COMMENTARY

on

CHAPTER A1

of the

GUIDELINES FOR SEISMIC RETROFIT OF EXISTING BUILDINGS

SEISMIC STRENGTHENING PROVISIONS FOR
UNREINFORCED MASONRY BEARING WALL BUILDINGS

PREPARED BY THE

STATE EXISTING BUILDINGS COMMITTEE
STRUCTURAL ENGINEERS ASSOCIATION OF CALIFORNIA

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TABLE OF CONTENTS

FOREWORD

This is a Commentary on Chapter A1 of the Guidelines for Seismic Retrofit of Existing Buildings (GSREB) Seismic Strengthening Provisions for Unreinforced Masonry Bearing Wall Buildings, 2003 International Existing Building Code IEBC).

The purpose of the Commentary is to discuss the basis for the technical provisions for the analysis and design of strengthening measures for existing unreinforced masonry building.

The contents of the commentary are summarized as follows:

- Part 1, Preamble, is a brief, general discussion of the retrofit problem.
- Part 2, Commentary on Chapter A1 is the detailed commentary on the chapter.
- Part 3 of the Commentary, Historical Background, is a brief summary of previous documents that have been developed for seismic retrofit of Unreinforced Masonry Bearing Wall Buildings (URM) buildings.
- Part 4, References is a list of useful references.

The provisions call for the selection of one of two procedures, either the General Procedure, or the Special Procedure. The Special Procedure is based on research on the behavior of URM buildings at their limit state and on the experience of engineers who have upgraded buildings in the Los Angeles hazard reduction program; the use of the Special Procedure is expected for most buildings. The General Procedure is a procedure for buildings with rigid diaphragms that do not qualify for the Special Procedure; it is a combination of a conventional code approach and some portions of the Special Procedure. The Commentary discusses the General and Special Procedures individually and in detail in appropriate places in the document.

"Building Code" means the current code adopted by the governing jurisdiction. Notations and Definitions as given in the Uniform Building Code (UBC) 1997 edition and in the International Building Code (IBC) 2003 edition are applicable to Chapter 1.

PREAMBLE

Unreinforced masonry (URM) bearing wall buildings have shown poor performance in past earthquakes: 1868 Hayward, California, 1906 San Francisco, California, 1925 Santa Barbara, 1933 Long Beach, California 1952 Kern County, California 1935 Helena, Montanna, 1964 Olympic, Washington, 1971 San Fernando, California, 1983 Coalinga, California ,1987 Whittier, California, 1989 Loma Prieta, California, 1994 Northridge, California, and 2001 Nisqually Earthquake, Washington.

California building codes changed after the 1933 Long Beach earthquake and few URM buildings have been built in California since then; however, there are large numbers of URM buildings that remain in the United States. Following the lead of the cities of Long Beach in the 1970's and Los Angeles in the 1980s, the State of California declared, through Senate Bill 547 (Sec. 8875 et seq. of the Government Code), that the hazard posed by this class of building is unacceptable and that communities must identify them. The Senate bill does not specify the level of performance required or expected, but leaves it up to each community. Although it was the Senate bill that prompted the predecessor to this document, the hazard of URM buildings is not limited to UBC Seismic Zones 3 and 4 (which are roughly equivalent to IBC Seismic Design Categories C and D) in California. This current edition of the GSREB is correlated to the mapped seismic hazards of the United States. The loss of lives and property damage in future earthquakes will be heaviest in buildings that exist today. The short-term impact of improved knowledge, codes, and design practices will be limited to those relatively few recent buildings that take advantage of this improved knowledge. Thus the dominant policy issues posed by earthquakes involve not new but existing buildings, particularly those structures that have obvious weaknesses and do not comply with the general intent and necessary requirements of current regulations. The issue before the public and the profession is how to set standards for these non-compliant existing buildings that are consistent with both the desire for safety and the limited resources available to achieve improved safety.

It is generally accepted that the intensity of earthquakes that could reasonably be expected to occur in the moderate and high seismic zones of the United States would be sufficient to cause buildings with minimal seismic resistance characteristics to be seriously damaged causing injury or death to the occupants or passers-by.

It is reasonable, when a real hazard exists, to take steps to significantly reduce the hazard. The objective of Chapter 1 is the reduction or elimination of seismic hazards associated with URM buildings.

The goal should be reduced life-safety hazards as best possible with the available resources. The efforts should be directed to insuring a coherent load path for lateral loads, reduction of out-of-plane wall failures, reduction of loss of support for floors and roofs, and reduction of falling parapets or ornamentation. Application of this Chapter will decrease the probability of loss of life, but loss of life cannot be prevented. In order to provide a basis for designing lateral force resisting systems and details for URM buildings, it is necessary to prescribe analysis and retrofit design procedures that are consistent with the desire to reduce the chances for failures of the types noted. We should be willing to accept some major and irreparable damage as long as there is a decrease in the likelihood of the falling of building elements or loss of support for horizontal framing.

The goal of this recommended strengthening is lower than the goal set for new construction. Chapter A1 of the GSREB recognizes that the economic difficulty of strengthening existing buildings necessitates reliance on building components with seismic performance characteristics that are less than ideal.

COMMENTARY ON CHAPTER A1

C101 - Purpose

Chapter A1 is intended for a hazard reduction program. It represents minimum standards required to reduce risk of life loss or injury. The objective is accomplished by reducing the possibility of damage and the extent of damage. However compliance with this Chapter will not necessarily prevent earthquake damage to rehabilitated buildings. Risk of life loss or injury is significantly reduced but not eliminated.

The risk reduction applies to the overall average performance of rehabilitated unreinforced masonry (URM) buildings. An individual building may have damage levels above or below the average depending on its structural characteristics and the local ground motion. If it is desired to further limit damage to any specific building or preserve its post-earthquake function, the Engineer should consider additional measures for strengthening the building.

Chapter A1 is not intended to be applicable in locations in the United States where new construction using plain or unreinforced masonry is allowed.

C102 - Scope

C102.1 General. This chapter is applicable to all existing buildings that have least one unreinforced masonry bearing wall. The provisions are not applicable to buildings having only non-bearing unreinforced masonry walls. The definition of a bearing wall in the 1997 UBC is given in Sec.1629.6.2 as a wall providing support for gravity loads. Sec. 2106.2.8 provides a definition of a non-bearing wall that defines the difference between a bearing wall and a non-bearing wall. The GSREB defines an unreinforced masonry bearing wall as a wall that supports more than 200 pounds per linear foot (2919 N/M) of vertical load in addition to its own weight. Unreinforced masonry walls constructed within concrete or steel frames are not bearing walls. These infilled frame buildings with unreinforced masonry walls are considered as potentially hazardous and they are not covered by this Chapter. Chapter 5 of the GSREB provides analysis procedures for non-ductile concrete frame buildings with unreinforced masonry walls infilled into the frames.

Table l-A lists elements of buildings that are regulated by this Chapter. This listing of elements is related to observed damage or partial collapse of these elements and spectral intensity of ground shaking. Separation of unbraced parapets has been reported for cities distant from a large magnitude earthquake source. The measure of spectral velocity S_{Dl} (or C_v for the 1997 UBC) is used instead of short period spectral acceleration because this is better related to potential instability of the parapet. The upper bound value of S_{Dl} given in Table l-A corresponds to descriptions of Seismic Design Categories B, C and D. The contours were generally the boundaries of UBC Seismic Zones 2A, 2B, 3 and 4.

Chapter A1 does not regulate non-structural systems. Requirements of the Building Code will apply to reconstructed, non-structural systems.

C102.2 Essential and hazardous facilities. The degree of earthquake hazard reduction anticipated in Chapter A1 is not considered acceptable for the 1997 UBC Occupancy Categories given in Table 16-K other than for Standard Occupancies, or 2000 IBC Seismic Use Groups II and III where Seismic Design Categories C, D, E, and F are required. These occupancies or Use Groups require special detailing considerations, increased seismic loading and, in the 2000 IBC, reduced allowable story drift.

C103 - Definitions

In general, the definitions given in this section are self-explanatory. The commentary expands on several of the definitions that are critical to the use of Chapter A1.

COLLAR JOINT is the space between wythes, space that may be empty or may be filled with mortar or grout. Although the condition of the collar joint may not be critical for in-plane wall shear stress, the procedure for determining allowable stress from in-place shear tests takes into account the probable effect of mortar in the collar joint. The condition of the joint is of greater importance for out-of-plane forces where it is necessary to have the wythes of the wall act integrally. A visual examination of the collar joint to determine its mortar coverage is required to allow the use of the increased height-to-thickness given in Table A1-B for buildings with crosswalls in all stories.

CROSSWALL is a light frame wall sheathed with new or existing materials. Light frame walls with shear panels have a desirable hysteretic behavior and can by their coupling strength cause adjacent diaphragms of different stiffness to have nearly the same relative displacement. They function as energy dissipaters when connecting diaphragms or a diaphragm to grade within the span of the diaphragm. They act similar to "shear" walls to the extent that they diminish the displacement of a floor or roof relative to the building base, but are not true shear walls. Their in-plane stiffness is not comparable with shear walls of masonry, concrete or lateral load resisting elements of structural steel. Moment resisting frames may also be designed as crosswalls.

FLEXIBLE DIAPHRAGM is a floor or roof constructed of wood framing with attached sheathing or a roof or floor sheathed with steel decking if the steel deck does not have a lightweight or regular weight concrete fill. The definition of a flexible diaphragm in Chapter 16 - Definitions in the 2000 IBC or Sec. 1630.6 of the 1997 UBC is not applicable to Chapter A1. A horizontal steel braced diaphragm with tension only braces is also considered a flexible diaphragm.

NORMAL WALL is a URM BEARING wall with loads perpendicular to the wall. When the earthquake loads are parallel to an URM bearing wall, the wall is considered as a shear wall.

OPEN FRONT is a term for a side of a building that does not have a shear wall, frame or braced frame at the exterior wall line. The open front may be at the first story or at all stories. The open front is defined as " on one side only". The definition is intended to be a limitation. Open front buildings on street corners must have lateral load resisting elements such as shear walls installed on one of the street-front sides.

POINTING is equivalent to repointing.

RIGID DIAPHRAGM is a floor or roof diaphragm of reinforced concrete construction. This floor or roof could be supported by reinforced concrete beams and girders or by structural steel members. The floor or roof could be a self-supporting slab or a pan-joist system. Chapter A1 does not consider semi-rigid diaphragms. It uses only two categories of the relationship of diaphragm in-plane shear stiffness to masonry wall in-plane shear stiffness. The Engineer must consider the relative stiffness and strength limit states of other materials, such as gypsum concrete roofs, to determine a rational analysis procedure for the existing structural materials. An alternate procedure would be to use the Building Code definition of a flexible diaphragm that is based on the shear stiffness only of the diaphragm and the shear walls below the diaphragm level.

UNREINFORCED MASONRY (URM) WALL is a wall constructed of solid clay or concrete units, with or without cores, or a masonry wall constructed of hollow units, either clay or concrete. If the wall is constructed of hollow units the net area of the masonry wall must be used for in plane shear calculations. The allowable height-to-thickness ratios given in Table A1-B are not affected by the solidity of the masonry unit. Only the height-to-thickness ratio affects the stability of the wall. The weight per square foot of the wall is not a parameter for out-of-plane wall stability.

The in-plane bed joint shear strength or tensile splitting strength of the masonry must be determined by testing. Field stone or adobe masonry does not have a reliable test method for determination of inplane shear strength. The Engineer should, with the concurrence of the Building Official, determine strength limit state materials properties.

Chapter A1 has been developed for unreinforced masonry. The tensile strength of bed joints is considered as zero. A wall is considered unreinforced if the amount of reinforcing is less than 25% of the minimum amount that would be required by the Building Code. The intention is to exclude engineered systems that have small but definite amounts of horizontal and vertical reinforcing.

In most cases, a wall will either contain reinforcing at or near the Building Code requirements, or it will have no reinforcing at all. The reinforcing must be both vertical and horizontal. Steel straps laid along bed joints exist in some older types of construction; these should not be considered as horizontal reinforcing since these straps could actually weaken the bed joint.

C104 - Symbols and Notations

Extended definitions are given to clarify the use of the following symbols:

P_D	The tributary dead load at the top of the pier under consideration. For uniformly loaded bearing walls, it consists of the dead load on a width of wall extending between the centers of the openings adjacent to an interior pier, or from the wall edge to the center of the adjacent opening for the exterior piers, or from wall edge to wall edge. For use in Formulae A1-4, A1-5, and A1-6, P_D is the superimposed dead load at the test location.
P_{D+L}	Stress resulting from the dead load plus actual live load in place at the time of testing, in pounds per square inch. For use in Formula (A1-3) the quantity P_{D+L} is the stress obtained by dividing the axial load on the wall by wall area, not the area of the test bed joints.
$\sum v_u D$	Is the sum of the diaphragm shear capacities of both ends of the diaphragm that are parallel to the direction of lateral load.
$\sum\sum v_u D$	Is the sum of the $\sum v_u D$ values for the diaphragms at and above the level under consideration.

C105 - General Requirements

C105.1 General. Chapter 16 of the Building Code provides the basic requirements for a seismic resisting system. Distribution of lateral loads shall consider the relative rigidity of horizontal diaphragms. Flexible diaphragms have inadequate relative stiffness to redistribute force between those elements that are considered as shear walls by the special procedure (SP). Diaphragms in buildings that use the general procedure (GP) are capable of redistributing the lateral loads, causing increased forces due to torsional response and coupling the weight at each story level with the shear walls.

Cl05.2 Alterations and repairs. The Building Code applies unless explicitly excluded or modified herein.

Cl05.3 Requirements for plans. These are minimum requirements for the plans to be submitted. Although this may seem an administrative requirement, this section is needed to record that a thorough investigation of the building has been made and is shown on required plans that are submitted to the Code Official.

The plans should include openings and their dimensions. Each diaphragm should be investigated at several locations. The lay-up of the sheathing should be observed and properly described. The continuity of the finish floor sheathing under partitions need not be verified.

Cl05.4 Structural observation. The construction phase of the retrofit program will discover unforeseen conditions. The Engineer or Architect responsible for the design must provide a more extensive observation of the project than that required for new construction.

Sec. C106 - Materials Requirements

Cl06.1 General. Existing materials as described in Table A1-D, new materials as described in Table l-E and masonry materials when tested as described in this section or categorized by f'_m may be used as a part of the seismic resisting structural system. New materials permitted by the Building Code may be used to supplement the strength of existing materials.

Cl06.2 Existing materials. Shall be in a sound condition or repaired. If replaced they must be replaced with new materials that are permitted by the Building Code for seismic resistance.

Cl06.3 Existing unreinforced masonry.

Cl06.3.1 General. It is essential for the Engineer to make a proper evaluation of the existing masonry. Allowable shear values for use in analysis of shear walls and allowable h/t values for normal walls that are not analyzed for out-of-plane loading are based on the assumption of adequate strengths and lay-up or the masonry. Non-conforming masonry must be removed or treated as veneer.

Cl06.3.2 Lay-up of walls.

Cl06.3.2.l Multi-wythe solid brick. The concern is for the integrity of the multiwythe wall acting as a whole in resisting out-of-plane forces. Facing means the wythe at the face of the wall, whether interior or exterior; backing means the inner wythes that are not normally visible. Lay-up means the pattern of masonry units in each wythe and in the interlocking of wythes. The primary requirement is that all wythes be adequately interlocked by header bricks. The quantity of mortar in the collar joints between wythes is also important, having an affect on the allowable h/t ratio for the wall as specified in the footnotes to Table A1-B.

An exception to the requirements of the common lay-up of multiwythe walls is permitted for lower seismic hazard zones. This exception is applicable in UBC Seismic Zones other than Zone 4 and where S_{D1} as defined and mapped in the IBC is 0.3g or less.

This exception permits veneer wythes with anchorage as specified by the Building Code and made composite with the backing masonry to be considered as a part of the structural wall. The wythes may be bonded by a combination of additional ties or the combination of grout and ties and should be checked to determine if they can transfer the calculated stress between wythes.

Cl06.3.3. Testing of masonry.

Cl06.3.3.l Mortar Tests.

Item 1. Mortar tests are performed by shearing of the mortar joints above and below the test masonry unit. The test load is recorded at the first sign of movement of the test brick. This can be detected by observation of sand grains detaching from the mortar joint.

Item 2. Many masonry walls in the United States have been constructed with mortars that have shear strength such that the mortar shear strength test is not possible (e.g., hollow masonry). The failure may be the bearing on the end of the masonry units at the jack. Where this occurs, an alternative testing method described in Sec.106.3.3.2 determines tensile splitting strength by the standard ASTM procedure.

C106.3.3.2 Alternative procedures for testing masonry. These procedures determine the tensile splitting strength of masonry. The tensile splitting strength is equivalent to the peak shear strength when pier shear strength, V_m, is calculated using $v_m A_n/1.5$.

Recently constructed masonry (post-1950) may be categorized by unit strength and mortar mix, Table 21-D of the UBC or Tables 2105.2.2.1.1 and 2105.2.2.1.2 of the IBC. These tables may be used to estimate f'_m of the masonry. The estimate will be a conservative prism strength. This prism strength may be used to calculate peak shear stress. The shear strength of a pier is calculated by Formula (A1-20).

The shear strength is an expected shear strength, no load factors or capacity reduction factors are used on Chapter A1.

C106.3.3.3 Location of tests. The test locations should be uniformly distributed over the wall surface. The exact location of each test shall be determined at the site by the Engineer. Relatively small areas that are pointed due to local deterioration should not be used for test areas since pointed areas will produce higher values that can skew the test results.

C106.3.3.4 Number or tests. Numbers and locations are specified to assure a representative sample of existing masonry; considering that the walls were not necessarily built with the same time with the same workmanship, workmanship may vary in a given wall, often being poorer near the top, and the walls have not been subjected to the same environmental influences. These, and other conditions, may make it desirable to make a division of walls into classes according to their relative overall quality, see Sec. A106.3.3.8. The number of tests in each class of wall should be increased to the maximum number specified in Sec. A106.3.3.4.3 to provide a body of data that is adequate to establish a 20th percentile value, $v_{t.}$

A common line of resistance is defined as a set of one or more masonry walls at an edge of a diaphragm. The wall may have offsets in that common line. A minimum of eight tests for a building is specified. This would be the minimum number for a single-story building. The area of 1500 square feet is gross area, including wall openings.

C106.3.3.5 Minimum quality mortar. A mortar shear test value, v_{to}, is obtained for each test location by dividing the test force, V_{test} by the sum of the areas of the upper and lower surfaces of the brick, and subtracting the axial stress in the wall at the time of testing.

The axial stress is defined to include actual live load where it is significant; ordinarily the use of the dead weight of the superimposed masonry is close enough. A set of values of v_{to} is assembled for the whole building.

When an individual wall has values of v_{to} that are consistently below 30 psi, the wall may be upgraded by pointing. Where this is done, the whole wall must be pointed. Then the upgraded wall is re-tested and improved values of v_{to} will be obtained. These improved values of v_{to} may be used in place of the original values for that wall, and a new set of values of v_{to} is assembled for calculation of v_{to} for this wall now represents a class of masonry.

Once a set of values of v_{to} is obtained for the building or a class of masonry, the mortar shear strength, v_t for the building or the class of masonry is determined as the value of v_{to} that is exceeded by eighty percent of the values of v_{to}.

When v_t is less than 30 psi for the building or a common line of resistance, all of the masonry in the building, or line of resistance, must be either removed or upgraded. It may be upgraded by repointing and then re-tested.

Cl06.3.3.6 Minimum quality of masonry. A minimum quality of an assemblage of units and mortar is required to have confidence in the determination of strength limit values. Hollow unit masonry can have its tensile splitting strength increased by pointing. Hollow unit masonry commonly has only the face shell supported on a mortar bed joint. Hollow unit masonry may also be grouted to increase its net area and tensile splitting strength.

Cl06.3.3.7 Collar joints. When a masonry unit is removed for the in-place shear test, the collar joint between the exterior wythe and the interior wythe is exposed. The percentage of mortar coverage of the collar joint is estimated and reported with the results of the in-place shear testing. Fifty percent of the collar joint must be filled to meet the requirements of footnotes to Table A1-B. Table A1-B specifies the allowable height-thickness ratios of unreinforced masonry walls.

Cl06.3.3.8 Unreinforced masonry classes. This section allows categorization of masonry into classes. A single wall may be defined as a class. The requirements that 80 percent of the tests made in this wall exceed the test value, v_{to} would require a minimum of five tests in that wall. A three-story or higher building will have a minimum of five tests in any wall. If categorization of a wall or walls as a class is contemplated in a building of fewer than three stories, additional tests will be required in each of the walls or line of walls.

Cl06.3.3.9 Pointing. Deteriorated mortar joints must be pointed. Mortar joints may be pointed before testing to increase the shear test values, but this pointing must be performed with special inspection as required by Sec. A107.1.

Pointing is maintenance work. Old mortars are subject to deterioration, and if deteriorated, they need to be renewed. Deterioration is most often due to moisture penetration and evaporation: water carries dissolved salts into the wall, or dissolves them from the masonry itself; upon evaporation of the water, crystals are left behind. The crystals break up the mortar matrix (and sometimes the brick itself), leaving it weak and powdery. The process of weathering propagates into the wall. In many cases, sound material will be found within an inch (or 1-1/2 inches at the most) of the original surface.

Sec. C107 - Quality Control

C107.1 Pointing. Preparation and pointing of mortar joints shall have special inspection. UBC Standard 21-8 provides guidance for the pointing work. The Building Officials may allow pointing of small, deteriorated areas at their discretion without special inspection.

C107.2 Masonry shear tests. Commentary provided in Sec.106.

C107.3 Existing wall anchors. Self-explanatory in text and UBC Standard 21-7.

C107.4 New bolts. Self-explanatory in text and UBC Standard 21-7.

Sec. C108 - Design Strengths

C108.1 Values. The materials resistance values used in Chapter A1 are strength values, but are not used with capacity reduction factors. Strength values obtained by experimental testing and inplace testing of unreinforced masonry are expected strength values. The strength values obtained by the ABK dynamic testing of ductile materials, such as sheathed diaphragms are strength limit values. It is recommended that strength values needed for retrofit materials that are not included in this Chapter be obtained from the latest edition of the NEHRP Recommended Provisions.

Design of new elements that are intended to supplement the existing material should be designed using strength procedures, but not with capacity reduction factors.

When designing new elements, special attention should be paid to the relative stiffness of the existing structural elements and the new elements. For example, if new shear walls or braced frames supplement the strength of an unreinforced masonry wall, the loading should not be reduced by an increased R Factor. However, if the existing structural system is strengthened so that the system's ductility is increased, a higher R Factor could be justified. Such an example would be where new concrete shear walls are added and are designed to carry the full lateral load and the unreinforced masonry piers are adequate to accommodate the expected sum of linear and nonlinear displacements without shear failure.

C108.2 Masonry shear strength.

Item 1. The correlation of v_t, and v_m was obtained by physical testing made by the ABK Joint Venture. Formula A1-4 is an empirical formula.

Item 2. Formula A1-5 is a theoretical formula adjusted for a probable coefficient of variation of the test data.

Item 3. Formulas A1-6 are similar to the shear values given in the Building Code. They are allowable strength values adjusted upward by about 1.7.

C108.3 Masonry compression. This maximum allowable stress is a conservative estimate of the compressive strength of a minimum acceptable masonry. No strength increase for seismic loading is permitted.

C108.4 Masonry tension. Text is self-explanatory.

C108.5 Existing tension anchors. Text is self-explanatory.

C108.6 Foundations. Foundation loads may be increased over existing loads for several reasons. First, consolidation of foundation materials has decreased the probability of settlement due to added loads. Second, the dynamic loading of soils by earthquakes can be tolerated by foundation materials without additional settlement. Third, in many cases, the dead load added by seismic strengthening is usually relatively insignificant.

The restrictions may be illustrated by the following example. Given original dead load = 100, added dead load = 20, live load = 50, and seismic = 30. The new dead load is 100 + 20 = 120. By the first sentence in Sec. A108.6, new dead load may be 1.25 x existing dead: in the example, 120 is less than 1.25 x l00 = 125; therefore, the added dead load is acceptable. By the second sentence, new dead plus live plus seismic may be 1.50 x existing dead plus live: in the example, new dead plus live plus seismic is 120 + 50 + 30 = 200, which is less than 1.5 x (100 + 50) = 225; therefore the added dead load is acceptable.

Every structure should provide adequate resistance to resist overturning effects. This is less important for existing unreinforced masonry elements since their lateral load carrying capacity is limited by their rocking capacity. However, where new elements are added, overturning can be a controlling issue. In these cases the new elements should be designed to resist overturning in general accordance with the Building Code.

Sec. C109 - Analysis and Design Procedure

Cl09.l General. Chapter A1 uses analysis and design procedures that are specific for unreinforced masonry buildings. The Chapter is for analysis and retrofit for existing buildings. The elements of the masonry buildings that are regulated by Chapter A1 are given in Table A1-A. This table exempts certain items from regulation in low to moderate seismic hazard regions.

Cl09.2 Selection of procedure. Unreinforced masonry buildings with concrete floors and roofs (rigid diaphragms) must be analyzed using the General Procedure. The General Procedure follows the Building Code in that the analysis base shear is calculated using the total mass of the building. The rigid diaphragms couple the shear walls of each level with the weight of the story level and normal walls. URM buildings with flexible diaphragms have an entirely different dynamic response characteristic. The mass of the floor, roofs and normal walls are coupled with the shear wall parallel to the direction of the seismic loading by a shear beam element that has a fundamental elastic period in the range where dynamic response is related to T, the elastic period of the diaphragm. This shear beam has significant ductility potential and the coupling of the story and normal wall mass with the shear wall will be limited to the shear capacity of the diaphragm at each story level. Section 111.1 provides the limits a building must satisfy in order for the special procedure to be used.

Sec. C110 - General Procedure

C110.1 Minimum design lateral forces. The base shear of the URM building with rigid diaphragms is given by Formula (A1-7). This formula is recognizable as being related to seismic design formula in the Building Code. It is a simplified form of the usual base shear formula.

The fundamental elastic period of the building is taken as being in the constant acceleration range, i.e. S_{DS} for the IBC or $2.5C_a$ for the UBC. The base shear is at the maximum value for the seismic hazard zone.

This base shear is reduced by 0.75 to be consistent with the policy that the analysis forces for existing buildings is three-quarters of the design forces specified for new buildings. The base shear is also divided by an R value of 1.5. This is consistent with Table 1617.6 of the IBC for bearing wall systems having ordinary plain (unreinforced) masonry shear walls. There is no comparable R factor in the UBC for bearing wall buildings with unreinforced masonry walls. It is recommended that this base shear be applied in a uniform loading pattern on a multi-degree of freedom model, if the shear wall has substantial perforations at each story level. The multi-degree-of freedom model is a shear-yielding model, not a flexural beam model. Story yield mechanisms may be only a single floor level if the yield mechanism is in-plane shear in the piers. If rocking of the piers is the yield mechanism, it is a possibility that yield mechanisms may form at several story levels. If the wall is basically solid, a triangular loading should be applied, as per the Building Code. The Engineer should make preliminary calculations as to probable story strengths and yield mechanisms to determine a rational distribution of the base shear over the height of a multi-story unreinforced masonry shear wall.

Global overturning is not a critical issue for perforated unreinforced masonry shear walls. Flexural tensile forces due to global overturning cannot be carried downward without a tension load path. Story height overturning moments cause a modification of the axial load on the pier, but equilibrium requires that the reduction in vertical pier loading be equal to the increase in vertical pier loading. The in-plane shear capacity and rocking shear capacity is affected by pier loading, but the total interstory shear capacity is not significantly changed.

C110.2 Lateral forces on elements of structures. The failure of building elements such as parapets and portions of walls represents a major source of hazards posed by URM buildings. All vulnerable elements should be identified and analyzed.

Diaphragms are analyzed for the loadings calculated by the distribution of base shear at the level of consideration. The allowable shear values of existing materials and new materials applied to existing materials given in Tables A1-D and A1-E are applicable to the General and Special Procedures. The Provisions require the same force factors on parts and portions as are required for a new building.

Although not specifically stated, in buildings being analyzed by the General Procedure having flexible diaphragms and crosswalls conforming to Section 111.3, use of the crosswalls to provide partial lateral support for the diaphragm loading has been used by some engineers and permitted by several jurisdictions.

Exception 1. Normal walls of unreinforced masonry need not be analyzed for flexural capacity but may be deemed adequate if the height-thickness ratios specified in Table A1-B are not exceeded. Use of this table is very conservative for rigid diaphragms. The rigid diaphragm does not amplify the story level acceleration like a flexible diaphragm. In addition to this effect, the overburden/wall weight ratio used in development of Table A1-B used the weight of wood floors and only one additional story for multi-story buildings. The probable vertical load of concrete floors and roofs would greatly increase the acceptable height-to-thickness ratio for normal walls.

Exception 2. Parapets that have height-thickness ratios less than specified in Sec. A113.6 need not be analyzed. Parapets with height-thickness ratios in excess of this limit shall be braced.

C110.4 Shear Walls (In-plane Loading). Once the story shear forces are determined by the methods of the General Procedure, the analysis of unreinforced masonry shear walls follows the same procedures as specified for the Special Procedure. The masonry is tested as prescribed by Sec.106. The allowable shear stress is determined in accordance with Sec.108. The piers in the shear wall are analyzed in accordance with Sec. A112.2.3.

Sec. C111 - Special Procedure

C111.1 Limits for the application of this procedure.

Item 1. The building must have flexible diaphragms at all story levels to qualify for this procedure and the seismic loading used in this section is exclusively for flexible diaphragms. The definition of rigid diaphragms and flexible diaphragms given in Sec. A103 are purposely very narrow. Investigation of existing unreinforced masonry bearing wall buildings will find materials that do not fit into one of these narrow definitions. Such a building would have to be modeled with the unique stiffness characteristics of the materials. Stiffness degradation should be assessed if a linear elastic analysis is made. It is outside the scope of Chapter A1 to provide guidance for analysis of unique buildings. The Engineer and the Building Official should make the decisions for an analysis procedure that is specific for a unique structural system.

Item 2. The vertical lateral-force-resisting system should have stiffness such that the relative stiffness ratio of the diaphragms and shear walls assumed for this procedure is maintained. A steel or reinforced concrete moment frame does not have the story stiffness of a reinforced concrete or masonry shear wall designed for the same lateral loading. A moment frame may be adequate to act as a shear wall in the middle portion of a flexible diaphragm or at an open front of an unreinforced masonry building. Sec. A111.3.5 has special limits on yield drift of a moment frame used as a crosswall. Sec. A111.6.4 requires use of the same design forces as for a shear wall and further limits the elastic drift of this moment frame. Use of an R factor in the design of the moment frame is prohibited.

Item 3. A minimum of two lines of vertical lateral-load-resisting elements on each axis of the building is required. Sec. A111.1.2 requires that each of these elements be "predominantly of masonry or concrete shear walls". If structural steel or reinforced concrete bracing is used for one or both of these vertical elements, a relative rigidity check of the bracing versus a shear wall should be made to confirm that the concept of" rigid wall-flexible diaphragm" response to earthquake is probable.

C111.2 Lateral forces on elements of structures. This section specifies that floor and roof diaphragms are only checked for a strength limit state and stiffness check. The requirement for the check is given in Sec. A111.4. In addition, requirements for diaphragm shear transfer shear walls, and out-of-plane forces are given in sections 111.5, 111.6, and 111.7 respectively. Other elements of structures are analyzed as prescribed by the Building Code, but the forces are those given in Sec. A110 and the materials resistance values are those given in Sec. A108 - Design Strengths. These design strengths are calculated using the principle of LRFD (Strength) methods. However, load is not factored and capacity reduction factors are not used for resistance values.

C111.3 Crosswalls.

C111.3.1 Crosswall definition. The definition of a crosswall is given and fully defined in this commentary in Sec. A103 and in Sec. A111.3.1. A crosswall is a light frame sheathed wall parallel to the direction of earthquake loading. A crosswall is not considered as a shear wall in that it need not be designed for the tributary loads of the flexible diaphragm or diaphragms. The crosswall decreases the displacement of the center of the diaphragm relative to the shear walls and provides damping of the response of the diaphragm to earthquake shaking.

The spacing of the crosswalls along the diaphragm span length is limited to 40 feet. This restriction is intended to limit the higher mode response of the diaphragm between crosswalls. In general, if crosswalls are used to increase the allowable height-thickness ratio of normal walls, the crosswalls must be in each story of the building. Existing and new crosswalls must extend the full story height between diaphragm levels.

Exception 1. Allows the use of crosswalls that extend only between the roof diaphragm and a floor diaphragm to couple these diaphragms into a combined diaphragm for analysis of the DCR of the combined diaphragms.

Exception 2. Allows a special condition that commonly occurs in residential occupancies. Often in this occupancy, many crosswalls interconnect the roof and floors. However, the first floor is commonly constructed over a crawl space without crosswalls. This exception allows the use of special crosswalls spaced as far apart as 40 feet to stiffen the lowest level diaphragm. The crosswalls must have adequate strength to limit the deformation of the diaphragm at the special crosswalls. The loading of the first floor diaphragm is its tributary load and the entire capacity of the crosswalls that are above the first floor.

C111.3.2 Crosswall shear capacity. The stiffness of sheathed systems such as flexible diaphragms and crosswalls is directly related to their peak strength, therefore the minimum strength of a crosswall is related to the strength of the diaphragm that it is restraining. The minimum strength of a crosswall must be 30 percent of the strength of the diaphragm shear capacity of the stronger diaphragm at or above the level under consideration.

C111.3.3 Existing crosswalls. Existing crosswalls generally are perforated by door openings. The effective length of the crosswall used for calculations of the required crosswall strength cannot include portions that have a height to length ratio less than 1.5. If the openings were 7 feet high doors, a section between doors or doors and the end of the wall of less than 4 1/2 feet cannot be used for calculation of crosswall capacity.

The capacities of the connections of the existing crosswall to the diaphragms above or below the wall need not be investigated if the crosswall extends to the diaphragm level above. If the crosswall only extends to ceiling joists that are separate from the floor or roof framing above, then continuity of the crosswall must be provided. Footnote 2 of Table A1-D limits the total capacity to 900 lbs. per foot regardless of the combined capacity of the existing materials on the crosswall. This limitation was provided in lieu of an investigation of the capacity of the connection of the crosswall to the diaphragm.

C111.3.4 New crosswalls. Connections of new crosswalls to diaphragms shall be designed by the applicable strength design sections of the Building Code. This includes the design of a vertical connection of the end of the crosswall to the foundation or the crosswall or support below the level of the crosswall. This connection is required to protect the floor framing from wall overturning effects. The dead load of the floor or roof framing supported by the crosswall cannot be used to resist the calculated overturning due to lateral forces. However, the calculated force at the end of the crosswall need not use an overturning moment accumulated from more than two stories, including the level of calculation.

C111.3.5 Other crosswall systems. Other systems may be used as crosswalls. A steel frame may be more desirable than a crosswall in a commercial space. Such a frame should be a moment frame rather than a braced frame, so as to have the damping effect of a crosswall rather than the rigid bracing effect of a shear wall. The prescription of yield story drift not to exceed one inch is intended to provide stiffness, an inelastic threshold and load-deformation behavior similar to that of existing crosswalls.

The frame used as a crosswall is intended to yield, and the frame sections at which yielding will occur must be braced to building elements that can provide adequate restraint against lateral buckling.

Considering the cost of adding a frame, regardless of the design criteria, it may be better to provide a moment frame designed as a shear wall. When used as a shear wall, the moment frame is designed to resist the loading of the tributary area of the diaphragm or diaphragms and its drift under this loading is limited by Sec. A111.6.4 to 0.015 times the story height.

C111.4 Flexible diaphragms.

C111.4.l Acceptable diaphragm span. Conventional diaphragm analysis is not required by the Special Procedure. The strength and stiffness of a flexible diaphragm is acceptable if its span length and DCR falls within Regions 1, 2, or 3 of Figure Al-l. Figure l-l is a plot of the estimated dynamic displacement of the center of the diaphragm of five inches measured relative to the shear walls that are shaking the diaphragm. The ground motions used for these calculations were scaled to the ATC-3 5% damped response spectrum having 0.40g Effective Peak Acceleration and 12 inches per second peak ground velocity. The analysis of the DCR of diaphragms is only required in UBC seismic zones 3 and 4, or in IBC seismic areas where S_{Dl} is 0.20g or greater. The probable ground motions of UBC Seismic Zone 3 are accommodated by the substitution of C_v in lieu of S_{Dl} in the calculation of DCR.

C111.4.2 Demand-capacity ratios. All of the DCR formulas assume that the resistance of each end of the diaphragm is nearly equal. If a large inequality exists, the total capacity of the diaphragm should be adjusted. A conservative assumption would be to use twice the capacity of the end with least depth. The analysis of a flexible diaphragm for a DCR assumed that the diaphragm shape in any span is relatively rectangular. Buildings with plan irregularities but parallel walls can be analyzed by this method. The span of the diaphragm is the distance between shear walls when ties or collectors develop the load transfer to the shear wall.

Demand-capacity ratios are calculated for the following four conditions:

Item 1. If there are no qualifying crosswalls above or below the diaphragm level analyzed, the total lateral load tributary to the diaphragm, multiplied by a lateral load coefficient and spectral response factor is divided by the total shear capacity of both ends of the diaphragm.

Item 2. The crosswalls in a single story building are also usually an additional load path between the roof diaphragm and the ground. The capacity of both ends of the diaphragm and the crosswalls below the diaphragm are added together to determine the capacity of the diaphragm.

Item 3. If the building has crosswalls in all levels of the building the calculation of demand-capacity ratio begins with the roof level and is made at each story level. The sum of the diaphragm loads above the level under consideration is used as the demand. The total capacity of all the diaphragms above the level under consideration and the capacity of the crosswalls below the diaphragm level analyzed is the combined capacity. The crosswalls at each level have a minimum capacity of 30 percent of the diaphragm capacity above and cause a nearly common displacement of the diaphragms relative to the shear walls.

Item 4. A special treatment of the roof diaphragm is provided in this paragraph. Roof diaphragms are generally more flexible than the adjacent floors. This provision allows the calculation of a DCR for the combination of a roof and the adjacent floor when only these two diaphragms are coupled by crosswalls. Crosswalls may or may not exist in the stories below the floor adjacent to the roof diaphragm.

C111.4.3 Chords. Diaphragms are checked using the demand-capacity formulas and Figure A1-1. The conventional analysis for diaphragm chords is not part of the Special Procedure and the analysis is explicitly excluded in the provisions. The reason is that there is not a significant flexural response in flexible wood diaphragms; the primary effect is shear yielding in the end zones. The source of flexural capacity of the diaphragm is the continuity inherent in the diaphragm construction. Since life threatening damage related to lack of diaphragm chords has not been observed and virtually every diaphragm has some form of edge restraint and/or internal continuity capable of providing adequate flexural capacity no rules for formal analysis of chords are included in Chapter A1.

C111.4.4 Collectors. The transfer of shear forces at the edge of a diaphragm to the shear resisting elements at that edge may require a collector at the edge of the diaphragm for distribution of the diaphragm shear to the elements of the shear wall. If the shear wall is the full depth of the diaphragm no need for a collector would exist as the uniformly distributed shear load of the diaphragm would be uniformly resisted along the length of the shear wall. If the shear wall strength and stiffness were at one end of that line of resistance, the collector will accumulate load from the diaphragm and act as a tie to the shear wall element. The calculated collector force may be resisted by existing or new elements. The relative rigidity of new and existing elements should be considered in apportioning the loads to the various elements.

A pier adjacent to a corner of the two intersecting shear walls is much stiffer than the usual pier analysis indicates. The effect of the flange, which is the other wall around the corner, is significant when the flange is in tension. The stiffness of this pier may cause the spandrel to have a significant collector stress. This should be considered in the design of the collector. Because of this increased collector force, a minimum required spacing of the first shear bolt from the corner is specified in Sec. A113.1.4 to provide an attachment of the collector immediately adjacent to the corner.

C111.4.5 Diaphragm openings.

Item 1. The analysis of a diaphragm for its DCR assumes a rectangular shape. The presence of a large opening can increase the flexibility of the diaphragm and cause localized stresses. The diaphragm shear is assumed to be an average shear stress and this assumption implies that the uniform shear in the diaphragm adjacent to the opening must be transferred by collectors to the portion of the diaphragm that is continuous past the opening. Sec. A111.4.9 describes both the stiffness and strength check; the reduction in stiffness cause by an opening is checked as indicated in item 3. If the strength of the existing diaphragm with the opening is adequate for the strength analysis, the transfer of uniform shear to the remaining section of the diaphragm could be assumed to be accomplished by the diaphragm itself, and the diaphragm edge adjacent to the opening could be assumed to have no shear stress.

Item 2. When the length of an opening is a significant part of the diaphragm span length and it is in the center half of the diaphragm, the DCR of the diaphragm that is continuous shall be checked. The demand is that part of the diaphragm load, W_d, within the length of opening and the span is the length of the opening measured parallel to the diaphragm span.

Item 3. Openings within the length of the diaphragm significantly affect its stiffness. The reduction of stiffness is approximated by using a capacity of the diaphragm that is calculated for the net depth rather than the full depth.

C111.5 Diaphragm shear transfer. The calculation of the capacity of the shear connection of the diaphragm to the shear wall is based on the coefficients for spectral acceleration, C_v for the UBC and S_{Dl} for the IBC, and a C_p factor that is based on the materials used for construction of the diaphragm. The values of the C_p range from 0.5 to 0.75. The least value is for a single layer of sheathing. The greatest value of C_p is for double or multiple layers of sheathing. A single layer of plywood with all edges supported on framing or blocked is considered as equivalent to a double layer system. Steel decks without fill material have C_p values in between the single and the double sheathed diaphragms. The stronger and stiffer diaphragms amplify the ground motions transmitted to the ends of the diaphragm by the shear walls more than single sheathed systems and are assigned a high C_p coefficient.

The evaluation of the required capacity of the shear connection of a diaphragm to the shear wall is not a calculation of the reaction of a beam with a loading that may be variable within its span length. The required capacity of the connection is one-half of the total dead load of the diaphragm, and normal walls, multiplied by the spectral response and construction coefficients. The analysis of diaphragms by the Special Procedure accepts nonlinear behavior of the diaphragm and limits the required capacity of the shear connection to the capacity of the diaphragm. The check for the acceptable DCR of the diaphragm is for displacement control. The limitation of the shear connection capacity to the shear capacity of the diaphragm provides a connection strength that causes shear yielding in the diaphragm.

C111.6 Shear walls (In-plane loading).

C111.6.1 Wall story force. The Special Procedure differs from the General Procedure in that the loading of the shear wall is determined by the diaphragm capacity rather than an arbitrary distribution of a base shear to the levels of the diaphragm. If there are no crosswalls, the story force of a diaphragm level is equal to the spectral response coefficient (C_v for UBC, S_{Dl} for IBC) times the tributary weight of the wall at the level of calculation, plus one-half of the dead load of the diaphragm and normal walls at the level of consideration. However the story force need not exceed the spectral response coefficient times the tributary weight of the shear wall plus the shear capacity of the diaphragm at the level of consideration.

C111.6.2 Wall story shear. The design shear load in a shear wall at any level is the total of the wall story forces calculated at the stories above by Formulas (A1-15) through (A1-16). The least value of calculated story shear is to be used for analysis of the existing shear wall.

C111.6.3 Shear wall analysis. The forces calculated in this section are used to analyze the existing shear walls by the procedures specified in Sec. A112.

C111.6.4 Moment frames. Moment frames may be used as shear walls in locations where a shear wall does not exist. A common use for moment frames is at the open front of a commercial building. The loading of the moment frame is calculated as if it were an existing shear wall. The interstory drift of the moment frame is limited to 0.015 of the story height. If the moment frame is used in a line of resistance with a URM wall, more severe limitations on interstory drift are prescribed by Sec. A112.4.2.

C111.7 Out-of-plane forces-unreinforced masonry walls.

C111.7.1 Allowable reinforced masonry wall height-to-thickness ratios. Unreinforced masonry walls need not be analyzed for forces normal to the wall surface. Instead of a usual flexural analysis, acceptable height-to-thickness ratios for the unreinforced masonry walls are specified. Walls conforming to these height-to-thickness ratios are stable due to dynamic rocking stability, not because of the tensile capacity of the bed joints. The concept of dynamic stability assumes that the bed joints are cracked at the wall anchorage levels and near the center of the wall between wall anchorage levels. If the height-to-thickness ratio of the unreinforced masonry wall exceeds these prescribed ratios, the wall must be braced to increase its stability.

The allowable height-to-thickness ratios are given in Table A1-B. The height-to-thickness ratios were determined by data obtained by dynamic testing of full size walls. The walls at the uppermost story level are most vulnerable because there is no overburden. Walls at the first story of the buildings have less vulnerability as the unamplified ground motion shakes one end of the wall. The increased ratios for buildings with crosswalls are due to the effect of damping by the crosswalls of the diaphragms. These height-to-thickness ratios, with the exception of buildings with crosswalls, are applicable to buildings analyzed by either the General or Special Procedure. Buildings with crosswalls analyzed by the Special Procedure may have larger height-to-thickness ratios if several special limitations are met.

Item 1. In Region 1, the increased height-to-thickness ratios are applicable only to buildings with crosswalls in all story levels and if the following conditions are met.

- The tested mortar shear strength shall not be less than l00 psi or where the visual examination of the collar joints indicates 50% or more of collar joint is filled with mortar and the tested mortar shear strength is 60 psi or more.
- The separation of the building from an adjacent building shall not be less than 5 inches, Sec. A113.10.

Item 2. A special condition exists when the DCR of the diaphragms falls in Region 2 of Figure A1-1. In this special condition, the height-to-thickness ratios for crosswalls may be used even though qualifying crosswalls are not present.

Item 3. This special exception allowing ratios of "with crosswalls" when no crosswalls are present is due to the presence of hysteretic damping of diaphragms that have DCR's in excess of 2.5.

C111.7.2 Walls with diaphragms in different regions. The DCR of both the upper and lower diaphragms that shake the out-of-plane wall shall be checked. If the DCR of either diaphragm falls into Region 3 of Figure l-l, the height-to-thickness ratios for "all other' buildings shall be used even if qualifying crosswalls are be present. The DCR of both diaphragms must fall in Region 2 for the height-to-thickness ratios for "with crosswalls to be used, even though crosswalls are not present. In the calculation of DCR for determination of h/t, the effect of crosswalls is excluded.

C111.8 Buildings with open fronts. It should be noted that "open-front" has an ordinary meaning referring to a building face that is open, i.e. free from significant solid walls, usually at one story; most commonly an open-front building is the ground floor of a commercial building. Buildings on corners of blocks often have open fronts on two sides.

If a moment frame complying with Sec. A111.6.4 is placed in the open front, then the building meets the requirement of 111.1.3 for two lines of vertical elements in the direction parallel to the front and the open-front procedure is not applicable.

Under the exception of Sec. A111.1.3, the open front of a single story building is permitted to remain open if the building satisfies the requirements of the open-front procedure in this section and the building contains crosswalls.

The open-front procedure is permitted only in one direction: if there are two open fronts, one of them must be provided with a braced frame or shear wall.

In case the open-front procedure is tried but cannot be satisfied, the solution would be to provide a braced frame or a moment frame in the open front. The open-front procedure uses Formula (A1-18) to determine an effective span, Li, the distance from the nearest shear wall to the open front is the length L. An additional length is calculated in order to account for the weight of masonry at the open front. This weight, which is a concentrated load at the end of the diaphragm is assumed to be distributed uniformly along a length (W_W/W_D) L. The augmented length is (W_w/W_d) L plus L. This length is then doubled to estimate the span, Li of an equivalent diaphragm that has a deflected shape similar to a shear beam spanning between two shear walls.

The diaphragm DCR is calculated by Formula (A1-19), using the spectral response coefficient, the sum of the diaphragm and normal wall dead load the weight of the wall at the open front, and the shear capacity of one end of the diaphragm and the required crosswall. This analysis determines the capacity of the crosswall that is required to control the displacement at the end of the diaphragm at the open front.

The required crosswall need not be right in the open front, but may be at some distance back, within the building; also it should be noted that a moment frame or other system might be used in place of a crosswall in accordance with Sec. A111.3.5. Where the crosswall or other system is at some distance back from the open front, a second analysis is required to determine the maximum distance that is permitted. This analysis is similar to the first, except that a crosswall capacity is not included in the calculation of Formula (A1-19). The maximum distance is L. A trial value of L is used in Formula (A1-18) , and the corresponding L_i and DCR are used to check the diaphragm. The process is repeated until an acceptable value of L is obtained.

Sec. 112 - Analysis and Design

C111.2.1 General. The requirements of this section are applicable to both the General and Special Procedures. The two procedures will determine different in-plane shear loading for the shear walls for buildings of equal floor area because of the difference in overall building weight and the coupling of the weight of normal walls with the shear walls.

C111.2.2 Existing unreinforced masonry walls.

C111.2.2.1 Flexural rigidity. The relative rigidity of piers in a wall may be determined on the basis of net area and height as in a shear beam. Wall piers rarely have span-depth ratios that exceed that ratio where flexural stress-strain relationships exist. Use of shear deformations only for calculation of relative rigidity is an appropriate procedure.

C112.2.2.2 Shear walls with openings. This section is for the analysis of a URM wall with windows and/or openings. The in-plane strength of the wall is the strength of the piers between the windows and doors. The pier shear capacity is its allowable shear, v_m, times the net area of the pier, A_u, and adjusted from peak shear stress to average shear stress. This adjusted shear strength is the average shear as used in the design of reinforced masonry and reinforced concrete. A second method of calculating the in-plane shear capacity is given by Formula (A1-21). This rocking shear capacity assumes that the bed joint at the top and bottom of the pier has cracked and the axial load on the pier acting eccentrically on the displaced pier provides that the restoring shear or rocking shear capacity. The effective lever arm of the axial loads on the ends of the pier is about ninety percent of the pier width. This restoring force is a strength limit state.

C112.2.2.2.1 Rocking controlled mode. When rocking shear capacity of each individual pier is less than that pier's shear capacity, the pier will rotate, or rock, as a rigid block and have a stable dynamic behavior. In this case, the capacity of the wall consisting of a series of piers is the sum of the rocking shear capacities of each pier and each pier is loaded with a portion of the total base shear. The rocking shear capacity, $P_D D/H$, is used as the relative rigidity of each pier.

C112.2.2.2.2 Shear controlled mode. When the axial load on any wall pier is large enough to prevent bed joint cracking prior to the formation of a diagonal shear crack, a shear failure in this pier may occur at a small displacement. In this case the load is proportioned to the piers by the relative shear rigidity. Flexural stiffness is not included in the relative rigidity calculations as the common pier shape is such that flexural deformations are those related to deep beams.

When the pier shear capacity (V_a) is less than the pier rocking capacity (V_r) in even one pier, the shear force in the wall (V_{wx}) is distributed to the piers in accordance with their relative rigidity D/H. The distributed load to a pier, V_p must be less than V_a and if $V_p < V_a$, the shear resistance of the wall piers is adequate. If $V_p < V_r$ for each pier and $V_p > V_a$ for one or more piers, then an incompatible pier capacity exists. The piers where $V_p > V_a$ are omitted from the analysis and the procedure is repeated using only the piers where $V_p > V_a$. This procedure is shown in a flow diagram in Figure A1-2.

C112.2.2.3 Masonry pier tension stress. Flexural moments and tensile stress are not calculated for unreinforced masonry piers. Flexural cracking may occur on bed joints due to out-of-plane flexure in maximum moment zones, but the pier system is stable when subjected to the dynamic loading of an earthquake.

C112.2.3 Shear walls without openings. These walls shall be analyzed as a single pier for in-plane shear capacity. The rocking shear computation uses one-half of the wall weight as the restoring force because the product of P_wD is one-half of the wall length, D.

C112.3 Plywood sheathed shear walls. Sheathed light frame shear walls may be used as shear walls in buildings with flexible diaphragms in conformity with the restrictions in the Building Code. These sheathed walls cannot share lateral forces with other materials in the line of the sheathed light frame walls.

C112.4 Combinations of vertical elements.

C112.4.1 Lateral-force distribution. The distribution of the lateral force should be based on realistic calculations of stiffness. The compressive modulus of unreinforced brick has been determined by testing to have values vary from 1×10^5 psi to 5×10^5 psi. Effective moduli of reinforced concrete and masonry shear walls vary from 10 to 20 percent of uncracked moduli. A special case is given in Sec.112.4.2 for assignment of lateral loads regardless of relative stiffness.

C112.4.2 Moment-resisting frames. Use of a combination of moment frame and URM shear wall in the same line of resistance is prohibited unless the piers in the wall have a rocking shear capacity that exceeds the pier shear capacity, see Formulas (A1-20) and (A1-21). If this condition is met, a moment frame may be used and 100 percent of the load shall be assigned to the moment frame. The minimum stiffness of the moment frame shall be twice that specified for other moment frames that are used to resist lateral loads (see Section 111.6.4). This section should not be interpreted to be applicable to a moment frame used in the open front of a URM building where the piers do not have significant stiffness. These masonry columns may support lintels that have a substantial load. The Engineer should assess the stability of the masonry bearing surface when the design drift of the open front occurs.

C113 - Detailed Systems Design Requirements

C113.1 Wall anchorage.

C113.1.1 Anchor locations. URM walls are anchored to all roofs and floors and to ceilings in certain cases. If the ceiling is forced to displace with the roof diaphragm by existing wood trussing, the anchor should also be at the ceiling level. The URM wall is forced to remain vertical within the depth of the truss and the flexural crack will occur at the ceiling level. A depth of truss that triggers ceiling anchorage has not been defined. Engineering judgement should be used to determine an acceptable depth of the roof trusses not having anchorage at the ceiling level.

Ceiling systems of substantial weight act as loading on the walls. The transfer of the ceiling weight to the shear wall could be made by the diaphragm above and its shear connection, or by an independent shear connection and diaphragm at the ceiling level.

C113.1.2 Anchor requirements. When existing wall anchors have been tested to confirm the adequacy of the embedded end, the connection of the existing anchor to the wood framing shall be analyzed by rational engineering methods for its adequacy.

C113.1.4 Anchors at the corners. The end pier in a system of piers has additional stiffness due to the effect of the intersecting wall acting as a flange. The placement of the first shear anchor within two feet of the end of the wall aids in transferring the shear load from the diaphragm to the corner pier.

C113.2 Diaphragm shear transfer. Text is self-explanatory.

C113.3 Collectors The transfer of shear forces at the edge of a diaphragm to the shear resisting elements at that edge may require a collector at the edge of the diaphragm for distribution of the diaphragm shear to the elements of the shear wall. If the shear wall is the full depth of the diaphragm, no need for a collector would exist as the uniformly distributed shear load of the diaphragm would be uniformly resisted along the length of the shear wall. If the shear wall strength and stiffness were at one end of that line of resistance, the collector will accumulate load from the diaphragm and act as a tie to the shear wall element. The calculated collector force may be resisted by existing or new elements. The relative rigidity of new and existing elements should be considered in apportioning the loads to the various elements.

A pier adjacent to a corner of two intersecting shear walls is much stiffer than the usual pier analysis indicates. The effect of the flange, which is the other wall around the corner, is significant when the flange is in tension. The stiffness of this pier may cause the spandrel to have a significant collector stress. Because of this increased collector force a minimum required spacing of the first shear bolt from the corner is specified to minimize the collector stress in the spandrel above the opening nearest to the building corner.

C113.4 Ties and continuity. Text is self-explanatory.

C113.5 Wall bracing.

C113.5.1 General. Normal walls with excessive height-thickness ratios may be supported by an additional line of wall anchors that reduce the unbraced height or by vertical members that support the wall when cracked.

C113.5.2 Vertical bracing members. The vertical member shall be attached to the diaphragms. The reaction shall not be transferred into the URM wall and to its anchorage system. The horizontal spacing of the wall braces is limited to a distance equal to one-half of the wall height in order to provide a uniformly distributed resistance to out-of-plane displacements. This bracing does not prevent tension cracking of the URM wall. The brace should be anchored to the wall with a wall anchor at the center of the wall height as a minimum. The design loading of the wall brace is determined by the requirements of the Building Code, and the stiffness of the wall brace must be sufficient to meet the deflection limitatons.

C113.5.3 Intermediate wall bracing. Braces that extend between floors or roofs can reduce the unbraced height of the URM wall. A horizontal girt on the face of the wall, anchored to the wall, can be used to distribute the wall anchor force to braces that are spaced greater than six feet. Vertical displacement of the anchorage of the brace to the floor and roof will cause an outward displacement or bending stress in the URM wall. The deflection of the brace anchorage point caused by live loading shall be investigated and minimized.

C113.6 Parapets. A maximum height of unbraced parapets above the wall anchorage points is specified. This height of unreinforced wall will be dynamically stable, but may be slid laterally by the diaphragm response. A minimum height of parapet above the wall anchorage is specified, as its tension capacity is dependent on the axial stress in the wall at the level of anchorage. An exception is provided for a minimum height of six inches. This exception is necessary as it was specified for standard details used for parapet hazard reduction prior to the development of comprehensive requirements for earthquake hazard reduction. It is not a recommended detail, and reinforced concrete beams may be slid laterally on the top of the wall by the anchorage forces. However failure of the anchorage system has not been observed.

C113.7 Veneer. Unbonded brick veneer and veneer poorly bonded with angled bricks or sporadic flat metal ties to the URM structural wall have been found to detach from the structural wall when shaken with 20 to 30 percent g ground motion. The exception in this section was taken from the 1930 Los Angeles Building Code and is applicable to seismic zones 3 and 4.

C113.8 Nonstructural masonry walls. A nonstructural wall must be isolated from the structure on three sides. If the wall is not isolated from the structure it must be considered as a shear wall and/or bearing masonry wall. The allowable height-to-thickness ratio is reduced from that allowed for a one-story bearing wall because vertical loading increases the stability of an unreinforced masonry wall. The height-to-thickness ratio may not exceed nine or the limitations of Sec. A113.5.3.

C113.9 Truss and beam supports. Conformance of the URM wall with the h/t limitations does not imply that these walls will be uncracked and deform as flexural beams. It is highly likely that a crack will occur at each anchorage point and in the center part of the wall where the design level of intensity of shaking occurs. These blocks of walls will rotate on these cracks but will be dynamically stable. The rotation angle between a block above and below a wall anchorage could be greater than five degrees. A truss or beam that has an unyielding bearing surface can load the edge of its bearing on the masonry and cause spalling of the bearing surface. The secondary support serves the same purpose as shoring that would be installed after the earthquake. A foundation is not required for the secondary support, and the secondary support loads need not be accumulative in a multi-story building.

C113.l0 Adjacent buildings. The increases in the allowable height-thickness ratios of URM walls, when crosswalls are present in each story, were predicated on allowing the diaphragms to have dynamic displacements that cause nonlinear displacements in the crosswalls and the diaphragm or in the diaphragm alone. Figure l-l was plotted by nonlinear analysis using ground motions scaled to 0.4g Effective Peak Acceleration. The acceptable demand-capacity-span boundary is equivalent to five inches of displacement of the center of the diaphragm relative to the shear walls at the end of the span. If an adjacent building restricts the diaphragm displacement, the increase in the allowable height-thickness ratio is not applicable.

TABLE l-A - ELEMENTS REGULATED BY THIS CHAPTER. This Chapter does not regulate unreinforced masonry elements where construction using unreinforced masonry is permitted by the Building Code. The requirements for UBC seismic zones 2A and 2B are separately listed, as the boundaries of those zones are 0.067 to 0.133g and 0.133 to 0.20g EPA respectively.

Observations of damage and analysis of URM buildings in the described seismic hazard zones provides the basis for the determination of regulated elements. Construction materials and methods used for URM buildings are very similar across the United States. The majority of URM buildings in Southern California and along the Pacific coast have been shaken with intensities correlated with UBC zones 2A and 2B. Collapse of parapets and separation of unanchored walls at the top story are observed failures. Strength and stiffness checks of existing diaphragms are required only in UBC seismic zones 3 and 4 and where IBC Seismic Design Category C and higher.

Diaphragms analyzed by the Special Procedure use Sec. A111.4.2 and Figure Al-l for a determination of adequate stiffness. Diaphragms analyzed by the General Procedure use the provisions of the Building Code.

TABLE A1-B - ALLOWABLE VALUE OF HEIGHT-TO-THICKNESS RATIO OF UNREINFORCED MASONRY WALLS. These values are applicable to all masonry walls both solid and constructed of hollow units. Allowable values of h/t ratios were determined by dynamic testing conducted by ABK. The values in Table l-B for the top story of a multi-story building in seismic zones 3 and 4 correspond to the ABK research. A very substantial conservatism was introduced into all other values. This conservatism was increased by the definition of seismic zones used in the Building Code versus the use of contours in the ABK Methodology. The boundary of seismic zone 4 is 0.3g Effective Peak Acceleration (EPA). The boundaries of seismic zones 3 and 2B are 0.2g EPA and 0.lg EPA respectively.

ABK tested brick with nearly solid collar joints. These joints were not purposely filled. but the high lime mortar flowed into the collar joints. Existing walls with unfilled collar joints have had portions of the exterior wythe between the header courses fall away. This has been observed at top stories and for walls that exceed the recommended heights. Footnotes 2 and 3 limit increases in the h/t ratio given for buildings with crosswalls when collar joints have limited coverage and when allowable inplane shear stress is near the minimum acceptable value.

TABLE A1-C - HORIZONTAL FORCE FACTOR, C_p. This table specifies the C_p factor to be used in Formula (A1-13) for determination of the shear transfer force at the edge of the diaphragm. It is not the C_p force used for either a strength or relative displacement analysis of the diaphragm.

TABLE A1-D - ALLOWABLE VALUES FOR EXISTING MATERIALS. The values for horizontal diaphragms were derived from ABK's cyclic and dynamic testing by 20- by 60-foot diaphragms. The values given are the strength limit state capacities obtained by the experimental testing. The values for crosswalls were derived from testing conducted by the Forest Products Laboratory, the Gypsum Institute, and the Southern California Hazardous Building Committee. Use of these values with the limitation of footnote 2 for crosswalls is acceptable without an investigation of nailing or edge connections. The required shear connection at the edge of the diaphragm does not require renailing of the diaphragm to blocking installed between the joists and rafters or to the existing edge member of the diaphragm. The blocking installed between joists at the edge of the diaphragm should be installed tightly between the joists or rafters.

TABLE A1-E - ALLOWABLE VALUES OF NEW MATERIALS USED IN CONJUNCTION WITH EXISTING CONSTRUCTION. The shear value of plywood applied over existing lx boards is limited to 675 lbs. per foot shear. This value was determined by cyclic and dynamic testing conducted by ABK. The reason for the low value is that the nails commonly used for application of plywood do not conform to the requirements of Chapter 25 of the Building Code. The thickness of the 1 x sheathing would limit the nail size used for plywood nailing to 4d. Substantial splitting of the old boards was also observed, especially at the closely spaced edge nailing of the perimeter of the plywood sheet. Testing of double board systems suggest that application of plywood over straight boards by use of staples of adequate length and a uniform distribution of the staples to each 1 x board can make a membrane similar to the double board system. The staples should be spaced along the length of every board. Concentration of staple spacing at the perimeter of the plywood sheet is not desirable or necessary.

HISTORICAL BACKGROUND

Long Beach, California

Since the early 1950's the City of Long Beach, California, has adopted a series of ordinances that addressed the earthquake hazard of URM buildings. Some other communities have used the current ordinance, adopted in 1976, as a model.

Palo Alto, California

The Palo Alto ordinance is mentioned here as the model of an ordinance that does not explicitly require owners to upgrade their buildings but does require them to have an engineering investigation performed. The ordinance includes detailed guidelines for the required reports. It is expected that when particular hazards become a matter of public record there will be socio-economic pressure on owners to strengthen the buildings. The City has also provided a number of planning and zoning incentives that make seismic upgrading more appealing. The technical provisions are based on Los Angeles Division 88 that is discussed in the following paragraph.

Los Angeles Division 88

The City of Los Angeles adopted an earthquake ordinance in January 1981. The ordinance, originally designated Division 68, became a model for other communities; it is commonly referred to as "Division 88" from its chapter number in the code as amended in 1985 and 1988. The purpose of the ordinance is reduction of life-safety hazards. The ordinance requires evaluation and upgrading of buildings that have bearing walls of unreinforced masonry. The allowable stress approach was used and the basic design force level was 0.133W. In this ordinance, the force levels were only applicable for Seismic Zone 4. The design forces were 0.10W for buildings with an occupant load of less than 100 (and 0.10W if they have crosswalls in all stories), and 0.186W for essential buildings. The City of Los Angeles adopted a Rule of General Application (RGA) in 1987. It is an alternative design method based on the ABK Methodology (ABK, 1984). The RGA and Division 88 were the basis for the Structural Engineers Association of Southern California (SEAOSC), October 29, 1987 Draft Model Ordinance and its subsequent drafts. Los Angeles Division 88 also added a design check of the roof diaphragm, using the Demand/Capacity ratio of the RGA, to their "General Procedure".

Division 88 includes a timetable for compliance, requiring more rapid compliance for higher risk buildings and allowing extension for installation of wall anchorage.

The ABK Methodology

The ABK Methodology, which addresses the unique behavior of bearing wall buildings of unreinforced masonry, was developed through research funded by the National Science Foundation. The summary report, published in January 1984, is *Methodology for Mitigation of Seismic Hazards in Existing Unreinforced Masonry Buildings: The Methodology* (ABK, 1984). The document is referred to herein by its report number TR-08. It provides the research background and formulation of the basic concepts used in the SP.

Guidelines for Historical Buildings

In January of 1986, the ABK Joint Venture published *Guidelines for the Evaluation of Historic Unreinforced Brick Masonry Buildings in Earthquake Hazard Zones* (ABK, 1986). This document provided a step-by-step procedure that embodied some further extensions of the TR-08 methodology.

The Los Angeles RGA

On April 21, 1987, the Los Angeles City Department of Building and Safety adopted a Rule of General Application (RGA) that accepted the TR-08 methodology as an alternative approach to the traditional code approach of Division 88. The RGA had been developed by the SEAOSC Hazardous Buildings Committee. In the RGA, the TR-08 force level and ultimate stresses were divided by about three in order to put the provisions on the same allowable-stress basis as other divisions of the Los Angeles Code. There was also a further expansion of the TR-08 concepts in order to resolve questions that had arisen during the early applications of the methodology.

The SEAOSC Draft Model Ordinance

The pioneering work of the SEAOSC Hazardous Buildings Committee continued and resulted in the publication on October 29, 1987, a *Draft Chapter A1 (1) Earthquake Hazard Reduction in Existing Buildings*. This document was a melding of the conventional code provisions of Los Angeles Division 88 with the RGA provisions that were based on TR-08. This document was disseminated in seminars, but formal publication was postponed due to the occurrence of the Whittier earthquake. It was expected that the Whittier experience would provide useful insights on the behavior of URM buildings; both those that had been strengthened and those that had not been strengthened under the Los Angeles hazard reduction program.

The SEAOC-CALBO URM Provisions

A Task Committee of the Structural Engineers Association of California (SEAOC) Seismology Committee developed the technical provisions. The California Building Officials (CALBO) developed the administrative provisions.

The Task Committee had two goals. The first goal was to develop a comprehensive set of technical provisions that embodies the thinking of four groups: engineers who have been involved with development of the ABK methodology, engineers who have completed evaluation and upgrade projects in Los Angeles, administrators of the program in Los Angeles, and engineers outside of Los Angeles who were unfamiliar with the methodology. The second goal was to develop a commentary that would discuss the origin and intent of the technical provisions and serve as a guide to engineers who have had no experience with the methodology, engineers to whom the provisions would appear to be a significant departure from traditional seismic design.

The Task Committee accepted the concepts of TR-08 and chose for its base document the Draft Chapter A1 (1) of October 29, 1987 and proceeded to draft the technical provisions for a model ordinance. A committee of CALBO accepted the task of developing recommendations for the administrative provisions.

One of the major difficulties faced by the Task Committee was the conflict between a need and a desire to deal with an especially hazardous class of buildings, and a desire to achieve a broad consensus on a document that includes concepts that were unfamiliar to engineers outside of Southern California. There were still some unknowns, but this document was the best available.

The Provisions were adopted into the 1991 Edition of the Uniform Code for Building Conservation (UCBC) without the reduction in base shear given in Division 88 of the Los Angeles City Building Code for low occupancy buildings.

1985 to 1997 Editions of the Uniform Code for Building Conservation

A version of Division 88 appeared in Appendix Chapter 1 of the 1985 and 1988 Editions of the Uniform Code for Building Conservation (UCBC). These versions addressed URM buildings in UBC Seismic Zone 4. The revision in the 1991 Edition addressed URM buildings in UBC Seismic Zones 1 through 4 and omitted the timetables for compliance. The 1994 Edition of the UCBC is basically the same as the 1991 Edition. In the 1997 UCBC a section addressing archaic material was added.

The California Seismic Safety Commission Model Ordinance

In December 1987, the California Seismic Safety Commission, in response to Senate Bill 547 (“the URM Law"), published a two-volume report (SSC, 1987). The report consists of (l) *Guidebook*, which offers assistance to local governments in meeting the requirements of the URM Law, and (2) *Appendix*, that repeats several pertinent codes and a model ordinance called *Rehabilitation of Hazardous Masonry Buildings: A Draft Model Ordinance* (SSC, 1985). The SSC model ordinance is based on Division 88 and has been recommended to local governments in UBC Seismic Zone 4 as a mitigation program that complies with the URM Law.

The NEHRP Handbook for the Seismic Evaluation of Existing Buildings

The Building Seismic Safety Council (BSSC) initiated at the request of FEMA in October 1988 preparation of two handbooks by a 22-member Retrofit of Existing Building (REB) Committee. The review of comments returned with the first ballot on the handbook resulted in re-ballot proposals that were submitted in January 1992. The Council reviewed comments that were resolved in February 1992.

The REB Committee developed Appendix C of the NEHRP Handbook. It was patterned on the 1991 Edition of the Uniform Code for Building Conservation (UCBC) but used the mapping of the 1988 NEHRP Recommended Provisions with the spectral response coefficient, A_v, for loading and materials resistance values at their strength limit state. Appendix C, Evaluation of Unreinforced Masonry (URM) Bearing Wall Buildings was incorporated into the NEHRP Handbook, FEMA -178 in June 1992.

Appendix C of FEMA-178 was revised in 1999 to conform to the seismic loading notation of the 1997 NEHRP Recommended Provisions and the concept of use of three-quarters of the seismic loading prescribed for design of new buildings as the acceptable analysis loading for existing buildings. This version has been balloted and accepted in ASCE 31-02 which is the successor to FEMA-310 which followed FEMA 178.

Chapter 1 of the 2001 Edition of the GSREB

Chapter 1 of the 2001 Edition of the GSREB was revised to include the seismic loading notation of the 1997 UBC and the 2000 IBC. A cross-reference between the notation of S_{DS} and S_{Dl} used in the 2000 IBC, and C_a and C_v used in the 1997 UBC is included in Sec. A103 - Definitions.

The formatting of Appendix Chapter 1 of the prior editions of the UCBC was retained for continuity. The concept of the use of three-quarters of the prescribed seismic loading for design of new buildings for the analysis of existing buildings was incorporated into the document. The definition of seismic loading in the UBC and IBC includes substantial increases in base shear for soil effects and for the proximity of the structure to major, active fault systems.

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