



IMPROVING SEISMIC PERFORMANCE OF UNREINFORCED MASONRY PIERS

T. R. Smith¹, D. P. Abrams²

Abstract

Flexural behavior of slender unreinforced brick masonry piers subjected to in-plane deflections is summarized relative to a new document addressing seismic rehabilitation of structures. Four rehabilitation techniques are judged relative to control piers with no rehabilitation for improving seismic strength, stiffness and deformation capacity. The techniques examined are: (a) adhered fiber-reinforced polymer strips, (b) reinforced shotcrete overlay, (c) ferro-cement surface coating, and (d) grouted reinforcing bars within drilled cores. Experimental results suggest that the displacement-based rocking behavior may be equal to, or superior to, that of retrofitted piers due to the large ductility capacity, and indicate that the new document conservatively characterizes seismic capacity for unreinforced masonry piers.

Key Words

existing buildings, earthquake engineering, rehabilitation, unreinforced masonry

1 Introduction

In modern design codes, the design strength of masonry is based on the lateral load capacity, limited by the onset of flexural cracking (MSJC 1999). Research has proven that unreinforced masonry (URM) walls can still resist lateral force after cracking and behave in elastic manner (Abrams 1992, Magenes 1997). Ductile elements dissipate energy through rocking or bed-joint sliding. This indicates that codes are conservative for seismic design because they do not take into account the ductility provided through this mechanism.

FEMA 356, *Prestandard and Commentary for the Seismic Rehabilitation of Buildings*, is a document released in November 2000 by the Federal Emergency Management Agency (FEMA) in the United States as part of the National Earthquake Hazards Reduction Program (NEHRP). The document describes procedures for analyzing and rehabilitating existing buildings using the concept of performance-based engineering. The guidelines allow an engineer or owner to specify a performance level for a building, evaluate the capacity of structural components, and prescribe a rehabilitation scheme to meet the desired performance level standards. The

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performance level is based on the amount of damage incurred to the structural components, which is determined as a function of lateral drift.

The purpose of this paper is to summarize the results of research completed by Shaun Franklin, Jaret Lynch, and Daniel Abrams on the effectiveness of rehabilitation methods for improving the seismic performance of URM shear walls (Franklin et al 2001). The research indicates that the seismic capacity of URM walls, as defined as the product of strength and a ductility factor, is greater than previously thought. Additionally, rocking behavior is indicated to be a stable mechanism for large displacements with minimal damage and little or no loss of strength. Comparing results to those predicted using FEMA 356 show that the document is overly conservative in the treatment of ductility of both nonrehabilitated and rehabilitated masonry components.

2 Outline of Experimental Work

A series of eight URM specimens were subjected to slowly applied reversals of lateral deflection to examine the relative differences in lateral in-plane strength, stiffness, and deformation capacity. The focus was to study changes in behavior of masonry piers with varying amounts of vertical compressive force as well different rehabilitation techniques, and to verify the accuracy of evaluation methods given in FEMA 356. The specimens were modelled as the lower half of a typical pier between openings in a perforated shear wall and had identical dimensions as shown in Fig. 1. The large height-to-length aspect ratio was established so that the piers would be governed by flexural behavior. The test apparatus is shown in Fig. 2.

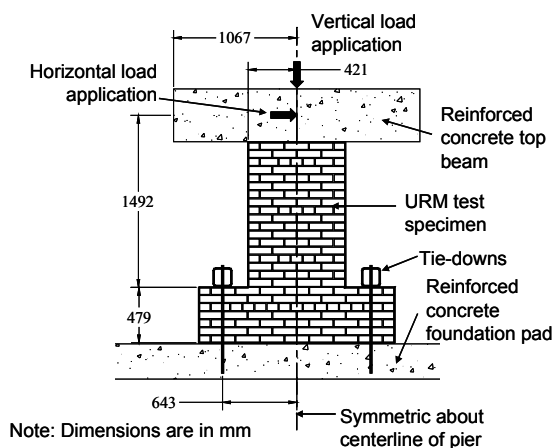


Figure 1 Test specimen

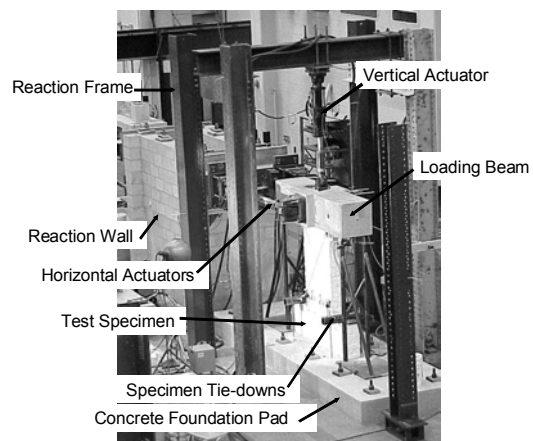


Figure 2 Test setup

Specimens 1F, 2F, and 6F were non-rehabilitated piers and served as control specimens; specimens 3F, 4F, 5F, 7F, and 8F were rehabilitated. The rehabilitation methods and loading stress level are described below with the experimental results.

3 Observations

For each of the eight test specimens, the initial stiffness, cracking load, and peak response were measured. A summary of this is listed in Table 1. It describes each specimen and reports the relevant parameters.

Table 1 Summary of experimental results

Specimen	Rehabilitation	Vertical stress	Initial stiffness	Cracking load	Net flexural tensile stress	Peak Response		
		MPa	kN/m	kN	MPa	Drift (%)	+kN	-kN
1F	None	0.29	20498	11	0.43	1.5	15	15
2F	None	0.17	15417	7	0.24	2.5	9	8
3F	FRP	0.29	16118	--	--	1.9	51	43
4F	Shotcrete	0.29	41697	41	2.49	1.5	49	45
5F	Surface Coating	0.29	31360	17	0.96	2.5	20	17
6F	None	0.59	31360	--	--	2.5	26	26
6Fb	None	0.83	--	--	--	1.5	34	34
7F	10 mm Reinforced Core	0.29	30659	27	1.32	2.0	29	31
8F	16 mm Reinforced Core	0.29	30835	29	1.35	2.5	29	39

Specimen 1F was the control pier, tested under the same compressive stress as the rehabilitated piers. The force-drift plot served as a basis of comparing energy dissipation, damage, and general behavior. The non-rehabilitated wall showed nonlinear elastic behavior after cracking, and rocked in a stable manner until 2% drift, where it was interrupted to prevent irreparable damage. The plot and a final damage photo are shown in Fig. 3. Notice that the only residual cracking occurred along the bedjoint, and nearly completely closed after completion of testing.

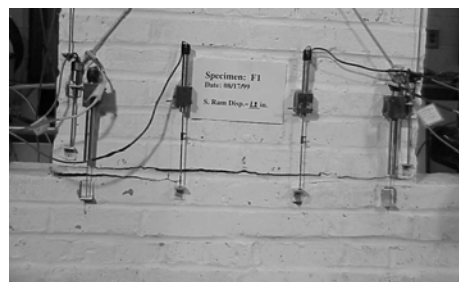
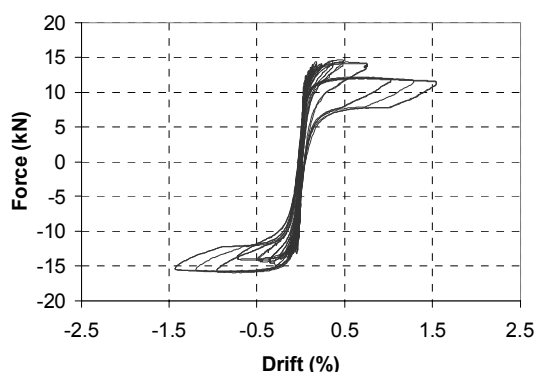


Figure 3 Force-deflection plot and final damage photo for non-rehabilitated pier

The behaviour and damage of the non-rehabilitated specimens 2F, tested under 0.17 MPa, and 6F, tested under 0.59 MPa, were very similar to that of the control specimen. Specimen 6F was predicted to fail due to toe-crushing rather than flexure; because it did not fail due to crushing with a compressive stress of 0.59 MPa, it was retested at a compressive stress of 0.83 MPa. It failed due to crushing at a load level significantly higher than predicted.

Specimen 3F was the same pier that was tested as 1F and 2F, rehabilitated with strips of 27 oz. Tyfo uni-directional glass fabric laminated to the surface with epoxy. The limiting behavior of this specimen was delamination of the FRP strips. A “U” shaped crack formed between the steel tie-down supports in the base of the pier. Large amounts of energy were dissipated through delamination of the strips in the lower portion of the pier and extensive cracking in the upper portion.

Specimen 4F was rehabilitated with 102 mm of reinforced concrete on one side of the pier to model the effects of shotcrete overlay. The specimen rocked about a base crack, elongating the vertical reinforcement into the plastic range. The force-deflection plot is shown in Fig. 4, which shows the initial linear elastic behavior followed by yielding and large quantities of energy dissipation. The strength increased slightly with increasing drift. Visible damage was confined to a “U” shaped crack along the base of the pier, as shown in the final damage photo in Fig. 4.

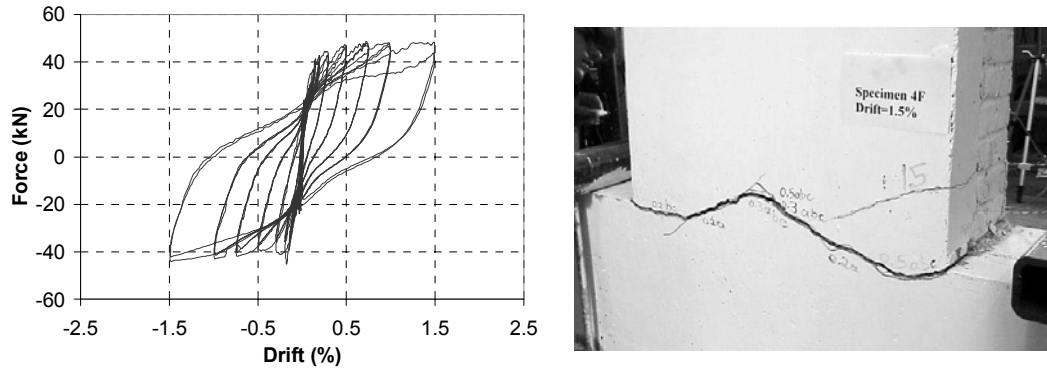


Figure 4 Force-drift plot and final damage photo for shotcrete rehabilitated pier

Specimen 5F was rehabilitated with a ferro-cement surface coating on one side of the pier, which consisted of 13 mm of concrete reinforced with 19 gage steel hardware cloth. The cloth fractured at 0.1% drift, and the wall proceeded to behave as a non-rehabilitated unreinforced masonry pier.

Specimens 7F and 8F were rehabilitated with reinforced grouted cores as indicated in Table 1. The force-drift relationship was initially linear elastic, followed by softening. The strength was considerably greater than that of the control specimen. Damage for both specimens was confined to a broad “U” shaped crack in the base of the pier. The force-drift plot and final damage photo are shown in Fig. 5 for specimen 8F.

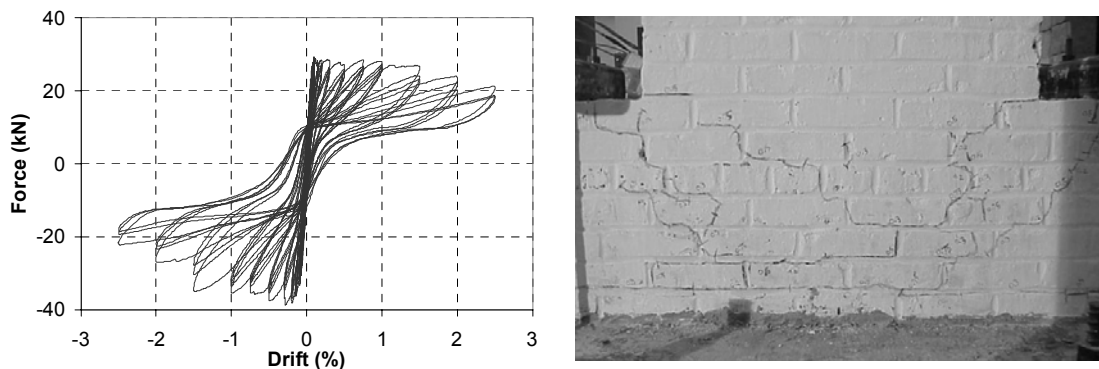


Figure 5 Force-drift plot and final damage photo for reinforced core rehabilitated pier

4 Interpretations of Behavior

4.1 Effects of Vertical Compressive Stress

The strength and initial stiffness of each test pier were increased by an increase in the axial compressive force. The axial stress was accurately included in the FEMA 356 strength estimation, but was neglected in the stiffness estimation. The m factor was decreased by increases in axial compression, and was dependent on the predicted failure mechanism (rocking or toe crushing) which FEMA did not predict accurately. The inaccuracy in prediction of the failure mechanism indicates that the transition from a deformation-controlled element to a force-controlled element is conservatively estimated.

4.2 Comparison of Estimated and Measured Strength, Stiffness, and Deformation Capacities

The non-rehabilitated strengths were calculated from Equations 7-4 and 7-6 from Section 7.4.2.2 of the *Prestandard* for rocking and toe-crushing, respectively, as applicable for the predicted failure mode. The rehabilitated strengths were evaluated according to the recommendations in Section C7.4.1.3 of the FEMA 356 *Prestandard*. Stresses were calculated using a fiber analysis following the assumptions listed in Sections 7.4.4.2.1, with the exception that an experimental stress-strain relationship was used instead of the recommended rectangular stress block.

The agreement between measured and estimated strengths was generally very good for the non-rehabilitated piers, the pier with shotcrete overlay, the pier with light reinforcement in the drilled core, and the pier with ferro-cement coating. Measured lateral strength for the more heavily reinforced grouted core was significantly less than predicted due to premature anchorage failure of one reinforcing bar, and the measured value of the FRP rehabilitated specimen was slightly less than predicted due to delamination of the strips. These values are compared in Fig. 6.

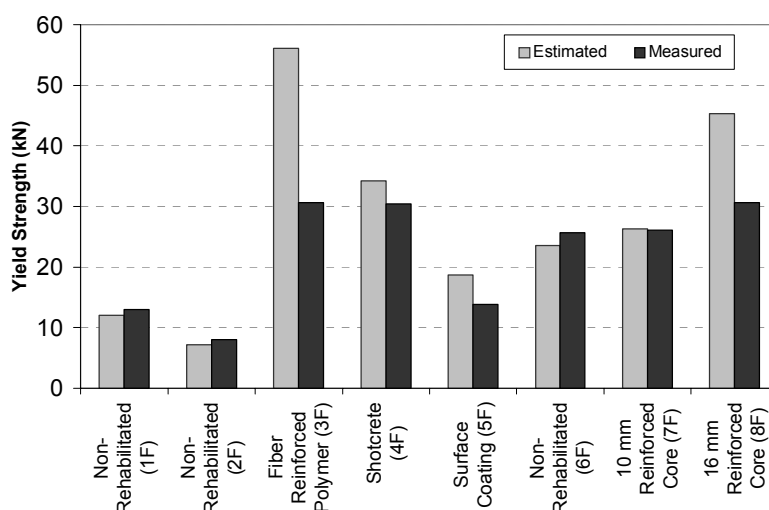


Figure 6 Strength comparison

Stiffness for non-rehabilitated specimens was estimated using Equation C7.4.2.1 of the *Prestandard*, and values for the rehabilitated specimens were calculated using the modulus of elasticity of an uncracked section in a transformed cross section. The initial stiffness of each pier was taken as the slope of the force-deflection curve within the linear range of behavior. The measured and estimated values generally agreed well, with the exception of overestimating the values for the shotcrete and FRP rehabilitation.

Deformation capacity is expressed in terms of a ductility factor, or m factor, which is defined as the deflection at a particular performance limit state divided by the apparent yield deflection. The FEMA 356 *Prestandard* prescribes values which are known to be conservative due to a margin of safety. The values are compared in Fig. 7.

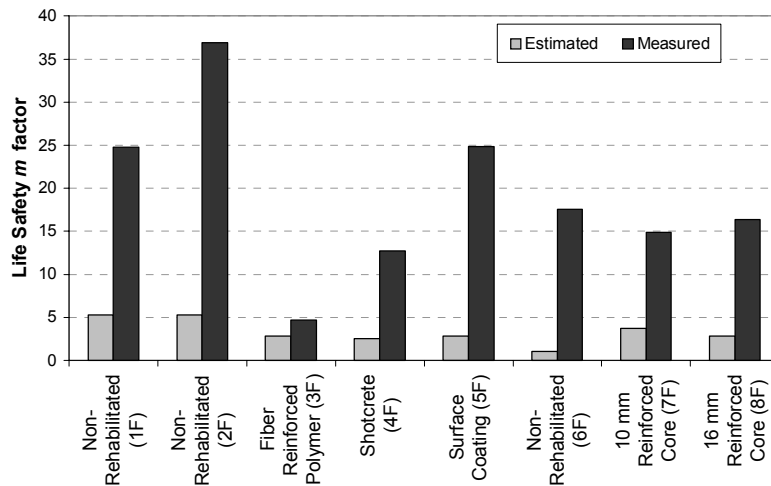


Figure 7 Measured and Estimated m factors

FEMA 356 presents a Linear Static Procedure (LSP) that establishes the premise of seismic “capacity,” defined as the product of pier strength times ductility factor. In general, the estimated product of strength times ductility is conservatively underestimated because of the conservative m factors for rocking mechanisms. This comparison is shown in Fig. 8 for the Life Safety performance level.

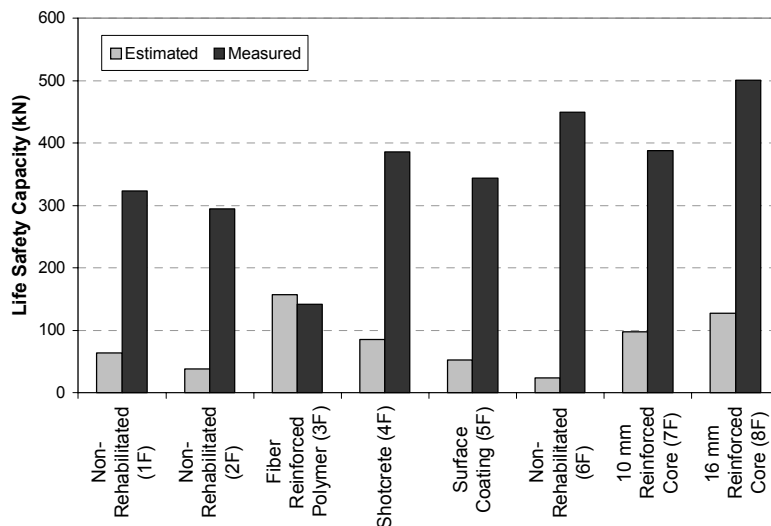


Figure 8 Comparison of capacity for life safety performance level

4.3 Effectiveness of Rehabilitation on Strength, Stiffness, and Deformation Capacity

As expected, lateral pier strength greatly increased with the application of FRP, shotcrete, and grouted reinforcement. The ferro-cement coating added little flexural strength over rocking because of the low tensile strength of the steel hardware cloth. The shotcrete rehabilitation increased the lateral stiffness. The application of FRP appeared to reduce lateral stiffness, but this may be attributable to damage incurred during previous testing to the pier.

Ductility was dependent on whether a rocking mechanism could occur; test piers with no rehabilitation and the pier with the ferro-cement coating responded with very high m factors because of rocking. The m factor was reduced considerably by the

other rehabilitation techniques that prevented rocking behavior. Ductility factors are compared in Fig. 7 for all specimens.

The ductility factor is important for determining the seismic capacity of the pier, described above as the product of strength times the ductility factor. Because of this, the product for the non-rehabilitated test piers was quite high. In comparison, although the strength of the piers was enhanced by addition of shotcrete or grouted reinforcement, the capacity was not increased dramatically, as can be seen in Fig. 8. The product for the FRP rehabilitation was decreased because of the limited ductility.

Rehabilitation of the piers generally induced more visible damage because the piers no longer displayed rocking behavior, and therefore cracking was no longer confined to one main crack. The rehabilitation did provide more energy dissipation when steel reinforcement yielded, but this also contributed to the visible damage as the reinforcement plastically deformed.

5 Conclusions

This paper has summarized the findings from research completed by Franklin, Lynch, and Abrams on the effectiveness of various rehabilitation techniques on unreinforced masonry piers. Several conclusions have been drawn from the presented information.

First, rehabilitation techniques must be considered in terms of their effect on the properties of the structural component. FRP is a good rehabilitation technique if an increase in strength is necessary but this usually is associated with a reduction in deformation capacity. Application of a shotcrete overlay can be effective in increasing the stiffness of a component though it can be expensive and disruptive to building operations. Both shotcrete overlays and grouted reinforced cores are good for increasing lateral seismic strength of a pier; however, deformation capacity of a non-rehabilitated pier through rocking may be sufficient by itself to meet seismic demand. In fact, one may consider inducing rocking through alteration of a pier's aspect ratio and/or vertical compressive stress to take advantage of the large deformation capacity of this mechanism. In all cases, the effect on the entire structural system should be addressed, and not just the effect on the component.

Second, the FEMA 365 *Prestandard* provides a conservative basis for the seismic capacity because of the large margin of safety provided in the prescribed ductility factors. This must be taken into account when evaluating a rehabilitation option using the guidelines.

Acknowledgements

Research on rehabilitation of unreinforced masonry piers was done as part of project ST-6 of the Mid-America Earthquake Center. This work was supported primarily by the Earthquake Engineering Research Centers Program of the National Science Foundation under award number EEC-9701785. The laboratory work was done by Shaun Franklin and Jaret Lynch during their graduate study toward Masters of Science degrees. Research was supervised by the Principal Investigator, Professor Daniel P. Abrams. Gratitude is extended to Omer Erbay for his general assistance.

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