NSF/RA-780132

PB 287478

METHODOLOGY FOR MITIGATION OF SEISMIC HAZARDS IN EXISTING UNREINFORCED MASONRY BUILDINGS

PHASE I

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CONTRACT WITH NATIONAL SCIENCE FOUNDATION
APPLIED SCIENCE AND RESEARCH APPLICATIONS
MARCH 31, 1978

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NOTICE

This research was conducted with the support of the National Science Foundation. However, any opinions, findings, conclusions or recommendations expressed in this report are those of the authors and do not necessarily reflect the views of NSF.

REPORT DOCUMENTATION PAGE	1. REPORT NO. NSF/RA-780132		PB 281478

Title and Subtitle
Methodology for Mitigation of Seismic Hazards in Existing Unreinforced Masonry Buildings, Phase 1

5. Report Date
March 31, 1978

6.

7. Author(s)
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8. Performing Organization Rept. No.

9. Performing Organization Name and Address
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10. Project/Task/Work Unit No.

11. Contract(C) or Grant(G) No.
(C) 7719523
(G)

12. Sponsoring Organization Name and Address
Applied Science and Research applications (ASRA)
National Science Foundation
1800 G Street, N.W.
Washington, DC 20550

13. Type of Report & Period Covered

14.

15. Supplementary Notes

Abstract (Limit: 200 words)

Unreinforced masonry buildings are studied in order to determine appropriate methods to deal with hazard mitigation and to study methods of retrofit so that design methods can be established. These design methods are to be devised with consideration of the particular structural conditions of unreinforced masonry construction, their earthquake response, the seismicity of the particular location, and the economics of retrofit. Phase 1 has studied the state-of-the-art of hazard mitigation and retrofit in order to determine those concerns requiring additional testing and analysis not currently available to complete the design methodology. An outline of the design methodology has been established and each item in the methodology has been discussed in sufficient depth to determine further study items. A set of suggested criteria have been proposed with omission of those items where additional work is required. Finally, a summary of these further study items has been included so that the work to be performed in Phase 11 can be identified.

17. Document Analysis a. Descriptors

Masonry	Construction	Hazards
Earthquakes	Earthquake resistant structures	
Buildings	Seismology	

b. Identifiers/Open-Ended Terms

Seismic hazards
Unreinforced masonry buildings

c. COSATI Field/Group

18. Availability Statement	19. Security Class (This Report)	21. No. of Pages
NTIS	20. Security Class (This Page)	69
		22. Price PC A04

(See ANSI–Z39.18) *See Instructions on Reverse* OPTIONAL FORM 272 (4–77)
(Formerly NTIS–35)
Department of Commerce

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PROJECT SUMMARY

Many types of building construction exist in active seismic areas. Of these, the unreinforced masonry building has been singled out by earthquake engineers and building officials as particularly hazardous due to its damage record in earthquakes of even moderate intensity. Most were built before the development of seismic design criteria for new construction.

This project is intended to study this type of existing building to determine appropriate methods for determining the need for hazard mitigation and to study methods of retrofit so that design methods can be established. These design methods are to be devised with consideration of the particular structural conditions of unreinforced masonry construction, their earthquake response, the seismicity of the particular location, and the economics of retrofit.

Phase I has studied the state-of-the-art of hazard mitigation and retrofit in order to determine those concerns requiring additional testing and analysis not currently available to complete the design methodology. An outline of the design methodology has been established (Fig. 1.1) and each item in the methodology has been discussed in sufficient depth to determine further study items. A set of suggested criteria (Chapter 9) have been proposed with omission of those items where additional work is required. Finally, a summary of these further study items (Chapter 10) has been included so that the work to be performed in Phase II can be identified.

CHAPTER 1. INTRODUCTION

Every review of the overall physical hazard to man presented by high intensity earthquakes has concluded that the stockpile of existing unreinforced masonry buildings is the greatest single hazard category. Recognizing this, the Code agencies in highly seismic areas have frequently upgraded the standards by which new masonry buildings have been built. It was hoped that normal attrition would slowly reduce the hazard presented by the existing structures. With the changes in the economy within recent years, however, it has become apparent that the rate of reduction of this hazard has been slowed by salvaging and retrofitting existing structures for further use. Practice varies on the criteria governing retrofit. Frequently retrofit is acceptable if the resistance of a building to lateral forces is not reduced from its existing condition by the retrofit work. If it were mandatory that existing buildings be brought up to the standards for new buildings when retrofitted, a strong deterrent to retrofit would be created. This would tend to leave the hazardous buildings in use but without any mitigation of hazards.

Building construction using unreinforced masonry predates the development of seismic criteria that guide the design and construction of present day buildings in highly seismic areas. Many of these buildings are still in use and are the subject of considerable concern to the communities containing them. [1,2,3,4,5] Some of these buildings have suffered some damage in previous earthquakes while others experiencing the same intensity of shakes have been unscathed. This leads to a considerable difference of opinion between the governing agencies and the owners of such structures as to the need for an increase in their seismic resistance.

Formulation of design methods and criteria are needed for determining the need for hazard mitigation and for methods of retrofit when such needs have been established. Even in areas where no mandatory earthquake regulations exist, the concern is rising for some definition of the minimum level of protection required. Some scattered research has been conducted in this field by independent investigators, but no compilation of the test data has yet been attempted to bring the earthquake analysis of existing unreinforced masonry buildings to a usable form.

This study, then, reviews the current state-of-the-art in seismic hazard mitigation of existing masonry buildings and determines the tests and studies that will be necessary to devise simple and reasonable design methods and criteria for retrofit at costs less than those presently required and which will produce a significant reduction of the earthquake hazard.

The design methods must consider the strengths and interactions of the components of the seismic resisting system of unreinforced masonry buildings, the earthquake response of the building, the seismicity of the particular area, and the cost of retrofit. These methods would be formulated for the use of those administering the safety requirements of a community.

A Flow Chart of procedures for determining the need for seismic retrofit and retrofit procedures are given in Fig. 1-1.

[ ] See Chapter 11 for references.

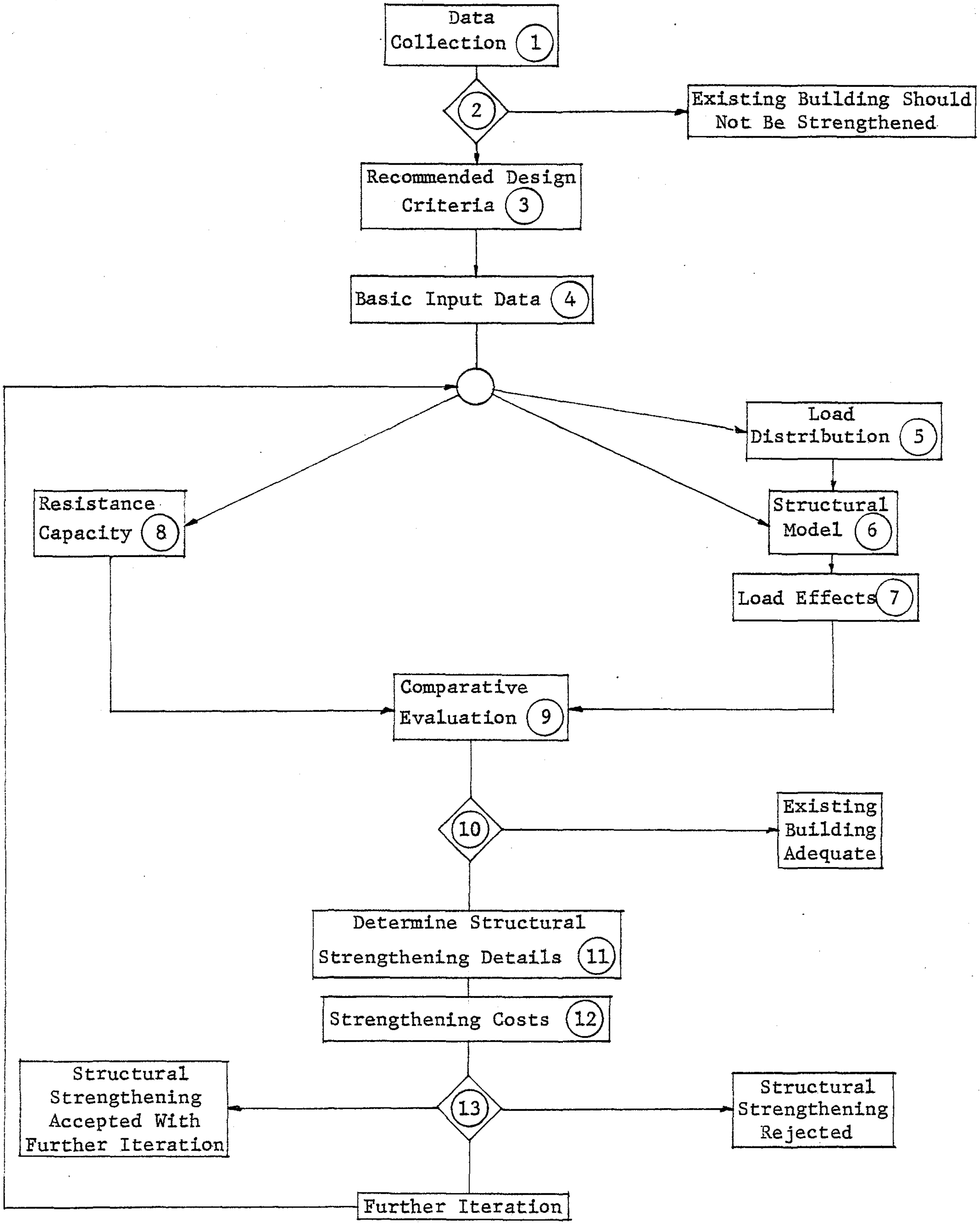


Fig. 1-1
FLOW CHART FOR DETERMINING NEED FOR SEISMIC RETROFIT AND RETROFIT PROCEDURES

CHAPTER 2. CONSIDERATIONS LEADING TO CHOICE OF DESIGN EARTHQUAKE AND RELATED BUILDING CAPABILITY

a. Description of Earthquake Hazards. The hazards induced by the exposure of a building to earthquake motions are many. They may be broadly categorized into the following types:

1. Psychological and sociological hazards.
2. Economic hazards.
3. Hazards to the life safety of building occupants and people in adjoining areas.

All these categories are interrelated and at times it is difficult to assign a specific building hazard to only one category. However, the first category would include such hazards as mental or social disruption of the normal routine and functioning of individuals or groups. The second would include the hazards resulting in damage only to the building or its contents. The last category would include those hazards resulting in either death or major physical injury to building occupants. This category is termed the life-safety hazard. It is the type of hazard considered within the scope of this work for which methods of hazard mitigation are to be devised.

It is to be noted that the emphasis in this work is to reduce existing hazards not to ensure that no hazard exists or to affect complete hazard abatement. Due to the random nature of any individual earthquake motion, a methodology to effect complete hazard abatement of the life safety earthquake hazard would be excessively costly and, as a result, could not be implemented. Thus the final end product to this study is to provide a methodology which will provide cost effective retrofit measures which will significantly reduce but not eliminate the risk of life-safety hazards.

If it is desired to mitigate the other two categories of earthquake hazard, including an effort to maintain operability of the building after a severe earthquake, measures beyond those covered in this methodology should be used.

As pointed out in ATC-3 [6] acceptable seismic risk is a matter of public policy determined by a specific government body. It should be based upon an evaluation of available technical knowledge, including the areas of seismicity, reasonable means available for protection, the magnitude of the earthquake risk, and the social impact of a major catastrophe.

b. Performance of Unreinforced Masonry Buildings. One of the more important evaluations necessary in order to establish an analytical methodology for the response of any building system is to review the overall performance of the system in past earthquakes, both severe and moderate. Application of the methodology to buildings which have been severely damaged and those which performed adequately will provide the information needed to judge whether the methodology will provide the desired level of hazard mitigation. This effort will be one of the major parts of the Phase II portion of this project once the methodology has been established.

Many references are available which describe the behavior of unreinforced masonry buildings to severe earthquake motions.[1, 7 through 12] The general consensus of opinion is that in an earthquake, unreinforced masonry walled buildings are extremely hazardous. The preponderance of results bear this out. However, these opinions are based on examples of performance during high seismic activity. And even in such cases there has been an occasional unreinforced masonry building in the heavily damaged area which suffered little or no apparent damage.

From a review of these references a number of conclusions can be reached.

1. Any new masonry construction in areas subject to severe earthquakes should be reinforced grouted masonry.

2. Most of the building damage which constitute a life-safety hazard were a result of

   - Inadequate connections between the masonry walls and floor or roof diaphragm.

   - Excessive deflection of the diaphragm system causing out-of-plane wall failure or failure of the interior vertical load system.

   - Differing dynamic response of component parts of complex buildings.

   - Collapse of parapet, cornice, veneer, and other building appendages particularly those over or adjacent to exits, exitways, or public ways.

3. Comparatively small number of severely damaged buildings were a result of in-plane shear or flexure failure of the lateral force resisting shear walls. This should not imply that the problem should be overlooked.

c. <u>Classification of Buildings Based on Hazard Exposure.</u>

1. <u>Seismicity of Building Location.</u> Obviously the degree of earthquake hazard is a function of the seismicity of the building site. Several attempts have been made to regionally map areas of approximately equal seismic hazard exposure. It is not the intent of this study to delve into these problems. In order to have a base upon which a methodology can be developed, it is our intention to use the most recently developed, which are the two maps in ATC-3[6] as being representative of the latest joint thinking of geologists, seismologists, and earthquake engineers. If another regionalization were to be used, the methodology would not change - only the detail.

2. <u>Occupancy Factors.</u> The hazard to life-safety is dependent on the frequency and density of persons using a building. Thus, a building that has a high density population and in continued use, such as a high rise building containing both office and living space, or penal institutions, would have a high risk for life safety hazard exposure. Conversely, a building such as a warehouse with stored materials seldom handled or a building infrequently used by a fairly large number of people would have a low risk of life safety hazard exposure.

The use of occupancy factors becomes important not only in selecting the design earthquake but also in establishment of priorities of hazard mitigation.

3. Anticipated Building Life. The exposure to the hazard of an earthquake occurrence is obviously greater if the building is expected to have a long life than one which would anticipate only a short life. Retrofit could extend the life of a building well beyond the average life span of the building considering its particular occupancy. To date, most investigations into the probabilistic occurrence of earthquakes have assumed a building life to be between 50 and 100 years. The average extended life of a retrofitted building (except historic buildings) probably would be less than comparable new buildings. As the requirements for space in a building do change, obsolescence would occur faster in a retrofitted building than a new one. Research into this factor will not be carried on in Phase II. Until further studies indicate differently, any assumption of building life should be assumed to be 50 years as was done in the ATC-3[6] study.

4. Social Importance. Some buildings have a social importance to the community that outweighs its population density and frequency of use. Buildings included in this special category would be those buildings in which continued operation is important during and immediately after the occurence of a damaging earthquake. These would include buildings housing emergency communication centers, disaster command posts, fire stations, hospitals, etc. Special requirements for these facilities should be rigorously maintained and when the anticipated seismic activity is such that the Effective Peak Acceleration is in excess of 0.15, unreinforced masonry should not be used in retrofitted buildings of this special category.

5. Social Significance. Some buildings have a social significance such that maintaining a comparatively high risk of earthquake damage is acceptable to the public. These would include historic buildings in which the cost of full retrofit would be prohibitive. For this type of building, it may be possible to mitigate significantly the earthquake hazards at a permissible cost, even though still retaining a fairly high risk of sustaining damage during a severe earthquake. Part of the hazard mitigation in these instances should be a public warning of the earthquake hazard visible to public users.

6. Speed of Retrofit. Another facet of social significance that may have an influence on the choice of the design earthquake would be the disruption to a community presented by the displacement of people during retrofit operations. The pace of retrofit could be slowed to the point where little overall disruption of community activities would result. This would extend the length of time to reach a reasonable level of hazard mitigation which, in turn, increases the risk of having buildings not retrofitted exposed to a damaging earthquake. Thus the pace of retrofitting and the extent to which retrofitting is required needs to be considered in deciding on the level of design earthquake.

d. <u>Current Criteria Used for Analysis</u>. The criteria used in current building codes is exemplified by the requirements in Uniform Code for Abatement of Dangerous Buildings (UCADB) 76.[13] None of the definitions of "dangerous buildings" (Section 302) explicitly address the problem of earthquake hazardous buildings except when the building has been damaged. It could be interpreted that definition number 4 would be applicable. This states that a building is "a dangerous building whenever any portion or member or appurtenance thereof is likely to fail, or to become detached or dislodged, or to collapse and thereby injure persons or damage property". Section 403 states in part "Any building declared a dangerous building under this ordinance shall either be repaired in accordance with the current building code or shall be demolished at the option of the building owner". From these it would appear that full compliance with the provisions for new buildings is required. However, a Board of Appeals is made available for establishing potential individual deviations of code provisions.

The Los Angeles Building Code [14] has the phrase "shall be brought up to a reasonable condition of stability and safety, or ...." This wording would permit the discretionary use of some code provision modifications with the concurrence of the Building Department. Also included in the Los Angeles City Building Code is the "parapet" ordinance. The essence of this ordinance is that no building shall have a parapet or appendage that is not adequately braced against earthquake motions represented by the code specified earthquake. Parapets were inspected by the Building Department, notices sent, hearings held to abate the hazard by the following procedures:

"1. Submit to the Department suitable corrective plans;
2. Obtain the necessary alteration permit; and
3. Complete all the work necessary or ordered" to demolish, reconstruct to conform to Code, or strengthened by bracing or other means to resist the forces of an earthquake.

Explicit means to accomplish the latter in a simplified manner were developed by the Building Department and made available to the public.

Specific regulations were adopted in the City of Long Beach, California, dealing with the problem of earthquake hazard mitigation. [5] The regulations were based on the recommendations of a consultant.[15] Similar provisions were introduced in 1975 as proposed changes to the Uniform Building Code [16], but were turned down as the Proposal [17] was not compatible with UBC 76. Among others, these provisions contain the following elements:

1. Area seismicity should be determined by community.

2. A method for grading the building hazard (with grading sheets).

3. A method for priority of hazard grading.

4. The lateral force capability of the building at the point of major structural failure is required.

5. A base shear representing a minimum tolerable level is provided to compare with the capability of the building.

6. Base shear modifiers are provided to reflect the building importance and life expectancy.

7. Base shear modifier for soil effects and fundamental building period were provided.

8. Base shear could be modified based on established damping factors.

9. A method for mandatory compliance with review bodies.

10. Minimum strength of masonry is given determined by core tests.

Regulations were adopted in the City of Santa Rosa, California, also based on a consultant's recommendations.[4] Similar criteria were used in Oroville, California, after the 1975 earthquake. Among others the provisions contain the following elements:

1. Preliminary review by City of buildings built prior to a cut-off date, except schools and one and two family dwellings.

2. A priority for this review based on occupancy was developed.

3. The preliminary review was by visual inspection only.

4. For those buildings found to be non-conforming to 1955 UBC by the preliminary review, a further investigation paid for by the property owner was required.

5. This further investigation was to be performed by a structural engineer and would include studies (tests if required) so that the engineer could make a statement relating to a code evaluation.

6. Giving an opinion of the safety of the building without regard to code requirements.

7. Relating to reinforcement needed to meet criterion for long term continued use, five year use, or one year use. These were to be determined based on an overstress on UBC 55 values permitted on various building components. Various overstresses were specified for each building life.

The specific requirements in current building codes which would not be applicable to an analysis to determine the degree of earthquake hazard of existing buildings are those which would eliminate existing system components from use. Some of these in UBC 76 [16] are as follows:

1. Section 2312(j)2.B. "All elements within structures located in Seismic Zones No. 2, No. 3 and No. 4 which are of masonry or concrete shall be reinforced so as to qualify as reinforced masonry or concrete ...."

2. Section 2516(b). "Wood members shall not be used to resist horizontal forces contributed by masonry or concrete construction in buildings over one story in height.

   EXCEPTION: Wood floor and roof members may be used in horizontal trusses and diaphragms to resist horizontal forces imposed by wind, earthquake, or earth pressure, provided such forces are not resisted by rotation of the truss or diaphragm."

3. Section 2310. "Concrete or masonry walls shall be anchored to all floors and roofs ... minimum force of 200 pounds per lineal foot of wall...." [No provision for variation with seismic zone.]

4. Section 2312(d). "... total lateral force ... V = ZIKCSW ... the product of CS need not exceed 0.14. The value of C shall be determined in accordance with the following formula: C = 1/15 T. The value of C need not exceed 0.12...."

   (All current force criteria for design of masonry buildings [16,18,19, 20,21,22] were developed and based on new construction. The direction of future studies should be to exphasize criteria appropriate to existing construction.)

It is recommended that these four items be studied in Phase II with consideration given to the discussions in this Chapter.

CHAPTER 3. DEFINITIONS AND TYPES OF MASONRY

The masonry considered in this study are those types normally used for building wall construction. Masonry for such items as bridge pier or abutments and retaining walls will not be discussed. Some of the terms frequently used in masonry construction are:

1. Ashlar Masonry. Masonry composed of rectangular units laid in mortar and properly bonded. The units are usually larger in size than brick, having sawed, dressed, or squared beds.

2. Bond. The mechanical disposition of masonry units by overlapping to break joints.

3. Brick. A rectangular unit made from burned shale, clay, or a mixture of the two and not less than 75% solid. The term applies to other materials as well, but is preceded by a modifier to indicate the major components such as concrete brick, fire clay brick, etc.

4. Buttress. A bonded masonry column integral with a wall to provide lateral stability to the wall. The buttress decreases in thickness from the base to the top though never thinner than the wall.

5. Chase. A continuous recess in a wall.

6. Column. A compression member with a width not exceeding four times its thickness and a height more than three times the least lateral dimension.

7. Concrete Block. A machine formed unit composed of Portland cement, aggregates, and water.

8. Coping. A cap on top of a wall, pier, or other masonry element to prevent penetration of water into the masonry below.

9. Corbel. Successive courses projecting from the face of a wall to increase the thickness or form a ledge.

10. Course. A continuous layer of bonded units.

11. Gross Cross-Sectional Area. The area enclosed by the outside plan dimensions of a masonry unit. Grooves or scores on the face of the unit shall be ignored in determining the gross area.

12. Grout. A mixture of cementitious material and fine aggregate with adequate water to pour without segregation of materials.

13. Grouted Masonry. Masonry in which the interior joints are filled with poured grout.

14. Header. A masonry unit laid flat across a wall, with end surfaces exposed, to bond two wythes.

15. Hollow Masonry Unit. Masonry unit which has at all horizontal sections a net cross-sectional area less than 75% of the gross cross-sectional area.

16. Masonry. As used herein will designate wall and column construction composed of shaped or molded units small enough to be handled by one man, bonded together with mortar or other cementitious material. Each unit would be plain material not reinforced with steel.

17. Mortar. A plastic mixture of cementitive material, fine aggregate, and water.

18. Net Cross Sectional Area. The gross cross-sectional area less the area of ungrouted cores or cellular spaces.

19. Parapet. The height of a wall above the roof line.

20. Pier. An isolated column where its horizontal dimension measured at right angles to the thickness does not exceed four times its thickness.

21. Pilaster. A bonded column built as part of a wall and projecting from one or both faces of the wall.

22. Solid Masonry Unit. Masonry unit which has at all horizontal sections a net cross-sectional area of 75% or more of the gross cross-sectional area.

23. Stretcher. A unit which has been laid with its greatest length parallel to the face of the wall.

24. Veneer. As used herein is a wythe of masonry securely fastened to a wall but not considered as adding to the strength of the wall.

25. Wall Masonry. Vertical or nearly vertical construction enclosing or dividing space within a building:

    a. Bearing Wall. A wall supporting vertical loads in addition to its own weight.

    b. Cavity Wall. A wall composed of two wythes separated by a space containing air, insulation, or other non-bearing materials. The two wythes are tied together with metal ties.

    c. Curtain Wall. A non-bearing exterior wall.

    d. Faced Wall. A wall of two or more wythes having the units in the wythes of different materials.

    e. Grouted Wall. Wall of two or more wythes having the space between wythes filled with grout or a wall of hollow units in which some or all cells are filled with grout.

    f. Non-Bearing Wall. A masonry wall supporting no loads other than its own weight.

    g. Party Wall. A wall on an interior property line used jointly by two buildings.

    h. Spandrel Wall. A curtain wall extending from the window head of one floor to the window sill of the next floor above.

26. Wythe. Each continuous vertical section of a masonry wall one unit in thickness.

The units used in masonry construction in buildings include:

- Brick of burned clay or shale, sand-line, or concrete.
- Concrete hollow load or non-load bearing or solid load bearing.
- Structural clay tile, load bearing or non-load bearing.
- Gypsum tile or block.
- Unburned clay units.
- Natural stone of many types including granite, marble, etc.

CHAPTER 4. TYPES OF BUILDINGS WITH UNREINFORCED MASONRY WALLS

Buildings using unreinforced masonry construction can be generally categorized into the following types:

a. Exterior Masonry Bearing Walls. Individual buildings more or less isolated from adjacent buildings having exterior masonry bearing walls. This type would include those having interior framing using bearing or non-bearing masonry or wood and wood stud, concrete or steel frames. This type of building can usually be analyzed as a single entity but the effect from or to adjoining structures cannot be overlooked.

b. Infill Walls of Masonry with Concrete or Steel Frame. Masonry infill walls have sometimes been ignored by inexperienced designers in analyzing building response to lateral forces. Unless separated, such walls are relatively stiff and will transfer lateral forces until they crack up. Thus the response of the building will not be as assumed if such walls are ignored.

   Tests of such infill walls, using inplane loading, show that failure will occur in diagonal tension or such walls may act in diagonal compression. When this occurs, the enclosing frame is subjected to shear or bending stresses or both. Many shear or bending failures have occurred during earthquakes in such enclosing frames.[23,24]

   Shear failures have been more common where reinforced concrete enclosing frames are involved than where the enclosing frames are structural steel. This has been because of the inherent shear capacity of steel shapes. Concrete frames can be provided with the required shear strength by design but this design has usually been overlooked in past construction.

c. Structural Frame Other Than Masonry. Non-load bearing masonry partitions and curtain walls and veneers have frequently been used in buildings whose structural frame is either steel, concrete, or wood construction. Consideration for the in-plane stiffness is frequently overlooked as in the case of infill walls. Provisions for support for out-of-plane forces are frequently deficient.

d. Row Buildings. In many areas of the United States there are rows of buildings with common walls or party walls or ownerships separated by a single division wall. Some row buildings may occupy an entire block. Where such buildings are constructed with unreinforced masonry walls and are considered hazardous, some legal problems may arise. In the case of party walls there may be a party wall agreement. This agreement may or may not state who has responsibility for retrofit involving such walls. Also, there is no earthquake separation between buildings. If no separation can be provided these units, when analyzed structurally, should be considered as one building. Sometimes adjacent units have different story heights or have different total height which complicates the structural analysis.

   In some cases where there are no common walls or party walls, there is no separation and buildings may pound each other in an earthquake. There has been considerable damage in past earthquakes from such pounding when the framing of the adjoining buildings are not compatible. The hazard from pounding should be considered for the individual cases. Usually it would be very difficult and expensive to create an adequate separation.

In some cases, adjoining properties with a common wall may have different occupancies and may be classified as possessing different life hazards. In a community where rehabilitation or demolition is required by law, priorities may be established such as to require the buildings with highest life hazard to be rehabilitated first. In such cases it would seem prudent to rehabilitate these adjoining buildings at the same time.

The concept of condominiums is fairly recent but in most jurisdictions there is nothing to prevent converting existing apartment houses to condominiums. The problem here is similar to that of row buildings with common walls in that there is not one but a group of owners. Usually there is a condominium agreement and an association of owners. This agreement may or may not cover the allocation of responsibilities for retrofit costs.

From the above discussion, a community which has decided to reduce the earthquake hazards of unreinforced masonry buildings should consider the legal problems involved when row buildings or similar structures are required to be retrofitted as multiple ownership is involved.

The solution to legal problems may differ in various states. It is suggested that research on the legal problems be compiled so that sound advise would be available to communities contemplating seismic hazard mitigation.

When such a block of buildings are considered as a unit, the transverse walls are typically quite frequent and have a minimal number of openings. Usually the floor frames into the transverse walls providing some bracing to the wall. Thus the transverse walls frequently may not require retrofit. The longitudinal walls are different, however. Usually many openings are found, particularly on the first floor. As the floor framing parallels these walls, bracing is not provided except by the transverse walls. Usually the determination of the hazards presented by this type of building would be analyzed by investigating these problem areas.

CHAPTER 5. DETERMINATION OF STRUCTURAL HAZARD IN BUILDINGS

a. Information Necessary for Evaluation. [6,25,26,27]

1. Availability of Plans, Calculations, and Soils Report. It is necessary to determine the seismic zone of the site so that the seismic forces to be used in analysis may be established. The size of components and the modulus of elasticity of materials are needed so that an analysis of the seismic load effects on the seismic resisting system can be made. This usually requires drawings and specifications so that reasonable assumptions of the capacities of the material can be made. If such detailed information is available, it should be verified at the site. If changes have been made, drawings should be corrected to show "as-built" conditions. If none are available, drawings should be made indicating in adequate detail the pertinent information on the elements of the framing system.

   The original structural calculations should be obtained if possible. In areas where no seismic analysis was required, calculations may include only vertical load design, or possibly vertical load plus wind. Even so, these will be of assistance in the evaluation.

   A soils report may be available giving recommended bearing pressures or pile capacities. If such a report is not available, the adequacy of the foundation design may be roughly estimated by the "test of time". Thus, if the building shows no evidence of settlement damage, the foundation design criteria originally used was probably adequate - at least for vertical loads. If there is evidence of settlement, which usually is indicated by diagonal cracks in walls or partitions, the adequacy of the foundation design criteria would be questionable. If no foundation or soils report is available, some information of value may be obtained from soils investigations of adjoining properties.

2. Field Inspection. Whether or not plans are available, a field inspection should be made. Plans and specifications should be verified and corrected to show any changes. If none are available, drawings should be made indicating in adequate detail the pertinent information on the elements of the framing system. A field inspection will also determine whether adjacent buildings are separated adequately to avoid pounding during an earthquake and whether they have common or party walls.

3. Field Analysis. The field inspection may reveal inadequacies in vertical load-carrying capacity which will permit an early feasibility decision. For instance, severe foundation settlement may have been noted. This could mean that a soils investigation, expensive underpinning, or soil stabilization would be required. If the structure were close to an unstable slope, the feasibility of the site itself might be questionable.

   Plans and field data may provide information that indicate that the vertical resisting elements are inadequate. It may be obvious without further analysis that additional vertical resisting elements will be required or that existing ones must be replaced or strengthened. A rough estimate of cost of such strengthening may show that it would not be economically feasible to proceed.

The data may show that the horizontal diaphragms are inadequate. They may have insufficient strength or may be incapable of transferring lateral forces to the vertical elements. A concrete diaphragm may not connect to an important shear wall. This would occur where the entire floor area adjacent to the wall is occupied by stairwells, shafts, and elevators. However, in many cases, the data may not be sufficient to permit such obvious conclusions. In such cases an analytical evaluation of the building would be needed.

There may be cases where information concerning critical elements is not available because of inaccessibility. A decision must be made on whether to obtain this information by removing finishes for observation or by testing. The cost of such cutting and/or testing can usually be roughly determined from past experience in the locale. Without adequate information about the critical elements, a proper analytical evaluation cannot be made.

4. Criteria Under Which Building was Constructed (Code, Inspection, and Enforcement). The criteria or code under which the building was constructed and the adequacy of enforcement and inspection may provide clues to items of construction not visible.

b. Determination of Critical Elements. In the design of new buildings, adequate strength can be provided to resist the design loads in all elements of the building. When evaluating the seismic hazard presented by an existing building, however, one additional design concept is needed. This is the determination of the "critical elements" of the building. This is necessary in order to reduce design and analysis time and to reduce repair costs by not requiring strengthening on elements when such strengthening would not significantly reduce the hazard.

In order to define "critical" elements it is necessary to discuss "important" elements. As used herein an important element is an element which, if it failed, would seriously reduce the capacity of the structure as a whole to resist vertical and lateral forces. Some members would not be considered important when deformed beyond their yield level deformations. Other members upon yielding could cause an important redistribution of stress paths. With many well-distributed similar elements, the redistribution of stress would increase in adjoining or parallel elements by only a small percentage. In a shear wall building, for instance, if a pier in a wall is highly overstressed and its force redistributed to the remaining piers without overstressing them, then the pier in question would not be considered "important". However, in a large multi-story structure supported on only four columns, the failure of one column could cause total collapse and, in such cases, the evaluation of columns, beams, and connections should be viewed with special care. Thus any "important" earthquake resisting element having a high ratio of design unit stress to allowable unit stress is termed a "critical element" and must be considered in the evaluation of the adequacy of the structural system.

In most buildings the critical elements will be the vertical resisting elements (shear walls, or moment resistant or braced frames) and the horizontal resisting elements (diaphragm or bracing). The earthquake loads normal to a wall are a function of the weight of the wall itself. Where a wall has a long span between floor diaphragms or between vertical frame elements, it might be a critical element if its failure due to these forces would produce collapse of the building as a whole. Good engineering judgment in selection of critical elements may save considerable time. If original calculations are available, stress ratios taken from these calculations may be useful in determining the critical elements even though the original design criteria may be different from that used for seismic evaluation.

c. Determination of Physical Properties of Critical Elements.

1. Diaphragms.

A. Where concrete floors or roofs act as diaphragms to distribute horizontal forces to vertical resisting elements, the concrete strength can be determined by taking and testing cores. Schmidt Hammer tests are feasible in certain locations but the average of several tests should be taken at each location. A pachometer uses a magnetic field to locate reinforcing and give the amount of concrete cover. If the cover is known, the size of reinforcing can be determined and vice versa. This test has been used frequently to determine if top bars have been "walked down" during construction. However, the presence of electrical conduit or other electrical field carriers may invalidate the results. The accuracy of pachometer readings should be verified in one or more locations by cutting or chipping to expose the reinforcing. The depth of cover through which tests are effective is limited. X-rays can sometimes be used to determine location of reinforcing steel, voids in concrete, and location of anchors. X-ray tests, however, are relatively expensive.

B. The physical properties of wood floors and roofs acting as diaphragms can usually be determined by visual inspection. The type and direction of sheathing, blocking, splicing or units and nailing should be determined.

Anchorage of the diaphragm to the vertical resisting elements is of utmost importance. Wood diaphragms in many older buildings are inadequately anchored for earthquake resistance. One of the typical anchorage types used was a self-releasing tee head anchor. This has some merit in the case of fire but little value in resisting lateral forces. Another type of anchorage used in the past was wood ledgers bolted to masonry walls. The floor or roof sheathing was nailed to the ledgers or to blocking. Many failures of this type of anchorage occurred in the San Fernando earthquake of 1971.[9]

A diaphragm acts as a horizontal girder and must have flanges (chords) capable of taking both tension and compression. Where unreinforced masonry is used it is usually necessary to provide chords of some type. The strengths of the various types of wood diaphragm may be found in most building codes.

C. The determination of the strength and flexibility of metal deck diaphragms is best made by a review of the plans and by inspection where visible. The shape of flutes, gage, quality and type of metal, welding or screw pattern to supports, and the connections at side laps are the necessary items. Specimens of the deck can be taken and tested to confirm gage and physical properties of the metal itself.

   Steel decks having a concrete fill with either normal or lightweight aggregates are frequently used. In some cases the concrete may be adequate in itself to resist diaphragm shears if properly dowelled or otherwise connected to resisting elements.

   Diaphragm values for most steel decks have been determined by tests. Code approved design strengths have been obtained by most manufacturers from code authorities (for instance, ICBO Research Recommendations). The Tri-Service Manual [28] discusses metal deck diaphragm strength and flexibilities in some detail.

D. Horizontal steel bracing[28] is sometimes used as a diaphragm. It is analyzed as a truss. The quality of steel can be checked by laboratory tests if needed. Bolting can be field checked for tightness. Visual inspection should be made for evidence of corrosion.

E. In all types of diaphragms, the adequacy of the anchorage to the vertical resisting elements and the development of chords must be determined.

2. <u>Masonry Walls</u>.[25,27,29,30,31]

The determination of the strength of masonry in existing buildings is difficult. However, the evaluation of the earthquake resisting capability of these walls can provide information so that reasonable decisions can be made on the necessity for strengthening. For this reason the determination of reasonable working values is required. UBC 76 for instance, does give values for unreinforced masonry for new work. Table No. 24-B for uninspected work allows tension or shear values varying from 4 to 12.5 p.s.i. depending on the type of masonry.

Development of methods for determining appropriate strength characteristics is one of the subjects recommended for study under Phase II. The quality of mortar used in existing masonry may not comply with that required by codes for new masonry. Thus, unless there is evidence of adequate inspection during construction, the value for uninspected masonry may be appropriate. If the mortar has less cement than called for in the Code, values even less than these must be used. If a core cannot be removed without damage or if a core tests to something less than 750 p.s.i. compression, it may be appropriate to assign no earthquake resistance to the masonry. On the other hand, most unreinforced brick masonry walls have some structural value. Therefore, in the case of cores testing under 750 p.s.i. compressive stress and in the case where a core cannot be removed, a working stress value of 2 or 3 p.s.i. in shear and tension could also be acceptable. For cases where cores test over 750 p.s.i. and the joints are well filled and the mortar is equivalent to Type N, then shear and tension values given in the Codes may be used. If the joints, particularly the vertical joints, have not been well filled or do not have a good bond, then lower values should be used.

In some old lime mortar brick masonry, a complete core cannot be removed intact. In such a case, the earthquake resistance of the masonry element is very poor with little or no value in resisting earthquake forces. Cores to permit a visual assessment of the quality of construction, as vertical joints can be observed and reinforcing, if any, can be located. Cores can be tested in compression, but such a test is of little relevance, since it is not in the direction of the actual vertical load. Mortar can be tested for cement, sand, and lime content. Bond between the masonry elements can be visually inspected. Coring in hollow unit masonry walls can confirm the location and quality of filled cells and the placement of reinforcing. Compressive shear tests on cores have little relevance in assessing shear capacity parallel to the plane of a wall.

Tests on cores where a load is applied at an angle to the bed joints can give bed joint shear strengths[23]. This method should be analyzed for a correlation with appropriate design strengths. It is recommended that this subject be part of Phase II.

Prism tests of existing walls can be of assistance in evaluating masonry strengths, if adequately sized prisms can be removed. This is difficult and expensive with existing masonry. If a prism can be satisfactorily removed, it can be tested in a laboratory for both shear and compression. It is frequently not possible, however, to remove a full, undamaged prism. Furthermore, the prism hole usually must be repaired. The problems of prism testing are recommended for investigation in Phase II.

Prisms have also been tested on the diagonal[31,32,33,34]. Many of the diagonal tests of prisms and cores have been made on masonry one wythe wide, probably because of the difficulty of removing and transporting multi-wythe prisms without damage. For this reason such tests will give values for one wythe only. In many cases, say in a three wythe wall of brick, the middle wythe usually has been laid with poor workmanship. Also the mortar used on exposed faces may be better than on non-exposed surfaces. Thus correlation of a single wythe to multi-wythe wall strength requires study and is recommended for Phase II.

Taking of prisms or cores does permit visual inspection so that with some judgment and experience the physical properties of the wall can be estimated.

Thus it is recommended that further study in Phase II be made to find a simple method of determining the physical properties of the existing unreinforced brick masonry. One suggested method to be investigated is a test similar to a Schmidt Hammer test but used with a small driving pin in mortar joints. Such a tool would be calibrated and a correlation between penetration and force of blow and the strength of the masonry would be made. This would test only the mortar joint quality.

Perhaps it could test bond. The condition beyond the depth of penetration might affect the ease of penetration and provide a clue. It is recommended that this subject be studied in Phase II.

3. <u>Foundations.</u> The adequacy of foundations to resist vertical loads can be generally appraised by field inspection to see if there are cracks of a pattern indicative of settlement. However, overturning effects from earthquake forces create positive and negative pressures on footings. The added compressive force may create overloads which can produce settlement by consolidation of the supporting soils. (Note that overload of soils may not always create a critical condition on a building.) Where a building is founded on loose or not very dense soils, the shaking of the ground in itself may consolidate the soil, producing settlement of the supported structure. Liquefaction of soils supporting foundations may occur during earthquakes in some types of soils containing excessive moisture.

   The concrete strength in footings can be checked by testing cores. The presence of reinforcing bars can sometimes be verified with a Pachometer. The width of footings may be verified by excavation if desired.

   If there is a question of the adequacy of the soil to resist both lateral and vertical loads a soils investigation should be made by competent foundation engineers. Soils information from adjacent properties may give adequate answers if no soil investigation was made at the site in question.

d. <u>Analysis of Lateral Force Resisting System.</u>

1. <u>Design Base Shear.</u> In order to establish a basis for the design of the lateral force resisting system, it is usually assumed that an adequate design is made if the strength of each member of the resisting system is equal to or greater than the load effects on the member resulting from the application of prescribed design loads. This may be expressed in the format for Load and Resistance Factor Design (LRFD) [35] as

$$(\phi R_n)_k \geqslant \gamma_o \left(\sum_{i=1}^{n} c_i \gamma_i Q_i \right)_j \quad \text{where}$$

where

$(\phi R_n)_k$ = factored nominal strength for limit state k

ϕ = resistance factor for the appropriate limit state.

R_n = nominal strength for the appropriate limit state.

$\left(\sum_{i=1}^{n} c_i \gamma_i Q_i\right)_j$ = factored internal force for load combination j.

c_i = influence factor by which the factored load intensity $\gamma_i Q_i$ is transformed into an internal force (i.e., bending moment, shear force, axial force, torque) by structural analysis.

γ_i = load factor for load type i

Q_i = load or load intensity i

γ_o = analysis factor.

Usually the Building Code regulations have been written in the form of "working stress design". When expressed in LRFD format, the design equation becomes

$$R_{nk} \geq \frac{\gamma_o \gamma_i}{\phi_k} \left(\sum_{i=1}^{n} c_i Q_i\right)_j$$ in which the load

factors (γ_i) and resistance factors (ϕ_k) are assumed to be constant for all types and strength limit states. Both ways of looking at the design equation envision an elastic analysis of determining the load effects from the imposed load (γ_o and c_i) with some modification or redistribution for non-linear effects. Thus to establish the appropriate loads to use in the design equation for seismic design, it is recommended that necessary tests and analyses be performed as part of Phase II of this study.

Current seismic design requirements (SEAOC 75[36], UBC 76[16], ANSI 74 [37] and ATC-3[6]) specify methods for determining a design base shear. In all cases the design base shear is not directly related to elastic response to anticipated ground motions. The elastic response is modified to reflect multi-mode response, variations in damping at high amplitude, permissible ductility of the framing system, soil structure interaction, general past behavior of the building system used. The one set of design requirements specifically dealing with the response of buildings using unreinforced masonry walls is in ATC-3. The method of base shear determination is repeated herein for the specific case of the buildings being discussed.

Base shear $V = C_s W$ where

$C_s = \frac{2.5A_a}{R}$ except for Type S_3 soils where $A_a \leq 0.3$. In this case, $C_s = \frac{2.0A_a}{R}$. It is also permitted to reduce the value of C_s if the fundamental period of the building were determined. Then the value of $C_s = \dfrac{1.2A_vS}{RT^{2/3}}$.

The terms in these equations are basically as follows (for more detailed explanation, see Reference 6):

V = Base shear.

W = Building dead load plus some other loads.

A_a = Coefficient representing Effective Peak Acceleration for which a country-wide map was provided.

A_v = Coefficient representing Effective Peak Velocity - related acceleration for which a country-wide map was provided.

S = Coefficient related to the soil profile characteristics of which S_1, S_2, and S_3 were given.

R = Response modification factor given for various framing systems.

T = Elastic fundamental period of the building with methods provided for simplified determination.

For the type of buildings considered herein, the vast majority would not use the period related equation. The base shear would be determined by $\dfrac{2.5A_a}{R}$ or $\dfrac{2.0A_a}{R}$ with most buildings requiring the first.

The values of R are 1-1/4 for unreinforced masonry bearing walls and 1-1/2 for a vertical load frame with filler walls. Most bearing wall buildings would be designed for

$$V = \frac{2.5}{1.25} A_aW = 2A_aW.$$

The load effects determined from the design base shear are to be equated to permissible stresses equal to 2.5ϕ times the stresses specified in working stress specifications. $\phi = 1$ for flexural stresses except tension across bed joints in which $\phi = 0$. $\phi = 0.4$ for masonry shear. It should be noted that these requirements were specifically for new construction and unreinforced masonry is not permitted in Building Categories C and D which are categories in high seismic areas and includes most buildings in areas having A_a and A_v greater than 0.15. An exception to this exclusion was given when it was permitted by the cognizant jurisdiction when justified in an analytical evaluation. No frame analysis is required for buildings in areas having A_a or $A_v \leq 0.05$. SEAOC 75[36] and UBC 76[16] specify a base shear $V = ZIKCSW$ in which

- V = Base shear.
- Z = Zone factor with map showing four zones in which $Z = 1.0$, 3/4, 3/8, and 3/16.
- I = Occupancy factor of 1.0, 1.25, or 1.50.
- K = Coefficient depending on lateral force resisting system = 1.0 for frame with filler walls and = 1.33 for bearing walls.
- $C = \frac{1}{15\sqrt{T}}$ but need not be greater than .12.
- S = Soil factor coefficient normally chosen at 1.5 unless special studies are made except CS need not be greater than .14.
- W = Building dead load plus some other load.

For most high frequency buildings, $V = Z\ 1.33 \times .14W = 0.186ZW$.

These forces are compared to working stress strengths with a 1/3 increase usually permitted.

It is difficult to have an all inclusive comparison between these two design procedures. However, assuming that UBC 76 in areas with Z = 3/8 the base shear is 0.070W. The comparable value from ATC-3 would be $\frac{2 \times 0.15W \times 1.33}{2.5} = 0.160W$ for flexure and $\frac{2 \times 0.15W \times 1.33}{2.5 \times 0.4 \times 1.5} = 0.266W$ for shear. For reinforced masonry the comparison is $\frac{0.160 \times 1.25W}{3.5} = 0.057W$ for flexure and $\frac{.266 \times 1.25}{3.5} = 0.095W$ for shear. These comparisons indicate when the seismic hazards of unreinforced masonry were specifically considered in ATC-3, design of unreinforced masonry lateral force resisting systems was severely penalized due to past performance in major earthquakes. This penalty imposed on new construction, however, does not solve the problem of the mitigation of existing hazards, if the cost of strengthening the entire lateral force resisting system makes retrofit impractical and unaccomplished.

In many existing buildings using masonry walls very flexible floor and roof diaphragms are found. The response characteristics of the building may well be dominated by the response of the diaphragm rather than the wall system. [38,39] It is recommended that part of the Phase II would be to study this effect on the magnitude of the base shear.

2. Vertical Distribution of Forces. All current design specifications prescribe methods to distribute horizontal forces vertically. These forces when numerically totalled are equal to the base shear described previously. These methods all include a triangular distribution for low buildings but are modified for taller buildings. These modifications are not discussed herein as most buildings within the scope are not tall buildings. These triangular distributions are applicable to buildings having fairly constant mass to stiffness ratios of building stories. Significant irregularities in the building frame are required to be considered in the vertical distribution of forces. One way to do this is by use of an elastic modal analysis of the building. Another is to modify the distribution based on experience or previous studies.

3. Torsion. The response of buildings to earthquake motions is composed of both translational and torsional components. This torsional response can be caused by any one or more of the following:

   A. The size of the building may be such that the input ground motion is not composed of constant translation across the width of the building. This results in a combination of translational and rotational input motion to the building. Usually past measurements of earthquake input have been taken at one point and records have not given a clear idea of the magnitude of the torsional input. Current programs in California of building instrumentation include provisions for obtaining this information in future earthquakes.

   B. Assuming elastic response of the building, the mass of the building may not be distributed in such a manner so that the center of mass is coincident with all the axes of rotation. When this occurs there would be torsional response.

   C. As discussed previously the elastic response of buildings is only providing for a simplified design procedure. Actually non-linear response is anticipated without building collapse and is considered implicitly in most building design specifications. When non-linearity does occur, the time history of earthquake motion becomes important and as all elements of the lateral force resisting system will not have concurrent and identical non-linear behavior, eccentricities will result which will amplify significantly the torsional response.

   Until recently the provisions for torsion were prescriptive. They did call attention to the problem even though not adequately describing or providing for torsional response. The latest proposal (ATC-3) at least philosophically more correctly addresses the problem. This statement is as follows:

   > "The shear and torsion in any horizontal plane shall be distributed to the various vertical elements of the seismic resisting system with consideration given to the relative stiffness of the vertical elements and the diaphragm. The design shall provide for the torsional moment M_t resulting from the location of the building masses plus the torsional moments M_{ta} caused by an assumed displacement of the mass each way from its actual location by a distance equal to 5 percent of the dimension of the building perpendicular to the direction of the applied forces."

   The details of how to give "consideration to the relative stiffness of the vertical elements and the diaphragm" have not been defined. The subject is recommended for consideration as part of Phase II.

In Reference 28, page 2-5, a discussion was presented with describes the methods and considerations frequently used in designing diaphragms. As part of this discussion it is suggested that for the design of new buildings torsion be resisted by distributing stiffness of the vertical element and its distance from the center of rigidity. It is suggested that the more flexible diaphragms not be designed to resist torsional shears at all. While this can be easily handled in the design of a new building, the evaluation of the earthquake hazard of an existing building should contain a reasonable assessment of all capabilities of the building. Thus the torsional capability of flexible diaphragms should be considered along with its behavior to transfer shear by flexure. Of course, the deformations and potential hazards derived therefrom should not be ignored. Analysis and study of torsional considerations is recommended to be part of Phase II.

4. Methods of Analysis of Multi-Pier Walls for In-Plane Forces. It has been customary to analyze such walls by determining the relative stiffness of the various piers and other wall elements by the application of a horizontal load at the diaphragm level, computing the shear and flexural deflection and then using the inverse of these deflections as a measure of relative stiffness. Examples of this[22,25,27,31] method can be found in several references. Many engineering offices have made design aid charts based on h/d values to assist in such calculations.

This method assumes that all elements of the shear wall remain elastic and have infinitely rigid spandrels and grade beams. It would be representative of the design of buildings where a maximum credible earthquake can be resisted without exceeding the elastic limit. However, with existing buildings, depending on occupancy or importance, the acceptable earthquake hazard should be determined by life safety rather than material damage. Some damage therefore may be acceptable depending on the political and economic conditions affecting a particular community or a particular owner.

As soon as one element of a shear wall cracks or sustains damage, the system is no longer completely elastic and the earthquake response is non-linear. The cracking of one pier, for instance, will cause a redistribution of the path of resistance to lateral loads. If adequately tied together at diaphragm and foundation levels, the vertical wall elements can be assumed to have equal, in-plane, lateral movement or deformation.

No generally accepted method for rational design recognizing the non-linear response of such walls has been found. It is recommended that part of the Phase II study be an effort to develop such an analysis. Until such research has been done, assumptions must be made that may or may not be appropriate. The progressive damage to the wall elements must be envisioned as well as where such damage will stop, if it does stop. For instance, if there are a series of different size piers in a shear wall, elastic computations usually indicate that the stiffest pier will have the highest unit stresses and be the first one damaged. Let us assume that this pier develops diagonal cracks. It has lost stiffness, but how much? What is its relative stiffness as a cracked section? Tests on individual piers have usually been stopped shortly after the load deflection curve begins to drop. Some tests have been made on multiple pier shear walls. A recent report described tests which were made on pairs of equal size piers[40]. Sixteen of the seventeen tests were made on reinforced concrete block masonry. Only one test was made on unreinforced masonry. Because the two piers were of similar dimension in each test, the results are not indicative of the redistribution of forces where piers vary in stiffness.

Design Assumptions. Prior to formulation of a rational design procedure, the following assumptions may be made to analyze multi-pier unreinforced shear walls deforming into the non-linear range:

A. If all the piers are well tied together at top and bottom, the sum of all pier widths in a wall could be used as a basis of analysis. However, if the height of piers vary significantly, then another factor may be considered. Consideration also might be given if the boundary conditions or the degree of fixity at top and bottom vary, thus affecting the distribution of forces after initial cracking. This method is similar to the requirements of SEAOC 75. [36].

B. It can be assumed that the stiffest pier will crack first and lose stiffness until it is only as stiff as the next stiffest pier. This process would reiterate until all piers are equally stiff. This assumption results in each pier resisting the same total force.

Modes of Failure. Tests on masonry piers have shown four modes of failure:

1. Diagonal tension.
2. Combination of shear and flexural tension through masonry units.
3. Compressive flexural crushing.
4. Shear along bed and/or head joints.

The type of failure varies with type of concurrent loading, reinforcement, size and shape, and boundary conditions.

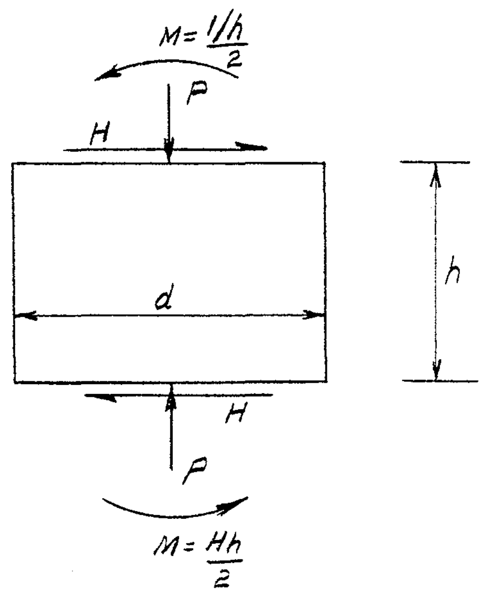
Effect of Lintel Over Openings. Many unreinforced brick masonry buildings have steel lintels to support loads over windows. These have about an 8-inch bearing at each end with no anchorage except embedment in masonry. The masonry surrounding steel lintels at the bearings may or may not have all voids filled. No lateral tests on unreinforced masonry elements containing such lintels have been found except for observations following actual earthquakes where considerable distress was found at these locations.

Arched rows of brick and reinforced concrete lintels have also been extensively used over openings in brick masonry walls. To the best of our knowledge there have been no tests on this type of construction. Where concrete floors or roofs occur, reinforced concrete lintels are usually found.

Effect of Vertical Loads. Tests that have been made on reinforced masonry show the variations in masonry in-plane shear capacity when vertical load is applied.[31] Vertical load will, of course, reduce vertical flexural tension and increase frictional shear strengths. However, earth movements may include a vertical acceleration. This has been recorded in some instances as high as 100% of the horizontal acceleration. Some codes have permitted 75% of dead load to be considered coincident with lateral forces. ATC-3 [6] has assumed 20% increase or decrease of dead load for this condition. At the present state-of-the-art, these assumptions are probably as good as can be found. No further study of this effect will be made during Phase II.

5. Methods of Pier Strength Determination. There are two methods for determining the strength of an unreinforced masonry pier to resist an imposed shear force. These are called "Uncracked Method" and "Cracked Method".

A. Uncracked Method.



Symbols:

A_w = Horizontal corss sectional area of wall pier = dt_w.

d = In-plane length of wall pier.

h = Height of wall pier.

H = Total permissible horizontal shear strength of wall pier.

F_{cw} = Permissible combined unit flexural and axial compression on wall pier.

F_{tw} = Permissible unit flexural tension on wall pier.

F_{vw} = Permissible unit shear on wall pier.

M = Moment on top and bottom of wall pier resulting from application of force $H = Hh/2$.

P = Axial load on wall pier concurrent with application of maximum horizontal shear on pier. Assume to be 80% of tributary dead load supported by wall pier.

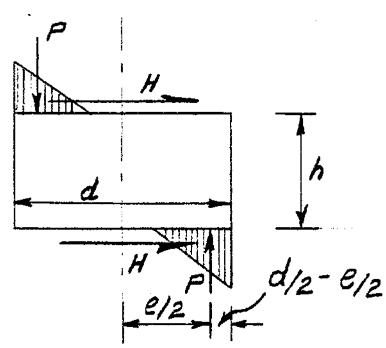
t_w = Thickness of wall pier.

This method assumes that the shear strength is determined by either the limiting horizontal shear stress, F_{vw}, or by the flexural tensile stress, F_{tw}, permitted on the masonry. The equations for determining the shear strength of the masonry pier are

$$H = F_{vw} A_w \text{ (shear)}$$

$$H = \frac{d}{3h} [F_{tw} A_w + P] \quad \text{(flexure)}$$

B. Cracked Method.



e = Eccentricity of P between point of application and point of reaction = Hh/P.

$$F_{cw} = \frac{2P}{3(d/2 - e/2)t_w}$$

This method assumes that the shear strength is determined by the limiting horizontal shear stress, F_{vw}, or by the flexural compression F_{cw}, permitted on the masonry. Also, no tension can exist on the masonry. The equations for determining the shear strength of the masonry pier are:

$$H = F_{vw} A_w \quad \text{(shear)}$$

$$H = \frac{Pd}{h}\left(1 - \frac{4P}{3F_{cw}A_w}\right) \quad \text{(flexure)}$$

A comparison of the two methods shows that both indicate the same shear capacity. The Uncracked Method yields a higher flexural capacity for stubby piers $\left(\frac{S_w h}{d} < 1\right)$ whereas the Cracked Method indicates a higher flexural capacity for the more more slender piers $\frac{S_w h}{d} > 1$.

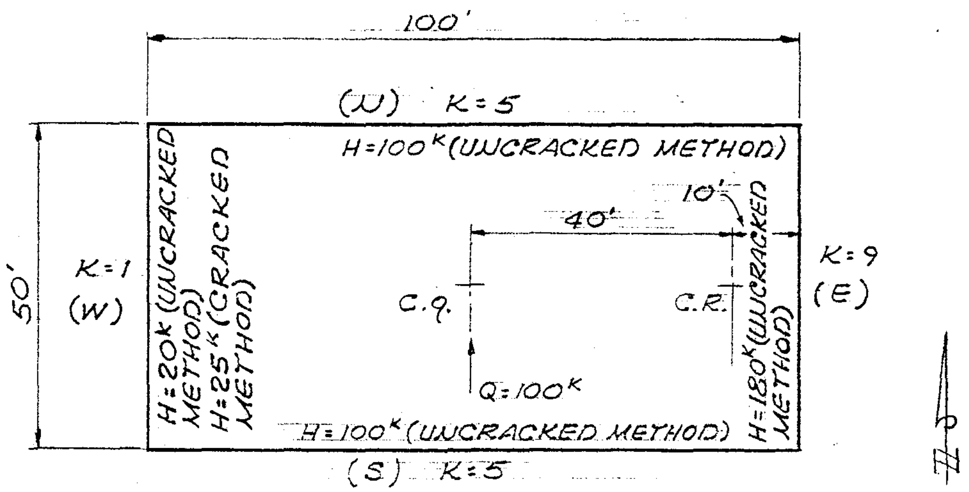
The Uncracked Method implies elastic response with the limit on the wall reached when any pier receives its limiting load, H, based on a distribution of wall load to the individual piers in proportion to their relative elastic regidities.

The Cracked Method implies a non-linear response and the shear strength of any wall, H_w, can be determined by the sum of H_i for all the piers in a wall, $\sum_{i=1}^{n} H_i$. In analysing a building with a flexible diaphragm, each wall is expected to be capable of resisting a tributary amount of the shears from the diaphragms. Thus if all H_w are greater than the sum of the tributary diaphragm shears plus the shear contributed by the wall mass, the building is adequate for in-plane shears.

When the floors and roof can be assumed to provide a rigid diaphragm, a simplified method of analysis will be needed to reflect the torsional characteristics of non-linear response. This might involve the assignment of a stress-strain curve to wall pier and/or a limiting deflection under design loads. It is recommended that this be part of the studies carried out under Phase II.

One possible design method would be to assume that walls overstressed when analyzed by the Uncracked Method may use the wall strength indicated by the Cracked Method if the stiffness of the wall is assumed to be 1/2 the elastic stiffness. The effect on design is indicated by the hypothetical example on Page 5-16. The uncertainties are the basic assumption of the elastic stiffness, the elastic in-plane limit strength of unreinforced masonry, and the capability of the masonry to deform to twice the elastic limit deflection without major reduction in shear strength. The studies under Phase II may clarify the choice of this criterion.

In the case of walls infilled into a building frame, it can be assumed that the frame provides restraint on the wall so that the force "H" would be determined by shear. The frame members are required to be verified by the forces induced in the frame after cracking.[24] The shear strength of the columns, V_c, should be equal to $\frac{2M_p}{3/4h_f} = \frac{8M_p}{h_f}$. The wall strength would be



ELASTIC DISTRIBUTION OF FORCES:

$$C.R. = \frac{1 \times 100}{10} = 10'$$

N.-S SEIS	K	d^2	Kd^2	$\times \frac{M_T}{\Sigma K d^2}$	$\div d$	$S_{N\text{-}S}$	$S_{E\text{-}W}$	K	$K+S$	DESIGN	REMARKS
E	9	100	900	.222	10	-20		90	70	90	
W	1	8100	8100		90	+20		10	30	30	30 > 20(N.G.)
N	5	900	4500		30	+33.3		0			
S	5	900	4500		30	-33.3		0			
			18,000								

$M_T = 100 \times 40 = 4000$

$$\frac{M_T}{\Sigma K d^2} = \frac{4000}{18{,}000} = .222$$

FOR INELASTIC DISTRIBUTION OF FORCES:

ASSUME WEST WALL $K = 0.5$, $C.R. = \frac{0.5 \times 100}{9.5} = 5.26'$

N.-S. SEIS	K	d^2	Kd^2	$\times \frac{M_T}{\Sigma K d^2}$	$\div d$	$S_{N\text{-}S}$	$S_{E\text{-}W}$	K	$K+S$	DESIGN	REMARKS
E	9	27.7	249	.326	5.26	-15.4		94.7	79.3	94.7	
W	0.5	8976	4487	.326	94.74	+15.4		5.3	20.7	20.7	20.7 < 25(OK)
N	5	900	4500	.326	30	+48.8					
S	5	900	4500	.326	30	+48.8					
			13,736								

$M_T = 100 \times 44.74 = 447.4'^K$

$$\frac{M_T}{\Sigma K d^2} = .326$$

HYPOTHETICAL
DESIGN EXAMPLE

$\sum_{i=1}^{n} V_{i\ col}$. If the existing column members do not provide the required shear strength, retrofit methods should be used.

e. Analyses of Building Component Parts and Building Contents.

Individual component parts of a building and building contents respond to the earthquake motions that are imposed on them at their interface with the building. Current practice is to design these elements using a force determined by multiplying the element weight by an appropriate factor based on previous experience.

The current proposals to ICBO[41] contain the following provisions for design of walls for normal loads: A basis force formula of $F_p = ZIC_p W_p$ in which I is 1.5 for elements of a life safety system. The factor Z is the zone factor using the map provided in UBC 76.[16] C_p comes from a table which specifies 0.3 and 0.8 factors for walls supported top and bottom and cantilever parapets, respectively. A footnote modifies C_p to 0.2 for walls supported at the ground level. These force levels are to be compared with working stress resistance capacities.

One set of design provisions (ATC-3[6]) have a much more sophisticated set of design provisions in which not only the type of component is considered but also are the location within the building, the seismic hazard exposure, and the desired performance level.

The ATC-3 provisions have a basic force formula of $F_p = A_v C_c P a_c a_x W_c$ where A_v is a factor representing the Effective Peak Velocity (similar to a zone factor), P is a performance factor varying from 0.5 to 1.5 depending on the seismic hazard exposure (similar to an occupancy factor), A_c is a factor dependent on the method of fastening to supports (for a wall $A_c = 1$), a_x is an amplification factor depending on the height of the element in comparison to the building height ($a_x = 1 + h_x/h_n$). C_c is the seismic coefficient which for exterior walls and public corridor partitions is 0.9 and for private corridors is 0.6. For stairs, elevators and exitways, C_c is 1.5. The forces thus obtained are to be compared with a resistance equal to the elastic strength of the element.

It is not recommended to delve into this subject in Phase II except as it affects the determination of the appropriate design force for assessing the seismic hazard of a wall resulting from earthquake motions perpendicular to the wall.

The response to earthquake motions normal to the plane of the wall can be resisted by arching action[24] or can be analyzed as a combined stress condition.[30,31]

f. Other Analyses. The deflection analyses required for new buildings on the lateral force resisting system are usually not critical on existing buildings with masonry walls. Some buildings with a large percentage of open area may require retrofit for both strength and stiffness. Deflection analyses are required when the following items are critical.

1. P-Δ effects on the lateral force resisting system usually become important only on tall framed structures using moment resisting frames. Seldom will this be an important analytical item on existing structures having masonry walls.

2. Relative deflection of frame in comparison with life-safety systems in the building. This may occasionally become important in the retrofit design of exitways and stairs. Elevators also are susceptible to damage if deflections become excessive, but elevator systems may not be considered a life-safety system. Thus provisions for operability of elevators may not be chosen for mandatory retrofit. Vertical piping could be affected depending on the support system and in some instances may be classified as a life-safety system.

3. All current design and analysis methods assume that some non-linear response to major earthquake motions will occur. The verification of vertical load stability of members of the vertical load system that are not part of the lateral force resisting system is important in existing structures as it is in new structures. Many existing masonry buildings are obviously stiff enough that intricate deflection analyses are not needed but an approximation should be made to verify the vertical load stability.

4. Deflection analyses are particularly relevant if adjoining buildings have floor or roof systems at different levels and insufficient separation is maintained to inhibit structural collapses due to one building impacting the other.

g. Critical Element Stress Ratio. The stress ratio on a critical element can be determined by dividing the maximum stress determined from the analysis by the permissible stress on the element. These ratios should be tabulated so that an overall evaluation of the seismic hazard of the lateral force resisting system can be made. From the tabulation of these stress ratios, decisions can be made on modification of critical element designations and methods of hazard reduction if required.

It is possible that more than one method of hazard reduction warrants consideration. For instance, inadequate masonry shear walls might be repaired, replaced or strengthened. Or, additional shear walls may be introduced. Inadequate horizontal diaphragms may be strengthened or the diaphragm span may be reduced by introducing additional intermediate shear walls. Tabulation of the critical element stress ratios can also assist in these determinations.

In the initial selection of a method of hazard reduction, the cost of different methods should be roughly determined. Such rough preliminary estimates may give sufficient data to enable a decision to be made as to whether or not to proceed with further analysis. Such preliminary cost estimates must consider the function of the building areas involved and the interruption of occupancy. For instance, the addition of additional shear walls should not reduce the room sizes materially if the present function of the room is to be maintained. Required exits should not be blocked. Where the ground floor is leased to retail or other commercial enterprises, replacing show window areas with shear walls may result in cancellation of leases with loss of substantial income.

If this rough evaluation indicates that strengthening the building may be economically feasible, a further and more comprehensive analysis should be made. This analysis should be sufficiently complete to permit a reasonably accurate contractor's estimate. However, the preliminary rough analysis may indicate that strengthening will not be economically feasible. In such cases, the program for structural strengthening may be rejected.

CHAPTER 6. RETROFIT METHODS

a. Masonry Walls - Strengthening.

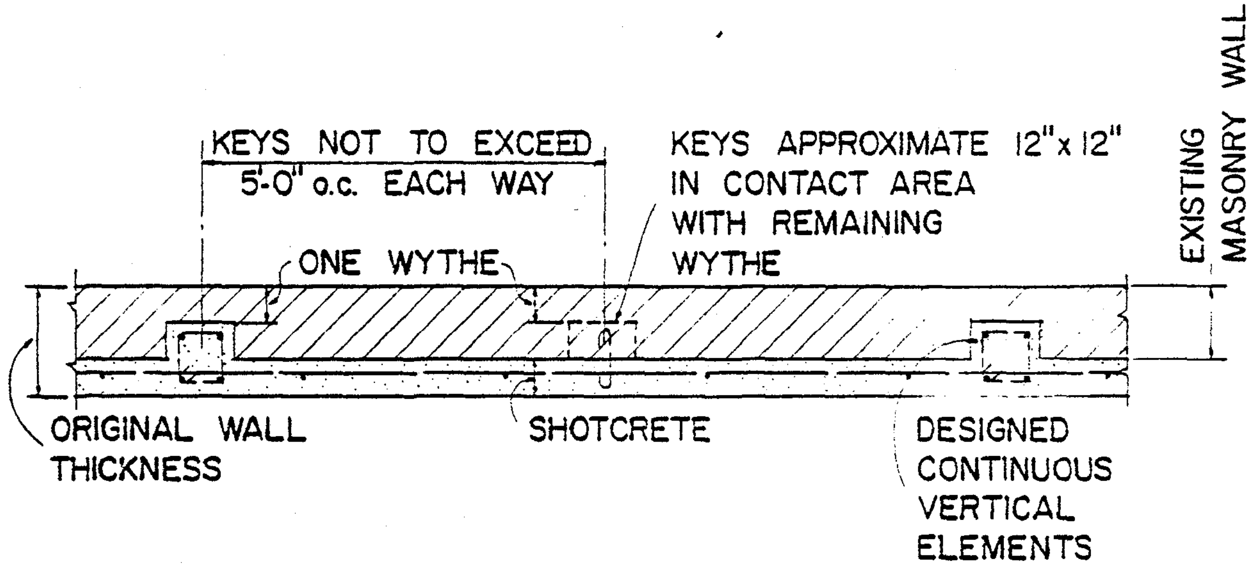
1. The most commonly used method for rehabilitating unreinforced brick masonry walls has been developed primarily in California for structures in highly seismic locations. This consists of stripping off one wythe of brick, either the inside or outside, and replacing this with reinforced shotcrete (gunite). Vertical chases are cut through to contact the farthest away wythe. These are also filled with reinforced shotcrete. They are spaced at approved intervals and near the sides of openings to provide tension reinforcing. In addition, "buttons" are placed at prescribed intervals to contact the farthest away wythe. Small reinforcing bar ties (button hooks) are placed in these buttons to help insure that the unreinforced elements do not fall out during high intensity earthquakes. This method is shown on Figures 6.1, 2, 3, & 4. This type of rehabilitation has been generally approved by the State of California agency enforcing school construction and by most California building officials.

   Adequate anchorage of walls to floor and roof diaphragms is necessary to resist both out-of-plane forces and to transfer in-plane shears from diaphragms. Reinforcement of the shotcrete is required to be properly anchored and lapped. This method has the advantage that very little mass is added to the building so the size of original footings is not affected by vertical dead loads. It is a relatively expensive method of retrofit.

   The strength of the strengthened wall is determined considering only the reinforced shotcrete element neglecting the masonry except for some bracing effect on the shotcrete shell. This method is primarily used only for retrofit of brick masonry.

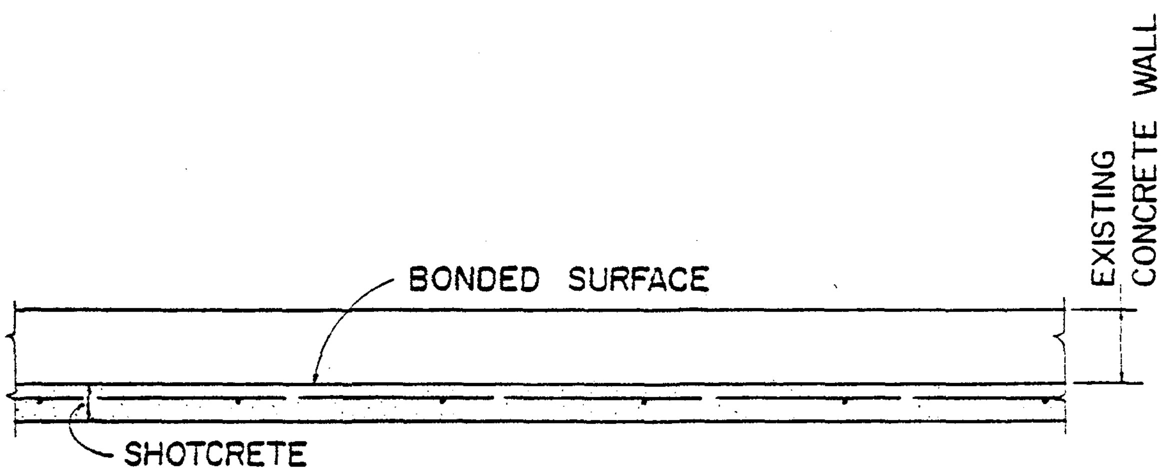
2. Another method sometimes used is replacement of unreinforced brick or block elements with elements of reinforced masonry or concrete. This may involve considerable temporary shoring in the case of load bearing walls. The accessibility for shoring will affect costs. Proper anchorage to floors, roof and foundations is required. In some cases, walls can be replaced in alternate sections, thus reducing the amount of shoring required at any one time.

3. A reinforced plaster membrane method of rehabilitation of unreinforced brick masonry was developed in 1959. This consists of a mesh reinforced membrane on each side. The system was tentatively accepted by the Schoolhouse Section of the Division of Architecture, State of California. The approval was subject to certain requirements as to the physical properties of the masonry. The reinforced membranes were either cement plaster or shotcrete. Bending normal to the plane of the wall is resisted by the reinforcing steel mesh acting in tension. Minimum shear values are taken by the masonry. Extra reinforcement is added at the edges of openings for resisting lateral forces in the plane of the wall. Ties through the wall are required at intervals.



BRICK MASONRY

(a)

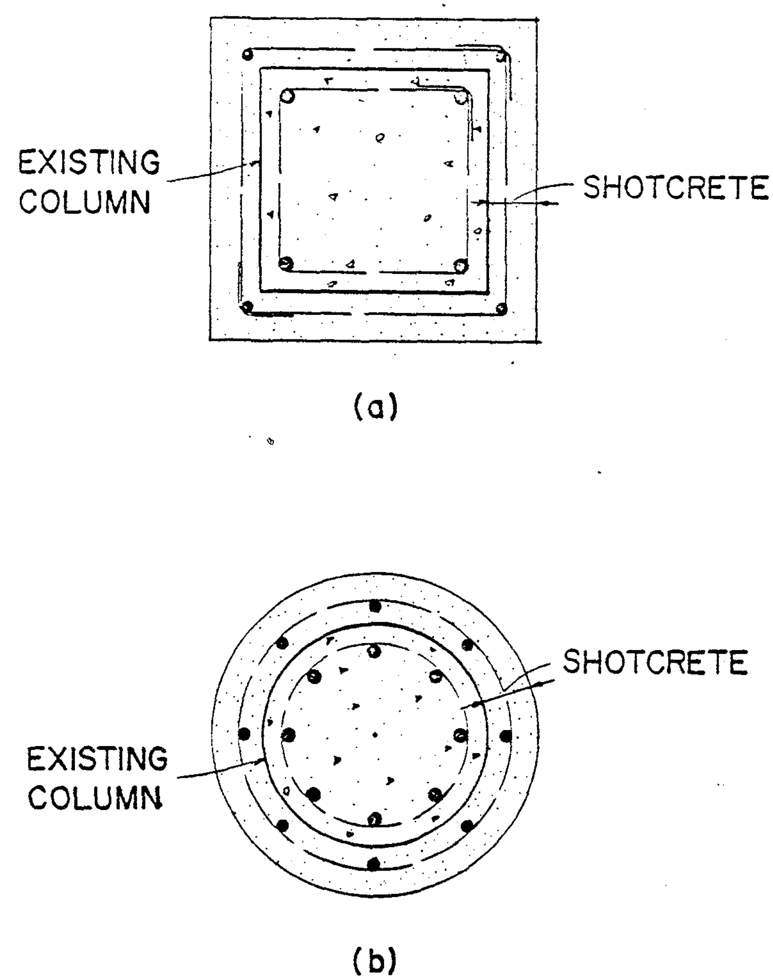


CONCRETE

(b)

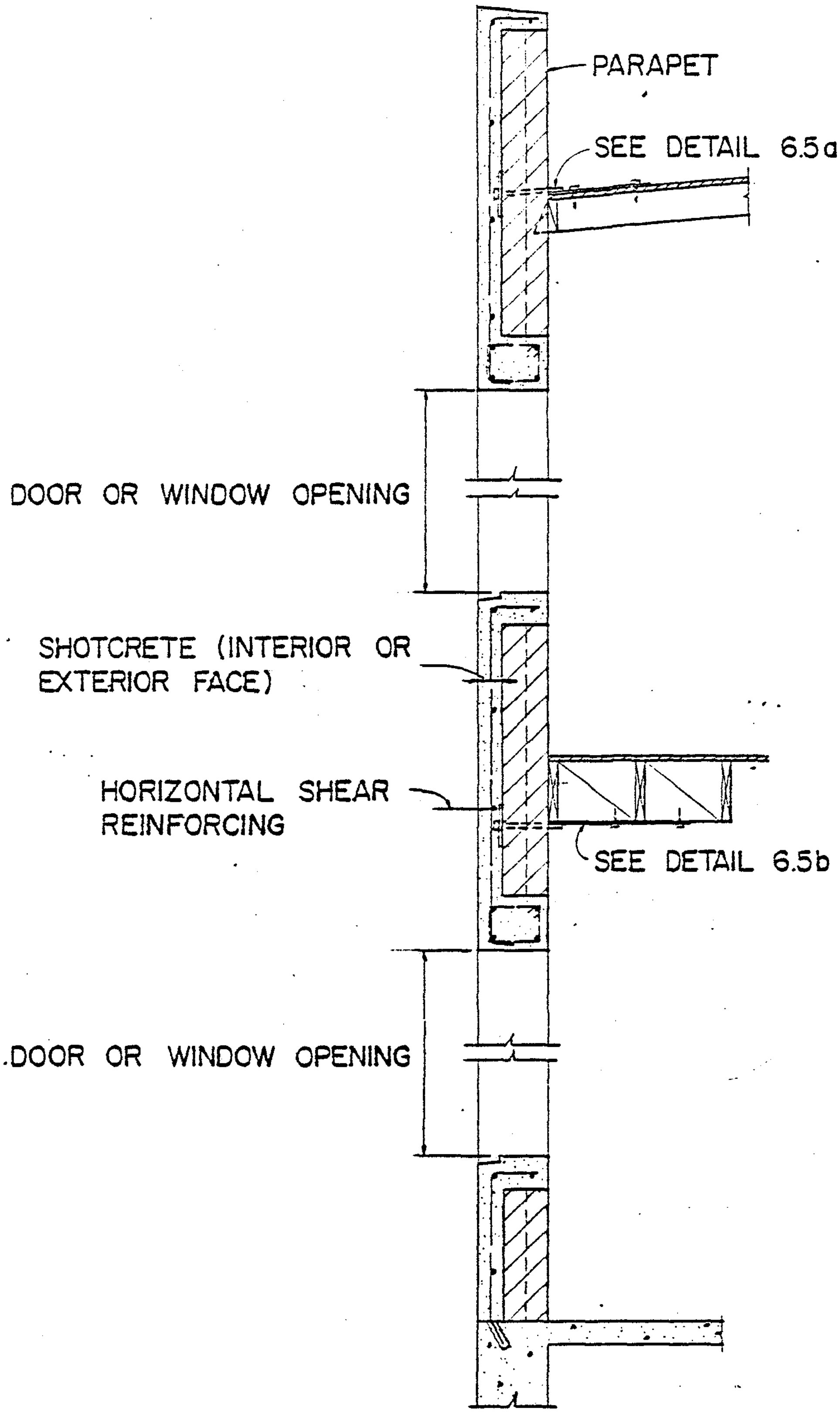
TYPICAL WALL STRENGTHENING

Fig. 6.1



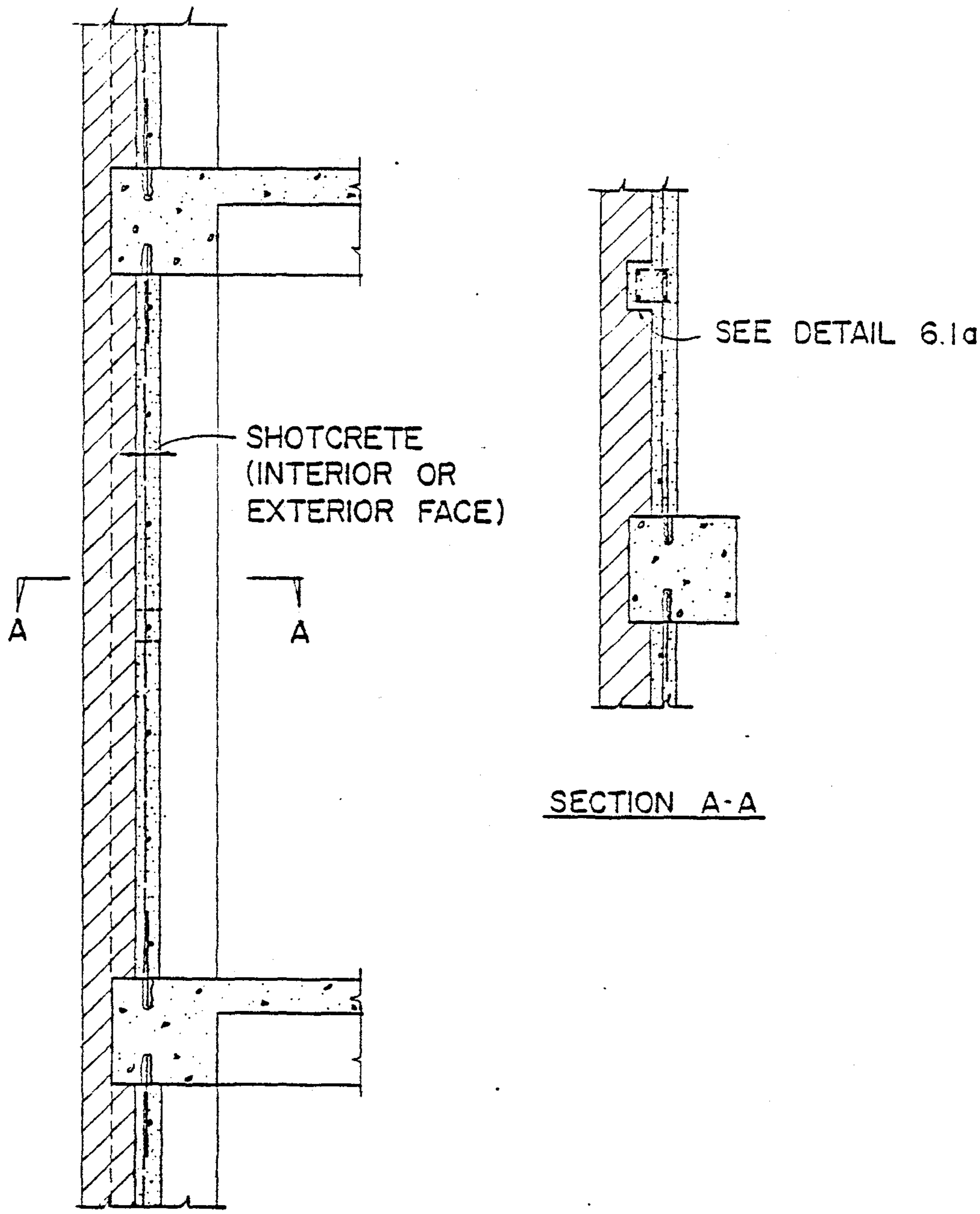
STRENGTHENING OF EXISTING CONCRETE COLUMNS & BEAMS

Fig. 6.2



TYPICAL STRENGTHING OF MASONRY WALLS

Fig. 6.3



TYPICAL STRENGTHENING OF BRICK MASONRY WALLS

Fig. 6.4

Anchorage of walls so modified to the floors and roof diaphragms were a problem. Some limited tests were made to develop working stresses. These tests were made under the auspices of the Los Angeles Board of Education. It was hoped this method of rehabilitation would prove more economical than the method described in Subsection 1. However, such anticipated savings were not realized. For all practical purposes this method has been discontinued. Nevertheless, further research in the use of this general method of rehabilitation is warranted. The method might prove economical in areas of less seismicity. However, it is recommended that only feasible anchorage details be studied in Phase II of this project. The method can also be used with concrete block masonry or with clay tile. Some weight is added by the thickness of the added membrane and thus the adequacy of foundations should be checked.

In certain cases, where unreinforced masonry is used as non-bearing partitions and are so positioned or isolated that they will not participate as shear walls for in-plane forces, plaster or shotcrete with mesh has been used to "basket" a wall with the mesh acting as reinforcement for forces normal to the plane of the wall. This requires lateral supports at top and bottom or at sides in addition to a separation detail. This system may be desirable when such unreinforced masonry walls are adjacent to exits or exit corridors.

The in-plane shear in this type of rehabilitation should be assumed to be all taken by the membrane. Further testing might show this assumpto be too conservative, depending on the relative thickness of the membrane to the masonry. To resist out-of-plane forces shear is assumed to be taken by the masonry. This requires a knowledge of the physical properties of the existing masonry, particularly its shear strength. One school building was rehabilitated by the Los Angeles School District by this method. In 1959, a circular[42] was issued putting additional limitations on this method in school construction in California. One was to limit the use of this method to two-story buildings.

4. Post-tensioning has been done by drilling vertical holes through a masonry wall from top to bottom, installing post-tensioning bars and grouting the holes. The bars are anchored to the footings. This method increases the shear value of the masonry because of the added compression from the post-tensioning. This tension can be varied to suit. The post-tensioning will resist flexural stresses in the wall also. For existing buildings the method has limitations in height of wall and access at top of wall for the drilling equipment. Low, one-story buildings are obviously much better adapted to this method than multi-story buildings.

5. Epoxy injection under pressure has been used to repair cracks in masonry walls. This was tried in several cases in the Los Angeles area after the San Fernando Earthquake of 1971. Where poor masonry was encountered, the epoxy filled a lot of voids in the inner wythes as well as the cracks. It took a lot of expensive epoxy and, in such cases, this method was not economical.

   Filling the voids in the inner wythes would no doubt increase the physical properties of the masonry. It would be difficult to determine, however, whether all voids were filled. Where this method was used, the intent was to repair cracks and not to increase physical properties.

In the case of unreinforced and ungrouted concrete block or other masonry with built-in voids, the volume of epoxy would be excessive and the method not economical. Cement grout would be less expensive.

6. Foam epoxy has been used to fill voids in masonry walls and behind veneers. Webbed terra cotta veneer has been bonded to the wall proper by this method. Foam epoxy can develop variable strengths depending on the mix. [43]

   Where the exterior facade has an historical meaning and it is desirable to maintain the appearance, this method has considerable merit. Suitable ties can be installed to the back-up masonry. The back-up masonry can be strengthened as required by Subsection 1, for instance, working from the inside. It is noted that a recent NSF contract has been awarded to Scientific Service, Inc., for "Use of Structural Foams to Improve Earthquake Resistance in Buildings". This contract may provide further inforon this method of retrofit.

7. A fiberglass membrane can be used on each side of unreinforced masonry. This can be applied similar to plaster. At least one racking test using this method has been made but the results have not been published. The test did show that a wall so reinforced would resist a much greater lateral in-plane force than the original uncracked masonry with which it was compared. For the racking test, additional steel reinforcement would probably be needed to take tension overturning. A large core or a prism tested diagonally would give values for diagonal tension and shear. [44]

   Primarily, unreinforced masonry is lacking in tension and shear qualities. Methods of retrofit must supply these.

8. If an existing masonry wall pier has been determined to be adequate to resist horizontal shear and the resultant of the vertical and lateral loads falls within the middle half of the wall pier, no tension reinforcement would be required. If the resultant falls outside the middle half then tension reinforcement would be required. This could be provided by properly developed vertical rebars at or near the face of pier. Where anchored in existing concrete, the use of epoxy in a hole drilled to receive the rebar has been found by tests to be very effective in a relatively short embedment length. It would not be necessary to provide a shotcrete shell as in Subsection 1 or the reinforcement of mesh and plaster as in Subsection 3. This assumes also that the wall is satisfactory to resist lateral foaces normal to the wall and properly anchored at supports. This method should be less costly than those described in Subsections 1, 2 or 3. It may be of particular merit in zones of low seismicity. We have no knowledge of tests on this limited type of retrofit. It is recommended that consideration of further studies of this method be given in Phase II of this project.

b. Wall Anchorage to Diaphragms is one of the most critical items to be considered in the response of buildings to earthquake forces. Anchorage should be adequate to transfer lateral forces to the vertical resisting elements from the diaphragm. The anchorage to the diaphragm and the anchorage to the wall are equally important. This anchorage should be adequate both in shear parallel to the plane of the wall and in tension or compression perpendicular to the wall. Some typical anchor details are shown on Figures 6.5 and 6.6.

As discussed elsewhere in this document, many older masonry wall buildings, not designed to resist earthquake forces, use self-releasing tee head anchors with the tee heads embedded in the walls. This self-releasing type of anchor was considered to be good in case of fire in that if the floor burned it would not pull the wall in.

Also as previously discussed, another type of anchorage in use in areas subject to high seismic risk was to rest wood joists or rafters on a wood ledger with bolts to the wall designed to transmit in-plane forces to the wall. The wood sheathing was nailed to the ledger or blocking to also deliver these forces. There were many failures of this type of anchorage in the 1971 San Fernando Earthquake.[9]

In considering the type of retrofit anchorage detail to be used, consideration should be given to supplying a tension chord to the diaphragm where required. A combination detail may be feasible.

Where joists or rafters are "firecut" into a multi-wythe brick wall, it may sometimes be advisable to cut off the projecting wood, particularly if there is any evidence of rot. A new seat such as a steel angle or wood ledger is required in such cases.

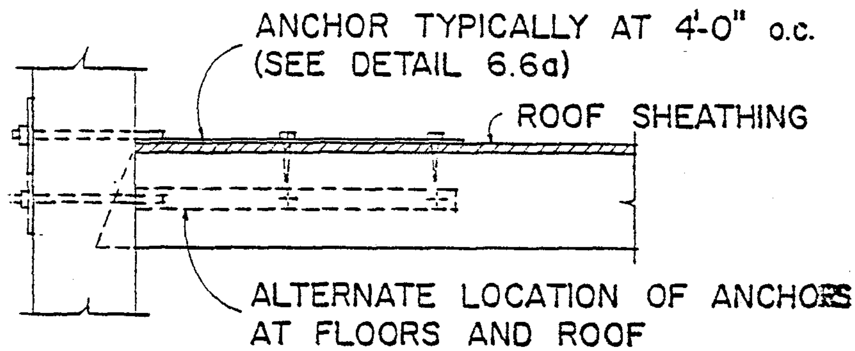
Values of bolts in unreinforced masonry as given in codes usually have a high margin of safety to test values because of the uncertainty of workmanship and characteristics of the masonry. Where large numbers of anchors are involved in a particular type and quality of masonry, in-place tests on existing bolts would provide added information for establishing values. New bolts or other types of anchors could be installed and tested for both shear and tension. This subject is recommended to be considered further in Phase II.

c. Diaphragms.

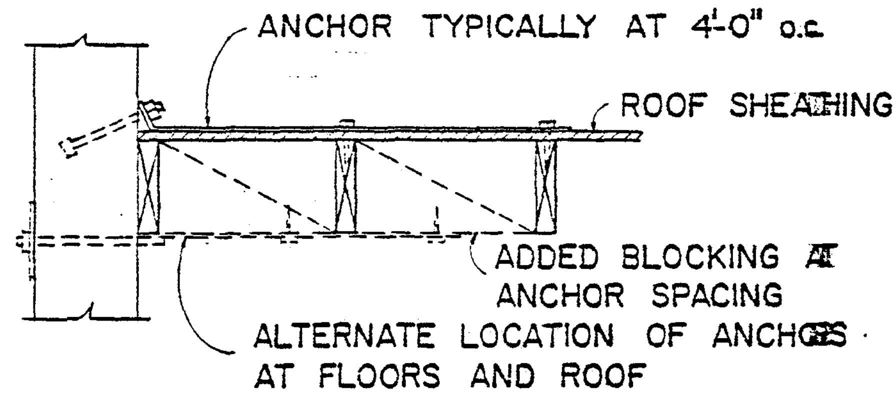
1. Concrete Slabs

A. Repairs. Where concrete floor or roof slabs have been severely damaged, the damaged portions should be replaced. Cracks in slabs may be repaired by epoxy injection.

B. Strengthening. Concrete floor and roof systems acting as diaphragms may require strengthening to properly transmit horizontal shears to the vertical resisting elements. Usually adequate dowels or other connections have not been provided to the vertical elements. To correct this, dowels are added. Such dowels must have adequate embedment at each end to be effective.



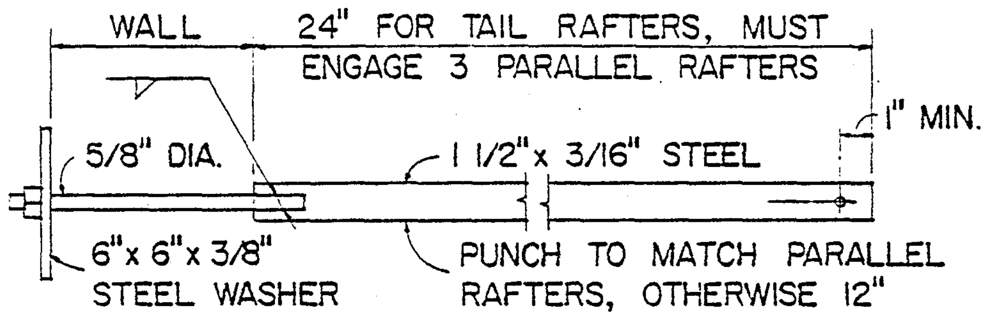
(a)



(b)

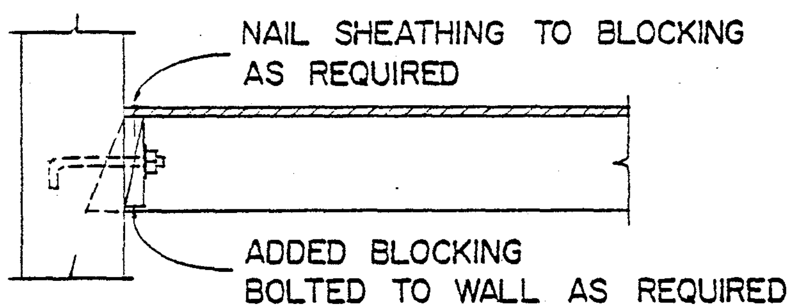
CONCRETE OR MASONRY WALL ANCHORS TO WOOD DIAPHRAGM

Fig. 6.5



ANCHOR DETAIL

(a)



WALL SHEAR TRANSFER

(b)

Fig. 6.6

Where diaphragm shear capacity or chord reinforcement is deficient, it is sometimes feasible to reduce requirements by adding shear walls to shorten the diaphragm span. The relative stiffness of new and existing shear walls and diaphragms must, of course, be considered in the re-analysis.

2. Wood Floors and Roofs. The sheathing of wood floors or roofs is often adequate except for the nailing. Plywood blocking, if not previously used, can be inserted in critical areas. In any case, a renailing program may be the best method of strengthening.

   Many older structures (those most likely to sustain earthquake damage) will be found to have floors with sheathing board laid normal to joists. Since the allowable shears for this type of diaphragms are based on nail couple capacities, they are usually inadequate, even though thoroughly nailed. This type of diaphragm may be strengthened by installing a plywood or a diagonal sheathed overlay designed to transmit the entire shear. Shear transfer from the overlay to the vertical resisting system elements must be provided for.

   Horizontal steel bracing systems used as the primary lateral force system with the wood sheathing carrying only vertical loads are frequently used. It may be strengthened, if inadequate, by a new plywood diaphragm overlay designed for the total shear. When overlay plywood sheathing is installed, the plywood joints should be staggered with the joints of the existing sheathing. Pre-subdrilling of nail holes, while costly, may often be advisable to minimize splitting of very dry existing members.

   When wood diaphragms are repaired or strengthened, adequate shear transfer may be provided at masonry or concrete walls by the installation of self-drilling, shear resisting anchor and bolt assemblies installed through existing wood ledgers or blocking (see Fig. 6.6). Additional shear bolting may also be provided by drypacking or grouting bolts into oversized holes in the walls. Obviously, repair or strengthening may also be accomplished by replacement of sheathing.

3. Steel Deck. The diaphragm shear values of many cold formed steel decks have been established by tests. Most manufacturers have obtained ICBO Research Recommendations or other code approvals, which may be used for design or calculations perhaps made in conformance with Reference 28. Where it is necessary to increase the strength of an existing diaphragm, some modifications of connections are necessary. When the maximum connections allowed have already been used or when the gage of the deck is inadequate, either replacement of the deck can be done in the deficient areas or a horizontal steel bracing system may be installed.

   Where a metal deck diaphragm is supporting a structural concrete slab, the slab alone may be considered to act as the diaphragm and, if necessary, strengthened as described for cast-in-place slabs. Connections to chords, tie struts, and vertical resisting elements must also be provided. As an alternative, a horizontal steel bracing system may be installed.

4. Horizontal Bracing. To strengthen a horizontal bracing system, the members may be increased in size and the connections strengthened accordingly. Sometimes strengthening may be accomplished by installing additional bracing in other bays.

d. Foundation Settlement Retrofit. The remedial method chosen will usually depend on the supporting soils. Therefore a thorough evaluation of soil characteristics should be made.

1. Underpinning. Where sizes are inadequate, footings may be increased by underpinning or they may be removed and replaced. Incremental underpinning may eliminate or reduce shoring requirements. New footings (caissons or piles) may be installed on each side of an inadequate existing footing. Beams supported by the new footing would be installed to carry the load. The portion of the building which has settled should then be jacked into position. This results in a space between the new beams and the existing footing, wall, or columns. This space should be solidly filled with concrete grout or drypack.

   For strengthening of pile footings that are subject to settlement, soil stabilization as well as underpinning should be considered. Soil stabilization procedures may reduce the need for additional piles which may be difficult to install. If soil stabilization is not feasible, it may be necessary to install additional piles and a new pile cap. This involves temporarily supporting the existing load bearing vertical element and then jacking it into position. Space for adding new piles must be available to provide vertical and horizontal clearance for a pile driver, if driven piles are used or for a drill rig if drilled cast-in-place concrete piles are used. Before a determination is made on the method of strengthening of pile footings, the soundness of the existing piles should be established. Entirely new foundations may be required if significant damage has occurred.

2. Soil Stabilization. There are many methods of soil stabilization and compaction that are used. Some involve consolidation by vibration, preloading, blasting, etc., which tend to increase the settlement of the area around the foundations and therefore are not easily adaptable to the repair of existing foundations. Other methods, such as pressure grouting or intrusion grouting with cement grout or chemicals, do not settle the ground surface and these methods are frequently used to increase the bearing capacity under existing buildings.

   Soil stabilization techniques are dependent on the specific soil characteristics. The size of granules, the moisture content, and the chemistry of the soils is important. An analysis of the effectiveness of soil stabilization is necessary since some soil conditions are not adaptable to any of the usual techniques.

   Pressure grouting, in some cases, may be used to raise or level footings or floor slabs. When soil-cement grout is used, the method has sometimes been called "mudjacking". Pressure grouting can be done to depths of 50 feet or more with proper equipment.

Most applications of chemical grouting are made into partially saturated soils. However, there are some instances in which chemical grouting has been successfully used in dry, granular, or fractured soils. This method should be used only when the soil chemistry is appropriate.

e. Additional Shear Walls. Many buildings not only may be lacking in total shear wall capacity to resist lateral forces but may have shear walls so located that severe torsional problems are involved. This case is frequently found where a street front of a building is mostly glass show windows. In some cases new shear walls can be located near such street fronts and the torsional hazard reduced.

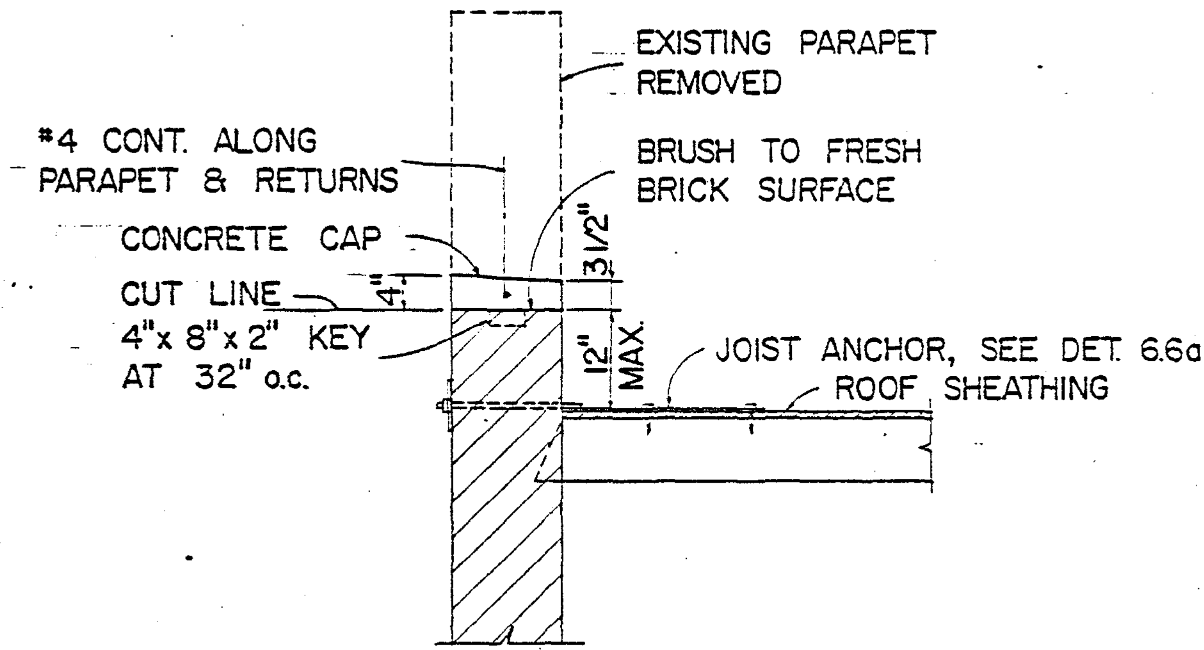
New footings will usually be required for new shear walls. Sometime vertical steel bracing can be used to avoid increasing dead load mass. Plywood sheathed wood shear walls, although less rigid than masonry walls may also be used in smaller buildings with wood floor and roof diaphragms.

Some building codes do permit and some do not permit the use of wood sheathed shear walls to resist lateral forces resulting from masonry masses. The basis for rejecting approval is that the plywood sheathed shear walls are not as stiff as the masonry. And yet all codes permit the use of plywood sheathed wood floor and roof diaphragm to resist such forces. This is not consistent. Where retrofit policy is to permit some damage but provide life safety, additional wood shear walls might be added to reduce requirement for the lateral forces on the existing masonry walls. Proper anchorage and overturning resistance would, of course, be necessary. Wood walls using plywood sheathing would be particularly useful for this purpose. Similarly, vertical steel braced frames can be added. These may be also less rigid than masonry walls in an uncracked condition, but after initial cracking they would be capable of carrying load. In some instances, existing walls can also be made acceptable to resist in-plane forces by adding new walls of masonry or concrete to the building.

f. Removal of Upper Stories. This is sometimes worthy of consideration. The mass of the building is reduced by this method so that the required capacity of the vertical resisting elements may be reduced. Consideration must be given, however, to the probable reduction in the natural period of vibration of the building which may cause the building to have a greater response to any given seismic motions.

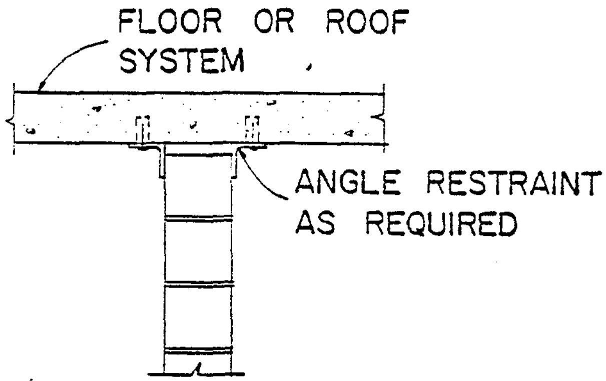
g. Parapet Walls. A typical detail for reducing the hazard from high masonry parapet walls is shown on Fig. 6.7.

h. Non-Bearing Partitions. Two typical details for bracing of masonry non-bearing partitions are shown on Fig. 6.8.

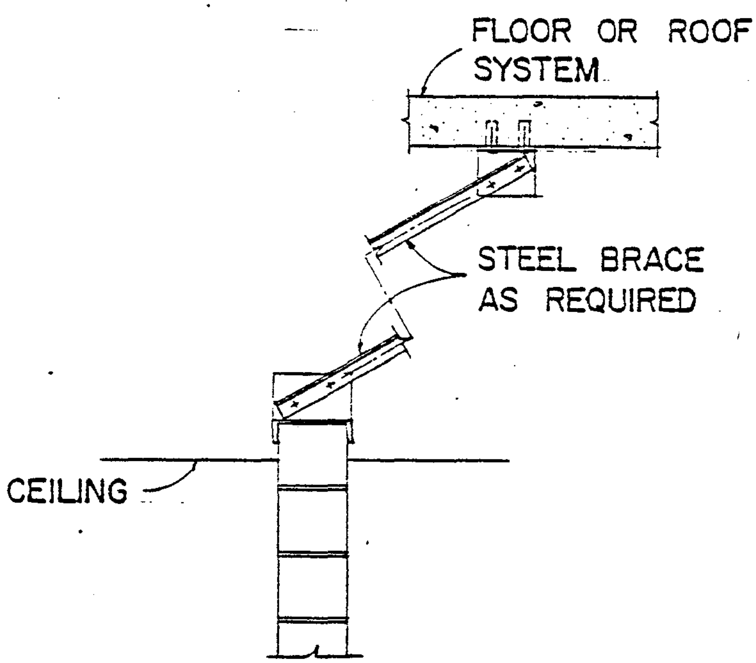


TYPICAL PARAPET CORRECTION DETAIL

Fig. 6.7



(a)



(b)

NON-BEARING PARTITION ANCHORAGE

Fig. 6.8

CHAPTER 7. ADEQUACY OF RETROFIT WORK

After an existing building has been retrofitted to some design standard, the overall building has become a new building in its response to an earthquake. An assessment of the adequacy of retrofit work based on the compliance with code design requirements by using the critical stress ratio may not give realistic values. It is recommended that part of the study to be conducted during Phase II of this project be an evaluation of various rating methodologies to determine their relevancy.

The adequacy of retrofit work has been found to be greatly affected by how well the retrofit work has been performed. In general, it can be said that the earthquake resistance of retrofit work can be greatly improved if the work is properly inspected by knowledgeable inspectors under the direction of a competent structural engineer. The costs of these inspections become a part of the overall cost of retrofit and should not be ignored in evaluation of retrofit measures.

CHAPTER 8. COST OF RETROFIT

The cost of retrofit does enter into the decision process of selecting acceptable design requirements. For instance, if a community had 5% or less of its buildings that could be classified as earthquake hazardous, a high level of retrofit could be called for without materially affecting the economic stature of the community. On the other hand, if a community had 70% or 80% of its buildings in an earthquake hazardous condition, then a serious economic impact would result if a high level of retrofit were to be required over a short period of time. Also, if the cost of retrofit becomes too large it would be more economical to raze the building and rebuild. This is counter to the current trend, which is to re-use, if possible, the existing inventory of buildings. The tendency, however, would be toward creation of additional urban sprawl and retention of hazardous areas without mitigating the hazard. The goal of this study, thus, is to create a simplified design methodology for general application to unreinforced masonry buildings, which would provide a minimum level of necessary retrofit at the lowest possible cost consistent with a reasonable level of earthquake hazard mitigation. Also, an easy modification within the methodology should be incorporated so that individual communities could create a higher or lower level of retrofit if it were desired by the community.

CHAPTER 9. DESIGN CRITERIA

The following design criteria are listed as a first draft. It is based primarily on the format and criteria found in the provisions of ATC-3.[6] Nomenclature and symbols are kept the same as found in ATC-3. Included in this draft by reference only are the two maps in ATC-3 showing A_a and A_v which are related to area seismicity. A_a is a coefficient related to the Effective Peak Acceleration and A_v is a coefficient related to Effective Peak Velocity. Discussion and background to these factors are not reproduced herein. It is recommended that these criteria be reviewed and evaluated after the studies and tests are completed in Phase II of this project.

PRELIMINARY DESIGN CRITERIA

a. Design Concept.

1. Load Effects. The strength of building components as described in Subsection C shall be compared with seismic effects prescribed in Subsection B. The decision of when retrofit is required to the lateral force resisting system would be based on whether components that have insufficient strength are critical to the structural stability of the building as a whole.

b. Design Loads.

1. Load Combinations. The effects on the building and its components due to gravity loads and specified earthquake forces shall be combined in the following manner:

$$U = 1.2Q_D + 1.0Q_L + 1.0Q_S \pm 1.0Q_E$$

$U = 0.8Q_D \pm 1.0Q_E$ except for partial penetration welded steel splices, unreinforced masonry, and other brittle materials, systems and connections for which $U = 0.5Q_D \pm 1.0Q_E$.

2. Local Modifications. Local evaluations of seismic intensity for retrofit design can modify the coefficients of Q_E in subsection 1.

3. Orthogonal Effects. When building components have significant load effects from seismic forces in each normal direction, the load effects from one direction should be combined with 0.3 of the load effects in the perpendicular direction.

4. Discontinuities in Strength of Seismic Resisting System. The analysis of a building shall consider the potential adverse effects when the ratio of the strength provided in any story to the strength required is significantly less than that ratio for the story immediately above and the strengths shall be adjusted to compensate for this effect.

5. Non-Redundant Systems. The analysis of a building shall consider the potentially adverse effect that the failure of a single member, connection, or component would have on the stability of the building and appropriate modifications shall be made to mitigate this effect.

6. Ties and Continuity. All parts of the building shall be interconnected and the connections shall be capable of transmitting the seismic force induced by the parts being connected. As a minimum, any smaller portion of the building shall be tied to the remainder of the building with elements having a minimum strength to resist $A_v/3$ times the weight of the smaller portion but not less than 5 percent of the portion's weight.

   As a minimum, a positive connection for resisting horizontal force shall be provided for each beam, girder, or truss to its support which shall have a minimum strength acting in the direction of the span of the member equal to 5 percent of the dead and live load reaction.

7. Concrete or Masonry Wall Anchorage. Concrete and masonry walls shall be anchored to the roof and all floors which provide lateral support for the wall. The anchorage shall provide a direct connection between the walls and the roof or floor construction. The connections shall be capable of resisting lateral forces of $1{,}000A_v$ (lbs) per lineal foot of wall. Walls shall be designed to resist bending between anchors where the anchor spacing exceeds 4 feet.

8. Collector Elements. Collector elements shall be provided which are capable of transferring the seismic forces originating in other portions of the building to the element providing the resistance to those forces.

9. Diaphragms. Diaphragms may be used to resist lateral forces in horizontal and vertical distributing and resisting elements, provided the deflection in the plane of the diaphragm, as determined by engineering analysis, does not exceed the permissible deflection of the attached elements. Permissible deflection shall be that deflection which will permit the attached element to maintain its structural integrity, even if cracked, under the individual loading and continue to support the prescribed loads without endangering the occupants of the building.

10. Bearing Walls. Exterior and interior bearing walls and their anchorage shall be designed for a force of A_vW_p normal to the flat surface with a minimum of $0.1W_p$. Inter-connection of dependent wall elements and connections to supporting framing systems shall have sufficient ductility to preclude fracture or brittle failure, or have sufficient strength to resist shrinkage, thermal changes, and differential foundation settlement when combined with seismic forces.

11. Load Path. A continuous load path, or paths, with adequate strength and stiffness, shall be provided which will transfer all forces from the point of application to the final point of resistance. The foundation shall be designed to resist the seismic forces transferred to it by the building. In the determination of the foundation design criteria, special recognition shall be given to the dynamic nature of the forces, the expected ground motions, and the design basis for strength and ductility of the structure.

12. Interaction Effects. The analysis shall consider the deformational effects on the structural system of rigid elements which are not part of the seismic resisting system. All separate portions not separated to inhibit damaging contact during the design earthquake shall be analyzed for the effect of that contact.

13. Deformational Compatibility. All structural elements not considered in the design to be part of the seismic resisting system shall be investigated and shown to be adequate for the vertical load-carrying capacity and the induced moments resulting from the total story design drifts Δ_s.

14. Base Shear. The building shall have a lateral force resisting system capable of resisting a force determined by the following formula:

$$V = C_s W$$

Where V = total seismic horizontal force at the base.

$$C_s = 2.5 \frac{A_a}{RD} \text{ except}$$

$$C_s = 2 \frac{A_a}{RD} \text{ when } A_a \geqslant 0.3 \text{ on Type } S_3 \text{ soils.}$$

S_3 soils is a profile with soft to medium-stiff clays and sands, characterized by 30 feet or more of soft to medium-stiff clays with or without intervening layers of sand or other cohesionless soils.

W = the total dead load of the building and applicable portions of other components including, but not limited to, the following:

1. Partitions and operating contents of permanent equipment.
2. For storage and warehouse usage, a minimum of 25 percent of the floor live load.
3. The effective snow load. [6]

R = The seismic response modification coefficient related to the building framing system employed as given in Table 9A.

D = Diaphragm response modification coefficient.

NOTE: The version of base shear determination does not contain a modifier based on the elastic fundamental period of the structure. It is recommended that this simplifying procedure be studied during Phase II and to see if period effects of diaphragms can be added to the table containing R, C_d, and D.

15. Vertical Distribution of Base Shear Force. The base shear shall be distributed over the height of the building in accordance to:

$$F_x = V C_{vx}$$

Where F_x = force assigned to Level x of the building.

TABLE 9A - FORCE AND DEFLECTION COEFFICIENTS FOR UNREINFORCED MASONRY WALL ANALYSIS

Type of Structural System	R	C_d	D
Bearing Wall System			
Building Frame System			
Moment Resisting Frame System			
Dual System			

NOTE: It is recommended that Types of Systems and Values for R and C_d be studied and selected during Phase II.

TABLE 9-B - ARCHITECTURAL, MECHANICAL, ELECTRICAL, AND STRUCTURAL MEMBER NOT PART OF SEISMIC RESISTING SYSTEM

Building Component	C_c
RECOMMENDED FOR COMPLETION DURING PHASE II	

$$C_{vx} = \frac{w_x h_x}{\sum_{i=1}^{n} w_i h_i}$$

16. Horizontal Distribution of Story Force. The shear and torsion in any horizontal plane shall be distributed to the various vertical elements of the seismic resisting system with consideration given to the relative stiffness of the vertical elements and the diaphragm.

    The design shall provide for the torsional moment M_t resulting from the location of the building masses plus the torsional moments M_{ta} caused by an assumed displacement of the mass each way from its actual location by a distance equal to 5 percent of the dimension of the building perpendicular to the direction of the applied forces.

17. Overturning. Every building shall be designed to resist overturning effects caused by the seismic forces determined in Subsection 15. At any level, the increment of overturning moment in the story under consideration shall be distributed to the various walls or frames in the same proportion as the distribution of the horizontal shears to those walls or frames.

18. Story Drift. The total story drift, δ_x, when needed shall be determined by

$$\delta_x = C_d \, \delta_{xe}$$

    Where

    C_d = Coefficient from Table 9A.

    δ_{xe} = Elastic deflection when base shear is V.

    Δ_s = Difference between adjacent story total drift, δ_x.

19. Foundations. It is recommended that foundation design, particularly for resisting overturning effects be studied during Phase II.

20. Architectural, Mechanical, Electrical, and Structural Members Not Part of the Seismic Resisting System. The effect on the response of the structural system and deformation compatibility of architectural, electrical, and mechanical systems or components shall be considered where there is interaction of the architectural, electrical, or mechanical system or component with the structural system. When it is determined that a high life-safety hazard is present, these systems and components shall be designed to resist a horizontal force equal to

$$F_p = A_v \, C_c \, W_c \, a_x$$

    Where

    F_p = The seismic force applied to a component of a building or equipment at its center of gravity.

A_v = The seismic coefficient representing the Effective Peak Velocity.

C_c = The seismic coefficient for components as given in Table 9B.

W_c = The weight of a component of a building or equipment.

a_x = The amplification factor at Level x related to the variation of the response in height of the building = $\left(1 + \frac{hx}{h_n}\right)$.

c. Design Strength of Building Components.

1. Masonry.

A. General. The quality and testing of masonry and reinforcement materials and the methods for determining the strength of existing and retrofit masonry shall conform to the requirements of Chapter 12 and 12A of ATC-3[6] except as modified by the provisions in this criteria.

B. Masonry Strength of Existing Materials.

(1) Compressive strength.

(2) Shear strength

(3) Tensile strength parallel to bed joints.

(4) Ø factors.

It is recommended that these items be furnished by studies in Phase II.

C. Limitation of Masonry Materials. The following materials shall not be used as structural components:

(It is recommended that the ATC-3 list of these materials be reviewed as part of Phase II.)

2. Diaphragms.

A. General. The quality and design of diaphragms in existing masonry buildings shall conform to the requirements of Chapters 9, 10, 11, and 12C of ATC-3[6] except as modified by the provisions in this criteria.

B. <u>Strength and Flexibility of Diaphragms.</u>

(1) Wood.

(2) Steel.

(3) Concrete.

(4) Gypsum.

(5) Ø factors.

(It is recommended that procedures for determining these items be furnished by studies in Phase II.)

C. <u>Limitations on materials used as diaphragms.</u> The following materials shall not be used as diaphragms in buildings:

(It is recommended that these items be developed by Phase II studies.)

3. <u>Retrofit Systems.</u> It is recommended that procedures for determining design strengths of retrofit methods be developed by Phase II of this project.

CHAPTER 10.' ITEMS RECOMMENDED FOR PHASE II RESEARCH

a. Introduction. In this Phase I document we have discussed a Methodology for Mitigation of Seismic Hazards in Existing Unreinforced Masonry Buildings. We suggest methods for determining whether or not a building is hazardous to life safety and if it is hazardous how to strengthen these unreinforced masonry buildings so they are no longer a life hazard. We have researched the available data and studied other research documents.[45] In the discussions we have mentioned a number of items for which additional research is recommended. We have also questioned the validity of some typical code provisions if applied to existing buildings. These items are summarized below in the order in which they occur in the text. The numbers given are "page numbers".

1. 2-6; 5-10. Further research is needed to review the present code limitations which do not permit wood members to resist horizontal forces (except for diaphragms) and which do not permit wood diaphragms to resist rotation. This may be a reasonable concept for new buildings but may be ultra-conservative for existing masonry structures. More information is needed on the elastic and inelastic response of wood shear walls and wood diaphragms of various types and the effect of their deflections on unreinforced masonry walls, especially in the non-linear range. This would involve analytical research plus large scale tests of wood diaphragms. The effect of roof covering and other diaphragm materials would be tested and/or analyzed.

2. 2-6; 6-6 & 6-18. The forces involved with anchorage of walls to diaphragms should be reviewed. Also, the tension, compression and shear design strength of anchors to masonry walls of various types and to diaphragms of various types need reanalysis. The design forces required for new construction would not necessarily be appropriate for existing masonry. It is recommended that customarily used anchor details be tested and analyzed. This testing would be done by independent commercial laboratories. Several tests of each type of anchor selected in various types of masonry would be performed and analyzed.

3. 2-6; 5-7, 8, 9. The determination of design base shear should be reviewed for existing building use. The effects of soil interaction and diaphragm flexibility should be considered. This is also related to comments on Page 5-11 considering the relative stiffness of horizontal diaphragms and vertical resisting elements. The need to determine the natural period of a building with, say, an average story height and regular shape should also be reviewed. A simplification in design analysis procedures would be advantageous in areas where there are many buildings of unreinforced masonry.

4. 5-4, 5, 6. The quality and physical properties of existing unreinforced masonry varies greatly with the area, date of construction, degree of inspection and supervision. To made any valid analysis of such masonry, the physical properties need to be determined. There have been many test programs on new reinforced masonry and some on new unreinforced masonry. The results vary considerably. A simple method of determining these properties is needed. Since there will be variations, even in the same building, some judgment will be required, but the range of this variability should be understood. Some methods that should be considered are:

- Pin Test. This is a variation of a Schmidt Hammer test and might be used to check mortar strength.

- Bed Joint Test. This can be done by cutting out around a masonry unit or units of masonry wall and jacking against the unit on one side.

- An evaluation of potential correlation of E values should be made comparing results of previous research with Pin Tests and Bed Joint Tests.

It is doubtful that 4' x 4' prism test specimens of old brick masonry can be removed from a wall and taken to a laboratory for testing on a diagonal. It is possible, however, that reasonably good concrete block or hollow construction of one unit thickness can provide adequate specimens.

Cores have been taken and tested on an angle. It has been difficult, however, to remove cores of old existing brick masonry more than one wythe thick. The feasibility of cores in hollow units should be determined. Cores may be of little value in hollow units as far as testing goes, but the holes left by coring are valuable for inspecting the quality of workmanship in multi-wythe masonry.

Consideration should be given to constructing prisms using masonry to match that of existing construction. By looking at cores, the type of workmanship and quality of mortar, existing masonry may be reasonably simulated. These specimens can be tested to provide additional data.

5. 5-6. Further study is needed on the overturning effects of in-plane forces. Where walls have been properly anchored laterally, experience in actual earthquakes has rarely shown failure from overturning. The frequency content of earthquake motions is no doubt involved.

6. 5-10. Detail procedures of giving "consideration to the relative stiffness of vertical elements and diaphragms" need to be studied.

7. 5-11. The torsional capabilities of flexible diaphragms needs further study. Most codes for new buildings prohibit the use of flexible diaphragms to transmit torsion. Nevertheless, they have some capability and the limitations should be established for existing structures.

8. 5-11 through 16. A rational method of analysis of in-plane forces accounting for non-linear effects in masonry shear walls is very desirable since the life-safety concept should permit some damage. To develop analysis procedures, tests on shear walls with multi-piers of variable h/w ratios would be of help. We know of no such tests at the present time on unreinforced masonry. These tests would require large scale facilities and expensive equipment capable of cyclic loading, but they should be considered for Phase II testing - at least the concept should be analyzed for potential development of criteria.

9. 5-18. Out-of-plane forces on a wall should be studied for appropriate design and evaluation criteria.

10. 6-7. Tests of masonry shear walls with reinforcing steel provided only around openings are recommended. Where shear is not critical, strengthening of this nature will be less expensive than retrofit by presently accepted methods. Tests of this type could be performed using the same facilities and equipment described in 8 above.

11. 7-1. Studies on the adequacy of various retrofited buildings should be made to determine the effectiveness of the design criteria.

12. 5-7 and Chapter 9. The adequacy of the design requirements suggested in Chapter 9 should be reviewed. Much of the proposed criteria has been taken from ATC-3. Their application to existing unreinforced masonry buildings merits further study. For instance, for regular buildings up to, say, 6 stories in height, are period calculations needed or can these be eliminated from the base shear formula? How much deviation can be expected if spectral analyses were used? Perhaps the R, C_d and D values can be varied with the general type of construction and diaphragm type to simplify procedures.

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