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Review of Research Literature on Masonry Shear Walls

C.W. C. Yancey
S. G. Fattal
R. D. Dikkers

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U.S. Department of Commerce
Robert A. Mosbacher, *Secretary*
National Institute of Standards and Technology
John W. Lyons, *Director*
Building and Fire Research Laboratory
Gaithersburg, MD 20899

CONTENTS

Abstract

A review of the technical literature on masonry shear wall tests was conducted to determine the range and depth of available research and to identify areas in need of additional research. The review covers documents published from 1976 to 1989 and includes approximately seven hundred masonry wall tests. Both U.S. and foreign research was included in the review. U.S. code and standard requirements for the design of masonry shear walls are discussed. Some of these requirements are highlighted in tabular and graphic form.

Technical information regarding experimental studies is tabulated for easy reference. A selected number of test programs are examined in greater detail to present the objective and scope, test variables and major findings.

Experimental data from comparable research studies are combined and analyzed to determine the influence of key design parameters on the performance of shear walls to in-plane lateral loading. Also included is a comparison of two experimentally-derived shear strength formulae with the shear strength provisions of the 1988 edition of the Uniform Building Code. The findings of the review are summarized and specific research needs are identified.

Key words: code requirements, design, full-scale tests, lateral loading, masonry shear walls, research, shear strength, standards, test methods,

1. INTRODUCTION

At the outset of the NBS[1] Masonry Research Program in FY83, there was a relatively well-defined need for additional information on the performance of plain and partially-reinforced masonry shear walls subjected to earthquake loading. At that time, there was no comprehensive national standard for the design of masonry structures, nor were there any provisions for strength design. In the intervening years, the masonry industry and design and construction practitioners have made progress to the point where there is now a national standard for masonry structures (ACI 530-88/ASCE 5-88) and one national model building code (1988 edition of Uniform Building Code) that contains provisions for strength design of reinforced masonry walls. Currently, the masonry industry is supporting the development of limit state design provisions, with a first draft scheduled for late 1990. In 1984, a Joint U.S.-Japan Technical Coordinating Committee on Masonry Research (JTCCMAR) was established under the sponsorship of the National Science Foundation (NSF) with two primary purposes: (1) to develop a basic knowledge of masonry material behavior based on material and small-scale masonry tests, and 2) to build and validate computer models for seismic response analysis and design using information from tests of small-scale masonry specimens, tests of masonry structural components and tests of a full-scale masonry research building. The respective scopes of the Japanese and U.S. research programs range from defining constitutive material properties to the testing of a five-story concrete masonry building. It is anticipated that the parallel programs will contribute significantly to seismic design and construction applied to masonry structures, particularly in areas of high seismic risk. These developments have created the need for reevaluating the objectives and scope of the NIST Masonry Research Program.

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[1] The National Bureau of Standards (NBS) was reorganized in August 1988 into the National Institute of Standards and Technology (NIST). In this report references are made to both organizational acronyms to maintain chronological accuracy.

2. OBJECTIVES AND SCOPE

The objectives of this study are: 1) to evaluate existing knowledge on the behavior of masonry shear walls under earthquake loads; 2) to identify research needs and priorities; and 3) to develop an NIST plan which compliments other masonry research activities.

Chapter 3 presents an overview of masonry shear wall research conducted at NBS/NIST (Sect. 3.1) and elsewhere (Sect. 3.2). The technical information presented is condensed from publications on masonry research conducted in the U.S. and abroad during the period 1976 through 1989. Chapter 4 discusses requirements for the design of masonry shear walls by U.S. codes and standards. Chapter 5 condenses and classifies available experimental information on shear walls contained in various publications. Chapter 6 presents analyses of existing research data and building code provisions, and identifies research needs based on this analysis. Chapter 7 summarizes the findings of this study and Chapter 8 identifies specific research priorities based on this study.

3. BACKGROUND

3.1 NBS/NIST Research

In September 1976, an NSF-sponsored workshop was held at the National Bureau of Standards in Boulder, Colorado to address existing problems in the areas of code requirements, design criteria, mathematical models, test standardization and material properties, and rehabilitation for masonry construction. The workshop participants were charged with identifying research needs to support the evolution of earthquake-resistant design and construction of masonry structures. Following the presentation of papers in a general session, the workshop was organized into working groups whose titles coincided with the problem areas cited above. The scopes of the working groups were established, and the groups were asked to recommend research topics that could potentially be funded by the National Science Foundation.

The <u>Code Requirements Working Group</u> developed the following prioritized list of research subjects aimed toward the development of requirements for seismic design of masonry structures.

1. Reinforcement requirements in masonry shear walls in terms of distribution, size, and minimum amount allowable.
2. Anchorage requirements and performance of bolts, connections, and hangers.
3. Ultimate strengths of masonry assemblages under static and cyclic loading.
4. Requirements pertaining to height-to-thickness ratios.
5. Roof diaphragm-shear wall connections.
6. Coupling and spandrel beams.
7. Reinforcement bond strengths and embedment requirements.
8. Floor slab-shear wall connections.
9. Proper splicing and placement of reinforcement.
10. Damping in masonry buildings.
11. Reducing vertical shrinkage cracks in concrete masonry walls.
12. Stack bond masonry performance.
13. Study effects of vertical acceleration and necessity for code requirements.
14. The possible need for separate code allowable stresses for different materials (e.g., hollow clay units, brick, concrete block, composite, etc.).
15. Methods to eliminate concrete block face shell bond failures.

The <u>Mathematical Models Working Group</u> cited the need for the development of a mathematical model for masonry structures that would serve as: 1) a research tool, 2) a design verification tool, and 3) a basis for developing simple models that may be used by designers. Specifically, it was recommended that a three-dimensional, nonlinear mathematical model be developed, incorporating hysteretic behavior and using a deterministic approach to simulate the earthquake response of masonry buildings. What was envisioned was a phased development process which would proceed from the formulation of stress-strain element models, through the assemblage of component and substructure models, using the Finite Element Method, and ending with the incorporation of substructure models into a mathematical model of an entire structure. It was acknowledged that a complex three-dimensional, dynamic analytical model would likely be utilized in other research programs and for the verification of rational design. The use of the analytical model in conjunction with full-scale laboratory testing could lead to the development of improved design procedures and improved simplified design models for designers.

The <u>Test Standardization and Material Properties Working Group</u> recommended the development of standard methods to evaluate compressive, shear and flexural strengths of masonry. It recommended research to improve the two existing methods of establishing compressive strength, f'_m: 1) testing of prisms consisting of the constituent materials and 2) the assignment of a value for f'_m based on the compressive strength of the masonry unit and the mortar type.

The use of ASTM Test Method E 519 as a means of establishing shear strength of walls was discussed. The group recommended research on this standard in four specific areas: 1) feasibility of reducing the size of the specimen from the specified four-foot square dimensions, 2) expanding the scope of the procedure to include tests of composite masonry specimens, 3) standardization of specimen size, loading fixtures, capping, loading rates, etc., and 4) standardized data interpretation.

The working group also recommended research to improve the state-of-the-art on the evaluation of flexural tensile properties of masonry. As an example, the group suggested research to correlated flexural test results obtained from small-scale testing (e.g., ASTM Test Method E 518) with those from full-scale masonry tests. Additional information on the NSF-sponsored workshop is found in reference [12].

In 1977, the National Earthquake Hazards Reduction Act was passed by the U.S. Congress. The National Bureau of Standards (NBS) was assigned a mission by this legislation, to assist in the development of improved design procedures for buildings subject to earthquakes. To support this legislated mission, the Center for Building Technology designed and built the Tri-directional Test Facility (TTF) during 1980-81. With the completion of the TTF, NBS developed the physical capacity to perform three-dimensional, cyclic tests on specimens as large as 3 m long by 3 m deep by 3.5 m high.

In the past two decades a series of experimental research projects were conducted at NBS to study the behavior of masonry elements under various loading configurations. The bulk of the experiments in the 1975-80 period focused on the performance of masonry shear walls, both unreinforced and containing horizontal joint reinforcement, under in-plane loads. Verification and interpretation of standard test methods such as ASTM E 564, E 72, and E 519, were primary objectives of these projects [12,33,86]. In particular, the ASTM E 519 Test Method was used extensively with masonry shear walls under various combinations of axial and diagonal loads. The results of these tests were used to corroborate NBS-derived failure hypotheses for predicting the strength (shear cracking limit state) of masonry shear walls [86].

During Fiscal Year 1984, NBS began a masonry research program which addressed a need identified at the 1976 NBS/NSF workshop and was linked to the NBS mission under the National Earthquake Hazards Reduction Program (NEHRP). The primary objective of the NBS Masonry Research Program was to characterize the performance of reinforced as well as unreinforced masonry shear walls. The scope of the program was heavily influenced by the actions and recommendations of Technical Committee 5 (on Masonry), of the Applied Technology Council (ATC), documented in ATC 3-06 Tentative Provisions for the Development of Seismic Regulations for Buildings. Committee 5 was responsible for the review and refinement of Chapters 12 (Masonry) and 12A (Masonry Construction) of the Tentative Provisions. The committee and affiliation held general formal meetings during 1980, and recommended substantial changes in the original draft of Chapters 12 and 12A. A number of the issues discussed at the committee meetings, as documented in the committee report "Review and Refinement of ATC 3-06 Tentative Seismic Provisions" (NBSIR 80-2111-5, National Bureau of Standards, Washington, DC, October 1980) dealt with the need for more information on the shear capacity of masonry walls, both reinforced and unreinforced. Such information was deemed critical in the development of Φ factors and mathematical formulations to predict shear strength. Thus, the NBS research plan as outlined in 1983 had as its major objective the defining of the shear capacity and behavior of shear-dominated walls.

The scope of the NBS Masonry Research Program encompassed testing of both unreinforced and reinforced masonry shear walls. Testing of unreinforced specimens was completed during FY 84-85 and the results were documented in three reports [80,81, & 82]. A total of thirty-two walls were subjected to a combination of lateral in-plane shear force of increasing magnitude and an axial compressive load of constant magnitude. The test variables were: magnitude of axial compressive stress, wall aspect ratio, type of mortar, and compressive strength of masonry units. At the beginning of FY87, a two-year test program was prepared to study the behavior of reinforced masonry shear walls. Fourteen concrete masonry walls containing grouted bond beams and varying amounts and distributions of horizontal reinforcement were built and tested by mid-FY88.

Subsequently, eight concrete masonry walls containing varying amounts and distributions of vertical reinforcement were built. In addition, twelve square, horizontally-reinforced walls were built for testing by ASTM Test Method E 519 (Standard Test Method for Diagonal Tension in Masonry Assemblages). The latter testing was an addendum to the original scope of the reinforced masonry phase of the program. It resulted from recommendations advanced by the masonry industry and by ASTM Subcommittee E06.11 (on Performance of Horizontal and Vertical Structures) that research was needed to confirm the applicability of Test Method E 519 to the testing of reinforced masonry walls for determining ultimate shear capacity. These tests have been delayed pending the completion of the study discussed in this report.

3.2 TCCMAR and Other Research

A substantial amount of the U.S. research in the area of seismic effects on masonry assemblages has been sponsored by the National Science Foundation (NSF) under the National Earthquake Hazards Reduction Program (NEHRP) and the masonry industry. The most comprehensive of the U.S. programs and the one that promises to make the most significant impact on masonry seismic design practice is being conducted by the Technical Coordinating Committee for Masonry Research (TCCMAR). TCCMAR was established in February 1984 under NSF sponsorship and is currently progressing toward the testing of a full-scale masonry research building. The TCCMAR research effort however, includes only fully-grouted, fully-reinforced wall elements, with the reinforcement distributed uniformly. Unreinforced walls and walls with nominal levels of reinforcement are not included in the TCCMAR program. Summarizing the current status of the TCCMAR program, the masonry materials studies have been completed, a large percentage of the component testing (i.e., floor diaphragms, story-high shear walls) has been completed and some of the assemblages (e.g., coupled shear walls and flanged shear walls) have been tested. In addition, several mathematical models (i.e., Finite Element Model and Structural Component Model) have been developed and are undergoing validation. Plans are also being developed for the construction and testing of a five-story masonry building.

The TCCMAR program is being conducted in tandem with a similar Japanese research effort and together they form the U.S.-Japan Coordinated Program for Masonry Building Research (JTCCMAR). The motivation for the Japanese participation in the coordinated research program was based on the recognized need for developing a strong technical information base on masonry materials and assemblage performance. Heretofore, the use of masonry for building construction in Japan was limited. Building codes set a height limitation of three stories and contained other largely empirical requirements. The experimental portion of the Japanese research program is complete, including the testing of a five-story, full-scale, reinforced concrete masonry building during 1987. The building was subjected to cyclic lateral loading up to its capacity.

The U.S. masonry industry, as represented by the Council for Masonry Research (CMR), Masonry Institute of America (MIA), Concrete Masonry Association of California and Nevada (CMACN), and Western States Clay Products Association (WSCPA), has been supportive of the TCCMAR program and also has conducted or funded independent research programs on masonry assemblages. CMR consists of national masonry organizations such as the Brick Institute of America, National Concrete Masonry Association, Portland Cement Association, the Mason Contractors Association of America and The Masonry Society. During the period of 1980 to 1982 industry-funded testing was conducted on 32 masonry walls under combined axial and out-of-plane lateral loads to demonstrate the performance of load-bearing masonry walls with slenderness ratios (h/t) greater than 25. The then-current model building codes imposed a slenderness ratio limitation of 25 on load-bearing masonry walls, primarily because of the lack of experimental data to support higher h/t values.

As the scope of the TCCMAR program extends only to the development of recommendations for design and performance criteria, the masonry industry is supporting a nationwide effort to develop a consensus standard for limit state design of masonry structures. The TCCMAR recommendations are being translated into a resource document which forms the basis for the draft limit state design standard. The Masonry Society (TMS) is formally charged with administering this program. The present timetable calls for completion of a draft standard by late-1990. Adoption as an American National Standards Institute (ANSI) consensus standard is expected by 1996.

4. CODE PROVISIONS FOR SEISMIC DESIGN

4.1 Scope of Review

Five documents, including building codes and recommended code provisions have been reviewed to examine their seismic provisions for masonry shear walls. Of particular interest were the following items: 1) special seismic provisions related to masonry construction, 2) the seismic risk map incorporated in the code or standard, 3) the building height limitations for buildings using masonry shear walls to resist lateral loads, and 4) provisions for Strength Design. Table 4.1 summarizes the seismic provisions for masonry walls as found in the: 1) Uniform Building Code - 1988 Edition, 2) Building Code Requirements for Masonry Structures, ACI 530-1988/ASCE 5-88, 3) NEHRP Recommended Provisions for the Development of Seismic Regulations for New Buildings - 1988 Edition, 4) Standard Building Code - 1988 Edition, and 5) BOCA National Building Code - 1987 Edition.

4.2 Synopses of Masonry Shear Wall Provisions

Uniform Building Code (1988 Edition)

The Uniform Building Code (UBC) is prevalent in 21 states, all but one of which are located west of the Mississippi River. The UBC incorporates a six-zone (0, 1, 2A, 2B, 3, & 4) seismic risk map with only parts of California and Nevada falling in the highest risk zone (i.e., Zone 4). Chapter 24 of the UBC contains the requirements for masonry construction and section 2407 contains special seismic provisions for masonry. As shown in table 4.2, there are no special seismic provisions required for buildings or structures in Zones 0 and 1. Masonry structures in Seismic Zone 2 are prohibited from using certain constituent materials in the structural frame. In addition, Section 2407 specifies the minimum area of vertical and horizontal reinforcement, locations that must be reinforced, and maximum spacing of the vertical and horizontal reinforcement (see figure 4.1). There are additional reinforcement requirements for stack bond construction. Structures constructed in Seismic Zones 3 and 4 have additional limitations (see figure 4.2). The additional provisions require that the minimum nominal thickness of reinforced masonry bearing walls be 6 in. The use of 4-in-thick load-bearing reinforced hollow-clay unit masonry walls is a permissible exception. Section 2407 requires shear reinforcement to be uniformly distributed and prescribes standard hooks as anchorage for shear reinforcement.

A building height limitation of 160 ft is specified for masonry buildings located in Seismic Zones 3 or 4. As an alternative to using the Working Stress Design method, Section 2412 contains a Strength Design approach for reinforced masonry shear walls. Load and resistance factors and formulae for calculating nominal axial and shear strengths are prescribed therein.

NEHRP Recommended Provisions for the Development of Seismic Regulations for New Buildings - 1988 Edition

Section 1.4 of the NEHRP provisions defines seismic performance as a measure of the degree of protection provided for the public and building occupants against the potential hazards resulting from the effects of earthquake motions on buildings. The level of seismicity and the Seismic Hazard Exposure Group are used in assigning buildings to Seismic Performance Categories. The seismicity level is expressed in terms of a Seismicity Index and is determined by referring to one of two seismic risk maps which present seven zones of Effective Peak Acceleration and Effective Peak Velocity-Related Acceleration.

Three Seismic Hazard Exposure Groups are defined. Group III contains buildings having essential facilities that are necessary for post-earthquake recovery. Group II covers buildings with large public assembly areas, jails, etc. and Group I includes all buildings not covered by Groups II and III. All buildings are assigned to a Seismic Performance Category based on the Seismicity Index and Seismic Hazard Exposure Group. There are five Seismic Performance Categories, A,B,C,D, and E, with Category E being assigned the highest level of design performance criteria.

Chapter 12 (Masonry) presents requirements for the design, construction, and quality assurance of masonry components that are to resist seismic forces. Strength reduction factors are listed for various masonry structural members in compression, shear, tension or other stress combinations and for member connections to account for uncertainties in material properties and construction quality. The Building Code Requirements for Masonry Structures (ACI 530-88/ASCE 5-88) and Specifications for Masonry Construction (ACI 530.1-88/ASCE 6-88) are cited as the reference documents upon which the design should be based. Response Modification Factors, R, for both reinforced and unreinforced masonry are included in NEHRP Table 3-2. R values are used in the calculation of the Seismic Coefficient, C_s, which is the multiplier for the building weight to obtain an equivalent seismic base shear. The allowable working stresses listed in ACI 530-88/ASCE 5-88 are multiplied by the factor 2.5 to obtain approximate strength values.

<u>Building Code Requirements for Masonry Structures (ACI 530-88/ASCE 5-88)</u>

In Chapter 5 of ACI 530-88/ASCE 5-88, service load combinations are defined and material properties are specified to support a Working Stress Design approach. Prescriptive provisions are included for such diverse topics as lateral load distribution, composite action in multi-wythe walls, minimum column dimensions and reinforcement, guidelines for calculating section properties, and design of anchor bolts which are solidly grouted in masonry. Chapter 6 covers requirements for the Working Stress Design of structures in which tensile resistance of masonry is allowed (unreinforced masonry), while Chapter 7 addresses the design of structures for which the contribution of tensile strength of masonry is neglected (i.e. reinforced masonry). Chapter 9 presents Empirical Design which incorporates prescriptive design criteria. Buildings designed by the empirical rules are restricted to Zones 2 or lower (using the Risk Map presented in ASCE A-7, "Minimum Design Loads for Buildings and Other Structures") and are limited to a maximum height of 35 ft, and a minimum wall thickness of 8 in.

Appendix A of the reference document contains special requirements for design of masonry building elements located in the five seismic zones highlighted on the Seismic Risk Map of ASCE A-7. As shown in table 4.2, there are no special design provisions for Zones 0 and 1. The provisions for Zone 2 specify minimum areas and maximum spacings of vertical and horizontal reinforcement (see figure 4.1). Additional reinforcement requirements are included for stack bond construction. The other Zone 2 provisions primarily address anchorage and connection details. The Zone 3 and 4 provisions further limit the height of shear wall buildings to 160 ft and prohibit the use of Type N mortar and masonry cement mortar. Masonry shear walls must be at least 8 in thick. Minimum steel ratios are cited for both horizontal and vertical reinforcement (see figure 4.2). Moreover, reinforcement required to resist in-plane shear must be uniformly distributed and be embedded in mortar or grout. A load factor of 1.5 is to be applied to the service load combinations in Chapter 5 and allowable working stresses, as prescribed in Chapter 7 are to be used in designing masonry shear walls. ACI 530-1988/ASCE 5-88 does not contain Strength Design criteria.

Standard Building Code(1988 Edition)

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The Standard Building Code, under the jurisdiction of the Southern Building Code Congress International, is used prevalently in ten southern states. Buildings and other structures are required to be designed to resist minimum lateral seismic forces in accordance with a formula for total base shear. The code uses the ASCE A-7 seismic risk map, overlayed with zones of effective peak velocity-related acceleration. A seismic zone factor ranging from about 0.10 to 1.00 is computed based on the acceleration obtained from the map. Section 1206 ("Earthquake Loads") contains provisions for obtaining the other site-related and occupancy-related factors needed to compute the total lateral base shear. Paragraph 1206.1 cites the occupancy group/zone combinations that are exempt from seismic design requirements. For example, structures in Group R-3 (one- and two-family dwellings) located in Seismic Zones 0,1 or 2 are exempt. Unlike other model building codes, the Standard Building Code defines "seismically reinforced-masonry shear walls." Such walls must contain reinforcement to resist tension and must contain specified minimum amounts of horizontal and vertical reinforcement. The sum of the areas of horizontal and vertical reinforcing must exceed a specified minimum. Maximum bar spacing requirements must be met. Shear walls containing less than the minimum amounts of reinforcement and plain masonry shear walls are assigned a higher horizontal force factor, K, than are seismically-reinforced masonry shear walls. The comparative values are 2.50 and 1.33 respectively.

The BOCA National Building Code (1987 Edition)

The BOCA National Building Code (hereafter referred to as the BOCA Code), published by Building Officials & Code Administrators International, Inc., is used primarily in the Midwest and Northeast, areas falling in Seismic Risk Zones 2 or less. The BOCA Code approaches the seismic design of buildings and other structures by incorporating an equivalent static lateral load. In computing the total base shear force, several site-dependent factors are used along with factors that depend on the occupancy importance and the type of structural system employed to resist the horizontal seismic forces. Section 1113.0 contains the earthquake load requirements in addition to addressing such design considerations as irregularity of geometry, anchorage requirements for building elements, overturning and lateral drift limitations. In the lateral force formulation, a numerical coefficient is assigned to the structure dependent upon its location on the 5-zone Seismic Risk Map of ASCE A-7. The BOCA Code classifies all buildings and structures into one of ten use groups. Buildings in Group R-3 (one- and two-family dwellings), located in Zones 0, 1, or 2 are exempt from the earthquake provisions. Buildings and structures are also assigned an Occupancy Importance Factor, between 1.0 and 1.5, depending upon the relative importance attributed to the nature of the occupancy. Buildings and structures designated as essential facilities receive a factor of 1.5. The relative ductility of lateral-load resisting systems is accounted for by assigning a Horizontal Force Factor, K. Unreinforced masonry shear walls are assigned a K-value of 4.00 while a value of 1.33 is used for reinforced masonry shear walls. The total base shear force is directly dependent on the magnitude of K.

The provisions of Article 14 govern the materials, design, construction, and quality of masonry. There are no special seismic design provisions, but this article references ACI 530-88/ASCE 5-88 where mininum areas and maximum spacing are specified for reinforcing bars. The BOCA code does not contain strength design provisions for masonry shear walls.

4.3 Summary and Conclusions

Based on the masonry design provisions reviewed, there are similar, but not uniform, provisions in the three model building codes for designing masonry construction for seismic conditions. Working stress design provisions are common to the three codes. In the case of the UBC, as well as in the NEHRP provisions, masonry construction may be based on strength design methods. However, the Strength Design provisions in Section 2412 of the UBC-1988 only apply to walls of hollow-units that are reinforced in accordance with minimum area and spacing requirements specified for Seismic Zones 3 & 4.

TABLE 4.1 - SEISMIC PROVISIONS IN CODES FOR MASONRY SHEAR WALLS

CODE OR STANDARD	CHAPTER, SECTION OR ARTICLE	SPECIAL SEISMIC PROVISIONS	SEISMIC RISK MAP	BUILDING HEIGHT LIMITATION	MIN. WALL THICKNESS	ULT. STRENGTH DESIGN
UNIFORM BLDG. CODE 1988 EDITION	CHAPTER 24 ART. 2312, 2407 - 2412	2407-ZONES 2, 3 & 4	UBC (6 ZONES)	ZONE 2 NONE STATED ZONES 3&4 $h \leq 160$ft.	$t \geq 6$ in	SEE SECT. 2412
ACI 530 - 88 ASCE 5-88	PART 3, CHAPTERS 5 - 9	APPENDIX A ZONES 2,3 & 4	ASCE/A7 (5 ZONES)	ZONE 2 35ft. ZONES 3 & 4 160ft.	$t \geq 8$ in $t \geq 8$ in	ONLY WORKING STRESS DESIGN
STANDARD BUILDING CODE 1988 EDITION	SECT. 1206 & CHAPTER 14	1206.1 - IDENTIFIES BLDG./STRUC. CATEGORIES THAT ARE EXEMPT FROM SEISMIC DESIGN. 1411-REINFORCED MASONRY	ASCE A7 PLUS EFFECTIVE PEAK VELOCITY-RELATED ACCELERATION	SEE SECTION 1404	SEE SECTION 1404	ONLY WORKING STRESS DESIGN
NATIONAL BUILDING CODE 1987 EDITION	1113.0 & ARTICLE 14	1113.1 IDENTIFIES BLDG./STRUC. CATEGORIES THAT ARE EXEMPT FROM SEISMIC DESIGN	ASCE A7 (5 ZONES)	NONE STATED	REFERS TO BIA BLDG. CODE REQUIREMENT & NCMA TR 75-B	ONLY WORKING STRESS DESIGN
NEHRP PROVISIONS 1988 EDITION	SECTION 1.4; CHAPTER 3	TABLE 12.1 IDENTIFIES 5 SEISMIC PERFORMANCE CATEGORIES	NEHRP-2 MAPS (Aa & Av) (7 ZONES)	CATEGORY A&B NONE STATED CATEGORY C 35 ft. CATEGORY D&E 160 ft.	$t \geq 8$ in	CHAPTER 12

Table 4.2. Seismic Provisions for Masonry Shear Walls *

Seismic Zone	Allowable Construction	General Design Requirements	Reinforcement Requirements
	ACI 530-88/ASCE 5-88		
0 and 1	No special limitations on type of units or mortar	Empirical or engineered design may be used.	No special provisions.
2	Veneer & non-struct. units can not be used to resist other than their weight.	Empricial or engineered design may be used	Vertical & Horizontal reinf. are required (see Fig. 4.1).
3 and 4	Type N mortar & masonry cement can not be used.	Engineered design (WSD).	Vertical & Horizontal reinf. must meet min. area & Spacing requirements (see Fig. 4.2).
	1988 Uniform Building Code		
0 and 1	No special provisions	Empirical or engineered design (WSD or SD).	No special provisions.
2	No Type O morar, masonry cement, or non-structural units can be used as part of the structural frame.	Engineered design (WSD or SD).	Vertical & Horizontal reinf. are required (see Fig. 4.1).
3 and 4	Type N mortar can be used as part of the structural frame.	Engineered design (WSD or SD).	Vertical & Horizontal reinf. must meet min. area & Spacing requirements (see Fig. 4.2).

*Provisions are taken from ACI 530-88/ASCE 5-88 Standards and the 1988 Uniform Building Code.

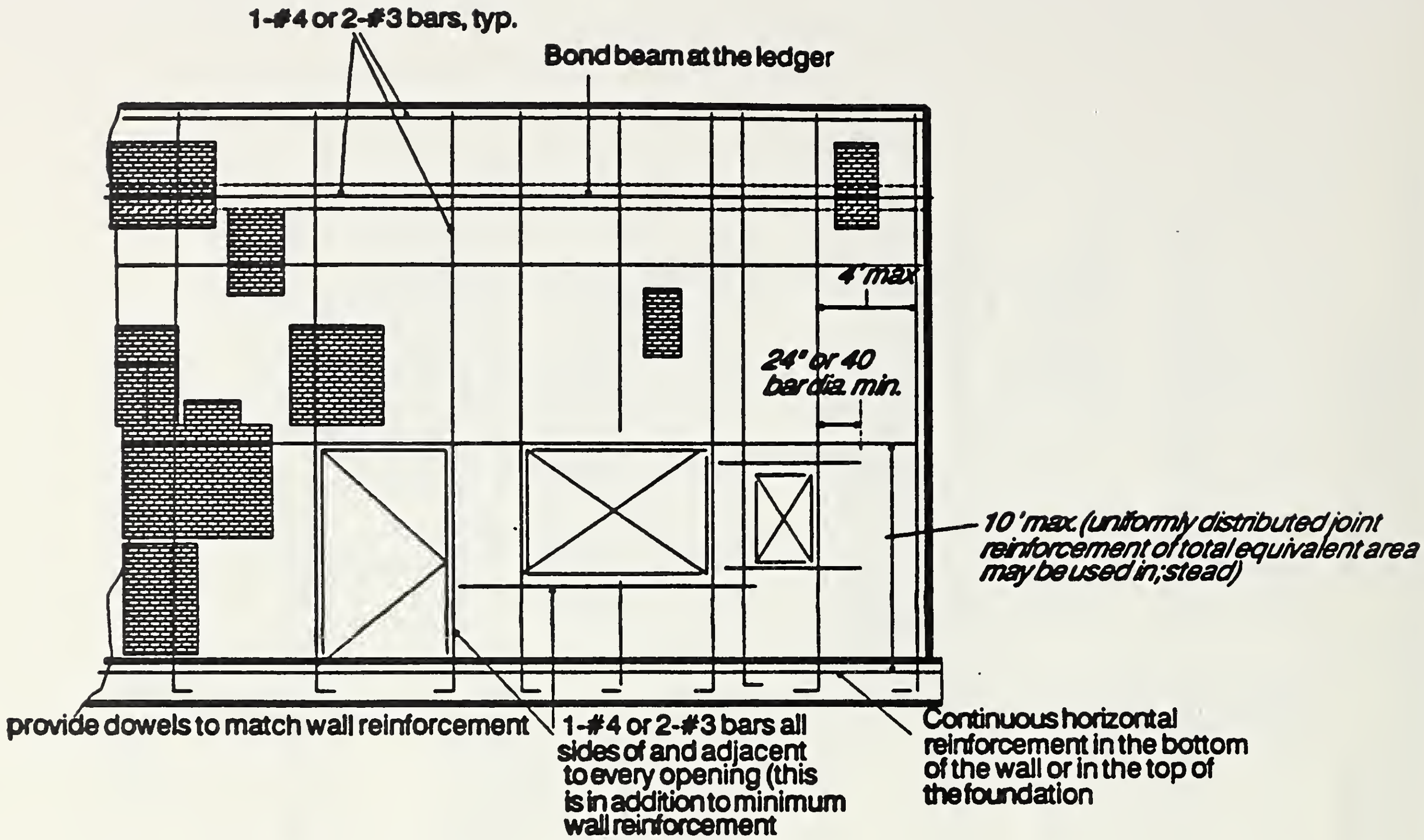


Figure 4.1 - Min. reinforcement for zone 2 (UBC/1988)

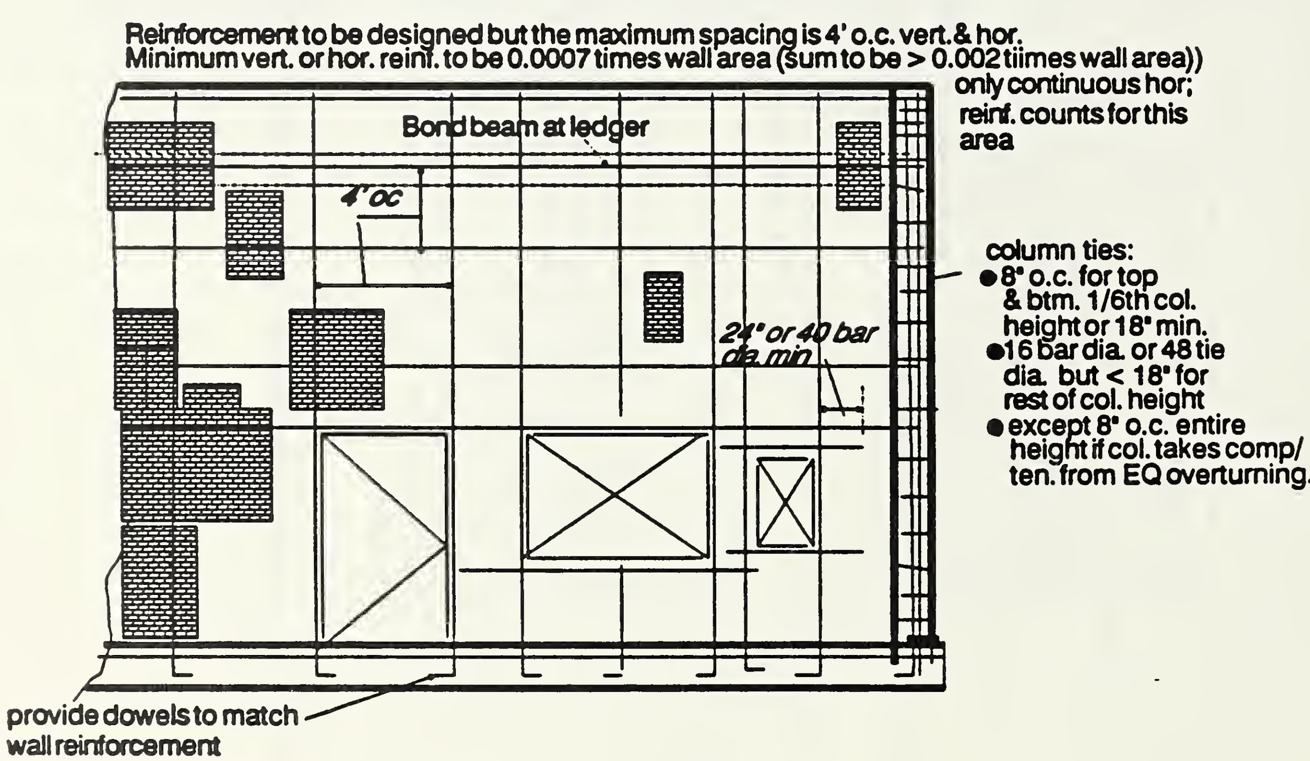


Figure 4.2 - Min. reinforcement for zones 3 & 4 (UBC/1988)

5. REVIEW OF RESEARCH

A search of the technical literature on masonry shear walls was conducted to determine the range and depth of available research and to identify areas in need of additional research. The review covers documents published in the United States and other countries since 1975. The scope of the review included clay and concrete masonry, plain and reinforced walls, single-wythe and multi-wythe construction, single-story and multi-story walls, three types of loading (i.e. monotonic, reversed cyclic, and dynamic) and walls with and without openings. Nearly ninety publications were reviewed, about half of which documented experimental research on masonry shear walls. The majority of the publications are in the form of technical papers presented at national and international masonry conferences or earthquake research workshops and symposia. This chapter presents a three-part summary of only those publications that contain experimental test results. Section 5.1 indicates the scope of experimental research from each source, in tabular form, giving specifics on the type of masonry, number of specimens of each type and their geometry, the prism strength, and test method used. To assist in identifying gaps in shear wall research and to aid in identifying high-priority research needs, the tests documented in section 5.1 are grouped into seven categories. The results of the subdivision process are displayed in the form of bar charts in section 5.2. In section 5.3, a selected number of test programs are examined where the objective and scope, test variables, and major findings for the respective programs are described in greater detail.

5.1 Tabular Summary of Technical Information

Table 5.1 presents a summary of the pertinent experimental information contained in the various publications. Following is an explanation of the entries presented in the table.

1. Reference numbers are listed in the first column of the table in accordance with the numerical order assigned to the references (chapter 9). The references are listed in chapter 9 in alphabetical order. The year of publication in the first column provides a chronological perspective on the projects.

2. Columns 2 to 4 define the dimensions of the wall specimens and indicate the height-to-length (also referred to by some authors as height-to-width) ratio, which is a principal test parameter. The sizes of bearing blocks, flange dimensions where applicable, and details about other attachments are excluded.

3. Columns 5 - 7 categorize the specimens according to the relative amounts of grout used in their construction. Excluded is information on the amount and location of grout in partially-grouted walls.

4. Columns 8 to 10 indicate the amount of vertical, horizontal, and joint reinforcement present, expressed in terms of reinforcement ratios. It was necessary to convert reinforcement ratio to a common basis of comparison as there are several different methods

used in the literature to compute reinforcement ratios. For example, most of the Japanese investigators compute vertical reinforcement ratio in the same manner as done for reinforced concrete beams, using only the area of the "flexural" bars located at either end of the wall. The denominator (wall area) is the product of the wall thickness and the distance from the compression face to the centroid of the flexural reinforcing bars. The horizontal reinforcement ratio is computed by dividing the total area of the horizontal bars by the product of wall thickness and the distance between the top and bottom layers of bars. Most American investigators compute the vertical reinforcement ratio by dividing the total area of vertical reinforcing bars by the gross cross-sectional area. The horizontal reinforcement ratio is computed by dividing the total area of horizontal steel by either the gross vertical cross sectional area or the same horizontal cross sectional area used to compute the vertical reinforcement ratio. In this review, the vertical reinforcement ratio is defined as the quotient of the total area of vertical bars divided by the gross cross sectional area of the wall. The horizontal reinforcement ratio is defined as the quotient of the total area of horizontal bars divided by the same cross sectional area used for the vertical reinforcement ratio.

5. Columns 11 to 13 indicate the numbers of specimens of each type, namely clay brick, concrete block or composite construction. Additional information, such as hollow vs solid brick, or multi-wythe walls of similar or dissimilar units, is covered under "Remarks."

6. Compressive strength values for masonry may be determined by conducting compressive tests on prisms. Column 14 list average compressive strength values (f'_{mt}) obtained from testing three-high or five-high prisms without modificiation of the results to account for slenderness effects.

7. Columns 15 & 16 indicate the top and bottom boundary conditions and are intended to reflect the test setup. Figure 5.1 presents schematics of the two most commonly-encountered test setups. For convenience, these test setups will be referred to as the "diagonal" and "lateral" test methods, respectively. The diagonal test method usually follows ASTM Test Method E 519 (Standard Test Method for Diagonal Tension (Shear) in Masonry Assemblages) directly, or is a modification thereof. Symbolically, $\underline{Q}$ represents the total edge load and $\underline{P}$ is the diagonal load. The lateral test method has been the most extensively used method in tests of masonry shear walls under simulated earthquake loads.

In table 5.1, the diagonal test method is identified by specifying free boundaries (FR) for the entries under "boundary conditions," supplemented by a note in the Remarks column to that effect. The boundary conditions for the lateral test method are identified as "FX" for a fixed end and "FR" for a free end. Thus a cantilevered wall test would be tabulated as FR and FX for the top and bottom boundary conditions, respectively.

8. Edge load, which is a principal parameter in shear wall tests, is tabulated in Columns 17 & 18 in units of stress and as a percentage of the prism strength, f'_{mt}.

9. Columns 19, 20, & 21 identify the manner of lateral loading as either one-directional (monotonic), cyclic, or dynamic. The entry in Column 19 designates a monotonically-increasing static load to failure. The entry in Column 20 represents cyclic loads or displacements applied at relatively slow rates. The amplitudes of load or displacement are repeated or increased according to predetermined test sequences. The third entry represents shake-table testing or testing at a relatively fast rate of loading or displacement (i.e. >1 Hz). Other loading situations are clarified in the Remarks column.

5.2 Distribution of Tests

The approximately seven hundred tests documented in table 5.1 are grouped into seven categories, according to: 1) the use of grouting, 2) scale factor, 3) types of construction, 4) test method used, 5) level of axial load, 6) aspect ratio, and 7) the amounts of horizontal and vertical reinforcement. Corresponding to these seven categories, bar charts (figures 5.2 through 5.8) are used to indicate frequency distributions of the test specimens.

Figure 5.2 shows the proportion of walls that were fully grouted, partially grouted or plain (ungrouted). About 62% of the walls tested in the past 15 years were fully grouted, 29% were ungrouted, and the remaining 10% were partially grouted. About one half of the tests on plain masonry walls were conducted at NBS [12, 33, 80, 81, 82, 83, & 86]. With the exception of reinforcement, all of the design parameters noted in section 6.2 were investigated during the NBS test series. The results of one of the NBS test series [86] were used to corroborate NBS-derived failure hypotheses to predict the shear cracking limit state in plain masonry shear walls.

Figure 5.3 indicates that 93% of the shear wall tests used "full-scale" specimens constructed with full-size masonry units. The only extensive small-scale model (using 1/3- and 1/2-scale) shear wall testing was reported by Tomazevic, et al, in a series of publications by the Yugoslavian Institute for Testing and Research in Materials and Structures [72, 73, 74, and 75](for more information refer to the discussions of these tests in section 5.3.3).

Figure 5.4 shows that 83% of the shear wall tests used single-wythe walls and 11% used walls of multi-wythe construction. In this report, "multiwythe" designates walls of brick-to-brick or brick-to-block units, with or without a collar joint. The collar joints were filled with either grout or mortar. The joint reinforcement consisted of either steel bars or mesh formed by longitudinal wires welded to cross wires. Two percent of the specimens were built with a rectangular-shaped flange connected perpendicular to the shear wall at its center or at one or both ends. In the United States, flanged walls have been tested only since the onset (i.e. late 1984) of the TCCMAR program. Priestley [56] investigated the asymmetrical behavior of a single flanged wall under dynamic loading (shake-table) applied in the plane of the web. The other flanged wall tests were conducted by Japan or the U.S. using cyclic loading.

Figure 5.5 categorizes the specimens according to the test method used. As noted in section 5.1, all shear wall tests belong to one of two main categories: "diagonal" or "lateral" load methods. The lateral load test method is further subdivided into three groups: monotonic, cyclic, and dynamic. One-third of the tests employed the diagonal test method patterned after ASTM Test Method E 519. In addition to imposing a monotonically increasing diagonal load, some tests incorporated a constant-magnitude axial load normal to the bed joints. This method was used at NBS to test plain masonry walls of various sizes, aspect ratios, and axial loads [12, 33, 86]. The NBS tests account for 72 percent of the reported diagonal tests, the rest being tests of fully-grouted plain walls reported by Arinaga, et al. [3] and wallette (smaller-scale walls) tests reported by the University of California at Berkeley [8, 26, 27, & 69]. No diagonal tests of reinforced walls have been reported.

Sixteen percent of the tests used monotonically-increasing (static) lateral loading up to failure. Seventy-five percent of these tests were conducted at NBS using unreinforced plain and grouted specimens of various aspect ratios and axial load [32, 80, 81, & 82]. In some NBS tests [e.g. 80,81, & 82], the load was increased monotonically until first cracking was observed. The direction of loading was then reversed to a displacement sufficient to cause cracking in the opposite direction, at which point the test was terminated. Static lateral load tests reported from other sources [3, 14, 46] used fully-grouted specimens with or without reinforcement.

The most common (about 47%) method of testing has been the cyclic lateral load test, often applied in conjunction with a constant-magnitude, axial load. (For purposes of this review, cyclic loading is defined as a lateral load or displacement, applied at a relatively slow rate, with increasing amplitudes in alternating directions until failure). Most frequently, the top and bottom surfaces were kept rotationally fixed to simulate a commonly-encountered in-situ condition. In other cases, the cantilevered wall test setup was used [14, 28, 41, 50, 72-75]. Most of the JTCCMAR experimental research has used servo-controlled cyclic loading.

Only three percent of the tests reported used dynamic loading (either shake-table tests or cyclic loading rates in excess of 1 Hz) [8, 43, 77]. In one series [43], duplicate specimens were tested using 0.02 Hz and 3 Hz cyclic loads. The results indicate that faster loading rates increase ultimate strength in the case of flexural failure and decrease it in the case of shear failure. It was also noted that more substantial anchorage of flexure reinforcement is needed in dynamic tests. Additional testing is needed to examine the significance of these findings.

Figure 5.6 shows the range of axial loads imposed on the specimens prior to testing under diagonal or lateral loads. The basis of comparison is the average axial stress q = Q/Ag expressed as a percentage of prism strength, (100 Q)/Ag f'_{mt}, where Q is the axial load, Ag is the gross cross-sectional area, and f'_{mt} is the compressive strength obtained from companion prism tests.

About one-third of the specimens were tested with no axial load, mostly (80%) by the diagonal test method. There was a relatively uniform distribution of average axial stress intensities on the remaining two-thirds of the specimens in the 0 - 20% range, with tests becoming scarce beyond this upper limit.

Figure 5.7 indicates that the majority of masonry shear wall tests have been conducted on walls with aspect ratios (height/length) of 1.5 or less. This trend may be due to the fact that researchers are often trying to affect a shearing, rather than a flexural, mode of failure in the wall. Depending on the magnitude of reinforcement, an aspect ratio of 1.5 or less should be sufficient to realize this objective. Just over 50% of the walls covered in this review had an aspect ratio of 1.0. It is noted that the standard diagonal compression test method (ASTM E 519) specifies an aspect ratio of 1.0. For this reason, most of the diagonal compression tests were performed on walls with a ratio of unity.

Figure 5.8 shows the distribution of horizontal and vertical reinforcement ratios as defined in section 5.1. Plain walls were excluded from this compilation. A range that includes <u>0%</u> reinforcement is explained by the fact that some experiments used reinforcement in only one direction. Thus, the ratio in the unreinforced direction is zero.

As indicated by the bar chart, reinforcement ratios in either direction are generally below 0.30%. Although not shown in figure 5.8, subdividing this range indicates a scarcity of tests using ratios in the range of 0.10 - 0.20% and almost no tests with ratios between 0 and 0.10%.

5.3 Synopses of Selected Publications

Some of the references listed in table 5.1 are described in greater detail in sections 5.3.1 through 5.3.3. The criteria for selection include scope of the experimental work, the reporting of element testing as opposed to assembly or sub-assembly testing, and comprehensiveness of the documentation of the experimental program. For each reference selected, the objective and scope, type and range of test variables, and major findings (results and conclusions) are summarized.

5.3.1 <u>General U.S. and U.S. TCCMAR Research</u>

[86] <u>Yokel, F.Y., and Fattal, S.G., "A Failure Hypothesis for Masonry Shear Walls"</u>

Objective and Scope:
Failure hypotheses are compared with the results of 32 plain single-wythe brick walls tested in diagonal compression. Three types of units designated by A, B and S were combined with two types of mortar, conventional and high

strength, designated by C and H, to build four types of walls (8 specimens of each type). All walls were 48 in square (h/L = 1).

Variables:
Type of Masonry - four types of construction, AC, AH, BH, and SH.

Axial Load (% of f'_{mt}) - 0 - 4.7%(AC), 0 - 9.3%(AH), 0 - 5.8%(BH), 0 - 3.3%(SH)

Prism strength - 3190, 4830, 5170, 6100 psi, for walls AC, AH, BH, SH, respectively.

Conclusions:
1. Shear failure under diagonal compression and axial load can occur by debonding along mortar joints or splitting of the masonry units.

2. For a given type of wall, failure can occur by joint debonding under low axial load and change to unit splitting under higher axial load.

3. Debonding strength is characterized by the linear relationship $\tau_c = \tau_o + c\sigma$ where τ_c = shear strength τ_o = shear strength at $\sigma = 0$, σ = axial stress, and c = constant. For the specimens tested, c = 0.4.

4. Splitting failure originates at the center of the specimen at a splitting strength governed by a critical relationship between the principal biaxial stresses.

5. Critical tensile stress and critical tensile strain hypotheses, respectively overestimate and underestimate capacity.

[29] Hirashi, H., "Flexural Behavior of Reinforced Masonry Walls"
Objective and Scope:
This paper reports test results of nine walls which were part of the test series reported in [48]. The nine specimens shared the following common properties: the same geometry (i.e., h/L = constant), fully grouted cores, constant flexural reinforcement in end cores, constant axial load, spiral reinforcement around flexural bars at critical compression zones, and rotational fixity of top and bottom surfaces. All specimens were subjected to the same lateral cyclic load history.

Variables:
1. Type of masonry - concrete block (6), hollow bricks (2), reinforced concrete (1).

2. Horizontal reinforcement - 0.29% and 1.16%

Conclusions:
1. Increase in the amount of horizontal reinforcement (four-fold) had no discernible effect on cracking and ultimate shear strengths.
2. Increasing the amount of horizontal reinforcement increases the maximum shear-to-maximum flexural strength ratio and signficantly improves deformation capacity: the ability to simultaneously develop large deformations without substantial strength degradation.

3. The ability of a shear wall to develop a large deformation capacity under cyclic load is largely attributed to the presence of spiral reinforcement at critical compression zones.
4. Proper splicing of vertical reinforcement (sufficient development length or mechanical coupling) permits the development of "nearly" full deformation capacity.
5. The ratio of maximum strength-to-cracking shear strength was in the range of 1.3 to 1.8, indicating that substantial post-cracking strength gain is possible in shear mode failures, depending primarily on the effective use, rather than the amount, of horizontal reinforcement.

[33] Jolley, R.H., "Shear Strength: A Predictive Technique for Masonry Walls"
Objective and Scope:
Finite element analysis was used to study the shear resistance of masonry walls tested under lateral or diagonal loads. A total of 87 specimens from nine different National Bureau of Standards test series were used as a basis of comparison. All the specimens were plain masonry except eight concrete block walls in which joint reinforcement was used in every other course. Three types of test setups were used, each patterned after a standard ASTM test: 1) lateral loading of cantilevered wall with tie-down rods at the loaded edge(ASTM E 72); 2) the same setup as (1) minus the tie-down rods(ASTM E 564); and 3) diagonal load method(ASTM E 519).

Variables:
1) Types of Masonry - concrete block and brick walls.
2) Aspect ratio - square (h/L = 1.0) and rectangular (h/L = 0.5).
3) Test Method - ASTM E 564, ASTM E 72, and ASTM E 519.
4) Axial Stress - 0 - 79% of f'_{mt}.
5) Horizontal Reinforcement - plain (79 walls), joint reinforcement (8 walls).

Conclusions:
1. Plain masonry walls, with or without joint reinforcement, can sustain in-plane lateral loads significantly in excess of the first cracking loads.
2. Shear strength depends on axial load and on the strengths of the individual constituents.
3. Distinct failure modes exist, depending upon the magnitude of the axial load.
4. The effect of axial load can be expressed by the relationship $\tau = c - k\sigma$, where, τ represents the shear strength, σ denotes the axial stress and c and k are constants which depend on constituent material strengths.

[12] Fattal, S.G., "The Capacity of Unreinforced Masonry Shear Walls Under Membrane Loads"
Objective and Scope:
Ninety-one plain and grouted masonry specimens were tested in diagonal compression to study the effect of size, aspect ratio and axial load on cracking shear response. The specimens were of different sizes and aspect ratios. Included were five circular grouted concrete block walls tested under diametric compression applied at five different angles with respect to bed

joint to study the effect of bed joint orientation on cracking capacity. Prisms having different slenderness ratios were tested to examine correlation with prism strength.

Variables:
1. Types of masonry - single-wythe brick, double-wythe brick with grouted collar joint, single-wythe hollow concrete block, and fully-grouted hollow concrete block.
2. Aspect Ratio - h/L = 0.5, 1.0, 2.0
3. Size - square (inches): 48 x 48, 32 x 32, 24 x 24, 16 x 16 angular (inches): 16 x 32, 32 x 16, 14 x 48, 48 x 24, 8 x 16, 16 x 8 circular (inches): diameter = 52
4. Axial stress - 0, 14 - 52% of f'_{mt}.
5. Bed joint orientation of circular walls with respect to applied load - 0, 22.5, 45, 67.5, 90 degrees.
6. Slenderness ratio - prisms with two slenderness ratios were tested.

Conclusions:
1. Good correlation of diagonal tests conducted on square specimens of different sizes indicates the feasibility of using smaller than 4-ft square specimens specified in ASTM E 519.
2. Cracking strength increases with increasing aspect ratio, while the characteristics of the failure mode changes from splitting to debonding (see definitions, chapter 6).
3. The use of kerf block in the construction of fully-grouted concrete block specimens creates planes of weakness normal to the bed joint where the ungrouted slots in the kerf units align vertically.
4. Effect of slenderness of brick prisms on compressive strength is at variance with strength correction factors specified by the Brick Institute of America.
5. Axial load increases diagonal load capacity to a point beyond which axial load can trigger compressive crushing at the diagonally-loaded corners before the diagonal cracking capacity can be fully developed.
6. Peak load was slightly above or the same as first cracking load.
7. Load deformation response was approximately linear, or, slightly non-linear at higher loads in the case of concrete block specimens under axial load.

[80] Woodward, K. and Rankin, F., "Influence of Block and Mortar Strength on Shear Resistance of Concrete Block Masonry Walls"

Objective and Scope:
To examine the effect of block and mortar strength on the in-plane shear resistance of seventeen 64-in high concrete masonry walls.

Variables:
1) The concrete block units had gross area unit strengths of 1300 and 1800 psi.
2) The mortar was proportioned according to ASTM C270 (Standard Specification for Mortar for Unit Masonry) as Type S or Type N.
3) The vertical compressive stress varied from about 100 psi to 400 psi based on the net cross-section area.

4) Thirteen walls were 64 in. long, two were 48 in. long and the remaining two were 96 in. long.

Conclusions:

1) For the lower levels of applied vertical compressive stress, the influence of block and mortar strengths on the maximum shear resistance was negligible. The influence of the component strengths became more significant as the vertical stress was increased.
2) The interaction effect of block and mortar strength on wall shear strength was greater than the effect of either component's strength taken alone.
3) In general, the linear relationship between maximum shear resistance and applied vertical compressive stress was unaffected by block or mortar strength. The high strength block-low strength mortar walls were an exception and exhibited a non-linear relationship.
4) The diagonal tensile strain threshold at which diagonal cracking occurred was unaffected by the variation in block and mortar strength. The range of threshold strain was between 110 and 165 microstrain.

[81] Woodward, K. and Rankin, F., "Influence of Aspect Ratio on Shear Resistance of Concrete Block Masonry Walls"

Objective and Scope:

To examine the influence of aspect ratio on the relationship between lateral in-plane load resistance and vertical in-plane compressive stress. Seven 64-in high, ungrouted and unreinforced concrete block masonry walls were tested with fixed-fixed boundary conditions.

Variables:

1) The aspect ratio was varied by using three different lengths of wall: 48, 80, and 96 in.
2) The axial stress magnitude varied for each length of wall. The axial stress levels for the two 48-in walls were 160 and 435 psi. The levels for the two 80-in walls were 230 and 390 psi. The stress levels for the three 96-in walls were 220, 310, and 410 psi.

Conclusions:

1) There was a relatively weak effect of aspect ratio on the diagonal cracking strength for aspect ratios less than or equal to 1. The diagonal tensile strain which defined the onset of diagonal cracking was unaffected by aspect ratio and was in the range of 75 to 150 microstrains.
2) There is a nearly linear relationship between axial compressive stress and maximum lateral resistance.
3) The maximum lateral load resistance was affected by aspect ratio for higher levels of axial compressive stress.
4) The longer walls developed maximum lateral load resistance greater than the resistance associated with diagonal cracking due to shear friction along horizontal cracks in the highly compressed regions of the walls.

[50] Porter, M.L., Wolde-Tinsae, A.M., and Ahmed, M.H., "Behavior of Composite Brick Walls."

[51] Porter, M.L., "Composite Brick and Block Masonry Walls"

Objective and Scope:

To study the behavior of five double-wythe brick walls [50] and six reinforced composite brick and block masonry walls [51] subject to gravity and in-plane shear loads. The walls were nominally 72 in. high, 48 in. long and 9 in. thick. The base of a wall was fixed, while the top was free to move. Distributed vertical axial load was applied first and maintained constant, followed by in-plane shear load. Both vertical and horizontal loading was applied as distributed loads along the top.

Variables:

The type of reinforcing in the 2-in collar joint was varied. Reinforcing consisted of either welded wire fabric or vertical and horizontal reinforcing bars. There was one control wall which had no grouted collar joint. There were a total of eleven walls tested in the entire program. Walls were tested in the presence of a uniformly-distributed axial load and a monotonically increasing horizontal load. The magnitude of the precompression axial load ranged between 135 and 180 kips for the all-brick walls and 120 and 180 kips for the brick-block walls.

Conclusions:

Brick and Block Composite Walls

1) Using a composite masonry wall with grouting and reinforcing the collar joint versus the reinforcing the block wythe increased the shear strength by 15%, the initial stiffness by more than 300%; and the first cracking load by 43%.
2) Precompression stresses had little effect on shear strength.
3) Overall wall ductility decreased as precompression stress increased.
4) Shear strength and stiffness increased by about 18% when mesh was welded to the base, which simulated the continuity of steel.
5) Some of the load-deflection curves obtained in this investigation can be approximated as a trilinear relationship, as proposed by R. Meli ("Behavior of Masonry Walls Under Lateral Loads," Proc. Fifth World Conference on Earthquake Engineering, v.1, Rome, 1974, pp. 853-862). More tests need to be conducted considering different parameters to find constants which define this curve.
6) The walls tested showed that the shear strength for composite walls cannot be predicted using a previously derived relationship for single-wythe walls without modifying constant values for coefficient of friction and ultimate shear-bond strength.
7) The assumption of composite action for the walls was valid for loads beyond those permissible according to ACI Standard 531-79.
8) The shear strength ranged from 151 psi to 205 psi.

Brick Walls

1) The failure modes for brick walls were mainly bearing failures in both wythes at the compressive corner followed by bond failure (i.e., separation) between the masonry wythes and the collar joint.
2) The precompression load had only a small effect on the shear strength. However, the authors concluded that a wider range of precompression loads should be considered in the future.

3) Williams' and Gerchwindner's [78] proposed Equation (1) for the bond stresses as applied to these composite walls provided a safety factor of 3.25.
4) Joint reinforcement reduced the shear strength, but not significantly.
5) Although the minimum amount of steel required by the ACI Code was used, the steel did not yield. Therefore, further studies should be done using less steel.
6) The shear strength ranged from 197 psi to 221 psi.
7) The load-deflection curve can be approximated as a trilinear relationship.
8) Ultimate shear strength can be estimated using the following equation: $v_{ult.} = 141 + 0.19\ \sigma_c$, where σ_c is the precompression stress.
9) The allowable value of shear stress as given in the ACI Code [$v = 1.5 \sqrt{f'_m} > 75$ psi] was applicable for these composite walls.

[43] <u>Mayes, R.L., Omote, Y., and Clough, R.W., "Cyclic Shear Tests of Masonry Piers, Vol. 1 - Test Results."</u>

Objective and Scope:
This report documents the first of five series of tests conducted at the University of California at Berkeley. The others are reported chronologically in references [8, 26, 27, and 69]. Seventeen concrete block masonry double-pier systems coupled with heavily-reinforced top and bottom spandrels were tested under cyclic lateral loading with and without initial axial load. The pier system was allowed to rotate at the top under lateral load applied to the top spandrel.

Variables:
1. Rate of loading - specimens were tested in identical pairs using slow and fast rate of loading (0.02 and 3 Hz).
2. Reinforcement - 1) none, 2) vertical end bars with two reinforcement ratios, 3) vertical end bars and horizontal bars with different reinforcement ratios, and 4) vertical end bars, horizontal bars and toe reinforcement in the form of perforated steel plates in bed joints.
3. Grouting - none, partial and full grout.

Conclusions:
1. Discrepancies in results attributed to the test setup were noted. Differences in response in opposite directions were attributed to deformations in the reaction frame supporting the push-pull forcing mechanism. Differences in axial loads on the coupled piers were caused by equal and opposite incremental axial loads to counter the overturning moment caused by the lateral force.
2. Sufficient amounts of horizontal reinforcement enhance the ductility of shear-mode response significantly.
3. Use of 1/8-in perforated steel plates in the toe area improves flexural-mode response.
4. Partial grouting improves the elasto-plastic shear-mode response compared with grouting.

5. Dynamic loading increases ultimate strength in case of shear mode failures and decreases ultimate strength in case of flexure-mode failures compared with strengths obtained from a slow rate of loading.

[8] Chen, Shi-wen J., Hidalgo, P.A., Mayes, R.L., Clough, R.W., and McNiven, H.D., "Cyclic Loading Tests of Masonry Single Piers," Vol. 2 -Height to Width Ratio of 1.

[26] Hidalgo, P.A., Mayes, R.L., McNiven, H.D. and Clough, R.W., "Cyclic Loading Tests of Masonry Single Piers," Vol. 1 - Height to Width Ratio of 2.

[27] Hidalgo, P.A., Mayes, R.L., McNiven, H.D., and Clough, R.W., "Cyclic Loading Tests of Masonry Single Piers, " Vol. 3 - Height to Width Ratio of 0.5.

Objective and Scope:
The cited references document the second through fourth series of Berkeley experiments in which the responses of single masonry piers under axial and lateral loads were examined. A total of 63 piers were tested using the same test setup. The initial vertical loads were applied through a system of springs and tie rods anchored to the test floor. The specimens were flanked by two hinged steel columns to inhibit rotation at the top. Lateral load was applied through a steel beam anchored to the top. In all these tests, the extension of the steel columns with increasing lateral displacement introduced substantial axial loads on the specimens at failure, with a corresponding "apparent" improvement in ultimate response. Diagonal compression tests of companion wallettes were used throughout these test series.

Variables (refer to table 5.1 for a detailed breakdown):
1. Type of masonry and grouting - Fully-grouted and partially-grouted concrete block and hollow brick and two-wythe solid brick with collar joint grouted [8]; fully-grouted and partially-grouted hollow brick and two-wythe solid brick with collar joint grouted [26]; and fully-grouted concrete block and hollow brick and two-wythe solid brick with collar joint grouted [27].
2. Aspect ratio - $h/L = 1$, 2 and 0.5, for series [8], [26], [27], respectively.
3. Axial stress - 1.7 to 3.2 % of f'_{mt}, initial, 4.0 to 15.3 % of f'_{mt}, final [8]; 1.4 to 1.8 % of f'_{mt} initial, 5.5 to 12.9% of f'_{mt}, final [26]; 2% of f'_{mt} initial, 3.8 to 8.7% of f'_{mt}, final [27].
4. Horizontal Reinforcement - 0 to 0.52% [8]; 0 to 0.54% [26]; 0 to 0.52% [27].
5. Vertical Reinforcement (only at ends) - 0 to 0.45% [8]; 0 to 0.92% [26]; 0.23 to 0.31% [27].

Conclusions:
1. Results were subject to distortions caused by variations of axial load and partial initial top-of-specimen rotation under lateral load, making it difficult to assess the effect of axial load on response.
2. Dynamic tests [8] require more substantial anchorage to develop yield capacity of vertical reinforcement than slow cyclic tests.

3. Mode of failure changes from combined shear-flexure; to shear; to shear, shear-sliding and flexure-sliding in walls with h/L = 2.0, 1.0 and 0.5, respectively.
4. Amount of vertical reinforcement generally had negligible influence on shear mode, hysteretic behavior and stiffness degradation.
5. In shear-mode failures, the difference between ultimate and cracking strengths was small except in grouted-core squat(h/L = 0.5) walls where the difference was more substantial, especially in concrete block walls.
6. With a few exceptions, the amount of horizontal reinforcement has no significant effect on shear strength, failure mode, stiffness degradation and hysteretic behavior.
7. Likewise, differences in response between partially- and fully-grouted walls tend to be small with regard to ultimate strength, inelastic behavior, energy dissipation and stiffness degradation, all comparisons being made on the basis of net grouted area.
8. All specimens exhibited substantial stiffness degradation under increasing lateral load.

[69] <u>Sveinsson, B.I., McNiven, H.D., and Sucuoglu, H., "Cyclic Loading Tests of Masonry Single Piers, Vol. 4 - Additional Tests with Height to Width Ratio of 1"</u>

Objective and Scope:
Thirty walls were tested under programmed cyclic load and controlled axial load (constant during testing) using the lateral load test method with top and bottom of walls kept rotationally fixed. Nine companion unreinforced wallettes were tested in diagonal compression.

Variables:
1. Types of Masonry - Grouted concrete block, grouted hollow brick, grouted two-wythe brick.
2. Types of Specimens - Square walls: 54 x 48 x (7 5/8 and 5 5/8) in. block; 54 x 48 x (7 3/8 and 5 5/8) in. brick; square wallettes: 32 x 32 x (7 5/8 and 5 5/8) in. block; 36 x 36 x (7 3/8 and 5 5/8) brick.
3. Test Method - Cyclic lateral load(walls), diagonal compression loading (wallettes).
4. Axial Load - 0.18% of f'_m (walls), 0% (wallettes)
5. Horizontal Reinforcement (including joint reinforcement) - 0.08 - 0.14% (walls), 0% (wallettes)
6. Anchorage of Horizontal Reinforcement - 1) hooked, 2) 90° bend, 3) welded to end plates.
7. Vertical Reinforcement - 0.13 - 0.67% (walls), 0% (wallettes)
8. Distribution of Vertical Reinforcement - 1) at ends only, 2) uniformly distributed .

Conclusions:
1. Increasing axial load increases lateral load resistance significantly.
2. Increasing axial load decreases ductility.
3. Increasing axial load changes the mode of response from flexural to shear.
4. Amount of horizontal reinforcement is not a significant factor in hysteretic behavior.

5. Effective anchorage of horizontal reinforcement in the most significant factor in improving response under lateral load.
6. Effect of axial load and horizontal reinforcement on stiffness degradation is not very significant.

[37] Limin, H. and Priestley, M.J.N., "Seismic Behavior of Flanged Masonry Shear Walls"

Objective and Scope:
A 79-inch high, wide-flange, T-section, reinforced concrete masonry shear wall was subjected to shake-table testing to confirm previously derived theoretical predictions of flexural response to seismic excitation. Both sinusoidal and simulated earthquake acceleration inputs were applied parallel to the web to quantify the asymmetric strength and stiffness characteristics. Two wall characteristics were of particular interest: 1) shear-lag effects in the flange and 2) behavior of the connection between the flange and the web.

Variables:
The shake-table test was conducted at the University of Cantebury, New Zealand and consisted of three stages: 1) free vibration tests, 2) sinusoidal excitation at gradually increasing levels of table acceleration amplitude, and 3) simulated seismic excitation by applying the 1940 N-S El Centro earthquake accelerograms to the shake table.

Conclusions:
The maximum experimental moment and displacement, with the web in compression and the flange in compression, were compared with predicted values. The predicted maxima were based on a set of design charts for flexural strength, effective stiffness, and curvature ductility capacity that were previously derived in an anlytical parameter study conducted by the same investigators. No definite conclusions were drawn as this was a preliminary study. For the one wall tested, it was observed that the experimental maximum moment and displacement with the web in compression were within $\pm$ 20% agreement with predictions. On the other hand, with the flange in compression, the maximum experimental moment was about 40% greater than its theoretical counterpart and the maximum experimental displacement was one-half the theoretical displacement value. This preliminary study served as the basis for a series of static and dynamic flanged-wall tests currently in progress [56].

[56] Priestley, M.J.N., and Limin, H., "Seismic Behavior of Flanged Masonry Shear Walls: Preliminary Studies"

Objective and Scope:
Flanged wall specimens are being tested as part of the TCCMAR program to determine their strength and ductility under reversed cyclic loading and to compare experimental results with predictions of response as obtained from analytical models such as the Lumped-Parameter Model, Structural Component Model, and Finite Element Model.

Variables:
The program is being conducted in two stages. Four static cyclic tests were conducted with the horizontal load being applied parallel to the web of the T-

shaped models. Also, there are plans for four dynamic tests on models otherwise identical to those tested under static cyclic loading. The four walls were identical in their dimensions, were subjected to a constant axial stress of 100 psi, and contained only vertical reinforcement. The spacing of the vertical reinforcement was held constant at 16 in o.c., but the bar size was varied. Two specimens contained #4 bars and the other two contained #6 bars. In addition, bed-joint reinforcement in the form of a stainless steel plate was placed in the toe of one wall.

[64] Shing, P.B., Schuller, M., Klamerus, E.W. and Noland, J.L., "Behavior of Sinle-Story Reinfroced Masonry Shear Walls Under In-Plane Cyclic Lateral Loads"

Objective and Scope:
To evaluate the validity of the 1988 UBC design formulae for ultimate shear capacity of reinforced masonry shear walls. Twenty-two 6-ft by 6-ft wall specimens were tested under in-plane cyclic loading. Sixteen walls were constructed using hollow-core concrete block units. The remaining six walls contained clay brick units.

Variables:
1) The amount of horizontal and vertical reinforcement. Vertical reinforcement ratio: 0.38 to 0.74%; horizontal reinforcement ratio: 0.14 to 0.26%.
2) The magnitude of the applied axial stress - 0 to 280 psi.
3) The type of masonry units - concrete block and clay bricks.

Conclusions:
1) The flexural strength of a square panel can be accuarately predicted by simple flexure theory based on the plane-section assumption.
2) The flexural strength of a shear wall subjected to seismic loads can be slightly higher than that predicted by flexure theory, due to the strain-hardening effect. The strain-hardening effect is reduced as the axial stress increases.
3) The ductility of a flexure-dominated wall can be substantially reduced by increasing the axial stress which leads to more severe toe crushing. However, ductile flexural behavior can be achieved under a high axial stress by using proper toe confinement.
4) The residual shear strength of masonry after diagonal cracking depends on the applied axial stress, the amount of vertical reinforcement, and the compressive strength of masonry. Based on these observations, the authors suggest that the UBC formulation appears to be overly-simplistic. A new shear strength formula has been proposed.
5) The UBC specification for the masonry shear strength is overly-conservative for the walls tested in this study.
6) The UBC specification tends to over-estimate the shear resistance of the horizontal reinforcement.
7) The authors' proposed formula for nominal shear strength appears to be more reliable and consistent with experimental results. However, the formula has to be further verified by additonal experimental data before it can be used for design.

[55] Priestley, M.J.N. & Elder, D.McG., "Cyclic Loading Tests of Slender Concrete Masonry Shear Walls"

Objective and Scope:
Three slender concrete block masonry walls were subjected to cyclic reversals of in-plane displacements to examine the ductility and strength degradation of such walls. The nominal 8-in block walls were approximately 20 ft high and 8 ft long. Reinforced concrete floor slabs, approximately 4 feet wide, were cast at the first and second floor levels and a reinforced concrete bond beam was placed at the top to distribute the lateral load and anchor the vertical reinforcement. The equivalent of a #5 bar was placed in each vertical cell, resulting in a center-to-center spacing of approximately 16 in. The vertical reinforcement ratio was 0.72% for each wall. The main vertical steel was lapped to "starter" bars which were anchored in the foundation beam and immediately above

Variables:
1) Two of the walls were subjected to an axial stress of 284 psi and one wall was subjected to an axial stress of 108 psi.
2) Confining plates were placed in the mortar beds in the compression zones of the potential plastic hinge area for one wall.
3) The lap length of vertical reinforcing was 3.2 ft for two of the walls, and 4.3 ft for the remaining wall.

Conclusions:
1) Results from the walls demonstrated the possible use of theoretical ductility capacity charts developed previously by Priestley.
2) The walls clearly indicated that problems must be expected from lapping starter bars within plastic hinge regions. Testing is needed to better define ductility of walls without lapping of vertical reinforcement in the plastic hinge zone.
3) The walls confirm that lack of ductility is more of a problem for tall shear walls than for squat walls previously tested.
4) Capacity of many conventionally-designed masonry shear walls may be suspect.

5.3.2 Japan Joint TCCMAR Research

[77] Wakabayashi, M. and Nakamura, T., "Reinforcing Principle and Seismic Resistance of Brick Masonry Walls"

Objective and Scope:
Six fully-grouted hollow brick walls were axially loaded and tested under dynamic lateral load. Top and bottom surfaces were built integral with heavily reinforced massive concrete beams and were kept rotationally fixed during cyclic testing. In addition, six walls constructed with 1/3-scale solid and hollow brick units were tested under cyclic lateral load. Analytical models are proposed for the prediction of strength and hysteretic behavior based on these tests. Earthquake-resistant design recommendations for reinforced brick shear walls are advanced. Additional tests of tall walls failing in flexure mode are not reported here.

Variables:
1. Size of Units - full- and one-third scale brick units
2. Axial stress (full-scale walls) - 2.5, 5 and 10% of f'_{mt}
3. Horizontal reinforcement (full-scale walls) - 0.28, 0.42, 0.85%

Conclusions:
1. Within the test ranges, maximum shear strength is not affected substantially by the amount of horizontal reinforcement and axial load.
2. A horizontal reinforcement ratio of 0.85% or greater is required for ductile shear failure .
3. Maximum shear capacity can be predicted reasonably by using a combination of truss and arch analogies and stress-strain relationship developed from diagonal compression and diagonal tension tests.
4. Hysteretic response curves can be developed using a combination of slip, degrading and bi-linear hysteresis models.
5. A design according to the proposed formulation will ensure a ductile shear mode response of masonry shear walls.

[39] <u>Matsumura, A., "Shear Strength of Reinforced Hollow Unit Masonry Walls."</u>

Objective and Scope:
Fifty-seven concrete masonry and twenty-three brick masonry walls were subjected to cyclic in-plane shear and constant axial loads. The purpose was to derive a formula for predicting the shear strength of reinforced masonry walls. A second objective was to determine the difference in shear strength between fully-grouted and partially-grouted walls.

Variables:
Nominal 8-in hollow concrete or 6-in clay brick units were used to construct reinforced, partially-grouted or fully-grouted walls of varying sizes. Full-size walls ranged in height from 63 to 71 in while the lengths ranged from 31 to 79 in. There was a set of smaller size walls with dimensions of 24 to 48 in high, 16 to 20 in long and 4 to 6 in thick. Two test setups were used: 1) fifty-five walls were subjected to horizontal shear loads with a fixed base and the top free to move horizontally (cantilever), and 2) twenty-five walls were laid horizontally and subjected to vertical shear loads like the loading of restrained deep beams. The former setup is called the "wall type" and the latter one is called the "beam type." The "beam-type" setup was generally used for small specimens as supplementary tests. Thirty-five of the "wall type" specimens were partially-grouted. Only the results of the "wall type" setup are discussed here. The effects of several factors were examined: axial stress, horizontal shear reinforcement and shear-span ratio.

Results:
Graphical presentations of the results were used to show the effect of the aforementioned variables on the shear strength. Mathematical expressions were derived to account for the influences of the several parameters.

Conclusions:
The author presents a formula for the prediction of the shear strength of reinforced masonry walls. The formula is a synthesis of the mathematical expressions derived to quantify the influences of the test variables.

Reduction factors are introduced to account for the use of partial grouting and for the cantilever test setup verus the fixed-fixed boundary condition. Matsumura's predictive formula was used in a comparative study discussed in section 6.5 of this report.

[40] Matsumura, A., "Effect of Shear Reinforcement in Concrete Masonry Walls"

Objective and Scope:
To quantify the effectiveness of horizontal reinforcement in increasing the shear strength of concrete masonry walls and to compare test results with previously derived (i.e., for reinforced concrete beams) formulae for estimating the initial shear crack load and the ultimate shear load. Six walls, nominally 72 in. high and 48 in. long, were tested under cyclic lateral loading and constant axial load (284 psi). All of the walls were fully grouted and contained vertical and horizontal reinforcement.
Variables:
The only variable was the volume of horizontal reinforcement; all horizontal bars were equivalent to #4 bars. The spacing was varied to effect different reinforcement ratios. Two specimens had two bars per course.

Conclusions:
1) It was confirmed that the effect of shear reinforcement on the shear strength can be expressed as $\tau_{sr} = 0.85\ p_h * \sigma_{yh}$, where p_h denotes horizontal reinforcement ratio and σ_{yh} denotes the yield stress of the horizontal reinforcing bars.
2) The initial shear crack load formula for reinforced concrete members is applicable to reinforced concrete masonry.
3) The ultimate load formulae for the lower bound strength of reinforced concrete members is applicable to reinforced concrete masonry.
4) Upon comparison of the Japanese data with data obtained at the Univ. of California at Berkeley (UCB), it was concluded that the UCB data do not clearly show the effectiveness of the shear reinforcement on shear strength.

[41] Matsumura, A. "Effectiveness of Shear Reinforcement in Fully Grouted Hollow Clay Masonry Walls"

Objectives and Scope:
1) To determine the shear strength and clarify behavior of reinforced, hollow unit clay masonry walls.
2) To examine the applicability of an empirical formula, previously suggested for concrete masonry walls, for estimating the effectiveness of shear reinforcement in clay masonry walls.

Five, fully-grouted walls, nominally 67 in. high and 43 in. long, were tested under cyclic lateral loading and a constant axial load (284 psi). The walls were fully grouted and reinforced with horizontal and vertical steel bars.

Variables:
The amount and spacing of horizontal bars were varied to produce different reinforcement ratios. Bars equivalent to #4 bars were used throughout for the horizontal reinforcement.

Conclusions:
1) Ultimate shear strength increases approximately in proportion to $p_h * \sigma_{yh} * f'_m$ where p_h denotes the horizontal reinforcement ratio, σ_{yh} denotes the yield stress of the horizontal reinforcing bars and f'_m is the prism strength.
2) The rate of increase in shear strength for grouted hollow clay masonry, as a function of increasing horizontal reinforcement ratio, is lower than that for grouted hollow concrete masonry.
3) This study used extremely high strength clay units compared with the strength of grout; more study is needed on the relationship between failure mechanisms and the properties of the constituent materials.
4) The previously derived formulae for predicting the ultimate shear strength of concrete masonry walls may have to be modified to more accurately reflect the constitutive properties of the brick masonry walls.
5) Shear crack strength is not affected by the presence of shear reinforcement. A formula is suggested for predicting first cracking strength.

[48] <u>Okamoto, S., Yamazaki, Y., Kaminosono, T., Teshigawara, M., and Hiraishi, H., "Seismic Capacity of Reinforced Masonry Walls and Beams</u>"

Objective and Scope:
The results of eighteen single-element shear wall tests, out of a total of 35 specimens tested, are included in this discussion. The remaining specimens were beams with and without an integral slab, and flanged walls. All specimens were fully grouted. The walls were tested under controlled axial load combined with programmed cyclic lateral loading applied in a manner that kept the top and bottom surfaces rotationally fixed. The vertical reinforcement ratio was constant with bars placed at each end and two bars spaced uniformly along the interior. Vertical reinforcement placed in the end cores was tied with spirals.

Variables:
1. Type of Specimen - hollow concrete block masonry, hollow brick masonry, reinforced concrete (control).
2. Aspect ratio - $h/L = 0.90, 1.6, 2.3$
3. Axial stress - 1.8 to 25.7% of f'_m
4. Horizontal reinforcement ratio - 0.17% and 0.67%

Conclusions:
1. Shear cracking load and ultimate shear strength increase at decreasing rates with increasing axial load. Gain in shear strength was in the 60-66% range with axial load increasing from 2 to 26% of prism strength. This conclusion is based on four masonry wall tests. The one reinforced concrete wall tested developed 10% greater shear strength under the same axial load than a comparable masonry wall.
2. Shear strength increases 20 and 30 precent respectively as the aspect ratio decreases from 2.3 to 1.6 and from 1.6 to 0.9. This conclusion is based on tests of three concrete block walls and three hollow brick walls.

3. Specimens failing in the shear mode had 50% of deformation capacity of those failing in the flexure mode.
4. Increasing the amount of shear reinforcement increases the ratio of ultimate shear strength to ultimate flexure strength and improves deformation capacity.
5. Spiral reinforcement improves deformation capacity of walls.

[70] Teshigawara, M., Isoishi, H., and Nakoka, A. "Effect of Transverse Walls Attached to Reinforced Concrete Masonry Walls"

Objective and Scope:
To determine the contribution of transverse walls to the shear strength and deformation capacity of shear walls. Six masonry walls were tested: three were rectangular and three were flanged.

Variables:
Of the three flanged walls, one was tee-shaped and two were cruciform-shaped (i.e. the flange was connected to the middle of the web). Two different horizontal reinforcement ratios (i.e. 0.167 and 0.668) were used. The vertical reinforcement ratio remained constant.

Conclusions:
1) The existence of the flanged walls increased the strength of the rectangular walls by 10 to 50 percent.

[31] Imai, H. and Miyamoto, M., "Seismic Behavior of Reinforced Masonry Walls with Small Openings"

Objective and Scope:
Repeated cyclic shear forces were applied to reinforced concrete masonry walls, with and without openings, to determine the behavior and performance of walls with openings. Tentative Japanese design guidelines predict the shear strength of walls with small openings by multiplying the shear strength of walls without openings by a reduction coefficient, which is estimated in the same manner as for reinforced concrete walls. These tests were conducted to confirm the magnitude of the proposed reduction factors. A total of six walls were tested.

Variables:
Four of the walls contained openings, two of the openings being one-block high and one-block wide. The other two openings were two-blocks wide and one-block high. The horizontal bar size, equivalent to #4, was constant throughout the testing. The horizontal bars were placed in every bed joint on five walls. The remaining wall, which had no openings, had horizontal bars spaced at every other bed joint. The vertical bars located near the ends of the walls were all equivalent to #6 bars. Half of the walls had two bars positioned at each end, while the other half had only one bar at each end.

Conclusions:
1) The reduction coefficient applied to openings in reinforced concrete walls can be applied to openings in reinforced masonry walls when calculating shear cracking stress and maximum shear strength.

2) The shear strength of the masonry walls can be predicted by the formula for the minimum shear strength of reinforced concrete walls.
3) Yield moment and maximum bending moment in masonry walls can be predicted by applying the respective formulae for calculating moment capacity of reinforced concrete walls, provided that concrete cylinder strength is replaced by the masonry prism strength.
4) Experimental failure modes, such as bending failure mode or shear failure mode, were in good agreement with the predicted lower-bound strength.

5.3.3 <u>Other Foreign Research Programs</u>

[53] <u>Priestley, M.J.N., "Seismic Resistance of Reinforced Concrete Masonry Shear Walls with High Steel Percentages"</u>

Objective and Scope:
The main purpose for conducting the shear tests was to establish that the maximum shear stresses allowed for masonry structures by existing and proposed draft New Zealand codes are unrealistically low. Cyclic shear tests were run on six reinforced concrete masonry walls with relatively high percentages of steel reinforcement.

The experimental ultimate loads were compared with theoretical and design ultimate loads. The ultimate experimental shear stresses was compared with the design allowable shear stresses. The authors analyzed the influence of base-course slip and compared experimental displacement ductility with code-required ductility.

Variables:
Variables investigated included reinforcement ratios (0.66 and 0.45%, vertical, and 0.66 and 0.34%, horizontal), magnitude of vertical axial load and the use of confining plates in the mortar beds near the bottom of the walls. Two walls were subjected to axial stress levels of 100 psi and the other four walls were not subjected to axial stresses. Three walls had thin stainless steel confining plates installed in the bottom three mortar courses.

Conclusions:
1) Provided that all shear is carried by adequately anchored horizontal steel, higher maximum shear stresses should be allowed for masonry walls by both existing and proposed New Zealand masonry codes.
2) The current (i.e. 1977) undercapacity factor, ϕ, for walls subjected to axial compression and bending, should be increased from 0.65 to 0.85.
3) The test results indicate that current New Zealand design practice overestimates the cracked stiffness of walls by a factor of more than 2.
4) Mortar-bed confining plates did not significantly reduce stiffness degradation in the walls, but did improve damage control in the compression toes.

[72] <u>Tomazevic, M. and Zarnic, R., "The Behavior of Horizontally Reinforced Masonry Shear Walls Subjected to Cyclic Lateral Loading- Part One."</u>

[73] Tomazevic, M. and Zarnic, R., "The Effect of Horizontal Reinforcement on Strength and Ductility of Masonry Walls - Part Two"

Objective and Scope:
The cited references document the first two of three test series conducted in Yugoslavia and partly sponsored by NBS. The third series [74, 75] is described separately because of marked differences in test setup and other factors. Both series used one-third scaled masonry units, an aspect ratio of 1.5, the same test setup, and only undeformed type horizontal reinforcement, placed in every bed joint and bent down at both ends outside the vertical edges of the specimens. Other similarities were: a constant axial load, rotationally-fixed top and bottom surfaces, and cyclic lateral load applied to the bottom of the specimens. Compressive strength was evaluated using horizontally reinforced and unreinforced prisms. Test results are described in great detail showing hysterises curves and envelopes, and all strain and deformation measurements.

Variables:
Mortar - two grades of high-strength cement mortars
Grout - full grouting using the same mortar grades
Horizontal Reinforcement - 0 - 0.37%
Type of Masonry - concrete block units with a U-shaped longitudinal depression at top for placement of horizontal bars [72], cut hollow brick units with six rectanglar cores.

Conclusions:
1. All specimens failed in shear; diagonal cracking initiated at the center.
2. Plain walls failed abruptly with little or no strength gain after shear cracking.
3. Plain walls exhibited lower cracking strength and deformation than reinforced walls.
4. The strength of mortar had the most significant effect on increasing cracking and ultimate strength of reinforced concrete block walls, and on the cracking strength of reinforced brick walls.
5. The strength of reinforcement was not fully activated beyond the minimum reinforcement level because of insufficient bond and anchorage. As a result, higher reinforcement percentages had no effect on ultimate strength and ductility.
6. Strength degradation under cyclic loading was of the order of 5% before and 30% after cracking in reinforced specimens.
7. Ductility factors for plain and minimally-reinforced block walls were 1.2 and 3.2, respectively.
8. The drift capacity of reinforced brick walls was about 4 times that of plain brick walls.

[74] Tomazevic, M., Lutman, M., Velechovsky, T., and Zarnic, R., "Seismic Resistance of Reinforced Masonry Walls, Volume 1: Test Results, Part One."

[75] Tomazevic, M., Lutman, M., Velechovsky, T., and Zarnic, R., "Seismic Resistance of Reinforced Masonry Walls, Volume 2: Test Results, Part Two"

Objective and Scope:
The two references document parts 1 and 2 of the third series of 16 scaled masonry wall tests conducted in Yugoslavia. The specimens were built with 1/2- scale concrete block units of two types: interior blocks contained six rectangular cores and had no grooves at the top as those used in the first series. Exterior blocks were two-core units similar to stretcher blocks used in the U.S. The test setup allowed rotation at the top where the lateral force was applied making the specimen behave as a simple cantilever. Axial load was maintained constant. Vertical reinforcement was placed only in the end cores and horizontal reinforcement having the configuration of a closed loop was placed in every course. Of the two types of mortar used in the first two series, the higher grade was used in this series.

Variables:
Aspect Ratio - h/L = 1.25 and 2.50 (equivalent to 2.5 and 5.0 for walls rotationally fixed at both ends)

Horizontal Reinforcement Ratio - 0 to 0.52%

Conclusions:
1. All walls without horizontal reinforcement failed in shear in a brittle manner.
2. Vertical reinforcement had no effect on resistance.
3. Vertical reinforcement in horizontally-unreinforced walls had no effect on ductility.
4. Horizontal reinforcement improved the shear resistance making the tall walls capable of responding in flexure before shear failure. Flexural failure was characterized by yielding of tensile steel followed by opposite corner crushing and buckling of compression steel.
5. Horizontally-reinforced squat walls typically failed by horizontal debonding near the base following considerable crushing of units and grout in that region.
6. In most cases both vertical and horizontal reinforcement were fully activated.
7. Ductility and deformation capacity of horizontally-reinforced walls were substantially greater than those without horizontal reinforcement.

[14] <u>Gallegos, H. and Casabonne, C., "Cyclic Test of Three Different Types of Masonry Walls."</u>

Objective and Scope:
To observe the mode of failure, the degradation of rigidity and strength and the degradation of ductility for three different types of reinforced brick walls.

The reported results are quite limited as this was a preliminary report on the experimental program. A table summarizing the load magnitude, shear stress, and horizontal displacement at the top of the wall at both first cracking and ultimate is presented. Hysteretic force-displacement curves are presented. The authors referred to the curves as being "stabilized" because the pre-determined displacement cycles were repeated until the hysteretic curves stabilized before advancing to the next higher level of displacement.

Variables:
Each of the three walls incorporated one of the types of masonry wall reinforcement permitted by the Peruvian building regulations. The Type I configuration consisted of a plain masonry wall bounded by a reinforced concrete frame. The Type II wall consisted of hollow units reinforced with vertical bars positioned in the cores and grouted in place and the horizontal reinforcement placed in the bed joint. The Type III specimen was a double-wythe wall, constructed with solid units separated by a reinforced, fully-grouted collar joint. As the scope of this literature review was confined to laterally self-supporting masonry shear walls, the Type I wall test results are not included in this summary. The 79-in square brick walls were subjected to servo-controlled cyclic, horizontal displacements, without the imposition of axial load.

Conclusions:
There are no conclusions presented in this paper. Rather, the authors contrast the modes of failure and the apparent ductility of the test walls. While the hysteresis curves for Types II and III walls were both described as trilinear force-deformation curves, the mode of failure for the Type II wall was more ductile. First cracking in the Type II wall was due to bending stresses in the loading area of the wall. Cracks propagated horizontally at first, followed by cracking along the diagonal. On the other hand, first cracking in the Type III wall was apparently caused by tensile stresses along the diagonals. There was very little reserve strength beyond the first cracking loading. The hysteresis curve was characterized by a relatively rapid strength degradation after reaching the ultimate load.

TABLE 5.1(a) – SCOPE OF MASONRY SHEAR WALL EXPERIMENTAL RESEARCH

REF.NO.	DEFINTION OF SPECIMENS													DEFINITION OF TEST METHOD							REMARKS
	H	H/L	t	Type/No. of Specimens			Reinforcement			Material/No. of Specimens				Boundary Cond.		Edge Load		Shear Load			
YEAR	(in)		(in)	Plain	P.G.	F.G.	Vert (%)	Horiz (%)	Joint	Brick	Block	Comp.	f'mt	Top	Bottom	(psi)	% f'mt	1-Dir.	Cyclic	Dyn.	
3 (87)	24 36 48	1.0 2.0 0.5				5					5		3400	FR (1)	FR (1)			X			(1) Diagonal test method
	24 24 48	1.0 0.5 2.0				3					3		2800	FR	FR			X			
	24 24 48	1.0 0.5 2.0				3					3		2350	FR	FR			X			
	24 24 48	1.0 0.5 2.0				3					3		3300	FR	FR			X			
	24 24 48	1.0 0.5 2.0				3					3		3960	FR	FR			X			
	24 24 48	1.0 0.5 2.0				3					3		5150	FR	FR			X			
	24 36 48	0.5 1.0 2.0				5				5			5100	FR	FR			X			
	24 24 48	1.0 0.5 2.0				3				3			4900	FR	FR			X			
	24	1.0				1				1			4000	FR	FR			X			
1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22

TABLE 5.1(b) - SCOPE OF MASONRY SHEAR WALL EXPERIMENTAL RESEARCH

REF.NO	DEFINTION OF SPECIMENS													DEFINITION OF TEST METHOD							REMARKS
	H	H/L	t	Type/No. of Specim			Reinforcement			Material/No. of Specimens				Boundary Cond.		Edge Load		Shear Load			
YEAR	(in)		(in)	Plain	P.G.	F.G.	Vert (%)	Horiz (%)	Joint	Brick	Block	Comp.	f'mt	Top	Bottom	(psi)	% f'm	1-Dir.	Cyclic	Dyn.	
							.00-1	.00-1													* Test variables: 4 reinforcement ratios,
4	24	1.0	5.9			15	.34-1	.52-1			15		N.A.	FR	FR			X			4 types of mortar
(89)							.60-12	.90-12						(1)	(1)						(1) Diagonal test method
							.89-1	1.33-1													
																					* Test variables: 2 reinforcement ratios,
	47	2.0	5.9			3	.00-1	.00-1			3		N.A.	FR	FR			X			2 types of joint mortar
							.60-2	.90-2													(1) Diagonal test method
	24	0.5	5.9			3	.00-1	.00-1			3		N.A.	FR	FR			X			* Test variables: 2 reinforcement ratios,
							.53-1														(1) Diagonal test method
					1	1	.00	.00			2		1330				4.1				* Test variables: hor. reinf., vert. reinf. & distr.,
8	56	1.2	7.6		0	1	.17	.00			1		1833	FX	FX	55	3.0			X	hor. reinf. anchor
(78)					1	1	.17	.08			2		1833	(1)	(1)	(2)	3.0				(1) Specimens flanked by 2 pinned steel column
					0	1	.17	.34			1		1833				3.0				(2) Axial load increased with lateral load
					1	1	.43	.00			2		1905				2.9				(do)
	56	1.2	7.6		1	1	.43	.17			2		1905	FX	FX	55	2.9			X	* Same as above
					0	1	.43	.48			1		1330			(2)	4.1				(1),(2) Same as above
					1	1	.00	.00		2			2535				2.2				
	56	1.2	7.4		0	1	.18	.00		1			2535	FX	FX	56	2.2		X		* Same as above
					1	1	.44	.00		2			2866			(2)	2.0				(1),(2) Same as above
					0	1	.18	.09		1			2722				2.1				
					1	0	.18	.09		1			2722								* Same as above
	56	1.2	7.4		0	2	.18	.44		2			2628(3)	FX	FX	56	2.1		X		(1),(2) Same as above
					1	1	.45	.18		2			2722			(2)					(3) Average of f'm 2535 psi and 2722 psi
					0	2	.45	.62		2			2628(3)								
						1	.00	.00				1									* Same as above
	56	1.2	10.0			1	.13	.00				1	2507	FX	FX	41	2.2		X		(1),(2) Same as above
			(2)			1	.13	.07				1				(2)					(4) Two-wythe brick composite walls
						1	.13	.32				1									
						1	.32	.00				1									* Same as above
	56	1.2	10.0			1	.32	.13				1	2507	FX	FX	41	2.2		X		(1),(2) Same as above
						1	.32	.46				1				(2)					(4) Same as above
	32	1	7.6			3				0	3	0									
	36	1	7.4			3				3	0	0		FR	FR			X			(4) Same as above
	36	1	10.0			3				0	0	3		(5)	(5)						(5) Diagonal test of F.G. companion wallettes

TABLE 5.1(c) – SCOPE OF MASONRY SHEAR WALL EXPERIMENTAL RESEARCH

REF.NO.	DEFINTION OF SPECIMENS													DEFINITION OF TEST METHOD							REMARKS
	H	H/L	t	Type/No. of Specimens			Reinforcement			Material/No. of Specimens				Boundary Cond.		Edge Load		Shear Load			
YEAR	(in)		(in)	Plain	P.G.	F.G.	Vert (%)	Horiz (%)	Joint	Brick	Block	Comp.	f'mt	Top	Bottom	(psi)	% f'mt	1–Dir.	Cyclic	Dyn.	
													5150			1385	27				(1) Based on prism tests w/ h/t = 2.2, 6.0, resp.
12	48	1.0	3.55	1						1			4309	FR	FR	(3)	(3)	X			(2) Diagonal tests
(77)													(1)	(1)	(1)						(3) Based on prism tests w/ h/t = 2.2
													3350			745	23				(1) Based on prism tests w/ h/t = 2.2, 4.6, resp.
	48	1.0	10.00			4						4	3423	FR	FR	1273	39	X			(2),(3): Same as above
													(1)			1389	42				(4) Two-wythe brick composite walls
	48	1.0	3.55	2		2				10		12	5150			1707	52				(2),(4): Same as above
	24		10.00	5		5						(4)	3350	FR	FR			X			(5) Single and double wythe brick prism tests,
	16		10.00	3		5							(5)								respectively
	8	0.50	3.55	2		5						20	5150								
	16	2.00	10.00	2		5				8		(4)	3350	FR	FR			X			(2),(4),(5): Same as above
	16	0.50	10.00	2		5							(5)								
	32	2.00	10.00	2		5															
																401	14				
	48	1.0	7.6	4					4				2962	FR	FR	627	21	X			(2) Same as above
																866	29				
																997	34				
																694	24				
	48	1.0	7.6			3			3				2847	FR	FR	981	34	X			(2) Same as above
																1264	44				
	24			5		5			18				2962								(2) Same as above
	32	1.0	7.6	2		2							2847	FR	FR			X			(6) Hollow and fully grouted block prism
	48			2		2							(6)								tests, respectively
	16	0.5		4		5							2962								
	32	2.0	7.6	4		2			26				2847	FR	FR			X			(2),(6): Same as above
	24	0.5		2		5															
	48	2.0		2		2															
	52	(7)	7.6			5			5				2847	FR	FR			X			(7): Circular grouted block specimens,
									(7)					(7)	(7)						diametral load tests
1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22

TABLE 5.1(d) - SCOPE OF MASONRY SHEAR WALL EXPERIMENTAL RESEARCH

REF.NO.	DEFINTION OF SPECIMENS													DEFINITION OF TEST METHOD							REMARKS
	H	H/L	t	Type/No. of Specimens			Reinforcement			Material/No. of Specimens			f'mt	Boundary Cond.		Edge Load		Shear Load			
YEAR	(in)		(in)	Plain	P.G.	F.G.	Vert (%)	Horiz (%)	Joint	Brick	Block	Comp.		Top	Bottom	(psi)	% f'mt	1-Dir.	Cyclic	Dyn.	
14 (83)	79	1.0	4-3 10-3			6	.06-3 .49-3	.05-3 .14-3		3		3 (1)		FR	FX			2	4		(1) Brick-brick composite
15,16 (85)	79	0.6	6	7					2	7			N.A.	FR	FX	110-5 (1) (2) 350-2		5	2		* All walls had flanges on each end (1) Axial load was in tension on all specimens (2) Moment applied at top of 2 specimens
26 (78)	80	1.9	7.4			2	.00-1 .51-1	.00-1 .00-1		2			4806 4502 (1)	FX (2)	FX (2)	60 (3)	1.3 (3)		.02 Hz		(1) Based on prism h/t = 1.5, 5.0 resp. (2) Specimens flanked by 2 pinned steel cols. (3) Initial axial load; % f'm based on lower strength
	80	1.9	7.4			4	.51-1 .51-1 .51-1 .51-1	.20-1 .30-1 .40-1 .50-1		4			4806 4502	FX	FX	60	1.3		.02 Hz		(1), (2), (3): Same as above
	80	1.9	7.4		3		.51-1 .51-1 .51-1	.00-1 .20-1 .30-1		3			4806 4502	FX	FX	60	1.3		.02 Hz		(1), (2), (3): Same as above
	80	1.9	10.0			5 (4)	.38-5	.00-2 .15-1 .22-1 .37-1		5 (4)			3384 3315 (5)	FX	FX	60	1.8		.02 Hz		(2), (3): Same as above (4) Two-wythe brick composite walls (5) based on prism h/t = 8.0, 5.0 resp.
	36	1.0	7.4 10.0			3 3				3		3 (4)		FR (6)	FR (6)			X			(4) Same as above (6) Diagonal test of fully grouted companion wallettes
1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22

TABLE 5.1(e) – SCOPE OF MASONRY SHEAR WALL EXPERIMENTAL RESEARCH

REF.NO.	DEFINTION OF SPECIMENS													DEFINITION OF TEST METHOD							REMARKS
	H	H/L	t	Type/No. of Specimens			Reinforcement			Material/No. of Specimens				Boundary Cond.		Edge Load		Shear Load			
YEAR	(in)		(in)	Plain	P.G.	F.G.	Vert (%)	Horiz (%)	Joint	Brick	Block	Comp.	f'mt	Top	Bottom	(psi)	% f'mt	1-Dir.	Cyclic	Dyn.	
27 (79)	40	0.5	7.6			3	.30–3	.00–1 .05–1 .10–1			3		3604 2988 (1)	FX (1)	FX (1)	52 (3)	1.7 (3)		.02 Hz		(1) Based on prism tests w/ h/t = 2.0, 5.0 resp. (2) Specimens flanked by 2 pinned steel cols. (3) Initial axial load; % f'm based on lower strength
	40	0.5	7.6			3	.30–3	.15–1 .20–1 .29–1			3		3604 2988	FX	FX	52	1.7		.02 Hz		(1),(2),(3): Same as above * Test variables: horiz. reinf.–all tests
	32	0.5	7.4			3	.31–3	.00–1 .08–1 .15–1		3 (4)			3589 2838	FX	FX	56	2.0		.02 Hz		(1),(2),(3): Same as above (4) Fully-grouted hollow brick walls
	40	0.5	7.4			3	.31–3	.23–1 .31–1 .52–1		3			3589 2838	FX	FX	56	2.0		.02 Hz		(1),(2),(3),(4): Same as above
	40	0.5	10.0			3	.23–3	.00–1 .06–1 .11–1		3 (5)			2948 2876	FX	FX	50	1.7		.02 Hz		(1),(2),(3): Same as above (5) Two–wythe grouted brick composite walls
	40	0.5	10.0			3	.23–3	.17–1 .23–1 .38–1		3			2948 2876	FX	FX	50	1.7		.02 Hz		(1),(2),(3),(5): Same as above
	32	1	7.6			3					3		3604 2988	FR (6)	FR (6)			X			(6) Diagonal tests of fully–grouted companion wallettes
	32	1	10.0			3						3 (7)	2948 2876	FR (7)	FR (7)			X			(7) Diagonal tests of fully grouted two–wythe solid brick companion wallettes
1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22

TABLE 5.1(f) - SCOPE OF MASONRY SHEAR WALL EXPERIMENTAL RESEARCH

	DEFINTION OF SPECIMENS													DEFINITION OF TEST METHOD							
REF.NO.	H	H/L	t	Type/No. of Specimens			Reinforcement			Material/No. of Specimens				Boundary Cond.		Edge Load		Shear Load			REMARKS
YEAR	(in)		(in)	Plain	P.G.	F.G.	Vert (%)	Horiz (%)	Joint	Brick	Block	Comp.	f'mt	Top	Bottom	(psi)	% f'mt	1-Dir.	Cyclic	Dyn.	
28 (84)	40	1.0	5.5		3 (1)	9 (1)	.07 : .16			12			592 981 : 1373(2)	FR (3)	FX (3)				X		(1) Mortar was used for grouting (2) f'm for P.G. and F.G. walls, resp. (3) Cantilever wall tests
29 (85)	71	1.5	?			6	.96	.17-3 .67-3			6		3367	FX	FX	71#		X			* Test variables: shear reinforcement, spiral reinforcement, splice of vert. reinf.
	71	1.5	?			2				2				FX	FX	71#		X			
	71	1.5	?			1 (1)						1 (1)		FX	FX	71#		X			(1) Reinforced concrete shear wall
31 (88)	71	1.5	7.6			6 (1)	.43-1 .50-2 .68-1 .75-2	.53-5 .26-1			6		3800	FX	FX	260	6.8		X		(1) Four walls had block-high openings
1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22

TABLE 5.1(g) - SCOPE OF MASONRY SHEAR WALL EXPERIMENTAL RESEARCH

REF.NO.	DEFINTION OF SPECIMENS													DEFINITION OF TEST METHOD							REMARKS
	H	H/L	t	Type/No. of Specimens			Reinforcement			Material/No. of Specimens			f'mt	Boundary Cond.		Edge Load		Shear Load			
YEAR	(in)		(in)	Plain	P.G.	F.G.	Vert (%)	Horiz (%)	Joint	Brick	Block	Comp.		Top	Bottom	(psi)	% f'mt	1-Dir.	Cyclic	Dyn.	
33 (76)	96	1.0	7.6	8					2 (1)		8		620	FR (2)	FX	33-2 115-3 231-3	5.4 18.5 37.2	X			* Cantilever wall tests (1) Truss joint reinforcement every other course (2) Tie rods at top loaded corner used for two specimens: modified ASTM E72 method
	48	0.5	7.6	10					3		10		620	FR	FX	56-3 112-4 224-3	9.0 18.0 36.0	X			* Cantilever wall tests (1) Same as above
	48	1.0	7.6	7					3		7		620	FR	FX	58-1 115-3 231-3	9.2 18.5 37.2	X			* Cantilever wall tests (1) Same as above
	96	1.0	3.6	12						12			3560-6 6200-6 (4)	FR (3)	FX	205-4 495-2 980-4 1380-2	3.3 : 27.5	X			* Cantilever wall tests (3) Tie rods at top loaded corner used for four specimens: modified ASTM E72 method (4) Regular and high bond mo hig o
	48	0.5	3.6	12						12			3560-6 6200-6	FR	FX	244-2 482-4 976-4 1358-2	6.9 : 27.1	X			* Cantilever wall tests (4) Same as above
	48	1.0	3.6	8						8			3560-4 6200-4	FR	FX	480-2 965-4 1363	13.4 : 27.4	X			* Cantilever wall tests (4) Same as above
	48	1.0	3.6	16						16			3560-8 6200-8	FR (5)	FX (5)	0 : 2647	0 : 42.7	X			(4) Same as above (5) Diagonal load tests
37,56 (88)	78	2.2	7.6			1 (4)	0.53	1.06			1		2800	FX	FX	12	0.4			X (2)	(1) T-shaped flanged wall (2) Free vibration, sinusoidal excitation and shake-table tests
	144	3.1	5.6			4 (1)	.68-2 .31-2				4		2150-2 1900-2	FX	FX	100	4.7-2 2.8-2		X		* Tests are in progress
1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22

TABLE 5.1(h) - SCOPE OF MASONRY SHEAR WALL EXPERIMENTAL RESEARCH

REF.NO. YEAR	H (in)	H/L	t (in)	Type/No. of Specimens: Plain	P.G.	F.G.	Reinforcement: Vert (%)	Horiz (%)	Joint	Material/No. of Specimens: Brick	Block	Comp.	f'mt	Boundary Cond.: Top	Bottom	Edge Load: (psi)	% f'mt	Shear Load: 1-Dir.	Cyclic	Dyn.	REMARKS
39 (87)	63	0.8-1 1.2-3	5.9		4		.11-1 .16-1 .90-2	.08-1 .12-3		4			2350-3 2550-1	FX	FX	0	0		X		* Unit strengths ranged from 3200 to 10500 psi based on gross area
	67	1.5	7.5			4	.61	0 .24 .48 .96		4			4150	FX	FX	284	6.8		X		
	71	1.5	7.5			1	.68	0			1		3230	FX	FX	284	8.8		X		
	71	1.1-1 1.5-2 2.3-1	5.9			4	.92-1 .94-2 1.18-1	.13-1 .18-2 .27-1			4		3160	FX	FX	71	2.2		X		
	71	1.5	7.5			1	.68	.29			1		3230	FX	FX	284	8.8		X		
	71	1.5	7.5			6	.68-2 1.04-4	.53			6		3230-2 3780-2 4200-1 4550-1	FX	FX	284	8.8-2 7.5-2 6.8-1 6.2-1		X		
	71	1.5	7.1			1	1.10	.56			1		3830	FX	FX	284	7.4		X		
	71	1.5	7.5			1	1.04	.62 (1)			1		3975	FX	FX	284	7.1		X		(1) Includes area of joint reinforcement placed every joint
	71	1.5	7.5			2	.68 1.04	1.06			2		3230	FX	FX	284	8.8		X		
1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22

TABLE 5.1(i) – SCOPE OF MASONRY SHEAR WALL EXPERIMENTAL RESEARCH

REF.NO.	DEFINTION OF SPECIMENS													DEFINITION OF TEST METHOD							REMARKS
	H	H/L	t	Type/No. of Specimens			Reinforcement			Material/No. of Specimens				Boundary Cond.		Edge Load		Shear Load			
YEAR	(in)		(in)	Plain	P.G.	F.G.	Vert (%)	Horiz (%)	Joint	Brick	Block	Comp.	f'mt	Top	Bottom	(psi)	% f'mt	1-Dir.	Cyclic	Dyn.	
39 (87)	71	1.3–1 1.4–4	5.9		5		.88–1 .90–4	0			5		1175–2 1380–2 2260–1	FX	FX	0–2 71–1 142–2	0–2 6.0–2 12.0–1		X		
	71	1.3–1 1.4–8	5.9		9		.83–1 .90–5 .16–2 .33–1	.09			9		1175–1 1380–3 2260–5	FX	FX	0–4 71–2 142–2 213–1	0–4 2.1–1 6.3–2 9.4–1		X		
	71	1.0–2 2.0–2	5.9		4		.69–1 .85–1 .87–1 1.23–1	.07–2 .12–2			4		1380–2 2260–2	FX	FX	0	0		X		
	71	0.9–2 1.3–1 1.4–2	5.9		5		.60–2 .88–1 .90–2	.12–2 .18–3			5		1175–1 1380–1 1280–2 2260–1	FX	FX	0–1 71–3 142–1	0–1 6.0–1 6.3–1 5.6–2		X		
	71	1.0–1 1.3–1 1.9–2			4		.56–1 .58–1 .64–2	.13–1 .17–1 .24–2			4		1280	FX	FX	71	5.5		X		
	71	1.4–2 1.3–1			3		.88–1 .90–2	.27			3		1175 1380 2260	FX	FX	71–1 0–1 142–1	6.0–1 0–1 6.0–1		X		
	71	1.3	5.9		1		.88	.41			1		1175	FX	FX	71	6.0		X		
40 (85)	71	1.5	7.5			6	.68–4 1.0–2	0–1 .29–1 .53–2 1.06–2			6		3230	FX	FX	284	8.8		X		
41 (88)	67	1.5	7.5			5	1.1–4 .56–1	0–1 .29–1 .53–2 1.06–2			5		3830	FX	FX	284	7.4		X		
43 (76)	64	2.0	5.6		2	15	0–1 .21–6 .46–2 .49–8	0–11 .48–2 .95–4			17		2430–2 2033–6 2362–7 2430–2	FX	FX	0–2 125–6 250–7 500–2	0–2 6.1–6 10.8–7 21.2–2		X		* Coupled double–pier tests. (1) 0.02 Hz; (2) 3 Hz; duplicates tested at 2 rates

TABLE 5.1(j) – SCOPE OF MASONRY SHEAR WALL EXPERIMENTAL RESEARCH

REF.NO.	DEFINTION OF SPECIMENS													DEFINITION OF TEST METHOD							REMARKS
	H	H/L	t	Type/No. of Specimens			Reinforcement			Material/No. of Specimens			f'mt	Boundary Cond.		Edge Load		Shear Load			
YEAR	(in)		(in)	Plain	P.G.	F.G.	Vert (%)	Horiz (%)	Joint	Brick	Block	Comp.		Top	Bottom	(psi)	% f'mt	1-Dir.	Cyclic	Dyn.	
48 (87)	70.9	1.6	7.5			4		.17			4		3314	FX	FX	71 284 569 853	2.1 8.6 17.2 25.7		X		Test variable: axial load
	70.9	0.9 1.6 2.3	7.5			3	.68	.17			3		2603 3314 2603	FX	FX	71	2.7 2.1 2.7		X		Test variable: h/l
	70.9	1.6	7.5			4	.68	.17 .17 .67 .67	X X		4		3314 3115 3115 3115	FX	FX	71	2.1 2.3 2.3 2.3		X		Test variable: horiz. reinf.; joint reinf.
50 (85)	72	1.5	9.1			5	.12–3 (2) .12–2	.17–3 .13–1 .04–1 (3)				5 (1)	2515	FR	FX	467–2 400–2 490–1	18.6 15.9 19.5	X			(1) Brick–brick w/reinforced clloar joints (2) Welded wire fabric in collar joint (3) Does not include area of joint reinforcement
51 (85)	72	1.5	9.1			6	.12–4 (2) .12–2	.17–4 (2) .13–1 0–1				6 (1)	1509	FR	FX	165	10.9	X			(1) Brick–brick w/reinforced clloar joints (2) Welded wire fabric in collar joint
53 (77)	65	0.7	5.6			6	.66–4 .34–2	.66–3 .45–3			6 (1)		2950	FX	FX	0–4 100–2	0–4 3.4–2		X (2)		(1) Open–ended bond beam block (2) 2 cycles to displ. ductility factor = 2, 4, ...
55 (82)	236	2.5	5.5			3	.72	.91			3 (1)		3625	FX	FX	275–2 108–1	7.6–2 3.0–1		X		(1) Open–ended bond beam block
1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22

TABLE 5.1(k) – SCOPE OF MASONRY SHEAR WALL EXPERIMENTAL RESEARCH

REF.NO.	DEFINTION OF SPECIMENS													DEFINITION OF TEST METHOD							REMARKS
	H	H/L	t	Type/No. of Specimens			Reinforcement			Material/No. of Specimens				Boundary Cond.		Edge Load		Shear Load			
YEAR	(in)		(in)	Plain	P.G.	F.G.	Vert (%)	Horiz (%)	Joint	Brick	Block	Comp.	f'mt	Top	Bottom	(psi)	% f'mt	1-Dir.	Cyclic	Dyn.	
64,65,66 (88)	72	1.0	5.6			3	.38-2 .74-1	.24-2 .14-1			3		2900-2 3000-1	FX	FX	200-1 270-2 200-1	6.9 9.3 9.0		X		
	72	1.0	5.6			3	.38-1 .74-2	.14			3		2600	FX	FX	0-2 100-1	0-2 3.8-1		X		
	72	1.0	5.6			3	.38-2 .74-1	.14-2 .24-1			3		3000	FX	FX	0 100 270	0 3.3 9.0		X		
	72	1.0	5.6			6	.40-4 .56-2	.26-4 .14-2		6			3800	FX	FX	100-1 280-2	2.6 7.4		X		
	72	1.0	5.6			3	.74-1 .38-2	.14-1 .24-2			3		3200	FX	FX	0-1 100-2	0-1		X		
	72	1.0	5.6			3	.54	.14-1 .24-2			3		3300	FX	FX	270-2 100-1	8.2-2 3.0-1		X		
	72	1.0	5.6			1	.74	.24			1		2500	FX	FX	270	10.8		X		
1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22

TABLE 5.1(I) – SCOPE OF MASONRY SHEAR WALL EXPERIMENTAL RESEARCH

REF.NO.	DEFINTION OF SPECIMENS													DEFINITION OF TEST METHOD							REMARKS
	H	H/L	t	Type/No. of Specimens			Reinforcement			Material/No. of Specimens			f'mt	Boundary Cond.		Edge Load		Shear Load			
YEAR	(in)		(in)	Plain	P.G.	F.G.	Vert (%)	Horiz (%)	Joint	Brick	Block	Comp.		Top	Bottom	(psi)	% f'mt	1-Dir.	Cyclic	Dyn.	
69 (85)	56	1.2	7.6			2	.17	.34			2		3359			273 437	8.1 13.0		X		* Test variables: axial load, horiz. reinf. and anchorage, vert. reinf. and distribution
	56	1.2	5.6–4 5.6			6	.44	.34–2 .17–2 .34–1 .17–1			6		2297–2 2196–2 2196–1 2196–1			400–2 400–2 400–1 100–1	17.4–2 18.2–2 18.2–2 4.6–1		X		* Do
	56	1.2	5.6			4	.44	.00–1 .17–1 .17–1 .17–1	.12–1 .35–1 .00–1 .00–1		4		2196			400–1 400–1 250–1 400–1	18.2–1 18.2–1 11.4–1 18.2–1		X		* Do
	56	1.2	7.4			3	.17	.44		3			3847			450–1 56–1 282–1	11.7–1 1.5–1 7.3–1		X		* Do
	56	1.2	5.6			6	.44	.23–3 .57–3		6			2198			400	18.2		X		* Do
	56	1.2	5.6			5	.68–1 .46–1 .46–1 .23–1 .44–1	.23–1 .57–1 .30–1 .74–1 .00–1	.00–4 .12–1	5			2918–4 4008–1			400–4 400–1	13.7–4 10.0–1		X		* Do
	56	1.2	10.0			6	.13	.06–1 .32–1 .06–1 .07–1 .06–1 .32–1				6 (1)				42–2 220–2 330–2	1.7–2 8.9–2 13.3–2		X		* Do (1) Two-wythe grouted brick composite walls
	32–3 36–6	1	7.6–1 7.4–1 5.6–4 10–3			9				3	3	3	2195 : 4007	FR (2)	FR (2)			X			(2) Diagonal tests of companion wallettes
1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22

TABLE 5.1(m) – SCOPE OF MASONRY SHEAR WALL EXPERIMENTAL RESEARCH

REF.NO.	DEFINTION OF SPECIMENS													DEFINITION OF TEST METHOD							REMARKS
	H	H/L	t	Type/No. of Specimens			Reinforcement			Material/No. of Specimens			f'mt	Boundary Cond.		Edge Load		Shear Load			
YEAR	(in)		(in)	Plain	P.G.	F.G.	Vert (%)	Horiz (%)	Joint	Brick	Block	Comp.		Top	Bottom	(psi)	% f'mt	1-Dir.	Cyclic	Dyn.	
70 (85)	71	1.5	8.0			6 (1)	.68	.22–4 .90–2			6		3480	FX	FX	289–4 72–2	8.4–4 2.1–2		X		(1) 3 walls were rectangular in shape; 3 walls were flanged
72 (84)	24	1.5	2.5			16 (1)		.00–4 .18–4 .32–4 .37–4(2)			16 (3)		788 893	FX	FX	241	30.6 27.0		X		(1) Solid units w/ notches at top (2) Fy = 45 psi, smooth steel bars (3) 1/3 scale units (4) All hollows filled with mortar
73 (84)	22	1.5	2.5			16 (6)		.00–4 .17–4 .30–4 .38–4		16 (5)			1419 1332	FX	FX (7)	258	18.2 19.4		X		(2),(3),(4): Same as above (5) Cored units (6) All cores filled with mortar (7) Displacement of bottom of specimens
73,74 (86,87)	30	1.2	3.9			16	.26–8	.00–4 .14–4 .28–4 .52–4			16 (8)		1306	FX	FX	142	10.9		X		(2),(4): Same as above (8) 1/2 scale units (9) Displacement of top of specimens
77 (84)	32	0.9	5.9			6	.76	.7		6 (1)			2800 (2)	FX (9)	FX	142 284	5.1 10.1 (2)			X	(1) Channel and I–shaped hollow units (2) Estimate
	10–4 11–1 20–1	0.9 1.0 2.5	1.3–4 2.0–2	4		2 (4)	.00–2 .66–2 .39–1 .59–1	.00–2 .00–2 .43–2		6 (3)			2800	FX	FX	?			X		(1),(2); Same as above (3) Two walls have flange walls at the ends
79 (80)	56	0.7	7.5	1		2 (1)	.06 (2)	.04 (2)			3		?	FR	FX	75–2 250–1			X		(1) Not grouted but plaster coating applied over WWF on each face (2) 2 walls externally reinforced with welded wire fabric
1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22

TABLE 5.1(n) - SCOPE OF MASONRY SHEAR WALL EXPERIMENTAL RESEARCH

REF.NO.	DEFINTION OF SPECIMENS													DEFINITION OF TEST METHOD							REMARKS
	H	H/L	t	Type/No. of Specimens			Reinforcement			Material/No. of Specimens				Boundary Cond.		Edge Load		Shear Load			
YEAR	(in)		(in)	Plain	P.G.	F.G.	Vert (%)	Horiz (%)	Joint	Brick	Block	Comp.	f'mt	Top	Bottom	(psi)	% f'mt	1-Dir.	Cyclic	Dyn.	
81 (85)	64	1.3	7.7	2							2		1367 (1)	FX	FX	77 232	5.7 17.0		X		* Variable: aspect ratio (1) Average prism strength for 2 wall specimen
	64	0.8	7.7	2							2		955 1058	FX	FX	129 206	13.5 19.5		X		
	64	0.7	7.7	3							3		980 1342 1445	FX	FX	103 155 206	10.5 11.5 14.2		X		
80 (85)	64	1.3	7.7	2							2		748 1084	FX	FX	85 222	11.4 20.6	X (1)			* Variable: block strength, mortar strength, axial stress (1) Basically monotonic until cracking; one reversed cycle
	64	0.7	7.7	2							2		774 1032	FX	FX	114 163	14.2 15.8	X (1)			* About half of the walls were constructed with face-shell bedding; the remainder were fully bedded
	64	1.0	7.7	4							4		1032	FX	FX	83 124 165 206	8.0 12.0 16.0 20.0	X (1)			
	64	1.0	7.7	2							2		774 1006	FX	FX	88 129	11.3 12.8	X (1)			
	64	1.0	7.7	2							2		1032 748	FX	FX	175 217	17.0 29.0	X (1)			
	64	1.0	7.7	3							3		841 1058 774	FX	FX	52 88 129	6.1 8.3 16.7	X (1)			
	64	1.0	7.7	2							2		1109 1109	FX	FX	175 217	15.8 19.5	X (1)			
1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22

TABLE 5.1(o) – SCOPE OF MASONRY SHEAR WALL EXPERIMENTAL RESEARCH

REF.NO.	DEFINTION OF SPECIMENS													DEFINITION OF TEST METHOD							REMARKS
	H	H/L	t	Type/No. of Specimens			Reinforcement			Material/No. of Specimens			f'mt	Boundary Cond.		Edge Load		Shear Load			
YEAR	(in)		(in)	Plain	P.G.	F.G.	Vert (%)	Horiz (%)	Joint	Brick	Block	Comp.		Top	Bottom	(psi)	% f'mt	1-Dir.	Cyclic	Dyn.	
83 (83)	64	1.0	7.7	8							8		680	FX	FX	18-2 30-5 38-1	1.1 4.4 5.6	2		6 (1)(2)	(1) Fatigue testing – 100,00 + cycles (2) Loading frequencies ranged up to 6.5 Hz
	64	1.3	7.7	5 (3)							5		700	FX	FX	25	3.6	3		2 (4)	(3) L-shaped walls; one leg – 64", – 48" (4) Loading frequency = 2 – 6 Hz
82 (84)	64	1.0	7.7	2							2		1850 1800	FX	FX	120 160	6.5 8.9	X (1)			(1) Basically monotonic until cracking; one reversed cycle after first cracking * Variable: axial load magnitude
	64	1.0	7.7	1							1		2150	FX	FX	240	11.2	X			Same note as (1) above
	64	1.0	7.7	2							2		1850 2100	FX	FX	300 320	16.2 15.2	X			Same note as (1) above
	64	1.0	7.7	2							2		2800 2050	FX	FX	410 410	14.6 20.0	X			Same note as (1) above
	64	1.0	7.7	1							1		2000	FX	FX	510	25.5	X			Same note as (1) above
1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22

TABLE 5.1(p) - SCOPE OF MASONRY SHEAR WALL EXPERIMENTAL RESEARCH

REF.NO.	DEFINTION OF SPECIMENS													DEFINITION OF TEST METHOD							REMARKS
	H	H/L	t	Type/No. of Specimens			Reinforcement			Material/No. of Specimens			f'mt	Boundary Cond.		Edge Load		Shear Load			
YEAR	(in)		(in)	Plain	P.G.	F.G.	Vert (%)	Horiz (%)	Joint	Brick	Block	Comp.		Top	Bottom	(psi)	% f'mt	1-Dir.	Cyclic	Dyn.	
85 (89)	56	1.2	7.6	1	9 (1)			0-1 .02-1 .18-2 .18-2 .25-3	4 (3)		10		1600 (2)	FX	FX	107	6.7		X		(1) Only bond beams were grouted (2) Estimated value (3) Two walls with joint reinf. at 8" o.c. and two walls with joint reinf. at 16" o.c.
86 (75)	75	1.0	3.6	32						32			3190-8 4830-8 5170-8 6100-8	FR	FR			X			* Diagonal tests
1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22

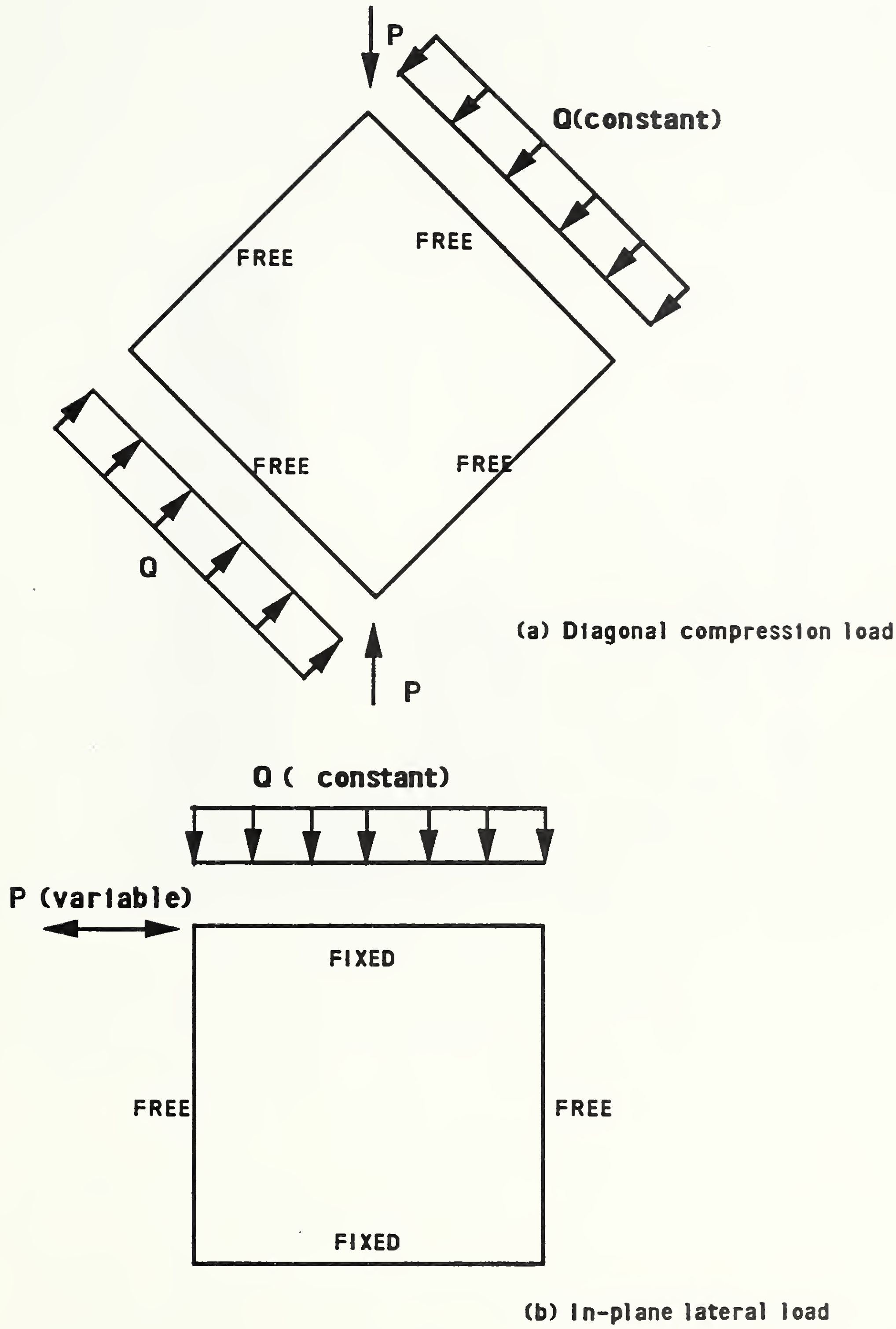


Figure 5.1 - Shear wall test methods

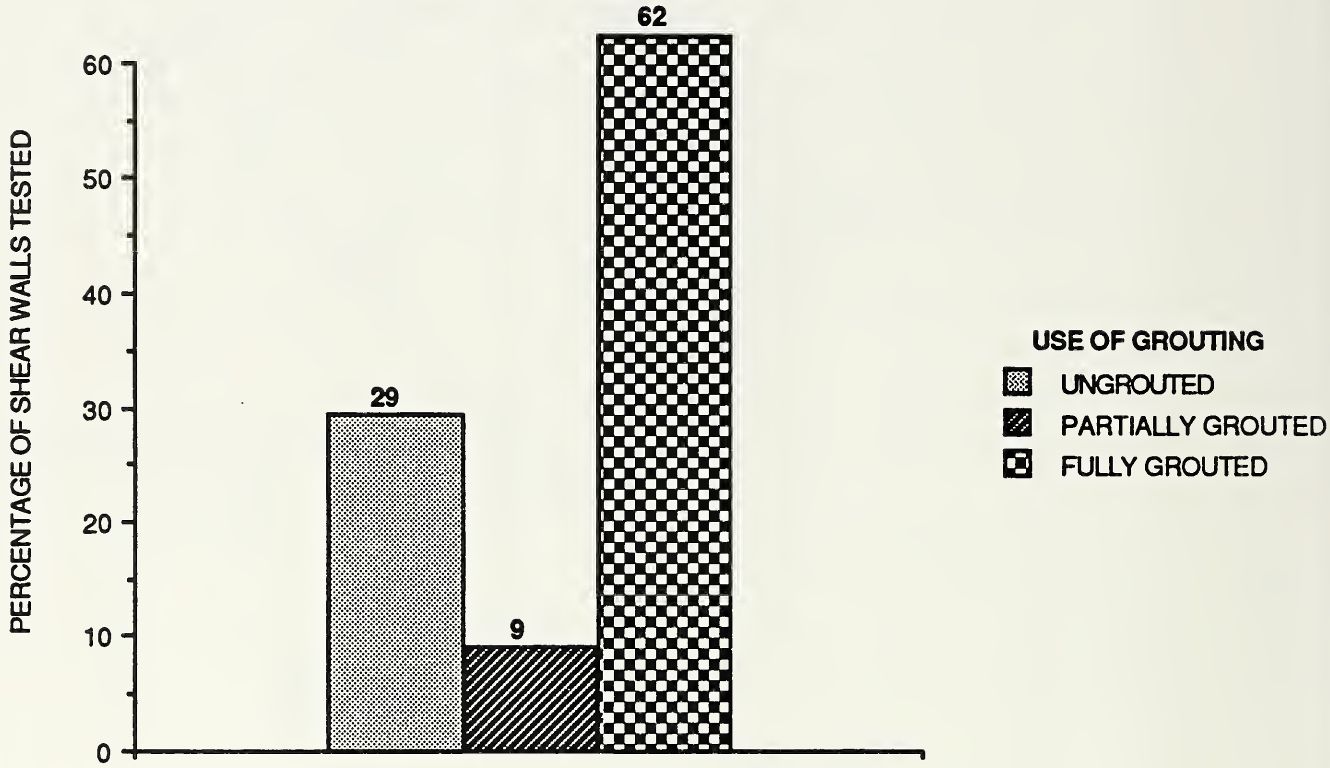


Figure 5.2 - Use of grouting in tested shear walls

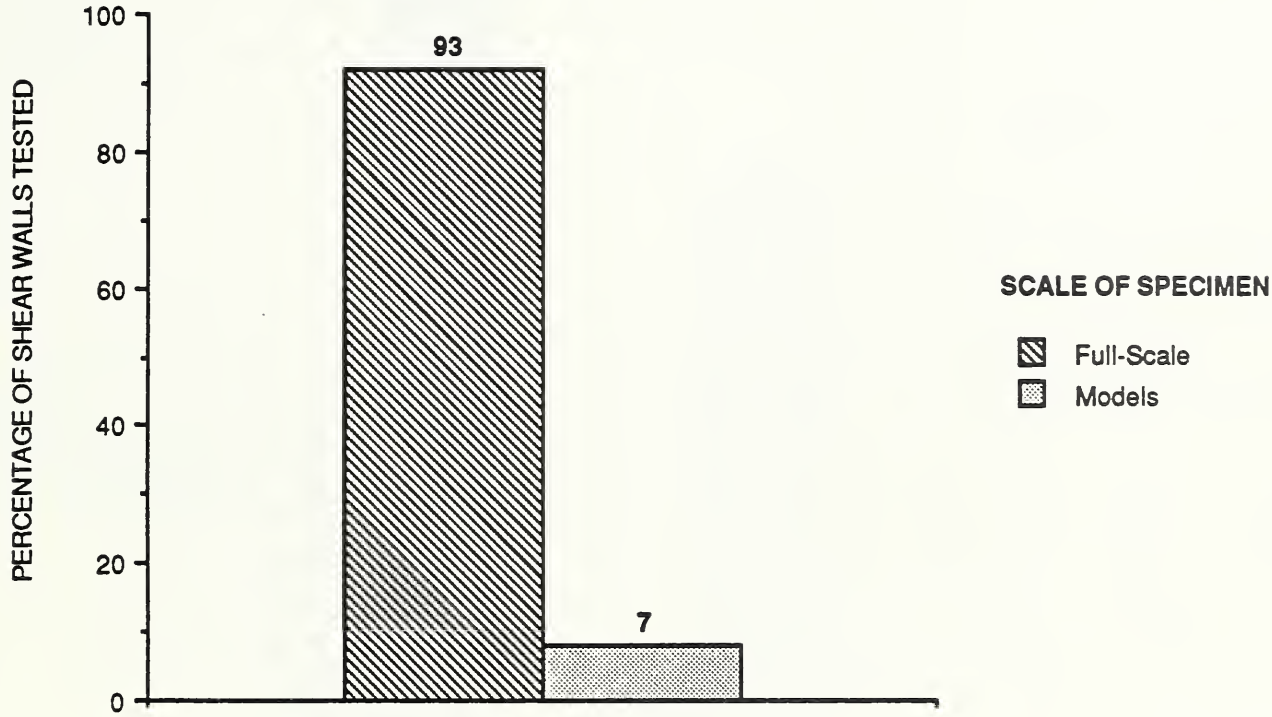


Figure 5.3 - Scale of shear walls tested

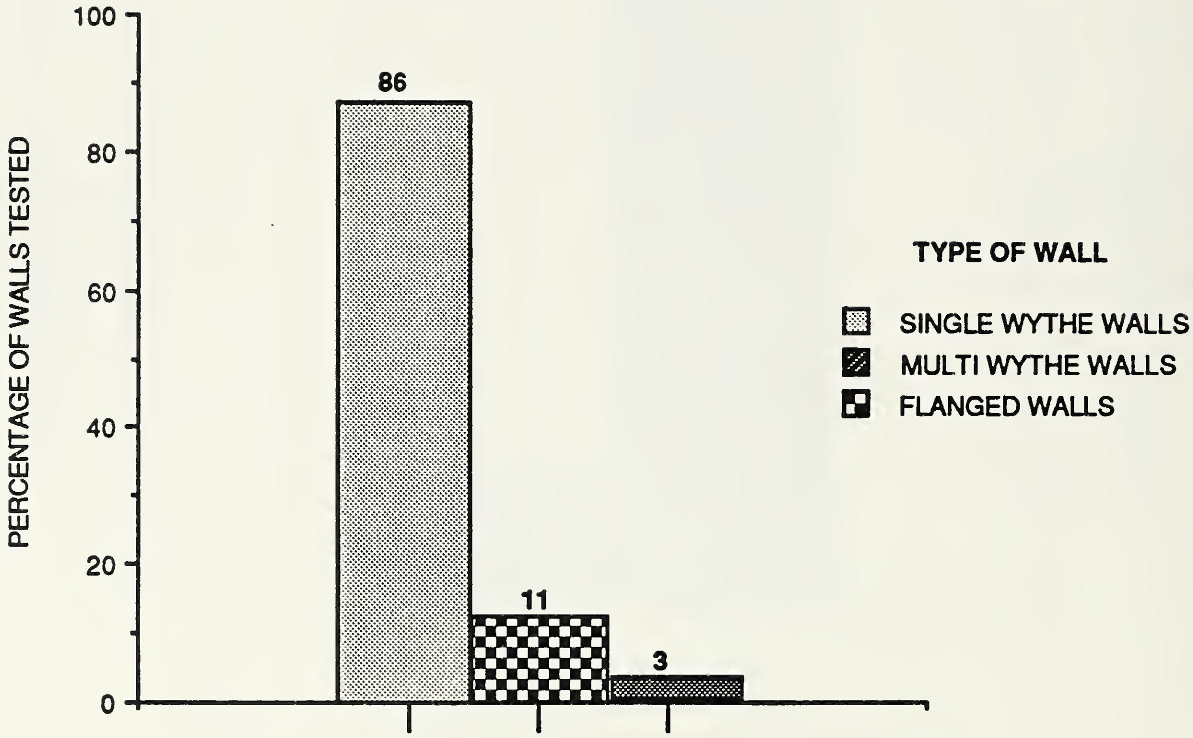


Figure 5.4 - Types of shear walls tested

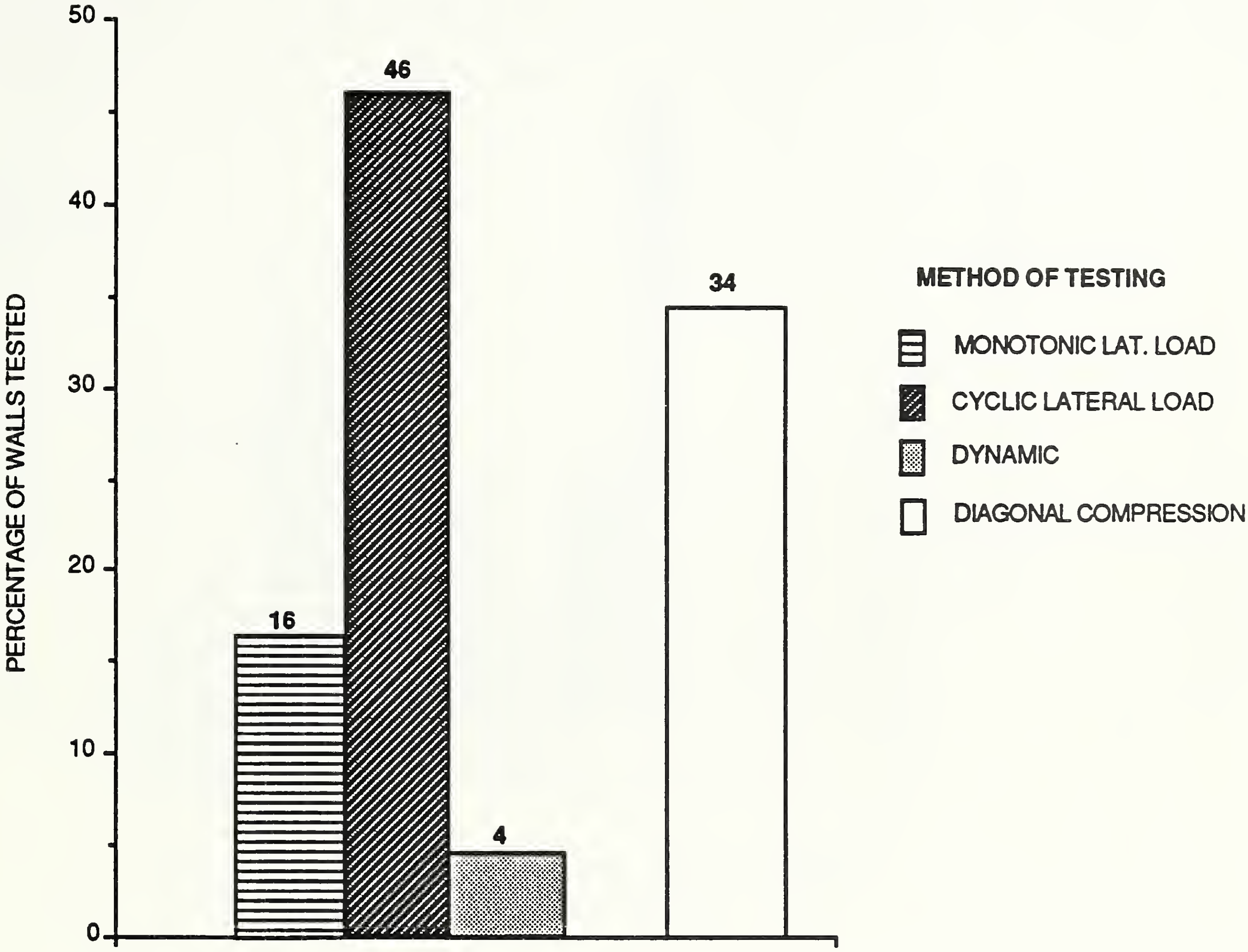


Figure 5.5 - Method of testing shear walls

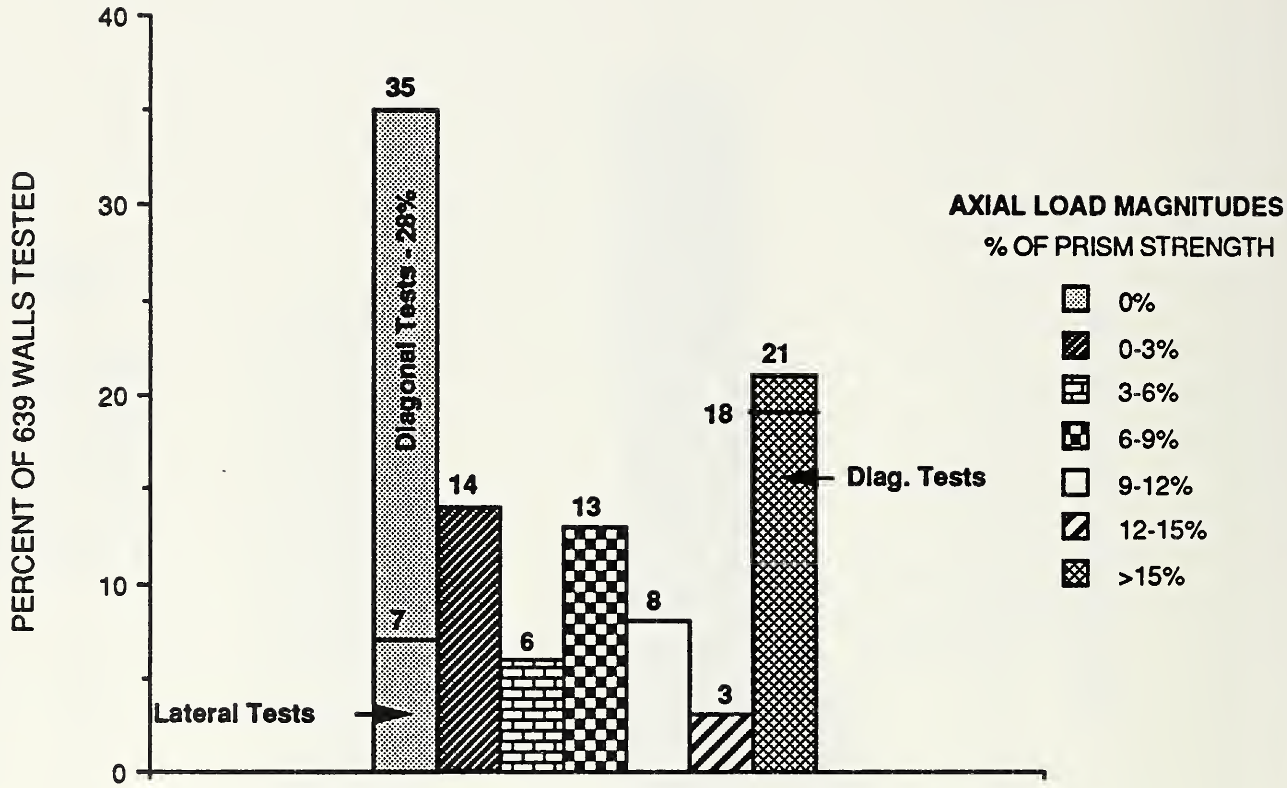


Figure 5.6 - Normalized axial stresses applied to shear walls

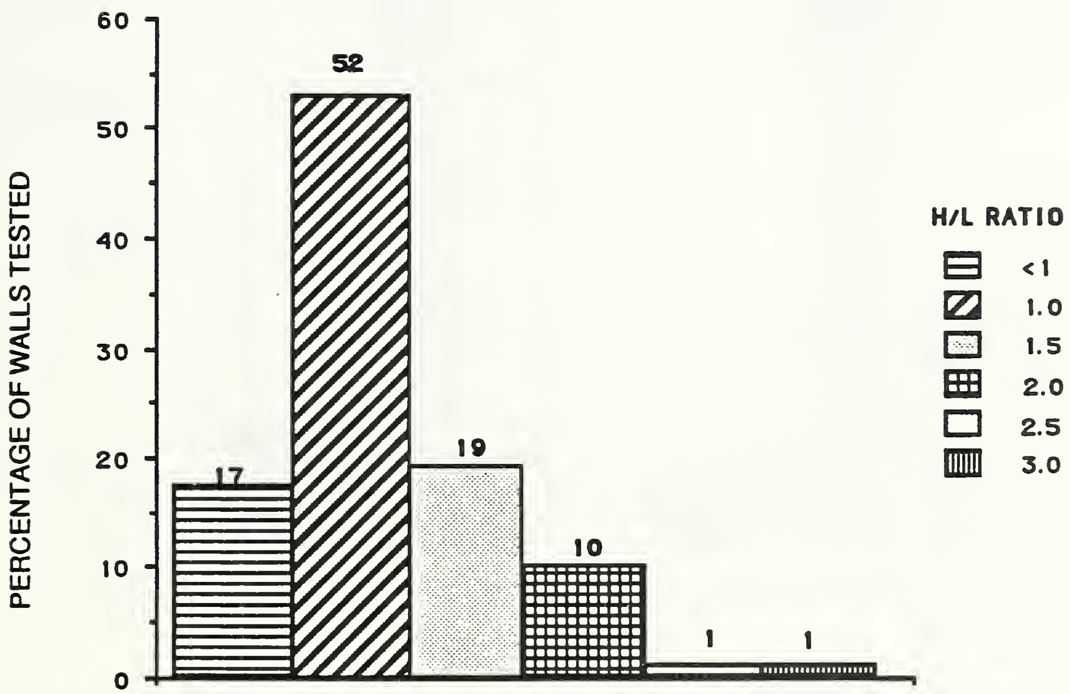


Figure 5.7 - Range of height/length ratios tested

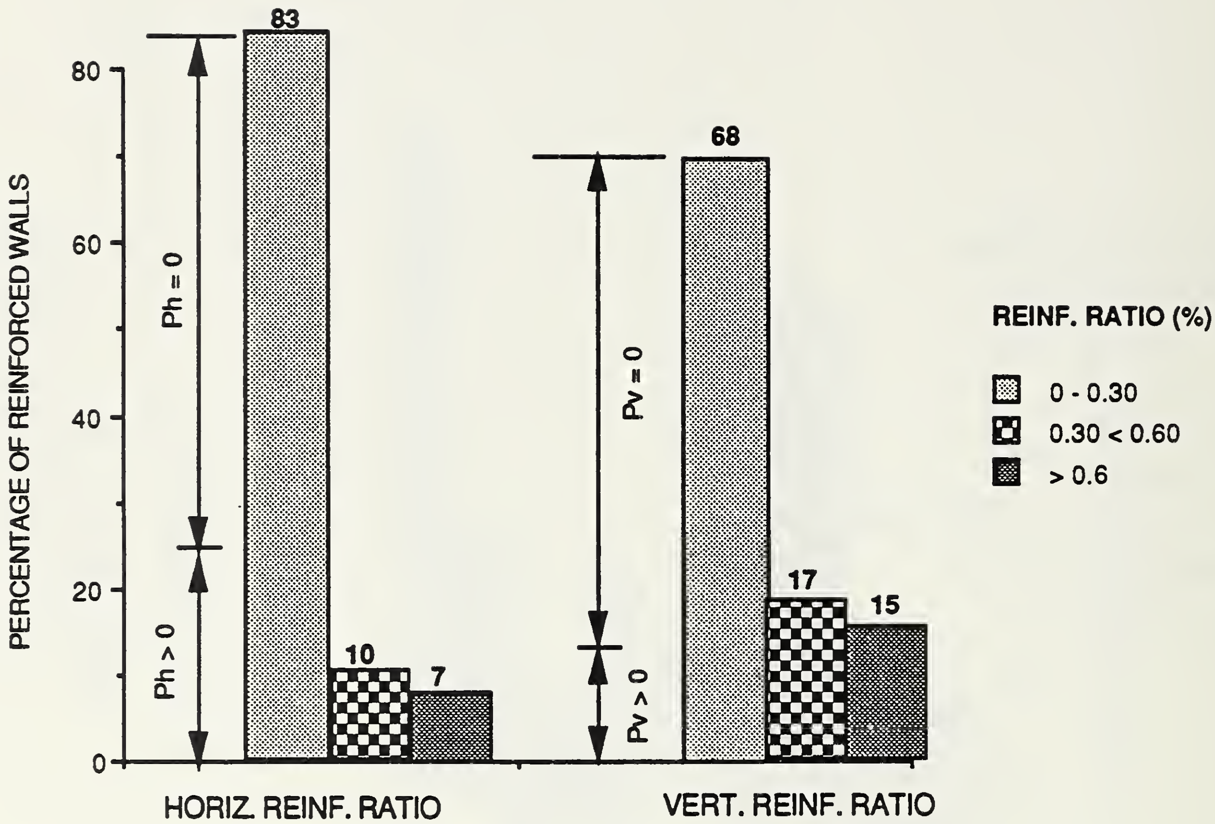


Figure 5.8 - Amount of vertical & horizontal reinforcement

6. ANALYSIS OF RESEARCH DATA

6.1 Introduction

In Chapter 5, experimental research information on masonry shear walls was condensed and presented in tabular form. The tables were supplemented by bar charts describing the frequency distribution of the various shear wall tests and by synopses of individual research programs to describe the objective and scope, test variables and conclusions.

This chapter presents an analysis of the information compiled in Chapter 5, using a format which highlights research needs in the area of shear-mode response of masonry walls under in-plane (membrane) loads. Section 6.2 outlines the procedure used to convert and analyze the experimental data in Chapter 5 into groupings according to type of construction and key design parameters. Analytical treatment of the principal design parameters is presented in sections 6.3 through 6.5.

6.2 Structural Properties and Key Design Parameters

6.2.1 Structural Properties

Research priorities for the study of seismic response of masonry shear walls can be identified by examining the following structural properties needed in their design.

1. Strength of uncracked walls
2. Strength of cracked walls
3. Stiffness
4. Ultimate deformations
5. Ductility
6. Energy absorption

The above properties are identified in conjunction with shear mode failures as distinct from flexural mode failures or compressive crushing failures at the wall corners. The first two modes may occur singly, simultaneously, or in succession, depending on the make-up of the wall and nature of loading. Compressive crushing, however, if it occurs first, impairs the wall from developing its shear and/or flexural capacity.

Shear-mode failures are in turn characterized by three types of rupture patterns, namely, (a) cracking of units across mortar joints, (b) debonding, which occurs by separation along mortar joints, whether or not this occurs as a result of cracking of the mortar or by breakage of bond between mortar and units, and (c) sliding, which is usually not an initial shear mode failure but follows after cracking or debonding.

The first two structural properties identified above are strength limit states. The strength of uncracked walls is considered to be the first limit state in shear mode response. It may simultaneously be the ultimate limit state, as is often the case for unreinforced or plain walls. The ultimate strength is taken as the maximum load capacity the wall develops whether or not it is capable of sustaining this level with increasing deformation.

Stiffness properties reflect the constitutive properties of masonry components. Element stiffnesses are used to convert the loads acting on a structure to the element level by analysis. Stiffness properties are needed in the analysis phase of the design process.

Ultimate deformation does not have a universally-accepted definition. The deformation at ultimate strength, the maximum deformation attained before or at complete collapse, or a factor applied to either of these, selected by convention, have been used to define this limit state. For the purpose of this study, the term is used to designate the deformation at ultimate strength.

Likewise, ductility and energy absorption are not uniquely defined terms. Ultimate and cracking deformation limit states are frequently used in their definition. Energy absorption is sometimes taken as the sum of the areas of hysteretic loops of a load-displacement cyclic response curve to ultimate load or deformation. It is also defined as the area under the envelope of this curve up to a certain defined level of displacement.

6.2.2 Key Design Parameters

In order to design a masonry shear wall, the prescriptive formulation of the design code should take into consideration all of the structural properties identified above. The design formulations defining the relationships of these characteristics are in turn influenced by the following design parameters of the element.

1. Definition of loads
2. Amount, type and distribution of reinforcement
3. Aspect ratio
4. Boundary conditions
5. Amount and distribution of grout
6. Constituent properties (units, grout, mortar, reinforcement, prisms, wallettes)

A design procedure is usually iterative. It uses the predefined parameters above in code-prescribed formulations of structural properties to check the adequacy of the element under investigation. If necessary, the cycle is repeated using a new set of values of parameters until code-prescribed requirements are met.

The empirical formulations are developed through experimental and analytical research in which the design parameters are treated as test variables to evaluate their effect on the structural characteristics. Therefore, by identifying design parameters that have not been well-researched, a research plan can be developed based on design priorities.

Loads of interest in shear wall analysis belong to three categories: gravity loads, lateral (or in-plane) loads, and out-of-plane loads. Loads of interest to this study are gravity loads and lateral loads resulting from earthquake excitations.

Reinforcement in shear walls consists of steel bars, usually placed horizontally in grouted bond beams, and vertically in grouted cores. Sometimes inclined bars are placed in grouted collars of multiple-wythe walls but this is not a common practice. Joint reinforcement placed in bed joints is frequently used with or without other reinforcement. Continuity and proper anchorage of reinforcing bars allows them to develop their strength and ductility potential and are considered to be key parameters in shear wall response. The distribution of reinforcement within the wall is another factor influencing shear wall response.

Aspect ratio alters the dominant mode of response. Generally walls with high aspect ratio respond in flexure and those with low aspect ratio respond in shear. As noted earlier, the two modes can occur simultaneously or in sequence before the wall develops its ultimate capacity. Shear mode response is the primary concern of this study.

Boundary conditions affect the external distribution of applied loads needed in the definition of critical states of internal stresses and deformations. Simulation of in-situ boundary conditions is a factor considered in the design of a test setup. The method most commonly used inhibits rotation of one or both horizontal surfaces while allowing one surface to translate under applied lateral load or displacement.

Grouting is typically used in reinforced walls. Full grouting is commonly specified for walls in regions of high seismicity, while partial grouting is more typically used for walls in regions of low-to-moderate seismic activity. Construction of grouted plain walls is not a common practice. Behavioral differences between fully and partially grouted walls are factors considered in their design.

Unit properties are specified by the design code according to standard test procedures. The prism strength is often used explicitly in prescriptive design formulations, while other properties are usually specified to meet minimum strength requirements. Unit tests are part of the routine in experimental research to provide a common basis of comparison of results.

In section 6.3, the bar charts of section 5.2 are used to identify the trends of past experimental research. Areas of major research thrust and, more importantly, areas which have not been sufficiently explored, are discussed.

The published test results may be utilized to examine the sensitivity of shear wall response to variations in the design parameters. The relative influence of design parameters on response is the type of information useful in the design of future experiments. However, only a few of the sources have comparable data. That is, only the data from sources which have used the same test setup and design parameters were compared.

In Section 6.4, comparable test data from two U.S. test programs are displayed on common plots to show the effect of axial load and reinforcement on cracking and ultimate shear response. In the other cases, the findings, as they relate to the effect of the various parameters on response, are discussed individually in Chapter 5.

In Section 6.5, results from three wall tests with shear modes of failure are compared with the ultimate shear strength formula presented in the design provisions of the 1988 UBC Code and to two other predictive formulae reported in the literature.

6.3 Interpretation of Research Information

6.3.1 Use of Grouting

Figure 5.2 indicates a relative scarcity of partially grouted masonry shear wall test data. Most of the shear wall tests conducted since the start of the Joint TCCMAR (JTCCMAR) program used fully-grouted, reinforced concrete masonry specimens. More than half of the reported tests of partially-grouted walls were conducted in Japan by Matsumura [39](35 walls), followed by the University of California at Berkeley [8,26,43](13 walls), and NIST [85](9 walls). The data from the first two sources reflect construction and reinforcement practices unique to high seismic risk areas. The last series of NIST tests reported in [85] can be explored to identify distinct behavorial trends that can be helpful in planning future experiments. This review indicates that a substantial amount of experimental work on partially-grouted masonry will have to be carried out to catch up with the body of knowledge on fully-grouted walls. This work needs to be given top priority because it reflects construction practices in regions of moderate to low seismicity.

6.3.2 Scale of Wall Specimens

Figure 5.3 illustrates the overwhelming preference of researchers for full-scale walls over small-scale models. The Yugoslavian small-scale model tests [72,73,74,75] were the only series utilizing exclusively scaled masonry units. The Yugoslavian specimen sizes were governed by the limited capacity (i.e. spatial and load application) of the testing facility. Generally, the capacity limitation and potential cost-savings are factors that dictate the use of small-scale models. Offsetting these potential benefits are the potential problems of workmanship and application of principles of similitude to a non-homogeneous material such as masonry. It is concluded that the dearth of model test results does not indicate a high-priority research need.

6.3.3 Type of Wall

Figure 5.4 indicates a need to supplement available research information on multi-wythe shear wall response under lateral loads. Most multi-wythe wall test data come from specimens of two-wythe brick construction. Other than the studies reported by Porter et al. [50,51], not much is known about the behavior of walls with dissimilar wythes, such as a brick wythe facing and concrete block backing. In section 6.4, similarities are noted in the responses of single- and double-wythe brick walls in the data from one source [69]. Reference [50] indicates separation of wythes in double-brick composite walls may have to be considered in predicting composite shear wall response.

In summary, the need for additional research on brick-brick and brick-block, double-wythe shear walls is indicated. However, factors such as industry statistics on the volume of multi-wythe vs. single-wythe construction in seismically-active areas will have to be considered before priorities can be assigned to composite masonry research.

6.3.4 Test Methods for Shear Walls

The statistical breakdown according to test methods used in experiments (see section 5.2 and figure 5.5) indicates the need to explore the potential of the diagonal test method in the study of reinforced masonry shear walls. The method is relatively simple to perform, inexpensive and within the load and size capacities of many commercial and research facilities. As a research tool, the method can be more effective if used in conjunction with lateral load tests of replicate specimens, all other parameters being identical. Figure 6.1a indicates the possibility of using diagonal tests to simulate a limiting boundary stress condition (i.e. zero axial load) for one class of in-situ piers in masonry structures. A comparison of limit states obtained from using the two methods will be effective in evaluating shear wall response with different boundary conditions and in defining the role of the diagonal test method in research and quality control testing.

6.3.5 Effect of Axial Load

Axial load has a strong influence on the in-plane shear performance of masonry shear walls (see section 6.4) mainly because it suppresses the tensile stress field in a material inherently weak in tension. Priestley and Woodward [55,82] have addressed the effect of this parameter on shear strength, ductility, and failure mode. Axial loading used in shear wall tests is intended to represent the range of gravity loads anticipated in an actual structure. Usually axial load is applied first and maintained constant by servo-controlled rams during testing. Axial stress (i.e. axial load divided by the gross area) however, becomes non-uniform to counter the overturning moment caused by increasing lateral loading, hence "nominal" axial stress is usually reported. Figure 5.6 points to the need to examine shear wall response under nominal axial stresses above 25% of the masonry compressive strength.

6.3.6 Test Wall Aspect Ratio

Aspect ratio as a parameter influences the response mode of a shear wall to in-plane lateral loading, depending on the boundary conditions at the top and bottom of the wall. Thus, the mode of failure of a cantilevered wall with a given aspect ratio should be compared with that of fixed-fixed wall having twice the height of the cantilevered wall. This would have the effect of shifting the distribution shown in figure 5.7 to the right.

There is a need to study walls with aspect ratios different from 1.0, mainly to develop a better understanding of the transition between flexural and shear modes of failure. Part of this research can be conducted analytically. However, selective testing will be needed to examine post-cracking behavior and to get a better understanding of the sequence of cracking modes up to ultimate load. For example, it is necessary to determine whether the shear cracking mode, if it occurs first, can have sufficient ductility to allow the specimen to develop its full flexural capacity. Even for the diagonal compression test method, the aspect ratio should not be limited to unity. It is relatively simple to fabricate corner loading fixtures that will facilitate the diagonal testing of specimens with higher and lower aspect ratios.

6.3.7 Reinforcement Ratios

Several investigators (e.g. [69]) have noted that increased horizontal reinforcement ratios, above certain levels, do not result in corresponding increases in shear strength. Thus, optimizing the amount of horizontal reinforcement becomes a design consideration.

Two important factors to be considered in addition to the amount of horizontal reinforcement are: the need for adequate anchorage to engage yield capacity of steel in the post-cracking region and the judicious placement of horizontal reinforcement at locations that allow the bars to cross the shear cracking plane and thereby become structurally engaged (refer to section 6.4).

The amount of vertical reinforcement has a bearing on whether or not a flexural yielding mode will develop before shear cracking occurs. Existing experimental results indicate that a sequence of flexural yielding followed by shear cracking failure results in more ductile response than in a case where shear cracking is followed by flexural yielding. This difference in ductility characteristic suggests that an upper limit be placed on the vertical reinforcement ratio when designing masonry shear walls.

6.3.8 Summary of Research Information

Based on state-of-the-art reports and papers published in 1976, it was observed that most pre-1976 masonry shear wall tests were conducted on unreinforced specimens and employed either the diagonal test method or the monotonic lateral loading test method. Beginning in the early 1980's and continuing to the present, emphasis has shifted to cyclic tests of fully-grouted walls containing relatively heavy reinforcement in one or both directions. This emphasis is explained by the concentration of research in areas of high seismicity. The transition zone between research on plain walls and that on heavily reinforced walls has not been adequately addressed. This inadequacy provides a strong basis for giving top priority, in future experiments, to studying the response of lightly-reinforced, partially-grouted masonry shear walls.

6.4 Effect of Design Parameters

Figure 6.2 shows the effect of axial load on cracking and ultimate shear strength based on results from Shing, et al. [64, 65, 66] and Sveinsson, et al. [69]. The following features are common: fully-grouted walls, nearly the same aspect ratio, uniformly-spaced reinforcing bars located in vertical and horizontal orientations, rotational fixity at the top and bottom edges, and application, along the top edge, of horizontal in-plane forces simulating earthquake loading conditions. In both studies, slow rates of cyclic loading were used.

In figure 6.2, solid and broken lines connect data points representing first cracking and ultimate shear stresses respectively. Each numbered pair of curves represents specimens which are identical in every respect except axial load. For instance, curves 1 identify specimens built with concrete block (BL), reinforced with hooked horizontal rebars (HK), and with vertical rebars located in the end cores (RE). The horizontal and vertical reinforcement percentages are 0.20 and 0.40, respectively. Additional explanations are given in the figure. The test results show the following major trends.

Within the range of axial stresses considered (0 to 18% of compressive strength), both cracking strength and ultimate shear strength increased with increasing axial load. The rate of increase in shear strength with increasing axial load, as well as shear strength under a given axial load, appear to be influenced by other parameters. The most significant gain in shear strength occurs in group 1 specimens relative to group 2. The specimens in both groups were built with concrete masonry units and belong to the same test series. Specimens in group 1 used 0.20% horizontal reinforcement with hooked ends, while those in group 2 used 0.29% horizontal reinforcement with 90° bent ends. The specimens in group 1 used three times as much vertical reinforcement as those in group 2, but a corresponding increase in shear strength should not occur because flexural reinforcement (vertical end bars) is not effective in resisting shear. In addition, group 2 with heavier horizontal reinforcement developed lower shear strength than group 1. This indicates that the type of anchorage used for the horizontal reinforcement (hooked vs. bent bars, in this case) had a significant positive effect on shear response under axial load; an observation which agrees with the findings of Sveinnson et al. [69] (refer to section 5.3.1).

Consider now the results of group 1 tests relative to concrete masonry tests (group 6) from Shing et al. [64]. Both groups use hooked horizontal bars. However, the reinforcement in group 6 is uniformly distributed in both directions. There are also differences in the range of axial loads considered and reinforcement percentages. Group 6 walls exhibit relatively flat response to axial load and lower overall shear capacity compared to group 1 walls. Shing et al. [64] assume that the horizontal bars near the top and bottom ends have no effect on shear resistance because they do not cross the diagonal rupture plane. On the other hand, the horizontal bars in group 1 walls were remote from the ends. These physical differences could have contributed to the differences in shear response.

In the case of double-wythe grouted brick specimens, the response of group 3 walls can be compared with those of group 5 because they are identical in every respect except in horizontal reinforcement ratios (0.06 and 0.27%, respectively). It is significant that more than a four-fold increase in the amount of horizontal reinforcement did not have a greater positive effect on shear response under axial load than the results indicate. It is possible that a 90° bend does not provide sufficient anchorage to develop the capacity of the bars.

A comparison between the results of single-wythe brick specimens (groups 4 and 7) shows differences indicative of physical differences between the two groups; type of anchorage, distribution of bars and differences in both reinforcement ratios. On the other hand, the response of group 4 walls, especially ultimate shear strength, is remarkably close to that of the two-wythe brick specimens of group 5. The two groups share common physical properties except for horizontal reinforcement ratios. Due to the absence of test replication, the only inference that can be made from these results is that under certain conditions the response of multi-wythe walls may be predictable on the basis of single-wythe tests. To verify this trend, additional testing will be required using replicate specimens of single- and double-wythe walls having otherwise identical physical properties.

The relationship between cracking strength (τ_c) and ultimate shear strength (τ_u) exhibits another common trend. In nearly every case, the ratio τ_u/τ_c decreases with increasing axial load. Although not shown, the results of tests reported by Sveinnson [69] indicate a decrease in ultimate deformation with increasing axial load. This dual response tendency, observed by other researchers as well [e.g. 50], has implications on ductility in situations where shear is the dominant mode of response.

The brick specimens exhibit significantly lower shear strength ratios τ_u/τ_c than the concrete masonry counterparts as evidenced by comparison of group 6 with group 7, or groups 3 and 5 with groups 1 and 2, respectively. Factors contributing to these behavioral differences cannot be identified because of insufficient data.

Figure 6.3 shows the effect of horizontal reinforcement on shear strengths using test results from the same two sources as in figure 6.2. However, most of the data in figure 6.3 are obtained from different test specimens than those used to examine axial load effects.

The most significant aspect of the results shown in figure 6.3 is a lack of trend in shear response with increasing horizontal reinforcement. For instance, an increase in horizontal reinforcement from 0.20 to 0.49 percent causes a maximum gain of 11 percent (group 2) and a maximum decrease of 9 percent (group 5) in cracking shear strength. The corresponding figures for the ultimate shear strength are, 12 percent increase (group 4) and 1 percent decrease (group 5), respectively. The trend of results from the other tests are similar. Low sensitivity of shear strength to changes in the horizontal reinforcement ratio has been also reported by others [29, 72, 73].

In group 1, two specimens were built using joint reinforcement with and without horizontal bars. It is noteworthy that the specimen in which only joint reinforcement was used developed ultimate and cracking shear strengths comparable to those from specimen reinforced with only horizontal bars. A single test does not establish a trend but draws attention to the effect of joint reinforcement as a possible research area which, according to this study, has not been adequately explored in the past.

The results of double-wythe brick walls (groups 7 and 8) shows the substantial effect of axial load on shear response of reinforced walls. An increase of axial load from 2 to 13 percent of prism compressive strength more than doubles the shear strength of these otherwise identical specimens (refer to figure 6.3).

In summary, the following comments can be made on the behavior of fully-grouted reinforced masonry specimens examined.

1. Axial load in the range of 0 to 18 percent of prism compressive strength has a significant positive effect on cracking strength as well as ultimate shear strength under lateral loading.

2. The increase in lateral load response due to the presence of axial load is further enhanced by using hooked rather than bent horizontal bar anchorage and by distributing horizontal reinforcement such that the extreme bars are inboard from the top and bottom edges of the wall.

3. Strength at first cracking and ultimate shear strength of walls under lateral loading are not significantly altered by the amount of horizontal reinforcement in the range of 0.05 to 0.50 percent.

4. The differences between shear strengths at cracking and ultimate limit states, as quantified by the ratio τ_u/τ_c, tend to decrease with increasing axial load. Although not demonstrated graphically, differences between cracking and ultimate deformations decrease in a similar manner. This declining trend is indicative of a decrease in post-cracking ductile behavior and energy dissipation capacity under hysteretic lateral loading.

5. The difference between first cracking strength and ultimate shear strength for brick walls is substantially narrower than that for concrete masonry walls.

6.5 Predictive Formulae Compared with Code Formula for Shear Strength

6.5.1 Uniform Building Code Formula

The 1988 UBC contains an empirical formula, shown below, for calculating the shear strength of reinforced masonry walls. The UBC formula is similar to procedures presented in ACI 318 for computing the shear strength of reinforced concrete walls. The nominal shear strength, where the limit state is ultimate strength, is obtained by adding two terms. One term is for the strength provided by the masonry. The second term accounts for the strength provided by the shear reinforcement. The UBC presents several design assumptions underlying the formula for nominal shear strength: 1) the nominal shear strength of a singly reinforced masonry wall cross section is based on applicable conditions of equilibrium and strain compatibility; 2) strains in the reinforcement and masonry are assumed to be directly proportional to the distance from the neutral axis; 3) maximum usable strain at the extreme masonry compression fiber is assumed to be equal to 0.003; 4) stress in the reinforcement below the specified yield strength, f_y, is taken as the elastic modulus, E_s, times the steel strain; and 5) for strains greater than the yield strain the stress in the reinforcement is equal to f_y.

UBC-88 Formula for Predicting Shear Strength

$$V_n = V_m + V_s$$
$$V_m = C_d A_{mv} \sqrt{f'_m}$$
$$V_s = A_{mv} \rho_n f_y$$

Where:

V_n = Nominal shear strength of the wall, (lb)
V_m = Shear strength provided by the masonry
V_s = Shear strength provided by the shear reinforcement
C_d = Masonry shear strength coefficient (ranges from 1.2 to 2.4 depending on ratio of moment to shear)
A_{mv} = Net area of masonry section bounded by the wall thickness and length of section in the direction of shear force considered, (in^2)
f'_m = Specified compressive strength of masonry at 28 days
ρ_n = Ratio of distributed shear reinforcement on a plane perpendicular to plane of A_{mv}
f_y = Specified yield strength of reinforcement, (psi)

6.5.2 Other Predictive Formula

Shing [64] concluded that the UBC formula for shear strength prediction is overly simplistic in that it does not adequately account for the complicated mechanisms at work after diagonal cracking occurs in a masonry shear wall. According to Shing, post-cracking strength of masonry, V_m in the UBC formula, is contributed to by three mechanisms: 1) the compression toe shear strength, 2) aggregate interlock forces which are dependent on applied axial stress and the amount of flexural steel, and 3) dowel action of the flexural vertical steel. Moreover, Shing points out that the UBC horizontal reinforcement shear strength component, V_s, can overestimate shear strength. When diagonal cracks propagate at approximately 45-degree angles, the central horizontal reinforcing bars are activated but the top and bottom bars have insufficient embedment lengths to develop tensile resistance. Hence, he has proposed a modification to the UBC term for shear strength provided by the shear reinforcement.

Shing used the experimental data from his series of 22 shear wall tests to determine the effect of axial compressive stress, σ_c, and the quantity $\rho_v f_y$ on the ultimate shear stress. Using a least-squares fit to the data, he found the rate of increase in the normalized masonry strength with respect to σ_c to be 0.0025. The normalized strength increased at the rates of $0.0016\rho_v f_y$ and $0.0014\rho_v f_y$ depending on the magnitude of the axial stress. He has used an average slope of 0.0018 in his proposed formula for V_m.

Shing's Formula for Predicting Shear Strength

$$V_n = V_m + V_s$$
$$V_m = [0.0018(\rho_v f_y + \sigma_c) + 2] A \sqrt{f'_m}$$
$$V_s = [(L - 2d')/s - 1] A_h f_y$$

Where:

V_n = Nominal shear strength of the wall, (lb)

V_m = Shear strength of masonry

V_s = Shear resistance of horizontal reinforcement

ρ_v = Ratio of the vertical steel

f_y = Yield strength of vertical steel, (psi)

σ_c = Axial compressive stress, (psi)

A = Area of horizontal cross section of masonry wall, (in^2)

f'_m = Specified compressive strength of masonry at 28 days, (psi)

L = Horizontal length of a wall, (in)

d' = Distance from the extreme vertical steel to the nearer edge of a wall,(in)

s = Vertical spacing of the horizontal reinforcement, (in)

A_h = Area of a horizontal reinforcing bar

Matsumura [39] conducted an experimental parametric study involving 57 concrete and 23 brick masonry walls. His objective was to develop a formula for predicting the shear strength of reinforced masonry walls as a function of such parameters as: shear reinforcement ratios, shear-span ratios, axial stresses, strengths of materials, and partial or full grouting. After each segment of the study, Matsumura proposed a relation to account for the effect of the parameter studied on the ultimate shear strength of the walls. The empirical formula presented below represents the synthesis of the individual components. The formula yields ultimate shear strength values in units of kilonewtons. Therefore, when considering shear walls designed in inch-pound units, conversion to metric units is necessary before using the formula.

Matsumura's Formula for Predicting Shear Strength

$$V_u = \{k_u\, k_p[(0.76 + (h/d + 0.7)) + 0.012]\,\sqrt{f'_{m(g)}} + [0.18\, \Gamma\, \delta \sqrt{\rho_{H\,H}\sigma_y\, f'_{m(g)}} + 0.2\, \sigma_{o(g)}]\}\, 10^3\, t\, j$$

<u>Symbols</u>

V_u = Ultimate shear strength (in kN)

k_u = 1.0 for fully-grouted masonry

= 0.8 for partially-grouted brick masonry

= 0.64 for partially-grouted concrete masonry

k_p = $1.17\, (\rho_t)^{0.3}$

ρ_t = Flexural reinforcement ratio = $a_t/(t\, d)$

a_t = Area of tensile steel at one end of the wall, (mm^2)

t = Wall thickness (mm)

d = Distance from compression face to centroid of tensile steel, (m)

h = Height of wall (m)

$f'_{m(g)}$ = Masonry prism strength on gross area (MPa)

Γ = 1.0 for brick masonry and fully-grouted concrete masonry

= 0.6 for partially-grouted concrete masonry

δ = 1.0 for walls with inflection point at mid-height

= 0.6 for cantilevered walls

ρ_H = Horiz. reinforcement ratio, computed by $A_H \div (t \times s)$, where A_H is area of a horizontal bar, t is wall thickness, and s is the spacing between the horizontal bars.

$_H\sigma_y$ = Yield strength of horizontal shear reinforcement (MPa)

$\sigma_{o(g)}$ = Axial stress on gross area of wall (MPa)

j = (7/8)d (m)

Table No. 24-K in the 1988 Edition of the UBC specifies two limiting values for nominal shear strength of masonry walls. The maximum nominal shear strengths are dependent upon the effective aspect ratio (i.e. M/Vd). The footnote to Table No. 24-K indicates that straight-line interpolation is permitted for M/Vd values between the two limits.

UBC-88 Maximum Nominal Shear Strength

From Table No. 24-K

$V_n = 6.0\ A_e \sqrt{f'_m}$ for $M/Vd \leq 0.25$

$V_n = 4.0\ A_e \sqrt{f'_m}$ for $M/Vd \geq 1.00$, where

M = Maximum bending moment that occurs simultaneously with the shear load V at the section under consideration.

d = Length of the wall

A_e = Effective cross-sectional area of wall

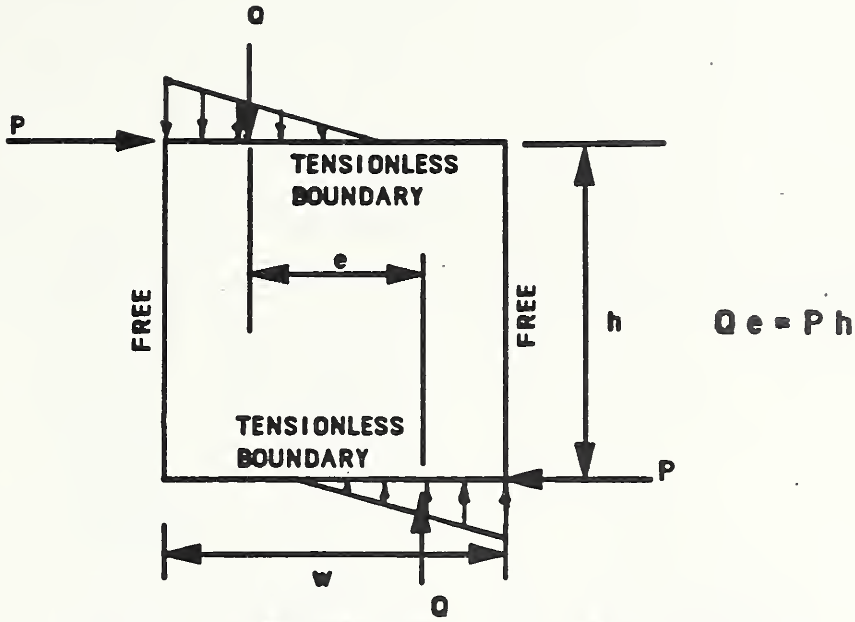
6.5.3 Comparison of Formulae

An abbreviated comparative study was conducted in which the three aforementioned predictive formulae were applied to three three sets of experimental reinforced masonry walls. One reinforced concrete masonry wall each was selected from the specimens tested by Shing [64], Matsumura [39] and Sveinnson et al. [69]. Table 6.1 summarizes the preliminary results. The extreme left column of the table presents a description of each wall, quantifying the parameters used in the respective formulae. The ultimate strength values obtained from the experiments are presented in the second right hand column. The extreme right column presents the maximun nominal shear strength values obtained in accordance with Table No. 24-K of UBC-88.

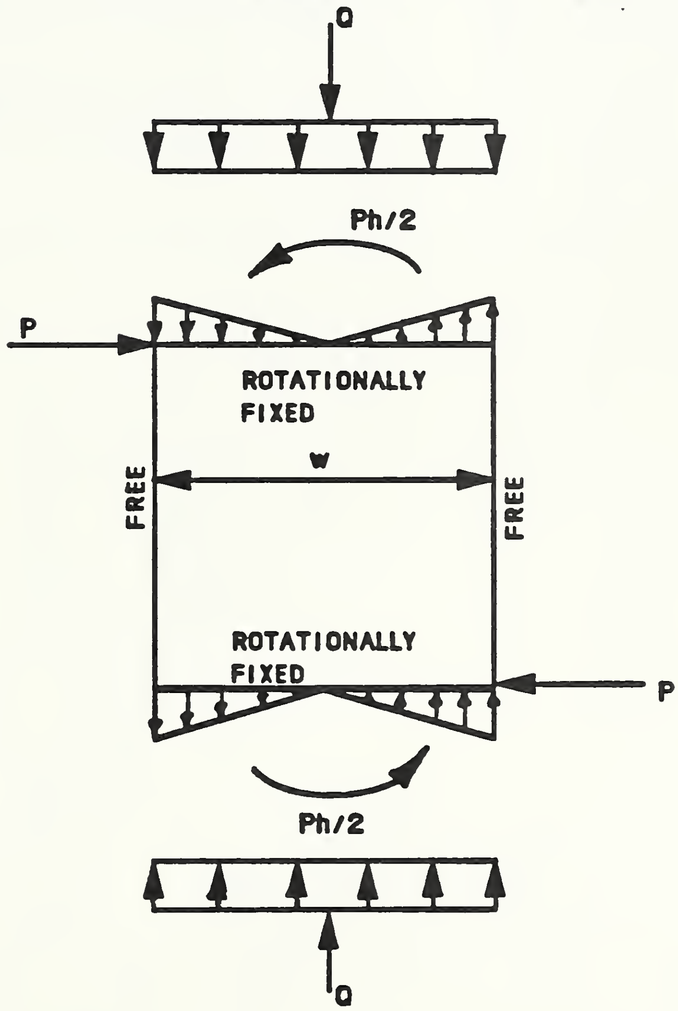
For the walls comprising the comparison, the UBC formula yields the most conservative estimates of the ultimate shear strength. The strengths obtained from Shing's formula lie relatively close to the experimental values without exceeding the test results. There is also relatively close agreement between the Matsumura formula results and the experimental values. The comparative study will be extended to all applicable walls in the three groups and the results reported in a separate document.

Table 6.1 - COMPARISON OF RESULTS FROM PREDICTIVE FORMULAE FOR SHEAR STRENGTH

	Ultimate Shear Strengths				
Wall Description	According to UBC-88 Formula τ_u (psi)	According to Shing's Formula τ_u (psi)	According to Matsumura's Formula τ_u (psi)	Exper Ultimate Stress τ_u (psi)	UBC Max. Nominal Stress τ_N (psi)
Shing's Wall #3; Fully Grouted Conc. Block; h = 72" L = 72", t = 7" nom.; Vert. Reinf.- 5 - #3; f'_m = 3000 psi; q = 270 psi; σ_y = 56 ksi.	175	234	267	247	292
Matsumura's Wall KW3-1; Fully Grouted Conc. Block; h = 71," L = 47," t = 6"nom; Vert. Reinf. - 4 - #7, 2 - #3, Horiz. Reinf. 4 - #3 f'_m = 3162 psi; q = 71 psi; σ_y = 55.8 ksi	150	207	241	178(-) 250(+)	261
Sveinnson Wall HCBL-11-15; Fully Grouted Conc. Block; h = 56," L = 48," t = 8"nom; Vert. Reinf. - 2-#5; Horiz. Reinf.- 4 - #5; f'_m = 3359 psi; q = 437 psi, $_h\sigma_y$ = 59 ksi; $_v\sigma_y$ = 67.5 ksi	319	311	322	345	294



(a) FIRST LIMITING CONDITION - Lateral load test with Q = 0



(b) SECOND LIMITING CONDITION - Lateral load test

Figure 6.1 - Two limiting boundary stress conditions

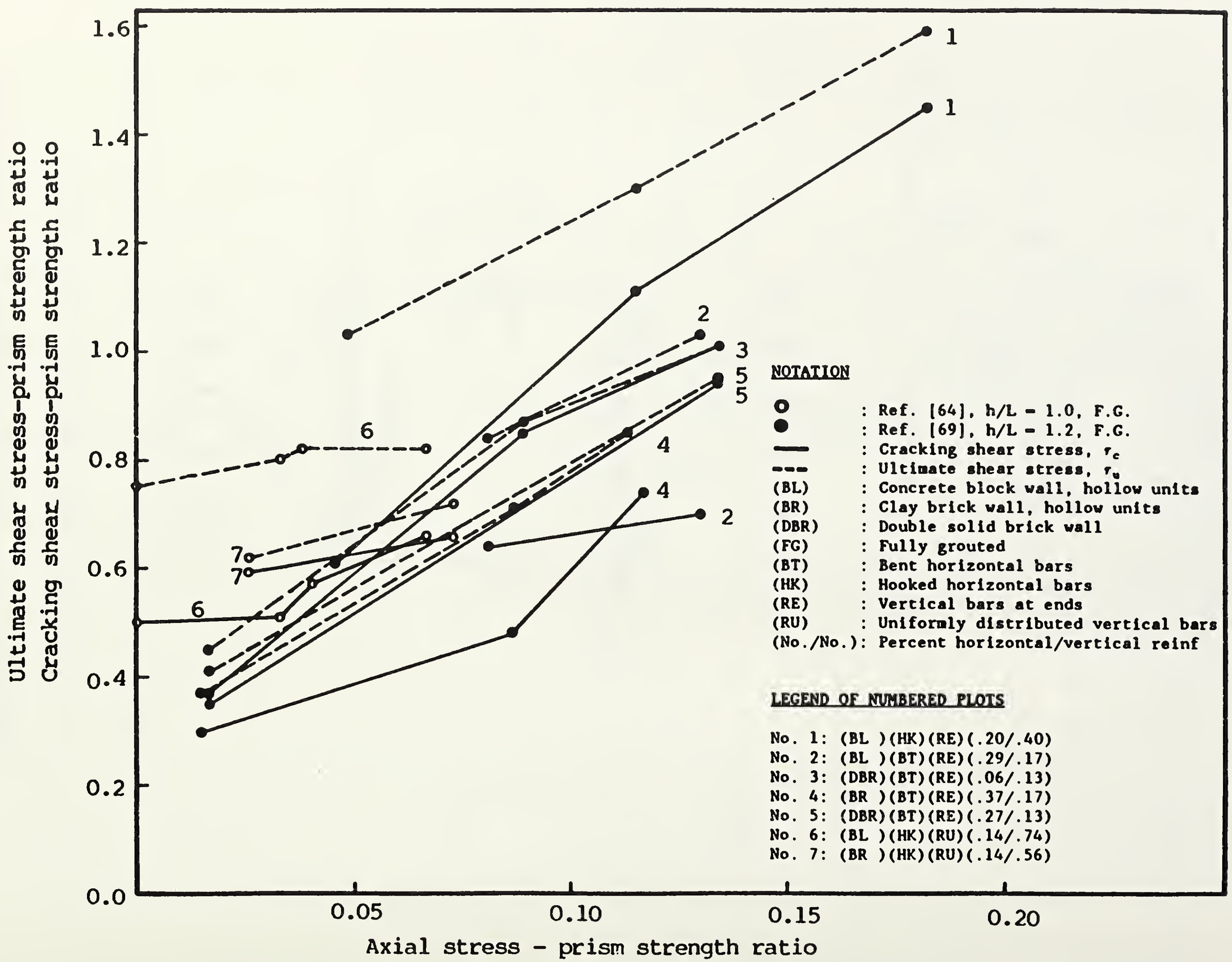


Figure 6.2 - Effect of axial stress on cracking and ultimate shear stresses

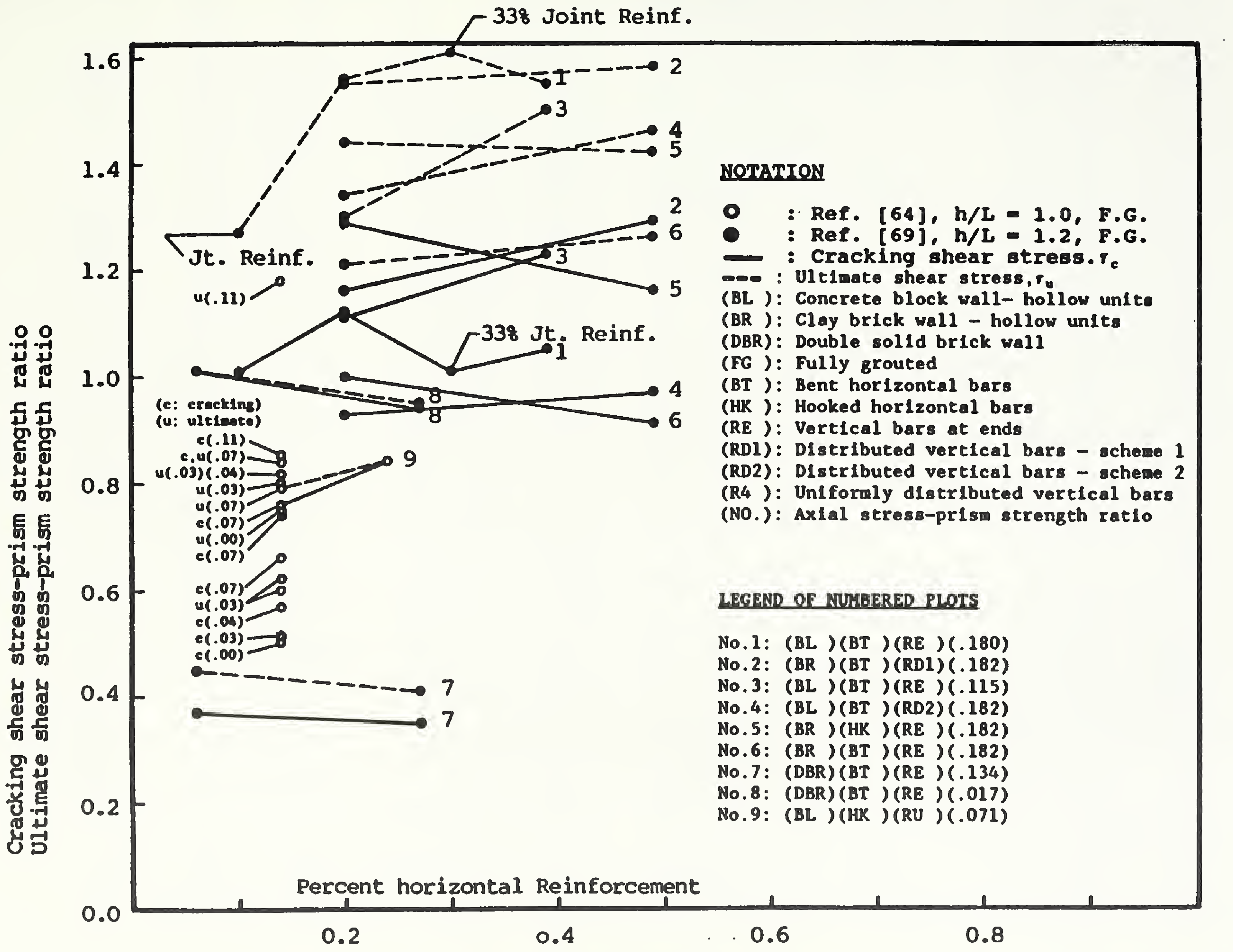


Figure 6.3 - Effect of horizontal reinforcement on cracking and ultimate shear stresses

7. SUMMARY AND CONCLUSIONS

7.1 Summary

This report documents a review of technical literature generated on masonry shear walls in the past 15 years. Special emphasis is placed on shear mode response and seismic effects. Needed improvements in shear wall design criteria are highlighted by comparison with experimental data, and the absence of design provisions for certain types of masonry construction is highlighted. Research needs and priorities are established by examining statistical groupings of experimental results according to tests variables. The effects of axial load and horizontal reinforcement on shear strength are examined using the test data from two sources. The major findings and conclusions drawn from this study are discussed in this chapter.

7.1.1 Design Requirements

Although progress has been made in the development of improved standard/code provisions for masonry, there is still no standard methodology for the rational design of masonry. The approach varies depending on the applicable code and type of masonry construction (solid or hollow unit, multi-wythe or single-wythe, reinforced or plain). The Working Stress Design approach is the most prevalent approach found in the six U.S. codes and standards reviewed in this study. One model code (1988 Edition of the Uniform Building Code) and one set of code provisions (the 1988 NEHRP Provisions) incorporate Strength Design provisions as an alternative to Working Stress Design. Some documents contain empirical design provisions, subject to building height limitations.

The Strength Design provisions for masonry shear walls in the 1988 UBC (Sec. 2412, Reinforced Masonry Shear Walls) are only applicable to fully-grouted walls constructed with hollow units and reinforced in accordance with requirements specified for Seismic Zones 3 & 4. In addition, f'_m is specified to be not less than 1500 psi nor greater than 4000 psi. Masonry shear walls in all other seismic zones must be designed or sized in accordance with Working Stress or empirical provisions unless they meet the minimum reinforcement requirements specified for Zones 3 and 4. If the latter requirements are satisfied, the walls may be designed using Strength Design criteria.

The 1988 UBC contains an empirical formula for predicting the ultimate shear strength of reinforced masonry shear walls. The two-term formula is patterned after the ACI 318-83 formula for reinforced concrete shear walls.

There is some experimental evidence to indicate that the UBC formula does not adequately account for the post-cracking mechanisms activated in reinforced masonry walls under in-plane shear and axial load.

7.1.2 Research

During the past fifteen years about 80 percent of the more than 700 in-plane shear tests have been conducted on single-wythe masonry walls. Less than 100 multiple-wythe walls have been tested.

Only about 60 partially-grouted masonry wall tests have been reported during the past fifteen years. Of this group of tests, approximately 50 were conducted in Japan as a part of the U.S.- Japan Joint TCCMAR program.

About one-third of the tests have employed the diagonal compression method of testing. However, the correlation between results obtained from this method and those obtained from in-plane shear tests with one or both edges fixed has not been established.

Since the formation of TCCMAR there has been heightened interest among researchers in developing predictive formulae for the shear strength of reinforced masonry walls. The same is true for the development of analytical methods applicable to reinforced masonry walls and confirming the methods by experimental results.

Based on a limited comparative study, formulae proposed by Shing and Matsumura for predicting the shear strength of reinforced masonry walls appear to more accurately estimate the ultimate strength than does the comparable formula in UBC-88. Shing's formula is only confirmed for a limited number of single-wythe, fully-grouted, uniformly reinforced masonry walls with aspect ratio equal to 1.0. The formula should be applied to walls with other aspect ratios and the predicted values compared with known experimental results.

7.2 Conclusions

In conclusion, the following research needs can be identified on the basis of this study:

1. In general, a strong case can be made for experimental studies of axial load effects on the response of masonry shear walls under lateral loads. In particular, emphasis should be given to axial loads in excess of 20 percent of prism strength, and reinforced walls which are partially grouted.

2. The effect of horizontal reinforcement on shear wall response to lateral loads needs to be studied more thoroughly. In the planning of a test program, particular emphasis should be given to the design of proper anchorages and distribution of the horizontal reinforcement so that its yield capacity can be fully utilized. More attention should be given to testing partially-grouted walls with horizontal reinforcement in the range of 0.05 to 0.25 percent.

3. The credibility of the observed trends in the experimental results examined is compromised by the absence of replicate testing to evaluate scatter. Past research has not generally given much emphasis to

replicate testing of full-scale specimens. Test replication should receive more attention in the planning of future tests.

4. Research is needed to examine the effectiveness of joint reinforcement, acting singly and in addition to horizontal bars, on shear wall response. Experimental data on walls containing only joint reinforcement is scarce.

5. There appears to be insufficient research data from in-plane shear tests of multiple-wythe walls to form a technical basis for rational design criteria for this class of masonry wall. However, the practical application of additional data collection must first be established. For example, if new composite wall construction is determined to be prevalent in seismic Zones 2, 3 or 4, experimental research would be needed to evaluate the shear response of composite walls in relation to the shear responses of their constituent wythes. The data examined indicate the possibility of predicting the response of a composite wall to in-plane lateral loads from the individual responses of its constituent wythes. If this relationship can be validated through exploratory tests, future testing of multi-wythe walls can be planned more selectively to fill gaps in the database.

8. RECOMMENDATIONS

Recommendations are presented for future analytical and experimental research on masonry shear walls toward advancing the state-of-the-art in masonry design and construction. The intended constituency for the research results includes: designers, building code promulgators, and masonry contractors. The proposed research should be conducted in a coordinated manner, with cooperation among industry, TCCMAR, univeristy researchers and NIST. The basic undelying premise is that NIST will maintain a long-term program in masonry research, with emphasis on the seismic resistance of masonry building components and that NIST can play a major role in coordination and technology transfer.

While acknowledging the milestones, realized and expected, from the TCCMAR program and the projected completion of a set of limit state design provisions for masonry construction, there will remain a number of significant research needs. As a set of guiding principles, the coordinated masonry research program should address the following needs: 1) development of a standardized test method for determining the in-plane shear strength of reinforced masonry walls, 2) experimental and analytical data with which to correlate results obtained from the Diagonal Test Method (e.g. ASTM E 519) with those obtained from the Lateral Load Test Method, 3) extension of the database of in-plane shear test results through replication of previously-run test setups, 4) production of additional technical data to promote the extension of limit state design provisions to all types of masonry wall construction, including partially-grouted, multi-wythe and solid-unit applications, 5) results from a systematic evaluation of the effects of the distribution and means of anchoring horizontal reinforcement, 6) gaining greater understanding of the effect of axial stress on the in-plane shear strength and ductility of masonry walls, 7) determining the in-plane shear performance of multi-wythe masonry walls for potential application in high seismic hazards areas, 8) confirmation of existing or newly-derived formulae for predicting the ultimate shear strength of fully-grouted and partially-grouted masonry walls over a broad range of key parameters, and 9) calibration of existing numerical analyses methods with experimental data.

Sections 8.1 and 8.2 outline, in priority order, recommended tasks to be performed as part of the NIST Masonry Research Program.

8.1 Analytical Tasks

Several investigators have proposed empirical formulae for predicting the shear strength of fully-grouted, reinforced masonry walls. In addition, the 1988 Edition of UBC contains an empirical predictive formula, as discussed in section 6.5. However, the existing formulae have not been tested against a wide range of available experimental data. The objectives of such a research effort would be: (1) to identify the formula(e) which can consistently give sufficiently accurate estimates of ultimate strength over a wide range of key parameters; (2) to define the range of applicability of the formulae for fully-grouted, reinforced walls; (3) to evaluate the applicability of formulae to partially-grouted and ungrouted (plain) masonry shear walls; and (4) to derive a predictive formula based upon the principles of structural mechanics.

A second area of analytical study that has been initiated within the past five years, but is in need of further development, is analytical modeling of masonry shear walls. Microcomputer-based models of fully-grouted and partially-grouted reinforced masonry walls would be useful tools for designers, code and standard developers and researchers. During the review of literature on experimental studies reported herein, several analytical models, including one Finite Element Model written for use on personal computers, were identified. One model is currently being calibrated against a selected set of experimental data. The objective of the NIST research should be the determination of the applicability of one or more analytical models when compared with a more extensive set of fully-grouted experimental masonry walls. In addition, the models can be excercised to determine their applicability to ungrouted and partially-grouted masonry walls.

Following is a list of analytical tasks recommended for the NIST Masonry Research Program. These tasks should be initiated prior to the conduct of laboratory-based studies to help provide a basis for such studies.

1. Initially, data analysis should be conducted to evaluate previous experimental results generated by NBS/NIST, U.S.TCCMAR, Japan's TCCMAR and other relevant U.S researchers. Test walls having comparable key parameters and failing in a shear mode can be selected from different studies and their results summarized for subsequent comparative analysis.

2. Further comparative study of existing predictive formulae is required to determine which is the best available formula for fully-grouted reinforced masonry walls. Selected formulae can be applied to the range of test results identified in Task (1) above.

3. The applicability of existing predictive formulae to partially-grouted and plain masonry walls should be studied.

4. Recently developed Finite Element Models should be evaluated to determine their limitations when applied to fully-grouted and partially-grouted, reinforced masonry walls. The study would involve performing numerical analyses on walls selected from the set summarized in Task (1) and comparing the analytical results with existing test results.

8.2 Experimental Studies

There are several identified research needs which can provide NIST with the opportunity to capitalize on its masonry shear wall test experience and the capabilities of the Tri-directional Test Facility. Recommended research topics include: 1) development of a standard test method for evaluating the in-plane shear response of fully-grouted and partially-grouted reinforced masonry walls, 2) extended study of the effect of varying the amount and distribution of horizontal reinforcement, 3) effect on ultimate strength, ductility and energy absorption of varying the anchorage detail for horizontal bars, and 4) in-plane shear performance of partially-grouted masonry walls. Following are recommended tasks directed toward the abovementioned research topics.

1) Conduct ASTM E 519 tests on masonry wall specimens, the results from which would be compared with results from in-plane shear tests previously conducted in the TTF.

2) Conduct parallel tests, using the Triaxial Test Facility for in-plane shear, and applying the ASTM E 519 test method on newly-constructed, single-wythe, partially-grouted reinforced masonry walls. The walls would consist of constituent materials similar to those used in the U.S. TCCMAR research program.

3) Conduct tests involving cyclic, in-plane lateral load on single-wythe walls to determine the effect on ultimate strength, ductility, and energy absorption of employing different anchorage details at the ends of horizontal reinforcing bars.

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11. ABSTRACT (A 200-WORD OR LESS FACTUAL SUMMARY OF MOST SIGNIFICANT INFORMATION. IF DOCUMENT INCLUDES A SIGNIFICANT BIBLIOGRAPHY OR LITERATURE SURVEY, MENTION IT HERE.)

A review of the technical literature on masonry shear wall tests was conducted to determine the range and depth of research conducted and to identify areas in need of additional research. The review covers documents published from 1976 to 1989 and includes approximately seven hundred masonry wall tests. Both U.S. and foreign research was included in the review. U.S. code and standard requirements for the design of masonry shear walls are discussed and some of the provisions are highlighted in tabular and graphic form.

Technical information regarding experimental studies is tabulated for easy reference and a selected number of test programs are examined in greater detail to present the objective and scope, test variables and major findings.

Experimental data from comparable research studies are combined and analyzed to determine the influence of key design parameters on the response of shear walls to in-plane lateral loading. Also included is a comparison of two experimentally-derived ultimate shear strength formulae with that included in the 1988 edition of the Uniform Building Code. The findings of the review are summarized and specific research needs are identified.

12. KEY WORDS (6 TO 12 ENTRIES; ALPHABETICAL ORDER; CAPITALIZE ONLY PROPER NAMES; AND SEPARATE KEY WORDS BY SEMICOLONS)

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