Paper No. 1417. (quote when citing this article)
Eleventh World Conference on Earthquake Engineering
ISBN: 0 08 042822 3

11 WCEE

PERFORMANCE OF RETROFITTED UNREINFORCED MASONRY BUILDINGS

BRIAN E. KEHOE

Wiss, Janney, Elstner Associates, Inc.
Emeryville, California U.S.A.

ABSTRACT

Unreinforced Masonry (URM) buildings have been known to perform poorly in earthquakes. Starting with the 1933 Long Beach earthquake jurisdictions have identified that these buildings are potentially hazardous and have implemented design requirements to reduce the life safety hazard of these buildings.

Initially, codes were enacted to improve the performance of new masonry construction. Research and experimentation have identified methods to reduce certain weaknesses in typical URM buildings that can be corrected to improve performance. This has led to the development of methodologies for the seismic retrofit of URM buildings. These methodologies are generally aimed at improving the life safety of the buildings without bringing the building into compliance with current code requirements.

The City of Los Angeles and many surrounding areas had enacted a program of mandatory seismic retrofit of URM buildings. A large number of these buildings had been retrofitted prior to the 1994 Northridge earthquake. More than 40 URM buildings were surveyed and additional data was obtained from building departments to evaluate the performance of the retrofit methodology.

Retrofitted URM buildings sustained less damage than URM buildings that had not been retrofitted. However, some buildings that had been retrofitted using the current methodologies were damaged. Although structural damage would have been expected, some buildings sustained damage in excess of what would be expected for life safety. Most of the damage in areas of strong shaking appeared to follow typical patterns. Evaluation of the performance of selected damaged URM buildings has identified aspects of the actual performance that are not properly considered in retrofit designs and in the retrofit methodology. Recommendations are made to improve the design methodology of seismic retrofitting of URM buildings.

KEY WORDS

Unreinforced Masonry; Rehabilitation; Damage; Mortar

INTRODUCTION

On January 17, 1994, an earthquake, magnitude 6.7, occurred near Northridge, California, which is in the San Fernando Valley area of Los Angeles. The earthquake affected many buildings in Los Angeles and Ventura Counties, and to a lesser extent in portions of Orange County. There are a number of unreinforced masonry buildings in this area that had been seismically retrofitted prior to the earthquake. The purpose of this study was to assess the performance of the buildings that had been retrofitted.

From February to June, 1994, to June, 1994, more than 40 unreinforced masonry buildings that had been seismically retrofitted were surveyed. In addition, data was gathered from the building departments of some of the cities in the affected areas including the retrofit criteria in effect prior to the earthquake, the number of unreinforced masonry buildings in the city that had been retrofitted prior to the earthquake, and the number of unreinforced masonry buildings that were damaged. Most of the buildings surveyed were in Los Angeles County. Several others were in Ventura County. Ground motion instrumentation was indicated peak ground accelerations in the vicinity of most of the areas in the survey exceeded 10 percent of gravity (Darragh, 1994).

REHABILITATION CRITERIA

There were many buildings in the area of Los Angeles affected by the January 17, 1994 Northridge Earthquake that had been seismically rehabilitated. A majority of these buildings were unreinforced masonry buildings in Los Angeles County that had been strengthened according to LA's Division 88, adopted in January 1981. This was a mandatory strengthening ordinance that applied to all unreinforced masonry buildings in the city. The methodology used for Division 88 is based on the ABK procedure (ABK, 1984). The methodology applies only to bearing wall buildings. This procedure intends to substantially reduce the life safety hazards associated with unreinforced masonry bearing wall buildings. The procedure concentrates on evaluation of the in-plane and out-of-plane strength of the exterior walls and on the lateral capacity of the horizontal diaphragms. These items had been shown to present the greatest collapse hazards. The ABK method reduces the collapse hazard associated with unreinforced masonry buildings. The procedure is not intended to bring the building up to the force level or detailing required for current construction.

All of the cities within Los Angeles County had enacted ordinances for rehabilitated unreinforced masonry buildings. This ordinance applies specifically to buildings with unreinforced masonry bearing walls. Nearly all of the unreinforced masonry bearing wall buildings in the City of Los Angeles, and many in the other cities, had been rehabilitated at the time of the earthquake.

The State of California ordinance that required mitigation of the hazards of unreinforced masonry buildings (SB 547-1984) applies to both bearing wall buildings and buildings with masonry infill. Standards for evaluation and design of seismic rehabilitation for masonry bearing wall buildings have been developed and used by the cities in the Los Angeles area for their unreinforced masonry building ordinances. Since there are no accepted standards for rehabilitated buildings with unreinforced masonry infill, there are fewer of these buildings that have been strengthened.

Many other cities in the affected area also had mandatory ordinances for seismic rehabilitation of unreinforced masonry buildings. The upgrade criteria used by these cities varied, but were generally based on a Division 88 or the Uniform Code for Building Conservation (UCBC) (ICBO, 1991), which includes the ABK procedure.

Most of the cities were near the completion of the program for bearing wall buildings, meaning that most of the buildings identified as unreinforced masonry had either been rehabilitated, or were in the process of rehabilitation, at the time of the earthquake.

Most of the buildings in the affected area had been rehabilitated by anchoring the exterior walls to the floor and roof diaphragms. These are typically wood floor and roof diaphragms that were anchored to the exterior masonry walls using through bolts attached to the floor joists.

Parapet bracing was typically done using diagonal steel braces. The only instance of problems with these braces was one building that had the braces installed too far below the top of the parapet so the parapet cracked and fell off above the brace. Several buildings replaced a portion of the parapet with reinforced concrete as a bracing method.

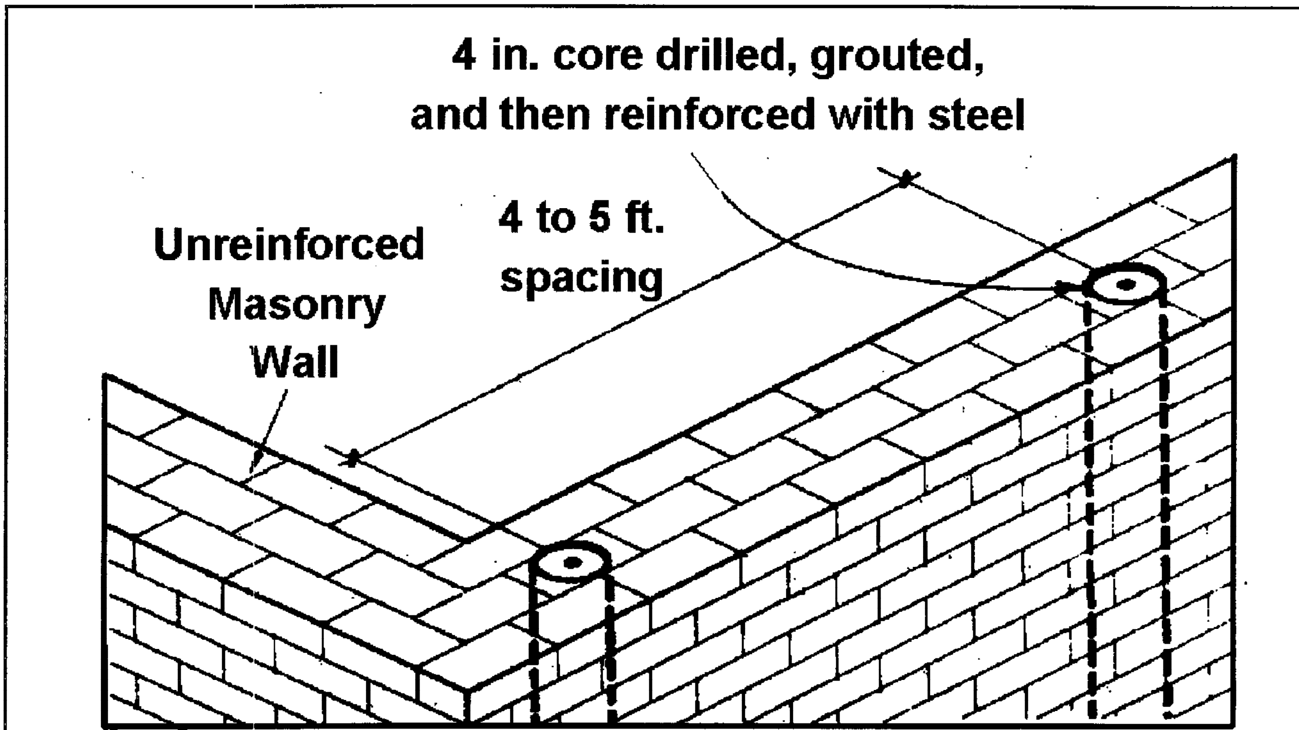

Figure 1 Center Core Technique

Several methods were used to brace the exterior walls for out-of-plane forces when the height to thickness ratio exceeded the allowable values. Two buildings, which were surveyed, used the center core technique. Several buildings used pilasters bolted through the walls. Other buildings used diagonal braces attaching to the floor or roof beams to the exterior walls at a height that reduces the height to thickness ratio. There was no damage associated with buildings using pilasters to strengthen the out-of-plane resistance of the walls. There were reports of damage due to out-of-plane movement of walls that were braced using diagonal braces.

The in-plane capacity of the masonry walls in some buildings was augmented. In some cases, window openings in the exterior wall were infilled with concrete or concrete block to create larger section of shear walls. The infilling of the window openings with concrete or concrete block, in some cases, created an area of increased damage. The new elements were stiffer, thus attracting more seismic force to the surrounding masonry.

Several buildings had shotcrete or gunite applied to the exterior face of the wall. These walls had a layer of the exterior brick removed to reduce the total weight on the foundation. One building had steel braced frames added to increase the strength and stiffness of the building in one direction.

Two buildings in the area that experienced strong ground shaking had been rehabilitated using the center core technique were surveyed. In this technique, a core is drilled down through the center of the exterior walls. The coring is spaced regularly along the exterior walls. Reinforcing is placed into the core holes and then the holes are filled with epoxy grout, as shown in Fig. 1. The purpose of the techniques is to provide

Table 1 Summary of Rehabilitated URM Data

City	Number of Rehabilitated URM's	Number of Damaged Rehabilitated URM's (Tagging)
Burbank	16	1 (Red)*
Glendale	267	17 (Red)*
Los Angeles	8242	146**
Moorpark	4	0
Santa Monica	89	22 (Yellow) 3 (Red)
Total	6276	>455

* Number of yellow tagged buildings not available.

** Damage greater than 10 percent

out-of-plane strength for the wall without the visual appearance of a new pilaster or strongback. The grouting and reinforcing also provide an increase in the in-plane strength of the wall.

RESULTS

Data gathered for retrofitted URM buildings included the posting of the buildings based on the post-earthquake damage assessments performed by the building departments, using the Applied Technology Council ATC-20 procedure (ATC, 1991). Table 1 contains a summary of the data on rehabilitation of unreinforced masonry buildings gathered from building departments in the area affected by the Northridge Earthquake. A task force established by the City of Los Angeles also surveyed damaged URM buildings and compiled data from the city's URM program (Schmid, 1995). The following results are based on our survey of forty-six rehabilitated unreinforced masonry buildings.

The buildings rehabilitated with the center core technique sustanied less dmage than other URM buildings located nearby. Both of the buildings strengthened using the center core technique were found to have damage to the exterior walls. The damage included diagonal cracking and falling bricks. The center core technique does not provide anchorage to all of the bricks in the wall, since the core is done at regular spacing along the length of the wall. The damage occurred between the strengthened cores.

Proper Retrofitting Improved Performance

Unreinforced masonry buildings that had been rehabilitated to one of the accepted design standards performed better than those buildings that had not been rehabilitated. While this may seem obvious, because of the large number of damaged unreinforced masonry buildings that had been rehabilitated, it is necessary to point out that those buildings in the same area that had not been rehabilitated were usually damaged more severely.

In the town of Moorpark, located about 33 km from the epicenter, at the time of the earthquake, there were five URM buildings. All of them are one or two stories high and are located within about 1 km of each other.

The peak ground acceleration measured nearby was 0.30 g. A survey of three of these buildings that had been rehabilitated prior to the earthquake did not indicate any damage. A similar building nearby that had not been rehabilitated suffered severe damage due to a parapet collapse that caused the top section of the wall to fall.

In the town of Fillmore, California, there were several building that did not have a complete retrofit. In one case, floor ties had been installed at the second floor line, but not at the roof line. The upper story exterior walls of this building collapsed. In another case, floor ties were installed along the longitudinal sides of the building, but not along the front. The masonry along the front of the building fell out.

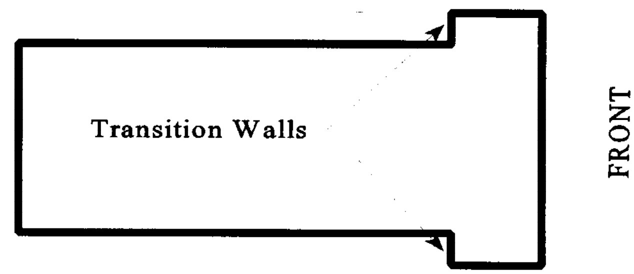

Figure. 2. Hammerhead Configuration

Configuration

Buildings with configuration problems experienced increased damage. Many apartment buildings in the Los Angeles area are T-shaped in plan, with small projecting ends at the front of the building, as shown in Fig 2. This is referred to as a hammerhead configuration. All of the buildings surveyed with this configuration experienced damage to the small transition wall where the building gets wider.

There are two factors that contribute to this type of damage. One factor is that the transition walls typically have window openings and low dead load compression stresses. This reduces the shear strength of these walls. The other factor is the difference in relative rigidity between the side walls of the front wings and the side walls of the remainder of the building. This causes twisting of the diaphragm in the wings that causes distortion of the transition walls.

Mortar Quality

The standards for retrofitting URM buildings recognize that the strength of the mortar has an effect on the in-plane strength of the masonry wall. A series of tests are specified to determine the shear strength of the mortar. The results of the tests are used, with a factor of safety as prescribed in the UCBC or Division 88. This value is then used to assign allowable in-plane shear stresses for the walls. The mortar strength tests however, are not considered in the evaluation of the walls for out-of-plane strength.

The quality of the mortar used in the construction of the walls also has a pronounced effect of the poor performance of the walls for out-of-plane response. Some of the potentially serious collapses occurred due to out-of-plane failure of the walls. An example of how the quality of the mortar may have affected the performance was found along Hollywood Boulevard in Los Angeles. Three similar URM buildings were located adjacent to each other. Two of the buildings sustained significant damage, while the third was undamaged. The mortar of the damaged buildings could be easily scraped away with a thumb nail. The mortar of the undamaged building was much stronger, and could not be easily scraped. Buildings that had damage typically had mortar that appeared to be weak.

The strength of the mortar also affects the strength of the anchor ties to resist pull-out forces. Most of the connections anchoring the exterior walls to the diaphragm were done with bolts drilled through the exterior walls and anchored to a steel plate located on the outside of the building. For nearly all of the wall failures, the anchor plate pulled through the masonry wall. This would indicate that is was the strength of the wall was the weak link and not the strength of the anchor rod or the diaphragm.

Several buildings were observed to have bricks that shifted during the earthquake. The bricks that shifted were located between the wall or parapet anchors. In most cases this occurred in the upper sections of the parapet due to weakness of the mortar surrounding the bricks and the lack of overburden walls.

Brick Veneer

Veneer brick is not always adequately anchored to the wall to prevent the bricks form falling out. Many building that had been rehabilitated experienced brick veneer falling off of the building. Much of the veneer brick was not attached via mortar to the remainder of the wall. In some cases, the veneer had been originally anchored to the structural wall by the means of sheet metal ties, which were inadequate and too widely spaced to hold the veneer brick to the building. None of the buildings surveyed had any signs of supplemental ties to anchor the brick veneer. The strength of the mortar may have affected to strength of the existing ties to restrain the veneer brick.

Many of the walls that failed out-of-plane, were observed to two wythes of brick with a third layer of brick veneer. The veneer wythe does not add to the out-of-plane strength of the wall and should not be considered in assessing the adequacy of the height-to-thickness ratio. The height-to-thickness ratio of some of these walls appeared to exceed that allowed for a two wythe wall. Apparently, the walls had been mistaken for a three wythe wall.

Corner Piers

The typical strengthening methods did not prevent damage at the corners of the buildings. Many of the unreinforced masonry buildings that were damaged experienced cracking of the masonry walls at the corners of the building. This usually occurred between a window or door opening and the corner at the top story. In these cases a diagonal cracks usually initiates from the top corner of the window to the corner before the end pier can rock, as shown in Fig. 3. In some cases, large portions of the corner fell off of the building.

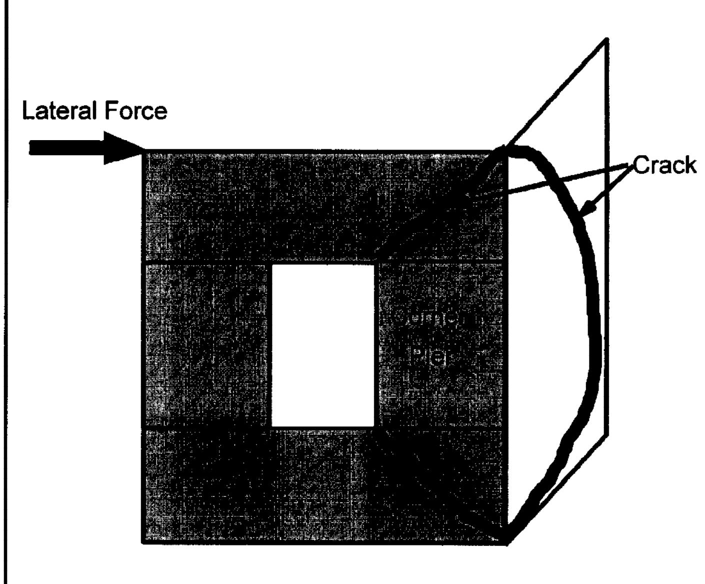

Figure 3 Corner Pier Cracking

The "Rocking Block" theory is used in both the City of Los Angeles Division 88 and the UCBC's Special Procedure for retrofitting URM's. This procedure checks the piers to verify that they will rock before they fail in shear. If the piers have sufficient shear strength, they will rock as a rigid block and not create a shear crack in the pier. A shear crack would be detrimental since the vertical capacity of the pier is reduced as the pier cracks. Rocking of the piers is considered to have stable dynamic behavior (SEAOC, 1992).

The rocking block methodology does not appear to adequately address the conditions at the upper story corner piers. For the pier to rock, the lateral force produces a moment at the top of the pier. This moment is resisted by moments in the spandrels on either side of the pier. When the flexural strength of the pier is small compared to the strength of the spandrels, a flexural crack will occur at the top of the pier. Since the pier is unreinforced, the maximum shear force that can be transferred to the pier will have been reached and a shear crack will not develop in the pier. The dead load on the pier increases the cracking strength, since the dead load stress in the pier must be overcome to develop a net tension in the masonry of the pier. The dead load also helps to provide stability to the block while it rocks.

Figure 4 Free Body Diagram of Corner Pier

In the typical areas of most buildings the depth of the spandrel is usually deeper than the width of the pier. Therefore, the moment at the joint will crack the top of the pier. At the top corners of the building, there are three conditions that prevent this theory from working. First, the moment at the top of the pier is resisted by only one spandrel beam. Second, the moment capacity of the pier is enhanced by the masonry around the corner of the wall. Third, at the top of the building, there is little dead load on the pier that needs to be overcome to produce a net tension - and thus cracking - due to the moment. These factors produce a condition where the rocking block cannot occur. A free body diagram at the unreinforced joint at the top corner demonstrates that a diagonal crack will develop, as shown if Fig. 4.

The moment in the pier is resisted by a moment in the spandrel These moments create diagonal tension through the joint. The strength of the joint to resist the diagonal tension controls the strength at the corner. Without additional strengthening, the joint develops a crack, which can then propagate around the corner of the building.

RECOMMENDATIONS

The standard methods for rehabilitating unreinforced masonry buildings are intended to protect life safety. URM buildings that had been retrofitted using these procedures prior to the Northridge earthquake sustained less damage than buildings that had not been retrofitted or had been partially retrofitted. The standard retrofit procedures for URM's do not eliminate damage to the buildings. Much of the damage observed to some of the retrofitted URM buildings required extensive repairs, which may necessitate the need for demolition or complete rebuilding of the structure following a moderate earthquake. Although there was no loss of life due to URM buildings, several URM's that had been retrofitted suffered partial collapses that may have caused serious injury had pedestrians been in the vicinity. This performance indicates that the existing methodology has some problems with the theory or the implementation.

One of the typical problems observed was cracking at the top corners of the building. Review of the statics at these joints indicates that the corner pier do not always follow the rocking block concept. The flexural strength of the pier can be limited by introducing a saw cut or other artificial joint across the width of the pier. Alternately, the shear strength of the joint at the top corner can be increased by reinforcing the joint with rebar or with concrete applied to the inside of the joint.

Other considerations in retrofitting URM's should include the effect of the strength of the mortar on the out-of-plane strength of the exterior walls. Low mortar strength also affects the strength of the masonry ties to prevent pullout of the anchors. Engineers also need to provide positive anchorage of veneer brick since existing sheet metal ties are often not sufficient when the mortar strength has deteriorated. Epoxy anchors, spaced regularly, should be added to anchor the veneer.

Mandatory seismic strengthening ordinances enacted in the Los Angeles area reduced the damage to unreinforced masonry buildings compared with similar buildings that had not been strengthened. Owners of unreinforced masonry buildings receiving strengthening need to be aware that the minimum requirements are only intended to reduce the life safety hazard and will not prevent substantial damage to the buildings. Building owners must then decide whether strengthening to minimum standards will be cost effective.

ACKNOWLEDGEMENT

The work that provides the basis for this report was funded by the Federal Emergency Management Agency, Mitigation Directorate, under the National earthquake Technical Assistance Contract, EMW-92-C-3852. The authors are solely responsible for the accuracy of the statements and interpretations in the report. Such interpretations do not necessarily reflect the views of the Federal Government.

REFERENCES

ABK A Joint Venture *(1984). Methodology for Mitigation of Seismic Hazards in Existing Unreinforced Masonry Buildings: The Methodology.*

Applied Technology Council (1991). *Procedures for Postearthquake Safety Evaluation of Buildings.*

Darragh, R. Cao, T, Huang, M., and Shakal, A. (1994) *Processed CSMIPS Strong-Motion Records From the Northridge, California Earthquake of January 17, 1994*: Releases 1, 2 and 3., California Department of Conservation, Sacramento, California.

International Conference of Building Officials (1991*). Uniform Code for Building Conservation.* Appendix Chapter 1, International Conference of Building Officials, Whittier, California.

Schmid, B. L. (1995). Performance of Unreinforced Masonry Bearing Wall Buildings During the Northridge Earthquake. *Proceedings of the Third National Concrete & Masonry Engineering Conference*, pp 697-708.

Structural Engineers Association of California (1992). *Commentary on Appendix Chapter 1 of the Uniform Code for Building Conservation.* Structural Engineers Association of California, Los Angeles, California.