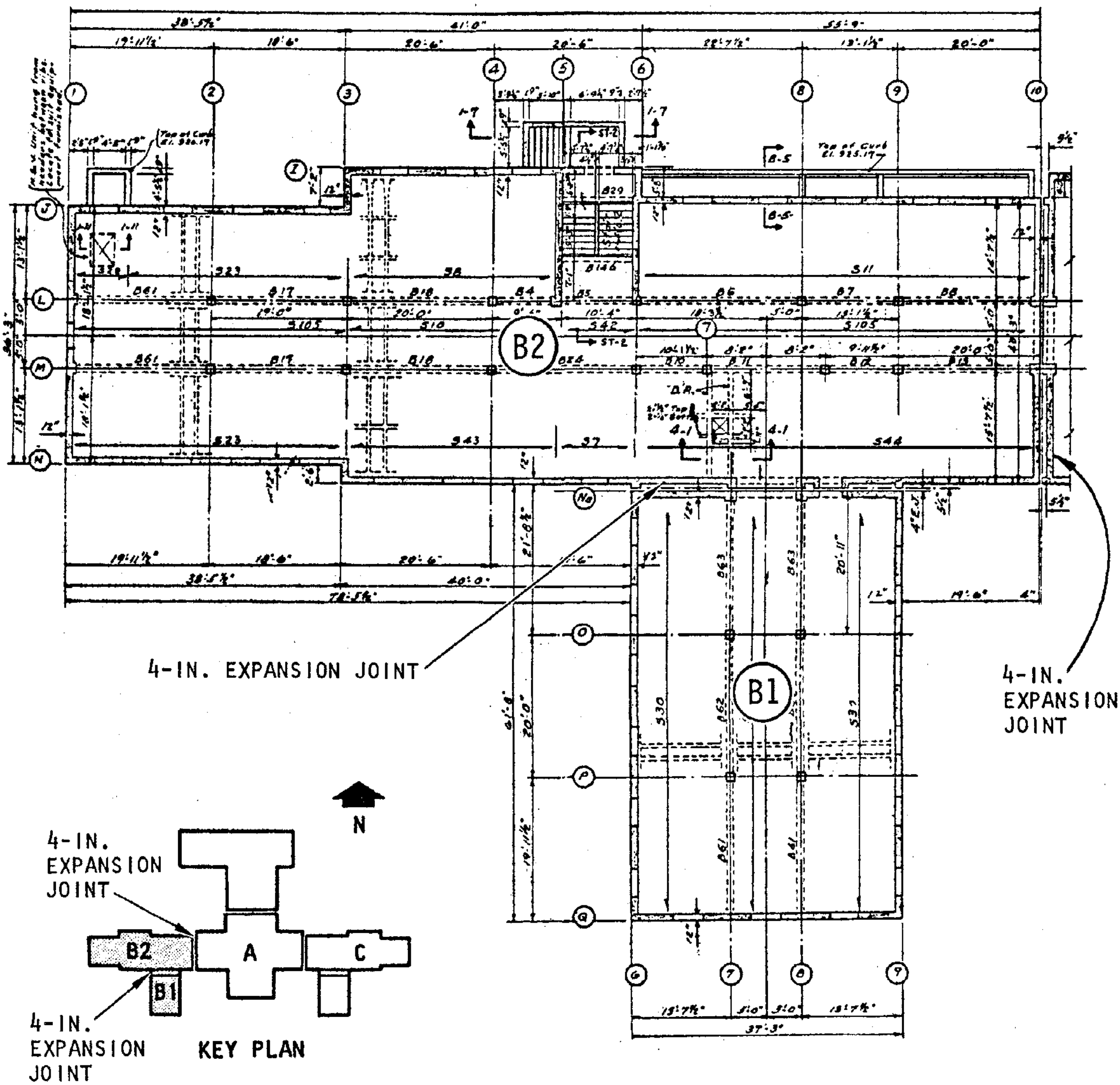



FIGURE 8. COMPARISON OF RESPONSE SPECTRA FOR 0.40 G SCALED NOOW 1971 ORION AND ATC FOR LOS ANGELES AREA



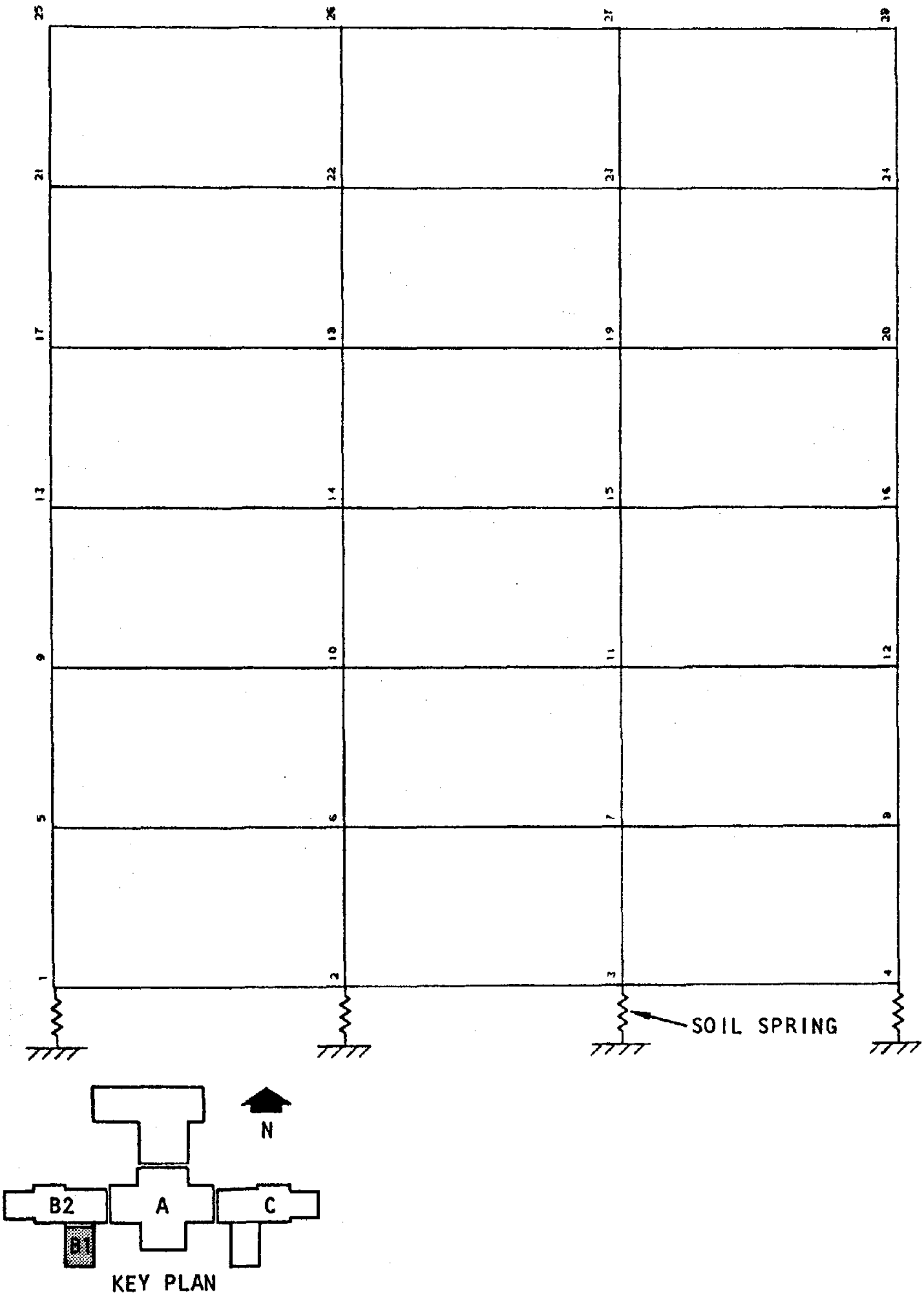

(a) Elevation (units A and B)

FIGURE 9. BUILDING 3




(b) First floor plan

FIGURE 9. (CONCLUDED)




(a) Building 3B1

FIGURE 10. TWO-DIMENSIONAL FINITE ELEMENT MESH FOR TYPICAL TRANVERSE SECTION

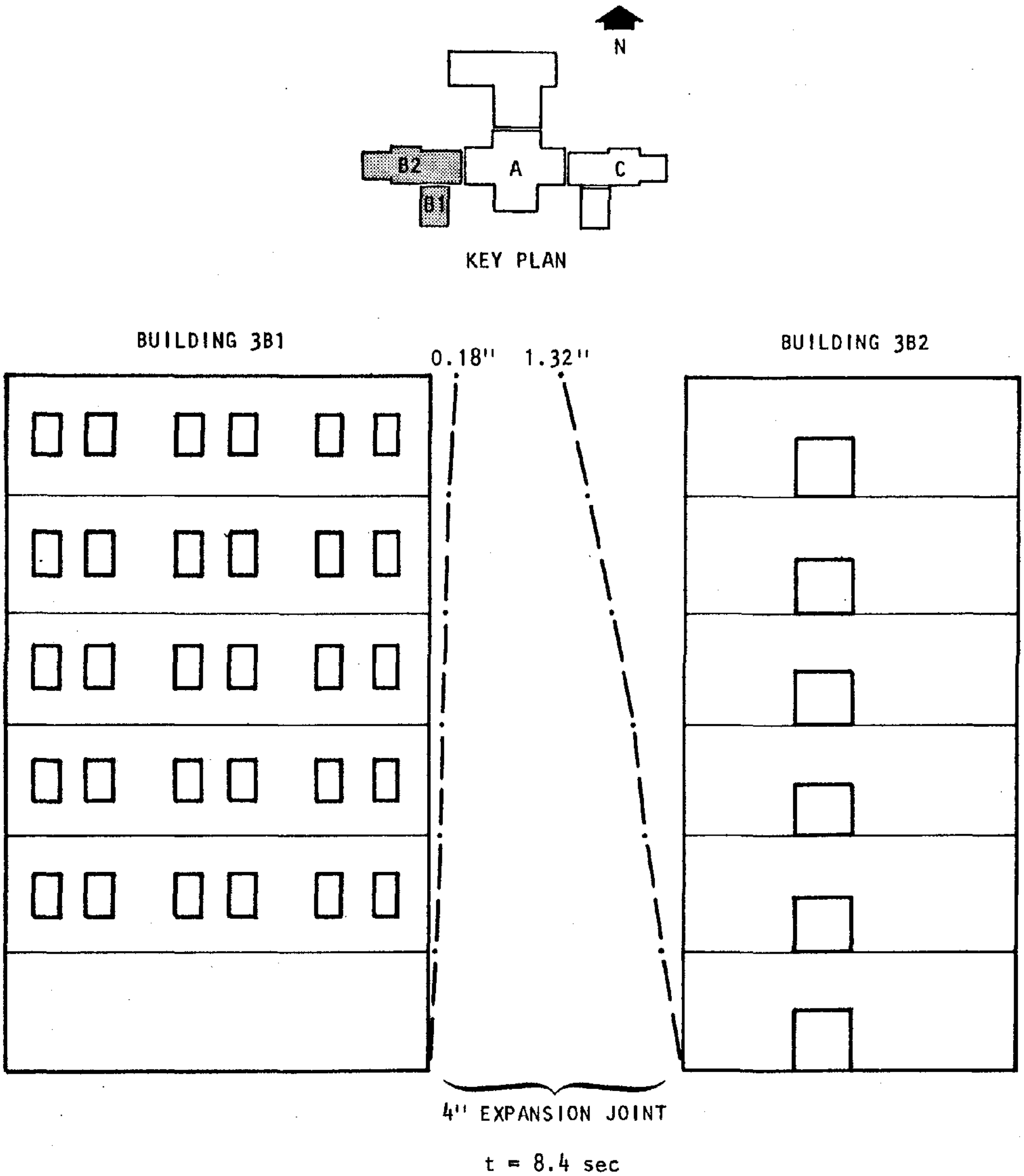



FIGURE 11. SCHEMATIC MAXIMUM RESPONSE FOR BUILDINGS 3B1 AND 3B2

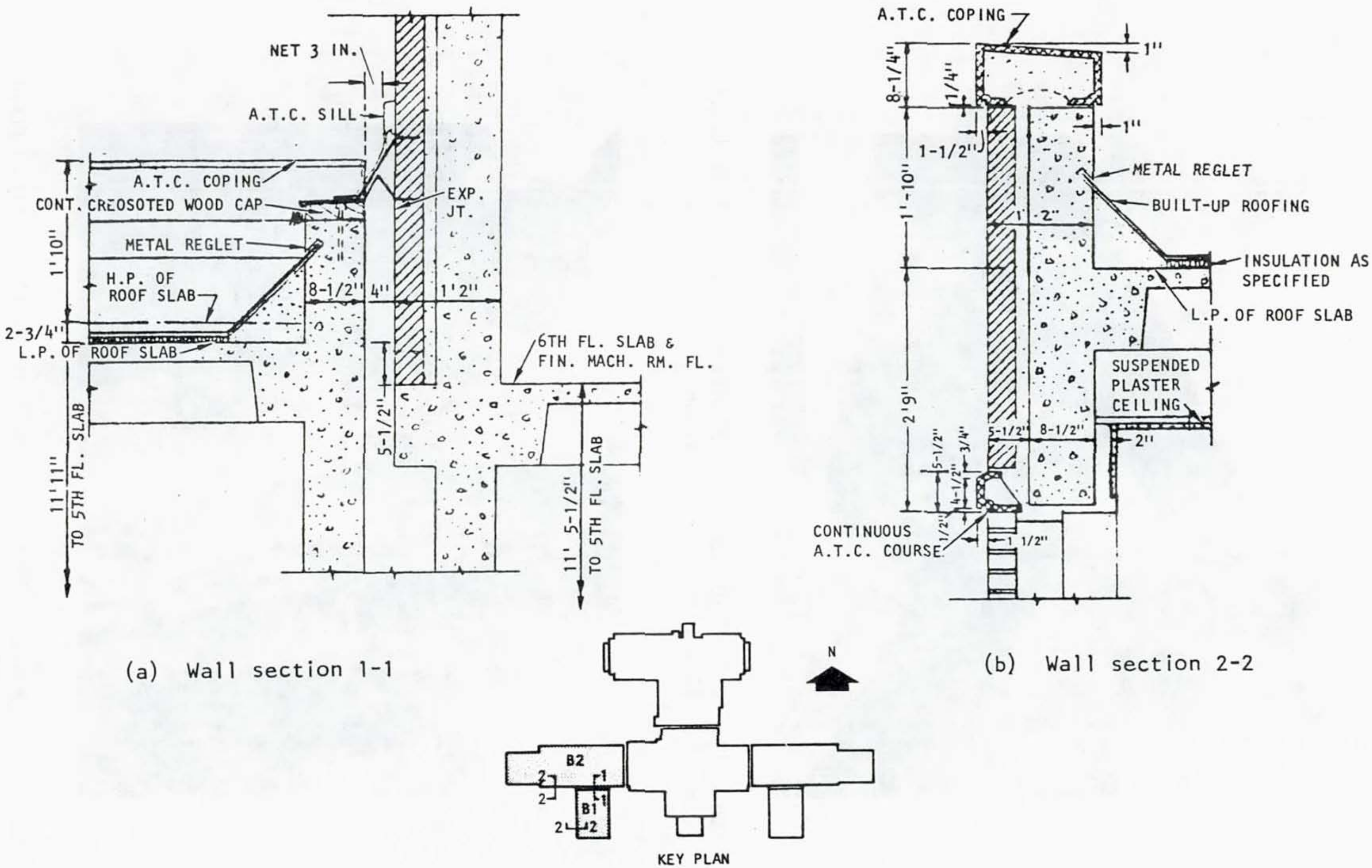



FIGURE 12. TYPCIAL WALL SECTIONS

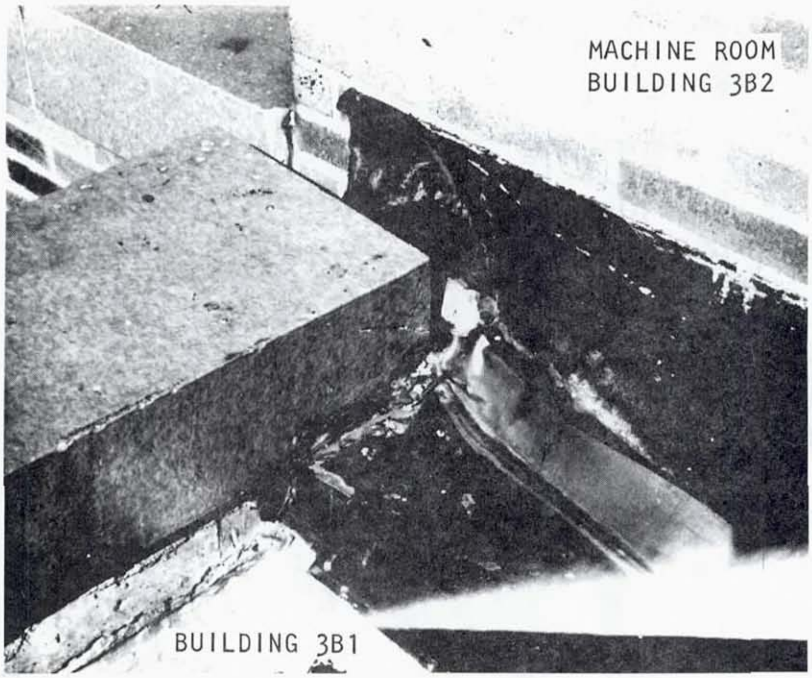



FIGURE 13. DAMAGE TO FLEXIBLE EXPANSION JOINT AT ROOF LEVEL (See Fig. 9 for location)

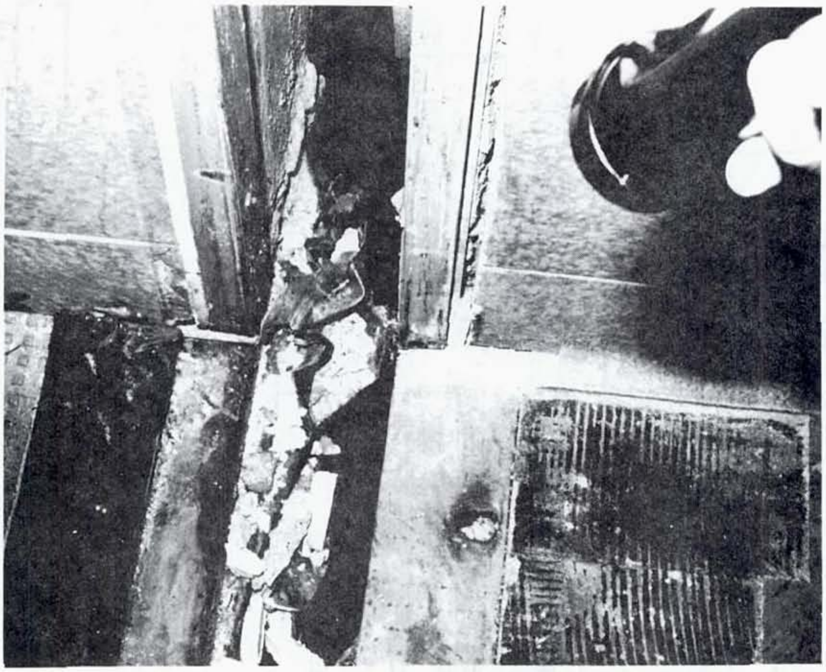


FIGURE 14. DAMAGE TO FLEXIBLE EXPANSION JOINT AT FIFTH FLOOR LEVEL OF BUILDING 3B

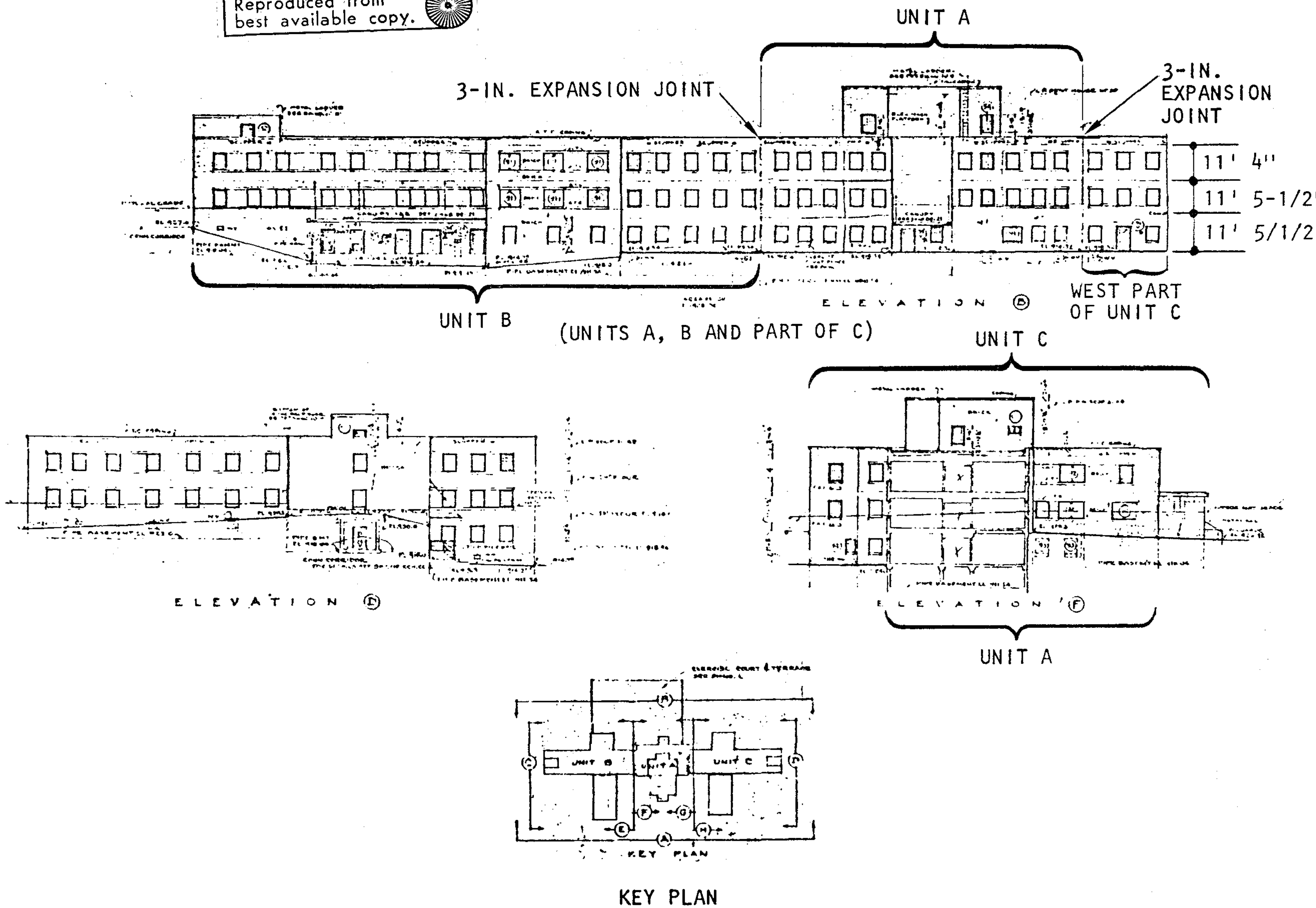



FIGURE 15. BUILDING 10: ELEVATION

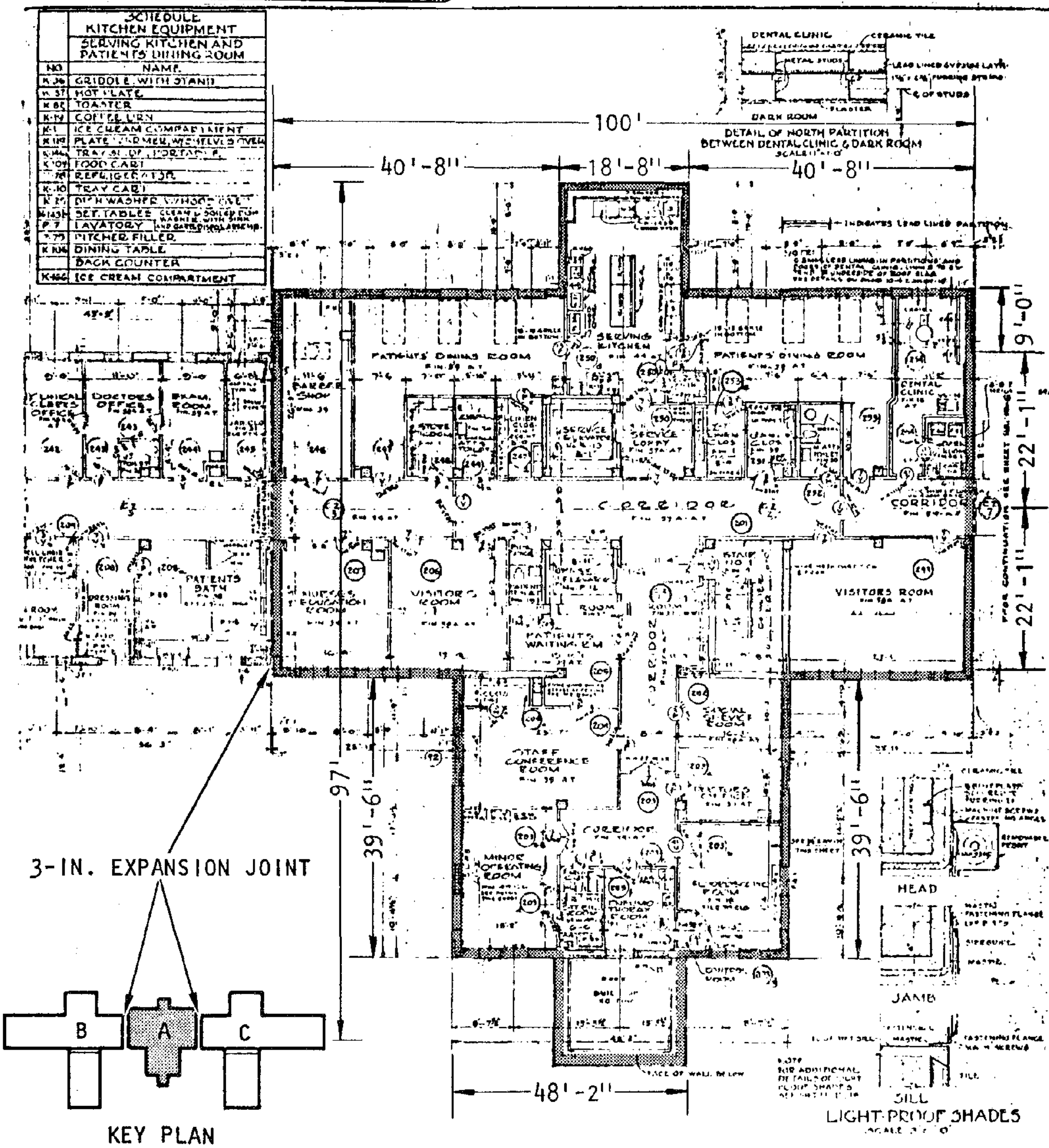



FIGURE 16. BUILDING 10: SECOND FLOOR PLAN — UNIT A

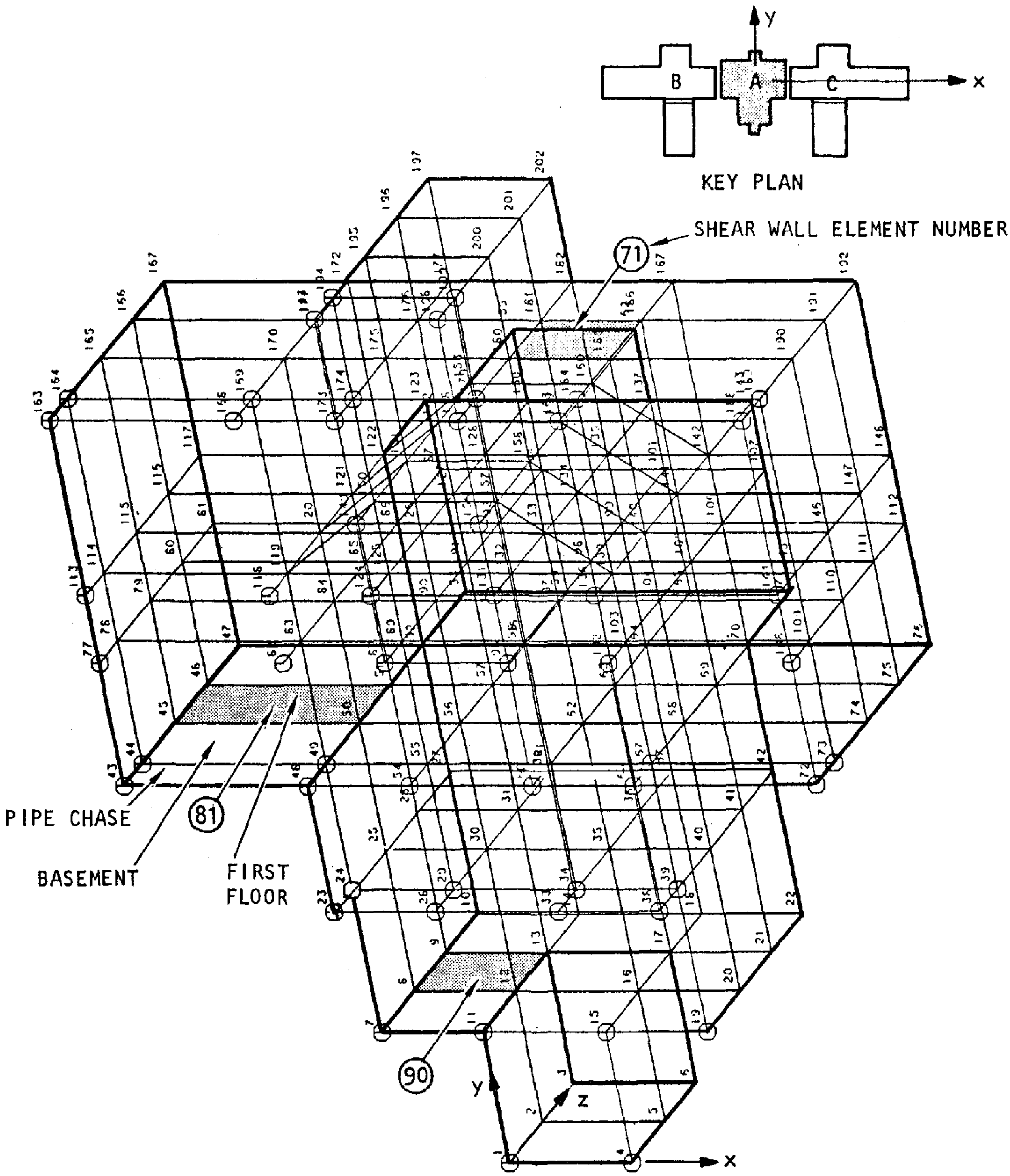



FIGURE 17. BUILDING 10A: FINITE ELEMENT MESH

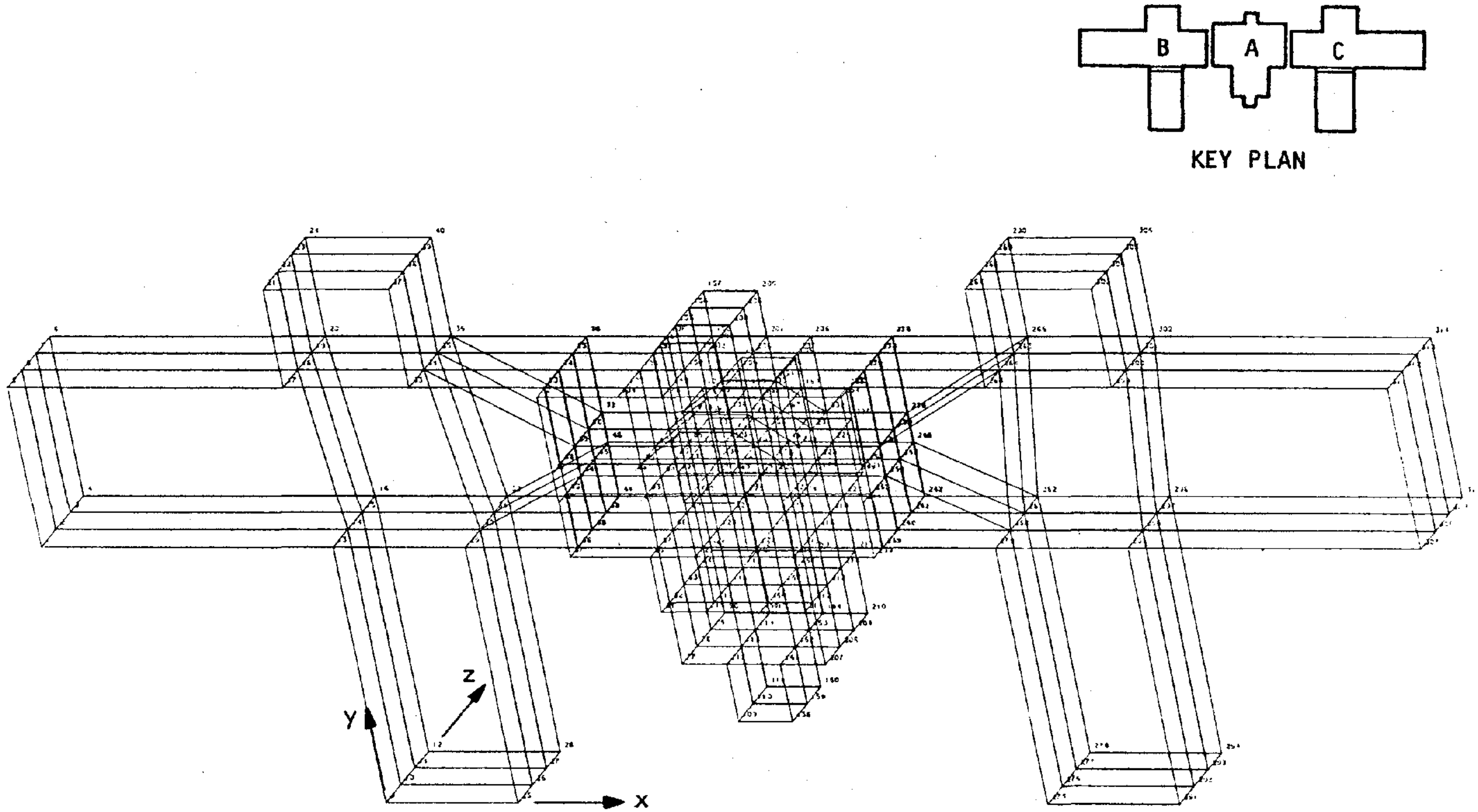



FIGURE 18. FINITE ELEMENT MODEL BUILDING 10 SEGMENTS A, B AND C TIED

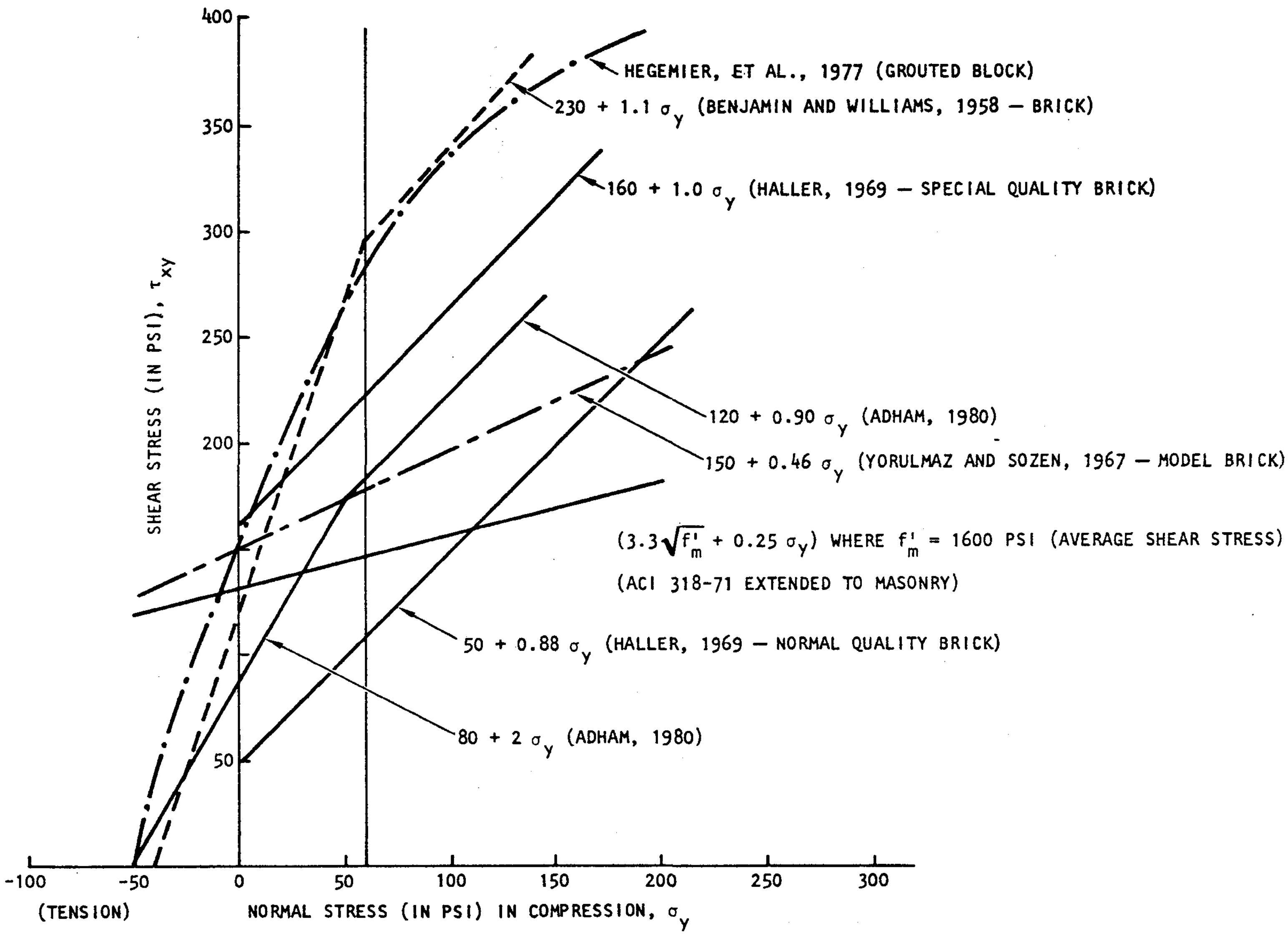



FIGURE 19. EFFECT OF NORMAL STRESS ON CRITICAL SHEAR STRESS

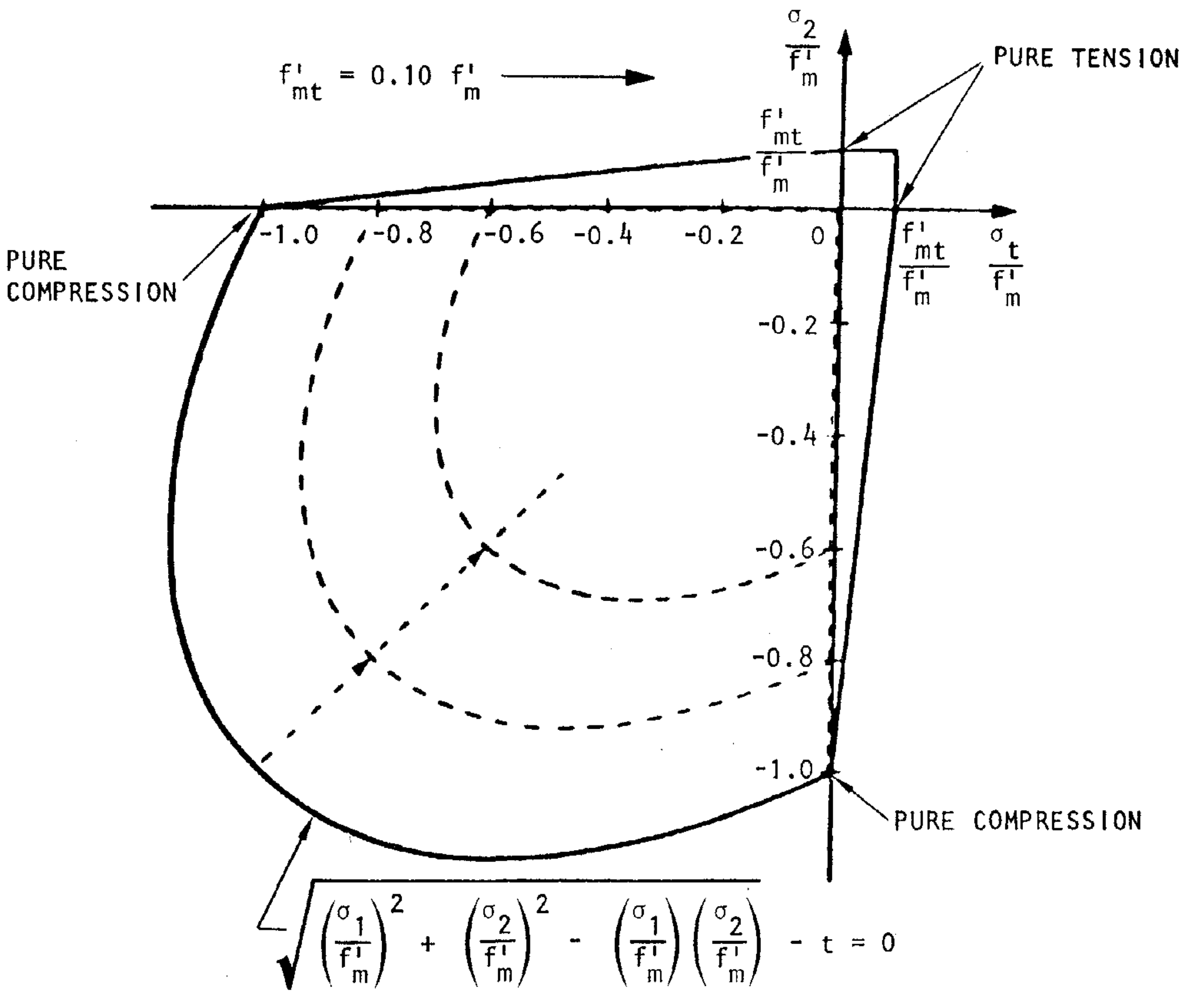
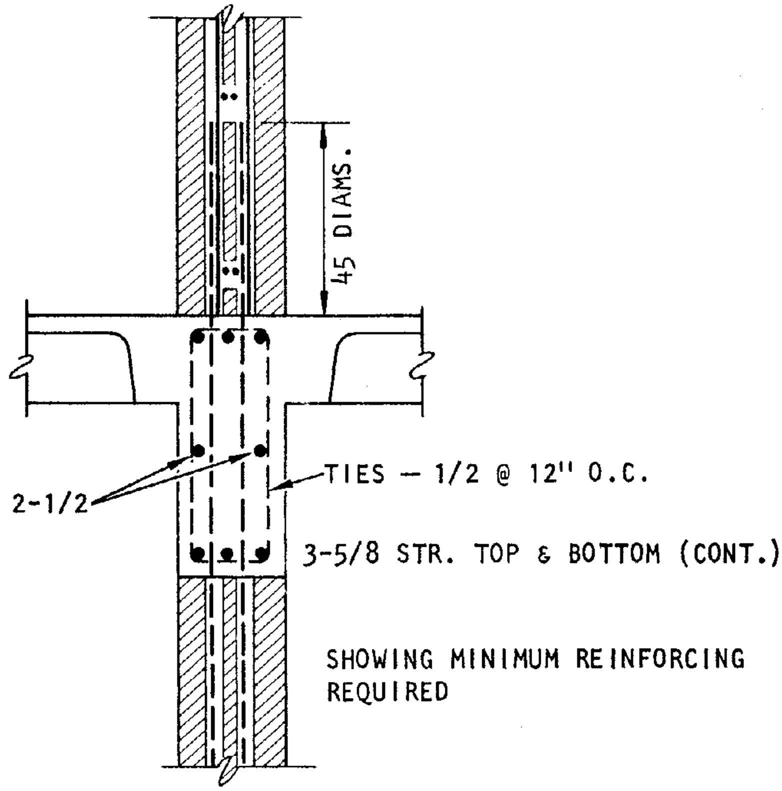
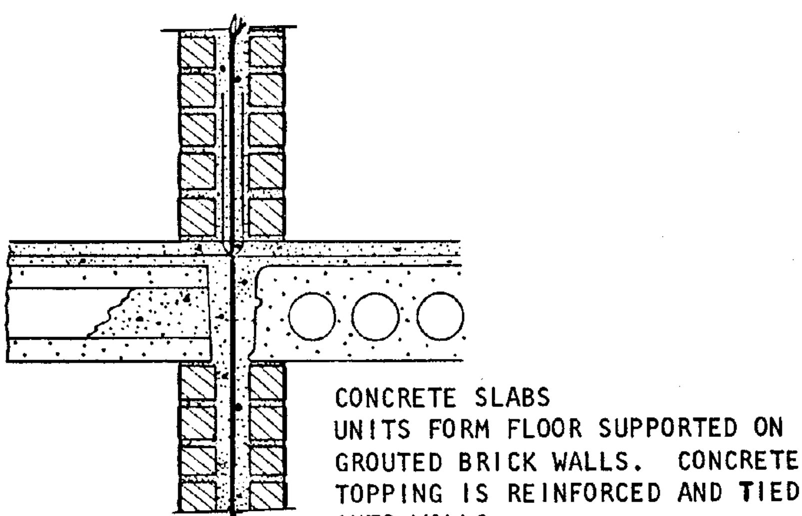



FIGURE 20. BIAXIAL FAILURE CRITERIA FOR BRICK MASONRY
(Adapted from Adham et al., 1975 and
Arya and Hegemier, 1978)

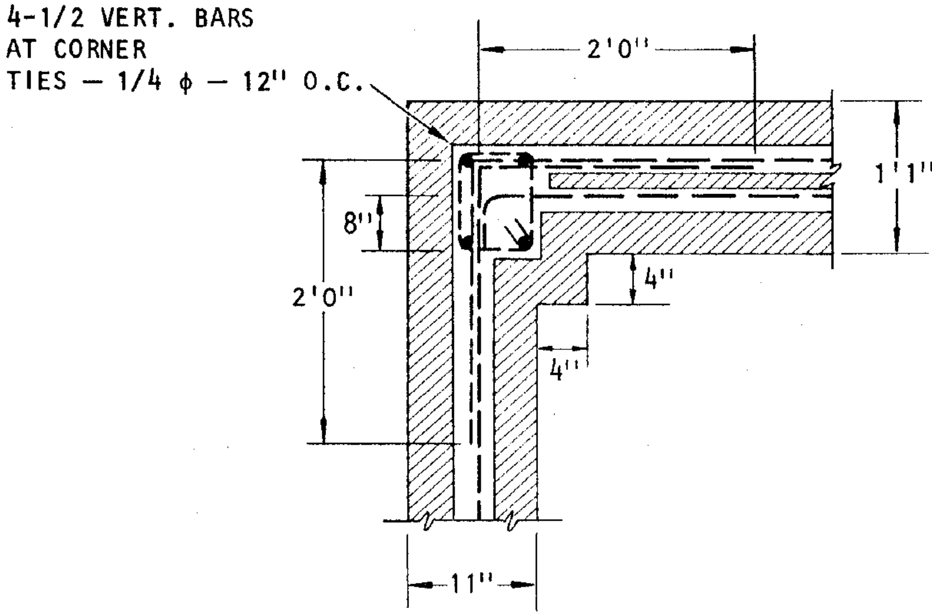



(a) Typical as built, 1954

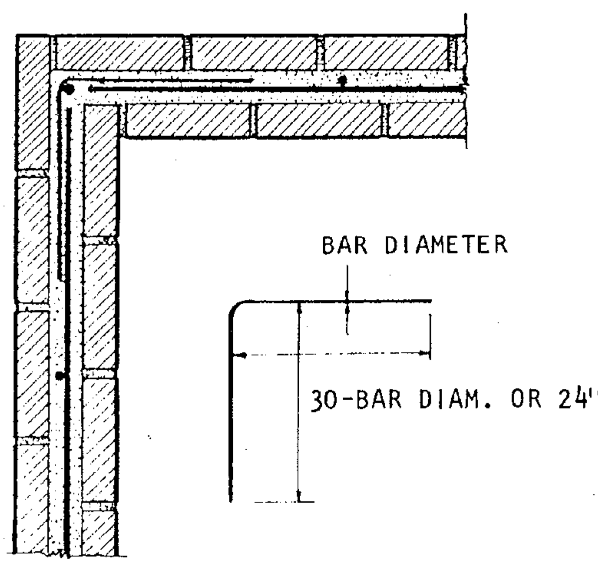



(b) Current recommended standard detail

FIGURE 21. WALL BEAM DETAIL (INTERIOR WALL)

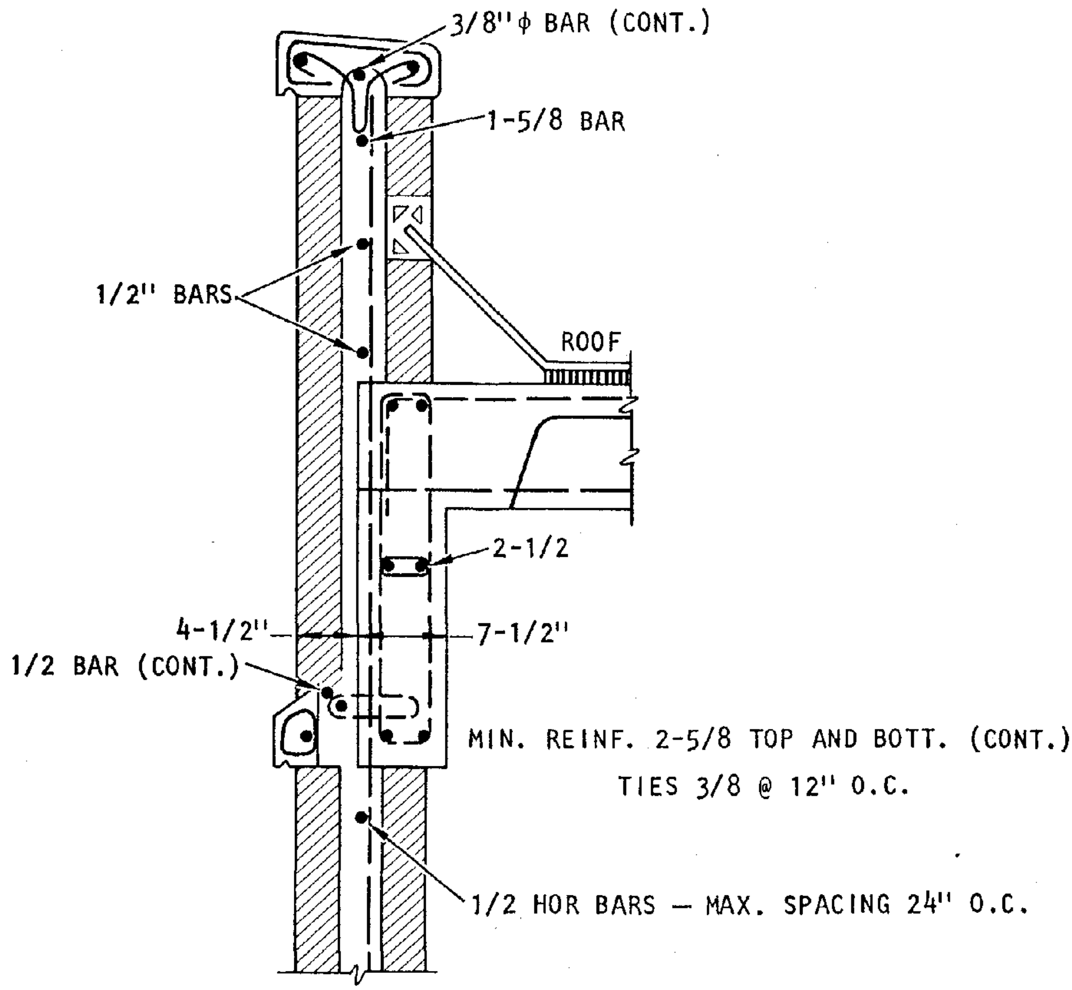



(a) Typical detail of corner piers (as built, 1954)

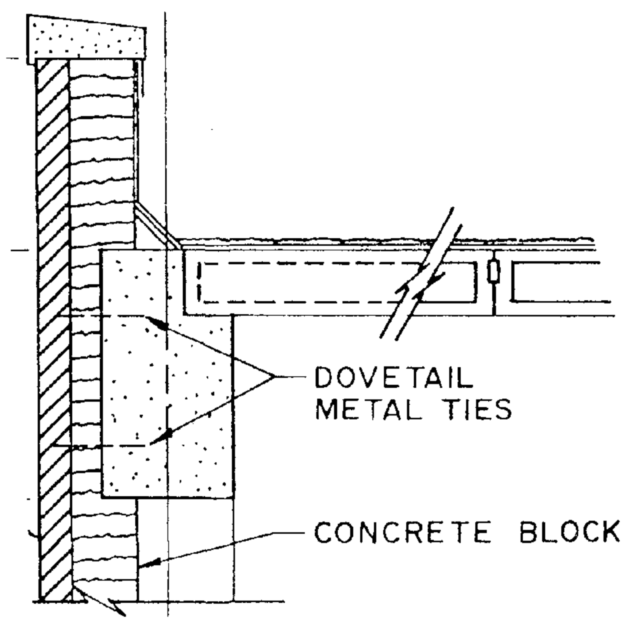



(b) Current standard detail of corner walls

FIGURE 22. DETAILS OF CORNER CONNECTIONS

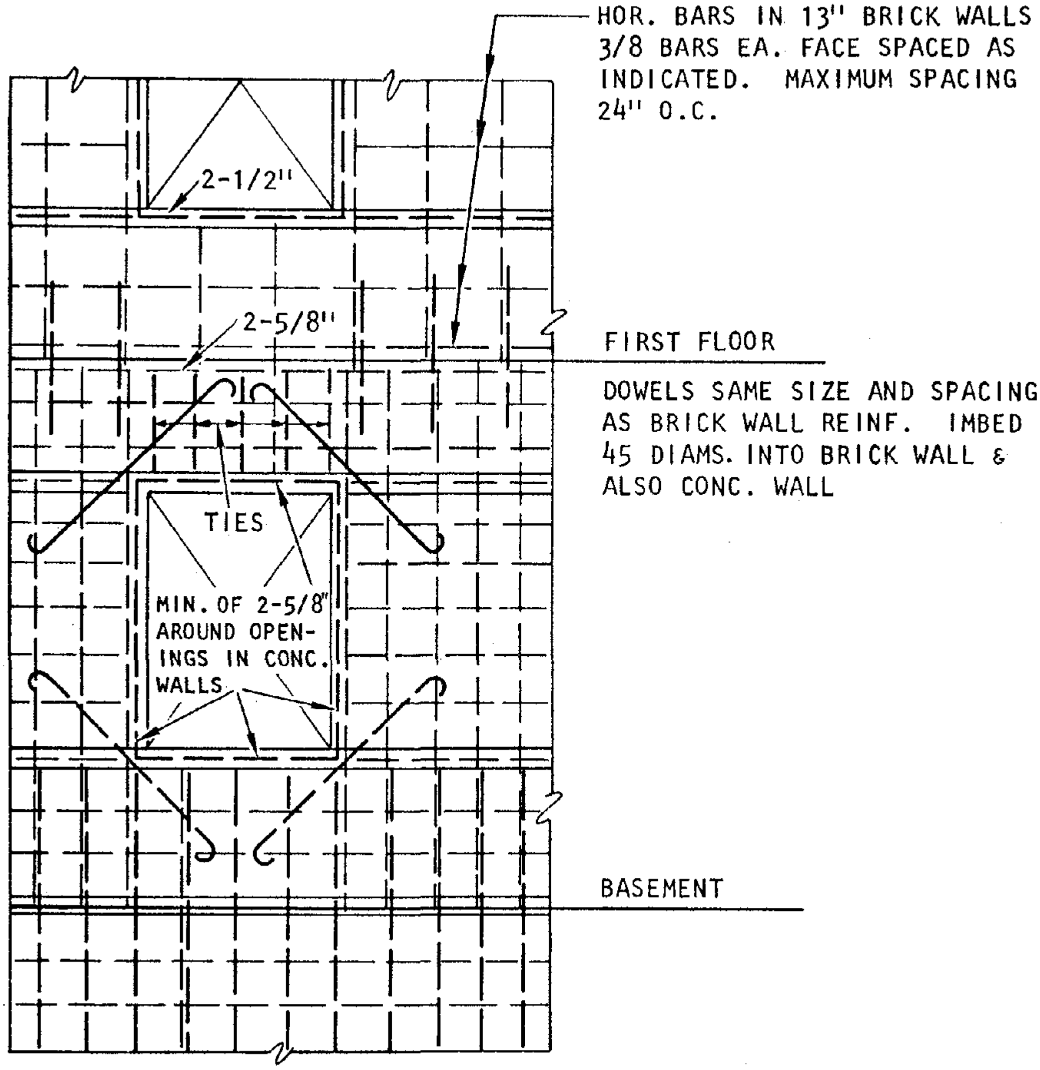



(a) Detail as built (1954)




(b) Current standard detail

FIGURE 23. PARAPET DETAIL




BARS AROUND WINDOW OR DOOR OPENINGS SHALL EXTEND AT LEAST 45 DIAMS., BUT NOT LESS THAN 24" BEYOND THE CORNER OF OPENINGS.
LAP BARS 45 DIAMS. AT SPLICES.

FIGURE 24. WALL REINFORCEMENT DETAIL (AS BUILT, 1954)

# AN EVALUATION OF NON-DESTRUCTIVE TEST METHODS APPLIED TO MASONRY

by: James L. Noland and R. H. Atkinson

ABSTRACT: The objective of the research is to assess the applicability of four non-destructive test (NDT) methods currently used for evaluation of soil, rock, and concrete to the NDT evaluation of masonry structures. The research was conceived in response to a national need to strengthen the nation's large inventory of existing unreinforced masonry buildings. If successful, NDT methods will complement and possibly replace assessment of structural capacity by destructive tests of specimens taken from buildings.

Four NDT methods to be evaluated are: hardness (Schmidt hammer), mechanical pulse velocity, ultrasonic pulse velocity, and dynamic response. Test specimens include thirty-six cantilever walls on which NDT measurements will be taken and approximately 100 companion small-scale assemblages which will be destructively tested to provide data for correlation. All specimens will be solid clay unit masonry for the purposes of the initial assessment of NDT methods.

# AN EVALUATION OF NON-DESTRUCTIVE TEST METHODS APPLIED TO MASONRY

By James L. Noland[1] and R. H. Atkinson[1]

## INTRODUCTION

At the present time, structural assessment of existing masonry buildings is based upon visual observations and data obtained from destructive test of small specimens taken from the structure being reviewed (6,15,23,24). These methods are limited because visual observations can only reveal gross defects, and testing of a sufficient number of specimens taken from a building to permit a comprehensive assessment may be prohibitive due to cost, time and aesthetic considerations.

This report describes a research project, funded by the National Science Foundation to assess the applicability of existing methods of non-destructive test (NDT) methods, which have been used successfully on other materials, to the NDT evaluation of masonry. Several advantages are possible if one or more NDT methods are found to be applicable, i.e.,

1) a more comprehensive and quicker examination of a structure,
2) a more economic evaluation for the same level of assessment, and
3) the structures would not be defaced.

Successful NDT methods could be a means of superior real-time quality control for new construction as well.

Possible disadvantages include:

1) equipment expense,
2) need for trained personnel, and
3) insufficient accuracy.

## BACKGROUND

Masonry buildings are designed to withstand loads which are predicted considering building function, location, geometry, and other factors. These loads are based upon current knowledge and represent the highest values to be reasonably expected. The loads are presented

---

[1]Consulting Engineer, Atkinson-Noland & Associates, Boulder, Colorado.

in the various codes and standards and are the minimum loads a building is expected to be able to withstand.

Technological advances and an increased data base in seismology, wind engineering, etc. have led to more accurate and often more severe design load requirements that have been previously stipulated. Hence, many older existing masonry buildings, designed to previously applicable codes and standards, and possibly deteriorated due to time and service, may not be safe.

Demolition and replacement of all existing masonry buildings not capable of satisfactory modern structural requirements is not economically feasible; the national inventory of such buildings is too great. Retrofit or limited use should be considered as a more realistic and economical solution.

## RESEARCH PLAN

Although other methods may be considered, four techniques currently used in the evaluation of rock, concrete, and soil properties will be investigated to assess their applicability to the structural evaluation of masonry. They are: the hardness method (rebound hammer), mechanical pulse velocity, ultrasonic pulse velocity, and dynamic response.

Tests will be conducted on laboratory-built large-scale unreinforced wall specimens of clay-unit masonry to establish response characteristics and the sensitivity of each method to detect variations in mortar properties, unit properties, and the presence of flaws. Companion small-scale specimens will be built and tested to enable correlations to be determined between non-destructive measurements and masonry properties.

Testing is restricted to one type of masonry, i.e., unreinforced, two-wythe, solid clay unit construction, because the project is basically exploratory in nature. The type chosen, however, is basically that of a large number of existing buildings.

Subsequent to NDT evaluation, destructive tests of the kind to be performed in the NSF funded project on methodology for mitigation of seismic hazards in unreinforced masonry buildings (24) will be performed on samples taken from the wall specimens. Relationships between these tests and NDT measurements will be studied.

## CANDIDATE METHODS

Prior use of the methods to be assessed for applicability to the investigation of masonry properties provides a substantial body of experience from which to begin (1,2,4,5,8,9,14,16,18,20). The principles behind each method and potential application to masonry are discussed in the following sections.

Schmidt Rebound Hammer.

The height of rebound of an impacting mass from a material is a function of the surface hardness of that material. The material characteristic "hardness" is derived for the elastic and strength properties of the material. The Shore scheroscope has been developed for hardness determination in metal materials while the Schmidt rebound hammer (16) was developed for concrete. Both have been successfully applied to the evaluation of rock properties (1).

The Schmidt rebound tester uses a spring activated hammer which is impacted against a steel plunger that is in contact with the test surface. The height of rebound of the hammer mass is taken as a measure of the surface hardness. The Schmidt hammer is commercially available in three energy levels (0.075 mkg, 0.225 mkg and 3.0 mkg) from a number of sources. The hammer weighs about 4 lbs, is portable and is easy to use in the field.

The rebound value obtained using the Schmidt hammer has been correlated to concrete compressive strength by a number of investigators, Malhotra (9). The calibration curve is not unique for all concrete mixes and thus a calibration test series is required if the hammer is to be used for quantitative compressive strength determinations. The Schmidt hammer is most useful in determining the uniformity of concrete in a structure and in comparing the quality of one concrete to another.

The Schmidt hammer has been adopted as one means of determining rock hardness by the International Society for Rock Mechanics (1). Schmidt hardness values of rock have been combined with abrasiveness test data and used successfully as a means to predict advance rates for tunnel boring machines (18).

The Schmidt rebound hammer was used by Deere and Miller (5) in their development of a series of index property tests that could be used to adequately predict the strength and deformation properties of intact rock. Using a Type L (0.075 mkg) hammer on laboratory sized specimens, good correlation was obtained between rebound number and both compressive strength and modulus of deformation. The Schmidt rebound number multiplied by the rock's dry unit weight had a correlation coefficient of = 0.943 with ultimate compressive strength and a coefficient = 0.876 with tangent modulus.

The nonhomogeneous nature of masonry will require that hardness values be obtained both for the clay or concrete unit and for the connecting mortar. One would expect significantly different hardness values to occur for these two materials and that correlation of hardness to overall masonry strength (or stiffness) to be a function of both values. This would reflect the contribution to masonry performance of these two components.

Use of the Schmidt hammer on masonry would require only that a relatively smooth area approximately 1/2 inch in diameter be prepared.

However, a special plunger adapter would have to be employed when thin mortar seams are to be tested. The Schmidt hammer is seen as providing information on the inherent strength or quality of the individual units and mortar that exist in a structure. Because of its limited zone of influence (probably 1 inch radius from the plunger face), this method of test may not be sensitive to effects of inter-wythe cracks, infill material, etc. on the performance of the overall masonry structure.

Mechanical Pulse Velocity Techniques.

The velocity of a stress wave in a material was shown by Rayleigh to be a function of the elastic constants of the material (E and $\mu$) and the material mass density, $\rho$. Three types of waves can be generated: the compression or p wave, the shear or s wave, and the Rayleigh or surface wave. Great use has been made of the compressive wave velocity as a non-destructive test method in concrete. Both the compression and the shear wave velocities are used to determine both the material quality and structural nature of soils and rocks that comprise building foundations.

In the mechanical pulse velocity technique a single mechanical pulse (wave) is generated in the test material and the velocity of its travel is measured. Typically the wave is generated by an energy source (hammer blow, piezoelectric crystal, explosive) with sensitive accelerometers used to measure wave arrival at particular points. Electronic timing equipment is used to determine the time elapsed between the energy input and wave arrival or between wave arrival at two points a known distance apart. The velocity and amplitude of the traveling wave in addition to being affected by E, $\mu$ and $\rho$ is affected by the presence of cracks and voids in the path of wave travel. In a nonhomogeneous material such as concrete or masonry, pulse velocity methods are anticipated to give information on "average" condition of the material including effects of internal flaws.

A summary of various studies relating compressive strength, flexural strength and static modulus to pulse velocity values is given by Whitehurst (20). The modulus determined from pulse techniques is the dynamic material modulus which is invariably higher (10% - 15%) than the static modulus. An adequate correlation can be established between the pulse velocity and compressive strength for a concrete in which the cement, aggregate and mix are constant. However, when previous correlation is not available, pulse velocity data alone cannot be considered to provide an adequate quantitative prediction of concrete strength. The pulse velocity method was also found by Deere and Miller (5) to be a satisfactory predictor of the deformation modulus of rock. A simple pulse or compressional wave was generated in the rock sample which was subjected to a low seating stress of approximately 150 psi. The measured dynamic wave velocity was found to have excellent reproducibility and when multiplied by the dry unit weight of the rock to be correlated to the static modulus of deformation with a coefficient = 0.929.

Pulse velocity techniques are often used to determine both the nature of foundation materials and their spatial distribution beneath building foundations. Techniques have been developed whereby both compressive and shear waves can be generated and the velocity and path of wave travel determined (14). The ability to generate and measure the shear wave velocity allows calculation of the shear modulus which is necessary for the determination of the building-foundation-soil response to seismic input motion.

Since the generated stress wave must pass through both the masonry unit and the mortar and will be influenced by both, measurement of wave velocity and wave amplitude attenuation may provide information on the overall quality of the masonry. When adequate bond between the unit and a high strength mortar is present, unimpeded wave transmission should occur. However, it would be expected that the presence of cracks, either gross visible cracks or very thin bond cracks, or a very soft or weak mortar would result in significant retardation or reflection of the wave and in significant attenuation of amplitude. As the placement of the pickup accelerometers and the impacting point can be easily changed, a masonry wall can be surveyed in vertical, horizontal and diagonal directions at numerous locations in a structure. Where access to only one side of the masonry is possible, the wave velocity measured will be for wave travel in the outer wythe only. Pickup transducers placed on opposite sides of a wall will measure travel time through a multiwythe wall, thus influence of inner wythes and collar joints can be determined.

The presence of reinforcing steel in concrete was found by Breuning and Bone (4) to occasionally influence readings in concrete. Reinforcing steel in modern masonry buildings could have the same influence which is to produce abnormally high velocity values when the direction of propagation is parallel to the bars. This effect would require study so that correction factors may be established if appropriate to do so.

Pulse velocity techniques in geomechanics include the generation, propagation and measurement of shear waves in soils and rocks. From the measured shear wave velocity the effective dynamic shear modulus of the material can be determined. Similar techniques, if developed and applied to masonry, would yield information on the shear modulus of the intact masonry material. The shear wave is detected by an accelerometer that is primarily sensitive to transverse motions. When the accelerometer signal is displayed on a CRT device, the arrival of the slower shear wave can be distinguished from the faster traveling P wave.

Equipment to measure mechanical pulse velocity consists of a hammer with a contact switch and a sensitive accelerometer or accelerometers. The elapsed time of wave travel between the accelerometer can be measured by an electronic counting device or both signals can be displayed on a dual channel oscilloscope with the time being measured on the CRT grid. Available devices used for soil and rocks studies provide a transit time resolution as close as $1 \times 10^{-6}$ seconds with a maximum time interval of 1 millisecond.

Ultrasonic Pulse Velocity Method.

In theory the use of ultrasonic frequency stress wave as a non-destructive test method is based on the same principles as the use of the mechanical pulse technique. A stress wave of high frequency (20 kHz to 200 kHz) is generated in the specimen by one transducer. A second transducer acts as a pickup with the elapsed time of wave travel measured electronically. Use of the higher frequencies are in general limited to smaller specimens and wall thicknesses.

The path length between transducers divided by the measured travel time gives the average velocity of wave propagation. Path length and transit times can generally be measured to an accuracy of 1 percent. Because the velocity of the pulses is almost independent of the geometry of the material through which they pass and depends only on elastic parameters, pulse velocity has been a very desirable technique for investigating structural concrete both in the laboratory and field. A comprehensive review of the use of ultrasonic techniques for non-destructive testing in concrete is given by Whitehurst (20) and by Malhotra (9).

The use of ultrasonic velocity techniques (20 kHz) to detect the presence and to determine the extent of cracks and deteriorated concrete was studied by Bruening and Bone (4). The effect of a crack is to decrease both the velocity and amplitude of the stress wave. By using several different stress paths, patterns of cracking and deterioration were determined in a bridge pier.

The presence of reinforcing steel in concrete affects the pulse velocity measurements since pulse velocity in steel is 1.2 to 1.9 times higher than velocity in concrete. Paths may be chosen which avoid the influence of reinforcement. When this is not possible, measured values have to be corrected by taking into account the proximity of the pulse path to the steel, the quantity and orientation of steel, and pulse velocity in concrete (9).

The foregoing effect would have to be investigated for reinforced masonry so that the influence of reinforcement over a range of parameters can be established. Since the location of reinforcement may not be known, procedures and equipment for identification of reinforcing steel location in concrete would have to be applied to masonry.

Ultrasonic test equipment is available from several commercial sources. A typical modern piece of equipment is a small battery powered portable device that can be adapted to transducers ranging from 36 kHz to 200 kHz. This device provides a direct digital reading of wave travel time and provision for CRT display of the wave.

Vibration Techniques.

A vibration procedure has been applied to masonry structures, Medearis (10,11). Microvibrations generated in the structure by wind, vehicular traffic, etc. are measured using sensitive seismic equipment. The dynamic characteristics determined experimentally are compared to those computed theoretically. For simple geometries, and known elastic modulus and Poisson's ratios, the natural frequencies can be computed using available theoretical relationships. For large or complex structures, a finite element model may be utilized to determine the dynamic characteristics of the structure. In either case, the equivalence of the experimental and calculated dynamic behavior becomes a basis for evaluating the masonry properties.

The microvibration approach would be advantageous since no forcing equipment is necessary. It has been reported (19) that the approach produces results essentially similar to high amplitude motion and, as previously noted, has been applied to large masonry structures. However, large amplitude motions will also be induced in the masonry wall specimens to provide a means for verification of the microvibration approach.

## RESEARCH TASKS

The project consists of two major components. Laboratory Investigation and Evaluation which are discussed in the following sections.

Laboratory Investigations.

Laboratory investigations and associated analyses are planned to:

1. Develop experimental procedures and select or modify equipment to maximize consistency of measurement as required for each NDT method.

2. Assess the capabilities of each NDT method to detect differences in masonry due to variations of mortar, units, and construction.

3. Assess the capabilities of each NDT method to detect flaws in masonry.

Each NDT method will be applied to cantilever masonry wall specimens built of various combinations of mortar mixes and clay unit strengths. The mortar mixes and unit strengths used will enable the NDT methods to be evaluated over a full range of solid clay unit masonry strength/stiffness parameters.

Companion small-scale subassemblages[1] will be built and tested to destruction to establish the strength and stiffness characteristics of each wall unit-mortar mix type. Three wall specimens and three of each subassemblage are planned[2].

Subsequent to NDT tests, two of the destructive tests proposed by ABK (24) will be performed. This will enable correlations between the results of these tests, results of NDT methods, and results on the prepared subassemblages to be established. Information generated will be forwarded to the ABK project. The ABK test specimens/methods are depicted in Figures 1 and 2.

Each well-constructed wall specimen will be tested using each of the NDT methods. The specimens will be tested at several time points after initial set of the mortar in order to detect the influence of curing. Initially, for the purpose of establishing or verifying procedures and to identify equipment needs and modifications the following factors will be considered:

1) sensor placement/orientation,

2) input signal and/or displacement amplitude,

3) surface preparation (as applicable),

4) equipment sensitivity, and

5) repeatibility of data.

Further, wave velocity methods will be evaluated for propagation:

1) vertically,

2) horizontally,

3) diagonally, and

4) through the wall.

Subsequently, NDT will be performed on flawed wall specimens. Flaws, e.g., joint delamination and mortarless joints, will be introduced to the well-constructed walls previously used in increasing amounts to establish NDT sensitivity and response change. The evaluation of the delaminated collar joint type of flaw will require that

---

[1]Subassemblages include compression prisms and shear-bond prisms built and tested in accordance with procedures used in prior programs (3,7,12,13), and flexural specimens in accordance with the applicable ASTM standard.

[2]The number of specimens of each kind to be built is an estimate, based upon prior experience, of the minimum number required to establish reasonable statistical confidence and is subject to revision after the degree of data scatter is observed.

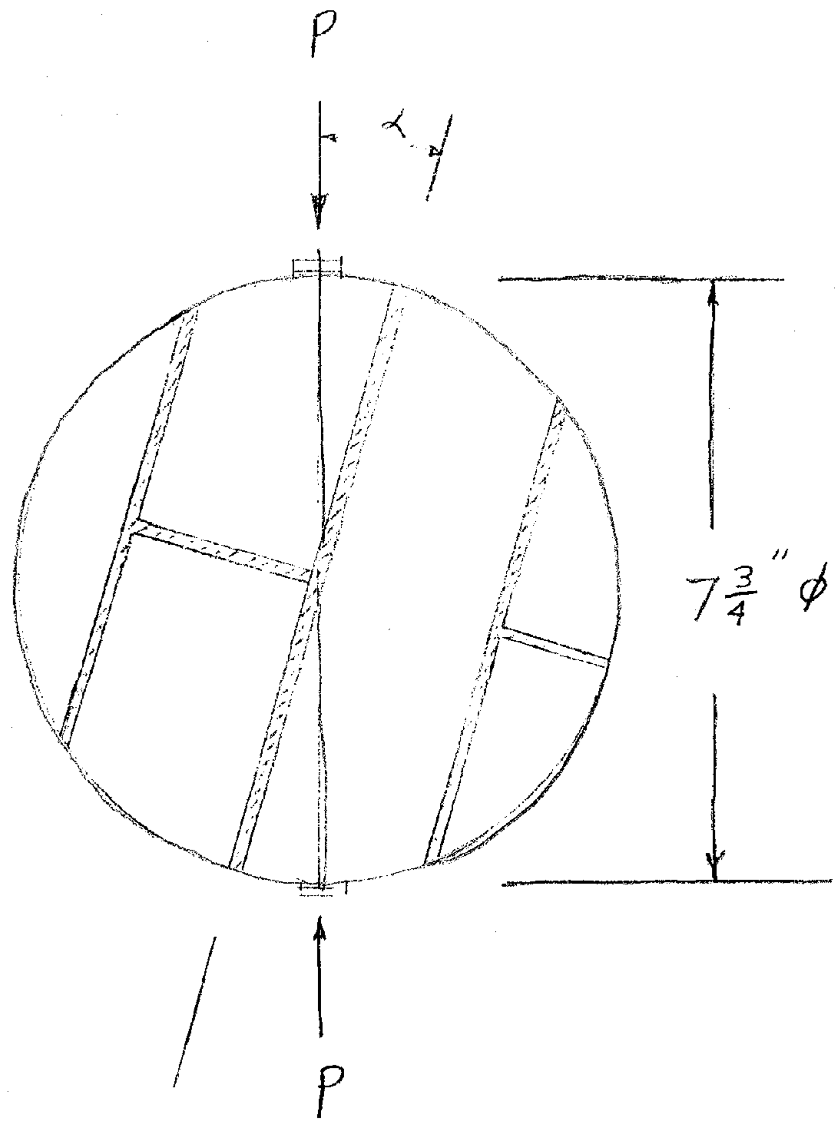



Figure 1. Joint Shear Test

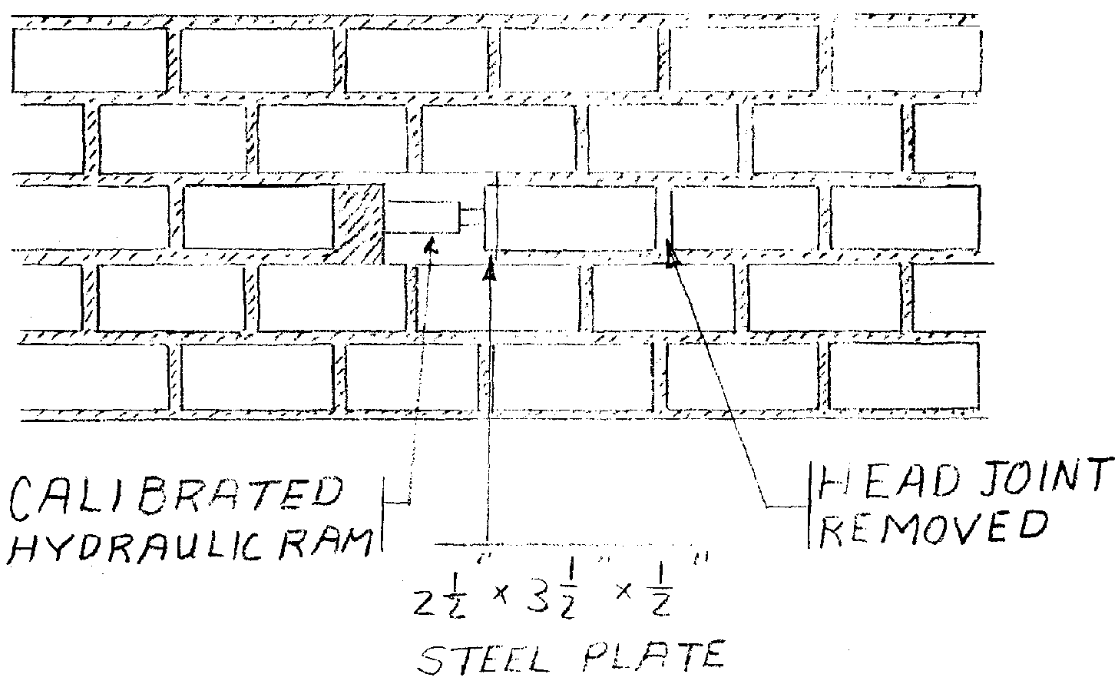



Figure 2. Joint Shear Test

special wall specimens be built to assure that flaws of known degree are present. The factors considered with respect to the well-constructed specimens will also be considered for the case of flawed specimens.

Each unique combination of unit and mortar is defined herein to be a masonry "type". In order to systematically establish the effect of the primary variables upon NDT response, specimens will be built representing the many combinations, for each series, of the variables presented in Table 1. There will be 12 such combinations, i.e., the number of masonry "types". Three specimens and three subassemblages (three each of compression, shear bond, and flexural subassemblages) are planned which leads to a target total of 36 well-constructed wall specimens and 108 subassemblages to be built. In addition, three wall specimens containing collar joint flaws will be constructed.

Table 1

MASONRY WALL SPECIMENS

| | |
|---|---|
| Wall Type: | Two wythe, solid clay unit with collar joint |
| Unit Strength[1]: | 3000, 7000, 10,000 psi |
| Mortar Mix[2]: | 1:¼:3, 1:½:4½, 1:1:6, 1:2:9 |
| Flaws: | Delaminated bed joints<br>Mortarless bed joints<br>Delaminated collar joint |

The wall specimens will be built upon footing sized to provide a fixed-base boundary condition for the vibration tests. A 35-day cycle is anticipated for each wall specimen test sequence, associated subassemblage testing, and other activities.

Specific NDT equipment items will be selected for use based upon market availability, ease of operation, and cost. Equipment will be selected which is representative, to the extent possible, of generic type, i.e., highly unique or single-source equipment will be avoided, if possible. Equipment to be used includes:

1) Schmidt hammer,

2) oscilloscope (dual trace with camera),

3) accelerometers and mechanical probe timer,

4) geophones,

5) ultrasonic testor,

---

[1]As determined by the flatwise compression test.

[2]Proportions by volume of portland cement, lime, and sand.

6) seismic recorder with transducers,

7) 8 inch $\phi$ core drill, and

8) miscellaneous other.

Evaluation.

From the construction, strength testing and NDT evaluation of each test wall, the following data will be acquired:

Wall Identification

Wall Characteristics

- Mortar strength data
- Unit strength data
- Prism strength (28 days)
- Prism modulus (28 days)
- Joint shear strength (28 days) using prepared sub-assemblage
- Joint shear strength (29-30 days) using ABK procedures (see Figure 1)

NDT Data (data to be acquired at 7, 14, 21 and 28 days after wall construction)

- Schmidt hardness data: mortar
  unit
- Dynamic response
- Mechanical pulse velocity: vertical direction
  horizontal direction
  $45^{\circ}$ angle to bed joint
  through wall
  shear velocity
- Ultrasonic velocity: unit velocity
  vertical velocity
  horizontal velocity
  $45^{\circ}$ angle to bed joint
  through wall

The use of NDT procedures will allow a significant number of experimental determinations to be made for each measurement. The exact number can only be determined after development and evaluation of each test procedure. Typically, however, 10 to 25 readings might be taken for a single test, e.g. ultrasonic velocity in a vertical direction on a specific wall type at 14 days age. The mean, standard deviation as well as any non-normal tendency of data will be determined for each set of readings. These values will be entered in the master data file.

The use of standard statistical routines will provide a means to determine what relationships and their strengths that exist between NDT observations and the measured strength and stiffness of the masonry types. Single and multiple regression analysis will be the primary means to establish the strength of relationships.

A data base file structure will permit regression analyses to determine correlation factor both for the entire data base and for selected groups within the data base.

Obviously, many combinations of possible relations are possible. A major task of the analysis phase will be to ferret out the meaningful relationships that exist between the NDT tests and the strength and stiffness data. Such relationships should have the following criteria:

- A sufficiently high coefficient of correlation.
- A confidence interval (at say a 90% probability) sufficient to provide adequate prediction of the engineering or design quantity desired.
- The minimum number of factors consistent with statistical accuracy.

## EXPECTED OUTCOME

The following are the specific results expected from the project:

1) Identification of the equipment required.

2) Experimental procedures.

3) Capabilities of each NDT method to detect the effect of mortar and unit property variations.

4) Capabilities of each NDT method to detect flaws.

5) Response data for each NDT method and expressions describing the relationships between NDT response, masonry parameter variation, and flaw degree.

6) Recommendations for further research, if appropriate, to extend the methodology to all masonry types and in-situ construction.

## REFERENCES


1. Atkinson, R.H., Suggested Methods for Determination of Hardness, Document No. 9, International Society for Rock Mechanics, Commission on Standardization of Laboratory and Field Test, 1975.

2. Barton, N. and Chovbey, V., "The Shear Strength of Rock Joints in Theory and Practice," v. 10, no. 1-2, Rock Mechanics, December 1977.

3. Baur, J., Noland, J., and Chinn, J., "Compression and Tension Bond Strength of Small-Scale Masonry Specimens," Proceedings of the North American Masonry Conference, Boulder, Colorado, August 1978.

4. Breuning, S.M. and Bone, A.J., "Soniscope Applied to Maintenance of Concrete Structures," Proceedings, 33rd Annual Meeting, Highway Research Board, pp. 210-216, 1954.

5. Deere, D.U. and Miller, R.P., "Engineering Classification and Index Projects for Intact Rocks," Air Force Weapons Laboratory Technical Report No. AFWL-TR-65-116, December 1966.

6. Fattel, S.G. and Cattaneo, L., Evaluation of Structural Properties of Masonry in Existing Buildings, NBSIR 74-520, Center for Building Technology, National Bureau of Standards, Washington, D.C.

7. Hanada, K. and Noland, J., "Effects of Height and End Lateral Restraints on Clay-Unit Prisms," Proceedings of the North American Masonry Conference, Boulder, Colorado, August 1978.

8. Hucka, U., "A Rapid Method of Determining the Strength of Rocks In-Situ," Int. J. Rock Mech. Mining Science, Vol. 2, pp. 127-134, 1965.

9. Malhotra, V.M., Testing Hardened Concrete: Non-Destructive Methods, Iowa State University Press and American Concrete Institute, 1976.

10. Medearis, K., An Investigation of the Dynamic Response of the Park Lane Towers to Earthquake Loadings, Report to the Colorado Masonry Institute, Denver, Colorado, 1974.

11. Medearis, K., An Investigation of the Strength of Masonry Walls Using Vibration Techniques, Report to the Colorado Masonry Institute, Denver, Colorado, 1976.

12. Miller, D., "Effects of Mortar and End Conditions on the Compressive Strength of Hollow Clay Unit Prisms," M.S. thesis, University of Colorado, (in progress).

13. Nuss, L., Noland, J., and Chinn, J., "The Parameters Influencing Shear Strength Between Clay Masonry Units and Mortar," Proceedings of the North American Masonry Conference, Boulder, Colorado, August 1978.

14. Power, J.H. and Real, C.R., "Shear Wave Velocity Propagation and Measurement," California Geology, Vol. 29, No. 2, February 1976.

15. Schmid, B., Kariotis, J., Schwartz, E., Tentative Los Angeles Ordinance and Testing Program for Unreinforced Masonry Buildings, 1978.

16. Schmidt, E., "The Concrete Scherometer," Proceedings, International Symposium on Nondestructive Testing on Materials and Structure, Vol. 2, RILEM, Paris, pp. 310-319, 1954.

17. Snell, L.M., "Non-Destructive Testing Technique to Evaluate Existing Masonry Construction," Proceedings of the North American Masonry Conference, Boulder, Colorado, August 1978.

18. Tarkoy, P.J., "Predicting Tunnel Boring Machine Penetration Rates in Selected Rock Types," Ninth Canadian Symposium on Rock Mechanics, Montreal, December 1973.

19. Trifunac, M., "Wind and Microtremor Induced Vibrations of a Twenty-Two Story Steel Frame Building," California Institute of Technology EERL Report, Pasadena, California, 1970.

20. Whitehurst, E.A., "Evaluation of Concrete Properties from Sonic Tests," American Concrete Institute Monograph No. 2, 1966.

21. __________, "Earthquake Resistant Masonry Construction: National Workshop," R. Crist, ed., NBS Building Science Series 106, U.S. Department of Commerce, National Bureau of Standards, Washington, D.C., 1977.

22. __________, Building Code Requirements for Masonry Construction, Colorado Masonry Institute Standard 301-76, Denver, Colorado, June 1976.

23. __________, Earthquake Hazard Reduction in Existing Buildings, Ordinance awaiting approval by Los Angeles City Council.

24. __________, "Methodology for Mitigation of Seismic Hazards in Existing Unreinforced Masonry Buildings: Phase II," Proposal by ABK, A Joint Venture, El Segundo, California, April, 1978.

# METHODOLOGY FOR MITIGATION OF SEISMIC HAZARDS IN EXISTING UNREINFORCED MASONRY BUILDINGS

by: John C. Kariotis[1], Robert D. Ewing[2] and Albin W. Johnson[3]

ABSTRACT: A methodology for the mitigation of seismic hazards in existing unreinforced masonry (URM) buildings on a nationwide basis is being developed as part of an ongoing two-year study program, sponsored by the National Science Foundation. The resulting methodology will be applicable to a broad range of buildings and will provide analysis methods and procedures for determining (1) seismic hazard and seismic input, (2) physical properties of URM, (3) requirements for hazard mitigation, and (4) methods of retrofit and strengthening when these needs have been established. Cost effectiveness is an important consideration.

The development of the methodology is based on field surveys that categorized existing buildings and combined analytical and experimental investigations. Analytical and experimental investigations used both computer and physical models excited by selected time histories.

Analysis verification by dynamic testing is discussed. Specimens for dynamic and static testing are described and procedures are outlined. Results to date of methodology development are presented. Experimental difficulties encountered in dynamic testing are discussed. Topics for ongoing research in masonry are presented.

KEY WORDS: unreinforced masonry; seismic hazard; dynamic testing; time histories; retrofit; mitigation

---

[1]Principal, Kariotis & Associates, Structural Engineers.

[2]Vice President - Computer Sciences, Agbabian Associates, Engineers and Consultants.

[3]Associate, S. B. Barnes & Associates, Structural Engineers.

# METHODOLOGY FOR MITIGATION OF SEISMIC HAZARDS IN EXISTING UNREINFORCED MASONRY BUILDINGS

by: John C. Kariotis[1], Robert D. Ewing[2] and Albin W. Johnson[3]

## IDENTIFICATION OF THE PROBLEM

Building construction using unreinforced masonry (URM) predated the development of the seismic criteria that guide the design and construction of present-day buildings. Many of these URM buildings are still being used in areas considered seismically active, even though investigation of earthquake damage has confirmed that this type of building has been a major contributor to loss of life during earthquakes. Yet the cost of rehabilitating URM buildings to new-construction standards is usually unacceptable.

Today, public agencies and the private sector are becoming more concerned about the potential for personal injury or death resulting from failure of these buildings. On the other hand, political jurisdications struggling with limited budgets can rarely afford the extensive research programs required to develop rehabilitation standards.

It seems apparent that a system of analysis methods and procedures is needed for determining realistic hazard-mitigation requirements and cost-effective methods of retrofit to fill such requirements. Research that can provide communities with usable tools to meet these goals have a major impact on cities squeezed between the threat to life safety and the economic constraints. The developed methodology and standards could reduce the enormous investment now required to make existing buildings conform to standards for new construction, or eliminate the economic loss ensuing from demolition.

## PROGRAM FOR PROBLEM SOLUTION

In 1977, the National Science Foundation (NSF) initiated a multi-phased program for the mitigation of seismic hazards. Three of the responding firms were convinced that methods and techniques could be developed for economical rehabilitation of URM buildings. With the concurrence of NSF, the three firms launched separate but interactive studies under Phase I funding to determine whether an applicable methodology was feasible (2, 4, 5). Phase I results were promising.

As a result, these three companies, Agbabian Associates, Steve B. Barnes and Associates, and Kariotis, Kesler & Allys, formed, in 1978, a

---

[1]Principal, Kariotis & Associates, Structural Engineers.

[2]Vice President - Computer Sciences, Agbabian Associates, Engineers and Consultants.

[3]Associate, S. B. Barnes & Associates, Structural Engineers.

joint venture to develop and test the proposed methodology (1). Under NSF Phase II funding, a two-year program has been under way for nearly one year.

## PHASE I STUDIES AND RESULTS

The research conducted in Phase I primarily identified trends in seismic response of the components of URM buildings, and determined what studies and testing would be necessary to arrive at a methodology suitable for nationwide usage. A preliminary sampling of existing masonry buildings was made in representative areas across the United States. Professional engineers and building officials from these areas were interviewed. The survey elicited the commonality of building construction methods, the range of building dimensions, and the various usages of masonry materials.

Analytical studies of a representative structure with a flexible horizontal diaphragm were made by typical design procedures and simplified mathematical models subjected to various ground motions. With current technology, the state of stress in the elements under dynamic loads cannot be determined within reasonable bounds by typical static analysis. However, the studies indicated that structural response of vertical and horizontal elements can be defined for analysis by simple parameters. We realized that additional research was needed to formulate procedures for dynamic analysis of these elements.

Prior and current research studies on material properties of masonry were reviewed to extract generic data applicable to URM. Current methods of sampling and testing URM buildings were examined to ascertain whether properties obtained by test can be predictors for the multiplicity of failure stresses that are related to failure modes. Gradually, it became apparent that static testing of URM was needed to determine the wide variation of mortar and masonry unit strengths. A requirement emerged for mathematical analysis and dynamic testing of full-scale unreinforced walls to determine their response to forces normal to their plane.

It was concluded that test programs related to the recommended research could furnish definitive data to improve analysis of undesigned elements. We also emphasized that analysis to determine the need for retrofit could be cost effective. Current retrofit and modification methods could be applied, and recently developed techniques for strengthening URM could be implemented.

The results of Phase I were encouraging. We agreed that the Phase I investigation demonstrated the feasibility of developing a methodology for mitigation of seismic hazards in URM buildings and had laid a foundation for the more specific objective of Phase II.

## PHASE II OBJECTIVES

The objectives of Phase II are four-fold:

1. To complete the preliminary Phase I evaluation of the state of the art for mitigating the seismic hazards of existing URM buildings.

2. To develop a methodology for the mitigation of these hazards.

3. To evaluate the methodology.

4. To outline a utilization plan for disseminating the information assembled by the research program.

A review panel of professional engineers and architects representing various sectors of the United States and potential users of the methodology is periodically evaluating the program's direction and content.

RESEARCH PLAN FOR PHASE II

The program has been scoped for two major efforts, one dealing with studies of existing data, the other with testing and verification. An abbreviated version of the research plan developed is given in the following overview:

Survey and assess current methods of evaluating URM.
Categorize existing URM buildings.
Evaluate seismic hazards.
Categorize URM damage from past earthquakes.
Assess failure theories.
Select methods of obtaining in-place material properties.
Assess and select analytical methods.
Assess and select retrofit methods.
Expand the data base through an interactive program of tests and analysis.

This paper reports on several of the tasks in the ongoing study that combine analytical and experimental investigations for the mitigation of seismic hazards in existing URM buildings.

SURVEYING AND CATEGORIZING EXISTING URM BUILDINGS

We undertook a qualitative but comprehensive survey of existing URM buildings to become familiar with the range of structures that could be analyzed by the developed methodology. This broad range has been categorized into classes of structures found in all seismic zones or specific to one or two seismic zones.

By choosing geographic survey areas that have equivalent seismic zones, we have minimized the number of structures that need to be analyzed as representative of the nationwide inventory of existing URM buildings. The three zones selected correspond to classifications defined by the Applied Technology Council Report "Tentative Provisions for the Development of Seismic Regulations for Buildings" (3) and are associated with parameters of effective peak acceleration (EPA) and effective peak velocity (EPV), which vary from 0.10 to 0.40 g.

St. Louis, The South Carolina area, and the New England area have EPA and EPV equal to 1/4 of the highest mapped. The Pacific Northwest, the Salt Lake City area, and Memphis have an EPA and EPV equal to 1/2 of the highest mapped. The geographic area that has the highest mapped ground motion is the California Pacific Coast. These three zones encompass the full range of seismicity in the United States.

Predominant types of URM structures were photographed during visits to the seven areas. Interviews with building officials, construction-materials associations, and architects and engineers furnished data necessary to classify the buildings by defined occupancy and to further identify them by common construction methods, which seem to be related to occupancy class.

The defined occupancy categories include 80 to 90% of the types of URM buildings in the geographical areas surveyed. When the methodology is complete, it will be tested by evaluating one or more structures of each category at each of the levels of seismicity.

For in-depth study, structures representative of the several categories were selected according to the combination of construction materials that is common to nearly all seismic zones and the combination of materials used in a significant number of URM buildings in any single seismic hazard zone. Categorization of each structure depended on the data elicited by the following questions:

: What kinds of construction materials are combined with URM elements?

: What is the size of the structure (height and general plan dimensions)?

: What are the interconnection details of vertical and horizontal elements?

: Was there a criterion used for lateral load design?

Materials - Construction materials having the most effect on structural response are those making up the floors and roofs and the vertical walls (other than URM) that interconnect floors and roofs Pre-1940 URM buildings have either wood-framed or concrete floors and roofs. Interior vertical elements other than URM are conventionally wood-framed walls with wall finishes of plaster or equivalent material. In post-1940 buildings, the floors, roofs and internal partitioning are generally constructed in a similar manner. However, wood-framed floors and roofs are sheathed in plywood, not boards. Floors and roofs framed with steel joists or beams and steel decking topped with concrete are also common in post-1940 construction. Steel-deck roofs will generally be covered with insulation board and roofing. Interior partitioning will probably terminate at the ceiling rather than interconnecting the framing levels.

Size - The size of the structures observed ranged from very large mill buildings more than 600 ft (183 m) long to residences with an end wall of about 15 ft ( 5 m). The height of the surveyed buildings ranged from 150 ft (46 m) high to a single story of 15 ft (5 m). It was interesting to note that post-1940 URM buildings are generally no more than two or three stories tall.

Distribution of URM Walls - The mill and industrial buildings have URM walls around the total perimeter and relatively uniform penetrations. Such a building is generally subdivided by fire walls having very few penetrations. Multiple housing, public buildings, schools and churches do not have consistent fire-wall subdivisions. Commercial buildings such as offices and retail stores have few or no URM elements at the lowest level facing the public ways. For most buildings, these elements are not anchored into a frame that could develop significant resistance to lateral displacement. In many cases, several stories of masonry above the street level may be supported on the framework.

Interconnection of Horizontal and Vertical URM Walls - Where wood framing is combined with URM walls, the beams and joists are usually anchored to the URM walls. However, a remarkable number of structures observed have no apparent anchorage at all. The inconsistency implies that the anchoring of the walls was an elective practice rather than customary.

Concrete floors are used with URM walls in two different ways. For pre-1940 buildings, the concrete floor was generally poured after completion of the masonry walls below the floor, and the masonry and concrete are in contact. These connections depend on the interlock of frictional surfaces. In later construction methods, the concrete frame/floor system was first completed and the masonry was infilled into the finished frame.

Criterion for Lateral Load Design for Existing URM Buildings - In general, we have found that these buildings were not designed for earthquake resistance. Some allowance for wind loads may have been designed into the buildings' elements, but development by design of a continuous load path for lateral load was unusual.

## STRUCTURES SELECTED FOR IN-DEPTH STUDY

The structures selected for study consist of:

a. Rectangular, six-story industrial building
b. Rectangular, four-story public school
c. Irregular, four-story plus basement public building
d. U-shaped, four-story apartment building
e. Rectangular, six-story and three-story office buildings
f. Rectangular, one-story and three-story industrial buildings

The industrial/mill building will be analyzed twice, with the second analysis assuming the floor framing to be equivalent to concrete (the stiffest floor system). The difference in response and performance will then be compared.

School buildings of URM still have full utilization in all zones except the California Pacific Coast Zone. A high level of life-safety is required for this occupancy class. The structure selected for study is one of several that have undergone preliminary evaluation in the Seattle area. This particular example, which has also been subjected to two recorded earthquakes, will be reinspected for confirmation or refutation of the damage prediction we will derive.

Although the public buildings vary considerably, they share a complexity of plan and an abundance of exterior embellishment. The building chosen for study is representative of this type, and the as-built plans are available for accurate modeling and analysis. Before this public building is studied in depth, it will be analyzed by a criterion comparable to that of current codes and the cost of conformance estimated. The results will be compared with reconstruction costs as determined by the developed methodology.

The construction materials of the composite apartment building developed for study will equal those commonly used in all zones.

The three-story commercial office building is typical of all zones and allows for a desirable comparative analysis for each level of ground motion. The response and performance of this structure will be considered when it stands alone and also when several buildings exist within a commercial block and share party walls. The six-story building is comparable to those observed in Memphis, St. Louis, and Seattle.

The two post-1950 industrial and commercial buildings will represent typical buildings observed in the eastern half of the United States.

## EVALUATION OF SEISMIC HAZARD

The Applied Technology Council 3-06 report is a state-of-the-art tool for describing earthquake ground shaking at various sites across the United States (3). This design oriented document describes procedures that can be used with ATC-developed maps and formulas to generate "design ground motion" for a specific site.

The two earthquake ground-shaking regionalization maps that were developed by ATC are used in accordance with the following ground rules: (1) the design lateral force should take into account the period of the structure and the distance from anticipated earthquake sources; (2) the probability of exceeding the design ground shaking should, as a goal, be roughly the same in all parts of the country; and (3) the regionalization maps should not attempt to delineate microzones.

The intensity of design ground shaking is represented by two parameters. These parameters are called the "effective peak acceleration" (EPA) and "effective peak velocity" (EPV). The EPA is proportional to spectral ordinates for periods in the range of 0.1 to 0.5 sec. whereas the EPV is proportional to spectral ordinates at a period of about 1 sec. The amplification of ground motion for a 5% damped spectra is 2.5 for both acceleration and velocity.

Since URM buildings are constructed of undesigned elements that are believed to have hysteretic or non-period-related response, they cannot be analyzed by spectral response methods. Accordingly, time histories will be required to evaluate the performance of this class of structure.

Time histories consistent with the design EPA and EPV of the seismic zones are constructed by scaling recorded earthquakes. A 5% damped spectrum of the time history is graphed and scaled to match the smoothed spectrum defined by ATC for the seismic zone.

Each time history will be selected from a library of earthquake strong-motion records based on geological and seismological information that includes (1) source magnitude, (2) source distance, (3) source focal depth, (4) attenuation relationship, and (5) peak acceleration, velocity, and displacement associated with seismic risk for each city.

CATEGORIZING PAST URM EARTHQUAKE DAMAGE AND FAILURE MODES

Most investigations and reports of earthquakes are quick to report the dramatic damage and collapse of URM buildings. The published data have led to the general consensus that all URM buildings are hazardous in an earthquake. What is also known, but not as well documented, is the fact that some quite similar structures in the same area did not show any appreciable damage. This became quite apparent during examination of buildings for failure modes. The following failure modes are based on observed damage or failure: *

1. Wall collapse due to (a) inadequate anchorage between walls and floor or roof diaphragms (probably one of the most common modes of failure), (b) out-of-plane bending failures, (c) in-plane shear or flexural failure, and (d) excessive deflection of the diaphragm system.

2. Diaphragm failures in (a) shear, (b) shear connection to walls or other resisting elements, (c) horizontal-shear connection chords (if any), and (d) chords (if any).

3. Excessive deflection of the diaphragm system, causing failures of interior vertical load systems or rigid, brittle elements.

4. Collapse of parapet, cornice, veneer, and other building appendages.

5. Differing dynamic response of component parts of complex buildings.

6. Incipient failure conditions due to previous strains on structure, such as foundation settlement, deterioration, or previous shaking.

7. Effect of infilled URM wall on building frames.

---

* Some listed modes of failure were inferred when investigations of the building systems showed conditions that are not consistent with present-day methods of seismic design and construction.

## COLLECTING STRENGTH AND TEST DATA

Available test data have been reviewed to determine the extent of correlation between failure theories and test results. All current methods of testing existing masonry have been reviewed and compared with research-oriented testing. Comprehensive studies or major masonry research and testing programs have yielded valuable information, particularly the ongoing efforts at Berkeley, sponsored by NSF and the Masonry Institute of America, and at the University of California, San Diego.

## SELECTING ANALYTICAL METHODS AND COMPUTER PROGRAMS

Nonlinear dynamic response of the following URM building components will be obtained:

: End walls rocking on their foundations (overturning)
: Stability of walls subjected to out-of-plane and out-of-phase motions
: Diaphragm response (hysteretic behavior)
: Torsional response (plan irregularity)
: Complete structures (all component responses included)

Linear and nonlinear static finite element analyses will be carried out for correlation with tests on panels, core specimens, and anchorages, as well as analysis of perforated walls and piers.

The seismic response of a URM building is not well described by the general methods used for design of new buildings. The URM building generally has stiff exterior wall elements that do not respond with ground motion amplification coefficients assumed for the basis of building codes. The Phase I analysis program (2) determined that response of masonry walls can be described by their height, height/width ratios and by the general classification of the soils under the walls.

An extensive study of all parameters that influence the response of walls such as seismic response mass that is attached to the wall by horizontal diaphragms, return walls at the ends of the wall and other typical conditions will be made to develop bounds of vertical element seismic response.

## ASSESSING AND SELECTING RETROFIT METHODS

The methodology is expected to focus on investigative and analytical processes that enable a designer to determine whether retrofitting is needed and, if so, the extent of retrofitting required. From that point, the details and costs of retrofitting will influence the decision to proceed with the hazard mitigation or to remove the building.

Depending on the geometry of the existing URM structure, the accessibility of its various components, and the method of original construction, one or more of several techniques can be considered for rehabilitation:

: Strengthening of masonry walls
: Adding or improving the anchorage of walls to diaphragms
: Repair and strengthening of diaphragms

: Amelioration of foundation settlement
: Addition of shear walls
: Removal of upper stories
: Parapet renovation
: Bracing of nonbearing partitions

In the course of our program, all of these methods will be assessed for both adequacy and relative costs. If innovative methods of retrofitting evolve during the study, they too will be considered. Depending on the results, certain methods will be selected for continued analysis, study, and testing. After completing the testing program of original and retrofitted construction, cost effectiveness, and performance, we will evaluate modification of selected systems.

ANALYSIS VERIFICATIONS AND RETROFIT TESTS

This phase of the research program comprises several interactive series of tests and analyses. The tests of actual, full-scale specimens will provide basic data on the static and dynamic characteristics of URM building components and will be used iteratively to correct, refine, and verify the mathematical models constructed to represent typical URM components. Our intent is to produce reliable analysis methods for predicting the behavior of URM building components subjected to earthquake ground motions.

Conducting the Test Program - The proposed test program consists of four related test series: (1) test diaphragms, (2) test out-of-plane URM walls for stability, (3) test sampling methods for determining in-plane URM strength, and (4) test anchorages.

Testing Diaphragms - The first series will test fifteen basic 20 ft by 60 ft (6.1 m by 18.3 m) diaphragms (including virgin specimens and repaired and retrofitted specimens) by static displacement and by dynamic, in-plane shaking. These tests are being conducted to (a) study the behavior of diaphragms under earthquake loading, (b) evaluate the potential effect that the energy-absorption capacity or, alternatively, the amplification characteristics of these diaphragms may have, (c) assess the effect that various kinds of retrofitted strengthening will have on the response of the diaphragms, and (d) correct, refine, and verify the mathematical models of typical building diaphragms existing in URM buildings.

Ten basic types of diaphragms will be built and five will be retrofitted at least once to repair damage or to augment stiffness. Two diaphragms will be made of steel decking, five of combinations of plywood, three of board sheathing applied diagonally, and/or applied straight across the diaphragm frame.

Testing Out-of-Plane Walls - The second series will dynamically test 22 masonry wall sections 6 ft by 10-to-16 ft high (1.8 m by 3.0 to 4.9 m) by out-of-plane and out-of-phase motions applied to the top and bottom of the specimens. Virgin and repaired specimens will be tested.

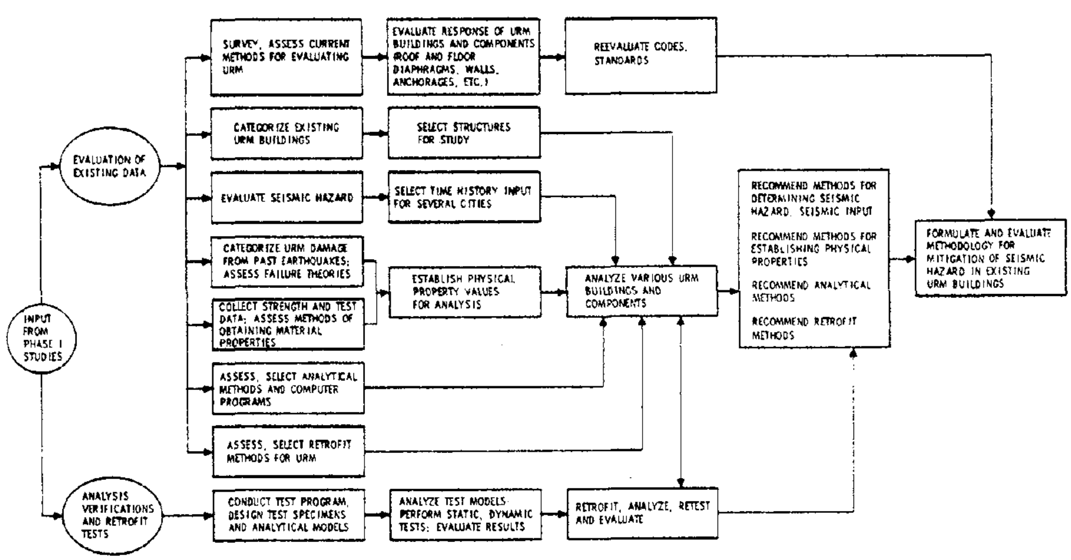
Testing unreinforced masonry wall sections in the out-of-plane direction is expected to reveal why actual walls fractured by out-of-plane motions often do not collapse in real earthquakes. It is possible that the fractured walls remain statically stable and thus are still capable of supporting compressive loads without collapse. The dynamics of the phenomenon will be investigated by computer analysis prior to testing, and the test program will be used to correct, refine, and verify the computer analysis.

Testing In-Plane Walls - The purpose of these tests is two-fold: (1) to ascertain methods for determining the strength properties of existing URM with an accuracy equivalent to the analysis techniques, and (2) to determine the strengthening of certain types of retrofit. The tests proposed will be limited to brick, concrete block, and hollow clay tile masonry of thicknesses nominally 12 in (0.3 m), 8 in (0.2 m) and 8 in (0.2 m), respectively. Because of the wide variation in workmanship and quality found throughout the nation, we expect to limit each basic type to good, medium, and poor quality masonry.

Testing Anchorages - Values have been established by industry for various types of anchors in new masonry. Since poor anchorage of walls to floor and roof diaphragms has been the cause of many failures in earthquakes, the placement of new anchors in existing URM buildings is an important part of any retrofit program. Therefore, we will test the frequently used types of anchors that may be installed in existing masonry, to determine shear and pull-out values.

RESULTS TO DATE

A schematic presentation of the research plan is shown below.




Flow Chart for Research Plan

Assessment of current methods for evaluating seismic hazards in URM buildings has been completed. This assessment is that new methods of evaluation need be developed. Current methods are generally related to codes intended for design and construction of new buildings. In fact the Uniform Building Code prohibits URM in certain seismic zones.

Existing URM buildings have been categorized and selection of structures for study is completed. Analysis of these structures is post-dynamic testing of components and retrofits.

Seismic hazard has been evaluated. Time histories of design earthquakes compatible with the seismic hazard of geographic zones have been selected and are now being used in the dynamic test program.

Categorization of URM damage in past earthquakes has been completed. Modes of failure have been categorized. The recent earthquake in the Imperial Valley gave an excellent opportunity to provide verification of failure modes and non-failure behavior of URM.

Strength data of unreinforced masonry has been collected from available sources. Methods of testing for determination of strength properties has been assessed. Field methods of determining in-place masonry have been reviewed and analyzed. Preliminary analyses of masonry systems indicate that determination of strength properties required to compare with analysis stresses is inadequate. The inadequacy is due to the difficulty of predicting tensile failure stresses from typical compressive or so-termed shear tests. This task of in-place strength determination is deferred til further static tests of URM are devised and performed.

Analytical methods and computer programs have been developed and used to direct the dynamic testing program. This direction only establishes bounds of dynamic testing and gives insight into instrumentation requirements.

The dynamic test program for analysis verification is now underway. Diaphragm testing has provided data on structural response. Data analysis is proceeding. Out-of-plane URM wall testing has been delayed due to technical equipment problems discussed under experimental difficulties. Due to the currency of this work, the conference presentation of this paper will provide a review of immediate results.

## EXPERIMENTAL DIFFICULTIES

The analysis verification of this program required material testing performed in a dynamic environment. It was recognized in the proposal preparation that equivalent static analysis as done for new structures is inadequate. These new buildings have inelastic behavior when subjected to design earthquakes. This guarantee of inelastic behavior for new structures is provided by limitations on construction materials and construction methods. Structural elements are designed by use of equivalent static loads to less than yield stress levels for combinations of seismic and gravity loads. These design techniques are not applicable to analysis. Analysis of existing buildings must consider the average response of the existing materials to a design seismic event. The dynamic tests must replicate the design seismic event to verify a computer response model identical to the test structure. Then the analytical response model can extrapolate the average response of existing building elements to the variety of construction materials and building dimensions encountered throughout the United States.

ABK prepared a test plan that described the dynamic and static tests and solicited price proposals. Responders to the price request were from the aerospace industry and one testing laboratory. A testing subcontractor was selected from the aerospace industry on consideration of price and their background of static and dynamic testing. The cost plus fixed fee was within cost allowances made in the contract with the NSF. The reporting of costs charged by the testing subcontractor lagged behind work progress and made an effective cost control program almost impossible.

The dynamic testing program also has had continuous mechanical problems that have challenged all available personnel and consultants within the testing subcontractor's staff. It has been generally ascertained that testing in a dynamic environment with random motions such as earthquake simulation is much different from tasks performed previously in the aerospace industry.

Hydraulic equipment, control systems, instrumentation and general equipment quality has been refurbished, rebuilt or replaced to run these dynamic tests. It has been determined that equipment that can perform in a static cyclic environment cannot be used without extensive rehabilitation. The aerospace industry generally tests large size materials within the elastic range or performs small scale dynamic tests. While large hydraulic force levels are not required by this program the wall test setup has been plagued by resonant feedback in the control system. Solution of this problem alone required two months after its discovery.

This dynamic test program is the forerunner for large size material tests to full scale ground motions and can probably be expected to have start up problems. The magnitude of these problems could not have been expected from extrapolation of past problems of the aerospace industry.

FUTURE RESEARCH

Our research to date has given us an understanding of the complexity of the performance of masonry subjected to combinations of dynamic and static loads. We have not yet devised methods of inexpensively testing strength properties of in-place masonry but have reached the conclusion that for URM the ultimate stress that must be determined is tension, often in a biaxial state of stress. Tests for ultimate tension stress have been made, but not with simple field procedures. Tension tests are easily flawed by test techniques. The ultimate tensile stress determined by test has typically a wide variation. This variation is reported by researchers to be associated with workmanship. Research directed to reduction of the workmanship variable in masonry will be of significant value for masonry used in new structures. Evaluation of existing masonry can indirectly benefit from this research. In many seismic zones, elastic behavior of URM is limited to elements having excess strength. Therefore, we have directed our URM analysis to predict the behavior of URM in a cracked state.

We are directing our continuing effort in developing a methodology for mitigation of seismic hazards in URM buildings. This methodology will describe analysis methods that can predict probable hazard of URM rather than attempting to predict initial cracking of URM.

## ACKNOWLEDGMENTS

The foregoing work is being conducted by the joint venture ABK comprising the firms of Agbabian Associates, S. B. Barnes & Associates, and Kariotis & Associates. It is authorized by the National Science Foundation in support of the ongoing earthquake hazard mitigation program under the aegis of the Division of Problem-Focused Research Applications (PFRA) of the Applied Science REsearch Applications Directorate (ASRA). The cooperation and encouragement of Dr. John B. Scalzi, Cognizant Program Official for NSF, is gratefully acknowledged. We wish to thank Agbabian Associates' staff member, Dr. Samy A. Adham, who conducted the seismic input studies.

## REFERENCES

1. ABK, A Joint Venture, "Methodology for Mitigation of Seismic Hazards in Existing Unreinforced Masonry Buildings: Phase II", P78-110, 4598, ABK, El Segundo, California, April 1978.

2. Adham, S. A. and Ewing, R. D., "Methodology for Mitigation of Seismic Hazards in Existing Unreinforced Masonry Buildings, Phase I", R-7815-4610, Agbabian Associates, El Segundo, California, March 1978.

3. Applied Technology Council (ATC), "Tentative Provisions for the Development of Seismic Regulations for Buildings, ATC 3-06", ATC, San Francisco, California, June 1978.

4. Barnes, S. B.; Johnson, A. W.,; and Pinkham, C. W., "Methodology for Mitigation of Seismic Hazards in Existing Unreinforced Masonry Buildings, Phase I", S. B. Barnes & Associates, Los Angeles, California, March 1978.

5. Kariotis, Kesler & Allys, "Mitigation of Seismic Hazards in Existing Unreinforced Masonry Wall Buildings", Kariotis, Kesler & Allys, South Pasadena, California, March 1978.

# AN OVERVIEW OF MASONRY RESEARCH IN CANADA

By Plewes, W. Gordon

ABSTRACT: Gives a brief account of masonry research in Canada since 1950 and a survey of projects completed or in progress. Research activity is shown to have increased greatly in the last decade, particularly at universities, with the additional benefit that a number of them now offer lectures or courses on masonry.

# AN OVERVIEW OF MASONRY RESEARCH IN CANADA

By W. Gordon Plewes[1]

## INTRODUCTION

My announced topic is an overview of masonry research in Canada today, but, rather than give a bare recitation of research projects, it is more interesting and meaningful to tell something of how it developed.

Before 1950, the Canadian Department of Energy, Mines and Resources had for many years done research into the ceramic properties of Canadian clays and shales. Perhaps some brick companies had also done a small amount of work along this line but this was about all the activity there was that could be construed as masonry research in Canada. From a structural or constructional point of view it was practically nil. Although the University of Toronto did some wall testing about 1895, they appear to have soon given this up and along with all the other universities became much more interested in reinforced concrete and steel for the next half century.

### N.R.C., Division of Building Research

In 1947 the National Research Council of Canada formed a Division of Building Research and within a few years, in response to many enquiries and the needs of our housing authority (Central Mortgage and Housing Corporation), engaged in studies of masonry leakage. The problem was particularly severe in our Atlantic Provinces where driving rains occur at certain times of the year. (It was not uncommon to find the East and North walls of houses covered with wood siding as a defense against such rains).

T. Ritchie undertook this work and over the years extended it into many areas of masonry materials. He is well known in the industry for his long list of papers on rain penetration, mortar properties, durability and soundness, brick properties, efflorescence and durability, and also for his many contributions to ASTM and Canadian Standards Association committees.

J. Ivan Davison joined NRC in 1958 and as the other member of the team carried on his research along similar lines at the Halifax laboratories. His interests have centred around masonry materials and their performance in Atlantic Canada also starting from the rain penetration problem, its contributing causes and compatible brick-mortar combinations. The properties of bricks and mortars have been studies in the laboratory, as well as in summer and winter site conditions. His laboratory has participated in many ASTM round robin tests such as that on the soundness test for type N dolomitic lime. At present he is studying the expansion occurring when bricks and mortars are

---

[1]Formerly Senior Research Officer, National Research Council, Ottawa, Canada

frozen. Davison is also well known for his publications and committee work as evidenced by the fact that he is immediate past chairman of ASTM Committee C12, Mortars for Unit Masonry.

These researchers have been particularly mentioned to pay tribute to the fact that their work has been continuous for 30 years. NRC is also active in other areas such as the fire resistance of masonry where T. Harmathy is a leader in the field. His work is leading to the direct design of masonry walls for fire resistance based on the basic properties of the materials. The 33 years of general research of the Division into heat and moisture regimes in buildings is also becoming more and more relevant to masonry in connection with serviceability and energy.

## NEED FOR RESEARCH

In 1965, when engineered masonry was first introduced into the National Building Code of Canada, government laboratory work was still the only masonry research in sight. The responsible committee was very conscious of the fact that in trying to re-establish masonry as a viable structural medium, its foundation needed improving. Although research and experience in Europe had shown that thinner walls and higher stresses could be justified, we still lacked much knowledge regarding the fundamental mechanics of masonry. Our materials standards were as good as any but constructional procedures, quality control and workmanship habits were inherited from less demanding rule-of-thumb design. To compensate, we tried to maintain a buffer of conservatism. Soon of course we were encouraged by the research, technical data and improved codes produced by the Structural Clay Products Institute, the National Concrete Masonry Association, the National Bureau of Standards and others.

Forty years ago there were those who claimed that Canada did not need a National Building Code or other engineering standards because we could get along quite nicely copying those of the U.S.A. and other countries. This attitude was not followed in our codes but still prevailed in 1965 so far as masonry research is concerned, which was no longer tolerable. This is not a nationalistic view for a constant exchange of technology between countries is highly desirable and necessary for us. However, it is naive to think that a sound and advanced masonry industry and good codes and standards can be maintained without doing something about our own particular problems. The promotion of modern masonry would in the long run be unsatisfactory without a home grown back-up of expertise and special knowledge such as enjoyed by steel and concrete in all regions. It seems incredible now that on at least one occasion, an industry association in Canada looking for masonry research could not find any non-governmental source with the interest and facilities to do it.

## THE LAST DECADE

A little over a decade ago a change began to take place. It is difficult to say just how, because it probably started in private discussions and committee meetings. It is certain however that there was a general feeling that we should do at least some masonry research in Canada for reasons much as described above.

The new engineered masonry codes produced a spark of interest at the universities and about 1970 C. Turkstra at McGill began to look at the mechanics of masonry. G. Suter at Carleton began to take an interest soon after and obtained his indoctrination by spending a year with Prof. Hendry at Edinborough.

In 1974 the Alberta Masonry Institute received a three year National Research Council grant under its Industrial Research Assistance Programme. The grant was for research into the field testing of mortars and as one of the conditions, the work was associated with the University of Calgary. Extensive sampling and testing was done in the field from a mobile laboratory and the results have been reported by Huizer, Jessop, Ward and Morstead.

About the same time the Canadian Masonry Contractors' Association formed a Canadian Masonry Research Foundation with an associated Canadian Masonry Research Council consisting of 12 advisors from a cross-section of engineering, architecture and construction. One of the first steps by the Council was to canvass themselves and their associates regarding what they considered important research needs. Twenty-seven were selected from a long list of topics and written up in a report regarding the state of knowledge, type of research required, etc. The list is too long to read here but actually there were few surprises. What the report did do was provide a good basis for selecting projects and setting priorities.

As with most such organizations, finances are not unlimited but seven projects and three state-of-the-art reports have been funded to date. These are regularly monitored by members of the Council.

Another activity was to initiate and support the preparation of a bibliography on masonry which was undertaken by the writer. This was considered high priority by the Council because of its intention to promote masonry research at the universities. It is fundamental that research should begin with a review of present knowledge but the fact was that most universities with ample libraries on concrete and steel had very small holdings on masonry. There were some initial difficulties getting the bibliography published as it contains almost 6000 references covering 1900 to 1977. Eventually it was produced by the International Masonry Institute and is now available.

All in all the Council has done some useful work but its very existence was perhaps its greatest achievement. It brought to the attention of researchers that perhaps the masonry industry was interested in research after all.

The next significant development, and a turning point in 1975, was a masonry seminar encouraged by the Council, hosted by McMaster University at Hamilton, Ontario, and sponsored by the Canadian Masonry Contractors' Association, the Clay Brick Association of Canada, the National Concrete Producers Association and all provincial masonry associations. This seminar was solely for professors from all the universities across Canada. Most of these had research experience in other fields but only a nodding acquaintance with masonry. All phases of masonry from materials through design and construction were covered, with Jim Amhrien doing most of the instruction. His meticulous preparation and boundless enthusiasm created a great deal of interest with the academic group and a revelationthat masonry was interesting and offered something to get their research teeth into. From that time on research at the universities grew to the point that 9 universities are now active in the field.

## SUMMARY OF RESEARCH

To avoid tedious repetition, the extent of past and present masonry research has been tabulated in Table 1. The list is thought to be quite exhaustive but no doubt somebody or something has been omitted, if so, apologies are due. The following comments are offered to bring out certain points.

1. It will be immediately noticed that in line with previous comments many of the projects are of fairly recent origin. The number of persons doing masonry research has grown a great deal in the last 10 years. Figure 1 shows the general trend in man-years. The increase is from around 12 to over 40, including researchers, students and technicians. The estimate is conservative to avoid exaggeration.

2. Partly by design and partly by accident there is not too much overlap of effort. Seismic research is naturally done in <u>British Columbia</u>, an earthquake zone. There is no attempt to duplicate the important work in California and they are concentrating on a piece of the problem that bothers builders in the Vancouver area.

3. Nova Scotia Technical College for obvious reasons is involved in rain resistance where they are surveying the extent of the problem today.

4. The National Building Code of Canada has, under the chairmanship of C. Turkstra, a task group on Limit States Design of Masonry. Many of the researchers are members. An initial draft LSD standard should be ready this year. The prime objective was to obtain ultimate strength analytical models consistent with tests for all types of masonry and all types of actions. The brief titles listed under <u>McGill</u> and <u>Alberta</u> do not convey the thoroughness with which this is being done.

Practical design methods are being compared and evaluated against all available data. At the moment a stimulating debate is going on concerning the relative merits of a modified, convenient, moment-magnifier method and a more accurate moment- curvature method.

5. Also contributing to development of ultimate strength design is the work at Carleton on shear and flexure in masonry beams. This is nearly completed and will be extended to shear wall research.

6. McMaster has an extensive programme on the mechanical properties of masonry of all kinds. Over a period of about two years about 1800 specimens of one kind or another were tested.

7. At Calgary, background on the creep, shrinkage, thermal and elastic movement characteristics of masonry is being developed which is badly needed for serviceability studies and design. Manitoba and Saskatchewan are starting observations on the behaviour of existing buildings.

8. National Research Council research has been discussed except for an extensive programme being carried out by Maurenbrecher on methods of test to determine $f^1m$ and the determination of reliable values for most of our common masonry units to form the basis of future codes.

9. The National Research Council has an arrangement whereby industry associations can sponsor a Research Fellow in their laboratories for 3 years. At present J. Kung is there on behalf of the Clay Brick Association and is studying durability of clay bricks with special emphasis on determining the conditions in an actual wall. Previously the National Concrete Producers had a Fellow working on the fire resistance of concrete block masonry.

10. An interesting recent development is the establishment of a Centre for Research and Development in Masonry at Calgary. it is under the wing of the University and the Alberta Masonry Institute and is an outgrowth of the NRC research grant previously held. Through the efforts of M. Ward, Head of the Department of Civil Engineering and E. Jessop who is Research Director, a substantial government grant was obtained from the Department of Industry, Trade and Commerce to set up the Centre. At the time of writing, a newly established board of directors had not yet met so it is too early to say much more at this time. Further information will no doubt be forthcoming.

## FINANCING

The writer has made a detailed estimate of the growth of masonry research funding since 1950. Such estimates are always precarious but it is safe to say that over the years the total value of masonry research including salaries, operating and overhead has grown from a few thousand to over a million dollars per year. Again as a pure estimate, about 10 - 15 percent comes from industry, 10 - 15 percent from funds found by the universities and 60 - 80 percent from public funds expended in government laboratories, grants to individual researchers at universities and grants to industry.

## EDUCATION

A quick way to get the results of research into use is through codes and standards, but this is not enough. Designers and builders have to be educated to use masonry well. A spin - off from the growth in masonry research in Canada has been a considerable increase in the teaching of masonry at universities. Without going into details at this time, quite a few engineering schools have managed to squeeze a few lectures or a whole course on masonry into their undergraduate time-tables. Several offer advanced courses and degrees in the subject. Previous to 1970, the words concrete block and brick were seldom if ever heard in our engineering class rooms.

At the same time, what is even more encouraging is that many technical schools are now offering masonry courses with about 600 hours of pre- apprenticeship masonry training. Their course outlines also generally indicate two other levels, technician and engineering technologist where masonry construction is part of the course. These programmes will require time to take effect but should help to produce the calibre of job supervisors that modern masonry needs.

## CONCLUSION

It is perhaps a lot to hope that masonry research in Canada will continue to grow at the rate indicated in Figure 1. It has been a pleasure however to be able to report that it is now at least at a fairly creditible level. The quality of the work you will be able to judge for yourselves when it is reported at the Second Canadian Masonry Symposium to be held in Ottawa in June of this year.

TABLE I

SUMMARY OF MASONRY RESEARCH IN CANADA

| ORGANIZATION | TOPIC | BEGUN | COMPLETED | IN PROGRESS | RESEARCHER |
|---|---|---|---|---|---|
| UNIVERSITIES | | | | | |
| BRITISH COLUMBIA | MINIMUM SEISMIC REQUIREMENTS FOR MASONRY | 1977 | | • | N.D. NATHAN<br>S. CHERRY<br>D.L. ANDERSON |
| ALBERTA | INTERACTION OF CONCRETE MASONRY BEARING WALLS AND CONCRETE FLOOR SLABS | 1978 | | • | J. WARWARUK<br>J. LONGWORTH |
| | EFFECTS OF REINFORCEMENT DETAILING FOR MASONRY COLUMNS | 1978 | 1979 | | J. WARWARUK<br>J. LONGWORTH |
| | SHORT CONCRETE MASONRY COLUMNS UNDER AXIAL AND ECCENTRIC LOADING | 1979 | | • | J. WARWARUK<br>J. LONGWORTH |
| McGILL | THEORETICAL AND EXPERIMENTAL STUDY OF BRICK WALL STRENGTH UNDER ECCENTRIC LOADS | 1970 | | • | C. TURKSTRA<br>J. OJINAGA |
| | THEORETICAL AND EXPERIMENTAL STUDY OF CONCRETE BLOCK WALL STRENGTH UNDER ECCENTRIC LOADS | 1970 | | • | C. TURKSTRA<br>J. OJINAGA |
| | DESIGN METHODS FOR LIMIT STATES DESIGN AND SAFETY INDICES FOR ALTERNATIVE APPROACHES | 1977 | | • | C. TURKSTRA<br>J. OJINAGA |

TABLE I (Continued)

| ORGANIZATION | TOPIC | BEGUN | COMPLETED | IN PROGRESS | RESEARCHER |
|---|---|---|---|---|---|
| McMASTER | COMPRESSION, TENSION, SHEAR AND COMBINED STRESS STRENGTH PROPERTIES OF MASONRY | 1975 | 1978 | | R. DRYSDALE |
| | FLEXURAL TENSION STRENGTHS NORMAL AND PARALLEL TO BED JOINTS - CONCRETE BLOCK | 1978 | 1979 | | R. DRYSDALE |
| | SPLITTING TENSILE STRENGTH OF 85 BLOCK WALLS | 1978 | 1979 | | R. DRYSDALE |
| | AXIAL STRENGTH OF CONCRETE MASONRY UNDER ECCENTRIC LOAD | 1978 | 1979 | | R. DRYSDALE |
| | EFFECT OF JOINT THICKNESS AND INFLUENCE OF JOINT REINFORCEMENT ON COMPRESSIVE STRENGTH | 1978 | 1979 | | R. DRYSDALE |
| | CAPACITY OF BRICK MASONRY FOR OUT-OF-PLANE LOADINGS | 1978 | 1979 | | R. DRYSDALE |
| | SHEAR STRENGTHS OF BRICK MASONRY | 1978 | 1979 | | R. DRYSDALE |
| | SPLITTING TENSILE STRENGTH OF BRICK MASONRY | 1978 | 1979 | | R. DRYSDALE |
| | TENSILE AND COMPRESSIVE CAPACITY OF TIES CROSSING CAVITIES | 1978 | 1979 | | R. DRYSDALE |
| | CONTINUATION OF STRENGTH AND DEFORMATION CHARACTERISTICS OF MASONRY | 1979 | | • | R. DRYSDALE |
| | CYCLIC LOADING ON EXTERNALLY REINFORCED MASONRY WALLS | 1971 | 1976 | • | W.K. TSO<br>A.S. HEIDEBRECHT |

TABLE I (Continued)

| ORGANIZATION | TOPIC | BEGUN | COMPLETED | IN PROGRESS | RESEARCHER |
|---|---|---|---|---|---|
| CARLETON | SHEAR STRENGTH OF REINFORCED MASONRY BEAMS | 1974 | 1979 | | G. SUTER<br>H. KELLER |
| | FLEXURAL STRENGTH OF REINFORCED MASONRY BEAMS | 1978 | | • | G. SUTER<br>H. KELLER |
| | MASONRY DEFORMATIONS | 1974 | | • | G. SUTER<br>H. KELLER |
| | SMALL SCALE MASONRY RESEARCH | 1975 | 1978 | | G. SUTER<br>H. KELLER |
| | FLOOR SYSTEMS | 1977 | 1979 | | G. SUTER<br>H. KELLER |
| | WINTER MASONRY CONSTRUCTION | 1975 | 1977 | | G. SUTER<br>H. KELLER |
| CALGARY | CREEP OF LIGHTWEIGHT CONCRETE MASONRY | 1977 | 1979 | | R. LOOV<br>E. JESSOP |
| | STRESS - STRAIN BEHAVIOUR OF MASONRY | 1977 | 1979 | | R. DePAIVA<br>E. JESSOP |
| | PREDICTION AND STUDY OF ELASTIC, CREEP AND SHRINKAGE PROPERTIES OF MASONRY | 1977 | | • | N. SHRIVE<br>R. LOOV<br>E. JESSOP<br>G. ENGLAND |
| | STUDY OF APPROXIMATE STRESS STATE IN MASONRY WALLS/PRISMS | 1977 | 1979 | | N. SHRIVE<br>E. JESSOP |
| | STUDY OF FAILURE MECHANISM OF MASONRY WALLS/PRISMS | 1979 | | • | N. SHRIVE<br>E. JESSOP |

TABLE I (Continued)

| ORGANIZATION | TOPIC | BEGUN | COMPLETED | IN PROGRESS | RESEARCHER |
|---|---|---|---|---|---|
| CALGARY (cont.) | STUDY OF ELASTIC AND THERMO-ELASTIC PROPERTIES OF MASONRY: MASONRY GEOMETRY AND PROPERTIES | 1979 | | • | R. DePAIVA<br>N. SHRIVE |
| MANITOBA | STRUCTURAL IMPLICATIONS OF THERMAL EFFECTS IN MASONRY | 1977 | | • | J. GLANVILLE |
| | EVALUATION OF STRUCTURAL DEFECTS IN MASONRY BUILDINGS IN WINNIPEG | 1979 | | • | J. GLANVILLE |
| | EXAMINATION OF MASONRY BEAMS | 1979 | | • | J. GLANVILLE |
| | SURFACE - BONDED MASONRY | 1979 | | • | J. GLANVILLE |
| SASKATCHEWAN | MEASUREMENT OF MOVEMENT IN MASONRY WALLS | 1979 | | • | V. NEIS |
| NOVA SCOTIA TECH. COLLEGE | RAIN PENETRATION OF MASONRY | 1979 | | • | F. EPPEL |

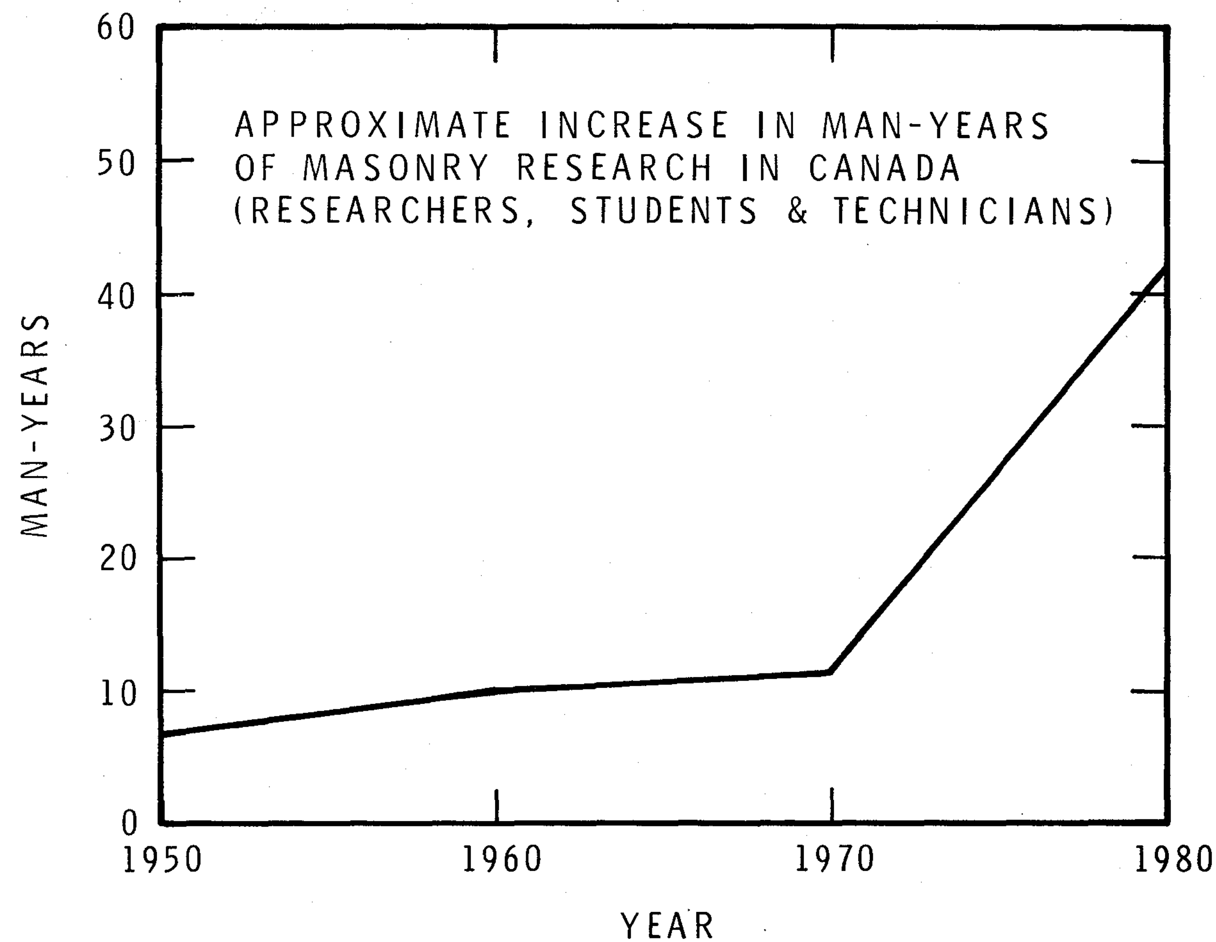
TABLE I (Continued)

| ORGANIZATION | TOPIC | BEGUN | COMPLETED | IN PROGRESS | RESEARCHER |
|---|---|---|---|---|---|
| NATIONAL RESEARCH COUNCIL OF CANADA | RAIN PENETRATION, MORTAR PROPERTIES, DURABILITY AND SOUNDNESS OF MORTAR, BRICK PROPERTIES, EFFLORESCENCE AND DURABILITY, COMPATIBILITY OF UNITS AND MORTAR, etc. | 1950 | 1978 | | T. RITCHIE |
| | MASONRY MATERIALS AND PERFORMANCE IN ATLANTIC CANADA, RAIN PENETRATION, COMPATIBILITY OF BRICK/MORTAR COMBINATIONS, PROPERTIES OF BRICKS AND MORTAR AS USED IN THE FIELD, MORTAR SOUNDNESS FREEZE-THAW EFFECTS ON BRICKS AND MORTAR, etc. | 1958 | | • | I. DAVISON |
| | VARIABILITY OF COMPRESSIVE STRENGTH AND MODULUS OF ELASTICITY OF MASONRY OF CLAY BRICK AND CONCRETE BLOCK, USE OF SMALL SPECIMENS TO PREDICT THE STRENGTH OF LARGER ELEMENTS, CHECK OF PRESENT CODE TABLES FOR $f'_m$, etc. | 1976 | | • | P. MAUREN-BRECHER |
| | PREDICTION OF FIRE ENDURANCE OF MASONRY FROM GEOMETRIC VARIABLES AND MATERIAL PROPERTIES. DESIGN FOR FIRE RESISTANCE | 1960 | | • | T. HARMATHY |
| | DESIGN OF MASONRY WALLS FOR SERVICEABILITY | 1947 | | • | VARIOUS |

TABLE I (Continued)

| ORGANIZATION | TOPIC | BEGUN | COMPLETED | IN PROGRESS | RESEARCHER |
|---|---|---|---|---|---|
| NATIONAL CONCRETE PRODUCERS ASSOC. | FIRE RESISTANCE OF CONCRETE BLOCK MASONRY | 1967 | 1971 | | L. ALLEN (INDUSTRIAL RESEARCH FELLOW AT N.R.C.) |
| CLAY BRICK ASSOC. OF CANADA | DURABILITY OF CLAY BRICK MASONRY | 1978 | | • | J. KUNG (INDUSTRIAL RESEARCH FELLOW AT N.R.C.) |
| CENTRE FOR RESEARCH AND DEVELOPMENT IN MASONRY | MORTAR RESEARCH | 1974 | 1979 | | E. JESSOP<br>M. WARD<br>H. MORSTEAD |
| | STRUCTURAL BEHAVIOUR OF AN INNOVATING INSULATED CONCRETE BLOCK WALL SYSTEM | 1979 | | • | E. JESSOP |
| | TIES AND ANCHORS | 1980 | | • | E. JESSOP<br>C. FENTON |
| GOVERNMENT OF CANADA, DEPT. OF ENERGY, MINES AND RESOURCES | SURVEYS OF CANADIAN CLAYS AND SHALES WITH REGARD TO THEIR FIRING PROPERTIES, SURVEYS OF AGGREGATE SOURCES, etc. | 1900 (±) | | • | VARIOUS |

FIGURE 1


APPROXIMATE INCREASE IN MAN-YEARS
OF MASONRY RESEARCH IN CANADA
(RESEARCHERS, STUDENTS & TECHNICIANS)
60
50
40
30
20
10
0
MAN-YEARS
1950
1960
1970
1980
YEAR

# STRENGTH OF CONCRETE MASONRY UNDER BIAXIAL STRESSES

by
Ahmad A. Hamid[1] and Robert G. Drysdale[2]

## INTRODUCTION

To rationally predict the strength of masonry elements under combined in-plane vertical and lateral loads, failure criteria for masonry under biaxial stresses should be established. The available failure hypotheses (5,9,10) for masonry are related to the failure theories for isotropic materials (8) such as Coulomb's theory of internal friction, the maximum stress theory and Mohr's theory of failure. Although it is known that these theories are not applicable in a generalized form to masonry they have been utilized (5,9,10) to predict failure of masonry assemblages under particular stress conditions. It has been shown (1,2,6) that masonry strength is highly sensitive to the orientation of stress with respect to the critical bed and head joint directions. Therefore, the failure theories for isotropic materials are not applicable for masonry because they were derived on the basis of the invarient state of stress concept where the stress orientation has no effect on the strength (8).

At McMaster University, Canada, an extensive research program has been conducted to study the strength and deformation characteristics of concrete masonry assemblages under combined stresses. In this regard, failure criteria are proposed for masonry under biaxial stresses taking into consideration its

---

[1]Assistant Professor, School of Civil Engineering and Environmental Science, The University of Oklahoma, Norman, Oklahoma.

[2]Associate Professor, Department of Civil Engineering and Engineering Mechanics, McMaster University, Hamilton, Ontario.

anisotropic nature as a composite material. The contributions of the different component materials to the assemblage strength under combined stresses are investigated.

## APPROACH

At the University of Edinburgh in Scotland, Sinha and Hendry (9) adopted the criterion of maximum tensile stress to determine the capacity of brick shear walls under combined shear and precompression. The proposed approach predicts the shear capacity of the joints but does not predict the capacity of the assemblage. At the National Bureau of Standards Yokel and Fattal (10) proposed a failure criterion for brick masonry under combined stresses adopting Mohr's theory of failure which considers a linear relationship between the major and minor principal stresses. The authors adopted the tensile strength as the apparent major stress at failure when the principal compression stress is equal to zero. The directional variation of the tensile strength (5), which is the unique characteristic of masonry as a composite material, was ignored.

For block masonry, grouting the cores provides partial continuity which reduces the degree of anisotropy of the composite. At the University of California in San Diego, Hegemier, et al, (5) suggested interaction failure envelopes for grouted concrete masonry for zero head joint normal stress. It was based on the assumption that the failure envelope in principal stress space is linear in the tension compression zone. This assumption can only be justified for the case under investigation where the material combination led approximately to isotropic material behavior from the macroscopic viewpoint. It is worthwhile mentioning that this approach offers a solution to the particular case under study and not in a generalized form to make it applicable to the cases where other combinations of component materials are used.

In the current study at McMaster University, a different approach (2,4) is adopted in an attempt to propose in a generalized form, failure criteria for masonry under biaxial stresses. The failure envelope is expressed as a function of the basic strength characteristics in the principle material directions (bed joint, head joint and out-of-plane directions). The failure theories of anisotropic materials (7) have been utilized with the modification to account for:

- the possible shear failure along either the bed or head joint directions and
- the interaction between the shear strength and the normal compression stress.

In the proposed equations, the stress field is expressed in the orthogonal material directions since the principal stresses concept is not applicable for masonry as a nonisotropic material.

The proposed criteria are based on a physical interpretation rather than being strictly phenomenological. Two modes of failure are considered with each describing a single mode of failure; a shear failure along one of the critical planes (bed and head joint directions) or a tension failure incorporating the interaction of the block, mortar and grout. The two criteria for shear and tension failures are to be used to predict the failure condition for any stress combination; the minimum of which would be the governing criterion.

## CONCLUSIONS

Although the predicted capacities using the proposed failure criteria indicated better agreement than other available criteria, they are not close to the experimental results as might be desired. However, failure modes were predicted accurately and in general it is suggested that the proposed criteria have a promising potential since the inherent anisotropic characteristics of masonry as a composite material have been rationally considered.

At present, work is continuing at McMaster University for the refinement of the proposed criteria to achieve better agreement with the experimental results. An extensive experimental program has been conducted to check the applicability of the proposed failure criterion for different loading and material combinations.

RECOMMENDATION

1. The available failure theories for isotropic materials are not applicable to masonry under biaxial stresses. Attempts should be directed to develop failure criteria which account for the inherent anisotropy of masonry as a composite material.
2. The following aspects need to be investigated for establishing a rational theory of failure for masonry under combined loading:
   a. Establishing universal testing procedures to uniquely determine the uniaxial strength characteristics of masonry. It requires the choice of the shape of the specimens so as to eliminate the effect of the end restrain.
   b. The applicability of the theory of internal friction to predict the shear strength of masonry along the bed and head joint directions especially under high precompression level.
   c. The effect of the stresses parallel to the failure plane on the shear and tensile strength capacity.
   c. The applicability of the superposition concept (11) in evaluating the contribution of grouting to the strength capacity of masonry under different unidirectional states of stresses.

e. The effect of aspect ration of the block, percent solid, shape of the block, thickness of the face shell and partial grouting on the ultimate strength of grouted masonry.

REFERENCES

1. Drysdale, R.G., Hamid, A.A., and Heidebrecht, A.C., "Tensile Strength of Concrete Masonry," Journal of the Structural Division, ASCE, Vol. 105, No. ST7, July, 1979, pp. 1261-1276.

2. Hamid, A.A., "Behavior Characteristics of Concrete Masonry," thesis presented to McMaster University at Hamilton, Ontario, Canada, in 1978 for partial fulfillment of the requirements for the degree of Doctor of Philosophy.

3. Hamid, A.A., Drysdale, R.G., and Heidebrecht, A.C., "Effect of Grouting on the Strength Characteristics of Concrete Block Masonry," Proceedings of the North American Masonry Conference, Boulder, Colorado, Aug. 1978, pp. 11/1-11/17.

4. Hamid, A.A., and Drysdale, R.G., "Failure Criteria for Masonry under Biaxial Stresses", Submitted to the ASCE, Journal of Structural Division, April 1979.

5. Hegemier, G.A., Nunn, R.O., and Arya, S.K., "Behavior of Concrete Masonry Under Biaxial Stresses," Proceedings of the North American Conference, Boulder, Colorado, August, 1978.

6. Johanson, F.B., and Thompson, J.N., "Development of Diametral Testing Procedures to provide a Measure of Strength Characteristics of Masonry Assemblages," Designing, Engineering and Construction with Masonry Products, F.B. Johanson, ed., Gulf Publishing Co., Houston, Texas, May, 1969, pp. 51-57.

7. Jones, R.M., "Mechanics of Composite Materials," 1st. Ed., McGraw-Hill, Washington, DC, 1975.

8. Nadai, A., "Theory of Flow and Fracture of Solids," Vol. 1, McGraw-Hill, New York, 1950.

9. Sinha, B.P., and Hendry, A.W., "Racking Tests on Storey Height Shear Wall Structures with Openings Subject to Precompression," Designing, Engineering and Construction with Masonry Products, F.B. Johanson, Ed., Gulf Publishing Co., Houston, Texas, May, 1969, pp. 192-199.

10. Yokel, F., and Fattal, G., "Failure Hypotheses for Masonry Shear Walls," Journal of the Structural Division, Proceedings of ASCE, ST3, Vol. 102, March, 1976, pp. 515-532.

11. ACI Committee 531, "Proposed ACI Standard: Building Code Requirements for Concrete Masonry Structures," ACI Journal, Proceedings V. 75, No. 8, Aug. 1978, pp. 384-403.

# SAFETY OF EXISTING STRUCTURES

By Yao, J. T. P.

ABSTRACT: The objective of this paper is to briefly review and summarize the state-of-the-art on (a) system identification in structural dynamics, (b) damage assessment of existing structures, and (c) general concept of structural identification. The inter-relationship of these topics with safety evaluation of existing structures are also discussed.

# SAFETY OF EXISTING STRUCTURES

By James T. P. Yao[1]

## INTRODUCTION

Some thirty five years ago, the late Professor A. M. Freudenthal published a historically significant paper entitled "The safety of structures" (11)[2]. His stated objective was "to analyze the safety factor in engineering structures in order to establish a rational method of evaluating its magnitude". Since then, much progress has been made in the subject area of structural safety and reliability. The state-of-the-art as of 1972 was summarized in a report of the American Society of Civil Engineers Structural Division Task Committee on Structural Safety (28). Subsequently, members of the same Task Committee presented a reliability-based design code format to the civil engineering profession (29). Meanwhile, the theory of structural reliability has been applied to solve safety-related problems in earthquake, wind, ocean, aerospace, and nuclear engineering (12). Recently, the reliability-based load and resistance factor design (LRFD) for steel buildings is considered to be the prototype for the next generation of structural design codes in the States (3,8,10,15,16,24,25,37).

To-date, most studies on structural reliability analysis have been concerned with the calculation of failure probabilities for certain mathematical representations of idealized structural systems and loading conditions. Even for simplified and idealized systems, relatively few special cases of the structural reliability problem have been completely solved. Although much improvements and refinements have been made in structural analysis in recent years, it is still difficult to obtain precise mathematical representations of the overall structural behavior in the nonlinear range corresponding to various stages of severe structural damage. Furthermore, certain significant factors in structural reliability cannot be evaluated in an objective manner. Consequently, available results of theoretical and analytical developments in structural reliability cannot be easily applied for the safety evaluation of complex existing structures.

The objective of this paper is to briefly review and summarize the state-of-the-art on (a) system identification in structural dynamics, (b) damage assessment of existing structures, and (c) general concept of structural identification. The inter-relationships of these topics with safety and reliability of existing structures are also discussed.

---

[1]Prof. of Civil Engrg., Purdue University, West Lafayette, IN.

[2]Numerals in parentheses refer to corresponding items in the Appendix I. -- References.

Portions of this paper are taken from a recent report by the writer (35).

SYSTEM IDENTIFICATION IN STRUCTURAL DYNAMICS

System identification refers to techniques which are developed for obtaining a mathematical representation of a specific physical system when both the input to the system and the corresponding output are known (9,27). A general system identification technique consists of the following three parts: (a) the determination of the form of the model with certain system parameters, e.g., a set of second-order ordinary differential equations for the description of the dynamic behavior of a given mechanical system; (b) the selection of a criterion function using some means of the "goodness-of-fit" from the model response to the measured response, when both the mathematical model and the actual system are subjected to the same input; and (c) the selection of an algorithm for the modification of system parameters so that the discrepancies between the behavior of the mathematical model and that of the actual system can be minimized.

During this past decade, structural response records with or without known forcing functions have been collected and analyzed to obtain better mathematical representations of the dynamical behavior of existing structures (5,18,20,21,23,26,30). During the first phase of this investigation, Chen et. al. (5,30) critically reviewed more than 40 references and tabulated their pertinent information concerning the the application of system identification techniques in structural engineering. The linear lumped-parameter models are found to be the most widely used ones because of their simplicity. Common methods of analysis include the use of modal expansion and transfer function, and various least-squares estimation techniques. For nonlinear models, invariant imbedding and synamic programming filters, least-squares filters, quasilinear methods, extended Kalman filters, and maximum likelihood methods have been used.

By necessity, dynamic testing of structures for the purpose of performing system identification must be conducted at small response amplitudies so that the serviceability and safety limit states are not reached during these tests. Consequently, the effectiveness of the resulting mathematical model is restricted to the linear or slightly nonlinear range of the structural behavior.

Natural hazards such as strong-motion earthquakes have caused severe damage to existing structures, and the safety evaluation of structures under such extreme loading conditions is very important indeed. With the "realistic" mathematical models resulting from system identification studies, it is possible to simulate the structural response to such extreme loads and thus to evaluate the serviceability and safety of the structure under consideration. However, there exists the paradox that (a) the applicability of most "realistic" models of the structure is limited to small-amplitude and linear response range, (b) the catastrophic loading conditions are likely to cause the structures to respond beyond the linear or "near-linear" behavior which is usually assumed, and (c) the severe loadings may cause serious damages in the structure and thus change the structural behavior appreciably from those in the mathematical model resulting from system identification studies.

It is important that the extent of damage in structures can be assessed following each major catastrophic event or at regular intervals for the evaluation of aging and decaying effects. On the basis of such damage assessment, appropriate decisions can be made as to whether a structure can and should be repaired (32,36).

## DAMAGE ASSESSMENT OF EXISTING STRUCTURES

The damage of a given structure can be studied both experimentally and analytically in case of need (4,17). Experimental studies include either field surveys or laboratory tests. Field surveys include the determination of exact locations of failed components and other evidences of distress, the application of various non-destructive testing techniques to the remaining structure, the discovery of poor workmanship and construction details, and proof-load and other load testing of a portion of a very large structure. Meanwhile, samples can be collected from the field and tested in the laboratory for strength and other mechanical and structural properties. Analytical studies frequently consist of the examination of the original design calculations and drawings, the review of project specifications, the performance of additional structural analyses incorporating filed observations and test data, and the possible explanation and description of the event under consideration. The state-of-the-art for damage assessment of existing structures has been reviewed recently (22,33). Although such general procedures are known to exist, the detailed methodology, especially the decision-making process, remain as privileged information for a relatively few experts. Such privileged information and specialized knowledge are being transmitted to younger engineers primarily through many years of working experience and the development of engineering "intuition" and professional "judgement", which are highly personal and subjective in nature.

## STRUCTURAL IDENTIFICATION

The concept of structural identification has been discussed at various stages of development since 1973 (18, 22, 26,33). Structural engineers are mainly interested in identifying the damage and reliability functions, in addition to obtaining the equations of motion. On the other hand, the updated equation of motion using test data and system identification can be a tool for the estimation of expected damage and reliability of existing structures in the future.

When a structure is inspected for the purpose of making damage assessment, a series of tests may be conducted and the resulting data can be analyzed accordingly. Quantities which can be measured and recoreded in testing structures include the load, the deformation (or strain), and the acceleration. From these experimental measurements, mechanical properties such as stiffness and strength and dynamic characteristics such as natural frequency and damping can be estimated. In addition, indications of damage such as cracks and local buckling in the plastic range can be detected visually be experienced inspectors. As an example, binoculars have been used by persons looking for color change in window panes in a certain tall building which indicate the presence of flaws causing the eventual breakage of window glasses. For metal structures which are subjected to repeated load applications, dye-check, ultrasonic or x-ray devices may be used to find and measure small and hidden fatigue cracks which indicate structural damage.

When a structure undergoes various degrees of damage, certain characteristics have been found to change. In testing a reinforced concrete shear wall under reversed loading conditions, free vibration tests were performed to estimate the fundamental natural frequency and damping ratios (31). Results of these tests as given by Wang, Bertero, and Popov (31) indicate the (a) the frequency decreased monotonically with damage while the damping ration increased initially and then decreased, and (b) the repaired specimen was not restored to the original condition as indicated by free-vibration test data. Similar results were reported by Hudson (20), Hilgardon and Clough (19), and Aristizabal-Ochoa and Sozen (1), among others.

Recently, comprehensive experimental results of dynamic full-scale tests were obtained for a multi-story building structure (14) and a 3-span highway bridge (2). Galambos and Mayes (14) tested a rectangular 11-story reinforced concrete tower structure, which was designed in 1953, built in 1958, and tested in 1976. The large-amplitude (and damaging) motions were induced with the sinusoidal horizontal movements of a 60-kip lead-mass which was placed on hardened steel balls on the eleventh floor. This lead-mass can be displaced up to $\pm$ 20 inches and the frequency capacity was 5Hz with the use of a servo-controlled hydraulic actuator, one end of which is fastened to the building frame. The maximum horizontal force range was $\pm$ 30,000 pounds. These test results indicate that the natural frequency decreased with increasing damage in general. Similarly, Baldwin et al (2) concluded from their testing of a three-span continuous composite bridge that changes in the bridge stiffness and vibration signatures can be used as indicators of structural damage under repeated loads.

## DISCUSSION AND CONCLUDING REMARKS

Ideally, the behavior of any physical system in given environmental conditions can be described with suitable mathematical expressions. In reality, however, it is difficult to obtain such a detailed and accurate mathematical representation for the damagement assessment and reliability evaluation of existing structures because of the following reasons:

(a) Although the linear and slightly nonlinear behavior of structures can be successfully analyzed mathematically or numerically, the analysis of structures through various stages of damaging loading conditions remains to be a challenging problem (14).

(b) Most existing civil engineering structures consist of extremely complex systems. The classification and identification of damage states for such complex systems require further studies.

(c) There exist many factors which cannot be evaluated objectively in the damage assessment and reliability evaluation of existing structures.

In current practice, a given structure can be investigated both analytically and experimentally. Voluminous data usually result from these analytical and experimental studies. On the other hand, a typical conclusion that is expected from these studies can be a relatively simple statement, such as "this particular structure has been severely damaged by the recent earthquake". For most structural engineers, the various analytical and experimental procedures in a given investigation can be readily understood. However, the complex decision-making process summarizing the many results of such an investigation into a simple concluding statement remains as privileged and specialized knowledge for a relatively few highly qualified structural engineers. Moreover, the transmission of the precious knowledge of such a decision-making process to younger engineers depends primarily on many years of close working relationships between experienced engineers and their apprentices.

The ultimate goal of the writer is to develope a more direct and systematical methodology so that more engineers can be trained in a shorter time period than it is required at present. With this long-term objective in mind, several approaches were examined following extensive library research into various aspects of the subject area (5,21,22,30, 33). An attempt was made to obtain a damage function (35). To seek a rational framework within which the problem can be formulated, the theory of pattern recognition was considered (13). In addition, an exploratory study of the theory of fuzzy sets was made for possible applications in finding decision functions (34). Preliminary results of analyzing available test data are also presented (6,7).

ACKNOWLEDGEMENT

The writer wishes to thank Jim Noland and Jack Scalzi for their invitation to prepare this review paper for inclusion in the Workshop Proceedings. The support of the National Science Foundation is gratefullly acknowledged. Moreover, the continued encouragement of Drs. M. P. Gaus, S. C. Liu, and J. Scalzi is appreciated.

## APPENDIX I -- REFERENCES

1. Aristizabal-Ochoa, J. D., and Sozen, M. A., Behavior of a Ten-Story Reinforced Concrete Walls Subjected to Earthquake Motions, SRS No. 4 1, Department of Civil Engineering, University of Illinois, Urbana, IL, October 1976.

2. Baldwin, J. W., Jr. Salane, H. J. and Duffield, R. C., Fatigue Test of a Three-span Composite Highway Bridge, Study 73-1, Dept. of Civil Engineering, University of Missouri-Columbia, June 1978.

3. Bjorhovde, R., Galambos, T. V., and Ravindra, M. K., "LRFD Criteria for Steel Beam-Columns", Journal of the Structural Division, ASCE, v. 104, n. ST9, September 1978, pp. 1371-1387.

4. Bresler, B., "Evaluation of Earthquake Safety of Existing Buildings", Developing Methodologies for Evaluating the Earthquake Safety of Existing Buildings, Earthquake Engineering Research Center, University of California at Berkeley, Report No. UCB/EERC-77/06, February 1977, pp. 1-15.

5. Chen, S. J. Hong, Methods of System Identification in Structural Engineering, M.S. Thesis, School of Civil Engineering, Purdue University, West Lafayette, IN 47907, August 1976.

6. Chen, S. J. Hong, and Yao, J. T. p., "Damage Assessment of Existing Structures", Proceedings, Third ASCE EMD Specialty Conference, University of Texas, Austin, TX, 17-19 September 1979, pp. 661-664.

7. Chen, S. J. H., Yao, J. T. P., and Ting, E. C., "Reliability Evaluation of Structural Damage Using Measurable Data", presented at the ASCE Specialty Conference on Probabilistic Mechanics and Structural Reliability, Tucson, AZ, 10-12 January 1979.

8. Cooper, P. B., Galambos, T. V., and Ravindra, M. K., "LRFD Criteria for Plate Girders", Journal of the Structural Division, ASCE, v. 104, n. ST9, September 1978, pp. 1389-1407.

9. Eykhoff, Pieter, System Identification-Parameter and State Estimation, John Wiley & Sons, 1974.

10. Fisher, J. W., Galambos, T. V., Kulak, G. L., and Ravindra, M. K., "Load and Resistance Design for Connectors", Journal of the Structural Division, ASCE, v. 104, n. ST9, September 1978.

11. Freudenthal, A. M., "Safety of Structures", Transactions, American Society of Civil Engineers, v. 112, 1947, pp. 125-180.

12. Freudenthal, A. M., Shinozuka, M., Konishi, I., and Kanazawa, T., Editors, Reliability Approach in Structural Engineering, Maruzen Co., Ltd., Tokyo, Japan. 1975.

13. Fu, K. S., and Yao, J. T. P., "Pattern Recognition and Damage Assessment", Proceedings, Third ASCE EMD Specialty Conference, University of Texas, Austin, TX, 17-19 September 1979, pp. 344-347.

14. Galambos, T. V., and Mayes, R. L., Dynamic Tests of a Reinforced Concrete Building, Research Report No. 51, Department of Civil Engineering, Washington University, St. Louis, MO, June 1978.

15. Galambos, T. V., and Ravindra, M. K., "Properties of Steel for Use in LRFD", Journal of the Structural Division, ASCE, v. 104, n. ST9, September 1978, pp. 1459-1468.

16. Hansell, W. C., Galambos, T. V., Ravindra, M. K., and Viest, J. M., "Composite Beam Criteria in LRFD", Journal of the Structural Division, ASCE, v. 104, n. ST9, September 1979, pp. 1409-1426.

17. Hanson, J. M., Private Communication, 11 June 1977.

18. Hart, G. C., and Yao, J. T. P., "System Identification in Structural Dynamics", Journal of the Engineering Mechanics Division, ASCE, v. 103, n. EM6, December 1977, pp. 1089-1104.

19. Hidalgo, P., and Clough, R. W., Earthquake Simulator Study of a Reinforced Concrete Frame, Report No. EERC 74-13, Earthquake Engineering Research Center, University of Californai, Berkeley, California, December 1974.

20. Hudson, D. E., "Dynamic Tests of Full-Scale Structures", Journal of the Engineering Mechanics Division, ASCE, v. 103, n. EM6, December 1977, pp. 1141-1157.

21. Ibanez, P., et al, Review of Analytical and Experimental Technical Techniques for Improving Structural Dynamic Models, Bulletin 249, Welding Research Council, New York, NY, June 1979, 44 pages.

22. Liu, S. C., and Yao, J. T. P., "Structural Identification Concept", Journal of the Structural Division, ASCE, v. 104, n. ST12, December 1978, pp. 1845-1858.

23. Miller, D. E., A Survey of Structural Identification Techniques, M.S. Thesis, Department of Mechanical Engineering, The University of New Mexico, Albuquerque, NM, August 1979.

24. Ravindra, M. K., Cornell, C. A., and Galambos, T. V., "Wind and Snow Load Factors for Use in LRFD", Journal of the Structural Division, ASCE, v. 104, n. ST9, September 1978, pp. 1443-1457.

25. Ravindra, M. K., and Galambos, T. V., "Load and Resistance Factor Design for Steel", Journal of the Structural Division, ASCE, v. 104, n. ST9, September 1978, pp. 1337-1353.

26. Rodeman, R., and Yao, J. T. P., Structural Identification - Literature Review, Technical Report No. CE-STR-73-3, School of Civil Engineering, Purdue University, December 1973, 36 pages.

27. Sage, A. P., and Melsa, J. L., System Identification, Academic Press, 1971.

28. Task Committee on Structural Safety, "Structural Safety--A Literature Review", Journal of the Structural Division, American Society of Civil Engineers, v. 98, n. ST4, April 1972, pp. 845-884.

29. Task Committee on Structural Safety, "Papers on Structural Safety: Reliability Bases of Structural Safety and Design, by A. H-S. Ang and C. A. Cornell; Rish-Based Evaluation of Design Criteria, by B. R. Ellingwood and A. H-S. Ang; Illustration of Reliability-Based Design, by M. K. Ravindra, N. C. Lind, and W. Siu; Reliability of Structural Systems, by F. Moses; Safety Against Dynamic Forces, by M. Shinozuka; and Fatigue Reliability and Design, by J. T. P. Yao", Journal of the Structural Division, American Society of Civil Engineers, v. 100, n. ST9, September 1974, pp. 1753-1836.

30. Ting, E. C., Chen, S. J. Hong, and Yao, J. T. P., System Identification, Damage Assessment and Reliability Evaluation of Structures, Technical Report No. CE-STR-78-1, School of Civil Engineering, Purdue University, February 1978, 62 pages.

31. Wang, T. Y., Bertero, V. V., and Popov, E. P., Hysteretic Behavior of Reinforced Concrete Framed Walls, Report No. EERC 75-23, Earthquake Engineering Research Center, University of California, Berkeley, California, December 1975.

32. Yao, J. T. P., "Assessment of Seismic Damage in Existing Structures", Proceedings, U.S.-S.E. Asia Symposium on Engineering for Natural Hazard Protection, Edited by A. H-S. Ang, Manila, Phillippines, September 1977, pp. 388-399.

33. Yao, J. T. P., "Damage Assessment and Reliability Evaluation of Existing Structures", Engineering Structures, v. 1, October 1979, pp. 245-251.

34. Yao, J. T. P., An Approach to Damage Assessment of Existing Structures, Technical Report No. CE-STR-79-4, School of Civil Engineering, Purdue University, W. Lafayette, IN, October 1979, 25 pages.

35. Yao, J. T.P., Reliability of Existing Buildings in Earthquake Zones-Final Report, Technical Report No. CE-STR-79-6, School of Civil Engineering, Purdue University, West Lafayette, IN, December 1979.

36. Yao, J. T. P., and Anderson, C. A., "Reliability Analysis and Assessment of Existing Structural Systems", presented at the Fourth International Conference on Structural Mechanics in Reactor Technology, San Francisco, CA, 15-19 August 1979.

37. Yura, J. A., Galambos, T. V., and Ravindra, M. K., "The Bending Resistance of Steel Beams", Journal of the Structural Division, ASCE, v. 104, n. ST9, September 1978, pp. 1355-1370.

Working Group Number: I

Topic: Design Technology Research and Development

Co-Chairmen: Daniel Shapiro[1]
Richard Gensert[2] (not present during Group Meeting)

## INTRODUCTION

Two working groups were held to discuss ideas concerning research and development priorities for the coming decade which are important to masonry design technology. Stimulated by presentations during the previous two days which described masonry research recently completed, in progress or contemplated, each group spent about 45 minutes brainstorming the subject. Both groups followed similar paths of reasoning. The results noted are derived by combining their conclusions.

## CONCLUSIONS

There was an emphasis on the need to develop the background information required for rational design by consistent mathmatical methods which would lead, in turn, to ultimate strength design of masonry.

Specific subjects cited were:

1. The determination of the true state of stress in masonry elements under various loading conditions.

2. Validation of H/t limitations

3. Serviceability considerations

4. Strength superposition concepts

5. Dynamic techniques including wind resistance considerations.

6. Fixity conditions at connections such as between walls and floor slabs

---

[1]Structural Engineer, Shapiro, Okino, Hom and Associates, San Francisco, CA

[2]Structural Engineer, Gensert, Peller and Associates, Cleveland, OH

Topic: DESIGN TECHNOLOGY RESEARCH AND DEVELOPMENT

Co-Chairmen: Daniel Shapiro and Richard Gensert

7. Termal and moisture caused stresses and deflections

8. Shear in composite walls

A second important area for study included the physical characteristics of various combinations of materials. Topics cited were:

1. Bonding - Reinforcing steel to grout
   - Masonry units to grout

2. Use of high stress reinforcing

3. Large diameter bar reinforcement in limited grout spaces as related to bond and bar development

4. Control and expansion joints

5. Quality control and degree of safety desired as they affect design

6. Differential movement caused by creep and thermal effects

7. Weather proofness of structural elements. Deterioration due to moisture

The third group of suggestions for study were in the general area of quality control. These included:

1. Tests required to determine true strength inter-relationships

2. Validation of prism testing techniques as a means of establishing strength - Effect of geometric characteristics on test results

3. Effects of competent inspection on strength

4. Location control on reinforcement. Importance.

Working Group Number : II

# CONSTRUCTION TECHNOLOGY RESEARCH AND DEVELOPMENT

By Howard Noziska[1] and Walter Dicky[2]

The construction phase is where common sense, knowledge and experience must be present to insure project sucess. It was the feeling of those attending this workshop that education and research is needed on many specific topics. The following is a summary of those discussed.

1. Quality Control by Professional Inspection

   a. The ability of the on-site inspector to insure proper construction procedures could be measurably improved by the availability of an "Inspectors Manual". This manual should be written in language that could readily be applied to the field situation. It was also felt that education and registration play an important part in providing quality control assurance.

   b. A complete review of the field test methods should be conducted to qualify and quantify them. Examples of such inspection methods are as follows:

      1. Masonry unit inspection such as the "Initial rate of absorption"

      2. Magnetic reinforcement finder

      3. Aggregate grading funnel

      4. Aggregate moisture content test

      5. Mortar flow tests

   c. Critical points to be covered at a preconstruction conference, and the need to continue to clarify code provisions were also determined to be important considerations. In addition the possibility of limiting uninspected masonry construction should be addressed.

1. Executive Director Minnesota Masonry Institute, Mpls. MN.

2. Consulting Engineer, Los Angeles, CA.

2. Construction Detailing

   a. An area thought to be critical is the connection of floors and roof to walls. Of particular concern was the construction feasibility and in use performance. This should be answered for both reinforced and non-reinforced masonry construction.

   b. Expansion joint and contraction joint configuration and spacing is of concern. Guidance as to types of materials and construction technique also should be considered.

   c. The veneer wall relating to the use of relieving angles as opposed to no horizontal joints should be investigated. Numerous climatic conditions should be included to complete the investigation.

3. Wall Bracing Techniques

   a. A need to develope field guidance to prevent blow over or collapse is evident.

   b. The provisions necessary to provide adequate support when the construction project is at an intermediate stage sould be developed. It is recommended that these requirements include consideration of the wall bracing necessary for the short term as well as for long term support.

   c. Professional responsibility should be established for these provisions.

   d. Methods should be developed to test anchorage capacity on the wall and at the base.

4. Grouting Techniques

   a. The puddle method as contrasted with the vibrating method of installation deserves further investigation.

   b. Limitations and technique to high lift grouting as compared with the low lift method should be determined and illustrated.

   c. Aggregate size and gradation affects on the characteristics of the grout should be reviewed.

   d. Placement of the reinforcing to insure coverage should also be investigated.

5. Reinforcement Concerns

   a. An investigation of reinforcement bar size limitations in the space available should be initiated. Along with this the proximity and size of anchors need to be reviewed and provided to the construction industry.

   b. Methods to hold reinforcing bars specifically in the designed location should be investigated.

   c. When footing dowels are to match vertical wall reinforcement the proper method (direct or in next cell) needs to be researched.

   d. The spacing of vertical wall steel is an important factor in the production and workmanship on the job. Some guidance as to what that spacing might be to accomodate the mason would certainly be beneficial.

6. Additional Topics of Concern

   a. Development of testing methods to determine the value of waterproofing materials and workmanship would be advantageous.

   b. A method to measure the strength of mortar as related to time after mixing should be developed.

   c. A method to determine the value of the corrosion resistance factor of anchors and ties is needed.

Working Group Number : III

Topic: Materials Technology Research and Development

Co-Chairmen: J. Gregg Borchelt[1]
Michael Ward[2]

## INTRODUCTION

This summary of research needs of masonry materials was developed by participants at the Conference on Masonry Research in Progress. Two separate sessions with contributors of various backgrounds were held. This report contains the findings of both sessions. Common subjects have been grouped together and the subjects are listed in order of rated importance with the first felt most needed. Masonry materials include units, as well as mortar and grout and their components, ties and anchors, and masonry assemblages.

## RESEARCH DIRECTIONS

Testing Procedures - Existing test methods and specifications for physical properties of all masonry materials need to be analyzed. Coefficients of variation for the different material properties and number of samples need to be determined to provide a measure of individual test accuracy. A handbook detailing testing procedures should be written to lead to more reliable reproducibility. Certification of testing laboratories, through a series of refereed tests, would aid in obtaining consistent results. It would then be possible to establish a data base for statistical analysis and a confidence level for design.

Unique test methods need to be developed to determine the physical properties of materials. Such tests would be independent of unit geometry or source of material. A test method which would provide a measure of the workmanship of masonry prisms, as a opposed to materials quality control, needs to be developed. Most failure modes in masonry are induced by a tension failure. A test for tension and its relation to other physical properties should be developed.

[1] Executive Director, Masonry Institute of Houston-Galveston, Houston, Texas

[2] Professor of Civil Engineering, University of Calgary, Calgary, Alberta, Canada

Ties and Reinforcing - The effects of corrosion on metal ties, anchors, and reinforcement need to be examined. The value of corrosion resistant coatings in various environmental conditions need to be determined. The amount of cover provided by mortar, grout, and masonry units to protect metal items in masonry is unknown.

Design values for pullout and compression of various ties are needed. Performance of existing tie size and spacing, especially for cavity walls, should be evaluated. Bond and pullout of reinforcing in mortar and grout need to be tested.

Metal Studs - The effect of the relative thickness of metal studs serving as back-up for masonry veneer needs to be evaluated. Load distribution and deflection criteria must be established. Loss of load carrying capacity due to corrosion of the tie/stud connection and the stud/floor or beam connection must be investigated.

Compatibility - The use of the proper mortar and brick combinations was recognized as a key factor in good performance of masonry. Proper bond between these two materials alleviates many potential problems. However, no adequate means of determining mortar/brick compatibility exists. The factors influencing this elusive property of masonry must be identified and a simple means of attaining compatibility achieved.

Additives and Coatings - A variety of additives are available for mortar and grout. These include materials which serve as colors, air entraining or water reducing agents, traditional material substitutes, accelerators, retardants, and cold weather aids. The effect of these additives on the durability and performance of mortar and grout and on the items embedded in mortar and grout must be known before they can be used.

Masonry unit manufacturers are experimenting with substitution of materials in the production of their products. Sawdust or coal is mixed with clay to reduce firing requirements, aggregate used in concrete masonry units may change from outside influences. The properties of the resulting units must be thoroughly examined. Existing specifications may not be adequate to provide sufficient durability.

Clear coatings are often applied to masonry walls to reduce water penetration. Little testing has been done to determine the properties these coatings should have to provide this water penetration resistance without causing damage in the masonry.

The remaining topics were felt to be of interest and should be examined:

- The majority of mortar testing is oriented to wall construction. Properties of mortar to be used in paving applications need to be determined.

- The best geometric configuration of cores in hollow brick to reduce bursting due to grout expansion.

- The effect of the age of cement on mortar and grout properties.

- The substitution of mortar for grout.

- Air penetration of walls.

- Water permeance of masonry units.

- Wet seive analysis of mortar.

# WORKING GROUP NUMBER: IV
# COMMUNICATION AND COOPERATION
# AMONG
# MASONRY RESEARCHERS, RESEARCH CONSUMERS AND RESEARCH SPONSORS

By E. L. Jessop[1] and J. E. Amrhein[2]

## PREAMBLE

The authors were co-chairmen of a discussion group seeking ways and means of effectively communicating the results of research to design and construction personnel, fostering cooperation between researchers working in different laboratories and bringing together researchers and research sponsors (industry and government) to jointly identify research needs.

This paper is based on the comments and ideas that were forthcoming from the group.

## COMMUNICATING THE RESULTS OF RESEARCH TO THE RESEARCH CONSUMER

The most effective means of communicating the results of research to the research consumer, i.e. design and construction personnel, are (in descending order of value, according to the participants):

1. Using the results to effect a code change. Since codes ought to reflect current knowledge, this is the most obvious way to communicate the results. It was recognized that the process of bringing about a change in any code is time consuming. Further, it was recognized that "Authorities having jurisdiction" would be negligent in accepting approaches to design and construction practice that were outside of the code unless (a) the proponent of the new approach was known to be competent and would accept the responsibility that goes with operating outside of the code, and (b) "adequate" research evidence was submitted to support the new approach to be adopted. Subject to the preceding requirements being met in a particular instance it was suggested that approval might be given on the basis of a code change only being made at a later date after satisfactory in-situ performance had been demonstrated.

[1]Director General, Centre for R & D in Masonry, Calgary, Alberta, Canada

[2]Director of Engineering, Masonry Institute of America, Los Angeles, California, U. S. A.

Under item (a) the proponent can obviously only be the designer; Under item (b) the responsibility for providing "adequate" research evidence should probably be a committee, which includes the researcher. Further, the committee should operate under an 'umbrella' organization to give it stature. The Masonry Society was suggested as the organization which should accept this responsibility.

2. Presenting the results at technical meetings. It was pointed out that there have been several major meetings held over the last ten years and each has resulted in a publication.

Major meetings include: International Brick Masonry Conference (U.S.A., 1967; England, 1970; Germany, 1973; Belgium, 1976; U.S.A., 1979); Load-Bearing Brickwork (British Ceramic Society, England, six conferences); First and Second Canadian Masonry Symposiums (Calgary, 1976; Ottawa, 1980); North American Masonry Conference (Boulder, Colorado, U.S.A., 1978).

To ensure that researchers publish their work as expeditiously as possible the suggestion was made that the Research Sponsor (frequently the National Science Foundation) require that the results be reported at a conference of the Research Sponsors choosing.

NOTE: It is of interest that in Canada, the Canadian Masonry Research Council recently commissioned three state-of-the-art reports on masonry topics and made it a condition of the Masonry Research Foundation of Canada grant that the reports be presented at the Second Canadian Masonry Symposium to be held in Ottawa, Ontario in June, 1980.

A further suggestion was made that the highlights of each technical meeting be made the subject of a slide/tape cassette presentation which could then be made available to interest groups across the continent. Once again, The Masonry Society was suggested as the organization which should perform this important task.

3. Writing articles in recognized technical journals. There are numerous such journals, i.e. Journal of the American Society of Civil Engineers, Structural Division, Journal of the American Concrete Institute, Canadian Journal of Civil Engineering. If, however, a researcher wishes to communicate with the contractor he may submit articles to "Masonry"; if he wishes to communicate with Clay Manufacturers he may submit articles to "Brick and Clay Record". And so on.

Not all issues of , say, the American Society of Civil Engineers contain articles on masonry topics. An article written in 'Masonry' on a construction topic may be of interest to a designer.

What is needed for the masonry industry is a recognized technical journal to which all who have an interest in masonry may subscribe. The Masonry Society has looked at the possibility of producing such a journal, and are still considering doing so. However, a journal has already been started, The International Journal of Masonry Construction, published by United Trade Press, England. With an editorial board under the general editorship of Professor Arnold Hendry of the University of Edinburgh, Scotland, U. K., the Journal fulfills the need outlined on the previous page.

4. Writing textbooks, on the basis of research results, for use in undergraduate and continuing education courses of engineering at universities and technical colleges.

While low in priority, because such an endeavour is inevitably time consuming and the process of education a slow one, this means of communication was nevertheless recognized as a very valuable one.

While there is nothing revolutionary in these recommendations for communication, each has its merits and drawbacks and obviously all should continue to be used. Of particular interest, however, was the recurring theme, particularly in the first three items, that some organization should accept responsibility for coordination - and that organization should be The Masonry Society.

## COOPERATION BETWEEN RESEARCHERS

Cooperative research programs, where several researchers working in different laboratories work on different aspects of the specified project according to their particular expertise and available facilities, were deemed to be generally desirable.

It was recognized that the sharing of ideas among potential participants in the planning stages of such a research program is extremely important. Two means of sharing ideas were discussed:

1. Invite prospective researchers to a specially convened meeting to discuss required research: this implies the need for a coordinating organization which can take the necessary initiative. The Masonry Society could fulfill this role. Such an organization would maintain an interest in any cooperative program established by this means, but the researchers themselves would be responsible for the program, acquisition of the necessary funds and the communication of the results.

2. Solicit proposals on particular topics in need of research and select from among the submissions those which best fit the concept of a cooperative program: this implies the need for a research oriented committee which not only defines the topics but also acts in a "management" role and as a "go-between" the researcher and the research sponsor.

NOTE: Such committees are, in fact, already established. The Canadian Masonry Research Council was created in 1975 to fulfill this role and a Research Advisory Committee to the Masonry Industry Council was established in 1979 in the U. S. A. for the same purpose.

Perhaps The Masonry Society should seek to have a voice in the Masonry Industry Council?

In any event it must be recognized that cooperative research projects don't just happen: they require considerable expenditures of time and effort on the part of an organizing body, and that body should either have the technical "know-how" itself or have access to an advisory committee of technical experts.

## COMMUNICATION AND COOPERATION BETWEEN RESEARCHERS AND RESEARCH SPONSORS

Researchers are always seeking funds and, it has to be admitted, do not always prepare their case adequately. In ignorance, because of a lack of information readily available to them, much duplication of effort does occur.

Research sponsors, on the other hand, particularly of the industry variety, have a tendency to be somewhat paranoid about the dangers of "re-inventing the wheel".

The participants felt that the only way to overcome these two problems would be to annually convene "Research Planned" conferences - a step before "Research in Progress" conferences - to which researchers, (possibly), research consumers and research sponsors could be invited.

## ROLE OF THE MASONRY SOCIETY

The Masonry Society was created in 1978, modelled after the World renowned and highly successful American Concrete Institute. Its uniqueness vis-a-vis existing masonry organizations is that its membership is open to all who have an interest in masonry, be they product manufacturers, contractors, union and non-union bricklayers, designers, researchers, educators and even the general public if they so desired. Further, the Society's interest embraces all types of masonry, be it clay, concrete, stone or other. No other masonry organization in existance in North America can make this claim - most, if not all, do not include researchers, designers and educators who are the new blook this industry so desperately needs. Indeed, the vast array of research talent and facilities housed at our Universities has largely been ignored by the masonry industry in the past in preference for confining their industry groups' technical activities to their own in-house laboratories and offices from which they can control what is published. The Masonry Society, unlike the secret societies of free-masons in history, holds forth the promise of openess, which is an essential ingredient if technical progress is to be achieved.

The participants, without dissent, recognized the value of such a Society and identified roles it could play in promoting effective communication and cooperation among researchers, research consumers and research sponsors.

## CLOSING REMARKS

In this paper the authors have not only attempted to synthesize the comments and ideas of a group discussion into a presentable format but have gone further, and included their own interpretation and views on this important subject.

## ACKNOWLEDGEMENT

The authors would like to express their gratitude to the National Science Foundation, which provided the funds to support the MASONRY RESEARCH IN PROGRESS seminar at which the discussions relating to this paper were held.

Appendix A

# REGISTRATION LIST

Conference on

MASONRY RESEARCH IN PROGRESS

Marina Del Rey, CA

March 11-12, 1980

Sponsored by the

NATIONAL SCIENCE FOUNDATION

John B. Scalzi - Program Manager
James L. Noland - Conference Organizer
& Co-Chairman
James E. Amrhein - Co-Chairman
Theresa K. Noland - Coordinator

A-2

SPEAKERS/AUTHORS


Samy Adham (Dr)
Agbabian Associates
250 N. Nash
El Segundo, CA 90245
(213) 640-0576

Subhash Anand (Dr)
Dept. of Civil Engineering
Clemson University
Clemson, S.C. 29631
(803) 656-3000

Russell Brown (Dr)
Dept. of Civil Engineering
Clemson University
Clemson, S.C. 29631
(803) 656-3000

Clayford T. Grimm
Arch. Engineering Department
University of Texas-Austin
1904 Wooten Dr.
Austin, TX 78758
(512) 471-1733

Gilbert A. Hegemeir (Dr)
Dept. of Applied Mechanics
and Engr. Sciences
B-010
University of California-
San Diego
La Jolla, CA 92093

John Kariotis
Kariotis, Kesler, & Allys
1414 Fair Oaks Avenue
South Pasadena, CA 91030

Ron Mayes (Dr)
Research Engineer
University of California-
Berkeley
Executive Director
Applied Technology Council
480 California Ave., Suite 205
Palo Alto, CA 94306
(415) 326-4029

Hugh McNiven (Dr)
Earthquake Engineering
Research Center
1301 S. 46th St.
Richmond, CA 94084
(415) 231-9559

James L. Noland (Dr)
Atkinson-Noland & Associates
2619 Spruce St.
Boulder, CO 80302
(303) 444-3620

Robert O. Nunn (Dr)
Dept. of Applied Mechanics
and Engineering Sciences
University of California-San Diego
La Jolla, CA 92093

Gordan Plewes
Consulting Engineer
40 Delong Drive
Ottawa, Ontario
Canada K1J 8H4
(613) 749-1142

Henry Russell (Dr)
Portland Cement Association
5420 Old Orchard Road
Skokie, Illinois 60077
(312) 966-6200

John B. Scalzi (Dr)
National Science Foundation
Room 1130
1800 G Street N.W.
Washington, D.C. 20550
(202) 632-0648

Gary Suter (Dr)
Dept. of Civil Engineering
Carleton University
Ottawa, Ontario
Canada K1S 5B6
(613) 231-3622

A-3

Ben Schmid
Consulting Engineer
2500 East Foothill Blvd.
Pasadena, CA 91107
(213) 795-2619

Roland Sharpe
Managing Director
Applied Technology Council
480 California Ave., Suite 205
Palo Alto, CA 94306
(415) 326-4029

RESPONDERS

James Amrhein
Director of Engineering
Masonry Institute of America
2550 Beverly Blvd.
Los Angeles, CA 90057
(213) 388-0472

Roger Benson
Benson and Gerdin
Consulting Engineers
3150 N. 7th Street
Phoenix, AZ 85014
(602) 264-0136

Russell Brown (Dr)
Dept. of Civil Engineering
Clemson University
Clemson, S.C. 29631
(803) 656-3000

Vincent Bush
Asst. Technical Director
International Conference of Building Officials
5360 S. Workman Mill Road
Whittier, CA 90601
(213) 699-0541

Walter Dickey
Consulting Engineer
1014 Fortune Way
Los Angeles, CA 90042
(213) 255-6049

Ahmad Hamid (Dr)
School of Civil Engineering and Environmental Science
University of Oklahoma
202 W. Boyd
Norman, OK 73019
(405) 325-5911

Gary C. Hart (Dr)
Ruthroff, Englekirk & Hart
3242 W. 8th Street
Los Angeles, CA 90005
(213) 385-9487

Clar Monk
Wiss, Janney, Elstner and Associates
330 Pfingsten Road
Northbrook, IL 70062
(312) 272-7400

Lawrence G. Selna (Dr)
Engineering Department
University of California-Los Angeles
3173 F. Engineering I
Los Angeles, CA 90024
(213) 825-5502

Earl Schwartz
Sr. Structural Engineer
Management Assistance Div.
Room 407
Dept. of Building & Safety
200 N. Spring St.
Los Angeles, CA 90012

John Tawressey
Consulting Engineer
KPFF
Lloyd Building
Seattle, Washington 98101
(206) 622-5822

A-5

## MODERATORS


James Amrhein
Director of Engineering
Masonry Institute of America
2550 Beverly Blvd.
Los Angeles, CA 90057
(213) 388-0472

Richard Henry Atkinson (Dr)
Atkinson-Noland & Associates
2619 Spruce St.
Boulder, CO 80302
(303) 444-3620

Albin Johnson
S. B. Barnes & Associates
2236 Beverly Blvd.
Los Angeles, CA 90057
(213) 382-2385

Gary Suter (Dr)
Dept. of Civil Engineering
Carleton University
Ottawa, Ontario
Canada K1S 5B6
(613) 231-3622

Donald Wakefield
Interstate Brick Co.
9210 S. 5200 W.
Box 517
West Jordan, UT 84084
(801) 561-1471

Michael Ward (Dr)
Civil Engineering Dept.
University of Calgary
2920 24th Ave. N.W.
Calgary, Alberta
Canada T2N 1N4
(403) 284-5820

## WORKING GROUP CO-CHAIRMEN

Daniel Shapiro
Shapiro, Okino, Hom & Assoc.
1736 Stockton Street
San Francisco, CA 94133
(415) 781-8105

Richard Gensert
Gensert, Peller & Associates
718 The Arcade
Cleveland, OH 44144
(216) 241-7078

Michael Ward (Dr)
Civil Engineering Dept.
University of Calgary
2920 24th Ave. N.W.
Calgary, Alberta
Canada T2N 1N4
(403) 284-5820

Gregg Borchelt
Masonry Institute of Houston-Galveston
5100 Westheimer
Houston, TX 77027
(713) 629-6024

Howard Noziska
Executive Director
Minneapolis Masonry Institute
7851 Metro Parkway, #103
Minneapolis, MN 55420

Walter Dickey
Consulting Engineer
1014 Fortune Way
Los Angeles, CA 90042
213-255-6049

James Amrhein
Director of Engineering
Masonry Institute of America
2550 Beverly Blvd.
Los Angeles, CA 90057
(213) 388-0472

Emlyn Jessop (Dr)
Centre for Research and Development in Masonry
105 4528 6A Street N.E.
Calgary, Alberta
Canada T2E 4B2
(403) 277-3024

A-7

# ATTENDEES

Hugh Burgess
College of Architecture
Arizona State University
Tempe, AZ 85281

James Usui
City of Los Angeles
Building Department
City Hall
Los Angeles, CA 90012

Irwin Salin
Office of the State Architect
107 S. Broadway
Los Angeles, CA 90012

Hal Dewdney
Office of the State Architect
107 S. Broadway
Los Angeles, CA 90012

Robert Alexander
Civil Engineering Dept.
California State University
Long Beach, CA 90840

Stuart Beavers
Concrete Masonry Association
of California & Nevada
83 Scripps Drive, Suite 303

Robert Ewing
Agbabian Associates
250 N. Nash St.
El Segundo, CA 90245

Joseph Warwaruk
Dept. of Civil Engineering
University of Alberta
Edmonton, Alberta
Canada T6G 2G7

Harry Wiewel
Ramset Fastening Systems
6023 Greentop St.
Lakewood, CA 90713

Ben McVicker
Long Beach City Building Dept.
333 West Ocean Blvd.
Long Beach, CA 90802

Saeed Amanullah
Los Angeles County
Building & Safety Division
532 South Vermont Ave.
Los Angeles, CA 90020

Theodore Morrison
Los Angeles County
Building & Safety Division
532 S. Vermont Ave.
Los Angeles, CA 90020

Lawrence Kahn
Civil Engineering Dept.
Georgia Institute of Technology
Atlanta, GA 30332

Raymond Anderson
Agbabian Associates
250 N. Nash St.
El Segundo, CA 90245

Timothy Healy
Agbabian Associates
250 N. Nash St.
El Segundo, CA 90245

Tom McNeilly, Director
Brick Development Research Inst.
Architecture Building
University of Melbourne
Parkville, Victoria 3052
Australia

Russell Mills
URS/John Blume and Associates
130 Jessie St.
San Francisco, CA

Jim Tanouye
U.S. Army
Corps of Engineers
San Francisco, CA

Norman Takajuji
City of Los Angeles
Dept. of Building & Safety
City Hall, Room 423
Los Angeles, CA 90012

Yuri Sheydayi
College of Architecture
Arizona State University
Tempe, AZ 85281

Daniel Abrams
Dept. of Civil, Environmental
and Architectural Engineering
University of Colorado
Boulder, CO 80309

Joseph Plecnik
Civil Engineering Dept.
California State University

ATTENDEES

George Fosdyke
George Fosdyke & Associates, Inc.
459 South Bixel St.
Los Angeles, CA 90017

Carl Engelbrecht
George Fosdyke & Associates, Inc
459 South Bixel St.
Los Angeles, CA 90017

Jack Doherty
George Fosdyke & Associates, Inc.
459 South Bixel St.
Los Angeles, CA 90017

Atis Malmanis
City of Santa Rosa
100 Santa Rosa Ave.
Santa Rose, CA 95401

J.B. Morgan
J.B. Morgan & Associates, Inc.
102 North Cascade, Suite 470
Colorado Springs, CO 80903

Leon Salveson
Masonry Industry of Washington
2366 Eastlake Ave. E.
Seattle, Washington 98008

Marvin Hopewell
Planning and Building Dept.
City of Long Beach
333 W. Ocean Blvd.
Long Beach, CA 90802

# Author Index