Development of Low-cost and Efficient Retrofitting Technique for Unreinforced Masonry Buildings

By

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DISSEPTION
Submitted in partial fulfilment
for the requirements of the degree of Doctor of Philosophy in
Civil Engineering

Department of Civil Engineering,
University of Engineering and Technology,
Peshawar, Pakistan, 2010

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DEPARTMENT OF CIVIL ENGINEERING
UNIVERSITY OF ENGINEERING & TECHNOLOGY
PESHAWAR

DECEMBER 2010

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Entitled:

DEVELOPMENT OF LOW COST AND EFFICIENT RETROFITTING TECHNIQUE FOR UNREINFORCED MASONRY BUILDINGS

be accepted in partial fulfillment of the requirements for the degree of:

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Dedicated to my parents
ABSTRACT

The main objective of this research work was to develop a low cost and efficient retrofitting technique for masonry buildings in Pakistan using indigenous materials, technologies and local skills. Three retrofitting techniques; ferrocement overlay, bed joint reinforcement and grout injection were investigated. Ferrocement overlay, also called reinforced coating, is a technique used for rehabilitation and retrofitting of unreinforced masonry buildings in which a steel welded wire mesh is connected to the surface of masonry wall and then plastered with a rich mortar. The efficacy of reinforced plaster is dependent on the bond between masonry and the plaster coating which is established through connectors, e.g. screws or bolts, and the bond between plaster and masonry.

This study was carried out through a number of quasi-static reverse cyclic testing on isolated piers, perforated walls and single room building. Shake table test of half scale model before and after retrofitting was also carried out as part of the study. Damage patterns and the force-deformation parameters such as lateral stiffness, lateral strength, deformation capacity, energy dissipation capacity, etc. of specimens tested before and after retrofitting were studied and compared.

It was concluded that the proposed cement-based grout might restore or even improve the pre-damaged state of unreinforced brick masonry buildings. Bed joint reinforcement in combination with grout injection might be utilized for local strengthening of cracked regions. Ferrocement overlay, on the other hand, was found very effective in enhancing the overall seismic performance of unreinforced masonry buildings. When applied to both sides of walls in single and double storey unreinforced masonry building, ferrocement overlay in combination with grout injection increased the lateral strength and stiffness by more than 100% without significantly affecting the deformation and energy dissipating capacities. This technique was found to be more effective on shear-critical piers than rocking-critical piers. This technique is very simple in application because of utilization of locally available materials and with no special skill requirements. This technique is not only simple and efficient but also economical. The total cost of retrofitting (ferrocement overlay and grout injection) is less than 20% of the cost of replacement of the building.

Guidelines for the application of ferrocement overlay and grout injection and detailed procedure for the design of the unreinforced buildings retrofitted with ferrocement overlay are developed. A simple analytical model is proposed for the seismic performance evaluation of unreinforced masonry buildings retrofitted with ferrocement overlay. This model is based on results obtained from experimental work and existing formulations for unreinforced masonry buildings and reinforced concrete members and is calibrated with the test results performed as a part of this study.
Performance modification factors for rocking-critical and shear-critical piers, corresponding to various damaged levels, are also proposed that are required for the performance evaluation of damaged buildings.
ACKNOWLEDGEMENTS

The author would like to express his sincere gratitude to his PhD advisor Prof. Dr. Akhtar Naeem Khan for his technical guidance, moral support and continuous motivation throughout this research. The author is indebted to him for spending a great deal of time, in spite of his busy schedule, reviewing his work and providing him valuable suggestions. The author was greatly impressed by his simple approach towards complicated and difficult task.

Thanks are extended to Prof. Dr. Qaisar Ali for sharing his idea with the author to work on “seismic retrofitting of masonry buildings” and providing technical assistance throughout the course of this study. He always remained a source of inspiration for the author. The author did hours of discussion with him coming out with mind full of inspiration and valuable ideas.

The author would like to acknowledge technical support from Prof. Amr Elnashai, Department of Civil and Environmental Engineering, University of Illinois at Urbana-Champaign, USA. The author had interaction with him during his stay at UIUC coming up with ideas helpful in improving the thesis report.

Thanks are presented to my colleagues Dr. Amjad Naseer and Dr. Mohammad Javed with whom the author discussed his ideas throughout the course of this study. They provided him valuable suggestions because of their ample experience in experimental work. The author also appreciates the assistance provided by his PhD fellow Engr. Khan Shahzada, Engr. Mohammad Shoaib and Engr. Mohammad Fahim in the experimental phase. The author must acknowledge the efforts from laboratory technicians and other supporting staff who spent many hours with the author even after their duty timings.

Finally the author presents his greatest gratitude to the Higher Education Commission, Government of Pakistan for providing financial support. University of Engineering and Technology Peshawar is thanked for granting him the study leave and providing access to its testing facilities. The author also acknowledges the donation of Ultra Chemicals, used in the study, from AHWA Chemicals Rawalpindi, Pakistan.
SYMBOLS

Roman Letters

\[ A_G : \text{Age of building, years} \]
\[ A_v : \text{Shear area of masonry pier} \]
\[ b : \text{Shear stress distribution factor} \]
\[ C_0 : \text{ASCE modification factor relating SDOF to MDOF systems} \]
\[ C_1 : \text{ASCE modification factor relating inelastic to elastic displacement} \]
\[ C_2 : \text{ASCE modification factor representing the effect of hysteresis loops} \]
\[ c : \text{Depth of neutral axis} \]
\[ c_m : \text{Cohesion in masonry} \]
\[ E_c : \text{Modulus of elasticity of plaster coating material} \]
\[ E_d : \text{Energy dissipated per cycle of hysteresis loop} \]
\[ E_{inp} : \text{Energy input per cycle of hysteresis loop} \]
\[ E_L : \text{Estimated life of a buildings, years} \]
\[ E_m : \text{Modulus of elasticity of masonry material} \]
\[ F_c : \text{Total force in plaster coating at toe of pier} \]
\[ F_m : \text{Total force in masonry at toe of pier} \]
\[ F_s : \text{Total force in wire mesh steel} \]
\[ f_b : \text{Compressive strength of masonry units, bricks} \]
\[ f_c : \text{Compressive strength of mortar cube} \]
\[ f_{cr} : \text{Tensile stress corresponding to cracking of coating material} \]
\[ f_E : \text{Frequency of Equivalent SDOF system} \]
\[ f_m : \text{Compressive strength of masonry} \]
\[ f_t : \text{Bed joint tensile strength of masonry} \]
\[ f_{td} : \text{Diagonal tensile strength of masonry} \]
\[ f_y : \text{Yield strength of wire mesh} \]
\[ G_m : \text{Modulus of rigidity of masonry material} \]
\[ H_o : \text{Height to the point of inflection from the base of pier} \]
\[ H_p : \text{Height of masonry pier} \]
\[ H_v : \text{Height of wall from base of building to the roof centre} \]
\[ I_g : \text{Gross moment of inertia of masonry pier} \]
$K_C$ : Cost of Retrofitting  
$K_P$ : Stiffness of pier  
$K_T$ : Cost of replacement of a structure  
$K_{RL}$ : Relative displacement of spandrels and pier  
$K_{ST}$ : Stiffness of top spandrel  
$K_{SB}$ : Stiffness of bottom spandrel  
$k$ : Vertical stress distribution factor in equivalent stress block  
$k_{eff}$ : Effective stiffness of masonry pier  
$k_E$ : Stiffness of equivalent SDOF system  
$k_b$ : Lateral Stiffness of a building  
$k_p$ : Elastic stiffness of masonry pier  
$L_p$ : Length of masonry pier  
$M$ : Mass matrix  
$M_B$ : Moment at the bottom of masonry pier  
$M_T$ : Moment at the top of masonry pier  
$m$ : Mass  
$m_E$ : Mass of equivalent SDOF system  
$m_i$ : Mass corresponding to storey level-i  
$n_p$ : Number of piers of a building in a certain direction  
$P$ : Total vertical load on pier  
$p$ : Vertical stress on masonry pier  
$p_g$ : Percentage of the remaining value of a structure  
$R_A$ : Response modification factor  
$S_a$ : Spectral acceleration  
$S_d$ : Spectral displacement  
$T_e$ : Effective fundament period of structure  
$t_c$ : Total thickness of plaster coating material  
$t_p$ : Thickness of masonry pier  
$V_b$ : Lateral capacity of a building  
$V_c$ : Shear capacity of plaster coating material  
$V_{cr}$ : Lateral load of pier corresponding to flexural cracking  
$V_d$ : Diagonal shear strength  
$V_{dc}$ : Lateral load of pier corresponding to diagonal cracking  
$V_l$ : Lateral load capacity of masonry pier  
$V_{max}$ : Peak lateral load from experimental force-deformation curve
$V_r$ : Residual Lateral load of pier  
$V_s$ : Shear capacity of steel welded wire mesh  
$V_{sl}$ : Lateral load of pier corresponding to shear sliding  
$V_{tc}$ : Lateral load of pier corresponding to toe crushing  
$V_u$ : Ultimate load from idealized force-deformation curve

**Greek Letters**

$\alpha_v$ : Shear ratio of masonry pier  
$\alpha_1$ : Ratio of stiffness of top spandrel to stiffness of bottom spandrel  
$\beta$ : Ratio of effective stiffness to elastic stiffness of pier  
$\psi$ : Boundary condition factor of masonry pier  
$\mu_n$ : Coefficient of friction of masonry material  
$\Delta$ : Displacement demand  
$\Delta_{cr}$ : Displacement at flexural cracking of pier  
$\Delta_{crw}$ : Roof level displacement corresponding to flexural cracking of pier  
$\Delta_e$ : Elastic displacement  
$\Delta_{tc}$ : Displacement at top of pier corresponding to toe crushing  
$\Delta_y$ : Yield displacement  
$\Delta_{yw}$ : Roof level displacement corresponding to yielding of pier  
$\Delta_u$ : Ultimate displacement  
$\Delta_{uw}$ : Roof level displacement corresponding to ultimate pier displacement  
$\mu_D$ : Displacement ductility  
$\mu_\Delta$ : Displacement ductility corresponding to displacement $\Delta$  
$\xi_{eq}$ : Equivalent viscous damping ratio  
$\rho_s$ : Reinforcement ratio, ratio of steel area to the gross area  
$\phi$ : Normalized mode shape with ordinate equal to one at top floor  
$\phi_i$ : Mode shape corresponding to any storey-i  
$\phi_h$ : Curvature at any height $h$ from the base of pier  
$\phi_{tc}$ : Curvature corresponding to toe crushing at the base of pier  
$\Gamma$ : Modal mass participation factor  
$\delta_t$ : Target displacement  
$\gamma_m$ : Specific weight of masonry material  
$\lambda_k$ : Stiffness modification factor for damaged pier
$\lambda_v$ : Strength modification factor for damaged pier

$\lambda_\Delta$ : Deformation modification factor for damaged pier
<table>
<thead>
<tr>
<th>ACRONYMS</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>ACI</td>
<td>American Concrete Institute</td>
</tr>
<tr>
<td>AFRP</td>
<td>Aramid Fiber Reinforced Polymer</td>
</tr>
<tr>
<td>ASCE</td>
<td>American Society of Civil Engineers</td>
</tr>
<tr>
<td>ASTM</td>
<td>American Society for Testing and Materials</td>
</tr>
<tr>
<td>CFRP</td>
<td>Carbon Fiber Reinforced Polymer</td>
</tr>
<tr>
<td>COV</td>
<td>Coefficient Of Variation</td>
</tr>
<tr>
<td>CM</td>
<td>Confined Masonry</td>
</tr>
<tr>
<td>CMR</td>
<td>Retrofitted Confined Masonry</td>
</tr>
<tr>
<td>CP</td>
<td>Collapse Prevention performance level</td>
</tr>
<tr>
<td>CSK</td>
<td>Cement Sand Khaka</td>
</tr>
<tr>
<td>CSM</td>
<td>Cement Sand Mortar</td>
</tr>
<tr>
<td>CSR</td>
<td>Composite Schedule of Rates</td>
</tr>
<tr>
<td>DS</td>
<td>Double Sided</td>
</tr>
<tr>
<td>FEMA</td>
<td>Federal Emergency Management Agency</td>
</tr>
<tr>
<td>GFRP</td>
<td>Glass Fiber Reinforced Polymer</td>
</tr>
<tr>
<td>IO</td>
<td>Immediate Occupancy performance level</td>
</tr>
<tr>
<td>KN</td>
<td>Kilo Newton</td>
</tr>
<tr>
<td>kips</td>
<td>Kilo pounds</td>
</tr>
<tr>
<td>ksi</td>
<td>Kilo pound per square inch</td>
</tr>
<tr>
<td>LS</td>
<td>Life Safety performance level</td>
</tr>
<tr>
<td>Lbs</td>
<td>Pounds</td>
</tr>
<tr>
<td>MDOF</td>
<td>Multi Degree Of Freedom</td>
</tr>
<tr>
<td>MPa</td>
<td>Mega Pascal</td>
</tr>
<tr>
<td>m</td>
<td>Meter</td>
</tr>
<tr>
<td>mm</td>
<td>Millimeter</td>
</tr>
<tr>
<td>O</td>
<td>Operational performance level</td>
</tr>
<tr>
<td>PGA</td>
<td>Peak Ground Acceleration</td>
</tr>
<tr>
<td>psi</td>
<td>Pounds per square inch</td>
</tr>
<tr>
<td>RC</td>
<td>Reinforced Concrete</td>
</tr>
<tr>
<td>SDOF</td>
<td>Single Degree Of Freedom</td>
</tr>
<tr>
<td>SEI</td>
<td>Structural Engineering Institute</td>
</tr>
<tr>
<td>SS</td>
<td>Single Sided</td>
</tr>
<tr>
<td>Abbreviation</td>
<td>Description</td>
</tr>
<tr>
<td>--------------</td>
<td>----------------------------------</td>
</tr>
<tr>
<td>SWWM</td>
<td>Steel Welded Wire Mesh</td>
</tr>
<tr>
<td>UET</td>
<td>University of Engineering and Technology</td>
</tr>
<tr>
<td>URM</td>
<td>Unreinforced Masonry</td>
</tr>
<tr>
<td>URMR</td>
<td>Retrofitted Unreinforced Masonry</td>
</tr>
</tbody>
</table>
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1. INTRODUCTION

1.1 Background and Motivation
Unreinforced masonry, because of its simplicity, durability, versatility and resistance to environmental effect, remained one of the favorable construction means for human-beings since remote ages. Unreinforced masonry can be found in different forms around the globe depending upon availability of construction materials, economic condition of the locality and local culture. In its simplest form, stones, bricks or blocks are placed one above the other with or without a binding mortar. Stone was most probably the first masonry units followed by sun-dried and fired clay bricks. Concrete blocks are the recent form of masonry units.

Most of the unreinforced masonry structures are constructed without any consideration towards their seismic resistance features. Moreover recent earthquakes around the world have proven the high seismic vulnerability of unreinforced masonry structures. Unreinforced masonry buildings are, therefore, in need of retrofitting/rehabilitation in order to improve their seismic performance.

A devastating earthquake of magnitude 7.6 struck Kashmir and Northern Areas of Pakistan on October 08, 2005 killing more than 73,000 people, injuring another 70,000 and affecting a total of 3.5 million people. According to Asian Development Bank and World Bank Report [ADB-WB 2005], the earthquake destroyed 203,579 housing units and damaged another 196,574. More than 75% of the damaged buildings consisted of stone and brick masonry constructions. A detail description of various kind of damages produced in masonry building can be found in [JKM-08].

Reconstruction of the damaged buildings after an earthquake may impose a huge demand on the national economy. Retrofitting of the damaged buildings is, therefore, a best alternative to improve the seismic performance of the damaged buildings and to reduce the associated seismic risk. Many conventional techniques, e.g. ferrocement overlay, grout injection, shotcrete overlay, reinforced concrete jacketing, application of steel elements, bed joint reinforcement, etc and non-conventional techniques, e.g. application of FRP, center core technique, post tensioning, etc are available around the world for retrofitting and rehabilitation of masonry buildings. A detailed discussion on these techniques is given in chapter No.2 of this report. These techniques are never verified to establish the requisite scientific parameters for indigenous materials and local skills. Furthermore, non-conventional techniques are costly and require special materials and technologies. A detailed study is, therefore,
required to ascertain the scientific merits of these techniques on the behaviour
of masonry buildings. Thus the problem may be stated as:

“Development of low-cost and efficient retrofitting techniques based on
scientific evidences, using indigenous materials and technologies, is must for
unreinforced masonry buildings in Pakistan”

The selected retrofitting technique shall not only be low cost but also efficient
in its implementation. Availability of local materials and skills is one of the
criteria on the basis of which a technique is selected. After a thorough
literature survey on various retrofitting technique following retrofitting
techniques were selected:

- Cement based grout injection
- Ferrocement Overlay
- Bed Joint Reinforcement

So far very little research (see Chapter No.2) has been reported on the seismic
performance of masonry buildings retrofitted with these techniques. Therefore
a scope is available to study in detail the performance of unreinforced masonry
buildings retrofitted with these techniques.

1.2 Objectives of the Research Work

This study focused on the development of cost-effective and efficient
retrofitting technique for masonry buildings in Pakistan. The primary
objectives of this research study are listed below:

1. To study, experimentally, the effect of selected retrofitting
   techniques on the following properties of masonry buildings:
   - Strength
   - Stiffness
   - Ductility
   - Energy dissipation

2. To develop guidelines for the design of retrofitted buildings

3. To develop guidelines for the application of retrofitting techniques

The findings of this research may be incorporated in Seismic Building Code of
Pakistan.

1.3 Scope and Limitations of This Research

The scope of this research includes in-plane seismic performance evaluation of
the unreinforced masonry buildings retrofitted with ferrocement overlay
through a number of static and dynamic tests. Quasi-static tests were carried
out on full scale isolated piers, masonry walls with openings and single room
masonry building before and after retrofitting to study the force-deformation
characteristics. Shake table test on half scale model of a single room was also
conducted to investigate its dynamic behavior before and after retrofitting.
Besides ferrocement overlay, the retrofitting technique of bed joint
reinforcement was also investigated by conducting quasi-static tests of isolated masonry piers.

The study is limited to brick masonry construction in the northern areas of Pakistan but can be extended to other types of masonry (stone and block Masonry) provided their material properties are experimentally investigated. The model proposed for the seismic performance evaluation is applicable only to brick masonry buildings repaired and retrofitted with grout injection and ferrocement overlay. The proposed analytical model is applicable to regular buildings (torsional effects are neglected) ignoring the effect variation in vertical stress and the flange effects. However these effects can be included after verification through some experimental or numerical studies.

1.4 Research Methodology

To achieve the research objectives following research methodology was adopted:

1. As a first step of this research various techniques and materials, available and used for seismic retrofitting of structures, around the world were explored with special emphasis on local materials and skills.

2. Based on low cost, availability of materials and skills, and efficiency in terms of application and performance, few of many available retrofitting techniques were selected for onward experimental performance evaluation.

3. An intensive and extensive experimental program shown in Figure 1.1 was devised to study the seismic performance of unreinforced brick masonry buildings retrofitted with techniques selected in step-2. Their details are as below:

- A total of sixteen unreinforced brick masonry piers made of locally available bricks placed in 1:4:4 cement-sand-khakka (stone dust) mortar, were tested under quasi-static reverse cyclic loading. First four piers, tested till complete failure could not be retrofitted. Last twelve damaged piers were retrofitted with different techniques and re-tested under the same loading conditions. Three techniques have been investigated; grout injection only, reinforced plaster with grout injection and bed joint reinforcement with grout injection. Two reinforcement ratios were used in reinforced plaster to investigate their effect on the capacity of wall.

- To investigate the effect of openings and pier-spandrel interaction two full scale perforated walls; unreinforced brick masonry wall and confined brick masonry wall, were tested under quasi-static loadings before and after retrofitting with reinforced plaster.

- The effectiveness of reinforced plaster applied on a full scale room was also investigated through quasi-static loading. The level of vertical load on in-plane walls was kept equivalent to internal walls of a single storey unreinforced masonry building.

- Finally a shake table test was conducted on half scale model of unreinforced brick masonry room to investigate the dynamic behavior after retrofitting with reinforced plaster.
4. Experimental test data was analyzed to develop guidelines for the performance evaluation of unreinforced masonry buildings retrofitted with reinforced plaster (ferrocement overlay). Moreover, guidelines for the application of grout injection and reinforced plaster were also developed to apply the techniques efficiently and effectively.

Figure 1.1: Outline of the Experimental Program

1.5 Organization of Thesis

This report titled “Development of Cost-effective and Efficient Retrofitting Technique for Masonry Buildings in Pakistan” is organized in ten chapters. The first chapter gives an introduction to the problem, research objectives and methodology adopted to achieve those objectives.

The second chapter gives an overview of the retrofitting strategies and state-of-the-art on seismic retrofitting of unreinforced masonry buildings. Various empirical relations to estimate the strength and deformation capacities of unreinforced masonry buildings, which can be extended to retrofitted buildings, are also discussed. Various static and dynamic tests used to investigate the behavior of masonry buildings are discussed towards the end of 2nd chapter.

The next four chapters from chapter No.3 through chapter No.6 deal with experimental test results and discussion. Chapters No.3 through No.5 discuss the reverse cyclic quasi-static test of isolated piers, full scale perforated walls and full scale room. Chapter No.6 elaborates on the dynamic shake table test of half scale model. A detailed description of specimens, materials, test setup,
test procedures, results before and after retrofitting and their comparison have been provided for all tests.

A model based on some simple formulations, for the seismic performance evaluation of masonry buildings before and after retrofitting with reinforced plaster, has been developed and discussed in chapter No.7 based on the experimental work from this study and by other researchers around the world. This model is also compared with full scale test performed on masonry buildings to authenticate its validity and reliability. As an example, the model is applied to a typical single storey masonry building to evaluate its seismic performance before and after retrofitting.

Chapter No.8 provides guidelines for the design of a building retrofitted with reinforced plaster. This chapter also presents guidelines for the application of grout injection and reinforced plaster to facilitate the person executing work at site.

Chapter No.9 provides cost analysis for the retrofitting or rehabilitation of a typical double storey unreinforced masonry buildings constructed in northern areas of Pakistan. The cost of retrofitting is represented as cost per unit area of the building and as percentage of the structural cost.

Finally chapter No.10 summarizes the whole project and enlist various conclusions and recommendations made based on the experimental work and numerical study on the seismic performance of unreinforced masonry buildings before and after retrofitting with reinforced plaster coating, cement based grout injection and bed joint reinforcement.
2. LITERATURE REVIEW

2.1 Introduction
This chapter deals with the theoretical background related to this research study. Starting with the definition of some basic terms used in the literature related to retrofitting/rehabilitation, retrofitting strategies and criteria are then discussed. Since before designing a retrofitting scheme the building has to be evaluated for existing damages, some guidelines for the damage evaluation are presented. A review of the empirical relations used for the capacity evaluation of unreinforced masonry is provided which is also used for the capacity evaluation of damaged and retrofitted structures with some modification. It is followed by a detailed literature review with regard to various retrofitting/rehabilitation techniques. Finally various test setup used for the in-plane quasi-static reverse cyclic load testing of masonry building and its components are discussed.

2.2 Terminologies
Repair, strengthening, seismic strengthening, seismic upgrading, rehabilitation and retrofitting are the words interchangeably used in research literature. According to the proposed terminology [MT-99] and [DT-94], word “repair” refers to the restoration of pre-earthquake state of a structure damaged by seismic ground motion without increasing its seismic resistance beyond its pre-earthquake state. “Strengthening, “seismic strengthening”, or “seismic upgrading” are the words used for technical intervention in the structural system that improve its seismic performance by increasing its strength and/or ductility. However strengthening a structure before an earthquake is called “rehabilitation”, whereas strengthening a damages structure after an earthquake is known as “retrofitting”. One can say that “retrofitting” is the combination of repair of the damages produced during earthquake to restore its pre-earthquake state and strengthening to enhance the seismic performance in future earthquakes.

2.3 Retrofitting Strategies
Various retrofitting strategies may be adopted depending upon the type and the existing condition of structure and target building performance objectives. Retrofitting strategies may broadly be classified into: (1) Increasing the seismic capacity of a structure and (2) decreasing the seismic demand on structure.
2.3.1 Retrofitting by Increasing the Capacity of Structure

The capacity of a structure may be enhanced by local or global stiffening or strengthening and/or increasing ductility, addition or removal of structural elements, etc shown in Figure 2-1 (left). Local modification in strength and/or ductility of a member or connection may be accompanied by stiffness increase resulting in an increased demand which must be taken care of in the design of retrofitted member/connection. Column/pier jacketing is an effective local modification to enhance strength/ductility.

![Figure 2-1: Retrofitting/Rehabilitation Strategies](image)

On the other hand global stiffening may be effective in case of very flexible structure with large deformation but small deformation capacity at component level. Construction of shear wall is effective measure for adding stiffness. When the overall performance of a structure is below a selected performance objective, global strengthening and ductility enhancement is the best solution. Ferrocement applied to whole structure is an effective solution to enhance the global performance of a single or double storey unreinforced masonry building.

2.3.2 Retrofitting by Decreasing Seismic Demand

The seismic demand on a structure is proportional to the mass and natural period of structural. Decrease in mass of a structure is accompanied by decrease in its natural period. Reduction in mass causes a direct decrease in the seismic demand. At the same time the period of structure will reduce which may increase the demand in case when the period falls on the descending branch of the response spectrum curve, Figure 2-1 (right). Low rise building usually falls on constant or ascending branch of response spectrum curve, reduction in mass (by reducing the number of storeys or reducing the weigh on structure) could be an effective retrofitting strategy.

Elongation of the period through base isolation of structure falling on the descending branch of the response spectrum curve will result in decrease of the seismic demand. Supplemental energy dissipation devices installed in the structure may also be effective in reducing the seismic demand as most of the energy from an earthquake is dissipated through these devices.

2.4 Retrofitting Criteria

The choice, whether to retrofit a structure or not, and the selection of appropriate retrofitting technique depend upon various factors. The most important is the existing capacity and the level of damages produced in the structure during an earthquake. If the existing structural capacity is adequate
and the damages occurred during an earthquake are within the permissible limit as predicted from the performance analysis, the structure will then be repaired in order to bring it to its pre-earthquake state. If, however, the level of damages produced during an earthquake are beyond the permissible limit, the structure then need to be retrofitted to improve its structural performance. Moreover, even if the damages occurred during past earthquakes are within the permissible limit but the predicted demand in future earthquake will be such that to produce damages beyond the permissible limit, the structure then need to be retrofitted to cope with the future demand. Other factors which should be kept in mind when deciding upon the retrofitting of a structure are:

2.4.1 Type of Structure
Different retrofitting schemes are used for different type of structures, e.g. for reinforced concrete frame structures the most effective technique is the use of fiber reinforced polymers (FRPs) for improving the performance in terms of strength and ductility.

2.4.2 Cost of retrofitting
The cost of retrofitting ($K_C$) shall not increase certain percentage ($p_g$) of the remaining value of the structure. The remaining value of building is determined by the cost of replacement ($K_R$) and the age of building ($A_G$) with respect to its estimated life ($E_L$). The criterion [RS-09] is expressed as follows:

$$K_C \leq p_g K_R \left( \frac{E_L - A_G}{E_L} \right) \quad 2-1$$

The value of ($p_g$) depends upon the type of structure and its historical importance. For a 25 years old building with estimated life of 50 years and retrofitting cost of 80% of the remaining value, the cost of retrofitting as calculated from above relation shall not increase 40% of the cost of replacement.

2.4.3 Materials, Technology and Skill Availability
The availability of suitable retrofitting materials, adequate technology and skilled workmanship are the factors that would define the retrofitting scheme. For a retrofitting scheme to be economical, it is important to utilize local materials and local technology.

2.4.4 Problem of Occupancy
In some cases the time available for retrofitting and the problem of occupancy may decide upon the type of retrofitting scheme. In that case the retrofitting scheme shall be fast enough to be completed in the available time frame.

2.4.5 Historical Importance
Similarly, historical importance of a building may decide upon the type of retrofitting scheme. The retrofitting scheme of historical buildings shall be such that it shall not alter the aesthetic of the building. The scheme may not, however, be cost-effective.
2.5 Investigation and Evaluation of Damaged Building

The main objectives of the investigation and evaluation of damaged building as listed in [RS-09] are:

- To identify structural components and elements of lateral-force-resisting system and their performance
- To observe and record damage to the components
- To distinguish, to the extent possible, between damage caused by the earthquake and damages that may have existed before

The seismic performance of a masonry building is function of the failure mode, elastic stiffness, yield strength and ultimate displacement of its individual structural components. The performance of a building is usually expressed in terms of global force-deformation curve of the whole system. The damages produced in a structural component during an earthquake may be transformed in terms of global displacement and stress. In case of irregular buildings the distribution of damages may be interpreted to estimate its torsional behavior.

Walls in a masonry building may be subjected to in-plane and out-of-plane actions depending upon the direction of earthquake force relative to the wall. Damage to a particular wall shall, therefore, be carefully examined to ascertain the type of crack. In out-of-plane action a wall is subjected to flexural action creating tensile stresses on one face of the wall and thus the cracks appear on one side of the wall. On the other hand a cracks penetrating through the thickness of the wall appearing on both sides is predominantly due to in-plane actions.

In a typical brick masonry buildings constructed in the northern areas of Pakistan, the structural walls are usually 9” thick. 4½” thick slender walls, sometimes used as partition, may be considered as non-structural components due their less resistance to in-plane and out-of-plane actions.

<table>
<thead>
<tr>
<th>Damage Level</th>
<th>Description of Damages</th>
</tr>
</thead>
<tbody>
<tr>
<td>Insignificant:</td>
<td>Damage does not significantly affect structural properties in spite of a minor loss of stiffness. Restoration measures are cosmetic unless the performance objective requires strict limits on non-structural component damage in future earthquake.</td>
</tr>
<tr>
<td>Slight:</td>
<td>Damage has small effect on structural properties. Relatively minor structural restoration measures are required for most components and behaviour modes. However this damage level is not used for unreinforced masonry.</td>
</tr>
<tr>
<td>Moderate:</td>
<td>Damage has an intermediate effect on structural properties. The scope of restoration measures depends on the component type and behaviour mode. Measures may be relatively major in some cases.</td>
</tr>
<tr>
<td>Heavy:</td>
<td>Damage has a major effect on structural properties. The scope of restoration measures is generally extensive. Replacement or enhancement of some components may be required.</td>
</tr>
<tr>
<td>Extreme:</td>
<td>Damage has reduced structural performance to unreliable levels. The scope of restoration measures generally requires replacement or enhancement of components.</td>
</tr>
</tbody>
</table>
FEMA-306 (Evaluation of Earthquake Damaged Concrete and Masonry Wall Buildings, Basic Procedure Manual) and FEMA-307 (Evaluation of Earthquake Damaged Concrete and Masonry Wall Buildings, Technical Resources) prepared by Applied Technology Council, are so far the most comprehensive documents for the investigation and evaluation of damaged concrete and masonry wall buildings. Various types of behavior modes are defined in the FEMA-306 for unreinforced masonry walls, piers and spandrels. Severity of damage for each component is classified as given in Table 2-1.

Performance based approach is used for the evaluation of damaged masonry buildings in these documents. Four performance levels as defined in ASCE standard (ASCE/SEI 41-06) namely Operational (O), Immediate Occupancy (IO), Life Safety (LS) and Collapse Prevention (CP) are used for the damage evaluation of buildings. Earthquake hazard is defined in either probabilistic manner as probability of exceedance over a defined time period or in terms of a characteristic earthquake likely to occur on a given fault. Before the evaluation of a building seismic performance objective has to be chosen. Seismic performance objective is the combination of performance level and a hazard. A common performance objective of a building may be to maintain life safety performance level when subjected to ground motion with a ten percent probability of exceedance in fifty years.

Damage caused by an earthquake may change the displacement demand and/or displacement capacity of a building. The global performance/capacity curve of a building is a function of the basic structural properties (stiffness, strength and deformation limits) of its components. For the evaluation of damaged building first the pre-event global capacity curve of building is formulated from the pre-event components data. The displacement capacity $d_c$ and displacement demand $d_d$ corresponding to the specified performance objective is then determined. The global capacity curve of the damaged building is reformulated using the modified force-deformation relationship of the components. The displacement capacity $d'_c$ and displacement demand $d'_d$ of the damaged building corresponding to the specified performance objective is then determined. The ratio $d_c/d_d$ and $d'_c/d'_d$ indicate the degree to which the building satisfies the specified performance objective in pre-event and damaged state respectively. If both the ratios are same, or nearly same, it may be said that the damage caused by the damaging earthquake has not significantly degraded the future performance for the performance objective under consideration.

2.6 Seismic Resistance Verification of Masonry Buildings

Per the ASCE standards for seismic evaluation of existing buildings [ASCE/SEI 31-03], linear or nonlinear, and static or dynamic analysis may be used to evaluate the capacity of existing building, identify potential deficiencies and to assess need for retrofitting. However a special analysis procedure is recommended for unreinforced masonry building with flexible diaphragm. According Abrams [DPA-00], for low-rise unreinforced masonry building, the simplest of the analysis methods, the linear static procedure is suitable for an assessment.
Figure 2-2: Force-Deformation Curve and ASCE/SEI 41-06 Performance Levels

The seismic performance of a building is expressed in the form of force deformation curve (Figure 2-2). Procedure used for the performance evaluation of new buildings may also be used for the seismic resistance verification of the existing buildings. The only difference is to take into account the age effect. Physical destructive and non-destructive test may be required to determine the mechanical and dynamic properties of the material used in the construction, either in laboratory or in-situ. Moreover the irregularities and connections should be properly modeled in the numerical analysis.

The in-plane walls are the stiffest part of a masonry building. If properly connected to the diaphragm, the in-plane walls transfer the earthquake forces from ground to the diaphragm as inertia forces. The out-of-plane walls may receive damage due to the excessive in-plane deflection of flexible diaphragms. Preventing the out-of-plane failure by proper measures, the lateral load carrying capacity of a masonry building thus depends on the capacity of in-plane walls. Provision of doors and windows in walls covert the in-plane walls in to a series of small piers and spandrels.

The in-plane force-deformation curve of a masonry building, in a certain direction, is obtained by superposing the in-plane force-deformation behaviour of individual piers in that direction. The force-deformation behaviour of a pier is defined by its lateral stiffness, strength and ultimate displacement.

### 2.6.1 Elastic Stiffness

The lateral in-plane elastic stiffness of a masonry cantilever piers can be estimated from the conventional principal of mechanics for homogeneous material, considering both flexural and shear deformation.

\[
k_p = 1 \left( \frac{H_p^2(3\psi - 1)}{6E_mI_g} + \frac{H_p}{A_vG_m} \right)
\]

Where \(k_p\) is the elastic lateral stiffness of piers, \(H_p\) is the pier height to the point of lateral load, \(E_m\) and \(G_m\) are modulus of elasticity and modulus of rigidity respectively, \(I_g\) is the gross moment of inertia, \(A_v\) is the shear area and \(\psi\) is a boundary condition factor varying between 0.5 (fixed ended piers) and 1.0 (cantilever piers). The relation as specified in ASCE/SEI 41-06 Section C7.3.2.1 (Eq. C7-2) may be obtained by putting \(\psi = 0.5\) for fix-fix piers and \(\psi = 1.0\) for fix-free piers.
2.6.2 Cracking Load:
Assuming the pier to be elastic before cracking, the load corresponding to
its initial in-plane flexural cracking is function of vertical stress and bed
joint tensile strength in addition to the geometric dimensions. By making
the tensile flexural stresses produced at heal due to lateral load, equal to the
bed joint tensile strength, one can obtain the following relation:

\[ V_{cr} = \frac{L_p^2 t_p}{6H_p} (p + f_t) \]  

Where \( V_{cr} \) is the cracking lateral load, \( L_p \) is the pier length, \( t_p \) is the pier
thickness, \( p \) is the total vertical stress on pier and \( f_t \) is the bed joint tensile
strength. The bed joint tensile strength is usually neglected but may play an
important role in increasing the cracking capacity of pier.

2.6.3 Strength Capacity:
Masonry, a combination of masonry units and mortar, is non-homogeneous
and anisotropic composite material. Due to the complex nature of masonry,
it is difficult to accurately predict its capacity. Depending upon the level of
vertical stress, boundary conditions, material strength and aspect ratio
(height to width ratio), masonry structural wall may behave in different in-
plane failure modes as given in Figure 2-3.

![Figure 2-3: Failure mode in masonry piers, (left) Rocking & toe crushing, (middle)
Diagonal shear cracking, (right) Shear sliding failure](image)

2.6.3.1 Flexural (Rocking) Failure Mode:
In rocking failure mode the masonry piers undergoes rigid body rotation,
thus producing cracks at the top and bottom bed joints. This type of
failure mode usually occurs in piers with large aspect ratio and low
vertical stress. Final failure may appear in the form of toe crushing due
to increased compressive stresses or walking (out-of-plane sliding). The
maximum horizontal shear which can be resisted by a rocking failure
mode under static in-plane loading may be expressed as [MC-97]:

\[ V_{rc} = \frac{L_p^2 t_p}{H_o} p \left(1 - \frac{p}{k f_m}\right) = \frac{L_p^2 t_p}{\alpha_p} p \left(1 - \frac{p}{k f_m}\right) \]  

Where \( H_o \) is the effective pier height (height from base to the point of
inflection), \( f_m \) is the compressive strength of masonry, \( k \) is the vertical
stress distribution factor at the toe of pier (\( k \) is usually taken equal to
0.85) and \( \alpha_v \) is called shear ratio which depends primarily on the
boundary condition and aspect ratio of pier. The shear ratio is given by:

\[
\alpha_v = \frac{M}{VL_p} = \frac{H_o}{L_p} = \frac{\psi H_p}{L_p}
\]

Two different equations are given in ASCE/SEI 41-06 Section 7.3.2.2
for flexural strength of masonry pier giving its rocking strength (Eq-2.5)
and toe crushing strength (Eq-2.6):

\[
V_{ro} = 0.9 \alpha \left( \frac{pL^2 t_p}{H_p} \right)
\]

\[
V_{wc} = \alpha \left( \frac{pL^2 t_p}{H_p} \right) \left( 1 - \frac{p}{0.7 f_m} \right)
\]

Where \( \alpha \) is a boundary condition factor equal to 0.5 for cantilever piers
and 1.0 for fixed-fixed piers. Eq-2.3) and Eq-2.6 are equivalent for \( \alpha = 1/2 \psi \).

2.6.3.2 Diagonal tension failure mode:
This unfavourable failure mode, appearing in the form of inclined cracks
along one or both diagonals (seizer cracks), occurs when the principal
tensile stresses, developing in a wall under a combination of horizontal
and vertical loads, exceeds masonry tensile resistance. The cracks may
follow the mortar bed and head joints or may pass through brick
depending upon the relative strength of mortar, brick-mortar interface
and bricks, and the level of vertical stress. Squat piers with large vertical
stress usually behave in diagonal tension failure mode. In case of
cantilever piers tested in laboratory under high vertical stress, it is
usually observed that the diagonal tension cracks appear well after the
appearance of flexural cracks.

The equation proposed by Turnsek and Sheppard [TS-80] for diagonal
tension failure is based on the assumption that diagonal cracking takes
place when the principle tensile stresses at the center of the pier exceed
the tensile strength of masonry:

\[
V_{dc} = \frac{f_{tu} L_p t_p}{b} \sqrt{1 + \frac{p}{f_{tu}}}
\]

Where, \( f_{tu} \) is the conventional tensile strength of masonry to be
determined by the diagonal shear test of the wall specimens. \( b \) is the
shear stress distribution factor and is dependent upon the aspect ratio.

\[
b = \frac{H_p}{L_p} \quad (1.0 \leq b \leq 1.5)
\]

The factor \( b \) is only dependent on the aspect ratio, thus neglect the effect
of boundary condition.
A second approach, applicable especially to the case where the diagonal cracking is associated with mortar bed and head joint failure, is based on Mohr-Coulomb formulation. Introducing a correction factor dependent on shear ratio, the diagonal shear strength \([MC-97]\) may be estimated from:

\[
V_d = L_p t_p \left( \frac{c_m + \mu_m P}{1 + \alpha_v} \right)
\]

Eq-2.8 was originated from the tests performed on doubly fixed piers, where the diagonal cracking is initiated at the centre of pier and therefore provides good estimate for the shear strength of fix-ended piers. However in the case of cantilever piers where the pier is already cracked due flexural stresses, the shear strength is calculated based on the effective un-cracked section length of pier. Assuming a linear compressive stress distribution the shear capacity based on cracked section may be expressed as:

\[
V_d = L_p t_p \left( 1.5c_m + \frac{\mu_m P}{1 + 3c\alpha_v / p} \right)
\]

Where \(c_m\) is the cohesion and \(\mu_m\) is the coefficient of friction.

**2.6.3.3 Shear sliding failure mode:**

Horizontal bed joint crack is developed in the case of shear sliding failure mode which usually appears in a pier with low aspect ratio, low vertical stress and poor quality mortar. This type of failure mode appears in the upper storey of a building where the vertical stress is relatively low but acceleration is high.

Neglecting the cohesion based on the assumption that the section is already cracked, the sliding shear capacity may be estimated from:

\[
V_{sl} = \mu_m pL_p t
\]

According to Magenes G. and Calvi G. M [MC-97], Eq-2.10 tends to under estimate significantly the shear capacity which correspond to the onset of sliding, because the sliding strength of a joint cracked in tension is higher than the residual sliding strength of a bed joint failing in shear. Eq-2.8 and Eq-2.9 could be considered more appropriate than Eq-2.10, if the case with a reduced value for the cohesion. However it is found that in Eq-2.9 the shear strength is inversely proportional to the cohesion and is not very much sensitive to the value of cohesion.

Shear sliding failure mode sometimes exists in combination with diagonal or flexure cracking failure modes.

**2.6.4 Deformation Capacity of Masonry Buildings**

Masonry walls, initially believed to be brittle, have been proven to be ductile elements capable of dissipating energy through rocking and bed joint sliding [ASL-07]. The deformation capacity of unreinforced masonry wall depends on the mode of failure. High deformations are notice in
A flexural-controlled wall during laboratory experiments. The shear mode of failure may be brittle exhibiting low deformation capacity.

ASCE/SEI 41-06 which is primarily based on FEMA 356, considers rocking as deformation control action and shear failure and toe crushing as force control action according to the criteria set forth by the standard (section 2.4.4.3). However, the force-deformation relations from lateral load test of piers have shown that all in-plane lateral failure modes (rocking, toe crushing, diagonal shear, etc) of masonry may be classified as deformation control action.

In ASCE/SEI 41-06 the deformation capacity of piers for deformation control action may be accounted for through m-factor (ductility factor) in linear static procedure (section 7.3.2.3.1) and by specifying the maximum deformation demand at the target displacement in non-linear static procedure (section 7.3.2.3.2) for various performance levels, Figure 2-2.

2.7 Typical Damages in Masonry Buildings

The global response of a masonry building depends upon the interconnectivity between various structural elements, i.e. in-plane walls, out-of-plane walls, diaphragm, etc. In the case where these elements are properly connected to each other, the damages are mostly concentrated in the in-plane walls appearing in the form of cracks in piers and spandrel, and at opening corners. Lack of interconnectivity and poor detailing, on the other hand, can cause local damage mechanism and the capacity of in-plane walls may not be fully utilized. The out-of-plane failure may happen due to the absence of connection between out-of-plane walls and diaphragm. Typical damages produced in a masonry building are shown in Figure 2-4. Detail discussion on the damages produced in masonry building in Oct.8 2005 Kashmir earthquake may be found in reference [JAM-08].

Damage to masonry buildings as shown in Figure 2-4 may be categorized in to the following groups:

- Damage to the out-of-plane walls appeared as horizontal cracks, usually on one side of the wall.
- Shear damage (rocking, diagonal cracking, shear sliding) to the piers of in-plane walls. The in-plane failure may be differentiated from the out-of-plane failure by examining the depth of cracks and the direction of seismic excitation.
- Corners failure appearing in the form of vertical cracks at the junction of in-plane and out-of-plane walls.
- Local failure due thrust from roof truss pushing the wall in outer directing.
- Radial cracks at the corner of openings due to stress concentration produced because of large size openings.
- Flexure failure of the spandrel.
2.8 Retrofitting Technologies

Considerable amount of research work has been carried out throughout the world on retrofitting of un-reinforced masonry in last two decades. M. Ashraf [ANN-09] and M. ElGawady [ELB-04] gave a review of various conventional and non-conventional retrofitting technologies used for URM buildings along with their relative advantages and disadvantages. Retrofitting method used now-a-days for URM buildings include surface coating, reinforced shotcrete overlays, application of FRPs, grout and epoxy injection, external reinforcement, post tensioning, central core technique, etc. Recently Japanese [MNM-06] proposed a new material, poly propylene band (called PP-band) for retrofitting of masonry building. The above-mentioned techniques are directly applicable on the surface of masonry walls. Indirect methods include the elongation of the building natural period using friction and fluid viscous dampers and thus reducing the seismic demand.

2.8.1 Retrofitting with Fibre Reinforced Polymers:

The disadvantages of conventional retrofitting techniques include: more intervention, occupancy problem, more time consuming, stiffness and mass increase and thus attracting more forces, requirement of foundation strengthening, etc. Many researchers have tried to overcome these problems by using Fiber Reinforced Polymers (FRP) and they were successful with regard to enhancing the strength capacity of URM buildings.

FRP composite is a high strength, corrosion-resistant and brittle material, formed by embedding high strength fibers (Glass, Carbon or Aramid) in a resin matrix (Epoxy, Polyester, and Vinyl ester resins) which binds the fiber together. Depending upon the type of fibers FRP may thus be classified as:

- Glass Fiber Reinforced Polymer (GFRP)
- Carbon Fiber Reinforced Polymer (CFRP)
- Aramid Fiber Reinforced Polymer (AFRP)

Typical mechanical properties of unidirectional FRP laminates are given in Table 2-2. [TCS-02]

**Table 2-2: Typical Mechanical Properties of FRP Laminates**

<table>
<thead>
<tr>
<th>Composite Type</th>
<th>Fiber Content (%by Wt)</th>
<th>Density (kg/m³)</th>
<th>Long. Tensile Modulus (GPa)</th>
<th>Tensile Strength (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>GFRP in Polyester</td>
<td>50-80</td>
<td>1600-2000</td>
<td>20-55</td>
<td>400-1800</td>
</tr>
<tr>
<td>CFRP in epoxy</td>
<td>65-75</td>
<td>1600-1900</td>
<td>120-250</td>
<td>1200-2250</td>
</tr>
<tr>
<td>AFRP in epoxy</td>
<td>60-70</td>
<td>1050-1250</td>
<td>40-125</td>
<td>1000-1800</td>
</tr>
</tbody>
</table>

FRP are available in the form of FRP fabric, FRP laminates and FRP rods. The fibers in FRP are arranged as unidirectional or multi-directional layout. Usually FRP fabric is applied on the whole surface of walls with high strength epoxy. FRP strips/plates are attached to the wall in horizontal, vertical and/or inclined direction depending upon the purpose for which it is being used. FRP rods are reinforcing bars used in center core technique and structural repointing for strengthening of masonry walls.

A review of modern retrofitting techniques using FRP is presented by ElGawady et al. [MEL-04] and [EHL-04]. According to his survey, strengthening/retrofitting of URM walls using FRP increase the lateral resistance by a factor ranging from 1.1 to 3. ElGawady et al. himself worked on the in-plane behaviour of URM walls upgraded with FRPs under static cyclic and dynamic loadings. He concluded that the application of FRP on masonry wall increases the strength by 1.3 to 2.9. However the increase in drift ratios was less than 1.2 times that of the reference wall. He also found out that FRP strips, use in diagonal pattern, are less effective than the bi-directional surface type materials (fabric).

![Figure 2-5: FRP Placement and Force-deformation Hysteresis Curves for FRP rehabilitated wall [ASL-07]](image-url)
D.P. Abrams et al. [ASL-07] tested cantilever rocking-critical pier under cyclic loading retrofitted with GFRP laminates (7.5 N, 27 oz. Tyfo unidirectional) applied to both faces of masonry with epoxy, Figure 2-5. A substantial increase (2.4 times) in the lateral strength of the rehabilitated pier over the non-rehabilitated was observed. Only 80% of the initial stiffness was restored. The deformation capacity was, however, substantially reduced.

Tim Stratford et al. [SPM-04] performed monotonic test to investigate the behaviour of URM walls reinforced with GFRP sheets, fixed with epoxy adhesive over the whole surface of walls. The technique increased the strength capacity by about 65%. However, the stiffness and deformation capacity was not changed by the GFRP. One of the important outcome of his experiments was that the adhesive used to apply the FRP to the masonry need not be strong.

Hernan Santa Maria et al. [MAL-06] tested masonry walls externally reinforced with CFRP. Two configurations were investigated (Figure 2-6): one with horizontal fabrics and other with diagonal fabrics. The authors concluded that externally bonded CFRP sharply increased the strength (60% to 80%) of URM walls and deformation capacity, spreading the cracks into several thinner cracks. They also concluded that the horizontal arrangement was more effective in spreading the cracks.

J. Gustavo Tumialan et al. [TGN-02] worked on strengthening of URM walls with near surface mounted (NSM) GFRP bars (Figure 2-7). They investigated both in-plane and out-of-plane behaviour of URM strengthened with NSM FRP bars. Vertical FRP bars were inserted in grooves formed in walls and subsequently filled with epoxy, to improve the out-of-plane behaviour of walls. The specimens were tested in simply supported conditions. Remarkable increase in out-of-plane bending resistance was noted (up to 14 times). For shear strengthening horizontal and/or vertical FRP bars were used and tested under diagonal compression. About 30% to 80% increase in strength was noted. A third type of strengthening technique was also investigated to improve the anchorage zone through vertical FRP bars.
A different study on the application of textile of carbon fabrics embedded in a cement based matrix (carbon fiber cement matrix, CFCM) for out-of-plane strengthening of masonry walls, was made by Holger Klosch (HK-98). The proposed system was found effective in increasing the out-of-plane strength equivalent to ground acceleration of 3g.

Similar study to the one discussed above was undertaken by Catherine G. Papanicolaou et al. [PTP-08] on the cyclic out-of-plane loading of masonry strengthened with textile reinforced mortar (TRM) versus fiber reinforced polymer (FRP). Two series of specimens A and B identified in Figure 2-8 (i) were constructed in cement-lime-sand mortar. In series A the plane of failure was parallel to the bed joints while in series B it was perpendicular to the bed joints.

Polymer-modified cement mortar and epoxy adhesive were used to fix textile, Figure 2-8 (ii) with TRM strengthened specimens and FRP strengthened specimens respectively. Substantial gain in strength and deformability was noted in the specimen reinforced with TRM. The strength of specimens reinforced with TRM was comparable with that of specimens reinforced with FRP. Higher deformations and energy dissipation capacity were, on the other hand, recorded in TRM reinforced specimens when compared with FRP reinforced specimens.

From the above discussion it may be summarized that FRP being a high strength brittle material enhances the lateral strength by several fold but
limits the inelastic deformation capacity producing a sudden failure due to de-bonding of the sheets. Thus FRP should be placed strategically on those piers that are likely to attract damage first (high demand to capacity ratio) so that alternate, and more ductile mechanism can occur [DPA-00]. The disadvantages of FRP includes, comparatively its high cost, low availability, requirement of skilled workmanship, common people acceptability, incompatibility with substrate material, etc.

2.8.2 Ferro-Cement Overlays:
Ferrocement overlay (Figure 2-9) also called reinforced-cement coating/jacketing, is the application of a thin layer of reinforced plaster on one or both faces of an unreinforced masonry wall to enhance its seismic performance by improving the lateral resistance and energy dissipating capacity. The method is cost-effective and efficient and is, therefore, widely used for strengthening of masonry buildings all over the world. The reinforcement is usually a welded wire steel mesh attached to the wall through screws or bolts.

The shear between masonry wall and the reinforced plaster is transferred through screws/bolts and bond between plaster and wall. A rich mix of cement-sand mortar is used as plaster. First the wire mesh is fixed with the walls with steel bolts/screws and steel washer as shown in Figure 2-9. The coating material is then applied on the surface passing through the mesh.

Ferrocement is old technique in terms of its application but relatively young in terms of the year devoted to its research for unreinforced masonry buildings. The first systematic work on retrofitting of URM buildings with ferrocement overlay was conducted by Prawel and Reinhorn [PR-85, RP-85]. They tested a series of masonry walls strengthened with a number of ferrocement layers using different mesh arrangement. The overall seismic performance of retrofitted walls was nearly double than those of non-retrofitted wall.

The minimum guideline set forth in [LHL-97] for ferrocement overlays are:

- Uni-axial strength of coating material should exceed a minimum of 1000 psi.
• Steel hardware cloth with a minimum diameter of 19g and maximum grid of 1/2" (12.7 mm) shall be placed at the centre of coating. The coating material should be passed through the cloth mesh rather than placing the mesh in the wet coating material.

• Minimum diameter of steel tie-down bolts perpendicular to the wall face shall be 1/4" (6.4 mm) placed 16" (400 mm) on centre.

• The thickness of plaster coating should exceed a minimum of 1/2" (12.7 mm), the wall height divided by 200 and the spacing between tie-down bolts divided 32.

According to Bret Lizudia et al. [LHL-97], the ferrocement overlay is an effective mean in enhancing both in-plane and out-of-plane performance (both in terms of strength and deformation capacity) of unreinforced masonry. Coating a single side of wall can provide the necessary enhancement in out-of-plane and in-plane strength, however, inelastic deformation capacity of walls coated on both sides will be greater because of the confinement effect. One more advantage of the ferrocement overlays is the decrease in the height-to-depth ratio which will improve dynamic stability and/or arching action if the boundaries of the wall panel are restrained [DPA-00]. However, since the reinforcement ratio is rather light, the added lateral strength of a wall is small.

![Figure 2-10: Specimen loading and dimensions [ASL-07] and Hysteretic Curves](image)

D.P. Abrams et al. [ASL-07] tested rocking-critical pier before and after retrofitting with ferrocement overlays. Tested specimen represented the lower half of a pier between two openings and the bottom foundation beam (Figure 2-10). Aspect ratio (length/height) of the pier was 0.56. The wire mesh was made of 19 gauge wire with 1/2" (12.7 mm) grid spacing. The compressive strength of plaster mortar was 1,000 psi (6.9 MPa). Initial cracking was heard at 0.05% drift. Visible flexure cracks appeared at 0.1% drift. The crack covered the whole length of pier at 0.2% drift and continued to rock up to 2.5% drift without any sign of additional damage. Slight increase in strength (Figure 2-10) was observed in comparison with non-rehabilitated specimen in the initial elastic portion. Once the mesh got fractured the strength reduced and the pier continued to rock like a non-rehabilitated specimen.
Alcocer et al. [ARP-96] investigated the technical feasibility of confined masonry retrofitted with steel welded wire mesh (SWWM). Four shear-critical full-scale damage specimens (one two-storey 3D building and three isolated walls) were rehabilitated and tested under alternate cyclic lateral load. Variables were the level of damage, type and size of specimens (two-storey 3D and one-storey 2D), mesh size and the type of anchors. Only in-plane walls of the ground storey of two-storey 3D specimen were retrofitted. All specimens were covered with 25 mm thick cement-sand plaster in volume ratio of 1:4. Before application of plaster the crush concrete in tie columns was replaced with new concrete and the diagonal cracks was repaired with cement-sand mortar. Specimen’s description and material properties used are summarized in Table 2-3.

Table 2-3: Specimen description and material properties

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Mortar Cube Strength, MPa</th>
<th>Masonry Com. Strength, MPa</th>
<th>Masonry Diagonal Strength, MPa</th>
<th>Steel Welded Wire Mesh</th>
<th>Both or Single Face</th>
<th>Anchors</th>
<th>Vertical Stress, MPa</th>
</tr>
</thead>
<tbody>
<tr>
<td>3DR (double storey) (Pre-cracked)</td>
<td>10.0</td>
<td>5.3</td>
<td>0.59</td>
<td>150 x 150, Φ 3.43</td>
<td>Single</td>
<td>50 mm nails North wall: 9 nails/m² South Wall: 6 nails/m²</td>
<td>0.49 MPa at Ground Level</td>
</tr>
<tr>
<td>M1 (2500 x 2500) (No damage)</td>
<td>12.2</td>
<td>5.2</td>
<td>0.69</td>
<td>150 x 150, Φ 4.88</td>
<td>Both</td>
<td>64 mm nails at 300 on one face and 450 on other</td>
<td>0.49 MPa</td>
</tr>
<tr>
<td>M2 (2500 x 2500) (No damage)</td>
<td>7.7</td>
<td>5.2</td>
<td>0.69</td>
<td>150 x 150, Φ 3.43</td>
<td>Both</td>
<td>64 mm nails at 300 on one face and 450 on other</td>
<td>0.49 MPa</td>
</tr>
<tr>
<td>M3 (2500 x 2500) (No damage)</td>
<td>14.1</td>
<td>5.2</td>
<td>0.69</td>
<td>150 x 150, Φ 6.35</td>
<td>Both</td>
<td>51 mm Hilti anchor at 450 both faces</td>
<td>0.49 MPa</td>
</tr>
</tbody>
</table>

Figure 2-11: Envelope curves for 3D model (left) and isolated 2D walls (right)

The technique not only improved the lateral strength (1.3-2.6 times the control specimen) but also the deformation capacity (1.2-1.7 times) of confined masonry walls (Figure 2-11). It was observed that the nail’s type and spacing plays an important role in the crack distribution; more uniform crack distribution was observed on face with higher density of nails. Initial
stiffness of the rehabilitated damaged specimens was found to be 2/3 that of the original specimen.

According to Miha Tomazevic [MT-99] the lateral resistance of wall, strengthened with thin ferrocement coating (less than 2.0", 50 mm), may be estimated by simply multiplying the shear resistance of the original wall by an experimentally obtained multiplier, i.e. the ratio between the lateral resistance of ferrocement coated and original wall.

From the above information it may be concluded that very little experimental work has been reported so far on the performance of unreinforced masonry walls retrofitted with ferrocement overlay. The reported work is either based on rocking-critical behaviour where the significance of ferrocement overlays is minimal or shear-critical behaviour but in confined masonry. One of the objectives of this research is to study the behaviour of shear-critical unreinforced brick masonry piers retrofitted with ferrocement under axial compression, diagonal compression and quasi-static loadings.

2.8.3 Shotcrete Overlays:

Shotcrete overlay and concrete jacketing are the application of reinforced concrete on one or both sides of a masonry wall. In shotcrete overlay concrete is sprayed on to the surface of wall, Figure 2-12 while in jacketing concrete is poured through a form work along the wall surface. To develop a composite action and to transfer shear between shotcrete and masonry, anchors are embedded into drilled holes with epoxy or cementicious grout [MT-99]. The bond between shotcrete and masonry surface may be developed by applying a strong bonding agent like epoxy on wall surface [KL-84]. The thickness of the shotcrete overlay depends upon the seismic demand. Generally the thickness varies from 3” to 6”. Reinforcing mesh generally consists of a steel mesh of about 1/4” to 3/8” diameter bars spaced at 10” on centre. 1/4” diameter anchor bolts spaced at about 9” are used to connect the steel mesh with the wall. In case of jacket on both sides of wall, it is preferable to connect both the jackets with a cross ties passing through the wall thickness.

![Figure 2-12: Application of Shotcrete on wall surface](image)

The capacity of wall retrofitted with shotcrete overlay is evaluated using equations developed for the analysis and design of reinforced concrete.
shear walls [MT-99]. The effect of original masonry being strengthened is generally ignored and the shotcrete overlay is assumed to resist all the seismic demand. According to D. P. Abrams [DPA-00] the assumption is reasonable for strength criteria as the strength of shotcrete can be many more times than that of masonry. However the assumption may lead to some cracking in the masonry violating performance objective for immediate occupancy or continued operation.

D. P. Abrams et al. [ASL-07] tested rocking-critical pier after retrofitting with 4" thick shotcrete overlay on one face. The reinforcing and anchoring detail and the force-deformation loops are shown in Figure 2-13. The thickness of the overlay was sufficient to provide a typical minimum concrete cover to the reinforcement.

The force-deformation curve is similar to that of a reinforced concrete shear wall behaving in flexure with an initial elastic range followed by yielding and high energy dissipation. The strength was found to be 3 times that of unreinforced wall, Figure 2-13.

Other researchers [MT-99], [KL-84], etc. also noted a high increase in both lateral resistance and energy dissipation capacity of unreinforced walls strengthened with shotcrete overlays. The increase in strength is inversely proportional to quality of original wall.

Shotcrete overlay, though, very much effective in increasing the strength and energy dissipation capacity of unreinforced wall, yet it has some disadvantages. The method is very expensive, causes disturbance to the occupants of the building, decreases the size of room, increases the stiffness of the wall and thus changes the overall performance of the building and last but not the least, changes the aesthetic of the building.

2.8.4 Centre Core Technique:
In centre core technique a three inch diameter or larger vertical cores are drilled through the entire height of an unreinforced masonry wall. Vertical reinforcements (deformed steel bars or FRP rods) anchored to the base of wall (or pre-stressing steel) are provided in the core subsequently filled with grout. Oil-well drilling technique is used for making core in the wall. The spacing between the successive cores along the length of wall depends upon the seismic demand and the existing capacity of the masonry wall. The technique is very effective in increasing in-plane and out-of-plane
flexural strength (if reinforcement is properly anchored with base), and in-plane shear strength of masonry wall.

Plecnik et al. [PCO-86] was probably the first one who proposed this technique for masonry wall strengthening. He performed shear test on walls strengthened with centre core technique. The cores were filled with different grout material, e.g. cement based grout (cement-lime-sand), epoxy-sand grout and polyester-sand grout. Test results showed that walls with cores filled with cement based grout were about 30% weaker than those with epoxy-sand or polyester-sand grout. Due to the high price of epoxy and polyester and satisfactory response of cement based grout, the cement based grout (8 parts of cement, one part of lime and 8 parts of sand) is recommended. The grout and reinforcement form a homogenous structural element much larger than the core itself [PCO-86] which increases both the in-plane and out-of-plane strength of wall.

O. O. Erbay and D. P. Abrams [EA-02] tested a wall (3648 x 1753) rehabilitated with centre core technique to investigate the role of dowel action of embedded vertical reinforcement on shear sliding behaviour. Four 75 mm diameter cores spaced at 1120 mm were drilled in the wall from top to bottom and extended 260 mm in the concrete pad at the base of wall. The wall was reinforced with four 16 mm conventional reinforcing bars (\(\rho = 0.01\%\)). Polyester-sand grout in volumetric ratio of 1.0:1.5 was used as filler material. The compressive strength of grout was 85.6 MPa. Test results revealed that the centre core technique enhanced the sliding shear capacity through dowel action by 20%. The behaviour of masonry rehabilitated with this technique was believed to be same as that of reinforced masonry.

![Figure 2-14: Specimen rehabilitated with centre core technique [ASL-07]](image)

Daniel Abrams et al. [ASL-07] investigated the effectiveness of centre core technique applied to two rocking-critical specimens (Figure 2-14) reinforced with different steel ratios. The specimens (842 x 1248) with bottom masonry base were rehabilitated with two reinforced cores (76 mm diameter) as shown in the Figure 2-14. One of the specimens was reinforced with 10 mm diameter bar of grade-60 steel in each core and the other with 16 mm bar one in each core. Epoxy-sand grout was used as a filler material. The two specimens were then subjected to a sequence of drifts of progressively increasing amplitude under a constant vertical stress.
of 0.29 MPa. The strength of the rehabilitated piers was found to be almost doubled and 2.4 times for specimens reinforced with 10 mm and 16 mm bars respectively. However the deformation capacity was limited in both the specimens.

The centre core technique is thus effective in enhancing the strength capacity of URM walls but at the same time limits the deformation capacity. The technique is very expensive and required special equipments for core drilling and grouting. The advantage of the technique is that it not only preserve the external appearance of the wall but also, do not disturb the functionality of the building since the whole operation can be done from the roof top.

2.8.5 **Grout and Epoxy Injection:**

Grout injection, for years, has been regarded as a suitable technique to restore the homogeneity, uniformity of strength and continuity of masonry walls [BST-06] eliminating the need for removing and rebuilding the damaged wall. The cause of cracks, however, should be carefully evaluated and addressed. Grout is injected in masonry cracks of width less than 3/8″ (10 mm) through pressure. In the case of cracks with width larger than 3/8″ (10 mm), the damage area should be reconstructed [MT-99]. Grout may be cement-based or epoxy based. In the case of fine cracks (less than 2 mm wide), the use of epoxy grout is recommended. Grout is also used to fill the internal voids in low quality masonry, empty collar joints in multi-wythe masonry and old deteriorated masonry in order to restore their original integrity. In the case of cement-based grout the injection pressure ranges from 15 to 60 psi (100 to 400 KPa) depending upon the width of cracks and/or voids. High pressure may however be required in cased epoxy-based grout because the cracks are very thin.

![Injection Machine and Injection Nozzles and Stoppers](image)

Cement-based grouts are usually recommended for masonry structures because of their porous nature and low cost. Epoxy may also be used, but it is very sensitive to moisture and it is very difficult to completely dry the masonry before injection. Cement-based grouts mainly consist of Portland cement, expansive agents and water with water-cement ratio ranges from 0.75 to 1.0. Slacked lime and Pozzolana like fly ash may be added to the mix. Miha Tomazevic [MT-99] used a mix of 90% of Portland cement and 10% of Pozzolana. In the case of relatively large cracks ranging from 3/16″ to 3/8″ (5.0 to 10 mm) fine sand may be added to the mix. The grout
recommended by the Department of Building and Safety, Los Angeles [DBS-08] for masonry building consisted of 3 parts of #60 silica sand, 1 part of #90 silica sand, 1 part of plastic Portland cement, 0.5 part of type-S lime and 0.5 part of type-F fly ash mixed with approximately 2.5 parts of water.

Cement-based grout mix is designed based on its penetrability, stability and effectiveness. A grout mix shall be fluid enough to be easily penetrated in to tiny cracks and voids but at the same time the grout must be stable enough to resist segregation and shrinkage. With addition of more water penetrability is increased but the stability is decreased. Addition of plasticizers will definitely increase the fluidity without degrading the stability of mix. Stability of grout mix can be improved with addition of fine aggregates, water retaining agents like fly ash, lime, etc.

Different steps involved in the cement-grout injection in unreinforced masonry may be summarized as follows:

- As a first step the plaster from the wall surface is removed from the cracked region.
- Holes, with 1/4” to 1/2” (6 to 12 mm) diameter and depth equal to more than half of the wall thickness, are drilled along the cracks at interval of about 1.0 to 2.0 ft (300 to 600 mm). In the case when the cracks appear on both side of wall, holes shall be drilled on both sides of the wall. The diameter of hole shall be as small as possible and shall be drilled into mortar joint. In case of fine cracks holes shall be placed as close as 4” (100 mm) on centre.
- Injection nozzles (Figure 2-15) about 4.0” (100 mm) long and 1/4” to 1/2” (6 to 12 mm) diameter are fixed in drilled holes using epoxy or fast binding mortar.
- The wall surface is cleaned and the cracks are sealed using mortar, epoxy or any other material capable of resisting injection pressure. During the injection the grout may force out through weak mortar joints in surrounding masonry. It is, therefore, recommended to seal the surrounding area also.
- To check whether the injection nozzles are active or not and also to remove dust and loose material from inside the cracks, water is first flushed through the nozzles at tap pressure. Starting from the top nozzles and coming down, continue flushing until the water, flowing out of the bottom hole, is clear. In the case of brick masonry, soaking of the bricks before grout injection shall be insured.
- Grout prepared in the required proportion shall then be injected through the nozzles. The injection pressure varies from 15 to 60 psi (100 to 400 KPa) depending upon the specific cases. In the case of collar joint injection, smaller pressure is recommended to prevent wall blowouts. In case of cracks in masonry walls Miha Tomazevic [MT-99] recommends to increase the pressure up to 45 psi (300 KPa) until the masonry absorbs the grout. After that the pressure is
increased to 60 psi (400 KPa) and kept constant for 5 to 10 minutes in order to density the mix and to drain excess water.

- After injection 2″ to 2 ½″ (50 to 65 mm) diameter inspection holes shall be drilled at appropriate location along the crack, when required, to confirm that the crack is fully grouted. The depth of inspection holes shall be equal to the depth of injection holes. The inspection holes shall then be packed with rich mortar afterward.

Teymour Manzouri et al. [MSS-96] investigated the effect of grout injection on the compressive strength, shear strength of masonry and lateral resistance of masonry walls. Two categories of 36 mixes were studied; coarse (sanded) and fine (with no aggregates) grouts were considered. Fine mix, consisted of 100 parts of cement 2.0 % of super plasticizers and 0.5 % of Ultra fines prepared with a water-cement ratio of 0.5 was finalized. While coarse mix, comprised 32.1 % of cement 4.8 % of lime, 7.9 % of fly ash, 55.2 % of #70 sand, super plasticize 2.0% (of cement) and Ultra fines 0.5% (of cement) mixed with water to get a water-cement ratio of 1.0 was finalized. The injection procedure was based on Los Angeles Rules of General Applications, 1991 [LAR-91]. It was found out that the grout mix can restore the initial compressive stiffness but may not fully restore its compressive strength. No increase in the shear strength was observed.

Experimental study on masonry walls [MT-99] repaired with cement or epoxy grout have shown that the original tensile strength of masonry is recovered or even improved but the rigidity in most cases is not. Grout injection is therefore the most desirable technique to restore the original strength of masonry.

An important aspect of the epoxy or grout injection is the compatibility of the injection material with that of wall material. The effectiveness of grout injection as outlined in [BST-06] depends on:

- The characteristic of the mix used,
- Mechanical properties of the mix
- The injection technique adopted
- Knowledge of the masonry type, i.e. size and distribution of voids in the wall.

Binda et al. [BMB-93] proposed a laboratory test to check the injectability of the grout material on the material sampled from the internal parts of the wall. The material is inserted in a transparent cylinder and then the grout is injected through the material from bottom (Figure 2-16). The injected cylinder, after curing can, then, be tested under direct compression test and split cylinder test. Another test proposed as part of this study is to test masonry specimens made from the masonry units and grout, under diagonal compression test (Figure 2-17) to evaluate the tensile strength of masonry injected with grout.

A flow test as proposed in [DBS-08] shall also be performed for the grout mix before injection. Grout mix is poured from a cylinder 2″Φ x 4″ (50 x 100 mm) high from 12″ (300 mm) height onto an impervious smooth level
surface. A 6” to 8” (150 to 200 mm) diameter puddle indicates a mix of proper consistency.

Figure 2-16: Injectability Test Proposed by Binda et al [BMB-93]

Figure 2-17: Pouring of grout material to cast specimen for diagonal compression test

2.8.6 Retrofit using Steel Elements:
The system of retrofit using steel elements is effectively used for strengthening of reinforced concrete frame structures. Very little work has reported so far on the retrofitting of masonry using steel elements. Researchers around the world have also tried to use this system for strengthening of masonry buildings. The steel elements are either attached to the wall with through thickness bolts [TBS-98] or applied externally to the foundation and top beams [RG-96]. In the case when steel strips are attached to the wall, anchor bolts play an important role in transferring shear between wall and strips. Inadequate connection may cause a premature failure of the wall.

Durgesh C. Rai and Subhash C. Goel [RG-96] investigated the behavior of perforated wall consisting of rocking critical piers and spandrels, reinforced with steel framing system consisting of vertical and horizontal elements around the wall, without any braces (Figure 2-18). Overall hysteretic
behavior (strength, stiffness and ductility) of the wall was found to be significantly improved. The strengthening system also controlled damage to the brittle wall piers and thus provided safety against sudden failure.

![Figure 2-18: Schematic diagram of masonry perforated wall strengthened with steel elements](image)

Figure 2-18: Schematic diagram of masonry perforated wall strengthened with steel elements

![Figure 2-19: Strengthening using steel strips and hysteretic response of unreinforced and reinforced walls](image)

Figure 2-19: Strengthening using steel strips and hysteretic response of unreinforced and reinforced walls

Taghdi et al. [TBS-98] tested concrete block masonry walls reinforced with steel trip system subjected to constant vertical stress and incrementally increasing lateral deformation reversals. The steel strip system consisted of diagonal and vertical strips attached to both faces of the wall using through-thickness bolts (Figure 2-19). The proposed system was found to be effective in significantly increasing both in-plane strength and ductility of low-rise unreinforced masonry building (Figure 2-19). A 450% increase in strength of strengthened wall was observed as compared to unreinforced
block masonry wall. The energy dissipating capacity was also enhanced with the proposed system.

Farooq et al. [FIG-06] investigated the strength behavior of masonry wall panels (4’x4’, 1220 x 1220 mm) strengthened with locally available vertical and horizontal steel strips (1.75” x 0.05”, 45 x 1.3 mm) under compression and lateral loading applied in small increments. 1/4” (6 mm) diameter and 1.75” (45 mm) long bolts screwed in plastic rivets were used to fix the strip on wall panel applied at the intersection of horizontal and vertical strip (Figure 2-20). The monotonic lateral load test was performed at a pre-compression stress of 184 psi (1.27 MPa). The tests were performed in a force control environment.

Figure 2-20: Wall panel Strengthened with Vertical and Horizontal Steel Strips and Shear Failure Pattern [FIG-06]

Table 2-4: Specimens Details: Retrofitting with Steel Elements

<table>
<thead>
<tr>
<th>Specimen ID</th>
<th>Spacing (H x V)</th>
<th>Single/double side</th>
<th>Compressive Strength, kips (% increase)</th>
<th>Lateral Strength, kips (% increase)</th>
</tr>
</thead>
<tbody>
<tr>
<td>US</td>
<td>N/A</td>
<td>N/A</td>
<td>141.0 (- )</td>
<td>20.4 (- )</td>
</tr>
<tr>
<td>FSM</td>
<td>9” x 6”</td>
<td>Single</td>
<td>162.1 (15%)</td>
<td>28.7 (40%)</td>
</tr>
<tr>
<td>SCM</td>
<td>9” x 9”</td>
<td>Single</td>
<td>160.4 (14%)</td>
<td>26.4 (30%)</td>
</tr>
<tr>
<td>DCM</td>
<td>9” x 9”</td>
<td>Double</td>
<td>180.9 (28%)</td>
<td>38.1 (87%)</td>
</tr>
</tbody>
</table>

The detail of specimens is given in Table 2-4. Appreciable increase in both compressive strength and lateral strength was recorded in the specimens strengthened with steel strips. The increase in strength for FSM and SCM was found to almost same but larger initial cracking load was observed in FSM. Specimen reinforced both sides showed a very high increase in both vertical and lateral strength due to the confinement effect. An appreciable increase in the ductility of strengthened was also observed.

From the above discussion it may be concluded that the system is effective in enhancing the performance of masonry building but at the same time the system may be uneconomical. There may be steel corrosion problem. The intervention changes the appearance of the wall. Cement-sand plaster, if applied on the wall surface may not adhere to the steel strip.
2.8.7 Structural Repointing:
Replacing a part of existing loose or weak mortar with a comparatively rich mortar is sometimes effective in enhancing the vertical and lateral load performance of a masonry wall. The technique is especially very effective in case of old masonry buildings subjected to overloading and/or severe weather conditions which result in a deterioration of old mortar. In case of brick/block masonry in which bed joints are leveled, reinforcing bars (1/4″-3/8″, 6-10 mm diameter) may be embedded in the bed joints to improve strength and ductility of the masonry (Figure 2-21). The reinforcing bars shall adequately be anchored to the wall through ties. In case of reinforcement provided on both faces of walls, cross ties shall be used. The numbers, spacing and size of reinforcing bars depend upon the thickness of mortar joints, depth of groove and the additional strength required. The new mortar is either based on cementitious material or polymeric materials.

High pressure water jet or a drill machine may be used to remove the existing mortar from bed joints. The depth of groove shall be less than 1/3 of the wall thickness and only one face of the wall shall be grooved at a time to avoid any accidental collapse of the wall during to intervention. Usually the depth of groove ranges from 1.5" to 3.0". The groove shall be cleaned with water and well moistened (in case of cement based mortar) before application of new mortar.
M. R. Valluzzi et al. [VBM-05] investigated experimentally and numerically the effect of bed joint reinforcement on the behaviour of double-brick masonry panels under monotonic compressive loads. The primary objective of the investigation was to verify the capability of the structural repointing to control dilatancy phenomena under vertical compressive loads. Two 1/4″ (6 mm) diameter bars were inserted in every third bed joint groove, 0.4″-0.6″ (10-15 mm) thick and 2.5″ (60 mm) deep. The intervention was performed only at one side of the wall panel. The bars were folded and fix by polymeric mortar at both the ends (Figure 2-22). Hydraulic lime mortar with expensive agent and polymeric mortar were used as repointing materials.

The intervention was found to be effective in controlling of vertical cracks due to overloading phenomenon (Figure 2-22) which was the primary objective of the research. No increase has been detected in strength. Polymeric mortar was not compatible with the masonry and therefore showed a reduced level of strength.

M. Corradi et al. [CTB-08] proposed a repair and preventive technique called deep repointing for double leaf stone masonry walls with appropriate mortar to bond the stones of the external leaves and thus increase the shear strength of masonry through confinement by the external leaves, Figure 2-23. Deep repointing in combination with grout injection was also studied. The repointing material was a lime-cement mortar with a compressive strength of 1560 psi (10.75 MPa) and the grout was a ready mix hydraulic lime with a compressive strength of 1015 psi (7.0 MPa). The depth of repointing was about 2.75″ to 3.15″ (70 to 80 mm). The technique was applied to three buildings belonging to the historic centres of Italy. Compression test, shear compression test and diagonal compression tests were performed before and after strengthening, on samples extracted from these buildings. Appreciable increase in strength has been noticed during testing in sample reinforced with deep repointing.

As stated under section 2.8.1 J. Gustavo Tumialan et al. [TGN-02] used FRP bars as vertical and/or horizontal reinforcement to enhance the out-of-plane and in-plane response of URM walls. The technique was found to be effective in increasing the diagonal shear strength of masonry by 30% to 80%.

Structural repointing is an easy technique to apply and does not change the appearance of the wall. So far little work has been reported on the lateral load behaviour of masonry reinforced with structural repointing. The selection of repointing mortar plays an important role in enhancing the
performance of masonry. The mortar must be compatible with the existing masonry materials and strong enough but not too stiff and have a good bond with the existing materials.

2.8.8 Confining URM Walls with Tie Columns:
The performance of confined masonry widely used in Europe, Asia and Latin America was found to be very promising during past earthquakes. An unreinforced masonry building may be confined by providing reinforced concrete tie columns at wall junctions and large openings. In order to be effective all tie columns shall be connected to each other at the lintel level with reinforced concrete tie beams. The intervention will not only improve the strength of wall but also the ductility and energy dissipation capacity. The confining elements are also effective in preventing the out-of-plane failure of masonry walls. Eurocode 6 and 8 provides guidelines for construction and design of new confined masonry buildings. Limited work has been done so far on the retrofitting of existing walls with confined elements.

Figure 2-24: Confinement of Unreinforced Masonry Wall

The procedure is to first provide temporary supports to the roof and then to remove the masonry from the wall junctions leaving tooting in the masonry (Figure 2-24). Four 1/2”Φ longitudinal bars properly anchored in the foundation are erected at the junctions with 3/8”Φ ties provided at 6” on centre. Concrete is then poured in the tie columns against the form work.

2.8.9 Post-tensioning:
The tensile stresses resulting from the lateral loads, which are the primary cause of cracking in masonry, may be reduced by inducing pre-compressive stresses in the masonry through post-tensioning. Post-tensioning being a new technique is comparatively expensive and is generally used for structures of historic importance.

2.9 Experimental Testing on Masonry:
The main objective of experimental tests in earthquake engineering is to study and understand the behavior of a structure or its component in a controlled environment of the laboratory. The experimental test setup shall be such that to reproduce the actual field conditions (loadings, boundary conditions, etc) in
laboratory as closely as possible. In the case of masonry buildings the experimental tests is performed on a complete full-size or scaled building, on a wall consisting of piers and spandrels or on a single pier. As the in-plane behavior of a masonry building is mostly controlled by its piers, tests on individual piers are preferred because of ease and economy. Different types of tests performed on masonry may be categorized as:

1. Monotonic static test
2. Quasi-static test
3. Shake table test
4. Explosion Test
5. Pseudo-dynamic test
6. Real-time hybrid test

2.9.1 Static Test

The first two are static tests which may be performed manually using hydraulic jacks or through control system using actuators. The static test is performed at a very slow rate (2.0 mm/sec). The specimen is usually tested until collapse and thus cover full spectrum of the load-deformation characteristic.

In the case of monotonic test load/displacement are applied only in one direction at very slow rate (Figure 2-25). The aim of test is to get the force deformation characteristics of specimen from which stiffness, strength and displacement ductility is determined.

In quasi-static test which is very common and also economical, increasing displacements are applied in reverse cyclic form. Each displacement cycle is repeated two are three times (Figure 2-26). The main objective of the test is to determine energy dissipating capacity, ductility and to study the crack propagation.

![Typical Displacement Pattern used in Static Test](MT-96)

Static test may be performed in force-controlled or displacement-controlled environments. The displacement-controlled static tests are more desirable because of safety reasons and the possibility to study the post-peak behaviour. Comparison of the force-deformation characteristics [MT-96] shows that the peak load and the ultimate displacement in case of monotonic load is on higher side than that of quasi-static load.
2.9.2 Shake Table Test

Shake table test is a dynamic test in which a reduced-scale or full-size structure or its component is mounted on a shake table and subjected to seismic base motion in the form of earthquake accelerogram (artificial or natural). The base motion may be in one direction (SDOF) or in multi-direction (MDOF). It is very difficult to control the motion to the desired level. The shake table motion is usually different from the input accelerogram. Selection of accelerogram is vital because the structural performance is different for different accelerogram due to the difference in the frequency contents and shaking time. Usually an accelerogram representative of the area and with frequency close to the natural frequency of the structure is selected.

Earthquake record (Figure 2-27) is applied to the structure in increasing intensities of amplitude. Response of the structure is recorded through accelerometer and displacement transducers. The recorded data then may be analysed to determine damping ratios and natural periods of the structure, and to develop the force deformation curve which may be further analysed for ductility ratio and response modification factor.

Shake table test is considered to be the best method to study the response of a structure to the earthquake ground motion. But at the same time this method is very expensive and time consuming, as a full structure (scaled or full-size) is to be constructed. It is also very difficult to predict the behaviour of prototype structure from the behaviour of scaled model tested on shake table. The model is rigidly connected to the shake table which again produced error in the results because the actual structure is constructed on flexible soil.

2.9.3 Explosion Test

In this method of testing a full-size structure constructed in the field is subjected to arrays of explosion to simulate earthquake ground motion [AN-07]. Weight of the charge, depth of explosion, distance of explosion from the structure, the number of explosion and the time delay between
successive explosions are the parameters which control the ground motion and shall be carefully designed.

2.9.4 Pseudo-Dynamic Test

Pseudo-dynamic test is basically a static type test in which the structure is loaded over an expanded time scale with the dynamic effects accounted for computationally. This test is considered to be a best alternative to the expensive shake table test in which displacements due to earthquake loads are calculated computationally using stepwise integration of dynamic equation of motion. The calculated displacements are applied statically to the specimens and the restoring forces are measured using load cells. These measured forces are fed back to the numerical model as input for the next calculation step.

The advantage of pseudo-dynamic test run on an expanded time scale is to allow the inspection of the specimen during test. Also as the test runs statically, it simplifies the equipment needed. Sufficiently large strong floor and strong wall is required for the pseudo-dynamic test of full-scale specimens. The drawback of pseudo-dynamic test is that the time-dependent behaviour is not included because the test is run at a very slow rate.

Sub-structuring technique may be incorporated in pseudo-dynamic testing in which some parts of a structure are simulated using a physical specimen while other parts are modelled numerically [BP-94]

2.9.5 Real Time Hybrid Test

Shake table test on full-size specimen simulating the true dynamic behaviour of a structure, is very costly and difficult to perform. The alternatives to the full-size shake table test are shake table test on reduced scale model and pseudo-dynamic test of full-size specimen performed on extended time scale. But it is significantly difficult to extrapolate the dynamic behaviour of structure particularly when the structure responds non-linearly or includes high-rate dependent components such as dampers.

The solution to these problems is a full-size pseudo-dynamic test but performed in real time, called real time hybrid testing, so that the specimen can respond dynamically. Like the pseudo-dynamic test real time hybrid test is the combination of real time physical testing and numerical modelling. Computer processing the numerical model should be fast enough to cope with physical testing in real time which is computationally very difficult. Until now only single degree of freedom or simple two degree of freedom test have been performed with this methodology.

2.10 Masonry In-plane Test Systems

Different researchers have used different test setup to study the in-plane behavior of masonry walls/piers under quasi-static loading. The differences are in the ways the boundary conditions are simulated and the vertical load is applied to induced vertical stresses in the wall. The test setup should be designed to reproduce the filed condition in laboratory as closely as possible.
The boundary conditions may vary from case to case. A pier with deep spandrel which is a part of a building with rigid diaphragm may be tested under fixed-ended conditions. On the other hand, a pier which is part of a building with flexible diaphragm may be tested as cantilever wall. Lower half portion of fixed-ended pier was tested as cantilever pier by Abrams et al. [ASL-07]. Other piers may have boundary conditions in between fixed and free.

Based on the boundary conditions a test setup can simulate, it may be broadly classified into cantilever setup and fixed-ended setup.

### 2.10.1 Cantilever Test Setup

The cantilever setup is used to test the wall fixed at the base and free to rotate and translate at the top. It is one of the simplest setup comprising mechanisms for vertical and horizontal loads applied to the specimen (Figure 2-28). Usually the specimen is constructed on a reinforced concrete beam which is later on fixed with the floor. Also a strong reinforced concrete or steel beam is provided at the top of the specimen in order to distribute the vertical load and to attach the horizontal jack/actuator with it. Rollers are provided between the vertical jack/actuator and the top beam to allow translation and also rotation. Some researchers have tried to use two jacks/actuators for vertical load application which may retrain the rotation of the specimen.

Horizontal jack/actuator must be attached to the top beam through two in-line hinges in order to allow free rotation and translation of the specimen. To protect the jack/actuator from the damaging effect of shear it is advisable to provide one hinge each at its back and front.

The main problem using manual control hydraulic jack in cantilever testing is difficulty in keeping the vertical load constant which varies as the specimen rocks. Dead load amplified through lever mechanism [LH-90] may be used as vertical load to keep vertical load constant during testing.

The masonry specimens tested in cantilever setup usually show a rocking behaviour under practical vertical stress. Though rocking is a stable and most desirable mode but most of the piers in a true building behave in shear sliding mode. The shear mode may be simulated by increasing the vertical stress to a level more than 20% of the compressive strength of the masonry. At this level of vertical stress diagonal crack produced passes through the bricks and thus giving a brittle failure. These disadvantages limit the use of cantilever test. It is usually used for comparative study of different masonry specimens and piers at the ground storey of multi-storey building with high vertical stress.
2.10.2 Fixed-Ended Test Setup

This type of test setup is applied to a wall specimen with both end fixed. The fixed ended conditions may be reproduced in laboratory by two different ways:

- Mechanical devices (Figure 2-29)
- Controlling the movement of vertical actuators using control algorithm
The function of both is to restrain the top rotation of the specimen but at the same time allow horizontal translation. Restraining the top rotation will induce bending moment at the top and thus creating a double-bending condition.

The function of the mechanical device is to keep the top beam horizontal. The rotation of the beam will only be possible if one of its members got elongated or shortened, which, in the case of stiff member is not possible.

In Figure 2-29 the mechanical device is placed above the top beam, however it may be placed below the top beam fixed to the floor. The first mechanical devise was probably used by Jiang an Jinqian [JJ-86] placed below the top beam. Miha Tomazevic [MT-96] and M. Javed [MJ-08] used a mechanical device placed above the top beam.

Recently a new mechanical device was proposed (but not implemented) at the EUCENTRE, Pavia, Italy [AC-07]. The devise manly consists of a torsionally very stiff tube and two arms connecting the tube with the top beam, Figure 2-30. If the specimen tends to rotate, one side will try to move down inducing torsion movement in the tube. On the other side of the tube an opposite torque will be created which will resist the rotation of the tube and hence the rotation of the specimen.
3. **QUASI-STATIC TEST OF URM ISOLATED PIERS**

3.1 Introduction

The most important objective of this research was to evaluate the effectiveness of the selected retrofitting techniques and to developed guidelines for their design and implementation. It was decided to study two retrofitting techniques namely reinforced plastering and structural repointing in combination of cement based grout injection.

This chapter presents an experimental study on the performance of isolated masonry piers tested under quasi-static loading before and after retrofitting. The chapter starts with a general description of the quasi-static test, followed by specimen’s description, mechanical properties of materials and test setup. Damage pattern and force-deformation behavior of each piers before and after retrofitting are discussed. Also the results before and after retrofitting are compared to evaluate the effectiveness of the selected techniques.

3.2 Quasi-Static Test: General

Quasi-static test was performed on isolated piers, full scale perforated walls and full scale rooms. The test procedure and data processing and analysis for all the quasi-static load tests were almost similar. The tests were conducted in displacement control environment, following almost similar displacement pattern, Figure 3-1. Each displacement cycle was repeated three times.
The data recorded during test were subjected to a noise filter using a three point moving average method. The corrected data was then used to plot the hysteresis loops and to determine other parameters like displacement ductility, equivalent viscous damping, etc. An Excel sheet in Visual Basic Application (VBA) was developed to filter the raw data, to plot the hysteresis loops of the specified gauges, envelope or backbone curves, elasto-plastic curves and variation of equivalent viscous damping with increasing displacement.

The envelope curve was developed by joining the points corresponding to the peak resistance of the adjacent displacement cycles. The elasto-plastic curve was developed from the average envelope curve (average of +ve and -ve curves) using the procedure proposed by Magenes and Calvi [MC-97], Figure 3-2, which is based on equal energy principle. According to their model, ultimate resistance, $V_u = 0.9 \, V_{max}$, maximum resistance and the effective stiffness $k_{eff}$ is calculated as the ratio of $0.75V_u$ to the corresponding displacement on the envelope curve. The yield displacement $\Delta_y = V_u/k_{eff}$ and the ultimate displacement, $\Delta_u$ corresponds to a point after peak resistance when the strength drops to $0.8V_u$. The displacement ductility is then calculated as the ratio of ultimate to yield displacement, $\mu_D = \Delta_u/\Delta_y$.

The equivalent viscous damping was calculated from the following expression:

$$\xi_{eq} = \frac{E_d}{2\pi E_{imp}} \tag{3-1}$$
Where $E_d$ is the energy dissipated in the wall per cycle, equal to the average area of three cycles at the same displacement level, and $E_{inp}$ is the input energy calculated as sum of the half product of maximum load and the corresponding displacements in positive and negative loading, Figure 3-3.

![Figure 3-3: Evaluation of Input Energy (left) and Dissipated Energy (right)](image)

### 3.3 Unreinforced Masonry Pier: Before Retrofit

Sixteen masonry piers, P1 through P16 were tested under quasi-static loading in the Structural Engineering Laboratory of department of Civil Engineering, UET Peshawar. Piers P1 to P4 (P-A series) were tested till their complete failure. Remaining 12 piers, to be retrofitted later on, were loaded approximately to the peak resistance in order to keep the damages produced within repairable limits and to avoid more damages during transportation. The damaged specimens were repaired and/or retrofitted with the proposed techniques and tested under the same test conditions.

#### 3.3.1 Test Specimens

Details of the test specimens are shown in Figure 3-4. Specimens P1 through P12 were different from P13 through P16. The differences were in the type of bricks, the thickness of bottom reinforced slab/beam, the level of vertical stress, the position of horizontal load with respect to the bottom of specimens and the masons who constructed the specimens. All the specimens were constructed in typical English bond with mortar joint thickness of about 1/2" (12.5 mm). The average dimensions of all the piers in the mentioned series are shown in Table 3-1.

![Figure 3-4: Geometry of Test Specimens: Piers Before Retrofitting](image)
Table 3-1: Average Geometrical Dimensions of Piers in each Series

<table>
<thead>
<tr>
<th>Series</th>
<th>Piers</th>
<th>Average Length (inch [mm])</th>
<th>Average Height (inch [mm])</th>
<th>Average Thickness (inch [mm])</th>
</tr>
</thead>
<tbody>
<tr>
<td>P-A Series P1-P4</td>
<td>47.25 [1200]</td>
<td>46.46 [1180]</td>
<td>9.00 [229]</td>
<td></td>
</tr>
<tr>
<td>P-B Series P5-P12</td>
<td>47.25 [1200]</td>
<td>46.46 [1180]</td>
<td>9.00 [229]</td>
<td></td>
</tr>
<tr>
<td>P-C Series P13-P16</td>
<td>48.00 [1220]</td>
<td>48.00 [1220]</td>
<td>9.00 [229]</td>
<td></td>
</tr>
</tbody>
</table>

3.3.2 Specimen Materials

Materials for the construction of specimens were selected according to the common practice in the Northern Pakistan.

All the specimens were constructed with solid burnt clay bricks of the same brand. First lot of bricks which was used in the construction of specimens P1 through P12 (P-A and P-B series) was found to be weaker in compression and stronger in tension than the second lot of bricks used in the construction of specimens P13 to P16 (P-C series), Table 3-2. The nominal size of bricks was 9"x4.5"x3" (229 mm x114 mm x76 mm). Water absorption of bricks was found to be 20%. All the bricks were thoroughly soaked in water before being laid in wall.

Table 3-2: Compressive Strength and Modulus Rupture of Bricks (ASTM C67)

<table>
<thead>
<tr>
<th>S.No</th>
<th>Specimens</th>
<th>Average Compressive Strength</th>
<th>Average Modulus of Rupture</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Psi (MPa)</td>
<td>C.O.V. (%)</td>
</tr>
<tr>
<td>1</td>
<td>P-A &amp; P-B Series P1 to P12</td>
<td>3170.3 (21.86)</td>
<td>13.8</td>
</tr>
<tr>
<td>2</td>
<td>P-C Series P13 to P16</td>
<td>4813.1 (33.19)</td>
<td>28.3</td>
</tr>
</tbody>
</table>

Table 3-3: Compressive Strength of Mortar Cubes (ASTM C-109)

<table>
<thead>
<tr>
<th>S.No</th>
<th>Specimens</th>
<th>Average Strength Psi (MPa)</th>
<th>C.O.V. (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>P1 and P5</td>
<td>1921.2 (13.25)</td>
<td>3.5</td>
</tr>
<tr>
<td>2</td>
<td>P2 and P6</td>
<td>1241.6 (8.56)</td>
<td>6.6</td>
</tr>
<tr>
<td>3</td>
<td>P3 and P7</td>
<td>949.9 (6.55)</td>
<td>10.0</td>
</tr>
<tr>
<td>4</td>
<td>P4 and P8</td>
<td>885.3 (6.10)</td>
<td>7.8</td>
</tr>
<tr>
<td>5</td>
<td>P9 and P10</td>
<td>1472.1 (10.15)</td>
<td>20.3</td>
</tr>
<tr>
<td>6</td>
<td>P11 and P12</td>
<td>1746.7 (12.05)</td>
<td>3.7</td>
</tr>
<tr>
<td>7</td>
<td>P13</td>
<td>1717.3 (11.84)</td>
<td>14.2</td>
</tr>
<tr>
<td>8</td>
<td>P14 and P15</td>
<td>1673.2 (11.54)</td>
<td>4.3</td>
</tr>
<tr>
<td>10</td>
<td>P16</td>
<td>1953.3 (13.47)</td>
<td>25.4</td>
</tr>
</tbody>
</table>
For all the specimens the mortar was prepared in the same weighted proportion (one part of ordinary Portland cement, 4 parts of locally available sand and 4 parts of khaka which is a stone dust from quarries) and same water cement ratio (w/c: 1.6). However variation in the compressive strength (Table 3-3) of mortar cube was noticed which may probably be due to variation in the type of sand and khaka used in the preparation of mortar. The water-cement ratio was decided based on the consistency of the mortar in order to achieve a workable mix. It was insured to use wet mortar with in one hour after being mixed with water in order to avoid excessive loss in strength with time. 2” (25 mm) mortar cubes were prepared and then kept for 28 days in potable water for curing at room temperature. The cubes were tested on the same date on which the corresponding specimens were tested.

Assemblages were also prepared to characterise the masonry behaviour in compression and diagonal tension. Five masonry prisms (Figure 3-5) of average size, 16.5”x8.6”x19.7” (420 x 218 x 500) were prepared in English bond using the above mentioned materials and tested under direct compression according to ASTM C1314. Compressive strength of masonry was calculated by dividing peak vertical load with the cross sectional area of prism. Modulus of elasticity was calculated from the stress-strain curve (Figure 3-5) as secant modulus between two points corresponding to 1/20th and 1/3rd of the peak vertical stress. Results of the experimental test are given in Table 3-4.

Table 3-4: Masonry Assemblage, Direct Compression and Diagonal Compression Test Results

<table>
<thead>
<tr>
<th>S.No</th>
<th>Description</th>
<th>Direct Compression Test</th>
<th>Diagonal Compression Test</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Compressive Strength, psi (MPa)</td>
<td>Mod. of Elasticity, ksi (MPa)</td>
</tr>
<tr>
<td>1</td>
<td>Average</td>
<td>719.6 (4.96)</td>
<td>204.0 (1407)</td>
</tr>
<tr>
<td>2</td>
<td>C.O.V (%)</td>
<td>12.7</td>
<td>27.9</td>
</tr>
</tbody>
</table>

Figure 3-5: Compression Test Masonry Prism (right), Stress-Strain Curve (left)
Similarly three square specimens (Figure 3-6) were prepared in English bond using the above mentioned materials, for diagonal compression test. Specimens were 27.6” (700 mm) high, 28.3” (720 mm) long and 8.86” (225 mm) on average. All the specimens were tested according to ASTM E519-02. The shear stresses and shear strains were calculated from the ASTM relation.

ASTM standard is silent with regard to the calculation of tensile strength from diagonal compression test. The tensile strength was, therefore taken as the principle tensile strength calculated from the equation below.

\[ f_{tu} = \frac{0.5P_u}{A_n} \]

3-2

Where \( P_u \) is the ultimate test load and \( A_n \) is the net area equal to product of the average thickness and the average of length and height.

The modulus of rigidity was calculated from the shear stress-shear strain curve (Figure 3-6) as the secant modulus between two points corresponding to 1/20th and 1/3rd of peak shear strength. Results of the diagonal test are given in Table 3-4. The test results for modulus of rigidity are not encouraging as the results are too low when compared with standard results.

3.3.3 Specimen Test Setup

All the specimens were tested in the loading frame of Structural Engineering Laboratory, Department of Civil Engineering, University of Engineering & Technology, Peshawar. The specimens were fixed at the base and kept free to rotate and translate in horizontal direction at top thus simulating cantilever boundary conditions. Horizontal and vertical loads were applied through hydraulic jacks controlled through a manual hydraulic pump. Load cells of 112.4 kips (500 KN) capacity were used to measure horizontal and vertical loads. Loading shoes attached to the ends of the top concrete beam through 4 numbers \( \Phi3/4” \) (19.0 mm) bolts (Figure 3-7 and Figure 3-8), were designed to transfer horizontal load from jack to the specimen. The idea behind the use of such a mechanism was to convert the pull load from the jack to a push load from the other side of the specimen, in order to subject the specimen to the same load condition during positive
and negative displacement cycle. Four horizontal lubricated rollers positioned as shown in Figure 3-7 on a steel plate placed on top of the top concrete beam, were used to allow free horizontal movement of the specimen. Vertical load was applied to a steel beam placed on the rollers.

Displacement at various locations was measured using displacement transducers, Figure 3-7 and Figure 3-8. Displacement transducer-1, measuring horizontal top displacement on the front face of the pier at the horizontal load level, was used as control gauge. Transducer-2 was installed on the back face of the wall at the same horizontal load level in order to record any possible rotation along the vertical axis of the pier. Transducers-3 and 4 were used to record the vertical rocking displacements.
of the pier. Transducers-5 and 6 were installed diagonally, recording the shear distortion of pier. Transducer-7 was attached to the bottom slab/beam recording any possible vertical lifting on the bottom slab/beam. The last transducer-8 was attached to top beam to record out-of-plane displacement.

The load cells and displacement transducers were connected to data acquisition system (UCAM-70A) as shown in Figure 3-8 (right). All the displacement transducers and load cells were calibrated before test.

### 3.3.4 Pre-Compression

Detail of pre-compression on each test pier is given in Table 3-5. Specimens in P-A series were subjected to different vertical loads in order to determine the level of vertical stress enough to simulate diagonal shear failure mode in the piers. Piers P1 and P2 were tested under vertical loads of 25.3 kips (112.6 KN) and 42.3 kips (188.2 KN) respectively, equivalent to a vertical stress of 60 psi (0.41 MPa) and 100 psi (0.69 MPa) respectively. Their behaviour was mainly governed by rocking mode. Therefore the remaining two piers P3 and P4 were subjected to an increased vertical load of 59.5 kips (264.8 KN) equivalent of a stress of 140 psi (0.97 MPa). A mixed shear-flexural behaviour was noticed in piers P3 and P4.

Based on the test results of piers in P-A series it was decided to test the remaining piers under a pre-compression stress of 140 psi (0.97 MPa). However piers in P-C series were tested under a vertical load of 66.1 kips (294.1 KN) equivalent to a stress of 155 psi (1.07 MPa). The reason behind this increase in pre-compression in P-C series was the increase in the height of position of horizontal load with respect to the base of specimen, Figure 3-4.

<table>
<thead>
<tr>
<th>S.No</th>
<th>Series</th>
<th>Specimens</th>
<th>Pre-compression Psi (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>P-A</td>
<td>P1</td>
<td>60 (0.41)</td>
</tr>
<tr>
<td>2</td>
<td>P-A</td>
<td>P2</td>
<td>100 (0.69)</td>
</tr>
<tr>
<td>3</td>
<td>P-A</td>
<td>P3, P4</td>
<td>140 (0.97)</td>
</tr>
<tr>
<td>4</td>
<td>P-B</td>
<td>P5-P12</td>
<td>140 (0.97)</td>
</tr>
<tr>
<td>5</td>
<td>P-C</td>
<td>P13-P16</td>
<td>155 (1.07)</td>
</tr>
</tbody>
</table>

### 3.3.5 Test Procedure

After completing the test setup, all the gauges were initialized to zero. Vertical stress was first gradually applied to the specimen to the desired value (140 psi (0.97 MPa) in case of piers P3 through P12 and 155 psi (1.07 MPa) in case of P13 though P16). The displacement transducers were once again initialized to zero after the application of vertical load. Displacement transducer-1 was used as controlled gauge.
Vertical load was found to be varying with increasing displacement cycles due to rocking of the piers. The variation in vertical load was adjusted during test using the hydraulic system. During and after each displacement cycle the piers were thoroughly examined for any crack produced. The cracks were marked with a number showing the displacement in millimetre of the corresponding displacement cycle. Photographs were also taken at the end each cycle (Figure 3-9).

![Figure 3-9: Marking of cracks produced during test](image)

Before retrofitting, the piers (P5 to P16) were tested up to a displacement level at which the horizontal load dropped or the damages produced were found to be severe enough to disturb the overall integrity of the wall. However, the piers after retrofitting were tested to the ultimate displacement where the horizontal load dropped by more than 20% from the peak or the pier moved in out-of-plane direction (walking).

Average of the data recorded by the displacement transducers 1 and 2 was used in the data analysis which corresponds to actual displacement at the centre of wall.

### 3.3.6 Failure Modes and Damaged Pattern

As already mentioned that the piers in P-A series were tested till complete failure, whereas, the remaining piers (P-B and P-C series) were tested up to the peak resistance in order to keep the damages with in repairable limits. The piers in P-A series were discarded without being subjected to retrofitting while the other piers of P-B and P-C series were retested after being subjected to different retrofitting techniques.

#### 3.3.6.1 P-A Series

Pier P1 and P2 of P-A series were tested under pre-compression of 60 psi (0.41 MPa) and 100 psi (0.69 MPa) respectively. Both the piers showed almost similar behaviour of rocking followed by out-of-plane sliding and rotation about the vertical axis of specimen (Figure 3-10). The very first flexural crack was observed at heal during 0.04” (1.0 mm) cycle. After the formation horizontal crack across the bed joint, the piers then started a rigid body rotation about the toe without a significant
increase in the strength. The damages were concentrated along the bed joint across the base of specimen. Due to the out-of-plane movement the test was stopped well before the ultimate conditions.

Figure 3-10: Damage Pattern of Specimen P1 and out-of-plane sliding (walking)

Figure 3-11: Damage Pattern of Specimen P3 (left), Vertical Side Splitting (right)

Remaining two piers P3 and P4 of P-A series were tested under a pre-compression of 140 psi (0.97 MPa). Similar to the first two piers the flexural cracks appeared during 0.04” (1.0 mm) displacement cycle.

Figure 3-12: Final damage Pattern of Specimen P4, in-plane movement at Toe
The damage mechanism started with appearance of vertical compression cracks followed by diagonal shear cracks and vertical splitting at high drift ratios.

In specimen P3 the two outer triangular wedges bulged out at high drift ratios, Figure 3-11. Whereas the final damaged condition of specimen P4 appeared in the form of outer movement of masonry at the toe regions, Figure 3-12. Due to high pre-compression the cracks were mostly passing through bricks, resulting in the crushing of bricks.

3.3.6.2 P-B Series

All the piers of P-B series (P5 to P12) were tested under a pre-compression stress of 140 psi (0.97 MPa). All the piers showed more or less similar behaviour, Figure 3-13. Starting with flexural cracks during 0.04” (1.0 mm) cycle, vertical cracks following the mortar bed and head joints were then produced which finally resulted in the formation of diagonal cracks during 0.39” (10 mm) cycle. The tests were stopped after the formation of diagonal cracks.

![Figure 3-13: Final damage Pattern of Specimen P5 (P-B Series)](image)

3.3.6.3 P-C Series

The specimens in P-C series were slightly different from those of P-A and P-B series. The major differences were in the specimen height and brick strength which were found to be on higher side. An attempt was made to test the first pier (P13) of P-C series under a pre-compression of 140 psi (0.97 MPa), but the specimen responded in rocking mode right from the beginning of test. The test was, therefore, stopped in 0.08” (2.0 mm) displacement cycle.

Pre-compression stress was increased to 155 psi (1.07 MPa) and the specimen was re-tested from the beginning. The specimen then behaved in rocking-shear mode. The remaining three specimens were then tested under a pre-compression stress of 155 psi (1.07 MPa). The first flexural crack appeared at a displacement of 0.04” (1.0 mm). Initially the pier rocked, followed by the formation of vertical cracks and finally a
diagonal crack which appeared at the peak lateral resistance, Figure 3-14. Bricks at the toe region were crushed during large displacement cycles.

![Figure 3-14: Damage pattern of Piers P16 (P-C Series)](image)

3.3.7 **Hysteretic Behaviour**

The force-deformation hysteresis loops along with the force-deformation envelope curves and idealized bilinear curves of all the tested piers are shown in Figure 3-15 through Figure 3-21.

### 3.3.7.1 P-A Series

The first two piers P1 and P2, which were tested under a comparatively low pre-compression stress, behaved in pure rocking mode, Figure 3-15. Piers P3 and P4, tested till the ultimate collapse, were considered as control specimens. The hysteretic response of these two piers was similar to each other. Both the piers responded in shear mode which is obvious from the shape of hysteresis loops, Figure 3-15. The average force-deformation envelope curves of P-A series are given in Figure 3-16. The behaviour of P3 and P4 was found to be similar in the elastic range. However the peak strength of P4 was slightly more than that of P3. Table 3-6 shows peak resistance, effective stiffness and ductility ratio (calculated from bilinear curve) of piers in P-A series. For linearly elastic system the elastic lateral stiffness is independent of the vertical stress. However the effective stiffness of pier P1 is comparatively smaller (though the mortar strength is double than that of P3 and P4) than other piers of the series, Figure 3-16. The reason behind this may be the inherent variation in the physical and mechanical properties of masonry system. However, no significant scatter was found in the ductility ratios of piers P3 and P4 tested under same vertical stress.

On the other hand the peak lateral resistance was found to be increasing with increase in vertical stress which is because of the increase in the shear strength with increase in the vertical stress.
Figure 3-15: Force Deformation Hysteresis Loops of Piers P1-P4

Figure 3-16: Force-Deformation Envelope Curves of Piers P1-P4

Table 3-6: P-A Series: Peak Resistance, Elastic Stiffness and Ductility Ratios

<table>
<thead>
<tr>
<th>Pier</th>
<th>Mortar Strength psi (MPa)</th>
<th>Vertical Stress psi (MPa)</th>
<th>Cracking Load Kips</th>
<th>Peak Lateral Resistance Kips (KN)</th>
<th>Elastic Stiffness Kips/in (KN/mm)</th>
<th>Ductility Ratio</th>
</tr>
</thead>
<tbody>
<tr>
<td>P1</td>
<td>1921.2 (13.25)</td>
<td>60 (0.41)</td>
<td>-</td>
<td>17.47 (77.8)</td>
<td>66.6 (11.67)</td>
<td>-</td>
</tr>
<tr>
<td>P2</td>
<td>1241.6 (8.56)</td>
<td>100 (0.69)</td>
<td>-</td>
<td>23.16 (103.1)</td>
<td>240.7 (42.18)</td>
<td>-</td>
</tr>
<tr>
<td>P3</td>
<td>949.9 (6.55)</td>
<td>140 (0.97)</td>
<td>10.60 (47.0)</td>
<td>27.02 (120.3)</td>
<td>268.5 (47.04)</td>
<td>5.43</td>
</tr>
<tr>
<td>P4</td>
<td>885.3 (6.10)</td>
<td>140 (0.97)</td>
<td>8.88 (39.5)</td>
<td>29.60 (131.7)</td>
<td>225.6 (39.53)</td>
<td>5.42</td>
</tr>
</tbody>
</table>
### 3.3.7.2 P-B Series

The force-deformation hysteretic response of piers in P-B series is shown in Figure 3-17 and Figure 3-18. A mixed shear-rocking behaviour was observed in most of the piers. All piers of P-B series except piers P5 and P10 were tested up to 10.0 mm displacement during which the diagonal cracks produced and the strength started degrading. Piers P5 and P10 were tested till 6.0 mm and 12.0 mm displacements respectively. Hysteresis loops were found widening with increase in the damages. The loops in the last cycle were much wider than the loops of 2nd last displacement cycle showing high damages in the last cycle during which the diagonal cracks formed.

![Graphs showing force-deformation hysteretic response](image)

Figure 3-17: Force Deformation Hysteresis Loops of Piers P5-P8 before Retrofittting

The force-deformation envelope curves of piers in P-B series are shown in Figure 3-19. The hysteresis loops and envelope curves are identical in pairs, i.e. P5 is similar to P8, P6 to P7, P9 to P10 and P11 to P12. One important aspect of this behaviour is the variation in the workmanship. The first four piers of P-B series were cast by one mason in two days; P5 and P8 on first day and P6 and P7 on second day. Likewise the last four piers were cast by another mason making two piers per day. Since the variation in the materials was not so much high, therefore the only reason for scatter in the behaviour seems to be due to the variation in the workmanship.

Table 3-7 gives the peak lateral resistance, elastic stiffness and cracking load of all piers of P-B series. Comparatively less variation was observed in the peak lateral resistance (COV = 9.03%) than the variation
in the elastic stiffness (COV = 44.08%) which is due to variation in the yield displacement of the piers. The elastic stiffness of P9 was found to be about 3 times that of P6. The scatter in the elastic stiffness may be attributed to the differences in the workmanship and material properties. The cracking load in Table 3-7 was calculated from the product of cracking displacement observed during test and the elastic stiffness of the pier.

Since piers P5 through P12 were tested till their peak resistances (i.e. the formation of diagonal cracks), their ultimate displacements and ductility ratios could not be established. The ductility ratio of the control specimens were utilized in the comparison of the test results before and after retrofitting.
Table 3-7: P-B Series: Peak Resistance, Elastic Stiffness

<table>
<thead>
<tr>
<th>Pier</th>
<th>Mortar Compressive Strength Psi (MPa)</th>
<th>Pre-Compression Stress Psi (MPa)</th>
<th>Peak Lateral Resistance Kips (KN)</th>
<th>Elastic Stiffness Kips/in (KN/mm)</th>
<th>Cracking Load kips (KN)</th>
</tr>
</thead>
<tbody>
<tr>
<td>P5</td>
<td>1921.2 (13.25)</td>
<td>140 (0.97)</td>
<td>29.69 (132.1)</td>
<td>193.8 (33.97)</td>
<td>7.63 (34.0)</td>
</tr>
<tr>
<td>P6</td>
<td>1241.6 (8.56)</td>
<td>140 (0.97)</td>
<td>27.46 (122.2)</td>
<td>125.6 (22.01)</td>
<td>7.42 (33.1)</td>
</tr>
<tr>
<td>P7</td>
<td>949.9 (6.55)</td>
<td>140 (0.97)</td>
<td>28.78 (128.1)</td>
<td>131.6 (23.06)</td>
<td>7.20 (32.1)</td>
</tr>
<tr>
<td>P8</td>
<td>885.3 (6.10)</td>
<td>140 (0.97)</td>
<td>31.73 (141.3)</td>
<td>181.6 (31.79)</td>
<td>7.14 (31.8)</td>
</tr>
<tr>
<td>P9</td>
<td>1472.1 (10.15)</td>
<td>140 (0.97)</td>
<td>27.06 (120.47)</td>
<td>386.8 (67.78)</td>
<td>7.61 (33.9)</td>
</tr>
<tr>
<td>P10</td>
<td>1472.1 (10.15)</td>
<td>140 (0.97)</td>
<td>25.23 (112.3)</td>
<td>288.6 (50.57)</td>
<td>11.36 (50.6)</td>
</tr>
<tr>
<td>P11</td>
<td>1446.7 (9.98)</td>
<td>140 (0.97)</td>
<td>24.97 (111.1)</td>
<td>163.1 (28.59)</td>
<td>9.63 (42.9)</td>
</tr>
<tr>
<td>P12</td>
<td>1446.7 (9.98)</td>
<td>140 (0.97)</td>
<td>24.86 (110.6)</td>
<td>157.8 (27.65)</td>
<td>9.30 (41.5)</td>
</tr>
<tr>
<td>Average</td>
<td>1354.4 (9.34)</td>
<td>140 (0.97)</td>
<td>27.47 (122.3)</td>
<td>203.6 (35.69)</td>
<td>8.41 (37.4)</td>
</tr>
<tr>
<td>C.O.V (%)</td>
<td>24.36</td>
<td>0.00</td>
<td>9.03</td>
<td>44.08</td>
<td>18.14</td>
</tr>
</tbody>
</table>

Figure 3-20: Force Deformation Hysteresis Loops of Piers P13-P16 before Retrofitting

3.3.7.3 P-C Series

As already reported, that the piers in P-C series were different from those in P-B series with regard to type of bricks, height of piers and the
amount of vertical pre-compression. The force-deformation hysteresis loops and envelope curves of piers in P-C series are shown in Figure 3-20 and Figure 3-21 respectively. Initially the hysteresis loops were very tight indicating a small amount of energy dissipation but with increasing displacement the loops were found to be widened.

The behaviour of all piers was found almost similar in positive and negative load directions. Good agreement between the average force deformation envelopes in terms of initial stiffness, peak strength and displacement capacity, can be seen from Figure 3-21.

Peak resistance and elastic stiffness of all piers in P-C series are shown in Table 3-8. The average value of peak resistance was found to be 36.52 kips (162.5 KN) with a coefficient of variation of 2.54%. The elastic stiffness of piers P16 was about 1.5 times the average of remaining three piers of the series which may be attributed to the inherent variability in the material properties of masonry.

<table>
<thead>
<tr>
<th>Pier</th>
<th>Mortar Compressive Strength Psi (MPa)</th>
<th>Pre-Compression Stress Psi (MPa)</th>
<th>Peak Lateral Resistance Kips (KN)</th>
<th>Elastic Stiffness Kips/in (KN/mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>P13</td>
<td>1717.3 (11.84)</td>
<td>155 (1.07)</td>
<td>37.68 (167.8)</td>
<td>344.6 (60.39)</td>
</tr>
<tr>
<td>P14</td>
<td>1673.2 (11.45)</td>
<td>155 (1.07)</td>
<td>35.97 (160.1)</td>
<td>342.6 (60.0)</td>
</tr>
<tr>
<td>P15</td>
<td>1673.2 (11.45)</td>
<td>155 (1.07)</td>
<td>36.81 (163.8)</td>
<td>350.6 (61.43)</td>
</tr>
<tr>
<td>P16</td>
<td>1953.3 (13.47)</td>
<td>155 (1.07)</td>
<td>35.62 (158.5)</td>
<td>525.2 (92.04)</td>
</tr>
<tr>
<td>Average</td>
<td>1754.2 (12.10)</td>
<td>155 (1.07)</td>
<td>36.52 (162.5)</td>
<td>390.8 (68.47)</td>
</tr>
</tbody>
</table>

C.O.V (%) | 7.66 | 0.00 | 2.54 | 22.96

Figure 3-21: Force-Deformation Envelope Curves of Piers P13-P16
3.3.8 Comparison of Experimental Results with Empirical Relations:
Before Retrofitting

The elastic stiffness $k_p$, cracking lateral load $V_{cr}$ and maximum lateral load $V_{max}$ as determined experimentally were compared with those obtained from empirical equations given in Section-2.6 and re-written below:

$$k_p = \frac{1}{H_p^3 \left( \frac{3E_m I_g}{A_m G_m} \right)}$$  \hspace{1cm} (3-3)

$$V_{cr} = \frac{L_p^2 t_p}{6H_p} (p + f_t)$$  \hspace{1cm} (3-4)

$$V_{ro} = 0.9 \alpha \left( \frac{pL_p^2 t_p}{H_p} \right)$$  \hspace{1cm} (3-5)

$$V_{ve} = \alpha \left( \frac{pL_p^2 t_p}{H_p} \right) \left( 1 - \frac{P}{0.7f_m} \right)$$  \hspace{1cm} (3-6)

$$V_{dc} = \frac{f_{tu} L_p t_p}{b} \sqrt{1 + \frac{P}{f_{tu}}}$$  \hspace{1cm} (3-7)

<table>
<thead>
<tr>
<th>Table 3-9: Material and Geometrical Properties of Piers in P-A and P-B Series</th>
</tr>
</thead>
<tbody>
<tr>
<td>Compressive Strength of Masonry, $f_{tu}$, psi (MPa)</td>
</tr>
<tr>
<td>Modulus of Elasticity of Masonry, $E_m$, ksi (MPa)</td>
</tr>
<tr>
<td>Modulus of Rigidity of Masonry, $G_m = 0.4 E_m$, ksi (MPa)</td>
</tr>
<tr>
<td>Diagonal Tensile Strength of Masonry, $f_{tu}$, psi (MPa)</td>
</tr>
<tr>
<td>Vertical Stress on Piers, $p$, psi (MPa)</td>
</tr>
<tr>
<td>Length of Piers, $L_p$, inch (mm)</td>
</tr>
<tr>
<td>Height of Piers, $H_p$, inch (mm)</td>
</tr>
<tr>
<td>Thickness of Piers, $t_p$, inch (mm)</td>
</tr>
<tr>
<td>Shear Area, $A_s = 0.8 A_p$, in$^2$ (mm$^2$)</td>
</tr>
<tr>
<td>Moment of Inertia of Pier, $I_g$, in$^4$ (mm$^4$)</td>
</tr>
<tr>
<td>Shear Stress Distribution Factor, $b$</td>
</tr>
<tr>
<td>Boundary Condition Factor, $\alpha$</td>
</tr>
</tbody>
</table>

Material and geometrical properties for piers in P-A and P-B series are given in Table 3-9. Putting values in above equations, one can obtain:

$$k_p = 217.6 \text{ kips/in} \quad (38.12 \text{ KN/mm})$$

$$V_{cr} = 9.17 \text{ kips} \quad (40.8 \text{ KN})$$

$$V_{ro} = 24.77 \text{ kips} \quad (110.3 \text{ KN})$$
\[ V_{tc} = 19.87 \text{ kips} \quad (88.46 \text{ KN}) \]
\[ V_{dc} = 30.83 \text{ kips} \quad (137.2 \text{ KN}) \]

It must be noted that Eq-3.6 provides an estimate of lower bound lateral strength of masonry piers. The expected value is taken as 1.3 times the lower bound value, ASCE/SEI 41-06. On the other hand Eq-3.5 gives directly the expected value of lateral strength. Minimum of \( V_{ro} \), 1.3\( V_{tc} \) and \( V_{dc} \) shall be taken as the maximum lateral strength of the piers which came out to be, \( V_{max} = 24.77 \text{ kips} \) (110.3 KN). Table 3-10 and Figure 3-22 below show the comparison of the experimental results with the empirical relations. From the table it is obvious that the empirical relations provide a very good estimate of the strength capacity of unreinforced masonry piers.

**Table 3-10: P-B Series: Comparison of the Experimental Results with the Empirical Relations**

<table>
<thead>
<tr>
<th>Properties (a)</th>
<th>Measured (b)</th>
<th>Estimated (c)</th>
<th>Ratio (d) = (b)/(c)</th>
</tr>
</thead>
<tbody>
<tr>
<td>( k ) = Elastic Stiffness, kips/in ((\text{KN/mm}))</td>
<td>203.6 ( (35.7) )</td>
<td>217.6 ( (38.12) )</td>
<td>0.94</td>
</tr>
<tr>
<td>( V_{cr} ) = Cracking Lateral Load, kips ((\text{KN}))</td>
<td>8.41 ( (37.4) )</td>
<td>9.17 ( (40.8) )</td>
<td>0.92</td>
</tr>
<tr>
<td>( V_{max} ) = Maximum Lateral Load, kips ((\text{KN}))</td>
<td>27.47 ( (122.3) )</td>
<td>24.77 ( (110.3) )</td>
<td>1.11</td>
</tr>
</tbody>
</table>

It is worth mentioning that the estimated diagonal shear strength is greater than the estimated rocking/toe crushing strength and thus the governing failure mode seems to be rocking instead of diagonal shear. However experimental results showed that though initially the piers behaved in a rocking mode but the final failure, in most of the cases, was the formation of diagonal shear cracks. The reason behind this discrepancy between experimental and empirical results may be cracking of some bricks in the toe region prior to the formation of diagonal shear cracks and thus causing a decrease in the diagonal shear capacity.

![Figure 3-22: Comparison of the Experimental Results with the Empirical Relations](image)

The deformation capacity of the tested piers may be compared with the acceptance criteria of the linear and non-linear static procedure of ASCE/SEI 41-06. The Immediate occupancy (IO), life safety (LS) and
collapse prevention (CP) performance levels correspond to 0.1% drift, 0.75 times the ultimate drift, and the ultimate drift respectively. Figure 3-23 shows a comparison of measured (average of P3 and P4) and estimated (ASCE/SEI 41-06 Section 7.3.2.3.1) m-factors used in linear static procedure for different performance levels. Good agreement between measured and estimated m-factors is found, Figure 3-23 (left). Similarly for non-linear static procedure the measured deformation capacities (average of P3 and P4) are compared with the acceptance criteria of ASCE/SEI 41-06 Section 7.3.2.3.2 in Figure 3-23 (right). The estimated deformation capacities seem to be very conservative.

![Figure 3-23: Comparison of estimated and measured m-factors (LSP) and drift ratios (NSP)](image)

### 3.4 Retrofitting of Damage Specimens

All the specimens of P-B and P-C series were subjected to different repair/retrofitting techniques, the details of which are given in Table 3-11. The repaired/retrofitted specimens were named by adding letter “R” with the name of piers before retrofitting, i.e. P5 was named as P5R.

<table>
<thead>
<tr>
<th>Piers</th>
<th>Plaster</th>
<th>Grout</th>
<th>DS Fine Mesh</th>
<th>DS Coarse Mesh</th>
<th>SS Coarse Mesh</th>
<th>Re-Pointing</th>
</tr>
</thead>
<tbody>
<tr>
<td>P5R</td>
<td>x</td>
<td>x</td>
<td>x</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>P6R</td>
<td>x</td>
<td>x</td>
<td>x</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>P7R</td>
<td>x</td>
<td>x</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>P8R</td>
<td>x</td>
<td>x</td>
<td>x</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>P9R</td>
<td>x</td>
<td>x</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>P10R</td>
<td>x</td>
<td>x</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>P11R</td>
<td>x</td>
<td>x</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>P12R</td>
<td>x</td>
<td>x</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>P13R</td>
<td>x</td>
<td>x</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>P14R</td>
<td>x</td>
<td>x</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>P15R</td>
<td>x</td>
<td>x</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>P16R</td>
<td>x</td>
<td>x</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
Broadly three repair/retrofitting techniques were studied; (a) cement-based grout injection, (b) reinforced plaster (ferrocement overlay) with grout injection and (c) structural re-pointing with grout injection. Grout was injected in all specimens except P15R which was retrofitted with coarse mesh applied on one side only. Three types of reinforced plaster were used: low gauge fine mesh applied on both faces (DS Fine Mesh), high gauge coarse mesh applied on both faces (DS Coarse Mesh) and high gauge coarse mesh applied on one face (SS Coarse Mesh).

3.5 Retrofitting Materials and Methods

This section deals with the properties of all repair/retrofitting materials used in the study which includes plaster, fine and coarse mesh, injection grout and bed joint reinforcements.

3.5.1 Repair Mortar

The crushed bricks in the toe region (if any) were replaced with new bricks placed in 1:4 cement sand mortar, Figure 3-24. Piers were supported from the top while replacing the crushed bricks.

![Figure 3-24: Replacement of Damage Bricks](image)

Loose joint mortar in the cracked region was scratched off with sharp tool (Figure 3-24, right) and replaced with new 1:4 cement-sand mortar.

![Figure 3-25: Application of Plaster (left), Plastered Specimens (right)](image)
### 3.5.2 Plaster

1:4 Cement-sand mortar, prepared from locally available sand and Portland cement with water-cement ratio equal to 1.0, was used to plaster the masonry surface. The compressive strength of mortar used in plaster for each of the repaired/retrofitted pier is given in Table 3-12.

<table>
<thead>
<tr>
<th>S. No</th>
<th>Piers</th>
<th>Average Strength psi (MPa)</th>
<th>C.O.V (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>P5R &amp; P12R</td>
<td>2792.7 (19.26)</td>
<td>28.2</td>
</tr>
<tr>
<td>2</td>
<td>P6R &amp; P8R</td>
<td>2609.9 (18.00)</td>
<td>19.1</td>
</tr>
<tr>
<td>3</td>
<td>P7R &amp; P13R</td>
<td>2424.4 (16.72)</td>
<td>5.7</td>
</tr>
<tr>
<td>4</td>
<td>P10R &amp; P11R</td>
<td>2548.4 (17.58)</td>
<td>20.8</td>
</tr>
<tr>
<td>5</td>
<td>P14R &amp; P15R</td>
<td>1869.7 (12.89)</td>
<td>9.4</td>
</tr>
<tr>
<td>6</td>
<td>P9 &amp; P16R</td>
<td>2424.4 (16.72)</td>
<td>5.7</td>
</tr>
</tbody>
</table>

Before application of plaster loose mortar along the cracks produced in the damaged piers was scratched off and cleaned. The average thickness of plaster was kept 0.75" (20 mm). In the case of pier on which mesh was already connected, the plaster was applied in two layers. The first layer was applied to fill the gape between mesh and masonry surface and the second layer was applied to cover the mesh and to get a smooth surface, Figure 3-25.

### 3.5.3 Injection Grout

The damaged specimens were repaired with cement-based injection grout designed based on its penetrability, stability and effectiveness. Penetrability can be improved by adding more water to the mix but it adversely affects the stability and effectiveness causing segregation and thus low strength. Ultra expansion agent which also includes plasticizers was, therefore, used to increase the fluidity without affecting the stability. At the same time it could control the shrinkage problems. Lime was added to increase the stability of mix. After a number of trials a grout mix was designed consisted of 10 parts of Portland cement, one part of lime, Ultra expansion agent at a rate of 250 grams per 50 kilograms of cement and portable water with a water cement ratio of 0.9. The basic idea was similar to as recommended by Miha Tomazevic (MT-99), but lime and expansion agent was used instead of pozzolana.

The water cement ratio of grout mix was decided based on the flow test proposed by Department of Building and Safety, Los Angeles [DBS-08]. In flow test, the grout filled in a cylindrical container 2" (51 mm) in diameter and 4" (102 mm) high is poured slowly and steadily from a height of 12" (305 mm) onto an impervious smooth levelled surface. A puddle of 6" to 8" (152 to 203 mm) diameter indicates a mix with proper consistency. The proposed grout mix formed a puddle of about 8" (203 mm), Figure 3-26.
The overall strength of the grout material was assessed with the help of a test proposed as part of this study. Soaked bricks were first arranged in a water tight mould, Figure 3-27 (left) and the grout mix was then poured in the spaces between bricks. Six specimens were prepared, 3 were subjected to water curing and the remaining 3 samples were just covered with jute bags without any water curing. The specimens, thus prepared, were subjected to diagonal compression test, Figure 3-27 (right).

The average diagonal tensile strength of cured and non-cured specimens was found to be 159 psi (1.10 MPa) and 154 psi (1.06 MPa) respectively, indicating that water curing plays a negligible role in the final strength of grout, Figure 3-28.
It is worth mentioning that the diagonal tensile strength of grouted specimen was very high as compared to the diagonal tensile strength of virgin masonry. Moreover the diagonal tensile strength in the actual grouted wall will be less than that of the tested specimen because in that case only the cracked region will be grouted and the remaining part will have the same virgin strength. However the experimental results showed that the grout injection not only filled the cracks but also the surrounding masonry causing a substantial increase in the strength. It was also observed during the diagonal test that the cracks passed through bricks and caused a sudden failure of specimen which was an indication towards the brittle nature of grouted masonry.

The grout material was injected through injection nozzle/ports as shown in Figure 3-29. The nozzles were 3/8" (10 mm) in diameter and 3" (76 mm) in length. Nozzles were inserted in predrilled holes with 1/2" (12.6 mm) in diameter and depth more than half of the wall depth, and fixed with the help of fast bonding ready-to-mix mortar. The distance between the nozzles was varying between 12" (305 mm) and 18" (457 mm), depending upon the nature of cracks. Stoppers (Figure 3-29) were used to close the nozzles whenever required.

Locally designed and fabricated injector assembly (Figure 3-30) was used to inject grout with specified pressure. The injector assembly comprised compressor, injector where grout was poured, and a couple of pressure pipes. Since agitators were not provided with in the injector, therefore the injector was manually shaken at frequent interval to avoid grout silting at the bottom.

The cracks in the walls were distributed through the wall surface and therefore the plaster, covering the whole pier, was utilized as sealant for the cracks. This technique not only avoided extra charges of the sealant but also minimize the chance of grout coming out of the un-cracked regions.

Before injecting the grout, water (at tap pressure) was passed through the nozzles with the purposes to (a) check the connectivity of nozzles, (b) to moisten the specimen and (c) to remove any dust and loose material from inside the pier. Starting from the top nozzles the water was passed through all the nozzles.

Figure 3-29: Injection Nozzles and Stoppers (left), Inserting and fixing the nozzles (right)
When the wall got sufficiently moistened, grout was poured in the injector and passed through the nozzles from bottom to top, Figure 3-31. The pressure during injection was kept constant at 3 bars for 2 to 3 minutes to allow the pier to absorb the grout. Further delay in maintaining the pressure was avoided because it resulted in chocking of the pipe. Any grout coming out of the adjacent nozzles was stopped with stoppers.

To check the interior texture of the masonry after being injected, a number of inspection cores, 2" (50.8 mm) in diameter (Figure 3-31) were taken from the cracked region. The cracks were found to be completely filled with the grout.

3.5.4 Galvanized Steel Welded Wire Mesh

Two types of steel welded wire meshes were investigated:

1. Low gauge fine mesh made of 0.04" (1.0 mm) wires spaced at 1/2" (12.7 mm) in both direction with a reinforcement ratio, $\rho_s$ of 0.054% of the gross area.

2. High gauge coarse mesh made of 0.06" (1.6 mm) wires spaced at 3/4" (18 mm) in both directions ($\rho_s = 0.092\%$).
The wires running in both directions were welded to each other. The average tensile strength of steel wires, taken from the mesh, was found to be 27,500 psi (189.6 MPa). The steel wire mesh was connected to the pier surface with the help of 1.5" (38 mm) long No.10 screws, plastic plugs and steel washers, Figure 3-32. It must be noted that the screws were fixed with in holes drilled in bricks and not in mortar. Steel nails may directly be hammered in to the bricks but hammering may aggravate the damage condition of the masonry and shall, therefore, be avoided in the case of fragile masonry. The density of the screws, in order to achieve proper connection and to avoid any wrinkle in mesh, was found to be 2 screws per square foot.

![Figure 3-32: Connecting Steel Wire Mesh with Piers](image)

![Figure 3-33: Comparison of Compressive Strength and Tensile Strength of Un-reinforced and Reinforced Specimens](image)

To determine the effect of reinforced plaster on the mechanical characteristics of masonry, three specimens each for direct compression and diagonal compression tests, reinforced with fine mesh applied on both sides and subsequently plastered with 1:4 CSM mortar, were tested. The geometry, materials and test setup were kept similar to those used for un-reinforced specimens. Appreciable increase in the compressive strength (about 24%) and diagonal tensile strength (about 74%) were noticed, Figure 3-33.

Since the application of steel mesh was accompanied by grout injection and the plaster was also utilized as sealant, therefore mesh was connected to the pier before fixing nozzles to avoid any hurdle during mesh application produced due to nozzles projected from pier surface.
3.5.5 Structural Re-pointing/Bed Joint Reinforcement

1/4" (6.4 mm) diameter plain steel bars were inserted in 1.5" (38 mm) deep grooves made in every third bed joint with the help of drilling machine. The average tensile strength of the reinforcing bars was found to be 34 ksi (234 MPa). Bed joint reinforcements, hooked at the end, were provided on both faces of pier, Figure 3-34 (left). The reinforcements on both faces were connected to each other by two, 1/4" (6.4 mm) diameter steel ties passing through the thickness of pier. One end of the tie was provided with 180⁰ hook and the other end was kept straight before inserting into a drilled hole. After inserting the tie, the straight end was bent by 90⁰, Figure 3-34 (right).

![Image of bed joint reinforcement placed in grooves and cross ties](image)

Figure 3-34: Bed Joint Reinforcement Placed in Grooves (left), Cross Ties (right)

After placing the bed joint reinforcement and connecting them through cross ties, injection nozzles were inserted along the cracks and then grooves were filled with 1:4 cement-sand mortar. Subsequently the whole pier surface was plastered with 1:4 cement-sand mortar.

3.6 Unreinforced Masonry Piers: After Retrofit

3.6.1 Test Setup and Procedure: After Retrofitting

All the specimens after repair/retrofitting were tested under the same test setup that was used for testing before retrofitting, Figure 3-35. The vertical loads, displacement time history for pseudo-static load, etc remained unchanged. The only difference was that the specimens after retrofitting were tested till their complete failure while the specimens before retrofitting were tested till their peak resistance. All the specimens were coated with lime white wash to enhance the visibility of cracks produced during test. To locate the cracks produced during the test, horizontal and vertical grid lines spaced at 6" (152 mm) were marked on all specimens.
3.6.2 Failure Modes and Damaged Pattern: After Retrofitting
This section elaborates on the failure modes and damage pattern of the specimens retrofitted with different techniques.

3.6.2.1 Piers Retrofitted with Grout Injection with Plain Plaster
Damaged piers P9, P10 and P16 were repaired by grout injection and plaster. The final damage patterns of each specimen before and after retrofitting are shown in Figure 3-38 through Figure 3-38.

Figure 3-36: Final Damage Pattern of Pier P9 (left) and P9R (right)

Figure 3-37: Final Damage Pattern of Pier P10 (left) and P10R (right)
Before retrofitting, the damage mechanisms were predominantly shear. Initially all the three piers behaved in rocking mode after retrofitting. The final failure modes appeared in the form of toe crushing for P9R, asymmetric seizer shear cracks in P10R and one side diagonal crack in P16R.

![Figure 3-38: Final Damage Pattern of Pier P16 (left) and P16R (right)](image)

### 3.6.2.2 Double Sided Low Gauge Fine Mesh with Grout Injection and Plaster

The damage piers P5, P6 and P8 were retrofitted with reinforced plaster (low gauge fine mesh) in combination with grout injection and retested as P5R, P6R and P8R respectively. All piers showed almost similar damage pattern before and after retrofitting, Figure 3-39 through Figure 3-41.

![Figure 3-39: Final Damage Pattern of Pier P5 (left) and P5R (right)](image)

![Figure 3-40: Final Damage Pattern of Pier P6 (left) and P6R (right)](image)
The dominating failure mode before retrofitting was diagonal shear, while after retrofitting the piers behaved in a rocking mode which finally resulted in out-of-plane sliding (walking), Figure 3-42. Besides major crack produced at the base due to rocking, some distributed inclined hairline cracks were also observed in the plaster. As against the specimens retrofitted with plain plaster, plaster remained connected with masonry throughout the test which was because of a strong bond established between plaster coating and masonry surface due to the presence wire mesh. The tests were stopped due to excessive out-of-plane movement endangering the conditions.

3.6.2.3 Double Sided High Gauge Coarse Mesh with Grout Injection and Plaster

Damaged piers P7 and P14 were retrofitted with double sided high gauge coarse mesh in combination with grout injection and plaster and retested as P7R and P14R respectively. The damage mechanisms were more the less similar to those observed in specimens reinforced with double sided fine mesh. The final damage patterns are shown in Figure 3-43 and Figure 3-44. The behaviour of pier P7 before retrofitting was predominantly shear while that of P14 was flexural rocking. However rocking followed by toe crushing and out-of-plane movement was the predominant behaviour after retrofitting for both P7R and P14R. Distributed hairline inclined cracks were also observed in plaster of retrofitted specimens.
3.6.2.4 Single Side Coarse Mesh with Grout Injection and Plaster

P13R was retrofitted with single sided high gauge coarse mesh ($\rho_s = 0.046\%$) with grout injection and plaster while specimen P15R was retrofitted with single sided high gauge coarse mesh and plaster without grout injection. No significant cracks appeared in P15 tested before retrofitting and therefore grout injection was avoided. The final damage pattern of both the piers tested before and after retrofitting are shown in Figure 3-45 and Figure 3-46.
The behaviour of both piers before and after retrofitting was predominantly flexural rocking appeared in the form of bed joint horizontal cracks at their base. In case of P13, however, a shear diagonal crack produced in the last cycle of test. The final failure of P13R and P15R appeared in the form of out-of-plane sliding and toe crushing respectively, Figure 3-47.

### 3.6.2.5 Bed Joint Reinforcement with Grout Injection and Plaster

Damaged piers P11 and P12 were retrofitted with bed joint reinforcement applied on both sides in combination with grout injection and plaster and retested as P11R and P12R respectively. Figure 3-48 and Figure 3-49 show the final damage patterns of these piers before and after retrofitting. During setting up the pier P12 for testing, it was damaged in the form of a horizontal crack three courses above the base. Later on the specimen was repaired with a strong mortar.

Before retrofitting both the specimens were tested up to drift ratio of 0.77%. As against other piers of P-B series, P11 did not show a clear diagonal crack; rather vertical cracks following the mortar joints were produced. A diagonal crack was, however, produced in P12. After retrofitting, both piers showed almost identical behaviour. The damage mechanism was rocking accompanied by toe crushing and some minor inclined cracks in plaster. Due to weak bond between masonry and plaster coating, the plaster also detached from masonry in toe regions.
3.6.3 Hysteretic Behaviour: After Retrofitting

The main difference between the hysteretic response of piers tested before and after retrofitting was the response mode. Before retrofitting most of the piers behaved in combination of rocking and shear but the final failure mode was diagonal shear. However after retrofitting, most of the piers behaved in rocking mode followed by toe crushing and/or out-of-plane sliding. Thus the proposed retrofitting techniques are said to effective in increasing the shear capacity of the piers, but the rocking response of the piers limited the increase in overall lateral strength.

In single and double storey brick masonry buildings with rigid diaphragm and deep spandrels, the piers may be considered as fixed at both ends and thus shear behaviour is expected in piers with low aspect ratio even though the vertical stress is very small. In such shear critical piers the techniques will be very much effective in enhancing their strength and deformation capacities as we will see in the test results of a full scale walls and room.

Below is discussed the hysteresis response of retrofitted piers and their comparison with virgin piers in terms of elastic stiffness, maximum resistance, ultimate deformation capacities and energy dissipating characteristics. The energy dissipating characteristic was found to be dependent on the behaviour mode and was not significantly affected by the retrofitting. In shear critical piers (P-B series before retrofitting) the damping was found to be increasing with increasing displacements due to the progressive damages produced with the increasing displacements. However, after retrofitting the behaviour mode changes from shear to
critical and the damage appeared as a single cracks at the base, resulted in less energy dissipation and hence less damping.

3.6.3.1 Piers Retrofitted with Grout Injection and Plain Plaster

The force deformation hysteretic response of piers retrofitted with grout injection and plain plaster is shown in Figure 3-50 in the form of hysteresis loops and envelope curves and their comparison with those before retrofitting. The technique restored the stiffness and strength to the pre-damaged state. The apparent increase in the deformation capacity is not due the intervention rather it is due to the change in behaviour mode from shear to rocking.

Figure 3-50: Force-Deformation Response of Piers Retrofitted with Grout Injection and Plaster

Figure 3-51 shows the force-deformation curves of all piers before and after retrofitting along with that of the control pier, P3 and Figure 3-52 gives a comparison of peak resistance, elastic stiffness and ultimate deformation. The technique is very effective in shear critical piers P9R and P10R and less effective in rocking critical P16R. The increase in peak strength was 13% and 25% in case of P9R and P10R respectively and P16R could achieve 90% of the pre-damaged strength. On the other hand the elastic stiffness was restored in all the cases providing an
increase of 29%, 25% and 26% for P9R, P10R and P16R respectively. In case of P9R and P10R, a very high increase in deformation capacity, 71% and 140% respectively was noticed.

Figure 3-51: Comparison of Envelope Curves: Plain Plaster with Grout

Comparison of equivalent viscous damping of piers retrofitted with grout injection of plain plaster with those tested before retrofitting (calculated from Eq-3.1) is shown in Figure 3-53. In case of P9 and P10 the damping before retrofitting is found greater than that after retrofitting. The reason is the change in the behaviour mode from shear to rocking. However the damping before and after retrofitting for pier P16 followed initially similar trend because the behaviour in both case was rocking. At drift ratio greater than 0.63%, a shear crack developed in P16 due to which more energy was dissipated which resulted in an increase in the damping. The damping in non-retrofitted pier increased with displacement. However the damping in retrofitted piers remained almost constant. The reason behind this behaviour is increase in the
dissipated energy with damages which produced in the non-retrofitted piers.

![Comparison: Equivalent Viscous Damping: Grout + Plain Plaster](image)

**Figure 3-53: Comparison: Equivalent Viscous Damping: Grout + Plain Plaster**

### 3.6.3.2 Piers Retrofitted with Double Side Fine Mesh with Grout Injection and Plaster

The dominating failure mode before retrofitting was diagonal shear, while after retrofitting the piers behaved in a rocking mode which finally resulted in out-of-plane sliding (walking). The force-deformation responses of the piers retrofitted with grout injection and reinforced plaster are shown in Figure 3-54 along with the results before retrofitting. Clearly one can see the change of behavioural mode from shear to flexural, which indicates that the technique is very effective in enhancing the shear capacity.

Figure 3-55 and Figure 3-56 compare the force-deformation behaviour before and after retrofitting. Significant increase in the effective stiffness is observed which is:

1. Partly because of the increase in thickness due to plaster acting monolithically with masonry and
2. Partly because of improvement in the mechanical properties due to grout injection and confinement effects

Apparently the increase in the lateral strength is not appreciable. It was because of high vertical stress and change in behavioural mode from shear to flexural which limited the increase in overall lateral strength. A high increase in strength is expected for shear-critical pier subjected to low vertical stress as is the case of typical single and double storey buildings. This phenomenon will be discussed in detail in Chapter 7.

Since the piers P5, P6 and P8 were tested till the formation of diagonal crack, their ultimate deformation is not known. Their ultimate deformation is determined as product of their yield strength and the ductility of the control piers. Once again the apparent increase in the deformation capacity, in comparison with control pier P3, is not because of the retrofitting; rather it is due to change in the behavioural mode. Since the ultimate drift ratio before retrofit is not known, it is calculated as the product of yield drift ratio times the ductility factor of control piers. There is no significant change in the deformation capacity. Test on
full scale room retrofitted with reinforced plaster have shown a decrease in the deformation capacity as discussed in Chapter 5 of this thesis report.

Difference in the behaviour, both before and after retrofitting, is due to the inherent variability in the properties of masonry. All the tests before
retrofitting were stopped after the formation of first diagonal crack which was assumed to be the peak strength based on the results of control piers P3 and P4. On average the stiffness is increased by 206% and peak strength by 21%.

A decrease in the equivalent viscous damping is once again observed in the retrofitted specimens, Figure 3-57. The damping of non-retrofitted piers was increasing with displacement and that of retrofitted specimens remains almost unchanged.
3.6.3.3 Piers Retrofitted with DS High Gauge Coarse Mesh with Grout Injection and Plaster

Damage piers P7 from P-B series and P14 from P-C series were retrofitted with high gauge coarse mesh, grout injection and plaster and retested as P7R and P14R. The hysteretic response of these piers before and after retrofitting is shown in Figure 3-58. The behaviour of P7R was shear before retrofitting and after retrofitting the pier behaved in combination of shear and rocking but the rocking was predominant failure mode. The behaviour of P14R, on the other hand, was more or less similar before and after retrofitting.

![Figure 3-58: Force-Deformation Response of Piers Retrofitted with Double Side High Gauge Coarse Mesh, Grout Injection and Plaster](image)

![Figure 3-59: Comparison of Envelope Curves: Piers Retrofitted with Double Side High Gauge Coarse Mesh, Grout Injection and Plaster](image)

The force-deformation behaviour before and after retrofitting is compared in Figure 3-59 and Figure 3-60. A 25% increase in lateral strength, 408% increase in elastic stiffness, and negligible change in the ultimate displacement of piers P7R was noticed after retrofitting. The
effective stiffness of pier before retrofit was very low due poor material properties. Grout injection and confinement due to reinforced plaster enhanced the mechanical properties and ultimately the lateral stiffness.

On the other hand the increase in the lateral strength and stiffness of P14R was negligible (4% and 30% respectively). However, some increase in the ultimate displacement (36%) of P14R was noted, Figure 3-60.

Several reasons may have caused this diversity in the behaviour of piers retrofitted with same technique including the type of masonry, amount of vertical stress and height of vertical load. It is point to be noted that none of the technique did affect appreciably the behaviour of piers in P-C series because their rocking behaviour before retrofitting.

The damping in specimen P7 (P-B Series) was found to be varying at very fast rate with increasing displacement due to its shear behaviour and that of P14 (P-C Series) remained almost constant, because its behaviour was more towards rocking, Figure 3-61. After retrofitting damping in both specimens (P7R and P14R) remained almost constant because of their rocking behaviour.
3.6.3.4 Piers Retrofitted with Single Side High Gauge Coarse Mesh with Grout Injection and Plaster

The force deformation hysteresis loops and envelope curves of piers P13 and P15 before and after retrofitting with single sided high gauge coarse mesh, grout injection and plaster are given in Figure 3-62 and Figure 3-63. Since both of piers belong to P-C series, they showed almost similar behaviour before and after retrofitting.

Figure 3-62: Force-Deformation Response of Specimens Retrofitted with SS Coarse Mesh, Grout Injection and Plaster

Figure 3-64 compares the force-deformation parameters of piers before and after retrofitting. The pre-damaged stiffness of piers is restored. About 87% of the pre-damaged strength is restored. The deformation capacity has, however, been increased by 67%. The behaviour of piers P13R and P15R retrofitted with single side mesh when compared with P14R retrofitted with double side mesh, it may be concluded that the double sided mesh is more effective than the single sided mesh. The reason may be the confinement effect which delayed the toe crushing, in addition to more reinforcement ratio in case of double side mesh.
The equivalent viscous damping of the piers before and after retrofitting is compared in Figure 3-65. Since both the piers behaved in rocking mode before and after retrofitting, therefore no signification variation was recorded in their damping characteristic.
with bed joint reinforcement, grout injection and plaster. Both of the piers belonged to P-B series and showed almost similar behaviour before and after retrofitting, Figure 3-67. The behaviour before and after retrofitting was predominantly shear and flexural respectively.

Force deformation parameters of P11 and P12 before and after retrofitting, Figure 3-68. The ultimate deformation capacity of piers before retrofitting was determined from the extrapolated force-deformation envelope curves. The strength, elastic stiffness and ultimate deformation were increased by 19%, 321% and 92% respectively for the retrofitted piers, Figure 3-68.

Figure 3-66: Force-Deformation Response of Specimens Retrofitted with Structural Re-pointing, Grout Injection and Plaster

Figure 3-67: Comparison of Envelope Curves: Piers Retrofitted Bed Joint Reinforcement, Grout Injection and Plaster
Similar to other piers of P-B series the damping in P11 and P12 was found to be increasing with displacement and remained almost constant in P11R and P12R, Figure 3-69. It was also noted that the damping in retrofitted specimens was less than that in non-retrofitted specimens which is due to the change in behavioural mode of the pier from shear to rocking.

Figure 3-68: Comparison: Piers Retrofitted with Structural Re-pointing, Grout Injection and Plaster

Figure 3-69: Comparison of Equivalent Viscous Damping: Piers Retrofitted Bed Joint Reinforcement, Grout Injection and Plaster
4. QUASI-STATIC TEST OF FULL SCALE URM AND CONFINED MASONRY WALLS

4.1 Introduction

Quasi static load test on isolated piers provides good estimate of the overall capacity of a building with strong spandrels where the damages are mostly concentrated in the piers. However, in a building with perforated walls, the damages may produce in spandrels (shear or flexural cracks) and/or at the corner of openings due to stress concentration. Thus the performance of isolated piers, in some cases, may not be accurately extrapolated to get the performance of the whole buildings. Since in-plane walls play main role towards the lateral load capacity of a building, it was, therefore, decided to experimentally study the effectiveness of the retrofitting technique on a full scale wall system with opening.

This chapter presents an experimental study on the performance of full scale unreinforced and confined masonry walls tested under quasi-static loading before and after retrofitting with reinforced plaster (ferrocement overlays) in the Structural Engineering Laboratory of the Department of Civil Engineering, University of Engineering and Technology, Peshawar, Figure 4-1. Starting with geometric and material properties, test setup and test procedure, a detailed description of the retrofitting technique is provided. It is then followed by a discussion on the damage mechanism and force-deformation behavior of each wall before and after retrofitting. In the last the results are compared to quantify the effect of retrofitting.
4.2 Experimental Program

Two full scale walls having same size and configuration and representative of a typical brick masonry building in Northern Pakistan, were constructed in structural engineering laboratory of department of Civil Engineering, University of Engineering and Technology, Peshawar, Pakistan. Bricks, mortar and concrete cubes, and masonry assemblages were tested in the laboratory to characterize their properties in compression.

The repair and retrofitting scheme consisted of replacement of loose joint mortar around the cracks, injecting grout in cracks, connecting steel welded wire mesh and applying plaster coating in cement sand mortar.

4.2.1 Material Properties

Material properties of bricks, mortar, and concrete elements were chosen to be representative of a typical brick masonry building in Northern Pakistan [QA-04]. The specimens were made in a 9" (229 mm) thick English bond pattern using solid burnt clay bricks laid in 1:8 cement-sand mortar. The nominal size of brick was 9"x4.5"x3" (230 x 115 x 75 mm). The reinforced concrete elements were cast with 1:2:4 cement-sand-coarse aggregates concrete. Mortar and concrete were prepared from ordinary Portland cement, locally available sand, crushed stones and portable water.

Table 4-1: Mechanical Properties of Masonry Materials

<table>
<thead>
<tr>
<th>Material Property</th>
<th>Number of Specimens</th>
<th>Strength, psi (MPa)</th>
<th>C.O.V. %</th>
</tr>
</thead>
<tbody>
<tr>
<td>Compressive Strength of Mortar, psi (MPa)</td>
<td>6</td>
<td>758 (5.23)</td>
<td>15.2</td>
</tr>
<tr>
<td>Compressive Strength of Bricks, psi (MPa)</td>
<td>5</td>
<td>3,170 (21.86)</td>
<td>13.8</td>
</tr>
<tr>
<td>Compressive Strength of Concrete, psi (MPa)</td>
<td>11</td>
<td>2947 (20.32)</td>
<td>15.1</td>
</tr>
<tr>
<td>Compressive Strength of Masonry, psi (MPa)</td>
<td>5</td>
<td>656.0 (4.52)</td>
<td>11.5</td>
</tr>
<tr>
<td>Modulus of Elasticity of Masonry, ksi (MPa)</td>
<td>5</td>
<td>175.0 (1207)</td>
<td>22.2</td>
</tr>
<tr>
<td>Yield Strength of Wire Mesh, ksi (MPa)</td>
<td>-</td>
<td>30.0 (207)</td>
<td>-</td>
</tr>
<tr>
<td>Compressive Strength of Plaster, psi (MPa)</td>
<td>23</td>
<td>3410 (23.5)</td>
<td>18.0</td>
</tr>
</tbody>
</table>
The mechanical properties of the masonry materials are given in Table 4-1. The mortar and concrete strength were based on 2” (51 mm) cube and 6” x 9” (152 x 229 mm) cylinder respectively. The compressive strength and modulus of elasticity of masonry is determined by testing masonry prisms (Figure 4-2) 16” x 18” x 9” (406 x 457 x 229 mm) according to ASTM E 447.

Cement-based grout material (Ultra-LSR) was used for filling cracks in masonry while epoxy-based grout (Ultra injection resin) was used to fill cracks in concrete elements. Steel welded wire mesh made of 0.04” (1.0 mm) wires spaced at 1/2” (12.7 mm) in both direction with a reinforcement ratio of \( \rho_s = 0.054\% \) of the gross area) was connected to the surface of wall using 2” (50 mm) long No.10 screws, steel washer and plastic plugs inserted in a pre-drilled holes in bricks. A 3/4” (19 mm) thick plaster coating was then applied on the surface of wall using 1:3 cement-sand mortar.

4.2.2 Test Specimens
Two full scale test specimens; unreinforced and confined brick masonry walls, were constructed from materials mentioned above. The walls were constructed by local mason in a manner similar to field practice. Bricks were soaked in water before use. Size and configuration of the walls were decided based on the local construction practice and limitation of the test facility. Each wall comprised three masonry piers with one opening each for a door and a window, Figure 4-3. Both the walls were constructed on a 6” (152 mm) thick and 21” (533 mm) wide reinforced concrete footing. An 18” x 15” (457 x 381 mm) concrete beam was cast on each wall to distribute the vertical load and to connect the horizontal load jack with the wall.
In case of confined wall, confining elements were cast monolithic with foundation and top concrete beam. 9" x 6" (229 x 152 mm) isolated and continuous horizontal RC members were provided as lintel in the case of unreinforced and confined masonry walls respectively. Openings in case of confined masonry wall were confined with vertical reinforced concrete elements, 4.5" x 9" (114 x 229 mm). The reinforcement provided in the confining elements and lintel beams are shown in Figure 4-4. Both the specimens were cured for at least 7 days through sprinkling of water 4 times a day. The walls were tested about two months after the completion of their construction.

4.2.3 Repair and Retrofitting of Damaged Walls

The damaged walls were retrofitted and retested under the same experimental environment. The retrofitting scheme included grout injection and application reinforced plaster (ferrocement overlay).

Epoxy based material (Ultra Injection Resin) was used to fill relatively thinner cracks produced in concrete elements of confined masonry wall. Injection ports/nozzles were first installed and the cracks were sealed from surface with an epoxy based sealant (Ultra Fairing Coat). Before injecting grout air was passed through nozzles to remove any dust inside the cracks. Grout was then injected at a high pressure exceeding 6 bars.
On the other hand ready to mix cement based grout material (Ultra-LSR) was used to fill relatively wider cracks produced in masonry of both unreinforced and confined wall. Injection ports/nozzles were first installed and the cracks were sealed from surface with a fast bonding mortar (Ultra-Grout with Ultra-CBR), Figure 4-5. Water was passed through the nozzles from top to bottom to check the connectivity between nozzles and to wet the masonry before injecting grout. The water was found coming out of surrounding areas adjacent to the cracks. It was then decided to seal the whole wall surface with plaster before injection. When the plaster got sufficient strength, the wall was moistened by passing water through nozzles and afterward grout material was injected at pressure ranging from 2 to 4 bars. The pressure was kept applied for about 2 minutes to consolidate the grout inside the cracks and to allow excessive water to spell out.

After repairing cracks in both masonry and concrete element, steel welded wire mesh was connected to the surface of both unreinforced and confined walls, wrapping it around walls and piers, Figure 4-6. It must be noted that the connecting screws were fixed in holes drilled within bricks, not in mortar joints. The distance between screws was 18” approximately. The minimum lap length at the discontinuous end of the wire mesh was kept 9” (229 mm). After fixing the wire mesh, wall surfaces were plastered with 1:3 cement-sand mortar to total thickness of 3/4” (19 mm).
4.2.4 Experimental Test Setup

The experimental test setup is shown in Figure 4-7. The walls were fixed at the bottom and free to rotate and translate from the top. The horizontal and vertical loads, applied through hydraulic jacks were measured with load cells having capacities of 112 kips (500 KN) and 56 kips (250 KN) respectively. These hydraulic jacks were connected to a hydraulic pump. The horizontal jack and load cell were connected to the top concrete beam through swivel in order to release the rotation and vertical translation of the wall. The height of horizontal load measured from the wall bottom was 10'-7.9" (3248 mm).

Sixteen displacement transducers were installed on wall to capture the displacement field at all important location as shown in Figure 4-7. Transducer 01, which was recording the in-plane displacement of the wall at the horizontal load level, was considered as control gauge. All the displacement transducers and load cells were connected to a data acquisition system UCAM-70 shown in Figure 4-7.

4.2.5 Test Procedures

Both the walls were subjected to increasing intensities of in-plane quasi-static reverse cyclic loads before and after retrofitting. A pre-compressive force of 22.5 kips (100 KN) including the weight of concrete and steel beams, was applied on both walls through vertical jack. Horizontal displacement (gauge-01) at the opposite end of the top concrete beam was used as the control displacement for the whole test. Each displacement cycle consisted of loading to a specified displacement level, unloading to zero load, reloading in negative direction to the same specified displacement and again unloading to zero displacement. Each displacement
cycle was repeated three times starting from 0.02″ (0.5 mm), Figure 4-8. An attempt was made to adjust the vertical load which was increasing with increasing displacement but the results were not encouraging because the load could not be released steadily. Therefore the vertical load was kept varying with increasing horizontal displacement. Cracks produced at the end of each cycle were marked on the walls. The tests were stopped after exhausting the maximum lateral resistance of the walls before retrofitting. After retrofitting the walls were tested up to ultimate state of the wall.

![Figure 4-8: Typical Displacement Pattern for Quasi-static Testing of Walls](image)

### 4.3 Test Results and Discussions

The data obtained from the cyclic test was first filtered through three point moving average method. The filtered data was used to plot the force-deformation hysteresis loops and envelope curves. Seismic resistance parameters such as stiffness, peak load, ultimate displacement, equivalent viscous damping, etc were determined from these plots. The envelope curves were produced by joining the points of peak loads in each displacement cycle. The equivalent viscous damping was calculated from the relation:

$$\xi_{eq} = \frac{E_d}{2\pi E_{inp}}$$  \hspace{1cm} 4-1

Where $E_d$ is the dissipated energy equal to the average area of three displacement cycles at the same displacement level, and $E_{inp}$ is the input energy calculated as sum of half product of peak load and the corresponding displacements in positive and negative loading.

#### 4.3.1 Unreinforced Brick Masonry Wall before Retrofitting

The damage pattern of unreinforced masonry wall tested before retrofitting is shown in Figure 4-9. The test was stopped after the last cycle with maximum drift of 0.28% during which the cracks, produced in spandrel above south pier, widened about 0.5″ (12.5 mm) and endangered the wall. The cracks in rest of the walls might be categorized as minor. No cracks were found in the end piers because they behaved in a pure rocking mode producing cracks at top and bottom. On the other hand the middle pier showed a mixed flexural and shears behaviour. Shear cracks originating from the window corners were also noticed. No cracks were observed in spandrel above openings.
The force-deformation hysteretic response of unreinforced masonry wall tested before retrofitting is shown in Figure 4-10. The loops are very tight indicating very small energy dissipation. The behaviour of wall in negative and positive load direction was identical in the initial elastic range up to a drift of 0.06%. After the formation of cracks the slope of force-deformation envelope curve decrease more rapidly in positive load direction than the negative load direction due to more damages produced in the former case. The test was stopped just after peak load at an average drift of 0.28%. Analysis of the displacements recorded at the pier level indicated that the deformations are mainly concentrated at the pier levels and the spandrel is just moving as a rigid body. However some residual displacements were seen in south pier at the end of each cycle which was due to the cracks produced above the south pier. All vertical gauges mounted on the wall recorded appreciable displacement which is an indication towards the rocking of walls and individual piers.

Variation of displacements recorded at the top of three piers with increasing wall displacement is given in Figure 4-12. The solid thick lines, drawn at 45°, act as reference. The displacement for north and middle piers are almost equal to the wall displacement indicating that the displacement was producing in piers and the spandrel was moving as a rigid body. The displacement at the top of south piers was initially equal to the wall displacement but after the appearance of cracks above south pier, the pier
was shifting southward, relative to the spandrel, with increasing wall displacement. The gauge-02 measuring displacement at top of south pier could not record the data beyond 0.45” (11.5 mm) displacement.

![Figure 4-11: URM before Retrofitting: Pier Displacement Vs Wall Displacement]

The vertical jack load was found to be varying with increasing lateral displacement. The total vertical load on wall consisted of jack load and the dead load of steel and concrete beams. The dead load was found to 7.5 kips (33.4 KN). Initially the jack load was set on 14.5 kips (64.6 KN) thus applying a total load of 22.0 kips (98.0 KN). The variation of total vertical load with increasing displacement is shown in Figure 4-12. The maximum variation in vertical load was 6.0 kips (26.7 KN) and 8.0 kips (36.6 KN) along the positive and negative loading direction respectively. This corresponds to 28% and 36% variation in positive and negative direction respectively. However the variation was less significant in the initial cycles. In the very last cycle the variation in vertical load was almost zero in positive direction. The vertical load could not be kept constant due to

![Figure 4-12: Variation of Vertical Load in Unreinforced Masonry wall Tested before Retrofitting]
manual control of the hydraulic system. The results may not be a true representation of the actual behaviour of the wall under constant vertical load. But since the retrofitted wall was also tested under the same loading environment, therefore the use of these results for comparison with retrofitted wall is justified.

### 4.3.2 Unreinforced Brick Masonry Wall after Retrofitting

Figure 4-13 shows the damage pattern of unreinforced wall tested after retrofitting.

All the three piers behaved in a pure rocking mode. The damages were concentrated at the top and bottom of the piers especially near the pier-spandrel connections. The location of cracks was different from those produced in the wall before retrofitting proving that the grout injection worked well in repairing the cracks. The cracks were very minor and distributed during the initial displacement cycles. The failure first started with the breaking sound of steel mesh and then appeared as cracks in the plaster. The first cracks appeared in the plaster at drift 0.15%. A major shear crack originating from the corner of the window, produced due stress concentration, was observed at a drift of 0.31%. Spalling of plaster from the cracked region was also observed.

The force deformation hysteretic response of the retrofitted unreinforced masonry wall is shown in Figure 4-14. The behaviour was almost similar in
positive and negative load direction up to a drift of 0.20%. Afterward the stiffness of wall in positive load direction decreased at faster rate than the stiffness in the negative load direction. However the strength degradation in negative direction started at drift of 0.34% while the load in positive load direction was still increasing. Both the curves coincided at a drift of 0.55%.

Figure 4-15: URM after Retrofitting: Pier Displacement Vs Wall Displacement

Figure 4-15 provides variation in pier displacements with variation in wall displacement. Similar to the URM wall tested before retrofitting, the pier displacements are almost equal to the wall displacement indicating that the spandrel was moving as a rigid body. However, in the case of negative load direction in middle pier and both positive and negative load direction in south pier, the pier displacement was found slightly lagging behind the wall displacement which is because of the global rocking of wall.

A more insight in the force deformation behaviour can be obtained by first discussing the variation in vertical load with increasing lateral displacement. Likewise the non-retrofitted unreinforced masonry wall, the vertical load in the retrofitted unreinforced masonry wall was found increasing with increasing lateral displacement, Figure 4-16.

Until a drift of 0.20% the increase in total vertical load was less than 20% in both positive and negative load direction. Beyond 0.20% the vertical load increased more rapidly in negative direction than in the positive direction which might be the reason for increase in the lateral load occurred at faster rate in negative direction, Figure 4-14. The variation in the vertical load was a function of the lateral displacement and the damages produced. More the damages produced, lesser was the increase in the vertical load. At peak resistance in the negative load direction the wall experienced severe damages in negative direction which resulted in a slow increase in the vertical load in negative load direction. However the increase in vertical
load which continued in positive load direction caused increased in the lateral resistance.

![Graph showing variation of vertical load in retrofitted unreinforced masonry wall](image)

**Figure 4-16: Variation of Vertical Load in Retrofitted Unreinforced Masonry wall**

### 4.3.3 Behaviour of Confined Masonry Wall before Retrofitting

The final damage pattern of confined masonry wall tested before retrofitting is shown in Figure 4-17. Middle and south piers showed a mixed rocking and shear behaviour. Vertical splitting at the joints between vertical concrete elements and the masonry was also prominent which was believed to be produced due improper concreting at the interface of both materials. The cracks first appeared at a drift of 0.09% in the lintel band near the door and window openings and in the first bed joint above lintel band. Cracks in the confining elements were noticed at a drift of 0.28%. South and middle piers showed more damages as compared to the north pier which was very slender. Some minor cracks were observed in the spandrel. At a drift of 0.43% the horizontal crack above lintel crossed the whole length of the wall. The test was stopped at a maximum drift of 0.49% at which the damages were of moderate nature.

![Damage pattern of confined masonry wall before retrofitting](image)

**Figure 4-17: Damage Pattern Confined Masonry Wall before Retrofitting**
The force-deformation response of the confined masonry wall before retrofitting is shown in Figure 4-18. The response is non-symmetric giving more energy dissipation in negative direction than that in positive direction because of more damages produced in positive direction. The response was identical in both directions till 0.04% drift. Beyond 0.04% drift there was an appreciable change in positive stiffness of wall which then continued till the end of the test. The positive stiffness, however, decreased gradually to almost zero at 0.1% drift and continued towards the end of test. More elaboration on the force-deformation behaviour will be made in the light of variation in the vertical load with increasing lateral displacement in positive and negative load directions.

The variation of total vertical load with lateral displacement is shown in Figure 4-19 which is symmetric in positive and negative directions. The non-symmetric behaviour of the wall is, thus, not because of the vertical load rather it was because of the damage pattern which initiated first in positive load direction. At a drift of 0.1%, 0.2% and 0.4% the increase in vertical load was about 10%, 26% and 43% of the vertical load at zero drift respectively.

The displacements at north and south pier in positive direction was slightly lagging behind the wall displacement which is because of global rocking and shear sliding of spandrel, Figure 4-20. The south pier displacement was, however, more than the wall displacement in negative load direction indicating some permanent deformation in south piers.
4.3.4 Behaviour of Confined Masonry Wall after Retrofitting

The retrofitted confined masonry wall was tested under reverse cyclic test up to drift of 0.31%. In the subsequent cycle the anchor bolts, connecting top concrete beam to the horizontal loading system, pulled out of concrete beam. Since it was not possible to continue the test under full cyclic loading, therefore it was decided to continue the test under half cyclic loading (push only). Very few cracks appeared in the wall till 0.31% drift. Most of the damages occurred in the half cyclic test. The final damage condition of the retrofitted confined masonry wall is shown Figure 4-21. Cracks were distributed throughout the whole surface of wall. The piers showed rocking failure mode. The north slender pier, however, also developed some vertical splitting cracks at the masonry-confining element connections which were believed to be produced due to high compressive stress in the half cyclic loading. Significant cracks in confining elements appeared in the wall after a maximum drift of 0.55%. Patches of plaster were also observed falling at high drift ratios.

The force-deformation response of confined masonry wall after retrofitting is shown in Figure 4-22. Significant decrease in the positive stiffness of wall was noticed at very small drift of about 0.02% and then continued unchanged till the end of test. The positive stiffness, however, changed gradually up to a drift of 0.05% and then continued almost unchanged till the end of the test. As already mentioned that the reverse cyclic test could not be continued after a drift of 0.3% due to the premature failure of the top beam anchor system, half cyclic test in positive direction was performed after 0.3% drift. The positive lateral load was increasing almost linearly with increase in lateral displacement.
The variation in total vertical load with increasing lateral displacement for confined masonry wall after retrofitting is shown in Figure 4-23. In the reverse cyclic test the increase in vertical load was more in negative
direction than the positive direction. Vertical load was increased by 5%, 22% and 41% in positive direction and 10%, 36% and 73% in negative direction at drift of 0.1%, 0.2% and 0.3% respectively. In the positive half cyclic the increase in vertical load was more than 100% at drift of 0.55%.

Variation in pier displacements with variation in wall displacement is given in Figure 4-24. The pier displacements are lagging behind the wall displacement both in positive and negative load direction, which is because of the global rocking of the wall.

![Figure 4-24: CM after Retrofitting: Pier Displacement Vs Wall Displacement](image)

### 4.3.5 Correction for Variation in Vertical Stress

The capacity of a wall increases with increase in the vertical stress. To account for variation in the vertical stress for URM walls before and after retrofitting, strength correction factors are developed based on the empirical relations. Since the behaviour of URM wall before and after retrofitting may be categorized as flexural rocking, therefore correction factors are developed based on relations available for rocking failure mode. The correction factor is basically the ratio of empirical strength at the experimental vertical stress and the required vertical stress.

The rocking capacity at toe crushing of unreinforced masonry pier before and after retrofitting with steel welded wire mesh can be estimated from (Chapter 7):

\[
V_{c,before} = \frac{pL_p^2t_p}{2\psi H_p} \left(1 - \frac{p}{0.85f_m}\right) \quad 4-2
\]

\[
V_{c,after} = \left(\frac{p + f_y\rho_s}{0.85f_m + 0.85f_c\frac{t_c}{t_p} + f_y\rho_s}\right) \left(1 - \frac{p + f_y\rho_s}{0.85f_m + 0.85f_c\frac{t_c}{t_p} + f_y\rho_s}\right) \quad 4-3
\]
Where $L_p$, $H_p$ and $t_p$ are the length, height and thickness of pier, $p$ is the vertical stress on pier, $f_m$ and $f_c$ are the compressive strength of masonry and plaster coating respectively, $f_y$ and $\rho_s$ are the yield strength and steel ratio of steel wire mesh respectively and $\psi$ is boundary condition factor equal to 0.5 in the present case.

The estimated capacity of wall, at the experimental and desired vertical stress, is determined as sum of the capacities of the three piers calculated from the above equations. The ratio of the two capacities for each displacement cycle is used as correction factor. The experimental and corrected force-deformation envelope curves for URM walls before and after retrofitting are shown in Figure 4-25.

![Experimental and Corrected Force-deformation Envelope Curves: URM before (top) and after (bottom) Retrofitting](image)

The behaviour of confined masonry is comparatively complex and there is no such relation available so far in the literature to estimate the capacity of confined masonry walls with different vertical loads. A detailed finite element analysis is the best alternative.

4.3.6 Comparison of Unreinforced Masonry Wall before and after Retrofitting

The average corrected force-deformation curves of unreinforced masonry wall before and after retrofitting is shown in Figure 4-26. There is a significant increase in the lateral stiffness (118%) and strength (74%) of wall tested after retrofitting. However, no appreciable change in the deformation capacity is noticed.
Table 4-2 provides a comparison of force-deformation parameters of unreinforced masonry wall before and after retrofitting.

Table 4-2: URM Wall Comparison: Force-deformation Parameters

<table>
<thead>
<tr>
<th>Description</th>
<th>Effective Stiffness kips/in (kN/mm)</th>
<th>First Significant Crack kips (KN)</th>
<th>Drift (%)</th>
<th>Peak Strength</th>
<th>kips (KN)</th>
<th>Drift (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Before Retrofitting, URM</td>
<td>86.4 (15.14)</td>
<td>6.00 (26.7)</td>
<td>0.040</td>
<td>8.9 (39.6)</td>
<td>0.28</td>
<td></td>
</tr>
<tr>
<td>After Retrofitting, URM</td>
<td>189.0 (33.1)</td>
<td>12.00 (53.4)</td>
<td>0.044</td>
<td>15.4 (68.5)</td>
<td>0.24</td>
<td></td>
</tr>
<tr>
<td>Percent Change (%)</td>
<td>+118</td>
<td>+100</td>
<td>+10</td>
<td>+74 (-14)</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

The equivalent viscous damping for unreinforced masonry wall before retrofitting started at value of about 7.7%. After a slight decreasing trend up to 0.15% drift ratio, the damping remained almost constant at 5.0%. On the other hand the damping of URM wall after retrofitting started at 4.0% and remained almost constant till 0.18% drift. The damping was found increasing from 4.0% to 12% at a drift of 0.36%. From the comparison of the damping before and after retrofitting one can conclude that the energy dissipating capacity of unreinforced masonry wall is not appreciably affected after retrofitting with steel wire mesh.
4.3.7  **Comparison of Confined Masonry Wall before and after Retrofitting**

The experimental force-deformation curves of confined masonry wall before and after retrofitting are compared in Figure 4-28. The initial stiffness of retrofitted wall is restored back to its pre-damaged condition. However, the lateral strength of wall is found to be increased by 26%. The increase in strength of unreinforced and confined wall is 6.5 kips (28.93 KN) and 5.4 kips (24 KN) respectively. Thus increase in strength is comparable, but the high strength of pre-damaged confined wall makes steel wire mesh less effective for the retrofitting of confined masonry buildings. The deformation capacity could not be obtained because of premature failure of top concrete beam.

![Figure 4-28: CM Wall Comparison: Force-deformation Envelope Curves](image)

Table 4-3 provides a comparison of force-deformation parameters of confined masonry wall before and after retrofitting.

Equivalent viscous damping of confined wall before and after retrofitting remained almost constant, Figure 4-29. Starting at 4.5% damping in both case, the damping before and after retrofitting remained constant at 5.6% and 3.9%. The retrofitting scheme has, thus, no appreciable effect on the damping characteristic of confined masonry wall.

<table>
<thead>
<tr>
<th>Description</th>
<th>Effective Stiffness kips/in (KN/mm)</th>
<th>First Significant Crack kips (KN)</th>
<th>Drift (%)</th>
<th>Peak Strength kips (KN)</th>
<th>Drift (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Before Retrofitting, CM</td>
<td>187.0 (32.8)</td>
<td>11.7 (52.1)</td>
<td>0.040</td>
<td>20.9 (93.0)</td>
<td>0.49</td>
</tr>
<tr>
<td>After Retrofitting, CM</td>
<td>191.0 (33.5)</td>
<td>12.0 (53.4)</td>
<td>0.04</td>
<td>26.4 (117.5)</td>
<td>0.43</td>
</tr>
<tr>
<td>Percent Increase (%)</td>
<td>+2</td>
<td>+3</td>
<td>0</td>
<td>+26</td>
<td>-12</td>
</tr>
</tbody>
</table>
4.3.8 Comparison of Confined and Unreinforced Masonry Walls

A significant difference in the strength, stiffness and deformation capacity of both the walls is noticed which is because of the presence of reinforced concrete elements in confined masonry wall. The strength, effective stiffness and deformation capacities of confined masonry wall are 134%, 120% and 75% respectively more than those of unreinforced wall. Thus it can be concluded that the performance of confined masonry walls is almost double than that of unreinforced wall. The strength and stiffness of retrofitted unreinforced wall is comparable with those of non-retrofitted confined masonry wall.
5. QUASI-STATIC TEST OF URM BUILDING AFTER RETROFITTING

5.1 Introduction

Chapter No.3 presented a study on the effect of retrofitting on the lateral load behavior of isolated piers which was extended to full scale walls in fourth chapter in order to include the effects of coupling due to the presence of spandrels, stress concentration at opening corners, variation in vertical stress on piers with lateral load, etc. This chapter extends the study to an unreinforced brick masonry system of a single storey single room full scale building, consists of in-plane and out-of-plane walls, openings, connections, diaphragm etc. The model, originally constructed and tested as part of PhD study [KS-10] at the department of Civil Engineering, has been retrofitted and tested as part of this study, Figure 5-1. The main objective of the test was to study and quantify the behavior of a full scale room, retrofitted with reinforced plaster (ferrocement overlays) and grout injection.

Figure 5-1: Full Scale Retrofitted Room
5.2 Experimental Description

The test structure was a single room unreinforced masonry building designed based on common construction practice in the northern areas of Pakistan, Figure 5-2. The 9” (229 mm) thick walls were made with clay bricks arranged in English bond pattern using 1:8 cement sand mortar. The structure was tested before retrofitting under reverse cyclic loading to investigate its seismic performance. The details may be found elsewhere [KS-10]. The damaged test structure was retrofitted and retested under the same load and boundary conditions that were used for testing before retrofitting.

![Figure 5-2: Test Structure After Retrofitting](image)

### 5.2.1 Retrofitting of the Test Structure

The retrofitting scheme included replacement of loose joint mortar, injection of cement based grout in cracked regions, connecting steel welded wire mesh and applying cement-sand plaster coating. The properties of materials used for retrofitting of the test structure are given in Table 5-1.

![Figure 5-3: Detailing of Steel Welded Wire Mesh](image)

Same mortars (1:4 cement-sand) were used both for plaster coating and replacement of loose joint mortar. The thickness of plaster coating was
ranging from 5/8" (16 mm) to 3/4" (19 mm). Compressive strength of mortar was based on 2" (50 mm) cube tested according to ASTM C 109.

Steel welded wire mesh (SWWM) was made with 0.04" (1.0 mm) diameter wires spaced at 1/2" (12.7 mm) in both directions. The steel ratio ($\rho_s$) for mesh applied on both sides of 9" thick wall is equal to 0.054%. Tensile strength of SWWM is determined by testing wires extracted from the mesh. SWWM was connected with surface of wall through 1.5" (38 mm) long No.10 screws, 1.0" (25 mm) diameter steel washer, and plastic plug. Number of screws per square foot of wall area was around two (22, screws/m²). Screws were fixed in 1/2" (12.5 mm) holes drilled in bricks. A minimum of 6" (150 mm) lap length both in vertical and horizontal direction were provided. The number of screws was doubled along the overlap. SWWM was applied on both sides of piers, pier-spandrel connection and walls intersections as detailed in Figure 5-2 and Figure 5-3.

<table>
<thead>
<tr>
<th>S. No</th>
<th>Material Description</th>
<th>Mean Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Compressive strength of 1:4 cement-sand mortar used for joint repair and plaster coating, psi (MPa)</td>
<td>1594 (11.0)</td>
</tr>
<tr>
<td>2</td>
<td>Tensile strength of steel welded wire mesh, ksi (MPa)</td>
<td>27.0 (186.2)</td>
</tr>
<tr>
<td>3</td>
<td>Diagonal tensile strength of prism made with bricks and cement based injection grout, psi (MPa)</td>
<td>200 (1.38)</td>
</tr>
<tr>
<td>4</td>
<td>Flow value for injection grout, inch (mm)</td>
<td>8.0 (203)</td>
</tr>
</tbody>
</table>

Cracks and internal voids in damage structure were filled with cement based injection grout, made with 10 parts of Portland cement, one part of lime, Ultra expansion agent mixed at a rate of 250 g per 50 kg of cement, and water with a water-cement ratio of 0.9. The mix was designed based on its effectiveness, injectability and stability. The water cement ratio for grout mix was decided based on the flow test from Department of Buildings and Safety, Loss Angles (P/BC 2008-056). The grout was injected through 3/8" (10 mm) diameter and 4" (102 mm) long injection ports spaced at distance of 9" (229 mm) to 12" (305 mm) along the cracks. The injection ports were fixed in 1/2" (12.5 mm) diameter and 4.5" (114 mm) deep predrilled holes with a fast binding mortar. The pressure during injection was kept constant at 45 psi (0.31 MPa) for 2 to 3 minutes after the wall has absorbed the grout. Further delay in maintaining the pressure was avoided because it resulted in chocking of the pipe. Any grout coming out of the adjacent ports was blocked with stoppers. The bond strength of grout with bricks was determined by diagonal compression test of 18" (457 mm) long, 18" (457 mm) high and 4.5" (114 mm) thick prisms, made by arranging bricks in a water tight mould and filling gapes between them with injection grout. The prisms were cured in an air-tight condition for 28 days and then tested under diagonal compression.
The procedure used for the execution of retrofitting scheme was to first remove the loose mortar from the cracked region and replace with new 1:4 cement-sand mortar, Figure 5-4 (left). Steel welded wire mesh was then connected with the surface of wall. After that Injection ports were installed along the cracks with fast binding mortar, Figure 5-4 (right). Whole masonry was then covered with plaster coating both from inside and outside, Figure 5-5 (left). After a 14 days moist curing of plaster coating, water was passed through ports from top to bottom to moisten the masonry and to check the connectivity of ports, Figure 5-5 (middle). Grout was, then, injected through the injection ports from bottom to top, Figure 5-5 (right).

5.2.2 Test Setup and Instrumentation

Test setup and instrumentation of the structure are detailed in Figure 5-6 and Figure 5-7. The loading system consisted of a hydraulic jack, two swivels at each end of the jack and a load measuring cell. It was connected to the top slab through two loading shoes one at each end of the room that were connected to each other through 4 bolts. Since the capacity of the loading system was about 45 kips (200 KN) and the capacity of room before retrofitting was less than 45 kips (200 KN), therefore one loading system was used to apply the lateral load. However the estimated capacity of retrofitted room was exceeding the capacity of loading system, therefore the lateral load was applied through two parallel loading systems separated by 24" (60 cm) on centre, Figure 5-6. The loads were measured with load cells having a capacity of 112 kips (500 KN).

The reinforced footing below the walls was firmly secured with the strong floor of the testing laboratory. Twelve displacement transducers were used
to record displacements at various locations, Figure 5-7. Gauge-01 was used as control channel. Gauges 02 through 05 were installed to record the torsional rotation of the room. The displacements at the top of each pier were recorded with gauges 06 through 10. Gauges 11 and 12 were used to measure the possible vertical displacement produced due to global rocking of the room. All the displacement gauges were installed on steel reference frames and connected to the walls through flexible wires. The load cells and displacement transducers were connected with a static data acquisition system UCAM-70A having a maximum data scanning speed of 20 channels per second.

Figure 5-6: Test Setup and Instrumentation of Test Structure

Figure 5-7: Instrumentation of Test Structure in Isometric Views

Figure 5-8: Reverse Cyclic Displacement Pattern
5.2.3 Test Procedure
The structure was tested under reverse cyclic loading in a displacement-controlled environment controlling the displacement recorded by gauge 01. The loads applied by the two parallel jacks were kept equal to avoid torsional moment. The controlled displacement pattern is given in Figure 5-8 where each displacement cycle was repeated three times. During and after each set of three equal displacement cycles, the test structure was thoroughly examined for cracks produced. The test was stopped after a strength degradation of 30%.

5.3 Results and Discussions

5.3.1 Damage Mechanism

![Figure 5-9: Final Damage Pattern of Test Structure after Retrofitting](image1)

![Figure 5-10: Damage Pattern of full scale room tested after retrofitting](image2)

The final damage pattern of the structure is shown in Figure 5-9 and Figure 5-10. Global and local rocking were the predominant behaviour modes of the structure. The global rocking appeared in the form of flexural cracks at the base of west solid wall and at the window sill level in east wall. No
shear cracks have been noticed in any of the five piers. The local rocking of piers produced diagonal cracks originating from opening corners and moving diagonally towards the end of walls. The middle pier of south wall, however, rocked about top and bottom horizontal cracks. Since the spandrels were not reinforced, some shear cracks of minor nature were observed in the south spandrel. As a whole the damages may be categorized as moderate.

When compared with the damage pattern of test structure tested before retrofitting (Figure 5-11), one can see a clear transformation from mixed compression-shear-flexural behaviour to a more stable rocking behaviour.

![Figure 5-11: Final Damage Pattern of Test Structure before Retrofitting](image)

### 5.3.2 Force-deformation Behaviour

The force-deformation hysteresis loops and envelope curves, drawn between total horizontal load (gauge 00) and percent storey drift recorded by control gauge 01, are shown in Figure 5-12. The envelope curves are drawn by joining the points corresponding to maximum load in each displacement cycle. The hysteresis loops are nonlinear elastic which is the characteristic of rocking mode. The behaviour both in negative and positive load direction remained nonlinear from the beginning of test with gradual stiffness degradation. The average peak strength of 49.5 kips was achieved at a storey drift of 0.12% after which strength degradation started. The strength degradation continued at a constant rate till a storey drift of 0.46% at which the lateral load was 36.0 kips. This residual strength remained almost constant till a storey drift of 0.57%.

![Figure 5-12: Force-deformation Response of Test Structure after Retrofitting: Hysteresis Loops (left) and Envelope Curves (right)](image)
5.3.3 Comparison of Force-deformation Parameters

The average force-deformation curves of the structure tested before and after retrofitting and their bilinear approximations, drawn on the basis of equal energy principle, are compared in Figure 5-13 (left). The ultimate drift on bilinear curve is defined as a drift at which the peak strength is degraded by 20%. Hysteretic damping (Figure 5-13, right) is calculated from the hysteresis loops as equivalent viscous damping, $\xi_{eq}$ in accordance with Eq-1.

$$\xi_{eq} = \frac{E_d}{2\pi E_{inp}}$$  \hspace{1cm} (5-1)

Where, $E_d$ is the dissipated energy equal to the area of a single hysteresis loop and $E_{inp}$ is the input energy equal to sum of the half product of peak load and the corresponding displacement in positive and negative load direction.

![Figure 5-13: Comparison of Force-deformation Behaviour before and after Retrofitting: Average Envelope and Corresponding Bilinear Curves (left), Hysteretic Damping (right)](image)

Table 5-2: Comparison of Force-deformation Parameters

<table>
<thead>
<tr>
<th>S. No</th>
<th>Force-deformation Parameters</th>
<th>Before Retrofit</th>
<th>After Retrofit</th>
<th>Ratio (After/Before)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Lateral stiffness, kips/inch (KN/mm)</td>
<td>421.3 (73.8)</td>
<td>963.8 (168.9)</td>
<td>2.29</td>
</tr>
<tr>
<td>2</td>
<td>Lateral peak strength from measured curve, kips (KN)</td>
<td>23.9 (106.4)</td>
<td>49.5 (220.3)</td>
<td>2.07</td>
</tr>
<tr>
<td>3</td>
<td>Lateral yield Strength from bilinear curve, kips (KN)</td>
<td>22.2 (98.8)</td>
<td>44.5 (198.0)</td>
<td>2.00</td>
</tr>
<tr>
<td>4</td>
<td>Ultimate storey drift (%)</td>
<td>0.50</td>
<td>0.37</td>
<td>0.74</td>
</tr>
<tr>
<td>5</td>
<td>Displacement ductility</td>
<td>12.5</td>
<td>10.6</td>
<td>0.85</td>
</tr>
</tbody>
</table>

The force-deformation parameters of the structure tested before and after retrofitting are compared in Table 5-2. Significant increase both in lateral stiffness (129%) and peak strength (107%) of the retrofitted structure is noticed. Such a high increase in lateral strength and stiffness is due to:

- The direct increase because of additional thickness of plaster coating.
- Improvement in the material properties of masonry with grout injection and confinement effect from plaster coating and wire mesh.
• The direct increase in shear strength because of steel welded wire mesh.

On the other hand the ultimate deformation capacity is decreased by 26%. However, the decrease in displacement ductility is negligible. Similarly, no significant change is noticed in the hysteretic damping. The slight decrease in damping is due to the change in behaviour mode.

### 5.3.4 Building Performance Levels

Three performance levels; immediate occupancy (IO), life safety (LS) and collapse prevention (CP), of the structure tested after retrofitting are shown in Figure 5-14. Points corresponding to yield and ultimate drifts on force-deformation curve, are considered as IO and CP performance levels respectively. LS is taken as a point where the lateral drift is 75% of the ultimate drift. The performance levels are selected based on criteria given in ASCE standard (ASCE/SEI 41-06).

![Figure 5-14: Building Performance Levels after Retrofitting](image)

The performance levels of the structure tested before and after retrofitting are compared in Table 5-3 in terms of storey drift and the corresponding lateral load. The small decrease in lateral drift of the retrofitted structure is certainly offset by a large increase in the lateral load and thus the overall performance of the retrofitted structure is significantly improved.

<table>
<thead>
<tr>
<th>Performance Level</th>
<th>Drift/Load</th>
<th>Before Retrofit</th>
<th>After Retrofit</th>
<th>Ratio (After/Before)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Immediate Occupancy, IO</td>
<td>Drift, %</td>
<td>0.04</td>
<td>0.035</td>
<td>0.88</td>
</tr>
<tr>
<td>Load, kips (KN)</td>
<td>16.5 (73.4)</td>
<td>39.6 (176.2)</td>
<td>2.40</td>
<td></td>
</tr>
<tr>
<td>Life Safety, LS</td>
<td>Drift, %</td>
<td>0.38</td>
<td>0.28</td>
<td>0.74</td>
</tr>
<tr>
<td>Load, kips (KN)</td>
<td>23.3 (103.7)</td>
<td>43.9 (195.4)</td>
<td>1.88</td>
<td></td>
</tr>
<tr>
<td>Collapse Prevention, CP</td>
<td>Drift, %</td>
<td>0.50</td>
<td>0.37</td>
<td>0.74</td>
</tr>
<tr>
<td>Load, kips (KN)</td>
<td>19.5 (86.8)</td>
<td>40.3 (179.3)</td>
<td>2.07</td>
<td></td>
</tr>
</tbody>
</table>
The spectral acceleration capacity, which is a measure of the overall performance of a system, corresponding to any drift ratio can be obtained by converting the real non-linear system to an equivalent elastic single degree of freedom system. For a single degree of freedom system the relation may be written as:

\[ S_a = \frac{k}{m} \Delta_y \sqrt{\frac{2\Delta}{\Delta_y}} - 1 \]  

Where, \( k \) is the effective stiffness of the bilinear curve and \( m \) is the mass of the system above mid height of the room which is calculated to be 0.238 kips-sec\(^2\)/inch for both retrofitted and non-retrofitted room, \( \Delta \) is the displacement at any point on capacity curve and \( \Delta_y \) is the yield displacement on bilinear curve.

![Figure 5-15: Spectral Acceleration Capacities: Before and After Retrofitting](image)

The spectral acceleration capacity corresponding to each performance level before and after retrofitting, as calculated from Eq.5-1 are given in Figure 5-15 and Figure 5-15. The capacity of retrofitted building in terms of spectral acceleration is more than double than that of non-retrofitted building for all performance levels.
6. SHAKE TABLE TEST OF HALF SCALE URM MODEL AFTER RETROFITTING

6.1 Introduction
A dynamic shake table test was performed on a retrofitted half scale model of the full scale unreinforced masonry room tested under quasi-static loading. The model was originally constructed and tested before retrofitting as part of PhD research at Department of Civil Engineering, University of Engineering & Technology Peshawar, Pakistan [KS-10]. The main objectives of the test were to study the dynamic performance of the retrofitted model and to compare the results with those obtained from the quasi-static test of full scale retrofitted room. Furthermore, the dynamic flexural response of out-of-plane walls, expected to be significantly improved after retrofitting, was also investigated.

6.2 Model Specimen and Materials
Plan and sectional views of the half scale model are given in Figure 6-1. All the physical dimensions were reduced to half of the prototype. The model was constructed on a 4" (102 mm) thick reinforced concrete slab which was then firmly secured with the shake table.
Simple model similitude requirements were incorporated in the construction and testing of model, i.e. the model was constructed using prototype materials, Table 6-1. Model bricks, cut from the prototype bricks, were laid in 1:6 cement-sand mortar using English bond at the face. No interior joints have been provided because the width of model bricks was similar to its length, Figure 6-1. A single course in model masonry was equivalent to two courses because the thickness of brick was double than that of the half scale brick. In order to keep the compression characteristic similar to the prototype masonry, the bed joint thickness was kept $\frac{1}{2}''$ (12.7 mm) instead of $\frac{1}{4}''$ (6.4 mm). On the other hand the thickness of head joint was kept equal to $\frac{1}{4}''$ (6.4 mm). The top slab was reinforced both ways with $\frac{1}{4}''$ (6.4 mm) diameter bars spaced at 3" (76 mm) on centre. The lintels were reinforced with 4, $\frac{1}{4}''$ (6.4 mm) diameter longitudinal bars and $\frac{1}{4}''$ (6.4 mm) stirrups spaced at 3" (76 mm) on centre.

### Table 6-1: Prototype and Model Material Properties

<table>
<thead>
<tr>
<th>S.No</th>
<th>Material Test Description</th>
<th>Prototype</th>
<th>Model</th>
<th>Ratio</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Brick Compressive Strength, $f_b$, psi  (MPa)</td>
<td>1803 (12.4)</td>
<td>1803 (12.4)</td>
<td>1.00</td>
</tr>
<tr>
<td>2</td>
<td>Mortar Cube Compressive Strength, $f_m$, psi (MPa)</td>
<td>733 (5.05)</td>
<td>1128 (7.8)</td>
<td>0.65</td>
</tr>
<tr>
<td>3</td>
<td>Masonry Compressive Strength, $f'_m$, psi (MPa)</td>
<td>438 (3.02)</td>
<td>570 (3.93)</td>
<td>0.77</td>
</tr>
<tr>
<td>4</td>
<td>Masonry Tensile Strength, $f_{tu}$, psi (MPa)</td>
<td>7.3 (0.05)</td>
<td>9.73 (0.07)</td>
<td>0.75</td>
</tr>
<tr>
<td>5</td>
<td>Concrete Compressive Strength, $f'_c$, psi (MPa)</td>
<td>3000 (20.7)</td>
<td>3000 (20.7)</td>
<td>1.00</td>
</tr>
<tr>
<td>6</td>
<td>Yield Strength of Steel Reinforcement, $f_y$, ksi (MPa)</td>
<td>40 (275.9)</td>
<td>40 (275.9)</td>
<td>1.00</td>
</tr>
<tr>
<td>7</td>
<td>Wire Mesh Reinforcement Ratio as percent of gross wall area (%)</td>
<td>0.054</td>
<td>0.086</td>
<td>1.59</td>
</tr>
</tbody>
</table>

### 6.3 Pre-Compression Stress

The main problem which comes across during dynamic testing of simple model is to simulate the damage mechanism without changing its dynamic
characteristics, i.e. natural period. To simulate the damage mechanism, the pre-compression stress in model must be kept equal to the stress in prototype. Using a dead weight at the roof level will definitely change the natural period of model which is not desirable. The problem may be solved by pre-stressing the model in order to produce the desired level of vertical stress in the walls. The pre-stressing, however, should not add to the lateral stiffness of the model.

Pre-stressing was, therefore, incorporated in the model testing. The pre-stressing force was applied through tightening bolts and measured with the help of a series of spring balance. The pre-stressing force was applied at three locations along each of in-plane walls, Figure 6-2. A pre-stressing force of 1267 lbs (5.64 KN) was applied at each location, equivalent to a total load of 760 lbs/ft (11.1 KN/m) of in-plane walls.

### 6.4 Retrofitting of Model

The damaged model, tested before retrofitting, was retrofitted with grout injection and reinforced plaster (ferrocement overlay). A steel welded wire mesh with the lightest gauge available in local market was used to retrofit the model. The percent steel ratio calculated over the gross area of wall was found to be 0.086%. The wire mesh was made with 0.035" (0.89 mm) diameter wires spaced at 1/2" (12.7 mm) on centre. The distance between wires in mesh was not scaled because of expected difficulty in application of plaster that might have resulted in an improper bond between wall surface and plaster.

Similar to isolated piers, perforated walls and full scale room, loose joint mortar was first removed from cracked locations. Wire mesh was then connected to both faces of in-plane and out-of-plane walls. As against full scale room, wire mesh was applied on out-of-plane walls with the purpose to investigate their dynamic behaviour. The wire mesh was connected with 1.0" (25 mm) long screws, steel washers and plastic plugs. The screws were spaced at rate of 4 per square foot (40 screws per square meter). An overlap of about 4.0" (100 mm) was provided at the discontinuous ends of the mesh, both vertically and horizontally. After connecting wire mesh with the surface of walls, injection nozzles/ports were then attached along the cracks spaced at distance of 6" (150 mm) roughly. The masonry walls were then plastered with 3/8" (10 mm) thick plaster in 1:4 cement-sand mortar.
6.5 Experimental Test Setup

The model, after retrofitting, was subjected to a test environment almost similar to the one used for its testing before retrofitting. The model was tested on 5ft x 5ft (1.5 m x 1.5 m) single degree of freedom shake table at the Earthquake Engineering Centre, University of Engineering and Technology, Peshawar, Figure 6-4. The shake table was operated by a displacement control actuator connected to the hydraulic and control system. The bottom pad of the model was firmly secured with shake table through 24 high strength bolts, 3/8” (10 mm) in diameter.

![Figure 6-3: Instrumentation of Half Scale URM Model After Retrofitting](image)

Table 6-2: Instrumentation Details, Half Scale Retrofitted URM Model

<table>
<thead>
<tr>
<th>Gauge ID</th>
<th>Type</th>
<th>Sensitivity mV/g, mV/in</th>
<th>Capacity</th>
<th>Location</th>
</tr>
</thead>
<tbody>
<tr>
<td>A0</td>
<td>Dytran Model 3056B3</td>
<td>4500</td>
<td>±2.0g</td>
<td>Shake Table</td>
</tr>
<tr>
<td>A1</td>
<td>Dytran Model 3191A</td>
<td>509.6</td>
<td>±10.0g</td>
<td>Bottom Slab along West Wall</td>
</tr>
<tr>
<td>A2</td>
<td>-d0-</td>
<td>509.0</td>
<td>-d0-</td>
<td>Top Slab along West Wall</td>
</tr>
<tr>
<td>A3</td>
<td>-d0-</td>
<td>496.2</td>
<td>-d0-</td>
<td>Bottom Slab along East Wall</td>
</tr>
<tr>
<td>A4</td>
<td>-d0-</td>
<td>510.1</td>
<td>-d0-</td>
<td>Top Slab along East Wall</td>
</tr>
<tr>
<td>D1</td>
<td>Celesco PT1DC Series</td>
<td>1000.7</td>
<td>0-20&quot; (0-500 mm)</td>
<td>Bottom Slab along West Wall</td>
</tr>
<tr>
<td>D2</td>
<td>-d0-</td>
<td>999.7</td>
<td>-d0-</td>
<td>Top Slab along West Wall</td>
</tr>
<tr>
<td>D3</td>
<td>-d0-</td>
<td>1000.0</td>
<td>-d0-</td>
<td>Bottom Slab along East Wall</td>
</tr>
<tr>
<td>D4</td>
<td>-d0-</td>
<td>1000.3</td>
<td>-d0-</td>
<td>Top Slab along East Wall</td>
</tr>
</tbody>
</table>

A total of five accelerometers and four displacement transducers were used to measure accelerations and displacements respectively at base and top of the
model as shown in Figure 6-3 and Figure 6-4 and detailed in Table 6-2. Accelerometer, A0 was mounted on shake table to record any relative movement between bottom slab and shake table. String pots displacement transducers were mounted on a reference frame erected off the shake table and connected to the model through strings, Figure 6-4.

![Figure 6-4: Reference Frame and Model Instrumentation](image)

The accelerometers and displacement transducers were connected to dynamic data acquisition system. The data was recorded at a sampling frequency of 200 Hz without applying any anti-aliasing filter.

### 6.6 Test Procedure

The retrofitted model was tested under the same conditions used for testing the model before retrofitting. After mounting the model on shake table and setting up the instruments, it was subjected to sinusoidal motion at frequency equivalent to the predominant frequency of JMA Kobe earthquake record. There are two main reasons for the selection of sinusoidal motion instead of true earthquake records.

- The working procedure of running shake table was to run a self check at frequent interval during the test in order to allow the system to develop its transfer functions. The transfer function of the system depends upon the dynamic characteristics (mass and stiffness) of the model, besides the components of the shake table (actuator, hydraulic system, rollers, etc). Without developing transfer function, the motion of table may not be controlled. In some of the previous test performed on the table, problems encountered during the development of transfer functions in the form of heavy unexpected shaking resulting in damaging the model. To avoid such circumstances in the present case, it was decided to run the table through a signal generator.

- The test before retrofitting was conducted using sinusoidal motion generated through signal generator. To compare results before and
after retrofit, it was necessary to conduct the test under the same environment that was used before retrofitting.

The predominant frequency of the JMA Kobe earthquake record is 2.8 Hz. To satisfy the similitude requirement of simple model, the test was run at a frequency around 5.5 Hz, Figure 6-5. The duration of the sinusoidal motion was kept around 20 seconds and the data was recorded for 25 seconds at a sampling frequency of 200 Hz without applying any anti-aliasing filter. To capture full range of the performance of the model, it was subjected to a series of sinusoidal motion with increasing amplitude. To identify the system, i.e. to determine the period and damping characteristic of the system, the model was subjected to a short duration sinusoidal motion at frequent interval. The motion was suddenly stopped to allow the system to vibrate freely.

![Figure 6-5: Typical Sinusoidal Motion](image)

Cracks produced during each test run were marked on the model and photographs were taken at the end of each test run. Later on the cracks were also mapped on 3D drawings. Two movie cameras were used to record each test run.

![Figure 6-6: Final Damage Pattern of Half Scale Model: West Wall (left), East Wall (right)](image)

### 6.7 Observed Behaviour of Model

The final damage pattern of the model at the end of the test is shown in Figure 6-6. From the very beginning of the test the model was deforming in a global rocking mode, producing cracks at the base of north and south walls. Minor cracks have been experienced by the model walls. The final damage condition of the model appeared in the form of excessive sliding at the base. Thus the overall behavior of model may be categorized as global rocking followed by in-plane rigid body sliding and torsional rotation at the base.
The observed behavior of the model during each test runs is given in Table 6-4 along with the model base acceleration.

Table 6-3: Observed Behaviour of Half Scale Model

<table>
<thead>
<tr>
<th>Test Run</th>
<th>Base Acceleration, (g)</th>
<th>Observed Behaviour</th>
</tr>
</thead>
<tbody>
<tr>
<td>TR-01 to TR-06</td>
<td>0.09-0.76</td>
<td>No damage except the cracks produced at the base of north and south wall due to global rocking of the model</td>
</tr>
<tr>
<td>TR-07</td>
<td>0.885</td>
<td>Cracks appeared along opening corners of west and east wall and internal side of north wall.</td>
</tr>
<tr>
<td>TR-14</td>
<td>0.920</td>
<td>Existing cracks were extended and some new cracks appeared including cracks above west and east walls below the roof.</td>
</tr>
<tr>
<td>TR-16</td>
<td>0.725</td>
<td>Cracked appeared in the base pad. The existing cracks extended on the west and east side.</td>
</tr>
<tr>
<td>TR-18</td>
<td>0.665</td>
<td>Bricks at the north-bottom corner of door in west wall came out of the walls crushing plaster and breaking the wire mesh.</td>
</tr>
<tr>
<td>TR-19</td>
<td>1.172</td>
<td>Model slid at the base pad to the north side along with some torsional rotation.</td>
</tr>
<tr>
<td>TR-20</td>
<td>0.937</td>
<td>More sliding was observed at the base.</td>
</tr>
</tbody>
</table>

6.8 Data Processing and Analysis

The time history records of displacements and accelerations were subjected to base line corrections and low pass filter in data analysis software, SeismoSignal (Version 4.0.0) by SeismoSoft. Quadratic base line correction was applied. Butterworth low pass filter with order equal to 8 and cutoff frequency ranging from 7 to 10 Hz was used as a filter. The cutoff frequency was kept more than the frequency of the sinusoidal motion.

Table 6-4 gives response displacement, shake table acceleration, base acceleration, response acceleration, and acceleration amplification factors of some of the test runs. Response displacement time histories, along west and east walls, were obtained by subtracting displacement time history recorded at bottom slab from the respective time history recorded at top slab. Response displacements in Table 6-4 is the average of maximum response displacements along west and east walls. Similarly base acceleration is the average of the maximum accelerations recorded at the base slab, while response acceleration is the average of maximum accelerations recorded at top slab. Acceleration amplification is calculated as the ratio of the response acceleration to base acceleration.

The accelerations recorded on shake table and at the base of the model are almost equal to each other showing that there was no relative movement between the shake table and model base. Since the model is very stiff having a very high fundamental frequency, the acceleration amplification (Figure 6-7) and relative displacements are very small. The high displacement in the last two runs is because of the sliding of the model along base pad.
Table 6-4: Shake Table Test Results

<table>
<thead>
<tr>
<th>Test Run</th>
<th>Shake Table Acceleration, (g)</th>
<th>Base Acceleration, (g)</th>
<th>Response Acceleration, (g)</th>
<th>Response Displacement, inch (mm)</th>
<th>Acceleration Amplification</th>
</tr>
</thead>
<tbody>
<tr>
<td>TR-02</td>
<td>0.096</td>
<td>0.089</td>
<td>0.100</td>
<td>0.004 (0.1)</td>
<td>1.098</td>
</tr>
<tr>
<td>TR-03</td>
<td>0.281</td>
<td>0.266</td>
<td>0.313</td>
<td>0.009 (0.2)</td>
<td>1.157</td>
</tr>
<tr>
<td>TR-04</td>
<td>0.439</td>
<td>0.410</td>
<td>0.493</td>
<td>0.022 (0.6)</td>
<td>1.155</td>
</tr>
<tr>
<td>TR-05</td>
<td>0.584</td>
<td>0.544</td>
<td>0.679</td>
<td>0.038 (1.0)</td>
<td>1.169</td>
</tr>
<tr>
<td>TR-06</td>
<td>0.790</td>
<td>0.757</td>
<td>1.006</td>
<td>0.069 (1.8)</td>
<td>1.207</td>
</tr>
<tr>
<td>TR-07</td>
<td>0.850</td>
<td>0.885</td>
<td>1.176</td>
<td>0.100 (2.5)</td>
<td>1.237</td>
</tr>
<tr>
<td>TR-14</td>
<td>0.954</td>
<td>0.920</td>
<td>1.301</td>
<td>0.150 (3.8)</td>
<td>1.394</td>
</tr>
<tr>
<td>TR-16</td>
<td>0.894</td>
<td>0.725</td>
<td>1.364</td>
<td>0.208 (5.3)</td>
<td>1.796</td>
</tr>
<tr>
<td>TR-18</td>
<td>0.812</td>
<td>0.665</td>
<td>1.426</td>
<td>0.230 (5.8)</td>
<td>1.702</td>
</tr>
<tr>
<td>TR-19</td>
<td>1.298</td>
<td>1.181</td>
<td>1.750</td>
<td>0.650 (16.5)</td>
<td>1.394</td>
</tr>
</tbody>
</table>

Figure 6-7: Acceleration Amplification

6.8.1 Response of Model

Figure 6-8 gives the base shear coefficient versus percent storey drift, relation of the model. Assuming the model is vibrating in its fundamental mode, base shear is calculated by multiplying the response acceleration with storey mass (3.39 kips 15.1 KN). Storey mass is calculated as sum of the masses of top slab and upper half portion of the masonry walls. Base shear coefficient is calculated as the ratio of base shear to the total weight of the model (5.44 kips, 24.2 KN) and storey drift is the ratio of response displacement to height of model (67.5″, 1715 mm). Although, there may be a time lag between the response acceleration and response displacement of the model, i.e. both the peaks may not necessarily have occurred simultaneously; this relation of peak values provides an overview of the loading history and the resulting performance [DPA-97]. The last two points in performance curve of the model are taken from the force-deformation loops of the test run TR-19.
6.8.2 Response of Prototype

Using the laws of simple model similarity given in Table 6-5, the response of model can be extrapolated to get the response of prototype. Since, the ratio of prototype to model strength was 0.77 (Table 6-1) instead of 1.0; the actual scale factors were different from the desired scale factors as shown in Table 6-5.

<table>
<thead>
<tr>
<th>Physical Quantity</th>
<th>Relationship</th>
<th>Desired Scale Factor</th>
<th>Actual Scale Factor</th>
</tr>
</thead>
<tbody>
<tr>
<td>Length (L)</td>
<td>( S_L = \frac{L_p}{L_m} )</td>
<td>2.00</td>
<td>2.00</td>
</tr>
<tr>
<td>Stress, Strength (f)</td>
<td>( S_f = \frac{f_p}{f_m} )</td>
<td>1.00</td>
<td>0.77</td>
</tr>
<tr>
<td>Strain (( \varepsilon ))</td>
<td>( S_\varepsilon = \frac{\varepsilon_p}{\varepsilon_m} )</td>
<td>1.00</td>
<td>1.00</td>
</tr>
<tr>
<td>Specific Mass (( \rho ))</td>
<td>( S_\rho = \frac{\rho_p}{\rho_m} )</td>
<td>1.00</td>
<td>1.00</td>
</tr>
<tr>
<td>Displacement (d)</td>
<td>( S_d = \frac{d_p}{d_m} = S_L )</td>
<td>2.00</td>
<td>2.00</td>
</tr>
<tr>
<td>Force (F)</td>
<td>( S_F = \frac{F_p}{F_m} = S_L^2S_f )</td>
<td>4.00</td>
<td>3.08</td>
</tr>
<tr>
<td>Time (t)</td>
<td>( S_t = \frac{t_p}{t_m} = S_L \frac{S_eS_\rho}{S_f} )</td>
<td>2.00</td>
<td>2.28</td>
</tr>
<tr>
<td>Frequency (( \Omega ))</td>
<td>( S_\Omega = \frac{\Omega_p}{\Omega_m} = 1/S_t )</td>
<td>0.50</td>
<td>0.44</td>
</tr>
<tr>
<td>Velocity (v)</td>
<td>( S_v = \frac{v_p}{v_m} = \sqrt{S_eS_\rho}/S_f )</td>
<td>1.00</td>
<td>1.14</td>
</tr>
<tr>
<td>Acceleration (a)</td>
<td>( S_a = \frac{a_p}{a_m} = S_f/S_LS_\rho )</td>
<td>0.50</td>
<td>0.39</td>
</tr>
</tbody>
</table>

Using the actual scale factors the response of prototype building can be obtained. Since the storey drift and the base shear coefficient are dimensionless quantities, they remain same for both model and prototype.
Three performance levels; immediate occupancy (IO), life safety (LS) and collapse prevention (CP) as marked in Figure 6-8 are given in Table 6-6. IO is assumed to correspond to first significant crack, i.e. the point where stiffness significantly changes, CP to drift in last test run and LS to 0.75 of CP. The higher deformation capacities corresponding to LS and CP are because of the model sliding along its base in the last test runs.

<table>
<thead>
<tr>
<th>Performance Level</th>
<th>Base Acceleration, g</th>
<th>Storey Drift, %</th>
<th>Base Shear Coefficient</th>
</tr>
</thead>
<tbody>
<tr>
<td>Immediate Occupancy, IO</td>
<td>0.35</td>
<td>0.20</td>
<td>0.30</td>
</tr>
<tr>
<td>Life Safety, LS</td>
<td>0.47</td>
<td>1.76</td>
<td>0.41</td>
</tr>
<tr>
<td>Collapse Prevention, CP</td>
<td>0.62</td>
<td>2.35</td>
<td>0.39</td>
</tr>
</tbody>
</table>

6.9 **Comparison of Force-Deformation Behavior Before and After Retrofit**

The detail model behavior before retrofitting may be found elsewhere [KS-10]. For comparison of the model behavior, tested before and after retrofitting, the final damage pattern of the model, tested before retrofitting, is reproduced in Figure 6-9. Global model rocking and local pier rocking can be clearly seen. The model could have not been tested till its complete failure. In the case of retrofitted model, the model was deforming in global rocking mode without any sign of local pier rocking.
The force-deformation behaviour of the model, tested before and after retrofitting, is compared in Figure 6-10. As stated earlier, the model, before retrofitting, could have been tested up to its peak load; the response is compared in terms of lateral stiffness and peak load. Comparison of the ultimate displacement is not possible. The lateral stiffness and peak loads are increased by almost 100% and 35% respectively. However in an actual building, one would expect more increase in lateral strength, where sliding of the single room would be restrained.
7. SIMPLE STATIC NONLINEAR ANALYTICAL MODEL

7.1 Introduction
This chapter deals with a simple non-linear static analytical model, proposed for the analysis URM building before and after retrofitting with reinforced plaster (ferrocement overlay). The chapter starts with definition of expression for various force-deformation parameters, e.g. lateral stiffness, strength and deformation capacity piers, followed by boundary condition factor. The piers force-deformation characteristics are extrapolated to get the capacity curve of a building. Building performance levels are then defined. The proposed analysis tool is compared with experimental work towards the end of chapter.

7.2 Analytical Model
In the proposed model the capacity of a building in a certain direction is obtained by superposing the capacities of all piers in that direction. The force deformation curve of a pier, as given in Figure 7-1 is defined by its lateral stiffness (effective stiffness), lateral strength, residual strength and ultimate displacement. The proposed analytical model is based on the following assumptions:

1. Strong spandrel and weak pier, i.e. the failure occurs in the piers of the building.
2. The diaphragm is assumed to be rigid and connected to the walls so that the displacements at the storey level at all locations remain same. The superposition of capacities of individual piers, to get total capacity of a building, is thus valid.
3. The effect of torsion is also not considered, meaning this model is applicable to regular buildings.
4. Flange effect, which causes an increase in the lateral capacity of a pier, is neglected in the analytical model.
5. Global rocking and variation in vertical stress with lateral load are also not considered in the proposed model.

The force-deformation curve of ASCE Standard, ASCE/SEI 41-06 (ASCE-06) is slightly modified to as shown in Figure 7-1. $V_L$ is the lateral load capacity, $V_r$ is the residual strength, $\Delta_y$ is the yield displacement and $\Delta_u$ is the ultimate displacement of the pier. Lateral strength, $V_L$ is taken as minimum of the
Flexural and shear strength of the pier and $V_r$ is taken as 0.6 times the lateral strength.

Different criteria are used for the ultimate displacement of pier corresponding to flexural and shear failure modes as detailed in the following paragraphs. Different parameters, defining the force-deformation curve of a single pier before and after retrofitting, are discussed in the following paragraphs.

### 7.2.1 Effective Stiffness of Pier

The effective stiffness of a masonry pier is given by:

$$k_{eff} = \beta k_p$$  \hspace{1cm} (7-1)

Where, $k_p$ is the elastic stiffness and $\beta$ is a factor representing the ratio of effective to the elastic stiffness. Since, the rocking behaviour is initially linear elastic, while the shear behaviour is non-linear from the very beginning of the test; therefore different criteria are used for the effective stiffness of rocking-critical and shear-critical piers. Based on the experimental results factor $\beta$ is taken 1.0 for rocking-critical and 0.9 for shear-critical piers.

The elastic stiffness of piers is calculated from a mechanics based equation (Eq.7-2) [MJ-08] considering both flexural and shear deformation, given below:

$$k_p = \frac{1}{6E_m I_g} \left( \frac{H_p^2 (3\psi - 1)}{L_p I_p} + \frac{H_p}{A_v G_m} \right)$$  \hspace{1cm} (7-2)

Where, $I_g = t_p L_p^3 / 12$, $A_v = 0.83 L_p I_p$, $L_p$, $H_p$ and $t_p$ are the length, height and thickness of pier, $E_m$ and $G_m$ are the moduli of elasticity and rigidity of masonry material respectively and $\psi$ is a boundary condition factor representing the ratio of height of the inflection point, $H_o$, to the height of pier ($\psi = H_o / H_p$).

In the case of wall coated with plaster the geometric properties, $I_g$ and $A_v$ in the above equation shall be calculated based on the transformed thickness $t_T$ given by:

$$t_T = t_p + \frac{E_c}{E_m} t_c$$  \hspace{1cm} (7-3)
Where, $t_c$ is the thicknesses of coating and $E_c$ is the elastic modulus of coating materials and is taken as function of compressive strength, $f_c$ of coating material (NIST-97):

$$E_c = 750f_c$$  \hspace{1cm} 7-4

### 7.2.2 Lateral Strength Corresponding to Flexural Cracking of Pier

The lateral strength corresponding to flexural cracking of pier without retrofitting is calculated by assuming linear stress distribution at the base of pier and is given by:

$$V_{cr} = \frac{L_p t_p}{6\psi H_p}(p + f_i)$$  \hspace{1cm} 7-5

Where, $f_i$ is the bed joint tensile strength of masonry and $p$ is the vertical stress on pier.

In the case of a retrofitted pier, with reinforced plaster coating continuous between spandrel and pier, the expression for lateral load capacity corresponding to the flexural cracking of the coating can be derived with reference to Figure 7-2. Strain is assumed to vary linearly along the length of pier and the tensile strength of masonry is ignored. Since the strain at the cracking of coating is very small, therefore the tensile stresses in steel wire mesh are also ignored. Stresses in coating material and masonry are assumed to be linear. Using equilibrium of forces and moments following equation is obtained:

$$V_{cr} = \frac{f_c t_c}{12\psi H_p} \left[ \left( \frac{E_m t_p}{E_c t_c} + 1 \right) \left( \frac{3L_p - 2c}{L_p - c} \right) + \left( L_p - c \right) \left( L_p + 2c \right) \right]$$  \hspace{1cm} 7-6

Where,

$$c = \frac{E_c}{E_m t_p} \left[ \left( L_p t_c + \frac{P}{f_{cr}} \right) + \left( L_p t_c + \frac{P}{f_{cr}} \right)^2 + 2 \left( \frac{E_m t_c}{E_c} \right) \frac{L_p^2 t_c}{2} + \frac{P L_p}{f_{cr}} \right]$$

And $P$ is the total vertical load on pier and $f_{cr}$ is the stress in coating material corresponding to its tensile cracking. The cracking stress of coating material is calculated from the expression given in ACI-318 (ACI-05) as:

$$f_{cr} = 6\sqrt{f_c}$$  \hspace{1cm} 7-7

Since the pier is with in elastic range, lateral displacement at top of pier corresponding to flexural cracking of pier may be obtained from:

$$\Delta_{cr} = V_{cr} / k_p$$  \hspace{1cm} 7-8
7.2.3 Toe Crushing in Retrofitted Pier

The lateral capacity of a retrofitted pier at the toe crushing may be calculated with reference to assumed linear variation of strain shown in Figure 7-2. The compressive stresses in masonry and coating are represented in the form of equivalent stress block with stress equal to 85% of the compressive strength and depth equal to 80% of the depth of neutral axis, $c$. The ultimate strain is assumed to be 0.0035, both for masonry and concrete. All the tensile steel wires are assumed to have yielded at the ultimate condition which is reasonable because the depth of neutral axis, $c$ is usually very small and the strain on the tensile side is very high. Using the equilibrium of vertical forces and moments, one can obtain:

$$V_c = \frac{L_p f_p}{2 \psi H_p} \left( p + f_y \rho_s \right) \left( 1 - \frac{p + f_y \rho_s}{0.85 f_m + 0.85 f_c \frac{t_c}{L_p} + f_y \rho_s} \right)$$

Where, $\rho_s$ is the vertical reinforcement ratio and $f_y$ is the yield strength of wire mesh reinforcement. The above equation is equally applicable to a non-retrofitted pier by putting $\rho_s$ and $t_c$ equal to zero.

$$V_c = \frac{L_p^2}{2 \psi H_p} \left( p \left( 1 - \frac{p}{0.85 f_m} \right) \right)$$

The displacement at the top of pier corresponding to toe crushing is calculated from linear distribution of curvature varying from maximum at the base to zero at the point of inflection. The curvature at the bottom of
pier, corresponding to toe crushing is given by, \( \phi_{tc} = \varepsilon_{tc} / c \). The curvature at any height, \( h \) is given by:

\[
\phi_h = \frac{d^2 \Delta}{dh^2} = \phi_{tc} \left( 1 - \frac{h}{\varepsilon p H_p} \right) = \frac{\varepsilon_{tc}}{c} \left( 1 - \frac{h}{\varepsilon p H_p} \right)
\]  

(7-11)

Double integration of the above equation from 0 to \( H_p \), and applying the boundary conditions, gives the following expression:

\[
\Delta_{tc} = \frac{\varepsilon_{tc}}{6c} H_p^2 \left( 3 - \frac{1}{\psi} \right)
\]  

(7-12)

Where, \( c = 1.25(p + f_s \rho_s) \left( 0.85 f_m + 0.85 f_c \frac{t_c}{t} + f_s \rho_s \right) \) for retrofitted pier and \( c = \rho/(0.68 f_m) \) for non-retrofitted pier.

In some cases, especially in slender pier with low vertical loads, the displacement corresponding to toe crushing is very high and the pier may slide (walking) or twist in out-of-plane direction well before the toe crushing. Therefore an upper limit must be imposed on the ultimate displacement. Based on the work done by past researchers an upper limit of \( \Delta_u = 0.010 H_p \) is used for non-retrofitted pier and \( \Delta_u = 0.006 H_p \) for retrofitted piers.

### 7.2.4 Shear Capacity of Retrofitted Piers

The shear capacity of piers retrofitted with reinforced plaster is calculated as the sum of shear capacity of masonry, coating and reinforcement.

\[
V_L = V_m + V_c + V_s
\]  

(7-13)

Where, \( V_m, V_c \) and \( V_s \) are the shear capacities of masonry wall, plaster coating and wire mesh reinforcement respectively.

The shear capacity of masonry, \( V_m \) is calculated as minimum of shear sliding capacity, \( V_{sl} \) and diagonal cracking capacity, \( V_{dc} \) given by:

\[
V_{sl} = L_p t (\mu_m p)
\]  

(7-14)

\[
V_{dc} = \frac{f_{tu} L_p t}{1 + \frac{p}{f_{tu}}}
\]  

(7-15)

Where, \( \mu_m \) is the coefficient of friction of masonry, \( f_{tu} \) is the diagonal tension strength of masonry computed from the diagonal compression test data and \( b \) is factor depending upon the aspect ratio \( H_p/L_p \) of pier and is given by: \( b = H_p / L_p \) (1 \( \leq b \leq 1.5 \))

Equation for diagonal cracking capacity, \( V_{dc} \) corresponds to a ductile failure mode, assuming that the cracks pass through the mortar bed and head joints. If the diagonal tensile strength, \( f_{tu} \) is found greater than the tensile strength of brick, \( f_{bt} \), the diagonal crack will more likely pass through the bricks and thus resulting failure mode will be non-ductile.
The shear capacities of the plaster coating and wire mesh reinforcement are obtained from the equations provided in ACI-318 (ACI-05) for the reinforced concrete members. The effective length of pier is taken 0.9 times the total length of the pier.

\[ V_c = 1.8 \sqrt{f_c L_p t_c} \]  
\[ V_s = 0.9 \rho_y f_y L_p t_s \]

Based on the experimental work done as a part of this study and the work done by previous researchers, the ultimate drift ratio for shear critical piers is taken as 0.6% before retrofitting and 0.4% after retrofitting.

### 7.2.5 Determination of Boundary Condition Factor, \( \psi \)

A method originally proposed by Muto [PP-92] and adopted by past researchers [MJ-08] was used to determine the boundary condition factor. The boundary condition at top and bottom of a pier depends on the relative stiffness of spandrels and pier, \( K_{RL} \), the relative stiffness of top and bottom spandrel, \( \alpha_i \), the storey level of the pier and the number of storeys in a building. The relative stiffness \( K_{RL} \) and \( \alpha_i \) are given by:

\[ K_{RL} = \left( K_{ST} + K_{SB} \right) / 2K_P \]

\[ \alpha_i = K_{ST} / K_{SB} \]

Where, \( K_{ST} \) and \( K_{SB} \) are the total stiffness of top and bottom spandrels on both sides of piers respectively and \( K_P \) is the stiffness of piers. In the case of ground storey, the bottom spandrels are considered infinitely rigid irrespective of their geometric dimensions, because they are directly attached to the foundation which is sufficiently rigid. The contribution from the reinforced concrete lintels and roof slab is also considered in evaluating the stiffness of spandrels.

The boundary condition factor, \( \psi \) is calculated as sum of two components; \( \psi_o \) and \( \psi_1 \) taken from Table 7-1 and Table 7-2 respectively. In case \( \alpha_i \) is greater than one, \( 1/\alpha_i \) shall be used but in that case \( \psi \) is calculated as difference \( \psi_o \) and \( \psi_1 \). In case if \( \psi \) is less than 0.5, \( \psi = 1 - \psi \).

<table>
<thead>
<tr>
<th>No. of Stories</th>
<th>Storey Level</th>
<th>( K_{RL} )</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>0.1</td>
</tr>
<tr>
<td>1</td>
<td>1</td>
<td>0.80</td>
</tr>
<tr>
<td>2</td>
<td>2</td>
<td>0.50</td>
</tr>
<tr>
<td></td>
<td>1</td>
<td>1.00</td>
</tr>
<tr>
<td>3</td>
<td>3</td>
<td>0.25</td>
</tr>
<tr>
<td></td>
<td>2</td>
<td>0.60</td>
</tr>
<tr>
<td></td>
<td>1</td>
<td>1.15</td>
</tr>
</tbody>
</table>
Table 7-2: Boundary Condition Factor, $\psi_j$

<table>
<thead>
<tr>
<th>$\alpha_j$</th>
<th>0.1</th>
<th>0.2</th>
<th>0.3</th>
<th>0.4</th>
<th>0.6</th>
<th>0.8</th>
<th>1.0</th>
<th>3.0</th>
<th>5.0</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.4</td>
<td>0.80</td>
<td>0.75</td>
<td>0.70</td>
<td>0.65</td>
<td>0.60</td>
<td>0.50</td>
<td>0.55</td>
<td>0.55</td>
<td>0.55</td>
</tr>
<tr>
<td>0.5</td>
<td>0.45</td>
<td>0.30</td>
<td>0.20</td>
<td>0.20</td>
<td>0.15</td>
<td>0.10</td>
<td>0.10</td>
<td>0.05</td>
<td>0.05</td>
</tr>
<tr>
<td>0.6</td>
<td>0.30</td>
<td>0.20</td>
<td>0.15</td>
<td>0.15</td>
<td>0.10</td>
<td>0.10</td>
<td>0.05</td>
<td>0.05</td>
<td>0.00</td>
</tr>
<tr>
<td>0.7</td>
<td>0.20</td>
<td>0.15</td>
<td>0.10</td>
<td>0.10</td>
<td>0.05</td>
<td>0.05</td>
<td>0.05</td>
<td>0.00</td>
<td>0.00</td>
</tr>
<tr>
<td>0.8</td>
<td>0.15</td>
<td>0.10</td>
<td>0.05</td>
<td>0.05</td>
<td>0.05</td>
<td>0.05</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
</tr>
<tr>
<td>0.9</td>
<td>0.05</td>
<td>0.05</td>
<td>0.05</td>
<td>0.05</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
</tr>
</tbody>
</table>

7.3 Force-deformation (Capacity) Curve of a Building

The force-deformation curve of a building in a certain direction is obtained by superposing the force-deformation curves of all the contributing piers in that direction. Since the capacity curve of a building is drawn between roof displacement and lateral strength, therefore the displacement at the top of pier must be transformed to the corresponding displacement at the roof level. Since the system remains within elastic limit, therefore, the displacements at the roof level corresponding to flexural cracking and yielding of pier are calculated by multiplying the corresponding pier drift ratio with the height up to roof level, $H_w$.

\[
\Delta_{crw} = \Delta_c \frac{H_w}{H_p} \quad 7-20
\]

\[
\Delta_{yw} = \Delta_y \frac{H_w}{H_p} \quad 7-21
\]

Where $\Delta_{crw}$ and $\Delta_{yw}$ are the displacements at the roof level corresponding to flexural cracking and yielding of pier.

Figure 7-3: Transformation of Pier Top Displacement to Building Top Displacement (left), Superposition of Force-deformation Curves of Piers (right)
In the case of ultimate displacement, however, the pier has got yielded but it is assumed that the spandrels are within elastic limits, Figure 7-3. The ultimate displacement at the roof level, $\Delta_{uw}$ can be proved equal to the following relation:

$$\Delta_{uw} = \left(1 + \frac{H_u}{H_y} \left(\frac{\Delta_u}{\Delta_y} - 1\right)\right) \Delta_{yw}$$  \hspace{1cm} 7-22

The extrapolated force-deformation curves of all the contributing piers can be superposed to get the force-deformation curve of a building, Figure 7-3.

### 7.4 Step-by-Step Procedure to get the Capacity Curve of a Building

The seismic capacity evaluation of an unreinforced masonry building before and after retrofitting with reinforced plaster goes through the following steps:

- Divide all the in-plane walls of the building into piers and spandrels as shown in Figure 7-4.
- Calculate vertical stress on the top of each pier which includes weight from the top stories, floors, and walls above the pier.
- Calculate boundary condition factor for each pier using the Muto method.
- Calculate the elastic stiffness, flexural cracking strength, toe crushing strength, shear sliding strength, and diagonal shear strength of each pier.
- If the toe crushing strength is less than shear sliding and diagonal shear strength, then classify the failure mode as flexural, otherwise shear.
Based on the failure mode, calculate the effective stiffness, cracking displacement, yield displacement and ultimate displacement of each pier.

Transform the pier top displacement to the roof top displacement and using the superposition principle construct the force-deformation curve of the building from the force-deformation curve of all contributing piers as shown in Figure 7-3 (right).

### 7.5 Building Performance Levels

The performance of a building is defined at various discrete levels on force-deformation (capacity) curve of a building. ASCE standard, ASCE/SEI 41-06 (ASCE-06) recommends three performance levels; Immediate Occupancy (IO), Life Safety (LS) and Collapse Prevention (CP) for the seismic rehabilitation of existing buildings. Originally both shear-sliding and rocking were considered as deformation-control actions in pre-standard FEMA-356 (Table 7-3), but in the ASCE standard only rocking is considered as deformation-control action. However the experimental work on unreinforced masonry has proved that the shear sliding also satisfies the criteria set forth in ASCE standard for deformation-control actions.

<table>
<thead>
<tr>
<th>Limiting Behavior Mode</th>
<th>Linear Static (m factors)</th>
<th>Non-Linear (Drift Ratios %)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>IO</td>
<td>LS</td>
</tr>
<tr>
<td>Flexural Rocking</td>
<td>1.0</td>
<td>3.0</td>
</tr>
<tr>
<td>Shear Sliding</td>
<td>(1.5 \frac{H_p}{L_p})</td>
<td>(\frac{H_p}{L_p})</td>
</tr>
</tbody>
</table>

In ASCE standard, the performance levels are associated with the force-deformation characteristics of piers/walls, not to the global performance of building. To associate different performance levels with global performance of a building, the criteria recommended by Javed M. [MJ-08] with slight modification is used.

Operational level, which is neither addressed in ASCE standard nor in FEMA pre-standard for rehabilitation of buildings, is associated with drift ratio corresponding to average flexural cracking of all piers. IO performance level is associated with a pier drift ratio of 0.1% in the case when at least one pier fails in shear and 0.15% when all the piers behave in rocking mode. LS performance level is associated with 75% (against 50% as recommended by Javed) of the drift ratio corresponding to CP. CP correspond to a drift ratio at which the overall capacity drops by 20%.

### 7.6 Global Displacement and Acceleration Demand

In seismic building codes including Pakistan Building Code [PBC 07], the seismic demand on a structure is usually given in the form of displacement or acceleration elastic spectra corresponding to a single degree of freedom.
system. On the other hand the capacity of a multi-degree of freedom (MDOF) system is represented in the form of inelastic force deformation curve. The displacement and acceleration demand on an inelastic multi-degree of freedom (MDOF) system can be obtained by first converting the inelastic MDOF system to a corresponding elastic MDOF system and then elastic MDOF system to equivalent single degree of freedom (SDOF) system.

The inelastic displacement demand \( \Delta \) on a MDOF system can be transformed to elastic displacement demand \( \Delta_e \) on a corresponding elastic system with reference to Figure 7-5.

The principle of equal energy holds good for frequency, \( f > 2 \text{Hz} \) which is the case for almost all masonry buildings. According to equal energy principle the response modification factor, which is the ratio of elastic force \( V_e \) to the inelastic force \( V_y \), is given by:

\[
R_\Delta = \sqrt{2\mu_\Delta - 1} \tag{7-23}
\]

Where \( \mu_\Delta \) is the ductility demand corresponding to a displacement demand, \( \Delta \) and is given by, \( \mu_\Delta = \Delta / \Delta_y \geq 1 \). The elastic displacement demand is calculated as:

\[
\Delta_e = \begin{cases} 
V_e / k_e = \Delta_y R_\Delta = \Delta_y \sqrt{(2\mu_\Delta - 1)} & \text{for} \Delta \geq \Delta_y \\
\Delta & \text{for} \Delta < \Delta_y 
\end{cases} \tag{7-24}
\]

The elastic displacement demand \( \Delta_e \), in MDOF system is converted to a displacement demand, \( S_d \) in equivalent single degree of freedom (SDOF) system using the following relations given in FEMA-274:

\[
S_d = \{\phi\}^T [M] \{\phi\} \Delta = \frac{1}{\Gamma} \Delta_e \tag{7-25}
\]

Where \([M]\) is the storey mass matrix and \( \Gamma \) is the modal participation factor corresponding to the fundamental mode of vibration \( \phi \). The mode shape \( \phi \) has to be normalized with ordinate equal to one at the top floor. A linear mode shape is the best alternative for masonry buildings, [MJ-08].
The spectral acceleration, $S_a$, in elastic SDOF system may directly be related to the spectral displacement, $S_d$, through the relation:

$$S_a = (2\pi f_E)^2 S_d$$  \hspace{1cm} (7-26)

$$f_E = \frac{1}{2\pi} \sqrt{\frac{k_E}{m_E}}$$  \hspace{1cm} (7-27)

$$m_E = \{\phi\}^T [M] \{\phi\} , \quad k_E = k_{\text{eff}}$$  \hspace{1cm} (7-28)

Where $f_E$, $m_E$ and $k_E$ are the frequency, mass and stiffness of equivalent SDOF system respectively.

Conservatively the ground acceleration is calculated by dividing the spectral acceleration of the SDOF system with the acceleration amplification factor of 2.5 which correspond to the horizontal portion of the response spectra recommended by Pakistan Building Code [PBC 07].

Alternatively the target displacement ($\delta_t = \Delta$) in a non-linear MDOF system corresponding to a spectral acceleration ($S_a$) in an elastic SDOF system may be calculated by using the Eq. 3-14 of ASCE standard (ASCE/SEI 41-06) given below:

$$\Delta = C_0 C_1 C_2 S_a \frac{T_e}{2\pi^2} g$$  \hspace{1cm} (7-29)

Where $C_0$ is modification factor relating displacement in elastic SDOF system to displacement in elastic MDOF system and is equal to the first mode mass participation factor, $C_1$ is modification factor relating inelastic displacement to elastic displacement, $C_2$ is modification factor representing the effect of hysteretic response and $T_e$ is the effective fundamental period of the building.

### 7.7 Comparison of the Proposed Model with Experimental Results

The proposed model is compared with the test results performed as part of this study and also by other researchers.

![Figure 7-6: Full Scale Room: Plan and Elevations](image)
### 7.7.1 Comparison with Quasi Static Test of Room

The proposed model is applied to the single room building, Figure 7-6 tested under quasi-static loading before and after retrofitting with reinforced plaster. The material and geometric properties of the building are summarized in Table 7-4.

<table>
<thead>
<tr>
<th>Material and Geometric Properties of Single Room Building</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Parameter</strong></td>
</tr>
<tr>
<td>Masonry Compressive Strength (Before Retrofit)</td>
</tr>
<tr>
<td>Masonry Diagonal Tensile Strength (Before Retrofit)</td>
</tr>
<tr>
<td>Elastic Modulus of Masonry (Before Retrofit)</td>
</tr>
<tr>
<td>Shear Modulus of Masonry (Before Retrofit)</td>
</tr>
<tr>
<td>Coefficient of Friction of Masonry (Before Retrofit)</td>
</tr>
<tr>
<td>Specific Weight of Masonry Material, pcf (KN/m³)</td>
</tr>
<tr>
<td>Height of Wall, inch (mm)</td>
</tr>
<tr>
<td>Thickness of Masonry Wall, inch (mm)</td>
</tr>
<tr>
<td>Total Thickness of Plaster Coating, inch (mm)</td>
</tr>
<tr>
<td>Compressive Strength of Plaster Coating, psi (MPa)</td>
</tr>
<tr>
<td>Modulus of Elasticity of Plaster Coating, ksi (MPa)</td>
</tr>
<tr>
<td>Tensile Strength of Plaster Coating, psi (MPa)</td>
</tr>
<tr>
<td>Yield Strength of Steel Wire Mesh, ksi (MPa)</td>
</tr>
<tr>
<td>Reinforcement Ratio (%)</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Pier Geometry and Load Data of Single Room Building</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Parameter</strong></td>
</tr>
<tr>
<td>No. of Piers</td>
</tr>
<tr>
<td>Length of Pier, inch</td>
</tr>
<tr>
<td>Height of Pier, inch</td>
</tr>
<tr>
<td>Boundary Condition Factor</td>
</tr>
<tr>
<td>Total Load on Pier Top, lbs</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Force-Deformation Parameters of each Pier: Before Retrofit</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Parameter</strong></td>
</tr>
<tr>
<td>Lateral Capacity, kips</td>
</tr>
<tr>
<td>Flexural Cracking Capacity, kips</td>
</tr>
<tr>
<td>Flexural Cracking Displacement, inch</td>
</tr>
<tr>
<td>Wall Yielding Displacement, inch</td>
</tr>
<tr>
<td>Wall Ultimate Displacement, inch</td>
</tr>
<tr>
<td>Elastic Stiffness of Piers kips/inch</td>
</tr>
<tr>
<td>Ratio of Effective &amp; Elastic Stiffness</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Failure Modes</th>
<th><strong>Symbol</strong></th>
<th><strong>Value</strong></th>
<th><strong>Unit</strong></th>
</tr>
</thead>
<tbody>
<tr>
<td>Shear, Flexural</td>
<td>( \text{Flexural} )</td>
<td>( \text{Shear} )</td>
<td>( \text{Flexural} )</td>
</tr>
</tbody>
</table>
The geometrical and load data for each pier is given in Table 7-5. Pier P1 and P2 are the east and west pier of north wall respectively and P3 and P4 are the middle and central piers of south wall respectively. The boundary condition factor is calculated as 0.5 for each pier using Muto method.

| Table 7-7: Force-Deformation Parameters of each Pier: After Retrofit |
|----------------|----------------|----------------|----------------|----------------|
| Pier ID        | P1             | P2             | P3             | P4             |
| V_{m}, Lateral Capacity, kips | 8.08           | 11.96          | 4.10           | 6.27           |
| V_{cr}, Flexural Cracking Capacity, kips | 5.50           | 8.25           | 2.76           | 4.42           |
| Δ_{cr}, Flexural Cracking Displacement, inch | 0.030          | 0.029          | 0.032          | 0.027          |
| Δ_{yw}, Wall Yielding Displacement, inch | 0.044          | 0.042          | 0.047          | 0.038          |
| Δ_{uw}, Wall Ultimate Displacement, inch | 0.622          | 0.417          | 0.419          | 0.415          |
| K_{w}, Elastic Stiffness of Wall kips/inch | 185.65         | 285.27         | 87.45          | 164.50         |
| Failure        | Mode, Shear, Flexural | Flexural | Flexural | Flexural |
| K_{eff}/K_{w}, Ratio of Effective & Elastic Stiffness | 1.00           | 1.00           | 1.00           | 1.00           |

Various parameters defining the capacity curve of each pier before and after retrofitting, calculated using the above mentioned procedure, are given in Table 7-6 and Table 7-7 respectively.

The empirical force-deformation curves, obtained using the proposed model, are compared with the corresponding experimental force-deformation curves of the building in Figure 7-7.

The proposed model accurately predicts the elastic response of the building but could not achieve good agreement with the post-elastic behaviour both in terms of strength and deformation capacities. The ratio between experimental strength and empirical strength is found to be about 1.2. The deviation of the empirical behaviour from the experimental behaviour is due to global rocking and flange effect of the external piers, which are ignored in the proposed model but observed in the experimental results. Moreover, in the retrofitted model, it is assumed that the grout injection just restored the pre-damaged mechanical properties of masonry and did not cause any improvement in them. The situation might, however, be different. The pre-damaged mechanical properties of masonry are very low and the grout injection definitely has improved these properties.

It is worth mentioning that the model has conservatively predicted the behaviour of both non-retrofitted and retrofitted unreinforced building. Since the factors of safety in both the cases are almost equal, therefore the model may be effectively used for the comparison of retrofitted and non-retrofitted buildings.

Figure 7-8 shows a comparison of the capacities in terms of ground acceleration capacity, calculated from the proposed analytical model, corresponding to various performance levels of the building before and
after retrofitting. It is found that the acceleration capacity increases by more than 2 times with the proposed retrofitting technique.

![Comparison of Experimental and Empirical Force-Deformation Curves before and after Retrofitting](image)

Figure 7-7: Comparison of Experimental and Empirical Force-Deformation Curves before and after Retrofitting

![Comparison of Performance Levels before and after Retrofitting](image)

Figure 7-8: Comparison of Performance Levels before and after Retrofitting

7.7.2 Comparison with Quasi-static Test of Unreinforced Masonry Wall

Material properties and geometry of the unreinforced masonry wall are given in Table 4-1 and Figure 4-3 respectively. The wall was retrofitted with steel welded wire mesh similar to full scale room.

The measured and estimated force-deformation response of the URM wall before and after retrofitting is compared in Figure 7-9. The estimated stiffness before and after retrofitting is found in good agreement with measured stiffness. The strength and deformation capacities are, however conservatively estimated. The estimated deformation is less than the measured one because of the global rocking of the wall. The global rocking also caused variation in vertical stresses on piers which finally resulted in a higher lateral strength.
7.7.3 Comparison with Experimental Results at Pavia

To verify the authenticity of the proposed model, it is also compared with the experimental force-deformation envelope curves by other researchers around the world. The comparison could be made only with the results before retrofitting because the model was not retrofitted with reinforced plaster for which the methodology has been developed.

Figure 7-10 shows the plane and elevation of a double storey model tested at University of Pavia, Italy under quasi-static loading [MKC-95]. To avoid flange effect wall A was disconnected from the orthogonal walls. The properties of masonry are given in Table 7-8. The diagonal tensile strength is assumed to be 0.035 times the compressive strength [MJ-08] and the modulus of rigidity is taken as 0.4 times the modulus of elasticity. The ground floor of Wall A has been divided into three piers out of which two end piers are identical. The dimensions, top load and boundary condition factor of each pier are provided in Table 7-11.
Table 7-8: Pavia Model: Mechanical and Physical Properties of Masonry Materials

<table>
<thead>
<tr>
<th>Property</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>$f_m$ Masonry Compressive Strength, psi (MPa)</td>
<td>900 (6.21)</td>
</tr>
<tr>
<td>$f_{dd}$ Masonry Diagonal Tensile Strength, psi (MPa)</td>
<td>31.5 (0.220)</td>
</tr>
<tr>
<td>$f_t$ Bed Joint Tensile Strength (Before Retrofit), psi (MPa)</td>
<td>0.0</td>
</tr>
<tr>
<td>$E_m$ Elastic Modulus of Masonry, ksi (MPa)</td>
<td>214.6 (1480)</td>
</tr>
<tr>
<td>$G_m$ Shear Modulus of Masonry, ksi (MPa)</td>
<td>85.8 (592)</td>
</tr>
<tr>
<td>$\mu$ Coefficient of Friction of Masonry (Before</td>
<td>0.57</td>
</tr>
</tbody>
</table>

Table 7-9: Pavia Model: Pier Geometry and Top Load Data

<table>
<thead>
<tr>
<th>Pier ID</th>
<th>P1</th>
<th>P2</th>
</tr>
</thead>
<tbody>
<tr>
<td>$n_p$</td>
<td>No. of Piers</td>
<td>2</td>
</tr>
<tr>
<td>$L_p$</td>
<td>Length of Pier, inch (mm)</td>
<td>45.25 (1150)</td>
</tr>
<tr>
<td></td>
<td></td>
<td>71.75 (1822)</td>
</tr>
<tr>
<td>$H_p$</td>
<td>Height of Pier, inch (mm)</td>
<td>84.50 (1232)</td>
</tr>
<tr>
<td></td>
<td></td>
<td>84.50 (1232)</td>
</tr>
<tr>
<td>$\psi$</td>
<td>Boundary Condition Factor</td>
<td>0.90</td>
</tr>
<tr>
<td>$P_T$</td>
<td>Load on Pier Top, kips (KN)</td>
<td>25.2 (112.2)</td>
</tr>
<tr>
<td></td>
<td></td>
<td>43.0 (191.4)</td>
</tr>
</tbody>
</table>

The force-deformation parameters of the individual wall element, obtained from the analysis of the model with the proposed methodology are given in Table 7-10. The total capacity of the building is obtained by superposing the results of individual wall elements. The force-deformation curve thus obtained is compared with the experimental force-deformation envelope curve in Figure 7-11. It is noted that the proposed model accurately predict the behaviour of the model. Although initial stiffness of the experimental curve is on higher side, the effective stiffness of the experimental curve is comparable with the stiffness of numerical curve.

Table 7-10: Force-Deformation Parameters of each Pier: After Retrofit

<table>
<thead>
<tr>
<th>Pier ID</th>
<th>P1</th>
<th>P2</th>
</tr>
</thead>
<tbody>
<tr>
<td>$V_m$</td>
<td>Lateral Capacity, kips (KN)</td>
<td>6.94 (30.9)</td>
</tr>
<tr>
<td></td>
<td></td>
<td>18.67 (83.1)</td>
</tr>
<tr>
<td>$V_{cr}$</td>
<td>Flexural Cracking Capacity, kips (KN)</td>
<td>2.50 (11.1)</td>
</tr>
<tr>
<td></td>
<td></td>
<td>6.76 (30.1)</td>
</tr>
<tr>
<td>$\Delta_{cr}$</td>
<td>Flexural Cracking Displacement, inch (mm)</td>
<td>0.098 (2.5)</td>
</tr>
<tr>
<td></td>
<td></td>
<td>0.087 (2.2)</td>
</tr>
<tr>
<td>$\Delta_y$</td>
<td>Wall Yielding Displacement, inch (mm)</td>
<td>0.273 (6.9)</td>
</tr>
<tr>
<td></td>
<td></td>
<td>0.241 (6.1)</td>
</tr>
<tr>
<td>$\Delta_u$</td>
<td>Wall Ultimate Displacement, inch (mm)</td>
<td>1.027 (26.1)</td>
</tr>
<tr>
<td></td>
<td></td>
<td>1.006 (25.6)</td>
</tr>
<tr>
<td>$K_w$</td>
<td>Elastic Stiffness of Wall kips/inch (KN/mm)</td>
<td>25.42 (4.45)</td>
</tr>
<tr>
<td></td>
<td></td>
<td>77.60 (13.60)</td>
</tr>
<tr>
<td>Failure</td>
<td>Failure Mode, Shear, Flexural</td>
<td>Flexural</td>
</tr>
<tr>
<td>$K_{eff}/K_p$</td>
<td>Ratio of Effective &amp; Elastic Stiffness</td>
<td>1.00</td>
</tr>
<tr>
<td></td>
<td></td>
<td>1.00</td>
</tr>
</tbody>
</table>
7.8 Assessment of a Typical Single Storey Residential Building

To assess the improvement in the performance of a building retrofitted with reinforced plaster and grout injection, a typical single storey building [AN-09] is selected. The plan view of the building is shown in Figure 7-12 in which various piers contributing towards the capacity in short direction are marked in hatch. The height of the building is 132" (3353 mm). The window sill level is 30" (762 mm) high and the door and window top is 84" (2134 mm) high. The thickness of wall is 9" (229 mm). Reinforced concrete slab is 6" (152 mm) thick. A total load of 75 psf (3.59 KPa) is assumed to be applied on slab.

The mechanical and physical properties of masonry materials are selected to be representative of typical brick masonry buildings in Peshawar (Northern Pakistan), Table 7-11. The properties of retrofitting materials, i.e. steel wire mesh and plaster coating of Table 7-4 were used in the analysis.

![Figure 7-12: Plan: Typical Single Storey Brick Masonry Building](image)

![Figure 7-11: Pavia Model: Comparison of Numerical and Experimental Curves](image)

Table 7-11: Mechanical and Physical Properties of Typical Masonry Materials
| $f_k$ | Masonry Compressive Strength (Before Retrofit), psi (MPa) | 720 (4.96) |
| $f_{tu}$ | Masonry Diagonal Tensile Strength (Before Retrofit), psi (MPa) | 35 (0.24) |
| $f_t$ | Bed Joint Tensile Strength (Before Retrofit), psi (MPa) | 0.0 |
| $E_m$ | Elastic Modulus of Masonry (Before Retrofit), ksi (MPa) | 200 (1379) |
| $G_m$ | Shear Modulus of Masonry (Before Retrofit), ksi (MPa) | 80 (552) |
| $\mu$ | Coefficient of Friction of Masonry (Before Retrofit) | 0.60 |
| $c$ | Cohesion in Masonry (Before Retrofit), psi (MPa) | 35 (0.24) |
| $\gamma_m$ | Specific Weight of Material, pcf (KN/m$^3$) | 95.0 (14.93) |

Table 7-12: Pier Geometry and Load Data of Typical Single Storey Building

<table>
<thead>
<tr>
<th>Pier ID</th>
<th>P1</th>
<th>P2</th>
<th>P3</th>
<th>P4</th>
<th>P5</th>
<th>P6</th>
</tr>
</thead>
<tbody>
<tr>
<td>$n_p$</td>
<td>No. of Piers</td>
<td>3</td>
<td>1</td>
<td>1</td>
<td>2</td>
<td>1</td>
</tr>
<tr>
<td>$L_p$</td>
<td>Length of Pier, inch</td>
<td>97.5</td>
<td>97.5</td>
<td>90.0</td>
<td>51.0</td>
<td>33.0</td>
</tr>
<tr>
<td>$H_p$</td>
<td>Height of Pier, inch</td>
<td>84.0</td>
<td>84.0</td>
<td>54.0</td>
<td>54.0</td>
<td>54.0</td>
</tr>
<tr>
<td>$\psi$</td>
<td>Boundary Condition Factor</td>
<td>0.5</td>
<td>0.5</td>
<td>0.5</td>
<td>0.5</td>
<td>0.5</td>
</tr>
<tr>
<td>$P_T$</td>
<td>Load on Pier Top, lbs</td>
<td>13825</td>
<td>7900</td>
<td>6000</td>
<td>5000</td>
<td>4250</td>
</tr>
</tbody>
</table>

A total of nine piers are identified in the shorter direction of building. Some of the piers are similar to each other in all respect, such that six different piers are identified. The geometric dimensions of all piers, their boundary conditions and the total vertical load on their top are given in Table 7-12.

![Figure 7-13: Typical Building: Comparison of force-deformation curve before and After Retrofitting](image)

The empirical force-deformation curves of the building before and after retrofitting with reinforced plaster and grout injection are shown in Figure 7-13. The lateral strength of the retrofitted building is about three times that of non-retrofitted building. The reason behind such a high increase in strength can be best explained with the help of Figure 7-14 where a graph is drawn between vertical stress (in dimensionless form) and increase in lateral strength
of retrofitted pier expressed as percentage of lateral strength of non-retrofitted pier. It is clear from the figure that the percentage increase is very high at low vertical stress and low at high vertical stress. Also the percentage increase is affected by the shear ratio of pier. At low shear ratio the increase is high and vice versa. In case of single storey building the stresses usually varies between 10-30 psi at which a high increase in lateral strength is expected.

It is also obvious from Figure 7-13 that the deformation capacity of the retrofitted building is less than the non-retrofitted building. The overall performance of the building before and after retrofitting may be compared in terms the ground acceleration corresponding to various performance levels, Figure 7-15. The performance of the retrofitted building is found to be much better than the non-retrofitted building.

![Percentage Increase vs Vertical Stress](image1)

**Figure 7-14: Variation of Percentage Increase in Lateral Strength with Increase in Vertical Stress**

![Performance Levels](image2)

**Figure 7-15: Typical Building: Comparison of Performance Levels**

### 7.9 Parametric Study Based on the Proposed Model

Various parameters contributing towards the capacity of a retrofitted building include geometry of pier, material properties, vertical stress, wire mesh reinforcement ratio, plaster coating thickness, etc. A parametric study is
presented in the following paragraph to evaluate the effect of various parameters (vertical stress, reinforcement ratio) defining the capacity of a pier calculated based on the proposed model. It must be mentioned here that the curves presented below are drawn for particular cases (material and geometry) to show the general trend and shall not be used for general cases.

7.9.1 Effect of Vertical Stress

Figure 7-16 shows variation in lateral strength of a pier with increasing vertical stress. It shows that the difference between lateral strength of pier before and after retrofitting is almost constant. At very low vertical stresses the difference is found to be increasing with vertical stress which then becomes constant at higher stresses.

From this effect it may be concluded that the proposed retrofitting technique is very effective, in terms of percentage increase in lateral strength, at low level of vertical stresses, Figure 7-14. At high vertical stress a higher reinforcement ratio may be required to sufficiently enhance the capacity of a pier.

![Graph showing the variation of lateral strength with vertical stress](image)

Figure 7-16: Variation of Lateral Strength of Pier with Vertical Stress (right) and Difference in Lateral Strength Before and After Retrofitting (left)

7.9.2 Effect of Wire Mesh Reinforcement Ratio

The lateral strength is increasing almost linearly with increase in the reinforcement ratio, Figure 7-17. Low reinforcement ratio is effective at low level of vertical stress. At high level of vertical stress high reinforcement ratio are required.
The compressive strength of plaster coating has a negligible direct role towards the capacity of retrofitted pier, Figure 7-18. The aspect, which is important in plaster coatings, is bond between wall and coating which is necessary to transfer shear between masonry and wire mesh. Weak mortar may not be able to establish a proper bond and therefore shall be avoided. A minimum of 1000 psi (6.90 MPa) is recommended as the compressive strength of plaster coating, based on the experimental work done as part of this study.

Another thing which is important is the thickness of plaster coating. Theoretically, increasing the thickness of plaster coating shall cause increase in the lateral capacity. It is true for a certain limit, depending upon the reinforcement ratio, beyond which premature failure of the plaster may happen.
8. DESIGN AND APPLICATION GUIDELINES

8.1 Introduction
This chapter presents guidelines for the design of an unreinforced masonry building retrofitted with ferrocement overlay and execution of the technique at the site. The design of a retrofitted building starts with the determination of material properties, existing condition assessment and evaluation of seismic damages. Seismic performance of the building is restored through performance restoration measures, e.g. grout injection and replacement of crushed bricks. If the capacity of the restored building is found less than the demand, the performance enhancement measures are, then, designed.

8.2 Design of a Retrofitted URM Building
The methodology proposed for the designed of unreinforced masonry building retrofitted with ferrocement overlay is based on the assumption of strong spandrel and weak piers behavior. It is assumed that the damages are mostly confined to the flexural and shear behavior of piers. However, it is the understanding of the author that the application of ferrocement overlay, when applied over the whole wall surface, will also enhance the capacity of building against other damage mechanism like opening corner failure, flexural cracking of spandrel, etc. The method is equally applicable to the retrofitting of a damaged building or to the strengthening of undamaged building. Design of retrofitting/strengthening scheme with ferrocement overlay goes through a number of steps detailed as follows.

8.2.1 Properties of Existing Masonry Materials
Almost all of the unreinforced masonry buildings in Pakistan are non-engineered, constructed without specifying the mechanical properties in the construction documents. The existing material properties of masonry may, therefore, be evaluated through field and/or laboratory test or some minimum values may be assumed based on the condition of the building. The following material properties are required for the performance evaluation of existing buildings:

- Compressive Strength of Masonry, $f_m$ (ASTM E-447) or Flatjack test (ASTM C1196-03)
- Diagonal Tensile Strength of Masonry, $f_{tu}$ (ASTM E-519-02)
- Modulus of Elasticity of masonry, $E_m$ (ASTM E-447)
• Modulus of Rigidity of Masonry, $G_m$ (ASTM E-519-02)
• Coefficient of Internal Friction of Masonry, $\mu$ (EN-1052-03)
• Bed Joint Tensile Strength, $f_t$ (ASTM C1072-00)

The material properties shall be obtained using the designated ASTM specification. Alternatively the compressive strength and the bed joint tensile strength of masonry may be determined using the condition assessment of the masonry described in the coming sections. Other properties may be calculated based on the relations below:

\[
\begin{align*}
  f_{tu} &= 0.04 f_m \\
  E_m &= 300 f_m \\
  G_m &= 0.4 E_m
\end{align*}
\]

8.2.2 Properties of Repair/Retrofitting Materials

The compressive strength of coating plaster ($f_c$) shall be based on the 28-days compressive strength of 2" (50.8 mm) mortar cube.

The tensile strength of plaster coating shall be based on either the split cylinder test (2" in diameter and 4" long) or shall be taken equal to the modulus of rupture strength calculated from the following expression, ACI-318 (ACI-2005):

\[
f_{cr} = 6\sqrt{f_c}
\]

The modulus of elasticity of coating material ($E_c$) shall be calculated from the expression (NIST-97):

\[
E_c = 750 f_c
\]

The welded wire mesh shall be corrosion resistant (galvanized). The tensile strength of steel wire mesh ($f_y$) shall be determined from the tensile strength test of wires extracted from the wire mesh.

Injection grout shall consist of 10 parts of Portland cement, 1 part of slacked lime and 0.05 parts of Ultra Expansion agent prepared with a water-cement ratio of 0.9

8.2.3 Condition Assessment

A procedure, similar to the one outlined in ASCE standard for seismic rehabilitation of existing buildings (ASCE/SEI 41-06), shall be used for the condition assessment of existing masonry. Masonry is classified in to three categories, good, fair and poor.

8.2.3.1 Good Condition Masonry

Masonry is said to be in good condition if there is no visual sign of deterioration, erosion and cracks in both bricks and mortar, further the bricks and mortar cannot be easily scratched off with a sharp tool. The masonry shall be laid in proper English bond (stretcher bond in case 4.5" thick wall) with little continuous vertical mortar joints (less than 5% of the total). The brick unit lapping in two adjacent courses shall be equal
to or greater than quarter brick length. The compressive strength and bed joint tensile strength of good masonry shall be taken as 600 psi and 10 psi respectively.

8.2.3.2 Fair Condition Masonry
Fair masonry on the other hand is the one made with relatively weak bricks laid in rather weak mortar which can be scratched off with a sharp tool but not with a finger. Continuous vertical joints shall be less than 15%. The compressive strength of fair masonry shall be taken as two-thirds of compressive strength of good masonry, i.e. 400 psi. The bed joint tensile strength shall be neglected.

8.2.3.3 Poor Condition Masonry
Any masonry other than the above two classes shall be categorised as poor masonry. The compressive strength of poor masonry shall be taken as one-third of compressive strength of good masonry, i.e. 200 psi. The bed joint tensile strength shall be neglected.

8.2.4 Damage Evaluation
Before going through the design of a retrofitted URM building, the level of seismic damages must be evaluated. If the building is found deficient in terms of demand to capacity ratio in comparison with pre-damaged building, the pre-damage state must be restored before being subjected to retrofitting with ferrocement overlay.

![Force Deformation Curve: Pre-damaged and Post-damaged States](image)

The procedure given in FEMA-306 may be used for the performance evaluation of a damage building. In this procedure the component force-deformation curves are modified according to the severity of damages produced in the components, Figure 8-1. The global capacity curve of the building may then be obtained from the superposition of capacity curve of the components. Various performance levels may be associated with deformation levels in the damage building. The displacement at any performance level may then be transformed to the corresponding ground acceleration.

Two types of behavioural modes are considered for the damage evaluation suffered by a pier during an earthquake; rocking followed by toe crushing or out-of-plane sliding (walking), and shear sliding along horizontal bed
joint or stair-stepped crack along bed and head joints. Third type of failure mode, which is not very common under low vertical stresses, the development of diagonal tension crack passing through masonry units is not considered in the performance evaluation.

Similar to FEMA-306, each of the two behavioural modes is divided in three damage state based on the severity of damages; insignificant, moderate and heavy, as outlined in Figure 8-2 and Figure 8-3. The severity of damages are detailed in terms of the intensity and width of cracks, Table 8-1.

Based on the experimental work done as part of this study and by M. Javed [MJ-08], performance modification factors are defined for each of the damage levels as tabulated in Table 8-2. The factors given in the table are slightly different from those given in FEMA-306, which were very conservative.

<table>
<thead>
<tr>
<th>Damage Level</th>
<th>Behavioural Mode Rocking Critical</th>
<th>Behavioural Mode Shear Critical</th>
</tr>
</thead>
<tbody>
<tr>
<td>Insignificant</td>
<td>Rocking cracks produced at top and bottom of pier</td>
<td>Stair-steeped or horizontal hairline crack</td>
</tr>
<tr>
<td>Moderate</td>
<td>Initiation of few cracks passing through units in toe region</td>
<td>When the head joint opened by approximately 1/4”</td>
</tr>
<tr>
<td>Heavy</td>
<td>Brick Crushing at the toe and/or out-of-plane sliding (walking) of the pier</td>
<td>When the head joint opened by approximately 1/2”</td>
</tr>
</tbody>
</table>

Table 8-2: Performance Modification Factors for Damaged Piers

<table>
<thead>
<tr>
<th>Damage Level</th>
<th>Rocking Critical Piers</th>
<th>Shear Critical Piers</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>$\lambda_k$</td>
<td>$\lambda_v$</td>
</tr>
<tr>
<td>Insignificant</td>
<td>0.9</td>
<td>1.0</td>
</tr>
<tr>
<td>Moderate</td>
<td>0.7</td>
<td>0.9</td>
</tr>
<tr>
<td>Heavy</td>
<td>0.5</td>
<td>0.8</td>
</tr>
</tbody>
</table>

Figure 8-2: Damage Levels of a Rocking Critical Pier
It is important to note that besides change in the future capacity of the building with damages, the demand may also change due to variation in the period of the structure.

8.2.5 Performance Restoration Measures

Before going to enhance the performance of a damaged unreinforced masonry building, its pre-damaged state shall be restored through certain measures called performance restoration measures. Replacement of the crushed bricks (if any) and application of grout injection are the two recommended performance restoration measures. Cement-based grout injection developed as part of this study is found to not only restore the performance of a pier but also to cause an increase in its strength and stiffness. Deformation capacity, however, is slightly decreased.

The in-practice masonry of northern Pakistan is very porous with partially filled bed and head joint. Grout injection fills those voids and improves the mechanical properties of masonry not only in the damaged but also in undamaged masonry buildings. The increase in mechanical properties of masonry is found to be a function of its existing mechanical properties. If the existing condition is very poor, then the increase in mechanical properties may be very significant. It is, therefore, recommended to apply same grout injection irrespective of the severity of damages produced in the piers.

For the analysis of a retrofitted building, in the absence of any experimental evidence, it is recommended to assume that the grout injection just restores the mechanical properties of masonry to its pre-damaged state ignoring any increase in them.

8.2.6 Strength, Stiffness and Behavioural Mode of Masonry Piers

The lateral load corresponding to flexural cracking \( (V_{cr}) \) in a retrofitted pier shall be calculated based on the tensile strength \( (f_{cr}) \) of the coating material using the following expression:

\[
V_{cr} = \frac{f_{cr} t_c}{12 \mu H_p \left( \frac{E_m t_c}{E_c t_c} + 1 \left( \frac{3L_p - 2c}{L_p - c} \right) c^2 + \left( L_p - c \right) L_p + 2c \right)}.
\]
Where,
\[
c = \frac{E_c}{E_m} \left\{ -\left( L_p t_c + \frac{P}{f_{cr}} \right) + \sqrt{\left( L_p t_c + \frac{P}{f_{cr}} \right)^2 + 2 \left( E_m t_c \left( \frac{L_p^2 t_c}{2} + \frac{PL_p}{f_{cr}} \right) \right)} \right\}
\]

For non-retrofitted pier the following equation shall be used:
\[
V_{cr} = \frac{pL_p^2 t}{6\psi H_p}
\]

The elastic stiffness of both retrofitted and non-retrofitted pier shall be calculated from the expression:
\[
k_p = \frac{1}{\left( \frac{H_p^2 (3\psi - 1)}{6E_m I_g} + \frac{H_p}{A_v G_m} \right)}
\]

In case of retrofitted pier $I_g$ and $A_v$ shall be based on the transformed thickness of the pier and $A_v$ shall be taken equal to 0.83 times the gross area of the pier.

The effective stiffness, $k_{eff}$ shall be taken as $1.0k_p$ and $0.9k_p$ for rocking-critical and shear-critical piers respectively.

The Lateral strength, ($V_m$) of masonry pier shall be taken as minimum of the $V_{tc}$, $V_{sl}$ and $V_{dc}$ calculated as:
\[
V_{tc} = \frac{L_p^2 t \left( p + f_s \rho_s \right)}{2\psi H_p} \left( 1 - \frac{p + f_s \rho_s}{0.85f_m + 0.85f_c t_c + f_s \rho_s} \right)
\]
\[
V_{sl} = L_p t (\mu_m p) + 1.8 \sqrt{\mu_m L_p t_c} + 0.9 \rho_s f_s L_p t
\]
\[
V_{dc} = \frac{f_m L_p t}{b} \sqrt{1 + \frac{p}{f_m} + 1.8 \sqrt{f_c L_p t_c} + 0.9 \rho_s f_s L_p t}
\]

If $V_{tc}$ is less than $V_{sl}$ and $V_{dc}$, the mode shall be classified as flexural mode; otherwise the mode shall be classified as shear mode.

### 8.2.7 Deformation Capacities of Masonry Piers

The yield displacement ($\Delta_y$) of a pier shall be calculated by dividing its lateral strength, $V_m$ with the effective stiffness, $k_{eff}$.

\[
\Delta_y = \frac{V_m}{k_{eff}}
\]

The ultimate displacement ($\Delta_u$) of a rocking critical (flexural mode) pier shall be taken as:
\[
\Delta_{uc} = \frac{0.0035}{6c} H_p^2 \left( 3 - \frac{1}{\psi} \right)
\]
Where, \( c = 1.25 \left( p + f_y \rho_s \right) \left( 0.85 f_u + 0.85 f_c \frac{t}{l} + f_y \rho_s \right) \)

The ultimate displacement (\( \Delta_u \)) in flexural mode shall not be greater than 0.010\( H_p \) in non-retrofitted pier and 0.006\( H_p \) in a retrofitted pier.

In case of shear critical pier (shear mode) the ultimate displacement (\( \Delta_u \)) shall be taken as 0.006\( H_p \) for non-retrofitted pier and 0.004\( H_p \) for retrofitted piers.

### 8.2.8 Force-Deformation Curve of Masonry Building

The force-deformation curve of a masonry building shall be obtained from a non-linear model, with non-linearity concentrated in the piers and is defined by the force-deformation curves of piers.

Alternatively, the force-deformation curve of a building in a certain direction may be obtained by superposing the force-deformation curves of all piers in that direction. Before superposition, the displacement at the top of pier shall be transformed to the corresponding displacement at the roof level using the following relations:

\[
\Delta_{\text{crw}} = \Delta_c \frac{H_w}{H_p} \quad 8-11
\]

\[
\Delta_{yw} = \Delta_y \frac{H_w}{H_p} \quad 8-12
\]

\[
\Delta_{uw} = \left( 1 + \frac{H_p}{H_w} \left( \frac{\Delta_u}{\Delta_y} - 1 \right) \right) \Delta_{yw} \quad 8-13
\]

### 8.2.9 Ground Acceleration Corresponding to Various Performance Levels

The displacement corresponding to various performance levels shall be determined based on the criteria set forth in Table 8-3:

<table>
<thead>
<tr>
<th>Performance Level</th>
<th>Displacement Criteria</th>
</tr>
</thead>
<tbody>
<tr>
<td>Operational Performance Level (O)</td>
<td>Average of the displacements corresponding flexural cracking of piers</td>
</tr>
<tr>
<td>Immediate Occupancy Performance Level (IO)</td>
<td>Displacement corresponding to a pier drift ratio of 0.1% in the case when at least one pier fails in shear and 0.15% when all the piers behave in rocking mode</td>
</tr>
<tr>
<td>Life safety Performance Level (LS)</td>
<td>Displacement equal to 75% of the displacement corresponding to CP</td>
</tr>
<tr>
<td>Collapse Prevention Performance Level (CP)</td>
<td>Displacement at which the lateral capacity of building drops by 20%</td>
</tr>
</tbody>
</table>

The ground acceleration, \( a_g \) corresponding to the displacement, \( \Delta \) of each performance level shall be calculated from:
\[
\alpha_g = \begin{cases} 
\frac{(2\pi f_E)^2}{2.5\Gamma} \Delta - - - - - - - - (\text{When } \Delta \leq \Delta_y) \\
\frac{(2\pi f_E)^2}{2.5\Gamma} \Delta \sqrt{\frac{2\Delta}{\Delta_y}} - 1 - - - - (\text{When } \Delta > \Delta_y)
\end{cases}
\]

Where, \( f_E = \frac{1}{2\pi} \sqrt{\frac{k_b}{\sum m_i \phi_i}}, \Gamma = \frac{\sum m_i \phi_i}{\sum m_i \phi_i^2}, \Delta_y = \frac{V_b}{k_b}\) and

\(V_b\) and \(k_b\) are the lateral strength and stiffness of building, \(m_i\) and \(\phi_i\) are the mass and mode shape corresponding to a storey-\(i\). The mode shape \(\phi\) has to be normalized with ordinate equal to one at the top floor.

### 8.3 Guidelines for Application of Grout Injection & Ferrocement Overlay

Before application of grout injection and reinforced plaster, loose mortar from the cracked region shall be scratched off and the crushed bricks shall be replaced with new ones in a rich mortar.

#### 8.3.1 Guidelines for Application of Grout Injection

- The injection ports also called injection nozzles shall have a diameter of 3/8” to 1/2” (10-12.5 mm) and length of 3” to 4” (75-100 mm). Threads for the stopper shall be provided at one end of the injection ports.
- Injection ports shall be fixed in predrilled holes with a strong cement-sand (1:2) or a fast binding ready-to-mix mortar.
- Holes shall be drilled in mortar joints along the crack. The depth of drilled holes shall be greater than half of the wall thickness in the case of one brick wall and two-third of the wall thickness in case of one and half brick wall.
- In case of a continuous crack, the distance between the injection ports shall not be greater than 12” (300 mm) for cracks with thickness exceeding 1/4” (6 mm) and 6” (150 mm) for cracks below 1/8” (3 mm) thickness. In the case of parallel cracks the distance between ports shall be decreased as required.
- The surface of wall shall be sealed before injecting grout through the ports. Sealing only the cracks may not be sufficient, because the grout may come out of the surrounding area. Injection after covering the whole surface of wall with plaster coating is found to be a best choice. In case, where the wall surface has to be covered with ferrocement overlay, injection ports shall be fixed after connecting wire mesh with the surface of wall.
- Before injecting grout through the ports, water shall be passed through the ports at tap pressure starting from the top port, with the purpose to moistened the masonry, to check the interconnectivity of
ports and to remove dust from the inside of cracks. Surface watering shall also be done 24 hours before injection to make sure that the masonry is saturated but surface dry

- Injection grout shall be prepared just before the injection and shall be continuously agitated during injection. The injection shall be started from the bottom most port and continue in the upward direction. Grout coming out of other ports shall be plugged with stoppers.

- Initially the injection pressure shall be kept about 2 bars. When the flow of grout stops the pressure shall be gradually increase to about 4 bars. The pressure shall be kept applied for a couple of minutes to consolidate the grout.

- Inspection cores of 2” (50 mm) diameter shall be taken, at least one from each component and at location where internal cavities are suspected, to insure proper injection in the field.

8.3.2 Guidelines for Application of Ferrocement Overlay

- Before connecting welded wire mesh with the surface of wall, grooves left after removal of loose mortar shall be filled through surface grouting with 1:4 cement-sand mortar.

- As far as possible reinforced plaster shall be applied on both sides of wall. However, one side reinforced plaster may be used in order to preserve the external aesthetic of the building.

- The thickness of plaster coating shall not be less than the 1/2” (12.6 mm), height of wall divided by 200 and spacing between the connected divided by 32 (NIST-97).

- The compressive strength of coating material shall exceed the compressive strength of masonry on which it is applied and 1000 psi (7.0 MPa), whichever is greater (NIST-97).

- The steel welded wire mesh shall be connected with wall using 1.5” (38 mm) long No.10 screws, 1/16” (1.5 mm) thick steel washers with external diameter equal or greater than to 3/4” (19 mm) and plastic rawal plugs inserted in pre-drilled holes. Steel nails shall be used only if it is insured that hammering will not cause further damage to the masonry.

- Screws or steel nails shall be inserted in bricks, not in mortar joints.

- Screws or nails shall be applied at a rate of 2 per square foot. Additional screws or nails may be required to remove any wrinkles from the mesh which may cause problem during plastering.

- A minimum overlap of 6” (150 mm) shall be provided at the discontinuous end of the mesh. The spacing of screws or nails shall be doubled at the overlap.
• Welded wire mesh shall be applied on the entire surface of wall for maximum effectiveness or at least over all potential zones including, weak piers, pier-spandrel connection, opening corners, etc.

• The gauge of wires in a mesh and the spacing between them shall be such that to provide a reinforcement ratio equal to or greater than 0.05% of the gross area of masonry.
9. COST ANALYSIS

This chapter deals with cost analysis of unreinforced brick masonry building retrofitted or rehabilitated with the proposed cement-based grout injection and ferrocement overlay applied to both sides of walls. First the cost analysis of the existing building is discussed which is then followed by the cost analysis of retrofitting materials.

Cost of rehabilitation is one the most important criteria for the selection of an appropriate rehabilitation technique. Cost of retrofitting or rehabilitation is usually represented as the cost per unit area of the building to be retrofitted or as percentage of the cost of replacement of the building. The total cost of rehabilitation may or may not include the cost of architectural finishes, electrification sanitation, etc. The construction and material cost of existing building is based on the Composite Schedule Rates (CSR) 2008 [CSR-08] with a cost factor of 1.1. The cost of retrofitting materials and the associated labour charges are, however, based on market survey conducted in year 2009 which also includes government tax and overheads according CSR-2008.

9.1 Cost of Existing Building

The typical building, selected in this study, is taken from the building survey made by Amjad Naseer [AN-09] as a part of his PhD work. The typical building is a double storey five Marla house constructed in northern regions of Pakistan. The architectural and structural details of the buildings are given in Figure A1 through Figure A3 of Appendix-A. Non-structural 4.5″ thick walls provided as partition for bath room, dressing room, parapets, etc are omitted. The total covered area of both floors is 1564.5 square feet.

Table 9-1: Cost Analysis of Existing Building

<table>
<thead>
<tr>
<th>S. No.</th>
<th>Description</th>
<th>Unit</th>
<th>Quantity</th>
<th>Unit Rate (Rs)</th>
<th>Amount (Rs)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Excavation in Foundation</td>
<td>cft</td>
<td>1224.4</td>
<td>3.00</td>
<td>3,673.20</td>
</tr>
<tr>
<td>2</td>
<td>PCC 1:4:8 in Foundation</td>
<td>cft</td>
<td>102.0</td>
<td>90.00</td>
<td>9,180.00</td>
</tr>
<tr>
<td>3</td>
<td>PCC 1:3:6 in Foundation</td>
<td>cft</td>
<td>329.5</td>
<td>108.00</td>
<td>35,586.00</td>
</tr>
<tr>
<td>4</td>
<td>PCC 1:2:4 in DPC</td>
<td>cft</td>
<td>31.6</td>
<td>130.00</td>
<td>4,108.00</td>
</tr>
<tr>
<td>5</td>
<td>Brick Masonry</td>
<td>cft</td>
<td>2418.2</td>
<td>115.00</td>
<td>278,093.00</td>
</tr>
<tr>
<td>6</td>
<td>RCC 1:2:4 in Lintels, Beams and Slab</td>
<td>cft</td>
<td>676.6</td>
<td>164.00</td>
<td>110,962.40</td>
</tr>
<tr>
<td>7</td>
<td>Steel Reinforcement</td>
<td>Tons</td>
<td>1.325</td>
<td>46,000</td>
<td>60,950.00</td>
</tr>
<tr>
<td>8</td>
<td>Plaster Coating</td>
<td>sft</td>
<td>7174.6</td>
<td>12.00</td>
<td>86,095.20</td>
</tr>
</tbody>
</table>
The detailed quantity analysis of the building is given Table A1 through A9 of Appendix-A of this document. Table 9-1 shows the quantities of various structural items and the associated unit cost taken from CSR-2008. The total structural cost of the building excluding the non-structural components and finishes is found to be about Rs.588,650.0 which is equivalent to Rs.376.3 per square foot. The cost of non-structural components and architectural finishes is about 1.25 times the cost of structural components. The total cost of building including non-structural components and architectural finishes is, thus, Rs.1,324,460.0 or Rs.846.6 per square foot of covered area.

**9.2 Cost of Retrofitting**

Cost of retrofitting, expressed as cost per unit area, includes the cost of cement based grout injection and ferrocement overlay. The ferrocement overlay is assumed to be made with low gauge fine steel welded wire mesh (0.04"Φ wires spaced at 0.5" on centre) and 3/4" thick plaster coating in 1:4 cement-sand mortar. The wire mesh is connected with a set of 1.5" long screws, steel washer and plastic plug at rate of 2 set per square foot. The grout mix consists of 10 parts of Portland cement, 1 part of lime and Ultra expansion agent at a rate of 250g per 50 Kg of cement.

### Table 9-2: Cost Analysis of Ferrocement Overlay (For 100 Square Feet)

<table>
<thead>
<tr>
<th>S. No.</th>
<th>Description</th>
<th>Unit</th>
<th>Quantity</th>
<th>Unit Rate (Rs)</th>
<th>Amount (Rs)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Dismantling Existing Plaster</td>
<td>sft</td>
<td>100.00</td>
<td>1.5</td>
<td>150.00</td>
</tr>
<tr>
<td>2</td>
<td>Mesh including 15% for Overlap and Wastage</td>
<td>sft</td>
<td>115.00</td>
<td>15.00</td>
<td>1725.00</td>
</tr>
<tr>
<td>3</td>
<td>1.5&quot; Long No.10 Screws with 10% Wastage</td>
<td>No.</td>
<td>220.00</td>
<td>0.70</td>
<td>154.00</td>
</tr>
<tr>
<td>4</td>
<td>1.0&quot; Φ Steel Washer with 20% Wastage</td>
<td>No.</td>
<td>240.00</td>
<td>0.30</td>
<td>72.00</td>
</tr>
<tr>
<td>5</td>
<td>Plastic Plug with 5% Wastage</td>
<td>No.</td>
<td>210.00</td>
<td>0.50</td>
<td>105.00</td>
</tr>
<tr>
<td>6</td>
<td>Labor Charges for Connecting Mesh</td>
<td>sft</td>
<td>100.00</td>
<td>4.00</td>
<td>400.00</td>
</tr>
<tr>
<td>7</td>
<td>Cement used in 3/4&quot; thick Plaster Coating</td>
<td>bags</td>
<td>1.54</td>
<td>350.00</td>
<td>539.00</td>
</tr>
<tr>
<td>8</td>
<td>Sand used in 3/4&quot; Plaster Coating</td>
<td>cft</td>
<td>7.70</td>
<td>10.00</td>
<td>77.00</td>
</tr>
<tr>
<td>9</td>
<td>Labor Charges for 3/4&quot; Plaster Coating</td>
<td>sft</td>
<td>100.00</td>
<td>10.00</td>
<td>800.00</td>
</tr>
<tr>
<td></td>
<td><strong>Total</strong></td>
<td></td>
<td></td>
<td></td>
<td><strong>4,022.00</strong></td>
</tr>
</tbody>
</table>

Details of the cost analysis for ferrocement overlay are given in Table 9-2. The cost of ferrocement overlay comes out to be about Rs.40.0 per square foot. The cost of grout injection depends upon the severity of damages and the internal pores in the existing masonry. Based on the experimental work done about 2 bags of cement are requires per 100 square feet of damaged area in a 9" thick masonry wall. Assuming the damaged area to be 25% of the total area, 0.5 bags per 100 square feet of the wall area is required. After including labor charges and cost of injection ports and stoppers, the unit cost of grout injection results in Rs.5.0 per square foot of masonry wall area. Thus the total cost of ferrocement overlay and grout injection is Rs.45.0 per square foot of masonry wall area.
Total masonry wall area for the selected double storey buildings is 5852 square feet. Assuming the wire mesh is applied on 75% of the total wall area (25% area is deducted for spandrels, etc.), the total cost of retrofitting comes out to be Rs.197,505.0, which is about 33.5.0% of the structural cost (Rs.588,650.0) of building and equivalent to about Rs.125.0 per square foot of the building.

The current value of an existing old building may be less than the value of newly constructed building due to the age effect. However the cost of retrofitting is not compared with existing building value, rather it is compared with the cost of replacement, which also includes the cost of dismantling, removal of debris, etc. The cost of retrofitting as percentage of the cost of replacement will certainly be less than 33.5%. Furthermore, the cost of retrofitting may be cut short by restricting it to the potential weak zones like piers, piers-spandrels connections, building corners, etc. Also the cost may be optimized by distributing the ferrocement overlay according to the demand in each orthogonal direction of the building. Since, buildings usually have larger capacity in the long direction with solid walls, than that in its short direction with perforated walls; one can apply reinforced plaster coating in short direction only. Thus the cost of retrofitting may be optimized to as low as 20% of the cost of replacement.

Based on the above calculations, it may be concluded that the retrofitting of damage unreinforced masonry building with cement-based grout injection and ferrocement overlay is a feasible and low cost solution.
10. SUMMARY, CONCLUSIONS AND RECOMMENDATIONS

10.1 Introduction
The research titled “development of low-cost and efficient retrofitting techniques for unreinforced masonry buildings” was aimed at the experimental evaluation of the effectiveness of some retrofitting techniques for masonry buildings, to select appropriate technique based on its cost, effectiveness and efficiency and to finally develop guidelines for the design and application of the selected technique.

This chapter summarizes the whole research work and provides its important conclusions. In the last some recommendation are made for the future work to study other aspects of the research.

10.2 Summary
Pakistan lies in one of the seismically active region of the world. In past this region has experienced some devastating earthquakes, e.g. 1885 Kashmir earthquake and 1905 Kangra earthquake in the north and 1935 Quetta earthquake and 1945 Makran earthquake in the south of Pakistan. Recently, 2005 Kashmir earthquake of magnitude 7.6 resulted in loss of many lives and properties. More than 400 thousands buildings got damaged or destroyed.

According to a survey made by the department of Civil Engineering, University of Engineering and Technology Peshawar, more than 70% of the buildings in northern areas of Pakistan, are made of unreinforced masonry. Unreinforced masonry buildings are seismically the most vulnerable type of construction showing very poor performance during earthquake shaking. The overall seismic behavior of an unreinforced masonry building is very complex because of variability in the material properties and non-homogeneous nature of the material. In-plane walls (walls in the direction parallel to earthquake shaking), being the stiffest parts of the building, are considered as the lateral load resisting elements. These walls are sometimes provided with openings for doors and windows thus dividing them into piers and spandrels. If spandrels are strong enough to resist shear and flexural stresses, piers are then considered as the weakest elements. The behavior of a pier may be flexural or shear depending upon the material properties, geometry of pier, boundary conditions and vertical stress on pier.

Different techniques are available for the seismic performance restoration and up-gradation of unreinforced masonry buildings around the globe. This study
is focused on repair and retrofitting with cement based grout injection, reinforced plaster also called ferrocement overlay and structural repointing. An intensive and extensive experimental program was devised to study the effectiveness of the above-mentioned retrofitting technique and to develop guidelines for their design and application.

As a first step the effect of above mentioned techniques on the material properties, e.g. compressive strength and diagonal tensile strength, was investigated. The techniques were then applied on isolated piers to study their performance under lateral loads. Piers were tested as cantilever wall before and after retrofitting under quasi-static loading. Very high vertical stress was applied on piers to simulate the shear behavior. To include the effect of openings, spandrels and pier-spandrel interaction and to simulate true boundary conditions two full scale perforated walls have been tested under quasi-static loading. The study was further extended to a full scale room to simulate nearly real field conditions in order to include flange effect and global rocking effect. It is worth mentioning that the tests on piers, walls and room were conducted under static loading. To include dynamic effects in the behavior of retrofitted building, a half scale unreinforced brick masonry model was tested on shake table.

The performance of specimens retrofitted with ferrocement overlay was very encouraging. Though increase in lateral strength of isolated piers, retrofitted with ferrocement overlay, was not significant because of very high vertical stress applied to simulate shear behavior. At low level of vertical stress, however, the performance could have been very high as was shown by the results of full scale walls and full scale rooms tested under low vertical stress and also verified by the proposed model. In the case of full scale walls and room the seismic performance after retrofitting was more than two times of that before retrofitting. The dynamic shake table test showed that the retrofitting technique is also effective in enhancing overall performance, in general and of out-of-plane wall performance, in particular.

Based on the experimental work performed as a part of this study and some other researchers simple analytical procedures were proposed for the damage evaluation procedure and the seismic performance evaluation of unreinforced masonry buildings before and after retrofitting with ferrocement overlay. The model conservatively estimated the capacity of buildings when compared with experimental results because of neglecting flange effect, global rocking, etc.

10.3 Conclusions

Based on the experimental work and the proposed analytical model, following conclusions are made for the unreinforced masonry building repaired and retrofitted with reinforced plaster (ferrocement overlays), structural repointing and cement based grout injection:

10.3.1 Grout Injection

Cement based grout injection was developed based on the three important properties; penetrability, stability and effectiveness. Grout must be fluid enough to penetrate into tiny cracks and internal voids in the masonry. At the same time grout must be stable to avoid bleeding and segregation of the
mix. The penetrability of a mix can be increased by increasing the proportion of water or by adding some plasticizer. The first option is not advisable because it can badly affect the stability of mix. Stability of a mix can be improved by adding some fine material or water retaining agent like lime, pozzolonic material, etc. In this study hydrated lime was used as stabilizing agent. Ultra expansion agent, which also includes plasticizer, was used to increase the penetrability and to reduce the shrinkage effects. Following conclusions are made from the study:

- Cement based injection grout, prepared in a proportion; 10 parts of cement, one part of hydrated lime, Ultra expansion agent at a rate of 0.5-1.0% of cement and 9 parts of water by weight, is found a suitable mix in terms of penetrability, stability and effectiveness in restoring the pre-damaged state of unreinforced masonry in Pakistan.

- Other mixes of injection grout may be used if they pass the flow test and diagonal compression test proposed in this study. The diameter of puddle in a flow test shall be around 8” (200 mm) and the diagonal tensile strength of the specimen made from masonry units and grout shall not be less than 2 times the diagonal tensile strength of existing masonry. In the case of wider cracks (crack width more than 1/4” (6 mm)) fine sand may be added to the grout to increase the stability.

- The proposed grout mix will not only restore the pre-damaged state but also enhance the performance of poor masonry by filling the internal voids and improving its mechanical properties.

- Pakistani masonry is very porous because of partially filled internal joints and porous nature of bricks. Sealing of the cracked region may not be sufficient, in most cases, to stop grout bleeding out of masonry during injection. If the wall has to be plastered later on, then covering the whole surface of wall with plaster before injection is a suitable solution to seal the surface.

- The masonry wall shall be well watered before injection with grout to avoid any early loss of water from the grout absorb by masonry.

- Depending upon the nature of masonry, the injection pressure may vary. For common type of masonry where the damages are of moderate level, the grout shall be injected at an initial pressure of 2.0 bars. The pressure may be increased to 4.0 bars and kept constant for a couple of minutes in order to consolidate the mix. In the case of very fragile masonry the pressure shall be kept less than 1.0 bar.

### 10.3.2 Ferrocement Overlay

Ferrocement overlay is the name given to steel welded wire mesh connected to the surface of wall which is subsequently plastered with cement-sand mortar. The shear between wall surface and plaster is transferred partly through screws, connecting mesh with wall surface, and partly due to bond between plaster and wall surface.

In this study the effect of ferrocement overlay on the seismic performance of unreinforced masonry building was investigated through quasistatic testing of isolated piers, perforated walls and full scale room and dynamic
shake table testing of a half scale model. Surface grouting, injection grouting and reconstruction of the crushed masonry were considered as performance restoration measures in addition to ferrocement overlay. Following are the conclusions drawn from the experimental work:

- Ferrocement overlay is an effective mean in enhancing the seismic performance of low-rise unreinforced masonry buildings damaged during earthquake.

- The compressive strength and diagonal tensile strength of masonry increases 24% and 75% respectively when coated with ferrocement overlay using fine mesh ($\rho_s = 0.054\%$)

- There is no significant change in the equivalent viscous damping, calculated from hysteresis loops, of the retrofitted piers, walls or room when compared with that before retrofitting. Equivalent viscous damping may, however, decrease when the failure mode changes from shear to flexural. In the case of flexural mode the damping remained almost constant while in the case of shear mode the damping increases with progressive damages. In the case of flexural mode, however, the damping increases after the start of crushing at toe.

- The in-plane flexural and shear capacities of a masonry pier are significantly improved by ferrocement overlay because of:
  - Reinforcement
  - additional thickness due to plaster
  - Confinement effect, if applied on both faces of pier

- Application of ferrocement overlay may, some times, change the behavioural mode of pier from shear to a more desirable flexural rocking mode.

- Ferrocement overlay with specified reinforcement ratio and plaster thickness adds a certain amount of capacity which is not significantly affected by the existing capacity and the amount of vertical stress on the wall. The lateral capacity of an unreinforced masonry wall, however, increases almost linearly with increase in the vertical stress.

- The increase in the overall lateral strength of isolated piers tested under a high vertical stress of 140 psi (0.97 MPa) was 20% for fine mesh ($\rho_s = 0.054\%$) and 25% for coarse mesh ($\rho_s = 0.092\%$). Such a low percentage increase in lateral strength was expected because of high vertical stress which made the capacity of existing capacity very high and in turn decreased the percentage increase in lateral strength.

- Ferrocement overlay shall be applied to the whole wall surface or at least to the potential zones for maximum effectiveness. Strengthening only piers may shift failure to initiate from other potential zones like opening corners, spandrels, etc and thus limiting the overall increase in lateral strength.

- There is a significant increase in the lateral stiffness and strength of an unreinforced masonry building retrofitted with ferrocement overlay.
• The seismic performance, in terms of spectral acceleration capacity, of a single and double storey unrenforced building, repaired and retrofitted with grout injection and ferrocement overlay, is almost doubled than the corresponding pre-damaged performance of the building.

• 1.5” (38 mm) long screws or steel nails inserted in bricks (not in mortar joints) at rate of 2 per square foot are enough to transfer shear stress between ferrocement overlay and the wall surface up to a mesh reinforcement ratio of 0.1%. Additional screws or nails may be required to remove any wrinkles from the mesh which may cause problem during plastering.

• Steel nails hammered in bricks and screws tightened in predrilled holes are found equally effective. Steel nails shall, however, be used only when it is insured that the hammering would not damage the fragile masonry.

• An overlap of 6” (150 mm) at the discontinuous ends of the wire mesh is found adequate both vertically and horizontally. However it was observed that the screws applied at rate of 2 per square foot were not sufficient. Screws at rate of 3 to 4 are recommended at the region where overlap is provided.

10.3.3 Structural Repointing
Structural repointing is a technique in which steel bars are placed in grooves made in bed mortar joints of masonry. The grooves are then filled with strong mortar. The effectiveness of structural repointing increases when applied in combination with grout injection which filled the spaces around the steel bars and thus producing a monolithic structure. This technique was originally developed to control the vertical cracks produced in old masonry due to overloading and/or severe weather conditions. The technique may be used to enhance diagonal shear capacity of piers. The technique is less effective in the case of in-plane flexural and shear sliding along horizontal bed joints. Thus the technique shall be used for local repair and retrofitting of diagonal cracks.

Effect of 1/4” (6 mm) diameter bars embedded in every third bed joint on both faces of the damaged piers was investigated under this research. The grooves were filled with 1:4 cement-sand mortar. The surfaces of piers were coated with a plaster in 1:4 cement sand mortar. Cement based grout was also injected in the wall to fill the cracks and voids in the masonry and around the bars. Following conclusions are made:

• The technique is effective in enhancing the diagonal shear capacity of piers but ineffective in other types of failure modes and shall be used to strengthened a local failure.

• The technique, applied on isolated piers and tested under high pre-compression of 140 psi (0.97 MPa), enhanced the lateral capacity of by about 5 kips (22 KN) in about a pre-damaged capacity of 25 kips (112 KN) which corresponds to a 20% increase. If the pier could have been tested under a lower vertical stress corresponding to single or double
storey building, a high percentage increase would have been expected because of the lower pre-damaged capacity.

- 1/4″ (6 mm) diameter cross ties applied at a distance of 16″ on centre in vertical joints are found enough to connect the steel bars.
- Longitudinal bars shall be bent at the ends to avoid any slippage along the bed joints.

10.3.4 Simple Analytical Model

Based on the experimental results a simple analytical model is proposed for seismic performance evaluation of unreinforced brick masonry buildings before and after retrofitting with ferrocement overlay. The model is based on the assumptions; a. the building is regular, i.e. the torsional effect is negligible, b. damages occur in piers only and the spandrels remain intact, i.e. weak pier-strong spandrel concept, c. vertical stress on piers remain constant d. global rocking and flange effect are negligible. Pakistani unreinforced masonry buildings are nearly regular with deep spandrels. Thus the first two assumptions are justified. The vertical stress on pier at the compression side increases with lateral load but at the same time it decreases on the piers at the tension side. There will be some variation in the local behaviour of the piers but the overall capacity will remain almost unchanged because variation in capacities of piers on tension and compression sides may cancel each other. In the case of perforated wall the chance of global rocking is minimal. The flange effect, however, may cause significant increase in the capacity of a masonry building in which the cross walls are properly connected to each other. Following conclusions are made from the numerical study using the proposed model:

- The proposed model was found in a good agreement when compared experimental results. It conservatively estimates the performance of masonry buildings both before and after retrofitting.
- The lateral strength of a typical Pakistani unreinforced masonry building retrofitted with ferrocement overlay is found three times that before retrofitting using the proposed model. The deformation capacity however decreased. The overall performance expressed in terms of ground acceleration capacity is, however, calculated to be two times the capacity before retrofitting.

10.4 Future Recommendations

The study has a limited scope and is not directly applicable to all type of unreinforced masonry buildings. Following is a list of some recommendations with regard to future extension of this research:

- It is recommended to investigate experimentally the effect of grout injection and ferrocement overlay on the performance of other kinds of masonry, e.g. stone masonry, block masonry, etc.
- The proposed model is compared with a single available experimental result. To validate the model and to get more confidence in its usage, experimental study on full scale walls and buildings is recommended.
• To optimize and economize the technique it is also recommended to investigate the effect of ferrocement overlay applied only on potential regions, e.g. top, bottom and toe regions of a rocking critical pier, middle centre of shear critical pier, opening’s corners, wall intersections, etc.

• The experimental study on isolated piers was carried out under cantilever setup where it is very difficult to simulate shear behavior at working vertical stress. But in the case of double bending (fixed ends) test shear behavior can be obtained at working vertical stress. It is therefore recommended to investigate the effect of ferrocement overlay on the performance of fixed ended isolated piers.

• A numerical study is recommended to evaluate the seismic performance of irregular URM buildings retrofitted with the proposed scheme.

• The proposed retrofitting scheme is verified for low rise unreinforced masonry buildings (one to three storeys). However, for relatively taller buildings (five to six storeys), the performance of retrofitted buildings need to be investigated.
11. REFERENCES


[ACI-05] ACI Committee-318, 2005, “Building Code Requirements for Structural Concrete”, American Concrete Institute, Farmington Hills, Mich


[DT-94] D. Todd. Let’s clean up our language, Editorial, EERI Newsletter, 28 (8), Earthquake Engineering research Institute, Oakland, 1994, P-3.


[EHL-04] M. ElGawady, J. Hegner, P. Lestuzzi, M. Badoux, “Static Cyclic Test of URM Wall before and after Retrofitting with Composites” 13th IB3MaC Amsterdam, July 4-7, 2004


[PTP-08] Catherine G. Papanicolaou, Thanasis C. Triantafillou, Myrto Papathanasiou and Kyriakos Karlos, “Textile reinforced mortar (TRM) versus FRP as strengthening material of URM walls:


Figure A1: Ground Floor and First Floor Plans of a Typical Double Storey Building
Figure A2: Sectional View of Typical Double Storey Building

Figure A3: Lintel Beam, Slab Beam and Foundation Details of Double Storey Masonry Building
### Table A1: Excavation in Foundation

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<th>Height (ft)</th>
<th>Volume (ft³)</th>
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### Table A8: Brick Masonry in First Storey

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Table A7: RCC 1:2:4 in First Floor Lintels, Beams and Slab

<table>
<thead>
<tr>
<th>S. No.</th>
<th>Description</th>
<th>Number</th>
<th>Length (ft)</th>
<th>Width (ft)</th>
<th>Height (ft)</th>
<th>Volume (ft³)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Lintel Beam</td>
<td>1</td>
<td>33.000</td>
<td>0.750</td>
<td>0.500</td>
<td>12.38</td>
</tr>
<tr>
<td>2</td>
<td>Floor Slab</td>
<td>1</td>
<td>34.000</td>
<td>24.000</td>
<td>0.417</td>
<td>340.00</td>
</tr>
<tr>
<td>3</td>
<td>Terrace Deduction</td>
<td>-1</td>
<td>10.000</td>
<td>6.750</td>
<td>0.417</td>
<td>-28.13</td>
</tr>
<tr>
<td>4</td>
<td>Stair Case Deduction</td>
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<td>8.000</td>
<td>5.250</td>
<td>0.417</td>
<td>-17.50</td>
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<tr>
<td></td>
<td><strong>Total:</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td><strong>324.25</strong></td>
</tr>
</tbody>
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