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APPENDICES: Development of Procedures to Enhance the Performance of Rehabilitated URM Buildings

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APPENDIX A

ANALYSIS OF EARTHQUAKE DAMAGE DATA

Preface

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 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 in projecting the performance of standard West Coast practice to moderate seismic zones.

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A.1 Summary

This appendix describes the data gathered and the analysis of data on URM bearing wall building retrofit performance data in the Northridge, Loma Prieta and Whittier Earthquakes. The more important conclusions are summarized below. The sections which follow provide a detailed discussion of the data collection process as well as the 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. A brief summary of information collected follows.

- 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 A-3 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 A-8 through A-12 show the relationship of the total inventory of Los Angeles retrofitted URM buildings, peak ground acceleration (PGA), peak ground velocity (PGV) spectral acceleration at periods (S_a) of 0.3 and 1.0 seconds, and the Modified Mercalli Intensity (MMI). These figures 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 A-12 shows, 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 0.30g, peak ground velocities below 25 cm/sec, 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) have been 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 9(c) 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 11(b). Note that at both MMI=VII and VIII, the EERI (1994a) predictions overestimate the actual damage in the Northridge event. Table A-12 shows a damage probability matrix using PGA as the ground motion parameter. As expected, damage increases as PGA increases. Other ground motion parameters generally show similar trends.

- 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% for 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. With the same assumption, the average damage ratio for retrofitted buildings was 1.0%, compared with 1.3% for unretrofitted 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.

Limitations of the Data

Based on this study and similar efforts in previous earthquakes, collecting useful damage data is difficult. There are many limitations in attempting to use information collected in the post-earthquake safety evaluation process to draw conclusions about the effectiveness of current retrofit provisions. These limitations include:

- Buildings usually are not dispersed homogeneously across the various levels of ground motion. For City of Los Angeles data from the Northridge Earthquake, for example, 93% of the buildings are in areas where the PGA is 0.30g or below. Drawing conclusions about performance expected in areas with higher ground shaking was thus compromised. Little data is available for retrofitted buildings in high intensity areas from the Loma Prieta or Whittier earthquakes, as well.

- Isolating building characteristics so that the influence of a single variable can be studied is often difficult. Residential buildings, for example, might appear to perform worse than nonresidential buildings, but what if residential buildings in the data set have some other common attribute which has a greater influence on damage and which, if eliminated or held constant, would then reveal that residential occupancy does not have a noticeable influence on damage?

- Poor quality in design and construction is often considered to be a leading cause of damage. According to the City of Los Angeles Task Force (LATF, 1994), for example, there was anecdotal evidence of low mortar strength in most of the buildings which experienced in-plane and out-of-plane failures of walls, parapet failures, and wall tie anchorage failures. Nonetheless, it has not been possible to perform statistical studies of the influence that either the quality of original construction or the quality of the retrofit design and construction on building performance because of the lack of comprehensive data collection.

- Building jurisdictions typically have limited resources and thus difficulty in justifying performing post-earthquake safety evaluations in areas where shaking or damage was believed to be lower than the average. Thus, inspections tend to be concentrated in areas with higher damage, and results are skewed. With the City of Los Angeles retrofitted building data, for example, only 13% of the buildings have inspection data.

Assumptions regarding the damage suffered by the uninspected buildings must be made to limit the bias. These assumptions introduce some uncertainty.

- It is difficult to determine from the available data the best estimate of damage immediately following the earthquake. In the City of Los Angeles, for example, the initial report was, in some cases, the most reliable because later reports were updated to reflect post-earthquake repairs. In other cases, later reports were more reliable because they were done by more experienced personnel. Figures A-6 and A-7 show the changes which resulted between the first report and the second.

- The experience and conscientiousness of the inspector clearly can dramatically influence the quality of the data. Many inspectors have limited experience with post-earthquake safety evaluations and may not correctly adhere to the criteria intended to be used in completing an assessment form. Even the best inspectors, however, can reach different conclusions when observing the same event, so consistency of reporting can be a significant issue.

- In many cases, the inspector may not have sufficient time or may be unable to enter the building so that interior damage will not be known. Similarly, without an interior inspection, it is generally difficult to determine the retrofit approach used on the building.

- Generally, the pre-earthquake history of a building is not known to the inspector, so existing cracks may be mislabeled as earthquake damage.

- Measures of damage, such as the cost of repair, require the inspector to draw a conclusion which may be influenced by later events. Will the owner decide to make a cosmetic repair? Or will the owner decide or be required to strengthen? The final cost depends on many factors over which the damage inspector has no control.

- In any large data collection effort, data entry errors will result. Obvious inconsistencies usually can be identified and eliminated after querying the data, but these records are then lost. There is, however, no way to verify data that initially appears reasonable, and potentially inaccurate data may thus remain in the dataset.

- Similarly, when "geocoding" or matching an address on a post-earthquake inspection form to the address in a geographic information system (GIS) database, there can be many addresses which will not match. In GIS parlance, a successful match is a "hit." Some inconsistencies can be eliminated and the hit rate improved, but this is a time-consuming process, and it is unlikely to ever achieve a 100% hit rate with very large databases.

A.2 Background

Unreinforced masonry buildings are unique as a construction class because of the comparatively extensive amount of data on unretrofitted building performance in past earthquakes. This is because URM construction has been used in both large cities and small towns throughout the United States for several hundred years; these buildings tend to be the most frequently damaged; the damage is typically easy to observe; and the buildings are clustered in identifiable districts, such as central business or industrial areas.

There is little uncertainty in the identification of common failure mechanisms of unreinforced masonry buildings. Several comprehensive studies have been undertaken in the last decade. Kariotis and Ewing (1979) lists seven basic failure modes inferred from observations of damaged buildings. ATC-14 (1987) includes a comprehensive literature search of earthquake damage examples for various types of construction, including unreinforced masonry buildings. Damage to URM buildings in the 1983 Coalinga, California earthquake was investigated and failure mechanisms identified in Reitherman et al. (1984). Rutherford & Chekene (1990) documents a comprehensive investigation of seismic retrofitting alternatives for San Francisco's unreinforced masonry buildings. Ultimately, these various studies identify many similar deficiencies and draw many of the same conclusions regarding failure mechanisms for URM buildings, including:

- Inadequate or nonexistent parapet bracing.
- Non-parapet falling hazards caused by inadequate cornice, veneer or other appendage anchorage.
- Inadequate connections between exterior brick bearing walls and floor/roof diaphragms for seismic forces.
- Wall failure in bending between diaphragms.
- Excessive diaphragm deflections.
- Corner damage.
- In-plane wall failure.
- Roof and/or floor collapse
- Soft-story or other configuration-induced failure.

Although there has been limited seismic retrofitting of existing URM buildings in California for many years, most of the work has been done in the last decade. Because seismic retrofitting is a relatively recent phenomenon and because it has not been undertaken consistently across the state's many communities, there have been few significant earthquakes to affect retrofitted buildings.

For this study, 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 is relatively small, whereas in the City of Los Angeles alone, data was collected on 5682 retrofitted URM buildings exposed to the Northridge event. The 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 in projecting performance of standard West Coast practice to moderate seismic zones. The final objective was to identify elements which should be further strengthened to obtain better performance for specified shaking intensities.

A.3 1994 Northridge Earthquake Data

The 1994 Northridge Earthquake is one of the most thoroughly studied natural disasters in the nation's history. Not only was it the first earthquake to be centered in a highly populated urban area with dense infrastructure, but ground motions were significant enough to cause building damage over a wide area. Hundreds of well-distributed strong motion recording instruments monitored the event, yielding the most complete ground motion information to date. Further, agencies such as the Federal Emergency Management Agency (FEMA), California Office of Emergency Services (OES), and the City of Los Angeles Department of Building and Safety (LADBS) were (as a consequence of recent earthquakes and other natural disasters) experienced in post-earthquake emergency evaluations and data collection efforts. The massive amount of data collected by the various disciplines may now be manipulated and correlated with increasing ease because of recent technological developments, including improvements in computer hardware and software and the development of Geographic Information Systems (GIS) for geographical analysis of data.

The Northridge Earthquake provided a major test of the performance of retrofitted unreinforced masonry buildings, and once again pointed out the vulnerability of URM buildings that have not been strengthened. Because of the passage of an ordinance in 1981 that required seismic strengthening (Division 88 of the City of Los Angeles Building Code), most URM buildings in the City of Los Angeles have been retrofitted. This ordinance requires that all URM parapets be removed or braced, that mortar joints be tested and walls strengthened if they do not meet minimum values for shear strength, that walls meet certain clear height/thickness requirements or be braced for out-of-plane forces, that walls be anchored to floor and roof diaphragms, and that diaphragms meet certain strength and stiffness requirements. Other cities in the Los Angeles area have ordinances similar to Division 88, though a few have not yet required strengthening. The vast majority of retrofitted buildings in the area affected by the earthquake are located in Los Angeles, and, as a result, data collection efforts for this study were directed there.

Retrofitted URM Inventory Database Development

Overview of Data

This section describes the development and limitations of the retrofitted URM building inventory used for the evaluation of the Northridge data. Each building in the evaluation inventory was tracked by LADBS in three databases: the *First Inspection Database*, the *Reinspection Database*, and the *Inspection Log Database*. In addition, we obtained the City's database of the buildings subject to Division 88 requirements (the total URM inventory). This *Division 88 Database* contains both building characteristics and retrofit compliance history. Geographical components within these databases were used to correlate individual building damage data with ground motion data. Each of these databases as well as the ground motion data are described in this section. A subset of this information has been completed for interested researchers; see Section A.6 for details.

The final building inventory used for our evaluation is termed the *Reduced First Inspection (RFI) Database*. It contains building location, structural attributes, detailed damage descriptions, and ground motion data for all of the URM bearing wall buildings which were inspected after the Northridge Earthquake. Basic characteristics of the URM building stock which are useful in this study of seismic vulnerability are location, height, length, width, and whether a basement exists. The damage descriptions which are of interest are, in part, wall cracking, diaphragm damage, roof/floor connection failure, chimney and parapet damage, and column, pilaster, or corbel failure. Street addresses were used to assign a latitude and longitude for each building. Finally, ground shaking parameters MMI, peak ground acceleration, peak ground velocity, and spectral acceleration (computed at periods of 0.3 seconds and 1.0 seconds) were correlated with the building's latitude and longitude and added to each building record.

First Inspection, Reduced First Inspection and Division 88 Databases

Immediately after the earthquake, LADBS personnel and OES volunteers formed survey teams to perform post-earthquake emergency evaluations. The purpose of the rapid inspection was to quickly (often with limited manpower) inspect and evaluate buildings to identify buildings which pose a danger to public safety. Many of the individuals providing this service are civil engineers or building inspectors. Some individuals have post-earthquake building safety evaluation experience, and others have participated in special training programs. These damage inspectors are familiar with building construction so that structural damage or any unusual situations, such as falling hazards, can be readily recognized. The expertise of structural engineers is preferred, but is not essential for this phase of the task. The LADBS coordinated building damage assessments using the ATC-20 (1989) post-earthquake safety evaluation method.

The purpose of the ATC-20 evaluation is

> *...to provide guidelines and procedures for the postearthquake safety evaluation of building types commonly found in the United States. Development of these*

procedures is intended to promote uniformity in the rating of building damage such that two individuals examining the same building will arrive at essentially the same conclusion regarding a structure's safety and its posting category (e.g., limited entry, unsafe) [ATC-20, 1989].

Since its publication, this manual has been used in many jurisdictions to classify building damage from earthquakes. The three damage classifications—Inspected, Limited Entry, and Unsafe—are defined in Table A-1. These classifications are entered on the ATC-20 Detailed Assessment Safety Evaluation Form shown in Figure A-1. Note that a recent addendum (ATC-20-2, 1995) has slightly modified this form. The LADBS developed their Rapid Screening Inspection Form (Figure A-2) from the 1989 version of the Detailed ATC-20 form. Building damage inspectors completed the LADBS Rapid Screening Inspection form and posted placards for each building inspected. The city-wide distribution of green-, yellow-, and red-tagged retrofitted URM buildings is shown in Figure A-3. At the time of these cursory inspections, the inspectors also provided rough estimates of repair costs, both as a cost estimate and as a percent of total replacement cost. A thematic map which shows the city-wide distribution of damage as a percentage of total replacement cost is show in Figure A-4. The City of Los Angeles reportedly accounts for over 75% of all inspected buildings (EQE/OES, 1995). The city boundaries include many locales such as Northridge, Reseda, Sherman Oaks, Van Nuys, Sylmar, Chatsworth, North Hollywood, and Encino.

Table A-1: ATC-20 Rating System (ATC, 1985)

Posting	Color	Description
Inspected	Green	No apparent hazard found, although repairs may be required. Original lateral load capacity not significantly decreased. No restriction on use or occupancy.
Limited Entry	Yellow	Dangerous condition believed to be present. Entry by owner permitted only for emergency purposes and only at own risk. No usage on continuous basis. Entry by public not permitted. Possible major aftershock hazard.
Unsafe	Red	Extreme hazard, may collapse. Imminent danger of collapse from an aftershock. Unsafe for occupancy or entry, except by authorities.

An initial database containing the first inspection building damage data for all types of building construction was created by the LADBS. The information contained in this database, termed the "EQ1-94 'Not for Sale' Database," has a restricted audience and is intended for damage assessment purposes only. At the time our study was initiated, the overall building damage database contained 126,286 buildings. The data fields are shown in Table A-2.

The EQ1-94 fields in Table A-2 constitute only a few of the fields available for each inspected building. We reviewed the information available in the EQ1-94 Database and concluded that without the remainder of the fields, which contain, in part, element-

Block__________ Parcel No.__________

ATC-20 Detailed Evaluation Safety Assessment Form

BUILDING DESCRIPTION:
Name: ______________________________
____
Address: ______________________________
____

No. of Stories: ________
Basement: Yes ☐ No ☐ Unknown ☐
Approximate Age: __________Years
Approximate Area:__________Square feet

Structural System:
Wood Frame ☐ Unreinforced Masonry ☐
Reinforced Masonry ☐ Tilt-up ☐
Concrete Frame ☐ Concrete Shear Wall ☐
Steel Frame ☐ Other ______________________
____

Primary Occupancy:
Dwelling ☐ Other Residential ☐ Commercial ☐
Office ☐ Industrial ☐ Public Assembly ☐
School ☐ Government ☐ Emer. Serv. ☐
Historic ☐ Other ______________________

OVERALL RATING: *(Check One)*
INSPECTED (Green) ☐
LIMITED ENTRY (Yellow) ☐
UNSAFE (Red) ☐

INSPECTOR:
Inspector ID ______________________
Affiliation ______________________

INSPECTION DATE:
Mo/day/year ______________________
Time ______________________ am pm

Instructions: Complete building evaluation and checklist on next page and then summarize results below.

Posting:	*Existing*	*Recommended*
None	☐	
Inspected *(Green)*	☐	☐
Limited Entry *(Yellow)*	☐	☐
Unsafe *(Red)*	☐	☐

Posted at this Assessment:
☐ Yes ☐ No
Existing posting by:
____

Recommendations:
- ☐ No further action required
- ☐ Engineering Evaluation required (circle one) Structural Geotechnical Other__________
- ☐ Barricades needed in the following areas: ______________________
____
- ☐ Other *(falling hazard removal, shoring/bracing required, etc.)*: ______________________
____

Comments *(Why posted Unsafe, etc.)*: ______________________
____
____
____
____

Figure A-1: ATC-20 (1989) Detailed Evaluation Safety Assessment Form

ATC-20 Detailed Evaluation Safety Assessment Form (Continued)

Instructions: Examine the building to determine if any hazardous conditions exist. A "yes" answer in categories 1, 2, or 4 is grounds for posting building UNSAFE. If condition is suspected to be unsafe and more review is needed, check appropriate Unknown box(es) and post LIMITED ENTRY. A "yes" answer in category 3 requires posting and/or barricading to indicate AREA UNSAFE. Explain "Yes", "Unknown" findings and extent of damage under "Comments."

Condition	Hazardous Condition Exists			
	Yes	*No*	*Unknown*	*Comments*
1. *Structure Hazardous Overall*				
Collapse/partial collapse	☐	☐	☐	______
Building or story leaning	☐	☐	☐	______
Other______	☐	☐	☐	______
______	☐	☐	☐	______
2. *Hazardous Structural Elements*				
Foundations	☐	☐	☐	______
Roof/floors (vertical loads)	☐	☐	☐	______
Columns/pilasters/corbels	☐	☐	☐	______
Diaphragms/horizontal bracing	☐	☐	☐	______
Walls/vertical bracing	☐	☐	☐	______
Moment frames	☐	☐	☐	______
Precast connections	☐	☐	☐	______
Other______	☐	☐	☐	______
______	☐	☐	☐	______
3. *Nonstructural Hazards*				
Parapets/ornamentation	☐	☐	☐	______
Cladding/glazing	☐	☐	☐	______
Ceilings/light fixtures	☐	☐	☐	______
Interior walls/partitions	☐	☐	☐	______
Elevators	☐	☐	☐	______
Stairs/exits	☐	☐	☐	______
Electric/gas	☐	☐	☐	______
Other______	☐	☐	☐	______
______	☐	☐	☐	______
4. *Geotechnical Hazards*				
Slope failure/debris	☐	☐	☐	______
Ground movement, fissures	☐	☐	☐	______
Other______	☐	☐	☐	______
______	☐	☐	☐	______

SKETCH:

Figure A-1: ATC-20 (1989) Detailed Evaluation Safety Assessment Form (cont.)

A. TYPE OF DISASTER:
☐ Fire ☐ Earthquake
☐ Flood ☐ Other ______

CITY OF LOS ANGELES
DEPARTMENT OF BUILDING AND SAFETY
RAPID SCREENING INSPECTION FORM

B. BUILDING USE:
☐ Residential
☐ Commercial

C. INCLUSIVE ADDRESS. COUNCIL DISTRICT:

D. OWNER: PHONE NO.:

MANAGER: PHONE NO.:

E. No of Stories: ______ No. of Living Units: ______ Basement: ☐ YES ☐ NO ☐ UNKNOWN

TYPE CONSTRUCTION: URM I II III IV V APPROX. SIZE ______ ft. x ______ ft.

PRIMARY OCCUPANCY: (Check one, only)

						☐ 21 THEATRE
☐ 01 DWELLING	☐ 04 AMUSEMENT	☐ 07 PVT GARAGE	☐ 10 HOSPITAL	☐ 13 OFFICE	☐ 16 RET STORE	☐ 22 WAREHOUSE
☐ 02 DUPLEX	☐ 05 APARTMENT	☐ 08 PUB. GARAGE	☐ 11 HOTEL	☐ 14 PUB ADMIN	☐ 17 RESTAURANT	☐ 35 CONDO
☐ 03 AIRPORT	☐ 06 CHURCH	☐ 09 GAS STATION	☐ 12 MFG	☐ 15 PUB. UTIL	☐ 18 SCHOOL	☐ 99 OTHER

F. INSTRUCTIONS: Examine the building to determine if any hazardous conditions exist. A "YES" answer in Categories 1, 2, or 4 is grounds for posting building UNSAFE. If condition is suspected to be unsafe and more review is needed, check appropriate Unknown box(es) and post LIMITED ENTRY. A "YES" answer in Category 3 requires posting and/or barricading to indicate AREA UNSAFE. Explain "YES", "UNKNOWN" findings and extent of damage under "Comments."

EXISTING HAZARDOUS CONDITIONS

Condition	YES	NO	UNK	Condition	YES	NO	UNK
1. *Structure Hazardous Overall*	☐	☐	☐	3. *Nonstructural Hazards*	☐	☐	☐
Collapse/partial collapse	☐	☐	☐	Parapets/ornamentation	☐	☐	☐
Building or story leaning	☐	☐	☐	Cladding/glazing	☐	☐	☐
Other ______	☐	☐	☐	Ceiling/light fixtures	☐	☐	☐
______				Interior Walls/partitions	☐	☐	☐
				Elevators	☐	☐	☐
2. *Hazardous Structural Elements*	☐	☐	☐	Stairs/Exits	☐	☐	☐
Foundations	☐	☐	☐	Electric/Gas	☐	☐	☐
Roof/Floors (vertical loads)	☐	☐	☐	Chimney	☐	☐	☐
Columns/pilasters/corbels	☐	☐	☐	Other ______	☐	☐	☐
Diaphragms/horizontal bracing	☐	☐	☐				
Walls/vertical bracing	☐	☐	☐	4. *Geotechnical Hazards*	☐	☐	☐
Moments Frames	☐	☐	☐	Slope failure/debris	☐	☐	☐
Precast connections	☐	☐	☐	Ground Movement, fissures	☐	☐	☐
Other ______	☐	☐	☐	Other ______	☐	☐	☐

COMMENTS: ______

G. Vacate Bldg.? ☐ YES ☐ NO Partially Vacate Bldg.? ☐ YES ☐ NO No. of Living Units Vacated ______

EST. DAMAGE: ______ % EST. DAMAGE: $ ______ PERMIT REQUIRED? ☐ YES ☐ NO

H. OVERALL RATING:

	Existing	Recommended
INSPECTED (Green)	☐	☐
____ Exterior Only		
____ Exterior and Interior		
LIMITED ENTRY (yellow)	☐	☐
UNSAFE (Red)	☐	☐
____ Building		
____ Area (See Section I-3)		

I. RECOMMENDATIONS: (Circle Number / Fill in data)
1. No Further Action required.
2. Detailed Evaluation required.
   ______ Structural ______ Geotechnical
3. Barricades needed in the following areas
   ______
4. Disconnect utilities:
   ____ Electric ____ Gas ____ Water

J. INSPECTOR
Name/I.D.: ______
Phone: ______

K. INSPECTED
Date: ______
Time: ______ a.m./p.m.

08—G-4 (Rev 6/90) DISTRIBUTION Original to file for Detailed Inspection (if required) / Duplicate to Data Entry / Triplicate to Dept Files

Figure A-2: City of Los Angeles Department of Building and Safety Rapid Screening Inspection Form

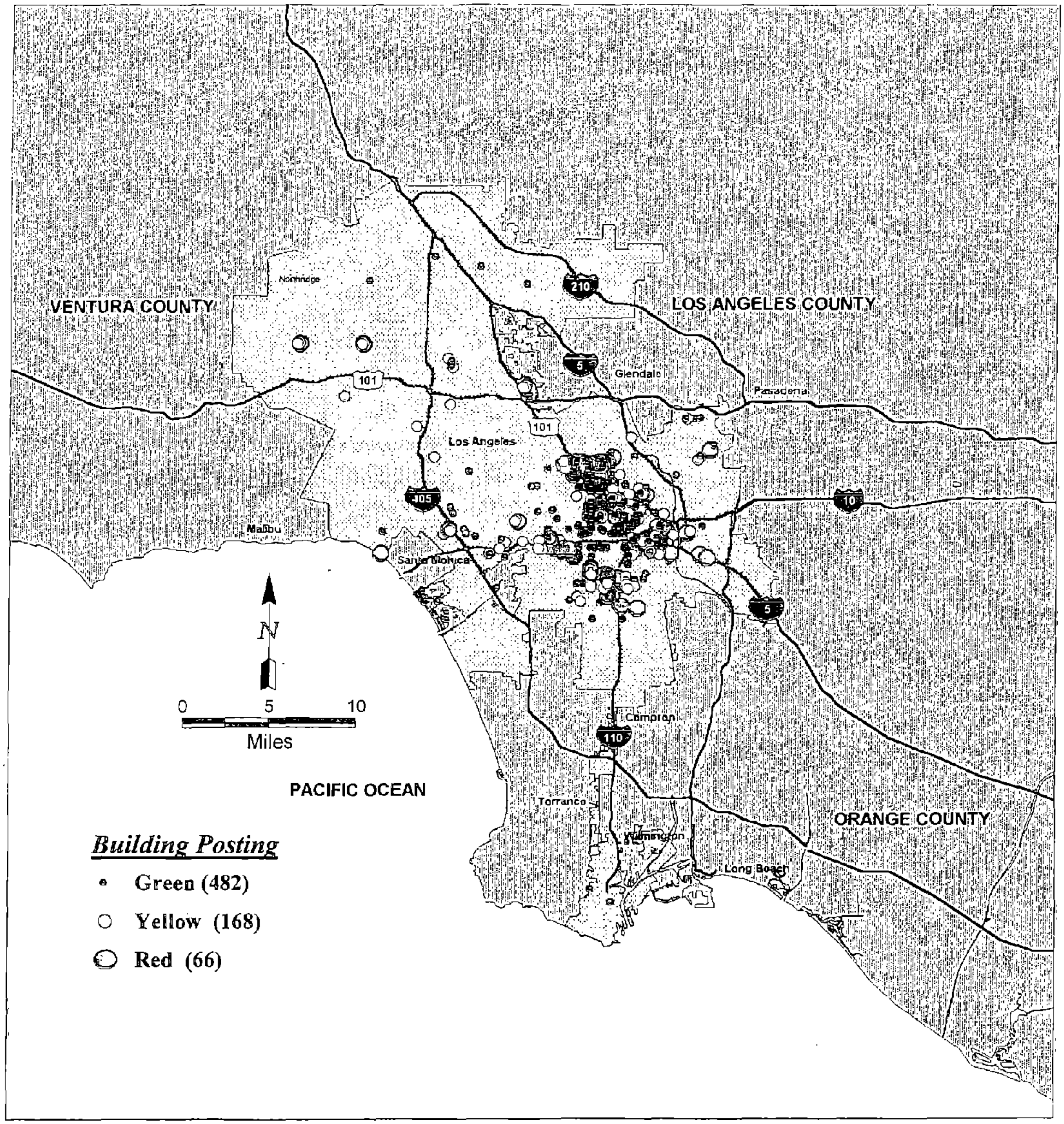


Figure A-3: Building and Safety Damage Assessment--Building Inspection Tagging

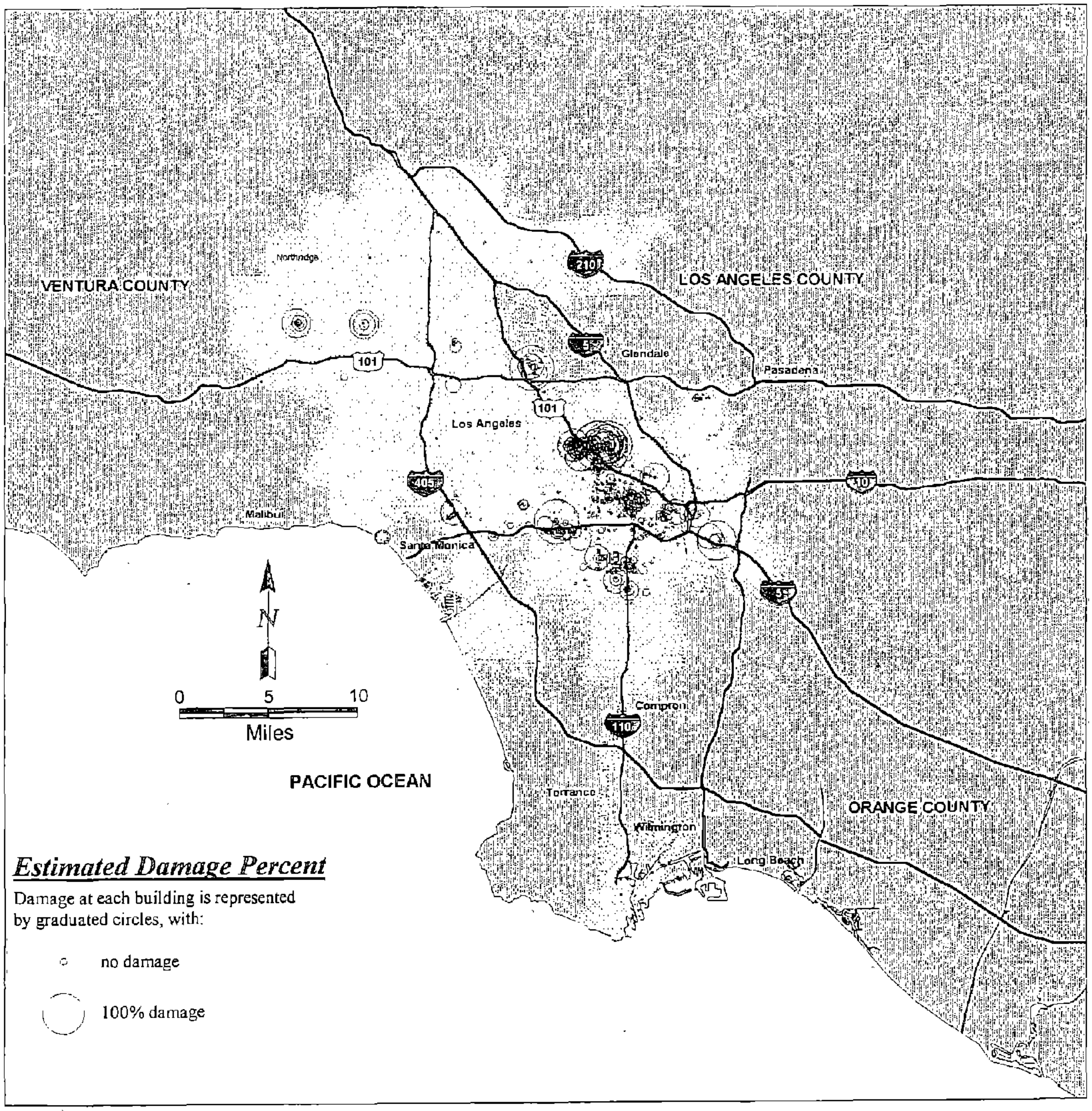


Figure A-4: City-Wide Distribution of Damage as a Percentage of Total Cost

specific damage descriptions and complete ATC-20 evaluation data, the usefulness of our investigations would be severely limited. Per our request, the remaining information for URM buildings was provided by the Resource Management Bureau of the LADBS. The resulting list has 1240 entries and is termed the *First Inspection Database.* Fields in this database which relate to this study are shown in Table A-4.

Considerable effort and resources were required in the preparation of the final study inventory of URM buildings from the First Inspection Database. A variety of errors and limitations in the databases were discerned in the preparation of the study database, through cross referencing/merging and address verification, but time and budgetary constraints prevented the elimination of all inadequacies. The final database, referred to as the *Reduced First Inspection (RFI) Database*, was prepared as follows:

1. The First Inspection Database list of 1240 was not limited to URM bearing wall buildings; it included all buildings which fall under the jurisdiction of the Earthquake Safety Division of the LADBS. Therefore, this First Inspection Database which was to be the main database in our study initially contained URM buildings, tilt-ups, and brick infills. Fortunately, as part of a separate verification effort by the city, the entire First Inspection Database of 1240 buildings was sent to the Earthquake Safety Division for verification and reinspection by the City's more experienced building inspectors. The Earthquake Safety Division evaluated approximately two-thirds of the buildings before reorganization of the Division precluded further investigations. The Earthquake Safety Division inspections helped identify many of the buildings which were not URM bearing wall construction, and we removed them from an interim version of the RFI Database. For the purposes of this study, the remaining third was assumed to have been properly identified as URM. The resulting interim version of the RFI Database had 1181 buildings.

2. The *First Inspection Database* does not indicate if the building complies with Division 88 retrofit requirements. A master list of the original 8242 URM buildings subject to Division 88 was obtained from the Earthquake Safety Division. This *Division 88 Database* contains building characteristics and a chronological log of the Division 88 permitting/compliance process. The fields of the list are shown in Table A-3.

3. Of the original 8242 buildings subject to Division 88, 1606 have been demolished and 178 are exempt, leaving 6458 buildings. Exemptions were given for residential buildings (occupancy less than five) and for those built after 1933 (allowed during WWII because of the shortage of steel). At the time of this study, 5682 of the remaining buildings were retrofitted to Division 88 requirements, 61 retrofitted with tension ties only, and 703 were noncomplying (unretrofitted) buildings. Twelve entries were duplicate addresses.

4. The interim RFI Database (1181 entries) was compared with the Division 88 Database to identify which buildings were found on both lists. The preliminary sort yielded 801 buildings matching the Division 88 list and 380 which did not.

5. The 380 buildings which did not match the Division 88 list were examined individually to determine if a cause could be identified—typically an error in address or the building was included in an address range given in the Division 88 list. The final match list contained 852 buildings. The remaining 329 were removed from the study database because they could not be verified as URM bearing wall construction nor identified as retrofitted.

6. Dimensional data (plan length and width) in the interim RFI Database was correlated with the Division 88 Database. Obvious errors in the RFI dataset were corrected and missing data was added.

7. The database was geocoded to assign a latitude and longitude to each building site using the MapInfo Professional geographical database analysis program.

8. Ground shaking contours representing MMI, peak ground acceleration, peak ground velocity and spectral acceleration at periods of 0.3 seconds and 1.0 seconds were overlain with the geocoded building database. The corresponding ground motion value was added to the data for each building record.

9. As anticipated, the extensive number of different inspectors required for this effort, their varied experience, and the circumstances under which the data was obtained resulted in some inconsistent entries. Consequently, the RFI Database was reviewed to determine if damage reported in the structural damage description fields was similarly reported in the "Hazardous Structural Elements" fields. We identified inconsistencies in the records for approximately 10 percent of the inventory. However, it was impossible to verify these inconsistencies as actual errors. Nevertheless, this effort allowed a generally favorable impression of the accuracy and completeness of the data to be made.

10. The RFI Database is described in detail in Section A.6. It is presented as an Excel workbook with three worksheets: *RFIR*—the reduced first inspection retrofitted database with 751 retrofitted buildings, *RFIU*—the reduced first inspection unretrofitted database with 93 unretrofitted buildings, and *TENSION TIE*—a database with 8 tension-tie-only buildings.

11. Because they had not been inspected, the damage to the balance of the retrofitted buildings is not known, but it is likely that damage is minor or nonexistent in most cases. Except for the limited parameter studies described later, these uninspected retrofitted buildings were assumed to be undamaged.

Reinspection Database

Following the initial inspection, a "blue form," also known as the "Disaster Re-Inspection Form G4GRI" was issued for each of the original 1240 buildings in the First Inspection Database (Figure A-5). The majority of these inspection forms were completed by

Table A-2: Fields in the EQ1-94 Database

Field	Item	Field	Item
1	Record Number	14	Estimated Number of Units Vacated
2	Street Number	15	Estimated Percent of Damage
3	Street Fraction-Begin	16	Estimated Repair Cost
4	Street Range	17	Vacate Building
5	Street Fraction-End	18	Permit Required
6	Street Direction Street Name	19	Building Posting
7	Street Type	20	Number of Stories
8	Unit Number	21	Construction Type (Per City of Los Angeles Building Code)
9	Council District	22	Earthquake Division
10	Damage Type (EQ, FIRE, FLOOD, etc.)	23	Approximate Building Size (Width x Length)
11	Building Use (Residential, Commercial, Mixed	24	Zip Code
12	Occupancy	25	Year Built
13	Estimated Total Dwelling Units		

Table A-3: Fields in the Division 88 Database

Field	Item	Field	Item
1	Division 88 List Number	13	Alt 1 Permit Compliance Date
2	Street Address	14	Alt 1 Complete Compliance Date
3	City	15	Anc 1 Permit Compliance Date
4	Class Code	16	Anc 1 Complete Compliance Date
5	Date Original Notice to Comply Sent	17	Anc 2 Permit Compliance Date
6	Affidavit Recording Date	18	Anc 2 Complete Compliance Date
7	Council District	19	Date Building Found Vacant
8	Building Number	20	Year Constructed
9	Number of Buildings	21	Essential Facility
10	Number of Stories	22	Historic Building
11	Building Width at Street	23	Date of Demolition
12	Building Depth	24	Exempt Building Data

inspectors in the Earthquake Safety Division, most with experience in URM construction. Database fields which are in any way related to this study are shown in Table A-5. This *Reinspection Database* mostly contains information pertinent to the evaluation, inspection, and permitting process. Detailed damage descriptions were reviewed.

Table A-4: Fields in the Reduced First Inspection Retrofitted Database

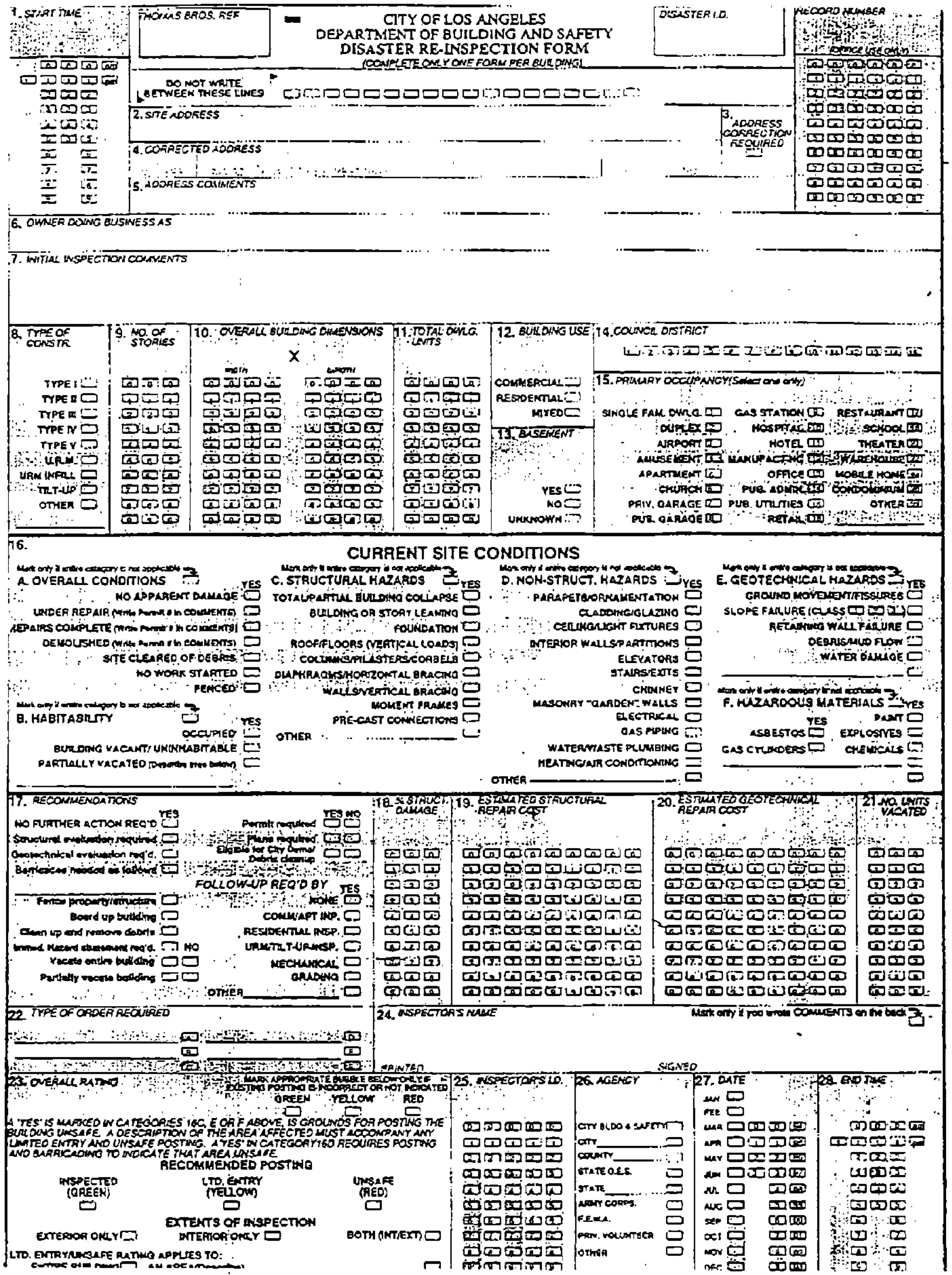
Field	Item	Field	Item
1	Record Number	33	Parapets/Ornamentation
2	Street Number	34	Cladding/Glazing
3	Street Fraction-Begin	35	Ceiling/Light Fixtures
4	Street Range	36	Interior Walls/Partitions
5	Street Fraction-End	37	Elevators
6	Street Direction Street Name	38	Stairs/Exits
7	Street Type	39	Chimney
8	Unit Number	40	Masonry Walls
9	Council District	41	Electrical
10	Disaster Type (EQ, FIRE, FLOOD)	42	Gas
11	Corner Address	43	Piping
12	Owner	44	Water/Waste Plumbing
13	Manager	45	HVAC
14	Number of Stories	46	Other
15	Number of Units	47	Geotechnical Hazards
16	Basement	48	Ground Movement
17	Construction Type (Per City of Los Angeles Building Code)	49	Slope Failure
18	Length	50	Class of Slide
19	Width	51	Retaining Wall Failure
20	Structure Hazardous Overall	52	Debris/Mud Flow
21	Collapsed	53	Other
22	Leaning	54	Damage Description
23	Hazardous Structural Elements	55	Damage Types
24	Collapse/Partial Collapse	56	Vacate Total
25	Building/Story Leaning	57	Vacate Partial
26	Foundations	58	Estimated Number of Units Vacated
27	Roof/Floor Vertical Loads	59	Estimated Percent of Damage
28	Column/Pilaster/Corbels	60	Estimated Repair Cost
29	Diaphragms/Horizontal Bracing	61	Permit Required
30	Walls Vertical Bracing	62	Recommendations (No Further Action, Structural, Geotechnical, Barricades, Disconnect Utilities)
31	Other	63	Inspector ID
32	Hazardous Nonstructural Elements		

Table A-5: Fields in the Reinspection Database

Field	Item	Field	Item
1	Record Number	30	Interior Walls/Partitions
2	Address Change	31	Elevators
3	Street Fraction-Begin	32	Stairs/Exits
4	Street Range	33	Chimney
5	Street Fraction-End	34	Masonry Walls
6	Street Direction Street Name	35	Electrical
7	Street Type	36	Gas
8	Unit Number	37	Piping
9	Council District	38	Water/Waste Plumbing
10	Address Comments	39	HVAC
11	Building Use (Residential, Commercial, Mixed	40	Other
12	Occupancy	41	Geotechnical Hazards
13	Estimated Total Dwelling Units	42	Ground Movement
14	Construction Type (Per City of Los Angeles Building Code)	43	Slope Failure
15	Overall Condition (No Damage, Under Repair, Repairs Complete, Demolished, Site Cleared of Debris, No Work Started, Fenced)	44	Class of Slide
16	Habitability (Occupied, Vacant, Uninhabitable)	45	Retaining Wall Failure
17	Structural Hazards	46	Debris/Mud Flow
18	Collapse/Partial Collapse	47	Water Damage
19	Building/Story Leaning	48	Hazardous Materials
20	Foundations	49	Paint
21	Roof/Floor Vertical Loads	50	Asbestos
22	Column/Pilaster/Corbels	51	Explosives
23	Diaphragms/Horizontal Bracing	52	Gas Cylinders
24	Walls Vertical Bracing	53	Chemicals
25	Other	54	Other
26	Nonstructural Hazards	55	Estimated Number of Units Vacated
27	Parapets/Ornamentation	56	Estimated Percent of Damage
28	Cladding/Glazing	57	Estimated Repair Cost
29	Ceiling/Light Fixtures	58	Description of Damage

Inspection Log Database

All inspection and repair activity for each building was also tracked by the LADBS in the *Inspection Log Database*. The fields of this database which are of interest to this study are the posting history and estimated damage history. Damage estimates made during the first inspection were based on limited observations; in some cases the interior of the structure was not accessed. Revised inspection reports, which include the revised damage estimates and reinspection dates, were filed after each reinspection. Examination of the



CITY OF LOS ANGELES
DEPARTMENT OF BUILDING AND SAFETY
DISASTER RE-INSPECTION FORM
(COMPLETE ONLY ONE FORM PER BUILDING)

1. START TIME — THOMAS BROS. REF — DISASTER I.D. — RECORD NUMBER (OFFICE USE ONLY)

DO NOT WRITE BETWEEN THESE LINES

2. SITE ADDRESS
3. ADDRESS CORRECTION REQUIRED
4. CORRECTED ADDRESS
5. ADDRESS COMMENTS
6. OWNER DOING BUSINESS AS
7. INITIAL INSPECTION COMMENTS

8. TYPE OF CONSTR.: TYPE I, TYPE II, TYPE III, TYPE IV, TYPE V, URM, URM INFILL, TILT-UP, OTHER
9. NO. OF STORIES
10. OVERALL BUILDING DIMENSIONS (WIDTH X LENGTH)
11. TOTAL DWLG. UNITS
12. BUILDING USE: COMMERCIAL, RESIDENTIAL, MIXED
13. BASEMENT: YES, NO, UNKNOWN
14. COUNCIL DISTRICT
15. PRIMARY OCCUPANCY (Select one only): SINGLE FAM. DWLG., GAS STATION, RESTAURANT, DUPLEX, HOSPITAL, SCHOOL, AIRPORT, HOTEL, THEATER, AMUSEMENT, MANUFACTURING, WAREHOUSE, APARTMENT, OFFICE, MOBILE HOME, CHURCH, PUB. ADMIN., CONDOMINIUM, PRIV. GARAGE, PUB. UTILITIES, OTHER, PUB. GARAGE, RETAIL

16. CURRENT SITE CONDITIONS

A. OVERALL CONDITIONS (YES): NO APPARENT DAMAGE; UNDER REPAIR (Write Permit # in COMMENTS); REPAIRS COMPLETE (Write Permit # in COMMENTS); DEMOLISHED (Write Permit # in COMMENTS); SITE CLEARED OF DEBRIS; NO WORK STARTED; FENCED

B. HABITABILITY (YES): OCCUPIED; BUILDING VACANT/ UNINHABITABLE; PARTIALLY VACATED (Describe area below)

C. STRUCTURAL HAZARDS (YES): TOTAL/PARTIAL BUILDING COLLAPSE; BUILDING OR STORY LEANING; FOUNDATION; ROOF/FLOORS (VERTICAL LOADS); COLUMNS/PILASTERS/CORBELS; DIAPHRAGMS/HORIZONTAL BRACING; WALLS/VERTICAL BRACING; MOMENT FRAMES; PRE-CAST CONNECTIONS; OTHER

D. NON-STRUCT. HAZARDS (YES): PARAPETS/ORNAMENTATION; CLADDING/GLAZING; CEILING/LIGHT FIXTURES; INTERIOR WALLS/PARTITIONS; ELEVATORS; STAIRS/EXITS; CHIMNEY; MASONRY "GARDEN" WALLS; ELECTRICAL; GAS PIPING; WATER/WASTE PLUMBING; HEATING/AIR CONDITIONING; OTHER

E. GEOTECHNICAL HAZARDS (YES): GROUND MOVEMENT/FISSURES; SLOPE FAILURE (CLASS); RETAINING WALL FAILURE; DEBRIS/MUD FLOW; WATER DAMAGE

F. HAZARDOUS MATERIALS (YES): PAINT; ASBESTOS; EXPLOSIVES; GAS CYLINDERS; CHEMICALS

17. RECOMMENDATIONS (YES): NO FURTHER ACTION REQ'D; Structural evaluation required; Geotechnical evaluation req'd.; Barricades needed as follows; Fence property/structure; Board up building; Clean up and remove debris; Limited Hazard abatement req'd.; Vacate entire building; Partially vacate building
(YES NO): Permit required; Plans required; Eligible for City Demo/Debris cleanup
FOLLOW-UP REQ'D BY (YES): NONE; COMM/APT INSP.; RESIDENTIAL INSP.; URM/TILT-UP INSP.; MECHANICAL; GRADING; OTHER
18. % STRUCT. DAMAGE
19. ESTIMATED STRUCTURAL REPAIR COST
20. ESTIMATED GEOTECHNICAL REPAIR COST
21. NO. UNITS VACATED
22. TYPE OF ORDER REQUIRED
24. INSPECTOR'S NAME — Mark only if you wrote COMMENTS on the back
PRINTED — SIGNED

23. OVERALL RATING — MARK APPROPRIATE BUBBLE BELOW ONLY IF EXISTING POSTING IS INCORRECT OR NOT INDICATED: GREEN, YELLOW, RED

A 'YES' IS MARKED IN CATEGORIES 16C, E OR F ABOVE, IS GROUNDS FOR POSTING THE BUILDING UNSAFE. A DESCRIPTION OF THE AREA AFFECTED MUST ACCOMPANY ANY LIMITED ENTRY AND UNSAFE POSTING. A 'YES' IN CATEGORY 16D REQUIRES POSTING AND BARRICADING TO INDICATE THAT AREA UNSAFE.

RECOMMENDED POSTING: INSPECTED (GREEN); LTD. ENTRY (YELLOW); UNSAFE (RED)
EXTENTS OF INSPECTION: EXTERIOR ONLY; INTERIOR ONLY; BOTH (INT/EXT)
LTD. ENTRY/UNSAFE RATING APPLIES TO:

25. INSPECTOR'S I.D.
26. AGENCY: CITY BLDG & SAFETY; CITY; COUNTY; STATE O.E.S.; STATE; ARMY CORPS.; F.E.M.A.; PRIV. VOLUNTEER; OTHER
27. DATE: JAN, FEB, MAR, APR, MAY, JUN, JUL, AUG, SEP, OCT, NOV, DEC
28. END TIME

Figure A-5: City of Los Angeles Department of Building and Safety Disaster Re-Inspection Form inspection chronology indicates that the elapsed time between inspections varies widely, complicating the selection of the damage estimate which most accurately represents the condition of the building immediately after the earthquake. The greater the length of elapsed time between reinspections, the more likely repairs had been initiated—lowering and thereby invalidating the original damage estimate. However, some damage reinspections, conducted within several days of the earthquake revised the estimated damage upward.

The revised damage estimate data was sorted to determine the effect of the damage revisions on the database. Of the 751 retrofitted buildings, 115 buildings had their original damage estimates revised upward and 336 downward; the remaining 300 did not change. The average percentage of damage for the 115 upward revisions increased from 6% to 19% whereas the average percentage of damage for the 336 downward revisions decreased from 11% to 2%. Since there is no consistent way to create a revised "first inspection" damage estimate which will not bias the data, the initial and re-inspection data are evaluated separately. The overall inventory is compared in Figure A-6. It is apparent from this figure that the reinspection damage estimates are biased towards lower damage. Assignment to ATC-13 (1985) damage state was made per the groupings in Table A-7. Further, the average percentage of damage decreases from 7.9% for the first inspection data to 5.3% for the reinspected data.

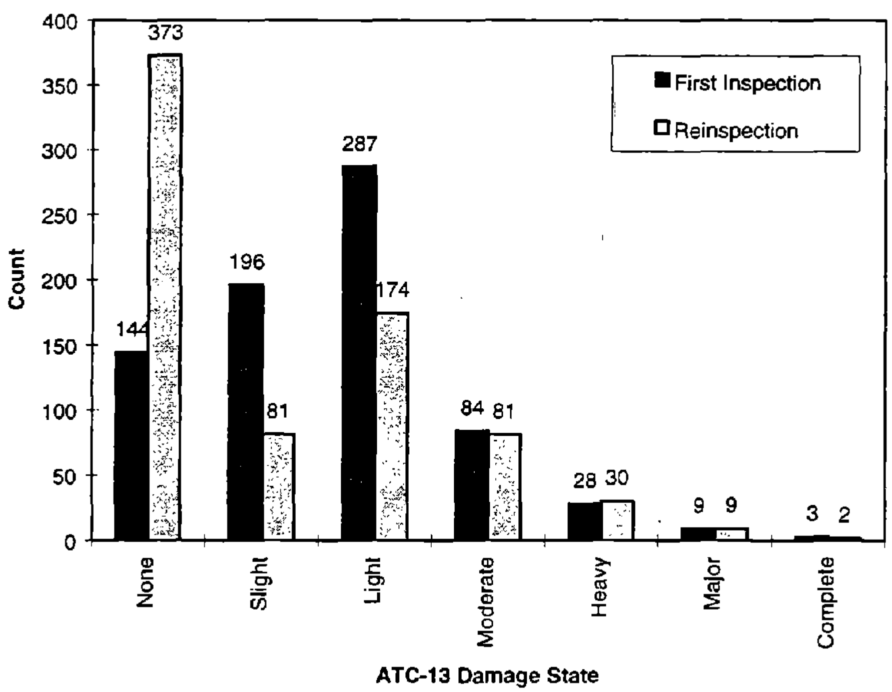


Figure A-6: Comparison of ATC-13 Damage State for First Inspection Damage Estimate and Reinspection Damage

Finally, by comparison, of the 751 buildings in the RFI Database, there were 66 red tags, 168 yellow tags, and 482 green tags. The remaining 35 did not receive a posting. In the reinspection dataset, there were 48 red tags, 137 yellow tags, and 541 green tags. The remaining 25 did not receive a posting. The first inspection posting and first reinspection posting are compared in Figure A-7(a). ATC-13 (1985) damage states are compared in Figure A-7(b).

Ground Motion Data

As described above, the primary goal of this task is 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. Four organizations collected the bulk of the ground motion data used for this task: (1) the California Division of Mines and Geology Strong Motion Instrumentation Program (CSMIP), (2) the U.S. Geological Survey National Strong Motion Program (USGS), (3) the University of Southern California Los Angeles Strong Motion Accelerograph Network (USC), and (4) the Los Angeles Department of Water and Power System Strong Motion Network (LADWP).

The epicenter of the main shock of the Northridge Earthquake was centered about one kilometer south-southwest of Northridge and 32 km west-northwest of downtown Los Angeles (the heart of the building inventory used in this task) at a focal depth of 19 km. The earthquake occurred on a south-southwest dipping thrust ramp below the San Fernando Valley, with a moment magnitude was estimated to be $M_w = 6.7$ (EERI, 1995). SAC (1995a) developed grid points which were intended to be used for smoothed contour maps of the maximum horizontal components of peak ground acceleration (PGA), peak ground velocity (PGV), and spectral acceleration (S_a). These grid points were used by Risk Management Solutions of Menlo Park, California to create the smoothed contour maps used in this study. In addition, an isoseismal map of Modified Mercalli Intensity (MMI) was developed by Dewey, et al. (1995). Each of these contour maps was converted into MapInfo format so that individual buildings could be assigned discrete values of PGA, PGV, S_a, and MMI. The relationship between each of these contour maps and our study inventory are described below.

<u>Peak Ground Acceleration</u>: The PGA contour map, overlain by the total retrofitted URM inventory is shown in Figure A-8. This map indicates that maximum horizontal accelerations exceeding 0.4g were restricted to the northwestern portion of the City of Los Angeles, nearest the epicenter. The seismic accelerations attenuate rapidly as they move southeast towards downtown Los Angeles, where accelerations of 0.15g to 0.25g are most common. The bar graph accompanying the figure shows the inventory count versus the PGA contour. Unfortunately, the building inventory was not particularly well-distributed across the contours; 93 percent of the total building inventory had a PGA of 0.30g or lower. A breakdown of the *total* retrofitted inventory (5682 Division 88 buildings) and *inspected* inventory (751 buildings) falling within four PGA bands 0.1g or less, 0.1g to 0.2g, 0.2g to 0.3g, 0.3g to 0.4g, and over 0.4g are shown in Table A-6. Little

data was available for retrofitted buildings in areas of high ground shaking; thus, drawing conclusions about performance expected in high earthquake zones was compromised. Not only were the buildings not dispersed homogeneously across the various PGA contours, but greater percentages of inspected buildings were located in the high ground shaking areas. Thus, the inspected inventory is biased in comparison to the total inventory.

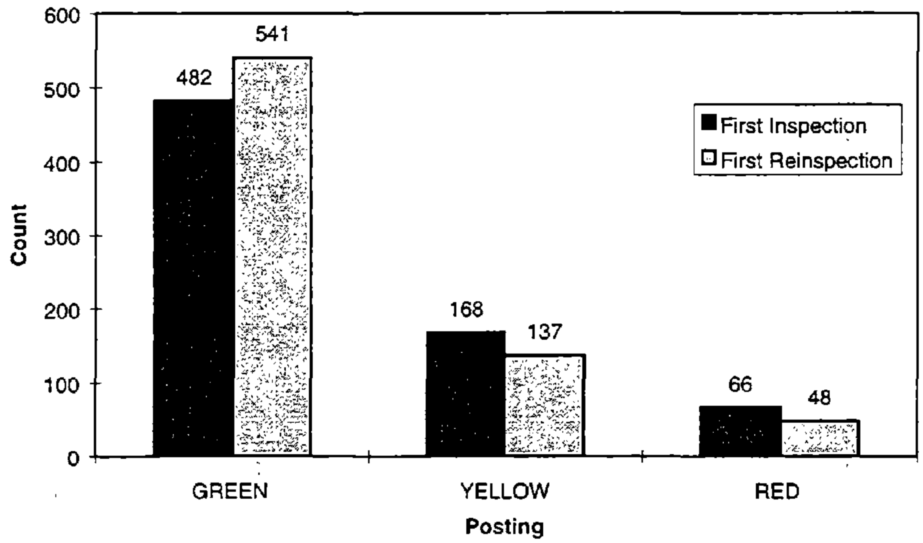
Peak Ground Velocity: The PGV contours, overlain by the total retrofitted URM inventory, are shown in Figure A-9. The geographical distribution of PGV was very similar to PGA. Figure A-9 indicates that maximum horizontal velocities exceeding 25-30 cm/sec were restricted to the northwestern portion of the City of Los Angeles, nearest the epicenter. Downtown Los Angeles was typically subjected to peak ground velocities between 10 and 20 cm/sec. The bar graph accompanying the figure shows the inventory count versus the PGV contour. Again, the building inventory is not particularly well-distributed across the contours; over 96 percent of the buildings experienced a PGV of 25 cm/sec or less.

Spectral Acceleration: Contour maps of spectral accelerations computed at periods of 0.3 seconds and 1.0 seconds were selected for this study, as these periods very roughly correspond to common periods of URM walls and diaphragms, respectively. These contours, overlain with the building inventory are shown in Figures A-10 and A-11, respectively.

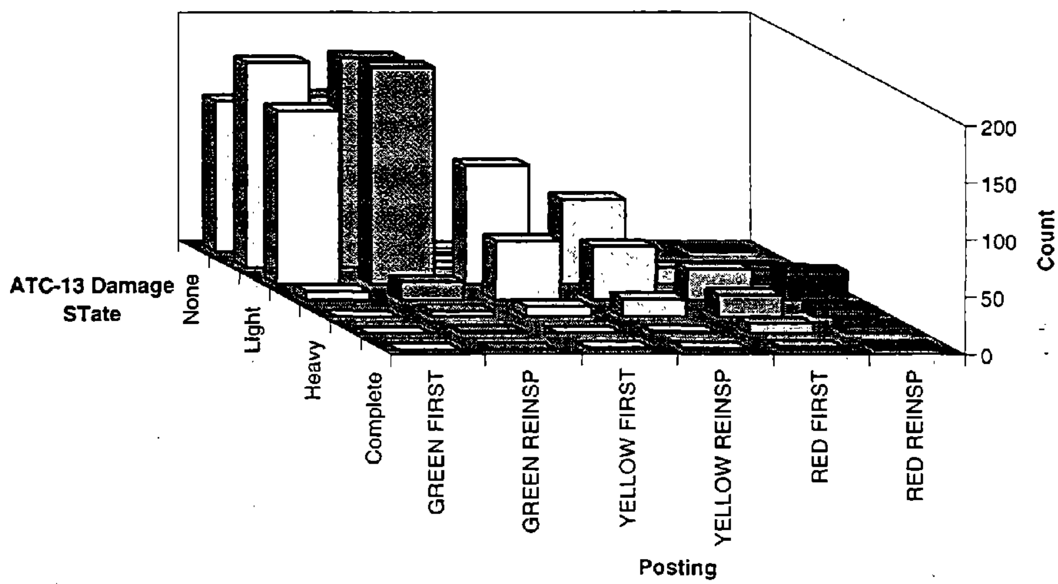
Modified Mercalli Intensity: The intensity of an earthquake at a location is a number that characterizes the severity of ground shaking at that location by considering the effects of the shaking on people, on man-made structures, and on the landscape. Intensities have for many decades been based on the Modified Mercalli Intensity Scale of 1931 (Wood and Neumann, 1931). The derivation of the scale relies heavily on direct building damage observations, particularly for intensities greater that V. Some of the criteria given in the original 1931 work which describe human reactions or effects resulting from ground failure are no longer given significant influence in the assigning of intensity values (Stover and Coffman, 1993).

A map of isoseismal lines and point observations of Modified Mercalli Intensity (MMI) was developed by Dewey, et al. (1995). A detailed explanation of the creation of this map is contained therein. MMI contours, overlain by the retrofitted URM inventory, are shown in Figure A-12. The bulk of the inventory is in the moderate intensity (VII) contour. Intensity VII is bounded by Glendale to the northeast, Torrance and Compton to the south, and extends as far west as Fillmore. Reports of IX were limited to the immediate vicinity of the epicenter and to isolated pockets in the San Fernando and Santa Clarita Valleys, Santa Monica, and south-central Los Angeles.

This MMI contour map, however, is not as useful as the other ground motion data used in this task because MMI maps are created, for the most part, from observations of damage to structures. Consequently, it can be argued that the MMI data cannot be used to



(a)



(b)

Figure A-7: Comparison of First Inspection Posting and First Reinspection Posting

Table A-6: Percent of Inventory in Each PGA Range

PGA Range	Percent of Total Inventory Falling within Range	Percent of Inspected Inventory Falling within Range
Less than 0.1g	2.3	0.4
0.1g to 0.2g	15.7	8.6
0.2g to 0.3g	74.7	78.4
0.3g to 0.4g	5.2	8.9
Over 0.4g	2.2	3.7

correlate ground shaking with general and/or element-specific damage because any conclusions would be self-fulfilling—MMI is determined by the existence of certain types of damage (URM buildings in particular) so observations of specific damage types are, necessarily, expected in the given MMI. Further, in some locations, the positions of isoseismals were determined from the distribution of red- and yellow-tagged buildings (Dewey, et al., 1995). Nevertheless, these contours are useful for comparing the actual damage to the damage predictions made in the literature.

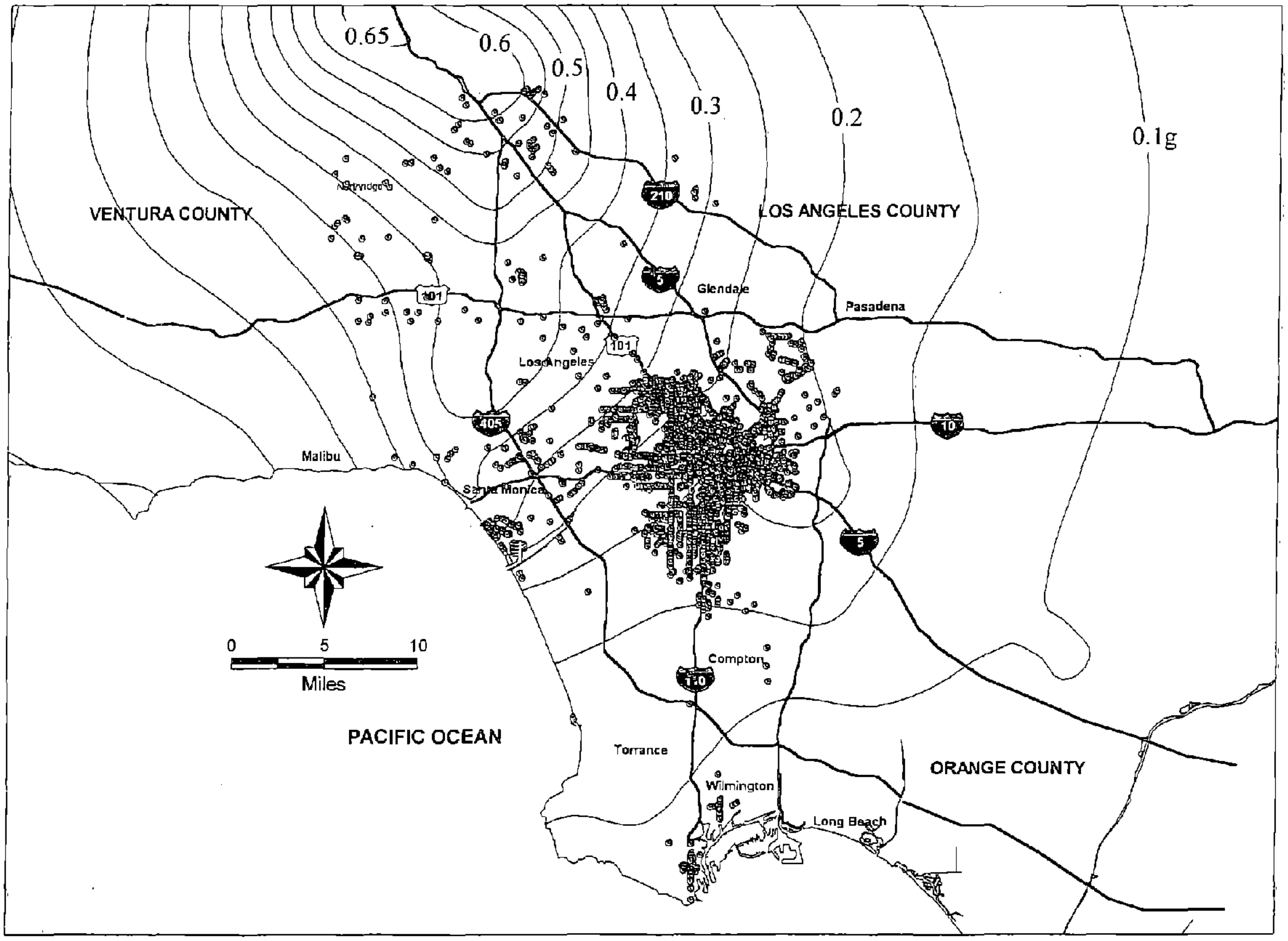
Comparison of Ground Motion Parameters: Comparing and contrasting the spectral acceleration contours, $S_a(0.3)$ and $S_a(1.0)$ with each other and with the MMI, PGA, and PGV contours leads to the following observations:

- The contours of PGA, PGV, and $S_a(0.3)$ are geographically similar (Figure A-13).
- The central contour (bull's-eye) of the $S_a(0.3)$ map aligns with the PGA and PGV central contours.
- The central contour of the $S_a(1.0)$ map is shifted southeast from the center contours of the others.
- $S_a(1.0)$ and PGV values attenuate more quickly than either PGA or $S_a(0.3)$.
- The spatial distribution of the total retrofitted inventory over the ground motion contours is similar; most of the inventory falls within a narrow band of low/moderate ground motion intensity.
- The central contours of the PGA, PGV, $S_a(0.3)$ and $S_a(1.0)$ maps do not intersect the MMI IX contour.

Analysis of the Damage

Character and Limitations of the Dataset

The study inventory is, fortuitously, somewhat homogeneous—general characteristics, attributes, and vintages are similar. Most of the URM buildings in the study database, for



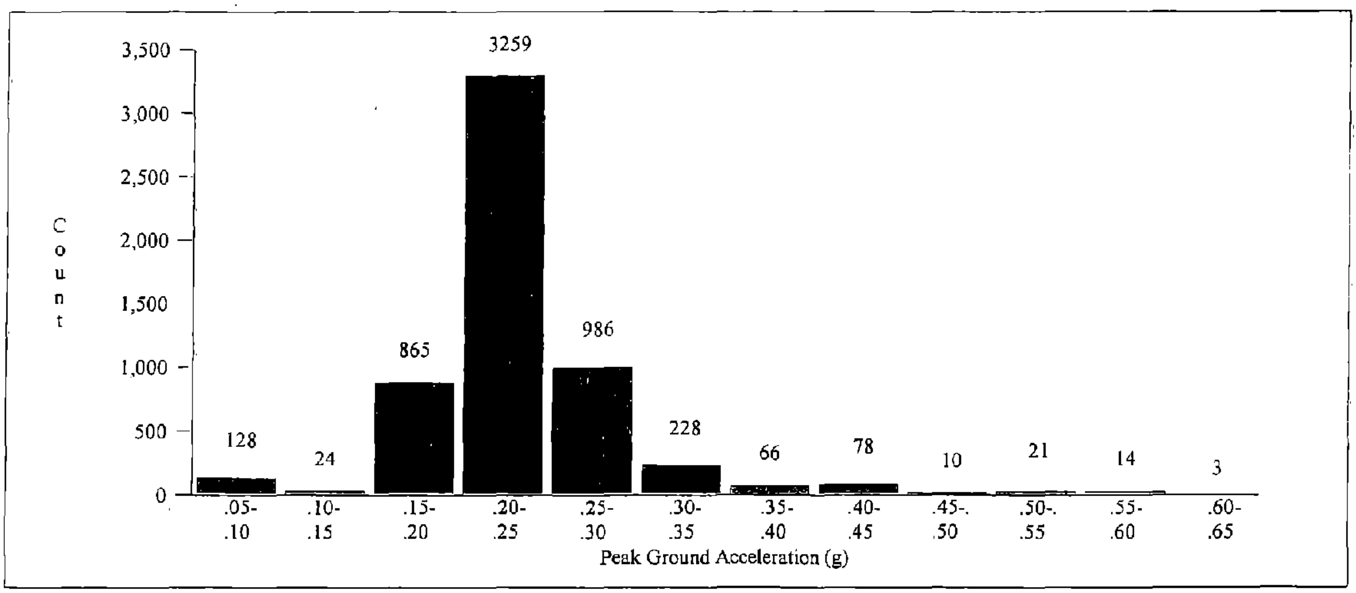
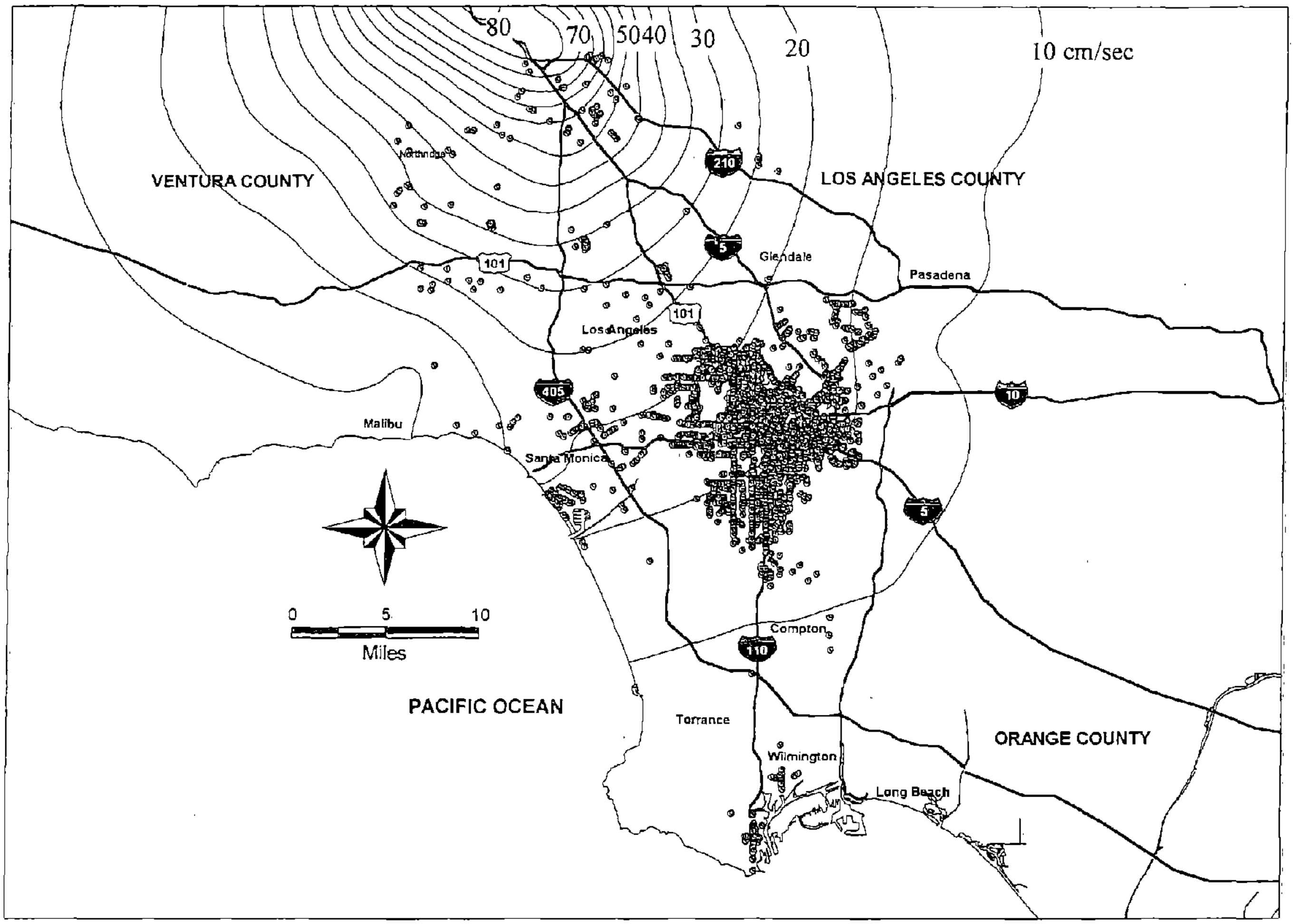


Figure A-8: Total Retrofitted Inventory vs. Peak Ground Acceleration (g)



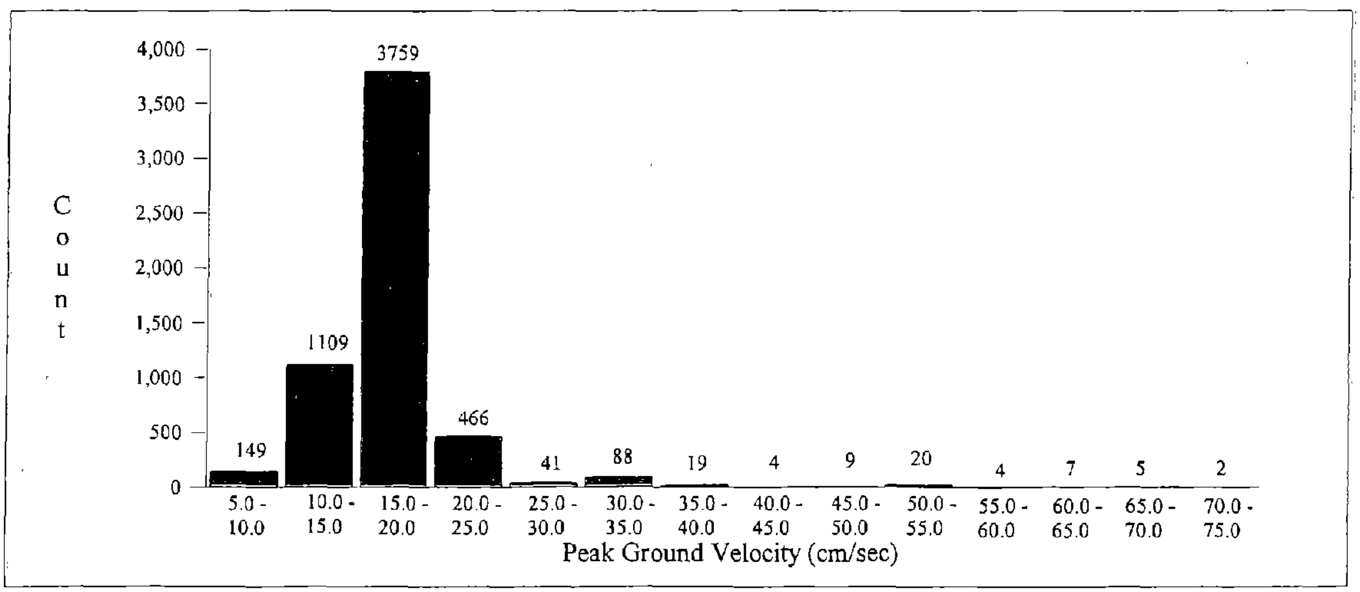
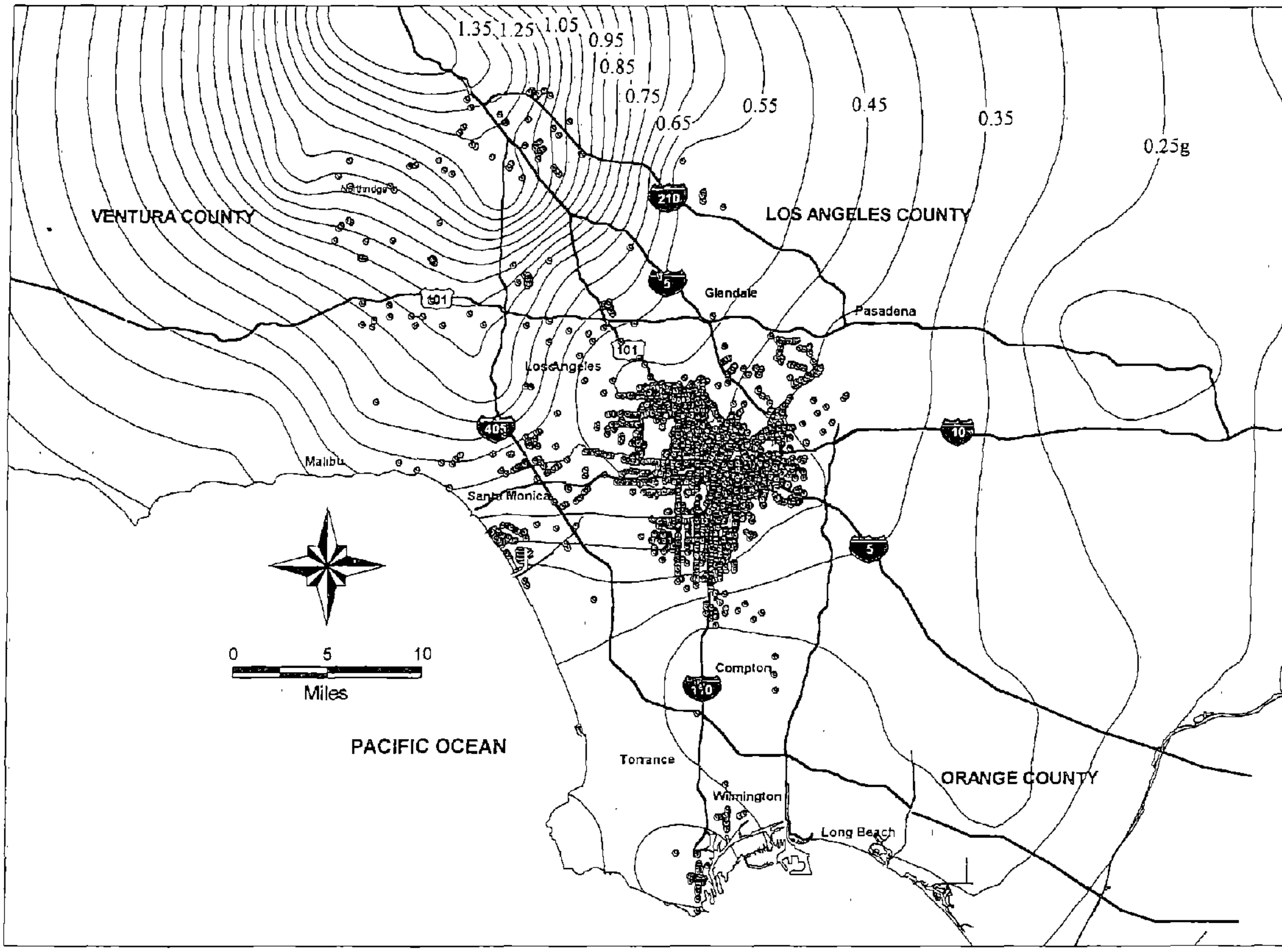


Figure A-9: Total Retrofitted Inventory vs. Peak Ground Velocity (cm/sec)



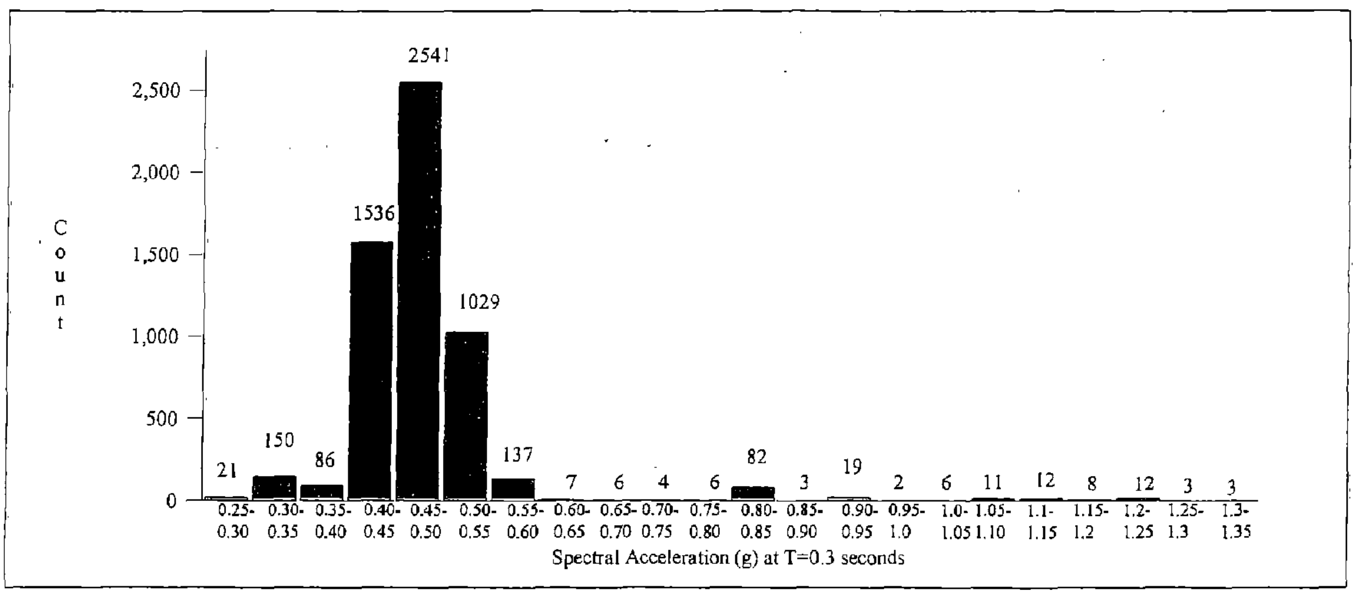
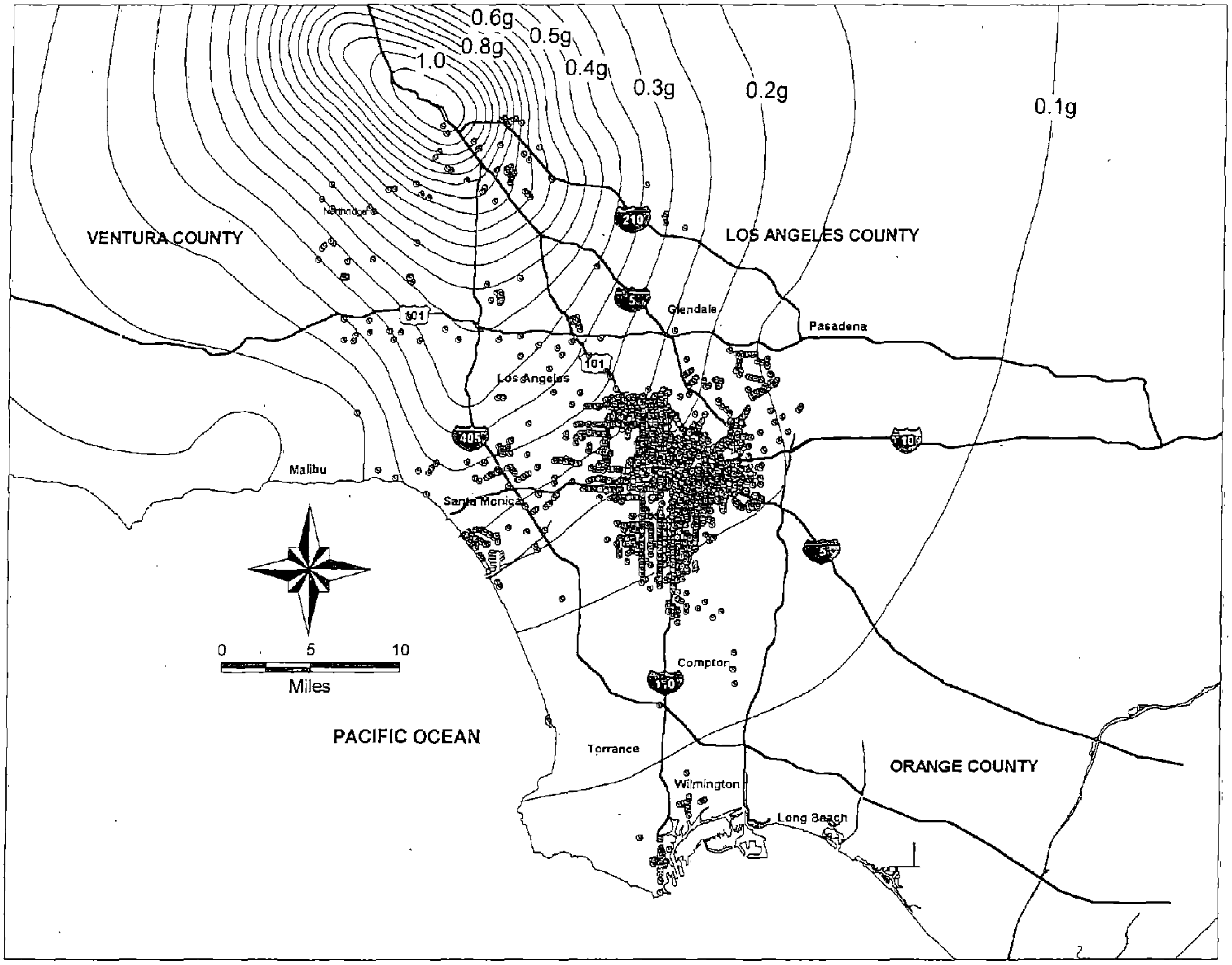


Figure A-10: Total Retrofitted Inventory vs. Spectral Acceleration (g) taken at T=0.3 seconds



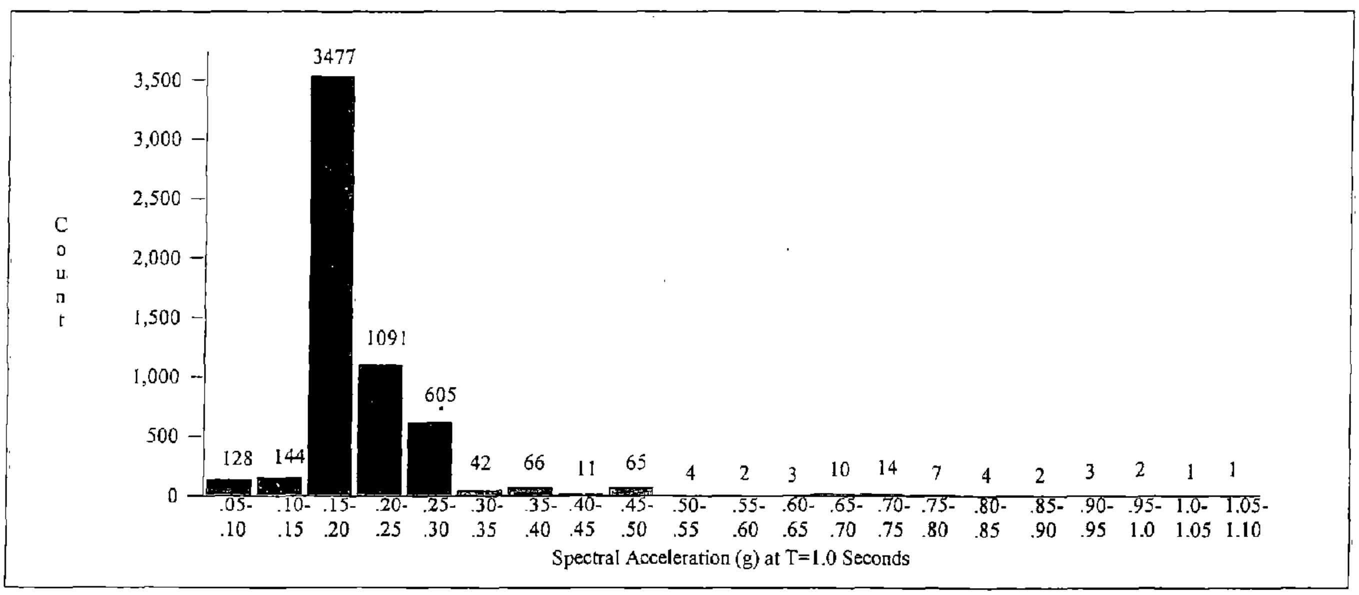
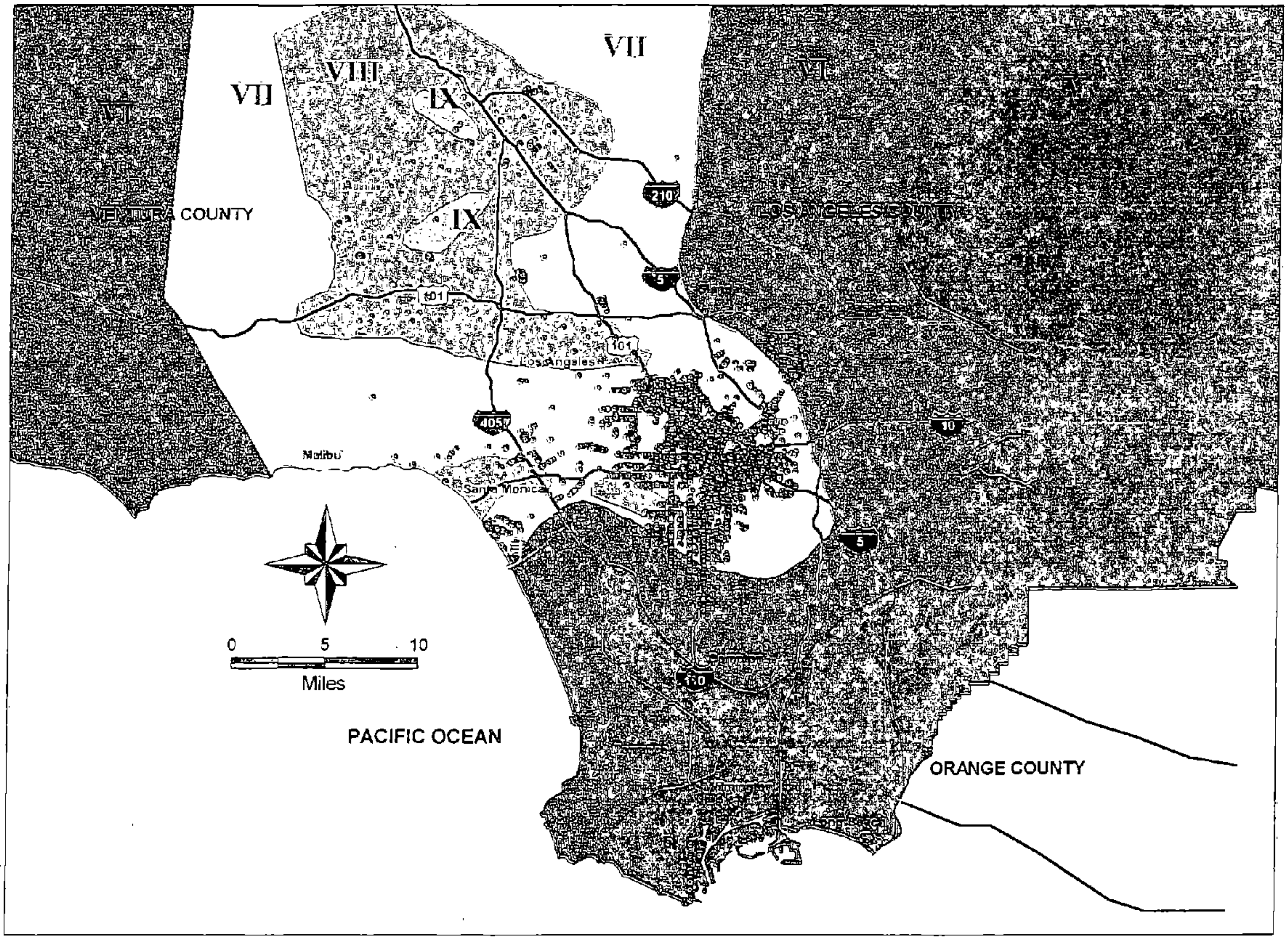


Figure A-11: Total Retrofitted Inventory vs. Spectral Acceleration (g) taken at T=1.0 Seconds



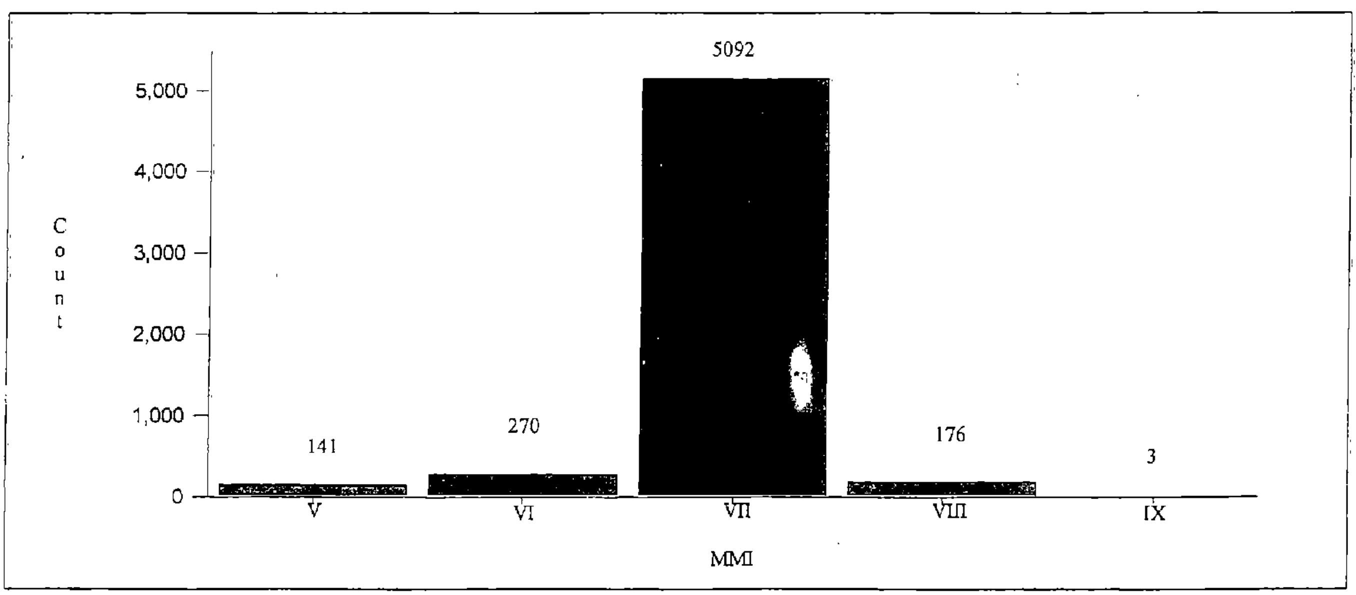
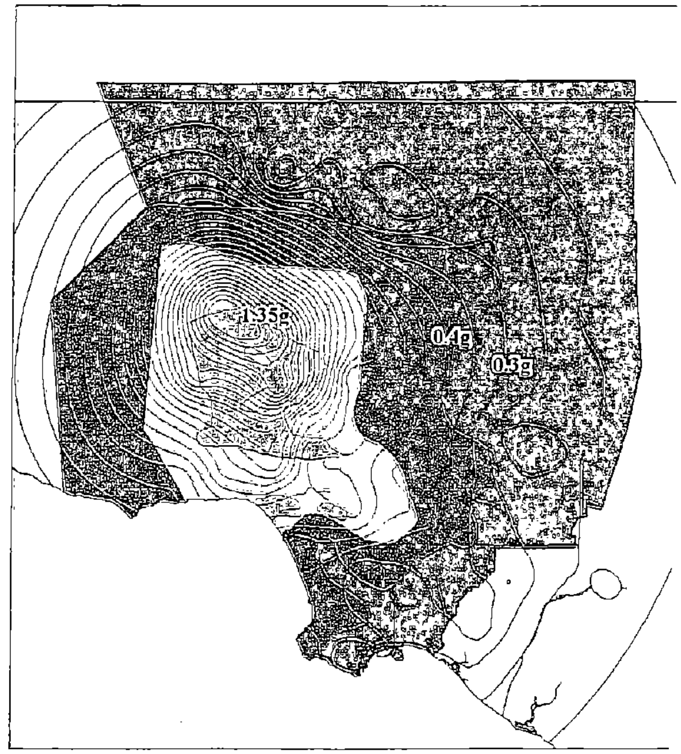
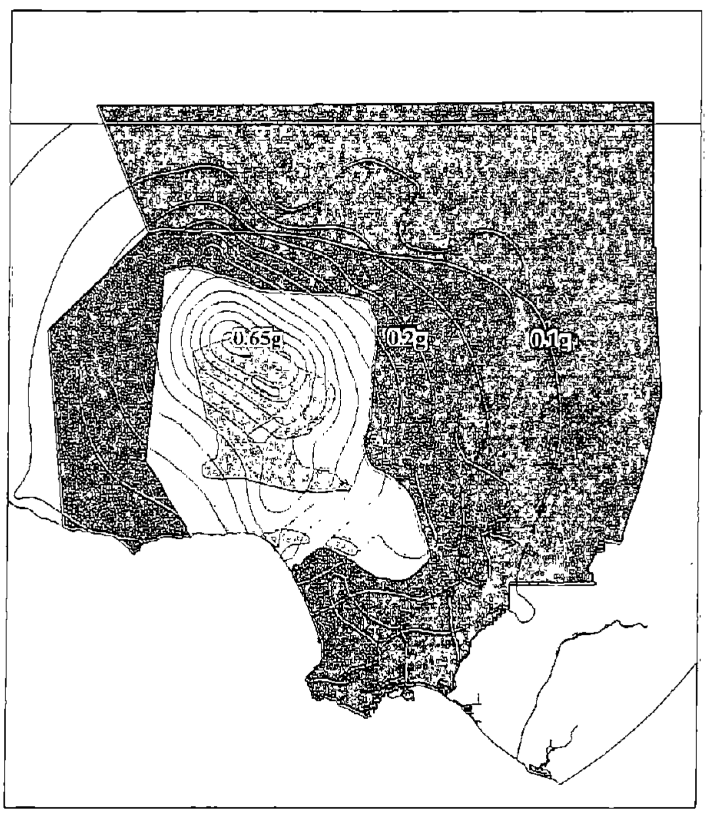


Figure A-12: Total Retrofitted Inventory vs. MMI

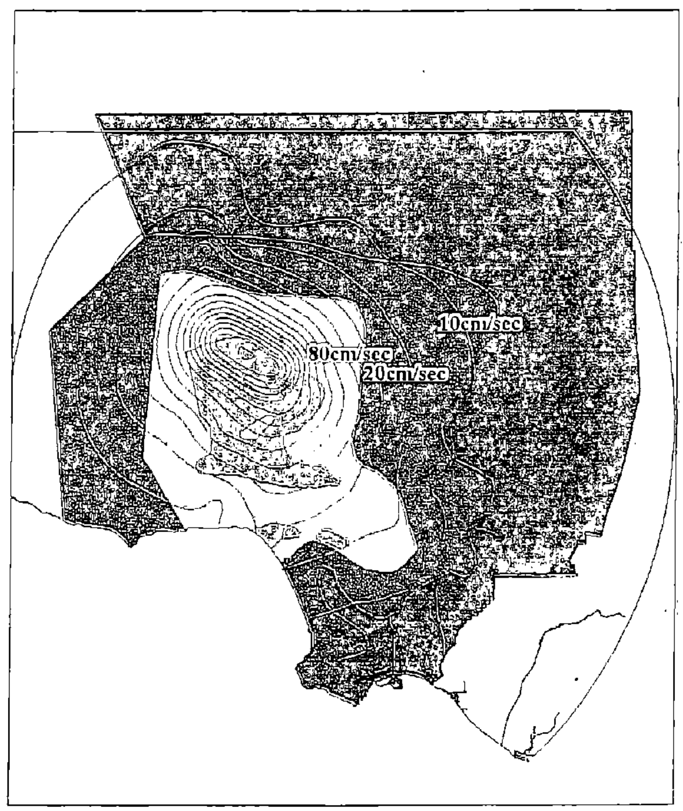
(a) MMI vs Sa(1.0) (g)



(b) MMI vs Sa(0.3) (g)



(c) MMI vs PGA (g)



(d) MMI vs PGV (cm/sec)

Figure A-13: Comparison of MMI Contours and other Ground Motion Contours

example, are low-rise (one- to three-story) residential or commercial structures built prior to 1933. Further, Figures A-8 through A-12 show that most of the inventory is concentrated in the older areas of Los Angeles, including Hollywood, West and Central Los Angeles, Santa Monica, and portions of the San Fernando Valley including North Hollywood, Glendale, and Burbank. In a very general sense, the overall flavor of the inventory used in this study is that it is somewhat homogeneous in construction, but not well-distributed across any range of ground shaking contours. There are many limitations in attempting to use information collected in the post-earthquake safety evaluation process to draw conclusions about the effectiveness of current retrofit provisions. These limitations were summarized in Section A.1.

Performance of Retrofitted and Unretrofitted Structures

A general observation made by reconnaissance teams in the Northridge Earthquake was that retrofitted URM buildings performed better than unretrofitted ones (EERI, 1996). A great majority of retrofitted URM buildings survived the earthquake with minimal or no damage. A number of these strengthened buildings suffered damage; a few experienced partial to near total collapse and others suffered severe structural damage but no collapse. In some cases, even though the buildings suffered damage, the retrofit measures probably succeeded in preventing collapse. Nevertheless, retrofit hardware was occasionally seen among the rubble of a collapse (EERI, 1996). Observed damage included in-plane shear failure and out-of-plane failure of unanchored bearing walls. Another type of damage observed was parapet failure. Several collapsed nine-inch thick parapets were either completely unanchored or anchored only according to a 1957 Los Angeles City anchorage ordinance (EERI, 1996). This 1957 anchorage was observed in some non-Division 88 conforming buildings, and in some cases these anchors performed satisfactorily, yet in others the parapets fell off the building because of ineffective anchorage or poor mortar strength. Failures of unreinforced masonry chimneys were also observed.

As a precursor to detailed damage evaluation, the overall performance of the retrofitted inventory (RFIR Database) was compared with the unretrofitted inventory (RFIU Database). The improvement in performance of retrofitted structures over unretrofitted URM buildings has been observed and documented in past studies, but it is confirmed here with somewhat greater statistical validity than has previously been published. The *RFIR* percent damage estimates for each of these inventories are aggregated using both the ATC-13 and the EERI damage scales. These damage ranges are shown in Table A-7 and A-8, respectively.

The count of the RFIR inventory broken down by MMI for each of the seven ATC-13 damage ranges is shown in Table A-9(a). This style of compilation has been used by previous researchers and is referred to in most of the literature as the ATC-13 damage probability matrix. The primary reason for selecting this tabulation over others, such as Whitman (1973), is for direct comparison with prior damage studies.

Table A-7: ATC-13 Damage Scale (ATC, 1985)

Damage State			Damage Factor Range (%)	Central Damage Factor (%)
1	None	No damage	0	0.0
2	Slight	Limited localized minor damage not requiring repair	0 - 1	0.5
3	Light	Significantly localized damage of some components generally not requiring repair	1 - 10	5.5
4	Moderate	Significant localized damage of many components warranting repair	10 - 30	20
5	Heavy	Extensive damage requiring major repairs	30 - 60	45
6	Major	Major widespread damage that may result in the facility being razed, demolished, or repaired	60 - 100	80
7	Destroyed	Total destruction of the majority of the facility	100	100

Table A-9(a) shows only the 751 retrofitted buildings which were inspected. As noted previously, these buildings were more likely to be located in areas of higher ground shaking and in locales where overall damage was greatest. As a result, they represent an upper bound on estimates of damage to the total inventory. A lower bound estimate of damage is shown in Table A-9(b). Here all of the uninspected buildings are assigned to the damage state of "None." The actual damage to uninspected buildings lies somewhere in between the bounds of Tables A-9(a) and A-9(b). Our best guess estimate of damage to uninspected buildings is shown in Table A-9(c). As the MMI increases, an increasing percentage of uninspected buildings are assigned to the higher damage states.

Damage for each MMI is summarized by the "mean damage factor," which is the sum of the central damage ratio times the percentage in each damage state. As a comparison of the mean damage factors shows, the estimates in Table A-9(c) lie between those of Tables A-9(a) and A-9(b), but are closer to A-9(b) than A-9(a). As a result, it is our opinion that not only is Table A-9(a) an upper bound damage probability matrix, but it is an unreasonably high estimate of damage to the total inventory.

Table A-8: EERI Modified Damage Scale (EERI, 1994)

Damage State			Damage Factor Range (%)
A	None	No damage, but could be shifted contents. Only incidental hazard.	0
B	Slight	Minor damage to nonstructural elements. Building may be temporarily closed but could probably be reopened after minor cleanup in less than 1 week. Only incidental hazard.	0 - 1
C	Moderate	Primarily nonstructural damage; there also could be minor but non-threatening structural damage; building probably closed for 2 to 12 weeks. Remote chance of life-threatening situation from nonstructural elements.	1 - 5
D	Extensive	Extensive structural and nonstructural damage. Long-term closure should be expected, due either to amount of repair work or uncertainty on economic feasibility of repair. Localized, life-threatening situations would be common.	5 - 30
E	Complete	Complete collapse or damage that is not economically repairable. Life-threatening situations in every building of this category.	30 - 100

Accompanying the table is a bar chart representation of the percent of inventory in each ATC-13 damage state for each MMI (Figure A-14). A similar bar chart showing the percent inventory in each damage state vs. MMI is shown in Figure A-15 for data aggregated using the EERI damage scale. In both Figure A-14(a) and A-15, the uninspected buildings are assumed to have no damage. Though the scales are defined differently, similar trends emerge as the MMI increases.

ATC-20 (1989) tagging information is presented in Tables A-9(d) and A-9(e). Assuming the uninspected buildings would have been tagged green, the percentage of red tags increases as the MMI is increased. The percentages of red-tagged buildings are 0.4%, 1.1% and 5.1% for MMI=VI, VII and VIII, respectively. A similar trend is shown for PGA: the percentages of damage are 0.0%, 0.8%, 1.0%, 4.3%, and 1.6% for PGA intervals of less than 0.1g, 0.1-0.2g, 0.2-0.3g, 0.3-0.4g, and over 0.4g.

The unretrofitted inventory is prepared and presented in the same manner; the count of the RFIU database is broken down by MMI for each of the seven ATC-13 damage ranges, as shown in Table A-10(a) and A-10(b). The mean damage factor increases with increasing MMI. For inspected buildings only, the mean damage factors are 2.8%, 2.4%, 10.2%, 9.1% for V, VI, VII, and VIII, respectively. With uninspected buildings assumed to have no damage, the mean damage factors are 0.3%, 0.2%, and 1.5%, and 1.7%. Note a jump in damage from MMI=VI to VII. There are too few buildings in MMI=VIII to draw conclusions about the difference between MMI=VII and VIII, but with a much larger sample of unretrofitted buildings, Lizundia et al. (1993) found a significant jump in damage from MMI=VII to VIII. This jump was confirmed both by observations and statistically.

A bar chart showing the percent of unretrofitted inventory in each ATC-13 damage state for each MMI is shown in Figure A-16. This chart may be compared with the retrofitted inventory, Figure A-14, to see the improvement in performance. This improvement in performance was even more pronounced when evaluating only the inspected buildings.

Table 10(c) shows a damage probability matrix for buildings which only had tension ties installed at the time of the earthquake. These buildings were concentrated exclusively in MMI=VII, so we can not to draw conclusions about the relationship between MMI and damage. The data sample is also quite small—only eight. The mean damage factor is 2.1.

In Table A-11(a) the percentage of the retrofitted inventory in each EERI damage state is compared with numerical damage projections for buildings retrofitted to the 1991 UCBC developed in EERI (1994). The EERI (1994) estimates, given as damage ranges in Table A-11(a), are based on the judgment of a group of structural engineers experienced in earthquake investigations and in writing building codes. Table A-11(b) shows the actual damage in the URM building data collected for this study. The uninspected buildings were assigned to the "None/Slight" category. The EERI (1994) projections tend to overestimate actual damage to *retrofitted* buildings. Lizundia, et al. (1993), similarly, found that the ATC-13 (1985) projections overestimated damage to *unretrofitted* buildings in the Loma Prieta Earthquake, as well as previous earthquakes.

Figure A-17 shows the estimated damage ranges as dashed range bars and the percentages of inventory in each damage state in bar chart format. Clearly, the damage predictions envelope the actual performance by a generous margin; retrofitted building performance was considerably better than predicted.

Correlations Between Building Damage and Quantitative Ground Motion Parameters

In the previous section, damage data were correlated with MMI, the traditional measure of ground motion intensity in past U.S. earthquakes. As strong motion recordings have become available in recent earthquakes—particularly the Northridge Earthquake—considerable attention can now be focused on quantitative correlations between strong motion parameters and damage data. One goal of these efforts is to find which ground motion parameters are best used to predict certain types of damage.

Table A-9(a): Retrofitted Building Inventory: ATC-13 Damage State vs. MMI (Inspected Buildings Only)

ATC-13 Damage Description	Central Damage Ratio	Inspection Status	MMI V Number	V Percent	VI Number	VI Percent	VII Number	VII Percent	VIII Number	VIII Percent	IX Number	IX Percent
None	0.000	Inspected	0		3		139		2		0	
		Uninspected	0		0		0		0		0	
		Total	0	0.00	3	17.65	139	20.09	2	5.26	0	0.00
Slight	0.005	Inspected	2		8		183		3		0	
		Uninspected	0		0		0		0		0	
		Total	2	66.67	8	47.06	183	26.45	3	7.89	0	0.00
Light	0.055	Inspected	1		5		264		17		0	
		Uninspected	0		0		0		0		0	
		Total	1	33.33	5	29.41	264	38.15	17	44.74	0	0.00
Moderate	0.200	Inspected	0	0.00	1	5.88	70	10.12	12	31.58	1	100.00
Heavy	0.450	Inspected	0	0.00	0	0.00	25	3.61	3	7.89	0	0.00
Major	0.800	Inspected	0	0.00	0	0.00	8	1.16	1	2.63	0	0.00
Destroyed	1.000	Inspected	0	0.00	0	0.00	3	0.43	0	0.00	0	0.00
Total Uninspected = 0 Total Inspected = 751 Total in Sample = 751			3	100.00	17	100.00	692	100.00	38	100.00	1	100.00
Mean Damage Factor				**2.17**		**3.03**		**7.24**		**14.47**		**20.00**

Table A-9(b): Retrofitted Building Inventory: ATC-13 Damage State vs. MMI (Uninspected Buildings Assumed to Have No Damage)

ATC-13 Damage Description	Central Damage Ratio	Inspection Status	MMI V Number	V Percent	VI Number	VI Percent	VII Number	VII Percent	VIII Number	VIII Percent	IX Number	IX Percent
None	0.000	Inspected	0		3		139		2		0	
		Uninspected	138		253		4400		138		2	
		Total	138	97.87	256	94.81	4539	89.14	140	79.55	2	66.67
Slight	0.005	Inspected	2		8		183		3		0	
		Uninspected	0		0		0		0		0	
		Total	2	1.42	8	2.96	183	3.59	3	1.70	0	0.00
Light	0.055	Inspected	1		5		264		17		0	
		Uninspected	0		0		0		0		0	
		Total	1	0.71	5	1.85	264	5.18	17	9.66	0	0.00
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**		**0.19**		**0.98**		**3.13**		**6.67**

Table A-9(c): Retrofitted Building Inventory: ATC-13 Damage State vs. MMI (Uninspected Buildings Assumed to Have Some Damage)

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 A-9(d): Retrofitted Building Inventory: MMI vs. Building Posting

ATC-13 Damage Description	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
Green	Inspected	3		14		451		14		0	
	Uninspected	138		250		4400		140		3	
	Total	141	100.00	264	98.88	4851	95.27	154	86.52	3	75.00
Yellow	Inspected	0	0.00	1	0.37	152	2.99	14	7.87	1	25.00
Red	Inspected	0	0.00	1	0.37	56	1.10	9	5.06	0	0.00
Blank	Inspected	0	0.00	1	0.37	33	0.65	1	0.56	0	0.00
Total Uninspected = 4931 Total Inspected = 751 Total in Sample = 5682		141	100.00	267	100.00	5092	100.00	178	100.00	4	100.00

Table A-9(e): Retrofitted Building Inventory: PGA vs. Building Posting

ATC-13 Damage Description	Inspection Status	Peak Ground Acceleration (g)									
		Less than 0.1		0.1 to 0.2		0.2 to 0.3		0.3 to 0.4		Greater than 0.4	
		Number	Percent	Number	Percent	Number	Percent	Number	Percent	Number	Percent
Green	Inspected	3		31		395		38		15	
	Uninspected	125		815		3660		233		98	
	Total	128	100.00	846	96.25	4055	95.43	271	90.33	113	89.68
Yellow	Inspected	0	0.00	23	2.62	123	2.89	13	4.33	9	7.14
Red	Inspected	0	0.00	7	0.80	44	1.04	13	4.33	2	1.59
Blank	Inspected	0	0.00	3	0.34	27	0.64	3	1.00	2	1.59
Total Uninspected = 4931 Total Inspected = 751 Total in Sample = 5682		128	100.00	879	100.00	4249	100.00	300	100.00	126	100.00

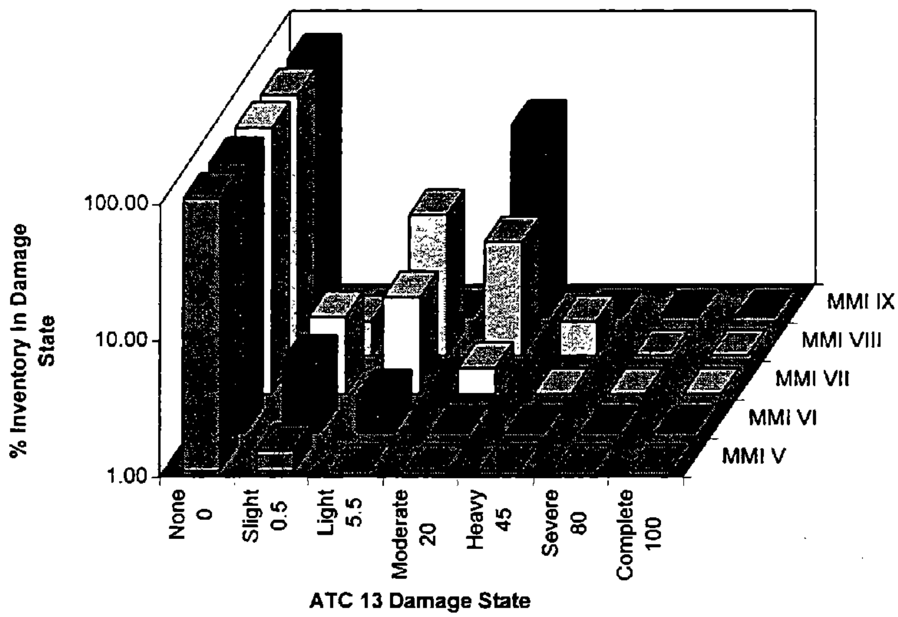


Figure A-14a: Percentage of Total Retrofitted Building Inventory vs. ATC-13 Damage State (Uninspected Buildings Assumed to Have No Damage)

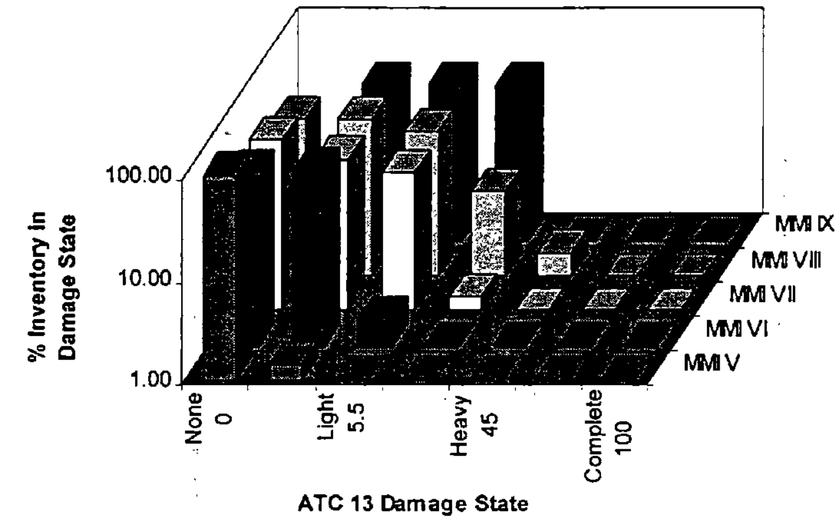


Figure A-14b: Percentage of Total Retrofitted Building Inventory vs. ATC-13 Damage State (Uninspected Buildings Assumed to Have Some Damage)

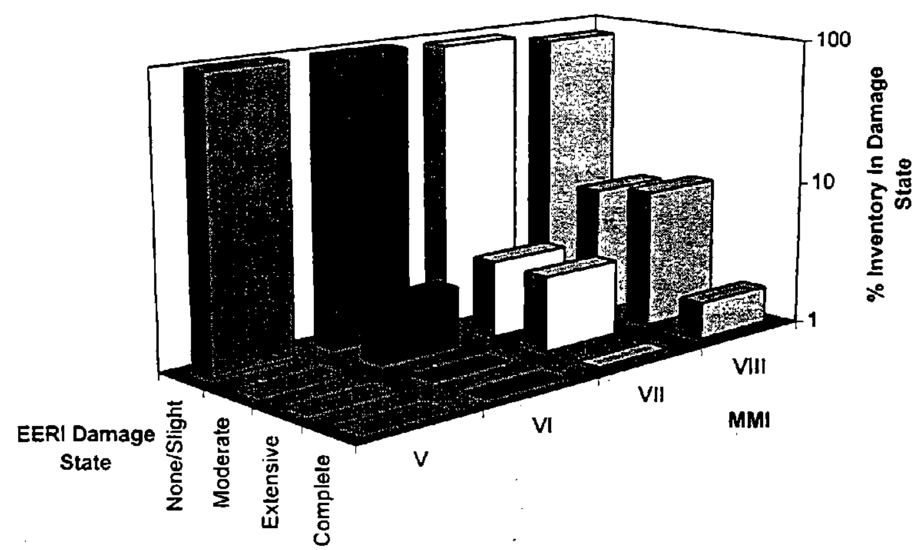


Figure A-15: Percentage of Total Retrofitted Building Inventory in Each EERI Damage State for MMI V, VI, VII, and VIII

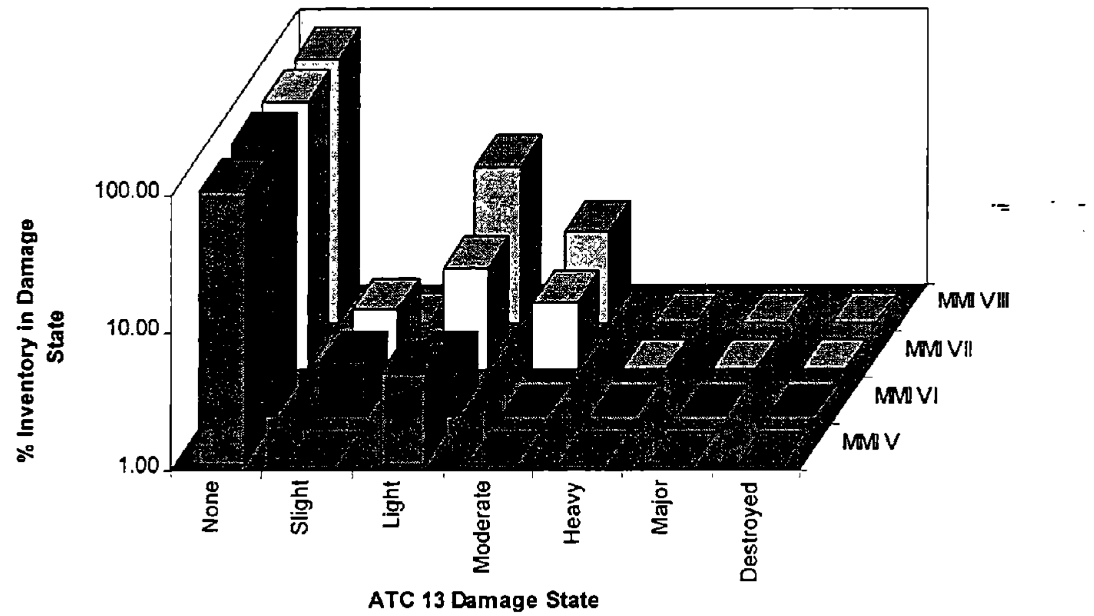


Figure A-16: Percentage of Total Unretrofitted Building Inventory vs. ATC-13 Damage State

Table A-10(a): Unretrofitted Building Damage Matrix: ATC-13 Damage State vs. MMI (Inspected Buildings Only)

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	1		1		11		0		0	
		Uninspected	0		0		0		0		0	
		Total	1	50.00	1	20.00	11	13.41	0	0.00	0	100.00
Slight	0.005	Inspected	0	0.00	2	40.00	16	19.51	0	0.00	0	0.00
Light	0.055	Inspected	1	50.00	2	40.00	32	39.02	3	75.00	0	0.00
Moderate	0.200	Inspected	0	0.00	0	0.00	18	21.95	1	25.00	0	0.00
Heavy	0.450	Inspected	0	0.00	0	0.00	3	3.66	0	0.00	0	0.00
Major	0.800	Inspected	0	0.00	0	0.00	2	2.44	0	0.00	0	0.00
Destroyed	1.000	Inspected	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
Total Uninspected = 0 Total Inspected = 93 Total in Sample = 93			2	100.00	5	100.00	82	100.00	4	100.00	0	100.00
Mean Damage Factor				**2.75**		**2.40**		**10.23**		**9.13**		**0.00**

Table A-10(b): Unretrofitted Building Damage Matrix: ATC-13 Damage State vs. MMI (Uninspected Buildings Assumed to Have No Damage)

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	1		1		11		0		0	
		Uninspected	20		74		498		18		0	
		Total	21	95.45	75	94.94	509	87.76	18	81.82	0	100.00
Slight	0.005	Inspected	0	0.00	2	2.53	16	2.76	0	0.00	0	0.00
Light	0.055	Inspected	1	4.55	2	2.53	32	5.52	3	13.64	0	0.00
Moderate	0.200	Inspected	0	0.00	0	0.00	18	3.10	1	4.55	0	0.00
Heavy	0.450	Inspected	0	0.00	0	0.00	3	0.52	0	0.00	0	0.00
Major	0.800	Inspected	0	0.00	0	0.00	2	0.34	0	0.00	0	0.00
Destroyed	1.000	Inspected	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
Total Uninspected = 610 Total Inspected = 93 Total in Sample = 703			22	100.00	79	100.00	580	100.00	22	100.00	0	100.00
Mean Damage Factor				**0.25**		**0.15**		**1.45**		**1.66**		**0.00**

Table A-10(c): Tension Tie Building Damage Matrix: ATC-13 Damage State vs. MMI (Uninspected Buildings Assumed to Have No Damage)

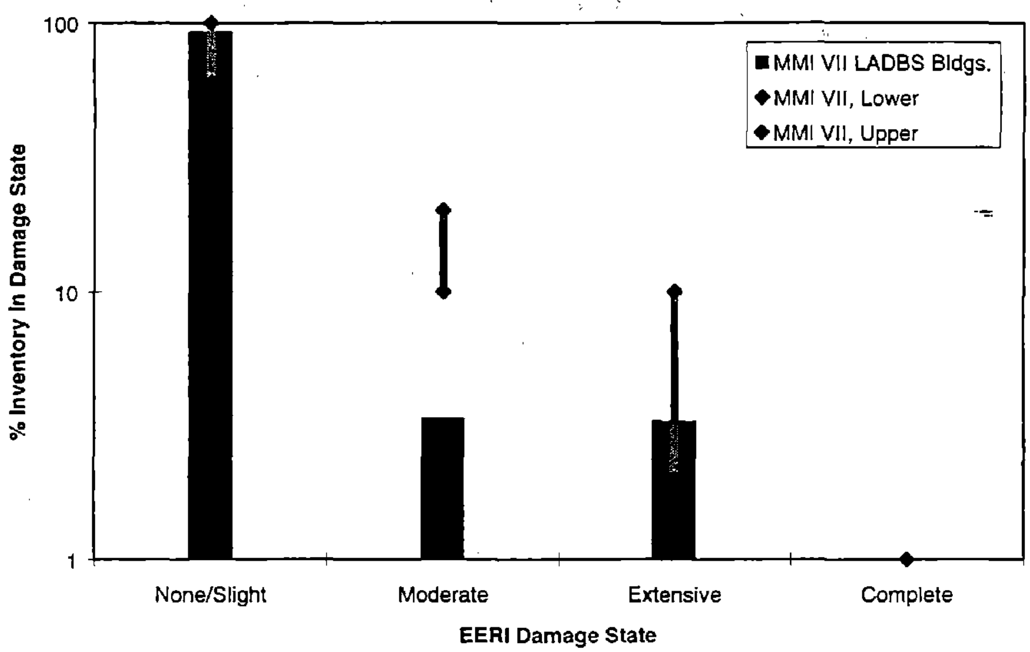
ATC-13 Damage Description	Central Damage Ratio	Inspection Status	MMI V Number	MMI V Percent	MMI VI Number	MMI VI Percent	MMI VII Number	MMI VII Percent	MMI VIII Number	MMI VIII Percent	MMI IX Number	MMI IX Percent
None	0.000	Inspected	0		0		2		0		0	
		Uninspected	1		3		47		2		0	
		Total	1	100.00	3	100.00	49	89.09	2	100.00	0	100.00
Slight	0.005	Inspected	0	0.00	0	0.00	2	3.64	0	0.00	0	0.00
Light	0.055	Inspected	0	0.00	0	0.00	3	5.45	0	0.00	0	0.00
Moderate	0.200	Inspected	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
Heavy	0.450	Inspected	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
Major	0.800	Inspected	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
Destroyed	1.000	Inspected	0	0.00	0	0.00	1	1.82	0	0.00	0	0.00
Total Uninspected = 53 Total Inspected = 8 Total in Sample = 61			1	100.00	3	100.00	55	100.00	2	100.00	0	100.00
Mean Damage Factor				**0.00**		**0.00**		**2.14**		**0.00**		**0.00**

Table A-11(a): Expected Performance of URM Buildings Rehabilitated Under the UCBC (EERI, 1994)

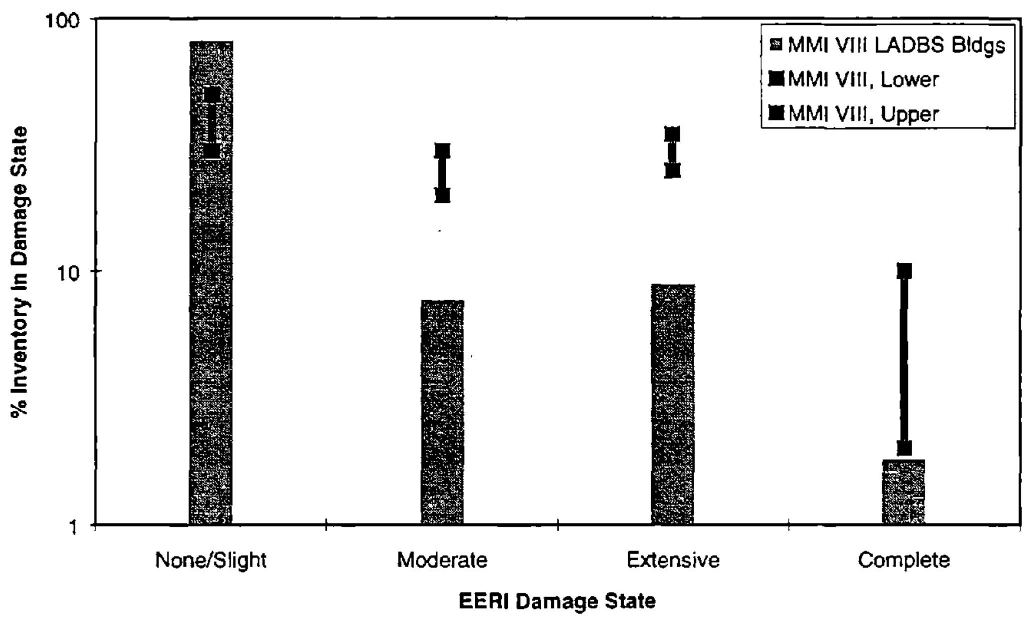
Percentage of Buildings Expected in Each Damage State for Various Shaking Intensities							
Size of Earthquake (Magnitude)			Damage States				
6.0-6.5	7.5-8.0	Expected	A	B	C	D	E
Distance to Fault		MMI	None	Slight	Moderate	Extensive	Complete
30 miles	50 miles	VII	40-60	20-40	10-20	2-10	<1
5 miles	40 miles	VIII	15-25	15-25	20-30	25-35	2-10
1 mile	30 miles	IX	2-10	5-15	25-35	40-60	5-15
--	3 miles	X	<1	2-10	10-20	50-70	15-25

Table A-11(b): Percentage of Damage in Various Damage States Compared with EERI (1994)

Ground Motion Level	Assumed EERI Damage State			
	None/Slight	Moderate	Extensive	Complete
EERI (1994) MMI VII	60-100	10-20	2-10	<1
Northridge MMI VII	92.8	3.3	3.2	0.7
EERI (1994) MMI VIII	30-50	20-30	25-35	2-10
Northridge MMI VIII	82.4	7.4	8.5	1.7



(a) MMI VII



(b) MMI VIII

Figure A-17: Percentage of Total Retrofitted Inventory in Each EERI Damage State vs. EERI Expected Performance Range

The percent damage data for the RFIR Database is broken down by ATC-13 damage state for ground shaking parameters PGA, PGV, $S_a(0.3)$, and $S_a(1.0)$ in Tables A-12 through A-15, respectively. In each of these tables, the original ground motion contours have been broadened from the original contours for ease of comparison and to help avoid statistical aberrations created by a small number of datapoints in a particular band. With few exceptions, mean damage factors increase with increasing ground shaking and experience the sharp increase described above at approximately the middle band of shaking intensity.

Since each successive ATC-13 damage state increases the level of damage, a good ground motion predictor of damage should increase as the damage state increases. Figure A-18 shows a comparison of four ground motion parameters PGA, PGV, $S_a(0.3)$, and $S_a(1.0)$, normalized and plotted against the ATC-13 damage state. The process of normalization involved taking each data point and dividing it by the largest value for that parameter. Theoretically, we are seeking a parameter that increases for higher damage states—the steeper the curve the better the predictor. It appears that PGA is the best predictor since it is the only one which monotonically increases. The lack of data in the higher damage states limits the effectiveness of this approach.

Correlations Between Building Characteristics and Building Damage

Among URM bearing wall buildings, there can be a wide range of types of buildings. ATC-13 (1985) separated the URM bearing wall buildings into two facility classes: low-rise (1-3 stories) and medium rise (4-7 stories). Rutherford & Chekene (1990) divided a database of 2007 buildings into 15 "prototypes" which were, in turn, combined into five groups for damage calculations. Because of the extensive nature of the RFIR Database, correlations between damage and a wide variety of attributes (number of stories, number of units, horizontal and vertical aspect ratios, building square footage, occupancy type, etc.) can be attempted. Of interest to this study were the number of stories and the horizontal and vertical aspect ratio correlations because, in part, taller more elongated URM buildings are more common in other parts of the country than in California.

Number of Stories: The ATC-13 damage probability matrix comparing low-rise and medium-rise URM building is given in Table A-16(a). Note that story data was not available for four buildings. The mean damage factor increases substantially—from 0.86% to 2.35%. This sharp increase would in all likelihood have been much more pronounced if the mid-rise buildings were distributed similarly to the low-rise buildings. As Figure A-19 shows, the mid-rise buildings were primarily in an area with lower ground motion, while the low-rise buildings were better distributed across all shaking levels. The inventories are further broken down by the ground motion parameter $S_a(0.3)$ in Table A-16(b).

Basement Influence: The presence or absence of a basement was reported in the RFIR Database with enough frequency (over 70%) to merit investigation. Table A-17(a) shows

Table A-12: Comparison of ATC-13 Damage State and Peak Ground Acceleration

ATC-13 Damage Description	Central Damage Ratio	Inspection Status	Peak Ground Acceleration (g)									
			Less than 0.1		0.1 to 0.2		0.2 to 0.3		0.3 to 0.4		Greater than 0.4	
			Number	Percent	Number	Percent	Number	Percent	Number	Percent	Number	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

Table A-13: Comparison of ATC-13 Damage State and Sa(0.3)

ATC-13 Damage Description	Central Damage Ratio	Inspection Status	Spectral Acceleration (g) taken at T = 0.3 sec									
			Less than 0.25		0.25 to 0.50		0.50 to 0.75		0.75 to 1.00		Greater than 1.00	
			Number	Percent	Number	Percent	Number	Percent	Number	Percent	Number	Percent
None	0.000	Inspected	0		96		43		5		0	
		Uninspected	0		3812		1000		70		49	
		Total	0	0.00	3908	90.19	1043	88.17	75	66.96	49	90.74
Slight	0.005	Inspected	0	0.00	143	3.30	44	3.72	7	6.25	2	3.70
Light	0.055	Inspected	0	0.00	195	4.50	72	6.09	19	16.96	1	1.85
Moderate	0.200	Inspected	0	0.00	58	1.34	16	1.35	8	7.14	2	3.70
Heavy	0.450	Inspected	0	0.00	19	0.44	6	0.51	3	2.68	0	0.00
Major	0.800	Inspected	0	0.00	7	0.16	2	0.17	0	0.00	0	0.00
Destroyed	1.000	Inspected	0	0.00	3	0.07	0	0.00	0	0.00	0	0.00
			0		521		183		42		5	
Total Uninspected = 4931 Total Inspected = 751 Total in Sample = 5682			0	0.00	4333	100.00	1183	100.00	112	100.00	54	100.00
Mean Damage Factor				0.00		0.93		0.99		3.60		0.86

Table A-14: Comparison of ATC-13 Damage State and Sa(1.0)

ATC-13 Damage Description	Central Damage Ratio	Inspection Status	Spectral Acceleration (g) taken at T = 1.0 sec									
			Less than 0.15		.15 to .20		.20 to .25		.25 to .50		Greater than 0.50	
			Number	Percent	Number	Percent	Number	Percent	Number	Percent	Number	Percent
None	0.000	Inspected	1		65		50		28		0	
		Uninspected	268		3138		873		604		48	
		Total	269	98.90	3203	92.12	923	84.60	632	80.10	48	90.57
Slight	0.005	Inspected	2	0.74	96	2.76	50	4.58	46	5.83	2	3.77
Light	0.055	Inspected	1	0.37	133	3.83	89	8.16	63	7.98	1	1.89
Moderate	0.200	Inspected	0	0.00	34	0.98	18	1.65	30	3.80	2	3.77
Heavy	0.450	Inspected	0	0.00	9	0.26	6	0.55	13	1.65	0	0.00
Major	0.800	Inspected	0	0.00	2	0.06	3	0.27	4	0.51	0	0.00
Destroyed	1.000	Inspected	0	0.00	0	0.00	2	0.18	1	0.13	0	0.00
Total Uninspected = 4931 Total Inspected = 751 Total in Sample = 5682			272	100.00	3477	100.00	1091	100.00	789	100.00	53	100.00
Mean Damage Factor				**0.02**		**0.58**		**1.45**		**2.50**		**0.88**

Table A-15: Comparison of ATC-13 Damage State and Peak Ground Velocity

ATC-13 Damage Description	Central Damage Ratio	Inspection Status	Peak Ground Velocity (cm/sec)									
			Less than 10		10 to 15		15 to 20		20 to 25		Greater than 25	
			Number	Percent	Number	Percent	Number	Percent	Number	Percent	Number	Percent
None	0.000	Inspected	0		10		112		17		5	
		Uninspected	146		1036		3255		354		140	
		Total	146	97.99	1046	94.32	3367	89.57	371	79.61	145	72.86
Slight	0.005	Inspected	2	1.34	16	1.44	133	3.54	32	6.87	13	6.53
Light	0.055	Inspected	1	0.67	28	2.52	200	5.32	35	7.51	23	11.56
Moderate	0.200	Inspected	0	0.00	14	1.26	40	1.06	18	3.86	12	6.03
Heavy	0.450	Inspected	0	0.00	3	0.27	12	0.32	8	1.72	5	2.51
Major	0.800	Inspected	0	0.00	2	0.18	4	0.11	2	0.43	1	0.50
Destroyed	1.000	Inspected	0	0.00	0	0.00	3	0.08	0	0.00	0	0.00
Total Uninspected = 4931 Total Inspected = 751 Total in Sample = 5682			149	100.00	1109	100.00	3759	100.00	466	100.00	199	100.00
Mean Damage Factor				**0.04**		**0.66**		**0.83**		**2.34**		**3.41**

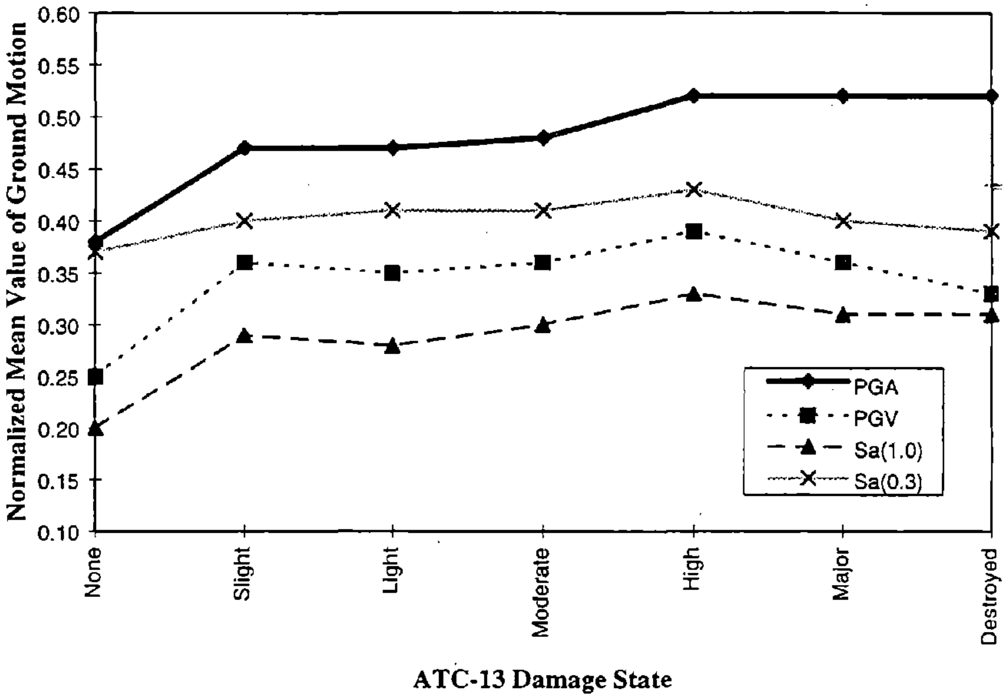


Figure A-18: Comparison of Normalized Ground Motion Parameters and Mean Damage Factor

Table A-16(a): Comparison of ATC-13 Damage with Number of Stories

ATC-13 Damage Description	Central Damage Ratio	Inspection Status	Number of Stories			
			1 through 3		4 through 6	
			Number	Percent	Number	Percent
None	0.000	Inspected	91		46	
		Uninspected	4698		240	
		Total	4789	91.67	286	63.00
Slight	0.005	Inspected	117	2.24	77	16.96
Light	0.055	Inspected	213	4.08	73	16.08
Moderate	0.200	Inspected	73	1.40	11	2.42
Heavy	0.450	Inspected	23	0.44	5	1.10
Major	0.800	Inspected	7	0.13	1	0.22
Destroyed	1.000	Inspected	2	0.04	1	0.22
Total Uninspected = 4938 Total Inspected = 740 Total in Sample = 5678			5224	100.00	454	100.00
Mean Damage Factor				**0.86**		**2.35**

Table A-16(b): Comparison of ATC-13 Damage with Number of Stories and Spectral Acceleration

ATC-13 Damage Description	Central Damage Ratio	Inspection Status	One to Three Stories, Spectral Acceleration (g), Taken at T = 0.3sec: 0.35-0.50 Number	0.35-0.50 Percent	0.50-0.65 Number	0.50-0.65 Percent	0.65-1.225 Number	0.65-1.225 Percent
None	0.000	Inspected	64		23		4	
		Uninspected	3613		931		154	
		Total	3677	92.67	954	90.43	158	78.61
Slight	0.005	Inspected	83	2.09	25	2.37	9	4.48
Light	0.055	Inspected	139	3.50	53	5.02	20	9.95
Moderate	0.200	Inspected	47	1.18	15	1.42	11	5.47
Heavy	0.450	Inspected	14	0.35	6	0.57	3	1.49
Major	0.800	Inspected	6	0.15	2	0.19	0	0.00
Destroyed	1.000	Inspected	2	0.05	0	0.00	0	0.00
Total Uninspected = 4698 Total Inspected = 526 Total in Sample = 5224			3968	100.00	1055	100.00	201	100.00
Mean Damage Factor				**0.77**		**0.98**		**2.34**

ATC-13 Damage Description	Central Damage Ratio	Inspection Status	Four to Six Stories, Spectral Acceleration (g), Taken at T = 0.3sec: 0.35-0.50 Number	0.35-0.50 Percent	0.50-0.65 Number	0.50-0.65 Percent	0.65-1.225 Number	0.65-1.225 Percent
None	0.000	Inspected	27		19		0	
		Uninspected	184		56			
		Total	211	61.70	75	66.96	0	100.00
Slight	0.005	Inspected	59	17.25	18	16.07	0	0.00
Light	0.055	Inspected	54	15.79	19	16.96	0	0.00
Moderate	0.200	Inspected	11	3.22	0	0.00	0	0.00
Heavy	0.450	Inspected	5	1.46	0	0.00	0	0.00
Major	0.800	Inspected	1	0.29	0	0.00	0	0.00
Destroyed	1.000	Inspected	1	0.29	0	0.00	0	0.00
Total Uninspected = 240 Total Inspected = 214 Total in Sample = 454			342	100.00	112	100.00	0	100.00
Mean Damage Factor				**2.78**		**1.01**		**0.00**

the ATC-13 damage probability matrix developed for buildings with and without basements. The mean damage factors presented in this table are intended for comparison with each other only—the Division 88 database does not indicate the presence or absence of a basement, so the appropriate balance of inventory could not be determined to indicate overall performance. The mean damage factor for buildings with basements was nearly double that of those without basements (9.0% vs. 5.8%). The median number of stories for buildings with basements is 3 while the median for buildings without basements is 1. Thus, despite Figure A-16(a) data showing taller buildings had higher damage, the inspected inventory showed a substantial decrease in the mean damage factor for buildings with a basement. The basement data was further broken down in Table A-17(b) by ground motion parameter $S_a(0.3)$, which showed that for similar spectral accelerations, buildings with basements generally performed better. This is a particularly interesting finding since basements are very common in much of the United States—much more so than in California.

Horizontal Aspect Ratio: The ATC-13 damage probability matrix developed for the horizontal or plan aspect ratio is shown in Table A-18. Plan aspect ratio (long dimension/short dimension) is presented in two categories: more flexible, with aspect ratio greater than 2.0 and less flexible, with aspect ratio less than 2.0. The mean damage factor of the more flexible diaphragm was larger than the less flexible (Figure A-20), and the damage factors for each damage state envelope (but not by much) those of the less flexible diaphragm. The plan aspect data was further broken down by ground motion parameters $S_a(0.3)$ and $S_a(1.0)$ in an attempt to discern which is a better predictor of damage. No obvious trends were found.

Vertical Aspect Ratio: The ATC-13 damage probability matrix developed for vertical aspect ratio is shown in Table A-19. Vertical aspect ratio (height of the building/shortest plan dimension) is presented in two categories: more flexible, with aspect ratio greater than 0.5 and less flexible, with aspect ratio less than 0.5. As expected, the mean damage factor of the more flexible building set was larger than that of the less flexible, and the damage factors for each damage state envelope those of the less flexible diaphragm (Figure A-21). Correlations between percent damage and the ground motion parameters $S_a(0.3)$ and $S_a(1.0)$ are also shown in Table A-19. Both parameters appear to correlate well with the vertical aspect ratio.

Correlations Between Damage Descriptions and Ground Motion Parameters

As described previously, the damage data in the RFIR Database was collected in completing the LADBS Rapid Screening Inspection Form (Figure A-2). Specifically, the data form includes damage categories for which the inspector checks YES/NO/UNKNOWN—a YES answer indicating an unsafe condition. A place for comments is also provided on the form. The damage categories of interest to this study are grouped below. The structural damage descriptions were taken from the comment fields.

Table A-17(a): Comparison of ATC-13 Damage for Buildings With and Without Basements

Damage State Number	Description	Central Damage Ratio	Without Basement		With Basement	
			Number	Percent	Number	Percent
1	None	0.000	41	14.75	58	22.66
2	Slight	0.005	66	23.74	72	28.13
3	Light	0.055	117	42.09	97	37.89
4	Moderate	0.200	32	11.51	20	7.81
5	Heavy	0.450	17	6.12	6	2.34
6	Major	0.800	4	1.44	3	1.17
7	Destroyed	1.000	1	0.36	0	0.00
Total	534		278	100.00	256	100.00
Mean Damage Factor				9.00		5.78

Table A-17(b): Comparison of ATC-13 Damage for Buildings With and Without Basements vs. Spectral Acceleration

Buildings with Basements								
Damage State Number	Description	Central Damage Ratio	Spectral Acceleration (g) taken at T = 0.3 sec					
			0.35 - 0.50		0.50 - 0.65		0.65 - 1.225	
			Number	%	Number	%	Number	%
1	None	0.000	36	18.85	22	36.67	0	0.00
2	Slight	0.005	54	28.27	16	26.67	2	40.00
3	Light	0.055	76	39.79	21	35.00	0	0.00
4	Moderate	0.200	16	8.38	1	1.67	3	60.00
5	Heavy	0.450	6	3.14	0	0.00	0	0.00
6	Severe	0.800	3	1.57	0	0.00	0	0.00
7	Complete	1.000	0	0.00	0	0.00	0	0.00
Total	256		191	100.00	60	100.00	5	100.00
Mean Damage Factor				6.68		2.39		12.20
Buildings without Basements								
Damage State Number	Description	Central Damage Ratio	Spectral Acceleration (g) taken at T = 0.3 sec					
			0.35 - 0.475		0.525 - 0.625		0.675 - 0.825	
			Number	%	Number	%	Number	%
1	None	0.000	24	14.12	14	18.92	3	8.82
2	Slight	0.005	44	25.88	15	20.27	7	20.59
3	Light	0.055	68	40.00	32	43.24	17	50.00
4	Moderate	0.200	21	12.35	5	6.76	6	17.65
5	Heavy	0.450	10	5.88	6	8.11	1	2.94
6	Major	0.800	2	1.18	2	2.70	0	0.00
7	Destroyed	1.000	1	0.59	0	0.00	0	0.00
Total	278		170	100.00	74	100.00	34	100.00
Mean Damage Factor				8.98		9.64		7.71

- General Damage Types
  - Collapse/Partial Collapse
  - Building or Story Leaning
  - Other Hazardous Condition
- Structural Damage Types
  - Foundations
  - Roof/Floor (Vertical Loads)
  - Columns/Pilasters/Corbels
  - Diaphragm/Horizontal Bracing
  - Walls/Vertical Bracing
- Nonstructural Damage Types
  - Parapets/Ornamentation
  - Cladding/Glazing
  - Interior Walls/Partitions
  - Chimney
- Structural Damage Descriptions
  - Wall Cracking
  - Diaphragm-To-Wall Ties
  - Shear Cracks
  - Veneer
  - Corner Damage

The data for each building also includes PGA, $S_a(0.3)$, and $S_a(1.0)$ values of ground motion at the building site, determined from the corresponding ground motion intensity contour. For each item above, the "percentage of buildings damaged" is the percentage of YES answers (indicating damage) of the total inventory (inspected and uninspected) in each ground motion intensity contour was computed. Summaries of each item are presented in Tables A-20 through A-22. To provide context, the table also shows, the "number of buildings damaged" which is the total count of buildings with YES answers in each contour. As previously discussed, not all of the buildings were inspected after the earthquake—only about 15 percent. Typically, buildings were inspected because there was some visible damage resulting in a request for inspection or because they were located in an area of heavy damage. If a building was not inspected it was much less likely to have been damaged. The uninspected inventory was considered to be undamaged; thus, the damage statistics given represent lower bound values.

In a similar fashion, damage versus ground motion matrices were prepared from the written damage descriptions available in the database. Text searches were only performed for items which appear frequently in the damage descriptions. Unfortunately, the written description is brief, allowing only an impression of the type and magnitude of damage to be made. Summaries, shown in Table A-23, include wall cracking, shear cracks, and corner damage. Additional queries were made for diaphragm-to-wall tie failure and veneer failure. The damage percentages reported for these items were skewed toward lower damage because narrative damage descriptions were made at the discretion

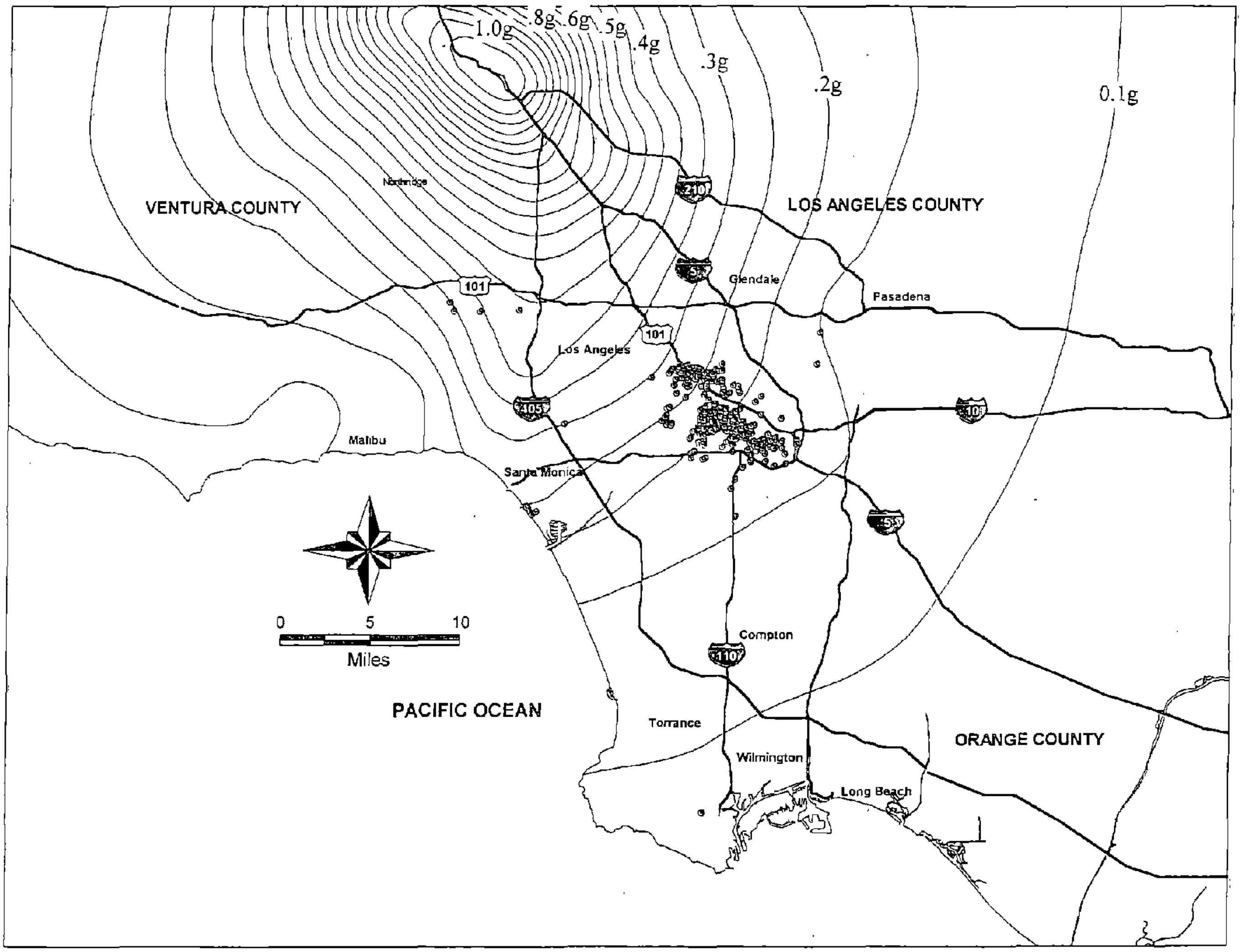


Figure A-19: Location of Midrise Inventory Overlain on Spectral Acceleration (g) taken at T=1.0 sec

Table A-18: Comparison of ATC-13 Damage State and Horizontal Aspect Ratio

Description	Central Damage Ratio	Horizontal Aspect Ratio: Less Flexible < 2.0 Number	Less Flexible < 2.0 Percent	More Flexible > 2.0 Number	More Flexible > 2.0 Percent
None	0.000	49	15.26	89	21.98
Slight	0.005	85	26.48	108	26.67
Light	0.055	138	42.99	136	33.58
Moderate	0.200	34	10.59	49	12.10
Heavy	0.450	11	3 43	16	3.95
Major	0.800	2	0.62	6	1.48
Destroyed	1.000	2	0.62	1	0.25
Total in Sample= 726		321	100.00	405	100.00
Mean Damage Factor			7.28		7.61

Description	Central Damage Ratio	Horizontal Aspect Ratio: Spectral Acceleration (g) taken at T = 0.3 sec — Less Flexible 0.35 - 0.60 Number	Percent	More Flexible 0.35 - 0.60 Number	Percent	Less Flexible 0.60 - 0.66 Number	Percent	More Flexible 0.60 - 0.66 Number	Percent	Less Flexible 0.66 - 1.225 Number	Percent	More Flexible 0.66 - 1.225 Number	Percent
None	0.000	34	15.81	57	20.28	14	15.91	30	30 93	1	5 56	2	7 41
Slight	0 005	62	28.84	79	28.11	17	19.32	24	24 74	6	33 33	5	18.52
Light	0 055	84	39 07	96	34.16	47	53.41	27	27.84	7	38 89	13	48 15
Moderate	0.200	24	11.16	34	12.10	8	9 09	9	9 28	2	11.11	6	22.22
Heavy	0.450	7	3.26	10	3.56	2	2 27	5	5 15	2	11.11	1	3.70
Major	0.800	2	0.93	4	1.42	0	0.00	2	2.06	0	0 00	0	0 00
Destroyed	1.000	2	0.93	1	0.36	0	0 00	0	0.00	0	0.00	0	0 00
Total in Sample= 726		215	100.00	281	100.00	88	100 00	97	100 00	18	100.00	27	100.00
Mean Damage Factor			7.67		7.64		5.88		7.48		9.53		8.85

Description	Central Damage Ratio	Horizontal Aspect Ratio: Spectral Acceleration (g) taken at T =1.0 sec — Less Flexible 0.075 - 0.20 Number	Percent	More Flexible 0.075 - 0.20 Number	Percent	Less Flexible 0.20 - 0.35 Number	Percent	More Flexible 0.20 - 0.35 Number	Percent	Less Flexible 0.35 - 0.75 Number	Percent	More Flexible 0.35 - 0.75 Number	Percent
None	0.000	29	17.58	36	19.35	19	14.18	51	27.42	1	4 55	2	5.88
Slight	0 005	43	26.06	56	30.11	35	26 12	45	24.19	7	31.82	7	20 59
Light	0.055	75	45.45	68	36 56	53	39.55	55	29.57	10	45.45	13	38 24
Moderate	0 200	12	7.27	21	11.29	20	14 93	21	11.29	2	9 09	7	20.59
Heavy	0.450	6	3.64	4	2.15	3	2 24	9	4.84	2	9.09	3	8 82
Major	0.800	0	0.00	1	0 54	2	1.49	4	2.15	0	0.00	1	2 94
Destroyed	1 000	0	0.00	0	0 00	2	1.49	1	0.54	0	0.00	0	0 00
Total in Sample= 726		165	100 00	186	100.00	134	100.00	186	100 00	22	100.00	34	97 06
Mean Damage Factor			5.72		5.82		8.99		8.44		8.57		12.65

Table A-19: Comparison of ATC-13 Damage State and Vertical Aspect Ratio

Description	Central Damage Ratio	Vertical Aspect Ratio: Less Flexible		More Flexible	
		Number	Percent	Number	Percent
None (Inspected)	0.000	65	1.43	81	7.08
None (Uninspected)	0.000	4126	91.00	801	70.02
Slight	0.005	94	2.07	102	8.92
Light	0.055	164	3.62	118	10.31
Moderate	0.200	58	1.28	30	2.62
Heavy	0.450	20	0.44	8	0.70
Major	0.800	5	0.11	3	0.26
Destroyed	1.000	2	0.04	1	0.09
Total in Sample= 5678		4534	100.00	1144	100.00
Mean Damage Factor			0.80		1.76

Description	Central Damage Ratio	Vertical Aspect Ratio: Spectral Acceleration (g) taken at T = 0.3 sec — Less Flexible 0.35 - 0.50		More Flexible 0.35 - 0.50		Less Flexible 0.50 - 0.65		More Flexible 0.50 - 0.65		Less Flexible 0.65 - 1.225		More Flexible 0.65 - 1.225	
		Number	Percent	Number	Percent	Number	Percent	Number	Percent	Number	Percent	Number	Percent
None (Inspected)	0.000	45	1.30	82	6.85	17	1.86	28	11.76	3	2.03	1	5.88
None (Uninspected)	0.000	3208	92.40	638	71.77	806	88.16	156	65.55	112	75.68	7	41.18
Slight	0.005	65	1.87	78	8.77	21	2.30	21	8.82	8	5.41	3	17.65
Light	0.055	99	2.85	86	9.67	50	5.47	27	11.34	15	10.14	5	29.41
Moderate	0.200	38	1.09	25	2.81	13	1.42	4	1.68	7	4.73	1	5.88
Heavy	0.450	11	0.32	7	0.79	6	0.66	1	0.42	3	2.03	0	0.00
Major	0.800	4	0.12	2	0.22	1	0.11	1	0.42	0	0.00	0	0.00
Destroyed	1.000	2	0.06	1	0.11	0	0.00	0	0.00	0	0.00	0	0.00
Total in sample= 5678		3472	100.00	889	100.00	914	100.00	238	100.00	148	100.00	17	100.00
Mean Damage Factor			0.68		1.79		0.98		1.63		2.44		2.88

Description	Central Damage Ratio	Vertical Aspect Ratio: Spectral Acceleration (g) taken at T =1.0 sec — Less Flexible 0.075 - 0.20		More Flexible 0.075 - 0.20		Less Flexible 0.20 - 0.35		More Flexible 0.20 - 0.35		Less Flexible 0.35 - 0.75		More Flexible 0.35 - 0.75	
		Number	Percent	Number	Percent	Number	Percent	Number	Percent	Number	Percent	Number	Percent
None (Inspected)	0.000	32	1.05	38	5.19	30	2.26	42	10.61	3	1.78	1	5.88
None (Uninspected)	0.000	2847	93.77	560	76.50	1154	86.90	236	59.60	124	73.37	6	35.29
Slight	0.005	45	1.48	55	7.51	38	2.86	44	11.11	11	6.51	3	17.65
Light	0.055	81	2.67	65	8.88	65	4.89	48	12.12	18	10.65	5	29.41
Moderate	0.200	21	0.69	13	1.78	30	2.26	15	3.79	7	4.14	2	11.76
Heavy	0.450	9	0.30	1	0.14	6	0.45	7	1.77	5	2.96	0	0.00
Major	0.800	1	0.03	0	0.00	3	0.23	3	0.76	1	0.59	0	0.00
Destroyed	1.000	0	0.00	0	0.00	2	0.15	1	0.25	0	0.00	0	0.00
Total in sample= 5678		3036	100.00	732	100.00	1328	100.00	396	100.00	169	100.00	17	100.00
Mean Damage Factor			0.45		0.94		1.27		3.13		3.25		4.06

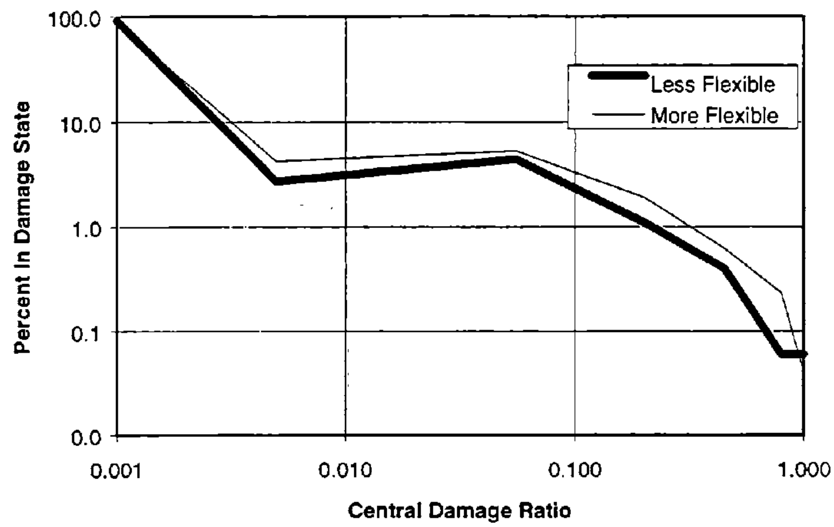


Figure A-20: Comparison of Central Damage Ratio for Less Flexible and More Flexible Horizontal Aspect Ratio

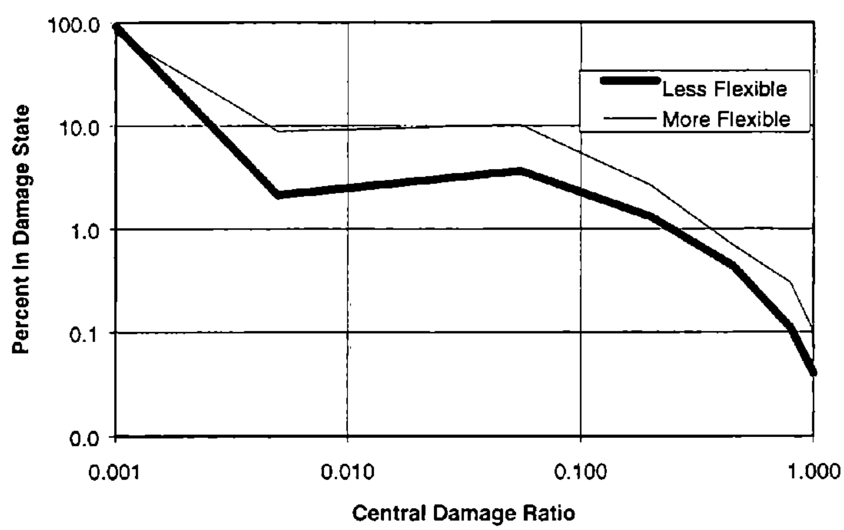


Figure A-21: Comparison of Central Damage Ratio for Less Flexible and More Flexible Vertical Aspect Ratio

of each inspector. Thus, the data was not accrued in a consistent manner. Even more vexing—the total number of occurrences of a parameter in the inventory (parapets or veneer, for example) is unknown. This made a consistent presentation of the "percent damaged in total inventory" impossible for these parameters. Nevertheless, these damage descriptions were interesting to evaluate because they allowed additional parameters to be explored. For example, there were 60 textual references to "corner damage," an item of interest, but one which is not available on the data collection form. Alternately, there were only 11 references to veneer cracking or failure even though the LATF (1994) and EERI (1996) reconnaissance reports indicate the problem to be much more widespread.

The percent of total inventory damaged for each damage type is shown in Table A-24. The number of buildings with chimneys, parapets, veneer and interior partitions is unknown; for this study the total number of buildings with either a YES or NO answer in these categories was used. Thus, since some of these buildings may actually lack such a feature, the damage statistics in Table A-24 represent lower bound values.

The final goal of this effort was to 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. As expected, the general trend for each of the items was increasing damage with increasing ground motion. Once again, there was too little data in the areas of high intensity to suggest the shape of a best fit curve (linear, exponential, etc.). Most items, however, did show a moderate to sharp increase in damage around ground motion contours $S_a(0.3)$=0.75-0.80g, $S_a(1.0)$=0.40-0.45g, and PGA=0.35-0.40g. The intensity values at which this increase occurs is termed "jump" in Table A-25. Recall that a major jump in damage was also observed between MMI VII and VIII and has been observed in past studies. In addition, the intensity value at which the percent damage reaches one percent, termed "one percent," is also shown.

From the tabulations of the data, the following observations, aggregations, and composite profiles can be made:

- The shaking intensity predominant at a damage state is of particular interest for projecting performance of standard West Coast construction to moderate seismic zones. The PGA at a damage state can be roughly correlated with the effective peak accelerations given in FEMA-178 (1992), and, to some extent, with the seismic zone map given the UBC. From Table A-25, most damage types (roof/floors, columns/pilasters/corbels, wall cracking, corner damage, and cladding) reach "one percent" at a PGA of 0.15-0.20g and exhibit a "jump" at PGA of 0.35-0.40g. The spectral acceleration, $S_a(0.3)$, at "one percent" and "jump" are typically 0.45-0.50g and 0.75-0.80g, respectively. The $S_a(1.0)$ at "one percent" and "jump" are typically 0.20-0.25g and 0.35-0.40g, respectively.

Table A-20: Summary of General Damage Types vs. Ground Motion

Collapse/Partial Collapse

Sa @ T=0.3 s (g)	Number of Buildings Damaged	Percentage of Buildings Damaged
0.30-0.35	0	0.0%
0.40-0.45	11	0.7%
0.45-0.50	31	1.2%
0.50-0.55	16	1.6%
0.55-0.60	6	4.4%
0.60-0.65	0	0.0%
0.70-0.75	1	20.0%
0.75-0.80	1	25.0%
0.80-0.85	5	6.1%
0.90-0.95	1	5.3%
1.05-1.10	1	9.1%
1.10-1.15	0	0.0%
1.20-1.25	0	0.0%

Collapse/Partial Collapse

Sa @ T = 1.0 s (g)	Number of Buildings Damaged	Percentage of Buildings Damaged
0.05-0.10	0	0.0%
0.10-0.15	0	0.0%
0.15-0.20	31	0.9%
0.20-0.25	15	1.4%
0.25-0.30	14	2.3%
0.30-0.35	0	0.0%
0.35-0.40	8	12.5%
0.40-0.45	2	20.0%
0.45-0.50	3	4.6%
0.60-0.65	0	0.0%
0.75-0.80	0	0.0%
Totals	73	1.5%

Collapse/Partial Collapse

PGA (g)	Number of Buildings Damaged	Percentage of Buildings Damaged
0.05-0.10	0	0.0%
0.15-0.20	14	1.6%
0.20-0.25	27	0.8%
0.25-0.30	16	1.6%
0.30-0.35	5	2.2%
0.35-0.40	6	9.1%
0.40-0.45	5	6.4%
0.45-0.50	0	0.0%
0.50-0.55	0	0.0%

Building or Story Leaning

Sa @ T=0.3 s (g)	Number of Buildings Damaged	Percentage of Buildings Damaged
0.30-0.35	0	0.0%
0.40-0.45	12	0.8%
0.45-0.50	29	1.1%
0.50-0.55	15	1.5%
0.55-0.60	6	4.4%
0.60-0.65	0	0.0%
0.70-0.75	0	0.0%
0.75-0.80	1	25.0%
0.80-0.85	3	3.7%
0.90-0.95	1	5.3%
1.05-1.10	0	0.0%
1.10-1.15	0	0.0%
1.20-1.25	0	0.0%

Building or Story Leaning

Sa @ T = 1.0 s (g)	Number of Buildings Damaged	Percentage of Buildings Damaged
0.05-0.10	0	0.0%
0.10-0.15	0	0.0%
0.15-0.20	26	0.7%
0.20-0.25	19	1.8%
0.25-0.30	13	2.2%
0.30-0.35	1	2.4%
0.35-0.40	5	7.8%
0.40-0.45	0	0.0%
0.45-0.50	3	4.6%
0.60-0.65	0	0.0%
0.75-0.80	0	0.0%
Totals	67	1.2%

Building or Story Leaning

PGA (g)	Number of Buildings Damaged	Percentage of Buildings Damaged
0.05-0.10	0	0.0%
0.15-0.20	9	1.0%
0.20-0.25	27	0.8%
0.25-0.30	20	2.0%
0.30-0.35	2	0.9%
0.35-0.40	6	9.1%
0.40-0.45	3	3.8%
0.45-0.50	0	0.0%
0.50-0.55	0	0.0%

Other Hazardous Conditions

Sa @ T=0.3 s (g)	Number of Buildings Damaged	Percentage of Buildings Damaged
0.30-0.35	0	0.0%
0.40-0.45	18	1.2%
0.45-0.50	69	2.7%
0.50-0.55	28	2.8%
0.55-0.60	6	4.4%
0.60-0.65	0	0.0%
0.70-0.75	1	20.0%
0.75-0.80	1	25.0%
0.80-0.85	8	9.8%
0.90-0.95	2	10.5%
1.05-1.10	0	0.0%
1.10-1.15	0	0.0%
1.20-1.25	0	0.0%

Other Hazardous Conditions

Sa @ T = 1.0 s (g)	Number of Buildings Damaged	Percentage of Buildings Damaged
0.05-0.10	0	0.0%
0.10-0.15	0	0.0%
0.15-0.20	55	1.6%
0.20-0.25	38	3.5%
0.25-0.30	24	4.0%
0.30-0.35	1	2.4%
0.35-0.40	11	17.2%
0.40-0.45	1	10.0%
0.45-0.50	3	4.6%
0.60-0.65	0	0.0%
0.75-0.80	0	0.0%
Totals	133	2.3%

Other Hazardous Conditions

PGA (g)	Number of Buildings Damaged	Percentage of Buildings Damaged
0.05-0.10	0	0.0%
0.15-0.20	20	2.2%
0.20-0.25	51	1.6%
0.25-0.30	44	4.5%
0.30-0.35	4	1.8%
0.35-0.40	10	15.2%
0.40-0.45	4	5.1%
0.45-0.50	0	0.0%
0.50-0.55	0	0.0%

Table A-21: Summary of Structural Damage Types vs. Ground Motion

Foundations

Sa @ T=0.3 s (g)	Number of Buildings Damaged	Percentage of Buildings Damaged
0.30-0.35	0	0.00%
0.40-0.45	1	0.06%
0.45-0.50	3	0.12%
0.50-0.55	1	0.10%
0.55-0.60	1	0.74%
0.60-0.65	0	0.00%
0.70-0.75	0	0.00%
0.75-0.80	2	50.00%
0.80-0.85	0	0.00%
0.90-0.95	0	0.00%
1.05-1.10	0	0.00%
1.10-1.15	0	0.00%
1.20-1.25	0	0.00%

Foundations

Sa @ T = 1.0 s (g)	Number of Buildings Damaged	Percentage of Buildings Damaged
0.05-0.10	0	0.00%
0.10-0.15	0	0.00%
0.15-0.20	2	0.06%
0.20-0.25	1	0.09%
0.25-0.30	2	0.33%
0.30-0.35	1	2.38%
0.35-0.40	1	1.56%
0.40-0.45	1	10.00%
0.45-0.50	0	0.00%
0.60-0.65	0	0.00%
0.75-0.80	0	0.00%
Totals	8	0.14%

Foundations

PGA (g)	Number of Buildings Damaged	Percentage of Buildings Damaged
0.05-0.10	0	0.00%
0.15-0.20	1	0.11%
0.20-0.25	2	0.06%
0.25-0.30	2	0.20%
0.30-0.35	0	0.00%
0.35-0.40	2	3.03%
0.40-0.45	1	1.28%
0.45-0.50	0	0.00%
0.50-0.55	0	0.00%

Roof/Floor (Vertical Loads)

Sa @ T=0.3 s (g)	Number of Buildings Damaged	Percentage of Buildings Damaged
0.30-0.35	0	0.00%
0.40-0.45	7	0.45%
0.45-0.50	34	1.34%
0.50-0.55	16	1.57%
0.55-0.60	4	2.96%
0.60-0.65	0	0.00%
0.70-0.75	0	0.00%
0.75-0.80	2	50.00%
0.80-0.85	0	0.00%
0.90-0.95	1	5.26%
1.05-1.10	0	0.00%
1.10-1.15	0	0.00%
1.20-1.25	0	0.00%

Roof/Floor (Vertical Loads)

Sa @ T = 1.0 s (g)	Number of Buildings Damaged	Percentage of Buildings Damaged
0.05-0.10	0	0.00%
0.10-0.15	0	0.00%
0.15-0.20	22	0.63%
0.20-0.25	23	2.12%
0.25-0.30	14	2.32%
0.30-0.35	1	2.38%
0.35-0.40	2	3.13%
0.40-0.45	1	10.00%
0.45-0.50	0	0.00%
0.60-0.65	1	25.00%
0.75-0.80	0	0.00%
Totals	64	1.12%

Roof/Floor (Vertical Loads)

PGA (g)	Number of Buildings Damaged	Percentage of Buildings Damaged
0.05-0.10	0	0.00%
0.15-0.20	9	1.01%
0.20-0.25	23	0.71%
0.25-0.30	25	2.54%
0.30-0.35	2	0.88%
0.35-0.40	3	4.55%
0.40-0.45	2	2.56%
0.45-0.50	0	0.00%
0.50-0.55	0	0.00%

Table A-21: Summary of Structural Damage Types vs. Ground Motion (cont.)

Columns/Pilasters/Corbels

Sa @ T=0.3 s (g)	Number of Buildings Damaged	Percentage of Buildings Damaged
0.30-0.35	0	0.00%
0.40-0.45	4	0.26%
0.45-0.50	53	2.09%
0.50-0.55	17	1.67%
0.55-0.60	6	4.44%
0.60-0.65	0	0.00%
0.70-0.75	0	0.00%
0.75-0.80	3	75.00%
0.80-0.85	4	4.88%
0.90-0.95	0	0.00%
1.05-1.10	0	0.00%
1.10-1.15	0	0.00%
1.20-1.25	0	0.00%

Columns/Pilasters/Corbels

Sa @ T = 1.0 s (g)	Number of Buildings Damaged	Percentage of Buildings Damaged
0.05-0.10	0	0.00%
0.10-0.15	0	0.00%
0.15-0.20	31	0.89%
0.20-0.25	27	2.49%
0.25-0.30	20	3.32%
0.30-0.35	1	2.38%
0.35-0.40	4	6.25%
0.40-0.45	2	20.00%
0.45-0.50	2	3.08%
0.60-0.65	0	0.00%
0.75-0.80	0	0.00%
Totals	87	1.52%

Columns/Pilasters/Corbels

PGA (g)	Number of Buildings Damaged	Percentage of Buildings Damaged
0.05-0.10	0	0.00%
0.15-0.20	14	1.57%
0.20-0.25	28	0.86%
0.25-0.30	32	3.25%
0.30-0.35	5	2.19%
0.35-0.40	4	6.06%
0.40-0.45	4	5.13%
0.45-0.50	0	0.00%
0.50-0.55	0	0.00%

Diaphragms / Horizontal Bracing

Sa @ T=0.3 s (g)	Number of Buildings Damaged	Percentage of Buildings Damaged
0.30-0.35	0	0.00%
0.40-0.45	3	0.19%
0.45-0.50	31	1.22%
0.50-0.55	9	0.88%
0.55-0.60	1	0.74%
0.60-0.65	0	0.00%
0.70-0.75	0	0.00%
0.75-0.80	2	50.00%
0.80-0.85	1	1.22%
0.90-0.95	0	0.00%
1.05-1.10	0	0.00%
1.10-1.15	0	0.00%
1.20-1.25	0	0.00%

Diaphragms / Horizontal Bracing

Sa @ T = 1.0 s (g)	Number of Buildings Damaged	Percentage of Buildings Damaged
0.05-0.10	0	0.00%
0.10-0.15	0	0.00%
0.15-0.20	15	0.43%
0.20-0.25	15	1.38%
0.25-0.30	13	2.16%
0.30-0.35	1	2.38%
0.35-0.40	1	1.56%
0.40-0.45	1	10.00%
0.45-0.50	1	1.54%
0.60-0.65	0	0.00%
0.75-0.80	0	0.00%
Totals	47	0.82%

Diaphragms / Horizontal Bracing

PGA (g)	Number of Buildings Damaged	Percentage of Buildings Damaged
0.05-0.10	0	0.00%
0.15-0.20	8	0.90%
0.20-0.25	13	0.40%
0.25-0.30	21	2.13%
0.30-0.35	1	0.44%
0.35-0.40	2	3.03%
0.40-0.45	2	2.56%
0.45-0.50	0	0.00%
0.50-0.55	0	0.00%

Table A-21: Summary of Structural Damage Types vs. Ground Motion (cont.)

Walls/Vertical Bracing

Sa @ T=0.3 s (g)	Number of Buildings Damaged	Percentage of Buildings Damaged
0.30-0.35	1	0.68%
0.40-0.45	30	1.93%
0.45-0.50	119	4.70%
0.50-0.55	44	4.33%
0.55-0.60	12	8.89%
0.60-0.65	0	0.00%
0.70-0.75	1	20.00%
0.75-0.80	3	75.00%
0.80-0.85	15	18.29%
0.90-0.95	2	10.53%
1.05-1.10	0	0.00%
1.10-1.15	0	0.00%
1.20-1.25	0	0.00%

Walls/Vertical Bracing

Sa @ T = 1.0 s (g)	Number of Buildings Damaged	Percentage of Buildings Damaged
0.05-0.10	1	0.78%
0.10-0.15	0	0.00%
0.15-0.20	97	2.78%
0.20-0.25	63	5.81%
0.25-0.30	40	6.63%
0.30-0.35	1	2.38%
0.35-0.40	13	20.31%
0.40-0.45	3	30.00%
0.45-0.50	9	13.85%
0.60-0.65	0	0.00%
0.75-0.80	0	0.00%
Totals	227	3.98%

Walls/Vertical Bracing

PGA (g)	Number of Buildings Damaged	Percentage of Buildings Damaged
0.05-0.10	1	0.78%
0.15-0.20	31	3.49%
0.20-0.25	93	2.85%
0.25-0.30	68	6.90%
0.30-0.35	12	5.26%
0.35-0.40	10	15.15%
0.40-0.45	12	15.38%
0.45-0.50	0	0.00%
0.50-0.55	0	0.00%

Table A-22: Summary of Nonstructural Damage Types vs. Ground Motion

Interior Walls/Partitions

Sa @ T=0.3 s (g)	Number of Buildings Damaged	Percentage of Buildings Damaged
0.30-0.35	1	0.68%
0.40-0.45	30	1.93%
0.45-0.50	125	4.94%
0.50-0.55	35	3.44%
0.55-0.60	6	4.44%
0.60-0.65	0	0.00%
0.70-0.75	0	0.00%
0.75-0.80	1	25.00%
0.80-0.85	4	4.88%
0.90-0.95	0	0.00%
1.05-1.10	0	0.00%
1.10-1.15	1	8.33%
1.20-1.25	0	0.00%

Interior Walls/Partitions

Sa @ T = 1.0 s (g)	Number of Buildings Damaged	Percentage of Buildings Damaged
0.05-0.10	1	0.78%
0.10-0.15	0	0.00%
0.15-0.20	113	3.24%
0.20-0.25	56	5.35%
0.25-0.30	21	3.48%
0.30-0.35	2	4.76%
0.35-0.40	5	7.81%
0.40-0.45	0	0.00%
0.45-0.50	2	3.08%
0.60-0.65	1	25.00%
0.75-0.80	0	0.00%
Totals	203	3.55%

Interior Walls/Partitions

PGA (g)	Number of Buildings Damaged	Percentage of Buildings Damaged
0.05-0.10	1	0.78%
0.15-0.20	23	2.59%
0.20-0.25	120	3.68%
0.25-0.30	45	4.56%
0.30-0.35	5	2.19%
0.35-0.40	6	9.09%
0.40-0.45	2	2.56%
0.45-0.50	1	10.00%
0.50-0.55	0	0.00%

Chimney

Sa @ T=0.3 s (g)	Number of Buildings Damaged	Percentage of Buildings Damaged
0.30-0.35	0	0.00%
0.40-0.45	0	0.00%
0.45-0.50	6	0.24%
0.50-0.55	0	0.00%
0.55-0.60	1	0.74%
0.60-0.65	0	0.00%
0.70-0.75	0	0.00%
0.75-0.80	1	25.00%
0.80-0.85	2	2.44%
0.90-0.95	0	0.00%
1.05-1.10	0	0.00%
1.10-1.15	0	0.00%
1.20-1.25	0	0.00%

Chimney

Sa @ T = 1.0 s (g)	Number of Buildings Damaged	Percentage of Buildings Damaged
0.05-0.10	0	0.00%
0.10-0.15	0	0.00%
0.15-0.20	2	0.06%
0.20-0.25	1	0.09%
0.25-0.30	3	0.50%
0.30-0.35	1	2.38%
0.35-0.40	2	3.13%
0.40-0.45	0	0.00%
0.45-0.50	1	1.54%
0.60-0.65	0	0.00%
0.75-0.80	0	0.00%
Totals	10	0.20%

Chimney

PGA (g)	Number of Buildings Damaged	Percentage of Buildings Damaged
0.05-0.10	0	0.00%
0.15-0.20	0	0.00%
0.20-0.25	3	0.09%
0.25-0.30	3	0.30%
0.30-0.35	1	0.44%
0.35-0.40	2	3.03%
0.40-0.45	1	1.28%
0.45-0.50	0	0.00%
0.50-0.55	0	0.00%

Table A-22: Summary of Nonstructural Damage Types vs. Ground Motion (cont.)

Parapets/Ornamentation

Sa @ T=0.3 s (g)	Number of Buildings Damaged	Percentage of Buildings Damaged
0.30-0.35	0	0.00%
0.40-0.45	18	1.16%
0.45-0.50	77	3.04%
0.50-0.55	30	2.95%
0.55-0.60	9	6.67%
0.60-0.65	0	0.00%
0.70-0.75	0	0.00%
0.75-0.80	2	50.00%
0.80-0.85	5	6.10%
0.90-0.95	3	15.79%
1.05-1.10	0	0.00%
1.10-1.15	0	0.00%
1.20-1.25	0	0.00%

Parapets/Ornamentation

Sa @ T = 1.0 s (g)	Number of Buildings Damaged	Percentage of Buildings Damaged
0.05-0.10	0	0.00%
0.10-0.15	0	0.00%
0.15-0.20	51	1.46%
0.20-0.25	46	4.24%
0.25-0.30	31	5.14%
0.30-0.35	1	2.38%
0.35-0.40	9	14.06%
0.40-0.45	1	10.00%
0.45-0.50	5	7.69%
0.60-0.65	0	0.00%
0.75-0.80	0	0.00%
Totals	144	2.60%

Parapets/Ornamentation

PGA (g)	Number of Buildings Damaged	Percentage of Buildings Damaged
0.05-0.10	0	0.00%
0.15-0.20	18	2.02%
0.20-0.25	50	1.53%
0.25-0.30	56	5.68%
0.30-0.35	6	2.63%
0.35-0.40	8	12.12%
0.40-0.45	6	7.69%
0.45-0.50	0	0.00%
0.50-0.55	0	0.00%

Cladding/Glazing

Sa @ T=0.3 s (g)	Number of Buildings Damaged	Percentage of Buildings Damaged
0.30-0.35	0	0.00%
0.40-0.45	9	0.58%
0.45-0.50	59	2.33%
0.50-0.55	29	2.85%
0.55-0.60	5	3.70%
0.60-0.65	0	0.00%
0.70-0.75	1	20.00%
0.75-0.80	1	25.00%
0.80-0.85	6	7.32%
0.90-0.95	2	10.53%
1.05-1.10	0	0.00%
1.10-1.15	1	8.33%
1.20-1.25	0	0.00%

Cladding/Glazing

Sa @ T = 1.0 s (g)	Number of Buildings Damaged	Percentage of Buildings Damaged
0.05-0.10	0	0.00%
0.10-0.15	0	0.00%
0.15-0.20	41	1.18%
0.20-0.25	34	3.13%
0.25-0.30	24	3.98%
0.30-0.35	1	2.38%
0.35-0.40	8	12.50%
0.40-0.45	1	10.00%
0.45-0.50	3	4.62%
0.60-0.65	1	25.00%
0.75-0.80	0	0.00%
Totals	113	1.98%

Cladding/Glazing

PGA (g)	Number of Buildings Damaged	Percentage of Buildings Damaged
0.05-0.10	0	0.00%
0.15-0.20	13	1.46%
0.20-0.25	45	1.38%
0.25-0.30	38	3.85%
0.30-0.35	4	1.75%
0.35-0.40	8	12.12%
0.40-0.45	4	5.13%
0.45-0.50	1	10.00%
0.50-0.55	0	0.00%

Table A-23: Summary of Structural Damage Descriptions vs. Ground Motion

Wall Cracking

Sa @ T=0.3 s (g)	Number of Buildings Damaged	Percentage of Buildings Damaged
0.30-0.35	1	0.68%
0.40-0.45	39	2.51%
0.45-0.50	102	4.03%
0.50-0.55	67	6.59%
0.55-0.60	8	5.93%
0.60-0.65	1	14.29%
0.70-0.75	0	0.00%
0.75-0.80	0	0.00%
0.80-0.85	11	13.41%
0.90-0.95	0	0.00%
1.05-1.10	1	9.09%
1.10-1.15	1	8.33%
1.20-1.25	1	12.50%

Wall Cracking

Sa @ T = 1.0 s (g)	Number of Buildings Damaged	Percentage of Buildings Damaged
0.05-0.10	1	0.78%
0.10-0.15	0	0.00%
0.15-0.20	105	3.01%
0.20-0.25	66	6.08%
0.25-0.30	40	6.63%
0.30-0.35	2	4.76%
0.35-0.40	8	12.50%
0.40-0.45	0	0.00%
0.45-0.50	8	12.31%
0.60-0.65	1	25.00%
0.75-0.80	1	16.67%
Totals	232	4.1%

Wall Cracking

PGA (g)	Number of Buildings Damaged	Percentage of Buildings Damaged
0.05-0.10	1	0.78%
0.15-0.20	17	1.91%
0.20-0.25	113	3.47%
0.25-0.30	65	6.59%
0.30-0.35	20	8.77%
0.35-0.40	2	3.03%
0.40-0.45	11	14.10%
0.45-0.50	2	20.00%
0.50-0.55	1	4.76%

Diaphragm to Wall Ties

Sa @ T=0.3 s (g)	Number of Buildings Damaged	Percentage of Buildings Damaged
0.30-0.35	0	0.00%
0.40-0.45	1	0.06%
0.45-0.50	3	0.12%
0.50-0.55	1	0.10%
0.55-0.60	0	0.00%
0.60-0.65	0	0.00%
0.70-0.75	0	0.00%
0.75-0.80	0	0.00%
0.80-0.85	3	3.66%
0.90-0.95	1	5.26%
1.05-1.10	0	0.00%
1.10-1.15	0	0.00%
1.20-1.25	0	0.00%

Diaphragm to Wall Ties

Sa @ T = 1.0 s (g)	Number of Buildings Damaged	Percentage of Buildings Damaged
0.05-0.10	0	0.00%
0.10-0.15	0	0.00%
0.15-0.20	3	0.09%
0.20-0.25	1	0.09%
0.25-0.30	0	0.00%
0.30-0.35	0	0.00%
0.35-0.40	2	3.13%
0.40-0.45	0	0.00%
0.45-0.50	2	3.08%
0.60-0.65	1	25.00%
0.75-0.80	0	0.00%
Totals	9	0.2%

Diaphragm to Wall Ties

PGA (g)	Number of Buildings Damaged	Percentage of Buildings Damaged
0.05-0.10	0	0.00%
0.15-0.20	0	0.00%
0.20-0.25	2	0.06%
0.25-0.30	2	0.20%
0.30-0.35	2	0.88%
0.35-0.40	0	0.00%
0.40-0.45	3	3.85%
0.45-0.50	0	0.00%
0.50-0.55	0	0.00%

Table A-23: Summary of Structural Damage Descriptions vs. Ground Motion (cont.)

Shear Cracks

Sa @ T=0.3 s (g)	Number of Buildings Damaged	Percentage of Buildings Damaged
0.30-0.35	1	0.68%
0.40-0.45	6	0.39%
0.45-0.50	22	0.87%
0.50-0.55	6	0.79%
0.55-0.60	2	1.48%
0.60-0.65	0	0.00%
0.70-0.75	1	20.00%
0.75-0.80	1	25.00%
0.80-0.85	0	0.00%
0.90-0.95	0	0.00%
1.05-1.10	0	0.00%
1.10-1.15	0	0.00%
1.20-1.25	0	0.00%

Shear Cracks

Sa @ T = 1.0 s (g)	Number of Buildings Damaged	Percentage of Buildings Damaged
0.05-0.10	1	0.78%
0.10-0.15	0	0.00%
0.15-0.20	17	0.49%
0.20-0.25	14	1.29%
0.25-0.30	7	1.16%
0.30-0.35	0	0.00%
0.35-0.40	2	3.13%
0.40-0.45	0	0.00%
0.45-0.50	0	0.00%
0.60-0.65	0	0.00%
0.75-0.80	0	0.00%
Totals	41	0.7%

Shear Cracks

PGA (g)	Number of Buildings Damaged	Percentage of Buildings Damaged
0.05-0.10	1	0.78%
0.15-0.20	3	0.34%
0.20-0.25	23	0.71%
0.25-0.30	12	1.22%
0.30-0.35	0	0.00%
0.35-0.40	2	3.03%
0.40-0.45	0	0.00%
0.45-0.50	0	0.00%
0.50-0.55	0	0.00%

Veneer

Sa @ T=0.3 s (g)	Number of Buildings Damaged	Percentage of Buildings Damaged
0.30-0.35	0	0.00%
0.40-0.45	2	0.13%
0.45-0.50	7	0.28%
0.50-0.55	1	0.10%
0.55-0.60	0	0.00%
0.60-0.65	0	0.00%
0.70-0.75	0	0.00%
0.75-0.80	0	0.00%
0.80-0.85	0	0.00%
0.90-0.95	1	5.26%
1.05-1.10	0	0.00%
1.10-1.15	0	0.00%
1.20-1.25	0	0.00%

Veneer

Sa @ T = 1.0 s (g)	Number of Buildings Damaged	Percentage of Buildings Damaged
0.05-0.10	0	0.00%
0.10-0.15	0	0.00%
0.15-0.20	6	0.17%
0.20-0.25	2	0.18%
0.25-0.30	2	0.33%
0.30-0.35	0	0.00%
0.35-0.40	0	0.00%
0.40-0.45	0	0.00%
0.45-0.50	1	1.54%
0.60-0.65	0	0.00%
0.75-0.80	0	0.00%
Totals	11	0.2%

Veneer

PGA (g)	Number of Buildings Damaged	Percentage of Buildings Damaged
0.05-0.10	0	0.00%
0.15-0.20	1	0.11%
0.20-0.25	6	0.18%
0.25-0.30	3	0.30%
0.30-0.35	0	0.00%
0.35-0.40	0	0.00%
0.40-0.45	1	1.27%
0.45-0.50	0	0.00%
0.50-0.55	0	0.00%

Table A-23: Summary of Structural Damage Descriptions vs. Ground Motion (cont.)

Corner Damage

Sa @ T=0.3 s (g)	Number of Buildings Damaged	Percentage of Buildings Damaged
0.30-0.35	0	0.00%
0.40-0.45	11	0.71%
0.45-0.50	30	1.16%
0.50-0.55	9	0.88%
0.55-0.60	6	4.44%
0.60-0.65	1	14.29%
0.70-0.75	0	0.00%
0.75-0.80	2	50.00%
0.80-0.85	1	1.22%
0.90-0.95	0	0.00%
1.05-1.10	0	0.00%
1.10-1.15	0	0.00%
1.20-1.25	0	0.00%

Corner Damage

Sa @ T = 1.0 s (g)	Number of Buildings Damaged	Percentage of Buildings Damaged
0.05-0.10	0	0.00%
0.10-0.15	0	0.00%
0.15-0.20	21	0.60%
0.20-0.25	18	1.66%
0.25-0.30	13	2.16%
0.30-0.35	2	4.76%
0.35-0.40	4	6.25%
0.40-0.45	1	10.00%
0.45-0.50	1	1.54%
0.60-0.65	0	0.00%
0.75-0.80	0	0.00%
Totals	60	1.1%

Corner Damage

PGA (g)	Number of Buildings Damaged	Percentage of Buildings Damaged
0.05-0.10	0	0.00%
0.15-0.20	8	0.92%
0.20-0.25	19	0.58%
0.25-0.30	21	2.11%
0.30-0.35	5	2.17%
0.35-0.40	5	7.35%
0.40-0.45	2	2.53%
0.45-0.50	0	0.00%
0.50-0.55	0	0.00%

Table A-24: Damage Summary (as Percentage of Total Inventory)

General Damage	**Percent Damaged**
Collapse/Partial Collapse	1.5
Building or Story Leaning	1.2
Other Hazardous Condition	2.3
Structural Damage Types	**Percent Damaged**
Foundation damage	0.1
Roof/Floor (Vertical Loads)	1.1
Pilaster, column, or corbel (Total count in inventory is unknown)	1.5 (Lower bound)
Diaphragms/Horizontal Bracing	0.8
Walls/Vertical Bracing	4.0
Nonstructural Damage Types	**Percent Damaged**
Parapets/Ornamentation (Total count in inventory is unknown)	2.5 (Lower bound)
Cladding/Glazing (Total count in inventory is unknown)	2.0 (Lower bound)
Interior Walls/Partitions (Total count in inventory unknown)	3.6 (Lower bound)
Chimneys (Total count of chimney inventory unknown)	0.2 (Lower bound)
General Damage Description Types	**Percent Damage**
Wall Cracking	4.1
Cracks Described as "Shear Cracks"	0.7
Corner Damage	1.1

Table A-25: Summary of Relationship Between Ground Motion Intensity Contours and Damage

Damage Type	Acceleration when a Sharp Increase in Damage Occurs, g (Jump)			Acceleration when 1% of the Total Inventory is Damaged, g (One Percent)		
	$S_a(0.3)$	$S_a(1.0)$	PGA	$S_a(0.3)$	$S_a(1.0)$	PGA
Building or Story Leaning	0.75-0.80	0.35-0.40	0.35-0.40	0.40-0.45	0.20-0.25	0.15-0.20
Foundations	0.75-0.80	0.40-0.45	0.35-0.40	0.55-0.60	0.30-0.35	0.35-0.40
Roof/Floors (Vertical Loads)	0.75-0.80	0.40-0.45	0.35-0.40	0.45-0.50	0.20-0.25	0.15-0.20
Columns/Pilasters/ Corbels	0.75-0.80	0.40-0.45	0.35-0.40	0.45-0.50	0.20-0.25	0.15-0.20
Diaphragms/Horizontal Bracing	0.75-0.80	0.40-0.45	0.35-0.40	0.45-0.50	0.20-0.25	0.20-0.25
Walls/Vertical Bracing (Cracking)	0.55-0.60	0.25-0.30	0.25-0.30	0.40-0.45	0.15-0.20	0.15-0.20
Cladding/Glazing	0.70-0.75	0.35-0.40	0.35-0.40	0.40-0.45	0.15-0.20	0.15-0.20
Wall Cracking (as reported in the narrative damage descriptions) narrative)	0.60-0.65	0.20-0.25	0.25-0.30	0.40-0.45	0.15-0.20	0.15-0.20
Shear Cracks (narrative)	0.55-0.60	0.35-0.40	0.35-0.40	0.55-0.60	0.20-0.25	0.25-0.30
Corner Damage (narrative)	0.75-0.80	0.40-0.45	0.35-0.40	0.45-0.50	0.20-0.25	0.15-0.20

The PGA at "jump" for walls/vertical bracing and wall cracking is 0.25-0.30g—lower than the other elements. The spectral acceleration values are also lower. It is possible that wall cracking, since it is so easily observed, was reported more frequently and with more consistency than the other types of damage, hence, the lower values. Further, the ATC-20 (1989) instructions state that wall cracking is to be reported only if it jeopardizes the vertical support of floor or roof framing—not just if cracking was observed. Perusal of the data gives a general impression that cracking was reported *if it was observed*, not *only if it jeopardized the vertical load-carrying system*. Thus, the evaluation for wall cracking is likely skewed towards greater damage. Nevertheless, wall cracking is indicated for 4.0% of the inventory; it appears to be a likely candidate for enhanced performance efforts.

- Veneer damage was reported for only 11 buildings and diaphragm-to-wall tie failure for only 9—not enough to evaluate statistically.

Damage Data from Other Sources

Damage data for other cities in the areas of strong ground motion is limited. SSC (1994) notes that in Glendale, 17 out of 267 retrofitted URM buildings were red-tagged and in Burbank one out of 16 was red-tagged. The number of yellow tags in each city is unknown.

SSC (1994) provides the following description of retrofitting history and Northridge damage to URM building in Santa Monica.

> *The City of Santa Monica has a long history of different hazard mitigation programs to address its URM bearing wall building inventory....In 1915 and 1921, Santa Monica adopted some unusual legislation requiring more stringent wall-to-diaphragm ties that other communities around the state. In 1978, about 249 buildings were identified which failed to comply with the 1933 Riley Act. In 1981, wall anchors were required for the 27 pre-1915 buildings. Work was to be completed by 1985 and needed to conform only to the 1915 and 1921 requirements. In 1986, SB547 was passed, and in a 4/11/89 response, Santa Monica adopted Ordinance 1489 which required engineer's reports on capacity of the URM lateral system, but no mandatory strengthening. Nonetheless, through 1991, about 95 of the original 249 buildings had been strengthened (to unknown requirements) and 23 demolished. After a protracted study, an EIR, and much public debate, the city adopted the 1991 UCBC for the remaining 131 buildings. Approximately 32 of these buildings had been strengthened by the time of the Northridge Earthquake and 4 more had been demolished. Thus, Santa Monica presents an interesting laboratory for observing the effectiveness of various seismic strengthening measures.*
>
> *The roughly east-west oriented I-10 freeway provides a simple geographic dividing line between the largely undamaged portion of the city to the south and the more significantly damaged portion to the north. Interestingly, the two areas are a similar distance from the epicenter, buildings do not appear to be significantly different in either area, the soil is said to be similar (although the water table may vary), and yet there are dramatic differences in the damage observed to URM buildings. This observation is reflected in the extent of ATC-20 tagging which had been performed by 1/27/94. A total of 61 of 152 URM bearing wall buildings were tagged in the northern portion of the city and only 4 of 70 in the southern portion. [Table A-26] summarizes the ATC-20 status for the 61 tagged buildings.*
>
> *It is interesting to compare the distribution of percentages in [Table A-26] to tagging records for the entire Santa Monica building stock. As of 1/27/94, there were 2348 total tags with 78% green, 15% yellow, and 5.6% red, a slightly better*

distribution than the results in [Table A-26] for URM buildings strengthened after 1991. These numbers are very similar to the results for all building types in Los Angeles County, where by 2/2/94 there were 67,497 tags with 77% green, 16% yellow, and 7% red.

Table A-26: ATC-20 Tagging vs. Strengthening Status for URM Bearing Wall Buildings in Santa Monica (SSC, 1994)

Strengthening Status	Total Tags		Green Tags		Yellow Tags		Red Tags	
	Number	Percent	Number	Percent	Number	Percent	Number	Percent
Unstrengthened	22	100	5	23	10	45	9	32
Pre-1991 Strengthening	28	100	12	43	9	32	7	23
Strengthening in 1991 or Later	9	100	6	67	2	22	1	11
Totals	61	100	23	38	21	34	17	28

Recommendations Made By Others Regarding Damaged URM Buildings

As noted in SSC (1994):

Hamburger and Mark (1994) summarizes some of the observations made by engineering reviews of damage to URM retrofitted buildings. They identify the attributing factors for the failures of retrofitted URM buildings as:

1. *Weak mortar. It is estimated most of the failed buildings had mortar with an ultimate shear strength of 40 psi or less.*

2. *Unbonded veneer courses.*

3. *Thin walls. Nine-inch walls in particular were observed to perform poorly, especially in the upper stories.*

4. *Poor detailing practice. In one observed case, steeply inclined diagonal "kicker" braces were used to bolster the midheight of a wall with an inadequate h/t for out-of-plane forces. Apparently these kicker braces, which were installed at an angle of approximately 60° with respect to vertical were unable to brace the wall adequately and failure occurred.*

5. *Steel lintels narrowing the effective shear width of the piers between the windows.*

6. *Plan irregularities.*

7. *Commercial buildings with large open spaces.*

8. *Partial retrofits.*

LATF (1994) summarizes the field observations and recommendations on proposed changes to Division 88 made by the Los Angeles Task Force committee which studied damage to URM buildings in the City of Los Angeles. They observed that:

1. *There was evidence of low mortar strength in most of the damaged buildings....*
2. *There were many failures in nine-inch thick walls. In some cases, these walls were covered with veneer, and in other cases, these walls were located in the upper story of multi-story buildings....*
3. *There were several failures noted in veneer....*
4. *There was a higher percentage of buildings with certain plan irregularities, such as "U"-, "L"- and "H"-shaped buildings, that had damage than rectangular-shaped buildings....*
5. *Corners of buildings were typical areas of damage....*
6. *Non-bearing URM walls were typical areas of damage....*
7. *The URM walls adjacent to openings in-filled with reinforced masonry were typical areas of damage....*
8. *Damage was seen in areas where the existing mortar was deteriorated due to extensive exposure to water, such as in the parapets, under window sills and adjacent to alleys....*
9. *In some of the failures, wall anchors were installed at the roof level but not at the ceiling level or vice versa. This was also the case where there were wall anchors installed at the h/t wall braces but not at the adjacent roof, floor or ceiling diaphragm....*
10. *Some of the buildings that were damaged in past earthquakes and repaired exhibited more damage than buildings that were not damaged in previous earthquakes....*
11. *There was a problem in some gunite panels pulling away from the URM walls. In addition, damage was observed adjacent to gunite shear panels that were not continuous from the foundation to the roof....*
12. *There was severe damage noted above some buildings with narrow piers....*

The excerpts below are taken from LATF (1994) and cover the changes proposed to the Los Angeles Building Code, discussion of the proposals and the final recommendations made by the Task Force:

1. *[Proposal:] Reduce the allowable shear values for low mortar test values. Discussion: A proposal was made to limit the allowable shear values to eight percent of the test values for mortar strengths between thirty psi and forty-five psi in lieu of the ten percent currently allowed for all mortar strengths. The study group felt that the shear values reported were not accurate and may have been substantially higher than the actual shear values of the mortar. Recommendation: It is recommended that this code provision not be changed.*

2. *[Proposal:] Revise the push test to include the use of a micrometer to determine first movement.*
   Discussion: The current method of testing the mortar strength requires the technician to observe first movement of the brick. The first movement is usually in the range of five thousands of an inch to two hundreds of an inch (0.005" to 0.020"). The use of a micrometer will help determine this amount of movement. Recommendation: It is recommended that the guidelines for in-place masonry shear tests, as shown in Appendix A of [the Task Force's report], be adopted.

3. *[Proposal:] Require specific considerations for the use of diagonal wall braces. Discussion: There was some damage noted in buildings containing wall braces that were at an angle to the wall. There was no consensus as to the cause of the problem. Some members suggested that the problem was due to the braces being too stiff while others felt that the problem was that the braces were too flexible. Damage from diagonal wall braces did not appear to be a widespread problem. Recommendation: It is recommended that the code not be changed until specific design considerations are developed.*

4. *[Proposal:] Require specific considerations for plan considerations.*
   Discussion: There appeared to be more damage to buildings with plan irregularities that regular shaped buildings. These plan irregularities include buildings with hammerhead-, "U"- and "L"-shaped configurations.
   Recommendation: It is recommended that the code not be changed, but that the design engineer use the special considerations for plan irregularities found in the Uniform Building Code.

5. *[Proposal:] Develop special requirements for building corners.*
   Discussion: There was a major amount of damage observed at the corners of buildings. The study group is still exploring ways to avoid this problem
   Recommendation: It is recommended that this item be studied further.

6. *[Proposal:] Require special considerations for the rocking of piers and design consideration relative to the existence of steel lintels over openings.*

Discussion: There was some disagreement as to what the actual problems and possible solutions are for these piers. There is concern that the steel lintels reduce the shear capacity of the pier.
Recommendation: It is recommended that this item be studied further.

7. *[Proposal:] Revise the method of pointing.*
   Discussion: There were several opinions regarding the structural advantages of pointing. Some felt that pointing greatly improves the strength of the walls, while others questioned the value of this procedure. In any case, there are improvements that can be made to the method currently being followed.
   Recommendation: It is recommended that the guidelines for pointing of URM walls, contained in Appendix B of [the Task Force's report], be adopted.

8. *[Proposal:] Require special considerations for the use of steel frames at open store fronts.*
   Discussion: There were problems observed with excessive deflections ... of frames at store fronts.
   Recommendation: It is recommended that the code not be changed. The subcommittee felt that the code adequately provides for deflection compatibility.

9. *[Proposal:] Require existing veneer ties to be checked for adequacy to support veneer.*
   Discussion: There were some buildings that had existing veneer ties that failed to support veneer. In some cases, these ties are deteriorated and inadequate to provide support of the veneer.
   Recommendation: It is recommended that the code be changed to require the testing of existing veneer ties.

10. *[Proposal:] Reduce the h/t ratios and increase the required mortar strength for non-bearing walls.*
    Discussion: There were some cases of failure in non-bearing walls. The subcommittee was not in agreement that the problem was with the h/t ratios and the mortar strength.
    Recommendation: It is recommended that the code not be changed in regard to non-bearing walls.

A.4 1987 Whittier Narrows Earthquake Data

The main shock of the Whittier Earthquake struck at 4:42 a.m. Pacific Standard Time on October 1, 1987. The main shock ruptured along a previously unrecognized thrust fault located just to the north of the Whittier Narrows at depths between 11 and 16 km (Hauksson et al., 1988). The epicenter was centered about 15 km northeast of downtown Los Angeles. The magnitude was estimated to be $M_L = 5.9$.

When the Whittier earthquake struck, Los Angeles was in the midst of the Division 88 (1985) program. The Division 88 program was phased over a period of several years, so that at the time of the earthquake, the status of the building inventory included the following categories: previously demolished, unstrengthened, those with only tension anchorage between the floors and walls completed (full strengthening was required at a later date), other forms of partial strengthening, and full Division 88 compliance. Though this M_L=5.9 event was of moderate size (most of Los Angeles was in the MMI=VI and VII area), a significant amount of data was obtained (SSC, 1994).

In the weeks following the earthquake, LADBS engineer/inspector teams surveyed 2431 buildings in the hardest hit areas. They reported 1633 buildings not damaged; 676 damaged, but still functional; and 122 vacated or partly vacated. Damage (ranging from nonstructural cracking to wall collapse) was reported to over 36% of the unstrengthened or partially strengthened buildings and 21% of the strengthened buildings. Further, for every strengthened building vacated there were almost 3 unstrengthened buildings vacated. Deppe (1988) summarizes information collected on these 2431 URM bearing wall buildings located in the most heavily affected areas. This figure represents approximately one third of the 7300 URM buildings remaining in Los Angeles at that time. Table A-27 provides the results of the survey for 2408 buildings.

Table A-27: Damage to URM Buildings in the Whittier Earthquake (Deppe, 1988)

Status	Buildings Surveyed	Buildings Damaged		Buildings Vacated	
	Number	Number	Percentage	Number	Percentage
U/PS[1]					
Residential	430	200	47	47	10.9
Commercial	1541	381	25	66	4.3
Total	1971	581	30	113	5.7
Strengthened[1]					
Residential	73	16	22	3	4.1
Commercial	364	69	19	6	1.6
Total	437	85	20	9	2.1

[1]"U/PS" categories include unstrengthened buildings, buildings with tension anchors only and other unspecified types of partial strengthening. "Strengthened" indicates presumed compliance with Division 88.

[2]"Damage" was defined as anything from non-structural cracking in plastered ceilings to the most major structural failure—an unreinforced masonry wall collapse.

Deppe (1988) reports that at the time of the Whittier Narrows Earthquake approximately 1100 buildings had been fully strengthened to Division 88 requirements, 1700 buildings had been issued a permit (and in some cases commenced) strengthening, 700 had been demolished, and 4500 were unstrengthened and with no permit. By way of comparison, based on compliance dates given in the Division 88 Database retrieved for this project, approximately 800 buildings had been fully strengthened to Division 88 (1985) requirements at the time of the Whittier Earthquake.

A database of building damage information for this event was supplied by the J.H. Wiggins Company. It contains 2549 building records, each with fields including address, latitude/longitude, several damage ratings, soil types, and MMI estimates. Of the 2549 buildings identified in the database, 219 were partially strengthened and 477 were fully strengthened to Division 88 (1985) requirements. This database was reduced to a format consistent with the current City of Los Angeles databases. The Whittier Database is more limited than the various Northridge Earthquake databases; the damage fields are all general descriptions or ratings and, thus, do not specify the presence (or absence) of element-by-element damage for each building in the database in a consistent manner. Figure A-22 shows the damaged and undamaged fully strengthened buildings.

The MMI information contained in this database was developed by the J.H. Wiggins Company. The MMI is based not on a digitized version of the actual USGS map, but on Wiggins-developed attenuation relationships which consider soil effects, magnitude, and distance from the building from the fault. The building data has been geographically correlated with the MMI contours.

Damage for the entire body of data is presented in Table A-28 grouped by ATC-13 (1985) damage states. The average reduction in damage to strengthened buildings is about 50%. Further, as shown in Table A-27, unstrengthened or partially strengthened residential buildings were 2.1 times as likely to be damaged as strengthened ones. The same ratio for commercial buildings is 1.3.

The principal modes of failure noted by Wiggins for both strengthened and unstrengthened buildings were:

1. Out-of-plane wall movement and partial wall collapse.
2. Wall separation from floors and roofs.
3. In-plane cracking with (a) "x" and rocking shear cracks in piers and (b) end pier shear cracks.
4. Upper corner cracking.
5. Arch cracking and collapse.
6. Wall cracking between upper and lower floors.

Failure statistics for these failure modes were not investigated by Wiggins.

Damage descriptions for each building were reviewed in an effort to supplement, or at least complement the Northridge data. Typically, the damage reported in the damage descriptions are skewed toward lower damage because narrative damage reporting is not done in a consistent manner. The only element-specific damage which was reported with any consistency was wall cracking. However, as described earlier, it is possible that wall cracking, since it is so easily observed, was reported more readily than the other types of damage. Thus, like the Northridge data, the evaluation for wall cracking is likely skewed towards greater damage. A total of 49 retrofitted buildings report cracking of (possibly) structural significance—4.4% of the total retrofitted inventory. This is quite close to the 4.1% value determined from the Northridge data.

Table A-28: Whittier URM Bearing Wall Building Damage Data (Based on Wiggins, 1994)

Damage State	Damage Factor Range (%)	"Geometric Average Central Damage"[1] (%)	Central Damage Factor[2] (%)	Unstrength-ened (%)	Partially Strength-ened (%)	Strength-ened (%)
None	0	0	0.0	66.4	57.1	77.9
Slight	0-1	1	0.5	6.4	8.7	3.8
Light	1-10	3.1	5.5	16.3	22.4	13.7
Moderate	10-30	17.3	20	8.7	10.9	3.4
Heavy	30-60	47.4	45	1.6	1	1.3
Major	60-100	77.4	80	0.5	0	0
Destroyed	100	100	100	0	0	0
Average Damage[1]				3.22	3.14	1.67
Mean Damage Factor[2]				3.79	3.91	2.04

[1] "Geometric Average Central Damage" is taken from Wiggins (1994). The "Average Damage" is calculated as the sum of the Geometric Average Central Damage times the percentage in damage state.

[2] "Central Damage Factor" is taken from ATC-13 (1985). The "Mean Damage Factor" is calculated as the sum of the Central Damage Factor and the percentage in each damage state.

A.5 1989 Loma Prieta Earthquake Data

Unreinforced masonry construction was the most severely damaged building class in the Loma Prieta Earthquake. Damage to URM structures was observed in nearly all of the areas that felt the earthquake. Most well-known types of failures were observed throughout the effected area. Damage was especially severe in the downtown areas of Santa Cruz, Watsonville, Hollister, and Los Gatos. Of the 62 fatalities, 8 were related to URM building failures. Five people died at Sixth and Bluxom in San Francisco when the top story of a URM building fell outward and crushed workers who were exiting the building. In Santa Cruz, the parapet on a URM building fell and killed a pedestrian, and a portion of the top story wall of a URM building fell through the wood roof of an adjoining URM building, crushing two workers.

Senate Bill 547, enacted in 1986, requires cities and counties to identify potentially dangerous unreinforced masonry buildings and adopt plans for mitigating hazards. Cities and counties can comply with the State mandate by simply surveying suspected URM

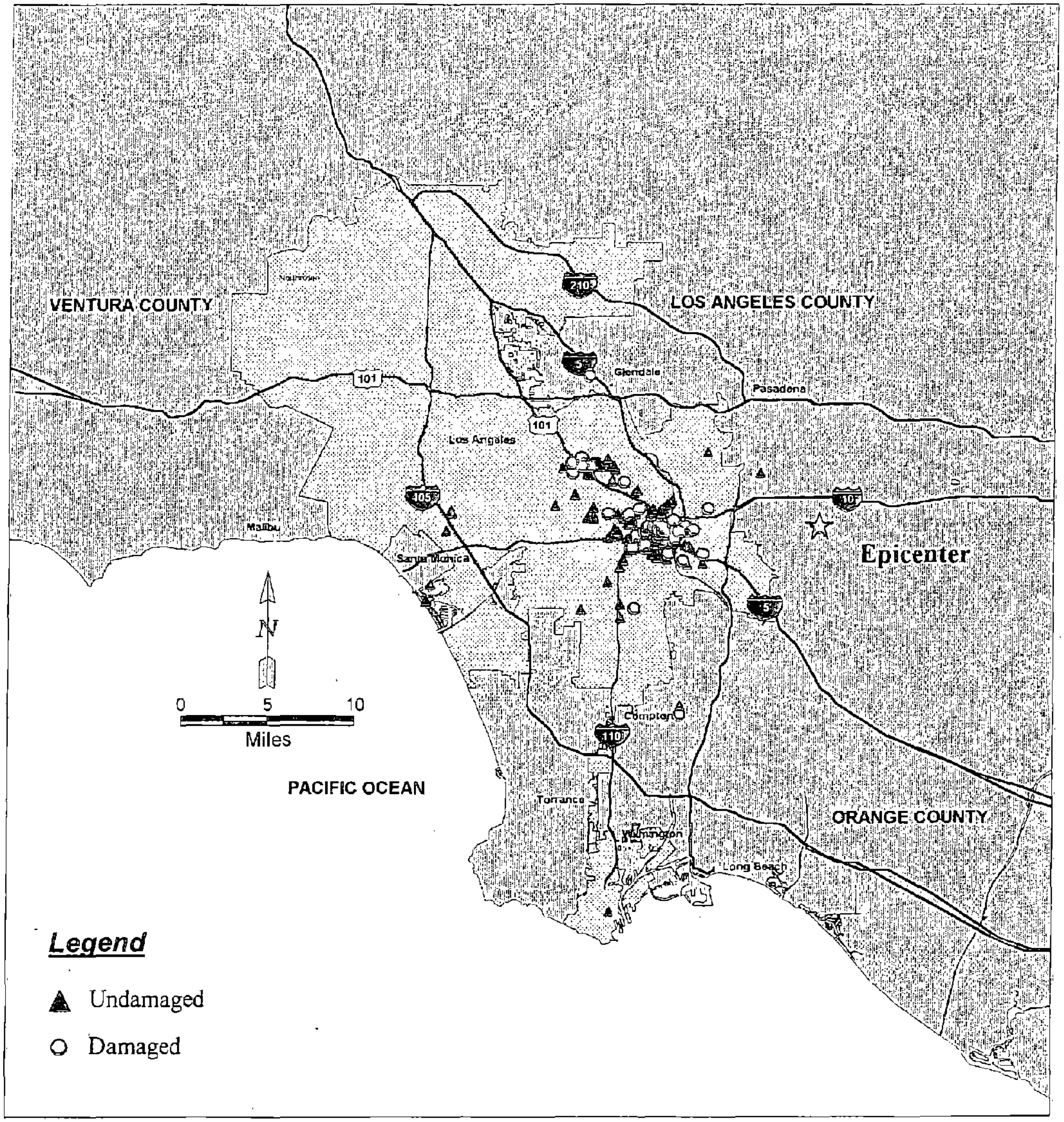


Figure A-22: Retrofitted URM Inventory, Whittier Narrows Earthquake

buildings and notifying owners of those that may constitute a hazard. SB547 does not require owners to strengthen their buildings. Except for Santa Rosa, the San Francisco Bay Area did not have any mandatory programs for strengthening of URM buildings when the Loma Prieta earthquake struck. As a result, there were relatively few buildings strengthened. (Following the earthquake, however, a number of communities passed mandatory URM building ordinances and a significant number of buildings have now been strengthened or will be in the near future.) Lizundia, et al. (1991) investigated damage in the Loma Prieta earthquake to unstrengthened and strengthened URM bearing wall buildings. The majority of these buildings were located in MMI=VI or VII areas. Damage occurred to some strengthened or partially strengthened buildings in MMI=VIII Santa Cruz and Watsonville and MMI=VII Hollister, Campbell and San Francisco. Results for the nine affected counties are shown in Table A-29; results for San Francisco are shown in Table A-30.

Table A-29: Damage to URM Bearing Wall Buildings in the 1989 Loma Prieta Earthquake (Lizundia, et al., 1991)

Strengthening Status	Total Buildings	Total Damaged		Total Vacated		Total Demolished	
	Number	Number	%	Number	%	Number	%
Unstrengthened[1]	6716	1203	18	410	6.1	51	0.8
	162	25	15	9	5.5	1	0.6

[1] These figures assume San Francisco's buildings with parapet bracing are "unstrengthened." The criteria for strengthening of the "strengthened" buildings vary significantly. Many are well below the level of the current UCBC.

A.6 Available Databases

Four Microsoft Excel databases are available. The first, entitled DIVIS88.XLS, describes the characteristics or attributes of the URM buildings which are on the Division 88 master list maintained by the Earthquake Safety Division of the Los Angeles Department of Building and Safety (LADBS). There are 6446 buildings in this database, 5682 which are fully retrofitted and 61 which are retrofitted with tension ties only and 703 which are unretrofitted. This database is presented as an Excel workbook, with worksheets for retrofitted, tension-tie-only and unretrofitted inventories. The second database, entitled INSPLOG.XLS, provides the tracking and repair activity for each building inspected by the LADBS after the Northridge Earthquake. Data for all of the 1240 buildings inspected by LADBS are available. The third database is presented in the form of an Excel workbook, entitled RFI.XLS. This workbook contains all the earthquake damage data for the fully retrofitted, unretrofitted, and tension-tie-only buildings inspected by LADBS. Data for the retrofitted inventory of 751 buildings is contained on the worksheet titled RFIR. Data for the unretrofitted inventory of 93 buildings is contained on the worksheet titled RFIU. Data for the tension-tie-only buildings is found on the worksheet titled TENSION TIE.

Table A-30: Damage to San Francisco URM Bearing Wall Buildings in the 1989 Loma Prieta Earthquake (Lizundia et al., 1991)

Strengthening Status	Damage Class	Central Damage Ratio[2]	Number of Buildings	Damage Averaged By Number of Buildings	Square Footage	Damage Averaged by Amount of Square Footage
Unstrengthened[1]	None	0.000	1203	0.0	18,502,644	0
	Slight	0.005	388	1.9	6,964,218	34,821
	Light	0.055	194	10.7	3,741,214	205,767
	Moderate	0.200	103	20.6	2,222,874	444,575
	Heavy	0.450	19	8.6	482,156	216,970
	Severe	0.600	16	9.6	608,938	365,363
	Totals		1923	51.4	32,522,044	1,267,496
	Average Damage Ratio[2]			**0.0267**		**0.039**
Strengthened[1]	None	0.000	34	0.0	No Data on Square Footage for the Strengthened Buildings	
	Slight	0.005	19	0.1		
	Light	0.055	13	0.7		
	Moderate	0.200	2	0.4		
	Heavy	0.450	0	0.0		
	Severe	0.600	0	0.0		
	Totals		68	1.2		
	Average Damage Ratio[2]			**0.0178**		

[1] These figures assume San Francisco's buildings with parapet bracing are "unstrengthened." The "strengthened" buildings include those presumably strengthened to San Francisco Building Code 104(f) as well as those which appear to not fully qualify with 104(f) requirements. The 104(f) standards are generally more stringent than those of the UCBC or Division 88.

[2] "Damage class" and "damage ratio" concepts are modified from ATC-13 (1985). The ratio in "central damage ratio" and "average damage ratio" is defined as the cost of repair divided by the cost of replacement.

DIVIS88

The Division 88 database is presented in the form of an Excel workbook, entitled DIVIS88.XLS. This database contains the building characteristics and attributes, plus a chronological log of the Division 88 permitting/compliance process. Ground motion data was added for each building. See Section A.3 for details. Data for the retrofitted inventory of 5682 buildings is contained on the worksheet titled RETROFITTED. Data for the unretrofitted inventory of 703 buildings is contained on the worksheet titled UNRETROFITTED. Data for the 61 tension-tie-only buildings is found on the worksheet titled TENSION TIE. The fields on each worksheet are identical. More specific information describing these databases follows:

Table A-31: DIVIS88 Database Fields

Field Name	Contents
D88LIST	Division 88 sequence number
STREET	Street address
CITY	City
NOBLDG	Number of buildings in complex
STORIES	Number of stories, excluding basement
WIDTH	Width (parallel to street)
DEPTH	Depth (perpendicular to street)
A1CMP_CD	Alt 1 comp compliance date (full Division 88 compliance)
A2ACM_CD	Anc 1 comp compliance date (tension ties only)
A2CMP_CD	Anc 2 comp compliance date (full Division 88 compliance)
VACATED	Date building vacated, as applicable
YEARBT	Year built
ESSENTIAL	Essential facility (Y/N)
HISTORIC	Historic building (Y/N)
MMI	Modified Mercalli Intensity
PGA_Ave	PGA (g)
PGV_Ave	PGV (cm/sec)
SA_03_Ave	Sa(0.3) (g)
SA_10_Ave	Sa(1.0) (g)

Definitions of these fields follow:

Division 88 Sequence Number: The building identifying code as defined by the Earthquake Safety Division. This number is also found in the RFI databases; it is the number used to link the DIVIS88 databases, with the RFI databases.

Street Address: The full street address of the building.

City: The city in which the building is located. Entries include "Los Angeles" and "Venice".

Number of buildings in complex: The number of buildings which share this address.

Number of Stories: Number of stories, excluding basement.

Width: Width (parallel to street).

Depth: Depth (perpendicular to street).

Alt 1 Comp: Full Division 88 compliance date. Division 88 (1988) originally allowed for two options for achieving full compliance: full compliance in one phase or full compliance in two phases. The two-phase option allowed a tension tie (only) retrofit to be completed in the first phase and the remaining work to be completed in the second phase. If this field has a date, then the one-phase option was selected. The date indicates when LADBS issued the document indicating that the work represented full compliance.

Anc 1 Comp: The date LADBS issued the compliance document for the first (tension-tie-only) phase of the two-phase option.

Anc 2 Comp: The date LADBS issued the compliance document for the second (full) phase of the two-phase option.

Year Built: Year building built.

Essential Facility: "Yes" indicates an essential facility.

Historic Building: "Yes" indicates a historic building.

MMI: Modified Mercalli Intensity assigned to the building.

PGA: Average of peak ground acceleration contours adjacent to the building (g).

Sa(0.3): Average of spectral acceleration (at T=0.3 seconds) contours adjacent to the building (g).

Sa(1.0): Average of spectral acceleration (at T=1.0 seconds) contours adjacent to the building (g).

INSPLOG

The INSPLOG database was provided by LADBS. It is in the form of an EXCEL file, entitled INSPLOG.XLS. This list provides the tracking and repair activity for each building inspected by the LADBS after the Northridge Earthquake. Data for 1239 of the 1240 buildings inspected by LADBS are available in this list. The database contains the following fields:

Definitions of these fields follow:

Inspection ID No.: The building identifying code established by LABDS during the post-earthquake inspection process. This number is not related to the D88LIST sequence number assigned by the Earthquake Safety Division. The Inspection ID Number is used in the RFI databases, also prepared by LADBS. It is the number used to link the INSPLOG database to the RFI databases.

Table A-32: INSPLOG Database Fields

Field Name	Contents
The following 10 fields typically repeat until inspection activity was completed.	
Inspection_ID_No.	Building inspection ID number (as assigned by LADBS)
Document_Type	Building damage assessment form name
Insp._Date	Date of inspection
Insp._Name	Inspector's name
%_Damage	Estimated repair cost divided by replacement cost
Str._Damage	Estimated structural damage as a dollar value
Geo._Damage	Estimated geotechnical damage
Unit_Vac.	Number of vacated units
Posting	Posting
Vacancy	Extent of vacancies
Final	Unknown
Time	Time of inspection

<u>Document Type</u>: The name of the LADBS disaster inspection form.

> "G4A": Emergency Call Slip (does not contain emergency data).
>
> "G4": Rapid Screening Inspection Form (See Figure A-2).
>
> "G4GRI": Disaster Re-inspection Form (See Figure A-5).
>
> "G4GRS": Disaster Re-inspection Form (Scantron version).
>
> "PMT": Building permit.
>
> "PL": Placard lite.

<u>Insp. Name</u>: Name of inspector.

<u>% Damage</u>: Estimated repair cost divided by building replacement cost.

<u>Str. Damage</u>: Estimated structural damage as a dollar value.

<u>Geo. Damage</u>: Estimated geotechnical damage as a dollar value.

<u>Units Vac</u>: Number of units vacated after the earthquake.

<u>Posting</u>: Building posting, based on the ATC-20 (1989) methodology. Entries include "Red", "Yellow", "Green", or blank.

Vacancy: Extent of vacancies. Entries include "T" for total, "P" for partial, and blank.

Time: Time of inspection.

RFI

The RFI database is presented in the form of an Excel workbook, entitled RFI.XLS. This database contains the building characteristics and attributes, retrofit status, earthquake damage data, and ground motion data for each of the retrofitted, unretrofitted, and tension-tie-only retrofitted buildings inspected by LADBS after the Northridge Earthquake. Data for the retrofitted inventory of 751 buildings is contained on the worksheet titled RFIR. Data for the unretrofitted inventory of 93 buildings is contained on the worksheet titled RFIU. Data for the tension-tie-only buildings is found on the worksheet titled TENSION TIE. The fields on each worksheet are identical. In addition, there are also worksheets titled RFI POSTING and RETROFITTED POSTING which contain building tagging information for the RFI dataset. More specific information describing these databases follows.

Definitions of these fields follow:

Inspection ID No.: The building identifying code established by LABDS during the post-earthquake inspection process. This number is not related to the D88LIST sequence number assigned by the Earthquake Safety Division. It the number used to link the INSPLOG database to the RFI databases.

Division 88 Sequence Number: The building identifying code as defined by the Earthquake Safety Division. This number is used to link the DIVIS88 databases with the RFI databases.

Document Type: The name of the LADBS disaster inspection form.

"G4A": Emergency Call Slip (does not contain emergency data).

"G4": Rapid Screening Inspection Form (See Figure A-2).

"G4GRI": Disaster Re-inspection Form (See Figure A-5).

"G4GRS": Disaster Re-inspection Form (Scantron version).

"PMT": Building permit.

"PL": Placard lite.

Table A-33: RFI Database Fields

Title	Contents
Inspection ID No.	Building inspection ID number (as assigned by LADBS)
D88LIST	Division 88 Sequence number (RFIR, RFIU, and TENSION TIE worksheets)
Document_Type	LADBS building damage assessment form name
Address_No.	Numeric street address
Street	Street name
Type	Street type
Dir	Direction
Address	Full street address
City	City
Use	Building use
CD	Council district
Alternate_Address	Alternate address
Stories	Number of stories
No._Units	Number of units
Basement	Basement (Y/N/U)
Length	Building depth (perpendicular to street)
Width	Building width (parallel to street)
Prim_Occ.	Primary occupancy
Hazardous	This field and the following fields are responses to the LABDS building damage assessment questions (form G4).
Collapse	
Leaning	
Other_1	
Description_1	
Hazardous_Elements	
Foundations	
Roof/Floors	
Col/Pil/Cor	
Dia/Hor/Brc	
Walls/	
Vertical_Bracing	
Moment_Frame	
Precast	
Other_2	
Description_2	
Nonstructural	
Parapets	
Cladding	
Light_Fixtures	
Interior_Partitions	

Title	Contents
Elevators	
Stairs	
Electric/Gas	
Chimney	
Other_3	
Description_3	
Geo._Hazard	
Slope_Failure	
Slide_Class	
Ground_Movement	
Comments	
Vacate	
Partial_Vacate	
Apt._Units_Vacated	
Permit_Req.	Additional permit required
Barricades	Barricade description
Retro_Status	Building retrofit status
Estimated_Damage%	Estimated repair cost divided by replacement cost
Estimated_Damage$	Estimated damage as a dollar value
ATC-13_Damage	ATC-13 damage state
MMI	Modified Mercalli Intensity
PGA_Ave	PGA (g)
SA_03_Ave	Sa(0.3) (g)
SA_10_Ave	Sa(1.0) (g)
PGV_Ave	PGV (cm/sec)
Firstpost	Initial building posting
Reinsppost	First reinspection building posting
Secinsppost	Second reinspection building posting

Address: The numeric portion of the building's address.

Street: The street name.

Type: The type of street, i.e., Av., Blvd., St., etc.

Direction: The direction portion of the street name, i.e., North, South, East, West.

Street Address: The full street address of the building

City: The city in which the building is located. Entries include "Los Angeles" and "Venice".

Use: The building use as defined by the G4 inspection form. These categories include "R" for residential, "C" for commercial and "M" (unknown designation).

Council District: The city council district in which the building is located.

Alternate Address: Alternate address.

Stories: Number of stories, excluding basement.

No. Units: The number of units which share this address

Basement: "Y", "N", or "U". Indicates the existence of a basement.

Length: Length of building (parallel to street) from the G4 inspection form.

Width: Width of building (perpendicular to street) from the G4 inspection form.

Primary Occupancy: The building use as defined on the G4 form. These categories include:

1. Dwelling
2. Duplex
3. Airport
4. Amusement
5. Apartments
6. Church
7. Private garage
8. Public garage
9. Gas station
10. Hospital
11. Hotel
12. Manufacturing
13. Office
14. Public Administration
15. Public Utilities
16. Retail Store
17. Restaurant
18. School
19. Theater
20. Warehouse
21. Condo
22. Other

The following entries are all taken from the LADBS G4 inspection form. Criteria for selecting "Yes" are presumably taken from ATC-20 (1989).

Hazard: "Yes" indicates a general hazardous condition exists.

Collapse: "Yes" indicates the building has partially or fully collapsed.

Leaning: "Yes" indicates the building or story is leaning.

Other 1: "Yes" indicates another general hazard.

Description 1: Description of general hazard.

Hazardous Str: "Yes" indicates hazardous structural elements exist.

Foundations: "Yes" indicates a hazardous condition exists.

Roof/Floor: "Yes" indicates damage to the roof and/or floors which compromises vertical load-carrying capacity.

Col/Pil/Cor: "Yes" indicates damage to columns, pilasters, or corbels.

Dia/Hor/Brc: "Yes" indicates damage to the diaphragms and/or horizontal bracing.

Walls/Vert Bracing: "Yes" indicates damage to walls and /or vertical bracing.

Moment_Frame: "Yes" indicates damage to moment frames.

Precast: "Yes" indicates damage to precast connections.

Nonstructural: "Yes" indicates nonstructural hazards exist.

Parapet: "Yes" indicates damage to parapets.

Cladding: "Yes" indicates damage to cladding.

Light Fixtures: "Yes" indicates damage to light fixtures.

Interior Partitions: "Yes" indicates damage to interior partitions.

Elevators: "Yes" indicates damage to elevators.

Stairs: "Yes" indicates damage to stairs and/or exits.

Electric/Gas: "Yes" indicates damage to electric and/or gas systems.

Chimney: "Yes" indicates damage to chimneys.

Geo. Hazard: "Yes" indicates a general geological hazard exists.

Slope Failure: "Yes" indicates slope failure and/or hazardous debris.

Sliding Class: Entries include "Y", "N", "U", and blank; criteria and meaning are unknown.

Ground Movement: "Yes" indicates ground movement and/or fissures.

Comments: General comments on damage.

Vacate: "Yes" indicates building must be vacated.

Partial Vacate: "Yes" indicates building must be partially vacated.

Apt. Units Vacate: The number of living units vacated.

Permit: "Yes" indicates a permit is required.

Barricades: "Yes" indicates barricades are needed.

Retro Status: The building's retrofit status as defined in the DIVIS88.XLS database, where:

full: Full Division 88 compliance.

unret: Unretrofitted

tt: Division 88 Tension-tie-only retrofit.

Estimated Damage%: Estimated repair cost divided by building replacement cost.

Estimate Damage$: Estimated damage as a dollar value.

ATC-13 Damage State: "None", "Slight", "Light", "Moderate", "Heavy", "Major", and "Destroyed" damage states as defined per ATC-13 (1985). See Table A-7 for descriptions of each damage state.

MMI: Modified Mercalli Intensity assigned to the building.

PGA: Average of peak ground acceleration contours adjacent to the building (g).

Sa(0.3): Average of spectral acceleration (at T=0.3 seconds) contours adjacent to the building (g).

Sa(1.0): Average of spectral acceleration (at T=1.0 seconds) contours adjacent to the building (g).

PGV: Average of peak ground velocity contours adjacent to the building (cm/sec).

Firstpost: Initial posting. "Red", "Yellow", "Green", or blank. This information is given in the RETROFITTED POSTING worksheet and the RFIR POSTING worksheet.

Reinsppost: First reinspection posting. "Red", "Yellow", "Green", or blank. This information is given in the RETROFITTED POSTING worksheet and the RFIR POSTING worksheet.

Secinsppost: Second reinspection posting. "Red", "Yellow", "Green", or blank. This information is given in the RETROFITTED POSTING worksheet only.

APPENDIX B
APPLICABILITY OF CURRENT PRACTICE NATIONWIDE

Preface

Current retrofit methodologies for URM 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 these guidelines 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. A summary of conclusions is contained in Section B.1; it is followed in Section B.2 by a more detailed explanation of the process used to gather information and our findings.

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B.1 Summary of Observed Construction Practices For Which Current Retrofit Provisions May Be Inappropriate

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

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. 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

Inherent to the special procedure methodology in current retrofit provisions is the assumption of 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 of the current retrofit provisions such as FEMA 178 (1992) and UCBC (1994) is an arbitrary level based upon compromise by code writers. In California, there are a relatively small number of buildings over this limit, 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 the building stock. Thus, establishing the applicability of the Special Procedure for these taller buildings is worth further investigation.

B.2 Regional Construction and Retrofitting Techniques

Several studies by the ABK Joint Venture team provided the foundation from which current retrofit provisions were developed. One of the first studies was the *Methodology for Mitigation of Seismic Hazards in Existing Unreinforced Masonry Buildings: Categorization of Buildings* (ABK, 1981a). This report describes a study to obtain information regarding the sizes, shapes, materials, and construction methods of roof and floor diaphragms and walls in URM buildings in six regions around the nation. 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. The nationwide survey form is included at the end of this appendix.

Figure B-1 shows a typical URM bearing wall building in California. Typical unreinforced masonry construction in five other 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 summary of findings includes: A brief description of the seismicity of the region, the 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. The results of this investigation are compiled below by geographical region.

Pacific Northwest (Seattle)

The Pacific Northwest is one of several active seismic regions in the United States. It is fairly well accepted that the Northwest may be subjected to great subduction earthquakes from the Cascadia Subduction Zone off the Oregon and Washington coasts. The western portions of Washington and now Oregon are assigned to Seismic Zone 3 in the UBC. The Puget Sound region, which includes Seattle, is an active region which has experienced over 1,080 felt earthquakes in the past 135 years. A large magnitude 7.1

earthquake in 1949 was centered about 40 miles southwest of Seattle. A report issued by the ASCE after the earthquake identified older URM buildings, circa 1890, as the most heavily affected type of construction. Poor performance was attributed to inferior brick, weak mortar, poorly anchored diaphragm elements, and bearing walls with many openings. Damage to older URM buildings in the July 29, 1965 earthquake (magnitude 6.5) was comparable to the 1949 event.

Both ABK (1981a) and a comprehensive study entitled *Seismic Hazards in Unreinforced Masonry Buildings in Small Towns in the Pacific Northwest* (Hawkins & Burke, 1985) discuss the structural characteristics of URM buildings; the first in a large metropolitan city (Seattle) and the latter in seven small cities and towns.

The findings of ABK (1981a) that pertain to this study follow in italics, and our comments are in regular type. A similar summary is contained in Lizundia, et. al (1993).

- *The size of downtown commercial buildings is large, both in plan and in height. Six or more stories are common. Walls are typically punched with a regular pattern of large windows.* URM bearing wall buildings of such heights and dimensions are uncommon in California, even in large cities. Further, buildings of this height exceed the six story limit of the Special Procedure in current retrofit provisions.

- *Large public schools, containing gable end walls, high pitched roofs and extensive fenestration, are also numerous. Many of these schools were extensively damaged in the 1949 and 1965 earthquakes. This damage included wall cracking, separation of walls and floors, collapse of chimneys, gable ends and cornices.* Gable walls and high-pitched roofs are often found in churches, but it is somewhat unusual to find them in other building types, at least in California.

- *Floors and roofs of URM school and commercial buildings are typically comprised of wood framing.* Other types of diaphragm construction are rare.

- *The quality of brick is good to excellent and the mortar quality is generally good. Weathering is not an extensive problem.* Further, ABK (1981a) suggests that local brick is superior to that used in California. Our own experience and observations in California confirm that mortar quality and test results vary significantly in different areas of California, as well as between different building types.

- *Use of terra-cotta and dressed stone in conjunction with brickwork is common for ornamentation. However, entire facades of terra-cotta or stonework are not common.* Terra cotta and dressed stone are used in California usually only in more monumental structures and only in facades visible to public viewing.

Hawkins and Burke (1985) report that the URM inventory in small cities and towns is predominantly four stories or less. Most URM buildings were built before 1900 during periods of rapid growth and speculation; they were erected quickly and are similar in construction. Wood diaphragms predominate. The absence of any kind of diaphragm-to-wall tie is common. Their report repeatedly mentions the great extent of deterioration that has occurred, relating it in part to the lack of maintenance caused by the typically long periods of vacancies. Typically, building condition is poor; mortar joints are deteriorated, parapets, cornices, appendages are unsecured; there is evidence of wood rot in the floor and roof joists, especially at the wall pockets; and foundations are inadequate and deteriorated. Evidence of water migration in the walls is widespread. Such a combination of existing conditions means these buildings are more vulnerable to lateral forces. In some small towns, buildings have deteriorated to such an extent that bricks fallen from chimneys, parapets, facades, and cornices are found around the building. It is interesting to note that the ABK (1981a) states that weathering and mortar deterioration were not significant in the Seattle inventory. A number of factors are likely responsible for the increased deterioration of the small town inventory, including older vintage, less maintenance, and lower construction quality. In our experience, the extent of deterioration described by Hawkins and Burke (1985) is not commonly found in Northern California buildings, regardless of community size.

Additional inventory information for Seattle is reported in Lizundia, et al. (1993). The results of this study (italicized) confirmed and/or determined the following:

- *There are numerous three- to four-story, 60x120 foot plan, soft story buildings.* Similar buildings are common in the San Francisco Bay area and in Los Angeles.

- *There are many buildings in Seattle built as late as even the early 1960s that use hollow (unreinforced) concrete block walls.* This is a somewhat unusual building system in California. Further, the current retrofit provisions were developed for brick construction. They may be applied to other unreinforced masonry materials only when certain conditions are satisfied. In the case of hollow CMU, the building must not exceed two stories in height, the units must be placed in a running bond pattern, and the shear stress does not exceed the allowable determined by in-place shear testing. These shear test values, however, are based on in-place shear tests intended for brick walls; they are not necessarily appropriate for CMU walls. Another significant issue concerning hollow block wall construction is the difficulty in providing adequate diaphragm-to-wall anchorage for out-of-plane forces.

- *About 300-500 URM unstrengthened buildings remain, but many retrofitted building designs have only addressed some deficiencies like parapets and wall-diaphragm ties. In-plane wall strengthening, for example, is not that common.*

Finally, a survey respondent indicates that structural clay tile is used with moderate frequency for construction of bearing walls. Structural clay tile is closely related to brick, but is finer in texture. Tile units typically consist of 12-inch-square units varying in thickness from three to six inches. The units have open ended dividers which create square or rectangular tubes through the unit. The tiles are often laid randomly, with the open ends facing horizontally or vertically. They are usually covered with plaster, and the workmanship is often poor. The UCBC addresses structural clay tile in the same abbreviated manner as hollow concrete block.

The building types identified herein for which current retrofit provisions may be inappropriate without modification include:

- Many buildings in excess of six stories.
- There are many buildings in Seattle built as late as even the early 1960s that use hollow unreinforced concrete block walls.
- Structural clay tile is used with moderate frequency for construction of bearing walls.

New Madrid (Kansas City, St. Louis, and Memphis)

The successions of shocks designated collectively as the New Madrid earthquakes began on December 16, 1811 and continued for over a year. There were three main shocks, all estimated to be $M_s = 8$ or greater. These shocks have not been equaled for number, continuance of disturbance, area affected, and severity anywhere in the United States. The tremors were felt for over 1,000,000 square miles—half of the entire United States. Historical records and geological evidence indicate that five strong shocks preceded the 1811 event. Damage to brick and stone buildings was reported as far away as Charleston, South Carolina (500 miles), Natchez, Louisiana (400 miles), and Cincinnati, Ohio (300 miles) (Fuller, 1912).

The attenuation of seismic waves is a significant factor in the seismicity of the area. The rock of the midwest can transmit earthquake energy much more efficiently than the fissured rock of California. Thus, locations such as Kansas City, 330 miles away and Chicago, 800 miles away, could experience significant shaking from a New Madrid event. The effect would likely predominate in the long period range. This could cause the taller structures to experience significant performance problems. Consideration should be given to the fact that the shorter and stiffer URM buildings may not be significantly vulnerable to long period vibrations.

Three cities with large commercial districts were surveyed in the New Madrid region: Kansas City St. Louis, and Memphis. These cities have very large inventories of URM buildings—possibly the largest in a significant seismic region.

Kansas City and St. Louis

The New Madrid fault is approximately 330 miles from Kansas City and approximately 200 miles from St. Louis. The Nemaha Uplift is nearby, giving both cities a Zone 2A UBC classification.

Thomas Heausler, a structural engineer in Kansas City with experience in URM building retrofit, assisted us in our determination of the range of URM building types in the New Madrid region. An engineer from Rutherford & Chekene accompanied Mr. Heausler on a tour of URM buildings in Kansas City which he determined to be of significance to this study. The purpose of the trip was to observe typical Midwestern and construction and retrofit practices and to retrieve information on local building stocks from historical preservation sources and city building inventories.

Log cabin, wood frame and unreinforced brick and stone masonry were the materials of choice for the early settlers in this midwestern town. The oldest date back to the 1850s. As the city grew, larger buildings, typically up to four stories, were constructed of unreinforced masonry with wood floors. Typical joist anchorage details are shown in Figure B-2. In the early 1900s, reinforced concrete floors became increasingly popular as there was a growing concern for fire resistance. Concrete floors were supported by concrete encased steel beams or reinforced concrete beams bearing on thick unreinforced masonry walls. Concrete floor framing is also reported to be common in St. Louis (ABK, 1981a). Later, about 1920, construction practices transitioned to beams supported on reinforced concrete or steel frames infilled with unreinforced masonry. During this period, the architecture of Kansas City was influenced by architects and construction practices used in Chicago, Boston, and New York.

Between 1925 and 1950, unreinforced masonry remained commonplace for smaller, one-story to three-story structures, especially churches, apartments (circa 1925), and residences (circa 1925 to 1950). Unreinforced masonry construction continues today for minor structures, including low-cost CMU garages and one-story retail stores, and motels. It is generally no longer specified by architects and structural engineers today, although URM buildings continue to be built by contractors; their construction is generally not prohibited in low seismic zones.

Although there is increased awareness of the seismic vulnerability of URM buildings, renovation generally is limited to architectural issues and repair of decayed structural elements only. Seismic rehabilitation is employed where convenient and cost-effective, but buildings are not strengthened to provide a performance level comparable to new construction requirements. Retrofit wall anchors have been added to numerous URM buildings. The motivation is not primarily to mitigate a seismic deficiency, but, instead, it is generally to correct a separation due to settlement or other undesirable movement of the wall relative to the floor.

The documented deficiencies of URM buildings found in California generally appear to be commonplace in Kansas City and St. Louis. The primary contrasts between unreinforced masonry construction in California versus the Midwest are outlined below:

- The commercial and industrial districts have significant populations of buildings taller than six stories. One survey respondent indicates the existence of URM construction up to 16 stories in height.

- The walls of the older URM buildings tend to be thicker than that of California. Typical wall thickness are shown in Table B-1.

- There was an extensive use of cast iron columns at the interior of the buildings as compared with wood post or stud walls as found in California. Typical mill construction, circa 1980, is shown in Figure B-3.

- Many buildings have basements excavated during construction. Rubble found in the excavation was often used for the basement wall masonry. This could also be supplemented with limestone quarried from numerous local quarries.

- Stone masonry construction is very common and often exhibits exceptional masonry craftsmanship. The coursing pattern often consists of horizontal bed joints which are interrupted by the vertical edge of large stones. This should provide added strength similar to running bond versus stack bond. Typical stone construction is shown in Figure B-4, Figure B-5, and Figure B-6.

- The face brick and mortar used on walls is often considerably stronger and more durable than the inner wythes.

- Rigid floor and roof diaphragms are common. Various types of concrete diaphragms, such as precast slab, cast-in-place slab, hollow plank, and precast joists, are shown in Figure B-7. Typical masonry diaphragms, including brick arches and HCT flat arches are shown in Figure B-8. HCT tile arch floors were observed in buildings up to 10 stories. A section through such as structure is shown in Figure B-9.

- There are many brick single family residences. Multi-family housing constructed prior to 1940 generally utilized unreinforced masonry walls.

- Long and narrow buildings are common in rail yards. These building types have few transverse shear walls and diaphragms with very high aspect ratios.

Memphis

The City of Memphis is the largest metropolitan region in the vicinity of the New Madrid fault zone. It is assigned to Seismic Zone 3 in the 1994 UBC. The URM inventory is described in ABK (1981a) as follows:

> *"The present city center has many large commercial URM buildings incorporated in its city plan. This and an adjacent industrial district include most of the URM buildings. Suburban commercial centers and multiple residential structures comprise the remainder of the existing buildings. Current URM construction is generally used for small commercial structures or single-story industrial structures. The pre-1940 buildings have wood framed or concrete floors. Post-1940 buildings generally use URM walls of concrete block rather than brick.*

These observations were verified by our survey respondents and our own investigations. The important characteristics of the existing URM inventory can be divided into two vintages as follows:

Pre-1940

- Joist-to-wall anchorage is uncommon.
- The quality of the masonry is poor to fair; there has been considerable deterioration. Upper floors are typically not occupied and typically in too poor a condition to rent.

Post-1940

- Concrete hollow block construction predominates (Figure B-10).
- Rigid floor and roof framing are common, including concrete floor slabs cast on metal deck forms, precast hollow plank and precast joists, and concrete slab spanning between steel I-beams (Figures B-6, B-7, B-8, and B-10).
- Metal deck and joist/beam framing generally replaced wood framing. Joist anchorage of metal framing is provided by welding framing to steel bearing plates anchored to bond beams. Typical construction details are shown in Figure B-11.
- Diaphragm flange reinforcement, drag strip, and ties to shear walls are not used.
- Cladding and veneer anchorage is nonexistent.
- Very few are retrofitted.

The building types identified herein for which current retrofit provisions may be inappropriate without modification include:

- Buildings taller than six stories.
- Unreinforced concrete hollow block wall construction.
- Rigid floor systems, including brick arch, HCT flat arch, concrete slab spanning between steel I-beams, hollow plank, cast-in-place concrete slab, precast concrete slab, and metal deck with concrete slab.
- Extremely steep roofs.
- Diaphragms with very high aspect ratios.

Wasatch (Salt Lake City)

Salt Lake City is the largest metropolitan area in the seismic region termed the "Wasatch Front." Seismic ground motion in this region can be divided into two categories: strong ground motion resulting from large, but very infrequent earthquakes and moderate ground motion due to smaller, relatively frequent earthquakes. The Wasatch fault, the East Great Salt Lake fault, and the Oquirrh fault dominate the strong ground motion hazard. Recent studies estimate characteristic maximum magnitudes of up to 7.1 in the region (Rutherford & Chekene, 1995). The 1994 UBC places the area in Seismic Zone 3. Historically, there has been a lack of small magnitude events associated with the Wasatch fault. There are other source zones, however, which are responsible for numerous small magnitude events.

ABK (1981a) identifies these important characteristics of the existing URM inventory:

- Construction is very similar to that found in California.
- Quality of masonry is very good compared to the Midwest.

- Industrial buildings are smaller than in the East.
- Single and multi-family housing commonly uses unreinforced masonry with wood diaphragms.
- Anchorage of wood framing to walls is similar to California.

One of our survey respondents indicates that thin concrete fill over open web steel joist diaphragm construction is not uncommon. Otherwise, we could identify no building type which differs significantly from that found in California.

New England (New York)

Most of New England, including New York City, is assigned a Seismic Zone 2A in the 1994 UBC. This geographic region has the largest inventory of URM buildings, dating from the early 1800's to current construction. The taller URM buildings are generally the oldest. Most buildings are commercial and manufacturing. Significant numbers of retrofitted buildings are found only in New York City. The important characteristics of the existing URM inventory identified in ABK (1981a) and by survey respondents are:

- Buildings, especially industrial buildings, are larger than the typical California stock, with many firewalls.
- Masonry is of higher quality than California. Mortar shows few signs of deterioration, even in the adverse climate.
- Brick cavity wall construction is quite common. Typical details are shown in Figure B-12 and Figure B-13.
- Rigid diaphragms are quite common. HCT flat arch and brick arch or concrete slab spanning between steel I-beams are typically found, especially in large buildings.
- Masonry foundations and footings are not uncommon. Various details are shown in Figure B-14.
- Metal deck diaphragms and rough-hewn timber beam floors are found in moderate numbers.
- Ungrouted CMU construction is common.
- Buildings are fully occupied in downtown areas. In outlying areas, typically only the first floor is occupied.
- Many historic buildings use stone in conjunction with brickwork
- Ornamentation and use of elaborate masonry details along the roof line are common.
- Very few parapets are braced.
- Churches are larger than in other regions, have more elaborate geometries, and more ornate ornamentation.
- Irregularly-shaped buildings are very common.

Cavity wall construction is so prevalent it merits additional discussion. A vintage building construction textbook (Kidder, 1916) provides valuable insight into typical East Coast masonry materials construction. In this era, solid brick wall construction was not the best choice for building in the damp, cold New England climate. Solid brick walls readily absorb moisture and transmit heat and cold. Heavy rains may penetrate two-wythe thick brick walls, dampening the interior walls. Damp interior walls act as a heatsink, as the moisture must be evaporated before the room and wall temperature can be raised. Furthermore, warm air of the interior deposits moisture and dirt on the interior surface of the cold walls. Consequently, solid walls built in this type of climate are often damp. Moisture in the brickwork compromises the integrity of the mortar (especially lime mortar) and promotes rot of the woodwork. These problems were recognized and addressed in the mid-1800's. Hollow wall construction, which provides an insulating air space between the interior and exterior surfaces, dissipates moisture and makes the building much cooler in summer and warmer in winter. This type of construction is common only in this region.

Hollow wall construction typically consists of an 8-inch-thick wall and a 4-inch-thick wall (the thicker portion on the outside) bonded together with metal ties or straps at 24 inches on center every fourth course. Residential construction typically uses two 4-inch-thick courses. Many building regulations circa 1900 required that both portions of the walls be at least 8-inches-thick if they were to be used as bearing walls. Floor joists normally extend across the air space to the exterior wall but are sometimes supported on the interior 4-inch-thick wythe only. Clearly, the first method negates the intended benefit of moisture protection while the second does not optimize the bearing support of the floors. In either case, hollow wall construction is especially vulnerable to lateral forces. Cavity walls cannot typically meet the h/t criteria offered in current retrofit provisions, even for very low levels of excitation.

A less common type of hollow wall construction also found in the region is hollow brick walls with brick wythes. Four-inch-thick inner and outer walls are connected with solid brick wythes, creating air spaces 4-, 8-, or 12-inches-thick. Figure B-13 shows typical brick hollow wall construction used for cheap cottage construction, two-story buildings, and for Congress Hall in Saratoga, NY, a seven story building.

The building types identified herein for which current retrofit provisions may be inappropriate without modification include:

- Buildings taller than six stories.
- Rigid floor systems: Brick arch, HCT flat arch and concrete slab spanning between steel I-beams.
- Ungrouted CMU construction.
- Brick or block cavity wall construction.
- Metal deck diaphragms.

Charleston, South Carolina

This geographic region has numerous URM buildings which date from the 17th and 18th centuries and thus predate the 1886 earthquake. Non-residential URM structures are predominantly commercial and manufacturing. Important characteristics of the existing URM inventory are as follows, with ABK (1981a) observations italicized:

- Bricks are softer than in the New England region and many pre-Civil War buildings used clay in lieu of lime mortar in the masonry walls. Lime mortar made from crushed sea shells is reported to be of the best quality—having weathered without distress for up to 250 years. Exterior brickwork is frequently protected with a lime plaster.

- Commercial buildings generally have open fronts on the public way and generally do not exceed five stories in height.
- *Residential and commercial construction predominates.* There are fewer industrial and manufacturing structures than in other regions.
- Wood floors and roofs predominate.
- Many buildings in coastal areas have been renovated and rehabilitated.

Considerable information about historic URM construction types and practices can be found in a reconnaissance report by the USGS published a year after the Charleston Earthquake (Dutton, 1887). Lengthy, glowing descriptions are given regarding the quality of masonry construction materials and exceptional quality of construction of buildings constructed prior to 1838. In 1838, a huge fire destroyed a large portion of the most populous portion of the city. So many wood houses were burned that new wood construction was banned from the fire region. Thus, new brick construction began in earnest. Many of the new buildings were built by contractors from the north who brought with them their own techniques and mortars for masonry construction. These techniques were felt by the author of the account to be inferior to those they replaced. Further, he observed

> *"more opportunity for slovenly work in this new bond, as anyone can see who will take the trouble to notice the progress of brick-work at present which is not properly supervised . . . Owing to the deceptions that can be practiced by bricklayers in laying new bond, an infinite amount of bad work has been done of late years."*

The duration of the 1886 earthquake is thought to have been from thirty-five to forty seconds. More than 60 people were killed, many by falling brick. Many buildings collapsed, exterior walls toppled into the street, gable walls, piazzas, and parapets fell, and heavy walls cracked. Diagonal shear cracks between windows were a frequent observation. Over 14,000 chimneys were toppled; USGS (1887) speculated that a greater loss of life would have resulted from falling brick if the shock had occurred during the business hours of the day.

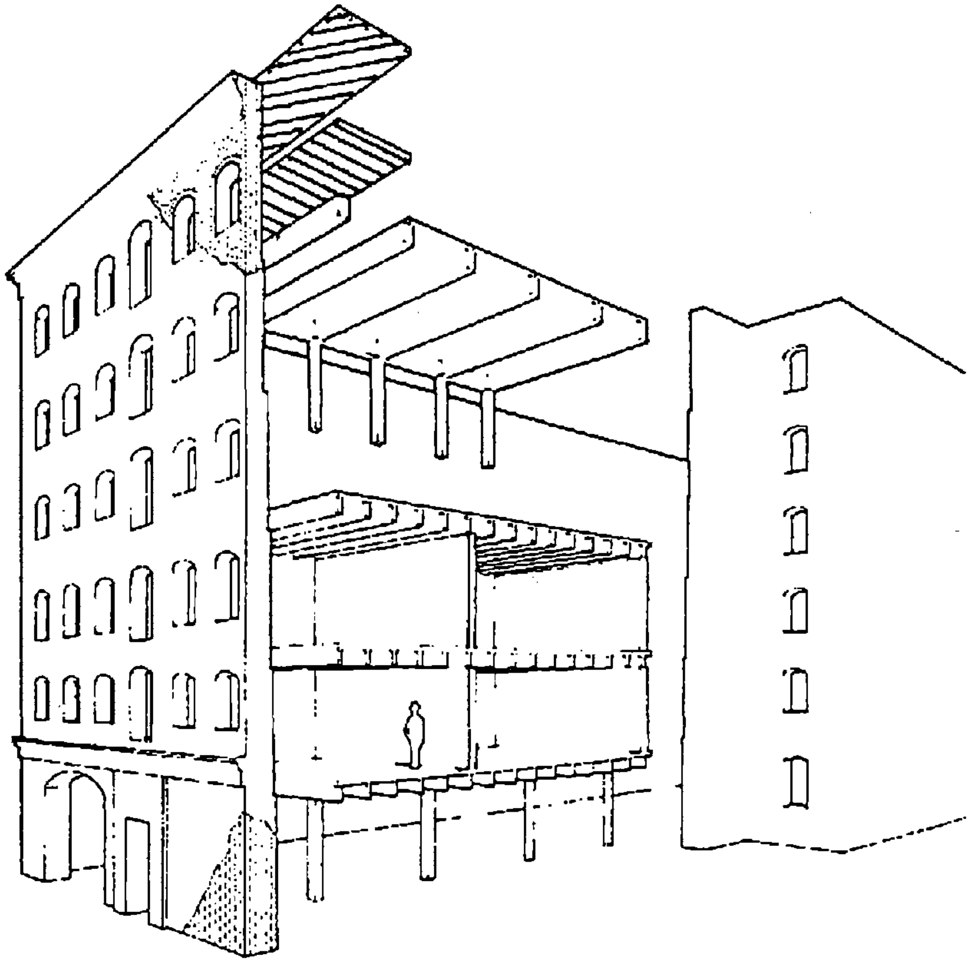
Current retrofit provisions appear applicable to the Charleston region.

Table B-1: Thickness (in) of Walls for Mercantile Buildings and, Except in Chicago, for All Buildings Over Five Stories in Height (Kidder, 1916)

Height and Location of Building		Stories 1st	2nd	3rd	4th	5th	6th	7th	8th	9th	10th	11th	12th
Two Stories	Boston	16	12										
	New York	12	12										
	Chicago	12	12										
	Minneapolis	12	12										
	St. Louis	13	13										
	Denver	13	13										
	San Francisco	17	13										
	New Orleans	13	13										
Three Stories	Boston	20	16	16									
	New York	16	16	12									
	Chicago	16	12	12									
	Minneapolis	16	12	12									
	St. Louis	18	13	13									
	Denver	17	17	13									
	San Francisco	17	17	13									
	New Orleans	13	13	13									
Four Stories	Boston	20	16	16	16								
	New York	16	16	16	12								
	Chicago	20	16	16	12								
	Minneapolis	16	16	12	12								
	St. Louis	22	18	18	13								
	Denver	21	17	17	13								
	San Francisco	17	17	17	13								
	New Orleans	18	18	13	13								
Five Stories	Boston	20	20	20	20	16							
	New York	20	16	16	16	16							
	Chicago	20	20	16	16	16							
	Minneapolis	20	16	16	12	12							
	St. Louis	22	22	18	18	13							
	Denver	21	21	17	17	13							
	San Francisco	21	18	17	17	13							
	New Orleans	18	17	18	13	13							
Six Stories	Boston	24	20	20	20	20	16						
	New York	24	20	20	20	16	16						
	Chicago	20	20	20	16	16	16						
	Minneapolis	20	20	16	16	16	12						
	St. Louis	26	22	22	18	13	13						
	Denver	26	21	21	17	17	13						
	San Francisco	21	21	17	17	17	13						
	New Orleans	22	18	18	18	13	13						

Table B-1, Continued: Thickness (in) of Walls for Mercantile Buildings and, Except in Chicago, For All Buildings Over Five Stories in Height (Kidder, 1916)

Height and Location of Building		Stories 1st	2nd	3rd	4th	5th	6th	7th	8th	9th	10th	11th	12th
Seven Stories	Boston	24	20	20	20	20	20	16					
	New York	28	24	24	20	20	16	16					
	Chicago	20	20	20	20	16	16	16					
	Minneapolis	20	20	20	16	16	16	12					
	St. Louis	26	26	22	22	18	18	13					
	Denver	26	21	21	21	17	17	17					
	New Orleans	22	22	18	18	18	13	13					
Eight Stories	Boston	28	24	20	20	20	20	20	16				
	New York	32	28	24	24	20	20	16	16				
	Chicago	24	24	20	20	20	16	16	16				
	Minneapolis	24	20	20	20	16	16	16	12				
	St. Louis	30	26	26	22	22	18	18	13				
	Denver	30	26	21	21	21	17	17	17				
	New Orleans	22	22	22	18	18	18	13	13				
Nine Stories	Boston	28	24	24	20	20	20	20	20	16			
	New York	32	32	28	23	24	20	20	16	16			
	Chicago	24	24	24	20	20	20	16	16	16			
	Minneapolis	24	24	20	20	20	16	16	16	12			
	St. Louis	30	30	26	26	22	22	18	18	13			
	Denver	30	26	26	21	21	21	17	17	17			
Ten Stories	Boston	28	28	24	24	20	20	20	20	20	16		
	New York	36	32	32	28	24	24	20	20	16	16		
	Chicago	28	28	24	24	24	20	20	20	16	16		
	Minneapolis	24	24	24	20	20	20	16	16	16	12		
	St. Louis	34	30	30	26	26	22	18	18	13	13		
	Denver	30	30	26	26	26	21	17	17	17	17		
Eleven Stories	Boston	36	36	32	28	28	24	20	20	20	20	16	
	New York	36	36	32	28	28	24	24	20	20	16	16	
	Chicago	28	28	24	24	24	20	20	20	16	16	16	
	St. Louis	34	34	30	30	26	26	22	22	13	18	13	
	Denver	30	30	26	26	26	21	21	21	17	17	17	
Twelve Stories	Boston	36	36	32	32	28	28	24	20	20	20	20	16
	New York	40	36	36	32	32	28	24	24	20	20	16	16
	Chicago	28	28	28	24	24	24	20	20	20	16	16	16
	St. Louis	34	34	34	30	30	26	26	22	22	18	18	13
	Denver	30	30	30	26	26	26	21	21	21	17	17	17

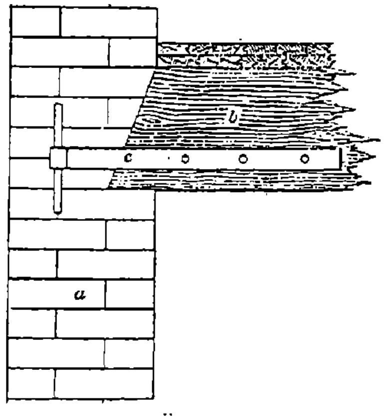


Typical roof/floor span system
Wood joists
Wood post and beam (heavy timber or mill construction)
Wood truss

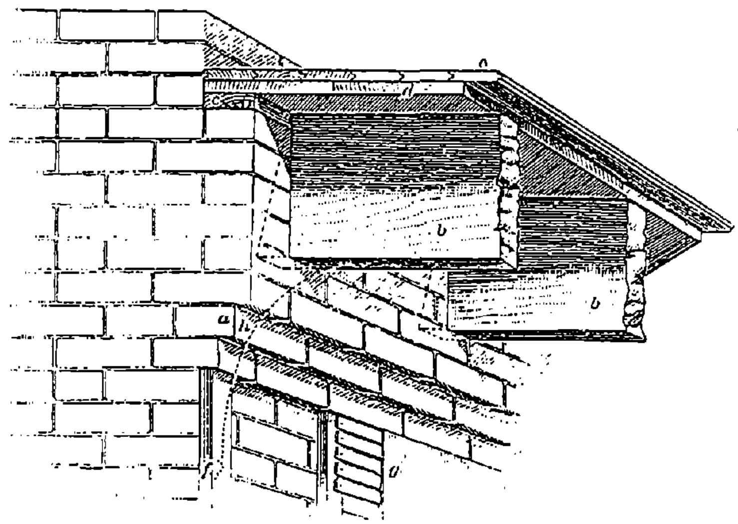
Typical roof/floor diaphragms
Diagonal Sheathing
Straight sheathing

Wall systems
Brick bearing wall, 2 to 5 wythes thick
Nonstructural wood stud interior partitions
Details (may or may not be present)
Unbraced parapet and cornice
Uniform fenestration on street facades
solid walls adjacent to other buildings
Light/ventilation wells
Large openings for ground floor shops

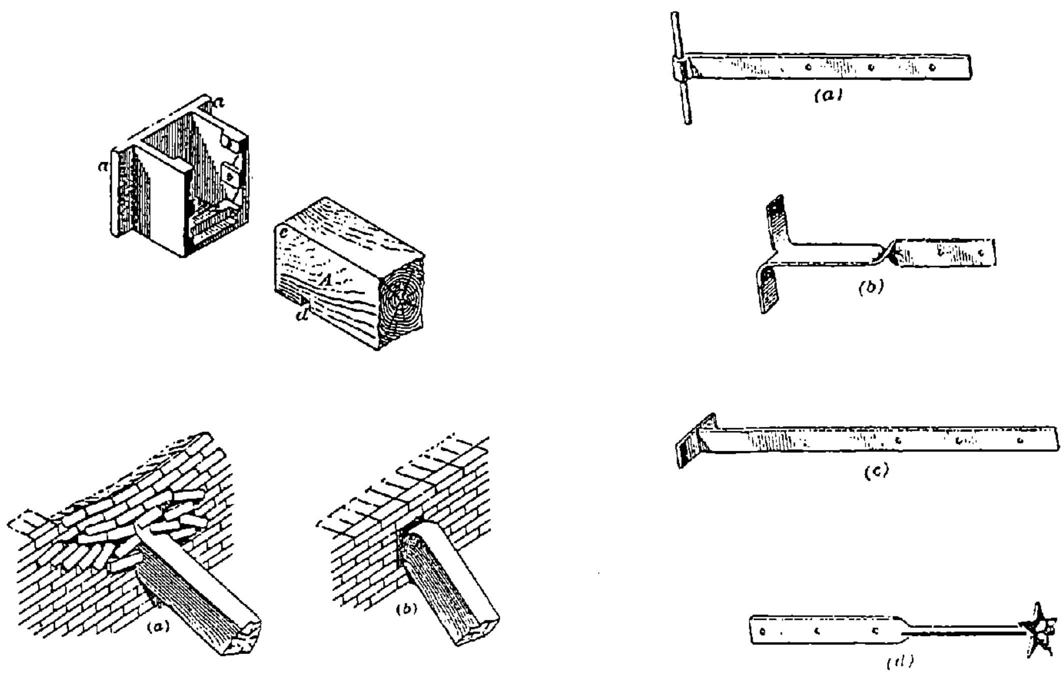
Figure B-1: Typical URM Bearing Wall Building For Which California-Type Retrofit Schemes Were Developed (Adapted from Wong, 1987)



(a) Typical Joist Anchor



(b) Corbel



(c) Typical Anchorage Hardware

Figure B-2: Typical Joist Anchorage Details and Hardware, Circa 1900 (Lavicka, 1980)

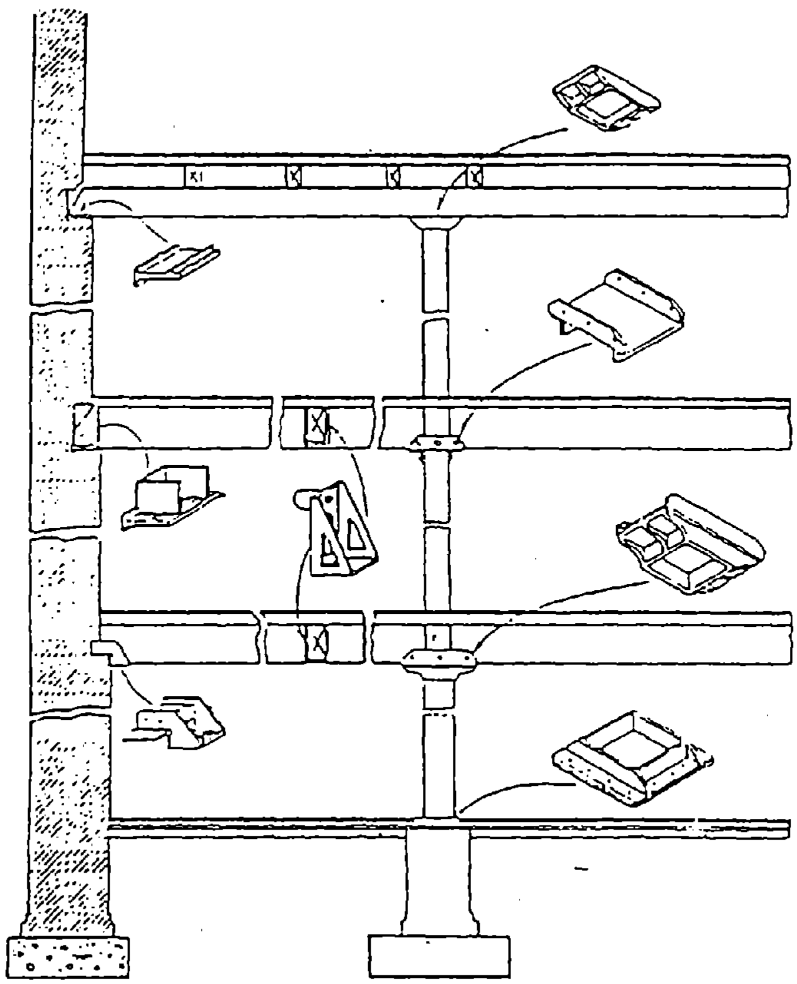
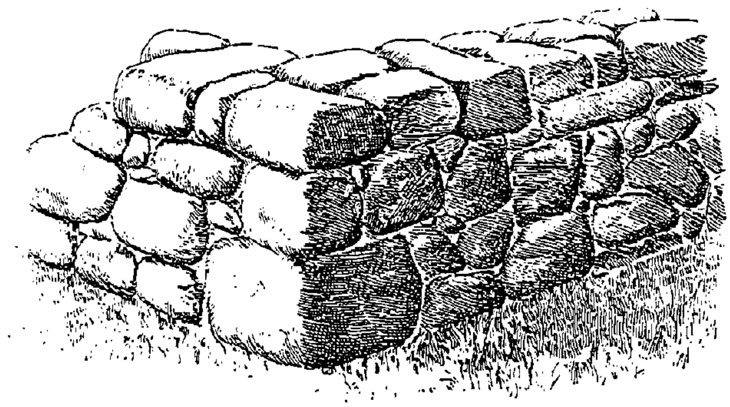
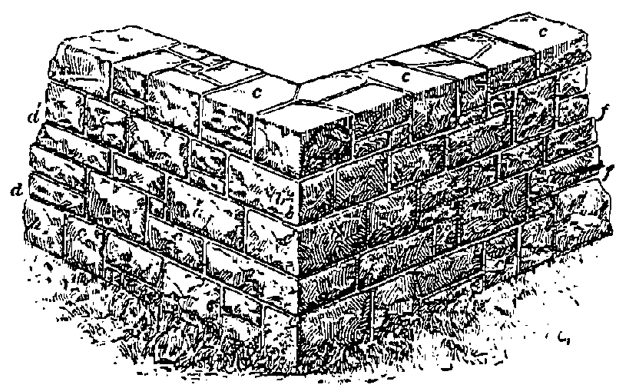


Fig. 79. Mill-construction. Combination Post-caps, etc.

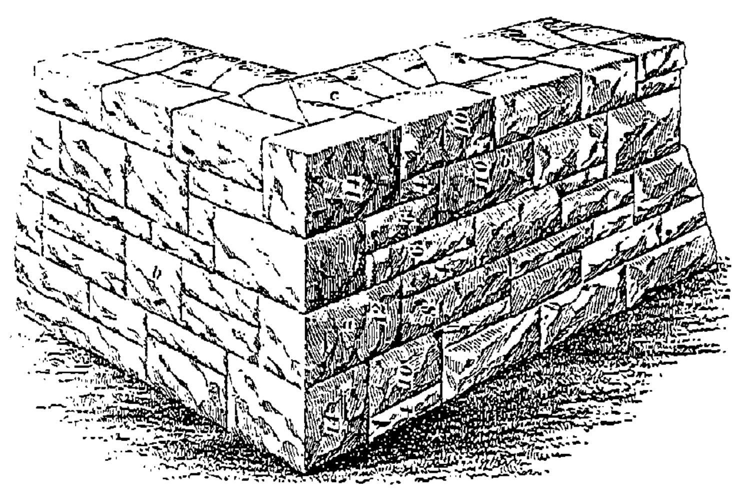
Figure B-3: Mill Construction Circa 1890. Steel I-beam Floor Framing and Cast-Iron Interior Column (Kidder, 1916)



(a) Stone



(b) Coursed Rubble



(c) Broken Ashlar

Figure B-4: Stone, Coursed Rubble, and Broken Ashlar Walls (Lavicka, 1980)

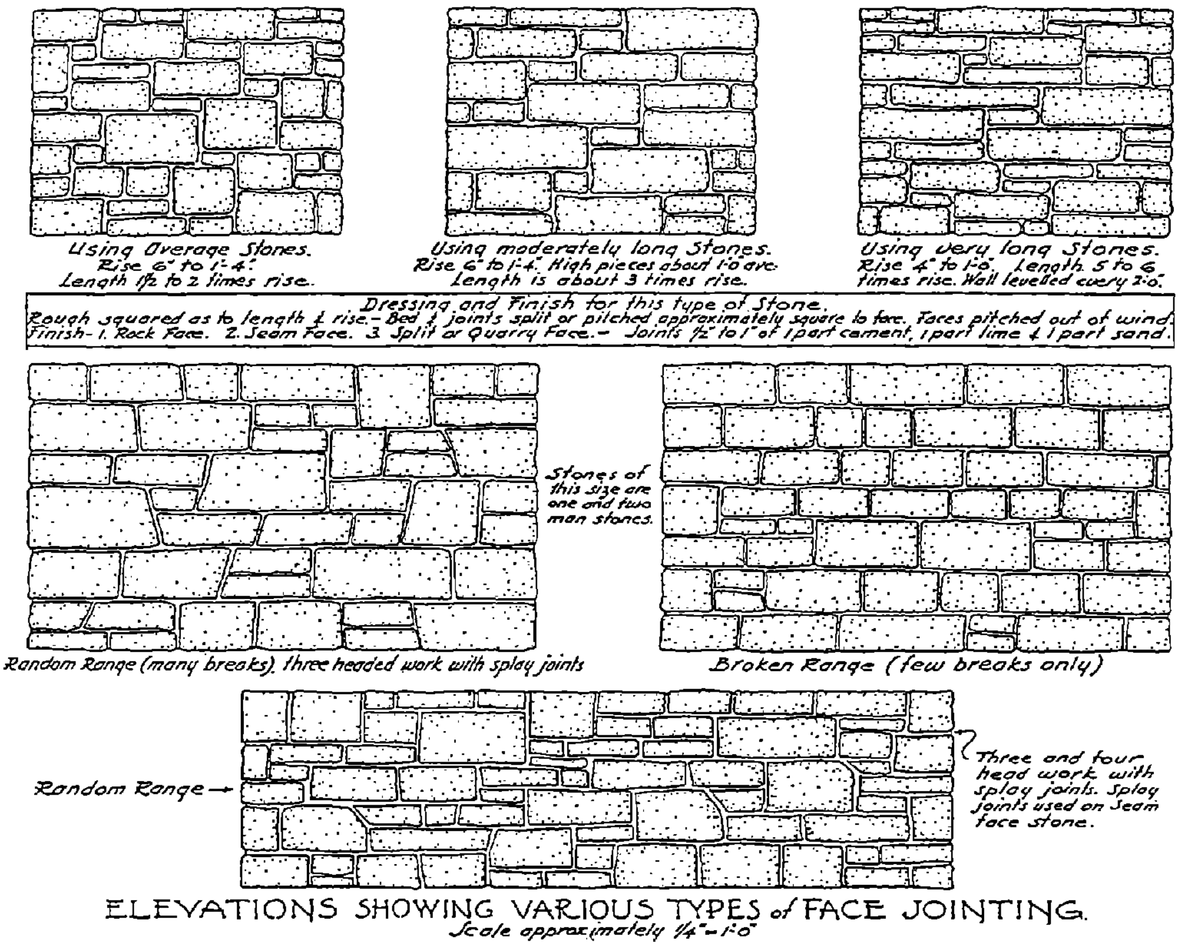
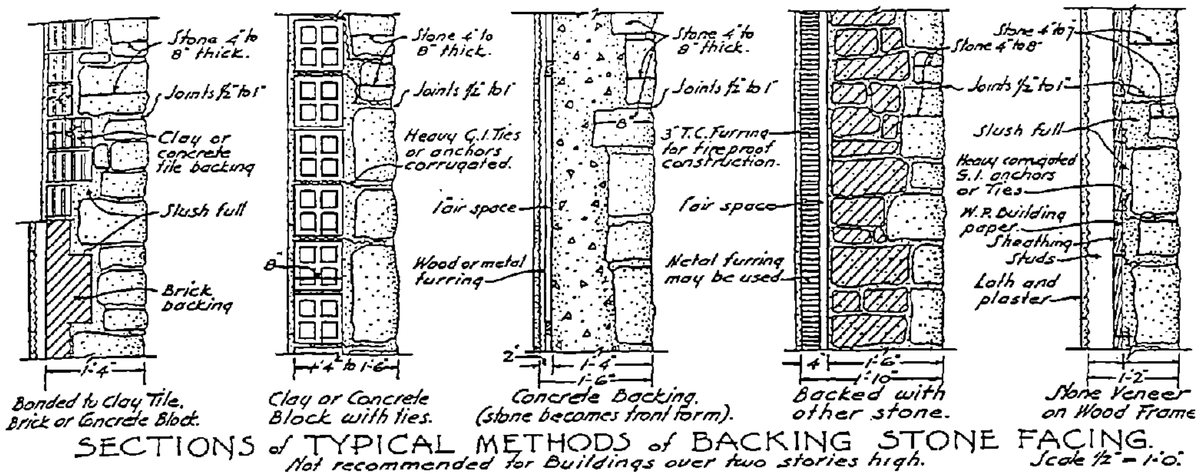
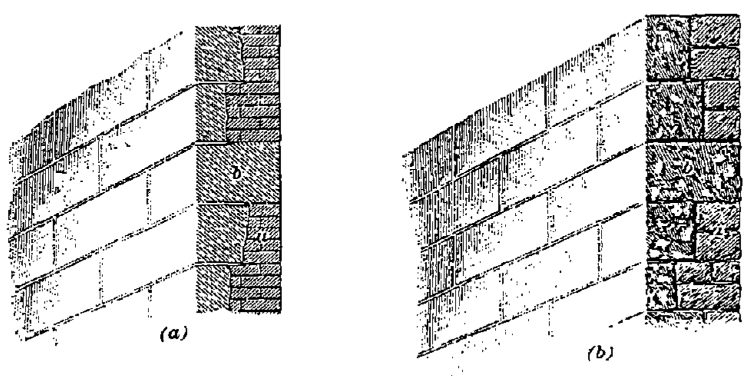


Figure B-5: Elevations Showing Various Types of Face Jointing (Belle, et al., 1991)



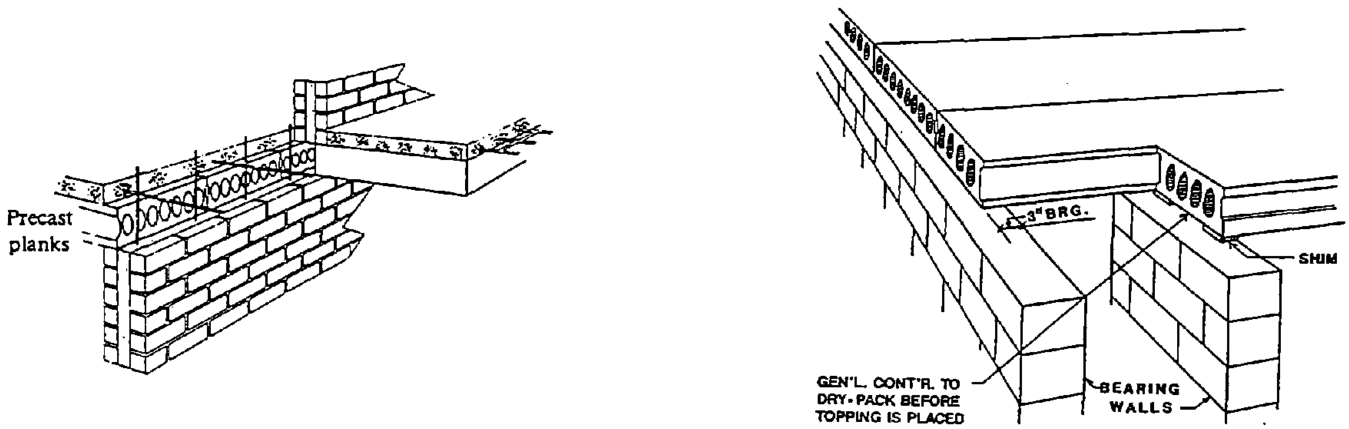
(a) Backing Stone Facing (Belle, et. al., 1991)



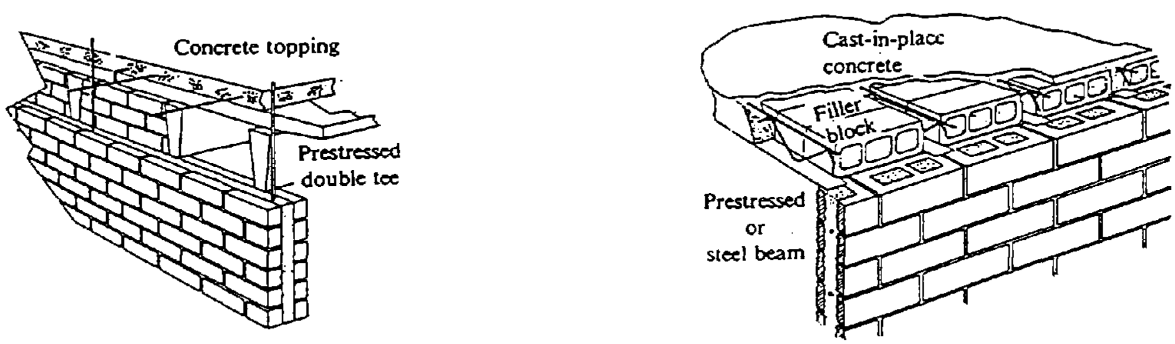
(b) Ashlar Facing with Brick and Stone Backing (Lavicka, 1980)

Figure B-6: Sections of Typical Methods of Backing

(a) Concrete Slab

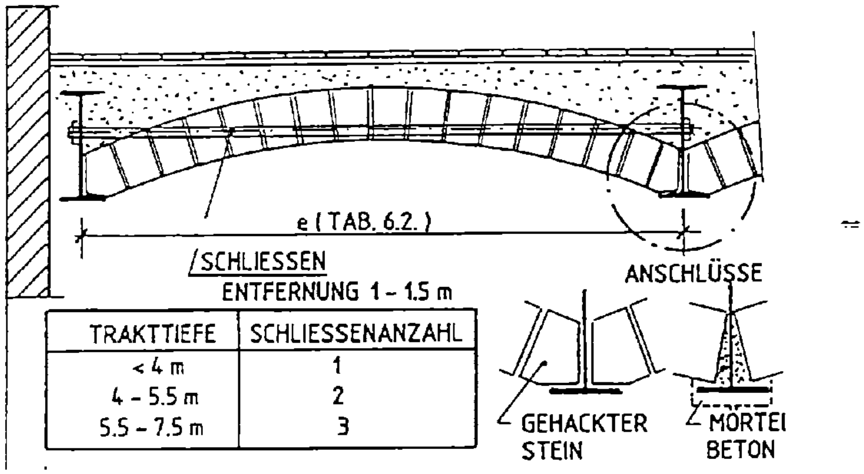


(b) Hollow Plank

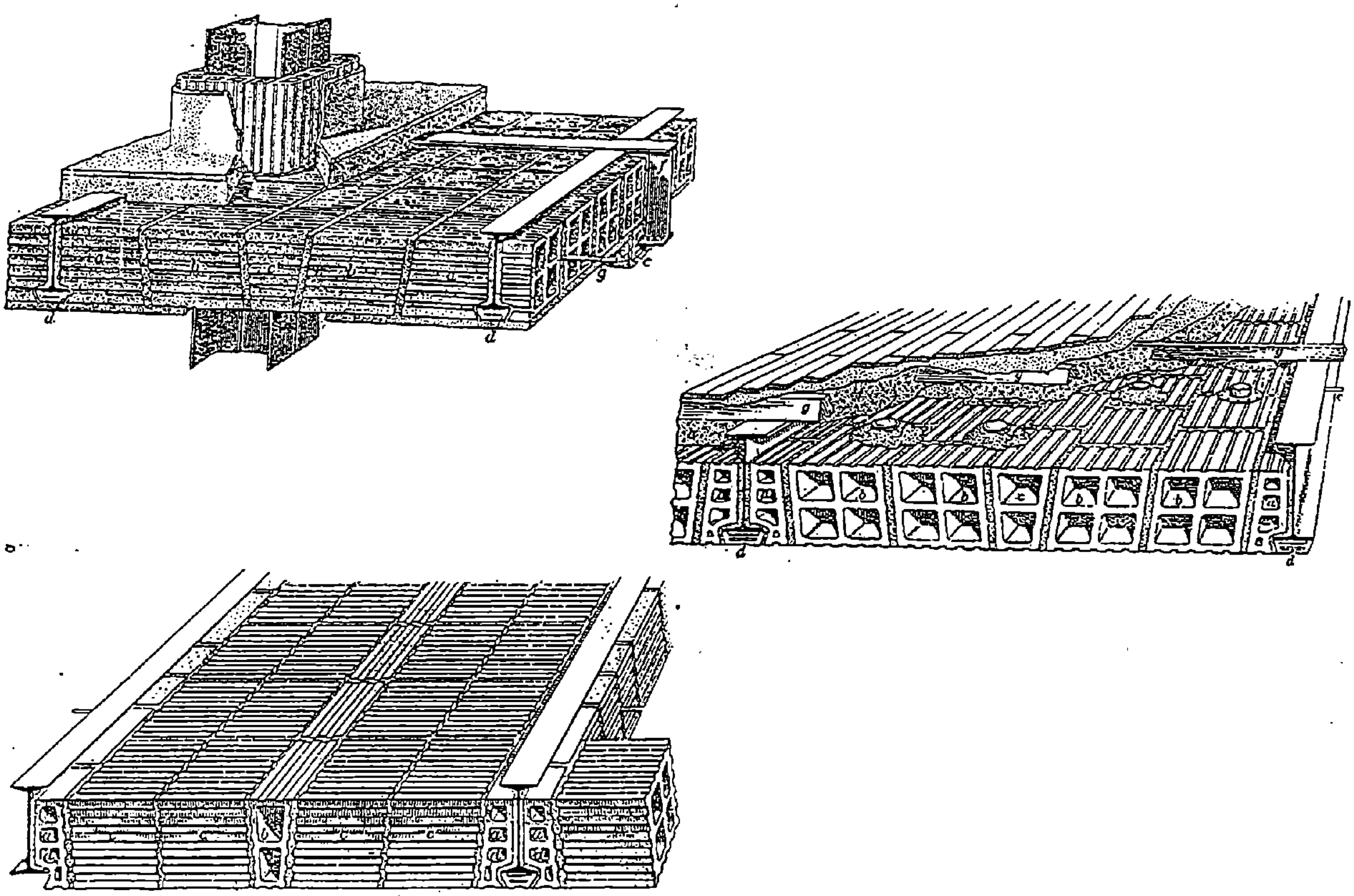


(c) Precast Joists

Figure B-7: Concrete Diaphragms (Amrhein, 1983)

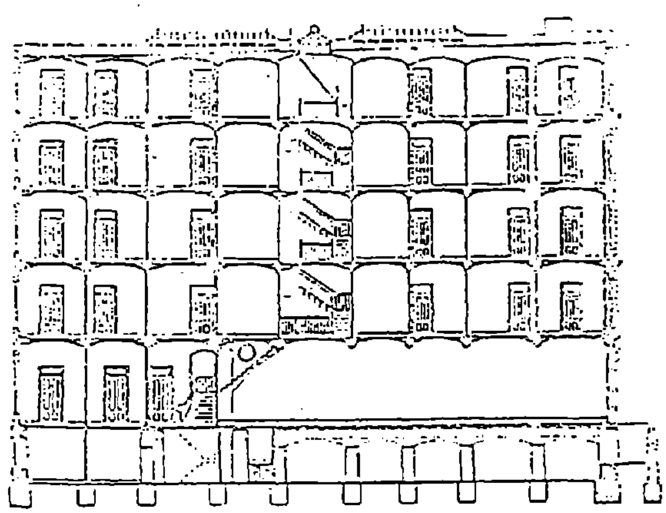


(a) Brick arch spanning between steel I-beams (Kolbitsch, 1989)



(b) Hollow clay tile flat arch spanning between steel I-beams (Lavicka, 1980)

Figure B-8: Masonry Diaphragms



(a) Section

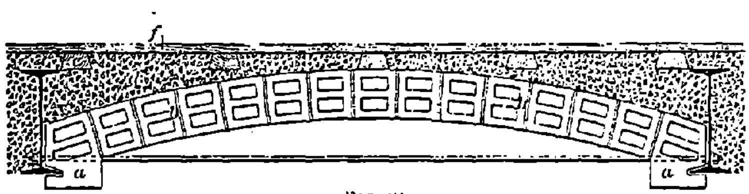


FIG. 84.

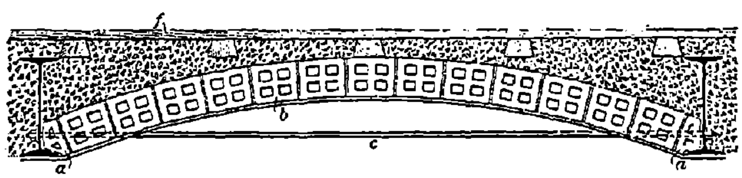
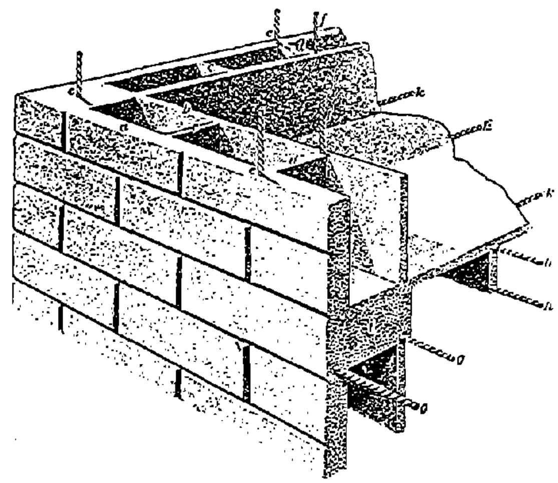


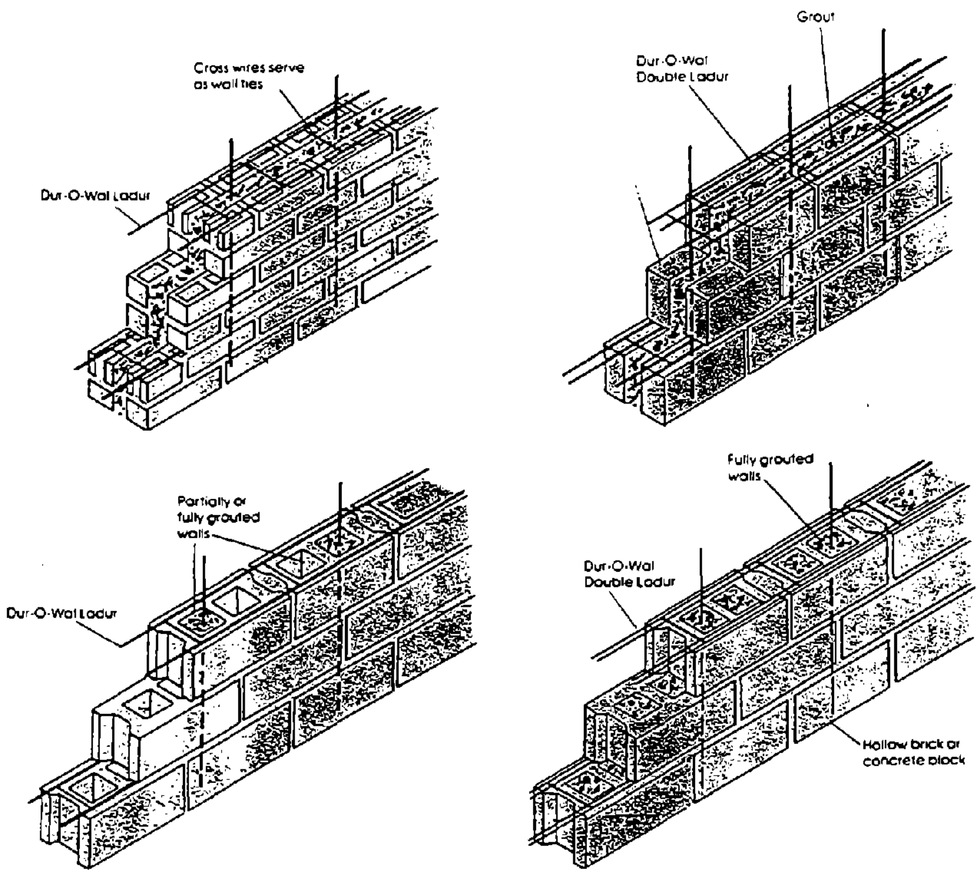
FIG. 83.

(b) Floor Detail

Figure B-9: Longitudinal Section through Manhattan Apartment Circa 1880 with Tile Arches from Cellar to Roof (Brickbuilder, 1901)



(a) Circa 1900



(b) Dur-O-Wal Construction, Present

Figure B-10: Hollow Concrete Wall Construction

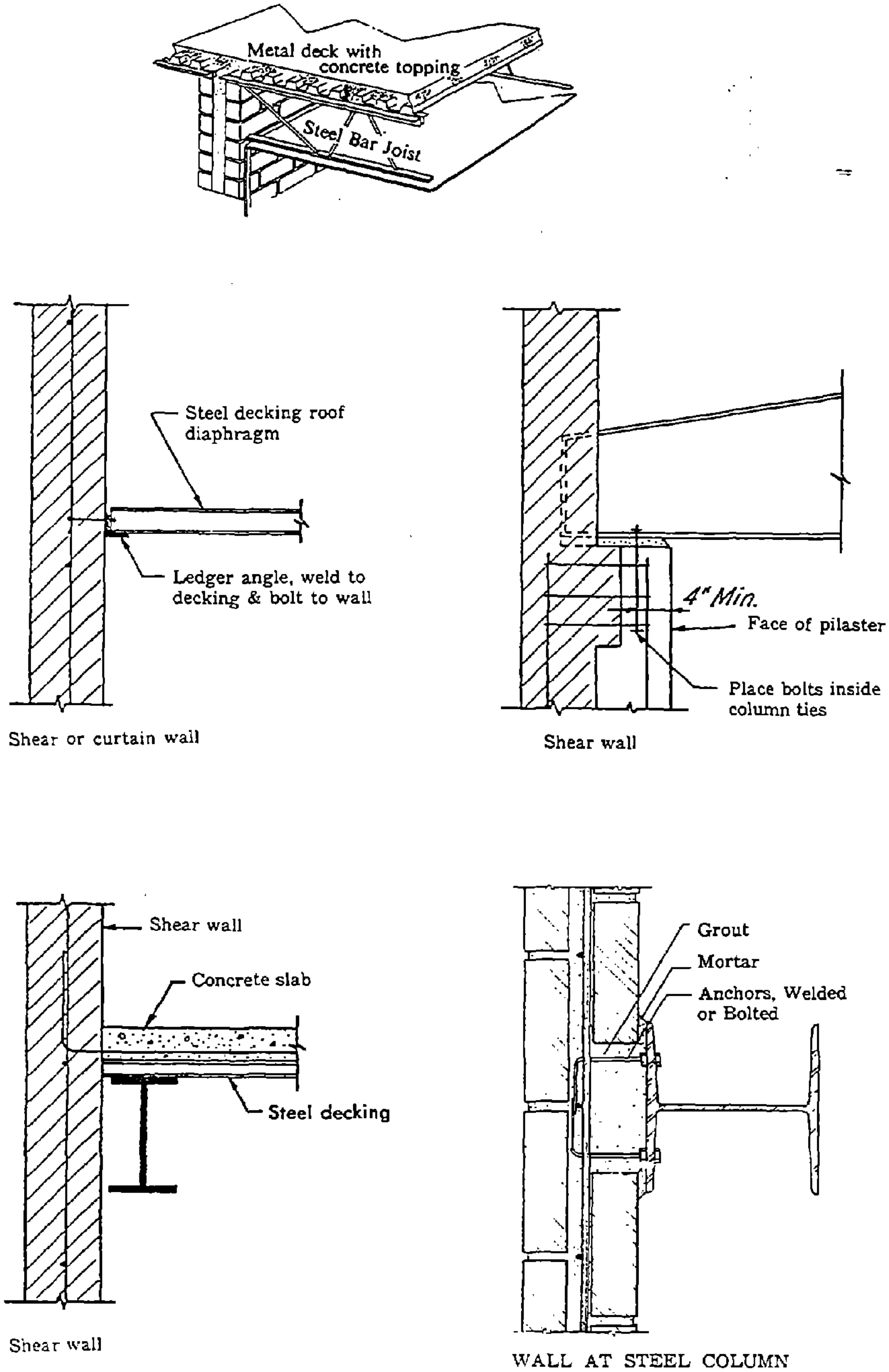


Figure B-11: Metal Deck Diaphragm Details (Amrhein, 1983)

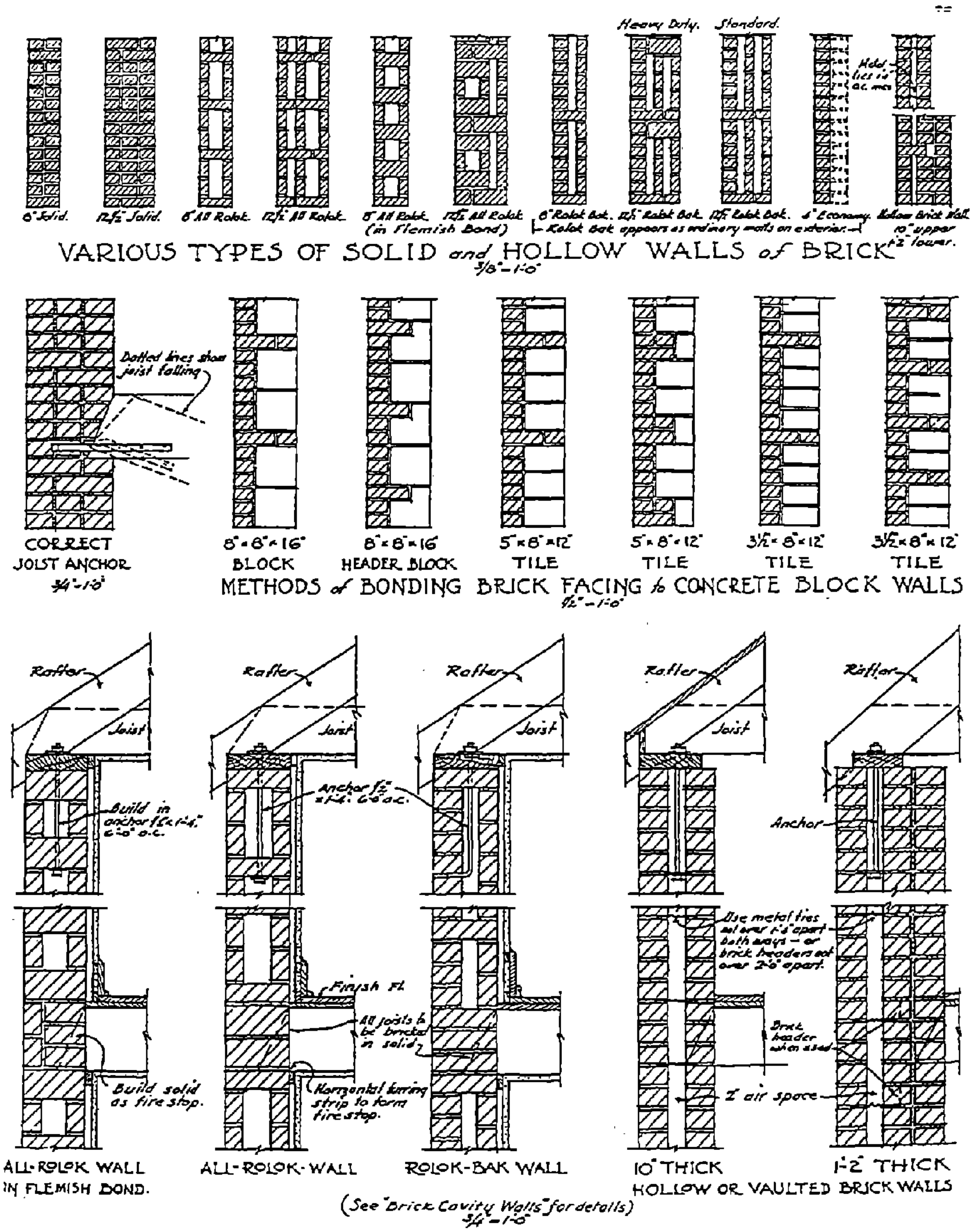
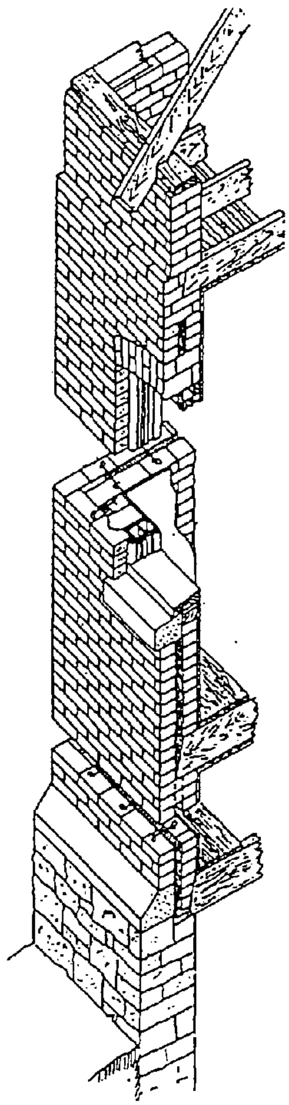
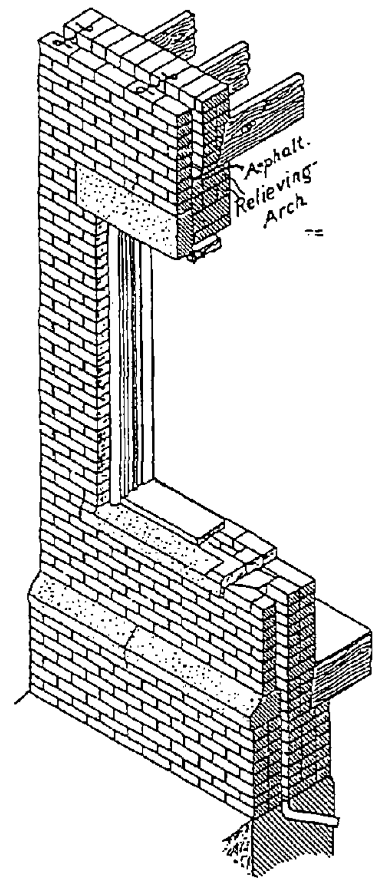


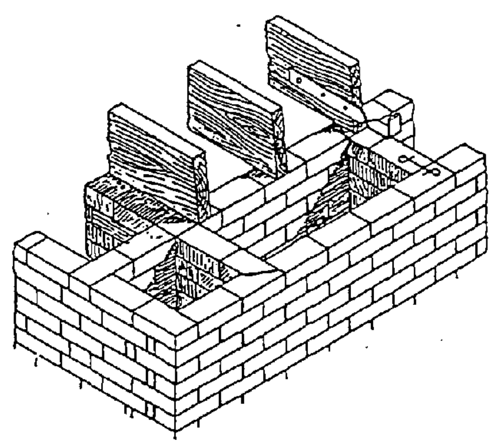
Figure B-12: Various Types of Solid and Hollow Walls of Brick (Belle, et al., 1991)



(a) Brick Cavity Wall Construction
for Inexpensive Cottage Construction

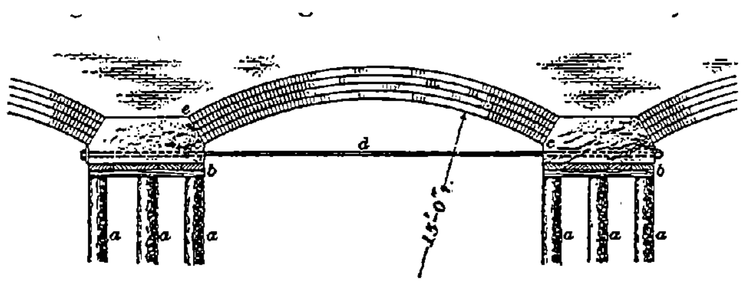


(b) Brick Cavity Wall Construction
for Two-Story Buildings

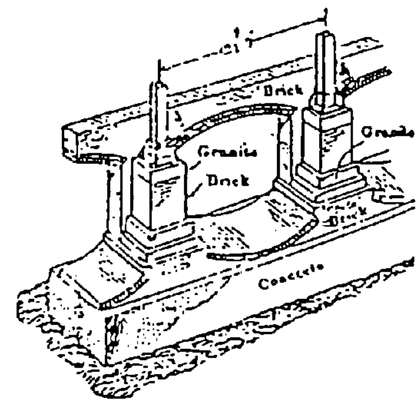


c) Brick Cavity Wall Construction.
Congress Hall, Sarratoga, N.Y.

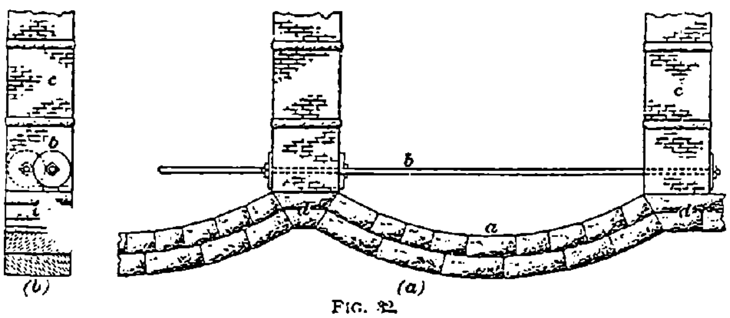
Figure B-13: Brick Hollow Wall Construction Types (Lavicka, 1980)



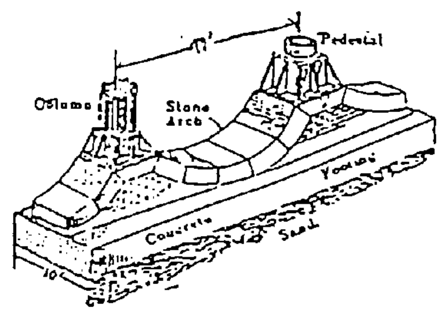
(a) Cluster Piles with Brick Anchors



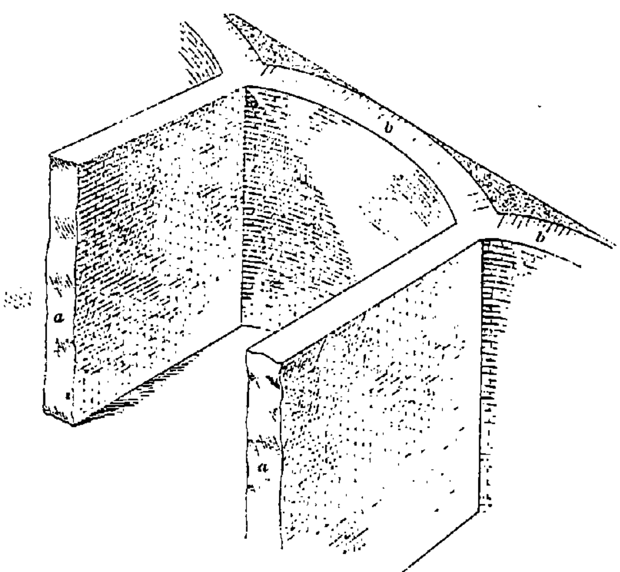
(b) Inverted-arch Footing World Building, New York



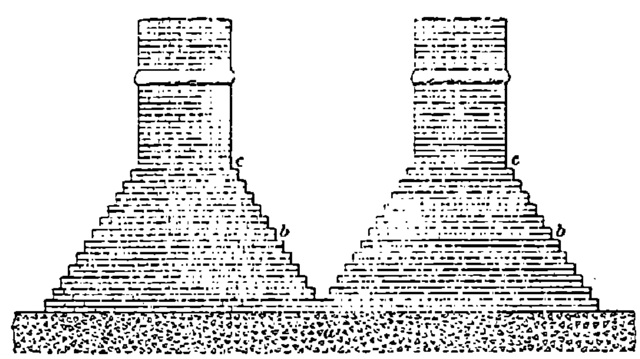
(c) Inverted Stone Arch



(d) Inverted-arch Footing, Drexel Building, Philidelphia



(e) Brick Vaults



(f) Stepped Footing

Figure B-14: Masonry Footings (Lavicka, 1980)

B.3 Survey Form

The following survey form was sent to a number of colleagues nationwide with experience in URM building retrofit. Survey respondents were:

- John Theiss, President, Theiss Engineers, Inc., St. Louis, MO
- John Hooper, Technical Director, EQE/RSP, Seattle, WA
- Leo Argiris, Ove Arup & Partners, New York, NY
- Gene Corley, Construction Technology Laboratories, Inc., Skokie, IL
- Warner Howe, Consulting Structural Engineer, Germantown, TN
- James Harris, JR Harris & Co., Denver, CO

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303 Second Street
Suite 800 North
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Tel: (415) 495-4222
Fax: (415) 546-7536

and Boise, ID

August 30, 1995

Dear :

We are presently engaged in a study for the National Institute of Standards and Technology (NIST) to develop a framework for nationwide guidelines for enhanced seismic performance of unreinforced masonry (URM) bearing wall buildings. To allow these guidelines to be meaningful nationwide, investigations must be made of regional construction and retrofitting techniques. The current Uniform Code for Building Conservation (UCBC, published by ICBO as part of the UBC) provisions were developed from research and development done in Southern California. These provisions are commonly used for retrofit in California, and they are applicable to solid masonry buildings with flexible diaphragms. However, some URM buildings have characteristics that do not fit the UCBC prototype and may require different analytical approaches and retrofit methods. Consequently, we are gathering information on the existing URM inventory for a number of geographical regions, including your area. Your assistance in this endeavor would be greatly appreciated.

Attached is a short questionnaire which I would like you to fill out at your convenience. The first page shows an isometric view and details of typical California UMB construction; the next shows the many variations of the prototypical URM building found in San Francisco. The two page questionnaire which follows solicits basic data on URM building types found in your local inventory which you consider to be exceptions to the typical URM building. Please return the questionnaire to me at the above address at your earliest convenience.

Thank you for your assistance.

Sincerely,

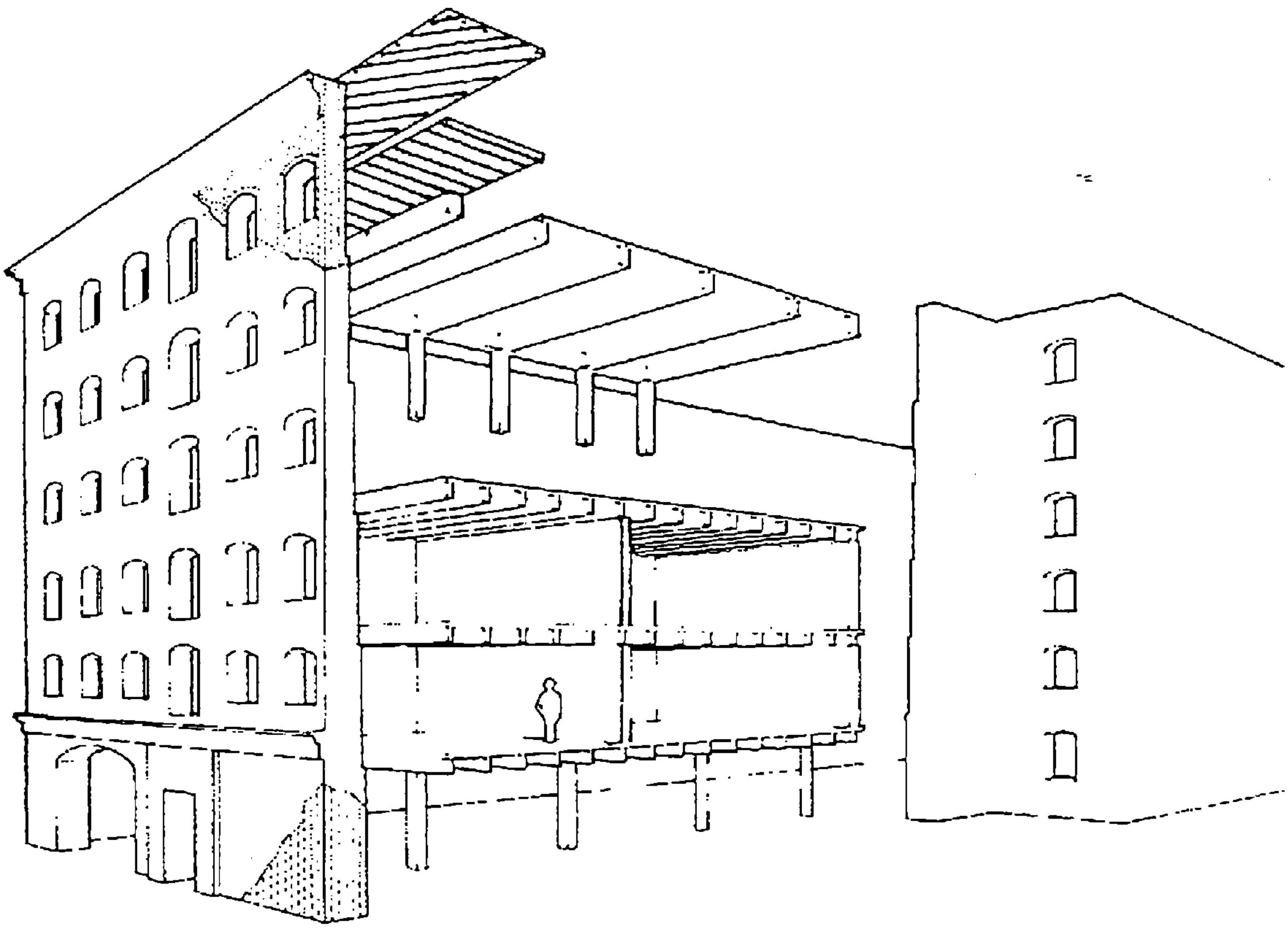
RUTHERFORD & CHEKENE

William T. Holmes
Principal

WTH/ml

Attachments

648ML.DOC



Typical Unreinforced Masonry Bearing Wall Building for which California-type Retrofit Schemes Developed

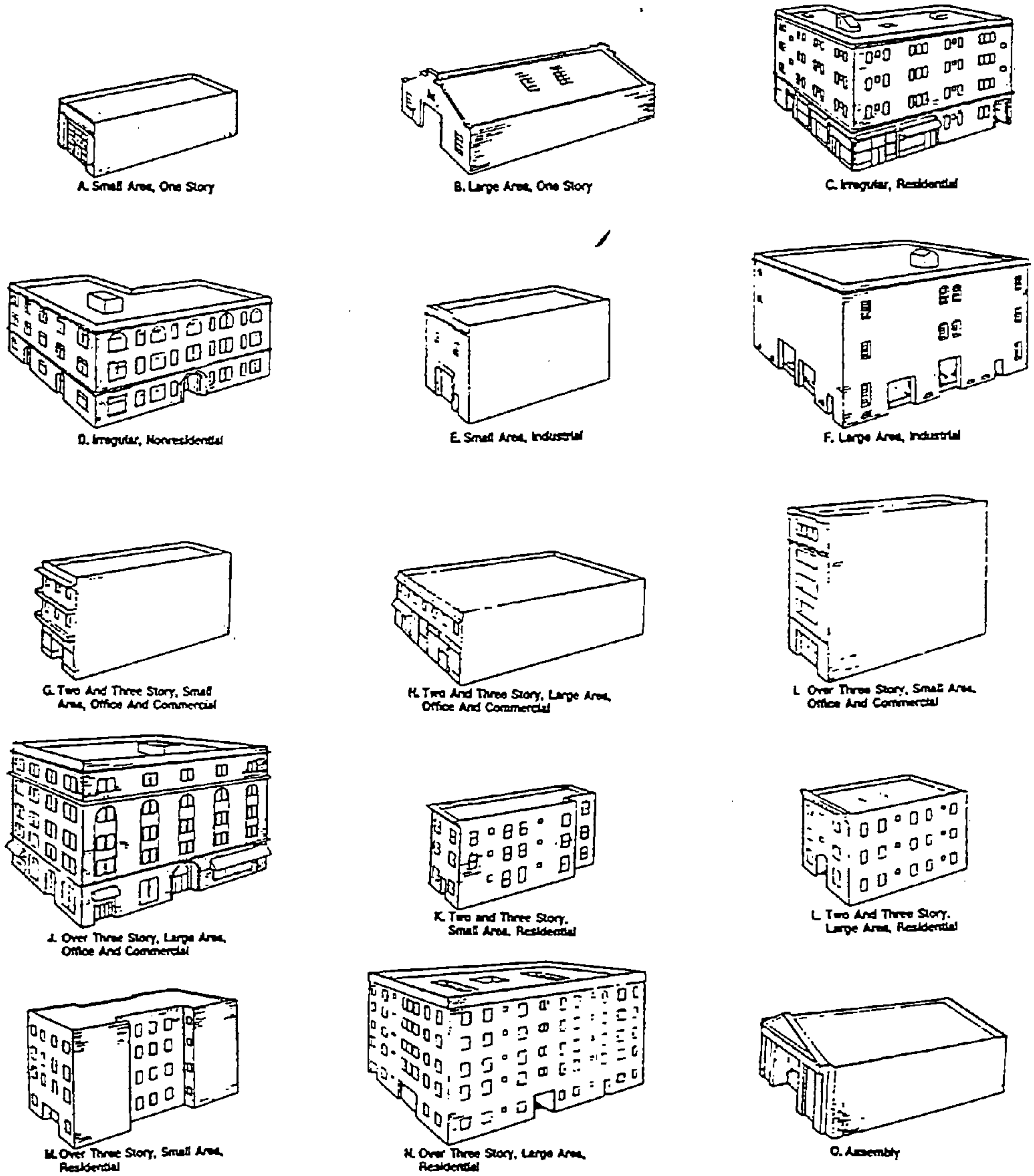
Typical roof/floor span system
Wood joists
Wood post and beam (heavy timber or mill construction)
Wood truss

Typical roof/floor diaphragms
Diagonal Sheathing
Straight sheathing

Wall systems
Brick bearing wall, 2 to 5 wythes thick
Nonstructural wood stud interior partitions

Details (may or may not be present)
Unbraced parapet and cornice
Uniform fenestration on street facades
solid walls adjacent to other buildings
Light/ventilation wells
Large openings for ground floor shops

Adapted from Lagorio, Freidman, and Wong, Issues for Seismic Strengthening of Existing Buildings.



Further Examples of Typical UMB Buildings

Please indicate the buildings which are common in your area
If possible, breifly describe or sketch any others which are exceptions to these prototypes

General

1.	Are there significant numbers of retrofitted URMs in your area?	YES	NO
2.	If you have retrofitted a URM, did you use the UCBC? If yes, did you find that the UCBC was applicable to your building types?	YES /YES	NO
3.	If you did not use the UCBC, which code or standard did you use?		
4.	Are URM buildings in your area typically found in pockets of similar vintage? If so, what is the vintage?	YES	NO
5.	After what date did the use of cement mortar become common?		
6.	Are interior walls typically covered with sheetrock or plaster	Sheetrock	Plaster

Exceptions	Frequency of occurrence		
	Predominant	Moderate	Few
I. **Diaphragms (Circle as many as are appropriate)**			
A. **Wood post and beam/wood sheathing (typical)**	☐	☐	☐
B. **Metal Deck**	☐	☐	☐
C. **Concrete diaphragms**			
1. Precast (hollow plank, precast joists)	☐	☐	☐
2. Cast-in-place concrete	☐	☐	☐
3. Pan joist systems	☐	☐	☐
4. Concrete slab spanning between steel I-beams	☐	☐	☐
5. Other	☐	☐	☐
D. **Masonry**			
1. Brick arch spanning between steel I-beams	☐	☐	☐
2. Hollow clay tile flat arch spanning between steel I-beams	☐	☐	☐
3. Other	☐	☐	☐
II. **Bearing Walls**			
A. Types			
1. Solid brick of multiwythe construction (typical)	☐	☐	☐
2. Cavity wall construction	☐	☐	☐
3. Rubble masonry	☐	☐	☐
4. Ashlar stone	☐	☐	☐
5. Structural clay tile	☐	☐	☐
6. Ungrouted CMU	☐	☐	☐
7. Other	☐	☐	☐
B. Masonry Mortar			
1. Weak and/or deteriorated mortar (insufficient strength for wall to act as a shear wall--probably lime mortar)	☐	☐	☐
2. Intermediate strength (Typical)	☐	☐	☐
3. Competent mortar (similar to modern construction--cement mortar)	☐	☐	☐
III. **Interior Partitions**			
A. Solid brick of multiwythe construction extending full height	☐	☐	☐
B. Hollow clay tile	☐	☐	☐
C. Wood or metal frame extending full story height	☐	☐	☐
D. Partial, with no significant restraint to interstory displacement	☐	☐	☐
IV. **Cladding and Veneer**			
A. Cladding and veneer types			
1. Stone	☐	☐	☐
2. Brick	☐	☐	☐
3. Terra-cotta	☐	☐	☐
4. Other	☐	☐	☐
B. Prevalence of cladding and veneer anchorage	☐	☐	☐

APPENDIX C
RESEARCH SUMMARIES OF URM WALL ENHANCEMENT METHODS

Preface

The following group of research summaries on seismic strengthening methods for unreinforced masonry walls was prepared by searching publications devoted to either earthquake engineering or masonry topics. Various literature sources were relied on including the information service at the National Center for Earthquake Engineering Research, the bibliography of masonry research from the International Masonry Institute, the ASCE listing of relevant research compiled for the ATC-33 project and the National Information Services Corporation's *Earthquakes and the Built Environment Index.* In addition, publications in Dr. Abrams' personal collection were reviewed for appropriate citations contained in research reports, or conference proceedings and journals on masonry or earthquake engineering.

Individual publications were reviewed for their relevance to the topic of seismic strengthening methods for unreinforced masonry walls. The essential content of appropriate publications was extracted to create the research summaries which follow. 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.

The objective was not to compress all information contained in a publication to fit within a few pages, but to provide a general overview of why the research was done, what was tested, how it was tested and what was found from an investigation. Selected illustrations and tables were scanned and reprinted in these summaries to help illustrate the test methods and test results. For further information on any one research investigation, the reader is encouraged to seek the original document.

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.

References

Grout or Epoxy Injections

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GI2. Binda, L., G. Baronio, and A. Fontana, "Strengthening and Durability of Decayed Brick-Masonry Repaired by Injections," *Proceedings of the Fifth North American Masonry Conference*, University of Illinois at Urbana-Champaign, June 1990, Vol. 2, pp. 839-852.

GI3. Binda, L., C. Modena and G. Baronio, "Strengthening of Masonries by Injection Technique," *Proceedings of the Sixth North American Masonry Conference*, Drexel University, Philadelphia, June 1993, Vol. 1, 1993, pp. 1-14.

GI4. Calvi, G.M. and G. Magenes, "Experimental Results on Unreinforced Masonry Shear Walls Damaged and Repaired," *Proceedings of the Tenth International Brick and Block Masonry Conference*, Vol. 2, 1994, pp. 509-518.

GI5. Faella, G., G. Manfredi and R. Realfonzo, "Cyclic Shear Behavior of Tuff Masonry Walls Strengthened by Grout Injections and Reinforcement," *Proceedings of the Tenth International Brick and Block Masonry Conference*, University of Calgary, July 1994, Vol. 1, pp. 239-248.

GI6. Manzouri, T., P.B. Shing, M.P. Schuller, and R.H. Atkinson, "Repair of Unreinforced Masonry Structures with Grout Injection Techniques," *Proceedings of Seventh North American Masonry Conference*, University of Notre Dame, June 1996, pp. 472-483.

GI7. Plecnik, J.M., J.E. Amrhein, J. Warner, W.H. Jay and C.V. Chelapati, "Repair of Earthquake Damaged Concrete Masonry Systems Subjected to Static and Dynamic Loads and Elevated Temperatures," *Proceedings of the Sixth World Conference on Earthquake Engineering*, New Delhi, 1977, pp. 2492-2498.

GI8. Roselund, N.A, and S. Pringle, "Repair of Cracked Unreinforced Brick Walls by Injection of Grout", *Proceedings of the Fourth U.S. National Conference on Earthquake Engineering*, 1990, Vol. 3, pp. 283-292.

GI9. Schuller, M.P., R.H. Atkinson, and J.T. Borgsmiller, "Injection Grouting for Repair and Retrofit of Unreinforced Masonry," *Proceedings of the Tenth International Brick and Block Masonry Conference*, Vol. 2, 1994, pp. 549-558.

GI10. Sheppard, P., and S. Tercelj, "The Effect of Repair and Strengthening Methods for Masonry Walls," *Proceedings of the Seventh World Conference on Earthquake Engineering*, Istanbul, Vol. 6, September 1980, pp. 255-262.

GI11. Shing, P.B., T. Manzouri, R.H. Atkinson, M. P. Schuller and B. Amadei, "Evaluation of Grout Injection Techniques for Unreinforced Masonry Structures," *Proceedings of the Fifth U.S. National Conference on Earthquake Engineering*, Chicago, July 1994, Vol. 3, pp. 851-860.

G12. Tomasevic, M., and V. Apih, "The Strengthening of Stone-Masonry Walls by Injecting the Masonry-Friendly Grouts," *Journal of European Earthquake Engineering*, 1993, pp. 10-20.

Surface Coatings

SC1. Alcocer, S.M., J. Ruiz, J.A. Pineda, and J.A. Zepeda, "Retrofitting of Confined Masonry Walls with Welded Wire Mesh," *Proceedings of Eleventh World Conference on Earthquake Engineering*, Acapulco, June 1996, Elsevier Science Ltd., Paper No. 1471.

SC2. Ehsani, M.R. and H. Saadatmanesh, "Seismic Retrofitting of URM Walls with Fiber Composites," *The Masonry Society Journal*, Spring 1996.

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SC5. Irimies, M., and L. Crainic, "Behavior of Repaired/Strengthened Unreinforced Masonry Shear Walls," *Proceedings of the Sixth North American Masonry Conference*, Drexel University, June 1993, pp. 555-563.

SC6. Jabarov, M., S.V. Kozharinov and A.A. Lunyov, "Strengthening of Damaged Masonry by Reinforced Mortar Layers," *Proceedings of the Seventh World Conference on Earthquake Engineering*, Vol. 15, No. 3, 1985, pp. 73-80.

SC7. Kahn, L.F., "Shotcrete Retrofit for Unreinforced Brick Masonry," *Proceedings of the Eighth World Conference on Earthquake Engineering*, San Francisco, July 1984, pp. 583-590.

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Grout or Epoxy Injections

GI1 Experimental Study on Decayed Brick-Masonry Strengthened by Grouting

L. Binda, G. Baronio, A. Fontana and G. Frigerio

Evaluation and Retrofit of Masonry Structures, JOINT USA-ITALY WORKSHOP, Aug. 19-29, 1987, pp. 111-122.

Objective

The objective of the research was to study the effect of grouting by injection on strength and durability of decayed masonry. In particular, the study focused on how grouting influences the mechanical behavior of masonry, and on compatibility problems related to strength, deformability and water absorption.

Rehabilitation Procedure

Grout is injected into internal voids of unreinforced clay-unit masonry.

Research Approach

A total of 25 test prisms were made with solid clay bricks and three types of mortar (M1, M2, M3). Prisms were nominally 25 x 52 x 60 cm. Some of the test prisms were subjected to crystallization to simulate decay. All of the prisms were tested in compression.

Test prisms that had not completely failed were repaired by injection, some with an epoxy resin at 18 °C, and some with a cement-polymer mortar. The injection process was performed after the external surface of the cracked prisms were sealed with a thick epoxy paint. After completing the injection and curing process, the prisms were again subjected to compression.

The durability of injected prisms to different environmental conditions was detected by a crystallization test and a compression test. Three of the prisms repaired by epoxy were subjected to the crystallization test.

The test prisms were sliced into 5 pieces to inspect the condition of the crack pattern after the salt damage.

Summary and Significant Findings

An increase in strength was observed with grout injections. The improved strength and elastic modulus for some test prisms were higher than values for the original specimens.

Due to the high porosity of the mortar, the mortar joint failed during the crystallization test.

The number of injections, the injection pressure and the quantity of injected epoxy resin influenced the masonry strength. Epoxy resin was found to be better than cement-polymer grout.

The degree of penetration and diffusion of the injected grout also influenced the water penetration inside the repaired masonry and its behavior under a temperature variation.

The salt decomposition and crystallization showed that the resin had impregnated the bricks instead of the filling voids and cracks. In most of the cases, bad diffusion of injection material was observed. The salt easily filled the voids along the joints or along the cracks and flaws. The cracks were widened by the crystallized salts.

Temperature variation influenced behavior of masonries repaired by polymeric grout as a result of the stress change induced by the deformation of the injected material.

Related Reference

Binda, L., G. Baronio, and A. Fontana, "Strengthening and Durability of Decayed Brick-Masonry Repaired by Injections," *Proceedings of the Fifth North American Masonry Conference*, University of Illinois at Urbana-Champaign, June 1990, Vol. 2, pp. 839-852.

GI2 Strengthening and Durability of Decayed Brick-Masonry Repaired by Injections

L. Binda, G. Baronio and A. Fontana

Proceedings of the Fifth North American Masonry Conference, University of Illinois, June 1990, Vol. 2, pp. 839-852.

Objective

The objective of this research was to investigate the influence and effectiveness of epoxy—formulated resin injection for improving strength and durability of decayed brick masonry.

Rehabilitation Procedure

Epoxy-formulated resins are injected into voids of an unreinforced brick masonry wall using a procedure that is similar to grout injection.

Research Approach

A series of masonry prisms (Table 1) were subjected to uniaxial compressive forces to determine the influence of injection grouting on compressive strength. One type of solid brick was used with three types of mortar: pozzolan-lime (M1), cement-lime (M2), and high strength cement modified with acrylic resins (M3). The third digit in the prism name of Table 1 represents one of these three mortar types. Some prisms were subjected to a crystallization test to simulate decay due to external aggressive environments before compression testing, and are noted in the table with a "T" as the fourth digit. Prisms which did not collapse were injected with an epoxy resin, or a grout composed of cement and 10% of the same epoxy resin, as a repair measure after compression testing.

A second series of eight prisms (250 x 510 x 600 mm) were constructed with solid bricks and a cement-lime mortar (1:3:5 parts of cement, lime and sand). Two resins were used: Types S and M as noted with the first digit of prism name in Table 2. Type S was a commercial resin with a compressive strength of 90 MPa and a flexural strength of 100 MPa. Type M resin was an experimental resin with a compressive strength of 105 MPa and a flexural strength of 90 MPa.

All eight prisms were loaded to failure in compression using a cyclic loading pattern. Specimens were loaded until 80% of the maximum compressive strength was reached, at which time they were unloaded and then reloaded to the onset of cracking. At that time four test prisms were injected with the Type S resin, while the remaining specimens were injected with the Type M. resin. After curing for 10 days, the test prisms were retested using the same loading history. Some of the specimens were also subjected to thermal cycles to check the durability of the injected assemblages with a temperature change

Summary and Significant Findings

A comparison of stress-strain curves for the first series of test prisms with and without epoxy resins is shown in Figure 1. The strength of the injected prisms approached a value of 85% of the strength of the original undamaged prisms. The injections did not change deformation characteristics appreciably, but the injected prisms were slightly more brittle than the virgin specimens.

Stress-strain curves for the second series of injected test prisms are contrasted with curves for virgin specimens in Figure 2. The use of epoxy resin injection for damaged specimens was shown to produce a stronger and stiffer test prism when compared to its original state. Resin injections increased the compressive strength of the prism 31.2% beyond that of an undamaged prism. Additionally, the elastic modulus of the injected prisms was 28% larger than that of the undamaged prisms.

When damaged masonry was injected under wet conditions, bond strength and durability under thermal cycles were reduced. Even with dry conditions, durability of repaired masonry under thermal cycles depended on the physical and mechanical properties of the resin.

Related Reference

Binda, L., G. Baronio, A. Fontana and G. Frigerio, "Experimental Study on Decayed Brick-Masonry Strengthened by Grouting," *Evaluation and Retrofit of Masonry Structure, Joint USA-ITALY Workshop*, Aug. 19-29, 1987, pp. 111-122.

Table 1: Summary of Test Results

Prisms name	type of grout.	N. of inject.	quant. of material daN	inject. pressure MPa	original strength MPa	strength after inject. MPa
MU11	epoxy	7	5.0	0.3	9.0	7.3
MU12	"	9	3.0	0.3	11.5	10.4
MU13*	"	3	1.0	0.3	13.0	
MU21*	"	6	4.5	0.3	15.5	
MU24	"	6	4.5	0.2	16.3	16.4
MU25	"	9	3.0	0.4	15.8	10.3
MU26	grout	5	1.8	0.35	6.5	3.8
MU32*	epoxy	3	0.8	0.3	17.0	
MU35	"	3	2.0	0.1	16.1	9.5
MU1T1	grout	6	5.0	0.25	7.1	6.8
MU1T2	epoxy	8	8.0	0.55	8.7	10.8
MU2T1	"	5	2.5	0.3	16.0	14.5
MU2T4	grout	6	1.4	0.6	17.0	3.7
MU3T1	epoxy	5	5.0	0.35	17.5	13.6
MU3T2	"	4	5.0	0.3	18.0	21.2
MU3T4	"	6	5.5	0.1	16.3	22.2
MU3T5	grout	5	0.5	0.7	17.1	3.7

* prisms which were not tested

Table 2: Summary of Test Results

Prisms name	N. of injection /m^2	quant. of epoxy N/m^3	f_{ud} MPa	f_{da} MPa	f_{ai} MPa
SD1	18	719	11.06	7.37	16.41
SD2	13	523	10.05	7.62	14.99
MD1	8	189	13.21	8.99	16.63
MD2	8	627	8.94	5.72	16.94
SW1	8	334	8.96	6.19	14.97
SW2	13	366	10.99	6.99	14.20
MW1	14	333	12.69	8.45	13.23
MW2	18	719	6.70	6.10	14.18

where: f_{ud} = strength of undamaged prisms
f_{da} = strength of damaged prisms after reloading
f_{ai} = strength of prisms after injection

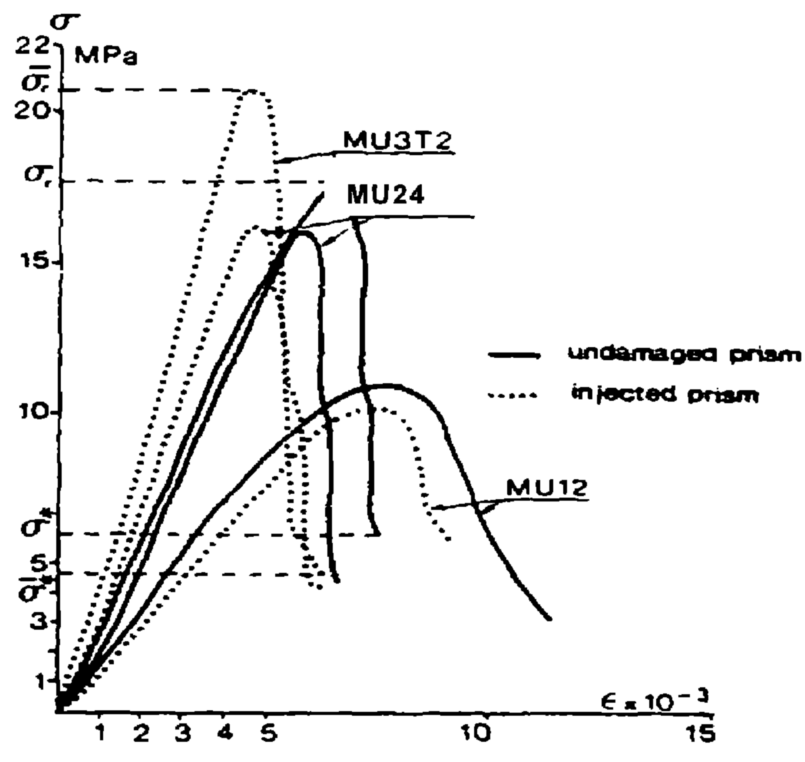
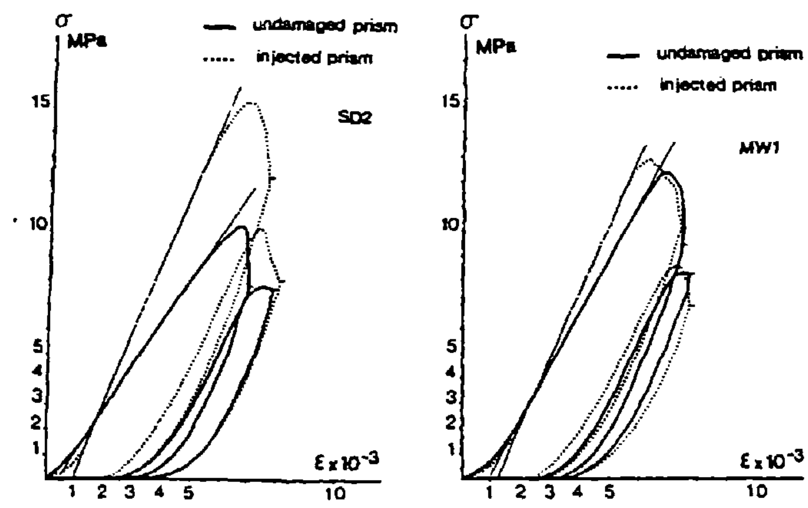


Figure 1: Measured Stress-Strain Curves for Test Prisms



(a) SD2 Prisms (b) MW1 Prisms

Figure 2: Typical Stress-Strain Curve for Type S Resin Injection

GI3 Strengthening of Masonries by Injection Technique

L. Binda , C. Modena and G. Baronio
Proceedings of the Sixth North American Masonry Conference, Vol. 1, 1993, pp. 1-14.

Objective

The purpose of the research was to investigate the effectiveness of grout injections for improving the in-plane shear strength of unreinforced clay-unit or stone masonry.

Rehabilitation Procedure

Cementitious or lime grouts are injected into an existing stone or brick masonry wall for the purpose of repair or rehabilitation.

The effectiveness of the method is dependent on the optimal combination of chemical, mechanical, and physical properties for masonry materials and injected materials.

Research Approach

Laboratory tests were done on full-scale wall specimens to compare the behaviors of original masonry and damaged masonry repaired by injection. Material tests were also done on mortars, stones and bricks, and grouts.

The first objective of the research was to define criteria to predict the injectability of existing masonry walls. The second objective was to quantify the effect of the injection method.

Experimental research was done to identify characteristics of stones and mortar used in the internal wythe of the wall. Mineralogical-petrographical analyses on mortar and stone, and chemical analyses on mortars were done.

One grout type was composed of hydrated lime and powdered bricks reproducing hydraulic mortars with the proportion of 1:2 and 1:3 (parts of lime to powdered brick). The second grout type was based on a hydraulic pozzolanic binder with a low content of soluble salts and no toxins.

In-situ tests were done using a simple shear test apparatus (Figure 1). A single horizontal force was applied at the mid-height of a test pier which was resisted at the top and bottom of the pier on the opposite side.

Grout injections were inserted into holes that were inclined 45^0 with the horizontal to facilitate the penetration of the admixture. After load testing, the injected wallettes were opened to observe penetration and diffusion of the grout.

Summary and Significant Findings

Measured force-deflection behavior of the test piers are shown in Figure 2a for Type 1 specimens (force applied to virgin panel) and in Figure 2b for Type 2 specimens (force applied in reversed direction after damaging panel). The subscript "i" represents those specimens that were injected.

Full penetration of the grout was not achieved because the grout flow was obstructed by the loose material in the interior of the wall. Some voids were filled with grout whereas other voids were not. Some of the largest voids were partially filled.

Pre-wetting of the masonry before grouting improved grout penetration.

The masonry shear strength was low (in the range of 0.02 to 0.05 MPa) but met values given in the Italian masonry building code specification for multiple wythe masonry.

The shear strength was directly related to the amount of internal voids.

The injection technique was effective for restoring, or improving, the original strength and stiffness of damaged walls.

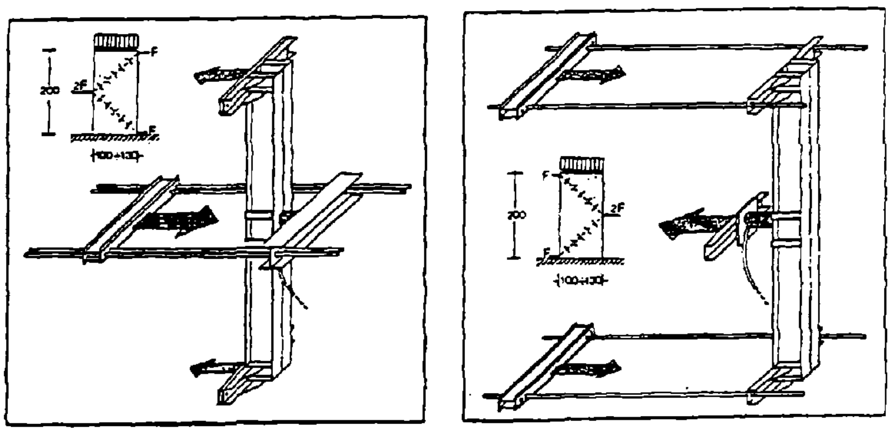
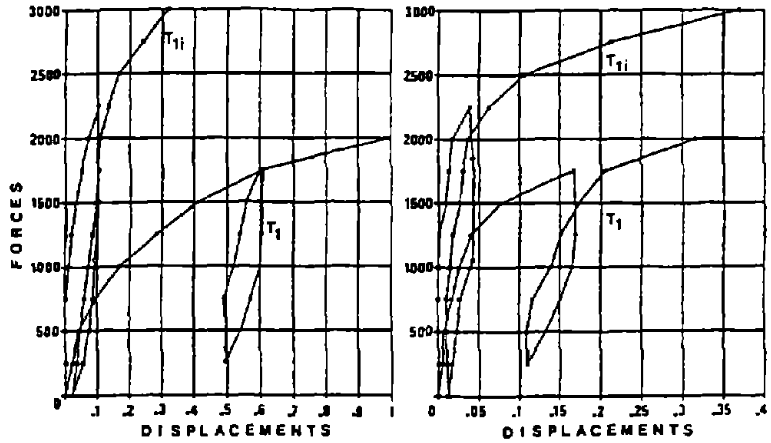
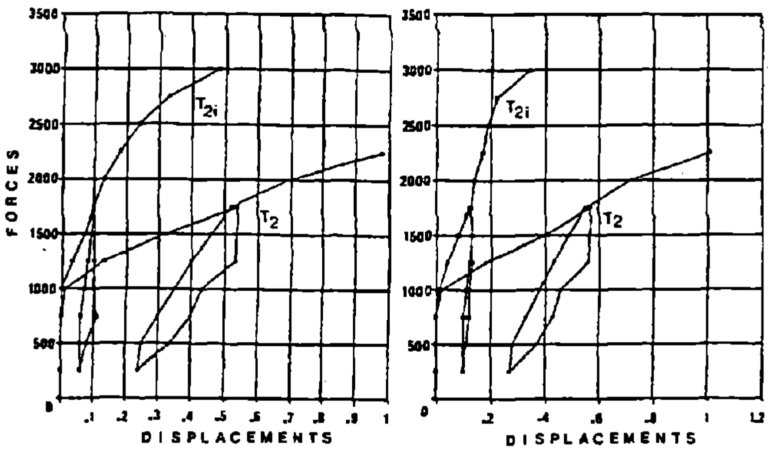


Figure 1 : Testing Apparatus for In-situ Shear Tests



(a) Test Type 1: Transducers 3/8 (compression) and 4/7 (tension)



(b) Test Type 2: Transducers 3 (tension) and 4 (compression)

Figure 2: Sample Force-Deflection Curves from In-situ Shear Tests

GI4 Experimental Results on Unreinforced Masonry Shear Walls Damaged and Repaired

G.M. Calvi and G. Magenes

Proceedings of the Tenth International Brick and Block Masonry Conference, Vol. 2, 1994, pp. 509-518.

Objective

The objective of the research was to investigate the use of either cementitious mortar or epoxy resin as an injection material for restoring the in-plane shear strength and stiffness of an unreinforced masonry wall.

Rehabilitation Procedure

The primary purpose of injection with cementitious or epoxy resin is to fill the voids and cracks in the masonry due to physical, chemical, or mechanical deterioration, and to restore the original integrity.

Grout is injected through core-drilled injection ports, which are located in the vicinity of a crack. The cracks are washed and first injected with a substrate conditioner to restrain the absorption of grout by the masonry. The grout is then injected and allowed to cure.

The injection of epoxy resin is also done through drilled ports. The cracks are first washed and allowed to dry. The epoxy is injected and allowed to cure.

Research Approach

Six masonry walls, previously cracked under shear and compression forces, were repaired and then tested with the same loading procedure used on the original walls.

Fired clay solid bricks and lime mortar, with similar mechanical properties of historical brick masonry, were used to construct the test walls. The test walls were 1.5 m wide, 0.38 m thick (three wythes, English bond), and varied in height from 2 to 3 m. The test walls had been previously subjected to cyclic or monotonically increasing shear forces with various amounts of vertical compressive stress. The top and bottom of the test walls were kept parallel during the horizontal shear loading using the testing apparatus shown in Figure 1. As shear force increased, the vertical force was maintained at a constant value by compensating for geometrical effects.

Four walls (MI1m, MI1, MI4, & MA) were injected with cementitious grout while the remaining two walls (MI2 & MI3) were injected with epoxy resin. The wall height was varied from 2.0 to 3.0 meters to examine behavior under two different moment-to-shear ratios. Hydraulic lime was used in the mortar for all specimens except for test wall MA which had common lime. Table 1 gives the specimen characteristics and cracking details for each wall.

Summary and Significant Findings

Measured force-deflection curves for original and repaired test walls are shown in Figure 2. Both cementitious mortar and epoxy resin injections were effective for restoring shear strength of damaged unreinforced brick masonry walls under in-plane shear. The injection of cementitious mortar resulted in moderate changes in terms of strength and stiffness with respect to the original material. The injection of epoxy resin led to a significant increase in strength, and moderate increase in stiffness.

Crack patterns were categorized into two groups: a concentrated crack with a large width and diffused cracking with many cracks of small width.

The original shear strength was essentially restored in all test walls. The strength of all test walls was found to increase except for wall MI4 in which small existing cracks were not injected with grout.

Cementitious grout was found to be more appropriate for panels with wider cracks, while the epoxy resin was found to be more effective for the panels with narrower cracks.

Table 1. Summary of Test Walls and Cracking

Specimen denomination	mortar type	height [m]	type of test before repair	crack pattern	crack width (mm)
MI1m	hydraulic lime	2.0	monotonic	SDM	≤ 10
MI1	hydraulic lime	2.0	cyclic	DDM	≤ 30
MI2	hydraulic lime	2.0	cyclic	DD	≤8
MI3	hydraulic lime	3.0	cyclic	dd	≤ 2
MI4	hydraulic lime	3.0	cyclic	DD	≤ 50
MA	common lime	2.0	cyclic	DD	≤ 30

SDM: single diagonal crack in one direction with minor adjacent cracks, mixed type failure (bricks and joints);
DDM: double diagonal cracks (X-shaped) in one direction with minor adjacent cracks, mixed type failure (bricks and joints);
DD: single diagonal cracks in both diections (X-shaped);
dd: diffused cracking, most crack widths less than 1 mm.

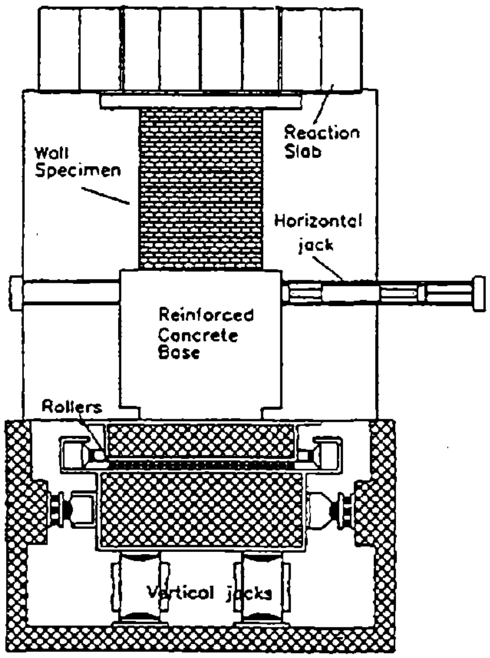
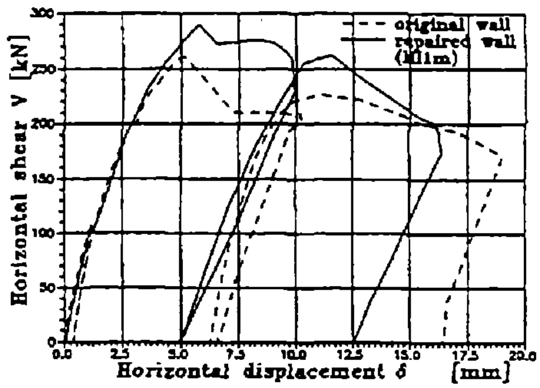
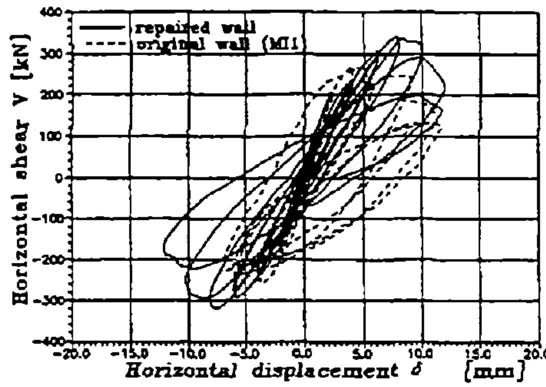


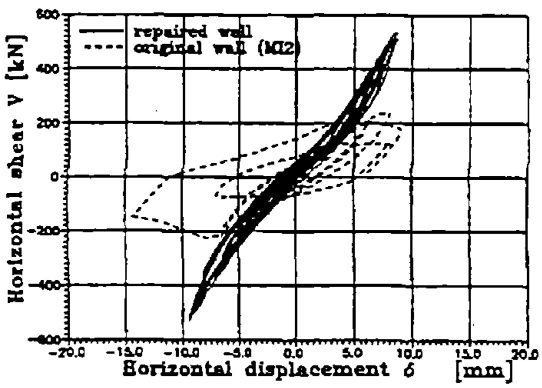
Figure 1: Testing Apparatus



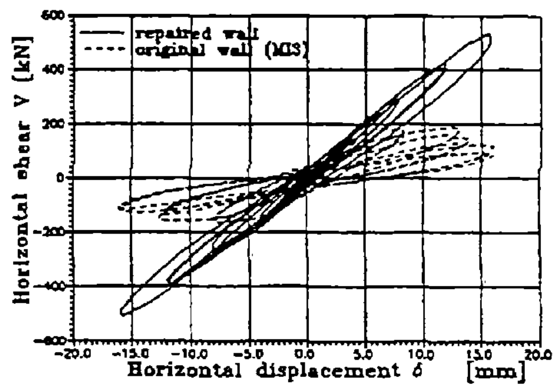
(a) Test Wall MI1m



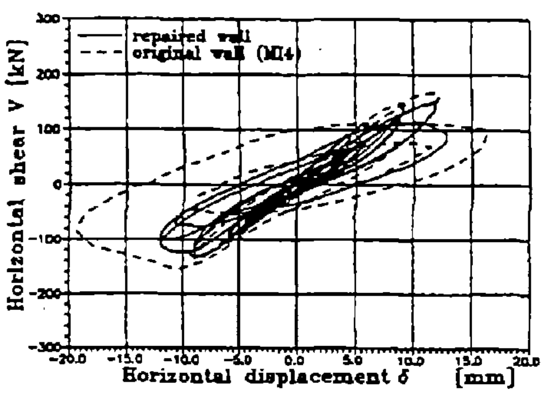
(b) Test Wall MI1



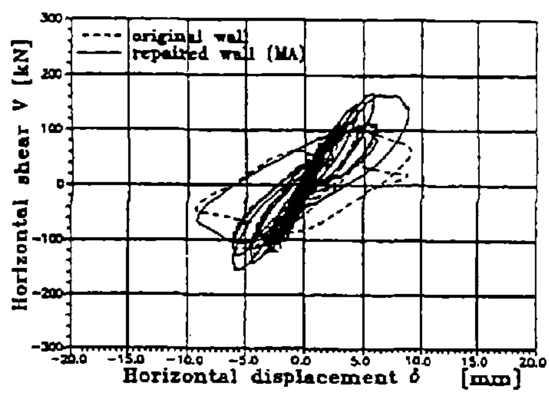
(c) Test Wall MI2



(d) Test Wall MI3



(e) Test Wall MI4



(f) Test Wall MA

Figure 2: Measured Force-Deflection Curves

GI5 Cyclic Shear Behavior of Tuff Masonry Walls Strengthened by Grout Injections and Reinforcement

G. Faella, G. Manfredi, and R. Realfonzo
Proceedings of the Tenth International Brick and Block Masonry Conference, University of Calgary, July 1994, Vol. 1, pp. 239-248.

Objective

The objective of the research was to investigate the effectiveness of cementitious grout injections and added steel reinforcing strips on in-plane lateral strength and behavior of masonry walls constructed with tuff stones.

Rehabilitation Procedure

Unreinforced masonry walls are strengthened by grout injection and steel reinforcement. Two pairs of steel strips are placed in vertical grooves on the external wall faces (Figure 1). The strips are anchored to the masonry with corrugated bars placed in sloping holes, and then injected with epoxy so that a grid of reinforced perforations is attained.

Research Approach

Tests were conducted on five types of walls. Type T1 walls were made of two outer leaves of tuff blocks filled with chips from tuff stones (faced masonry), so that the two leaves were partially tied (Figure 1). Type T2 walls were constructed using blocks laid in a bonded pattern through the wall thickness. Type T3 walls were Type T1 walls strengthened by grout injections. Type T4 walls were Type T2 walls strengthened by reinforcement. Type T5 walls consisted of Type T1 walls strengthened by grout injections and reinforcement.

Fixed-based masonry walls were subject to cyclic horizontal force under a constant vertical load using the testing apparatus shown in Figure 2. The test frame was comprised of a stiff reaction frame with a control mechanism to apply a constant vertical load. Additionally, lateral and vertical actuators were coupled together by means of transducers to ensure that the reaction beam supporting the wall remained horizontal. Tests were performed by imposing cyclic displacements of 0.5 mm increments with a step-wise increasing amplitude.

Summary and Significant Findings

The experimental data demonstrates that the grout injections and reinforcement resulted in a dramatic improvement in lateral strength. Envelope curves of the horizontal force-displacement relationships are shown in Figure 3. For the grout-injected wall type T3, the observed strength is approximate twice that of type T1 and incrementally larger than type T2. With the addition of reinforcement for wall types T4 and T5, an increasing trend in strength is seen with respect to wall types T2 and T3 respectively.

An evaluation of ductility was made using envelope curves for each test wall. For the purpose of this research, ductility was measured by the index,

$$\mu_1 = \frac{s_u}{s_f}$$

with s_f and s_u are the displacements corresponding to the cracking and ultimate forces respectively. Table 1 shows the mean values of μ_I for all wall types considered. The test data show that the rehabilitation methods provided an increase in shear strength, while producing a reduction in ductility.

The measure of stiffness degradation for each specimen was given by the index,

$$k_m = \frac{|H^+{}_{\max,i}| + |H^-{}_{\max,i}|}{|s^+{}_{\max,i}| + |s^-{}_{\max,i}|}$$

where $H^+_{max,i}$ and $H^-_{max,i}$ are the maximum lateral forces for each direction of loading.

Wall stiffness decreased, and energy dissipation increased with lateral drift (Figures 4 and 5).

Wall energy dissipation capacity improved with grouting and reinforcement. A significant amount of energy was dissipated as wall stiffness degraded.

Table 1: Test Types and Data

WALL TYPE		N	Reinf.	H_{max} [kN]	μ_I
		3	-	108	
T1	Faced masonry	4	-	124	3.4
T2	Bonded masonry	6	-	166	3.3
		7	-	201	
T3	Faced masonry	8	-	233	
	[Strengthened by	9	-	203	1.9
	grout injections]	10	-	154	
		11	A1	271	
T4	Bonded masonry	12	A1	208	
	[Strengthened by	13	A3	394	2.4
	reinforcement]	14	A3	254	
		15	A1	239	
		16	A1	304	
T5	Faced masonry	17	A1	231	
	[Strengthened by	18	A1	240	1.6
	grout injections	19	A3	329	
	and reinforcement]	20	A3	274	
		21	A3	294	
		22	A3	253	

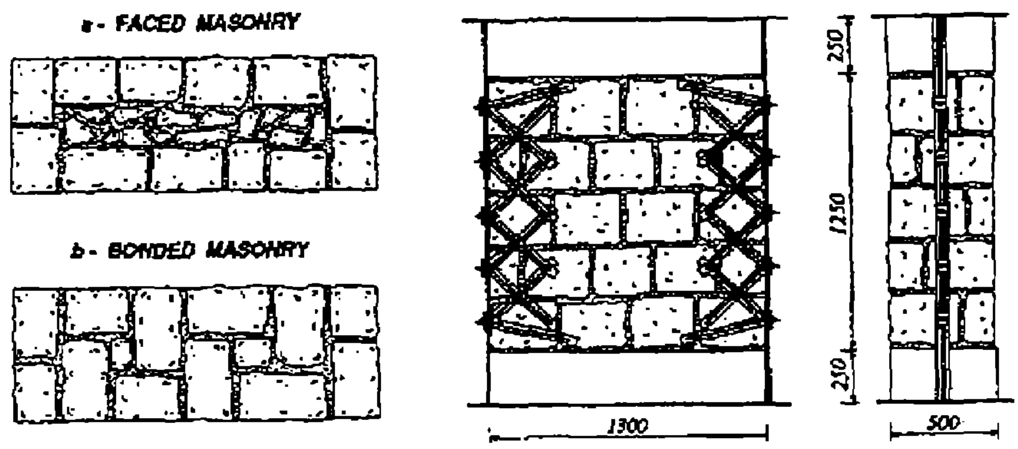


Figure 1: Masonry Types and Arrangement of Reinforcement

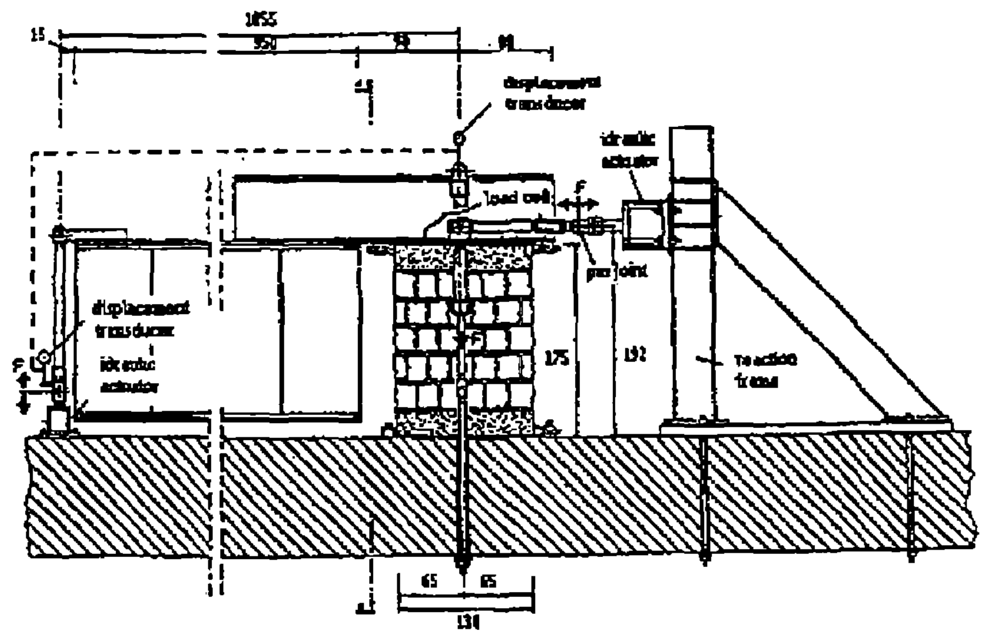


Figure 2: Testing Apparatus

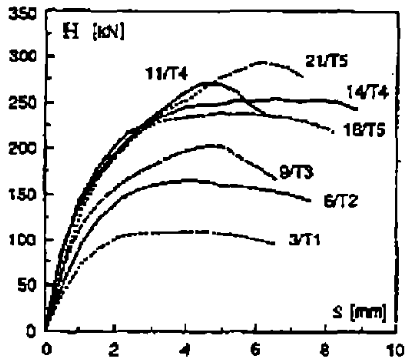
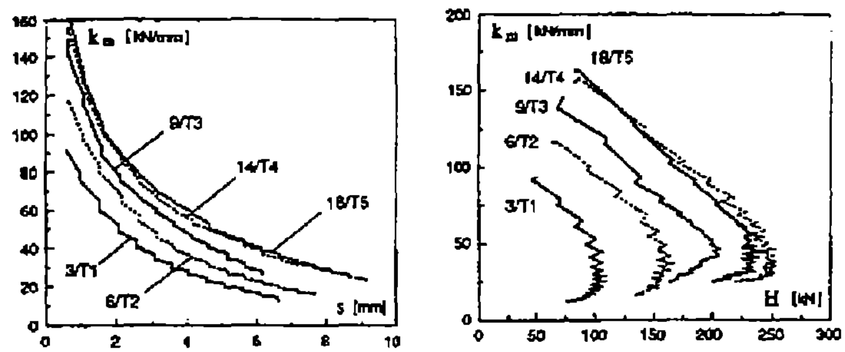


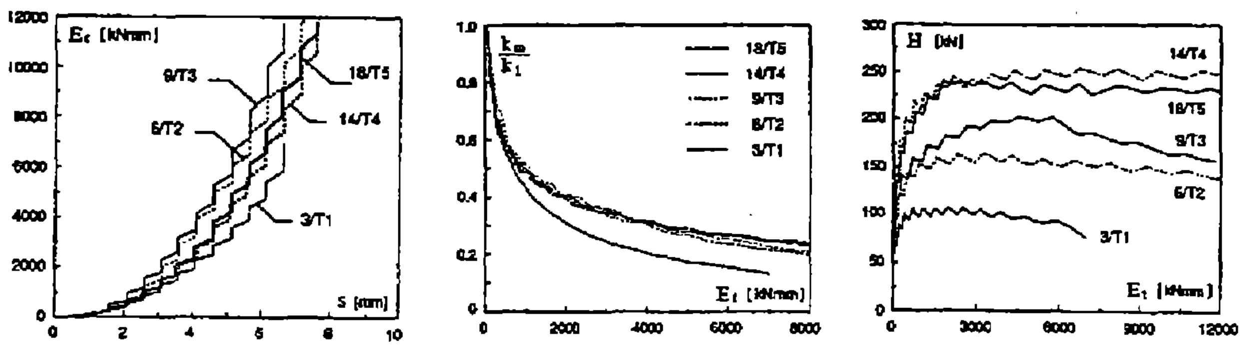
Figure 3: Measured Force-Deflection Relations for Test Walls



Stiffness k_m vs displacement s.

Stiffness k_m vs shear force H.

Figure 4: Lateral Stiffness of Test Walls vs. Displacement and Force



Dissipated energy E_t vs displacement s.

Stiffness k_m vs dissipated energy E_t.

Dissipated energy E_t vs shear force H.

Figure 5: Dissipated Energy of Test Walls vs. Displacement, Stiffness and Force

GI6 Repair of Unreinforced Masonry Structures with Grout Injection Techniques

T. Manzouri, P.B. Shing, M.P. Schuller, and R.H. Atkinson

Proceedings of Seventh North American Masonry Conference, University of Notre Dame, June 1996, pp. 472-483.

Objective

The objective of the research was to investigate the effectiveness of injection grouting for repair of unreinforced clay-unit masonry walls. In addition, the use of added pin joint reinforcement and added vertical and horizontal reinforcement were studied.

In addition to grouting, one test wall (#3) was retrofitted with Helifix dry-fix remedial anchors. These reinforcing ties were used for pinning of the wythes in the toe area, the pinning of the toe to the base slab, and as horizontal reinforcement which was placed into selected bed joints that were repointed (Figure 1).

One other test wall (#4) was repaired by grouting and placement of vertical and horizontal standard deformed reinforcing bars between exterior wythes (Figure 2).

Rehabilitation Procedure

Damaged test walls were repaired by first replacing cracked units and mortar joints with new materials, and then by injecting with grout. Cracks wider than 0.06 inch, internal voids and collar joints were injected with a coarse grout. Smaller cracks with widths ranging from 0.008 to 0.06 inch were then injected with a fine grout. Mix proportions for grout mixes are given in Table 1.

Research Approach

A total of four unreinforced clay-unit masonry walls were constructed and tested. Three walls were solid and had identical geometries (Figure 3) while the fourth wall had a window opening (Figure 4). Each test wall was laid in three-wythe running bond.

The first three test walls were subjected to a constant vertical compressive stress equal to 55, 85 and 150 psi respectively. For the fourth test wall the pier compressive stress was 70 psi. Each wall was tested in its original unretrofitted state, and then repaired and retested.

Test walls were subjected to repeated and reversed in-plane lateral forces until substantial damage occurred.

Table 1: Grout Mixes

Grout Type	Cement Type[1]	w/c2 Ratio	Solids (% by weight)				Admixtures[3] (% by wt.)	
			Cement	Lime Type S	Fly Ash Class F	Sand #70	SP[4]	Grout Aid
Fine	III	0.50	100	-	-	-	2.0	0.5
Coarse	I/II	1.00	32.1	4.8	7.9	55.2	2.0	0.5

1 ASTM C 150 Portland Cement
2 Ratio of weight of water to weight of cement
3 Percent by weight of cement
4 SP=Superplasticizer (modified naphtalene sulphonate formaldehyde base)

Summary and Significant Findings

In general, the test walls behaved as flexural elements with first bed-joint cracking in the heel region, followed by toe crushing and then diagonal tension cracking. Sliding along shear cracks was often observed, and in some cases sliding was observed along open flexural cracks.

Measured force-deflection behavior of test wall #3 (Figure 5a) demonstrated the effectiveness of adding horizontal reinforcing ties in the bed joints. Comparison of behavior of wall #4 (Figure 5b) revealed that the added horizontal and vertical reinforcement suppressed the formation of diagonal tension cracks and base sliding. The reinforcement tended to keep the masonry intact and thus resulted in less inelastic action, less hysteretic energy dissipation and less damage.

Results of the experiments indicate that both strength and stiffness of the damaged walls can be restored with grout injections. Furthermore, the strength and ductility of the test walls could be enhanced with the introduction of steel reinforcement. Grout injection proved to be a reliable means for bonding the new reinforcement to the masonry.

Related Reference

Manzouri, T., P.B. Shing, B. Amadei, M.P. Schuller, and R.H. Atkinson, "Repair and Retrofit of Unreinforced Masonry Walls: Experimental Evaluation and Finite Element Analysis," Report No. CU/SR-95-2, Department of Civil, Environmental and Architectural Engineering, University of Colorado at Boulder, 1995.

Schuller, M.P., R.H. Atkinson, and J.T. Borgsmiller, "Injection Grouting for Repair and Retrofit of Unreinforced Masonry," *Proceedings of the Tenth International Brick and Block Masonry Conference*, Vol. 2, 1994, pp. 549-558.

Shing, P.B., T. Manzouri, R.H. Atkinson, M. P. Schuller and B. Amadei, "Evaluation of Grout Injection Techniques for Unreinforced Masonry Structures," *Proceedings of the Fifth U.S. National Conference on Earthquake Engineering*, Chicago, July 1994, Vol. 3, pp. 851-860.

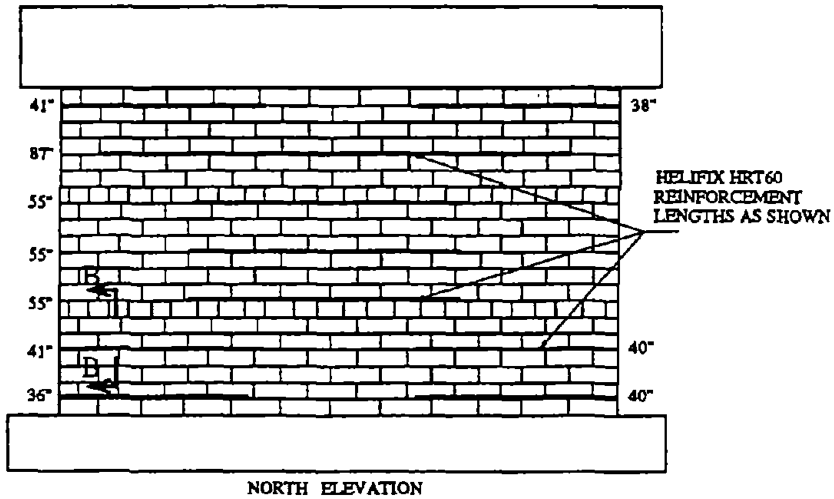


Figure 1: Placement of Horizontal Helifix Ties (Wall #3)

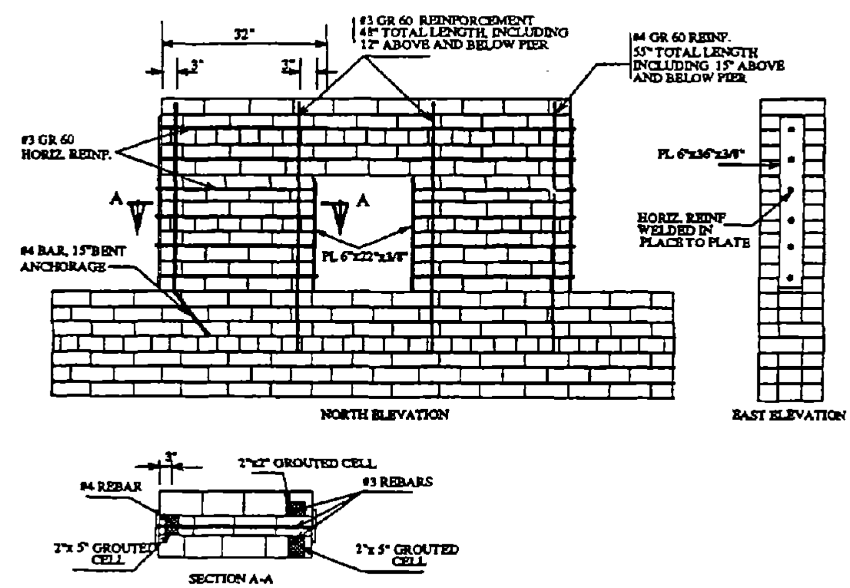


Figure 2: Placement of Horizontal and Vertical Reinforcement (Wall #4)

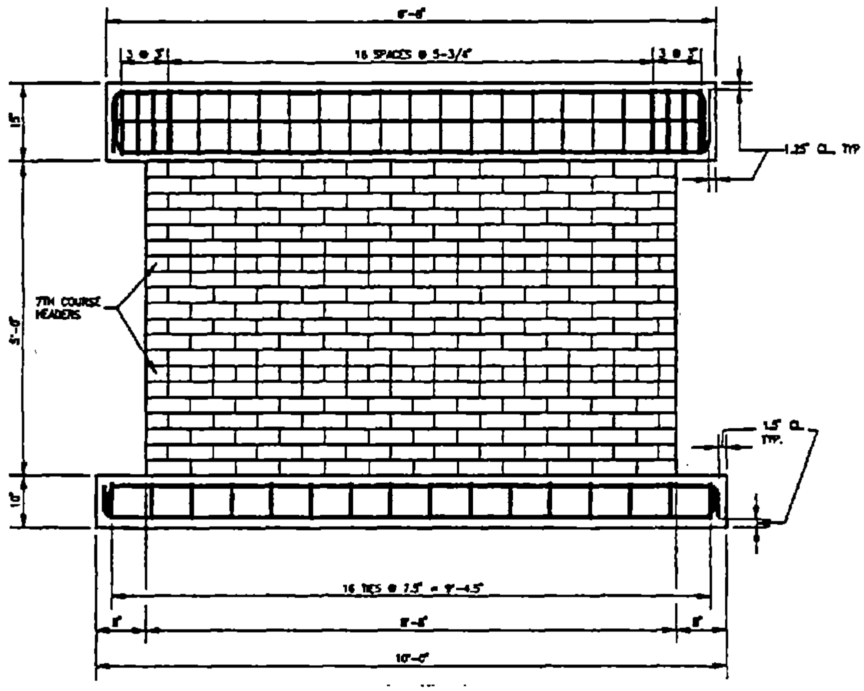


Figure 3: Description of Test Walls #1 - #3

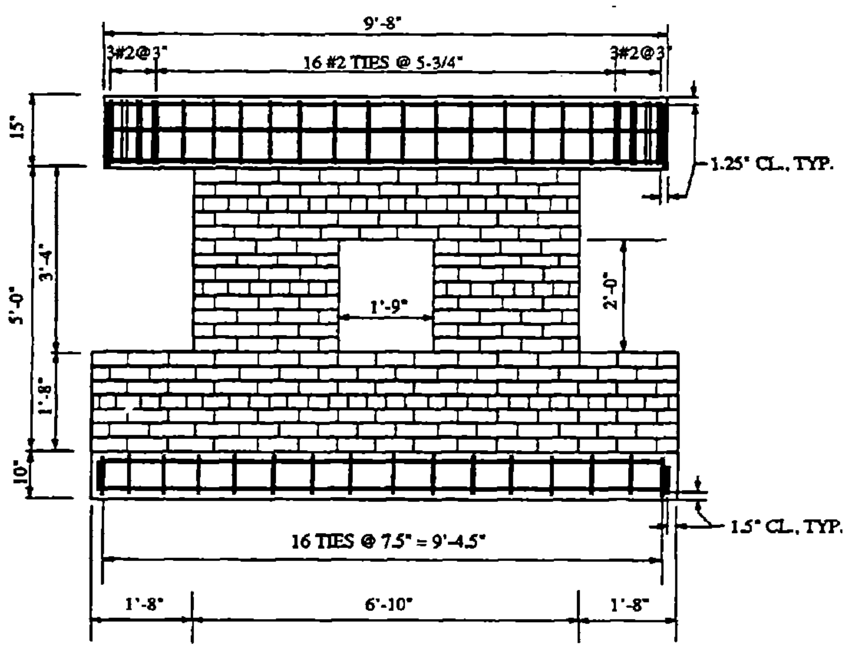


Figure 4: Description of Test Wall #4

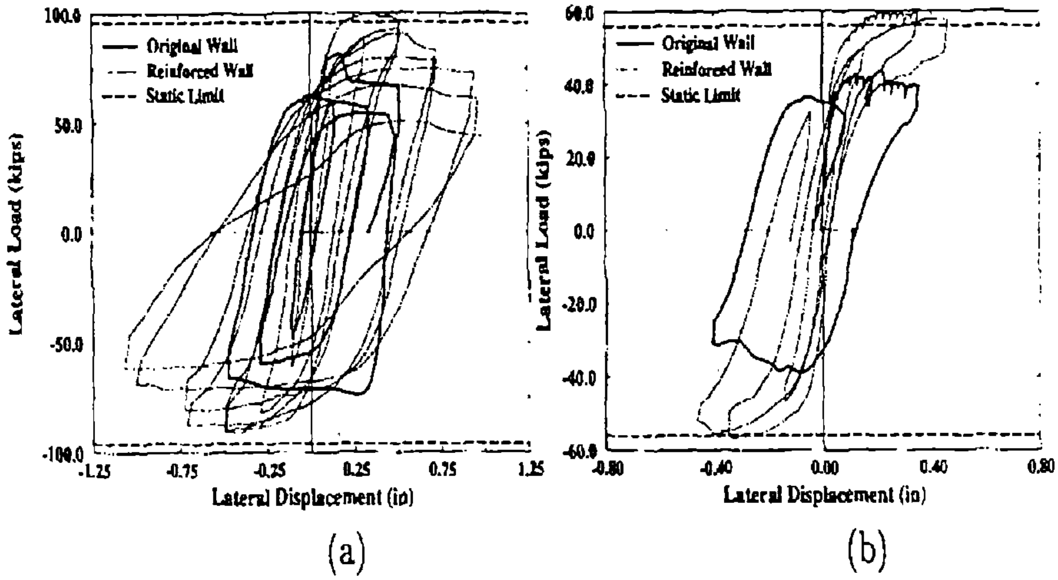


Figure 5: Measured Force-Deflection Relations for Walls #3 and #4

GI7 Repair of Earthquake Damaged Concrete Masonry Systems Subjected to Static and Dynamic Loads and Elevated Temperatures

J.M. Plecnik, J.E. Amrhein, J. Warner, W.H. Jay and C.V. Chelapati
Proceedings of the Sixth WCEE, New Delhi, 1977, pp. 2492-2498.

Objective

The objective of the research was to investigate the effectiveness of epoxy injections to improve the strength of concrete masonry shear walls. Experimental parameters included the type of epoxy adhesive, the width of cracks to be repaired, and exposure to fire.

Rehabilitation Procedure

Cracks in a concrete masonry wall are injected with epoxy as a repair measure. One injection method consists of premixing the epoxy resin and hardener, and then injecting it into a wall with a common caulking gun. Another method consists of pumping epoxy under pressure for better penetration into cracks.

For cracks more than 1/4 in. wide, an epoxy mixture consisting of epoxy adhesive as a binder and various fillers such as sand or cements can be used. For thinner cracks, epoxy adhesives without aggregate fillers can be used.

Advantages of epoxy injection include low repair cost, short repair time, little change in appearance, and good strength. Disadvantages include sensitivity of epoxy adhesive to temperature effects, and the lack of verification during actual earthquakes.

Research Approach

Test samples consisted of individual jointed concrete masonry units which were subjected to axial compression or shear. The direction of the applied compressive force was varied with respect to the bed joint. One set of specimens (Figure 1) served as the control group and were simply concrete blocks joined with mortar. For one other set of test specimens (Figure 2), epoxy was injected between the mortar joint and the unit. A third set of specimens (Figure 3) were fabricated exclusively of grout segments that were joined with epoxy.

Block test specimens (10 cm x 15 cm) were saw cut from standard 6 in. concrete masonry units. Grout specimens (15 cm x 25 cm x 10 cm) were constructed to approximate the cross sectional area of the cavity of the 6 in. standard block. Cracked grout specimens were immediately repaired with epoxy.

In addition, rectangular panels with repaired diagonal cracks were subjected to vertical compression force. The panels varied from 16" x 16" to 32" x 24".

All tests on the epoxy repaired specimens were conducted at least seven days after epoxy injection to allow for curing. The optimum viscosity for repair was a function of many variables including crack size, relative difference in temperature between epoxy and structural materials, speed of injection, and injection pressures.

Summary and Significant Findings

Failure of nearly all test specimens was initiated in the construction material, not within the epoxy adhesive. Slight debonding occurred in only two specimens among 120 compression tests.

The width of cracks and the viscosity of the epoxy adhesive did not greatly affect the strength.

Compression and shear test results for the three types of specimens are shown in Tables 1, 2 and 3 for crack angles of 60, 75 and 90 degrees.

Dynamic strength always exceeded static strength for the control block rib specimens and for the epoxy joined specimens, and exceeded static strength for two of the four joined grout specimens.

The failure mechanism for rectangular panels consisted of debonding of blocks, or compressive crushing of the units or grout.

Bond and tensile strength of epoxy repaired specimens was nearly zero for temperatures exceeding 400 °F.

The strength of the epoxy repaired specimens was approximately the same for both high and low viscosity epoxy adhesives.

Related Reference

Plecnik, J.M., J.E. Amrhein, W.H. Jay, and J. Warner, "Epoxy Repair of Structures," *International Symposium on Earthquake Structural Engineering*, St. Louis, August 1976, pp. 1023-1036.

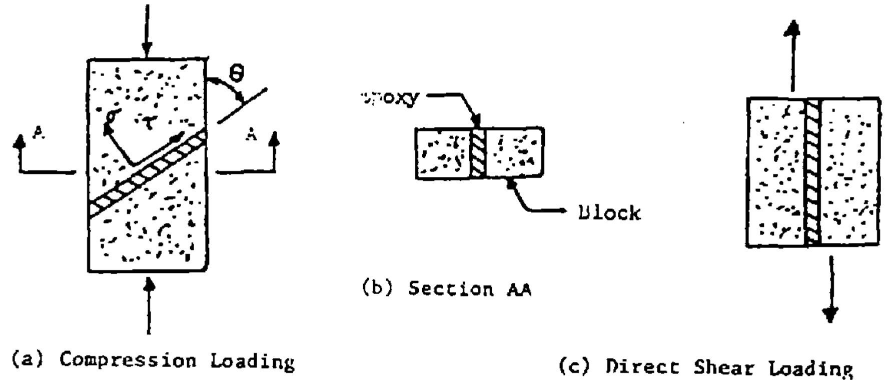


Figure 1: Control Block Rib Specimens

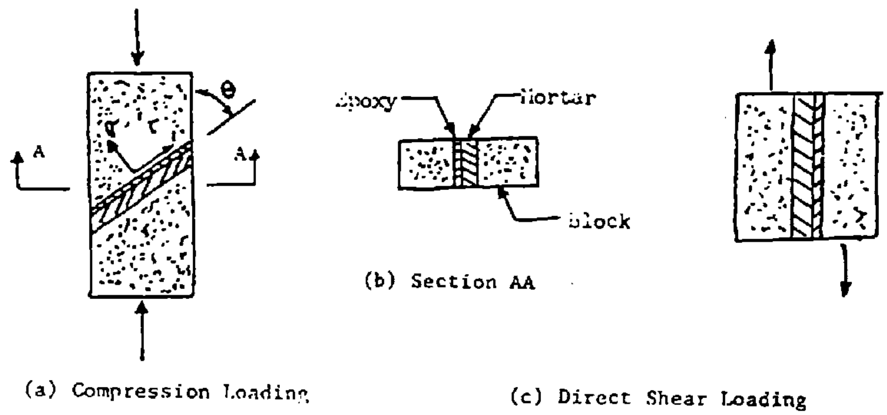


Figure 2: Masonry Epoxy Joint Specimens

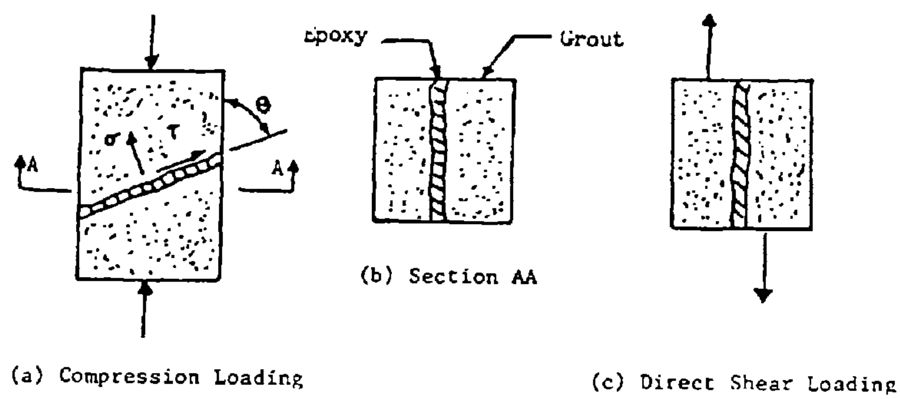


Figure 3: Epoxy Joined Grout Specimens

Table 1 Compression Strength, Shear Strength and Standard Deviation for Block Rib Specimens

Test Condition	Static Strength (Kg/cm^2)		Dynamic Strength (Kg/cm^2)	
	Average	Standard Deviation	Average	Standard Deviation
Compression -90°	171.8	14.7	238.4	22.3
Compression -75°	210.1	15.6	241.3	24.0
Compression -60°	179.2	13.0	252.3	29.7
Direct Shear	33.5	*	37.5	*

Table 2 Compression Strength, Shear Strength and Standard Deviation for Masonry Joint Specimens

Test Condition	Static Strength (Kg/cm^2)		Dynamic Strength (Kg/cm^2)	
	Average	Standard Deviation	Average	Standard Deviation
Compression -90°	140.7	24.5	209.6	24.6
Compression -75°	161.0	21.7	215.2	18.0
Compression -60°	148.7	24.3	197.8	21.6
Direct Shear	**	**	**	**

Table 3 Compression Strength, Shear Strength and Standard Deviation for Grout Specimens

Test Condition	Static Strength (Kg/cm^2)		Dynamic Strength (Kg/cm^2)	
	Average	Standard Deviation	Average	Standard Deviation
Compression -90°	142.2	11.8	202.9	30.9
Compression -75°	196.5	45.3	195.2	28.6
Compression -60°	228.6	27.6	207.9	20.8
Direct Shear	16.6	*	28.2	*

*Standard Deviation not calculated due to inadequate number of tests.

**Direct Shear strength was nearly zero since failure always occurred at the non-epoxied mortar-block interface.

GI8 Repair of Cracked Unreinforced Brick Walls by Injection of Grout

N.A. Roselund and S. Pringle

Proceedings of Fourth U.S. National Conference on Earthquake Engineering, 1990,Vol. 3, pp. 283-292.

Objective

The objective of this research was to investigate the use of grout injection for rebonding loose masonry fragments and for restoration of lateral strength to an unreinforced masonry wall.

Rehabilitation Procedure

Before injection, loose mortar and mortar easily debonded are removed from the open joint. The crack is then cleaned by flushing with water, and filled with mortar to a depth of about 1/2 inch. The mortar is tooled to match the adjacent joints. Bricks that can be removed by hand are removed from the wall and reset in new mortar. Then, holes for injection and for verification of flow are drilled into and adjacent to the crack (see Figure 1). The holes are drilled from one side of the wall to the depth of the near face of the far wythe. The crack and holes are flushed with water to clean and saturate the wall. Each injection hole and verification hole are flushed until water runs clean from adjacent holes. Flushing is repeated. Grout is injected into the wall through an injection wand. Injection starts from the far collar joint and continues to the near collar joint, and progresses from bottom to top. Verification ports are plugged and spillage must be washed from the wall. When the grout is firm, plugs are removed from the ports, and ports are pointed with mortar.

Grout of various constituents was injected into walls that were later demolished for observation. The grout mix that was developed is shown in Table 1. It produces a grout that has a slow setting rate and higher water retention due to the lime; has excellent fluidity due to the fine aggregate and the lubricating qualities of fly ash and plasticizing agents of the plastic cement; bonds to brick better than the original mortar; has insignificant shrinkage; has a compressive strength of about 1200 psi, which can be stronger than the original mortar; uses low cost, readily available materials; and is a cementitious material having permeability qualities compatible with the original construction.

Table 1. Grout Recipe

Parts measured by volume

Silica #60	Sand #90	Plastic Portland Cement	Type S Lime	Type F Fly Ash
3	1	1	1/2	1/2

Research Approach

No research was done. The article describes a case study.

Summary and Significant Findings

The injection process forces fluid grout into the crack and into the interconnected system of voids in the collar (Figure 2) and head joints. Generally, the voids in the collar and head joints allow grout to flow well beyond the crack and to make contact with the adjacent bricks. The hardened grout bonds loosened bricks into the wall and restores strength lost when the original bond of mortar to brick was lost due to cracking.

Related Reference

Pringle, S., "Repair of Cracked Unreinforced Brick Walls by Injection of Grout," *Proceedings of 1993 U.S. National Earthquake Conference*, Memphis, TN, May 1993.

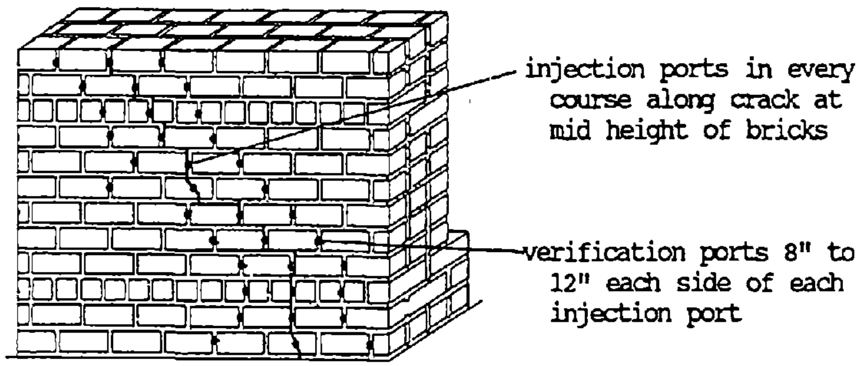


Figure 1: Pattern of Injection and Verification Ports

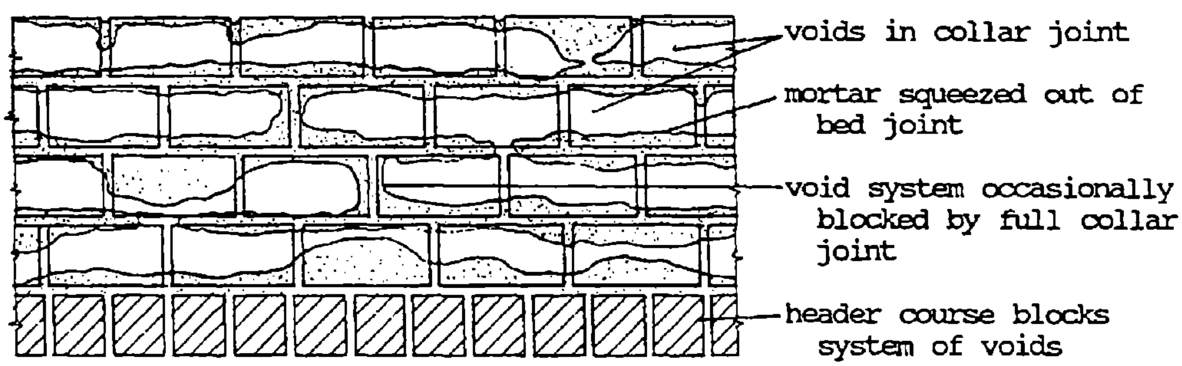


Figure 2: Condition of Mortar in Collar Joint

GI9 Injection Grouting for Repair and Retrofit of Unreinforced Masonry

M.P. Schuller, R.H. Atkinson, and J.T. Borgsmiller

Proceedings of the Tenth International Brick and Block Masonry Conference, University of Calgary, July 1994,Vol. 2, pp. 549-558.

Objective

The objective of the research was to investigate grouting procedures and specific mix formulations for rehabilitation of old unreinforced masonry buildings.

Rehabilitation Procedure

A survey and mapping of a wall is done to identify the extent and size of any visible cracks, mortar joint delaminations, or other damage. This is followed by a nondestructive evaluation to identify subsurface cracks and voids.

Grout is injected into fine cracks through either drilled injection ports or surface mounts. The spacing and placement of these ports is dependent on the width and roughness of the crack as noted in Figure 1. For cracks less than 1 mm wide, drilled ports are recommended. For cracks larger than 1 mm wide, surface mounted ports placed over the crack are recommended. For injection of collar joints, ports are inserted into a drilled hole and sealed with silicone or epoxy.

Surface cracks and mortar delaminations that may promote grout leakage during injection must be sealed. After the sealant is cured, the masonry must be washed with water to remove any dust or debris and to saturate the masonry. The washing should be done 24 hours before injection, and the masonry should be saturated-surface dry at the time of injection.

For both cracks and collar joints, the injection process should begin at the lower most injection port. Holes above the injection port should be plugged with a wooden dowel when grout flows from them. Grout should be continued to be injected until refusal, then pressure should be applied for an additional 30 to 60 seconds to consolidate the grout.

The removal of wooden dowels can be done 30 minutes after injection, but injection ports should remain installed for 24 hours after injection. Also, any injection port holes at mortar joints should be repaired with stiff mortar similar in color and composition to the original mortar. Finally, verification of the adequacy of the injection by means of nondestructive testing or in situ testing should be done.

Research Approach

A series of eight (8) masonry piers were constructed to investigate the effect of injection grouting on masonry behavior. Each pier was constructed using solid pressed clay units reclaimed from a building constructed circa 1915. Mortar with proportions of cement:lime:sand of 1:2:9 (by volume) was used to model old and deteriorating mortar. The interior wythe consisted of broken, uneven units, with collar joints being predominately empty.

Each pier was loaded to failure in compression to determine its compressive characteristics. After load removal, test piers were reloaded to determine the compressive behavior in the damaged state. Piers were then removed from the loading apparatus and grout was injected into collar joints and cracks. After a 28-day minimum curing period, the piers were again loaded to failure in compression. Different types of grouts were used for filling spaces ranging from very narrow cracks to empty joints and large voids.

Summary and Significant Findings

A variety of grouts were developed for filling spaces ranging in size from very narrow cracks to large voids and empty cracks. These grouts were injected into the damaged piers. For a typical pier, a comparison of its rehabilitated behavior with that of the as-built and damaged conditions showed that grout injection can be an effective rehabilitation method. A typical test specimen stress-strain curve is shown in Figure 2. This demonstrates how the grout injection was effective in restoring the initial stiffness of the masonry pier. In effect, the behavior in the working stress range was completely restored. Additionally, the compressive strength of the rehabilitated pier was approximately 80% of the original undamaged pier. This demonstrates that the use of grout injection can be a viable option for the rehabilitation of unreinforced masonry structures.

Related Reference

Manzouri, T., P.B. Shing, M.P. Schuller, and R.H. Atkinson, "Repair of Unreinforced Masonry Structures with Grout Injection Techniques," *Proceedings of Seventh North American Masonry Conference*, University of Notre Dame, June 1996, pp. 472-483.

Shing, P.B., T. Manzouri, R.H. Atkinson, M. P. Schuller and B. Amadei, "Evaluation of Grout Injection Techniques for Unreinforced Masonry Structures," *Proceedings of the Fifth U.S. National Conference on Earthquake Engineering*, Chicago, July 1994, Vol. 3, pp. 851-860.

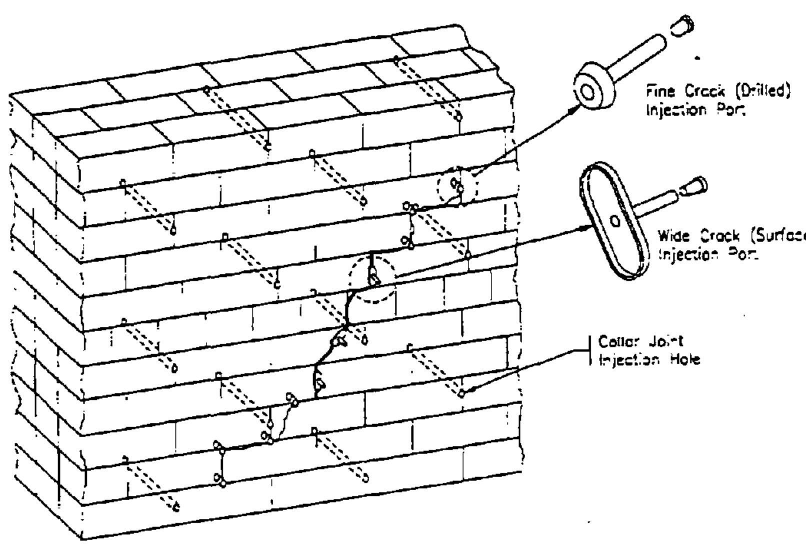


Figure 1: Placement of Grout through Ports

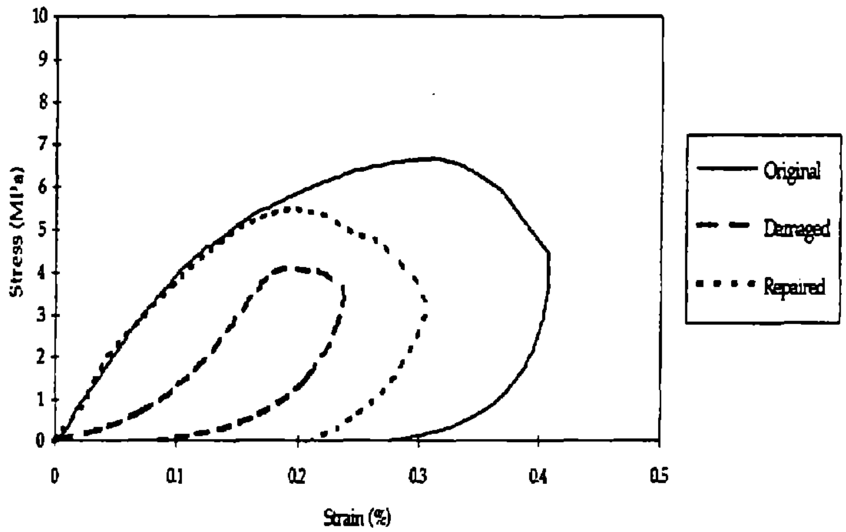


Figure 2: Compressive Stress-Strain Relation for Plain and Grouted Piers

GI10 The Effect of Repair and Strengthening Methods for Masonry Walls

P. Sheppard and S. Tercelj

Proceedings of the Seventh World Conference on Earthquake Engineering, Istanbul, Vol. 6, September 1980, pp. 255-262.

Objective

The objective of the research was to investigate the use of injection grouts and reinforced coatings for enhancing in-plane performance of unreinforced stone or brick masonry walls.

Rehabilitation Procedure

The paper addresses grouting as well as reinforced surface coatings.

Grouting

Cement grouting is used as a repair method for brick walls and as a strengthening method for stone masonry walls. Cracked areas are first sealed in a damaged wall, and then injection ports are drilled. Grout is under pressure into the cracked portions of a wall. The grout consists of 90% Portland Cement and 10% pozzolana, diluted with water to a 1:1 ratio by weight.

Surface Coating

A reinforced cement-plaster coating is applied to both surfaces of walls constructed with bricks and blocks. The coating is reinforced with a welded mesh consisting of 6-mm diameter wires at 15 cm centers. The coating thickness is 30 mm, and layers on opposite sides of a wall are tied together with 6-mm horizontal stirrups passing through pre-drilled holes. Ten stirrups per square meter are placed.

The nominal compressive strength of the cement plaster is 200 kg/cm^2.

Research Approach

A series of 20 masonry walls were subjected to in-plane lateral forces and vertical compressive stress. Five different wall types were studied. Some specimens were repaired by grout injections while others were strengthened with a plaster coating.

Summary and Significant Findings

Grouting

As noted in Table 1, the lateral shear strength of block walls was retained or was slightly increased when grouted whereas the strength of stone walls was increased by a factor of 2 or 3 with grouting. Shear strength of block walls covered with cement plaster coatings was increased to the extent that flexural modes controlled behavior.

Typical force-deflection curves for stone and brick masonry walls are shown in Figures 1 and 2 respectively. Walls repaired by grouting exceeded the strength of the unrepaired walls whereas the shear stiffness did not vary appreciably with grouting. Deformation capacity was increased considerably with grouting for the brick walls.

In general, if the cracked zones are well cement-grouted, the shear resistance of the original, undamaged wall can be attained.

Surface Coating

Walls strengthened with the coating were limited by flexural action. As noted in Table 2, for normal strength walls, lateral force capacity was increased by a factor of 2, and a factor of 1.25 for wall with high tensile strength.

Related References

Sheppard, P., and M. Tomasevic, "In-Situ Tests of Load Bearing Capacity of Walls of Old Masonry Buildings," *Proceedings of Fourth National Congress on Earthquake Engineering*, Vol. 2, pp. 86-92, 1986 (in Serbo-Croatian).

Tomasevic, M., and V. Apih, "The Strengthening of Stone-Masonry Walls by Injecting the Masonry-Friendly Grouts," *Journal of European Earthquake Engineering*, 1993, pp. 10-20.

Tomasevic, M., and P. Sheppard, "The Strengthening of Stone-Masonry Buildings for Revitalization in Seismic Regions," *Proceedings of the Seventh European Conference on Earthquake Engineering*, Vol. 5, Athens, September 1982, pp. 275-282.

Table 1: Summary of Grouted Test Specimens

τ_o-comp.	BLOCK TYPE & NOM. STRGTH (kp/cm²) WALL THICKNESS & CROSS-SECTION	WALL DESIGNATION	AV. MORTAR STRENGTH	BASIC WALL		WALL REPAIRED BY GROUTING		FACTOR OF INCREASE OF τ_o-comp.
				σ_n	τ_o-comp.	σ_n	τ_o-comp.	
			kp/cm²	kp/cm²	kp/cm²	kp/cm²	kp/cm²	
1.5 kp/cm²	STONE-MASONRY t = 60cm F = 6000 cm²	C	5	0·20	0·39			
		D	5	0·24	0·43	1·01	1·06	2·47
5.0 kp/cm²	NORMAL-FORMAT SOLID BRICKS (200) t = 25cm F = 2375cm²	B1	5	0·69	1·32	1·08	1·71	1·30
		B2	30	1·98	2·48	2·48	2·87	1·16
8.0 kp/cm²	PULV. FUEL ASH. BLOCKS (150) t = 29cm F = 2930 cm²	E1	13	1·44	2·46	1·39	2·41	0·98
		E2	13	1·56	2·57	2·20	3·16	1·23
	MODULAR BRICK BLOCKS (200) t = 28,5 cm F = 2850 cm²	9B	48	1·52	2·54	2·60	3·50	1·38
		11A	61	1·86	2·85	1·80	2·80	0·98
	EXP. CLAY AGG. BLOCKS (75) t = 29cm F = 2900 cm² — ORD'Y MASRY	7B	29	1·91	2·90	2·81	3·67	1·27
	COMPACT MASRY	6A'	58	4·22	4·79	4·65	5·11	1·07

EXPERIMENTAL RESULTS FROM COMBINED HORIZ/VERT. LOADING TESTS (LOADS GIVEN IN Mp)

WALL DESIG.	BASIC WALL H	BASIC WALL V	REP'D WALL H	REP'D WALL V	WALL DESIG.	BASIC WALL H	BASIC WALL V	REP'D WALL H	REP'D WALL V
C	2·2	8·0			E1	5·6	12·5	6·8	21·5
D	2·45	8·0	6·9	11·5	E2	5·9	12·5	8·85	20·9
B1	3·2	12·5	4·15	12·5	9B	6·1	15·0	10·7	27·3
B2	6·0	12·5	7·40	15·0	11A	6·2	11·0	8·3	25·0
					7B	7·1	15·0	10·1	20·0
					6A'	14·8	28·0	18·4	43·0

Table 2: Summary of Coated Test Specimens

BLOCK TYPE & NOM. STRGTH (kp/cm²) WALL THICKNESS & CROSS-SECTION (BASIC/STRGTHD.)	WALL DESIGNATION	AV. MORTAR STRENGTH.	BASIC WALL		STRENGTHENED WALL				FACTOR OF INCREASE OF $H_{comp.}$
			SHEAR		SHEAR		FLEXURE		
		kp/cm²	σ_n kp/cm²	H_{comp} Mp	σ_n kp/cm²	H_{comp} Mp	σ_w/β /	$H_{comp.}$ Mp	
MODULAR BRICK BLOCKS (200) t = 28.5/34.5 cm F = 2850/3450 cm² $(\sigma_n/\beta)_{rep.}$ = 0.25	9A	52	1.86	8.1	14.1	40.6	0.35	$\boxed{15.4}$	1.9
	9A'	50	1.85	8.1	18.6	51.2	0.29	$\boxed{16.3}$	2.0
PULV. FUEL ASH. BLOCKS (200) t = 29/35 cm F = 2930/3640 cm² $(\sigma_n/\beta)_{rep.}$ = 0.10	E3	13	1.59	7.6	4.35	17.8	0.45	$\boxed{15.5}$	2.0
	E4	13	1.71	7.95	4.15	17.2	0.45	$\boxed{15.5}$	2.0
NORMAL-FORMAT SOLID BRICKS (200) t = 25/31 cm F = 2375/2950 cm² $(\sigma_n/\beta)_{rep.}$ = 0.10	D2	4	0.54	3.4	3.45	$\boxed{12.3}$	0.52	$\boxed{11.8}$	3.5
CONCRETE BLOCKS $(\sigma_n/\beta)_{rep.}$ = 0.15 t = 29/35 cm F = 2900/3500 cm² (LIGHT AGG'TS)	1A'	51	4.52	*12.5	11.5	35.0	0.28	$\boxed{16.7}$	1.3
	5A'	63	3.80	12.9	10.4	32.3	0.30	$\boxed{16.2}$	1.25
t = 19/25 cm F = 1900/2500 cm² ORD'Y (190)	B3	34	3.30	7.7	4.7	12.9	0.56	$\boxed{9.8}$	1.25

* FLEXURAL FAILURE

EXPERIMENTAL RESULTS (LOADS GIVEN IN "MP")

WALL DESIG.	BASIC WALL - METHOD "D"		STRGTHD. WALL - METHOD "D"		WALL DESIG.	BASIC WALL - METHOD "D"		STRGTHD. WALL - METHOD "B"
	H	V	H	V		H	V	P
E3	5.95	12.5	20.2	42.2	9A	6.2	11.0	99.2
E4	6.25	12.5	19.6	41.8	9A'	6.9	15.0	131.0
D2	2.8	12.5	10.65	15.0	1A'	17.25	38.0	82.5
B3	7.7	15.0	12.85	20.0	5A'	13.65	27.0	74.5

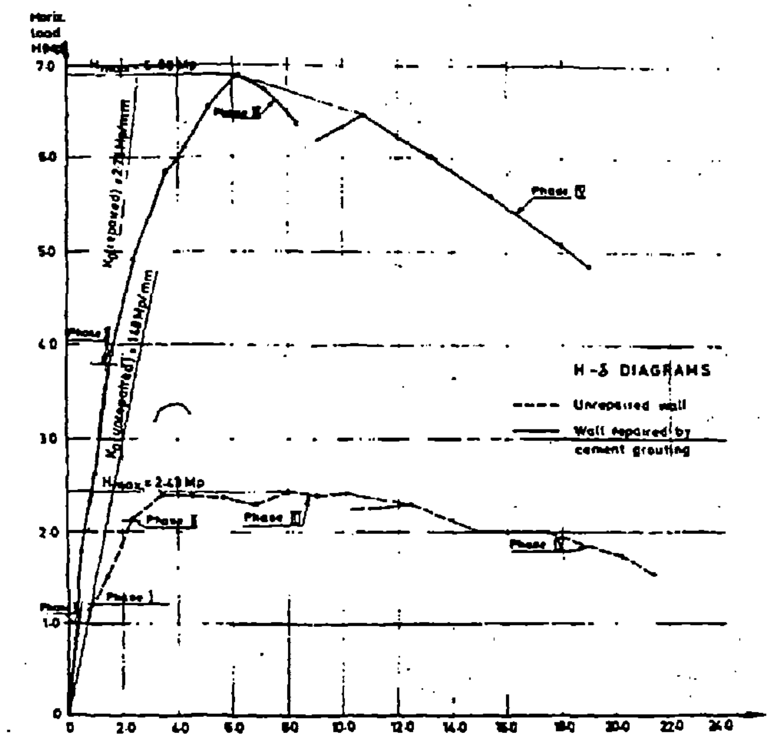


Figure 1: Typical Force-Deflection Curves for Plain and Grouted Stone Walls

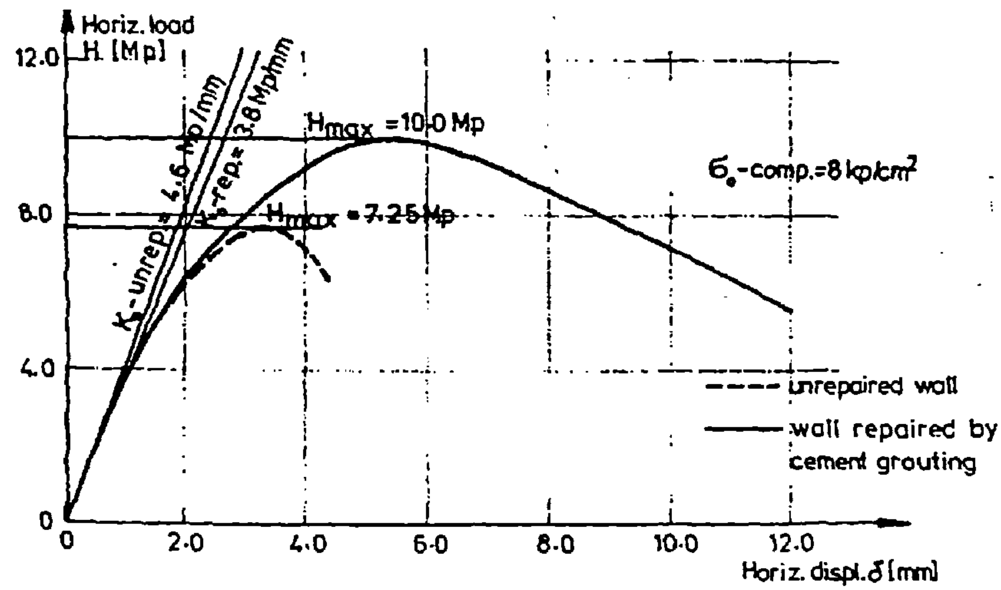


Figure 2: Typical Force-Deflection Curve for Plain and Grouted Brick Walls

GI11 Evaluation of Grout Injection Techniques for Unreinforced Masonry Structures

P.B. Shing, T. Manzouri, R.H. Atkinson, M.P. Schuller, and B. Amadei
Proceedings of the Fifth U. S. National Conference on Earthquake Engineering, Vol. 3, 1994, pp. 851-860.

Objective

The purpose of this research was to evaluate different grout injection materials for the repair of unreinforced masonry structures.

Rehabilitation Procedure

Unreinforced masonry walls are repaired by first removing and replacing damaged masonry units, followed by the injection of cracks with grout. The grout injection requires a very fluid grout with fine particle sizes for old masonry. The grout must be stable and resistant to segregation and shrinkage.

Research Approach

Compression tests were done on masonry piers rehabilitated with grout injections. In addition, direct shear tests were done on masonry bed joints with and without grout injection.

Eight masonry piers, with dimensions of 4½ x 2½ x 8½ in., were constructed and subjected to axial compressive stress. Test piers were constructed of reclaimed brick units and a Type O mortar which was used to simulate weak mortar in older masonry structures. Four of the piers had full mortar bed, head and collar joints, while the rest had poorly filled collar joints, partially filled head joints and furrowed bed joints representing older construction.

Most of the specimens were loaded to their maximum compressive strength then unloaded and reloaded to examine their behavior after damage. Damaged specimens were repaired and allowed to cure for 30 days, at which time they were tested again. Four types of grout mixes were used. They consisted of Type I or Type III Portland cement, with a water/cement ratio ranging from 0.75 to 1.40 by weight. The grout mixes used to fill the interior voids had 55% sand by weight.

In-plane direct shear tests were done on 20 specimens. Each of the specimens were two-units long and constructed with reclaimed brick units and Type O mortar. Two types of specimens were tested. The first type had intact joints while the other had bed joints injected with the grout mix. Each specimen was subjected to constant normal stress and cyclic shear stress reversals. Three constant normal compressive stresses of 50 psi, 100 psi, and 150 psi were used. At least 28 days before testing were required for curing.

Summary and Significant Findings

From the compression tests, the use of a grout mix consisting of Type III Portland cement, superplasticizer and Grout-Aid, with a water/cement ratio of 0.75, was found to be most effective in restoring the compressive strength of a damaged pier, as shown in Figure 1. The large increase in strength was attributed to the addition of microsilica to the grout mixture. The microsilica produced a very stiff grout compared to the stiffness of the masonry.

The results of the direct shear tests showed that grout injected joints achieved the same shear resistance and behavior as the intact mortar joints, as shown in Figure 2. This indicated the effectiveness of the grout as a bonding agent.

The increase in shear strength was proportional with the increase in normal compressive stress (to the left in Figure 3) suggesting a constant coefficient of friction.

The injection technique provided excellent performance in terms of injectability and bond strength. As a result, restoration of original compressive and tensile strengths for cracked masonry, and shear strength of mortar joints, was attained.

Related Reference

Manzouri, T., P.B. Shing, M.P. Schuller, and R.H. Atkinson, "Repair of Unreinforced Masonry Structures with Grout Injection Techniques," *Proceedings of Seventh North American Masonry Conference*, University of Notre Dame, June 1996, pp. 472-483.

Schuller, M.P., R.H. Atkinson, and J.T. Borgsmiller, "Injection Grouting for Repair and Retrofit of Unreinforced Masonry," *Proceedings of the Tenth International Brick and Block Masonry Conference*, Vol. 2, 1994, pp. 549-558.

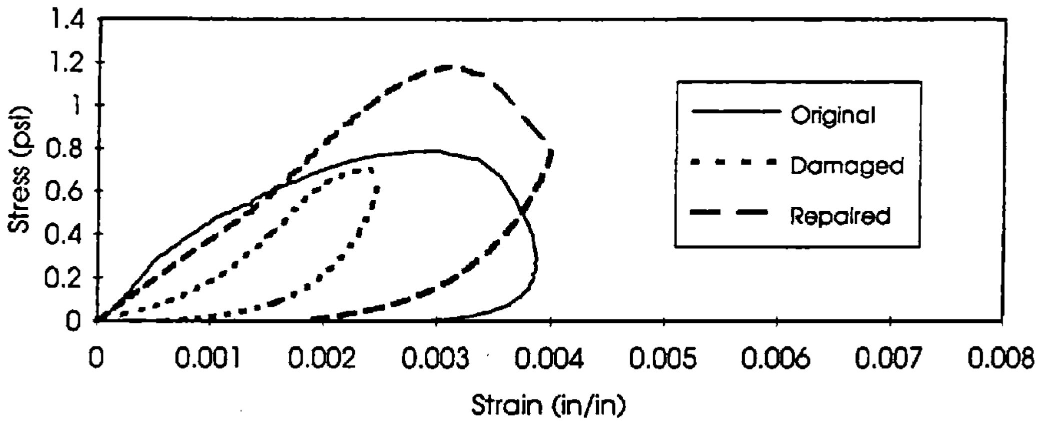


Figure 1: Compressive Stress-Strain Behavior of a Pier

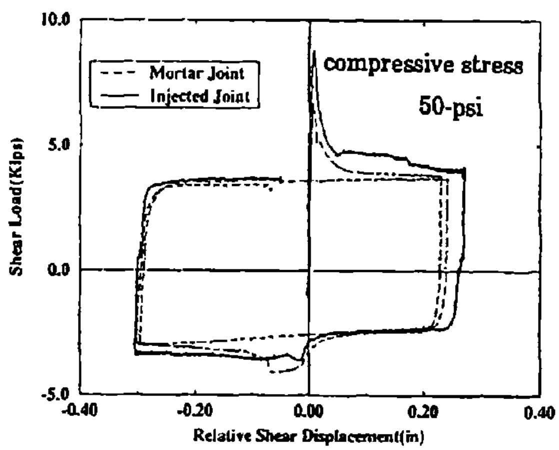


Figure 2: Measured Shear Force-Deflection Relations

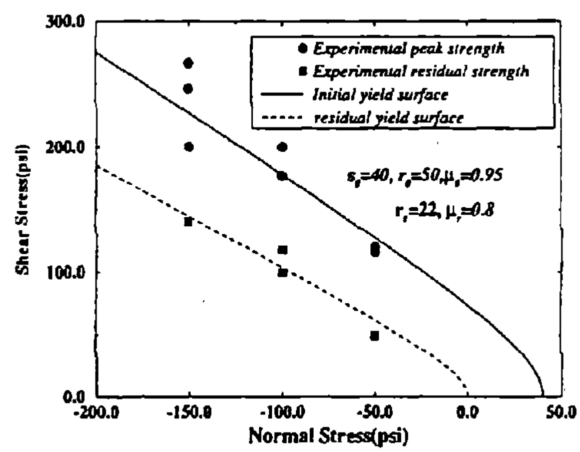


Figure 3: Shear Stress vs. Normal Stress

GI12 The Strengthening of Stone-Masonry Walls by Injecting the Masonry-Friendly Grouts

M. Tomazevic and V. Apih

Journal of European Earthquake Engineering, 1993, pp. 10-20.

Objective

The objective of the research was to investigate the use of injection grouts for in-plane strengthening of stone masonry walls. Attention was focused on the introduction of foreign material that could result in negative effects for the historic fabric of the masonry.

Rehabilitation Procedure

Grout consisting of 90% Portland Cement and 10% pozzolana is used. Grout is injected through injection tubes placed between the stones. The procedure is done from the base of a wall upward using the higher ports as verification holes.

Research Approach

The effect of grouting on lateral force behavior of stone masonry walls was examined experimentally with laboratory and insitu tests of existing and grouted walls. The paper provides a broad overview of research results regarding injection grouting, and provides detailed results from a single experimental study.

A series of stone masonry piers were constructed (Figure 1) and subjected to static lateral forces while a vertical compressive stress of 1.0 MPa was maintained.

Summary and Significant Findings

Systematic cement grouting can more than double the lateral resistance of stone-masonry walls. The level of improvement depends on the quality of the original wall or pier. Typical values of mechanical properties for plain and grouted walls are given in Table 1.

Typical relationships between shear stress and rotation angle for two plain and grouted walls are shown in Figure 2. The walls were a combination of stone and brick and tested by Sheppard and Tomasevic (1986).

A typical force-deflection curve from the experimental study reported in this paper is presented in Figure 3. As expected all piers failed in shear as a result of diagonal cracking. Some cracks passed through the stone units. All specimens could be classified into a single group with respect to their mechanical properties despite the fact that they were injected with grouts of different mechanical properties.

Related References

Sheppard, P., and S. Tercelj, "The Effect of Repair and Strengthening Methods for Masonry Walls," *Proceedings of the Seventh World Conference on Earthquake Engineering*, Istanbul, September 1980, Vol. 6, pp. 255-262.

Sheppard, P., and M. Tomasevic, "In-Situ Tests of Load Bearing Capacity of Walls of Old Masonry Buildings," *Proceedings of Fourth National Congress on Earthquake Engineering*, Vol. 2, pp. 86-92, 1986 (in Serbo-Croatian).

Tomsasevic, M., and P. Sheppard, "The Strengthening of Stone-Masonry Buildings for Revitalization in Seismic Regions," *Proceedings of the Seventh European Conference on Earthquake Engineering*, Vol. 5, Athens, September 1982, pp. 275-282.

Table 1: Summary of Mechanical Properties from Grouted Wall Tests

Type of masonry	*Description of tests*	*Grout strength* (MPa)	*Compressive strength* f_c (MPa)	*Tensile strength* f_t (MPa)	*Elastic modulus* E (MPa)	*Shear modulus* G (MPa)
Uncoursed stone, two layers, muddy sand (1)	(a), 1 wall					
	original:	-	0.5	0.02	197	70
	grouted:	33	1.0	0.12	825	100
Uncoursed stone, two layers, clean sand (2)	(a), 6 walls					
	original:	-	0.77	0.10	390	87
	grouted:	33	2.14	0.25	2744	145
Uncoursed stone, two layers, clean sand (4)	(b), 1 wall					
	original:	-	-	0.10	-	-
	grouted:	31	-	0.14	-	100
Uncoursed stone, mixed, clean sand (3, 4)	(b), 3 walls					
	original:	-	-	0.14	-	40
	grouted:	24	-	0.19	-	450

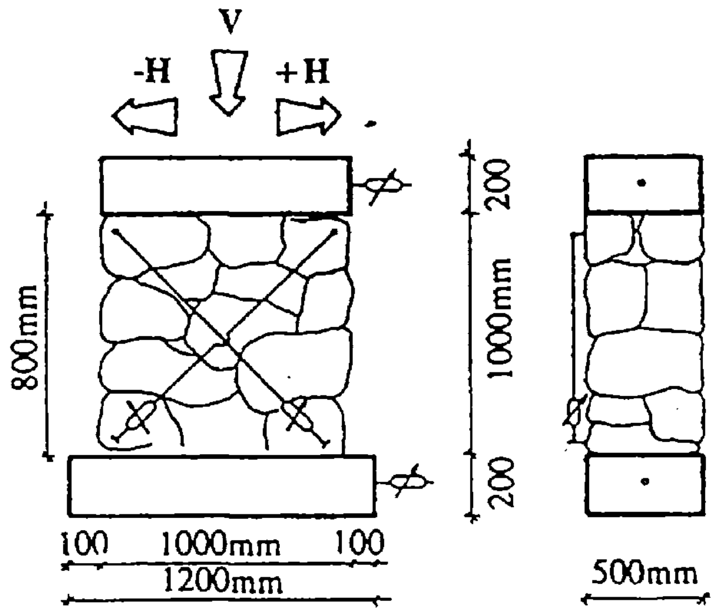


Figure 1: Description of Stone Masonry Pier Specimen

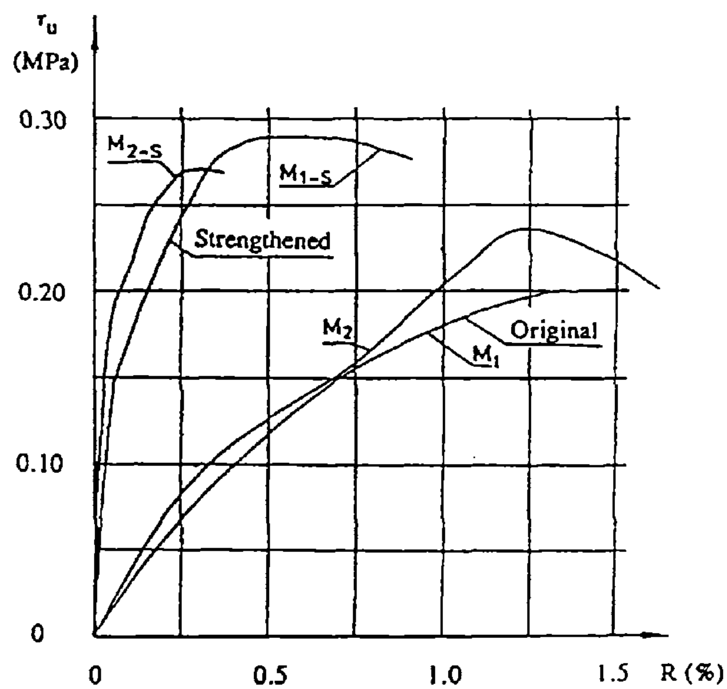


Figure 2: Measured Shear-Deflection Curves for Grouted and Plain Walls

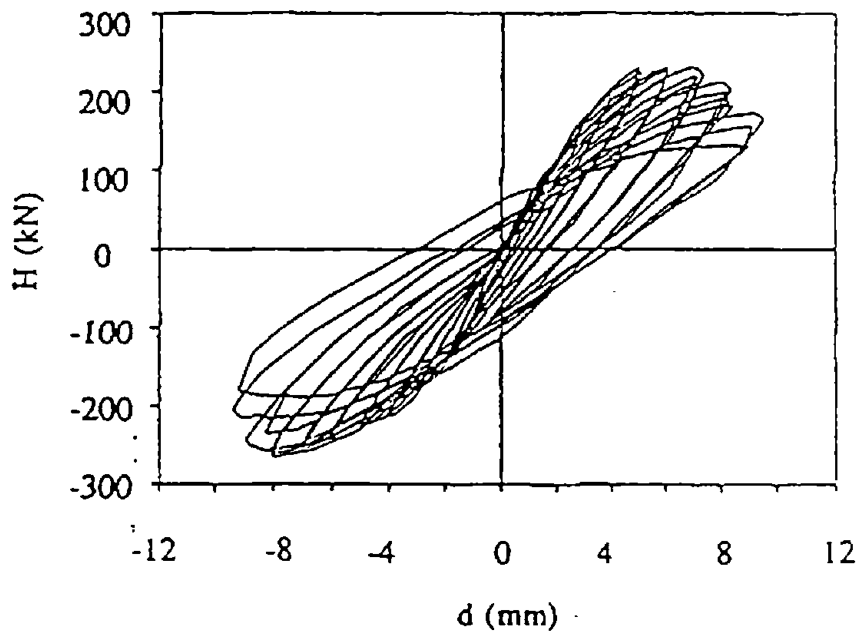


Figure 3: Typical Shear-Deflection Curve for Grouted Pier

Surface Coatings

SC1 Retrofitting of Confined Masonry Walls with Welded Wire Mesh

S.M. Alcocer, J. Ruiz, J.A. Pineda, and J.A. Zepeda

Proceedings of Eleventh World Conference on Earthquake Engineering, Acapulco, June 1996, Elsevier Science, Ltd., Paper No. 1471.

Objective

The objective of this research was to investigate the effectiveness of a reinforced concrete coating on enhancing in-plane seismic performance of confined masonry walls.

Rehabilitation Procedure

A cement plaster coating is parged on to the surface of a confined brick masonry wall. Welded wire fabric is embedded in the coating. The coating is anchored to the wall surface with bolts that are embedded into the masonry.

A confined masonry wall is an unreinforced brick wall that is surrounded with reinforced concrete columns and beams that are cast in place after the brickwork is in place.

Research Approach

Full-scale wall specimens were rehabilitated and subjected to a series of static lateral force reversals. Experimental parameters were the level of damage, number of stories, the diameter of the wire used for the mesh, and the types of anchors used to attach the coatings to the masonry.

A summary of test walls, cracking patterns and shear stress vs. drift relations are shown in Figure 1.

The two-story test structure was first loaded to cause damage (Specimen 3D) that was repaired by filling cracks with cement mortar and brick pieces, and then applying the wire mesh and surface coating (Specimen 3DR). Nails (40 mm long) were driven into the masonry by hand to serve as anchors for the coating. Metal bottle caps were used as spacers between the wall surface and the mesh. Nine nails per square meter were used in the north side of the walls while six nails per square meter were used in the south wall face.

The one-story walls were strengthened using the same reinforced coatings as for the two-story walls. The steel mesh was attached directly to the wall surface with no spacers. For test walls M1 and M2, nails (64-mm long) were driven by hand at a spacing of 300 mm for one face and 450 mm for the other face. In wall M3, 51-mm long Hilti anchors were driven with an impact wrench and were spaced at 450 mm on both faces.

Specimen M1 was reinforced with the minimum amount of horizontal steel specified in current Mexican masonry code provisions. Specimens M2 and M3 had approximately two and three times this amount of reinforcement respectively. Specimen M0 was a coated confined masonry wall with no reinforcement in the panel. Specimen MA was a confined masonry wall without a coating but with similar amounts of horizontal reinforcement in the bed joints as contained in the coating of M1.

The thickness of the coating was 25 mm for all specimens.

Summary and Significant Findings

The rehabilitation method was found to be an effective technique for improving seismic resistance. Application of a surface coating resulted in a more uniform diagonal cracking pattern and a much higher lateral strength.

For the two-story test structure, most of the damage occurred at the first story. Diagonal cracks in the first story were concentrated for the plain masonry walls but widely distributed for the coated walls. Strengthening of the first level walls resulted in an increased shear for the upper level that was unstrengthened. Several nails lost their anchorage or were bent loose. Bottle cap spacers increased the flexibility of the anchor and thus reduced its anchor strength in shear.

For the one-story specimens, diagonal crack patterns were again concentrated for the plain masonry and distributed for the coated masonry. For specimens M1 and M2, nails were found to be well anchored following testing. The Hilti anchors used in M3 also were observed to behave well. Cracking was distributed more in wall faces with sixteen anchors per square meter than in surfaces with nine anchors per square meter, however, the less dense spacing was found to result in acceptable behavior.

The rehabilitation technique resulted in a significant increase in lateral shear strength, and provided substantial inelastic deformation capacity. Force-deflection relations for rehabilitated test walls showed much more energy dissipation than for the control walls. Rounding of curves in the loading branch were attributed to yielding of panel reinforcement. Pinching in the load-reversal region was credited to wall shear deformations as well as local mortar damage.

The influence of the rehabilitation procedure on stiffness, strength and deformation capacity is illustrated in Figure 2 where envelopes of measured shear stress vs. drift ratio are presented. The lowest maximum drift for the rehabilitated walls was 0.7%.

The contribution of the wire mesh depended on the amount of steel, the type of anchor and the mortar quality.

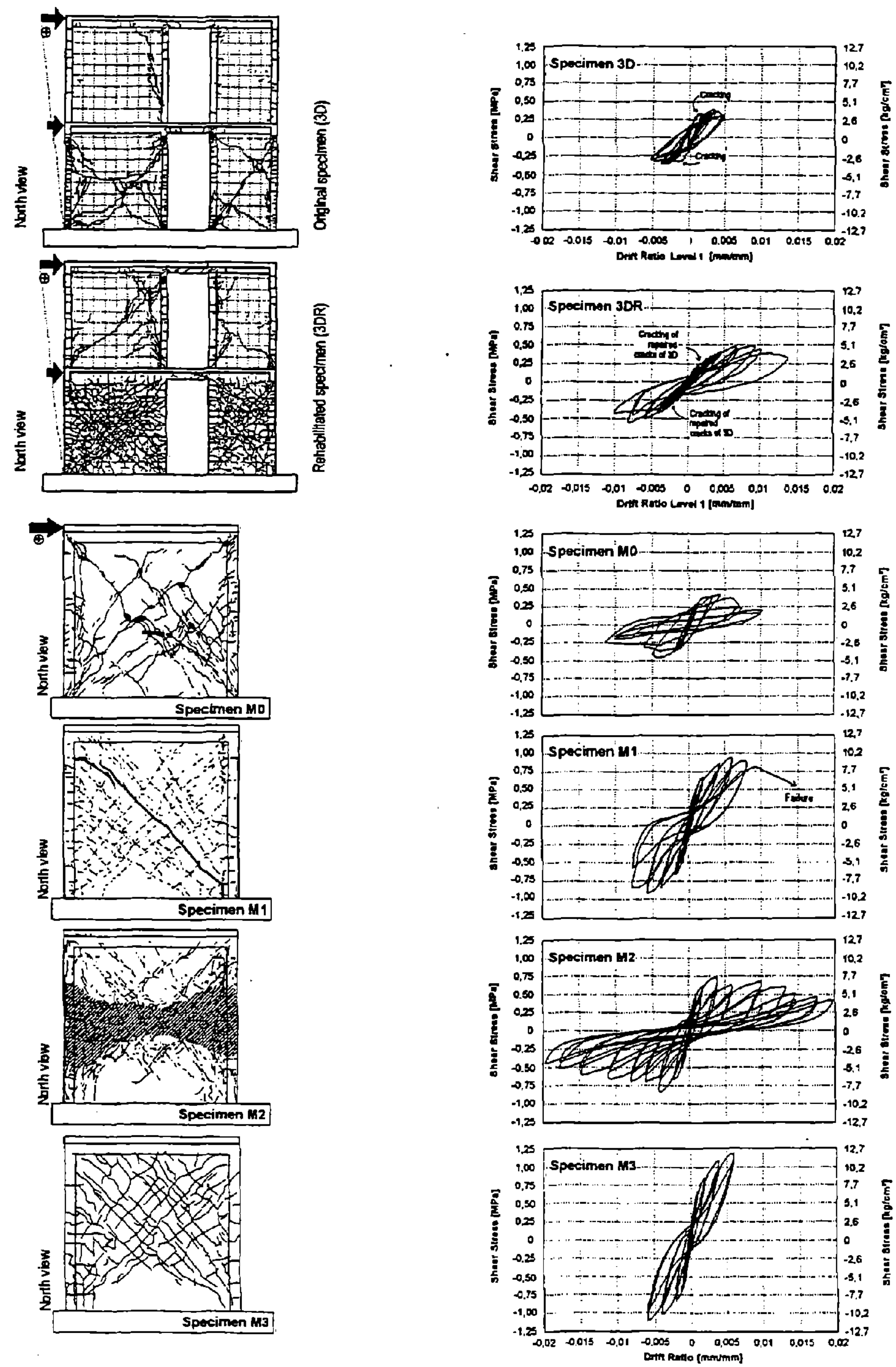


Figure 1: Final Crack Patterns and Force-Deflection Curves for Test Walls

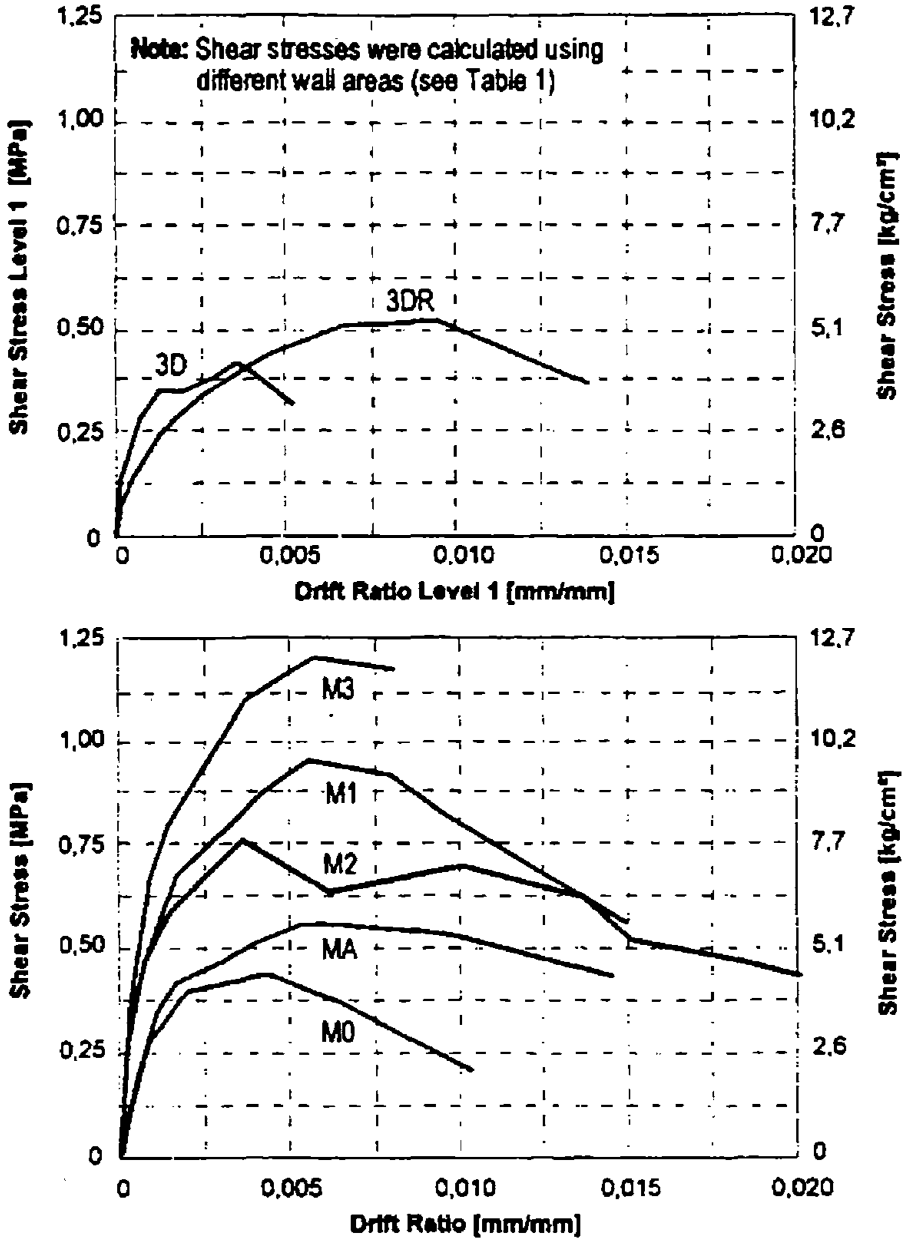


Figure 2: Measured Force-Deflection Envelope Curves

SC2 Seismic Retrofitting of URM Walls with Fiber Composites

M.R. Ehsani and H. Saadatmanesh
The Masonry Society Journal, Spring, 1996.

Objective

The objective of this research was to investigate the effectiveness of using composite fiber mesh beneath externally applied coatings to enhance the in-plane strength of unreinforced masonry walls.

Rehabilitation Procedure

The surface of a masonry wall is sandblasted and cleaned of any loose particles. Loose masonry units are reset and shear cracks are filled with mortar.

Three-foot wide fabrics are applied to the wall in vertical strips and pressed against an uncured epoxy which has been applied in thin layers to the wall surface (Figure 1). Successive strips of fabric are added with sufficient overlap. The bottom edge of the fabric is anchored to the existing footing with steel anchors and bolts. The top edge of the fabric is connected to the exterior surface of the existing wall parapet.

On the interior wall surface, the top of the fabric is wrapped around blocking which is fastened to the floor joists. After the fabrics are attached to the wall, a second coating of epoxy is applied to the exterior surface. Finally, the wall is covered with a special ultraviolet-protective layer of coating.

Research Approach

The flexural behavior of unreinforced masonry walls was examined by testing small masonry beams with four-point bending. Each beam specimen consisted of clay bricks stacked in a single wythe. Beam dimensions measured 8½ in. wide, 4 in. deep and 57 in. long.

Two types of epoxy were employed for this experiment. The first one was a two-component epoxy having consistency similar to cement paste. The second epoxy was also a two-component epoxy, but it had a lower viscosity than the first.

Two types of mortar were used for this study. The first, (Type M) had a cement:lime:sand ratio of 1:¼:3, while the second type (Type M*) had a 1:¼:5 ratio. The compressive strengths were found to be 4650 and 4100 psi for Type M and M* respectively.

Three fabric types of various strengths were used to investigate the various failure modes. The first was a glass fabric with an acrylic polyvinyl finish, while the remaining two contained unidirectional E-glass fibers. Stress-strain relations are compared in Figure 2 for glass and carbon-fiber composites, and steel.

Direct shear tests were done using the three types of fiber meshes. Each specimen consisted of three clay bricks covered with a composite fabric on both faces. The fabric pieces measured 4.5 in. wide by 8 in. long.

Summary and Significant Findings

Previous research has shown that flexible lightweight fiber composite materials are extremely strong in tension. This research has shown that flexural and shear strength, and ductility, can be enhanced significantly with the use of fiber composites. In particular, composite fabrics were found to strengthen shear transfer at the mortar-unit interface.

Retrofitted beams could resist loads equivalent to 24 times their dead weight. The shear specimens resisted high loads and failed in a ductile manner. Force-deflection curves for specimens tested in shear are shown in Figure 3.

The mode of failure was governed by the strength of the fabric. Lighter fabrics failed in tension while heavier fabrics were able to maintain the integrity of the specimen until the masonry units reached their capacities.

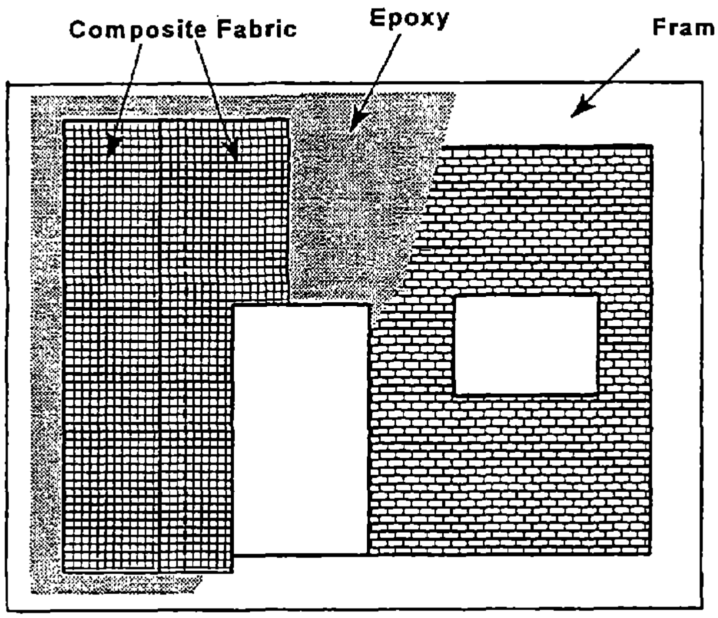


Figure 1: Application of Composite Fabric System

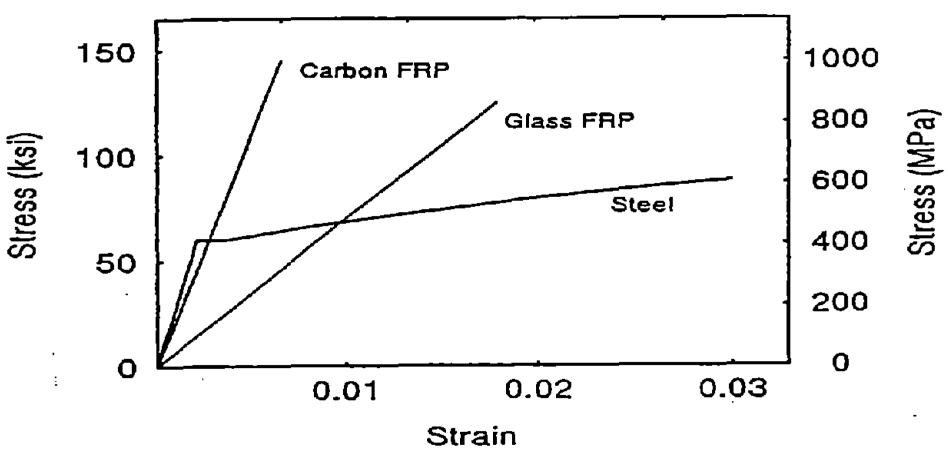


Figure 2: Typical Stress-Strain Curves for Fiber Composites

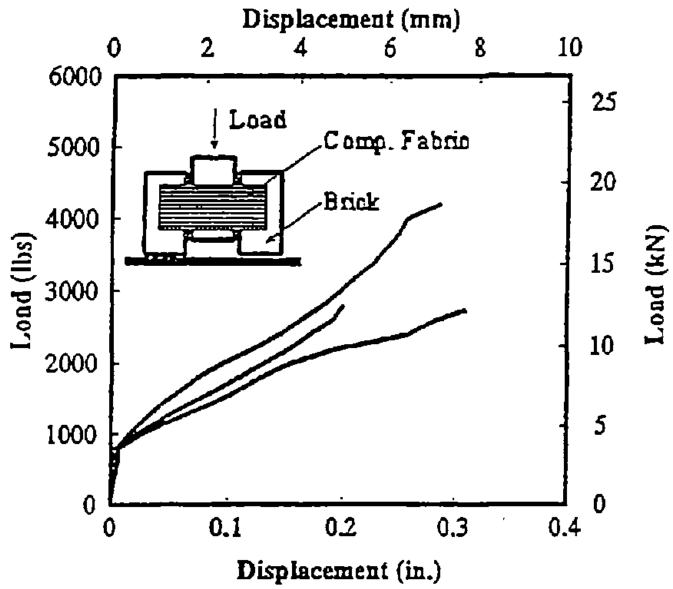


Figure 3: Load-Deflection Curves for Shear Tests

SC3 Repair and Strengthening of Masonry Assemblages Using Fibeг Glass

G. Ghanem, M. A. Zied, and A.E. Salama
Proceedings of the Tenth International Brick and Block Masonry Conference, University of Calgary, July 1994,Vol. 2, pp. 499-508.

Objective

The objective of the research was to investigate the effect of using external fiber glass reinforcement for increasing the in-plane strength of unreinforced masonry walls.

Rehabilitation Procedure

A coat of resin is applied to the surface of a masonry wall which is followed by the placement of a reinforcing mat, and a second resin coat.

Research Approach

A total of 24 grouted and ungrouted specimens at 1/3-scale were constructed to investigate the splitting tensile strength of concrete masonry. Two specimen shapes (Figure 1) were used to study tensile strength for different orientations of force relative to the mortar bed joint direction. Square specimens were used to apply stresses at directions normal and parallel to the bed joints. Octagonal specimens were used to apply stresses at 45 degrees to the bed joints.

Four model blocks were placed on top of each test panel to simulate gravity stress on the bed joints. After 24 hours, six specimens (3 of the square shape and 3 of the octagonal shape) were grouted using normal grout.

Fifteen specimens were strengthened with fiber glass reinforcement. Fiber reinforcement was applied to the central strip of panel (Method I in Table 1) or to the outer two-thirds of panel (Method II in table).

All specimens were tested to failure by splitting in tension. Nine unstrengthened specimens were repaired with fiber glass reinforcement and re-tested to failure.

Summary and Significant Findings

Using a fiber glass mat as reinforcement skin can significantly improve the tensile strength of ungrouted masonry. The specimens strengthened with fiber glass reinforcement showed an increase in strength of up to 600% relative to the unstrengthened test panels.

The increase in strength with reinforcing depended on the location of the fibers (Methods I or II in Table 1).

The tensile strength of the repaired specimens showed an improvement in strength relative to the original specimen. The increase in strength ranged from 10% to 60% and can be attributed to the good bond achieved between the fiber glass and the wall, in addition to the high tensile strength of the fiber glass.

Tensile strength was dependent on the orientation of the applied compressive force. Tensile debonding along the bed joints at the initiation of the first crack limited panel strength, and occurred at various force levels depending on the angle between the force and the bed joints. A summary of tensile strengths with load direction, reinforcing method and grouting procedure is shown in Figure 2.

Table 1: Summary of Test Results

Load Direction	Hollow	Grouted	Strengthened		Repaired Hollow
			MethodI	MethodII	
	Tensile Strength psi	Tensile Strength psi	Tensile Strength psi		Tensile Strength psi
Parallel	51	110	191	166	49
Normal	71	Not Tested	197	191	81
Diagonal	65	125	220	204	72

Note: Each Value is the Average of Three Readings
145 psi = 1 mpa

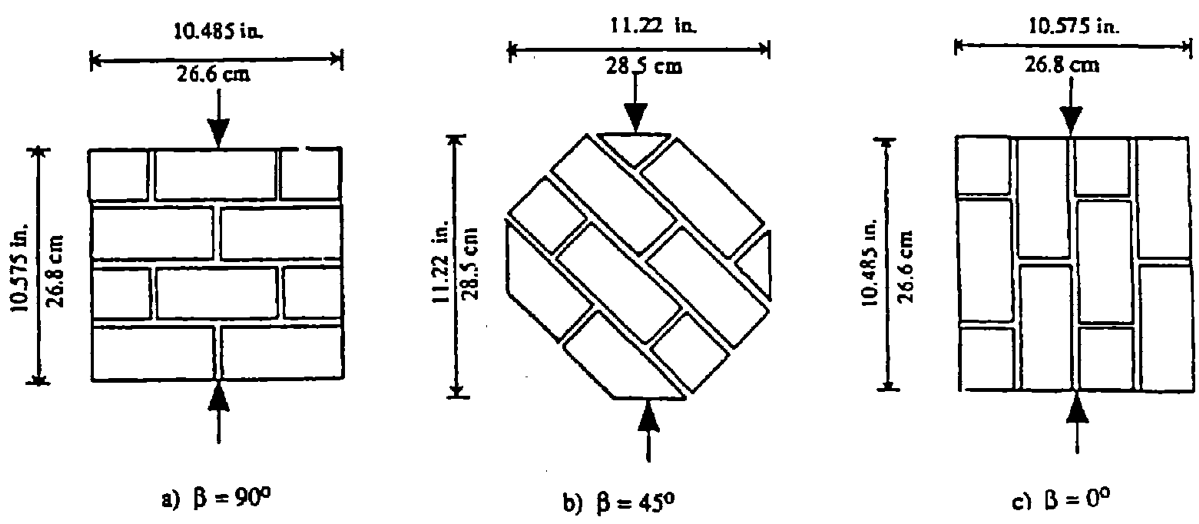


Figure 1: Description of Test Panels

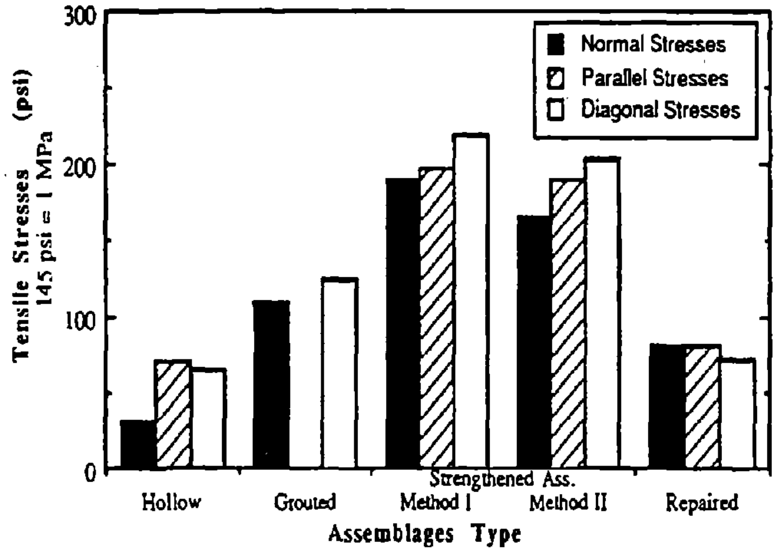


Figure 2: Tensile Strengths for Test Panels

SC4 Laboratory Testing of a Variety of Strengthening Solutions for Brick Masonry Wall Panels

D.L. Hutchison, P.M.F. Yong and G.H.F. McKenzie

Proceedings of the Eighth World Conference on Earthquake Engineering, Vol. 1, 1984, pp. 575-582.

Objective

The objective of the research was to investigate the effectiveness of various coating methods for improving the in-plane strength and stiffness of unreinforced masonry walls.

Rehabilitation Procedure

A common strengthening method for damaged brick masonry walls is to apply a layer of sprayed, reinforced concrete. Reinforcement may consist of a welded wire fabric or steel mesh anchored to the wall before application of the coating. Alternatively, reinforcing fibers can be mixed directly in the concrete, as is done with glass fiber reinforcement.

Research Approach

Six test walls were constructed and strengthened by various methods and subjected to cycles of in-plane lateral forces. The coating materials used in this study were normal concrete, glass-fiber reinforced concrete (GRC), steel-fiber reinforced concrete (FRC), and a ferrocement coating. In addition, two of the walls were externally prestressed and did not receive any coating material.

Prestressed walls - Specimens SW1 and SW2 were two wythe walls prestressed with 200 kN and 400 kN respectively.

Sprayed concrete wall - Specimen SW3 (Figure 1a) was a single-wythe wall constructed in running bond. This wall received a layer of sprayed ready-mix concrete on one side only.

GRC coated wall - Specimen SW4 was a single-wythe brick wall (Figure 1b). Both faces received a single coating of GRC by hand plastering.

FRC coated wall - Specimen SW5 was a single-wythe wall on which a FRC coating was applied to both sides. The coating used "enlarged end" fibers with an overall length of 18 mm. Dowels were inclined for the FRC wall and mounted on an angle section which was connected to top and bottom beams by four D24 bars.

Ferrocement coated wall - Specimen SW6 was a single-wythe wall on which a steel mesh was anchored to the backing brickwork and then a 20-mm thick coating was applied.

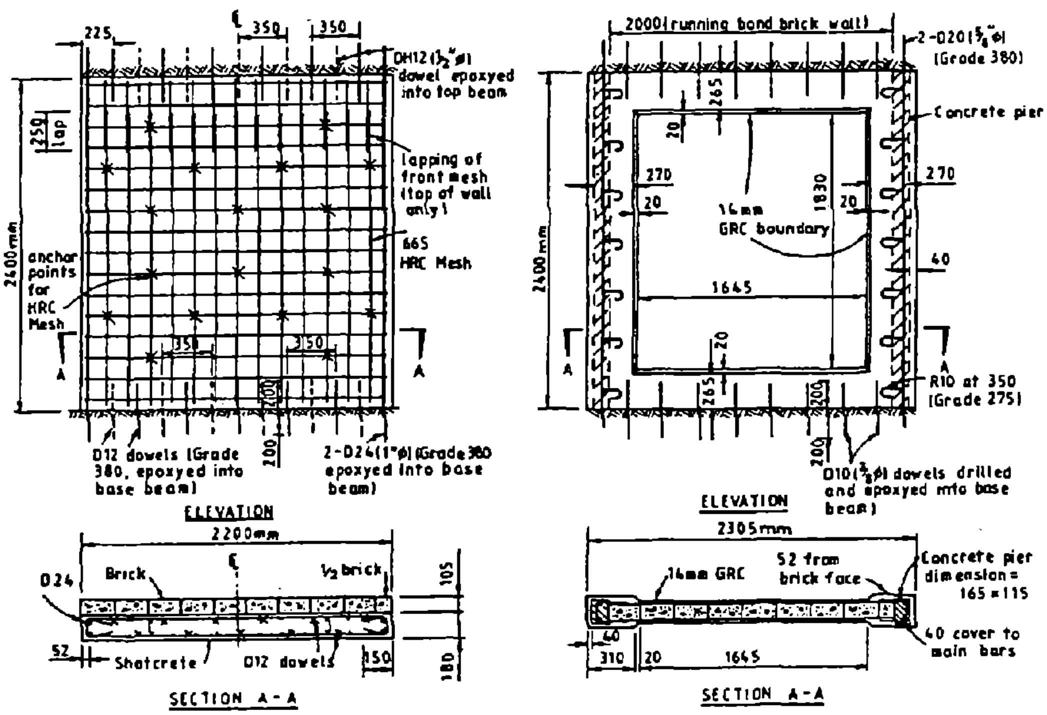
Summary and Significant Findings

A sample force-deflection hysteresis curve is shown in Figure 2 for the GRC coated wall. The initial stiffness of the wall reduced with cracking. Disjointed diagonal cracks were observed in the coating material. A significant loss of strength was observed at an 8-mm deflection as a result of the sudden development of a major diagonal crack.

Lateral force-deflection behavior for the series of six test walls is presented in Figure 3. The prestressed walls (SW1 and SW2) did not perform as expected because sliding was not restrained. From these tests the general applicability of prestressing as a strengthening solution could not be assessed.

Behavior of the ferrocement coated wall (SW6) did not compare well with the other coated walls due to poor bonding between the coating and the brickwork.

Walls with sprayed concrete (SW3), glass-fiber reinforced concrete (SW4), and steel-fiber reinforcement concrete (SW5) were the most promising methods for improving in-plane strength of unreinforced brick masonry walls.



(a) Sprayed Concrete Wall (b) Glass Reinforced Cement Wall

Figure 1: Description of Test Walls

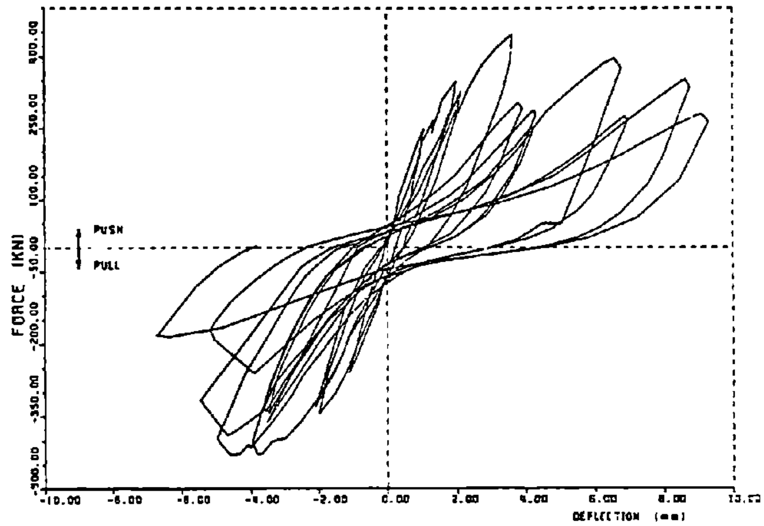


Figure 2: Measured Force-Deflection Behavior of GRC Coated Wall

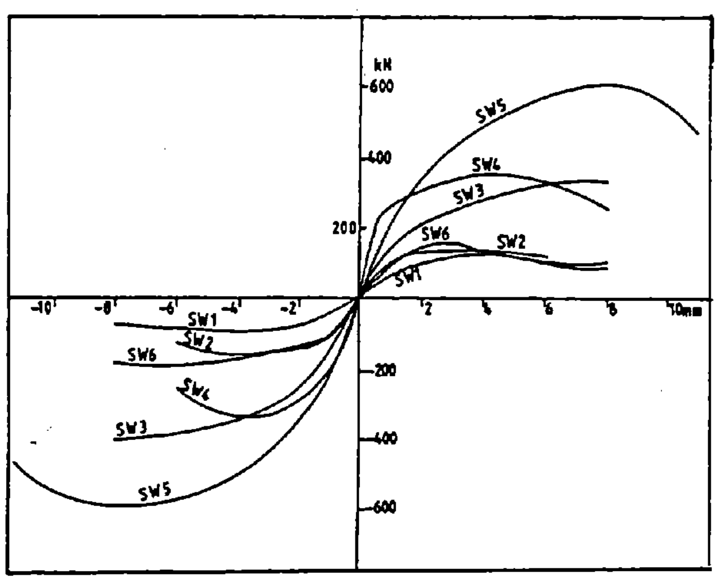


Figure 3 : Envelope Force-Deflection Curves for Test Walls

SC5 Behavior of Repaired/Strengthened Unreinforced Masonry Shear Walls

M. Irimies and L. Crainic

Proceedings of Sixth North American Masonry Conference, Drexel University, June 1993, pp. 555-563.

Objective

The objective of the research was to investigate the effectiveness of repairing damaged masonry walls with cement paste injected into cracks and in-plane strengthening by application of a reinforced mortar coating.

Rehabilitation Procedure

Mortar layers, 30-mm thick, are applied to both surfaces of a wall after cracks are filled with cement paste. The mortar coating is reinforced with welded wire fabric (wire diameter is 6 mm with a mesh of 250 mm, wire strength is 280 MPa).

Alternatively, mortar pumped layers are applied to both faces of a wall without filling cracks with cement paste. The layers are 30 mm thick and are reinforced with wire mesh (wire diameter is 4 mm with a mesh of 200 mm, wire strength is 500 MPa).

No connectors are used to tie the mesh to the wall surface.

Research Approach

A series of six, two-story shear wall test structures (Figure 1) were constructed and subjected to in-plane lateral forces until failure. Walls were constructed with clay bricks (compressive strength equal to 10 MPa) and a mortar mix of cement, lime and sand (1: 2.8 : 13) with a compressive strength of 1.0 MPa. Walls were constructed with flanges so that behavior of the webs could be examined under high shear forces.

Test walls were first loaded to observe behavior without remedial measures (test walls P1, P2 and P3), and then repaired and rehabilitated to examine the effectiveness of the combination of injected cement paste and mortar coating (P2s) and the pumped mortar layers procedure (P3s). Two additional walls (P1s) were repaired by injecting cement paste into cracks, but no mortar coating was applied.

Static, lateral forces were applied in an inverted triangular distribution. Vertical compressive stress equal to 0.25 MPa was applied and maintained at a constant level during the lateral load tests.

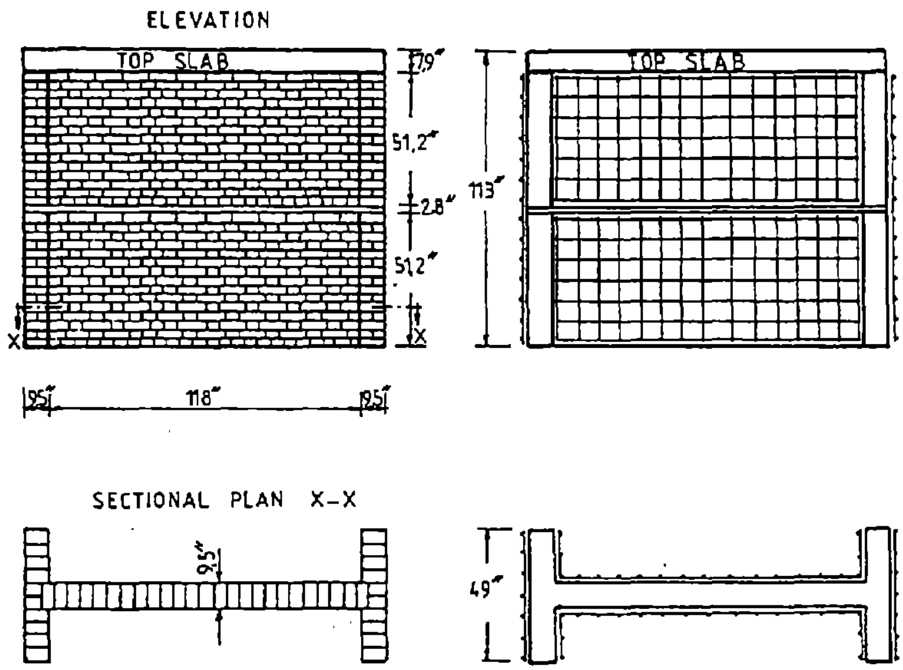
Summary and Significant Findings

Behavior of virgin walls was a result of horizontal cracking in the flanges and diagonal cracking in the webs (Figure 2). Cracking occurred in the mortar joints. After cracking, the force capacity of the test walls remained constant and was governed by friction along the bed joints creating a ductile response.

Walls repaired by filling cracks with cement paste cracked at the same force level as for virgin specimens (Figure 3). The resulting behavior was similar to that of the virgin wall.

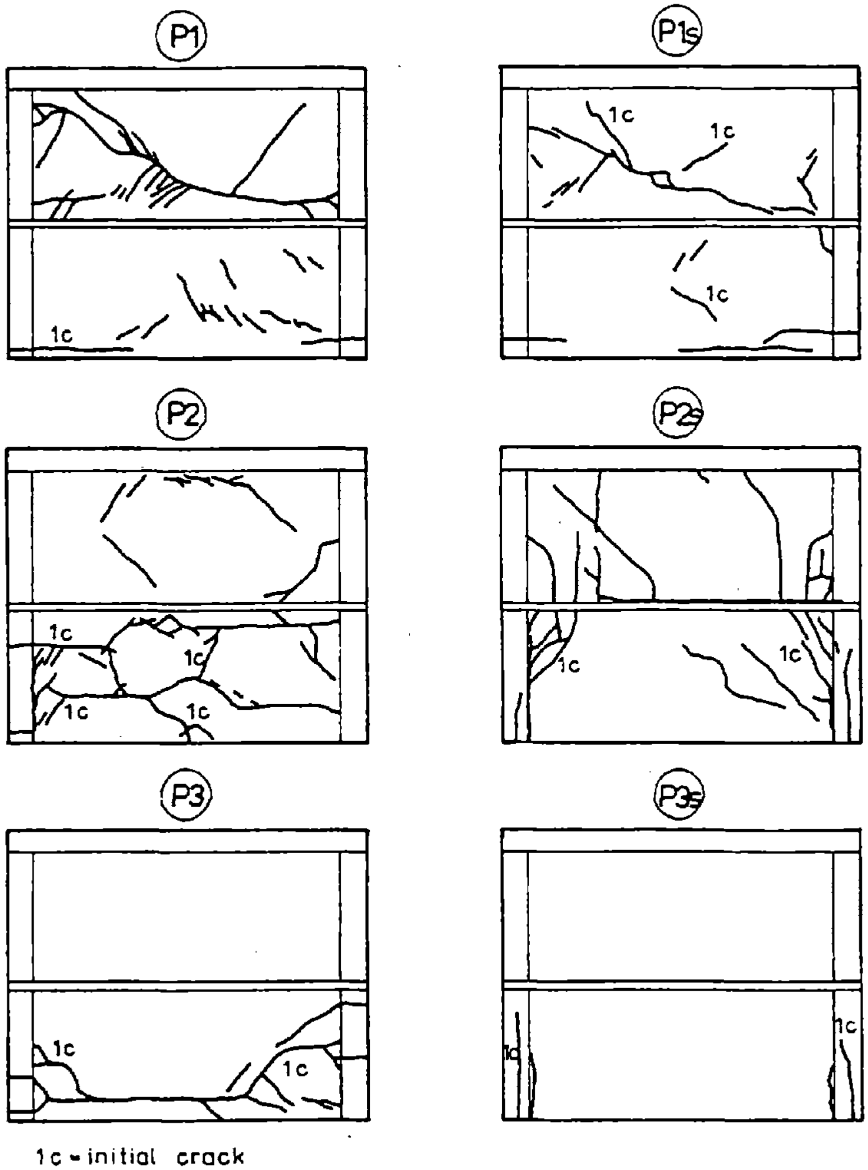
Both rehabilitation methods resulted in a substantial increase in stiffness. The walls with a mortar coating rocked about their base. When this rotation was restrained with external devices, a concentration of cracking in the compressed flanges developed. Vertical cracks occurred in the coating of the exterior face of the compressed flange spalling the masonry, and vertical cracks developed in the bricks and in the mortar on the flange, as well at the flange to web interface. No spalling of the coating was observed.

Applying the mortar coating with a pump without filling cracks with cement paste resulted in similar behavior as for the wall that had the cracks filled with paste.



(a) wall dimensions (b) coating reinforcement

Figure 1: Description of Test Walls



(a) Plain Walls

(b) Repaired/Rehabilitated Walls

Figure 2: Cracking Patterns for Test Walls

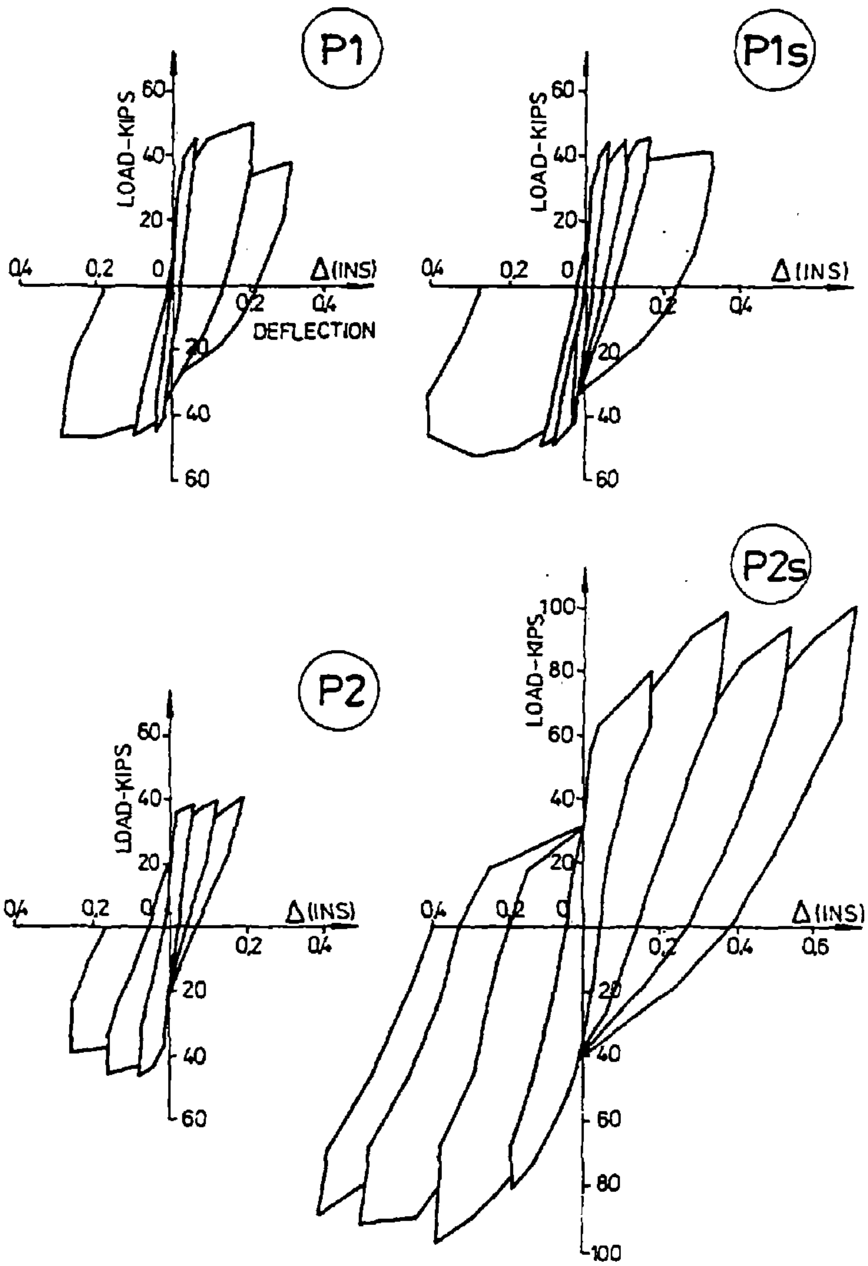


Figure 3: Measured Force-Deflection Curves for Test Walls

SC6 Strengthening of Damaged Masonry by Reinforced Mortar Layers

M. Jabarov, S.V. Kozharinov, and A.A. Lunyov
Proceedings of the Seventh World Conference on Earthquake Engineering, Vol. 15, No. 3, 1985, pp. 73-80

Objective

The objective of the research was to examine the effectiveness of repairing damaged unreinforced clay-unit masonry walls with a coating of reinforced mortar.

Rehabilitation Procedure

A cement mortar is parged on the surface of a cracked brick wall. The mortar layer is approximately 25 mm thick and is reinforced with a wire mesh or reinforcing bars placed in a diagonal direction.

Research Approach

Two parallel masonry walls with openings were subjected to in-plane, static lateral forces. The test walls (Figure 1) were 5.6 m high by 7.0 m long, and were 380 mm thick. The walls were joined with a 100 mm thick concrete slab at the first and second levels where the lateral forces were applied.

After the first series of tests on the unstrengthened wall, a mortar layer was applied to the faces of the two exterior piers at each story. Diagonal reinforcing bars (3 - 5B-I bars) were embedded in the mortar layer for these exterior piers (Figure 2). After testing of the partially repaired structure, the interior piers at each of the two stories were strengthened with a parged cement coating. A steel mesh (200 x 200 mm grid) was placed within the coating for the interior pier.

Lateral forces were applied at static rates. At various stages of loading, the test walls were subjected to forced harmonic vibrations to examine dynamic characteristics including frequency and damping ratios.

Summary and Significant Findings

The strength and stiffness of masonry strengthened by reinforced layers depend on the layer thickness and cement mortar strength, the reinforcement quantity and the means of the bonding to the wall.

Observed cracking patterns for the test walls are depicted in Figure 3. For the unstrengthened wall, cracking was initiated at approximately two-thirds of the peak lateral force. Cracks continued to propagate along the diagonals of the piers until a peak force of 910 kN was reached.

After strengthening the exterior piers, the lateral force capacity was increased to 1175 kN. Small cracks in the exterior piers were detected at approximately a third of this value. The force capacity of the test walls with the interior piers strengthened were 2.9 times the capacity of the unstrengthened wall.

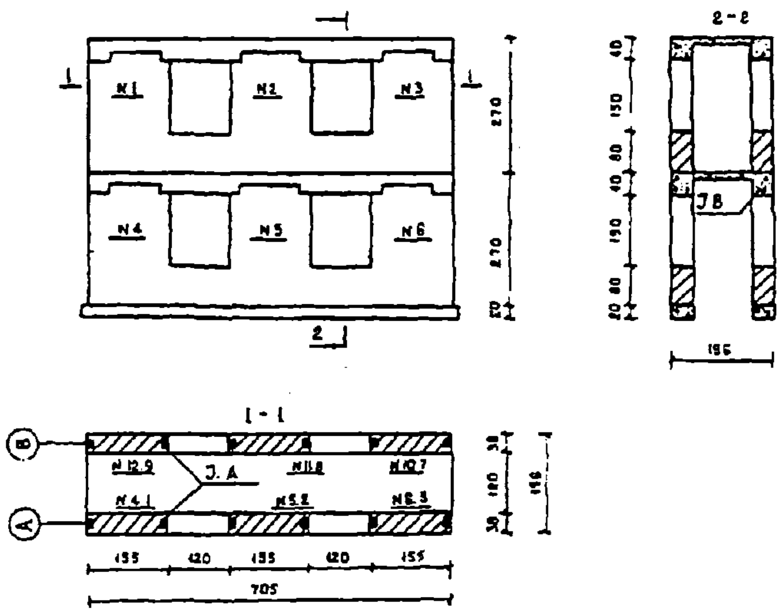


Figure 1: Description of Test Structure

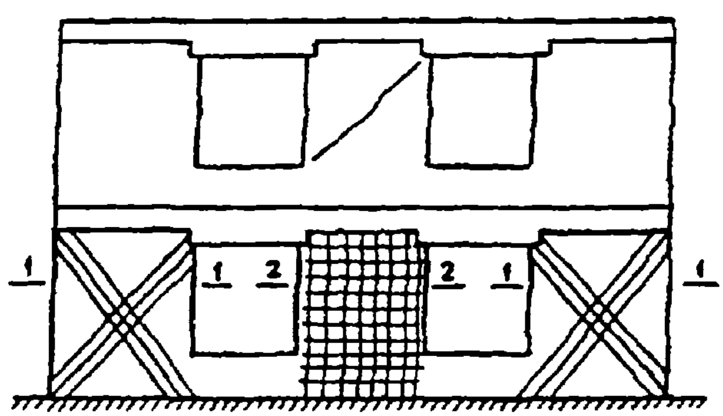
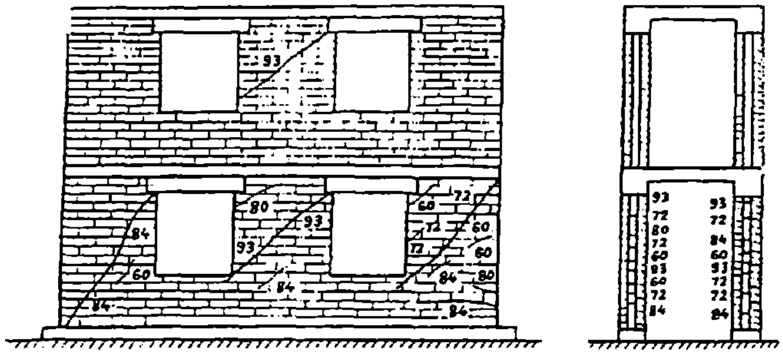
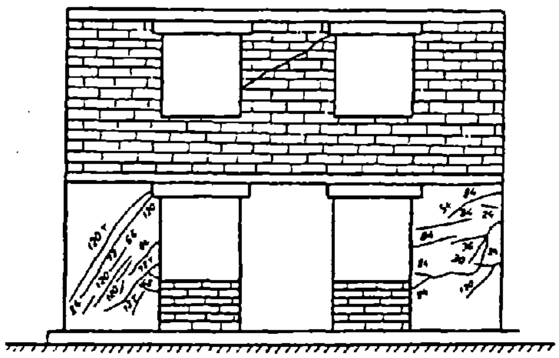


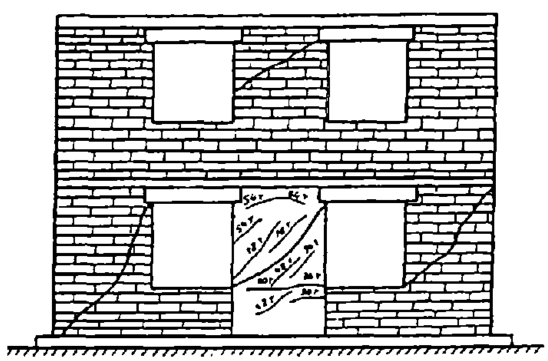
Figure 2: Reinforcing of Repaired Piers



(a) unstrengthened test wall



(b) test wall with strengthened exterior piers



(c) test wall with strengthened interior pier

Figure 3: Observed Crack Patterns

SC7 Shotcrete Retrofit for Unreinforced Brick Masonry

Lawrence F. Kahn

Proceedings of the Eighth World Conference on Earthquake Engineering, Vol. 1, 1984, pp. 583-590.

Objective

The objective of this research was to investigate the effectiveness of using reinforced shotcrete to enhance the seismic performance of unreinforced brick masonry walls. In particular, in-plane composite behavior of masonry and shotcrete was studied.

Rehabilitation Procedure

Shotcrete is sprayed on to the surface of an unreinforced brick masonry wall over a layer of reinforcement. Reinforcement consists of either welded wire fabric or conventional reinforcing bars. Dowels are drilled into a wall panel to provide better composite action.

Research Approach

Seventeen brick panels were constructed, of which fifteen were coated with a layer of shotcrete. The shotcrete in each panel was reinforced with a welded wire fabric (W4 plain bars in 6 inch centers each way). Nine single wythe 3 x 3 foot panels were used to investigate the effects of surface bond with the panel either dry, wet or epoxy coated prior to shotcreteing.

Six double wythe 4 x 4 foot panels were used to determine whether dowels can enhance the connection bond between masonry and shotcrete. Additionally, a single wythe, 3 x 3 foot panel and a double wythe 4 x 4 foot panel were tested without a shotcrete surface.

Each panel was oriented at 45° and tested with a single, static reversed cycle loading across the diagonal. After reaching the ultimate load, deflections were increased until the load dropped to about one-half of ultimate. When the load decreased to zero, the panel was rotated 90° and compressive load was applied.

Summary and Significant Findings

Application of a layer of reinforced shotcrete to unreinforced masonry panels was shown to be an effective method for greatly increasing the in-plane diagonal strength plus providing reversed cyclic and inelastic deformation capacity. Measured force-strain curves for shotcreted walls (Figure 1) illustrate stable behavior.

In all single and multiple-wythe specimens, the masonry cracked diagonally through the bricks.

Dowel bars epoxy bonded into drilled holes did not improve the composite panel response or the brick-shotcrete interaction. The increase in strength appears to be provided by the shotcrete.

Panels reinforced with the welded wire fabric responded with significant increases in strength after first cracking and large inelastic deflection capacity compared to the unreinforced panels. Panels without welded wire fabric showed no post cracking strength. Panels with shotcrete and reinforcement were able to deflect inelastically and remain intact even after being subjected to fully reversed cyclic loading.

Related Reference

L. Kahn, "Shotcrete Strengthening of Brick Masonry Walls," *Concrete International*, July 1984, Vol. 6, Issue 7, pp. 34-40.

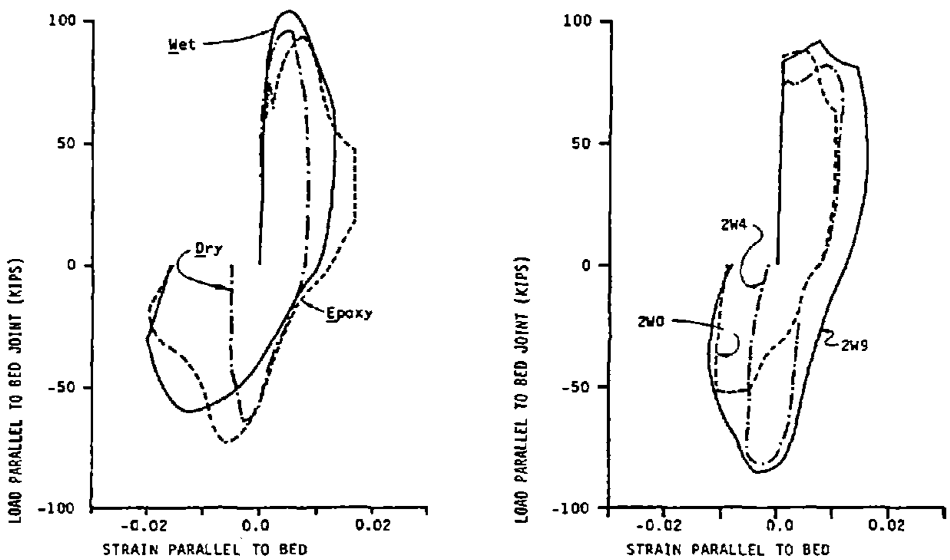


Figure 1: Measured Force-Strain Behavior of Shotcrete Panels

SC8 The Seismic Renovation and Repair Potential of Ferrocement Coatings Applied to Old Brick Masonry Walls

H. H. Lee and S.P. Prawel

Proceedings of the Sixth Canadian Conference on Earthquake Engineering, June 12-14, 1991, pp. 663-670.

Objective

The objective of this research was to investigate the effectiveness of thin ferrocement coatings to improve the in-plane and out-of-plane strength and behavior of unreinforced clay-unit masonry walls.

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 is embedded into the coating. The coating is adhered to the wall with a series of connectors in addition to surface bonding.

Research Approach

The first series of tests were done to determine the bonding characteristics between the coating and a masonry wall, and the required connector size and spacing. Diagonal compression tests were done for this purpose using 42 in. square panels that were 8-in. thick.

Three of the panels were coated on both sides with the coated ferrocement overlay, each using a different size of mesh while one plain panel was left as a control specimen. A 1/2 in. coating of ferrocement was applied. Force-deflection relations are given in Figure 1.

The second series of tests were done to study the hysteretic behavior of the rehabilitated masonry walls with ferrocement. A series of test walls were subjected to reversed cycles of lateral force. Lateral forces were applied either parallel or transverse to the plane of the walls.

The third series of tests were done to investigate the dynamic properties of the coated masonry walls. Test walls were subjected to simulated earthquake motions using a shaking table.

A total of 16 walls each 6 ft. wide, 8 ft. high and 8 in. thick were built from reclaimed old bricks, half of which were coated with a layer of ferrocement on each side. A 1/2 in. x 1/2 in. x 19 gauge mesh was placed in the cement coating. 1/4 inch bolts spaced at 12 inches were placed through the wall thickness.

Summary and Significant Findings

The mode of failure for both the coated and plain wall specimens subjected to either out-of-plane (Figure 2) or in-plane loading (Figure 3) was flexural. For both in-plane and out-of-plane loadings, the original stiffness of coated masonry walls was increased up to two times as much as that for the uncoated walls. The shear strength was increased from 50% to 100%, and the flexural strength was increased approximately three times with the retrofit method (Figures 4 and 5).

Energy dissipation capacity, ductility, and stiffness were increased with the coatings.

Performance of the test walls when subjected to dynamic loads was enhanced.

Related References

Prawel, S.P., and A.M. Reinhorn, "Seismic Retrofit of Structural Masonry Using a Ferrocement Overlay," *Proceedings of the Third North American Masonry Conference*, University of Texas at Arlington, June 1985, pp. 59-1 to 59-19.

S.P. Prawel, A.M. Reinhorn and S.A. Qazi, "Upgrading the Seismic Resistance of Unreinforced Brick Masonry Using Ferrocement Coatings," *Proceedings of the Eighth International Brick/Block Masonry Conference*, Dublin, Ireland, September, 1988, pp. 785-791.

Prawel, S.P. and H.H. Lee, "The Performance of Upgraded Brick Masonry Piers Subject to In-Plane Motion," *Proceedings of the Fourth U.S. National Conference on Earthquake Engineering*, Palm Springs, May 1990, Vol. 3, pp. 273-282.

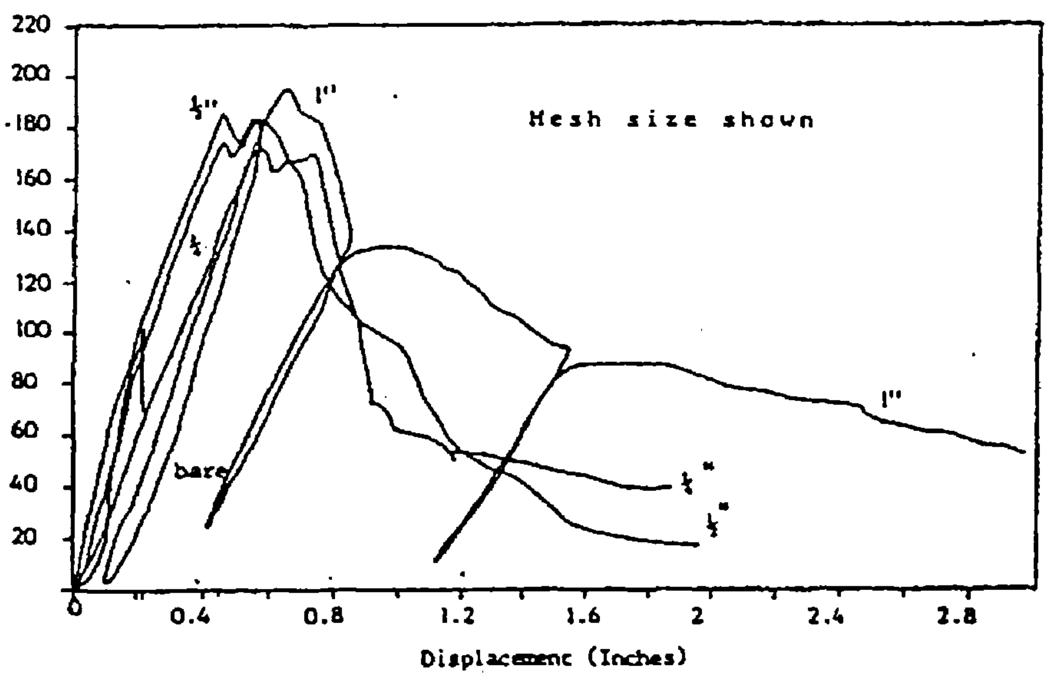
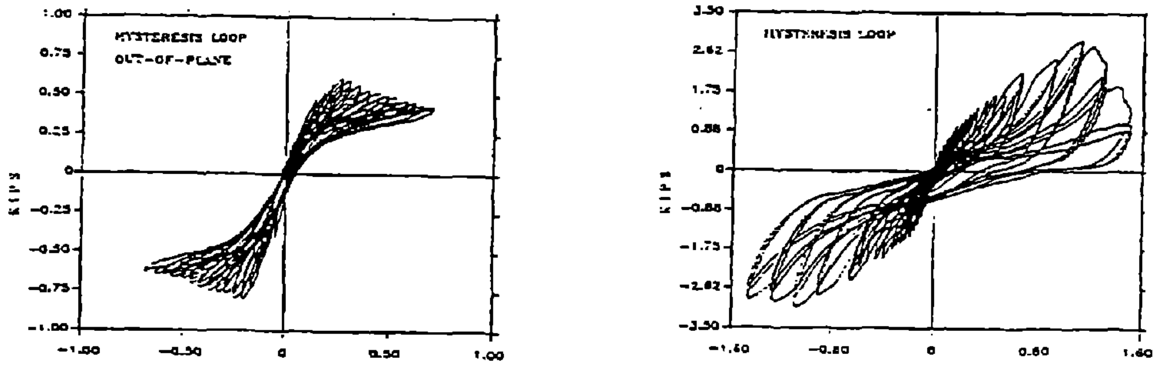


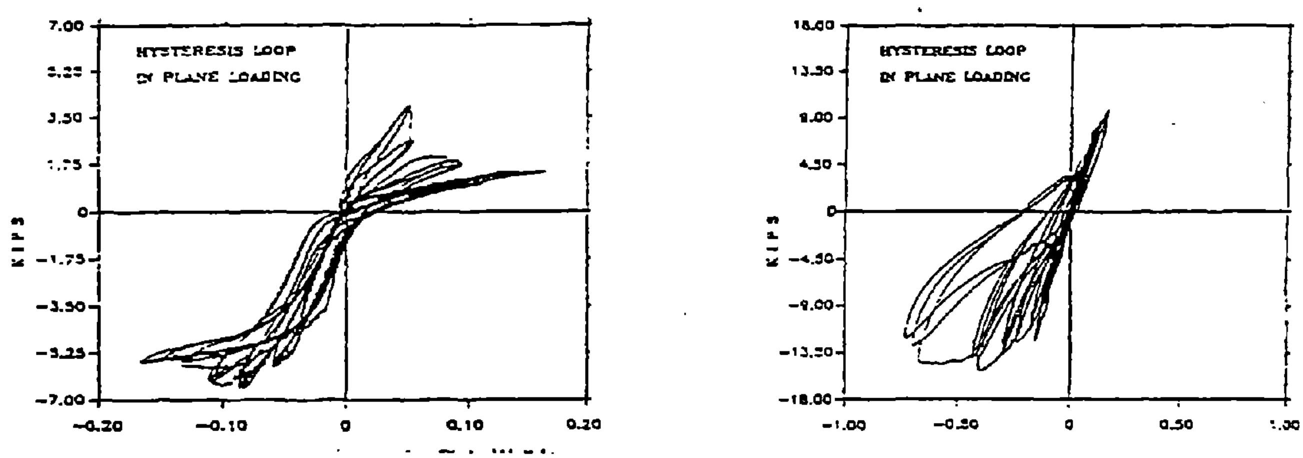
Figure 1: Load vs. Vertical Deformation from Diagonal Split Test



(a) Plain Wall

(b) Coated Wall

Figure 2: Measured Out-of-Plane Behavior



(a) Plain Wall

(b) Coated Wall

Figure 3: Measured In--Plane Behavior

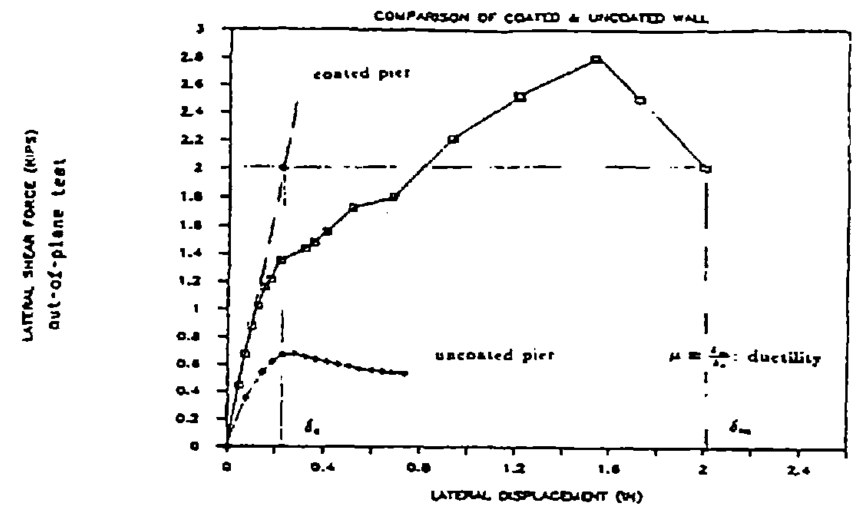


Figure 4: Out-of-Plane Behavior for Coated and Plain Walls

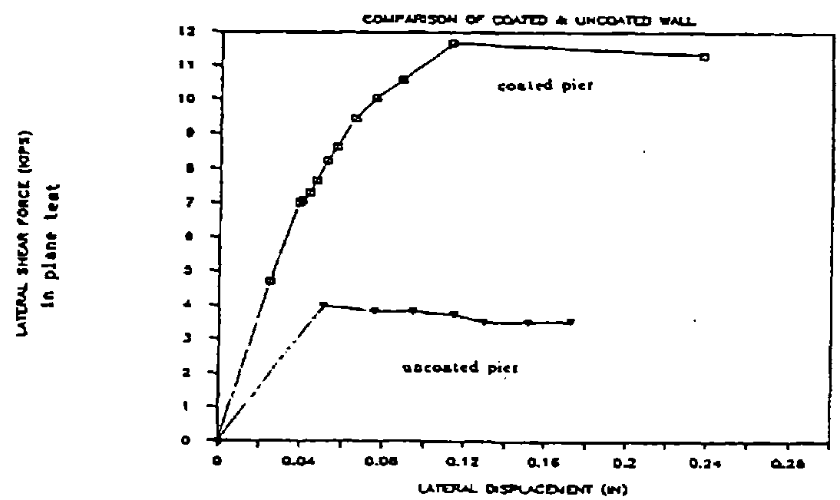


Figure 5: In-Plane Behavior for Coated and Plain Walls

SC9 Calculating Methods of Strengthened and Repaired Brick Masonry Structures for Earthquake Resistance

Zezhen Niu
Proceedings of USA-PRC Joint Workshop on Earthquake Disaster Mitigation through Architecture, Urban Planning and Engineering,
Beijing, pp. 21-1 to 21-16.

Objective

The paper summarizes methods for estimating in-plane shear strength enhancements for unreinforced clay-unit masonry walls, columns and chimneys based on test data from various research institutes in the People's Republic of China. For masonry walls, the following three rehabilitation methods are addressed: cement mortar coating, reinforced cement mortar coating, and reinforced concrete columns with tie beams. Equations are given to represent the lateral strength of unreinforced masonry components rehabilitated per these methods.

Rehabilitation Procedure

An unreinforced masonry wall is strengthened by applying a cement mortar coating to both wall faces (Figure 1). The thickness of the coating varies from 1.5 to 3.0 cm (0.6 to 1.2 inch). As an added measure, the coating may be reinforced with wire mesh, and S-shaped reinforcing bars are placed through the wall thickness to tie the mesh. The thickness of the reinforced coating is from 2.5 to 4.0 cm (1.0 to 1.6 inch).

The paper also mentions a strength enhancement procedure where reinforced concrete columns are placed at the ends of a wall panel. A beam or steel rod is placed to tie the columns together at the top of the wall panel. The intention of this method is to improve lateral strength and ductility of the wall panel through added confinement of the panel.

Strength of Coated Walls

Based on tests of unreinforced masonry wall panels within the PRC, the author has formulated the following expression to estimate the lateral strength, P, of a wall strengthened with external coatings.

$$P = \frac{m}{\xi}\left(\alpha_c P_c + \alpha_s P_s + \alpha_g P_g\right) \qquad (1)$$

where:

m is a coefficient of construction condition (typically 0.8)

ξ is a coefficient for non-uniform shear stress distribution (1.2 for rectangular section)

P_c, P_s, and P_g are the lateral shear strength of the masonry, the coating and the reinforcement respectively (see Table 1)

α_c, α_s, and α_g are coefficients of effectiveness for the masonry, the coating and the reinforcement respectively (see Table 1)

The author has also formulated the following equation for the lateral stiffness, B, of a strengthened wall.

$$B = \beta E_c A_c + E_s A_s \qquad (2)$$

where:

β is the coefficient of quality of the original wall considering effects of cracking (1.0 for uncracked wall, 0.84 for cracked wall)

A_c is the cross sectional area of the masonry

A_s is the cross sectional area of the additional mortar coating

E_c is the modulus of elasticity of the masonry

E_s is the modulus of elasticity of the coating

For walls strengthened with unreinforced or reinforced coatings, the contribution of the masonry shear strength is added with that of the coating. For uncracked walls, the participation of the masonry is limited by the tensile strength of the masonry. Expressions for α_c in Table 1 are based on the coating reaching its cracking strength for unreinforced coatings, or yielding of reinforcement for reinforced coatings. For cracked walls, the masonry shear strength is taken as simply a frictional coefficient times the vertical compressive force, and its participation is taken at 84% when used with an unreinforced coating and at 100% when used with a reinforced coating.

The shear strength of the mortar coating is taken as an index value of mortar shear stress times the area of the coating. The participation of the coating for uncracked walls is taken as 100% for unreinforced coatings and 84% for reinforced coatings. Participation of the coating for cracked walls is again taken as 100% for unreinforced coatings but is neglected entirely for cracked, reinforced coatings.

Wall shear strength attributable to the coating reinforcement is considered in full for either uncracked or cracked walls.

Comparison of wall strength values per Equation 1 with measured values from experiments showed a good correlation.

Table 1: Strength Parameters for Coated Walls

	Wall Condition	Masonry α_c	Masonry P_c	Coating α_s	Coating P_s	Reinforcement α_g	Reinforcement P_g
Unreinforced Coating	uncracked	$0.2+0.13\sigma_o/R_j$	$R_\tau A_c$	1.00	$R_{sj}A_s$	-	-
	cracked	0.84	$f\sigma_o A_c$	1.00	$R_{sj}A_s$	-	-
Reinforced Coating	uncracked	$0.1+0.06\sigma_o/R_j$	$R_\tau A_c$	0.84	$R_{sj}A_s$	1.00	$2n_sR_g\alpha_g l/s$
	cracked	1.00	$f\sigma_o A_c$	0.00	$R_{sj}A_s$	1.00	$2n_sR_g\alpha_g l/s$

where:

f masonry frictional coefficient (typically 0.7)
l length of wall
n_s number of reinforcing layers
R_g tensile strength of reinforcement
R_j tensile strength of brick masonry
R_s compressive strength of mortar coating
R_{sj} shear strength of mortar coating, approximately $R_{sj} = 2\sqrt{R_s}$
R_τ shear strength of masonry, $R_\tau = R_j\sqrt{1+\frac{\sigma_o}{R_j}}$
s spacing of reinforcement
α_g area of reinforcing bar or wire
σ_o vertical compressive stress

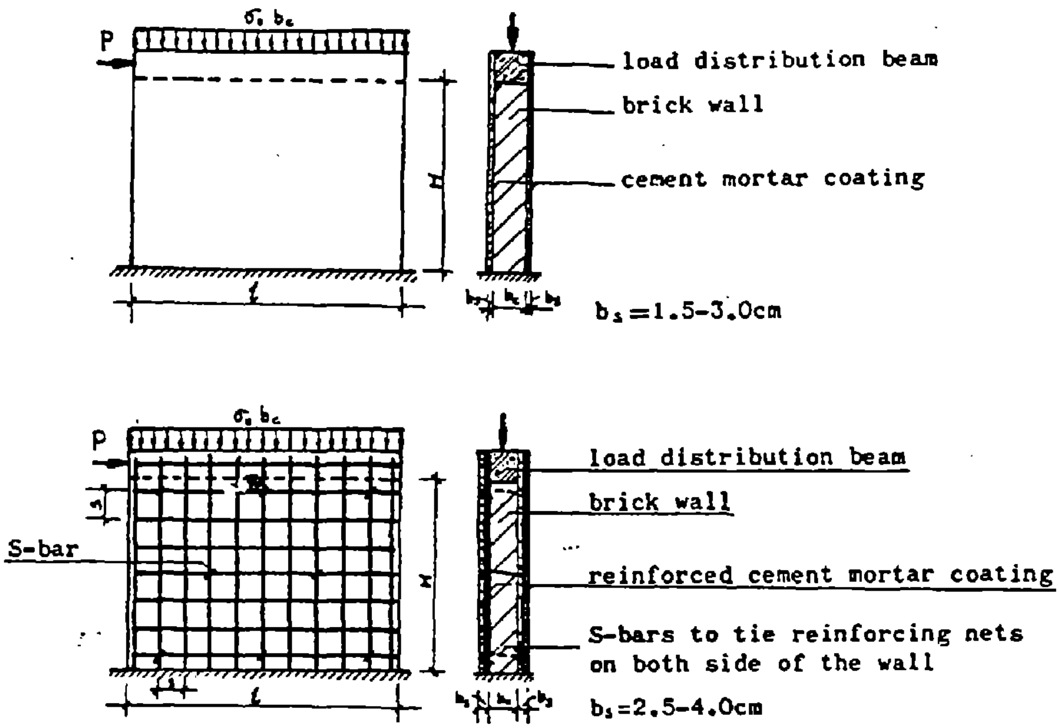


Figure 1: Unreinforced and Reinforced Coating Methods

SC10 The Performance of Upgraded Brick Masonry Piers Subject to In-Plane Motion

S. P. Prawel and H. H. Lee
Proceedings of the Fourth U.S. National Conference on Earthquake Engineering, Palm Springs, May 1990, Vol. 3, pp. 273-282.

Objective

The objective of the research was to investigate the effectiveness of ferrocement coatings for enhancing in-plane seismic resistance of unreinforced masonry walls.

Rehabilitation Procedure

A mortar coating is parged on the surface of a masonry wall. A wire mesh is embedded into the mortar. In addition to surface adhesion, bolts are used to secure the coating to the masonry.

Research Approach

The testing program was designed to investigate the in-plane behavior of masonry walls strengthened with ferrocement coatings. In particular, the research examined ultimate strength, ductility requirements, energy dissipation, and strength/stiffness degradation of masonry walls with and without coatings.

Test walls consisted of two-wythe brick walls which were 6'-8" long. Test walls were constructed using reclaimed bricks.

Half of the test walls were coated with a one-half inch thick layer of ferrocement, which was applied to each side of a wall. Each ferrocement layer consisted of two layers of No. 19 gage wire mesh with a one-half inch grid embedded in a mortar coating. One-quarter inch diameter bolts were placed through the wall thickness to help anchor the coatings to each wall face. Spacing of the bolts was nominally 12 inches.

The effect of the retrofit procedure was examined by subjecting a pair of identical test walls to static, lateral forces, and comparing their hysteresis behavior with that for a pair of control test walls which were not rehabilitated. In addition to the static tests, an identical pair of retrofitted test walls were subjected to simulated earthquake motions on a shaking table to compare traits of dynamic response with that for a second pair of non-retrofitted walls which were also tested on a shaking table.

Summary and Significant Findings

Inelastic action of the uncoated piers when tested statically was a result of flexural cracking in addition to sliding and rocking movements. First cracking was observed during the second cycle of loading (Figure 1b). For the coated piers, one specimen failed in flexure while the other failed due to a collapse of the loading device. For the coated specimen limited by flexure (Figure 1a) a horizontal crack developed along the wall base, but no rigid body motion about the base was observed.

The results of uncoated and coated piers tested on the shaking table were almost identical to the results from the cyclic loading tests. The uncoated piers cracked in flexure which was followed by sliding and rocking. For the coated piers, the ferrocement was able to prevent early splitting of the masonry and to prevent development of internal cracks. Except for cracks at the two bottom ends of a pier, no significant cracks were found in the masonry or the ferrocement coating.

Ultimate strength and stiffness are plotted versus lateral deflection in Figures 1c and 1d respectively. Comparisons are made for plain and coated walls. The static strength and stiffness of the plain walls were increased by 250% with retrofitting.

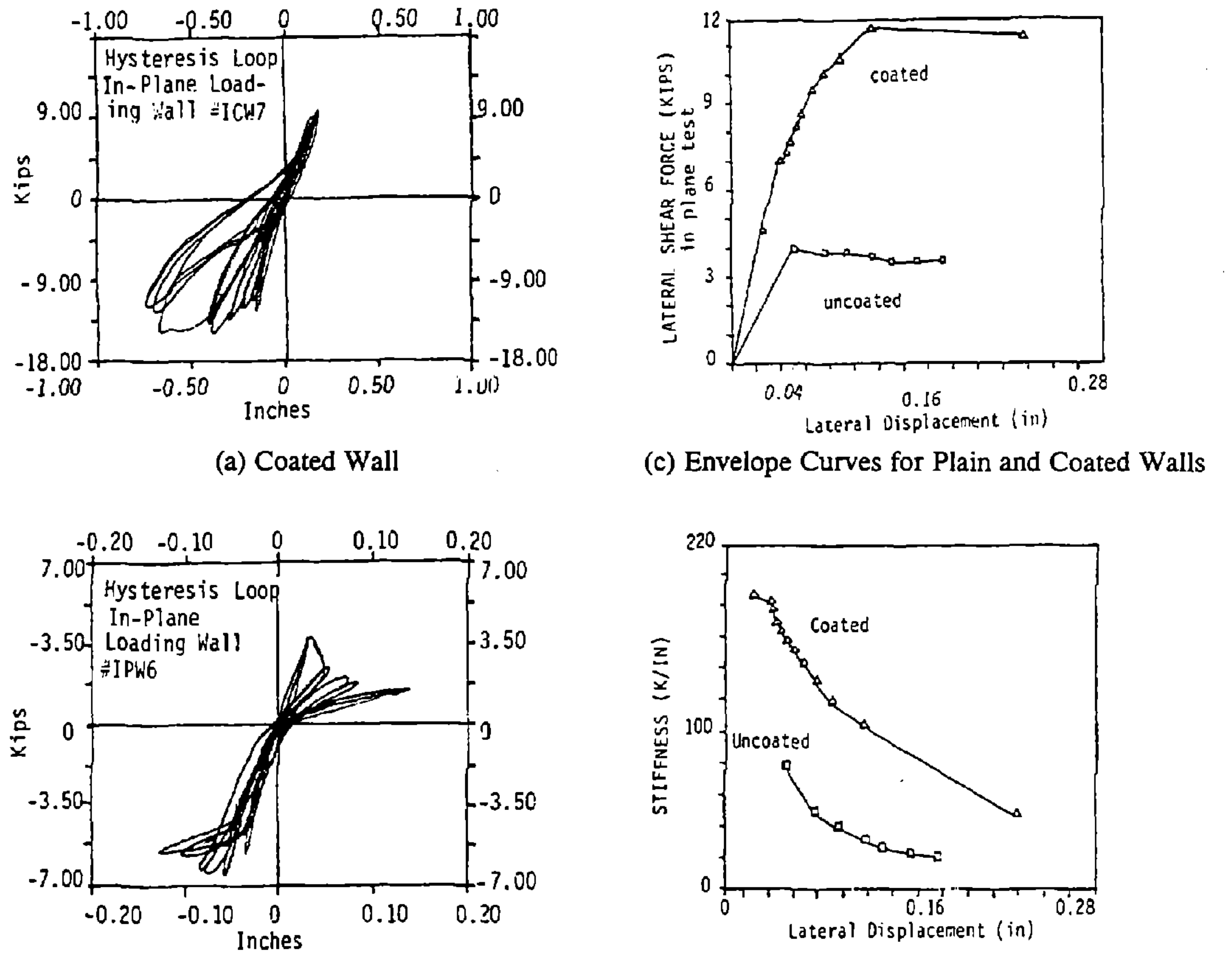
The retrofit procedure increased the energy dissipation capacity by 300% (Figure 2a). The damping factor was higher for the uncoated wall. (Figure 2b). The natural frequency of a coated wall was 1.5 times that of an uncoated wall (Figure 2c).

Related References

Prawel, S.P., and A.M. Reinhorn, "Seismic Retrofit of Structural Masonry Using a Ferrocement Overlay," *Proceedings of the Third North American Masonry Conference*, University of Texas at Arlington, June 1985, pp. 59-1 to 59-19.

S.P. Prawel, A.M. Reinhorn and S.A. Qazi, "Upgrading the Seismic Resistance of Unreinforced Brick Masonry Using Ferrocement Coatings," *Proceedings of the Eighth International Brick/Block Masonry Conference*, Dublin, Ireland, September, 1988, pp. 785-791.

Lee, H. H., and S.P. Prawel, "The Seismic Renovation and Repair Potential of Ferrocement Coatings Applied to Old Brick Masonry Walls," *Proceedings of the Sixth Canadian Conference on Earthquake Engineering*, June 1991, pp. 663-670.



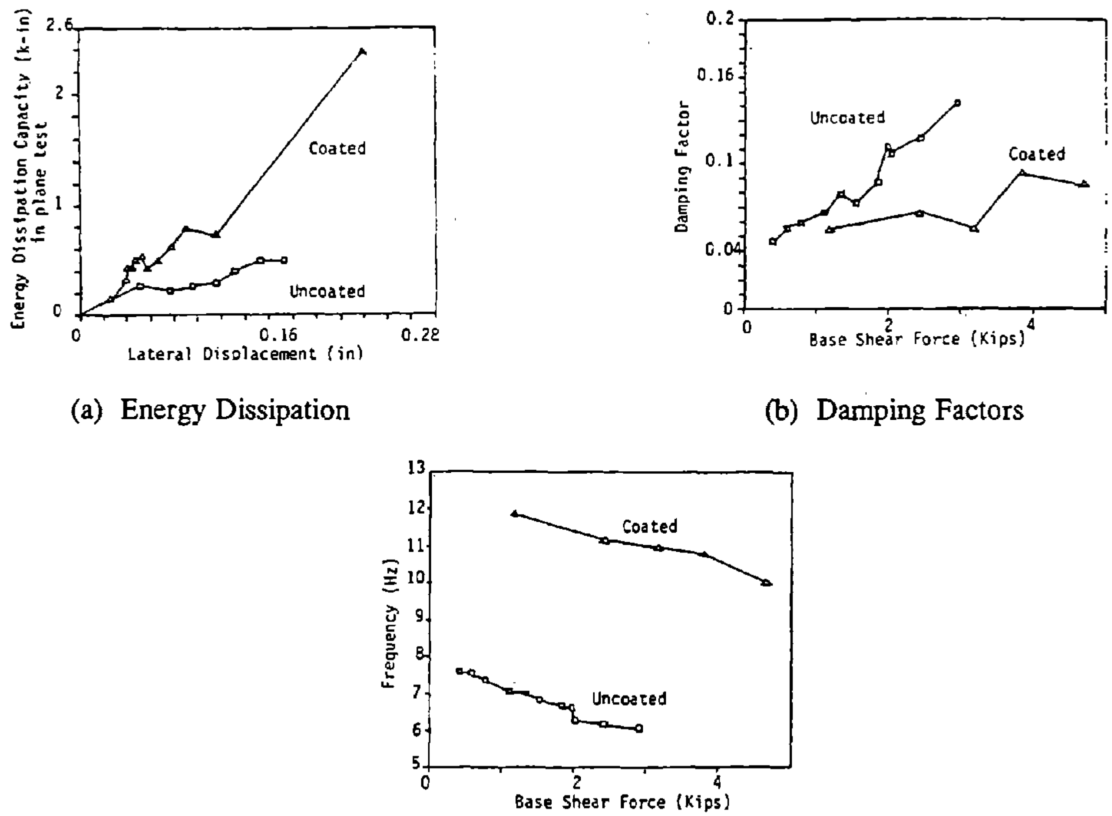
(a) Coated Wall

(c) Envelope Curves for Plain and Coated Walls

(b) Plain Wall

(d) Stiffness of Plain and Coated Walls

Figure 1: Measured In-Plane Behavior of Plain and Coated Walls



(a) Energy Dissipation

(b) Damping Factors

(c) Frequencies

Figure 2: Energy Dissipation, Damping and Frequencies for Plan and Coated Walls

SC11 Seismic Retrofit of Structural Masonry Using a Ferrocement Overlay

S.P. Prawel and A.M. Reinhorn

Proceedings of the Third North American Masonry Conference, University of Texas at Arlington, June 1985, pp. 59-1 to 59-19.

Objective

The objective of this research was to investigate the use of ferrocement coatings for the in-plane rehabilitation of unreinforced masonry walls.

Rehabilitation Procedure

Ferrocement is an orthotropic composite material having a high strength cement mortar matrix and reinforced with layers of fine steel wires in the form of a mesh.

Wire meshes are placed and secured to the wall by means of tie wires passing through the wall at a spacing of approximately 8 in. Spacers are used between the meshes to control positioning. A mortar is then passed between the meshes, aided by a surface vibrator, and allowed to cure.

Research Approach

The testing program included two uncoated brick masonry test panels (B1-1 & B1-2) and five coated test panels (SZ1 to SZ5), each having a different spacing of reinforcing meshes. Each masonry panel was tested in a diagonal split test (Figure 1) to investigate in-plane shear forces.

The test panels were 25.5 in. square, and constructed of stack bond brick masonry, 8 in. thick. The coated specimens were strengthened with an ½ in. layer of ferrocement with various amounts of galvanized welded wire fabric. The wire spacing in the mesh was varied from 1/8 in. to 2 in. with the ferrocement layer being varied to maintain a constant reinforcement volume ratio.

Summary and Significant Findings

The load deflection curves for all specimens are shown in Figure 2. The bare masonry specimens (B1-1 & B1-2) behaved with a distinctly nonlinear force-deflection relation over almost the entire load range while the coated specimens maintained an almost proportional pattern up to yielding. The difference in behavior for the uncoated specimens was due to differing construction procedures.

As shown in Figure 2, each of the coated specimens developed a maximum strength which was approximately twice that of the bare masonry test panels. The measured strength was essentially independent of the reinforcing spacing.

The surface coating improved not only the ultimate deformation range but also extended the elastic range.

Stiffness degradation was reduced with the coating (Figure 3). The coated specimens behaved in nearly an ideal plastic manner whereas the stiffness of the non-retrofitted test panels reduced rapidly.

The bond anchors between the masonry and the coatings had a dominant effect on the enhancement of strength and ductility. The strength, ductility and secant stiffness of the coated walls were nearly twice those for the uncoated walls. Composite strength did not appear to depend on mesh size.

Related References

S.P Prawel, A.M. Reinhorn and S.K. Kunnath, "Seismic Strengthening of Structural Masonry Walls with External Coatings," *Proceedings of the Third U.S National Conference on Earthquake Engineering*, Charleston, SC, August 1986, Vol. 2, pp. 1323-1334.

S.P. Prawel, A.M. Reinhorn and S.A. Qazi, "Upgrading the Seismic Resistance of Unreinforced Brick Masonry Using Ferrocement Coatings," *Proceedings of the Eighth International Brick/Block Masonry Conference*, Dublin, Ireland, September 1988, pp. 785-791.

Prawel, S.P. and H.H. Lee, "The Performance of Upgraded Brick Masonry Piers Subject to In-Plane Motion," *Proceedings of the Fourth U.S. National Conference on Earthquake Engineering*, Palm Springs, May 1990, Vol. 3, pp. 273-282.

Lee, H.H., and S.P. Prawel, "The Seismic Renovation and Repair Potential of Ferrocement Coatings Applied to Old Brick Masonry Walls," *Proceedings of the Sixth Canadian Conference on Earthquake Engineering*, June 1991, pp. 663-670.

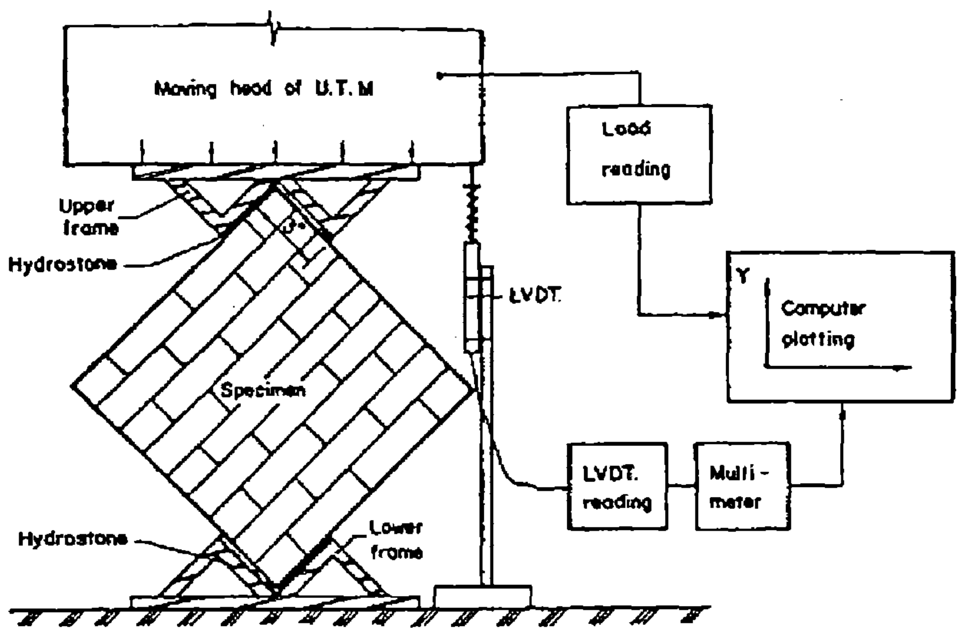


Figure 1: Test Set-up

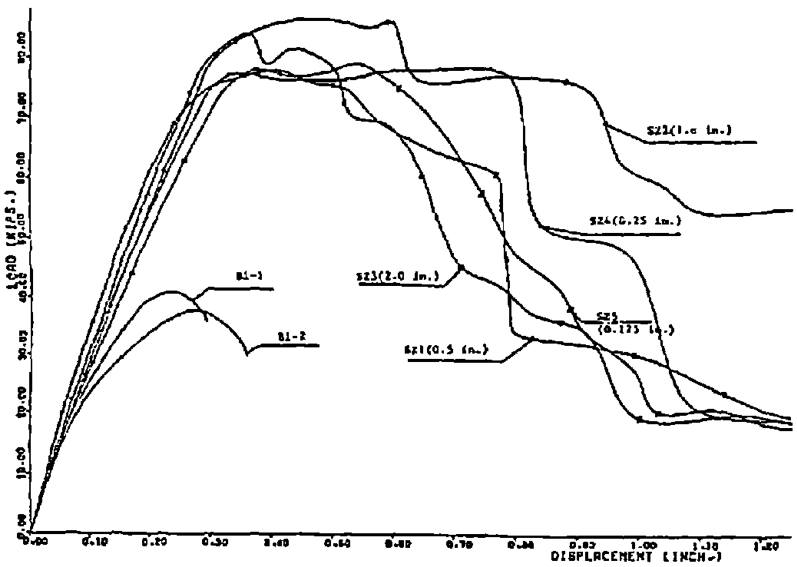


Figure 2: Measured Load Deflection Curves

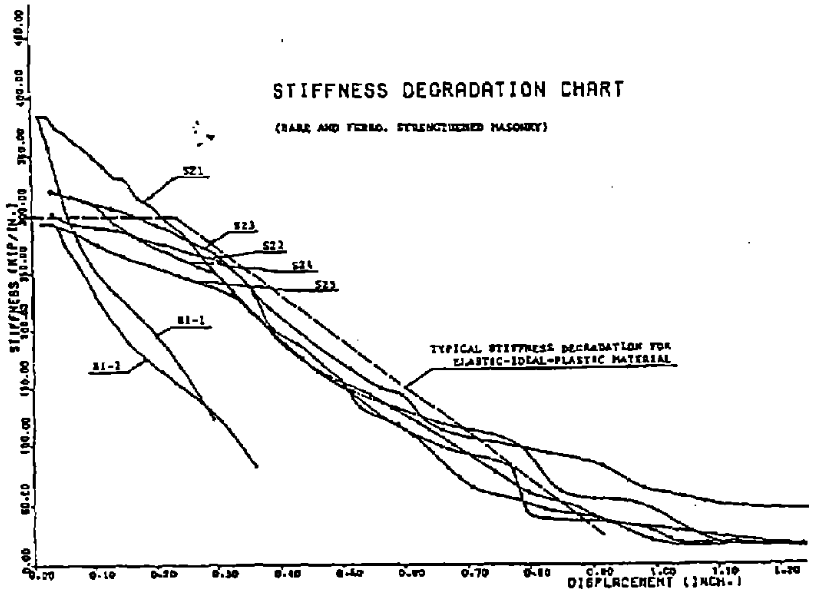


Figure 3: Lateral Stiffness vs. Deflection of Test Panels

SC12 Upgrading the Seismic Resistance of Unreinforced Brick Masonry Using Ferrocement Coatings

S.P. Prawel, A.M. Reinhorn and S.A. Qazi

Proceedings of the Eighth International Brick/Block Masonry Conference, Dublin, Ireland, September, 1988, pp. 785-791

Objective

The objective of this research was to investigate the use of ferrocement coatings for the in-plane rehabilitation of unreinforced masonry walls. In particular, development of bond between brick and a cement plaster coating was examined.

Rehabilitation Procedure

Ferrocement is an orthotropic composite material having a high strength cement mortar matrix and reinforced with layers of fine steel wires in the form of a mesh.

Wire meshes are placed and secured to the wall by means of tie wires passing through the wall at a spacing of approximately 8 in. Spacers are used between the meshes to control positioning. A mortar is then passed between the meshes, aided by a surface vibrator, and allowed to cure.

Research Approach

The first series of tests was done to determine connector size and spacing requirements. Diagonal splitting tests were run on ten square plates of coating material (42 in. x ½ in. thick) which were joined with steel connectors. The size and spacing of the connectors was varied until the ultimate strength of the coating material could be achieved.

The second series of tests was done to investigate bond mechanisms between the coating and a brick masonry surface. Diagonal splitting tests were run on 13 square panels of masonry (42 in. x 8 in. thick). Reclaimed bricks (type L in Table 1 with a strength of 1.04 ksi) and new type NW bricks (type B in Table 1 with a strength of 1.32 ksi) were used with a type N Portland cement mortar (1:1:3.6).

Summary and Significant Findings

Based on the tests, 1/4 in. bolts spaced at about 12 in., and an extra bolt in each loaded corner, were appropriate to insure maximum participation of the coating material.

Force-deflection curves for specimens constructed with new brick (B type) and old brick (L type) specimens are shown in Figures 1 and 2.

The experimental results indicated that the strength of coated masonry panels in all cases was almost two to three times that of the bare masonry panels.

Attaching thin sheets of ferrocement to a brick wall is a viable retrofitting method in terms of strength and ductility.

Related References

Prawel, S.P., and A.M. Reinhorn, "Seismic Retrofit of Structural Masonry Using a Ferrocement Overlay," *Proceedings of the Third North American Masonry Conference*, University of Texas at Arlington, June 1985, pp. 59-1 to 59-19.

Prawel, S.P. and H.H. Lee, "The Performance of Upgraded Brick Masonry Piers Subject to In-Plane Motion," *Proceedings of the Fourth U.S. National Conference on Earthquake Engineering*, Palm Springs, May 1990, Vol. 3, pp. 273-282.

Lee, H.H., and S.P. Prawel, "The Seismic Renovation and Repair Potential of Ferrocement Coatings Applied to Old Brick Masonry Walls," *Proceedings of the Sixth Canadian Conference on Earthquake Engineering*, June 1991, pp. 663-670.

TABLE I
Description of Specimens

	Masonry Prop.		Prop. of 1/2" thick ferrocement Note: 2 layers of mesh per sheet				Reinforcement strength	
Name	Mortar strength ksi	Masonry strength ksi	Mesh size in wire gage	Mortar Prop.	Mortar strength ksi	Vol.* Ratio	Yield ksi	Ult. ksi
B10	1.014	2.61	no coating					
B11	0.795	2.42	no coating					
B5	0.939	2.75	no coating					
B6	1.034	2.96	.5 x .5 # 19	1C:2S W/C-0.05	3.72	0.0219	92	138
B2	1.233	3.54.	.5 x .5 # 19	1C:2S W/C=0.48	5.25	0.0289	92	138
B3	1.074	3.82	1/4in x 1/4in # 23	1C:2S W/C=0.46	3.31	0.0363	88	133
B4	1.034	3.31	chicken wire	1C:2S W/C=0.48	3.28	0.0098		
B7	1.321	2.15	1 in x 1 in # 16	1C:2S W/C=0.48	3.62	0.0198	74	110
B9	1.014	2.43	2in x 2in # 14	1C:2S W/C-0.48	3.32	0.0175	55	82
L5	3.183	2.50	no coating					
L4	2.665	2.75	1/4in x 1/4in # 23	1C:2S W/C=0.60	3.34	0.0363	88	133
L3	2.546	3.12	1 in x 1 in # 23	1C:2S W/C=0.60	4.17	0	74	110
L1	1.854	2.82	.5 x .5 # 19	1C:2S W/C=0.60	3.96	0.0181	92	138

* Vol. fract. based on measured coating thickness.

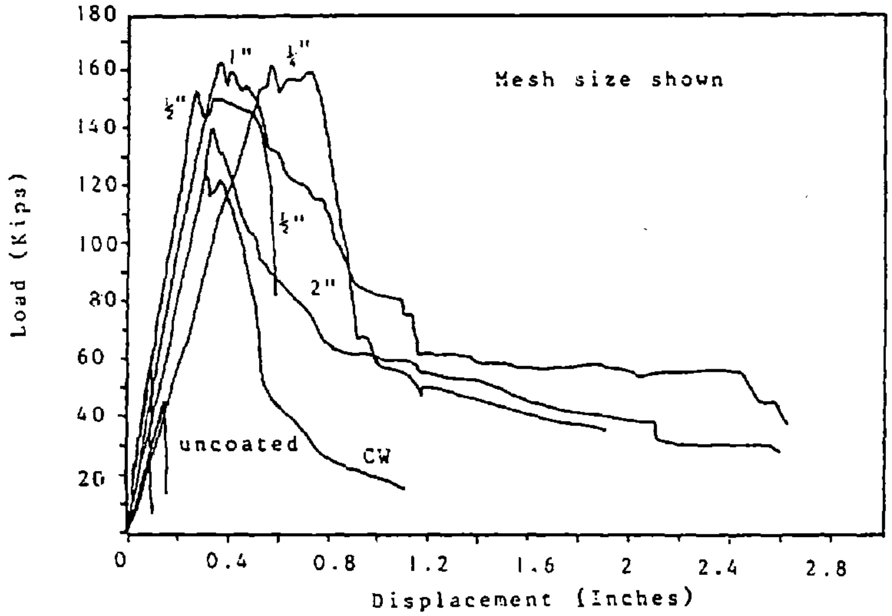


Figure 1: Force-Deflection Relations for New Brick Panels (B type)

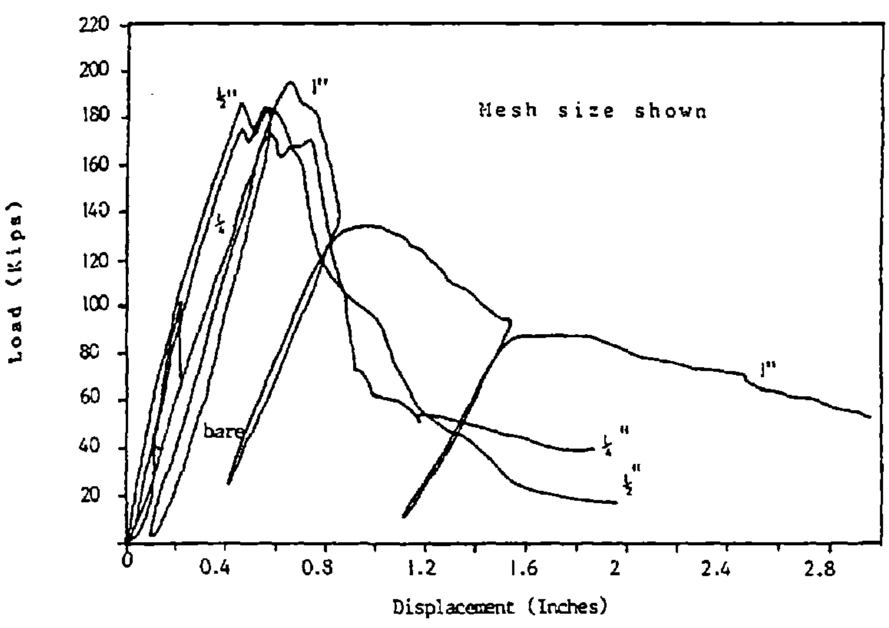


Figure 2: Force-Deflection Relations for Old Brick Panels (L type)

SC13 Evaluation of TYFO-W Fiber Wrap System for Out-of-Plane Strengthening of Masonry Walls

A.M. Reinhorn and A. Madan
Department of Civil Engineering, State University of New York at Buffalo, Report No. AMR 95-0001, March, 1995.

Objective

The objective of the research was to evaluate the effectiveness of a proprietary fiber-wrap system for improving out-of-plane strength and deformation capacity of unreinforced brick walls.

Rehabilitation Procedure

A high strength fiber composite material is attached to the surface of an unreinforced masonry wall with epoxy. The wall is wrapped with the fiber material across the entire wall surface, or is placed in bands. Multiple layers of the epoxy-fiber material are applied, followed by application of a protective coating of epoxy.

Research Approach

Two brick masonry wall specimens were subjected to static, out-of-plane lateral forces. The first specimen was not coated while the second was coated with two layers of fiber reinforcement.

Each face of the second specimen was coated with a different method. The south face was reinforced with bands of fiber fabric which were epoxied bonded to the wall surface. The material was a unidirectional fabric of E-glass rovings woven with Kevlar yarns in one direction. Two 6-inch wide vertical bands were bonded to the wall in a symmetrical pattern at a spacing of 48 inches. The north face was coated with a continuous web fiber fabric overlay using a commercial adhesive.

The epoxy used to bond the fabric to the wall surface was a two-part, ambient curing resin which had good weathering, adhesion and shear strength properties.

Each test wall was 72 inches wide by 70 inches high, and was laid in double wythe running bond. Wall specimens were subjected to a two-point loading using a servo-hydraulic actuator to simulate face loading on a vertical strip of wall (Figure 1). Loads were reversed and repeated, and continued until fracture of the fabric occurred.

Summary and Significant Findings

Measured force-deflection relations for the continuous fiber wrapped wall is shown in Figure 2. Each fabric orientation increased out-of-plane strength from 3.7 to 4.2 times the strength of the plain masonry wall.

Both banded and continuous fabric systems improved the cracking performance of an unreinforced masonry brick wall. The fabric controlled masonry cracking. Uncertainty in cracking strength estimates was reduced because of the controlling influence of the fabric.

The adhered fabric enhanced both the strength and deformation capacities substantially. Hysteretic energy dissipation was increased with the fabric, and strength deterioration was reduced.

Failure of the coating can occur as a result of debonding of the fabric from the wall surface or fracture of the coating.

The continuous fiber wrap provided a greater degree of bi-directional confinement to the masonry than the banded wrap, and thus a larger strength.

Related References

Reinhorn and M.S. Madan, "Evaluation of TYFO-W Fiber Wrap System for In-Plane Strengthening of Masonry Walls," Department of Civil Engineering, State University of New York at Buffalo, Report No. AMR 95-0002, August 1995.

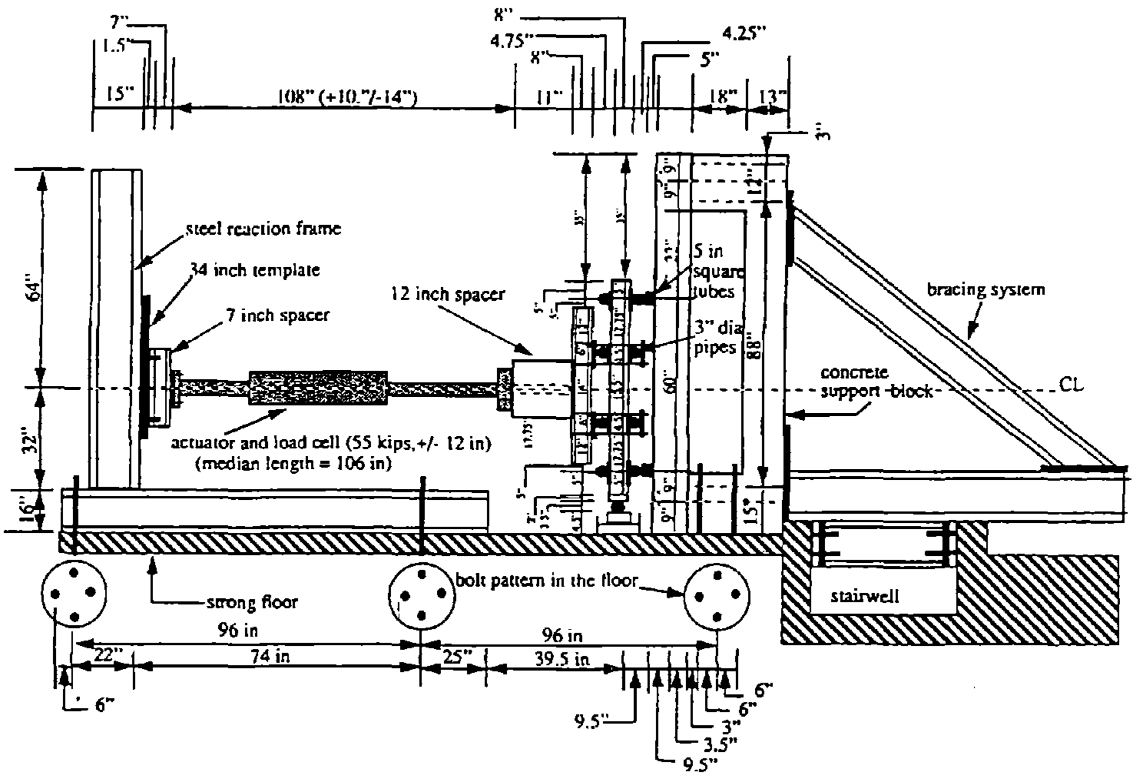


Figure 1: Test Set-up

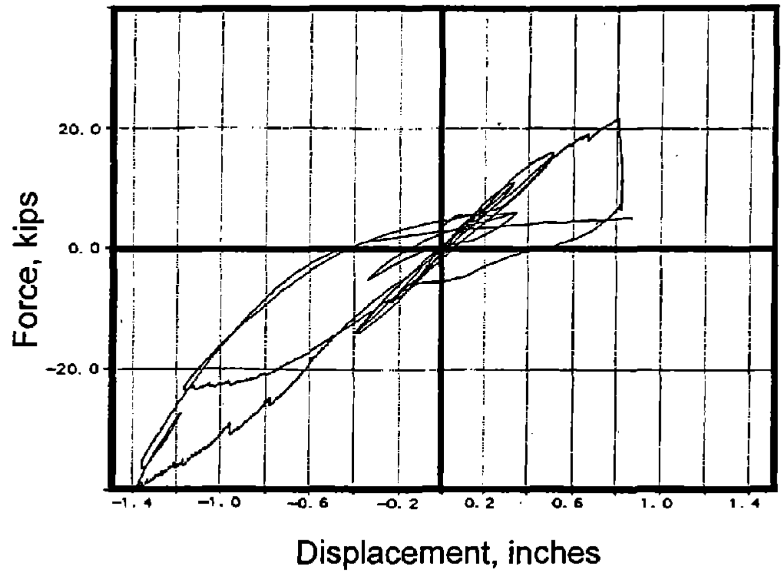


Figure 2: Measured Load Deflection Curves

SC14 Evaluation of TYFO-W Fiber Wrap System for In-Plane Strengthening of Masonry Walls

A.M. Reinhorn and A. Madan
Department of Civil Engineering, State University of New York at Buffalo,
Report No. AMR 95-0002, August 1995.

Objective

The objective of the research was to evaluate the effectiveness of a proprietary fiber-wrap system for improving in-plane strength and deformation capacity of unreinforced brick walls.

Rehabilitation Procedure

A high strength fiber composite material is attached to the surface of an unreinforced masonry wall with epoxy. The wall is wrapped with the fiber material across the entire wall surface, or is placed in bands. Multiple layers of the epoxy-fiber material are applied, followed by application of a protective coating of epoxy.

Research Approach

Two brick masonry wall specimens were subjected to static, in-plane lateral forces. The first specimen was not coated while the second was coated with two layers of fiber reinforcement.

Each face of the second specimen was coated with a different method. The west face was reinforced with bands of fiber fabric which was epoxied bonded to the wall surface. The material was a unidirectional fabric of E-glass rovings woven with Kevlar yarns in one direction. Two 6-inch wide vertical bands were bonded to the wall in a symmetrical pattern at a spacing of 48 inches. The east face was coated with a continuous web fiber fabric overlay using a commercial adhesive.

The epoxy used to bond the fabric to the wall surface was a two-part, ambient curing resin which had good weathering, adhesion and shear strength properties.

Each test wall was 72 inches wide by 70 inches high, and was laid in double wythe running bond. A constant vertical axial compressive force was applied while lateral forces were applied in a reversed cyclic manner using the test setup shown in Figure 1.

Summary and Significant Findings

Damage to the plain wall was a result of diagonal cracking which caused a sudden and sharp reduction in strength. Damage to the rehabilitated test wall was a result of debonding of the fabric on the west face which was followed by debonding of the fiber bands on the east face. Debonding of both fiber reinforcements was concentrated along the diagonals where tensile stresses were high. Lateral strength of the rehabilitated wall dropped when diagonal tension cracks developed.

Measured force-deflection relations for the plain and fiber wrapped walls are compared in Figure 2. The retrofit procedure enhanced both in-plane strength and inelastic deflection capacity. The strength of the wrapped wall was 120% more than the plain wall, and the maximum deflection was 150% larger for the wrapped wall than the plain wall. The hysteretic energy dissipation was also enhanced with the retrofit procedure.

The fiber wrapping prevented the falling of debris after the wall failed.

Related References

A.M. Reinhorn and M.S. Madan, "Evaluation of TYFO-W Fiber Wrap System for Out-of-Plane Strengthening of Masonry Walls," Department of Civil Engineering, State University of New York at Buffalo, Report No. AMR 95-0001, March 1995.

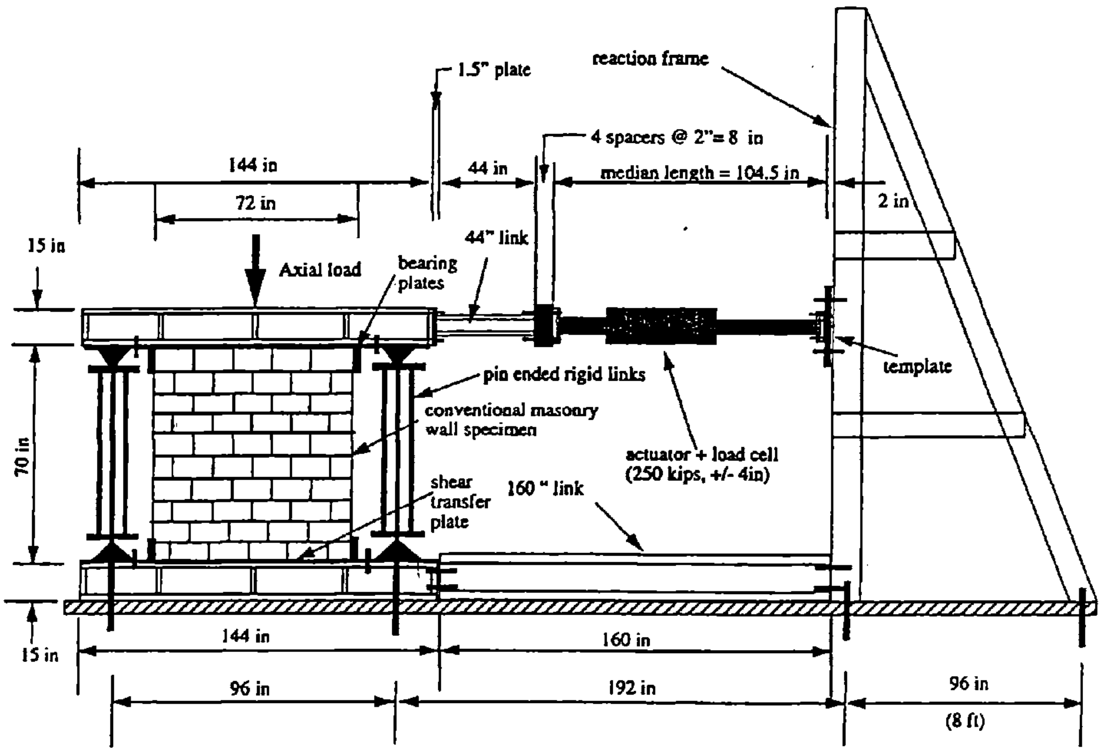


Figure 1: Test Set-up

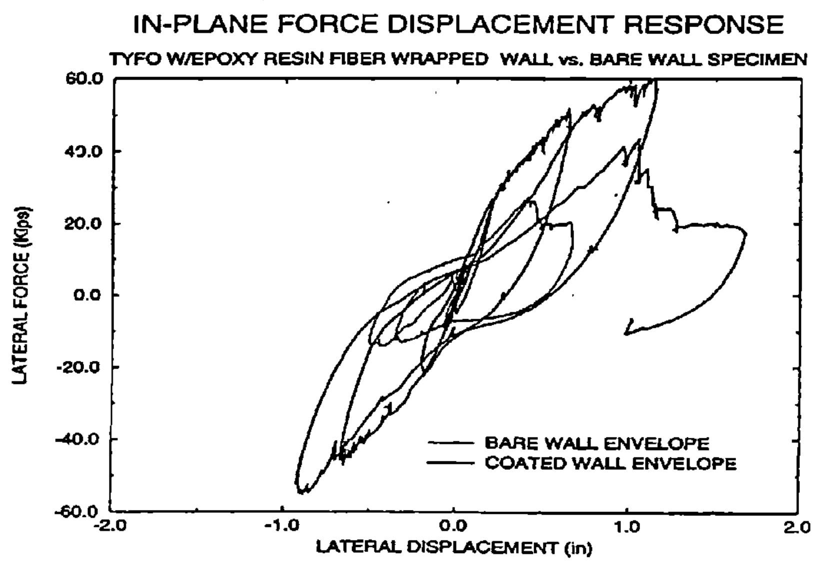


Figure 2: Measured Load Deflection Curves

SC15 Earthquake Resistance of Masonry Structures Strengthened with Fiber Composites

G. Schwegler and P. Kelterborn

Proceedings of the Eleventh World Conference on Earthquake Engineering, Acapulco, June 1996, Elsevier Science, Ltd., Paper No. 1460.

Objective

The purpose of this paper is to illustrate the use of carbon fiber reinforced plastic sheets for the in-plane rehabilitation of unreinforced masonry walls. Case studies are presented where two 6-story residential buildings were rehabilitated in Zurich using a procedure that was researched with numerous large-scale tests of strengthened masonry walls.

Rehabilitation Procedure

Carbon fiber sheets are glued to the surface of a clay-unit masonry wall and anchored into concrete slabs above and below the wall panel.

The CFRP sheets are a combination of unidirectional high strength carbonfibers with an epoxy resin matrix. CFRP sheets are better than steel sheets with respect to corrosion, fatigue behavior and strength.

The sheets are applied diagonally on the surface of a masonry wall (Figure 1). Before application, the wall surface is sanded smooth and holes are filled with epoxy mortar.

Tests have shown that sheets may be applied to only one side of a wall.

Research Approach

Research is summarized in the related references.

Summary and Significant Findings

Application of CFRP sheets can significantly increase lateral strength and ductility without increasing wall thickness appreciably.

Related References

Schwegler, G., "Verstarken von Mauerwerk mit Hochleistungsfaserverbundwerkstoffen" dissertation, Eidfenossische Materialprufungs und Forschungsanstalt Dubendorf, EMPA-Bericht Nr. 229.

Schwegler G., "Masonry Construction Strengthened With Fiber Composites in Seismically Endangered Zones," *Proceedings of the Tenth European Conference on Earthquake Engineering, August-September 1994,* Vienna, A.A. Balkema, Rotterdam, 1995.

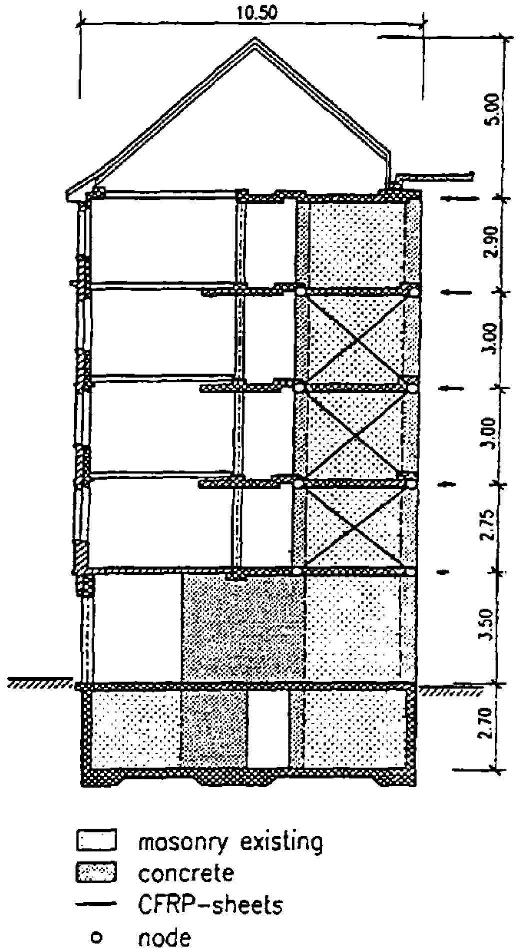


Figure 1: Location of Sheets

(dimensions in meters)

SC16 Research on Strengthening Methods for Earthquake Damaged Masonry

M. Simonici

Proceedings, Joint USA/ITALY Workshop on Seismic Repair and Retrofit of Existing Buildings, May 7-11, 1984, Rome, Italy, pp. 242-251.

Objective

The objective of the research was to investigate the effectiveness of using a welded wire mesh embedded in a surface coating to enhance in-plane strength of unreinforced clay-unit masonry walls.

Rehabilitation Procedure

Welded wire mesh is bonded to each face of an unreinforced masonry wall with a surface coating of mortar. The coating is 3 to 4 cm thick and consists of cement to sand in a 1 to 3 ratio by volume.

Two kinds of reinforcement methods are used. The first method consists of external reinforcement with welded wire mesh. The second method consists of discontinuous horizontal and vertical reinforcement.

Research Approach

Unstrengthened masonry test walls were subjected to a lateral in-plane force until significant cracking occurred. The test walls were then strengthened by applying the surface coatings reinforced with wire mesh, and retested.

In a second test series, unstrengthened walls were again tested until significant damage occurred, then strengthened with external reinforcing bars, and retested. Primary reinforcing bars were anchored in the foundation and in the floor slabs (Figure 1). Other reinforcement was evenly distributed across the wall surface and not anchored.

Summary and Significant Findings

Behavior of the unstrengthened wall specimen (ZS in Figure 2) was generally limited by crushing of masonry at the wall toe, and for certain specimens, by diagonal tension cracking. When the wall was strengthened with welded wire mesh, the mortar coating delaminated because of poor adhesion to the surface of the masonry. As a result, the retrofit scheme did not increase the lateral in-plane strength or stiffness of the wall (ZC in Figure 2).

For the second series of tests, the strength of the unstrengthened wall (MS) was limited by masonry compressive stress at the wall toe. Failure of the strengthened test wall (MC) was a result of yielding of reinforcement. Delaminations of the coating were not observed despite the lack of anchorage. The retrofit method resulted in significant enhancements in both strength and stiffness (Figure 3).

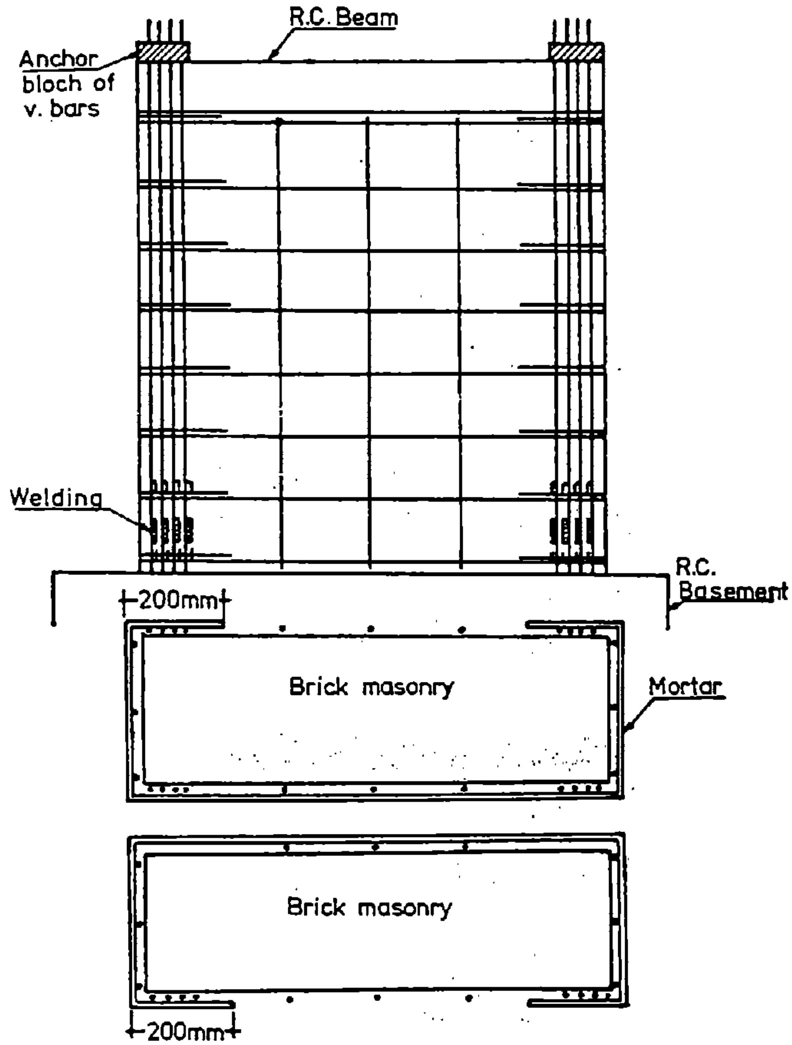


Figure 1 Description of Test Wall Strengthened with External Reinforcement (MC)

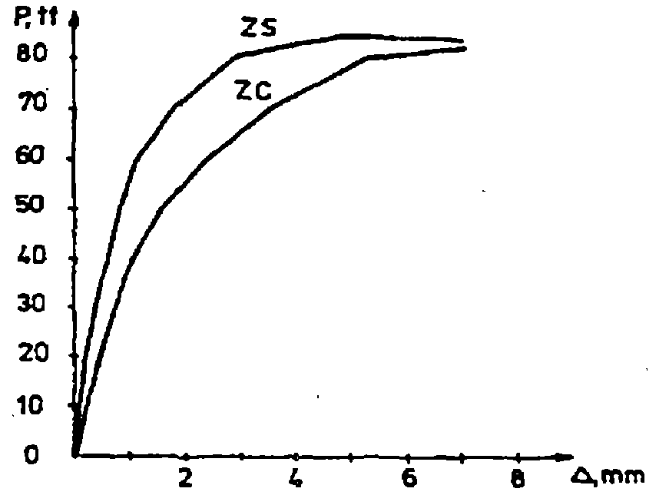


Figure 2 Measured Load-Deflection Relations (ZS and ZC Specimens)

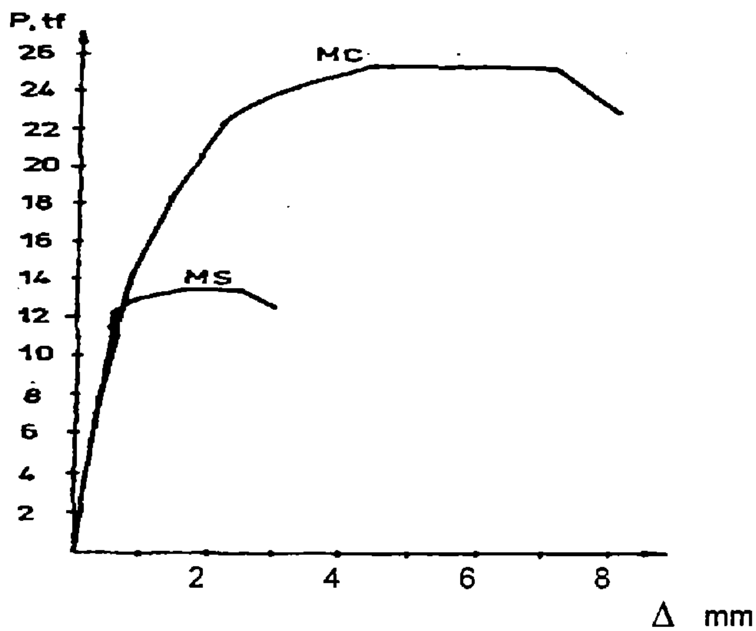


Figure 3 Measured Load-Deflection Relations (MC and MS Specimens)

SC17 Cyclic Loading on Externally Reinforced Masonry Walls

W.K. Tso, E. Pollner and A.C. Heidebrecht

Proceedings of the Fifth World Conference on Earthquake Engineering, Rome, Italy, 1974, pp. 1177- 1186.

Objective

The objective of the research was to investigate the effectiveness of external reinforcement on seismic behavior of unreinforced concrete masonry walls.

Rehabilitation Procedure

Expanded metal sheets are bonded to one or both sides of an unreinforced concrete block wall with a one-inch thick mortar surface coating to increase in-plane lateral strength and stiffness.

Research Approach

Each half-scale test wall (Figure 1) was 6'-8" long by 4'-8" high and constructed using standard 6" hollow concrete units. Concrete blocks had an average compressive strength of 1200 psi. The mortar (1:3 ratio of cement to sand) reached an average cube strength of 1000 psi. Each test wall was encased in a steel frame. External reinforcement is not shown in Figure 1.

External reinforcement consisted of 1.5-in. wide by 16 gauge expanded metal sheets (0.05 in. thick). Mortar for the surface coating had an average strength of 2800 psi. Seven 1/4 inch diameter bolts with 2" x 2" x 1/8" plates welded at the ends were used to increase anchorage.

The test program included: (a) walls with external reinforcement on both faces, (b) walls with reinforcement on one face only, and (c) damaged walls repaired with external reinforcement.

Summary and Significant Findings

1. *Wall Reinforced on Both Sides (Wall No. 2)*: A maximum lateral load of 100 kips was reached in the second cycle (Figure 2). Stiffness degradation for repeated cycles is shown in Figure 3. Energy absorption is shown in Figure 4.
2. *Wall with Reinforcement on One Side (Wall No. 4)*: Seven anchor bolts were used to increase the bonding of the reinforcing layer to the masonry wall. The load-deflection curves (Figure 5) and stiffness degradation curves (Figure 6) were similar to those of the wall reinforced on both sides (Wall No. 2). With reinforcement on only one side of a wall, there was some difference in energy absorption capacity (Figure 7) and damage patterns.
3. *Damaged Wall with Reinforcement on Both Sides (Wall No. 3):* The test wall was loaded until cracks developed along the joints. Then, two reinforcing layers were applied to the damaged wall and the wall was retested. Load deflection curves for the first four cycles of unreinforced wall are shown in Figure 8. The maximum load in all cycles remained below 33 kips. The load deflection curves for the first four cycles of the repaired wall are shown in Figure 9. The stiffness of the repaired wall was much better than that of the undamaged wall. The stiffness degradation and the energy absorption capacity for the unreinforced and the

repaired wall are shown in Figures 11 and 12.

Walls with external reinforcing performed better than unreinforced walls when subjected to cyclic loading. Walls reinforced on both faces performed better than walls reinforced on one side only.

Damaged walls repaired by external reinforcing behaved similarly to undamaged reinforced walls.

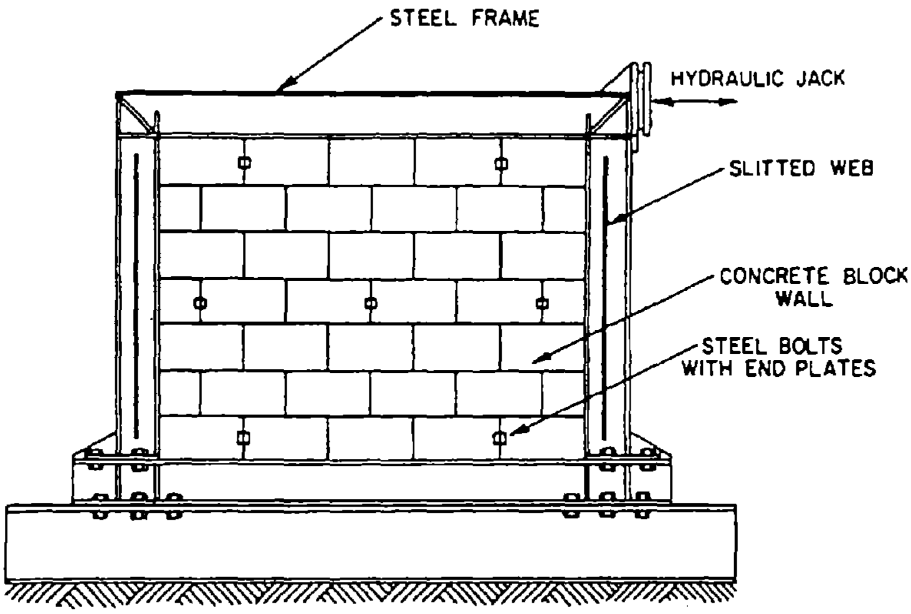


Figure 1: Description of Test Wall

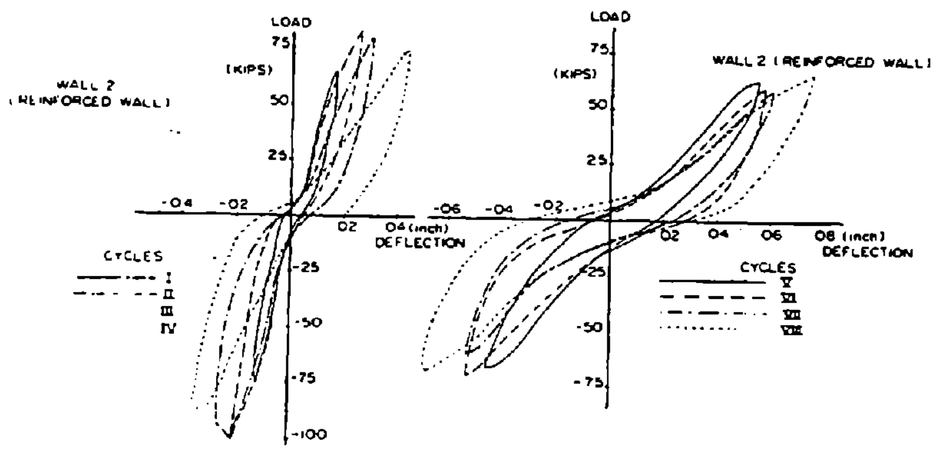


FIG. ? LOAD DEFLECTION CURVES FOR REINFORCED WALL

Figure 2: Load-Deflection Relations for Wall 2

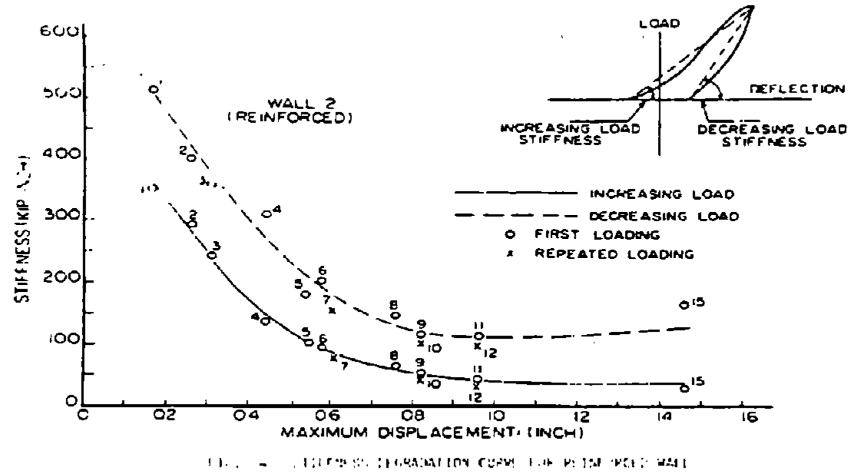


Figure 3: Stiffness Reductions for Wall 2

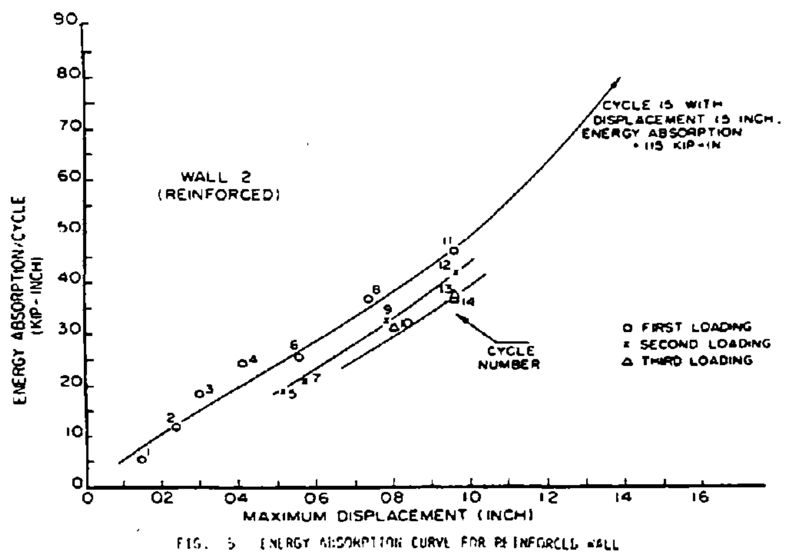


Figure 4: Energy Dissipation for Wall 2

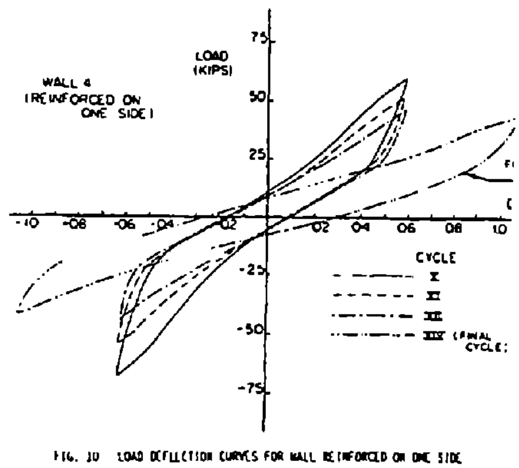


Figure 5: Load-Deflection Relations for Wall 4

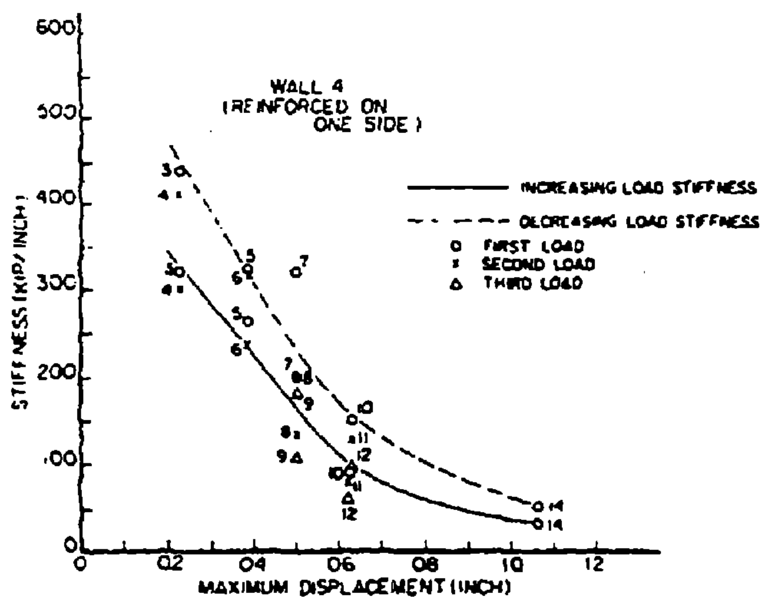


Figure 6: Stiffness Reductions for Wall 4

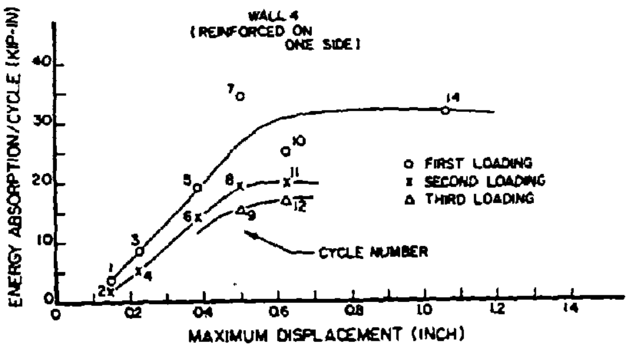


FIG. 9 ENERGY ABSORPTION CURVE FOR WALL REINFORCED

Figure 7: Energy Dissipation for Wall 4

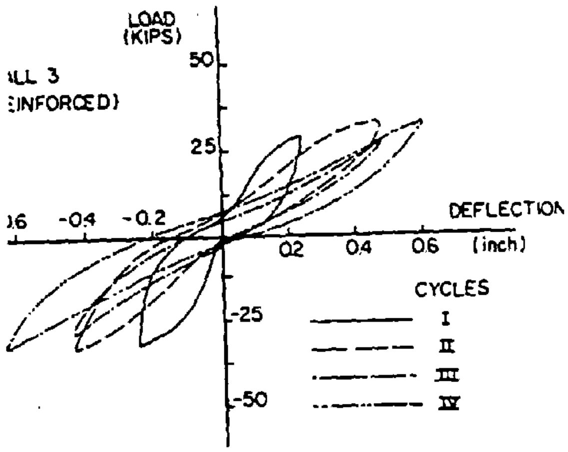


FIG. 11 LOAD DEFLECTION CURVE FOR UNREINFORCED WALL

Figure 8: Force-Deflection Relation for Unstrengthened Wall 3

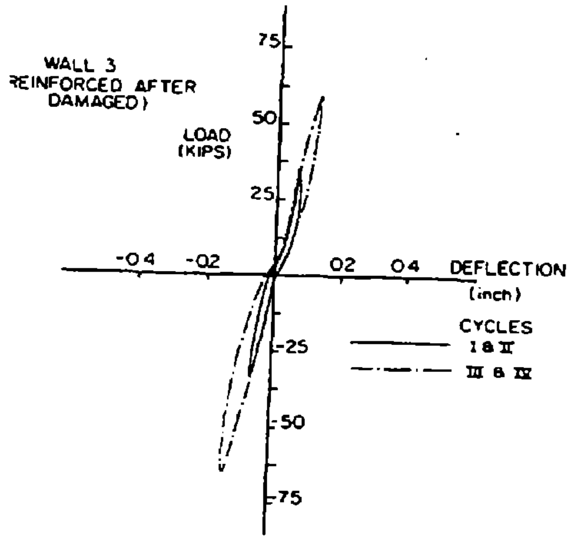


FIG. 12 LOAD DEFLECTION CURVE FOR REPAIRED WALL

Figure 9: Force-Deflection Relation for Repaired Wall 3

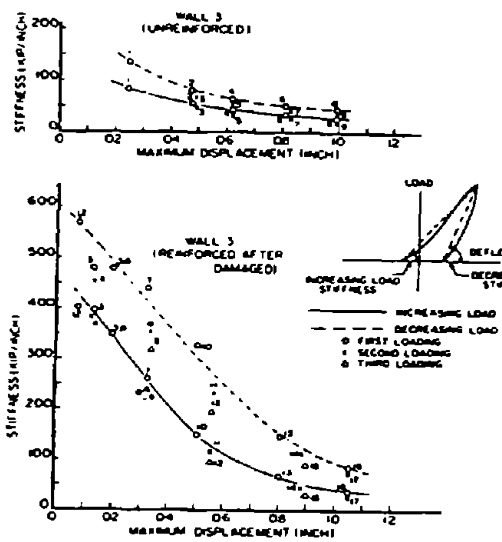


FIG. 13 STIFFNESS DEGRADATION CHARACTERISTICS

Figure 10: Stiffness Reductions for Wall 3

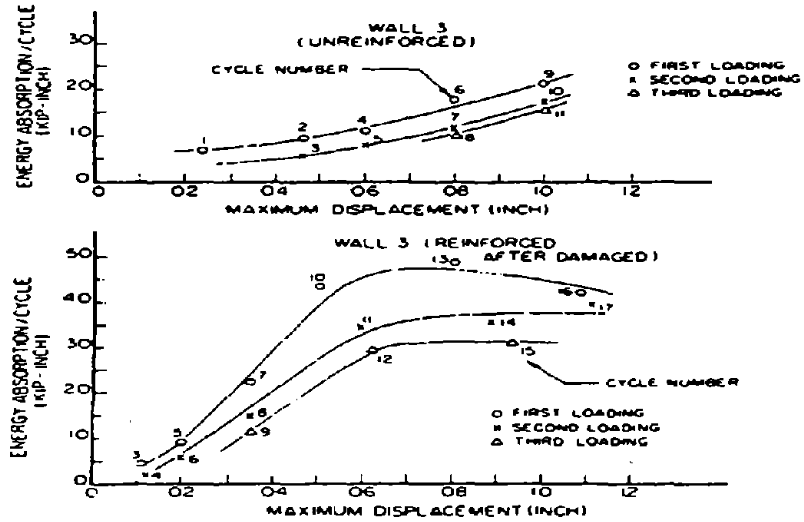


FIG. 14 ENERGY ABSORPTION CURVES

Figure 11: Energy Dissipation for Wall 3

Reinforced or Post-Tensioned Cores

PC1 Center-Core Seismic Hazard Reduction System for URM Buildings

D. C. Breiholz

Proceedings of the Tenth World Conference on Earthquake Engineering, Madrid, July 1992, Vol. 9, pp. 5395-5399.

Objective

The objective of the paper is introduce and illustrate the CenterCore strengthening system.

Rehabilitation Procedure

The CenterCore strengthening system consists of grouting a reinforcing bar within a vertical core drilled through an unreinforced masonry wall.

A dry drilling process is required to remove and collect debris. Reinforcing bars (#6 or #9 bars are typical) are bonded within a core with a polyester sand grout for strong bonding of inner and outer wythes.

A positive and negative air system vacuums the brick dust to a filtered, dust-controlled container for removal from the site. Improved quality control measures for the grout components (sand, polyester resin, and catalyst) have provided a more predictable product, and now viscosity can be controlled without reducing bond strength.

Summary and Significant Findings

High strength values for both in-plane shear and out-of-plane bending were the result of adequate bond capacity between reinforcement and a grouted core.

The in-plane shear capacity was enhanced with the use of polyester grout.

The design strength of CenterCore for out-of-plane flexure is based on a yielding of the steel prior to any crushing of the masonry using a conservative value of masonry compressive strength.

Related Reference

Breiholz, D.C., "CenterCore Strengthening System for Seismic Hazard Reduction of Unreinforced Masonry Bearing Wall Buildings," *Structural Engineering in Natural Hazard Mitigation*, pp. 319-324.

PC2 Strengthening of Masonry Structures with Post-Tensioning

Hans Rudolf Ganz
Proceeding of the Sixth North American Masonry Conference, June 6-9, 1993, Philadelphia, Pennsylvania, pp. 645-655.

Objective

The paper presents three case studies where post-tensioning has been used for seismic strengthening of unreinforced clay-unit masonry buildings. Although no research information is provided, the case studies suggest possible applications for seismic rehabilitation.

Rehabilitation Procedure

Cores are drilled vertically through the height of an unreinforced brick masonry wall. Prestressing tendons are inserted into the cores, grouted in place at their bases, and post tensioned. The method can be used to enhance shear or flexural capacities of masonry walls subjected to either in-plane or out-of-plane forces.

Post-tensioning of existing masonry walls is done to improve strength and deformation capacity with respect to in-plane and out-of-plane lateral forces. The imposed vertical compressive stress increases the flexural cracking strength to resist bending moments as well as shear strength.

Behavior of unreinforced masonry elements (curve 1 in Figure 1) or under-reinforced elements (curve 2) can be improved substantially with prestressing (curve 3). The improved behavior according to curve 3 may be attained with either bonded non-prestressed and/or prestressed reinforcement, or with unbonded prestressed reinforcement. Unbonded prestressed reinforcement will improve shear resistance along bed joints through the increased normal compression without losing effective prestressing forces during loading reversals.

Case Studies

Los Gatos Brick Castle: The 100-year old building located in Los Gatos suffered considerable structural and non-structural cracking during the 1989 Loma Prieta earthquake. The structural system consists of exterior unreinforced clay-unit masonry walls (8 inches thick), timber floors, roof and partitions and a stone foundation. In addition to post-tensioning of the masonry walls, structural repair consisted of grouting of cracks in masonry walls, reconstruction of parapets, the addition of continuous steel chords and anchors along all the floor-wall connections, and the addition of a reinforced concrete beam at the roof level.

A total of 15 monostrand tendons were placed vertically in 200-mm thick unreinforced clay-unit masonry walls. Each tendon was anchored into the stone rubble foundation at the wall base, and into the reinforced concrete tie beam at the roof level. Bending moments resulting from tendon eccentricities were minimized and resisted by cross walls. To prevent long term corrosion, strands were greased and sheathed.

General Post Office, Sydney, Australia: The historic building is more than 100 years old and consists of sandstone walls.

As part of the restoration, a tower was strengthened with four vertical post-tensioning tendons (each consisting of 19 monostrands with a 0.5 inch diameter) which were placed in cores drilled through the 35-meter height of sandstone columns. Large steel bearing plates were used to spread the large tendon loads (2500 kN) to the masonry. The tower was tied together at floor levels with 35-mm horizontal stressbars.

Holy Cross Church, Santa Cruz: The church was severely damaged in the 1989 Loma Prieta earthquake. The church was constructed of unreinforced clay-unit masonry walls built on a stone rubble foundation. Timber trusses span from buttresses across the width of the church, and provide support for the roof.

In addition to post-tensioning of masonry, retrofit measures included grouting of cracks in the masonry, reconstruction of the bell tower in steel and timber, addition of reinforced concrete beams on top of the buttresses and a new roof diaphragm system with steel trusses. Connections between the steel trusses and the buttresses were designed to yield so that the force delivered to the buttresses would be limited.

A total of 26 tendons were used to stress the unreinforced clay-unit masonry walls and buttresses to enhance shear and flexural strength. Tendons in the end walls and towers consisted of seven 12-mm diameter strands which were placed into vertical drilled cores 100-mm in diameter. Tendons in the buttresses included 12 strands which were placed within drilled cores 175 mm in diameter. Bare strands were bonded were bonded to the foundation with grout while the remaining length of the tendons placed within cores were greased and sheathed.

Summary and Significant Findings

Post-tensioning of existing unreinforced masonry walls can improve strength and ductility to resist lateral seismic forces. Continuous prestressed tendons can improve cracking strength, and thus seismic performance under low and moderate earthquakes. The method is best suited to structures with a masonry compressive strength exceeding 700 psi on the gross area.

Related Reference

Ganz, H.R., "Recent Experience with Post-Tensioned Masonry in Switzerland," *Proceedings of the Sixth North American Masonry Conference*, June 6-9, 1993, Philadelphia, Pennsylvania, pp. 657-667.

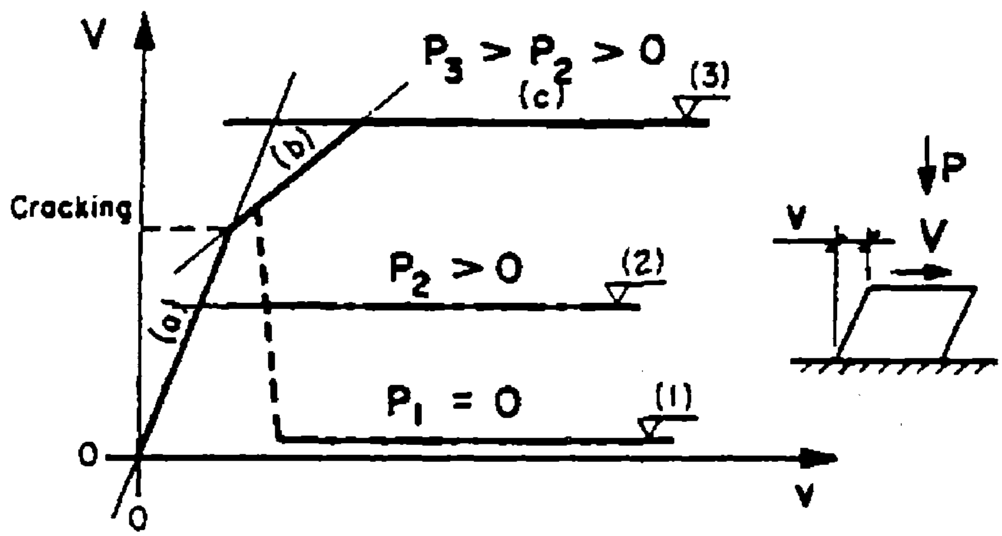


Figure 1: Effect of Vertical Prestress on Lateral Force-Deflection Behavior of a Wall

PC3 Recent Experience with Post-Tensioned Masonry In Switzerland

Hans Rudolf Ganz

Proceeedings of the Sixth North American Masonry Conference, June 6-9, 1993, Philadelphia, Pennsylvania, pp. 657-667.

Objective

The paper provides a summary of case histories where post-tensioned masonry has been used for design of new buildings in Switzerland.

Rehabilitation Procedure

Vertical prestressing tendons (Figure 1) are anchored within the concrete foundation below a wall, and are stressed against a concrete anchorage block at the top of wall. Monostrand tendons are fed through the stressing anchorage and 28-mm duct into a self-activating dead-end anchorage. The tendons are stressed with a light hydraulic jack to 75% of their tensile strength(200 kN), and locked off.

Anchorages at both the top and bottom of wall are filled with a special grease for the purpose of protecting corrosion of the prestressing steel. Low relaxation 7-wire strand of 0.6 in. diameter with ultimate strength of 265 kN is used. Pre-assembled chairs at the dead-end anchorage and caps on the top of each duct segment are provided for temporary protection. A minimum masonry compressive strength of 8 MPa (based on gross cross sectional area) is specified.

Case Studies

Factory Fire Proof Wall, Regensdorf: A single leaf masonry wall, 36.2 m long, 6.1 to 8.8 m high and 250 mm thick (Figure 2) was post-tensioned with 17 tendons at an average spacing of 2.0 m. The dead-end anchorages were cast into a 1m high concrete pad while the stressing anchorages were placed in precast concrete cubes of 250 mm length on the top.

Movie Theater, Wattwil: All perimeter walls and one inside wall were designed and constructed in post-tensioned masonry. The plan view of the building and a cross section of a post-tensioned wall are shown in Figure 3. The masonry walls, 26.5 m long, 5.15 m high and 180 mm thick, were post-tensioned with 33 tendons at a spacing 1.7 to 2.2 m.

Industrial Center, Altendorf: Masonry walls, 43.45 m and 9.5 m long were post-tensioned (Figure 4). The dead-end anchorages were cast 0.5 m into the concrete wall. A total of 71 tendons were placed at a spacing of 0.57 m and 0.95 m to resist wind loads.

Summary and Significant Findings

Post-tensioned masonry provides a suitable design for single-wythe masonry walls subjected to transverse loads.

Related Reference

Ganz, H.R., "Strengthening of Masonry Structures with Post-Tensioning," *Proceedings of the Sixth North American Masonry Conference*, June 6-9, 1993, Philadelphia, Pennsylvania, pp. 645-655.

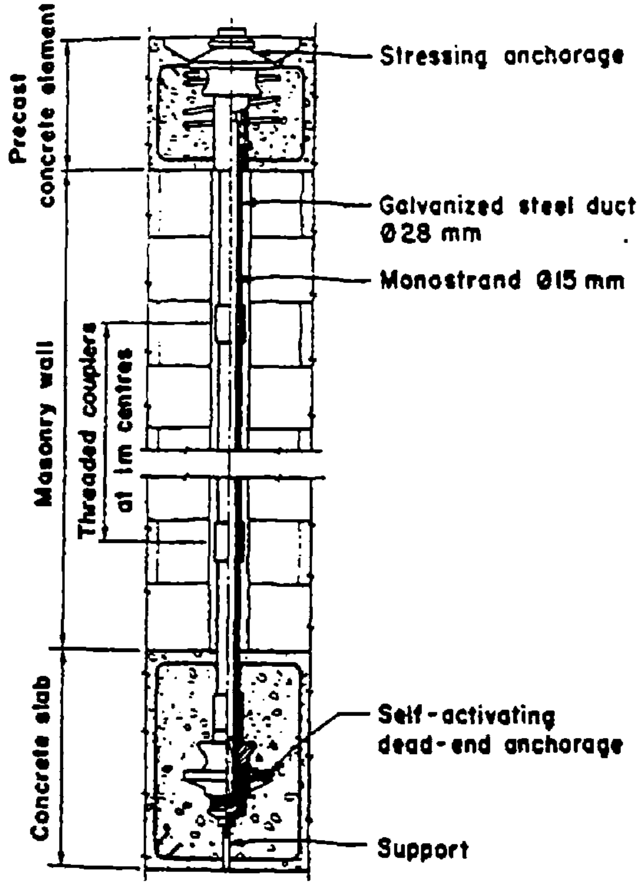


Figure 1: Typical Tendon for Post-Tensioned Masonry

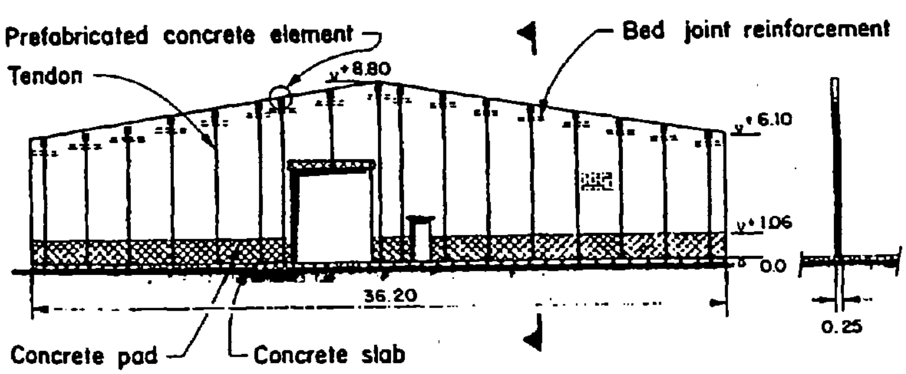


Figure 2: Post-Tensioned Masonry Walls for Factory in Regensdorf

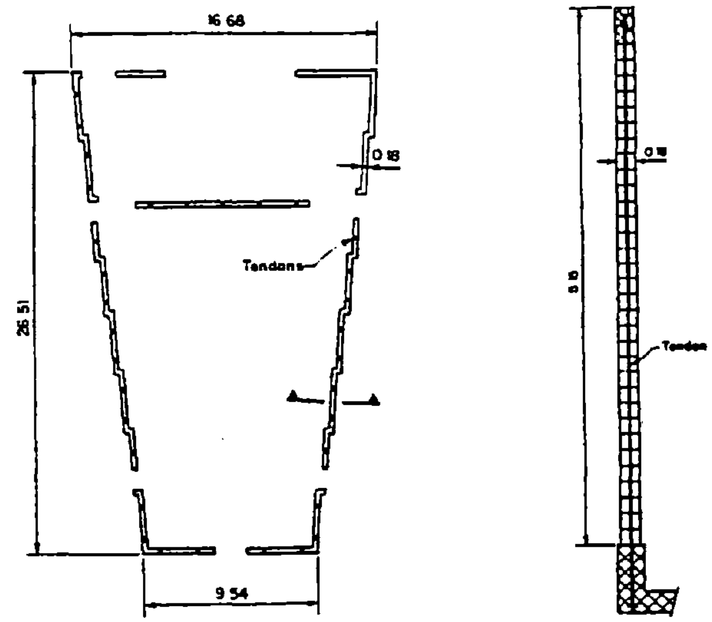


Figure 3: Plan of a Movie Theater and a Cross Section of a Post-Tensioned Wall

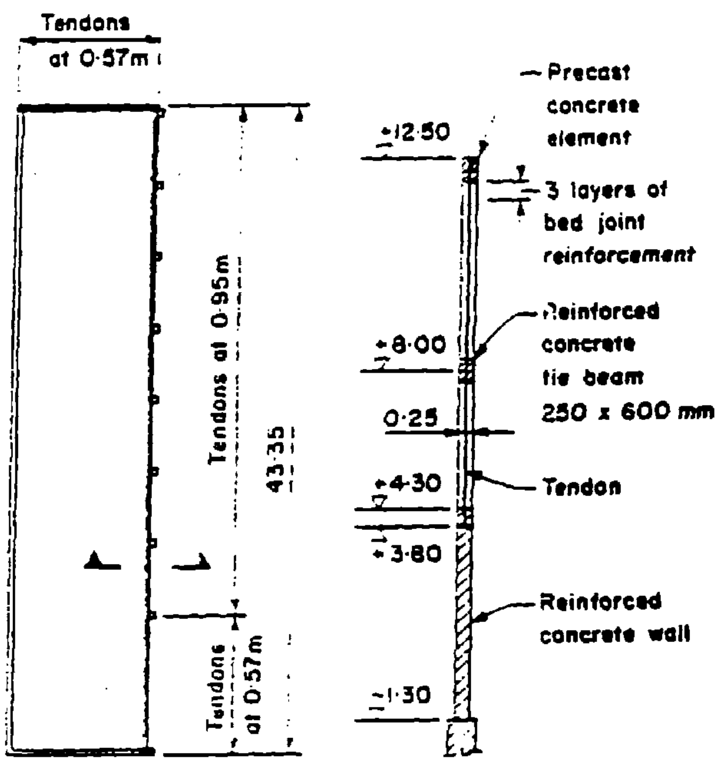


Figure 4: Plan of Warehouse and a Section through the Post-Tensioned Wall

PC4 Performance of a Post-Tensioned, Single-Wythe, Clay Brick Masonry Wall Tested in Shear

A. Huizer and N.G. Shrive

Proceedings of the Fourth Canadian Masonry Symposium, University of New Brunswick, 1986, Vol. 2, pp. 609-618.

Objective

The objective of this research was to investigate the use of post-tensioning for improving the flexural and shear capacity of hollow clay-unit masonry walls.

Rehabilitation Procedure

Unreinforced masonry walls are post-tensioned using Dywidag bars in both the horizontal and vertical directions as shown in Figure 1. These bars are placed and tensioned 14 days after construction of the wall, and are left unbonded.

Research Approach

A single-wythe, story high, hollow clay masonry test wall was constructed using hollow clay units (Figure 2). The compressive strength of the masonry units was measured as 10.4 Mpa. A Type S mortar with an average 28-day compressive strength of 10.5 MPa was used for construction.

A total of eight Dywidag bars were used to post-tension the test wall with four (bars 1-4) being oriented in the vertical and four (bars 5-8) in the horizontal direction. Post-tensioning forces applied to each bar are shown in Figure 1 along with locations of displacement transducers (LVDT). Strains for each prestressing bar at various stages of the investigation are summarized in Table 1.

The masonry units were found to have a horizontal compressive strength of one-fifth that of the vertical compressive strength. As a result the horizontal prestressing force was less than that of the vertical prestress. The reduction in strains between release and the start of the test was attributed to anchorage and relaxation losses in the steel and creep in the masonry. In addition, the wall was constructed with the vertical prestress bars placed at an eccentricity of 15 mm from the center line of the wall.

Summary and Significant Findings

The post-tensioned test wall was subjected to in-plane shear force which was increased monotonically until failure occurred. The force-deflection curves for the tension side of the wall at different transducer locations are given in Figure 3. Unlike a wall without post-tensioning, the test wall responded in a ductile manner until a toe compressive failure occurred in the third to sixth course above the base. With increasing lateral force and displacement, the wall softened until failure. Cracking and spalling of the wall occurred on only the side as a result of the higher compression force caused by the eccentric vertical prestress. This zone of the wall also corresponded to the location of minimum lateral compression due to the horizontal prestress. Hollow masonry was found to be stronger in bi-axial compression than in uniaxial compression.

Masonry shear capacity was enhanced due to the increase in normal force provided by the vertical prestress. A value of allowable stress equal to 0.10 MPa was calculated for the brick masonry shear wall based on 0.04

$\sqrt{f'_m}$. For the recorded peak load of 131 kN, the stress was 0.43 MPa based on the gross wall area.

The prestressing force not only improved the shear and flexural behavior of the wall, but also increased the ductility of the test wall. The wall remained intact after unloading..

Table 1: Strains in Post-Tensioning Dywidag Bars

Tendon	At Release Level (10^{-6} m/m)	At Start of Shear Test (10^{-6} m/m)	At End of Shear Test (10^{-6} m/m)
#1	3900	3520	4900
#2	3900	3580	4740
#3	3900	3520	3040
#4	3900	3460	2650
#5	1450	1180	1560
#6	1450	1380	2000
#7	2100	1730	2230
#8	2100	1820	1720

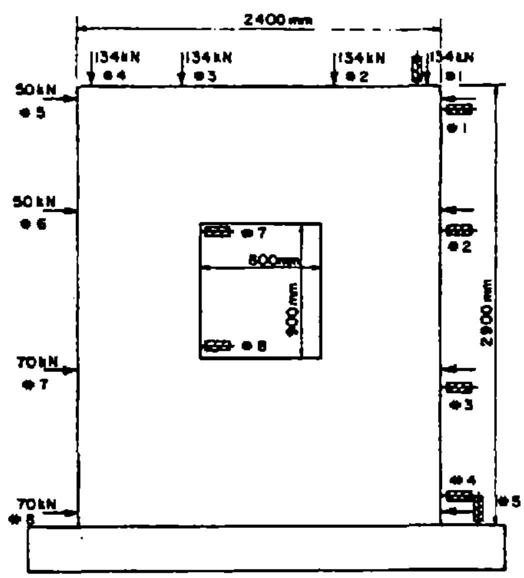


Figure 1: Description of Post-Tensioned Masonry Test Wall

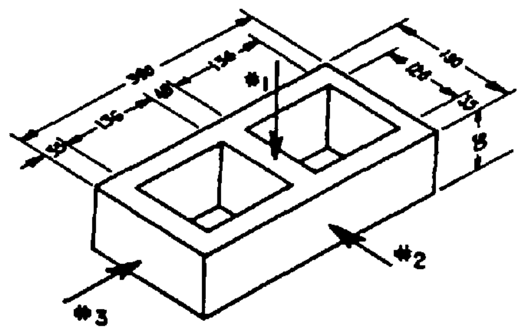


Figure 2: Typical Hollow Clay Unit

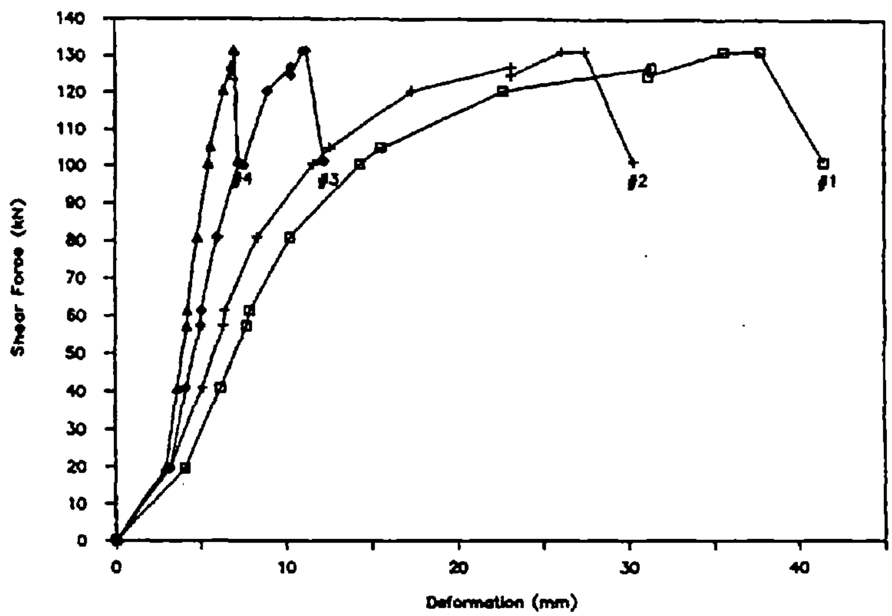


Figure 3: Force-Displacement Relations for Test Wall

PC5 Shaking Table Tests of Three Four-Storey Brick Masonry Models: Original and Strengthened by RC Core and by RC Jackets

D. Jurukovski, L. Krstevska, R. Alessi, P.P. Diotallevi, M. Merli and F. Zarri

Proceedings of the Tenth World Conference on Earthquake Engineering, July 1992, Madrid, A A Balkema, Rotterdam, Vol. 5, pp. 2795-2800.

Objective

The primary purpose of this research was to develop an appropriate strengthening technique for buildings, constructed with mixed structural systems of unreinforced brick masonry and reinforced concrete. The research consisted of tests of models on a biaxial shaking table. Two strengthened models by RC core and by RC jackets were employed. The secondary purpose of this project was aimed at pointing out the differences in the dynamic behavior of those two models.

Rehabilitation Procedure

Cores are drilled into unreinforced brick walls and reinforced. Steel jackets are wrapped around unreinforced brick walls.

Research Approach

A prototype building was considered consisting of a mixed concrete frame and masonry wall system at the lower story, and unreinforced brick masonry walls at the upper three stories. One-third scale models of portions of the building systems were subjected to simulated earthquake motions on a shaking table as well as forced vibration tests.

The first test structure consisted of brick masonry walls and reinforced concrete frames only at the first floor. The second test structure was strengthened by external reinforced concrete walls. The third test structure was strengthened by a central core.

Simulated earthquakes were based on motions measured at El Centro 1940, Parkfield 1966, Montenegro 1979 records obtained at Bar and Petrovac and Friuli 1976 records at Breginj-Slovenia.

Summary and Significant Findings

The two strengthening methods increased the lateral strength of the building systems. Each technique resulted in a failure mechanism (Figure 1) that distributed the energy over the height of the structure, and provided a high energy absorption capacity.

Behavior of the first structure was characterized by intensive damage to the first story with slight damage to the second story. Masonry damage was observed across all stories for the second and third structures.

Measured acceleration histories recorded at the fourth floor (Figure 2) indicated that the first test structure had little or no amplification of base accelerations while the amplification factors for the second and third structures were approximately 3 and 2 respectively. These tendencies were related to the different damage mechanisms for the three structures.

Measured deflection histories at the fourth floor (Figure 3) indicated that the first structure deflected almost twice as much as the second structure. This was a result of the greater extents of damage for the first test structure.

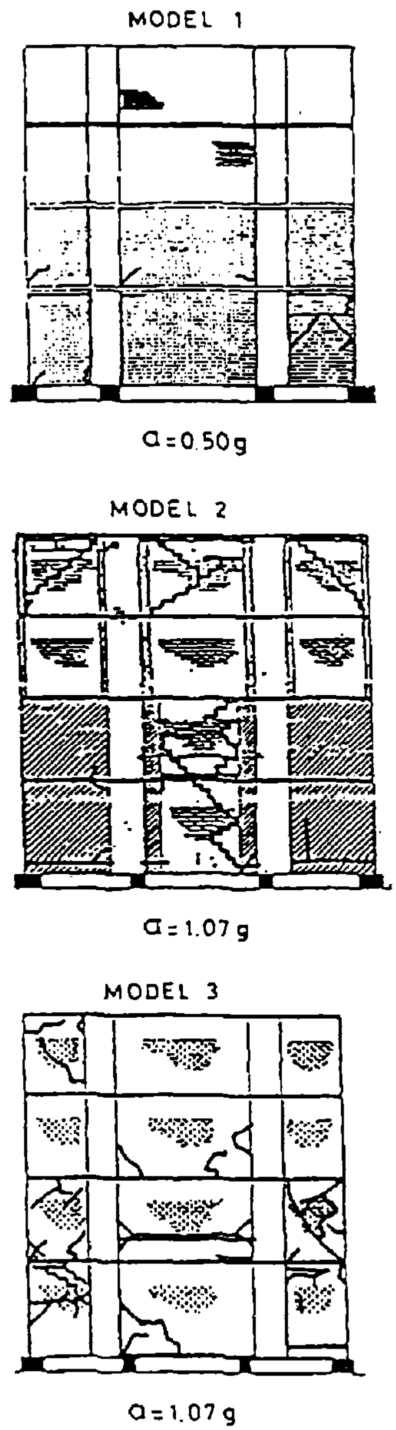


Figure 1 : Observed Cracking Patterns for Test Walls

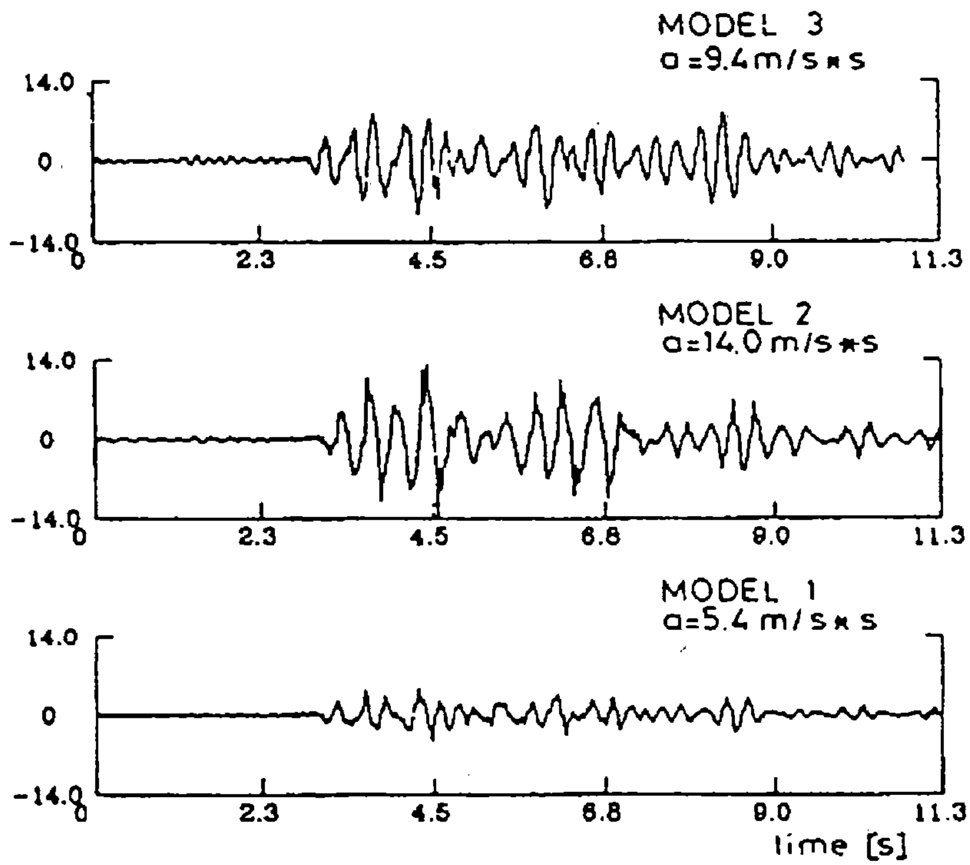


Figure 2: Measured Acceleration Histories for Three Test Structures

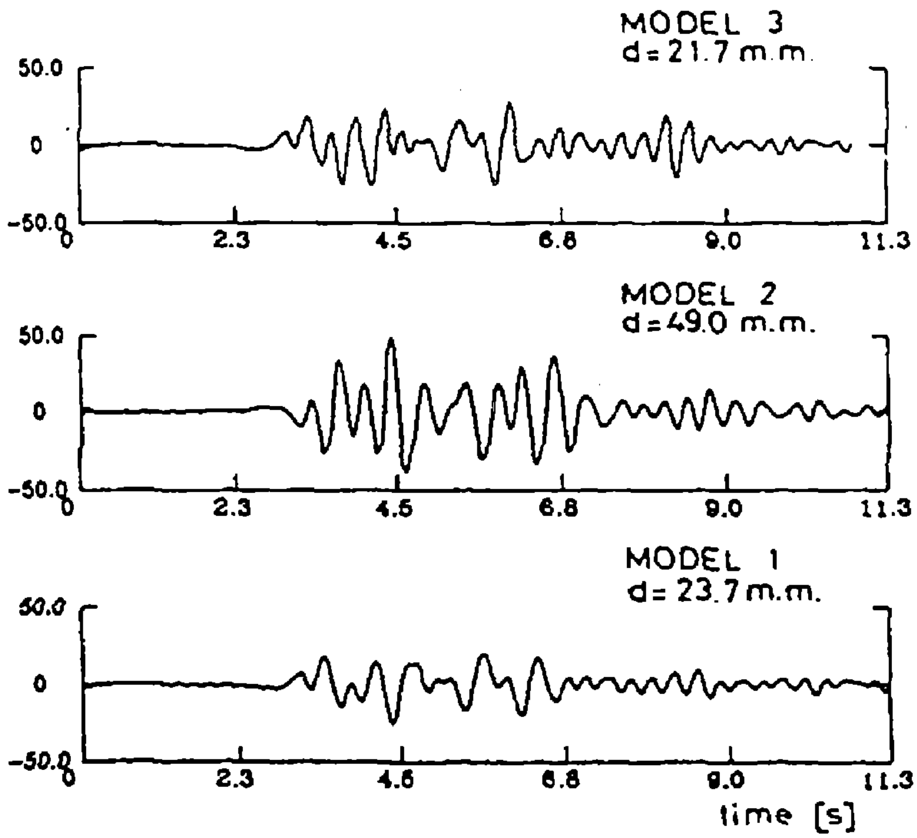


Figure 3: Measured Displacement Histories for Three Test Structures

PC6 Strengthening of Unreinforced Masonry Buildings

Joseph Plecnik, Thomas Cousins, and Edward O'Conner
Journal of Structural Engineering, Vol. 112, No. 5, May 1986, pp. 1070-1087.

Objective

The objective of the research was to determine the strength of newly manufactured small specimens made with cement grout, sand/polyester grouts and sand/epoxy grouts as filler materials, and optimum parameters such as core diameter and flow and strength characteristics of the core filler materials.

Rehabilitation Procedure

The proposed method involved strengthening multi-wythe, unreinforced brick masonry walls for seismic load in the out-of-plane and in-plane direction. These walls were usually from 1 to 3 stories high.

A vertical core is drilled through the wall to the foundation. A reinforcing bar is placed in the core hole with filler material poured into the hole. The filler material can be unfilled or filled epoxy, or polyester, and cement grout. The distance between the vertical holes, the size and type of reinforcing, and the size of the core depend on the seismic design requirements of the wall.

Research Approach

Over 70 small scale specimens were built, subjected to a static shear load and tested to failure. One type of brick and #5 reinforcing steels or fiberglass rods were used in the specimens. Different mortar strengths were used to determine the effect and the contribution of mortar strength on the shear strength of the test specimens. Three types of core filler materials were used: cement grout, a sand/polyester mix, and a sand/epoxy mix. Different mix ratios of these three materials were employed to determine the mixing ratio yielding the optimum result in terms of strength and costs.

Three buildings located in the Raleigh, North Carolina, were chosen as typical of the Type III (1982 UBC classification) URM brick masonry construction and were designated as Buildings #3, #4, and #5.

After determining the compressive strength of the brick and performing shove tests, some portions of the walls in these buildings were strengthened. Panels and prisms were cut out of these strengthened walls and transported to the laboratory for testing. Panels were loaded cyclically for resistance to in-plane shear loads (Figures 1 and 2) and out-of plane forces (Figures 3 and 4).

Summary and Significant Findings

Large-diameter cores result in a greater flow of the filler material into collar joints, and provide a shear transfer mechanism to attain ultimate out-of-plane moment capacity. The cores also provide a greater effective area to resist in-plane shear forces.

Strength and flow characteristics of sand-filled epoxy or sand-filled polyester grouts are better than cement grouts. Polyester grouts are more widely used because of the higher cost of epoxy grouts.

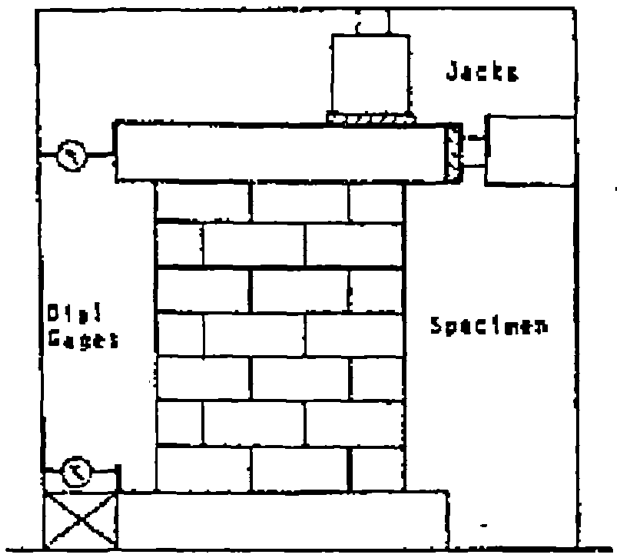


Figure 1: In-Plane Test Setup

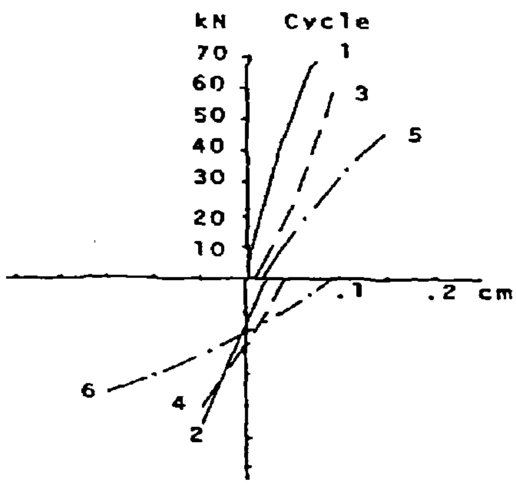


Figure 2: Measured Force-Deflection Relation for In-Plane Loading

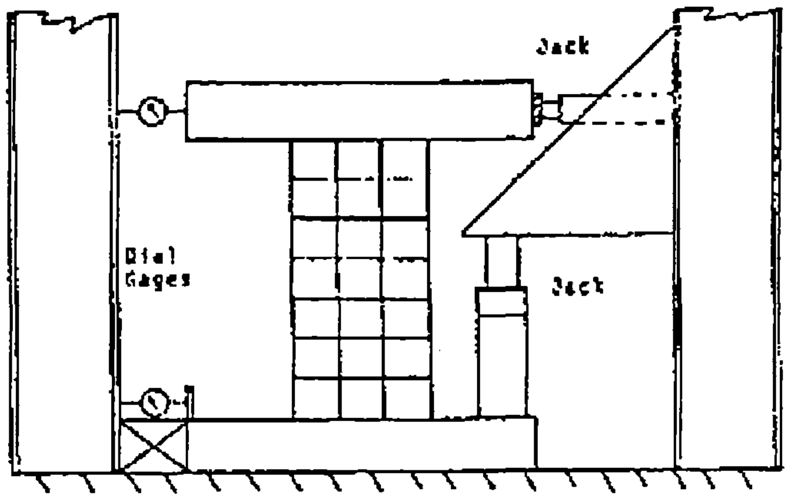


Figure 3: Out-of-Plane Test Setup

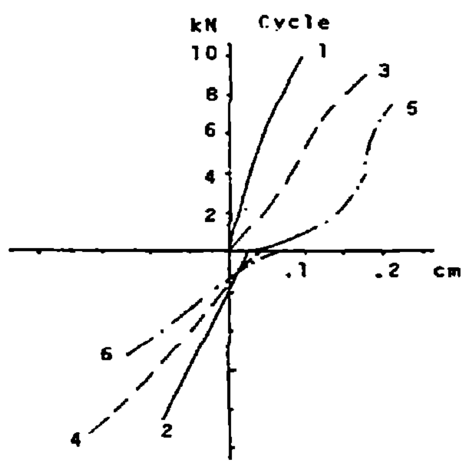


Figure 4: Measured Force-Deflection Relation for Out-of-Plane Loading

Miscellaneous Rehabilitation Techniques

MR1 Dynamic Response of Unreinforced Masonry Buildings with Flexible Diaphragms

A.C. Costley and D.P. Abrams

Structural Research Series Report No. 605, Department of Civil Engineering, University of Illinois at Urbana-Champaign, October 1995, pp. 281.

Objective

The objective of the research was to investigate dynamic response of unreinforced clay-unit masonry buildings with flexible diaphragms. One rehabilitation procedure that was investigated consisted of enlarging window openings so that piers would be controlled by rocking.

Rehabilitation Procedure

Portions of a masonry wall below window openings are removed so that the height-to-length aspect ratio of adjacent piers is increased, and as a result, lateral force-deflection behavior is governed by a rocking mechanism. Overall story shear strength is reduced, but deformation capacity is increased.

Research Approach

Two reduced-scale test structures were constructed and subjected to simulated earthquake motions on a shaking table. The 3/8th scale buildings consisted of two parallel, perforated shear walls that were tied together with flexible diaphragms at each of two levels.

Shear walls of the first test structure (S1 in Figure 1a) were comparable in lateral strength with a "door wall" and a "window wall". Test structure S2 was constructed by eliminating the portions of masonry below the windows of the S1 window wall, and filling portions of masonry at the base of the door openings of the S2 door wall (Figure 1b). This resulted in significantly different lateral strength for each of the two walls.

Each test structure was subjected to a series of progressively increasing base motions until the capacity of the earthquake simulator was reached.

Summary and Significant Findings

Global force-deflection behavior of each test structure is compared in Figure 2. Base shear forces have been determined from measured accelerations of the structures and divided by the total weight. Lateral deflections at the top of the first story have been divided by the story height to express in terms of a drift percentage.

Initial cracking was observed for both structures at less than 0.1% drift. Maximum in-plane drifts were in the range of 1%.

Test Structure S2 was weaker than S1 as a result of enlarging the window openings; however, its performance was better because less damage occurred to the base-story piers. Cracks were observed at the top and bottom bed joints of the slender piers of the "door wall" of S2. Little or no damage was observed on the opposite "window wall" because lateral forces were limited by the shear force that could be resisted by the weaker of the two walls. Horizontal cracks in the piers closed following the earthquake simulations as a result of gravity stress. Structure S1 was also controlled by rocking, but had more extensive cracking because shear forces were higher.

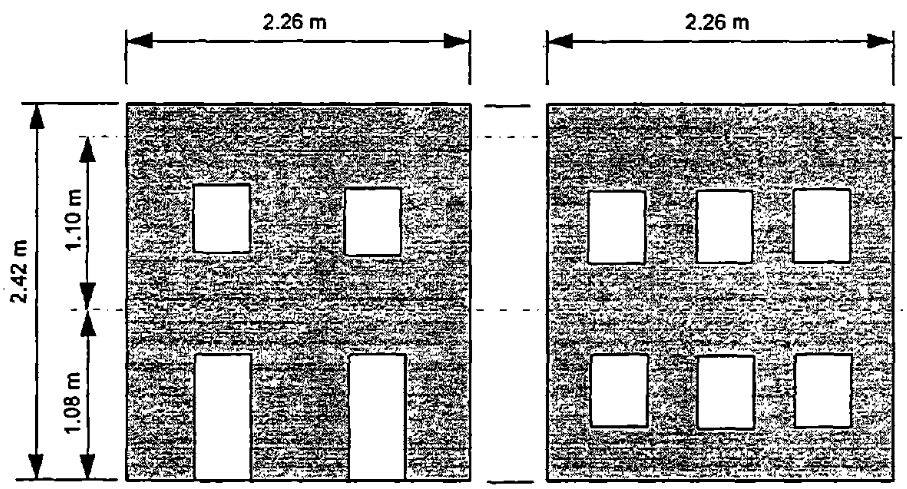
Related References

Costley, A.C., D.P. Abrams and G.M. Calvi, "Shaking Table Testing of an Unreinforced Brick Masonry Building," *Proceedings of Fifth U.S. National Conference on Earthquake Engineering*, Chicago, July 1994, pp. 127-136.

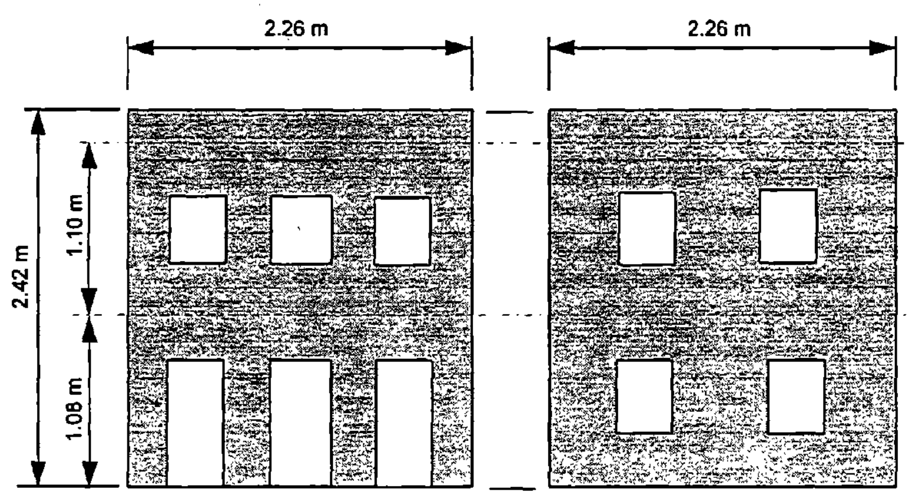
Costley, A.C., and D.P. Abrams, "Seismic Response of URM Buildings," *Proceedings of Seventh Canadian Masonry Symposium*, McMaster University, Hamilton, Ontario, June 1995, pp. 72-83.

Costley, A.C., and D.P. Abrams, "Response of Building Systems with Rocking Piers and Flexible Diaphragms," *Proceedings of the ASCE Structures Congress IX*, Chicago, April 1996.

Abrams, D.P., "Response of Unreinforced Masonry Buildings," *Journal of Earthquake* Engineering, Imperial College Press, Vol. 1, No. 1, November 1996.



(a) Test Structure S1



(b) Test Structure S2

Figure 1: Description of Test Structures

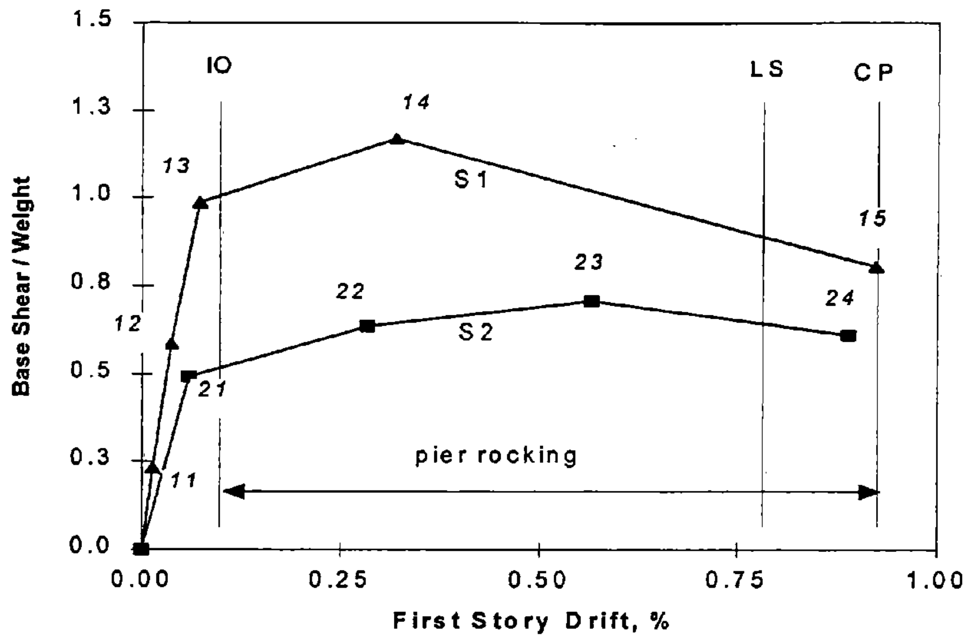


Figure 2: Overall Behavior of Each Test Structure

MR2 Earthquake Resistant Behavior of Brick Wall Strengthened by Additional RC Columns with Steel Tie Rods

Zezhen Niu, Qi Du, Jianyou Cui and Runtao Yu
Institute of Earthquake Engineering, China Academy of Building Research, Beijing, People's Republic of China, May 1984.

Objective

The objective of the research was to investigate the effectiveness of strengthening unreinforced clay-unit masonry walls with reinforced concrete columns and steel tie rods. A series of experiments were done on test walls to develop a set of expressions for estimating lateral strength of enhanced walls.

Rehabilitation Procedure

Reinforced concrete columns are placed at the ends of a wall panel. A beam or steel rod is placed to tie the columns together at the top of the wall panel. The intention of this method is to improve in-plane strength and ductility of the wall panel through added confinement of the panel.

Research Approach

A total of 16 half-scale test walls were subjected to lateral forces to investigate enhancements in strength and ductility with the rehabilitation procedure. A typical test wall is shown in Figure 1. Reinforced concrete columns were cast at the ends of the test walls and tied to the masonry with reinforcement. In addition, steel tie rods were used to tie the columns together. Two test walls were not strengthened to serve as control specimens.

Test walls were subjected to various levels of vertical compressive stress ranging from 2.0 to 4.5 kg/cm^2 (28 to 64 psi). Thirteen walls were 480 cm (15.7 ft) long while three walls were 240 cm (7.9 ft) long. Wall thickness was 24 cm (9.5 inch).

Summary and Significant Findings

The force-deflection behavior for the plain, unstrengthened and unreinforced masonry walls (Figure 2) revealed a stable hysteresis loop. Frictional forces resisted shear along bed joints after the formation of initial stair-stepped diagonal cracks.

The experimental investigation revealed three failure modes (Figure 3) for masonry walls strengthened with the rehabilitation procedure: (a) flexural cracking in the columns followed by diagonal tension cracking, (b) diagonal tension cracking with some flexural cracking in the columns and (c) diagonal tension cracking with no flexural cracking in the columns. The first mode occurred for the walls with the shorter length. The third mode occurred with the walls with the higher vertical compressive stress.

The steel tie rods working together with the reinforced concrete columns confined the unreinforced brick masonry walls, and thus, enhanced their lateral strength and inelastic deformation capacity.

Related Reference

Niu, Z., Q. Du, J. Cui, and R. Yu, "A Study of a Seismic Strengthening for Multi-Story Brick Building by Additional R/C Columns," *Proceedings of the Eighth World Conference on Earthquake Engineering*, San Francisco, Vol. 1, 1984, pp. 591-598.

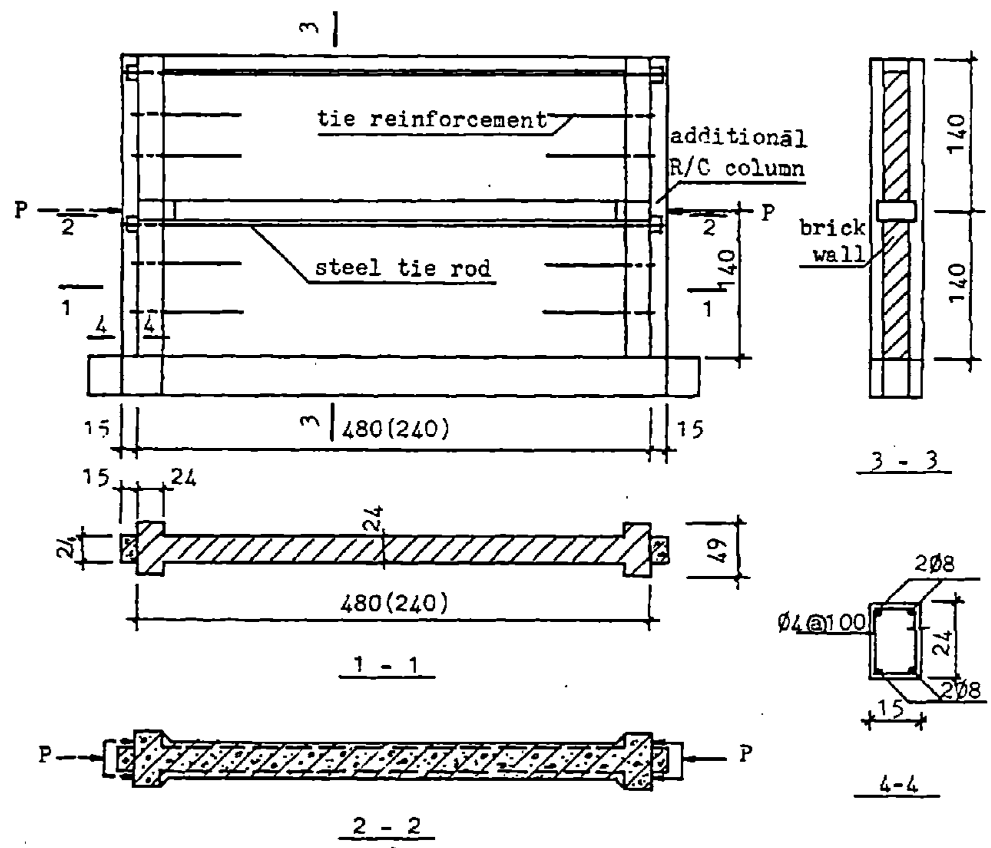


Figure 1: Description of Test Wall

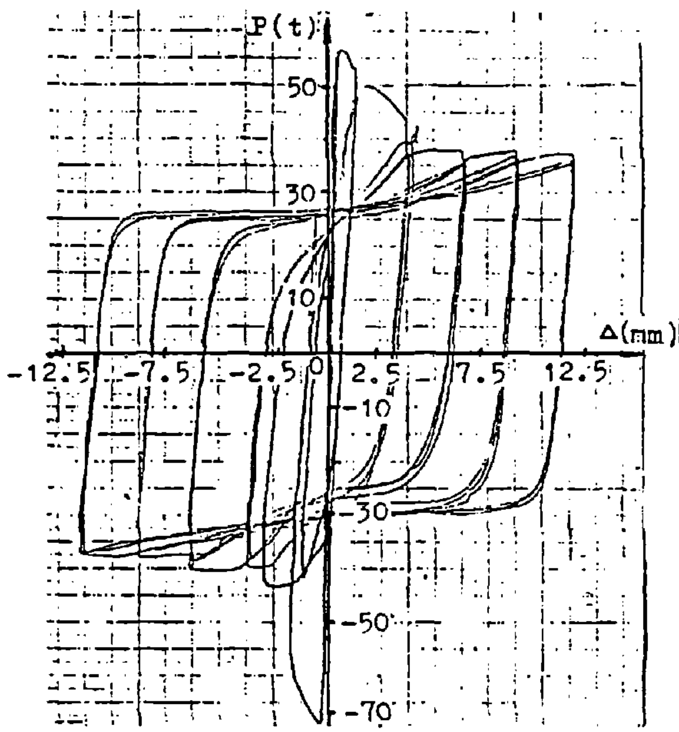


Figure 2. Measured Force-Deflection Curve for Unstrengthened Wall

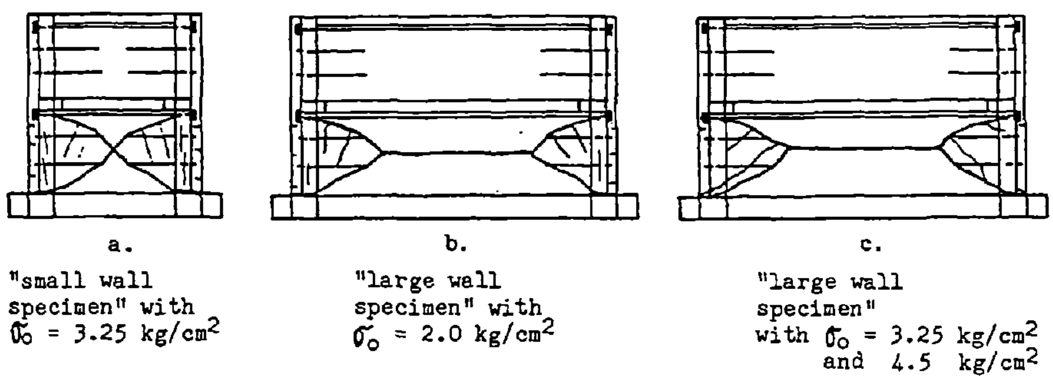


Figure 3. Observed Failure Modes for Strengthened Walls

MR3 Seismic Strengthening of URM Buildings with Steel Bracing

D.C. Rai, S.C. Goel, and W.T. Holmes
Proceedings of the Fifth U.S. National Conference of Earthquake Engineering, Chicago, July 1994, Vol. 3, pp. 697-705.

Objective

The objective of this research was to investigate the use of steel bracing for in-plane strengthening unreinforced masonry buildings.

Rehabilitation Procedure

Steel bracing members are placed across an unreinforced brick masonry wall.

Research Approach

A half-scale model (Figure 1) of a third-story exterior window wall was constructed, fitted with steel bracing and tested subjected to a series of slowly applied reversals of lateral displacement.

The test wall was two wythes thick and measured 1.68m high and 2.34m long (Figure 2). Three openings, 76 mm wide, were included in the test wall leaving four piers with dimensions of 0.53 m by 1.00 m.

Reclaimed bricks were used with a Type N mortar to construct the test wall. Average compressive strength of test prisms was 6.18 MPa. Flat-wise compressive strength of brick units was 9.13 MPa and compressive strength of mortar cubes was 12.0 MPa.

The steel braces were 2.5"x 1.5"x 3/16", ASTM A500 Grade B tubes. To prevent out-of-plane buckling of the braces and damage to the wall, the braces were oriented with the weak axis perpendicular to the plane of the wall. Additionally, pin-ended vertical steel members were provided at both ends of the wall to resist overturning moments. Horizontal forces were applied at an eccentricity equal to 0.74 m to simulate overturning moments from an upper story.

Summary and Significant Findings

Strength of the retrofitted system was limited by strength of a weld connecting a bracing member to a steel gusset plate at a story drift equal to 0.75%. This was due to inadequate penetration of the weld on one side of the gusset plate resulting in differential stress concentrations in the unbalanced fillet welds.

The steel bracing members behaved independently of the masonry elements. Measured behavior of the system was quite similar to that expected for the steel bracing members alone (Figure 3).

Due to the increased hold-down forces in vertical steel members of the bracing system, the rocking capacity of the masonry piers was significantly increased. The observed maximum shear was 147 kN which was much larger than the estimated 9 kN rocking shear for a non-retrofitted wall.

Vertical members of the steel bracing system apparently increased the effective overburden load on the piers, resulting in an increase in shear strength. A comparison of the forces in the vertical members and the shear in the wall is given in Figure 4.

Related Reference

Rai, D.C., "Hysteretic Behavior of Unreinforced Masonry Piers Strengthened with Steel Elements," *Proceedings of Eleventh World Conference on Earthquake Engineering*, Acapulco, June 1996, Elsevier Science, Ltd., Paper No. 501.

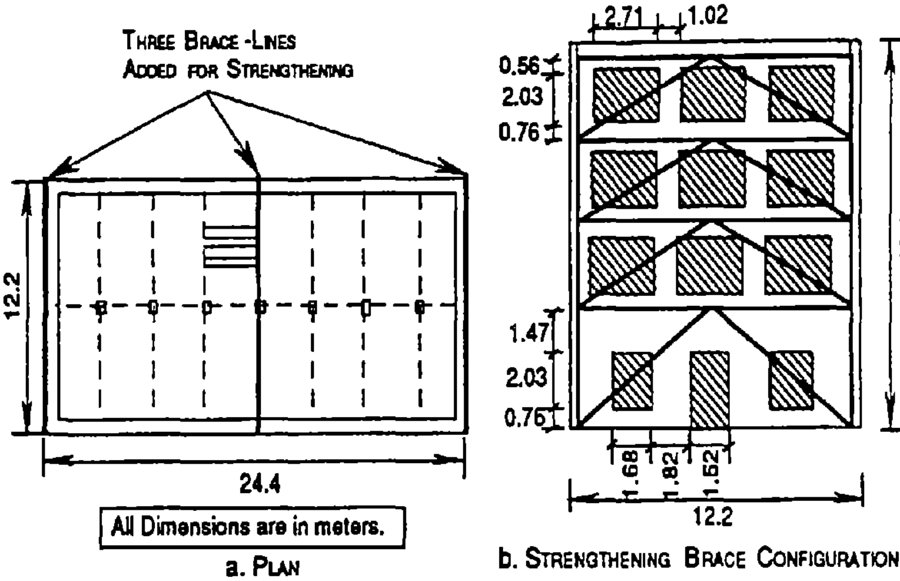


Figure 1: Prototype Building Used in Investigation

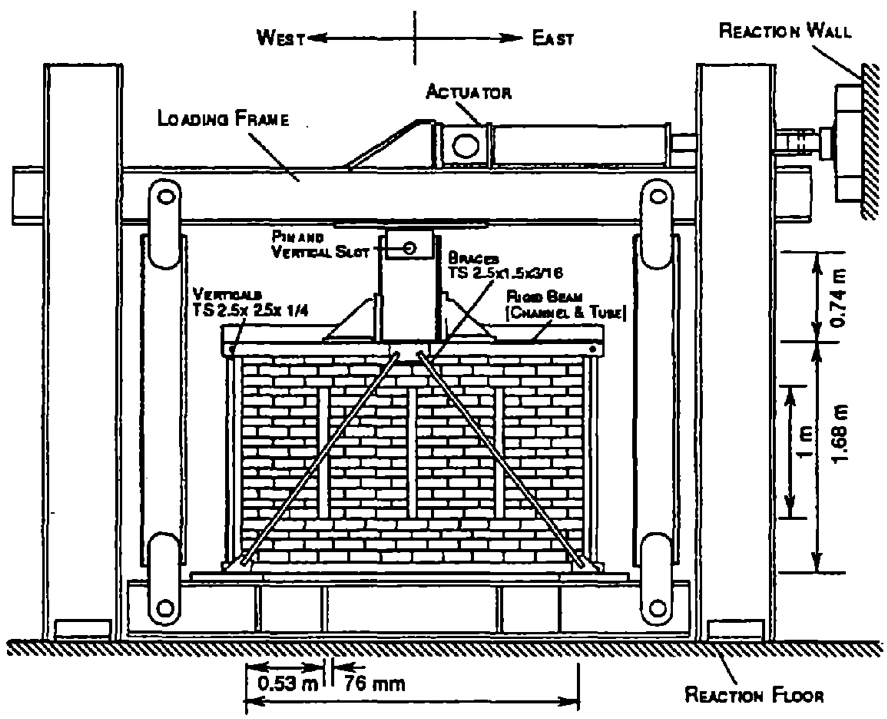


Figure 2: Test Specimen and Loading Frame

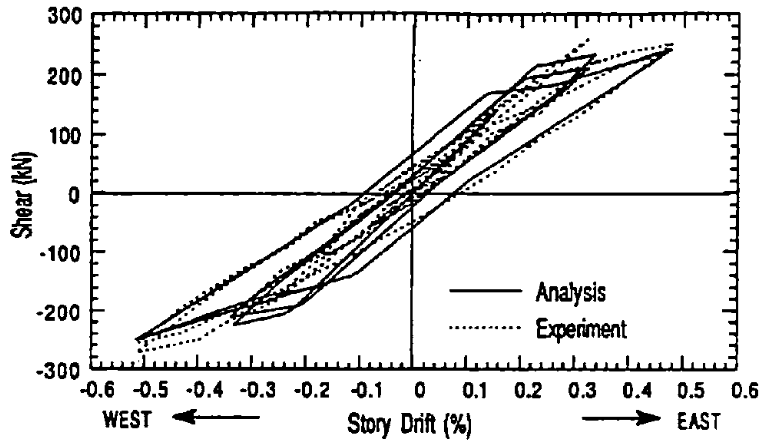


Figure 3: Measured and Calculated Force-Deflection Behavior of Bracing Members

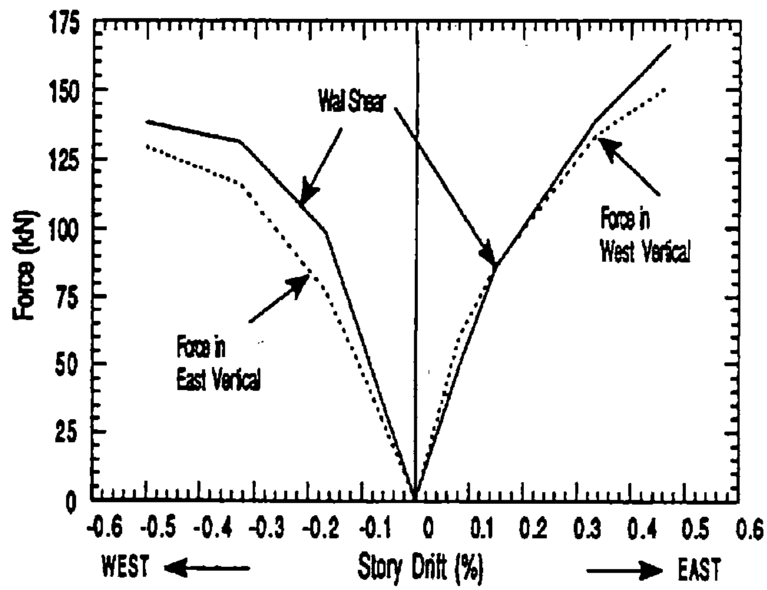


Figure 4: Shear Resisted by Wall and Forces in Vertical Members

MR4 Hysteretic Behavior of Unreinforced Masonry Piers Strengthened with Steel Elements

D.C. Rai

Proceedings of Eleventh World Conference on Earthquake Engineering, Acapulco, June 1996, Elsevier Science Ltd., Paper No. 501.

Objective

The objective of the research was to evaluate the effectiveness of a surrounding steel unbraced frame for improving the rocking performance of unreinforced masonry piers.

Rehabilitation Procedure

A steel frame is placed around an unreinforced masonry pier. Vertical steel members provide hold-down forces to stabilize rocking controlled piers and increase pier shear strength.

Research Approach

Individual masonry pier components were subjected to in-plane lateral forces using the testing rig shown in Figure 1. Vertical steel elements (TS 2.5 x 2.5 x 1/4) were placed adjacent to each pier edge. In one specimen, the test pier was centrally located in the steel frame to simulated an interior pier, and in another case, the test pier was located asymmetrically to simulate an exterior pier.

The test piers were 21-in. wide by 39-in. high. The unreinforced masonry piers were laid in running bond with Type N mortar and reclaimed clay-masonry units. The average prism compressive strength was 1060 psi. The average in-place shear strength was 110 psi.

A finite element study as well as a simplified analytical model were developed to help understand the experimental observations.

Summary and Significant Findings

Measured force-deflection relations for the interior and exterior test piers are shown in Figure 2.

For the interior pier, the first flexural cracks were observed at approximately 0.2% drift. The piers then continued to rock until severe crushing of the toe occurred at approximately 2.2% drift where shear strength decreased to 60% of maximum strength.

Similar behavior was observed for the exterior pier however the shape of the hysteresis loop was asymmetrical. Cracking was observed in the sill near the toe at 0.75% drift. A significant strength decrease was observed at 3.0% drift when toe crushing occurred. The cracking pattern was asymmetrical as were the peak strengths for each direction of loading. The ultimate limit state was vertical splitting at a drift of 4.5%.

The analytical study confirmed the experimental observation that the stabilizing moment of a rocking pier can be enhanced by the vertical members of a steel frame. Axial tensile forces in these vertical members result in vertical compressive forces applied to the piers which increase rocking strength.

The strength, stiffness and ductility of unreinforced masonry piers were substantially enhanced with the introduction of an unbraced steel frame. The rehabilitation method also controlled damage.

Related References

Rai, D.C., S.C. Goel and W.T. Holmes, "Seismic Strengthening of URM Buildings with Steel Bracing," *Proceedings of the Fifth U.S. National Conference on Earthquake Engineering*, Chicago, July 1994, Vol. 3, pp. 697-705.

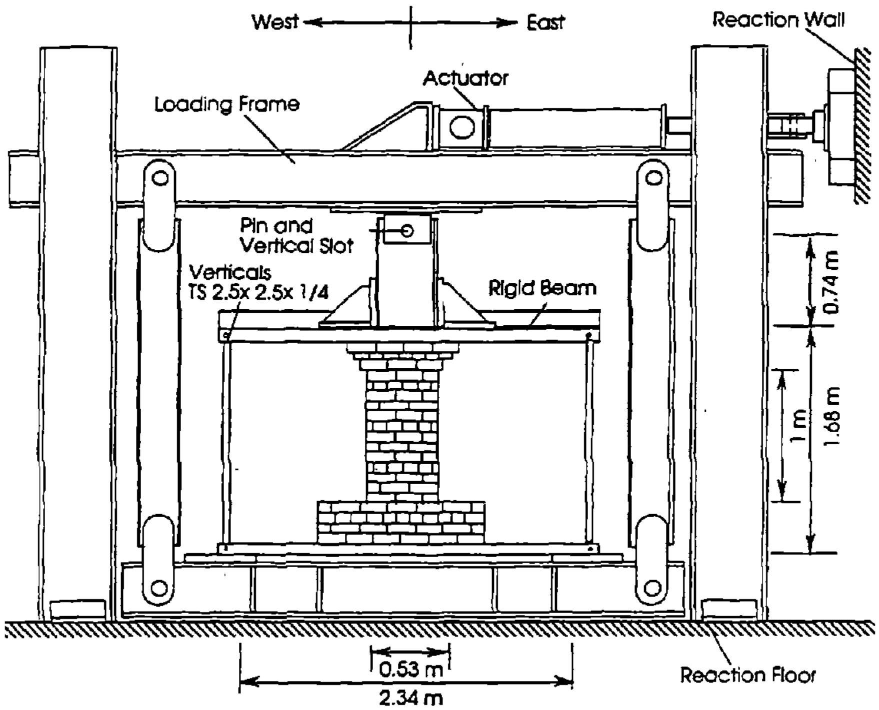
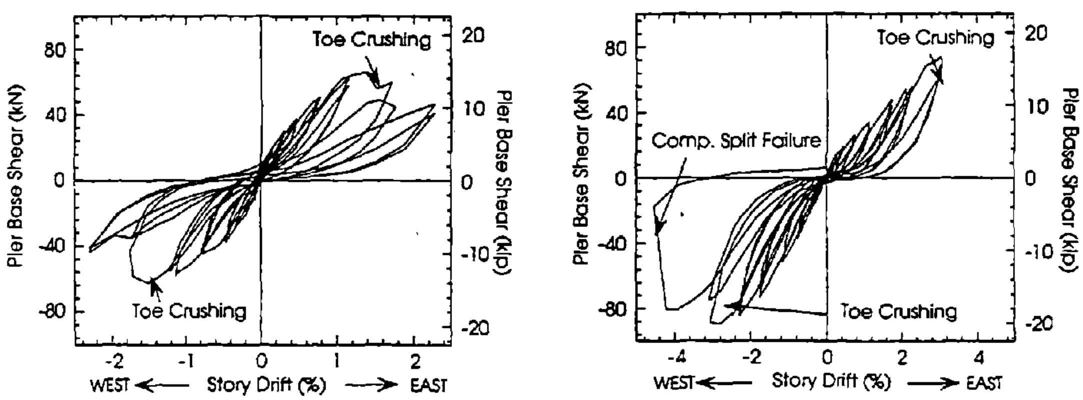


Figure 1: Test Set-up



(a) Interior Pier

(b) Exterior Pier

Figure 2: Measured Load Deflection Curves

APPENDIX D
BENEFIT-COST CONSIDERATIONS

Preface

This appendix provides benefit-cost information for enhancement procedures, grouped into three categories: 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 capacity 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.

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D.1 Summary

The following is a summary of enhancement procedures and their range of estimated costs. Detailed discussion is contained in the following sections.

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). 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-place 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 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.

Table D.1: 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-place 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 this Appendix.
2. "*" indicates cost of scaffolding comprises more than 33% of total estimated cost.

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, 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% of construction costs to as much as 10% of construction costs.

Peer Review/Plan Check

Three levels of peer review can contribute to improved design quality. The first level occurs prior to commencing evaluation and design, when the evaluation and design methodology 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 D.2: 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 this Appendix.		

Table D.3: 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. Site visits provide the engineer of record the opportunity to review construction for general conformance with the design intent and can alert the contractor to non-conforming conditions. It can also facilitate problem solving by providing firsthand observation of existing conditions and by opening a channel of direct communication with the contractor.

Estimated costs for special inspection and testing for nine wall enhancement techniques applied to the prototype building are included in Table D.8 of this Appendix.

Table D.4: 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 rehabilitating specific activities are presented in Table D.5. Estimates are based on costs reported in FEMA 156 (1988) and are adjusted to 1996 cost for construction in San Francisco.

Table D.5: Estimated Cost of Specific Element Enhancement

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 enhancement to the building's expected seismic performance.	

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 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 to an A_a=0.40 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 and very high areas of seismicity are presented in Table D.6.

Table D.6: 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.

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 strong backs 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 D.7: 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 D.8 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.

D.2 Quality of Design and Construction

Improved Knowledge of the Building

Improved knowledge of the building comes principally through field investigation. Field investigation begins at the start of the evaluation process and can continue throughout the design of the retrofit solution. The engineer must frequently decide what unknown conditions are pertinent to understanding the building's construction, its behavior under seismic loading, and the eventual retrofit solution. Improved knowledge can be achieved through investigation of existing conditions and tests of material capacity. Investigations often require removing localized floor, wall and ceiling areas, and require the services of contractors. Testing, such as in-plane shear tests and out-of-plane flexural tests, is performed by testing labs. Both investigation of existing conditions and material testing require that the engineer coordinate with contractor, owners and/or testing labs to describe the required work and ensure it is satisfactorily performed.

Exposing Masonry Wall-to-Diaphragm Connections

Rationale: Masonry wall-to-diaphragm connections form a critical link in the lateral load path. Connections transfer in-plane loads from the diaphragm to lateral force-resisting walls. The same connections may serve to transfer wall out-of-plane forces to the diaphragm. Joists ends often embed into masonry walls for bearing support. Embedded ends subject to moisture, as is often the case at ground floor joists, will often decay. Thorough knowledge of connection geometry and material conditions are required if an engineer is to correctly evaluate connection capacity, or, assuming retrofitting is required, correctly design a retrofit for the connection.

Procedure: Exposing masonry wall-to-diaphragm connections requires removing approximately an 18" x 36" area of the floor or ceiling in various locations in the building. Openings should expose conditions of joists framing perpendicular to walls and joists framing parallel to walls. As framing may change from floor to floor or roof, openings should be made at each floor level and at the roof. First floor exposure may not be required where the crawl space allows access. Lath and plaster ceilings are easier to remove than wood floors, particularly where floors are finished with hard woods. Removing ceilings has the disadvantage of requiring ladders or scaffolding for overhead work.

Table D.8: 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.

Cost Estimate and Assumptions: Assuming two 18" x 36" openings made in the first, second, and third floors and the third story ceiling of the prototypical three-story building by a carpenter and a helper, and including an engineer's time spent coordinating and recording the conditions, the estimated cost ranges between $0.10 and $0.17 per square foot. Patching is not included in the estimate, as it is assumed to be done during seismic retrofitting.

Exposing Crosswall-to-Diaphragm Connections

Rationale: Crosswalls must be connected to diaphragms if they are to provide damping. Older crosswalls are typically constructed of lath and plaster on wood studs; modern walls generally have gypsum wall board in place of lath and plaster. Bottom plates are frequently connected with 16d nails on 12" to 16" centers. Top plates are commonly nailed to joists from above, with toe nailing through joists. Crosswalls may be non-bearing walls added during prior remodeling with minimum connections to floors and ceilings; ceiling connections may be as nominal as spaced 16d nails through lath and plaster into joists. To enhance the crosswalls' ability to accept and absorb energy from the diaphragm, connections should have the capacity to transfer the calculated wall shear capacity to the diaphragms above and below the wall.

Procedure: Exposing crosswall-to-diaphragm connections requires removing lath and plaster or gypsum board from studs and/or removing ceilings. Bottom plate to floor connections can be exposed by making openings approximately 48" long by 12" high at the base of walls. Top of wall-to-joist connections can be exposed by removing a 18" x 18" area of ceiling, or, where plates are nailed to joists from below, by removing wall finishes. Connections to joists framing parallel to walls and perpendicular to walls at each level should be exposed.

Cost Estimate and Assumptions: Assuming two 48" x 12" openings made at the base of walls and two 18" x 36" openings made in ceilings at the top of walls at the first, second, and third stories by a carpenter and a helper, and engineer's time spent coordinating and recording the conditions, the estimated cost ranges between $0.12 and $0.20 per square foot. Patching is not included in the estimate, as it is assumed to be done during seismic retrofitting.

Verifying Wall Cross Section

Rationale: The engineer must know the cross sectional properties of the wall in order to correctly calculate height-to-thickness (h/t) ratios. Walls constructed with a cavity may require two h/t calculations, one for each thickness of wall section. Veneers and the space behind veneers must be subtracted from gross wall thickness. Drilling into the wall can help identify cavities and locate masonry behind veneer where cavity wall construction is suspected or where veneers are noted.

Procedure: Drilling walls to establish cross section depends on the drill operator's ability to note changes in drilling resistance as the drill bit passes through masonry into an air space. A lightweight drill and small diameter (3/8" to 1/2") carbide bit are used to drill through exterior wythes. Holes can be made through mortar to minimize damage to masonry and facilitate patching. The engineer should be on site periodically during the drilling process to review results and vary locations if necessary to obtain better information. The engineer should drill a few of the holes to get a "feel" for the wall construction. A minimum of two tests per wall per floor are made where cavity construction is suspected or veneers noted. As cavities are typically towards the exterior of the wall and veneers are on the exterior of walls, drilling is done from the exterior of the building.

Cost Estimate and Assumptions: Assuming two holes drilled through the exterior wythe at each wall and at each floor by a testing laboratory, a report, and on site observation by an engineer, estimated costs are $0.37 to $0.52 per square foot. Scaffolding accounts for $0.26 per square foot of this estimated cost. Estimated costs include mortar patching.

In-Place Mortar Shear Strength Tests on Exterior and Interior of Perimeter Walls

Rationale: The UCBC requires in-plane shear testing in locations representative of the varying mortar conditions throughout the building. As the entire wall cross section is used when determining in-plane wall strength, knowing the interior wythe mortar shear strength is as important as knowing the exterior wythe mortar shear strength. In contrast to exterior wythes, interior wythes of brick may be more poorly laid up and the mortar shear strength may be weaker. Testing mortar at interior wythes of brick as well as exterior wythes will provide a more representative estimate of mortar strength.

Procedure: A brick is removed from a running bond course as is the mortar from the head joint of an adjacent brick. A ram is inserted into the void left by the removed brick and pushed against the adjacent bricks. Load to the adjacent brick is increased until the first signs of slippage occur. The load at which slippage occurs is recorded and used to determine the shear strength of the mortar. In-plane shear tests are made on exterior and interior wythes of brick. Interior tests have the advantage of not requiring scaffolding. They have the disadvantage of at times requiring finish removal. A minimum of one interior test location on each wall on each floor should be made. Where great variation of mortar strength is encountered, such as very high strength on one wall and very low strength on the next, more tests should be made.

Cost Estimate and Assumptions: Assuming one exterior and one interior in-plane shear tests made on each wall at each floor for a total of 24 tests, each test location requiring removal of lath and plaster over furring over a 4' x 4' square by a carpenter and a helper, production of a test report and an engineer's time spent selecting test locations and coordinating tests, estimated costs are $0.75 to $1.05 per square foot. Scaffolding accounts for $0.33 per square foot of this estimated cost.

Verifying the Presence of Veneer Ties and Spacing by Pacometer Testing

Rationale: Pacometer (i.e., metal detector) testing can provide a non-destructive method of determining the presence of veneer ties and their spacing. Pacometer testing is particularly suited for this task as ties make up virtually the only metal embedded in veneer. It provides an alternative to investigation with a borescope where tie condition is not suspect, or where no space exists between veneer and masonry wall to permit use of a borescope. Testing laboratories often make pacometer readings and summarize findings in a report.

Procedure: A pacometer is passed over the surface of the veneer and the presence of metal chalked on the veneer. A minimum of four areas, approximately four feet by four feet each, at each floor, should have pacometer readings taken. More areas should have readings taken where variation in tie spacing occurs. Results are mapped onto an elevation of the building for review by the engineer.

Cost Estimate and Assumptions: Assuming four areas investigated per floor at only the front wall, estimated costs are $0.13 to $0.18 per square foot. Scaffolding accounts for $0.08 per square foot of this estimated cost. Scaffolding may already be in place from borescope investigation. Estimated costs include production of a test report and an engineer's time spent selecting locations to make pacometer readings.

Verifying Veneer Tie Condition and Spacing by Borescope Testing

Rationale: Veneers have frequently peeled away from structural masonry walls during earthquakes. Veneer ties between veneer and wall must be spaced sufficiently close to adequately tie veneer to the structure. Ties must be in good condition, ties often corrode when wetted by leaks in veneer. Tie spacing and condition can be investigated by inserting a viewing scope into the space behind veneer through holes drilled through veneer. Testing laboratories usually perform this investigation and summarize findings in a report.

Procedure: Holes are drilled through the veneer at each corner of a two foot square area using a light weight drill and 3/4" diameter carbide bit. A borescope is inserted into the hole and tie spacing and condition investigated. A minimum of two areas of veneer per floor are investigated. More areas should be investigated where variation in tie spacing or condition occurs. Where tie condition appears suspect, veneer should be removed. The engineer should be on site to personally view tie condition. Often, only the front of a building has veneer.

Cost Estimate and Assumptions: Assuming two areas investigated per floor at only the front wall, without scaffolding, and on-site observation by an engineer, estimated costs are $0.16 to $0.22 per square foot. Scaffolding accounts for $0.07 per square foot of this estimated cost. Estimated costs include mortar patching, production of a test report and an engineer's time spent selecting test locations and observing ties.

Exposing and Pull-Testing Veneer Ties

Rationale: Observations of veneer delamination after the Northridge Earthquake noted instances where ties remained embedded in the masonry backing while the veneer pulled away from the tie, and other instances where ties tore out of the masonry backing, allowing the veneer to fail as well. Failures may have been caused by poor material conditions, such as rusted ties or cracked masonry, or by inadequate tie strength. While investigating by borescope and pacometer readings can give an indication of tie condition and spacing, exposing ties can provide better observation of material condition. Tie type and spacing may meet UCBC minimum requirements, but improper installation or hidden corrosion may weaken ties. Pull testing exposed ties can provide an indication of tie strength.

Procedure: Veneer is removed from a 12" x 12" area around a tie. The condition of the tie and mortar are closely noted. Ties in good condition are pull tested using a ram. Sufficient ties should be exposed to provide a representative sample of ties with a minimum of two exposures per wall per floor. In addition, veneer should be removed where tie condition appears suspect. The engineer should be on site to personally view tie condition. Often, only the front of a building has veneer. This method only addresses the capacity of the tie in the backing, not the capacity in the veneer.

Cost Estimate and Assumptions: Assuming two areas investigated per floor at only the front wall by a testing laboratory, a report from the laboratory, and on site observation by the engineer, estimated costs are $0.19 to $0.28 per square foot. Scaffolding accounts for $0.07 per square foot of this estimated cost. The cost to patch veneer is not included as patching is assumed to be done during rehabilitation. Scaffolding may already be in place from borescope investigation or pacometer reading.

Identifying Interior Wall Construction

Rationale: Partition wall construction can include clay brick, hollow clay tile, concrete masonry units and plaster or gypsum board on studs. Buildings may contain two or three different construction materials. Partitions constructed of clay masonry or grouted concrete masonry units may significantly contribute to a building's mass and/or stiffness. Often plaster limits the ability to identify wall construction. Chipping off plaster can expose wall material. Voids encountered while drilling into walls can determine if hollow clay tile is used or if concrete masonry units are partially or wholly ungrouted. Section A111.3.1 of the 1994 UCBC requires that walls be wood framed to qualify as crosswalls.

Procedure: Identifying interior wall construction utilizes fairly unsophisticated investigative techniques which an engineer can usually perform. The engineer should walk through the building and note any variation in wall types, as each wall type requires identification. Two walls per floor should be identified at a minimum. An engineer wielding a geologist's pick can easily chip off plaster to expose the masonry substrate. One 18" x 18" location is sufficient to identify substrate material and profile. A lightweight drill and small diameter carbide bit (1/8" to 1/4") are used to drill through masonry when investigating for hollow clay tile or ungrouted cells in concrete masonry units. The drill operator must note changes in drilling resistance as the drill bit passes through masonry into an air space. Holes can be drilled at areas where plaster has been removed. As concrete masonry units may be partially grouted, holes should be drilled at 8" increments along a horizontal plane to determine grout spacing. Engineers can easily identify a wood stud wall sheathed with gypsum wall board by rapping on the wall with their knuckles. Areas between studs will make a hollow sound, areas at studs will make a solid sound. Stud location can be confirmed by drilling a small diameter hole through finish material into studs or by a stud finder. Stud spacing can be confirmed in a similar matter.

Cost Estimate and Assumptions: Assuming an engineer makes a walkthrough survey of the walls and investigates two walls per floor (including chipping and drilling) and records results, estimated costs are $0.05 to $0.07 per square foot. Costs for patching chipped plaster are not included as patching is assumed to be done during seismic retrofitting.

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, 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% of construction costs to as much as 10% of construction costs.

Peer Review/Plan Check

Peer review provides the opportunity for an independent review of the proposed seismic rehabilitation. Peer review is performed by experienced engineers, at various stages in the evaluation and design process. Early on in a project, a general review of evaluation and design methodology can be made. Reviews can occur as the plan progresses, often at the 100% design development and/or construction document phase. Plan check provides a third type of review. Here plans are submitted to the governing municipality which then checks them for conformance to governing codes. Currently some areas of the country mandate plan check while others do not.

Design Methodology and Criteria Review

Rationale: Peer review at the beginning of a project can help ensure that the evaluation and design methodology selected for the project are consistent with the intended performance objective.

Procedure: A single structural engineer or a peer review panel composed of three or four structural engineers convenes with the engineer of record to review the methodology the engineer of record proposes to use when evaluating and designing the building. The engineer of record presents the building, the owner's selected performance objective, the proposed evaluation and design methodologies, and the rationale for their selection to the reviewers. The reviewing panel and engineer of record conclude the meeting when they concur on the methodology. In some cases, a follow-up meeting and analysis by the engineer may be necessary to reach concurrence.

Cost Estimate and Assumptions: Assuming a review panel of three structural engineers and the engineer of record meeting for approximately 2 hours, and including time for preparation, transportation and a summary memo, estimated costs for this level of peer review are \$0.21 to \$0.31 per square foot.

Schematic Design Review

Rationale: Peer review of the evaluation findings and schematic or conceptual retrofit design can help ensure that the retrofit design address identified seismic deficiencies and meets the performance objective. In some cases, a follow-up meeting and analysis by the engineer may be necessary to reach concurrence.

Procedure: The same engineers which composed the design methodology and criteria panel meet with the structural engineer of record to review the schematic design developed to address deficiencies identified in the seismic evaluation. The engineer of record presents the findings of the seismic evaluation and the proposed schematic design (or designs), and explains how the design addresses the deficiencies. The reviewing panel and engineer of record conclude the meeting when they concur that the schematic design addresses the deficiencies and will meet the performance objective.

Cost Estimate and Assumptions: Assuming a review panel of three structural engineers and the engineer of record meeting for approximately 4 hours, and including time for preparation and transportation, estimated costs for this level of peer review are \$0.28 to \$0.41 per square foot.

Construction Document Review

Rationale: Peer review of the completed construction documents can help ensure that the final design has developed the schematic design concepts to a construction document level and that the design addresses identified seismic deficiencies. Review can include suggestions for design modifications which may decrease construction costs or improve performance.

Procedure: This review is principally a review of the final design and calculations, and can include review of building and member loading, member sizing, detailing and constructability. Suggested improvements in the design and specifications may be made. Completed plans and specifications are reviewed by an experienced structural engineer. After completing the review, the reviewing engineer and engineer of record meet to discuss the reviewer's comments and work out solutions to design issues. Often review comments are submitted to the engineer of record for written response. The review process is complete when the reviewing engineer and engineer of record concur that the final design addresses the identified seismic deficiencies and will meet the specified performance objective.

Cost Estimate and Assumptions: Assuming a single structural engineer reviews the plans and specifications, meets with the engineer of record and summarizes review comments in written form, estimated costs are $0.53 to $0.96. Note that construction document review costs are greatly affected by the size and complexity of a project. Review for small and simple projects may cost one third of the estimated costs in this report while large complex projects may cost three or more times the estimated costs. For some projects, reviews may occur at earlier stages in the design process, such as at the end of the design development phase. Such additional reviews will have an associated cost.

Plan Check

Rationale: A plan check review can help ensure that construction documents, specifications, design criteria and calculations conform to applicable codes and standards.

Procedure: Final construction documents, calculations and design criteria are submitted to the appropriate plan check agency. This is often the city or county building department. A plan check engineer reviews the documents for conformance to applicable codes. Structural calculations are reviewed, including loading and seismic coefficients. Elements which do not conform to the applicable code and errors in calculations are summarized in written form. The engineer of record makes the appropriate corrections to the construction documents and calculations, and resubmits them for final review.

Cost Estimate and Assumptions: Plan check costs are based upon 1994 Uniform Building Code, which sets the fee for plan check review as a function of construction cost, as with permitting fees. Fees incrementally increase as construction cost increases. Table D.3 of this appendix provides plan check fees.

Field Review

Field review provides for quality control in the construction process. It includes inspection by trained, certified special inspectors, material testing by testing laboratories, and site visits by the engineer of record. Depending on the type of construction, the required amount of special inspection and testing will vary for each job. Construction involving concrete or shotcrete placement requires special inspection of reinforcing and concrete placement, and slump tests and compression tests of concrete. Steel welding procedures should be reviewed for conformance to applicable welding standards, and welds should be inspected. Often the engineer must decide what elements of construction require inspection and testing, and must write the inspection and testing specifications. This is particularly true when the retrofit design employs new or innovative construction techniques, such as center coring or grout injection. The engineer must also decide on the appropriate level of site visits. In California, site visits are typically made on a monthly basis and/or at major steps in the construction, such as prior to concrete pours or shotcrete placement or after structural steel placement or diaphragm nailing. More frequent site visits can enhance the building's reliability to withstand earthquakes by better ensuring the construction conforms to the design intent.

Special Inspection and Testing

Cost estimates for special inspection and testing are given for nine different wall enhancement techniques in Section D.4, Rehabilitation Methods, of this report. Other elements of construction commonly found in seismic retrofit construction, such as diaphragm nailing or tension tie connections, require special inspection and/or testing, but are not included in this discussion of costs.

Site Visits by the Engineer of Record

Rationale: Site visits provide the engineer of record firsthand knowledge of the construction process. The engineer can review the construction for general conformance with the design intent and can alert the contractor to non-conforming conditions. The engineer can judge the quality of the contractor's work, and can judge the constructability of the design. The latter can prove extremely helpful in future retrofit designs. Site visits can also help develop a good working relationship with the contractor, which should aid problem solving during the course of the job.

Procedure: The engineer decides upon the appropriate level of site visits. At a minimum, site visits are made on a monthly basis, or at critical phases in the construction process. An enhanced level of site visits would require weekly visits during critical portions of the construction, and bi-weekly visits thereafter.

Cost Estimate and Assumptions: Assuming construction lasting 6 months, with monthly site visits by a structural engineer, and providing time for travel and site visit reports, the estimated cost of a minimum level of site visits for the prototypical building is $0.22 to $0.41 per square foot. Assuming an enhanced level of site visits for the same duration of construction, with 3 months of weekly site visits and 3 months of bi-weekly site visits by a structural engineer, and providing time for travel and site visit reports, the estimated cost of an enhanced level of site visits for the prototypical building is $0.81 to $1.24 per square foot.

D.3 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 retrofit methodologies. Estimated cost of rehabilitating specific activities are presented in Table D.5. Estimates are based on costs reported in FEMA 156 (1988) and are adjusted to 1996 cost for construction in San Francisco.

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 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.

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 to 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 and very high areas of seismicity are presented in Table D.6. Estimates are based on methodology presented in FEMA 156 (1994) for construction in San Francisco. While San Francisco is in an area of very high seismicity, as defined by the FEMA 156 (1994) document, the methodology considers seismicity and geography separately in determining estimated cost. Thus, it was possible to estimate costs for construction in San Francisco as if it were an area of low seismicity.

D.4 Rehabilitation Methods

Traditional Out-of-Plane Bracing Alternatives

Wall out-of-plane failures occur with greater frequency than in-plane failures, yet more techniques have been developed to address in-plane failure than out-of-plane failure. Two 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.

Diagonal Bracing

Rationale: Walls with excessive height-to-thickness (h/t) ratios may experience out-of-plane failure. Diagonal bracing attempts to reduce the h/t ratio by introducing a horizontal bracing line between the floor and roof level.

Procedure: A steel channel section is attached to the interior face of the masonry wall. Diagonal brace elements, such as angles, are attached to the channel and strutted back up to the roof or floor joists. Bracing elements are located at about 6' on center along the length of the channel. The elevation of the channel is selected such that the h/t ratio of the wall below and above the channel are within acceptable limits. Bracing element spacing is such that the channel can span in flexure between braces. Diagonal bracing members can be designed to bolt onto the channel in the field, which will simplify installation.

Cost Estimate and Assumptions: Assuming an MC 6 x 12 channel is placed along each wall at each floor, and braced with 2" x 2" x ¼" angles at 6' on center, the estimated cost of the enhancement technique for the prototype building is $3.29 to $4.91 per square foot.

Strongbacks

Rationale: Strongbacks address excessive height-to-thickness ratios by helping to brace the walls against out-of-plane forces. Strongbacks carry the wall out-of-plane load in flexure to the diaphragms above and below.

Procedure: A wood or steel section is placed vertically along the interior face of the building. The masonry is attached to the member along the member's length with brackets, which fasten to the masonry, at approximately 4' to 6' on center. The base and top of the strongback is securely attached to the floor diaphragms above and below. Tube sections are often used as strongbacks. Wood sections may be quite large when used as strongbacks.

Cost Estimate and Assumptions: Assuming strongbacks made of 4" x 4" x ¼" tube steel sections are introduced at 6 feet on center along solid walls and between windows at perforated walls, the estimated cost of the enhancement technique for the prototype building is $5.39 to $7.87 per square foot.

Wall Enhancement Methods

Several wall rehabilitation methods are available for enhancing seismic performance of walls. The method most appropriate for a building will depend upon a number of factors, cost among them. This section describes nine enhancement techniques and provides a range of estimated costs for each technique in Table D.8. Estimates are based on a prototypical three-story, 40' x 80', unoccupied commercial building, with floors and roofs constructed of wood sheathing over wood joists. Figures D-1 through D-9 illustrate the enhancement techniques and describe the scope of work from which a base cost estimate for each technique was made. Section D.5 contain the base cost estimates. Base cost estimates are made for a generic facade module that is a 20' width of three-story end wall, and are given in $/square feet of wall area. Figures D-10 and D-11 illustrate the wall area used to calculate the base estimate, as well as the total wall area over which the enhancement applies. The base estimate is converted into $/plan square feet by multiplying it by the ratio of the area of the wall surface over which the enhancement applies to the area of wall surface used in the base estimate, and dividing the product by the total plan area of the prototypical building. Because the prototypical building is small and costs are generally higher for small buildings, and as estimated labor costs are based on union labor rates, estimated base costs reflect the higher end of a range of probable costs. To calculate the estimated range for each technique, estimated costs were increased by 10% and decreased by 40%. Testing and inspection costs are taken from Section D.6.

Grout and Epoxy Injection

Rationale: Hollow walls, walls with cavities or walls with numerous voids often do not have the capacity to resist in-plane or out-of-plane seismic loading. Veneers often peel away from structural walls during earthquakes. Injecting voids with grout or epoxy strengthens wall in-plane shear strength and flexural strength. Out-of-plane resistance may be enhanced by increasing wall effective thickness, thereby reducing the h/t ratio.

Procedure: Injection ports are drilled into voids at regular spacing. Holes and voids are thoroughly flushed with water prior to injecting grout. Loose mortar is repointed to prevent grout from leaking out. Grout is injected, starting from the lowest ports and working upward in closed cell cement masonry unit walls, and laterally in cavity walls or behind veneers. As grout begins to flow out of higher ports or lateral ports, the injection port is plugged and injection continues through adjacent ports. Injection may be done from the exterior or interior of the building. Void flushing and grout injection require continuous special inspection. A testing laboratory verifies that grout has filled voids and grout achieves the specified compressive strength.

Cost Estimate and Assumptions: Assuming a 12 inch concrete masonry unit wall constructed of closed cell standard block, and injecting a cementitious grout from the interior of the building, the estimated cost of the enhancement technique for the prototype building is $8.63 to $15.81 per square foot. The estimated cost of special inspection and testing is $0.35 per square foot.

Surface Coatings

Rationale: Masonry walls often lack sufficient flexural and/or shear strength to resist in-plane and/or out-of-plane seismic loading. Surface coatings provide a method of increasing wall in-plane flexural strength and shear strength, out-of-plane flexural strength, and inelastic deformation capacity for in-plane loading.

Procedure: Loose paint, plaster, dirt, etc. is removed from the wall surface. Loose masonry is reset and deteriorated or cracked joints repointed. The surface is then washed with water, and a 19 gauge, ½" x ½" mesh hardware cloth attached to the masonry with ¼" diameter expansion anchors spaced at 16 inches on center. A layer of cementitious coating is applied over the mesh. Coating can be applied to a single side or both sides of a wall. When coating on the inside of the wall, floors and roof must be cut away from the masonry wall and reattached through the coating. Final wall preparation, hardware cloth installation and coating application require special inspection.

Cost Estimate and Assumptions: Assuming a clay brick masonry wall with a surface coating applied to interior and exterior surfaces, the estimated cost of the enhancement method for the prototypical building is $12.71 to $23.31 per square foot. The estimated cost of special inspection and testing is $0.66 per square foot. Note that final estimated costs are based on the assumption that fastening hardware cloth to the masonry will cost $8.00 per square foot, as opposed to $14.50 per square foot shown in the estimate in Section D.5.

Adhered Fabrics

Rationale: As with surface coatings, adhered fabrics provide a method of increasing wall in-plane flexural strength and shear strength, out-of-plane flexural strength, and inelastic deformation capacity for in-plane loading.

Procedure: Loose paint, plaster, dirt, etc. are removed from the wall surface. Sandblasting may be required to completely remove unacceptable material from the wall surface. Loose masonry is reset and deteriorated or cracked joints repointed. Epoxy is evenly applied to the masonry surface, followed by the fabric, which is embedded into the epoxy. The bottom of the fabric is anchored to the foundation with a steel angle, and a second coat of epoxy is applied over the fabric. Fabric can be applied to a single side or both sides of a wall. Fabric applied to the interior side of a wall can be fit around joists, floors must be cut back to permit fabric continuity between floors. On exterior walls, the cured epoxy surface is painted with a paint which protects against ultraviolet light. Surface preparation, fabric application and angle installation require special inspection.

Cost Estimate and Assumptions: Assuming a clay brick masonry wall with fabric applied to interior and exterior surfaces, the estimated cost of the enhancement technique for the prototype building is $11.53 to $21.14 per square foot. The estimated cost of special inspection and testing is $1.68 per square foot.

Shotcrete Overlay

Rationale: Shotcrete overlays add substantial flexural and shear strength to masonry walls, for both in-plane and out-of-plane loading. Overlays are often designed to resist the entire lateral load when placed against a masonry wall. Shotcrete overlays are very similar to cast-in-place concrete shear walls. On the West Coast, shotcrete has been used extensively to seismically rehabilitate buildings.

Procedure: Wall surfaces are prepared by removing loose paint, plaster, dirt etc. Usually wire brushing of the surface is sufficient preparation. Rebar dowels are set into the wall at approximately 3' centers. Dowels tie the existing masonry wall to the shotcrete. Horizontal and vertical steel reinforcing, similar to that used in concrete shear walls, is tied to the dowels. Shotcrete is sprayed onto the wall. The thickness varies depending on strength requirements, but it is usually not thinner than six inches. New foundations, attached to the existing foundation, are frequently required under shotcrete overlays. Shotcrete may be applied to either face of a wall. When shotcrete is applied to the inside of the wall, the floors and roof must be cut away from the masonry wall and reattached to the new shotcrete. The quality of the shotcrete is extremely dependent on the skill of the person applying the shotcrete. Nozzle operators demonstrate their skill by applying shotcrete to a test panel which closely represents the wall areas most difficult to apply shotcrete. A special inspector observes the test panel shotcrete placement. The testing laboratory cores the test panels to observe the quality of the shotcrete placement and takes cores which they test for compressive strength. A special inspector inspects dowel installation, reinforcing steel placement and continuously observes shotcrete placement. The testing laboratory verifies proper dowel installation by torque or pull testing dowels, and checks shotcrete compressive strength by testing shotcrete cores taken from the finished walls.

Cost Estimate and Assumptions: Assuming a 6" thick layer of shotcrete with 0.25% horizontal and vertical reinforcement, and limited foundation strengthening, the estimated cost of the enhancement technique for the prototype building is $7.20 to $13.20 per square foot. The estimated cost of special inspection and testing is $0.43 per square foot.

Reinforced Cores

Rationale: Preservation of ornate exterior or interior finishes may preclude the use of surface applied enhancement techniques. In these cases, reinforced cores provide an enhancement alternative. Reinforced cores increase wall in-plane flexural and shear strength, out-of-plane shear strength, and in-plane inelastic deformation capacity.

Procedure: Four-inch diameter or larger cores, centered in the wall, are drilled from the top of the wall and extend into the foundation. Spacing varies depending on project specific requirements; often cores are located at each end of piers and at six feet on center elsewhere. The entire length of the wall is cored. Reinforcing steel is centered in the core where upon the core is filled with polyester grout. Prior to grouting, masonry joints which might leak grout are repointed. Cores may be drilled using either a wet or dry drilling process. Cores drilled wet must sufficiently dry before placing grout. The exterior is repointed to help prevent leaks from the grout and/or drilling water. Any water which leaks out of the interior face of the brick will run down the furring space behind the plaster. Vacuum ports can be drilled through the interior plaster if necessary to remove the water. A special inspector inspects cores, reinforcing steel placement, grout mixture and grout placement. A testing laboratory tests the grout samples for appropriate compressive strength.

Cost Estimate and Assumptions: Assuming 4" diameter cores drilled at the ends of piers and filled with polyester grout, the estimated cost of the enhancement technique for the prototype building is $13.90 to $25.48 per square foot. The estimated cost of special inspection and testing is $0.17 per square foot. Note that final estimated costs are based on the assumption that cores will cost $150 per linear foot as opposed to $220 per linear foot, as shown in the estimate in Section D.5.

Post-Tensioned Masonry

Rationale: Preservation of ornate exterior or interior finishes may preclude the use of surface applied enhancement techniques and reinforced cores may not sufficiently strengthen walls. Post-tensioned masonry can increase wall in-plane flexural and shear strength, and out-of-plane flexural strength more than can reinforced cores.

Procedure: The procedure for post-tensioning masonry is very similar to the procedure for reinforced cores. Cores are drilled from the top of the into the foundation. As with reinforced cores, spacing varies depending on project specific requirements, cores are often located at each end of piers and at six feet on center elsewhere. The entire length of the wall is cored, and exterior joints which might leak grout are repointed. Any water which leaks out of the interior face of the brick will run down the furring space behind the plaster. Vacuum ports can be drilled through the interior plaster if necessary to remove the water. A tendon is placed in the core and anchored to the foundation with primary grout. After the primary grout has cured, the tendon is stressed, and the core may be filled with secondary grout. A special inspector inspects cores, tendon placement and stressing, grout mixture and grout placement. A testing laboratory test the grout samples for appropriate compressive strength.

Cost Estimate and Assumptions: Assuming 4" diameter cores drilled at the ends of piers and filled with polyester grout, the estimated cost of the enhancement technique for the prototype building is $14.94 to $27.31 per square foot. The estimated cost of special inspection and testing is $0.22 per square foot. Note that estimated costs are based on the assumption that cores will cost $150 per linear foot as opposed to $220 per linear foot, as shown in the estimate in Section D.5.

Infilled Openings

Rationale: Infilled openings provide an inexpensive but often aesthetically unpalatable method to increase the shear strength of the wall.

Procedure: Openings are infilled with masonry of size and strength similar to that of the original masonry. Masonry lay-up should match the original lay-up. Mortar at jambs and sills should be removed, and new masonry should interlace with existing masonry. Mortar strength should match that of the original mortar. No special inspection is required.

Cost Estimate and Assumptions: Assuming a three-wythe wall of standard clay brick, 13 inches thick, the estimated cost of the enhancement technique for the prototype building is $2.65 to $4.85 per square foot.

Enlarged Openings

Enlarging openings can alter the behavior of a pier from a shear-controlled mode to a rocking-critical mode. This will reduce the in-plane yield strength of the wall but will increase the deformation capacity of the wall.

Procedure: Openings are enlarged by sawcutting masonry below the sill and in line with the jambs. The masonry is then removed. Non-structural infills such as studs with exterior siding and interior gypsum board may replace removed masonry. No special inspection is required.

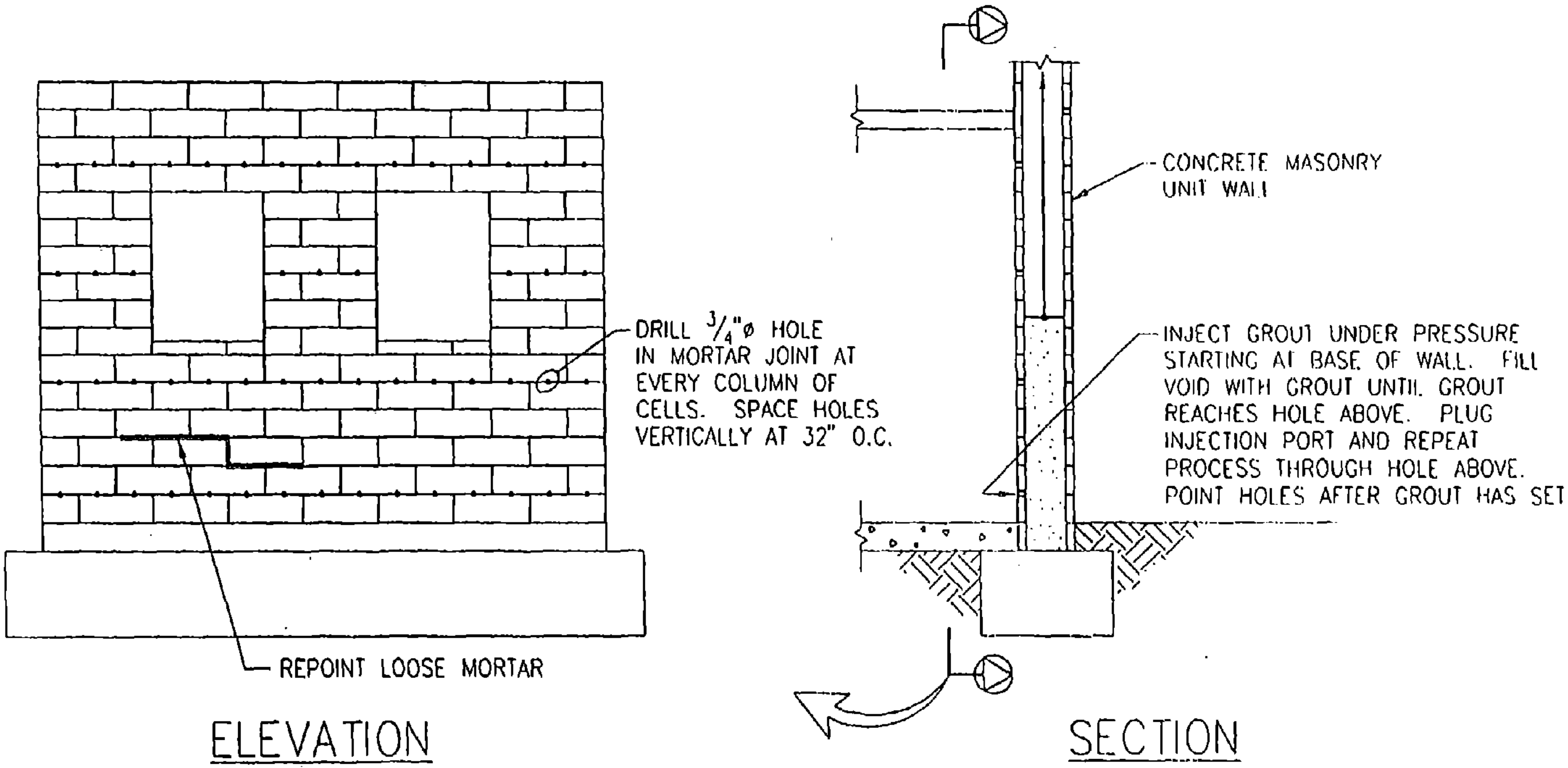
Cost Estimate and Assumptions: Assuming a three-wythe wall of standard clay brick, 13 inches thick, the estimated cost of the enhancement technique for the prototype building is $2.81 to $5.15 per square foot.

Steel Bracing

Rationale: Steel braced framing decreases the in-plane lateral load demand on existing masonry walls. Braces placed on or near walls share lateral loads with the masonry wall based on their relative rigidity.

Procedure: Steel braced frames are inserted into the building. Braces may be constructed of a variety of steel shapes, including tube shapes, wide flanges, pipes and angles. It is often necessary to add collectors to bring diaphragm shear forces to the braces. Braces must attach to, or pass through, existing floor construction. Braces often require new foundations to transfer load from the brace to the ground. Chevron, diagonal or X-brace configurations may be used. Foundation reinforcing steel and concrete placement require on-site special inspection, as does field welding. Steel mill certificates and welding procedure specifications require review by a special inspector. Shop welding requires special inspection. Concrete must be tested for compressive strength.

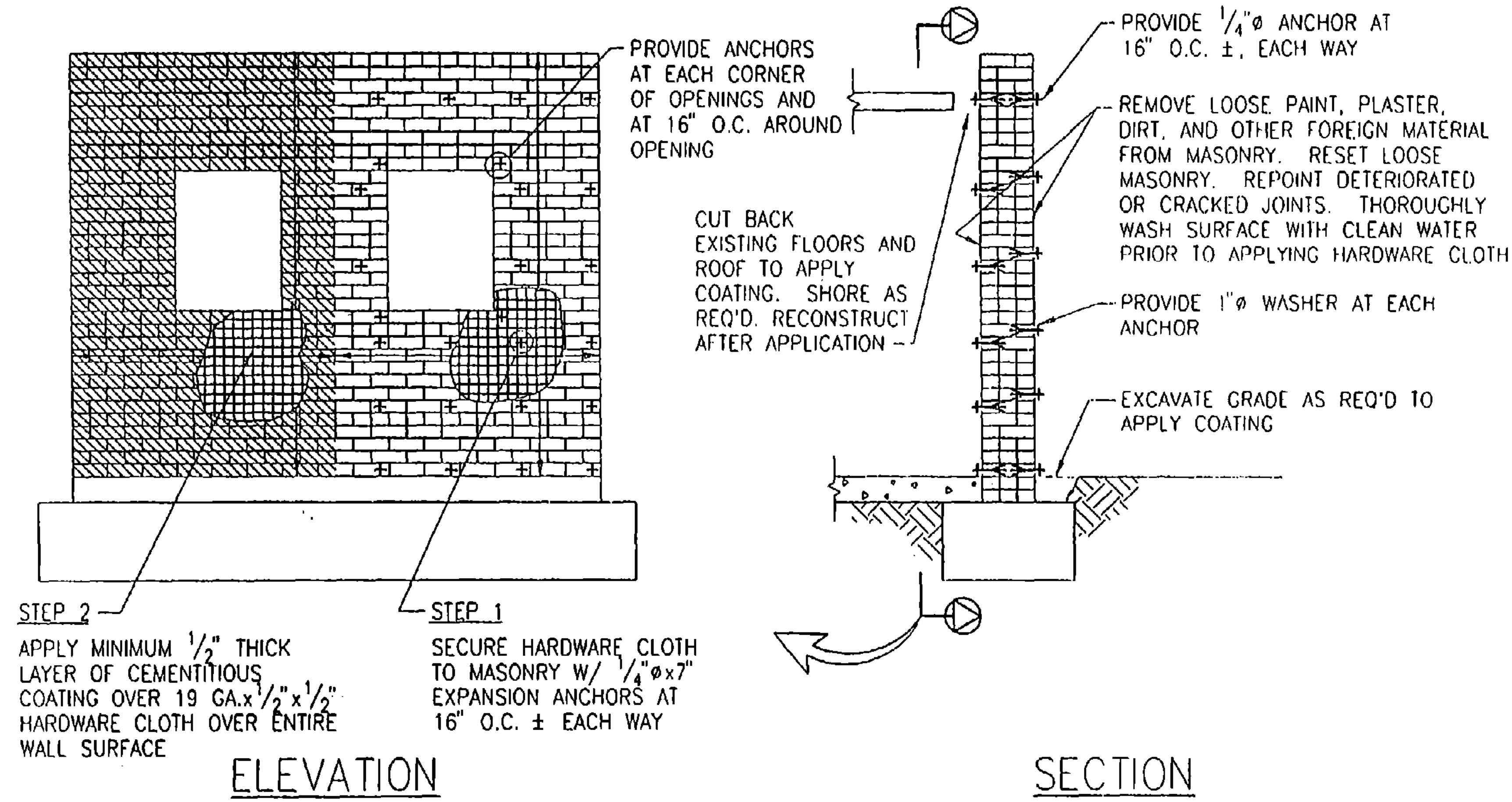
Cost Estimate and Assumptions: Assuming bracing as shown in Figures D-9 and D-11 constructed on each end of the prototypical building, and limited foundation strengthening, the estimated cost of the enhancement technique for the prototypical building is $9.29 to $17.03 per square foot. The estimated cost of special inspection and testing is $0.35 per square foot.



NOTES:

1. INJECTION MAY BE DONE FROM EITHER THE EXTERIOR OR INTERIOR SIDE OF THE WALL.
2. THOROUGHLY FLUSH INSIDE OF WALL WITH WATER 24 HRS. PRIOR TO INJECTION.
3. GROUT COMPOSITION IS 1 PART TYPE I OR II CEMENT, $^1/_2$ PART FLYASH, $^1/_2$ PART TYPE S LIME AND 4 PARTS CLEAN SAND. IMMEDIATELY CLEAN GROUT OFF OF WALL SURFACE.
4. ON-SITE INSPECTOR SHALL CONTINUOUSLY INSPECT ENTIRE FLUSHING AND GROUTING PROCEDURE AND TEST GROUT FLOW WITH DRILLED VERIFICATION HOLES AND/OR NONDESTRUCTIVE METHODS.
5. TESTING LABORATORY SHALL TEST GROUT SAMPLES FOR COMPRESSIVE STRENGTH.
6. HORIZONTAL SPACING OF INJECTION PORTS IS BASED ON THE ASSUMPTION THAT CLOSED END MASONRY UNITS ARE USED. WHERE OPEN END UNITS ARE USED, SPACING MAY BE INCREASED.
7. ENHANCEMENT PROCEDURE MAY BE USED TO FILL VOIDS IN CAVITY WALLS, WALLS WITH RUBBLE CORES AND BEHIND VENEERS.

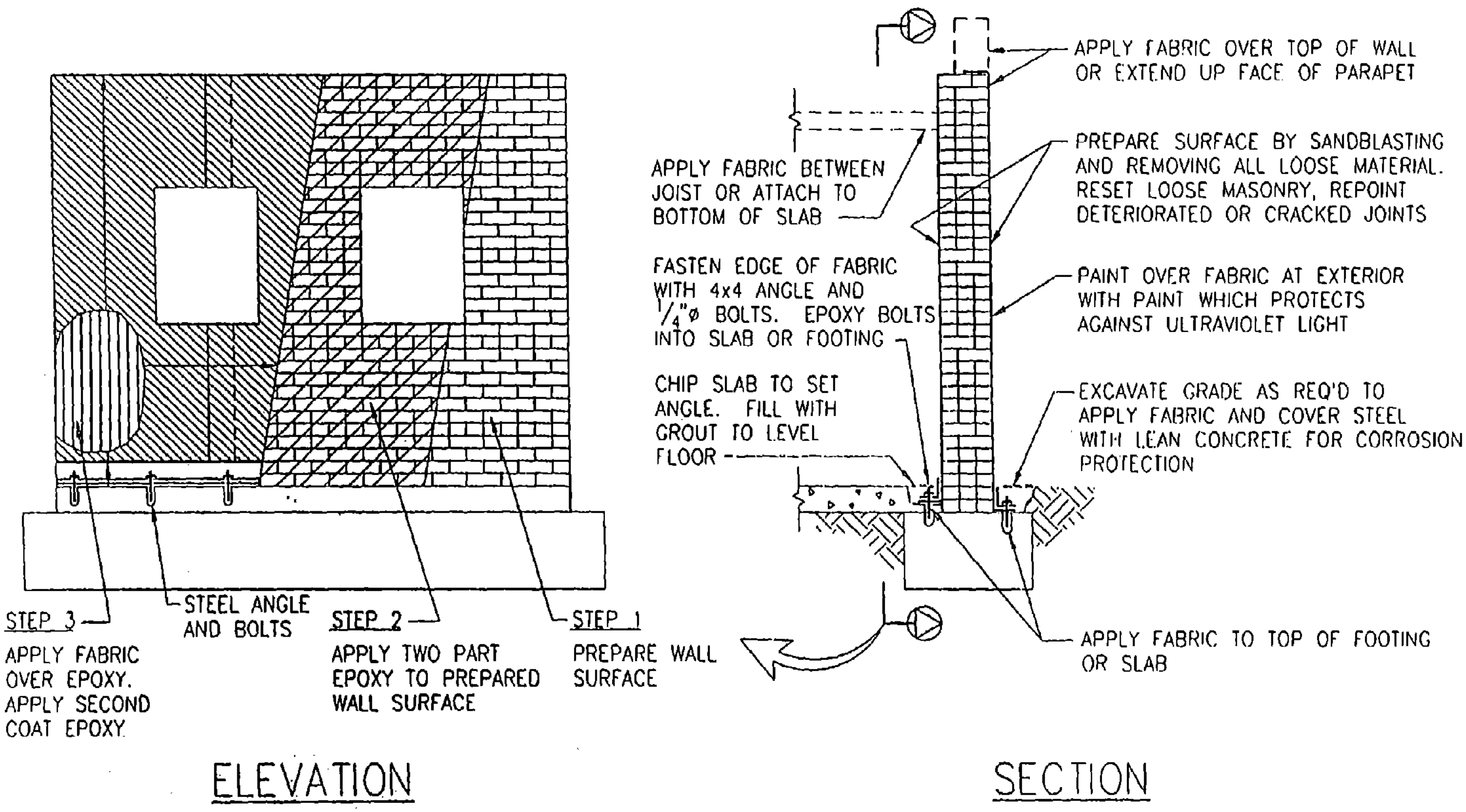
Figure D-1: Grout and Epoxy Injection



NOTES:

1. COAT BOTH INTERIOR AND EXTERIOR SURFACES.
2. COATING COMPOSITION IS 1 PART TYPE I OR II PORTLAND CEMENT TO 3 PARTS CLEAN SAND. ADD CLEAN WATER TO ACHIEVE THE CONSISTENCY OF PLASTER.
3. ON-SITE SPECIAL INSPECTOR SHALL INSPECT FINAL SURFACE PREPARATION, HARDWARE CLOTH INSTALLATION AND COATING APPLICATION.
4. TESTING LABORATORY SHALL TEST CEMENTITIOUS PLASTIC SAMPLES FOR COMPRESSIVE STRENGTH.

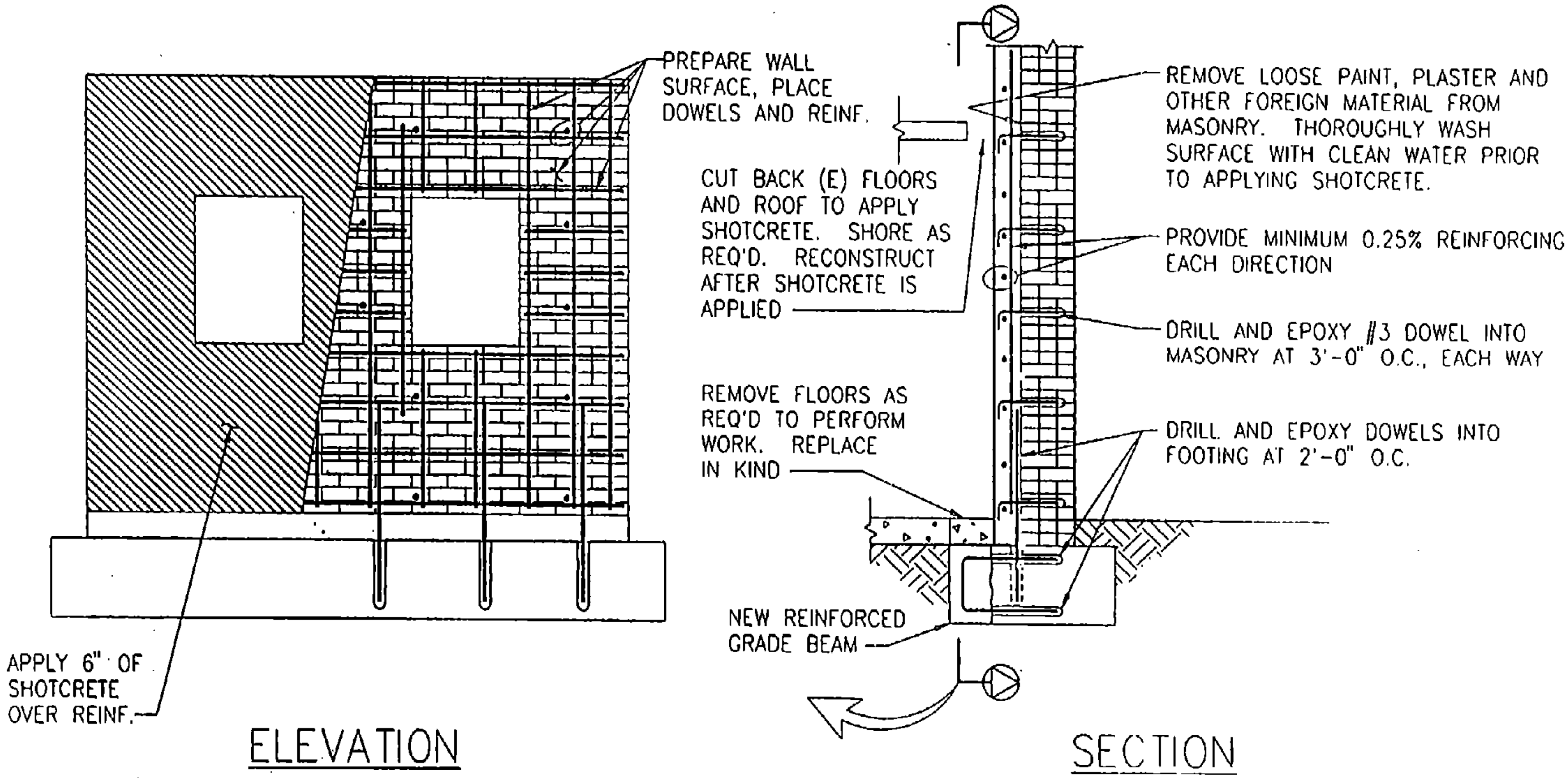
Figure D-2: Surface Coatings



NOTES:

1. COAT INTERIOR AND EXTERIOR SURFACES.
2. ON-SITE SPECIAL INSPECTOR SHALL INSPECT SURFACE PREPARATION, FABRIC APPLICATION AND ANGLE INSTALLATION.
3. FABRIC IS HIGH STRENGTH FIBER MATERIAL, SUCH AS FIBERGLASS OR FIBER COMPOSITE

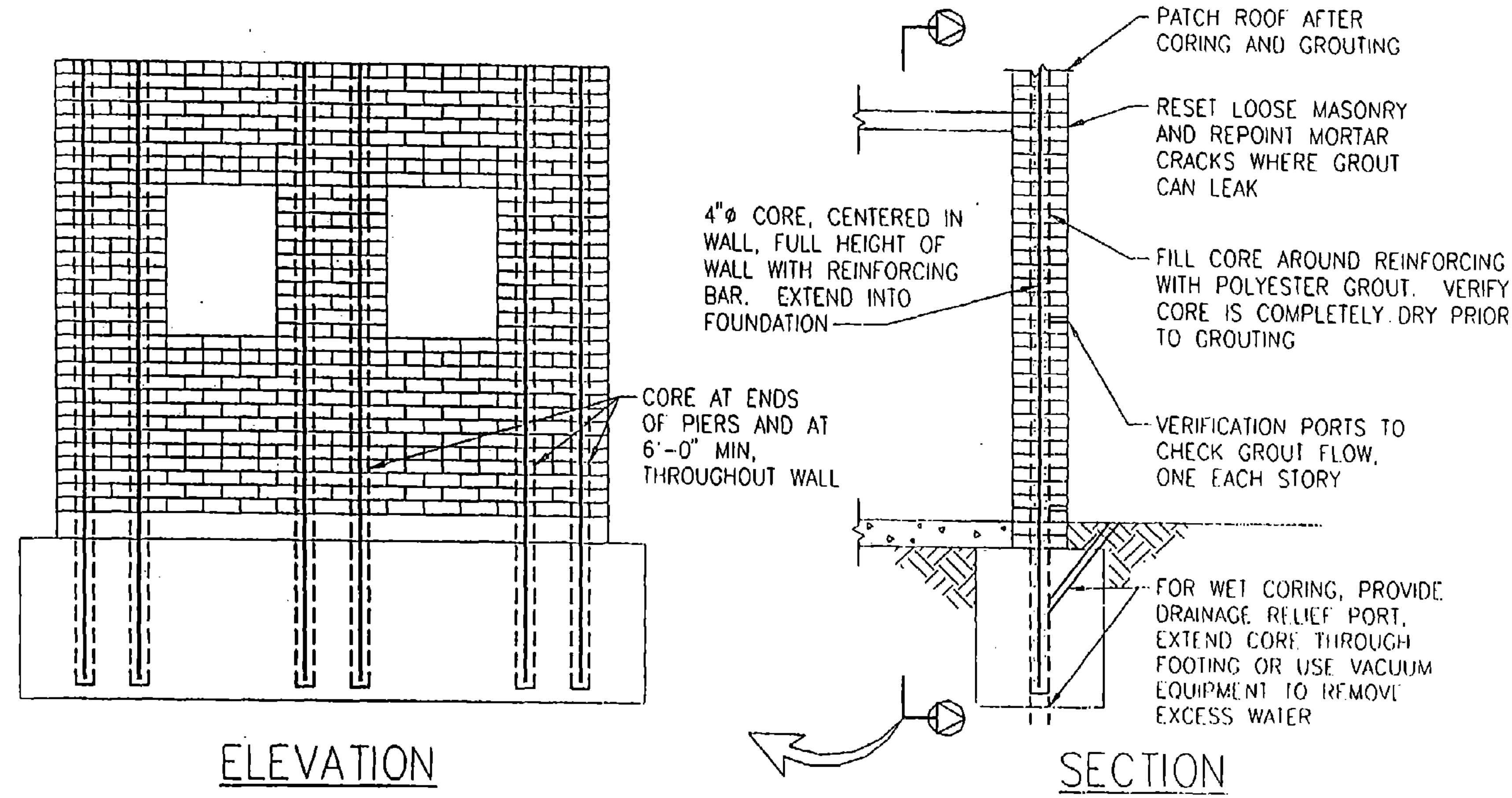
Figure D-3: Adhered Fabrics



NOTES:

1. A TEST PANEL MOCKUP SHALL BE MADE. SPECIAL INSPECTOR SHALL INSPECT TEST PANEL REBAR AND SHOTCRETE PLACEMENT.
2. ON-SITE SPECIAL INSPECTOR SHALL INSPECT ALL DOWEL AND REINFORCING STEEL INSTALLATION, CONCRETE PLACEMENT AND SHOTCRETE PLACEMENT.
3. TESTING LABORATORY SHALL TORQUE TEST 10% OF DOWELS AND TEST CONCRETE FOR STRENGTH.

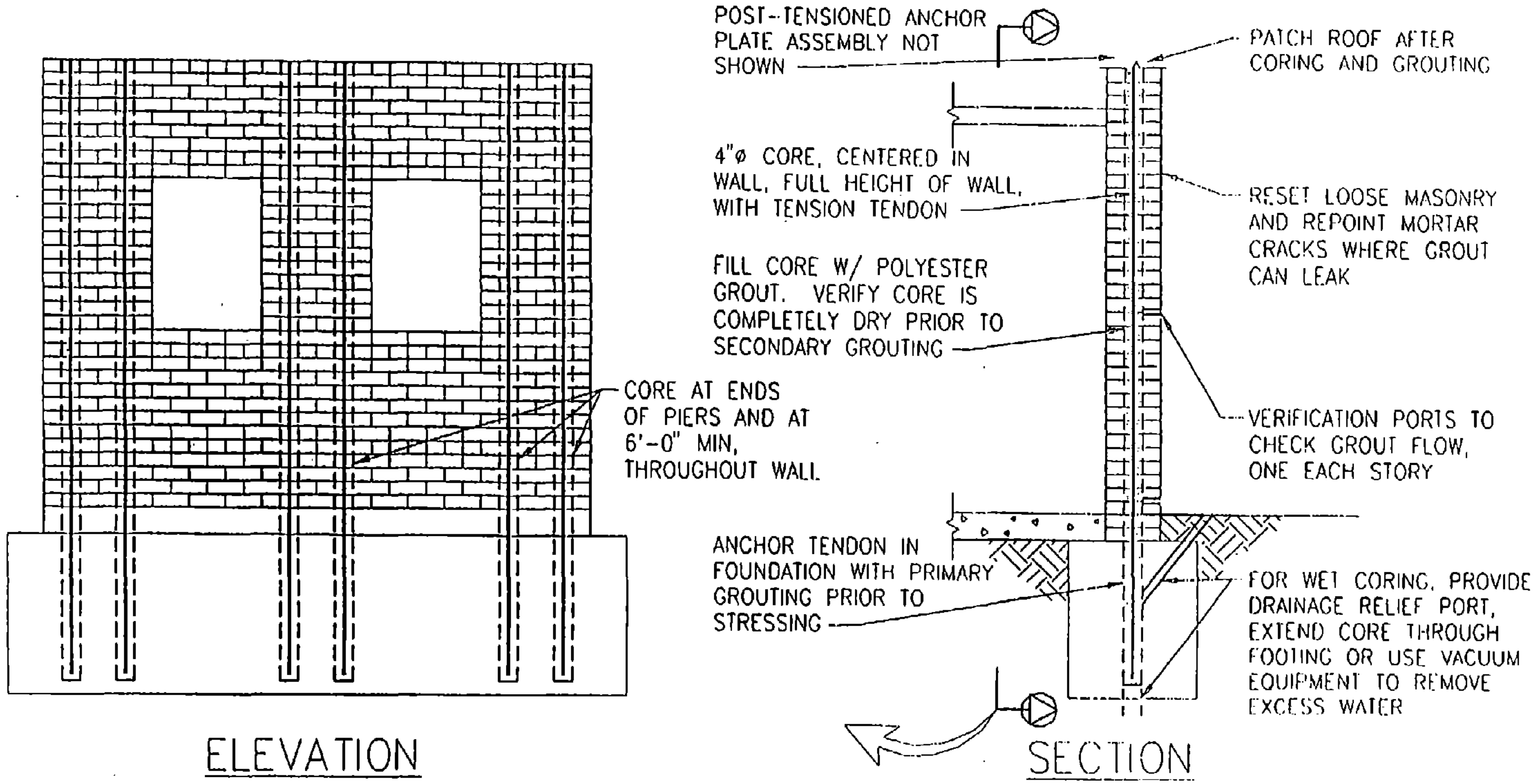
Figure D-4: Sholcrete Overlay



NOTES:

1. ON-SITE SPECIAL INSPECTOR SHALL INSPECT CORES, REINFORCING STEEL, GROUT MIXTURE AND PLACEMENT.
2. TESTING LAB SHALL TEST GROUT SAMPLES FOR COMPRESSIVE STRENGTH.
3. CORES MAY BE DRILLED WITH WET OR DRY DRILLING EQUIPMENT. FOR DRY CORING, PROVIDE DUST PROTECTION. FOR WET CORING, VERIFICATION HOLES ALSO SERVE AS DRAINAGE RELIEF PORTS.
4. NON-SHRINK CEMENTITIOUS GROUT MAY BE USED IN LIEU OF POLYESTER GROUT. FLUSH WITH WATER PRIOR TO GROUTING.

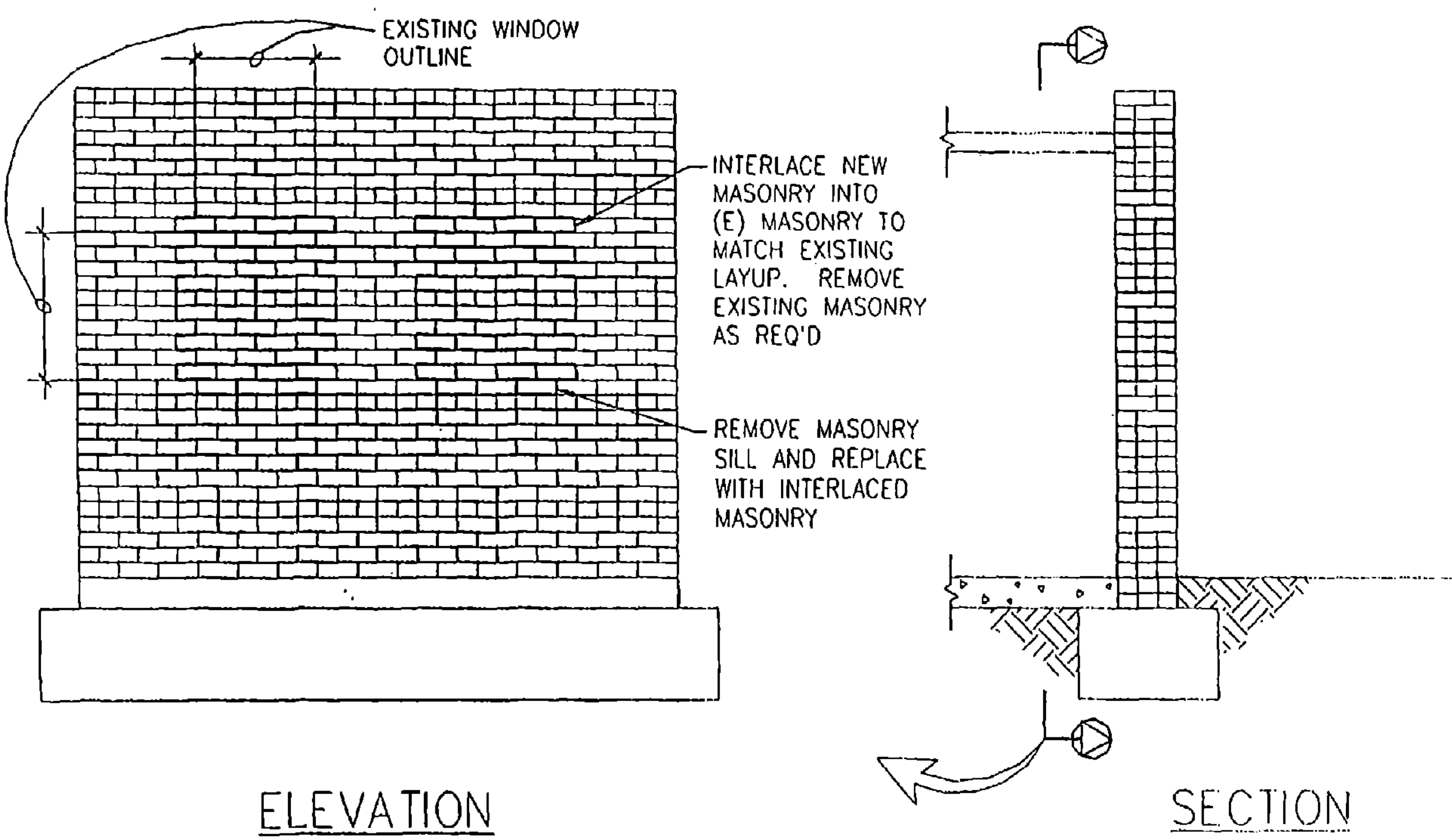
Figure D-5: Reinforced Cores



NOTES:

1. ON-SITE SPECIAL INSPECTOR SHALL INSPECT CORES, PLACEMENT AND STRESSING OF TENDONS, GROUT MIXTURE AND PLACEMENT.
2. TESTING LAB SHALL TEST GROUT SAMPLES FOR COMPRESSIVE STRENGTH.
3. CORES MAY BE DRILLED WITH WET OR DRY DRILLING EQUIPMENT. FOR DRY CORING, PROVIDE DUST PROTECTION. FOR WET CORING, VERIFICATION HOLES ALSO SERVE AS DRAINAGE RELIEF PORTS.
4. NON-SHRINK CEMENTITIOUS GROUT MAY BE USED IN LIEU OF POLYESTER GROUT. FLUSH WITH WATER PRIOR TO GROUTING.

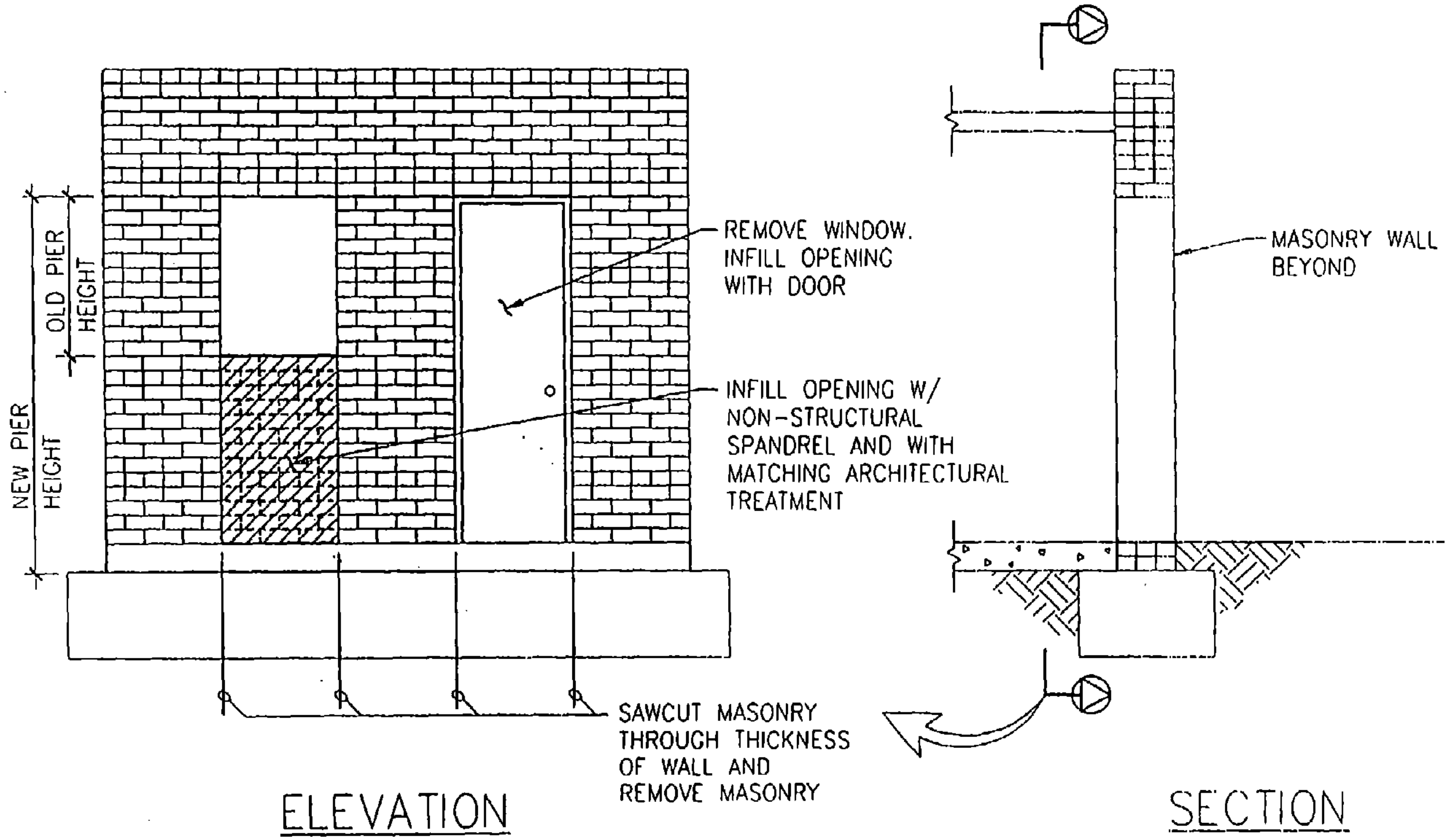
Figure D-6: Post-Tensioned Masonry



NOTES:

1. NO SPECIAL INSPECTION REQUIRED.
2. SIZE AND STRENGTH OF NEW MORTAR AND MASONRY TO CLOSELY MATCH EXISTING MORTAR AND MASONRY.

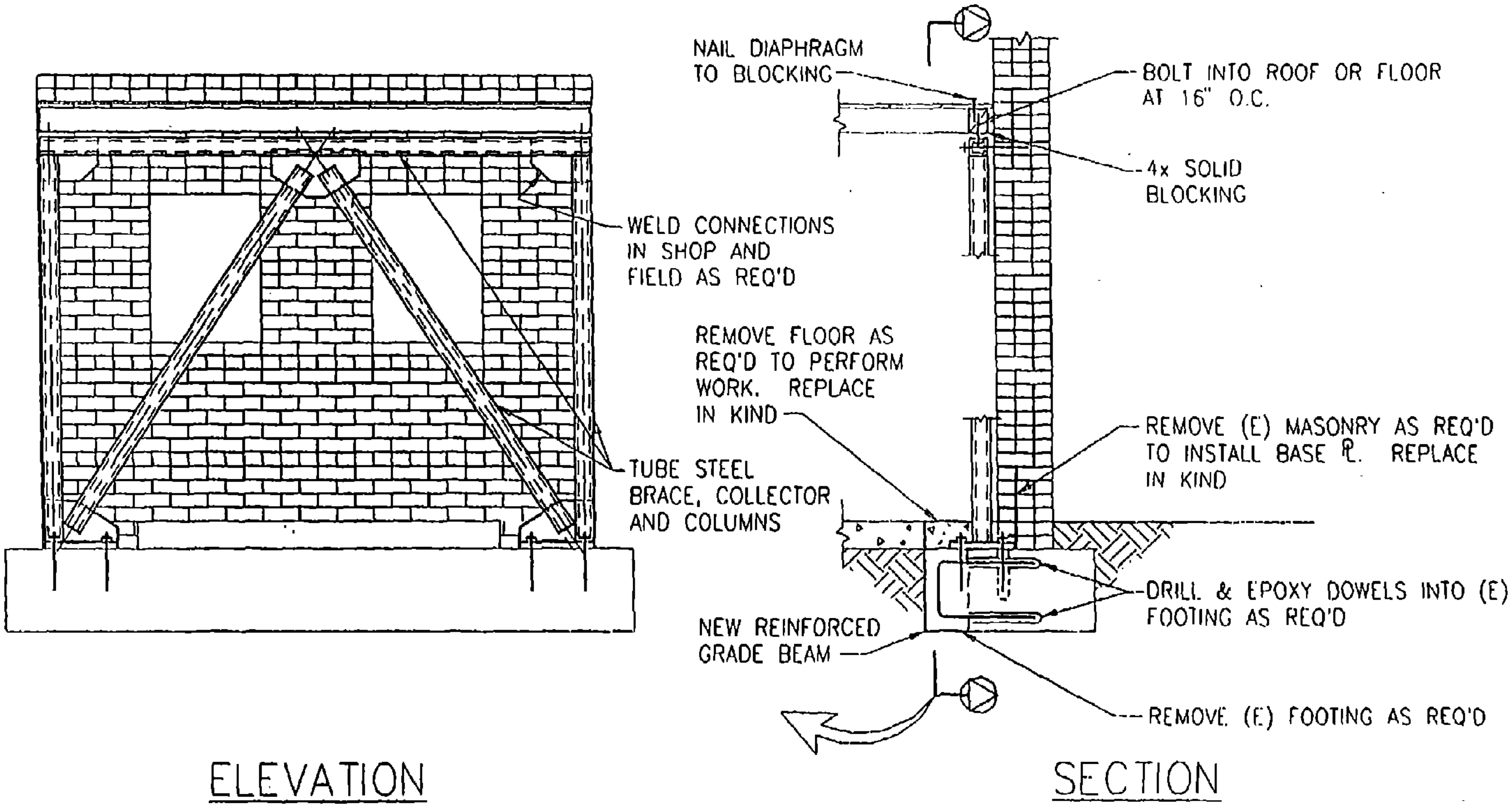
Figure D-7: Infilled Openings



NOTES:

1. NO SPECIAL INSPECTION REQUIRED.

Figure D-8: Enlarged Openings



NOTES:

1. SPECIAL INSPECTOR SHALL REVIEW STEEL MILL CERTIFICATES AND WELDING PROCEDURE SPECIFICATIONS AND SHALL INSPECT SHOP WELDING.
2. ON-SITE SPECIAL INSPECTOR SHALL INSPECT CONCRETE REINFORCING, CONCRETE PLACEMENT AND FIELD WELDING.
3. 'X' BRACING AND DIAGONAL BRACING ARE FREQUENTLY USED IN ADDITION TO CHEVRON BRACING.

Figure D-9: Steel Bracing

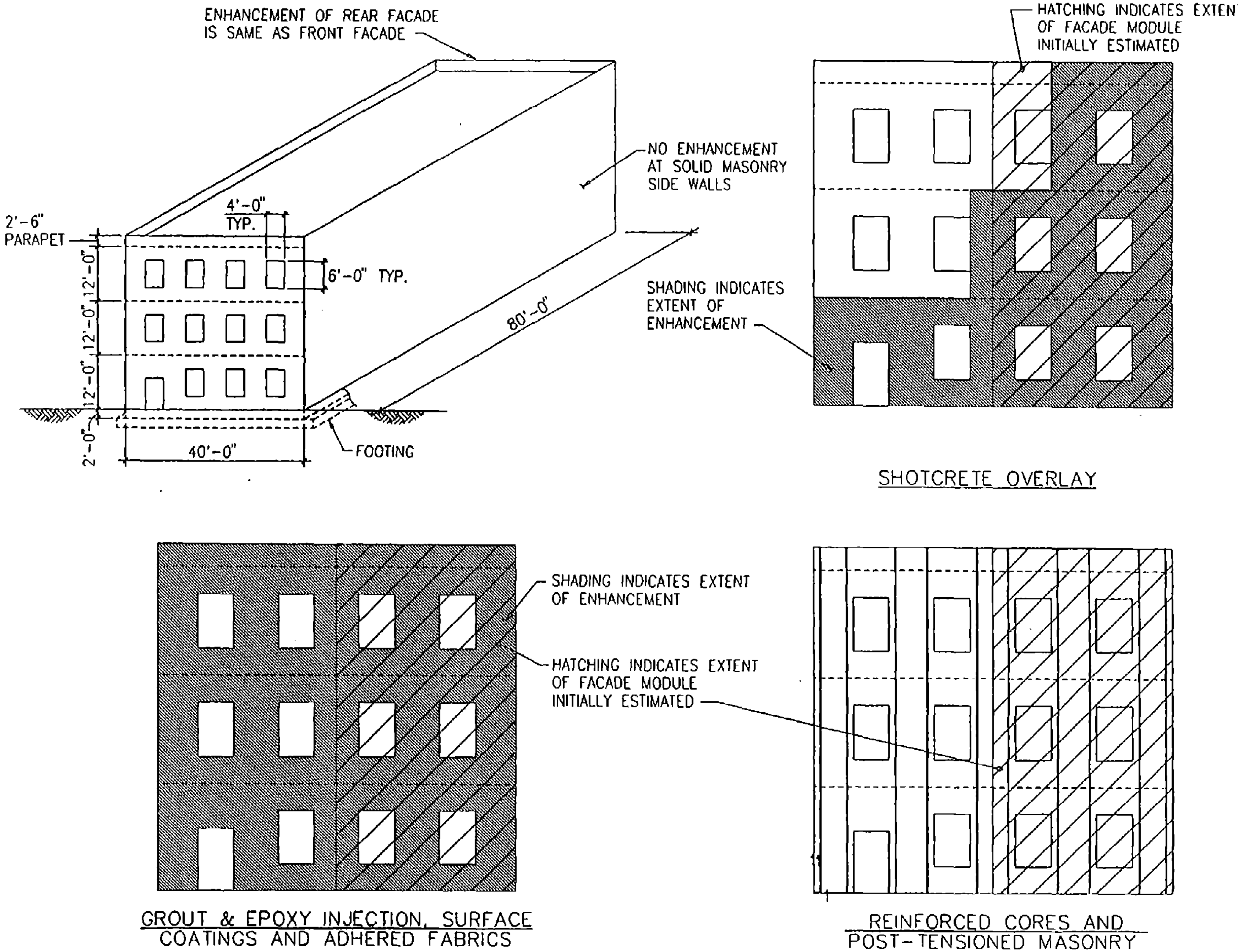


Figure D-10 : Model Building and Facades

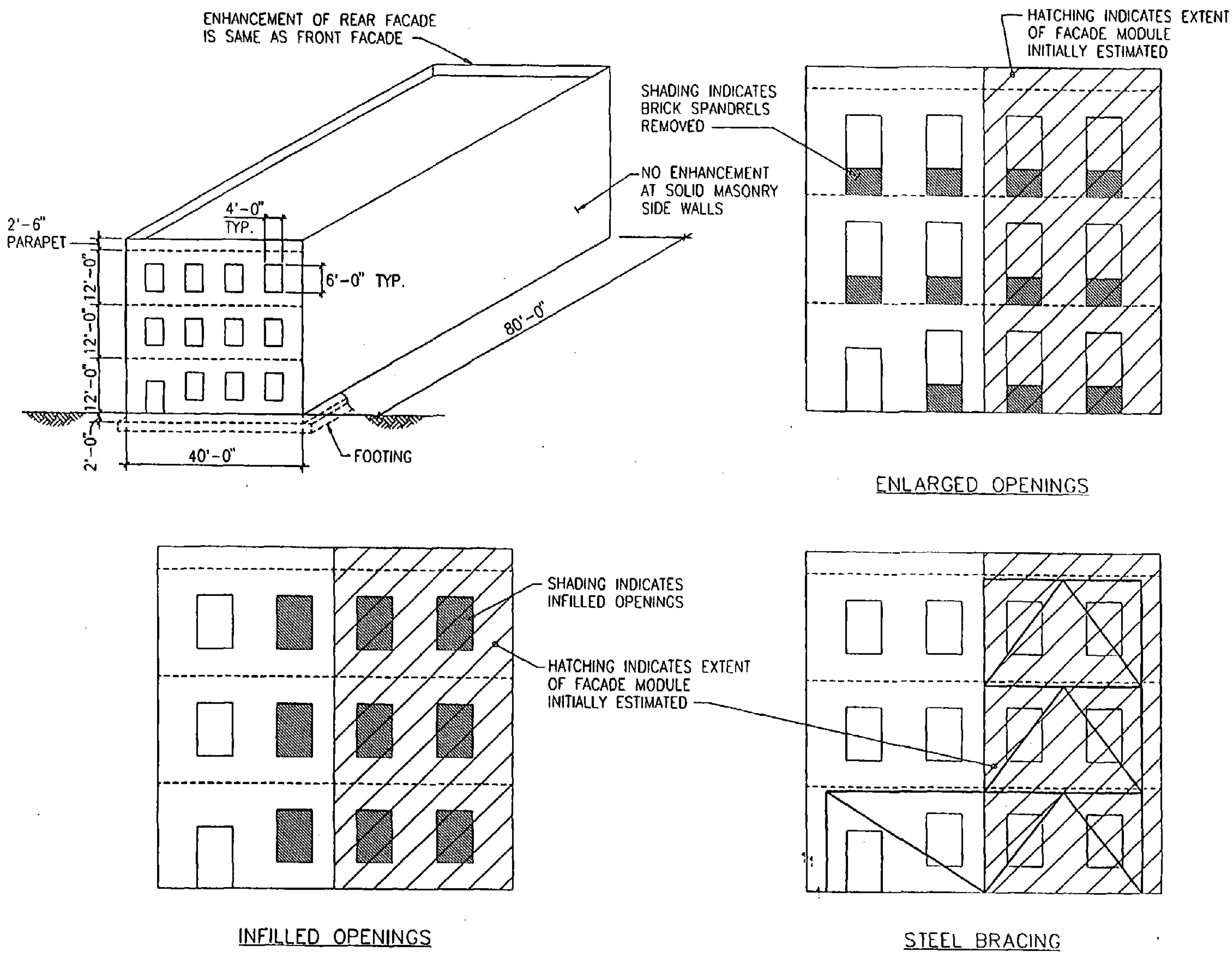


Figure D-11 : Model Building and Facades

D.5 Estimate of Increase in Shear Strength Provided by Wall Enhancement Methods

Background

In order to obtain a rough idea of the cost effectiveness of the various wall enhancement methods, this section provides quantitative estimates of the increase in in-plane capacity for each method using the design guidelines in Section 2. As noted in Section 2, there are four basic in-plane behavioral modes for URM walls. In order to provide a consistent means of comparison, the increase in capacity of a deformation-controlled shear mode is estimated. Note that this may not be the primary reason for selecting a particular method. To make quantitative estimates, a great number of assumptions have to be made; they are given below. The final values for the increases are contained Table D.8.

General Assumptions

The following general assumptions are made for all the enhancement methods:

- The linear static procedure of FEMA 273 (1996) is used.
- The life safety performance level is assumed.
- Walls are considered "primary components".
- Plain and enhanced wall capacities are calculated for the lower story of the prototypical buildings shown in Figures D-10 and D-11, using the information shown in Figures D-1 to D-9.
- Assume deformation-controlled behavior will persist following enhancement.
- Compare plain versus enhanced capacities using the FEMA 273 (1996) Equation 3-17 with $\kappa_{enhanced} = \kappa_{plain} = 1$, so that the increase in shear capacity provided by the enhancement is defined as $(m_{enhanced})(Q_{CEenhance})/(m_{plain})(Q_{CEplain})$

Specific Assumptions for the Plain Brick Wall

The following assumptions are made for the existing plain or "unenhanced" brick masonry walls in Figures D-2 to D-9:

- $v_{te} = 70$ psi
- $f_{me} = 1{,}000$ psi
- $Q_G = 0.9Q_D$ governs over $Q_G = 1.1(Q_D + Q_L + Q_S)$
- $Q_D = 35$ psf for floor loads and 27 psf for roof loads
- The tributary width of floor and roof bearing loads to the URM wall is 10'

Specific Assumptions for the Grouted CMU Wall

- No adjustment made for the increase in mass caused by the grout
- 12" nominal closed end block wall
- $f_{ge} = 2{,}000$ psi

Specific Assumptions for the Surface Coatings

- $f_{ce} = 2000$ psi

Specific Assumptions for the Shotcrete Overlay

- The enhancement capacity is based solely on the shotcrete shear capacity; any contribution from the masonry is ignored.
- No adjustment is made for the increase in mass caused by the shotcrete.
- $f_y = 60{,}000$ psi
- $f_c = 4{,}000$ psi
- 6" thickness of shotcrete
- $\propto_c = 3$
- Assume pier behavior is governed by shear (FEMA 273) Table 6-19) so that $m_{enhanced} = 2$

Specific Assumptions for the Post-Tensioned Masonry

- Accounts only for increase in shear capacity due to the increase in effective compressive stress provided by the post-tensioning
- #5 - 150 ksi Dywidag threadbar used as tendon
- Effective prestress is 0.60 of ultimate capacity

Specific Assumptions for the Steel Bracing

- Enhanced capacity only includes the capacity of the steel as if the masonry were not present; the interaction between the steel and masonry is ignored.
- The weakest member in the lowest story is the single diagonal brace above the door. This member is assumed to be a TS8x8x1/2; others are assumed to be TS6x6x1/2 members.
- $m_{enhanced} = 2$ (cold formed tubes) + 1 (special gusset plate detailing)

D.6 Subconsultant Cost Estimate of Wall Enhancement Methods

Hanscomb, Inc. provided estimates of the construction cost of the wall enhancement methods described in Figures D-1 to D-9. They were instructed to estimate a generic 20' wide module of an end wall shown in Figures D-10 or D-11. The entire three-story height of the wall was assumed to be enhanced. The conversion from this module to the actual extent of the 40' wide facade which is assumed to be enhanced is described in Section D.4. Hanscomb was also instructed to use the following additional assumptions. Hanscomb's report is given at the end of the appendix.

Cost Basis:	San Francisco Bay Area, 1996 dollars.
Costs Include:	All costs borne by the contractor which the owner will ultimately pay.
Building Type:	Commercial building.

Occupancy:	Unoccupied. Some dust protection required at exterior. No provisions for noise. Provide a premium for an occupied building.
Wall Module:	Three-story tall, composed of first floor with slab-on-grade, second and third floor with wood joist, roof with wood joist, and 2'-6" parapet. 2' wide by 1½' deep concrete strip footing under wall, 3' deep footing at wall with post-tensioned masonry.
Wall Dimensions:	12' interstory floor height by 20' wide. Two windows 4' wide by 6' high symmetrically placed in wall. Block wall is composed of standard 8"x12"x16" block, closed cell. Brick walls are three-wythe bricks, 13" wide.
Floor Construction:	2x12 joists at 16" on center with 1x sheathing and under-layment.
Roof Construction:	2x12 joists at 16" on center with 1x sheathing. Hot mop roofing with gravel ballast.
Interior Partition:	Facade module is intersected by one plaster over wood stud partition intersecting wall at each story.
M/E/P:	No major obstacles on wall, no asbestos abatement, no large plumbing runs, no large mechanical or electrical equipment. Minor electrical exists such as conduit, receptacles, light switches.
Demolition:	Interior of wall finished with plaster on furring. Exterior of wall plain brick, unpainted. Lath and plaster ceiling, carpet floors.
Wall Preparation:	10' of repointing required each face of wall, each story, except at reinforced cores and post-tensioned masonry, where 30' of repointing is required on the exterior surface. Prepare surface with mechanical wire brush typical except at adhered fabric. Sandblast at brick surfaces to receive adhered fabric.
Finishes:	Basic costs include carpet floors, plaster on interior walls and ceilings, paint walls and ceiling with two coats paint, (primer and finish), extend ceiling paint 10' back from wall. Provide a premium for a moderate degree of finishes which includes quarry tile or hard wood floor, plaster and paint on ceiling and walls same as minor level, wood base boards, window moldings.
Braced Frame:	TS6x6x½ for all tubes.

Reinforced Core:	Dry core.
Post-Tensioned Cores:	Dry core.

D.7 Subconsultant Cost Estimate of Special Inspection and Testing of the Wall Enhancement Methods

Special inspection and testing requirements for the wall enhancement methods are given in the notes of Figures D-1 to D-9. Often these requirements are estimated as a percentage of total construction cost. This method is more appropriate for large, new construction projects. With rehabilitation work, particularly where innovative techniques are being used, such an approach is less appropriate. Applied Materials Engineering, Inc. (AME), a San Francisco Bay Area testing and inspection firm, provided estimates for the required testing and inspection as if they were developing a fee proposal to perform the work. Their estimates are for the scope of work shown in Figures D-10 and D-11 and include the time required for inspection, administration (including writing summary reports), and the cost of material tests. Costs have been converted to $/plan square foot for Table D.8. AME's report is given at the end of the appendix.

DEVELOPMENT OF PROCEDURES TO ENHANCE THE PERFORMANCE OF REHABILITATED URM BUILDINGS

WALL ENHANCEMENT COST STUDY
SEPTEMBER 18, 1996

CLIENT

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SAN FRANCISCO, CA 94107

COST CONSULTANT

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750 BATTERY STREET, SUITE 400
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SF/E3296890/GEC

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INTRODUCTION

This cost study has been prepared to reflect the anticipated comparative cost of nine alternative methods for enhancing the seismic performance of unreinforced masonry (URM) walls.

This document is based on the measurement and pricing of quantities wherever information is provided and/or reasonable assumptions for other works not covered in the drawings or specifications, as stated within this document. Unit rates have been obtained from historical records and/or discussion with contractors. The unit rates are composites of labor, material and equipment and reflect current bid costs in the San Francisco Bay Area. All unit rates relevant to subcontractor works include the subcontractors' general conditions,overhead and profit.

The following were used in preparation of this opinion:

Figures D-1 to D-11
Cost estimate assumptions, provided by Rutherford & Chekene
Conversations with rutherford & Chekene

Exclusions

The following items are excluded:

Inspection costs
Consultant fees and expenses
Any associated alteration work apart from the masonry enhancement
Legal and financing costs
Owner's fees for testing construction materials
Other associated owner's costs

Items affecting the cost estimated:

Items which may change the estimated construction cost include, but are not limited to:

- Modifications to the scope of work included in this estimate.
- Unforeseen or hidden conditions.
- Special phasing requirements.
- Restrictive technical specifications or excessive contract conditions.
- Any specified item of equipment, material, or product that cannot be obtained from at least three different sources.
- Any other non-competitive bid situations.

Assumption:

The following assumptions have been made:

1. Normal working hours.
2. Sufficient space will be provided to the contractor to house temporary site storage and accommodation within the vicinity of the site.
3. Building is located in the San Francisco area, is unoccupied and has easy access (premium for building being occupied is shown in the executive summary)

Escalation:

Prices in this opinion reflect current bid costs at an ENR Building Cost Index of 3238.97

Contingencies:

A design pricing contingency allowance has been included at 10%. This is to allow for items not included in the drawings or specifications undefined at this stage, (including any addendums produced during bidding stage). It is also to allow for items included in the front end document, i.e. special contractual provisions including liquidated damages and minority stipulations, restrictions on working conditions etc.

It is prudent for all program budgets to include an allowance for change orders which occur during the construction phase and impact total project cost. This opinion does not include an allowance for construction contingency.

This opinion has been based on a competitive open bid situation with a recommended 5-7 bona fide reputable bids from general contractors and a minimum of 3 bidders for all items of sub-contracted work. Experience indicates that a fewer number of bidders may result in higher bids, conversely an increased number of bidders may result in more competitive bids.

Since Hanscomb has no control over the cost of labor, materials, or equipment, or over the contractor's method of determining prices, or over competitive bidding or marketing conditions, the opinion of probable construction cost provided for herein is made on the basis of professional experience and qualifications. The opinion represents Hanscomb's best judgment as a professional construction consultant familiar with the construction industry. However Hanscomb cannot and does not guarantee that proposals, bids, or the construction cost will not vary from opinions of probable cost prepared by them.

DEVELOPMENT OF PROCEDURES TO ENHANCE THE PERFORMANCE OF REHABILITATED URM BUILDINGS
WALL ENHANCEMENT COST STUDY

Estimate Stage: Concept
Wall Area: 810 SF
Prepared/Checked by: GEC/GB

Ref.	WALL ENHANCEMENT TYPE	Architectural Costs	Structural Costs	Electrical Costs	Total Construction Cost	Special Architectural Premium	Occupancy Premium
	EXECUTIVE SUMMARY						
D-1	GROUT & EPOXY INJECTION	9,306	24,837	356	34,500	998	10,000
D-2	SURFACE COATINGS	13,649	53,613	713	67,975	2,463	10,000
D-3	ADHERED FABRICS	14,397	31,020	713	46,130	2,463	10,000
D-4	SHOTCRETE OVERLAY	13,649	28,824	713	43,186	2,463	10,000
D-5	REINFORCED CORES	911	77,959	0	78,870	0	2,000
D-6	POST-TENSIONED MASONRY	911	82,676	0	83,586	0	2,000
D-7	INFILLED OPENINGS	3,019	11,095	0	14,114	55	10,000
D-8	ENLARGED OPENINGS	6,261	4,856	119	11,235	998	10,000
D-9	STEEL BRACING	14,268	40,738	713	55,719	11,749	10,000

Occupancy premium includes for provision of facilities that would otherwise be available within the building (e.g. storage space), and (where appropriate) for dust/security screens and isolation of working area adjacent to wall, and removing same on completion.

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DEVELOPMENT OF PROCEDURES TO ENHANCE THE PERFORMANCE OF REHABILITATED URM BUILDINGS
WALL ENHANCEMENT COST STUDY

Estimate Stage: Concept
Wall Area: 810 SF
Prepared/Checked by: GEC/GB

Ref.	Section	System Quantity	Unit	$/Unit	Total Cost
	CONSTRUCTION SUMMARY				
D-1	**GROUT & EPOXY INJECTION**				
	ARCHITECTURAL	810	SF	11.49	9,306
	SPECIAL ARCHITECTURAL	810	SF	1.23	998
	STRUCTURAL	810	SF	30.66	24,837
	ELECTRICAL	810	SF	0.44	356
	GROUT & EPOXY INJECTION			43.82	35,497
D-2	**SURFACE COATINGS**				
	ARCHITECTURAL	810	SF	16.85	13,649
	SPECIAL ARCHITECTURAL	810	SF	3.04	2,463
	STRUCTURAL	810	SF	66.19	53,613
	ELECTRICAL	810	SF	0.88	713
	SURFACE COATINGS			86.96	70,438
D-3	**ADHERED FABRICS**				
	ARCHITECTURAL	810	SF	17.77	14,397
	SPECIAL ARCHITECTURAL	810	SF	3.04	2,463
	STRUCTURAL	810	SF	38.30	31,020
	ELECTRICAL	810	SF	0.88	713
	ADHERED FABRICS			59.99	48,593
D-4	**SHOTCRETE OVERLAY**				
	ARCHITECTURAL	810	SF	16.85	13,649
	SPECIAL ARCHITECTURAL	810	SF	3.04	2,463
	STRUCTURAL	810	SF	35.59	28,824
	ELECTRICAL	810	SF	0.88	713
	SHOTCRETE OVERLAY			56.35	45,649

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DEVELOPMENT OF PROCEDURES TO ENHANCE THE PERFORMANCE OF REHABILITATED URM BUILDINGS WALL ENHANCEMENT COST STUDY	Estimate Stage: Concept Wall Area: 810 SF Prepared/Checked by: GEC/GB

Ref.	Section	System Quantity	Unit	$/Unit	Total Cost
D-5	REINFORCED CORES				
	ARCHITECTURAL	810	SF	1.12	911
	SPECIAL ARCHITECTURAL	810	SF	0.00	
	STRUCTURAL	810	SF	96.25	77,959
	ELECTRICAL	810	SF	0.00	
	REINFORCED CORES			97.37	78,870
D-6	POST-TENSIONED MASONRY				
	ARCHITECTURAL	810	SF	1.12	911
	SPECIAL ARCHITECTURAL	810	SF	0.00	
	STRUCTURAL	810	SF	102.07	82,676
	ELECTRICAL	810	SF	0.00	
	POST-TENSIONED MASONRY			103.19	83,586
D-7	INFILLED OPENINGS				
	ARCHITECTURAL	810	SF	3.73	3,019
	SPECIAL ARCHITECTURAL	810	SF	0.07	55
	STRUCTURAL	810	SF	13.70	11,095
	ELECTRICAL	810	SF	0.00	
	INFILLED OPENINGS			17.49	14,169
D-8	ENLARGED OPENINGS				
	ARCHITECTURAL	810	SF	7.73	6,261
	SPECIAL ARCHITECTURAL	810	SF	1.23	998
	STRUCTURAL	810	SF	5.99	4,856
	ELECTRICAL	810	SF	0.15	119
	ENLARGED OPENINGS			15.10	12,233
D-9	STEEL BRACING				
	ARCHITECTURAL	810	SF	17.61	14,268
	SPECIAL ARCHITECTURAL	810	SF	14.51	11,749
	STRUCTURAL	810	SF	50.29	40,738
	ELECTRICAL	810	SF	0.88	713
	STEEL BRACING			83.29	67,468

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	DEVELOPMENT OF PROCEDURES TO ENHANCE THE PERFORMANCE OF REHABILITATED URM BUILDINGS WALL ENHANCEMENT COST STUDY	Estimate Stage: Concept Wall Area: 810 SF Prepared/Checked by: GEC/GB			
Ref.	**Description**	**Quantity**	**U.o.M.**	**Unit Cost**	**Total**
D-1	**ARCHITECTURAL**				
	Remove existing drywall lining to inner face	720	SF	0.80	576
	Remove existing drywall ceiling to provide access to wall	60	LF	10.50	630
	5/8" gypsum wallboard on furrings; painted	720	SF	4.20	3,024
	Repair drywall ceiling up to wall	60	LF	40.00	2,400
	Repaint ceiling	600	SF	0.70	420
	Sub-Total				7,050
	General Conditions, Overhead and Profit			20%	1,410
	Contingency			10%	846
	ARCHITECTURAL				9,306
D-1	**SPECIAL ARCHITECTURAL**				
	Remove existing wood base	60	LF	0.40	24
	Remove window molding	120	LF	0.35	42
	New wood base; paint	60	LF	4.00	240
	New window molding; paint	120	LF	3.75	450
	Sub-Total				756
	General Conditions, Overhead and Profit			20%	151
	Contingency			10%	91
	SPECIAL ARCHITECTURAL				998

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	DEVELOPMENT OF PROCEDURES TO ENHANCE THE PERFORMANCE OF REHABILITATED URM BUILDINGS WALL ENHANCEMENT COST STUDY	**Estimate Stage: Concept Wall Area: 810 SF Prepared/Checked by: GEC/GB**			
Ref.	**Description**	**Quantity**	**U.o.M.**	**Unit Cost**	**Total**
D-1	STRUCTURAL				
	Scaffolding	770	SF	2.50	1,925
	Drill 3/4" hole in masonry face; grout on completion	434	EA	20.00	8,680
	Injected grout to fill 12" hollow masonry wall	810	SF	10.00	8,100
	Remove loose mortar and repoint, exterior	30	LF	2.00	60
	Remove loose mortar and repoint, interior	30	LF	1.70	51
	Sub-Total				18,816
	General Conditions, Overhead and Profit			20%	3,763
	Contingency			10%	2,258
	STRUCTURAL				24,837
D-1	ELECTRICAL				
	Remove and reinstall receptacle	6	EA	30.00	180
	Remove and reinstall light switch	3	EA	30.00	90
	Sub-Total				270
	General Conditions, Overhead and Profit			20%	54
	Contingency			10%	32
	ELECTRICAL				356

	DEVELOPMENT OF PROCEDURES TO ENHANCE THE PERFORMANCE OF REHABILITATED URM BUILDINGS WALL ENHANCEMENT COST STUDY	Estimate Stage: Concept Wall Area: 810 SF Prepared/Checked by: GEC/GB			
Ref.	**Description**	**Quantity**	**U.o.M.**	**Unit Cost**	**Total**
D-2	**ARCHITECTURAL**				
	Remove existing drywall lining to inner face	720	SF	0.80	576
	Remove existing drywall ceiling to provide access to wall	60	LF	10.50	630
	Remove stud partition adjacent to exterior wall	36	LF	7.50	270
	Remove carpet adjacent to wall	60	LF	0.50	30
	Remove roof finish adjacent to wall	20	LF	7.50	150
	5/8" gypsum wallboard on furrings; painted	720	SF	4.20	3,024
	Repair drywall ceiling up to wall	60	LF	40.00	2,400
	Repair stud partition up to wall	36	LF	22.00	792
	Relay carpet adjacent to exterior wall	60	LF	8.00	480
	Repair roof finish up to wall	20	LF	55.00	1,100
	Repaint ceiling	600	SF	0.70	420
	Repaint partition	720	SF	0.65	468
	Sub-Total				10,340
	General Conditions, Overhead and Profit			20%	2,068
	Contingency			10%	1,241
	ARCHITECTURAL				13,649
D-2	**SPECIAL ARCHITECTURAL**				
	Remove existing wood base	60	LF	0.40	24
	Remove window molding	120	LF	0.35	42
	Premium for removing ceramic tile in lieu of carpet	60	LF	2.50	150
	New wood base; paint	60	LF	4.00	240
	New window molding; paint	120	LF	3.75	450
	New ceramic floor tile in lieu of carpet	60	LF	16.00	960
	Sub-Total				1,866
	General Conditions, Overhead and Profit			20%	373
	Contingency			10%	224
	SPECIAL ARCHITECTURAL				2,463

DEVELOPMENT OF PROCEDURES TO ENHANCE THE PERFORMANCE OF REHABILITATED URM BUILDINGS
WALL ENHANCEMENT COST STUDY

Estimate Stage: Concept
Wall Area: 810 SF
Prepared/Checked by: GEC/GB

Ref.	Description	Quantity	U.o.M.	Unit Cost	Total
D-2	STRUCTURAL				
	Excavate externally to expose face of wall; backfill; dispose of surplus	3	CY	38.00	114
	Shoring to existing floor or roof	60	LF	100.00	6,000
	Cut back existing floor or roof	60	LF	15.00	900
	Reinstate floor or roof structure	60	LF	50.00	3,000
	Scaffolding	770	SF	2.50	1,925
	Clean off masonry:				
	- exterior face	810	SF	0.30	243
	- interior face	810	SF	0.30	243
	Remove loose mortar and repoint, exterior	30	LF	2.00	60
	Remove loose mortar and repoint, interior	30	LF	1.70	51
	Hardware cloth, 19ga x 1/2" x 1/2", with 1/4" x 7" expansion anchors at 16" o.c. each way:				
	- to exterior face	810	SF	14.50	11,745
	- to interior face	810	SF	14.50	11,745
	Cementitious plaster:				
	- to interior face of wall	720	SF	3.00	2,160
	- to exterior face of wall; paint	810	SF	3.00	2,430
	Sub-Total				40,616
	General Conditions, Overhead and Profit			20%	8,123
	Contingency			10%	4,874
	STRUCTURAL				53,613
D-2	ELECTRICAL				
	Remove receptacle and install in new position	6	EA	60.00	360
	Remove light switch and install in new position	3	EA	60.00	180
	Sub-Total				540
	General Conditions, Overhead and Profit			20%	108
	Contingency			10%	65
	ELECTRICAL				713

	DEVELOPMENT OF PROCEDURES TO ENHANCE THE PERFORMANCE OF REHABILITATED URM BUILDINGS WALL ENHANCEMENT COST STUDY	**Estimate Stage: Concept Wall Area: 810 SF Prepared/Checked by: GEC/GB**			
Ref.	**Description**	**Quantity**	**U.o.M.**	**Unit Cost**	**Total**
D-3	ARCHITECTURAL				
	Remove existing drywall lining to inner face	720	SF	0.80	576
	Remove existing drywall ceiling to provide access to wall	60	LF	10.50	630
	Remove stud partition adjacent to exterior wall	36	LF	7.50	270
	Remove carpet adjacent to wall	60	LF	0.50	30
	Remove roof finish adjacent to wall	20	LF	7.50	150
	5/8" gypsum wallboard on furrings; painted	720	SF	4.20	3,024
	Repair drywall ceiling up to wall	60	LF	40.00	2,400
	Repair stud partition up to wall	36	LF	22.00	792
	Relay carpet adjacent to exterior wall	60	LF	8.00	480
	Repair roof finish up to wall	20	LF	55.00	1,100
	Repaint ceiling	600	SF	0.70	420
	Repaint partition	720	SF	0.65	468
	UV protective paint to exterior	810	SF	0.70	567
	Sub-Total				10,907
	General Conditions, Overhead and Profit			20%	2,181
	Contingency			10%	1,309
	ARCHITECTURAL				14,397
D-3	SPECIAL ARCHITECTURAL				
	Remove existing wood base	60	LF	0.40	24
	Remove window molding	120	LF	0.35	42
	Premium for removing ceramic tile in lieu of carpet	60	LF	2.50	150
	New wood base; paint	60	LF	4.00	240
	New window molding; paint	120	LF	3.75	450
	New ceramic floor tile in lieu of carpet	60	LF	16.00	960
	Sub-Total				1,866
	General Conditions, Overhead and Profit			20%	373
	Contingency			10%	224
	SPECIAL ARCHITECTURAL				2,463

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	DEVELOPMENT OF PROCEDURES TO ENHANCE THE PERFORMANCE OF REHABILITATED URM BUILDINGS WALL ENHANCEMENT COST STUDY	Estimate Stage: Concept Wall Area: 810 SF Prepared/Checked by: GEC/GB			
Ref.	**Description**	**Quantity**	**U.o.M.**	**Unit Cost**	**Total**
D-3	STRUCTURAL				
	Excavate externally to expose face of wall; backfill; dispose of surplus	3	CY	38.00	114
	Remove floor or roof decking	60	LF	5.00	300
	Reinstate floor or roof decking	60	LF	6.00	360
	Scaffolding	770	SF	2.50	1,925
	Sandblast masonry:				
	- exterior face	810	SF	2.00	1,620
	- interior face	810	SF	2.00	1,620
	Remove loose mortar and repoint, exterior	30	LF	2.00	60
	Remove loose mortar and repoint, interior	30	LF	1.70	51
	Two coat epoxy and high strength fiberglass fabric to walls:				
	- to exterior face	810	SF	10.00	8,100
	- to interior face	810	SF	10.00	8,100
	4" x 4" angle with 1/4" diameter bolts:				
	- exterior	20	LF	25.00	500
	- interior	20	LF	25.00	500
	Cut groove in slab on grade for new angle	20	LF	12.50	250
	Sub-Total				23,500
	General Conditions, Overhead and Profit			20%	4,700
	Contingency			10%	2,820
	STRUCTURAL				31,020
D-3	ELECTRICAL				
	Remove receptacle and install in new position	6	EA	60.00	360
	Remove light switch and install in new position	3	EA	60.00	180
	Sub-Total				540
	General Conditions, Overhead and Profit			20%	108
	Contingency			10%	65
	ELECTRICAL				713

DEVELOPMENT OF PROCEDURES TO ENHANCE THE PERFORMANCE OF REHABILITATED URM BUILDINGS
WALL ENHANCEMENT COST STUDY

Estimate Stage: Concept
Wall Area: 810 SF
Prepared/Checked by: GEC/GB

Ref.	Description	Quantity	U.o.M.	Unit Cost	Total
D-4	ARCHITECTURAL				
	Remove existing drywall lining to inner face	720	SF	0.80	576
	Remove existing drywall ceiling to provide access to wall	60	LF	10.50	630
	Remove stud partition adjacent to exterior wall	36	LF	7.50	270
	Remove carpet adjacent to wall	60	LF	0.50	30
	Remove roof finish adjacent to wall	20	LF	7.50	150
	5/8" gypsum wallboard on furrings; painted	720	SF	4.20	3,024
	Repair drywall ceiling up to wall	60	LF	40.00	2,400
	Repair stud partition up to wall	36	LF	22.00	792
	Relay carpet adjacent to exterior wall	60	LF	8.00	480
	Repair roof finish up to wall	20	LF	55.00	1,100
	Repaint ceiling	600	SF	0.70	420
	Repaint partition	720	SF	0.65	468
	Sub-Total				10,340
	General Conditions, Overhead and Profit			20%	2,068
	Contingency			10%	1,241
	ARCHITECTURAL				13,649
D-4	SPECIAL ARCHITECTURAL				
	Remove existing wood base	60	LF	0.40	24
	Remove window molding	120	LF	0.35	42
	Premium for removing ceramic tile in lieu of carpet	60	LF	2.50	150
	New wood base; paint	60	LF	4.00	240
	New window molding; paint	120	LF	3.75	450
	New ceramic floor tile in lieu of carpet	60	LF	16.00	960
	Sub-Total				1,866
	General Conditions, Overhead and Profit			20%	373
	Contingency			10%	224
	SPECIAL ARCHITECTURAL				2,463

	DEVELOPMENT OF PROCEDURES TO ENHANCE THE PERFORMANCE OF REHABILITATED URM BUILDINGS WALL ENHANCEMENT COST STUDY	**Estimate Stage: Concept**	**Wall Area: 810 SF**	**Prepared/Checked by: GEC/GB**	
Ref.	**Description**	**Quantity**	**U.o.M.**	**Unit Cost**	**Total**
D-4	**STRUCTURAL**				
	Remove slab on grade 1'6" wide	20	LF	15.00	300
	Excavate internally for new footing; backfill; dispose of surplus	4	CY	45.00	180
	Clean face of existing footing	30	SF	0.60	18
	Concrete in footing extension	1	CY	160.00	160
	Dowel in footing extension	31	LB	1.55	48
	Drill footing for dowel; grout in	30	EA	35.00	1,050
	Repair slab on grade	20	LF	25.00	500
	Shoring to existing floor or roof	60	LF	100.00	6,000
	Cut back existing floor or roof	60	LF	15.00	900
	Reinstate floor or roof structure	60	LF	50.00	3,000
	Scaffolding	770	SF	2.50	1,925
	Clean off masonry:				
	- interior face	810	SF	0.30	243
	Remove loose mortar and repoint, exterior	30	LF	2.00	60
	Remove loose mortar and repoint, interior	30	LF	1.70	51
	6" thick shotcrete to interior face of wall	810	SF	5.60	4,536
	Rebar in shotcrete	972	LB	0.70	680
	Dowels in shotcrete	55	LB	1.55	85
	Drill masonry for dowel; grout in	84	EA	25.00	2,100
	Sub-Total				21,837
	General Conditions, Overhead and Profit			20%	4,367
	Contingency			10%	2,620
	STRUCTURAL				28,824
D-4	**ELECTRICAL**				
	Remove receptacle and install in new position	6	EA	60.00	360
	Remove light switch and install in new position	3	EA	60.00	180
	Sub-Total				540
	General Conditions, Overhead and Profit			20%	108
	Contingency			10%	65
	ELECTRICAL				713

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	DEVELOPMENT OF PROCEDURES TO ENHANCE THE PERFORMANCE OF REHABILITATED URM BUILDINGS WALL ENHANCEMENT COST STUDY	Estimate Stage: Concept Wall Area: 810 SF Prepared/Checked by: GEC/GB			
Ref.	**Description**	**Quantity**	**U.o.M.**	**Unit Cost**	**Total**
D-5	**ARCHITECTURAL**				
	Remove existing coping and flashing	20	LF	7.00	140
	New coping and flashing	20	LF	27.50	550
	Sub-Total				690
	General Conditions, Overhead and Profit			20%	138
	Contingency			10%	83
	ARCHITECTURAL				911
D-5	**STRUCTURAL**				
	Scaffolding	770	SF	2.50	1,925
	Premium for dust protection	770	SF	1.50	1,155
	4" diameter core drilled dry and vertically through masonry wall; reinforing bar inserted in core and grouted in	252	LF	220.00	55,440
	Drill verification port in exterior face of masonry; grout on completion	18	EA	20.00	360
	Remove loose mortar and repoint, exterior	90	LF	2.00	180
	Sub-Total				59,060
	General Conditions, Overhead and Profit			20%	11,812
	Contingency			10%	7,087
	STRUCTURAL				77,959

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	DEVELOPMENT OF PROCEDURES TO ENHANCE THE PERFORMANCE OF REHABILITATED URM BUILDINGS WALL ENHANCEMENT COST STUDY	**Estimate Stage: Concept Wall Area: 810 SF Prepared/Checked by: GEC/GB**			
Ref.	**Description**	**Quantity**	**U.o.M.**	**Unit Cost**	**Total**
D-6	**ARCHITECTURAL**				
	Remove existing coping and flashing	20	LF	7.00	140
	New coping and flashing	20	LF	27.50	550
	Sub-Total				690
	General Conditions, Overhead and Profit			20%	138
	Contingency			10%	83
	ARCHITECTURAL				911
D-6	**STRUCTURAL**				
	Scaffolding	770	SF	2.50	1,925
	Premium for dust protection	770	SF	1.50	1,155
	4" diameter core drilled dry and vertically through masonry wall; reinforing tendon inserted in core and grouted in	258	LF	220.00	56,760
	Premium for two stage grouting and post-tensioning anchor plate	6	EA	350.00	2,100
	Drill verification port in exterior face of masonry; grout on completion	18	EA	20.00	360
	Remove loose mortar and repoint, exterior	90	LF	2.00	180
	Remove loose mortar and repoint, interior	90	LF	1.70	153
	Sub-Total				62,633
	General Conditions, Overhead and Profit			20%	12,527
	Contingency			10%	7,516
	STRUCTURAL				82,676

	DEVELOPMENT OF PROCEDURES TO ENHANCE THE PERFORMANCE OF REHABILITATED URM BUILDINGS WALL ENHANCEMENT COST STUDY	Estimate Stage: Concept Wall Area: 810 SF Prepared/Checked by: GEC/GB			
Ref.	**Description**	**Quantity**	**U.o.M.**	**Unit Cost**	**Total**
D-7	**ARCHITECTURAL**				
	Remove existing window	144	SF	2.05	295
	Remove existing window sill	24	LF	2.50	60
	5/8" gypsum wallboard on furrings infilling openings	144	SF	6.00	864
	Joint new wallboard to existing	120	LF	5.00	600
	Repaint wall	720	SF	0.65	468
	Sub-Total				2,287
	General Conditions, Overhead and Profit			20%	457
	Contingency			10%	274
	ARCHITECTURAL				3,019
D-7	**SPECIAL ARCHITECTURAL**				
	Remove window molding	120	LF	0.35	42
	Sub-Total				42
	General Conditions, Overhead and Profit			20%	8
	Contingency			10%	5
	SPECIAL ARCHITECTURAL				55

	DEVELOPMENT OF PROCEDURES TO ENHANCE THE PERFORMANCE OF REHABILITATED URM BUILDINGS WALL ENHANCEMENT COST STUDY	Estimate Stage: Concept Wall Area: 810 SF Prepared/Checked by: GEC/GB			
Ref.	**Description**	**Quantity**	**U.o.M.**	**Unit Cost**	**Total**
D-7	**STRUCTURAL**				
	Scaffolding	770	SF	2.50	1,925
	13" thick infill masonry, facing brick externally	144	SF	35.00	5,040
	Bond to existing masonry	72	LF	20.00	1,440
	Sub-Total				8,405
	General Conditions, Overhead and Profit			20%	1,681
	Contingency			10%	1,009
	STRUCTURAL				11,095

	DEVELOPMENT OF PROCEDURES TO ENHANCE THE PERFORMANCE OF REHABILITATED URM BUILDINGS WALL ENHANCEMENT COST STUDY	Estimate Stage: Concept Wall Area: 810 SF Prepared/Checked by: GEC/GB			
Ref.	**Description**	**Quantity**	**U.o.M.**	**Unit Cost**	**Total**
D-8	**ARCHITECTURAL**				
	Remove existing drywall lining in isolated areas	84	SF	1.25	105
	Remove existing window sill	24	LF	2.50	60
	New window sill	24	LF	40.00	960
	Stud infill panel; insulation; tile finish externally to match brickwork	84	SF	37.50	3,150
	Repaint wall	720	SF	0.65	468
	Sub-Total				4,743
	General Conditions, Overhead and Profit			20%	949
	Contingency			10%	569
	ARCHITECTURAL				6,261
D-8	**SPECIAL ARCHITECTURAL**				
	Remove existing wood base	60	LF	0.40	24
	Remove window molding	120	LF	0.35	42
	New wood base; paint	60	LF	4.00	240
	New window molding; paint	120	LF	3.75	450
	Sub-Total				756
	General Conditions, Overhead and Profit			20%	151
	Contingency			10%	91
	SPECIAL ARCHITECTURAL				998

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	DEVELOPMENT OF PROCEDURES TO ENHANCE THE PERFORMANCE OF REHABILITATED URM BUILDINGS WALL ENHANCEMENT COST STUDY	Estimate Stage: Concept Wall Area: 810 SF Prepared/Checked by: GEC/GB			
Ref.	**Description**	**Quantity**	**U.o.M.**	**Unit Cost**	**Total**
D-8	**STRUCTURAL**				
	Scaffolding	770	SF	2.50	1,925
	Sawcut 13" thick masonry	42	LF	35.75	1,502
	Remove masonry walling	84	SF	3.00	252
	Sub-Total				3,679
	General Conditions, Overhead and Profit			20%	736
	Contingency			10%	441
	STRUCTURAL				4,856
D-8	**ELECTRICAL**				
	Remove and reinstall receptacle	3	EA	30.00	90
	Sub-Total				90
	General Conditions, Overhead and Profit			20%	18
	Contingency			10%	11
	ELECTRICAL				119

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DEVELOPMENT OF PROCEDURES TO ENHANCE THE PERFORMANCE OF REHABILITATED URM BUILDINGS
WALL ENHANCEMENT COST STUDY

Estimate Stage: Concept
Wall Area: 810 SF
Prepared/Checked by: GEC/GB

Ref.	Description	Quantity	U.o.M.	Unit Cost	Total
D-9	ARCHITECTURAL				
	Remove existing drywall lining to inner face	720	SF	0.80	576
	Remove existing drywall ceiling to provide access to wall	60	LF	10.50	630
	Remove stud partition adjacent to exterior wall	36	LF	7.50	270
	Remove carpet adjacent to wall	60	LF	0.50	30
	Remove roof finish adjacent to wall	20	LF	7.50	150
	5/8" gypsum wallboard on furrings; painted	720	SF	4.20	3,024
	Repair drywall ceiling up to wall	60	LF	40.00	2,400
	Repair stud partition up to wall	36	LF	22.00	792
	Relay carpet adjacent to exterior wall	60	LF	8.00	480
	Repair roof finish up to wall	20	LF	55.00	1,100
	Repaint ceiling	600	SF	0.70	420
	Repaint partition	720	SF	0.65	468
	Paint steel sections	469	SF	1.00	469
	Sub-Total				10,809
	General Conditions, Overhead and Profit			20%	2,162
	Contingency			10%	1,297
	ARCHITECTURAL				14,268
D-9	SPECIAL ARCHITECTURAL				
	Remove existing wood base	60	LF	0.40	24
	Remove window molding	120	LF	0.35	42
	Premium for removing ceramic tile in lieu of carpet	60	LF	2.50	150
	New wood base; paint	60	LF	4.00	240
	New window molding; paint	120	LF	3.75	450
	New ceramic floor tile or hardwood flooring in lieu of carpet	60	LF	16.00	960
	Gypsum wallboard on studs in furring to steel bracing	1407	SF	5.00	7,035
	Sub-Total				8,901
	General Conditions, Overhead and Profit			20%	1,780
	Contingency			10%	1,068
	SPECIAL ARCHITECTURAL				11,749

	DEVELOPMENT OF PROCEDURES TO ENHANCE THE PERFORMANCE OF REHABILITATED URM BUILDINGS WALL ENHANCEMENT COST STUDY	Estimate Stage: Concept Wall Area: 810 SF Prepared/Checked by: GEC/GB			
Ref.	**Description**	**Quantity**	**U.o.M.**	**Unit Cost**	**Total**
D-9	**STRUCTURAL**				
	Remove slab on grade 1'6" wide	20	LF	15.00	300
	Excavate internally for new footing; backfill; dispose of surplus	4	CY	45.00	180
	Clean face of existing footing	30	SF	0.60	18
	Concrete in footing extension	1	CY	160.00	160
	Dowel in footing extension	17	LB	1.55	26
	Drill footing for dowel; grout in	20	EA	35.00	700
	Repair slab on grade	20	LF	25.00	500
	Shoring to existing floor or roof	60	LF	100.00	6,000
	Cut back existing floor or roof	60	LF	15.00	900
	Reinstate floor or roof structure	60	LF	50.00	3,000
	Scaffolding	770	SF	2.50	1,925
	Hollow metal section framing	8257	LB	2.00	16,514
	4" x 6" solid wood blocking	60	LF	6.30	378
	Remove loose mortar and repoint, exterior	30	LF	2.00	60
	Remove loose mortar and repoint, interior	30	LF	1.70	51
	Cut out and repair masonry for base plate	2	EA	75.00	150
	Sub-Total				30,862
	General Conditions, Overhead and Profit			20%	6,172
	Contingency			10%	3,703
	STRUCTURAL				40,738
D-9	**ELECTRICAL**				
	Remove receptacle and install in new position	6	EA	60.00	360
	Remove light switch and install in new position	3	EA	60.00	180
	Sub-Total				540
	General Conditions, Overhead and Profit			20%	108
	Contingency			10%	65
	ELECTRICAL				713

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APPLIED MATERIALS & ENGINEERING, INC.
980 41st Street
Oakland, CA 94608
Tel: (510) 420-8190
FAX: (510) 420-8186

September 19, 1996

Mr. Bret Lizundia
RUTHERFORD & CHEKENE
303, Second Street, Suite 800 North
San Francisco, CA 94107

Project Number 96342T

Subject: Testing and Inspection Fees
Development of Procedures to Enhance the
Performance of Rehabilitated URM Buildings

Dear Bret,

We have prepared structural testing and inspection estimates for the following wall enhancement techniques:

1. Grout and Epoxy Injection.
2. Surface Coatings.
3. Adhered Fabric.
4. Shotcrete Overlay.
5. Reinforced Cores.
6. Post-Tensioned Masonry.
7. Steel Bracing.

For each scheme, testing and inspection fees are determined based on the shaded areas shown in Figure D-10 and D-11, "Model Building and Facades".

SCOPE OF WORK & FEES

1. Figure D-1: Grout and Epoxy Injection
Basis: Wall Area: 2656 sq. ft.
One crew utilized for grouting. A crew grouts approximately 400 sq. ft. per day. Days of inspection: 6.

a)	Six, 8 hour trips for inspection; 48.0 hours @ $ 55.00 per hour	$ 2,640.00
b)	Six sets of 3 grout compression samples: 18 grout test cylinders @ $ 30.00 each	540.00
c)	Administration: 2.0 hours staff engineer @ $ 80.00 per hour	160.00
	Sub-Total	$ 3,340.00

2. Figure D-2: Surface Coatings

Basis: Wall Area: 5312 sq. ft.

One crew utilized for coating. Each crew coats approximately 500 sq. ft. per day. Two days for inspection of surface preparation and hardware cloth installation. Ten days for coating application inspection.

a)	Twelve, 8 hour trips for inspection; 96.0 hours @ $ 55.00 per hour	$ 5,280.00
b)	Ten sets of 3 plaster cementitious compression samples; 30 cementitious plaster test cylinders @ $ 30.00 each	900.00
c)	Administration: 2.0 hours staff engineer @ $ 80.00 per hour	160.00
	Sub-Total	$ 6,340.00

3. Figure D-3: Adhered Fabric

Basis: Wall Area: 5152 sq. ft.

One crew utilized for coating. Each crew prepares and coats approximately 300 sq. ft. per day. Days of inspection: 17.

a)	Seventeen, 8 hour trips for inspection; 136.0 hours @ $ 55.00 per hour	$ 7,480.00
b)	ASTM D3039 Composite Sample Tensile Tests: 17 tests @ $ 500.00 each	8,500.00
c)	Administration: 2.0 hours staff engineer @ $ 80.00 per hour	160.00
	Sub-Total	$16,140.00

4. Figure D-4: Shotcrete Overlay

Basis: Wall Area: 1708 sq. ft.

One crew utilized for shotcreting. Crew shotcretes 850 sq. ft. per day. Days of inspection: 2.

a)	Test Panel Inspection:	
	One, 4 hour trip; 4.0 hours @ $ 55.00 per hour	$ 220.00
	One set of 3 cores @ $ 100.00 per hour	300.00
b)	Dowel Installation and Testing: Three, 8 hour trips; 24.0 hours @ $ 55.00 per hour	1,320.00
c)	Concrete Placement Inspection of Grade Beam: Two, 4 hour trips; 8.0 hours @ $ 55.00 per hour	440.00

d) Concrete Cylinder Tests:
Two sets of 3 concrete cylinders @ $ 95.00 per set — 190.00

e) Shotcrete Placement and Testing:
Two, 8 hour trips; 16.0 hours @ $ 55.00 per hour — 880.00
Two sets of 3 shotcrete cores; 6 cores @ $ 100.00 each — 600.00

f) Administration:
2.0 hours staff engineer @ $ 80.00 per hour — 160.00
Sub-Total $ 4,110.00

5. Figure D-5: Reinforced Cores
Basis: Twenty, 42-1/2 feet deep cores. Initial inspection of coring operation for a day. Cores to be grouted in two days. Inspection time: 3 days.

a) Three, 8 hour trips; 24.0 hours @ $ 55.00 per hour — $ 1,320.00

b) Test 2 sets of 3 grout samples:
6 grout cylinder tests @ $ 30.00 each — 180.00

c) Administration: 2.0 hours staff engineer @ $ 80.00 per hour — 160.00
Sub-Total $ 1,660.00

6. Figure D-6: Post-Tensioned Masonry
Basis: Twenty, 43-1/2 feet deep post tensioned cores. Initial inspection of coring operation for a day. Primary grouting in one day. Tensioning/secondary grouting two days. Inspection time: 4 days.

a) Four, 8 hour trips; 32.0 hours @ $ 55.00 per hour — $ 1,760.00

b) Test 2 sets of 3 grout cylinders;
6 grout cylinders @ $ 30.00 each — 180.00

c) Administration:
2.0 hours staff engineer @ $ 80.00 per hour — 160.00
Sub-Total $ 2,100.00

7. Figure D-9: Steel Bracing
Basis:
a) Two, 1 story and two, 3 story braced frames.
b) Single-pass fillet welds to be inspected intermittently.
c) Shop welding inspection: 5 days.
d) Field welding inspection: 10 days.

a) Concrete placement inspection of grade beams:

b)	Concrete Cylinder Tests: Two sets of 3 cylinders @ $ 95.00 per set		$ 190.00
c)	Review of Welding Procedure Specifications: 4.0 hours staff engineer @ $ 80.00 per hour		320.00
d)	Shop and Field Welding Inspection: Three, 8 hour trips; Twelve, 4 hour trips; 72.0 hours @ $ 55.00 per hour		3,960.00
e)	Administration: 2.0 hours staff engineer @ $ 80.00 per hour		160.00
		Sub-Total	$ 5,070.00

Please call if you have questions regarding the above.

Sincerely,

APPLIED MATERIALS & ENGINEERING, INC.

Dushyant Manmohan
Principal