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Development of Procedures to Enhance the Performance of Rehabilitated URM Buildings

Building and Fire Research Laboratory
National Institute of Standards and Technology
Gaithersburg, Maryland 20899

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Prepared for:
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By:
Bret Lizundia, William T. Holmes, Margaret Longstreth, and Alan Kren
Rutherford & Chekene
303 Second Street, Suite 800 North
San Francisco, California 94107

In association with:
Daniel P. Abrams
University of Illinois at Urbana-Champaign
Urbana, Illinois

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U.S. Department of Commerce
William M. Daley, Secretary
Technology Administration
Gary R. Bachula, Acting Under Secretary for Technology
National Institute of Standards and Technology
Robert E. Hebner, Acting Director

Abstract

The 1994 Northridge Earthquake provided a major test of the effectiveness of current rehabilitation methods for unreinforced masonry bearing wall (URM) buildings. A large sample of retrofitted URM buildings were shaken, many at high levels of ground motion, and a substantial amount of ground motion and damage data was collected. The majority of the strengthened buildings had been rehabilitated to the City of Los Angeles standards in Building Code Division 88 (1985), which is similar to others such as UBC (1994) and FEMA 178 (1992). These standards all aim to reduce the risk of life safety.

While no lives were lost due to retrofitted URM building damage in Northridge Earthquake, there was nonetheless a substantial amount of property damage, and a number of buildings had to be vacated. In some cases, the damage was significant and the buildings were eventually demolished because repair was deemed economically unfeasible.

This study was to develop procedures that can be used to enhance the performance of rehabilitated URM buildings. The report is organized into three sections in the main body of the report (NIST GCR 97-724-1) and four appendices in a separate volume (NIST GCR 97-724-2), which address specific tasks in the study' workplan. These tasks are: (1) analysis of the earthquake damage data, (2) applicability of current practice nationwide, (3) study of techniques to enhance the performance of URM walls, (4) benefit-cost considerations, and (5) development of procedures to enhance the performance of rehabilitated URM buildings.

Appendix A presents the results of Task 1. Appendix B presents the results of Task 2. Appendix C contains the Task 3 research summaries documenting relevant experimental studies of URM wall enhancement methods. Appendix D contains the cost and limited benefit information developed in Task 4 for selected enhancement procedures.

The main body of the report summarizes, synthesizes, and evaluates the results contained in the appendices. Section 1.3 provides a summary of each of the five tasks. Section 2 contains recommended enhancement procedures for URM rehabilitation. Section 3 provides a discussion of the unresolved issues and research needs identified during the study for certain construction practices, rehabilitation methods, and collection of empirical performance data.

Project Responsibility

Rutherford & Chekene served as the prime contractor to NIST for this study. The involvement of subconsultants to Rutherford & Chekene is as follows:

Subconsultant	*Involvement*
Daniel P. Abrams University of Illinois at Urbana-Champaign Urbana, Illinois	Compilation of Appendix C research summaries, preparation of Section 2 design guidelines for wall enhancement techniques, and Section 3 wall enhancement research needs
Thomas Heausler Heausler Structural Engineers St. Louis, Missouri	Assistance with preparation of Appendix B
Gordon Beveridge and Geoff Canham Hanscomb, Inc. San Francisco, California	Appendix D cost estimates for wall enhancement methods
Dushyant Manmohan Applied Materials Engineering Oakland, California	Appendix D cost estimates for special inspection and testing for wall enhancement methods

Acknowledgments

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Disclaimer

The primary purpose of this study was to develop procedures to enhance the performance of rehabilitated URM buildings. Enhanced performance means either the increased reliability of achieving an intended Performance Level or the successful achievement of a Performance Level higher than expected by the current standard of practice. Although our recommendations represent our best judgment, the research on which our recommendations are based is limited, and there is also limited experience in actual earthquakes of the proposed enhancement procedures. As a result, actual earthquake performance may not meet the intended performance objective.

This study also contains summaries of 1994 Northridge Earthquake damage data collected by others, and it contains generic cost estimate data developed for use in comparing various methods of providing enhanced performance. While this report is believed to be valid for its intended purposes, users of the damage data and cost estimates assume all liability arising from such use. The material presented should not be used or relied upon for any specific application without careful examination and verification of its suitability and applicability.

Color Figures

This report contains several ground motion figures, which are available from NIST in color versions.

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Section 1: Introduction

1.1 Background

The 1994 Northridge Earthquake provided a major test of the effectiveness of current rehabilitation methods for unreinforced masonry bearing wall (URM) buildings. A large sample of retrofitted URM buildings were shaken, many at high levels of ground motion, and a substantial amount of ground motion and damage data was collected. In part because of its urban setting, scrutiny--by the media, the public, building owners, government agencies, and design professionals--was high.

The majority of the strengthened buildings had been rehabilitated to City of Los Angeles standards contained in Division 88 of the City of Los Angeles Building Code (Division 88, 1985). This standard is similar to others in common practice, such as the UCBC (1994) and FEMA 178 (1992). These standards all aim to reduce the risk to life safety posed by URM buildings; reductions in property damage reduction and loss of function are not primary performance objectives.

While it can be argued that since no lives were lost due to retrofitted URM building damage in the Northridge Earthquake and thus the performance expectations were theoretically met, there was nonetheless a substantial amount of property damage, and a number of buildings had to be vacated. In some cases, the damage was significant, and the buildings were eventually demolished because repair was deemed not to be economically feasible.

While life safety protection has long been the primary goal of seismic rehabilitation efforts, there has been a growing desire to develop methodologies which can meet more stringent performance objectives. Until recently, however, industry design standards were not available which could be used to meet such objectives. This is rapidly beginning to change, though, with the emergence of documents such as FEMA 273 (1996) and FEMA 274 (1996) which broadly address performance-based rehabilitation for all building types.

The focus of this study was on developing procedures which can be used to enhance the performance of rehabilitated URM buildings. More specifically, the objectives for this project were:

- To perform limited statistical analyses on the damage data collected from previous earthquakes.
- To correlate element-specific damage data collected from the 1994 Northridge Earthquake with different ground shaking parameters.
- To investigate URM building construction in different locations of moderate and high seismicity around the United States to determine if certain aspects of construction

typical of each region differ significantly from that of California, particularly when "enhanced" performance is desired.

- To summarize the theoretical background and practical applications of new URM wall strengthening procedures, including: grout and epoxy injections, surface coatings, adhered fabrics, shotcrete overlays, reinforced cores, post-tensioned masonry, infilled openings, enlarged openings, and steel bracing.
- To develop procedures for enhancing the performance of rehabilitated URM buildings.
- To develop benefit-cost information for the proposed enhancement procedures.

1.2 Organization of the Report

This report is organized into three sections in the main body of the report and four appendices. The appendices address specific tasks identified in the study's workplan and can be used as stand-alone documents.

Appendix A presents the results of Task 1, which was the analysis of earthquake damage data to URM buildings. It primarily focuses on rehabilitated URM buildings in the City of Los Angeles shaken by the Northridge Earthquake. General building damage was organized into damage probability matrices; element-specific damage data was also collected. An explanation of the databases of collected damage information is included. These databases are available to interested researchers. Appendix B presents the results of the Task 2 investigation into nationwide construction practices for URM buildings. Areas where current rehabilitation methodologies may require modification are identified. Appendix C contains the Task 3 research summaries documenting relevant experimental studies of URM wall enhancement methods. Appendix D contains the cost and limited benefit information developed in Task 4 for selected enhancement procedures.

The main body of the report summarizes, synthesizes, and evaluates the results contained in the appendices. Section 1.3 provides a summary of each of the five tasks. Section 2--the heart of the study--contains recommended enhancement procedures for URM rehabilitation. The recommended procedures have been organized into three broad categories which address improving the quality of design and construction, changing traditional design criteria, and the use of innovative rehabilitation methods. Development of a formal methodology and commentary is beyond the scope of this project; instead, the procedures are intended to serve as informal recommendations to the design engineer. Cost information is provided to help aid the engineer in deciding on the scope of the selected enhancement procedures. Section 3 provides a discussion of the unresolved issues and research needs identified during the study for certain construction practices, rehabilitation methods, and collection of empirical performance data.

The procedures in Section 2 address unreinforced masonry bearing wall buildings constructed from any combination of brick, stone, concrete block, or structural clay tile.

Other types of URM buildings, such as steel and concrete frame buildings with interior or exterior infill walls, buildings constructed of adobe or glass block, and single family dwellings are not addressed by this study.

1.3 Summary of Tasks Completed for this Study

The overall study effort was organized into the five tasks:

- Task 1: Analysis of the earthquake damage data
- Task 2: Applicability of current practice nationwide
- Task 3: Study of techniques to enhance the performance of URM walls
- Task 4: Benefit-cost considerations
- Task 5: Development of procedures to enhance the performance of rehabilitated URM buildings

The following discussion summarizes each task and lists important conclusions which were identified. Additional details are contained in the appendices.

Task 1: Analysis of Earthquake Damage Data

Building damage data from three earthquakes was analyzed to find trends in damage patterns and relationships between damage to retrofitted buildings and several ground motion parameters. The selected earthquakes were the 1994 Northridge Earthquake, the 1987 Whittier Narrows Earthquake, and the 1989 Loma Prieta Earthquake. The inventory of retrofitted buildings subjected to the latter two earthquakes was relatively small, whereas in the City of Los Angeles alone, data was collected for 5682 retrofitted URM buildings exposed to the Northridge event. The Northridge data was interpreted by creating general and element-specific damage matrices for each ground motion parameter similar to those in ATC-13 (1985) and Lizundia, et al. (1993).

The primary goal of this task was to investigate damage to retrofitted URM bearing wall buildings, correlate it with different ground motion parameters, and attempt to relate general damage and element-specific damage with ground motion to show which building elements are the most vulnerable and at what level of shaking they begin to fail. The ground motion predominant at a damage state is of particular interest for use in projecting the performance of standard West Coast practice to moderate seismic zones. A summary of the data collected and some of the more important conclusions from this study are given below; Appendix A provides a detailed discussion of the data collection process and analytical results.

General Building Damage Data and Conclusions

A wealth of Northridge Earthquake data was collected on general building and element-specific damage to URM buildings in the City of Los Angeles, and it was correlated with several measures of ground motion. Some of this information includes:

- This study collected information on 5682 retrofitted URM buildings of which 751 were inspected following the earthquake, 703 unretrofitted URM buildings of which 93 were inspected, and 61 buildings with only tension-tie retrofits of which 8 were inspected.

- Figure 1-1 shows the distribution of post-earthquake safety ratings based on the City of Los Angeles version of the ATC-20 (1989) rating system. There were 716 retrofitted URM buildings with ratings, yielding 482 green tags, 168 yellow tags and 66 red tags. The remaining 35 of the 751 inspected buildings did not have an ATC-20 (1989) rating.

- Figures 1-2 through 1-5 show the relationship of the total inventory of Los Angeles retrofitted URM buildings and the Modified Mercalli Intensity (MMI), peak ground acceleration (PGA), and spectral acceleration (S_a) at periods of 0.3 and 1.0 seconds. Appendix A provides a similar figure for peak ground velocity. Figures 1-2 through 1-5 show that the building inventory was concentrated in the central Los Angeles area, some distance from the epicentral region to the northwest in the community of Northridge. The histogram in Figure 1-2 indicates, for example, that the majority of the buildings (97%) were located in the MMI=VII contour or below. At MMI=VII, noticeable and noteworthy damage generally begins to occur in unrehabilitated URM buildings. The majority of the buildings also experienced peak ground accelerations below 0.25g, short period (0.3 seconds) spectral accelerations below 0.55g and longer period (1.0 seconds) spectral accelerations below 0.30g. As a result, this earthquake was largely a test of retrofitted buildings subjected to moderate seismicity, such as that in 1994 UBC Zone 2B or FEMA 178 (1992) zones with A_a or A_v below 0.20. Nonetheless, given the tremendous number of total buildings, some data is available for the larger ground motions consistent with higher seismicity areas.

- To compare ground motion and building damage, damage probability matrices similar to ATC-13 (1985) and Lizundia, et al. (1993) were created. Because only the most heavily damaged buildings were inspected, assumptions were made regarding the damage experienced by the uninspected buildings to attempt to provide a consistent estimate of damage to the total building inventory. Parameter studies were run to understand the sensitivity of these assumptions. Table 1-1 shows the ATC-13 damage states vs. MMI with our best guess estimate of the damage to the uninspected buildings. It is compared with the EERI (1994a) predictions for retrofitted URM buildings in Table 2-3. Note that at both the MMI=VII and VIII, the EERI (1994a) predictions overestimate the actual damage in the Northridge event. Table 1-2 shows a damage probability matrix using PGA as the ground motion parameter. As expected, damage increases as PGA increases. Appendix A provides a number of other damage probability matrices.

- The performance of retrofitted and unretrofitted buildings was compared. The unretrofitted and tension-tie-only building inventory is quite small, so conclusions are limited. In general, though, retrofitted buildings performed better than unretrofitted buildings. Of the inspected buildings, 55% of the retrofitted buildings had ATC-13 (1985) ratings of "Light" or higher, compared to 67% of the unretrofitted buildings.

The average damage ratio for inspected retrofitted buildings was 7.7%, compared with 10.0% for the unretrofitted buildings. If the uninspected buildings are assumed to have no damage, then 7.2% of the retrofitted buildings have ratings of "Light" or higher, compared to 8.8% of the unretrofitted buildings. With the same assumption, the average damage ratio for retrofitted buildings was 1.0%, compared with 1.3% for retrofitted buildings.

Additional Conclusions

Additional conclusions include the following:

- Buildings with basements performed better than buildings without basements, even though they are typically taller. Buildings without basements had a mean damage factor 50 percent higher than those with basements. This increase was more pronounced where ground motion intensity was lower.

- The aspect ratio of the short and long plan dimensions of a building had an small impact: higher ratios (thus, more flexible diaphragms) were marginally more likely to be damaged.

- The aspect ratio of height of the building vs. the short dimension in plan had an impact: ratios over 0.5 (thus, a more flexible vertical lateral-force resisting system) were more than twice as likely to be damaged as those with ratios under 0.5.

- Ground motion thresholds when noticeable damage began to occur and when there was a sharp increase in damage were obtained for various building components. The PGA at which approximately 1% of the buildings reported evidence of damage to most elements was 0.15-0.20g. For diaphragms and foundations, slightly higher PGAs (0.20-25g and 0.35-0.40g, respectively) were required to cause damage in 1% of the building population. A sharp increase in damage occurred at about 0.35-0.4g except for wall cracking, which showed a sharp increase earlier, at about 0.25-0.3g. Thus, these thresholds appeared to be relatively similar for all types of damage except for wall cracking. While it is possible that this was actually the case, it seems unlikely and may indicate inadequacies or discrepancies in the data.

- Veneer failures were rarely reported--inspectors mentioned veneer failures only 11 times in an inspected inventory of 751 retrofitted buildings. This is inconsistent with anecdotal evidence from various sources such as EERI (1996) which indicated more extensive veneer failures.

Table 1-1: Comparison of ATC-13 Damage State and MMI

ATC-13 Damage Description	Central Damage Ratio	Inspection Status	MMI V Number	MMI V Percent	MMI VI Number	MMI VI Percent	MMI VII Number	MMI VII Percent	MMI VIII Number	MMI VIII Percent	MMI IX Number	MMI IX Percent
None	0.000	Inspected	0		3		139		2		0	
		Uninspected	138		128		2200		56		0	
		Total	138	97.87	131	48.52	2339	45.93	58	32.95	0	0.00
Slight	0.005	Inspected	2		8		183		3		0	
		Uninspected	0		125		1320		55		1	
		Total	2	1.42	133	49.26	1503	29.52	58	32.95	1	33.33
Light	0.055	Inspected	1		5		264		17		0	
		Uninspected	0		0		880		27		1	
		Total	1	0.71	5	1.85	1144	22.47	44	25.00	1	33.33
Moderate	0.200	Inspected	0	0.00	1	0.37	70	1.37	12	6.82	1	33.33
Heavy	0.450	Inspected	0	0.00	0	0.00	25	0.49	3	1.70	0	0.00
Major	0.800	Inspected	0	0.00	0	0.00	8	0.16	1	0.57	0	0.00
Destroyed	1.000	Inspected	0	0.00	0	0.00	3	0.06	0	0.00	0	0.00
Total Uninspected = 4931 Total Inspected = 751 Total in Sample = 5682			141	100.00	270	100.00	5092	100.00	176	100.00	3	100.00
Mean Damage Factor				0.05		0.42		2.06		4.13		8.67

Table 1-2: Comparison of ATC-13 Damage State and Peak Ground Acceleration

ATC-13 Damage Description	Central Damage Ratio	Inspection Status	Less than 0.1 Number	Less than 0.1 Percent	0.1 to 0.2 Number	0.1 to 0.2 Percent	0.2 to 0.3 Number	0.2 to 0.3 Percent	0.3 to 0.4 Number	0.3 to 0.4 Percent	Greater than 0.4 Number	Greater than 0.4 Percent
None	0.000	Inspected	0		14		118		9		3	
		Uninspected	125		825		3656		227		98	
		Total	125	97.66	839	94.38	3774	88.90	236	80.27	101	80.16
Slight	0.005	Inspected	2	1.56	13	1.46	157	3.70	15	5.10	9	7.14
Light	0.055	Inspected	1	0.78	23	2.59	226	5.32	28	9.52	9	7.14
Moderate	0.200	Inspected	0	0.00	11	1.24	59	1.39	9	3.06	5	3.97
Heavy	0.450	Inspected	0	0.00	2	0.22	20	0.47	4	1.36	2	1.59
Major	0.800	Inspected	0	0.00	1	0.11	6	0.14	2	0.68	0	0.00
Destroyed	1.000	Inspected	0	0.00	0	0.00	3	0.07	0	0.00	0	0.00
Total Uninspected = 4931 Total Inspected = 751 Total in Sample = 5682			128	100.00	889	100.00	4245	100.00	294	100.00	126	100.00
Mean Damage Factor				0.05		0.59		0.99		2.32		1.94

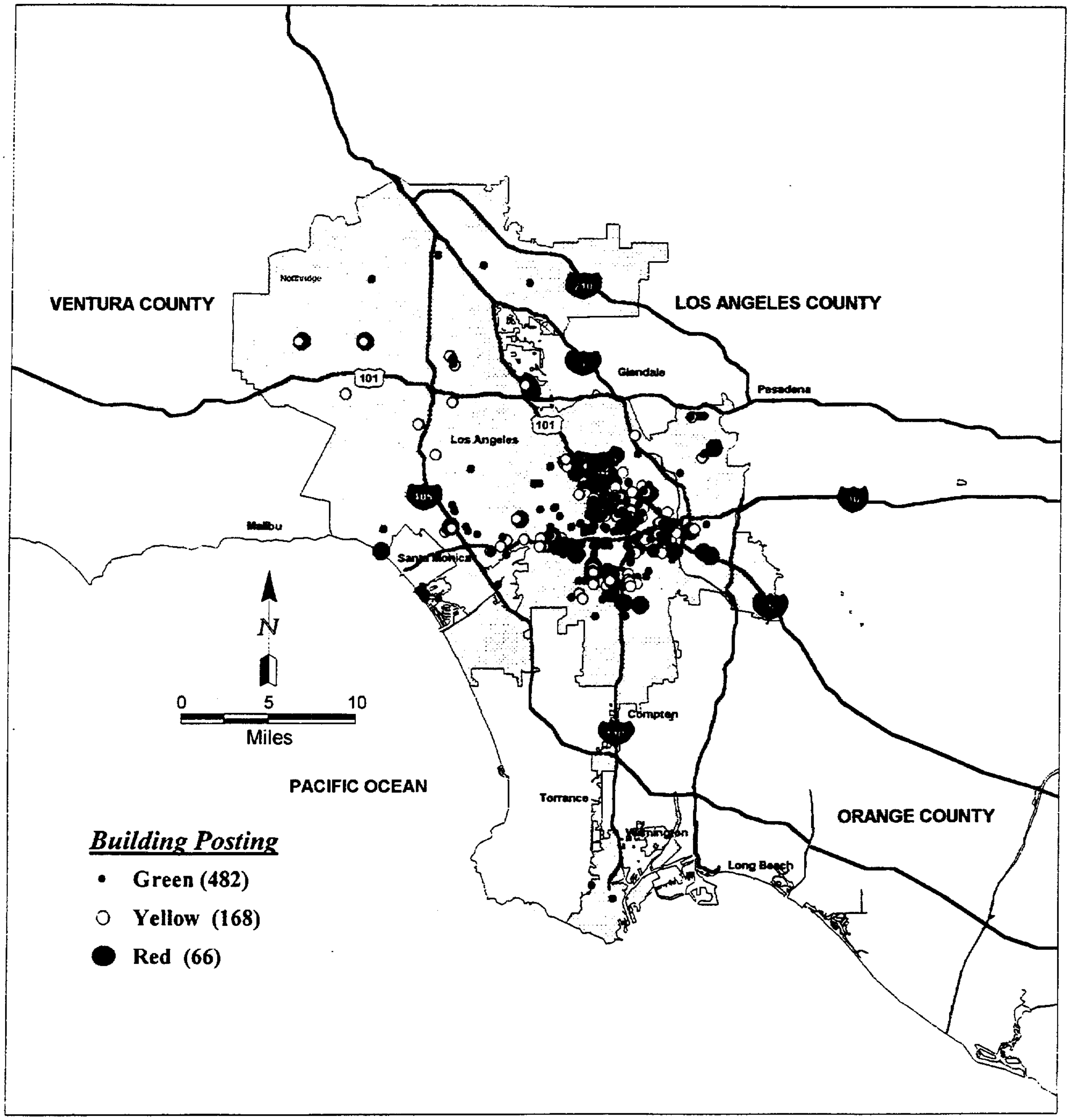
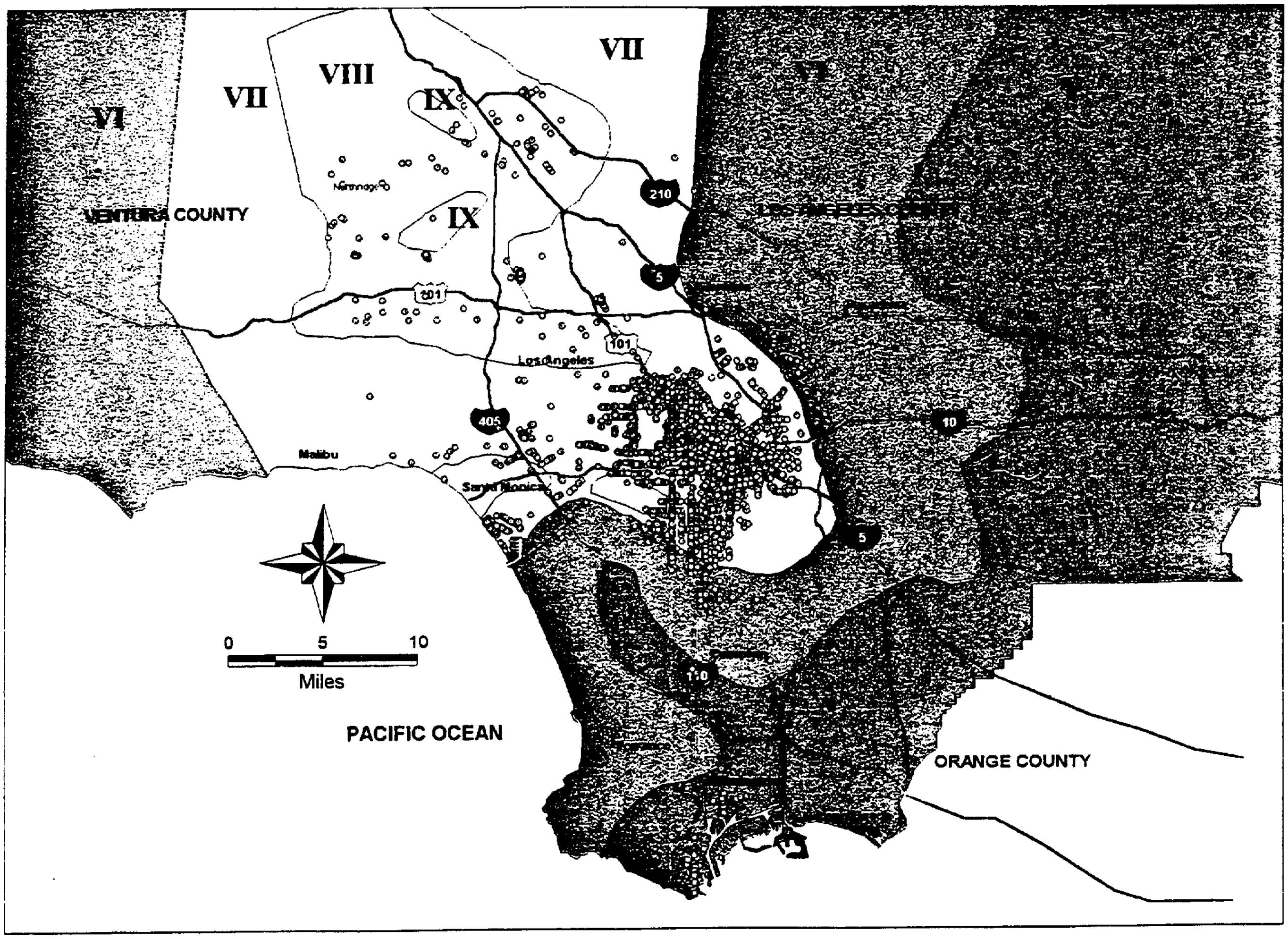


Figure 1-1: Building and Safety Damage Assessment--Building Inspection Tagging



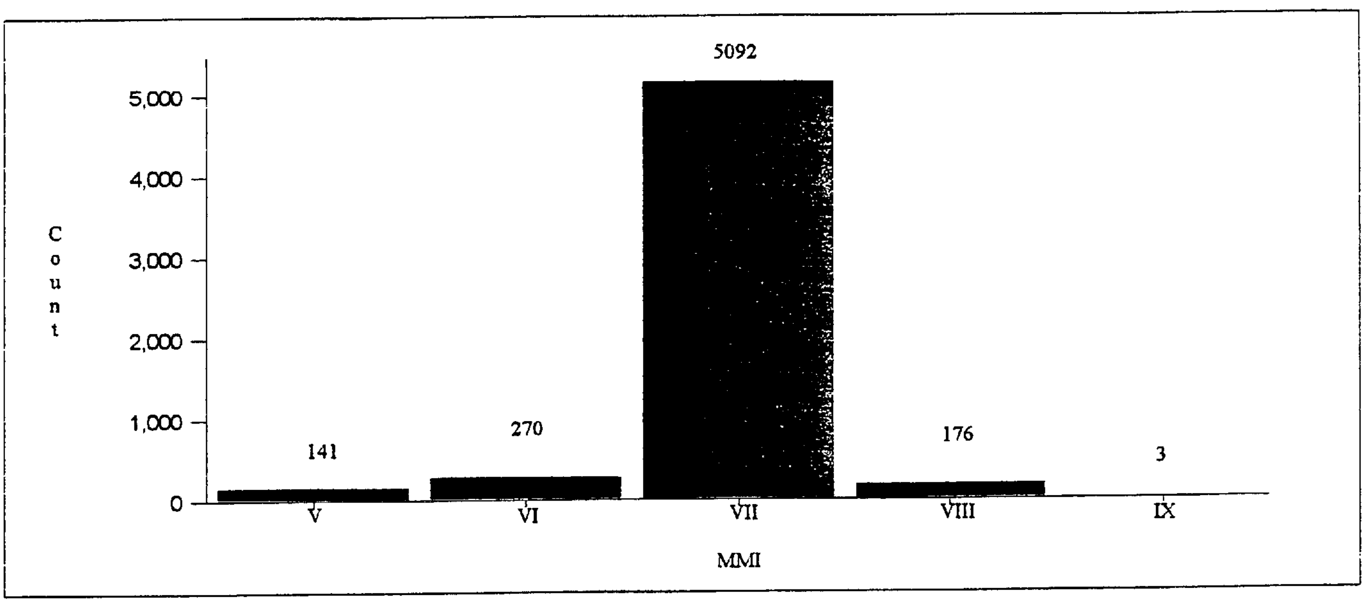
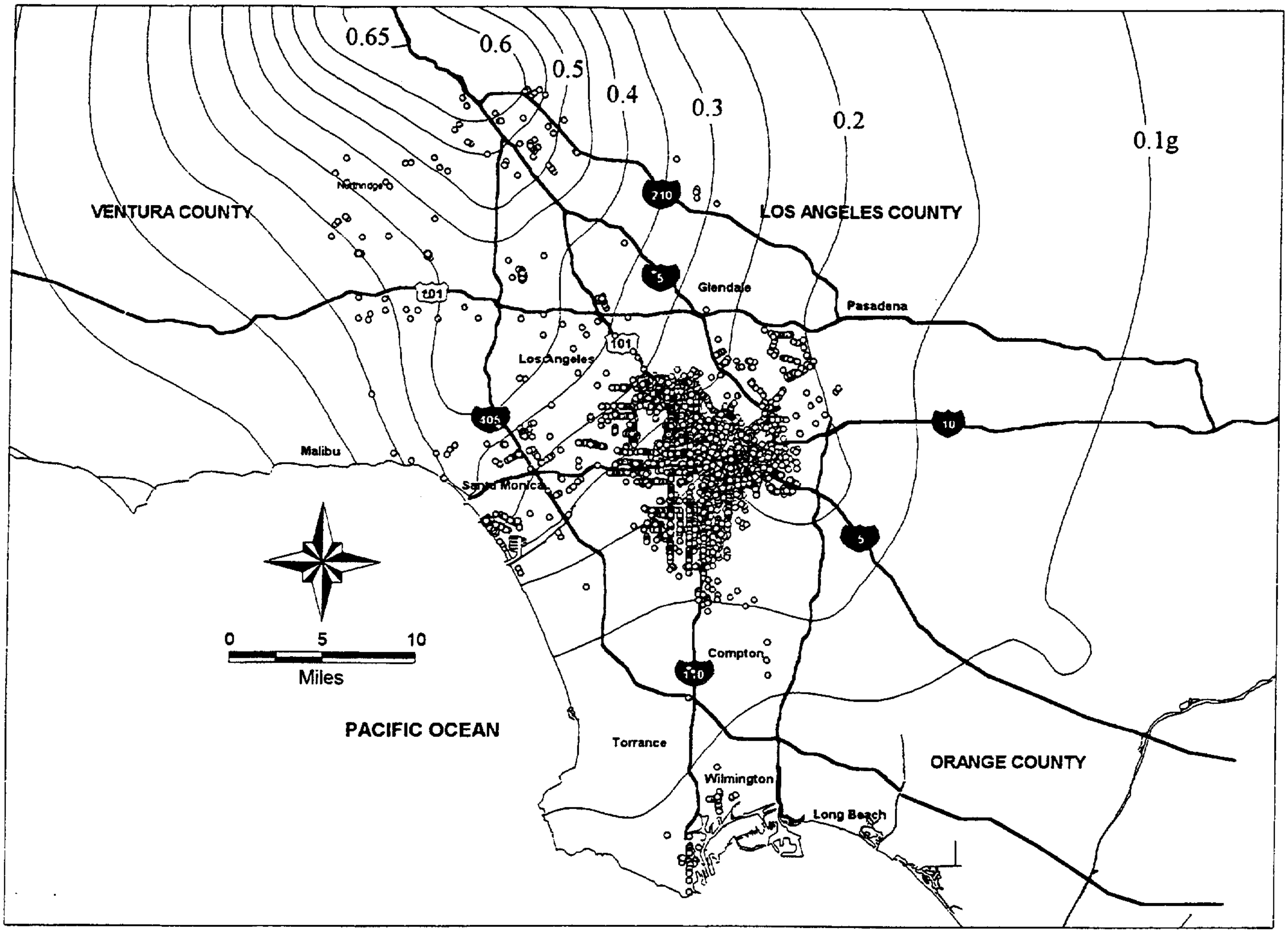


Figure 1-2: Total Retrofitted Inventory vs. MMI



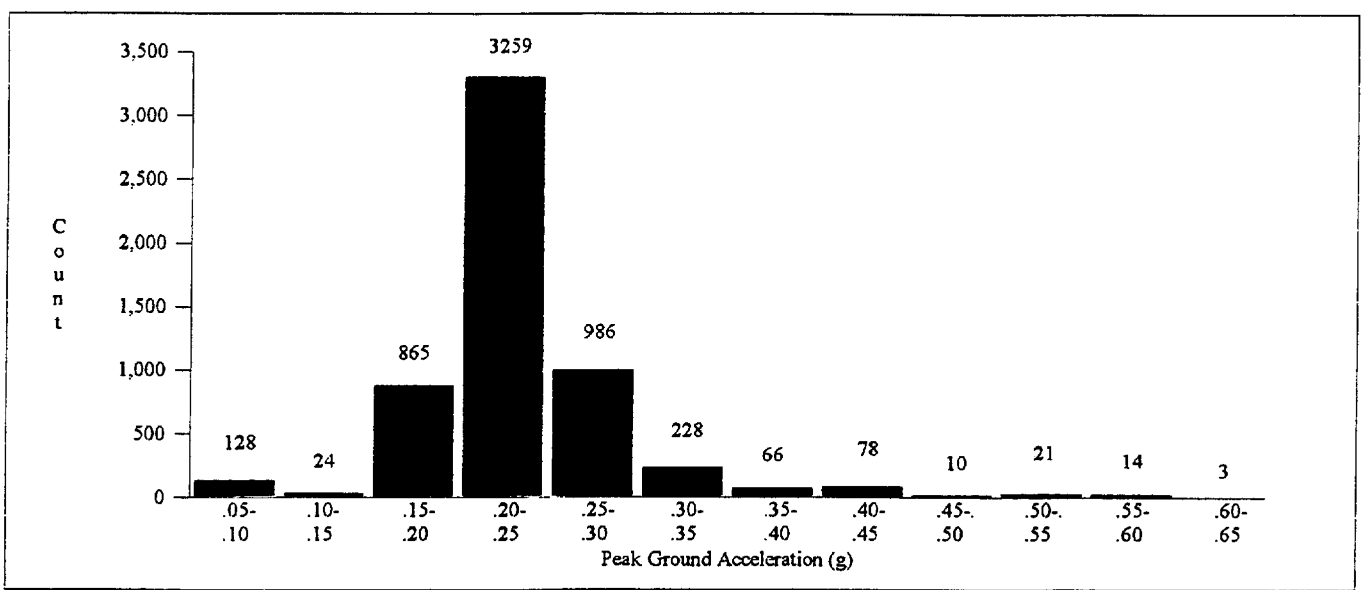


Figure 1-3: Total Retrofitted Inventory vs. Peak Ground Acceleration (g)

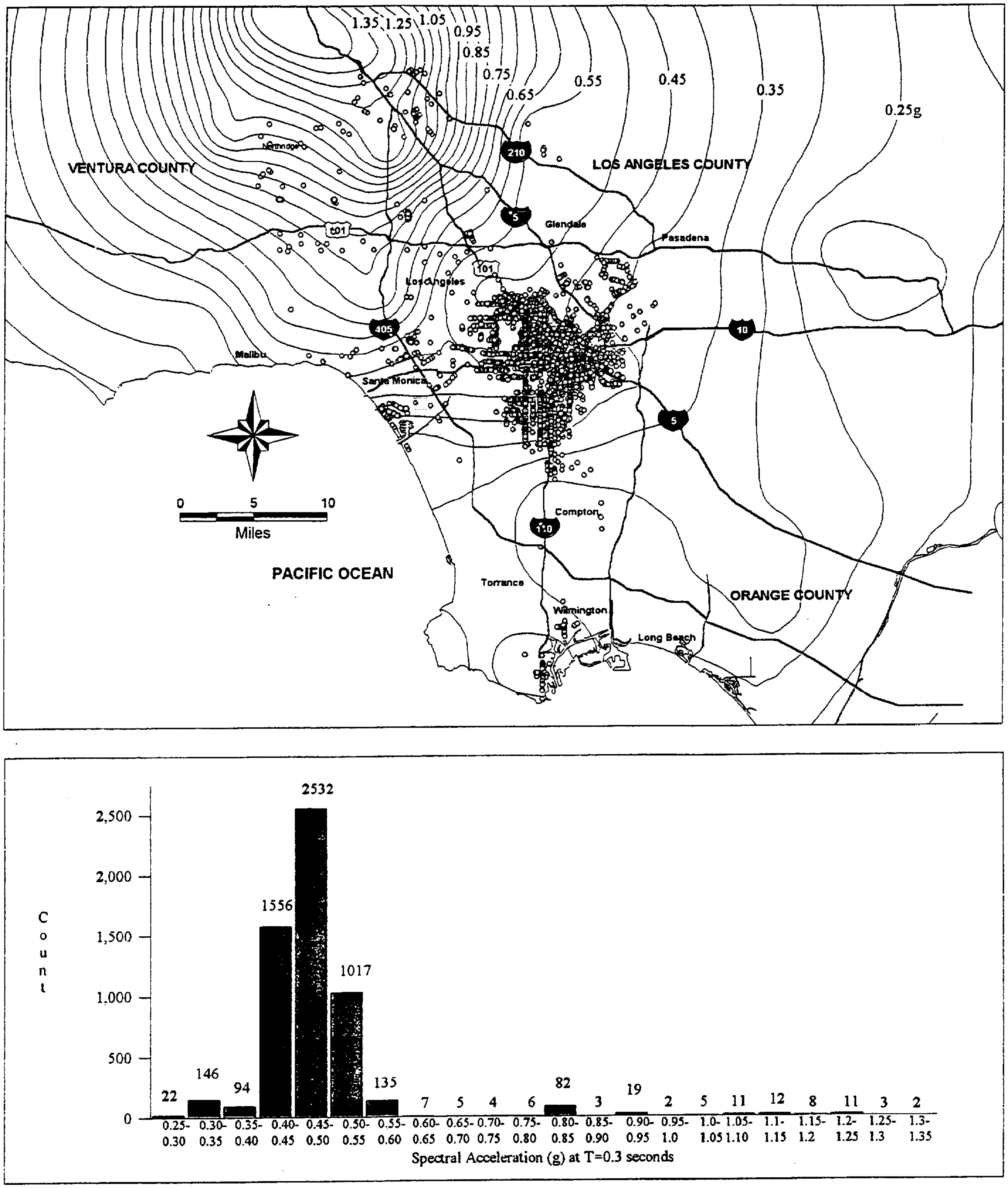


Figure 1-4: Total Retrofitted Inventory vs. Spectral Acceleration (g) taken at T=0.3 seconds

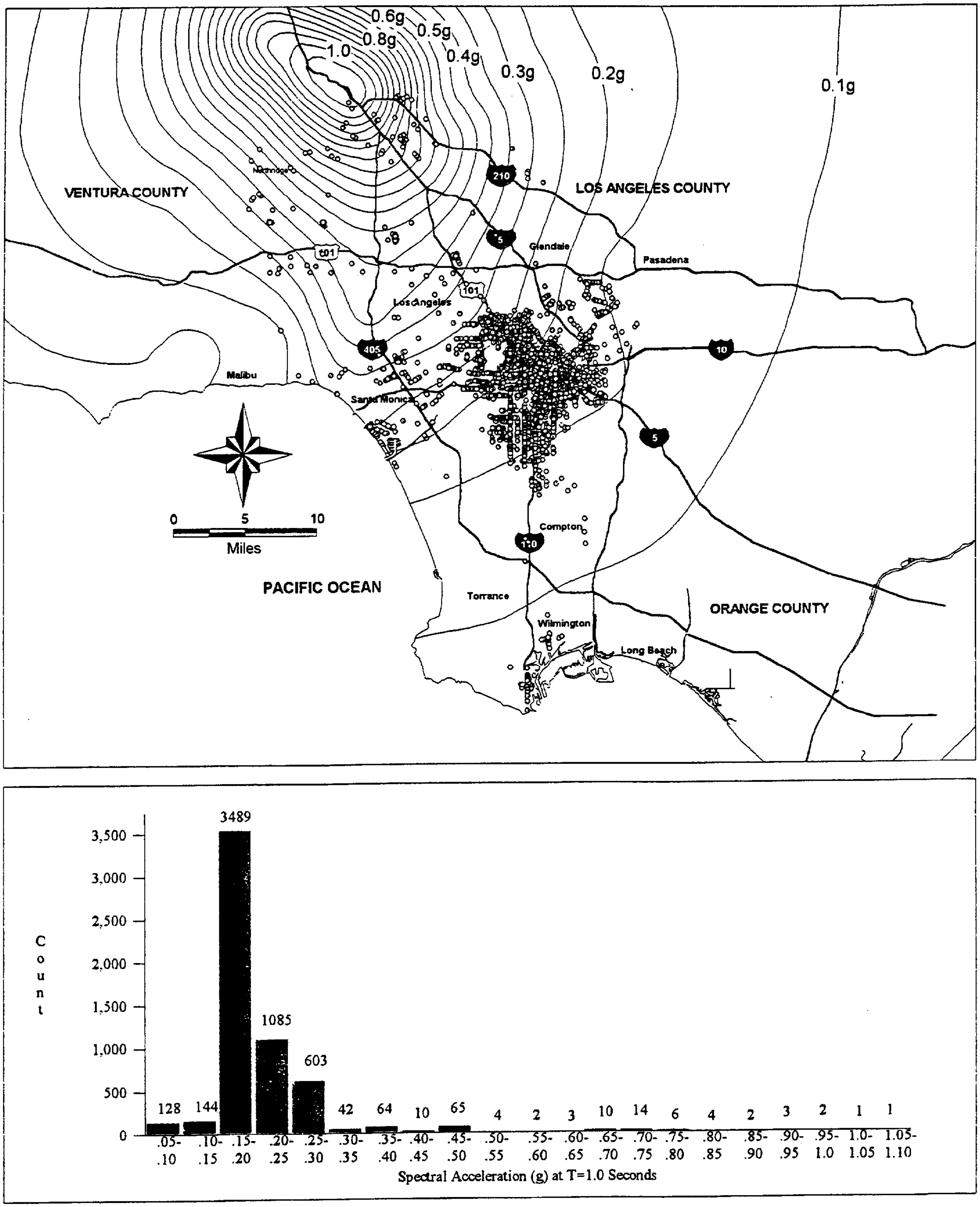


Figure 1-5: Total Retrofitted Inventory vs. Spectral Acceleration (g) taken at T=1.0 Seconds

Task 2: Applicability of Current Practice Nationwide

Background

Current retrofit methodologies for unreinforced masonry buildings are based primarily on the type of construction common to California. Some URM buildings, however, have characteristics that do not fit the California prototype and may require different analytical approaches and retrofit methods. To allow the proposed enhancement procedures to be meaningful nationwide, investigations were made of regional construction and retrofitting techniques in areas of moderate and high seismicity to determine if there are significant numbers of other types of hazardous URM buildings or construction techniques that need additional consideration, particularly when "enhanced" performance is desired.

Typical unreinforced masonry construction in five regions was explored: the Pacific Northwest (Seattle), the Wasatch region (Salt Lake City), the Central United States (Kansas City, St. Louis, and Memphis), New England (Boston), and the Carolinas (Charleston). The findings, summarized in Appendix B, include a brief description of the seismicity of the region, conclusions given in ABK (1981a), supplementary information for building categorization, and, finally, identification of building characteristics or elements which are inadequately addressed in current retrofit provisions. We reviewed and supplemented this data with additional information on regional building stocks gathered from a literature review, a consultant, and a nationwide survey of colleagues. Conclusions of the investigation are summarized below.

Summary of Observed Construction Practices for which Current Retrofit Provisions May Be Inappropriate

Several construction practices were found in the review of areas outside of California for which current retrofit provisions may be inappropriate or require refinement. These practices include ungrouted hollow masonry unit bearing walls, cavity wall construction, rigid diaphragms, and, to a lesser degree, buildings taller than six stories.

<u>Ungrouted Hollow Masonry Unit Bearing Walls</u>: Although current retrofit methodologies allow a wide range of URM materials, they were developed primarily for solid brick masonry units. Many areas outside of California have a large stock of buildings which have bearing walls made of ungrouted hollow concrete masonry units (CMU). In some areas, structural clay tile (SCT) or hollow clay tile (HCT) bearing walls are used as well. Current retrofit provisions can be applied to non-solid brick unit unreinforced masonry materials only when certain conditions are satisfied: the units are placed in a running bond pattern, the building does not exceed two stories in height, and the shear stresses do not exceed the allowable determined by in-place shear testing. What should be done with buildings which do not meet these criteria? Other issues which current methodologies do not cover include how to perform shear tests in hollow materials, whether there is an increased likelihood of toe crushing with the thin face shell, and how to provide adequate wall-diaphragm anchorage.

Cavity Wall Construction: Current evaluation methodologies assume a monolithic wall. With cavity wall construction, the wall may not act monolithically. H/t provisions may no longer apply or may be impossible to meet if each wythe is viewed as a separate thin wall.

Rigid Diaphragms: Although current retrofit methodologies allow rigid diaphragms, the original research on which these methodologies were based primarily addressed flexible diaphragms. A variety of types of rigid diaphragms are used in areas outside of California. These include concrete slabs spanning between steel I-beams, hollow concrete planks, brick arches, and HCT flat arches. These rigid floor systems have dynamic characteristics which differ significantly from flexible diaphragms. Buildings with rigid diaphragms will respond to earthquake shaking in a substantially different manner than those with flexible diaphragms. Also, capacities for some of the more unusual rigid diaphragms are difficult to establish and are not given in current retrofit provisions.

Buildings Taller than Six Stories: The Special Procedure methodology used in current retrofit provisions assumes rigid, unamplified, in-plane wall response. This assumption becomes less valid for systems in which a more flexible response is expected--i.e., taller walls or walls punched with numerous window and door openings--because amplification of the ground base acceleration is more likely. The six-story limit used in the Special Procedure is an arbitrary level based upon compromise by code writers. In California, there is a relatively small number of these buildings, and they are concentrated primarily in San Francisco and Los Angeles. In other urban areas of the country, taller buildings appear to comprise a larger percentage of the building stock. Thus, establishing the applicability of the Special Procedure for these taller buildings is worth further investigation.

Task 3: Study of Techniques to Enhance the Performance of URM Walls

Background

The objectives of this task were to summarize the theoretical background of various URM wall enhancement methods and to develop design guidelines to use with each method. Appendix C contains a group of research summaries documenting relevant experimental studies on wall enhancement methods. The summaries were prepared by Dr. Daniel Abrams and were selected by searching publications devoted to either earthquake engineering or masonry topics.

Individual publications were reviewed for their relevance to the topic of seismic strengthening methods for unreinforced masonry walls. Material is organized to explain the objective of the research, the rehabilitation procedure, the research approach, and the significant findings from the research. Where multiple publications have been written on the same research project, related references are given.

Summaries are organized with respect to the following general categories of strengthening procedures: grout of epoxy injections, surface coatings, reinforced or post-tensioned cores and miscellaneous rehabilitation techniques. In any one category, they are arranged in alphabetical order of the last name of the first author.

Recommendations

The design guidelines developed in this subtask are contained in Section 2 and provide procedures for developing practical design parameters for the rehabilitation methods found to have potential for enhancing seismic performance. For each wall enhancement method, the design guidelines provide a description of the enhancement procedure, the minimum construction requirements, and quantitative guidelines for determining the capacity of an enhanced wall. FEMA 273 (1996) guidelines were used as the basis for these provisions. Where possible, "m" factors and simplified force-displacement relationships similar to those utilized by FEMA 273 (1996) were generated. When this was not possible, additional research is identified and recommended in Section 3.2 that should be performed for verification and/or development of rehabilitation methods for enhanced performance objectives. Needed research consists of experimental studies on methods for strengthening URM walls as well as analytical studies that are needed for estimation of seismic demands for various Performance Levels.

Task 4: Benefit-Cost Considerations

The objective of Task 4 was to develop cost data and limited benefit information on the proposed enhancement procedures. The focus was on the wall enhancement methods defined in Task 3 and, to a lesser extent, on other enhancement procedures. The intent was to provide cost information that can be used by the design engineer to help determine which procedures are most cost effective on a specific project.

Cost information was obtained for a variety of sources including FEMA 156 (1988 and 1994) and FEMA 157 (1988 and 1995), Rutherford & Chekene (1990 and 1993) and engineering experience and judgment. For the wall enhancement methods, Hanscomb, Inc. provided cost estimates for the construction work, and Applied Materials Engineering provided estimates for special inspection and testing.

Because there is limited actual empirical data available on the performance of walls rehabilitated with these methods and because probabilistic estimates of the reduction in damage which could be expected when these methods are used is beyond the scope of this project, information on the benefits of the procedures was limited to qualitative discussions for most of the procedures. For the wall enhancement methods, generic estimates of the increase in shear capacity were calculated, where possible, using the design guidelines developed in Task 3.

Task 5: Development of Procedures to Enhance the Performance of Rehabilitated URM Buildings

Synopsis of the Evolution of the State of Current Practice for URM Rehabilitation

To help place the recommended procedures in context, the following synopsis of the history of URM rehabilitation is provided.

The 1933 Long Beach Earthquake left a large toll of damaged and collapsed unreinforced masonry buildings, especially school buildings. As a consequence, California's legislature passed the Field Act in 1933 which prohibited the use of unreinforced masonry in all new school buildings. In 1939, the Garrison Act was passed which required corrective steps for existing school buildings, including URM buildings. Rehabilitation lagged until the passage of the Greene Acts in 1967, 1968 and 1974 (SSC, 1979).

The first retroactive unreinforced masonry seismic ordinance of any local government was the 1949 Parapet Correction Ordinance, passed by the City of Los Angeles. This ordinance required that parapets above public access or above exits be either braced or removed. Another early retroactive unreinforced masonry seismic ordinance was implemented by the City of Long Beach in 1959 (O'Connor & Reitherman, 1984). Consulting engineer John Wiggins introduced the concept of balanced risk in his evaluation and revision of the Long Beach Ordinance, and he attempted to have the City Council set a basic acceptable risk level, which would then be converted and applied to the many different unreinforced masonry cases, such as combinations of soil types, state of repair, occupancies, etc. (Wiggins & Moran, 1971).

Following the 1971 San Fernando Earthquake, a serious commitment was made to strengthen URM buildings. In 1977, The National Science Foundation (NSF) supported private industry in a study to derive a methodology for the mitigation of seismic hazards in existing unreinforced masonry buildings. This joint venture, termed the ABK Joint Venture, consisted of three firms, Agbabian Associates, S.B. Barnes & Associates, and Kariotis & Associates in contract with the National Science Foundation. This research was reported in ABK (1981a, 1981b, 1981c, 1984, 1986).

The Los Angeles City Earthquake Hazard Reduction Ordinance (commonly known as Division 88), was adopted on February 13, 1981 and incorporated the results of early ABK investigations. ABK (1984) also provided the technical basis for the Rule of General Application (RGA) adopted by the City of Los Angeles in 1986 as an alternative to Division 88. Los Angeles' ordinance required all unreinforced masonry bearing wall buildings to be demolished or seismically rehabilitated. Exemptions were permitted for one- and two-unit single family construction and detached apartment buildings less than five units. The mandatory strengthening program requires building owners to strengthen their buildings within established time frames ranging from one to more than seven years, depending on the number of building occupants.

The first general California state law regarding unreinforced masonry buildings, Senate Bill 445, passed in 1979. This bill urged local governments to voluntarily identify buildings for strengthening. By the date of the Coalinga Earthquake in 1983, few buildings had been strengthened. SB 547 was passed by the legislature in 1986; it required local jurisdictions to prepare an inventory of URM buildings and develop a hazard mitigation program in their communities by January 1990. The *Guidebook to Identify and Mitigate Seismic Hazards in Buildings* (SSC, 1987) was prepared by the Seismic Safety Commission to help local jurisdictions comply with the law. It contains a model ordinance based upon Division 88 (1985).

The Structural Engineers Association of California (SEAOC) and the California Building Officials (CALBO) joined forces in 1988 to review the existing ordinances, including Division 88 (1985), the Seismic Safety Commission Model Ordinance in SSC (1987), the UCBC (1988), and the Los Angeles RGA, in order to develop comprehensive technical provisions based on the current state-of-the-art concepts on hazard reduction and to write a commentary to explain those provisions. The joint SEAOC/CALBO URM Provisions were formally approved in September, 1990 as the 1991 UCBC Appendix Chapter 1 (UCBC, 1991). The State of California adopted the UCBC as its Model Ordinance. This ordinance has, subsequently, been used by many communities. An ultimate strength version of the UCBC was adopted as an appendix in the FEMA 178 (1992) evaluation document. As part of the ICBO code update process, UCBC Appendix Chapter 1 was modified in 1994 (UCBC, 1994).

The UCBC contains two methodologies for evaluation and strengthening of URM buildings: a General Procedure and a Special Procedure. The General Procedure references the UBC's provisions for new buildings and uses a traditional lateral force analysis but provides allowable material stresses for the unreinforced masonry walls. The Special Procedure uses the ABK methodology.

In this report, the phrase "current retrofit practice" is used to mean the standard of practice embodied in the current UCBC since it is the most complete method for rehabilitation.

Relationship of the Recommended Procedures and FEMA 273 (1996)

The purpose of this project was to identify procedures to obtain improved seismic performance in rehabilitated URM buildings and to describe the procedures in a common framework to facilitate comparison and implementation by others. The identified procedures were not necessarily intended to be mandated code changes, but rather options that could be exercised by engineer and owner to either obtain more reliability in expected performance or to reduce expected damage.

During completion of Tasks 1 through 4 described above, a framework for such a set of procedures came into focus in a major national FEMA-funded project, which has resulted in the *NEHRP Guidelines for the Seismic Rehabilitation of Buildings* (FEMA 273, 1996) document. FEMA 273 (1996) is structured around the concept of voluntary selection of various Performance Levels, and appropriate nomenclature and vocabulary has been created and defined. Methods and criteria to achieve several different levels of seismic protection are included. Performance is defined by selecting a Rehabilitation Objective, which consists of a desired Performance Level, such as Immediate Occupancy, Life Safety, or Collapse Prevention, for either a specific earthquake or a ground motion with a certain probability of occurrence (see Figure 1-6). A desired level of performance can be selected for an individual building, or mandated as a minimum by a community when adopting a risk reduction ordinance. The document warns that due to the many variables associated with unknown conditions in existing buildings, deterioration of materials, site conditions, and particularly with ground motions, compliance with a given set of requirements should not be considered a guarantee of the intended specified performance.

The bulk of FEMA 273 (1996) is devoted to development of the Systematic Rehabilitation Method, which requires complete analysis of the building structure and verification of the adequacy of both existing and new structural elements for realistic deformations expected during earthquake shaking. A Simplified Rehabilitation Method is also included which may be applied to certain small and simple buildings as specified in the document. These simplified techniques generally require few and simpler calculations and may be applied to either the most hazardous aspects of a building or the entire building. The Simplified Rehabilitation Method adopts FEMA 178 (1992) as the basic reference. Although the intent of commonly used URM ordinances such as the UCBC is often described vaguely as "risk reduction," FEMA 178 (1992) is described in FEMA 273 (1996) as providing a Rehabilitation Objective of Life Safety for an event with a 10% chance of exceedance in 50 years (the standard level of shaking used in most codes). Use of Simplified Rehabilitation Methods for URM buildings is limited to buildings without irregularities and to buildings with two stories in high seismicity zones and three stories in moderate and low seismicity zones.

The results of this study have been put into the framework of FEMA 273 (1996), particularly the design guidelines for enhancing performance of URM walls. Based on the results in the Northridge Earthquake and other studies within this project, the procedures that are described will either increase the reliability of reaching a specified performance, or can be used as part of a rehabilitation aimed at improved damage control, as defined by FEMA 273 (1996). Selection of a higher Performance Level in the damage control range could be driven by a desire to reduce the cost of repairs or the time the building may have to be vacated, or by concern for performance in ground motions larger than implied by current codes and standards.

The evaluation and design methodologies of FEMA 273--estimating expected displacements from earthquake motion and comparing them directly with acceptability criteria deduced from laboratory testing--had previously not been contained in U.S. codes or standards. Many of the numerical coefficients, particularly the displacement acceptability criteria, have therefore not been calibrated with extensive use in actual evaluation and design. The ballot and comment portion of the BSSC consensus process may generate changes to the ballot draft in the values or format of the design acceptability criteria of materials, particularly masonry. Users should check compatibility of this document with the latest version of FEMA 273 and validate results using alternate methodologies and engineering judgment.

Building Performance Levels and Ranges

Performance Level: the intended post-earthquake condition of a building; a well-defined point on a scale measuring how much loss is caused by earthquake damage; in addition to casualties, loss may be in terms of property and operational capability.

Performance Range: a range or band of performance, rather than a discrete level.

Rehabilitation Objective: The combination of a Performance Level or Range with Seismic Demand Criteria.

Structural and Nonstructural: Separate structural and nonstructural performance levels or ranges are combined to form the overall building performance level/range.

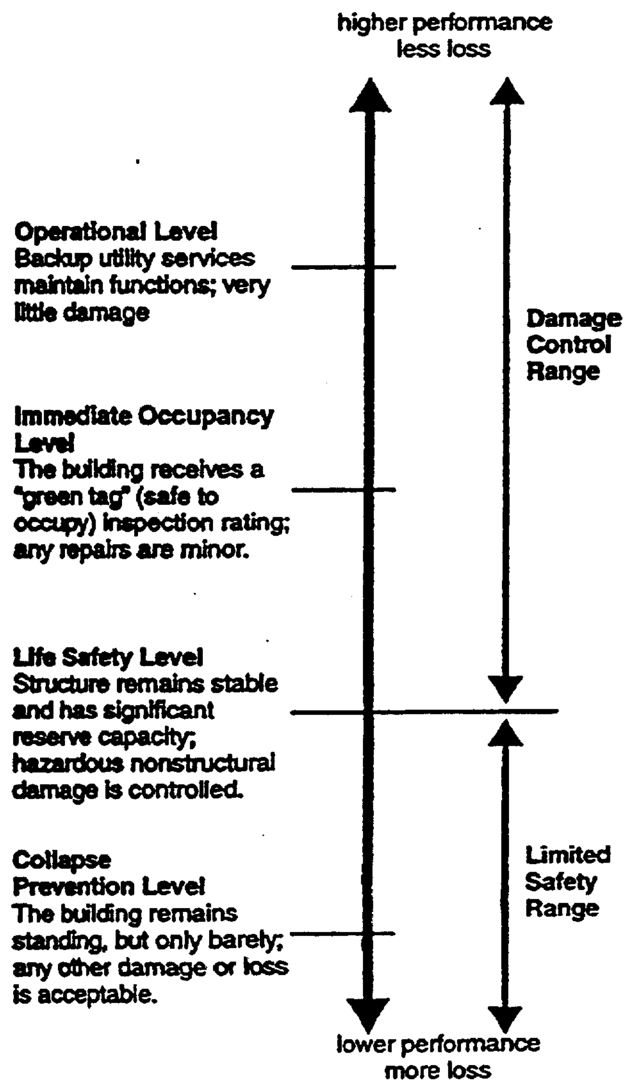


Figure 1-6: FEMA 273 (1996) Building Performance Levels and Ranges

Section 2 : Procedures to Enhance the Performance of Rehabilitated URM Buildings

2.1 Scope

Enhanced performance as used in this study includes either increased reliability of achieving an intended Performance Level or successful achievement of a Performance Level higher than expected from the current standard of practice. Figure 2-1 illustrates this concept with three curves (probability density functions) of performance. Curve 1 shows how performance might be distributed in a design earthquake for buildings rehabilitated using current methodologies and standards of practice. Instead of a discrete point, the target Performance Level is represented more realistically as a range of performance. Most of the buildings will lie within or above the target performance range, but some percentage of poor performing buildings will fall below. One method of reducing this percentage is to implement procedures which provide increased reliability, such as those which enhance the quality of design and construction. Curve 2 shows graphically the resulting distribution. The shaded area shows the percentage of buildings which were performing below the target range that have now been moved up into the target range. The percentage of buildings beyond the target range does not change appreciably, since the best buildings already are likely to have high quality design and construction. A second method of enhancing performance is to aim for a higher target performance range for all buildings. This can be accomplished with enhancement procedures which cover a greater scope of work, use higher design criteria, or which explicitly protect against damage (not just life safety). Curve 3 shows that this is likely to simply shift the entire distribution upwards. Note that this will also result in fewer buildings falling below the original target range. In practice, it may be difficult to distinguish between procedures which increase reliability and those which aim for a higher target range; the end result of most rehabilitation schemes is likely to result in a distribution which is a cross between Curves 2 and 3.

The procedures described in this section are those that were identified as a result of completion of the background tasks of this project, as well as study of other published reports concerning the Northridge Earthquake, and are not suggested to be the results of an exhaustive review of all aspects of the current practice in the rehabilitation of URM buildings.

It was found in Task 2 of this study that a significant number of existing URM buildings outside of California have characteristics that cause uncertainty in the correct application of current practice or are not covered at all. For example, buildings with rigid diaphragms or cavity wall construction have not been extensively rehabilitated and are not covered by current standards in the same detail as the typical West Coast URM. Extension of current practice for these building types without additional guidelines may produce inconsistent and unreliable performance. However, there is no damage history or research results upon which additional recommendations can be based for these buildings and elements, and the best that can be done within the context of this project is to include such informational needs in Section 3.

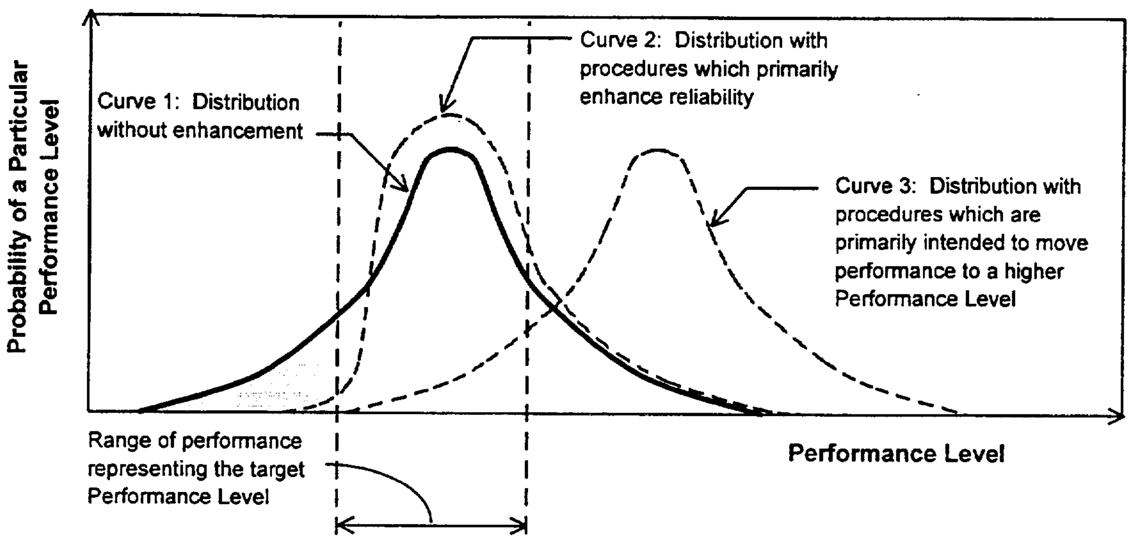


Figure 2-1: Probabilistic Depiction of URM Rehabilitation Performance Enhancement

The recent BSSC/USGS national ground motion mapping effort, called Project 97, may result in a redefinition of traditional design ground motions. Spectral ordinates are mapped rather than effective peak ground acceleration, and the results of large, rare events are weighed more heavily than in the current standard of practice. FEMA 273 (1996) defines a Rehabilitation Objective as achievement of a given Performance Level for a specified ground motion. Both the Performance Level and the severity of the ground motion can be varied by the user. This project has focused on parameters that measure the performance portion of a Rehabilitation Objective, and the potential effects of changes in national ground motion mapping have not been studied.

As discussed in Section 1, the procedures are generally described in the context of FEMA 273 (1996), although many of the recommendations are independent of the particular code or standard being used. The URM provisions contained in FEMA 178 (1992) and the Uniform Code for Building Conservation (UCBC, 1994) are also referenced because at this time they more closely describe the current state of the practice for seismic rehabilitation of URM buildings. The UCBC has been updated several times since FEMA 178 (1992) has been published and best represents the current standard of practice in the west.

The procedures suggested to enhance performance are described in Section 2.2 and the potential cost effectiveness of their application is discussed in Section 2.3. Procedures to enhance performance of URM buildings are divided into three categories: 1) improving the quality of design or construction, 2) changing traditional design criteria, and 3) implementing various types of rehabilitation methods.

2.2 Procedures to Improve the Quality of Design and Construction

In general, improving the quality of design and/or construction should result in improved reliability of performance. Damage data, particularly for URM buildings, consistently indicates a wide variety of performance for similar levels of shaking. For retrofitted buildings in the Northridge Earthquake, much of the poor performance was attributed to poor quality design or construction (LATF, 1994 and SSC, 1995). Each of the procedures described below is expected to reduce the possibility of having unexpectedly or unusually poor performance.

Improve Knowledge of the Building

Determination of the appropriate level of investigation of existing conditions is always an issue for seismic rehabilitation of existing buildings. Building owners are often reluctant to budget testing and investigation services sufficient to minimize unknowns that may affect performance. Extensive experience with rehabilitation of typical California URM buildings has facilitated development of a set of minimum information requirements for that building type contained in the UCBC. In addition, FEMA 273 (1996) contains guidance on the minimum amount of configuration and materials information required for design of rehabilitation measures. However, a certain degree of interpretation is possible concerning both of these requirements, particularly when considering the variations in URM construction nationally.

To increase the reliability of reaching or exceeding the Performance Level incorporated into the desired Rehabilitation Objective, it is recommended that investigation of in-situ conditions be thorough and complete. The information about the building required by the UCBC, if responsibly collected, when coupled with appropriate material testing, should yield a cost effective basis for the design, regardless of Rehabilitation Objective. However, if the engineer of record will not provide construction observation and will thus not be able to adjust details for unknown conditions, more detailed information may be needed in the design phase. Similarly, if initial investigation and testing indicates that field conditions are highly variable, additional exposures or testing may be needed to adequately analyze the building and develop representative details. The UCBC (1994) list of required information is duplicated below in italics, and the significance of the various information requirements is discussed.

Requirements for Plans. The following construction information shall be included:

1. *Dimensioned floor and roof plans showing existing walls and the size and spacing of floor and roof framing members and sheathing materials. The plans shall indicate all existing and new crosswalls and shear walls and their materials of*

construction. The location of these walls and their openings shall be fully dimensioned and drawn to scale on the plans.

Discussion: Original construction documents are often not representative of field conditions or are not available at all. However, the overall configuration and material make-up is key to estimates of mass and structural stiffness. The floor and roof section and interface with the masonry wall must be known. Although not required by the UCBC, confirmation of the adequacy of the crosswall-to-diaphragm load transfer mechanism will increase reliability of performance if crosswalls are used.

2. *Dimensioned wall elevations showing openings, piers, wall classes..., thickness, heights, wall shear test locations, and cracks or damaged portions requiring repairs, the general condition of the mortar joints and if and where pointing is required. Where the exterior face is veneer, the type of veneer, its thickness and its bonding and/or ties to the structural wall masonry shall also be noted.*

   Discussion: Determination of the type, quality and dimensions of the masonry is essential. The type, layup and dimensions of the masonry will influence not only in-plane wall strength but also the appropriate h/t for out-of-plane considerations. Improper assumptions in this regard can lead to peeling of outer wythes or complete out-of-plane failures. Coring may be necessary to determine the characteristics and effective thickness of walls with veneers, cavity wall construction, and two-wythe stone masonry. For veneer and cavity wall construction, the spacing and condition of steel ties and relief angles must be determined.

   Determination of in-plane failure modes and performance characteristics is sensitive to length-to-height ratios of walls and piers, so representative wall elevations also must be developed or confirmed in the field.

   Tests are almost always required to determine the quality and material properties of the masonry. Weak mortar, near the lower limits of acceptability, leads to increased damage (LATF, 1994), and consideration should be given to strengthening or reducing the demand in such walls. Pointing of the outside face of walls should not be assumed to restore integrity to masonry without evaluation of the condition of the mortar throughout the wall.

   Locating push tests on both inside and outside faces of wall is a good method for decreasing uncertainty of masonry characteristics and variability.

3. *The type of interior wall and ceiling materials and framing.*

   Discussion: Interior walls may be concrete block or clay tile with plaster finish. These walls will not only contribute to the mass of the building but may form a significant portion of the building stiffness and damageability. The extent and make-up of other interior partitions is also important, either for approximations of mass or for use as crosswalls (per the UCBC Special Procedure).

Plaster or gyp-board ceilings framed separately from the floors or roof must be identified. The stiffness of such ceilings or their framing is seldom included in analysis but has caused damage to both ceiling and URM walls at their perimeter.

4. *The extent and type of existing wall anchorage to floors and roof when used in the design.*

   Discussion: Existing wall anchors must be tested if they are to be considered effective. Tests should be made on anchors representative of the typical conditions. However, because of unknown conditions that may affect each existing anchor, more reliable performance can be obtained from new anchors.

5. *The extent and type of parapet corrections which were previously performed, if any.*

   Discussion: Previous parapet corrections may have been placed in accordance with outdated criteria or may have deteriorated and should be checked for adequacy and consistency with the current Rehabilitation Objective.

6. *Repair details, if any, of cracked or damaged unreinforced masonry walls required to resist forces specified in this chapter.*

   Discussion: Evidence of wall repair could indicate damage from a previous earthquake or settlement. Repairs are often cosmetic, and the full extent of the damage should be determined before a wall capacity is assumed. The LA Task Force reported that previously damaged buildings appeared to suffer more damage than others, probably due to inadequate repairs (LATF, 1994)

Thorough Design

To obtain optimum performance from a given rehabilitation scheme, the detailed knowledge of the characteristics of an individual building, as discussed in the previous section, must be fully utilized in design. To further reduce the risk of significant damage, characteristics that may not be specifically covered by codes and guidelines that tend to cause or increase damage must be identified and, to the extent possible, mitigated. The following issues should be considered.

Completeness of Drawings and Specifications

If field conditions are not adequately covered on the drawings, inapplicable or inadequate details may be developed in the field. This is often the case when typical details—developed for wall-floor connections, installation of plywood over sheathing, or other recurring conditions—do not apply. In such cases, field adaptations may be made that reduce the effectiveness of the detail.

Detailing at Corners

It was observed in the Northridge Earthquake that URM walls at or near corners of buildings received abnormally high damage, often in the form of vertical cracks. Similar damage has been observed in other earthquakes as well. This may be due to the incompatible stiffness of walls in-plane and out-of-plane, in addition to slight shear slippage of diaphragms tending to push out the perpendicular wall at the corners. This damage can probably be reduced by placing a steel tension tie, similar to a diaphragm chord, at each corner at each level, extending 8-10 feet from the corner in both directions and connected together at the corner. If practical, additional similar ties could be placed between diaphragm levels.

Special Consideration of Rigid Ceilings

Even though FEMA 273 (1996) emphasizes deformation compatibility and encourages inclusion of all significant elements in analysis, it is unlikely that the effects of rigid ceilings located apart from floor framing will be considered unless an especially thorough design is undertaken or unless specifically required by the governing code (e.g. the UCBC specifically requires ceiling-to-wall anchorage in certain circumstances). Rigid ceilings can cause damage to URM walls if they move independent of floors, or, in some cases, if they force damaging deformations in the wall by providing unexpected lateral stiffness. The plan configuration of the ceiling, as well as its rigidity and vertical and lateral connection to the floor, must all be considered to decide if damage will be minimized by anchoring the ceiling to the walls, releasing the ceiling from the walls, or changing the support conditions of the ceiling.

Special Consideration of Veneers

Veneers, or any outer wythe of a multi-wythe wall not adequately tied back, have consistently demonstrated a vulnerability for separation from the remainder of the wall and collapse to the ground. Such conditions must be identified and the existing support system exposed and investigated for strength and deterioration. The condition of metal ties should be checked at their connection both to the veneer and to the base wall material. Reliability can be enhanced by additional exposures or in-situ tests for strength. Performance under extreme loadings can be enhanced by decreasing the currently required 24" spacing of ties.

Nonbearing URM Walls

The presence of a large number of interior masonry partitions or nonbearing walls represents potential damage that is difficult to control. Often, interior masonry walls will not be capable of either supporting tributary loads from a flexible diaphragm or undergoing the expected lateral displacements without significant damage. Removing or strengthening the walls or freeing them from diaphragm movement is often very expensive. It is important, however, to recognize the damage potential and study alternate means of mitigation.

Damaged or Deteriorated Masonry

If damage is suspected based on historical records or field evidence, it is important to determine the extent and severity and realistically consider the effect on capacity. Many of the wall enhancement methods discussed in this document can be used to provide additional capacity to damaged walls.

Configuration Irregularities

The fact that irregularities in configuration cause or increase damage is not unique to URM buildings. Any rehabilitation undertaken with a special concern for performance should eliminate to the extent possible vertical irregularities. In most instances, plan irregularities cannot be eliminated and must be mitigated with careful detailing and/or new elements placed to minimize torsion or incompatible response of adjacent elements. Detailing should consist of securely tying the building together horizontally with collectors at reentrant corners, and corner ties, as described above, at outside corners.

Written Design Criteria

To improve communication with the owner and to facilitate internal and external checking, it is recommended that written design criteria be developed, including the intended Rehabilitation Objective, the primary design standard, and any revisions and additions to this standard. Due to inherent limitations in reliability of URM under seismic loading, it should be emphasized that the Rehabilitation Objective should only be considered the goal for the design, and a given Performance Level cannot be guaranteed.

Plan Check/Peer Review

To further assure that performance will not be compromised by lack of thoroughness or inadvertent omission or error, it is recommended that the completed plans and specifications be checked for conformance with the project design criteria. Depending on the thoroughness and competence of the jurisdictional plan check, as well as the complexity of the project, an independent peer review may be justified.

Field Review

Field review and construction quality control is an important aspect of overall quality control in all construction projects. It is especially important in the seismic rehabilitation of URM buildings due to the high likelihood of discovering unforeseen conditions during construction. A significant step in increasing reliability of rehabilitation of URM buildings consists of a field quality control program created and monitored by the engineer of record, which includes special inspection and procedures for review of required submittals and proposed changes, and augmented by structural observation by the engineer of record.

2.3 Procedures Related to Design Criteria

Changes in design criteria are generally intended to move performance to a higher Performance Level rather than to increase the reliability of reaching a specified Performance Level. It was originally assumed that statistical analysis of the detailed damage database (see Appendix A) would yield vulnerability levels of various URM building elements as a function of ground shaking intensity that could be used to deduce performance-sensitive changes in design criteria. The database proved to contain inadequate detail to obtain such information, and the only available parameter for study was *percent damage*, a nonspecific measure of the overall damage level. The relationship of percent damage to ground motion intensity was studied, both to deduce the potential results of global design criteria and to compare results with previously published damage probability matrices. The results of those studies are summarized in this section. Other possible variations in overall design criteria are also discussed.

The Scope of Regulated Elements

FEMA 178 (1992) indicates which elements typically found in URM buildings must be checked for conformance with numerical acceptance criteria (see Table 2.1). The premise of the table is that, for low levels of shaking represented by A_V less than 0.2, certain deficiencies are extremely unlikely to cause unacceptable damage. In the FEMA 273 (1996) document, FEMA 178 (1992)--through Simplified Rehabilitation--can be used for regular, lowrise URM buildings to achieve a Rehabilitation Objective of Life Safety for a 500-year event. For building configurations outside this range or for enhanced Rehabilitation Objectives, Systematic Rehabilitation procedures are required for which all elements of the lateral force-resisting system must be checked against the selected performance criteria. For the purpose of this document, it is recommended that, for any rehabilitation project for which the reliability or the level of performance is an overriding concern, *all* elements of the lateral force-resisting system be checked, not just those checked in Table 2.1.

Balancing Design Criteria for Cost Effectiveness

In any situation where there is interest in rehabilitation measures beyond the prescriptive minimum, either for a specific Performance Level or simply for cost effectiveness, a common procedure is to use the principles of capacity design to determine the most cost effective increase in the minimum global design levels. This is most effectively done as follows:

Table 2-1: Elements Regulated by FEMA 178 (1992)

Building Elements	A_v			
	≥0.05 <0.1	≥0.1 <0.2	≥0.2 <0.3	≥0.3
Parapets	x	x	x	x
Walls, anchorage	x	x	x	x
Wall, h/t ratios		x	x	x
Walls, in-plane shear		x	x	x
Diaphragms[a]			x	x
Diaphragms, shear transfer[b]		x	x	x
Diaphragms, demand capacity ratios[b]			x	x

[a] Applies only to buildings designed according to the General Procedure.
[b] Applies only to buildings designed according to the Special Procedure.

1. Using the minimum prescribed seismic demand and the minimum rehabilitation scheme designed to meet this demand, calculate the Demand/Capacity (D/C) ratios of key elements such as existing URM walls, added lateral force-resisting elements, wall-to-diaphragm ties, and diaphragms. D/C ratios can be applied to force- or deformation-controlled actions. Consider the example rehabilitated building represented in Table 2.2. For a particular direction of earthquake shaking, the key lateral force-resisting elements are four walls, the shear and tension ties between the diaphragm and walls, and the diaphragm itself. Due to the relatively fixed building configuration for most rehabilitations, D/C ratios for these elements are likely to be inconsistent. The D/C ratios in the column "Minimum Required" represent the condition after the originally deficient building has been rehabilitated to a given minimum standard such as FEMA 178 (1992) or UCBC (1994). The shaded cells indicate the elements that were originally deficient and required strengthening. Wall #1 and Wall #2 were in some way strengthened, and due to use of standard details or the relative lengths or the walls, Wall #2 provides slightly more strength than minimally required (D/C=0.9). Shear and tension ties were placed at logical, even spacing that also provided slightly more strength than required (D/C=0.9).

2. Estimate the cost of a logical increase in overstrength by matching the entire design to the element with the highest D/C ratio less than 1.0. Due to overstrengths inherent in the system, all that is required to provide a lateral force-resisting system with 11% more capacity than the original minimum is to strengthen Wall #1 to the new level. The cost of that increase is little since only one element requires

additional work. The D/C ratios of each element when the demand is increased by 11% is shown in the "Minimum Demand x 1.11" column. This example indicates an overall cost increase of only 5% as shown in the "Cost Factor" line.

Table 2-2: Use of Demand/Capacity Ratios to Determine Cost Effective Design Levels

Element	Demand/Capacity Ratio for a Given Demand			
	Minimum Demand	Minimum Demand x 1.11	Minimum Demand x 1.25	Minimum Demand x 1.42
Wall #1	1.0	1.0	1.0	1.0
Wall #2	0.9	1.0	1.0	1.0
Wall #3	0.8	0.89	1.0	1.0
Wall #4	0.7	0.78	0.87	1.0
Diaphragm	0.6	0.67	0.75	0.85
Tension ties	0.9	1.0	1.0	1.0
Shear ties	0.9	1.0	1.0	1.0
Cost Factor	1.0	1.05	1.2	1.6

1. Shaded cells indicate that strengthening is required to achieve increase.
2. Cost factor represents cost relative to minimum.

3. Find other logical increments with similar cycles. Fully utilizing the inherent strength still left in Wall #3 (D/C=0.89) is more difficult than implementing the initial strengthening of Wall #1 described in step 2 above, because it requires strengthening of Wall #1 and Wall #2, as well as the wall ties. A cost for this increase is estimated at a 20% premium.

4. Continuing the process, if the element with the next highest D/C ratio is fully utilized (Wall #4), relatively major strengthening would be required for Walls #1, #2, and #3 as well as wall ties. Although a 42% strength increase has been achieved, a large cost premium of 60% is required.

5. Consider the probable failure sequence of the building elements and resulting effects on global behavior and adjust the apparent benefits of the increments accordingly. For example, if strengthening a wall has changed the failure mode from rocking to shear, the benefit of the overall increase in capacity may be deceiving.

6. Present cost increases and probable benefits to the owner. In the example case, the 11% increment is almost "free" and probably would be accepted. (Depending on their engineering philosophy, some engineers may provide an improvement such as this without consultation with the owner.) The next increase represents a significant cost increase but also a significant increase in capacity, and the decision would depend on the importance of enhanced performance to the owner. The

increase in capacity represented by the last column represents a major cost increase and possibly major reconstruction of the building. Development of such an extreme strengthening level, however, forms an upper bound of performance possibilities and, in conjunction with the first two study levels, provides a useful relationship between cost and performance.

It should be noted that rehabilitation experience in California indicates that, in the highest seismic zones, economically obtainable capacities of most URM buildings are already somewhat balanced by demand. In these areas, capacity design analyses of this type will seldom find significant excess global capacity. However, in moderate and low seismic zones, large increases in capacity may be available for small increases in construction costs.

Lessons Based on Analysis of Damage Patterns from the Northridge Earthquake

Damage data was obtained for over 750 buildings rehabilitated in accordance with the requirements of the City of Los Angeles (essentially equivalent to UCBC/FEMA 178). These damage data were integrated with the overall inventory of URM buildings maintained by the City of Los Angeles, yielding a total database of 5682 buildings. A simplifying assumption was made that all buildings not included in the damage data (not investigated by the City) had no (literally zero) damage. Other possible assumptions of distribution of damage in this large inventory would tend to add several percentage points to overall damage in the lesser damage categories but would not affect analysis of the more extensive damage states.

Extensive analysis of this large database, unfortunately, yielded few conclusions that could be used to develop guidelines to enhance performance of rehabilitated URM buildings, particularly specific changes in design criteria. The characteristics and limitations of the database and our analysis are discussed in Appendix A.

However, an overall review of the data suggests the following general conclusions pertinent to enhancing performance of rehabilitated URM buildings:

- The overall damage level, including the well documented cases of damage caused by nonconformance with applicable standards, was less than estimated by EERI in the publication *Expected Seismic Performance of Buildings* (EERI, 1994), as shown in Table 2.3. There were essentially no rehabilitated URM buildings exposed to MMI IX. Based on these data, the level of damage expected from the current state of practice can be moderated, *particularly if design and construction quality is high*. This could affect the need for and desirability of additional enhancements.

- Statistically, certain building configurations suffered more damage. Buildings of 4-6 stories were more damaged than those of 1-3 stories; buildings with high vertical aspect ratios were more damaged than more squat buildings; buildings with high aspect ratios in plan were more damaged than more squarish buildings--

Table 2.3: Percentage of Retrofitted URM Buildings in Various Damage States Compared With EERI (1994)

Ground Motion Level	EERI Damage State			
	None/Slight	Moderate	Extensive	Complete
EERI MMI VII	60-100	10-20	2-10	<1
Actual MMI VII	92.8	3.3	3.2	0.7
EERI MMI VIII	30-50	20-30	25-35	2-10
Actual MMI VIII	82.4	7.4	8.5	1.7

due to more flexible diaphragms; and buildings without basements performed worse than those with basements. Although no specific change in design criteria can be recommended from these results, it could be noted than attempting to obtain enhanced performance in buildings with any of these characteristics may require additional effort.

- It appears that the buildings in this database, rehabilitated to UBC Zone 4 (A_a=0.4) criteria, suffered little or no damage for ground motions less than associated with A_a of about 0.2. It can be expected, therefore, that buildings in moderate seismic zones that are rehabilitated to similar standards would also suffer little or no damage, similar to the FEMA 273 (1996) Performance Level of Immediate Occupancy, for ground motions limited to an A_a of 0.2 ($Sa_{0.3}$ of approximately 0.75). Similarly, damage should be expected to be somewhat proportionally reduced for buildings at sites with A_a between 0.2 and 0.4, if they are designed for criteria intended for 0.4.

- It can also be projected that URM buildings located in areas of A_a = 0.4 will suffer less damage if designed for a larger demand. However, short of complete reconstruction, it is difficult to rehabilitate most URM buildings for more than about 1.5 times the UCBC demand requirements in high seismic zones. In any case, in areas where extreme ground motions and significant durations are possible, URM buildings probably should not be depended upon to provide high Performance Levels such as Immediate Occupancy unless extraordinary measures are taken.

Control of Probable Wall Behavior Modes

Lateral force-deflection behavior of unreinforced masonry shear walls can be governed by force-controlled actions that result in a severe loss of strength when reaching a peak stress level, or by deformation-controlled actions that are somewhat independent of the stress level but dependent on the amount of imposed lateral deflection.

Although inelastic behavior of URM wall and pier components subjected to lateral in-plane forces can be a complex interaction of several mechanisms, behavior, in general, can be classified into the following categories:

- diagonal tension cracking through units
- crushing at the compression toe
- shear sliding along bed joints
- rocking about the compression toe.

The first pair of actions are force-controlled because they are limited by a stress whereas the second pair of actions are deformation-controlled because they are essentially ductile actions that are limited by extents of lateral deflection. Because the difference between one action and another can be quite subtle, yet significant for seismic performance, discussion is provided in the subsequent Section 2.4 on these distinctions for shear and flexural behavior.

It is useful when projecting a Performance Level for a building to identify the probable behavioral mode of each wall or pier because each implies different types of damage, different safety levels, and different repair implications. The characteristics as they might affect performance are summarized in Table 2.4.

2.4 Rehabilitation Methods

Improvements in reliability, strength, or economy of all aspects of rehabilitation methods for URM buildings are needed to facilitate use of enhanced Rehabilitation Objectives. Enhancement procedures are given below for systems to anchor walls to diaphragms, to increase out-of-plane wall capacity, and to enhance in-plane wall performance.

Special Consideration of Wall Anchorages

In the majority of URM buildings, the most important, and in many cases the most vulnerable, feature related to seismic performance is the out-of-plane wall-to-diaphragm tie system. There were many instances of failures in this system in the Northridge Earthquake; examples with recommended enhancement procedures include the following:

Table 2.4: Characteristics of URM Wall Behavior Modes

Mode	FEMA 273 (1996) Control Parameter	Safety/Occupancy Implications	Repair Implications
Diagonal Tension Cracking	Force	Failure to be avoided. Results in rapid loss of lateral strength. If seismic demand continues after failure, building stability could be threatened. Any significant cracking of this type could cause ATC-20 (1989) red-tag building closure.	Grouting seldom acceptable. Local reconstruction, doweling across cracks, or overlay should be expected (expensive).
Toe Crushing	Force	Failure to be avoided. Strength degrades. Could compromise local vertical load-carrying capacity. Could cause ATC-20 red-tag building closure.	Local reconstruction required (expensive).
Bed-Joint Sliding	Deformation	Small crack widths are of little consequence. As crack widths increase, unit-to-unit bearing for vertical load could be compromised. Larger deformations could cause ATC-20 yellow- or red-tagging.	Grouting acceptable for most cases (inexpensive). Large permanent deformation may require local reconstruction, doweling or overlay (expensive).
Rocking	Deformation	Small to moderate deformations are of little consequence. Larger deformations could create local damage.	No repair required for moderate deformations (inexpensive). Grouting or overlay may be required for large deformations (moderately expensive).

- Anchors pulling through the wall due to poor quality masonry: The poor quality masonry in some cases was building-wide and in other cases was probably only at consideration such as project specific tests or oversized or continuous outside wall washers.

- Anchors pulling through the wall due to lack of overburden or edge distance: Although not formally documented through published tests, anchors to masonry walls near the tops of walls or near openings have indicated a reduced capacity both in actual earthquakes and in private tests (Rutherford & Chekene, 1965). It is recommended to avoid placement of anchorages at such locations, or if necessary, to limit allowable capacities to 33% of the full value, to employ oversize or continuous wall washers, or to install a concrete bond beam, doweled to vertically confine the masonry. When dowels must be installed near edges, in the absence of actual test data, the requirements which the City of Los Angeles Department of Building and Safety has placed on proprietary epoxy anchors are recommended: *"anchors shall be placed no closer to an edge than 24" (3 brick lengths) to attain full capacity. They may be spaced as close to an edge as 12" (1-1/2 brick lengths) provided the load on the anchor is also reduced to 33% of full capacity" (LADBS, 1995)*. Particular care should be taken when the anchors are loaded in shear toward the edge of the brick wall.

- Damage or failure at the connection to floor elements: Similar to performance in tilt-up buildings, there were many failures of proprietary timber hardware devices due to the code-approved inadequate factor of safety and lack of ductility (EERI, 1996). In addition, even without complete failure, much damage was created by excessive flexibility, which allowed walls to pull away from diaphragms, in some cases causing a loss of vertical support. It is recommended that strength and deformation of ties be considered at actual expected earthquake demands.

The procedures above primarily address enhancing performance reliability near the Life Safety Performance Level. If a higher Performance Level is sought in a high seismicity zone, additional measures may be necessary. Even a well detailed tension tie, for example, has a finite capacity, and tie spacing can only be reduced so far before more the addition of ties will have little beneficial impact. The next step of enhancing out-of-plane performance will be reducing load to the anchor through alternative means. One procedure is to spread the load though the connection between vertical strongbacks and the wall itself. These strongbacks, discussed below, also help brace the wall itself. A final step beyond this is simply to eliminate reliance on the masonry as part of the tie, by using concrete bond beams or--in the most extreme case--by placing shotcrete on the inside of the masonry surfaces. Gutting the interior and building an internal concrete structure is expensive and architecturally disruptive, but there are a great number of buildings in California which have been rehabilitated in this manner.

Traditional Out-of-Plane Bracing Alternatives

Two traditional techniques which address out-of-plane deficiencies are diagonal bracing and strongbacks. Several walls retrofitted with diagonal bracing were observed to have failed during the Northridge Earthquake; using strongbacks in lieu of diagonal braces may enhance performance.

Wall Enhancement Methods

Almost all research and testing associated with rehabilitation of URM buildings has been concentrated on improving in-plane wall capacity, but many of the methods tested are infrequently, if ever, used, due to lack of design guidelines. FEMA 273 (1996) has provided a framework in which such testing can be more easily integrated into practice. This section, to the extent possible, provides design guidelines for the use of wall strengthening methods that have been tested and summarized in Appendix C. The focus is on in-plane enhancement, but guidelines are also provided for out-of-plane enhancement where appropriate.

Types of Wall Enhancements

Several methods have been proposed and researched for enhancing seismic performance of unreinforced masonry wall or pier components. In the subsequent sections of this report, enhancement methods are itemized in one of the following nine categories:

- grout and epoxy injections
- surface coatings
- adhered fabrics
- shotcrete overlays
- reinforced cores
- post-tensioned masonry
- infilled openings
- enlarged openings
- steel bracing.

For each category, a brief description of each procedure is given which is followed by suggested guidelines on minimum construction requirements and information for Simplified Rehabilitation per FEMA 273 (1996), as well as a Systematic Rehabilitation with these same guidelines.

When adequate information has been given in the research literature on a particular wall enhancement scheme, quantitative information is given on modeling lateral stiffness of rehabilitated masonry components and estimating lateral strengths and acceptable deformations. When little or no numerical research data can be found in the literature for a particular wall enhancement scheme, but it has promise for application or has been used in practice, then general statements are made regarding the possible ranges for strength and deformation capacity that are credible with the enhancement method.

Most of the wall enhancement methods that have been researched have been aimed at increasing the lateral in-plane strength of a component. For example, grout and epoxy injections have proven to be useful for increasing the shear resistance of unreinforced brick or stone walls with large internal voids. Surface coatings, shotcrete overlays or adhered fabric have been found to also increase shear resistance of unreinforced masonry components as well as in-plane and out-of-plane flexural strength. Reinforced cores and prestressing are known to increase flexural strength of unreinforced brick walls and piers. In contrast to research on improving lateral strength, little, if any, of the research has been focused on improving the inelastic deformability or lowering seismic force demands.

Each wall enhancement method is discussed within the context of the newly developed FEMA 273 (1996) and FEMA 274 (1996). FEMA 273 (1996) is based on achieving a desired level of performance for a specific rehabilitation measure. The analysis methods used with the Systematic Rehabilitation approach are displacement–based so that computed parameters may apply to various Performance Levels. Since FEMA 273 (1996) addresses component acceptability in terms of both the seismic demand and capacity, discussions of wall enhancements include considerations of inelastic deformability as well as reducing seismic demand.

In FEMA 273 (1996), a clear distinction is made between "deformation-controlled" actions and "force-controlled" actions. As a preface to the descriptions of each wall enhancement method, the implications of these two types of actions on component acceptability of rehabilitated unreinforced masonry components are discussed. This is followed by a general discussion of rehabilitation philosophies to increase the likelihood that deformation-controlled actions will be retained or attained. Finally, a disclaimer is presented alerting a potential user to possible concerns other than structural engineering considerations that should be addressed before a rehabilitation scheme is implemented.

Component Acceptability Criteria per FEMA 273 (1996)

Linear Static Analysis: With the linear analysis procedures of the Systematic Rehabilitation approach of FEMA 273 (1996), component acceptabilities are distinguished for actions that are controlled either by deformations or by forces. For deformation-controlled actions, expected component strength, Q_{CE}, is multiplied by an m factor greater than 1.0, and compared with the unreduced elastic demand forces, Q_E, and the gravity forces, Q_G, in accordance with the following equation.

$$Q_G + Q_E \leq mQ_{CE} \quad (1\text{-}1)$$

Although terms in Equation 1-1 are member force quantities such as shear, moment or axial force that are obtained from an elastic analysis, the equation is intended to serve as a check on anticipated displacements.

The *m* factor is similar to a ductility factor and is based on the premise that elastic and inelastic deflections will be the same for an elasto-plastic oscillator. See FEMA 273 and 274 (1996) for details. Factors taken from Section 7.4.2.3 of FEMA 273 (1996) are presented in Table 2.5 for non-rehabilitated, unreinforced masonry wall or pier components that are loaded with forces parallel to their plane. The rationale for the *m* factors is explained in FEMA 274 (1996). Factors are given for three Performance Levels (Immediate Occupancy, Life Safety and Collapse Prevention), and they cover components of both primary elements (lateral force-resisting and vertical load-carrying) and secondary elements (vertical load-carrying). More slender walls (length-to-height aspect ratio less than 1.0) are assigned a greater ductility than squat walls because a rocking mechanism tends to produce larger nonlinear displacements than for a shear-controlled action. Ductility of components subjected to light vertical compressive stress is limited by flexural tensile stress, and are thus assigned a low *m* factor. Ductility of components with a high amount of vertical compressive stress (150 psi or greater) is limited by toe compressive stress, and is given a smaller factor than for moderate levels of vertical stress (in the range of 75 psi). As was noted in Section 1.3, the numerical coefficients—such as *m* factors—used in FEMA 273 (1996) have not been calibrated with extensive use in actual evaluation and design. Users should validate results using alternate methodologies and engineering judgment.

For a building system with unreinforced masonry shear walls, the Q_E term is that value which results from application of an equivalent unreduced base shear, V to the system as given by the following equation from Section 3.3.1.3 of FEMA 273 (1996):

$$V = C_1 C_2 C_3 S_a W \tag{1-2}$$

where the C_i terms are intended to provide equivalency between elastic and inelastic displacements and second-order effects, S_a is the elastic spectral acceleration and W is the total weight of the building and an anticipated portion of the live loading. The total base shear force is distributed to adjacent shear walls to give the seismic demand force Q_E in Equation 1-1.

For force-controlled actions, per Section 3.4.2.1 of FEMA 273 (1996), the maximum seismic forces that can be delivered to a component from the combination of gravity and seismic effects is compared with a lower bound estimate of strength, Q_{CL} in accordance with the following equation:

$$Q_G + \frac{Q_E}{C_1 C_2 C_3 J} \le Q_{CL} \tag{1-3}$$

where:

$$J = 1.0 + S_{DS} \tag{1-4}$$

and S_{DS} is the FEMA 273 (1996) short period spectral acceleration.

In this case, Equation 1-3 is intended to represent actions resulting from actual forces rather than having force terms represent displacements as in Equation 1-1. As a simple, general approximation for comparison purposes, the seismic demand force can be taken equal to one half of Q_E assuming the product of $C_1 C_2 C_3 J$ equal to 2.0 (typical for low-rise buildings in moderate seismic zones). Lower-bound shear strengths of URM walls, Q_{CL} are known to be as low as 60% times expected strengths, Q_{CE}.

For lateral strength of shear wall components, the term Q_G in Equations 1-1 and 1-3 can be dropped, assuming that gravity forces are balanced about shear walls and thus do not result in wall shear forces or overturning moments.

Substituting 0.5 times Q_E for the seismic demand force in Equation 1-3, and 0.6 times Q_{CE} for Q_{CL} results in the following equivalent equation for force-controlled actions.

$$Q_E \leq 1.2 Q_{CE} \tag{1-5}$$

Comparing Equations 1-1 and 1-5 shows that rehabilitation of a force-controlled shear wall will always be more restrictive than for a deformation-controlled wall if the *m* factor exceeds a value of 1.2. Since typical Life Safety Performance Level *m* factor values for unreinforced masonry shear walls are in the range of 1.5 to 4, it is apparent that efforts should be made where possible to restrict masonry shear walls to behave in deformation-controlled modes. In fact, using the values assumed above, the strength of a force-controlled wall would have to be (1.5 to 4)/1.2 = 1.25 to 3.33 times the strength of a deformation-controlled wall to achieve an equivalent level of capacity. Not only should rehabilitation schemes be chosen to introduce deformation-controlled behavior but also care should be taken so that an existing deformation-controlled wall will not become a force-controlled component through an improper selection of a rehabilitation scheme.

Nonlinear Static Analysis: As an alternative to the linear static procedure, FEMA 273 (1996) permits the use of a nonlinear static analysis in Section 3.3.3. With the nonlinear procedure, individual structural components are modeled with a simplified nonlinear force-deflection relation, as shown in Figure 2-2. Lateral static forces are applied incrementally as various components reach their force capacities. When lateral deflections of a system reach expected target displacements, nonlinear component deformations are determined and compared with acceptable values. If ductility of a component is insufficient for the system to reach its target displacement, the strength or deformation capacity of the component may be increased through some rehabilitation scheme. The system is then analyzed again to confirm that adequate ductility will be available for the rehabilitated component.

FEMA 273 (1996) specifies in Section 7.4.2.3.A factors to define the shape of idealized force-deflection curves for unreinforced masonry wall and pier components. These shape coordinates are repeated in Table 2-6. Variables *d* and *e*, representing nonlinear deformation capacities for primary and secondary components, are expressed in terms of story drift percentages as defined in Figure 2-2.

For components of primary lateral-force resisting elements, collapse should be considered at lateral drift percentages exceeding values of d in the table, and the Life Safety Performance Level should be considered at approximately 75% of the d value. For components of secondary elements, collapse should be considered at lateral drift percentages exceeding the values of e in the table, and the Life Safety Performance Level should be considered at approximately 75% of the e value in the table. Drift percentages based on these criteria are given in Table 2-5 for convenience.

For determination of the c, d and e values and the acceptable drift levels using Table 2-6, the vertical compressive stress, f_{ae}, should be based on an expected value of gravity compressive force per the load combinations given in Equations 3-1 and 3-2 of FEMA 273 (1996).

Bed-Joint Sliding vs. Diagonal Tension Cracking

If the masonry units are weak in tension relative to the shear strength of the bed joints, diagonal cracking may propagate directly through the units and across the wall. Just as an unreinforced concrete wall would behave upon reaching a critical shear stress level, the unreinforced masonry wall will have a rapid loss of lateral strength behaving in this mode. However, if the converse is true (strong units and weak mortar), then diagonal tension stress will result in an initial separation of head joints and a subsequent sliding of units along the bed joints. As a result of frictional forces from vertical compressive stress, the wall will continue to resist substantial lateral in-plane forces even though head joint separations may be very wide. Thus, combining high-strength units with low-strength mortars can result in a deformation-controlled action, whereas the reciprocal combination can result in a force-controlled action. Despite the lower shear strength with the deformation-controlled action, the criteria for component acceptability can be less stringent because an m factor is multiplied by the expected capacity (Eq. 1-1) rather than using the lower bound estimate of strength (Eq. 1-3) with no m factor.

Expected in-plane shear strength of an unreinforced masonry wall governed by bed-joint sliding is given in Section 7.4.2.2.A of FEMA 273 (1996) with the following equation:

$$Q_{CE} = V_{bjs} = v_{me} A_n \tag{1-6}$$

where v_{me} is the expected masonry shear strength based on the in-place shear test, and A_n is the net mortared area. Note that sliding may occur on a stair-stepped crack which follows the sliding bed joints and opening head joints or it may occur along a horizontal bed joint plane in the wall or the interface between the wall and the foundation.

For shear walls with a length-to-height aspect ratio between 0.67 and 1.00, the lower bound lateral in-plane strength can be estimated with the following equation given in Section 7.4.2.2.B of FEMA 273 (1996) and presented in Kingsley (1995):

$$Q_{CL} = V_{dt} = f'_{dt}\, A_n \left(\frac{L}{h_{eff}}\right)\sqrt{1+\frac{f_a}{f'_{dt}}} \tag{1-7}$$

where f'_{dt} is the diagonal tension strength of the masonry, f_a is the lowest estimate of vertical compressive stress, L is the length of the wall or pier, and h_{eff} is the height to the resultant of the lateral force.

A component will be displacement-sensitive if the expected bed-joint sliding shear strength is less than the diagonal tension shear strength, and force-sensitive if the converse is the case. Ideally, expected values of strength such as V_{bjs} should not be compared directly with lower bound values such as V_{dt}; however, in the interest of simplicity and needed conservatism, the following inequalities are used to distinguish one action from the other:

if $V_{bjs} < V_{dt}$ then component is displacement-sensitive
if $V_{dt} < V_{bjs}$ then component is force-sensitive.

As an example, take a shear wall with a 1:1 aspect ratio and subjected to a vertical compressive stress of 75 psi. Assume that the tensile strength of clay units is one-thirtieth of the flat-wise unit compressive strength. As a lower bound estimate for typical American brick, assume a unit compressive strength equal to 6000 psi which gives a lower-bound diagonal tension strength limited by cracking of the unit equal to 200 psi. Substitution of this value for f'_{dt} into Equation 1-7 gives the following lower bound estimate of the wall diagonal tension strength.

$$V_{dt} = (234psi)A_n \tag{1-8}$$

The wall will remain displacement-controlled if the expected masonry shear strength, v_{me}, is less than 234 psi. This value corresponds to an average value of bed-joint shear strength per the in-place shove test, v_{te}, equal to 524 psi in accordance with Equation 7-1 of FEMA 273 (1996). Even as an average value, 524 psi is larger than most test results which suggests that bed-joint sliding should control over diagonal tension for walls of this example with typical mortars, and thus be displacement-controlled. Only for cases where the mortar is strong enough to result in test values exceeding 524 psi would the acceptability of the wall be judged in accordance with that for a force-controlled component (Equation 1-3), which would be more restrictive since an m factor could not be used.

Rocking vs. Toe Compression

If the shear strength of an unreinforced masonry wall or pier component is sufficiently high, flexural modes of behavior will control. In such cases, lateral strength will be controlled by toe compressive stress for relatively squat walls with large amounts of vertical compressive stress, or by rocking for relatively slender walls with small amounts of vertical compressive stress.

Rocking of a wall or pier component will occur when the lateral force exceeds a value given by the following equation:

$$Q_{CE} = V_r = 0.9aP_{CE}\left(\frac{L}{h_{eff}}\right) \tag{1-9}$$

where α is a factor equal to 0.5 for a cantilevered shear wall and equal to 1.0 for pier components fixed against rotation at both their top and bottom, P_{CE}, is the expected vertical compressive forces resisted by the component, L is the length of the component and h_{eff} is the height to the resultant of the lateral force applied to the component. For a rocking mechanism to occur, bed-joint cracks will develop and propagate across the length of the component. Although the effective shear area will be reduced with this bed-joint cracking, shear forces will still be transferred as a result of friction at the wall toe. Rocking behavior can be depicted with an elasto-plastic model where the strength is equal to that given by Equation 1-9 and the lateral displacement is equal to the rigid-body rotation times the component height. Lateral deflection is limited by stability concerns, or by twisting (if the component is asymmetrical in plane, such as L-shaped) or out-of-plane "walking" of the component with repeated and reversed loading cycles. However, because there is no sudden loss of lateral strength, a rocking mode is classified as a deformation-controlled action. Component acceptability is per Equation 1-1 with m factors in the range of 3 to 5 for primary components.

If lateral forces attracted to a wall or pier component do not exceed the rocking strength of the pier (as determined with Equation 1-9), and shear strength is sufficiently high, then crushing at the wall toe may be likely. Toe crushing occurs when vertical splitting cracks form when the vertical compressive stress exceeds the compressive strength of the masonry. A sudden reduction in lateral strength will occur as the resistance to flexural compression is lost. Because flexural tension stress at the wall heel will tend to open bed-joint cracks along the wall base, the lateral-force deflection behavior of the wall will be represented with a curve of continuously lessened slope until toe crushing occurs. However, the ultimate limit state is related to toe compressive stress and is thus classified as a force-controlled action.

The lower bound lateral strength of an unreinforced masonry wall or pier component that is controlled by toe compressive stress is given in FEMA 273 (1996) as:

$$Q_{CL} = V_{tc} = aP_{CL}\left(\frac{L}{h_{eff}}\right)\left(1-\frac{f_a}{0.7f'_m}\right) \tag{1-10}$$

where P_{CL} Is a lower bound estimate of vertical compressive force, f_a is an upper bound estimate of vertical compressive stress, and f'_m is a lower bound estimate of the masonry compressive strength. Equation 1-10 has been derived using statics with a triangular distribution of compressive stress, and assuming no tensile strength of the masonry.

Similar to the reasoning to promote bed-joint sliding over diagonal tension, a preferred action is rocking over toe compression so that a deformation-controlled action will govern and component acceptability can be judged using Equation 1-1 rather than Equation 1-3. A wall will be displacement-sensitive if the rocking strength is less than the toe compressive strength, and force-sensitive if the converse is true. Again, expected values of strength should not be compared directly with lower bound values; however, as done for the shear modes of behavior, the following inequalities are used to distinguish one flexural action from the other:

if $V_r < V_{tc}$ then component is displacement-sensitive
if $V_{tc} < V_r$ then component is force-sensitive.

A rocking mechanism can be forced to occur by increasing the length-to-height aspect ratio and/or decreasing the vertical compressive force. These may result in a less stringent acceptability (even with a lower strength) because the wall will be displacement-controlled and an m factor is then multiplied by the expected strength.

Governing Actions

In the previous two sections, direct comparisons were made to distinguish force- and displacement-controlled actions for related mechanisms that were separately grouped into categories of shear (bed-joint sliding or diagonal tension stress) or flexure (rocking or toe compressive stress). The final decision on whether the component will be force- or displacement-sensitive is, however, based on the action that results in the lowest in-plane lateral strength from either category.

As currently written, FEMA 273 (1996) requires that when the initial determination of governing mode is made, the values used in the force-controlled equations will represent lower bound material properties and axial loads, but for deformation-controlled modes, expected properties, and axial loads are used. If deformation-controlled modes are considered preferable, then this is a conservative approach. An alternative, more rational approach is to make the *initial* comparison using the *same* properties and axial loads. If a deformation-controlled mode is predicted to govern, then the *design* strength should be based on expected values. If a force-controlled mode is predicted, then the *design* strength should be based on lower bound values. A similar philosophy of applying the factor of safety (or strength reduction factor) after comparing nominal strengths is used by the UBC to determine whether a concrete wall is governed flexure or shear.

As a rule for improved performance, the following hierarchy in behavior modes should be striven for when prescribing a rehabilitation method for a component:

- rocking
- bed-joint sliding

- toe crushing
- diagonal tension cracking.

The most favorable mode is rocking because it is a flexural action that is displacement-controlled. Damage will be limited to a single bed-joint crack at the bottom of a shear wall, or at the top and bottom of a pier. During rocking, additional damage is not likely to occur, and thus large nonlinear deflections can be tolerated. Following the seismic excitation, the gravity stress resisted by the component will tend to close these cracks. The lateral force-deflection relation for rocking behavior can be approximated with an elasto-plastic curve characteristic of reinforced materials with considerable deformation capacity.

Bed-joint sliding is a shear action that is displacement-controlled. Cracks follow the head and bed joints rather than propagating through the units. Because the frictional forces developed along the bed joints remain constant to resist applied shear forces, large in-plane lateral deflections can be tolerated even though head joints may separate considerably.

Toe crushing is a flexural action that is force-controlled. This action is limited to components that are subjected to large amounts of vertical compression stress prior to application of lateral force. Damage will consist of vertical splitting cracks at the toe region of the wall. Before the toe crushes, some redistribution of stress will occur as the resultant of the resisting vertical compressive stress migrates towards the toe. The lateral in-plane stiffness will gradually reduce until toe crushing occurs, providing some residual capacity before failure.

The least favorable mode is diagonal tension because it is a shear action that is force-controlled. When principal tensile stresses exceed diagonal tensile strength, an inclined crack will propagate through relatively weak units, leaving little or no frictional resistance. Failure will occur suddenly, and damage will be extensive.

Rehabilitation Philosophies to Help Insure Improved Performance

If the lateral capacity of an existing wall or pier component is governed by a deformation-controlled action, a rehabilitation scheme will often be most effective if it preserves or enhances that type of action. In contrast, if the rehabilitation scheme alters the lateral behavior of a component from a deformation-controlled to a force-controlled action, then the strengthened component may not be as acceptable unless substantial additional strength is provided. If the existing component is governed by a force-controlled action, then the rehabilitation may be most effective if it somehow alters the behavior to a deformation-controlled action. In many situations, though, it may not be feasible, cost effective or desirable to provide for a conversion to deformation-controlled behavior. In these circumstances, the rehabilitation method must provide sufficient strength to avoid a brittle failure. Some of the issues and difficulties in promoting deformation-controlled behavior are discussed below in terms of the wall enhancement options given in subsequent sections.

Grout or epoxy injections will increase the shear strength of an unreinforced masonry wall or pier component. If an existing wall is prone to bed-joint sliding, injecting of grout into an open collar joint may make it vulnerable to diagonal tension. Thus, the deformation-controlled action of bed-joint sliding would be replaced with the force-controlled action of diagonal tension. The retrofit scheme may result in a larger attraction of shear force because of the increased stiffness and strength but may take away from the inelastic deformation capacity.

Surface coatings, shotcrete overlays or adhered fabric may likewise influence the ductile bed-joint sliding mechanism of an existing masonry wall. If the new materials add sufficient shear strength, the failure mode can change to diagonal tension, which is a force-controlled action. In order to be effective, increased strength of the composite wall must be sufficient to provide an equivalent deformation capacity. If a pier component is prone to rocking, the vertical reinforcement in the coating or shotcrete, if anchored into the adjoining components, may increase the flexural strength such that the component will become shear critical, and thus again cause it to be classified as a force-controlled action if diagonal tension strength is low.

Infilling window or door openings may reduce the potential for pier components to rock, and create a shear critical condition in the wall acting as a whole component. Bed-joint sliding mechanisms may also be prevented if the new masonry that infills an opening has a relatively strong mortar. In contrast, enlarging window openings to increase the rocking potential of adjacent piers can change a force-controlled condition to a deformation-controlled one. The actual impact of enlarging a window opening will depend upon the relative strengths of each mode of behavior. For example, there are two ways that openings can be enlarged: by making them taller and by making them wider. If the openings are made taller, the rocking, diagonal tension and toe crushing strengths of adjacent piers will diminish, but the bed joint sliding strength will remain the same. Depending on the relative values of the various strengths, it is possible to go from a force-controlled mode to a deformation-controlled mode and vice-versa. If the openings are made wider, then the bed joint strength of adjacent piers diminishes and the other three strengths increase. Again, the governing mode will depend on relative values.

Adding vertical reinforcement may increase flexural strength such that shear becomes critical. Grout cores for added reinforcement may also preclude bed-joint sliding shear mechanisms because of the dowel-type action that they may provide. In such cases, an existing deformation-controlled component becomes force-controlled, and the acceptability criteria become more stringent.

Vertical post-tensioning of unreinforced masonry walls can also increase flexural strength such that shear becomes critical; however, the added vertical compressive stress should increase shear strength as well. Bed-joint sliding mechanisms will be enhanced through increased frictional forces. The added vertical force will, however, not change the shear mode to diagonal tension because diagonal tension strength is also increased with the larger vertical compressive stress. If the cores are grouted, the bed-joint sliding will be restrained through the keying action provided by the grouted cores, and diagonal tension may govern for shear. If an existing component is vulnerable to rocking, the increased vertical compressive force with post-tensioning will increase the rocking strength. This will increase the acceptability of the component if shear strength exceeds the enhanced rocking strength.

The addition of steel bracing parallel to the plane of an unreinforced masonry shear wall can add strength and ductility to an otherwise brittle component, provided that the wall shear strength is governed by diagonal tension and the brace is stiff enough to participate significantly in resisting the load. If the wall strength is governed by bed-joint sliding or rocking, a retrofit scheme may only be effective if it increases the vertical compressive force on the masonry.

Disclaimer on Wall Enhancement Methods

Many of the URM wall enhancement methods described below have been developed with limited laboratory testing, and they may also have a limited history of use in actual retrofit projects. It is important to remember that satisfying the required structural criteria is not the only measure of acceptability. The engineer of record (and possibly other members of the design team such as the architect, materials conservator, code consultant, etc.) considering innovative techniques will often find it necessary to research the proposed technique in greater detail than normal to confirm that issues such as the following are appropriately addressed:

- Is there adequate data on the long-term stability of the materials being used?
    - Do they creep, crack, shrink, lose strength, debond, rust, etc. over time?
    - Will contact with rain on the surface or moisture within the masonry lead to problems?
    - Are the materials adequate for the expected temperature range?
    - Is efflorescence a concern?
    - Is ultraviolet light an issue?
    - Are harmful emissions, such as volatile organic compounds, a problem?
- Does the strengthening technique meet fire safety requirements?
- Does the strengthening technique meet aesthetic and historic preservation requirements?
- Do installation methods create other potential problems (vibration, moisture, etc.)?

- Can an adequate field quality assurance program be developed to verify that in-situ properties meet design assumptions?

Grout and Epoxy Injections

Description of Rehabilitation Procedure

Cement grouts, or lime grouts, or epoxy resins are injected into voids and cracks in a masonry wall comprised of solid units. Grout or resin is injected through core-drilled injection ports which are located in the vicinity of cracks and voids. The method is common for restoration of older brick and stone masonry for non-seismic applications, as well as for repair of seismically damaged masonry. However, the method may also be used as a rehabilitation measure for enhancing lateral-force resistance particularly for cases when large voids in a collar joint are present or for ungrouted CMU cells. In such cases, the purpose of the procedure is to increase:

- flexural strength for walls subjected to out-of-plane loadings.
- shear strength for walls subjected to in-plane loadings.

Before grout or resin injection is done, a visual survey should be done to identify the extent and size of any surface cracks, mortar delaminations or voids. This should be followed by a nondestructive evaluation to identify any subsurface cracks or voids. Ultrasonic and sonic wave velocity tests can provide reliable data for mapping of these internal flaws.

Minimum Construction Requirements

Injection and flow-verification holes should be core drilled where voids and cracks are suspected. The holes should be drilled from one side of a wall to the near face of the far wythe. Surface cracks and mortar delaminations that may result in grout leakage must be sealed.

A masonry wall should be washed with water 24 hours before injection. The masonry should be saturated but surface dry at the time of injection. Cracks and voids should be flushed with water through the injection holes before grout is pumped.

The injection process should start at the lowest port and proceed upward. Holes above the injection port should be plugged when grout is seen to flow out of them. Grout should be injected until refusal; then pressure should be applied for an additional minute to consolidate the grout.

For walls with multiple voided collar joints, injection should proceed from the far collar joint and proceed towards the near collar joint.

Injection and verification ports should be plugged after placement of grout or resin. Plugs should be removed when the grout or resin is firm and ports should be repointed with a stiff mortar similar in composition and color to the original mortar.

The adequacy of the injection process should be verified with nondestructive tests.

Simplified Rehabilitation

Injecting grout or resins into cracks should be done with older or damaged masonry to insure that it is sound, and so that the Simplified Rehabilitation Method of Section 10.3.3.3 of FEMA 273 (1996) can be used.

Injecting grout into the CMU cells, a cavity between wythes, or into large voids in a collar joint can improve the wall shear strength as a result of the increased shear area. The shear strength as calculated per Section 10.3.3.3.B of FEMA 273 (1996) can be based on the total wall thickness if the CMU cell, cavity, or collar joint is fully grouted. Furthermore, the height-to-thickness ratio can be based on the total wall thickness. Thus, by grouting the collar joint, the wall slenderness can be reduced to the extent that it may comply with Simplified Rehabilitation Method measures that are based on the height-to-thickness ratio.

General Enhancements

Stiffness: When cracks are filled with grout in accordance with the minimum construction requirements, the lateral stiffness of a component may be determined based on the gross, uncracked section and the elastic modulus of the original masonry. Other recommendations for modeling in-plane stiffness of unreinforced wall and pier components per Section 7.4.2.1 of FEMA 273 (1996) may be considered.

The lateral stiffness of a masonry wall or pier component with a grouted cavity or collar joint may be determined considering composite action between the grout and the masonry. Differences in elastic moduli for the two materials should be considered when determining properties of the composite section.

Strength Acceptability Criteria: Injection of grout or resins into cracks will not appreciably increase the effective shear area because of the small amount of injected material relative to the area of existing masonry. Moreover, bed-joint sliding resistance will not be improved with grouting because the mortar-unit interface along the bed joints is usually not affected with grouting. Therefore, injections, as a rehabilitation measure, should be considered as a method for insuring that a wall will indeed perform as a homogeneous medium and can thus be analyzed with confidence using the procedures in FEMA 273 (1996) for unreinforced masonry walls.

When grout is injected to fill CMU cells, a cavity between wythes, or large voids in a collar joint, the grout and masonry will act compositely to resist in-plane shear forces. Care must be taken so that a ductile bed-joint sliding mechanism for an existing wall or pier will not be overcome with a brittle mechanism governed by diagonal tension of the grouted space between wythes.

Expected strength of unreinforced masonry wall or pier components, loaded laterally parallel with their plane, should be the lesser of the expected shear strength or the expected rocking strength. Grouting of CMU cells, a cavity, or large voids in a collar joint can increase shear strength more than rocking strength, and thus can be an effective way to alter the behavioral mode from shear to rocking.

The expected shear strength of a grouted wall, $V_{gw,}$ shall be the sum of the bed-joint sliding shear strength of the masonry, V_{bjs}, and the shear strength of the grouted section in accordance with Equation 2-1.

$$Q_{exp} = V_{gw} = v_{me}A_n + 2\sqrt{f_{ge}}A_{grout} \qquad (2\text{-}1)$$

where:

A_n = area of net mortared masonry section, inches^2

A_{grout} = area of grout, inches^2

f_{ge} = expected compressive strength of grout, psi

L = length of wall or pier, inches

v_{me} = expected bed-joint sliding shear strength per Section 7.3.2.4 of FEMA 273 (1996), psi.

The flexural tension strength of a wall with a grouted cavity will be slightly larger when rehabilitated because of the tensile strength of the added grout material. After the first loading cycle that cracks the grout in flexural tension, the rocking strength will reduce to that of the ungrouted section.

The lower bound strength of unreinforced masonry wall or pier components with a grouted cavity, loaded laterally parallel with their plane, should be the lesser of strengths limited by diagonal tension stress or toe compressive stress determined in accordance with Section 7.4.2.2.B of FEMA 273 (1996). The term A_n in Equation 7-5 of FEMA 273 (1996) should be replaced by the combined area of the existing masonry and injected grout. In Equation 7-6 of FEMA 273 (1996), the vertical compressive stress, f_a, should be determined based on the combined area of the existing masonry and injected grout. If one of these lower bound strengths is less than the shear strength per Equation 2-1 or the rocking strength, then the component shall be considered as force-sensitive.

<u>Deformation Acceptability Criteria</u>

A. Linear Analysis Procedures

A wall or pier with injected cracks should be considered as an unreinforced component, and be subject to the provisions of Sections 7.4.2 and 7.4.3 of FEMA 273 (1996). The *m* factors should be the same as given in Table 2-5.

If CMU cells, a cavity between wythes, or large voids in a collar joint are grouted, the wall or pier component should be considered as an unreinforced masonry component and be subject to the provisions of Sections 7.4.2 and 7.4.3 of FEMA 273 (1996). The *m* factors should be the same as given in Table 2-5.

B. Nonlinear Analysis Procedures

If the nonlinear static analysis procedure given in Section 3.3.3 of FEMA 273 (1996) is used, injected walls and pier components should be assumed to deflect to nonlinear lateral drifts as given in Table 2-6 for plain, unreinforced masonry walls.

Surface Coatings

Description of Rehabilitation Procedure

A thin cement plaster coating is parged on one or both sides of an unreinforced brick wall. A layer of steel hardware cloth or metal strips are embedded into the coating. The coating is adhered to the wall with a series of connectors in addition to surface bonding.

The purpose of the procedure is to increase one or more of the following:

- flexural strength for walls subjected to out-of-plane loadings
- flexural strength for walls subjected to in-plane loadings
- shear strength for walls subjected to in-plane loadings
- inelastic deformation capacity for walls subject to in-plane loadings
- vertical compressive strength.

Coating a single side of a wall can provide the necessary enhancements in out-of-plane and in-plane strength; however, inelastic deformation capacity of walls coated on both sides will be greater because of the confinement effects that a dual-faced coating will provide the interior, cracked brickwork.

Minimum Construction Requirements

The coating can be applied to one or both sides of a masonry wall. The coating should be applied to the entire surface of a masonry wall or pier component for maximum effectiveness.

The uniaxial compressive strength of the coating material should exceed a minimum of 1000 psi and should exceed the compressive strength of the masonry to which it is adhered. The coating material should consist of a mixture of Portland cement sand in a 1 to 3 volume ratio.

The compressive strength of the coating material can be determining by casting 2-in. diameter cylinders in steel molds in accordance with the latest version of ASTM C31:

"Practice for Making and Curing Concrete Test Specimens in the Field," and testing them in accordance with ASTM C39: "Test Method for Compressive Strength of Cylindrical Concrete Specimens".

The coating should be reinforced with one of the following:

- Steel hardware cloth with a minimum diameter of 19g and a maximum grid of 1/2 inch placed at the center of the coating. The coating material should be passed through the cloth mesh rather than placing the mesh in the wet coating material.
- Metal strips with a minimum thickness of 16g and a minimum width of 1.5 inches spaced at a maximum of 8 inches.

Steel tie-down bolts perpendicular to the wall face with a minimum diameter of 1/4-inch should be spaced sufficiently close so that the coating will not delaminate from the wall surface. Research has shown that a minimum horizontal and vertical spacing of 16 inches is sufficient for this purpose. Anchorage of the bolts should be satisfactory if they extend through the wall thickness or are embedded into the wall 6 inches or more. Washers or steel should be placed at the end of each bolt to disperse bearing stresses on the coating.

The thickness of the cement plaster coating should exceed all of the following requirements:

- a minimum of 0.5 inch
- the wall height divided by 200
- the minimum spacing between tie-down bolts divided by 32.

Simplified Rehabilitation

Application of surface coatings can be a simple and effective method for increasing the shear strength of a wall as calculated per Section 10.3.3.3.B of FEMA 273 (1996). The shear stresses in the masonry and the coating can be estimated by considering a transformed section where the area of the coating has been multiplied by the ratio of elastic moduli for the coating and masonry. Stresses attracted to the coating and the masonry should then be checked relative to allowable shear values for each material.

A second advantage of surface coatings is that the height-to-thickness ratio can be reduced. This can be a corrective measure when the slenderness of a parapet or a wall exceeds minimum values as set forth by FEMA 178 (1992) or FEMA 273 (1996).

General Enhancements

Stiffness: The lateral stiffness of a coated masonry wall or pier component may be determined considering composite action between the coating and the masonry. Differences in elastic moduli for the two materials should be considered when determining properties of the composite section.

The elastic modulus of the coating material should be taken as 750 times the expected compressive strength of the material unless tests are done to determine the modulus.

The initial stiffness of a coated wall or pier should be considered to be linear and proportional with the geometrical properties of the uncracked section.

Strength Acceptability Criteria

A. In-Plane Strength

Expected strength of coated, unreinforced masonry wall or pier components, loaded laterally parallel with their plane, should be the lesser of the expected shear strength or the expected rocking strength.

The expected shear strength of a coated wall, $V_{coat,}$ shall be the sum of the bed-joint sliding shear strength of the masonry, V_{bjs}, and the shear strength of the coating in accordance with Equation 3-1.

$$Q_{CE} = V_{coat} = v_{me} A_n + 2\sqrt{f_{ce}}\, A_c \tag{3-1}$$

where:

A_n = area of net mortared/grouted masonry section, inches^2

A_c = area of coating, $t_c L$, inches^2

f_{ce} = expected compressive strength of coating material, psi

L = length of wall or pier, inches

t_c = total thickness of coating layers, inches

v_{me} = expected bed-joint sliding shear strength per Section 7.3.2.4, psi.

The expected rocking strength of a coated wall or pier component is the same as for an uncoated component per Section 7.4.2.2.A of FEMA 273 (1996) provided that the coating reinforcement is not anchored into the adjoining member above or below the component.

When the vertical coating reinforcement is continuous above or below the component, as may be done with a pier component between window openings, the expected rocking strength can be increased by the tensile strength of the coating reinforcement; however, the deformation capacity of the strengthened component will then be limited by the plastic elongation of the coating reinforcement.

The lower bound strength of coated, unreinforced masonry wall or pier components, loaded laterally parallel with their plane, should be the lesser of strengths limited by diagonal tension stress or toe compressive stress determined in accordance with Section 7.4.2.2.B of FEMA 273 (1996). The term A_n in Equation 7-5 of FEMA 273 (1996) should be replaced by the combined area of the existing masonry and the coating, A_n

plus A_c. In Equation 7-6 of FEMA 273 (1996), the vertical compressive stress, f_a, should be determined based on the combined area of the existing masonry and injected grout. If one of these lower bound strengths is less than the shear strength per Equation 2-1 or the rocking strength, then the component shall be considered as force-sensitive.

Coating an unreinforced masonry wall can increase in-plane shear strength because of the added area effective in resisting shear. However, care must be taken so that a ductile bed-joint sliding mechanism for an existing component will not be overcome with a brittle mechanism governed by diagonal tension of the coating. Reinforcing the coating with wire mesh will help insure that such a brittle mechanism can be avoided.

B. Out-of-Plane Strength

Transverse walls must resist out-of-plane forces resulting from the wall mass accelerating across an individual story height. According to FEMA 273 (1996), the lower bound, out-of-plane strength of an individual wall panel must be less than the maximum inertial force. Because an elastic demand force applied normal to a wall plane cannot be obtained from a static linear analysis of the global structural system subjected to an equivalent lateral force, the maximum credible inertial force is taken as a coefficient times the wall weight.

The lower bound out-of-plane strength of a wall without arching or vertical compressive stress is limited by the flexural tension strength of the masonry. A coating applied to one or both surfaces of a wall can enhance out-of-plane flexural strength by increasing the section modulus as well as the tensile strength of the material at the extreme tensile fibers. As long as the minimum construction requirements are met, composite action between the coating material and masonry can be assumed. Thus, the flexural strength for a strip of unit width spanning vertically or horizontally can be taken as:

$$M_{CL} = \frac{f_{tcoat}}{n} S_{trans.} \tag{3-3}$$

where f_{tcoat} is the tensile strength of the coating material, n is the ratio of moduli of elasticity for the coating and masonry respectively, and S_{trans} is the section modulus of the composite section where the width of the strip is increased by multiplying the unit dimension by n. The tensile strength of the coating, f_{tcoat} should not exceed half of the masonry compressive strength to insure that the masonry will not crush.

Vertical compressive stress will enhance the out-of-plane strength appreciably because it will counteract flexural tensile stress and will create a force couple that will provide flexural strength even after initial flexural cracks form. However, if the wall is resisting vertical compressive stress before the coating is applied, no transfer of stress to the coating can be considered unless the gravity force is temporarily removed and reapplied after the coating has developed strength. Thus, the introduction of the coating material may not significantly enhance out-of-plane strength if its tensile strength is appreciably less than that of the original vertical compressive stress on the masonry.

Post-cracked behavior of unreinforced masonry walls with little or no vertical stress can be considered if a nonlinear dynamic analysis is done to estimate the expected lateral deflection that may be imposed on two opposing wall segments. Since the dynamic stability will depend on the relative height-to-thickness ratio for the wall, application of a coating to the wall surface can enhance performance of the wall.

Deformation Acceptability Criteria

A. Linear Analysis Procedures

If a reinforced coating is applied to one or both surfaces of an unreinforced wall, inelastic deformations should be based on the *m* factors as given in Table 2-8. These factors are in general slightly higher than those for a plain, unreinforced masonry wall because of the added ductility that a reinforced coating should provide. If the coating is not reinforced, then this added ductility will not be provided, and *m* factors should be taken from Table 2-5 for plain walls.

B. Nonlinear Analysis Procedures

Nonlinear force-deflection relations, and acceptable deflections, for walls rehabilitated with a reinforced coating should be based on the shape coordinates given in Table 2-8. These factors are based on the premise that the reinforced coating will provide added inelastic deformation capacity. If the coating is not reinforced, shape coordinates and acceptable deflections should be taken from Table 2-6 for plain walls.

Adhered Fabric

Description of Rehabilitation Procedure

Overlapping strips of a high strength fabric are epoxied to the surface of an unreinforced masonry wall.

The purpose of the procedure is to increase one or more of the following:

- flexural strength for walls subjected to out-of-plane loadings
- flexural strength for walls subjected to in-plane loadings
- shear strength for walls subjected to in-plane loadings
- inelastic deformation capacity for walls subject to in-plane loadings.

Minimum Construction Requirements

The fabric can be applied to one or both sides of a masonry wall. Fabric strips should be aligned with their principal strength parallel to the shorter of the vertical or horizontal wall span.

The wall surface should be sandblasted and cleaned of any loose particles. Loose masonry units should be reset and any cracks should be filled with mortar paste or grouted.

The fabric should be pressed against an uncured epoxy that has been applied to the wall surface. Successive strips of fabric should be added with sufficient overlap to insure full development of the fabric at any location.

When continuity is desired for vertical strips, the bottom edge of fabric should be anchored to the existing footing or floor slab with steel anchors. The top edge of fabric should be connected to the underside of slab or floor joists or to the surface of an upper parapet for an exterior surface. For horizontal strips, the side edges should be anchored to the wall edges with steel anchors or to the adjacent columns with steel anchors.

A second coating of epoxy should be applied to the exterior wall surface over the fabric. An ultra-violet protective coating should be applied over the fabric on any wall surface exposed to sunlight.

Fabrics should be selected with strengths sufficiently high so that the strength of the masonry in shear and flexure can be developed. Research experiments have investigated the use of a glass fabric with an acrylic polyvinyl finish and unidirectional E-glass fibers.

Simplified Rehabilitation

Application of fabric coatings can increase the shear strength of a wall or pier, but the increase in strength cannot be expressed in terms of the Simplified Rehabilitation methods of Section 10.3.3.3.B of FEMA 273 (1996) because the fabric does not increase the wall thickness appreciably, and the provisions were not developed with the intention of using high-strength materials.

General Enhancements

<u>Stiffness</u>: The lateral stiffness of a masonry wall or pier component with a fabric overlay may be determined considering composite action between the fabric and the masonry. Differences in elastic moduli for the two materials should be considered when determining properties of the composite section.

<u>Strength Acceptability Criteria</u>

A. In-Plane Shear Strength

Expected strength of unreinforced masonry wall or pier components with fabric, loaded laterally parallel with their plane, should be the lesser of the expected shear or flexural strength. Because of the high tensile strength of the fabric and the possible anchorage of the fabric to the foundation, the governing failure mode may be changed from rocking to tensile straining of the fabric or anchorage, or from bed-joint sliding to shearing of the fabric. The lateral strength for each of these modes needs to be estimated to determine which mode may govern and perhaps whether application of a fabric may be justified.

Flexural and shear strength of a rehabilitated component should be estimated based on composite action of the existing masonry and adhered fabric. Data needs to be obtained for the tensile strength and elastic modulus of the fabric material as well as for the existing masonry to define a transformed section and limiting stress.

B. Out-of-Plane Strength

Adhered fabrics can substantially increase the out-of-plane strength of an unreinforced masonry wall or pier component because of the high tensile strength of the fabric which is placed at the extreme fibers of the section. For resistance to reversed inertial loads, the fabric must be placed on both surfaces of a wall.

A simple, lower-bound estimate of out-of-plane flexural strength can be determined by neglecting the tensile strength of the masonry since the strength of the fabric will be many more times that of the masonry.

Deformation Acceptability Criteria

A. Linear Analysis Procedures

If a fabric is applied to one or both surfaces of an unreinforced wall per the minimum construction requirements given above, then inelastic deformations should be based on the m factors as given in Table 2-7. These factors are in general slightly higher than those for a plain, unreinforced masonry wall and are thus only justified when a check has been made that the high-strength fabric will result in an enhanced ductility over that of the plain, unreinforced masonry wall. If the fabric will convert the rehabilitation wall to force-controlled behavior, then sufficient lower bound strength must be provided.

B. Nonlinear Analysis Procedures

Nonlinear force-deflection relations, and acceptable deflections, for walls rehabilitated with adhered fabrics should be based on the shape coordinates given in Table 2-8. These factors are again based on the premise that the fabric will be governed by deformation-controlled behavior and will provide added inelastic deformation capacity.

Shotcrete Overlays

Description of Rehabilitation Procedure

Shotcrete is sprayed onto the surface of an unreinforced masonry wall over a layer of reinforcement. Reinforcement typically consists of conventional reinforcing bars placed in the horizontal and vertical directions. Wire fabric is not commonly used for reinforcement except for some applications where the shotcrete layer is thin. Dowels are drilled into a wall to provide improved composite action and to anchor the masonry to the shotcrete for out-of-plane loadings.

The purpose of the procedure is to increase one or more of the following:

- flexural strength for walls subjected to out-of-plane loadings
- flexural strength for walls subjected to in-plane loadings
- shear strength for walls subjected to in-plane loadings
- inelastic deformation capacity for walls subject to in-plane loadings.

Minimum Construction Requirements

Shotcrete can be applied to one or both sides of a masonry wall. The coating should be applied to the entire surface of a masonry wall or pier component for maximum effectiveness.

The uniaxial compressive strength of the shotcrete material should exceed a minimum of 3000 psi and should exceed the compressive strength of the masonry to which it is adhered.

Shotcrete should be reinforced with reinforcing bars placed in the horizontal and vertical directions. Minimum amounts of reinforcement should comply with the definition of reinforced masonry in Section 7.8 of FEMA 273 (1996) if the rehabilitated wall is to be considered as reinforced.

Simplified Rehabilitation

Application of shotcrete can be an effective method for increasing the shear strength of a wall as calculated per Section 10.3.3.3.B of FEMA 273 (1996). The shear strength can be estimated using the increased overall thickness of the wall with the shotcrete. If the shotcrete material is stronger in shear than the masonry, this approach will result in a conservative estimate of shear strength.

General Enhancements

Stiffness: The lateral stiffness of a shotcreted masonry wall or pier component may be determined considering composite action between the coating and the masonry. Differences in elastic moduli for the two materials should be considered when determining properties of the composite section.

If the shotcrete is reinforced with at least the minimum amounts of reinforcement, its stiffness should be determined based on Section 6.8 of FEMA 273 (1996) for a reinforced concrete wall.

Strength Acceptability Criteria

A. In-Plane Shear Strength

If the shotcrete is reinforced with at least the minimum amounts of reinforcement, the in-plane shear strength of the rehabilitated wall or pier component should be estimated using the same procedures as given for reinforced concrete walls in Section 6.8 of FEMA 273 (1996).

For cases where the shotcrete contains less than minimum amounts of reinforcement, the in-plane shear strength of the rehabilitated wall or pier component should be estimated using Equation 3-1 for coated walls.

B. Out-of-Plane Strength

If the shotcrete is reinforced with at least the minimum amounts of reinforcement, the out-of-plane flexural strength should be estimated using the same procedures as given for reinforced concrete walls in Section 6.8 of FEMA 273 (1996).

For cases where the shotcrete contains less than minimum amounts of reinforcement, the out-of-plane flexural strength should be estimated using the procedures for coated walls.

Deformation Acceptability Criteria

A. Linear Analysis Procedures

If shotcrete is applied per the minimum construction requirements given above and minimum amounts of reinforcement are provided in the shotcrete, then inelastic deformations should be based on the m factors as given in Section 6.8 of FEMA 273 (1996) for reinforced concrete walls. If the shotcrete layer contains less than minimum reinforcement, m factors should be based on values given in Table 2-7 for coated walls. In certain cases, the m factors for concrete may be lower, but in most cases, the overall capacity of the shotcrete wall will be higher than the plain wall.

B. Nonlinear Analysis Procedures

If shotcrete is applied per the minimum construction requirements given above and minimum amounts of reinforcement are provided in the shotcrete, nonlinear force-deflection relations, and acceptable deflections, for shotcreted walls should be based on the shape coordinates given in Section 6.8 of FEMA 273 (1996) for reinforced concrete walls. If the shotcrete layer contains less than minimum reinforcement, force-deflection curves and acceptable deflections should be based on values given in Table 2-8 for coated walls.

Reinforced Cores

Description of Rehabilitation Procedure

A conventional reinforcing bar is bonded with grout within a vertical and/or horizontal core that is drilled into an unreinforced masonry wall.

The purpose of the procedure is to increase one or more of the following:

- flexural strength for walls subjected to out-of-plane loadings
- flexural strength for walls subjected to in-plane loadings
- shear strength for walls subjected to in-plane loadings
- inelastic deformation capacity for walls subject to in-plane loadings.

Minimum Construction Requirements

The diameter of drilled cores should not be less than four times the diameter of the embedded reinforcing bar.

The deviation of the drilled core from the centerline trajectory should not exceed one-sixth of the wall thickness.

Internal voids within the masonry wall should be filled with grout before core drilling to improve the accuracy of the drilling procedure.

Vertical cores should be drilled from the top of a wall down to the foundation. When anchorage is necessary, adequate embedment within the foundation should be provided to develop the full tensile strength of the bar.

Development lengths shall be in accordance with MSJC (1995).

Sufficient vertical and horizontal cores should be provided so that the minimum amounts of reinforcement will comply with the definition of reinforced masonry in Section 7.8 of FEMA 273 (1996). Insufficient data exists to provide design guidelines for the typical case for in-plane strength when horizontal cores are not provided.

The grout material should be cement grout, sand/epoxy grout or sand/polyester grout. Strength and flow characteristics of epoxy and polyester grouts have been shown to be better than for cement grouts. Polyester grouts are typically less expensive than epoxy grouts.

Simplified Rehabilitation

An unreinforced masonry wall with reinforced cores meeting the requirements above can be considered as a reinforced masonry wall and thus subject to the requirements for the Simplified Rehabilitation Method as prescribed in Section 10.3.3.3 of FEMA 273 (1996).

General Enhancements

Stiffness: The lateral stiffness of an unreinforced masonry wall with reinforced cores should be considered to be the same as a reinforced masonry wall, and thus should be determined in accordance with Section 7.4.4.1 of FEMA 273 (1996) for in-plane loadings, and with Section 7.4.5.1 of FEMA 273 (1996) for out-of-plane loadings.

Strength Acceptability Criteria: The lateral strength of an unreinforced masonry wall with reinforced cores should be considered to be the same as for a reinforced masonry wall, and thus should be determined in accordance with Section 7.4.4.2 of FEMA 273 (1996) for in-plane loadings, and with Section 7.4.5.2 of FEMA 273 (1996) for out-of-plane loadings. Adequate shear strength should exist, or be provided, so that the vertical reinforcement may yield.

Deformation Acceptability Criteria

A. Linear Analysis Procedures

Inelastic deformations should be based on the *m* factors as given in FEMA 273 (1996) for reinforced masonry walls (Section 7.4.4.3.A). These factors are in general higher than those for a plain, unreinforced masonry wall because of the added ductility that the reinforcement will provide.

B. Nonlinear Analysis Procedures

Nonlinear force-deflection relations, and acceptable deflections, for core-reinforced walls should be based on the shape coordinates given in FEMA 273 (1996) for reinforced masonry walls (Section 7.4.4.3.B).

Post-Tensioned Masonry

Description of Rehabilitation Procedure

A steel bar or tendon is placed within a vertical and/or horizontal core that is drilled into an unreinforced masonry wall or pier component. The bar or tendon is tensioned so that the wall is subjected to an axial compressive stress that counteracts flexural tensile stresses resulting from lateral seismic forces. Vertical prestressing bars or tendons are anchored within a concrete foundation below a masonry wall and are stressed against an anchorage block placed at the top of the wall. Prestressing bars or tendons are bonded to the core with grout at their base, and they may be unbonded or bonded along the length of the core.

The purpose of the procedure is to increase one or more of the following:

- flexural strength for walls subjected to out-of-plane loadings
- flexural strength for walls subjected to in-plane loadings
- shear strength for walls subjected to in-plane loadings
- inelastic deformation capacity for walls subject to in-plane loadings.

Minimum Construction Requirements

The diameter of drilled cores should not be less than four times the diameter of bonded prestressing steel, or twice the diameter of unbonded reinforcing steel.

The deviation of the drilled core from the centerline trajectory should not exceed one-sixth of the wall thickness.

Internal voids within the masonry wall should be filled with grout before core drilling to improve the accuracy of the drilling procedure.

Vertical cores should be drilled from the top of a wall down to the foundation. Adequate embedment within the foundation should be provided to develop the full tensile strength of the prestressing bar or tendon.

Development lengths shall be in accordance with MSJC (1995).

Post-tensioning bars or tendons should be stressed to no greater than 75% of their tensile strength.

For bonded prestressing steel, the grout material should be cement grout, sand/epoxy grout or sand/polyester grout. Strength and flow characteristics of epoxy and polyester grouts have been shown to be better than those of cement grouts. Polyester grouts are typically less expensive than epoxy grouts.

The masonry compressive strength must be sufficiently high to resist the post-tensioning stress in addition to the flexural compressive stresses resulting from the lateral seismic forces, and should exceed a value of 1000 psi. In the case of horizontal prestressing, the masonry compressive strength in the horizontal direction should be considered which can be significantly lower than the strength in the vertical direction.

Simplified Rehabilitation

Vertical post-tensioning of existing unreinforced masonry wall or pier components will increase vertical compressive stress. As a result, allowable wall shear stresses as prescribed in Section 10.3.3.3 of FEMA 273 (1996) can be increased, per MSJC (1995).

General Enhancements

Stiffness: The lateral stiffness of a post-tensioned masonry wall or pier component should be considered to be the same as an unreinforced masonry wall with a superimposed compressive stress, and thus should be determined in accordance with Section 7.4.2.1 of FEMA 273 (1996) for in-plane loadings, and with Section 7.4.3.1 of FEMA 273 (1996) for out-of-plane loadings.

Stiffnesses can be larger for prestressed walls because the flexural cracking moment will be increased with prestressing.

Strength Acceptability Criteria: The lateral strength of a wall enhanced with unbonded post-tensioned tendons should be considered to be the same as for an unreinforced masonry wall with an applied vertical compressive stress, and thus should be determined in accordance with Section 7.4.2.2 of FEMA 273 (1996) for in-plane loadings, and with Section 7.4.3.2 of FEMA 273 (1996) for out-of-plane loadings.

If the tendons are grouted, strength of the wall may be considered to be the same as for a reinforced masonry wall, and thus should be determined in accordance with Section 7.4.4.2 of FEMA 273 (1996) for in-plane loadings, and with Section 7.4.5.2 of FEMA 273 (1996) for out-of-plane loadings.

Deformation Acceptability Criteria

A. Linear Analysis Procedures

A wall or pier that is prestressed in the vertical direction with unbonded tendons or bars should be considered as an unreinforced masonry component subjected to a superimposed vertical compressive stress. Acceptable deformations should be per Section 7.4.2.3.A of FEMA 273 (1996). The m factors should be the same as given in Table 1. Factors for wall or pier components that are prestressed with bonded tendons or bars may also be taken from Table 2-5, but they may be increased to account for plastic straining of the prestressing tendons or bars.

As implied in Table 2-5, a moderate increase in vertical compressive stress with prestressing will result in a larger tolerable nonlinear deflections because rocking or bed-joint sliding mechanisms can be developed. An excessive amount of vertical compression stress will result in a reduction in deformation capacity because a toe compression failure may be likely.

B. Nonlinear Analysis Procedures

Nonlinear force-deflection relations, and acceptable deflections, for prestressed walls should be based on the shape coordinates given in Section 7.4.2.3.B of FEMA 273 (1996) for unreinforced masonry walls (and repeated here in Table 2-6).

Values in Table 2-6 are given in terms of the vertical compressive stress which may include the effects of prestressing.

Infilled Openings

Description of Rehabilitation Procedure

Existing window and/or door openings are filled with masonry to provide additional cross sectional area to resist shear and flexural stresses. The method is intended primarily for unreinforced masonry walls.

The purpose of the procedure is to increase one or more of the following:

- flexural strength for walls subjected to in-plane loadings
- shear strength for walls subjected to in-plane loadings
- vertical compressive strength.

Minimum Construction Requirements

The length of opening to be filled should be less than 40% of the overall length of masonry wall per Section 7.4.1.3.A of FEMA 273 (1996). New and existing masonry units should be interlaced at the perimeter of the opening to be filled per Section 7.4.1.3.A of FEMA 273 (1996).

Compressive strength and deformability of new masonry should match that of existing masonry.

Simplified Rehabilitation

Infilling openings in unreinforced masonry walls is a simple way to reduce shear stresses resulting from lateral seismic forces per Section 10.3.3.3.B of FEMA 273 (1996). The larger cross sectional area will result in a lower horizontal shear stress for comparison with allowable shear stresses.

General Enhancements

<u>Stiffness</u>: An infilled opening should be considered to act compositely with the surrounding masonry. The lateral in-plane stiffness of an unreinforced masonry wall with infilled openings should be estimated considering differences in elastic moduli and strengths for new and existing masonries. Other stiffness assumptions should be the same as for an unreinforced wall in accordance with Section 7.4.2.1 of FEMA 273 (1996) for in-plane loadings.

<u>Strength Acceptability Criteria</u>: The lateral in-plane strength of an unreinforced masonry wall with infilled openings should be estimated considering differences in strengths for new and existing masonries. Other strength criteria should be the same as

for an unreinforced wall in accordance with Section 7.4.2.2 of FEMA 273 (1996) for in-plane loadings.

Deformation Acceptability Criteria

A. Linear Analysis Procedures

A wall or pier with infilled openings should be considered as an unreinforced masonry component. Acceptable deformations should be per Section 7.4.2.3.A of FEMA 273 (1996). The *m* factors should be the same as given in Table 2-5.

B. Nonlinear Analysis Procedures

Nonlinear force-deflection relations, and acceptable deflections, for walls with infilled openings should be based on the shape coordinates given in FEMA 273 (1996) for unreinforced masonry walls (Section 7.4.2.3.B and repeated in Table 2-6).

Enlarged Openings

Description of Rehabilitation Procedure

Portions of a perforated masonry wall are removed to alter the height of pier components adjacent to an opening. The purpose of the procedure is to alter the behavior of a pier from a shear-controlled mode to a rocking mode. Thus, lateral in-plane strength will be reduced, but deformation capacity will be increased. Furthermore, cracking will be limited to bed-joint flexural cracks at the top and bottom of the pier.

Minimum Construction Requirements

Masonry portions should be removed without disturbing masonry units, mortar or grout of the remaining masonry.

Simplified Rehabilitation

Increasing the height-to-length aspect ratio of piers will not be an appropriate measure for the Simplified Rehabilitation Method.

General Enhancements

Stiffness: Methods for determination of lateral in-plane pier stiffness are the same as prescribed in Section 7.4.2.1 of FEMA 273 (1996).

Strength Acceptability Criteria

A. In-Plane Shear Strength

Expected in-plane strength of unreinforced masonry piers should be in accordance with Section 7.4.2.2 of FEMA 273 (1996).

B. Out-of-Plane Strength

Out-of-plane strength of unreinforced masonry piers should be in accordance with Section 7.4.3.2 of FEMA 273 (1996).

Deformation Acceptability Criteria

A. Linear Analysis Procedures

A wall with enlarged openings should be considered as a set of unreinforced masonry pier components. Acceptable deformations should be per Section 7.4.2.3.A of FEMA 273 (1996). The *m* factors should be the same as given in Table 2-5.

Increasing the height of door or window openings will result in a lower length-to-height aspect ratio for the piers adjacent to the opening and thus larger *m* factors from Table 2-5.

B. Nonlinear Analysis Procedures

Nonlinear force-deflection relations, and acceptable deflections, for walls with enlarged openings should be based on the shape coordinates given in FEMA 273 (1996) for unreinforced masonry walls (Section 7.4.2.3.B and repeated in Table 2-6).

Increasing the height of door or window openings will result in a lower length-to-height aspect ratio for the piers adjacent to the opening, and thus larger amounts of nonlinear deflection will be tolerable, as noted in Table 2-6.

Steel Bracing

Description of Rehabilitation Procedure

Steel bracing members are added to a building system to share lateral forces with the in-plane unreinforced masonry walls. The purpose of the procedure is to increase the lateral strength and deformation capacity of the structural system.

Minimum Construction Requirements

Steel bracing should be designed in accordance with the requirements set forth in local codes for construction of new steel buildings.

Connections should be provided to transfer lateral forces from the floor and roof diaphragms to the steel bracing elements and the primary unreinforced masonry shear walls.

Simplified Rehabilitation

Adding steel bracing is not an appropriate measure for the Simplified Rehabilitation procedure because no provisions are given for mixed steel-masonry systems.

General Enhancements

Stiffness: The lateral stiffness of a structural system comprised of existing unreinforced masonry walls and new steel bracing should be estimated assuming each element resists its portion of load per relative rigidity. The in-plane stiffness of masonry components shall be determined in accordance with Section 7.4.2.1 of FEMA 273 (1996). The stiffness of steel concentrically braced frames shall be in accordance with Section 5.5.2.2 of FEMA 273 (1996). The stiffness of steel eccentrically braced frames shall be in accordance with Section 5.5.3.2 of FEMA 273 (1996).

Strength Acceptability Criteria: In-plane strength of unreinforced masonry components shall be in accordance with Section 7.4.2.2 of FEMA 273 (1996). Strength of structural steel members shall be in accordance with Chapter 5 of FEMA 273 (1996).

Deformation Acceptability Criteria

A. Linear Analysis Procedures

Because masonry walls are not retrofitted when bracing elements are added to the structural system, acceptable deformations are the same as for unreinforced masonry components per Section 7.4.2.3.A of FEMA 273 (1996). The *m* factors should be the same as given in Table 1.

Acceptable deformations for the steel bracing members are the same as given in Sections 5.5.2.3 and 5.5.3.3 for concentrically and eccentrically braced frames.

B. Nonlinear Analysis Procedures

Nonlinear force-deflection relations, and acceptable deflections, for masonry walls are the same as for unreinforced masonry components per Section 7.4.2.3.B of FEMA 273 (1996). Shape coordinates and acceptable deformations for masonry wall and pier components are given in Table 2-6.

Force-deflection curves and acceptable deflections for steel bracing are the same as given in Sections 5.5.2.3 and 5.5.3.3 for concentrically and eccentrically braced frames.

Table 2-5: Linear Static Procedure:
Recommended *m* Factors for Non-Rehabilitated, Unreinforced Masonry In-Plane Walls and Piers

f_{ae} (psi)	L/h_{eff}	"m" Factors				
		Primary			Secondary	
		IO	LS	CP	LS	CP
0	0.5	1.0	1.5	2.0	1.5	2.0
	1.0	1.0	1.2	1.5	1.2	1.5
	2.0	1.0	1.0	1.0	1.0	1.0
75	0.5	3.0	4.0	5.3	8.0	10.6
	1.0	2.6	3.5	4.7	7.0	9.3
	2.0	2.2	3.0	4.0	6.0	8.0
150	0.5	1.5	3.0	4.0	6.0	8.0
	1.0	1.3	2.2	3.0	4.4	6.0
	2.0	1.0	1.5	2.0	3.0	4.0
source: Section 7.4.2.3 of FEMA 273 (1996)						

Table 2-6: Nonlinear Static Procedure:
Recommended Simplified Force-Deflection Relations for Non-Rehabilitated Unreinforced Masonry In-Plane Walls and Piers

f_{ae} (psi)	L/h_{eff}	c	d %	e %	Acceptance Criteria				
					Primary			Secondary	
					IO %	LS %	CP %	LS %	CP %
0	0.5	0.0	0.10	0.10	0.08	0.08	0.10	0.08	0.10
	1.0	0.0	0.10	0.10	0.08	0.08	0.10	0.08	0.10
	2.0	0.0	0.10	0.10	0.08	0.08	0.10	0.08	0.10
75	0.5	0.7	0.60	1.20	0.25	0.45	0.60	0.90	1.20
	1.0	0.6	0.50	1.00	0.15	0.38	0.50	0.75	1.20
	2.0	0.5	0.40	0.80	0.10	0.30	0.40	0.60	0.80
150	0.5	0.5	0.40	0.80	0.20	0.30	0.40	0.60	0.80
	1.0	0.4	0.30	0.60	0.10	0.22	0.30	0.45	0.60
	2.0	0.4	0.20	0.40	0.08	0.15	0.20	0.30	0.40
source: Section 7.4.2.3 of FEMA 273 (1996)									

**Table 2-7: Linear Static Procedure:
Recommended *m* Factors for Coated[1] Unreinforced Masonry
In-Plane Walls and Piers**

f_{ae} (psi)	L/h_{eff}	"m" Factors Primary IO	Primary LS	Primary CP	Secondary LS	Secondary CP
0	0.5	1.0	1.5	2.0	1.5	2.0
	1.0	1.0	1.2	1.5	1.2	1.5
	2.0	1.0	1.0	1.0	1.0	1.0
75	0.5	3.0	4.0	5.3	8.0	10.6
	1.0	3.0	4.0	5.3	8.0	10.6
	2.0	2.5	3.5	5.0	7.0	10.0
150	0.5	1.5	3.0	4.0	6.0	8.0
	1.0	1.5	3.0	4.0	6.0	8.0
	2.0	1.2	2.0	3.0	4.0	6.0

[1] May also apply to walls with a fabric overlay, or multi-wythe walls with a grouted collar joint.

**Table 2-8: Nonlinear Static Procedure:
Recommended Simplified Force-Deflection Relations for
Coated[1] Unreinforced Masonry In-Plane Walls and Piers**

f_{ae} (psi)	L/h_{eff}	c	d %	e %	Acceptance Criteria Primary IO %	Primary LS %	Primary CP %	Secondary LS %	Secondary CP %
0	0.5	0.0	0.10	0.10	0.08	0.08	0.10	0.08	0.10
	1.0	0.0	0.10	0.10	0.08	0.08	0.10	0.08	0.10
	2.0	0.0	0.10	0.10	0.08	0.80	0.10	0.08	0.10
75	0.5	0.8	0.60	1.20	0.25	0.45	0.60	0.90	1.20
	1.0	0.7	0.60	1.20	0.25	0.45	0.60	0.90	1.20
	2.0	0.6	0.40	0.80	0.15	0.38	0.50	0.75	1.00
	0.5	0.6	0.40	0.80	0.20	0.30	0.40	0.60	0.80
	1.0	0.6	0.40	0.80	0.20	0.30	0.40	0.60	0.80
150	2.0	0.5	0.30	0.60	0.12	0.22	0.30	0.45	0.60

[1] May also apply to walls with a fabric overlay, or multi-wythe walls with a grouted collar joint.

2.5 Cost Effectiveness

Background

Each of the procedures identified above has both an initial cost and a potential benefit of enhancing performance in an earthquake. There are several ways to estimate the potential benefit.

One method is to review empirical data from past earthquakes to compare performance of buildings with and without the enhancement procedures. Unfortunately, some of the enhancement procedures described above have limited or no actual usage; more importantly, past damage data generally is not broken down by such specific variables. See Appendix A for a discussion of the limits of past data.

Perhaps the most common method of estimating benefits is to use probabilistic seismic risk analysis techniques which relate ground shaking to generalized earthquake performance. Damage probability matrices such as those of ATC-13 (1985) link the ground motion to the percentage of repair vs. replacement cost. Thus, the benefit is measured in the reduction in this percentage. Similar relationships can be developed to estimate loss of function. Rutherford & Chekene (1990) provides an example of this method. Not only is such a loss estimate procedure far beyond the scope of this project, but there is little data to support the refinements which would be necessary to develop damage probability matrices which could capture the potential variation between each enhancement procedure.

Another method is to use experimental results to categorize the enhancement procedure benefits in terms of variables such as drift, strength and ductility. Benefits could be measured by the reported amount of drift reduction or strength and ductility increase. For the quality of design and construction enhancement procedures, there is, however, no experimental data. For other enhancement procedures, such as the wall rehabilitation methods, the experimental data is of limited benefit because, as Section 3.2 notes, nearly all of the studies were done without the benefit of coordination with other test programs, and, as a result, specimen geometries, materials, loading techniques, data presentation and types of conclusions vary considerably.

Although the experimental data are difficult to compare, the design guidelines in Section 2.4 are in a deliberately consistent format. Thus, we have developed estimates of the increase in shear capacity provided by the wall enhancement methods. Note that several methods lack explicit quantitative design guidelines at this point in time and a significant number of assumptions must be made to apply the quantitative guidelines for the other methods. Table 2.16 provides a summary; Appendix D provides greater detail.

Of course, such estimates do not apply for most of the other enhancement procedures. Instead, the discussion of benefits is qualitative, so the engineer can determine together with the cost estimates provided which procedures are most cost effective on a specific project. It was possible to provide cost estimates for most, but not all, of the procedures in Sections 2.2-2.4.

The following discussion of benefit-cost information for enhancement procedures is grouped into the three categories of Sections 2.2-2.4: Quality of Design and Construction, Design Criteria, and Rehabilitation Methods. For each procedure, there is a rationale describing the procedure's qualitative benefits, a description of the procedure, and a summary of the assumptions used in preparing the cost estimate. For the wall enhancement methods, a quantitative estimate of the increase in shear strength provided by the enhancement is also given. Cost estimates, given on a dollar per square foot basis, are based on a prototypical three story, 40' x 80', unoccupied commercial building, with floors and roofs constructed of wood sheathing over wood joists. In order to reflect variation in costs due to variation in labor rates and building size, a high and low estimate is reported for each procedure.

Quality of Design and Construction

Improved Knowledge of the Building

In general, improving the quality of design and/or construction should result in improved reliability of performance. In the Northridge Earthquake, much of the poor performance was attributed to poor quality design and construction (LATF, 1994 and SSC, 1995). Poor knowledge of the building construction will limit the quality of the seismic evaluation and hinder design of suitable or necessary details for rehabilitation. Poor knowledge of material properties will prevent reasonable estimates of material capacity to resist seismic loads. The procedures described below, by improving the knowledge of the building, should improve the evaluation and design process which should lead to enhanced performance in a retrofitted building.

- Exposing masonry wall-to-diaphragm connections will provide information that can be used to properly detail tension tie and shear transfer connections.
- Exposing the crosswall-to-diaphragm connections can provide information to verify the adequacy of the load-transfer mechanism and the assumption of crosswall participation in the seismic response of a building.
- Exposing connections requires removing floor and ceiling finishes.
- Verifying wall construction can provide information necessary to determine in-plane strength and height-to thickness (h/t) ratios.
- In-plane push tests on both interior and exterior wall surfaces can provide better estimates of mortar shear strength.
- Drilling into walls can identify the presence of cavities.
- Veneer tie spacing can be determined by pacometer testing or investigating with a borescope; veneer may require removal to determine tie condition.
- Pull testing veneer ties can determine tie capacity to resist out-of-plane forces.

- Identifying interior wall construction can help refine estimates of building weight and stiffness and can confirm which walls may be used as crosswalls under the UCBC (1994) Special Procedure. Stud wall construction can be identified by means as simple as "sounding" walls by tapping on them. Drilling and borescope investigation can determine whether masonry walls are constructed of clay brick, hollow clay tile, or concrete masonry units. They can also determine whether concrete masonry unit cells are grouted or ungrouted.

Thorough Design

Thorough design requires that the finished set of construction documents correctly address seismic deficiencies identified through field investigation, testing, and structural evaluation. A set of documents so designed will not principally rely on typical details, many of which may not apply to actual conditions, but instead will contain details which reflect existing conditions. Other aspects of thorough design include: detailing at corners; special consideration of rigid ceilings; special consideration of veneers, nonbearing URM walls, damaged or deteriorated masonry, configuration irregularities; special consideration of wall anchorage; and written design criteria. Section 2.2 of this report describes a number of other aspects included in thorough design. The cost of thorough design can vary enormously from building to building. Small, single-story buildings will require much less time to investigate, test, understand and document, while large, complex buildings or buildings which have been extensively or frequently remodeled will require a much greater effort. Engineers' fees can vary from as little as 0.025% to as much as 10% of construction costs.

Table 2-9: Estimated Costs of Various Field Investigations

Investigation Activity	Estimated Cost ($/sf)	
	Low	High
Exposing masonry wall-to-diaphragm connections	0.10	0.17
Exposing crosswall-to-diaphragm connections	0.12	0.20
Verifying wall cross section *	0.37	0.52
In-plane mortar shear strength tests on exterior and interior of perimeter walls *	0.75	1.05
Verifying the presence of veneer ties and spacing by pacometer testing *	0.13	0.18
Verifying veneer tie condition and spacing by borescope testing *	0.16	0.22
Exposing and pull-testing veneer ties *	0.19	0.28
Identifying interior wall construction	0.05	0.07

1. Estimated costs are for the prototypical building. A description of the scope of work for each activity is given in Section D.2 of Appendix D.
2. "*" indicates cost of scaffolding comprises more than 33% of total estimated cost.

Peer Review/Plan Check

Three levels of peer review can contribute to a thorough design. The first level occurs prior to commencing evaluation and design, when the evaluation and design methodologies proposed by the engineer of record are reviewed to help verify that they will meet the performance objective. The second occurs after schematic design, to help verify that the schematic concept uses the specified design methodology and will meet the performance objective. Upon completion of construction documents or at discrete stages during construction document preparation, a third, detailed review of the completed design drawings and specifications can be made, to review the details of the design and possibly recommend changes and improvements.

Plan check review by the governing municipality, where not otherwise code mandated, can help to enhance quality and improve performance.

Table 2-10: Estimated Cost of Peer Review

Review Level	Estimated Cost ($/sf)	
	Low	High
Design Methodology and Criteria Review	0.21	0.31
Schematic Design Review	0.28	0.41
Construction Document Review	0.53	0.96
Estimated costs are for the prototypical building. A description of the scope of work for each level of peer review is given in Section D.2 of Appendix D.		

Table 2-11: Cost of Plan Check Review

Construction Cost	Plan Check Fee
$25,000 to $50,000	$229 plus $5.85 for each additional $1000 of construction
$50,001 to $100,000	$377 plus $4.06 for each additional $1000 of construction
$100,001 to $500,000	$582 plus $3.25 for each additional $1000 of construction
$500,001 to $1,000,000	$1856 plus $2.76 for each additional $1000 of construction
over $1,000,000	$3221 plus $1.78 for each additional $1000 of construction
Fees are based on Section 107.3 of the 1994 Uniform Building Code.	

Field Review

Field review can help ensure that construction conforms to the contract documents. Inspection by on-site special inspectors can identify non-conforming construction which might otherwise go unnoticed. Testing, such as shotcrete compression tests, can verify correct material properties, or verify proper installation, as with dowel pull and torque tests. Estimated costs for special inspection and testing for nine wall enhancement techniques applied to the prototype building are included in Table 2-16. Site visits provide the structural engineer with the opportunity to review construction for general conformance with the design intent and can alert the contractor to non-conforming conditions. Site visits can also facilitate problem solving by providing firsthand observation of existing conditions and by opening a channel of direct communication with the contractor.

Table 2-12: Estimated Cost of Site Visits by the Engineer of Record

Minimum Level of Site Visits		Enhanced Level of Site Visits	
Low ($/sf)	High ($/sf)	Low ($/sf)	High ($/sf)
0.27	0.41	0.81	1.24
Appendix Section D.2 defines the level of site visits and describes the assumptions upon which estimated costs are based. Estimated costs are for the prototypical building.			

Design Criteria

The Scope of Regulated Elements

One means of enhancing performance in low and moderates zones of seismicity is to quantitatively evaluate and, if required, to strengthen elements which are not required in current standards of practice. The estimated costs of specific rehabilitation activities are presented in Table 2-13. Estimates are based on costs reported in FEMA 156 (1988) and are adjusted to 1996 cost for construction in San Francisco.

Table 2-13: Estimated Cost of Specific Rehabilitation Activities

Activity Required for High Seismicity Region	1996 Cost in San Francisco ($/sf)
Wall Bracing (h/t)	0.66
Plywood Shear Walls	1.16
Shotcrete	1.12
Roof Diaphragm & Roofing	0.93
Floor Diaphragm	0.38
1. Costs per square foot are the structural costs based on FEMA 156 (1988) for the prototypical building. 2. Activities are not required by FEMA 178 (1992) in low seismicity regions. Implementation of these specific activities will provide performance enhancement.	

Lessons Based on Analysis of Damage Patterns from the Northridge Earthquake

Based upon the analysis in Appendix A, buildings rehabilitated to UBC Zone 4 (A_a = 0.4) criteria, suffered little or no damage for ground motions less than those associated with A_a of about 0.2. It can be expected, therefore, that buildings in moderate seismic zones that are rehabilitated to similar standards would also suffer little or no damage, similar to the FEMA 273 (1996) Performance Level of Immediate Occupancy, for ground motions limited to an A_a of 0.2 ($Sa_{0.3}$ of approximately 0.75).

Based on these assumptions, the cost increase required to obtain little or no damage is the *difference* between a life safety rehabilitation in an A_a=0.2 zone and in an A_a=0.4 zone. A very rough estimate of the range of the cost increase can be obtained from using FEMA 156 (1994) cost estimation methods. Estimates for the total cost of rehabilitating the prototype building for moderate (roughly A_a=0.2) and very high areas of seismicity (such as those with A_a=0.4) are presented in Table 2-14.

Rehabilitation Methods

Traditional Out-of-Plane Bracing Alternatives

Out-of-plane failures of masonry walls occur with greater frequency than in-plane wall failures. Unacceptable height-to-thickness (h/t) ratios have frequently been mitigated by installation of strongbacks or by installation of diagonal braces. Walls with diagonal braces were observed to have failed in the Northridge Earthquake. Using strongbacks in lieu of diagonal braces may enhance performance.

Table 2-14: Range of Estimated Costs for Seismic Rehabilitation of the Prototypical Building

Area Seismicity	Range of Cost ($/sf)		
	Low	Median	High
Moderate	5.28	13.19	32.73
Very High	8.90	22.24	55.17

1. Costs per square foot are the structural costs based on FEMA 156 (1994).
2. The range of values are based on a confidence interval of 50% for a single building, the life safety performance objective, and they are estimated using Option 2 with 1996 dollars.

Table 2-15: Estimated Cost of Wall Out-of-Plane Enhancement

Enhancement Technique	Estimated Cost ($/sf)	
	Low	High
Diagonal Bracing	3.29	4.91
Strongbacks	5.39	7.87

Wall Enhancement Methods

Several methods of wall strengthening can enhance wall performance under in-plane and/or out-of-plane loading. Methods such as applying shotcrete to a wall surface or center-coring walls have been extensively implemented on the West Coast, while others, such as adhered fabrics or grouting, have seen limited usage. Costs are given in Table 2-16 for wall enhancement methods. An estimate of the increase in shear capacity provided by the enhancement is also given; see Section D.7 for details.

Table 2-16: Estimated Construction Costs of Wall Enhancement Methods[1]

Fig. No.	Wall Enhancement Methods	Structural Construction Costs ($/sf)		Architectural Costs[2] ($/sf)		Premium if Occupied[3] ($/sf)	Testing & Inspection Costs ($/sf)	Enhanced Shear Capacity/ Plain Shear Capacity ($m_{enhanced} \times Q_{CEenhanced} / m_{plain} \times Q_{CEplain}$)
		Low	High	Low	High			
D-1	Grout & Epoxy Injection	8.63	15.81	0.25	0.46	4.17	0.35	3.0[4]
D-2	Surface Coatings	12.71	23.31	0.62	1.13	4.17	0.66	1.2-1.3[5]
D-3	Adhered Fabric	11.53	21.14	0.62	1.13	4.17	1.68	NA[6]
D-4	Shotcrete Overlay	7.20	13.20	0.41	0.75	2.78	0.43	2.4[7]
D-5	Reinforced Cores	13.90	25.48	0.00	0.00	0.83	0.17	NA[8]
D-6	Post-Tensioned Cores	14.94	27.31	0.00	0.00	0.83	0.22	2.0[9]
D-7	Infilled Openings	2.65	4.85	0.01	0.02	3.13	0.00	1.5
D-8	Enlarged Openings	2.81	5.15	0.25	0.46	4.17	0.00	NA[10]
D-9	Steel Bracing	9.29	17.03	1.96	3.59	2.78	0.53	3.2[11]

1. Estimated costs are based on the scope of work depicted in Figures D-1 to D-11. See text for additional assumptions.
2. Low costs include addressing the impact to carpet floors and plaster on adjacent interior walls and ceilings (including repainting). High costs include a premium for higher quality finishes including quarry tile or hard wood floors, wood base boards and window moldings.
3. The premium for working in an occupied building includes provision of facilities that would otherwise be available in the building (e.g. storage space), and (where appropriate) includes dust/security screens, isolation of working area adjacent to the wall, and removing the same on completion of the work.
4. Does not account for increase in wall mass due to added grout.
5. Lower value is for 1/2" coating on each side; higher value is for 1" coating on each side.
6. Equations and methodology need to be developed through additional research.
7. Does not account for increase in mass due to shotcrete; enhanced capacity only includes shotcrete contribution, and it ignores interaction with masonry issues.
8. Design guidelines do not apply without horizontal reinforcing, but limited tests have shown substantial increases in shear capacity with vertical-only reinforced cores.
9. Only accounts for increase in shear capacity due to increase in compressive stress.
10. Purpose is to change behavior to rocking mode, not to increase shear strength.
11. Enhanced capacity only includes steel contribution; issues related to interaction with the masonry are ignored.

Section 3: Unresolved Issues and Recommended Testing and Research

The recommendations in Section 2 represent a first step toward future development of formal guidelines for enhanced performance of rehabilitated URM buildings. Based on the work done in this study and summarized in the appendices and other previous studies of URM damage, resolution of many issues will require additional testing and/or research. The sections that follow provide recommendations on these issues.

3.1 Research Recommendations for Construction Practices Common in Areas Outside of California

Information about the existing URM building inventory for geographical regions around the United States was obtained from publications on historic building construction, early city building codes, consultants, and a survey of engineers in several moderate and high seismicity areas. Appendix B summarizes the results of these investigations and draws conclusions about building construction practices where modifications or refinements to current retrofit methodologies are necessary. The procedures given in Section 2 address these issues in part, but some issues remain largely unresolved, and additional testing and research is recommended. These issues include hollow clay tile or structural clay tile bearing walls, cavity wall construction, and rigid diaphragms.

Ungrouted Hollow Masonry Unit Bearing Walls

Although current retrofit methodologies allow a wide range of URM materials, they were developed primarily for solid brick masonry units. Many areas outside of California have a large stock of buildings which have bearing walls made of ungrouted hollow concrete masonry units (CMU). In some areas, structural clay tile (SCT) or hollow clay tile (HCT) bearing walls are used as well. In attempting to apply current retrofit provisions to hollow units, a number of issues arise, given the paucity of testing on these materials; these issues include:

- The current in-place push test provisions were developed for and originally correlated to component testing and finite element modeling of solid brick. CMU, SCT and HCT walls often are only one-wythe thick and may require modification of current equations to account for differences in material properties, lay-up, and the lack of a collar joint. In addition, where mortar is relatively strong in comparison to the unit, special precautions and variations in testing procedures may be necessary to avoid failing the unit wall before yielding the bed joint.

- Because they are hollow, HCT, SCT and, to a lesser extent, CMU piers all are more likely to be susceptible to toe crushing failure mechanisms than solid brick. Testing is necessary to determine quantitatively when toe crushing is likely.

- The most basic component of current retrofit provisions is the requirement to tie the diaphragms to the URM walls. Drilled epoxy dowels or throughbolts are the two typical retrofit anchor approaches. A substantial amount of proprietary static testing has been done for these products. To a lesser degree, a few companies have done testing of products for hollow units. Capacities are substantially lower because spalling of the face shell is usually the governing failure mechanism. Additional cyclic or dynamic testing is necessary to determine the adequacy of such anchors, both when they are placed in the face shell and in the bed joint. These tests should also cover the retrofit approach used by some engineers of pressure grouting the course where the unit is to be anchored.

- Current retrofit provisions typically disallow masonry which is laid in stack bond. Hollow masonry walls are more likely than solid walls to be laid in stack bond, and thus there is greater need to determine acceptability criteria and retrofit strategies for stack bond walls.

Cavity Wall Construction

Current retrofit methodologies are based on the typical California-style brick bearing wall building, where the wall is assumed to behave monolithically. Cavity wall construction, widely used in many parts of the country outside of California, often involves construction where the wall wythes are separated by an air gap. In modern buildings, a typical example is where one-wythe of brick veneer covers a CMU backing wall. Other examples, common to the Northeast and shown in Appendix B, have the air gap in the center of the wall. Two basic issues require additional study: the adequacy of veneer provisions and the adequacy of h/t provisions for the wall itself.

Current retrofit provisions for veneer were intended to address the typical case where the collar joint between the bearing wall and the brick is at least partially filled with mortar. In this situation, it is unlikely that the veneer will begin to respond in a dynamically incompatible mode from the out-of-plane behavior of the wall. In addition, this partial bond provides some ability to transfer in-plane shear forces between the bearing wall and the veneer.

Many of the general testing needs discussed in Section 3.3 for veneer apply to cavity wall veneer, but they also need to account for the assumed lack of monolithic behavior between the veneer and backing. Thus, it is even more important that tests be dynamic. Basic questions that testing can help address include:

- What spacing and types of existing anchorage are adequate?

- Are h/t values including only the backing thickness adequate? How should the split in overburden between the veneer and backing be addressed?

- Are retrofit anchors developed for veneer with partially filled collar joints adequate for cavity wall construction?

- If a ledger angle is used to support the veneer, what are the criteria for acceptability?

- Are there any compatibility problems between the veneer and the backing during in-plane loading?

Perhaps an even more basic question with cavity wall construction, particularly with a central cavity, is whether the h/t provisions developed for solid walls can be applied. If the engineer assumes the thickness of the wall is limited to the narrowest portion on one side of the cavity, it may not be possible for any walls to meet the requirements. Solid walls subjected to out-of-plane test loads in the ABK (1984) program were found to crack at midspan and rock. The reliability of such models would appear to be reduced when a cavity exists at the plane of rocking. Testing is warranted as well as further guideline development.

Buildings with Rigid Diaphragms

Although current retrofit methodologies allow rigid diaphragms, the original research on which these methodologies were based primarily addressed flexible diaphragms. There are a variety of types of rigid diaphragms found in URM buildings, primarily in areas outside California. These include concrete slabs spanning between steel beams, brick and HCT arches spanning between steel beams, and hollow concrete planks. Buildings with rigid diaphragms will respond to earthquake shaking in a substantially different manner than those with flexible diaphragms.

With flexible diaphragm buildings, the assumption is that the ground motion is delivered, essentially unamplified, by rigid in-plane walls to the floor and roof diaphragms where the flexible diaphragm and coupled out-of-plane wall response is amplified substantially. In large events, the diaphragm is assumed to yield and limit the force transmitted between the wall and diaphragm. In a rigid diaphragm building, diaphragm yielding is unlikely and the frequency of response of the walls and diaphragms is likely to be much closer. As such, current retrofit provisions for rigid diaphragms are limited in the UCBC (1994) to the General Procedure approach which references the typical seismic provisions for new buildings for all elements of the building except in- and out-of-plane wall provisions. There remain, however, several issues for which additional research and testing may be desirable; these include:

- Current h/t provisions were established when wall assemblages were tested using input motions derived primarily from relatively flexible diaphragms. With more rigid diaphragms, are the provisions adequate?

- With flexible diaphragm buildings, a tributary area analysis is usually assumed in determining the load tributary to a wall. Current Simplified Rehabilitation provisions address each lateral force-resisting wall line separately. The capacity of one wall line is not assumed to have any effect on other lines. In rigid diaphragm buildings, this is clearly not a reasonable assumption, since the diaphragm is likely to be capable of redistributing forces to other wall lines when the capacity of one is reached. The FEMA 273 (1996) Systematic Rehabilitation Nonlinear Static and Nonlinear Dynamic Procedures can be used to address this redistribution, but further guideline development is necessary to develop appropriate Linear Static and Linear Dynamic Procedures as well as simplified provisions.

- There are no values for checking floor systems such as brick or HCT spanning between steel beams. Besides cracking or crushing at boundary edges, such systems also have the potential for sliding within the diaphragm between the masonry and steel and for out-of-plane failures if the arch compression is lost. Testing should be performed, and empirical earthquake performance data should be collected for these systems.

3.2 Needed Research for Wall Enhancement Methods

General Research Needs

Consistent Research

The summaries of past research on various strengthening methods for walls illustrate the diversity of the methods studied as well as the range of research perspectives, experimental techniques and modes of reporting. Nearly all of the studies were done without the benefit of coordination with other test programs, and as a result, specimen geometries, materials, loading techniques, data presentation and types of conclusions vary considerably. Efforts to consolidate such information into a set of comprehensive guidelines are encumbered by these variances. Future research projects need to be motivated and directed to achieve a common product, such as a set of improved rehabilitation guidelines, rather than be planned and executed serendipitously by individual researchers.

Needed Parameters for Performance-Based Design

In general, testing of various wall enhancement methods needs to be done with a common perspective towards performance-based design. Few, if any, of the publications summarizing laboratory test programs provide data on behavior that can be used to index various Performance Levels such as Immediate Occupancy, Life Safety or Collapse Prevention. For future research, drift levels corresponding to these Performance Levels should be reported by researchers observing laboratory experiments. Data should be presented using the newly defined parameters contained in FEMA 273 (1996) such as an *m* factor for deformation-based acceptance criteria and standard dimensions defining the nonlinear force-deflection curves.

Enhancing Deformation Capacity

The acceptance criterion for deformation-controlled components per FEMA 273 (1996) Linear Static Procedure is based on the product of an *m* factor (representing component ductility) times the expected strength. Thus, seismic performance can be enhanced by increasing strength, as is traditionally been done, and/or by increasing inelastic *deformation* capacity. Moreover, the acceptance criterion per the Nonlinear Static Procedure is based on the capability of a component to *deform* a specific amount within the inelastic range. Nearly all of the past research has focused on retrofit methods that will increase strength, but few studies have been directed towards methods to implicitly increase deformation capacity. Future research should develop strengthening techniques that will not necessarily compromise deformation capacity, as well as techniques that are primarily intended to increase ductility.

Undesirable Interventions

One rehabilitation technique may alter behavior from a deformation-controlled mode to a force-controlled mode while another technique may result in a reciprocal alteration. For example, a ductile bed-joint sliding mode may be precluded if an open collar joint between wythes is filled with a strong, but brittle grout. Or, conversely a shear-critical pier can be transformed to a pier controlled by a ductile rocking mode by increasing its height-to-length aspect ratio. Before the influence of the intervention measure can be known, the force-deflection relation for the original unreinforced masonry component must first be assessed. Research needs to be done to improve methods for evaluating behavioral modes for existing unreinforced masonry wall and pier components, and to identify which rehabilitation techniques may result in non-compatible properties with respect to stiffness, strength and deformation capacity.

Research on Specific Rehabilitation Techniques

The summaries of individual research projects in Appendix C provide an overview of the types of information that has been developed with past research. The following research needs for specific techniques have been formulated by contrasting this collection of information with the content of the new FEMA 273 (1996) and common rehabilitation practices for unreinforced masonry components.

Grout Injections

Research needs to be done to define the interaction between a grouted collar joint and existing clay-unit masonry shear walls. Expressions need to be improved for estimating the lateral in-plane strength as a function of the relative participations of the existing masonry and the grouted collar joint. Research is needed to investigate the effectiveness of grouting hollow-unit masonry for improving in-plane lateral force-deflection behavior.

Surface Coatings

A considerable amount of tests have been done on coated masonry walls. The experimental database, however, needs to be expanded to investigate the role of the coating on improving deformation capacity for original walls governed by rocking, bed-joint sliding or diagonal-tension cracking.

Adhered Fabrics

Research results on adhered fabrics is only now just emerging. Equations and/or procedures need to be formulated to express the lateral in-plane strength of wall panels enhanced with adhered fabrics. Continued efforts are needed to examine how fabrics may alter behavior modes for deformation-controlled components. Also, long-term effects of epoxy adhesive need to be studied as well as the cost effectiveness of using high-strength fabrics.

Shotcrete

Despite the popularity of shotcrete, little experimental research has been done to quantify the composite characteristics between the shotcrete and the original masonry for walls resisting in-plane loadings. Common engineering practice is to assign all of the shear to the new shotcrete overlay and to neglect the contribution of the original masonry. This is an admissible practice for estimating a lower-bound shear capacity but does not give a reliable estimate of the expected capacity which is needed for deformation-controlled components per FEMA 273 (1996). Furthermore, extensive cracking of the original masonry may need to occur before the reinforcement in the shotcrete develops its strength. Research is needed to define the extent of masonry damage so that more stringent acceptance criteria can be formulated for Performance Levels in the range of Immediate Occupancy.

Reinforced Cores

Few experiments have been done on wall samples with grouted and reinforced cores to determine similarities with reinforced masonry construction. The behavior of wall panels with only vertical reinforcement is of interest since horizontal coring is generally impractical. Research is needed to determine how shear strength can be enhanced to be consistent with the increased flexural strength that vertical cores provide. Specifically, future research needs to define what enhancements in shear strength are provided by the grouted cores and how these enhancements may alter shear behavior from a bed-joint sliding mode to a diagonal tension mode.

Post-Tensioned Cores

Testing of post-tensioned wall panels is needed to identify differences in force-deflection behavior for walls post-tensioned with ungrouted and grouted tendons. Walls with ungrouted tendons will respond nonlinearly but elastically and may amplify seismic motions differently than walls with grouted tendons that respond as nonlinear inelastic systems. The applicability of the Linear Static Procedure of FEMA 273 (1996) needs to be investigated for such nonlinear systems that are elastic. If the procedure is found appropriate, m factors need to be developed that will represent these systems. Behavior of walls with grouted tendons should be contrasted with behavior of reinforced masonry walls. The influence of grouted cores on increasing shear strength through keying action also needs to be investigated with respect to the alteration of behavior modes.

Infilled Openings

Few tests, if any, have been done to examine how a wall will behave when window or door openings are filled with new masonry. If the stiffness of the new masonry is much different than that of the original masonry, significant differences in stresses will be developed which could result in undesirable cracking and reduced deformation capacity. Testing can also help establish minimum construction requirements for preparation of masonry around the perimeter of the infilled opening and the maximum size of opening that can be infilled.

Enlarged Openings

Enlarging door or window openings so that the height-to-length aspect ratio of adjacent piers will be increased has been found to be a suitable method for increasing deformation capacity because behavior can be reversed from a shear-critical mode to a rocking mode. Testing needs to be done to determine the rocking behavior of a group of piers of various heights and aspect ratios. Differential vertical deflections of rocking piers could result in an uplifting of gravity forces from an individual pier and a reduction in rocking strength for that pier. Research needs to be done to determine whether the lateral in-plane strength of the pier group is the same as the sum of the rocking strengths of the individual piers.

3.3 Testing Needs for Retrofit Techniques Unrelated to Wall Strengthening

Review of publications on damage to URM buildings and experience with URM retrofitting has led to the conclusion that, in addition to wall strengthening techniques, there are other retrofit techniques which require more testing to help resolve concerns about their adequacy to provide enhanced performance.

Veneer

Many URM buildings in the Northridge Earthquake, including those which had been retrofitted, had veneer delamination failures. To a lesser degree, similar delamination was observed in the Loma Prieta Earthquake. As noted by EERI (1996) for the Northridge Earthquake:

> *There were numerous observations of masonry veneer that had separated from the supporting walls and fallen on the ground or on adjacent buildings....It has been suggested that the veneer was not anchored during the retrofit because it was not identified. Another cause of failure was that the original veneer anchors were inadequate either by the original design or by deterioration. Additionally, exact demands on rigid or brittle veneer anchors from in- and out-of-plane deformation of the wall have not been defined. Current veneer anchorage requirements concentrate on horizontal support of the inertial forces of the veneer rather than deformation compatibility.*

Existing and retrofit ties represent two fairly different conditions and are discussed separately below.

Existing Ties

FEMA 178 (1992) and UCBC (1994) define the type, spacing and condition requirements for existing ties. Corrugated, galvanized iron strips laid in the bed joints are assumed. These ties sometimes have a nail-like protrusion which is placed in a cut in the top of the brick to form a mechanical tie. Otherwise, the ties rely on the unit-to-mortar and mortar-to-tie bond for resisting out-of-plane demands. Review of the literature has not uncovered any actual testing which has been done to confirm that, if current provisions requirements are met, then the expected performance (which is that the brick does not fall) can be reliably assumed. Though the majority of most delamination failures may, in fact, be because veneer ties were missing, it is recommended that, since delamination was found in some walls with existing ties, testing be performed to determine existing tie acceptability. The most desirable type of test would be a shaking table test of at least a wall/veneer assemblage where the variables addressed would include: input motion, wall backing displacement/strain, veneer displacement/strain, bed joint mortar shear strength, collar joint fill, tie spacing, tie condition, tie type, whether the tie has a positive hook into the veneer unit, and the depth of tie anchorage into the backing. Alternative tests could involve removing the brick veneer and pull-testing the tie to help confirm the anchorage into the backing.

Retrofit Ties

The UCBC (1994) defines spacing for retrofit ties and references the UBC for demand loads. In addition to removal of the veneer, there are several standard retrofit tie approaches. These approaches and potential limitations are given below. Testing similar to that noted above for existing ties is recommended.

- Interior drilled dowel: When there is access from the inside face of the backing, a drilled dowel is installed which penetrates through the backing into the veneer. The dowel is set in epoxy. Since the dowel must be set back from the exterior face of the wall to limit spalling during dowel placement, a basic issue is whether enough capacity can be developed in the connection between the dowel and the veneer unit. Also, it is usually difficult with such "blind" drilling to locate the dowel in the center of the unit.

- Placing a tie in the center of the veneer unit and patching the hole: The tie can be 1) a stainless steel rod placed in an epoxy filled hole or 2) a proprietary spiral tie which involves drilling a pilot hole smaller than the tie and then mechanically driving the tie into the pilot hole. Although either of these approaches are likely to be adequate structurally, they are aesthetically unpalatable and are often considered architecturally unacceptable, given the difficulty of color and texture matching the stone patches.

- Epoxy dowels placed in the bed joint: This is a common approach to avoid damaging the veneer unit. The basic question remains: Is the mortar-to-veneer bond adequate in a large earthquake?

- Placing a mechanical tie in the bed joint: There are two basic types: wedge anchors and spiral ties. The proprietary double wedge anchor systems involve drilling a hole into the brick backing, where the first wedge anchor is placed and tightened, then a second anchor is placed against the stone veneer and tightened. An additional 9 gauge "seismic" wire can be placed in the bed joint for added reliability (and added repointing cost). This system can be less expensive than epoxy ties, but the size of the holes are about 1/2"-5/8", which may be too large for the narrow bed joints. Also wedge anchors relax over time, and there do not appear to be any realistic cyclic tests or long term tests. The second type of proprietary mechanical tie is a spiral. When a spiral tie is installed in the bed joint, the question becomes: Is a tie in the bed joint reliable enough to prevent the veneer units from falling? Even if the tie anchors the bed joint mortar, will the mortar-to-veneer bond be adequate in a large event?

Wall-Diaphragm Anchorage

Although the majority of retrofitted URM buildings in the Northridge Earthquake met the intended performance objectives, there were a number of disconcerting examples, particularly in Santa Monica, where buildings with closely spaced throughbolted wall-diaphragm anchors did not prevent the wall from pulling away from the diaphragm. Slip was observed along the bed joints beneath the plate washer in some cases where the wall had not yet completely pulled away; in other cases, the wall had completely failed leaving the anchor holding nothing but air. Although many of these anchors were on thin two-wythe walls, no comprehensive investigation has been done to determine if these anchors were properly designed to current retrofit provisions, what the mortar shear strength was, and if they were correctly installed. More broadly, there has been very little dynamic testing of anchors which accounts for connection flexibility and dynamic loading; testing to date, though extensive, has typically been with static tests. The basic questions which are not completely resolved at this time include:

- What level of reliability is offered by correctly installed anchors (both throughbolts and epoxy anchors) meeting current provisions?
- Are connection flexibility concerns an issue?
- For enhanced performance, how should anchorage be addressed: reducing the demand/capacity ratio by increased loads, reduced spacing, etc. or should detailing limitations also be required?

Out-of-Plane Wall Bracing

Vertical Bracing

Full-scale dynamic testing has been performed on URM walls subjected to out-of-plane loading to determine h/t acceptability provisions. No testing has been done to confirm that the wall bracing required by current bracing is adequate. In fact, the history of how the current provisions has evolved from early requirements is interesting. Originally, Division 68 (the precursor to Division 88) required that vertical bracing be twice the gross stiffness of the URM wall. This was believed to be very conservative (Kariotis, no date), and Division 88 was modified to include two vertical bracing approaches: a stiffer brace with a deflection limitation of 1/10 the wall thickness, and a flexible brace which was designed, once the wall cracked at midspan, "to provide a predictable force to cause the center of the wall to track with the ground and diaphragm dynamic displacements" (Kariotis, no date). Division 88 (1985) specified that this flexible brace must deflect between 1/4 and 1/3 of the wall thickness at allowable force demands. When the Division 88 provisions were incorporated in the UCBC (1991) and then into FEMA 178 (1992), the flexible brace option was eliminated. Some of the basic questions which remain are:

- Does testing support the stiffness requirements for the stiffer brace (or the flexible brace) when performance per the current provisions is desired?
- How should the requirement be altered to obtain enhanced performance: stiffer brace, strain compatibility check, strain limits?
- What demand should be used on the brace-to-wall anchor?

Intermediate Wall Bracing

In addition to vertical bracing with strongbacks, an alternative means of reducing the unbraced height and lowering the h/t ratio to within allowable levels is to provide "kickers" down from the diaphragm framing to an intermediate point on the wall. In this type of brace, care must be taken to avoid having the brace serve as a knee brace for diaphragm live loads or vertical seismic accelerations. With a flexible floor and stiff wall, the brace, for example, could actually serve to load the wall, rather than to "brace" it. Current provisions provide no quantitative stiffness criteria. The anecdotal evidence of poor performance in the Northridge Earthquake of some walls braced with kickers led Los Angeles Task Force members to consider potential code changes. Consensus was not reached, in part because of a lack of data (LATF, 1994). Without adequate testing and greater empirical data gathering, resolution of this issue is unlikely.

Wood Diaphragm Capacities

Current retrofit provisions provide two different methodologies for determining the acceptability of the diaphragm: in the General Procedure, a traditional check of strength is made; in the Special Procedure, the diaphragm displacement is limited. In both approaches, values are provided for a number of different existing conditions. There are several common existing conditions and retrofit possibilities, however, for which no capacity values are provided. These include:

- Original testing (ABK, 1981b) was done with supporting joists at 24" o.c. How much reduction should be provided for wider support spacing?
- What values should be assigned to two layers of diagonal sheathing?
- What is the effect of a plaster ceiling below a diaphragm? Since these stiff, brittle elements are likely to crack at displacements below the diaphragm deflection limit, how should existing provisions be altered to provide increased reliability against ceiling cracking?
- Plywood retrofit overlays on top of existing sheathing are limited in current provisions to a low capacity unblocked overlay and higher capacity blocked overlay. Since installing blocking can be quite disruptive and costly, it would be worthwhile to test alternative retrofits without blocking to see if higher capacities can be justified. Two alternatives, for example, are 1) stapling the plywood at very close spacing to simulate the conditions provided by finished flooring over straight sheathing, and 2) adding 1-1/8" tongue and groove plywood over existing straight sheathing.

3.4 General Issues Which Are Not Addressed in These Guidelines But Which Are Worth Additional Study

Low Seismicity Zones

Many geographical areas are assigned to lower seismicity zones because local seismicity, though potentially capable of producing large magnitude events, does so at a very long return period. Thus, under current probabilistic seismic zonation methodologies, the return period can dominate the magnitude, and a low zonation can result. When a large event eventually strikes, the structure may not have sufficient resistance. The Systematic Rehabilitation Method in FEMA 273 (1996) addresses this issue explicitly, but additional work is needed to provide a simple collapse prevention check for the Simplified Rehabilitation Method.

Near Fault Ground Motions

The original research on which the current retrofit provisions are based (ABK, 1984) utilized a suite of ground motions which do not include (because, in part, they were not available) near field effects. Thus, it is unclear to what extent large velocity pulses and higher than typical UBC Zone 4 ground motions will have on buildings retrofitted to current provisions. All building types face similar problems, and code writers are struggling with the best way to implement these effects, but with URM buildings, it could be argued that, with their inherently lower levels of ductility, they are more sensitive than many other buildings to near-field effects. In addition, some modes of URM wall behavior, such as in-plane and out-of-plane rocking, would appear to be sensitive to the dominant period of input motion.

Multistory In-plane Wall Provisions

Current retrofit provisions view individual stories separately; the effects of overturning on the wall are not addressed. It would be desirable to validate this approach by modeling or testing, particularly when enhancement performance is desired.

Spandrels

Current retrofit provisions require investigating several behavioral modes for piers. Spandrels failures are ignored, in part because it has not been possible to develop consensus on the issue and in part because spandrel failure is viewed as a much lower threat to Life Safety. Observations from recent earthquakes have found examples of spandrel cracking; in the Loma Prieta Earthquake, a review of 1888 unretrofitted URM buildings in San Francisco found 179 with "X" cracking in spandrels and 179 with vertical cracks at spandrel edges, as compared with 204 piers with "X" cracks and 199 piers with horizontal cracks and the top or bottom (Lizundia, et al., 1991). When enhanced performance is desired, additional research is needed to understand spandrel behavior, determine acceptability requirements and develop possible retrofit approaches.

In-plane Strength/Stiffness Incompatibility

Current retrofit provisions place relatively few limitations on combining vertical lateral-force resisting elements with different strength and stiffnesses on a single wall line. There is also not a consistent standard of practice in the industry. Some engineers will add a new element such as a braced frame or a concrete shear wall and design it to take 100% of the load tributary to the wall line. No check is made to determine if the load attracted to other elements exceeds allowable levels (such as the $3/8R_w$ UBC deformation compatibility check). Other engineers will design each element for its attracted load (as required by current retrofit provisions), but generally the ductility differences between different materials are not explicitly considered. The FEMA 273 (1996) Systematic Rehabilitation Nonlinear Static and Nonlinear Dynamic Procedures explicitly account for these differences because they are displacement-based methods, but additional guideline development is needed to develop Linear Static and Linear Dynamic and simplified procedures for addressing ductility differences.

Weak Mortar

Anecdotal evidence from LATF (1994) indicates that many of the damaged retrofitted buildings in the Northridge Earthquake had low mortar strength. As summarized by EERI (1996):

> *Both in-plane and out-of-plane failures of walls and parapet failures have been attributed to weak mortar....It has been suggested that the mortar strength had been inaccurately reported prior to retrofitting.*

In their final report, the LATF decided not to take any action over this issue, but since a thorough investigation of the buildings was not done to confirm the reported strength, the actual strength, and whether the design and construction met the intent of the provisions, it is not possible to completely confirm whether current provisions are inadequate or whether mortar testing requirements simply need to be better enforced. For enhanced performance, the same question remains.

3.5 Recommendations for Future Collection of Empirical Performance Data for Retrofitted URM Buildings

Empirical performance data has long been the primary way in which seismic performance provisions are revised. URM buildings are no exception. In fact, this study, together with several previous studies, represent attempts to go beyond anecdotal or case study approaches to studies of URM performance and to accumulate a substantial database of information so that more objective statistical comparisons and studies can be performed. Appendix A summarizes performance data collected for retrofitted URM buildings in the Northridge, Loma Prieta and Whittier Earthquakes. Based on analysis of the data, together with experience collecting analyzing unretrofitted URM building data from the Loma Prieta Earthquake (Lizundia, et al., 1991 and 1993), collection of useful statistical data is difficult.

In a theoretically perfect world, one could make an observation such as many buildings with diagonally braced walls were damaged, form a hypothesis that vertical braces will perform better than diagonal braces, compare the performance of a set of buildings with each bracing type with all other variables being equal, reach a conclusion that there is or is not a statistically significant difference in performance, and make a recommendation on potential changes. Of course, such a scenario can only be approximated with any empirical data collection effort. As the summary in Appendix A indicates, there are many reasons why using ATC-20 (1989a) damage information (which is primarily for establishing post-earthquake occupancy safety requirements) generally does not provide the detailed information on element-specific damage needed to assess the adequacy of current retrofit provisions. Given the difficulties and costs of comprehensive data collection, future collection efforts will need revised methods; recommendations include:

- <u>General Survey To Obtain Statistical Information</u>: Take advantage of the increasingly common post-earthquake volunteer safety evaluation process but supplement it with an additional form which covers more specific questions. A sample supplementary form was used in San Francisco following the Loma Prieta Earthquake (Lizundia, et al., 1991). It is essential that assessment personnel be properly trained in how to fill out the form, so that consistent and relevant statements are made. The more representative the scope of the assessment, the more reliable the conclusions will be. For example, safety evaluations tend to concentrate resources in the most heavily damaged areas; thus, conclusions will tend to be skewed by buildings which performed worse than the average. Follow-up investigations will be needed to verify missing information, assessment errors and data entry errors.

- <u>Case Study Investigations of a Limited Building Sample</u>: Given the scope of the building stock in many communities, the limits of general rapid surveys, the perishable nature of most damage data, and the cost of non-volunteer assessments, a sample of buildings should be selected for more thorough case study investigations. This is not a new idea. A very thorough set of case studies were performed following the San Fernando Earthquake (Murphy, 1973). Following the Northridge Earthquake, a number of case study projects, such as SAC (1995b), SAC (1995c), and SSC (1996), have been initiated for a variety of building types. The most relevant example for URM buildings is the Los Angeles URM Task Force work which, though never published, forms the basis of the final report recommendations (LATF, 1994). In the LATF study, 33 buildings were analyzed in the field by two- and three-member teams. Such a field review is not enough. In addition, it is necessary to review the original retrofit drawings, calculations, and testing reports and to perform supplementary push testing to help provide a more rigorous answer to the question: if design and construction are consistent with current retrofit provisions, will the stated performance goals generally be met or exceeded and with what level of reliability?

References

ABK (Agbabian, Barnes, Kariotis Joint Venture), 1981a, *Methodology for Mitigation of Seismic Hazards in Existing Unreinforced Masonry Buildings: Categorization of Buildings*, ABK-TR-01, Agbabian Associates, El Segundo, CA, December.

ABK (Agbabian, Barnes, Kariotis Joint Venture), 1981b, *Methodology for Mitigation of Seismic Hazards in Existing Unreinforced Masonry Buildings: Diaphragm Testing,* ABK-TR-03, Agbabian Associates, El Segundo, CA, December.

ABK (Agbabian, Barnes, Kariotis Joint Venture), 1981c, *Methodology for Mitigation of Seismic Hazards in Existing Unreinforced Masonry Buildings: Wall Testing, Out-of-Plane*, ABK-TR-04, Agbabian Associates, El Segundo, CA, August.

ABK (Agbabian, Barnes, Kariotis Joint Venture), 1984, *Methodology for Mitigation of Seismic Hazards in Existing Buildings: The Methodology*, ABK-TR-08, Agbabian Associates, El Segundo, CA, January.

ABK (Agbabian, Barnes, Kariotis Joint Venture), 1986, *Guidelines for the Evaluation of Historic Unreinforced Masonry Buildings in Earthquake Hazard Zones*, ABK Joint Venture, El Segundo, CA.

Amrhein, J.E., 1983, *Reinforced Masonry Engineering Handbook, Clay and Concrete Masonry*, Fourth Ed., Masonry Institute of America, Los Angeles, CA.

ATC-13 (Applied Technology Council), 1985, *Earthquake Damage Evaluation Data for California*, Applied Technology Council Report ATC-13, Redwood City, CA.

ATC-14 (Applied Technology Council), 1987, *Evaluating the Seismic Resistance of Existing Buildings*, Applied Technology Council Report ATC-14, Redwood City, CA.

ATC-20 (Applied Technology Council), 1989, *Procedures for Post-Earthquake Safety Evaluation of Existing Buildings*, Applied Technology Council Report ATC-20, Redwood City, CA.

ATC-20-2, 1995, *Addendum to the ATC-20 Postearthquake Building Safety Evaluation Procedures*, Applied Technology Council Report ATC-20-2, Redwood City, CA.

ATC-31 (Applied Technology Council), 1992, *Evaluation of the Performance of Seismically Retrofitted Buildings*, Applied Technology Council Report ATC-31, Redwood City, CA.

Belle, J., J.R. Hoke and S.A. Kliment, 1991, *Traditional Details for Buildings Restoration, Renovation, and Rehabilitation*, John Wiley & Sons, Inc.

Collins, George R., *The Transfer of Thin Masonry Vaulting from Spain to America.*

Deppe, K., 1988, "The Whittier Narrows, California Earthquake of October 1, 1987--Evaluation of Strengthened and Unstrengthened Unreinforced Masonry in Los Angeles City," *Earthquake Spectra*, Volume 4, Number 2, Earthquake Engineering Research Institute, Oakland, CA, May.

Dewey, et al., 1995, *Intensity Distribution and Isoseismal Maps for the Northridge, California, Earthquake of January 17, 1994*, USGS Open-File Report 95-92.

Division 88, 1985, *Division 88: Earthquake Hazard Reduction in Unreinforced Masonry Buildings*, City of Los Angeles Building Code, Los Angeles, CA.

Dutton, C.E., 1887, "The Charleston Earthquake of August 31, 1886," *Ninth Annual Report of the United States Geological Survey*, Government Printing Office.

EERI (Earthquake Engineering Research Center), 1994a, *Expected Seismic Performance of Buildings*, EERI Ad Hoc Committee on Seismic Performance, EERI Publication Number SP-10, Oakland, CA, February.

EERI (Earthquake Engineering Research Center), 1994b, *Northridge Earthquake January 17, 1994, Preliminary Reconnaissance Report*, Publication Number 94-01, EERI, Oakland, CA.

EERI (Earthquake Engineering Research Center), 1995, "Northridge Earthquake of January 17, 1994 Reconnaissance Report, Volume 1," *Earthquake Spectra*, Supplement C to Volume 11, Publication 95-03, EERI, Oakland, CA, April.

EERI (Earthquake Engineering Research Center), 1996, "Northridge Earthquake of January 17, 1994 Reconnaissance Report, Volume 2," *Earthquake Spectra*, Supplement C to Volume 11, Publication 95-03/2, EERI, Oakland, CA, January.

EQE/OES, 1995, *The Northridge Earthquake of January 17, 1994: Report of Data Collection and Analysis, Part A: Damage and Inventory Data*, EQE International Inc. and the Geographic Information Systems Group of the Governor's Office of Emergency Services for the Governor's Office of Emergency Services of the State of California, Sacramento, CA, May.

FEMA 156, 1988, *Typical Costs for Seismic Rehabilitation of Existing Buildings, Volume I - Summary*, Englekirk and Hart Consulting Engineers for FEMA, Washington, D.C., September.

FEMA 156, 1994, *Typical Costs for Seismic Rehabilitation of Existing Buildings, Volume I - Summary*, Second Edition, Hart Consultant Group, Inc for FEMA, Washington, D.C., December.

FEMA 157, 1988, *Typical Costs for Seismic Rehabilitation of Existing Buildings, Volume II - Supporting Documentation*, Englekirk and Hart Consulting Engineers for FEMA, Washington, D.C., September.

FEMA 157, 1995, *Typical Costs for Seismic Rehabilitation of Existing Buildings, Volume II - Supporting Documentation*, Second Edition, Hart Consultant Group, Inc. for FEMA, Washington, D.C., September.

FEMA 178, 1992, NEHRP *Handbook for the Seismic Evaluation of Existing Buildings*, Developed by Building Seismic Safety Council (BSSC) for the Federal Emergency Management Agency, BSSC, Washington, D.C.

FEMA 273, 1996, NEHRP *Guidelines for the Seismic Rehabilitation of Buildings*, prepared by Applied Technology Council (ATC-33 Project) for the Building Seismic Safety Council and funded by the Federal Emergency Management Agency, Washington, D.C., September, Ballot Version.

FEMA 274, 1996, NEHRP *Commentary on the Guidelines for the Seismic Rehabilitation of Buildings*, prepared by Applied Technology Council (ATC-33 Project) for the Building Seismic Safety Council and funded by the Federal Emergency Management Agency, Washington, D.C., September, Ballot Version.

Fuller, M.L., 1912, "The New Madrid Earthquake," Department of the Interior, USGS Bulletin 494, Washington, D.C., reprinted by Center for Earthquake Studies, Southeast Missouri State University, Cape Girardeau, MO, November, 1989.

Hart, G.C., J. Kariotis, J. and J.L. Noland, 1988, "Masonry Building Performance Survey: The Whittier Narrows Earthquake of October 1, 1987," *Earthquake Spectra*, Volume 4, Number 1, Earthquake Engineering Research Institute, Oakland, CA, February.

Hauksson, et al., 1988, *The Whittier Narrows Earthquake in the Los Angeles Metropolitan Area, California*, Science.

Hawkins, Neil and Padraic Burke, 1985, *Seismic Hazards in Unreinforced Masonry Buildings in Small Towns in the Pacific Northwest: Findings on the Structural Conditions of Unreinforced Masonry Buildings in Seven Study Towns*, National Science Foundation, Washington, D.C., April 15.

Kariotis, J., 1984, "Survey of Unreinforced Masonry Buildings," *Coalinga, California Earthquake of May 2, 1983 Reconnaissance Report*, EERI Report No. 84-03, EERI, Oakland, CA, January.

Kariotis, John, no date, "Bracing of Unreinforced Masonry Walls, Out-of-Plane," in *Earthquake Hazard Mitigation of Unreinforced Masonry Building Built Prior to 1934*, Seminar Notes, Structural Engineers Association of Southern California, Whittier, CA.

Kariotis, John and Robert D. Ewing, 1979, "Seismic Hazard Mitigation in Unreinforced Masonry: Methodology Overview," *Proceedings, 48th Annual Structural Engineers Association of California Convention*, October 4, 5, 6, 1979, Coronado, CA.

Kidder, F.E., 1916, *The Architects' and Builders' Pocketbook, A Handbook for Architects, Structural Engineers, Builders, and Draughtsmen,* Sixteenth Edition, Rewritten, John Wiley & Sons, New York, NY.

Kingsley, G.R., 1995, "Evaluation and Retrofit of Unreinforced Masonry Buildings," *Proceedings of Third National Concrete and Masonry Engineering Conference*, San Francisco, CA.

Kolbitsch, Andreas, 1989, *Altbauer Konstruktionen*, Springer-Verlag, Vienna.

LABDS (Los Angeles Department of Building and Safety), 1995, "Research Report: RR 25029: General Approval - Revision - CIA-Gel Anchors for Use in Unreinforced Solid Brick Walls."

LATF (Los Angeles Task Force), 1994, "Findings and Recommendations of the Unreinforced Masonry Building Subcommittee of the City of Los Angeles Department of Building and Safety and the Structural Engineers Association of Southern California: Task Force on Evaluating Damage from the Northridge Earthquake," City of Los Angeles Department of Building and Safety, Final Report, August 12.

Lavicka, W.L., 1980, *Masonry, Carpentry, Joinery, The Art of Architecture, Engineering and Construction in 1899*, selections from the from International Library of Technology, Scranton International Textbook Company beginning in 1899, Chicago Review Press, Chicago.

Lizundia, Bret, et al., 1991, *Damage to Unreinforced Masonry Buildings in the October 17, 1989 Loma Prieta Earthquake, San Francisco*, CA, report prepared for the National Science Foundation and the California Seismic Safety Commission, Rutherford & Chekene Consulting Engineers, San Francisco, CA.

Lizundia, Bret, et al., 1993, *Analysis of Unreinforced Masonry Building Damage Patterns in the Loma Prieta Earthquake and Improvement of Loss Estimation Methodologies: Technical Report to the USGS*, Rutherford & Chekene Consulting Engineers, San Francisco, CA.

MSJC (Masonry Standards Joint Committee), 1995, *Building Code Requirements for Masonry Structures*, ACI 530-95/ASCE 5-95/TMS 402-95, American Concrete Institute, Detroit, MI, American Society of Civil Engineers, New York, NY, and The Masonry Society, Boulder, CO.

Murphy, L.M., editor, 1973, *San Fernando, California, Earthquake of February 9, 1971: Effects on Buildings Structures, Volume I*, NOAA, Washington, D.C.

O'Connor, Ed, and Robert Reitherman, 1984, "An Interview with Edward M. O'Connor: Retroactive Earthquake Regulations in Long Beach," *Building Standards*, September-October.

Reitherman, Robert, et al., 1984, "Performance of Unreinforced Masonry Buildings," *Coalinga, California Earthquake of May 2, 1983 Reconnaissance Report*, EERI Report No. 84-03, January.

Reitherman, Robert, et al., 1984, *Seismic Damage to Unreinforced Masonry Buildings*, Scientific Services, Inc. for the National Science Foundation.

Rutherford & Chekene, 1965, *Report on Bolt Pullout Tests in Brick Walls: The Cannery*, Rutherford & Chekene Consulting Engineers, San Francisco, CA.

Rutherford & Chekene, 1990, *Seismic Retrofitting Alternatives for San Francisco's Unreinforced Masonry Buildings: Estimates of Construction Cost and Seismic Damage for the San Francisco Department of City Planning*, Rutherford & Chekene Consulting Engineers, San Francisco, CA.

Rutherford & Chekene, 1993, "Engineering Analysis," Socioeconomic and Engineering *Study of Seismic Retrofitting Alternatives for Oakland's Unreinforced Masonry Buildings*, consultant team led by Recht Hausrath & Associates, Oakland, California, March.

Rutherford & Chekene, 1995, "Geotechnical Investigation, Manufacturing Unit Number 1 (MU-1), Lehi, Utah Campus, Micron Technology, Inc.," Rutherford & Chekene Consulting Engineers, San Francisco, CA, August 23.

SAC (SAC Joint Venture Partnership), 1995a, *Technical Report: Characterization of Ground Motions During the Northridge Earthquake of January 17, 1994*, Report No. SAC-95-03, Sacramento, CA, December.

SAC (SAC Joint Venture Partnership), 1995b, *Technical Report: Analytical and Field Investigations of Buildings Affected by the Northridge Earthquake of January 17, 1994*, Report No. SAC-95-04, Parts 1 and 2, Sacramento, CA, December.

SAC (SAC Joint Venture Partnership), 1995c, *Technical Report: Case Studies of Steel Moment Frame Building Performance in the Northridge Earthquake of January 17, 1994*, Report No. SAC-95-07, Sacramento, CA, December.

SSC (Seismic Safety Commission), 1979, *The Field Act and California Schools*, SSC Report No. 79-02, Seismic Safety Commission, State of California, Sacramento, CA.

SSC (Seismic Safety Commission), 1987, *Guidebook to Identify and Mitigate Seismic Hazards*, SSC Report No. 87-03, Seismic Safety Commission, State of California, Sacramento, CA.

SSC (Seismic Safety Commission), 1994, "Background Report B18: Performance of Retrofitted Buildings," *A Compendium of Background Reports on the Northridge Earthquake (January 17, 1994) for Executive Order W-78-94*, SSC Report No. 94-08, Seismic Safety Commission, State of California, Sacramento, CA.

SSC (Seismic Safety Commission), 1995, *Turning Loss into Gain*, Report to Governor Pete Wilson in Response to Governor's Executive Order W-78-94, SSC Report No. 95-01, Seismic Safety Commission, State of California, Sacramento, CA.

SSC (Seismic Safety Commission), 1996, *1994 Northridge Earthquake Buildings Case Studies Project, Proposition 122: Product 3.2*, SSC Report No. 94-06, Seismic Safety Commission, State of California, Sacramento, CA.

Schmidt, Ben, 1994, "Report of Field Observation of Unreinforced Masonry Buildings in Areas Most Affected by the 1/17/94 Northridge Earthquake," January 29.

Stover, C.W., and J.L. Coffman, 1993, *Seismicity of the United States*, 1568-1989 (Revised), U.S. Geological Survey Professional Paper 1527.

UCBC, 1988, *1988 Uniform Code for Building Conservation,* Appendix Chapter .1, "Seismic Strengthening Provisions for Unreinforced Masonry Bearing Wall Buildings," International Conference of Building Officials, Whittier, CA.

UCBC, 1991, *1991 Uniform Code for Building Conservation,* Appendix Chapter 1, "Seismic Strengthening Provisions for Unreinforced Masonry Bearing Wall Buildings," International Conference of Building Officials, Whittier, CA.

UCBC, 1994, *1994 Uniform Code for Building Conservation,* Appendix Chapter 1, "Seismic Strengthening Provisions for Unreinforced Masonry Bearing Wall Buildings," International Conference of Building Officials, Whittier, CA.

Whitman, Robert, 1973, *Damage Probability Matrices for Prototype Buildings,* MIT Department of Civil Engineering, Seismic Design Decision Analysis; Report No. 8

Wiggins, J.H., J. Breall, and R. Reitherman, 1994, *Life Safety and Economic and Liability Risks Associated with Strengthened, Unreinforced Masonry Buildings*, J.H. Wiggins Company, Redondo Beach, CA, September 1.

Wiggins, J.H. and D.F. Moran, 1971, *Earthquake Safety in the City of Long Beach Based on the Concept of Balanced Risk*, J.H. Wiggins Company, Redondo Beach, CA.

Wong, K., 1987, *Seismic Strengthening of Unreinforced Masonry Buildings, A Design and Cost Guide for Architects and Engineers*, Center for Environmental Design Research, University of California, Berkeley.

Wood, Harry O., and Neumann, Frank, 1931, "Modified Mercalli Intensity Scale of 1931," *Bulletin of the Seismological Society of America*. Vol. 21, No. 4.