STRENGTH ENHANCEMENT OF BRICK MASONRY COLUMN WITH FERROCEMENT

By

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DEDICATED

TO

MY LOVING FAMILY

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Abstract

Prior to concrete becoming a widely used building material, brick masonry walls and columns were mainly provided in buildings in most parts of the Indo-Pak Sub-Continent. Quite a large number of these old brick masonry buildings have lost strength, which needs to be restored. A research work was carried out to enhance the strength of brick masonry columns through confinement with ferrocement. This configuration acted as a retrofit arrangement in restoring strength of old brick masonry columns. Uniaxial compression tests were performed on three uncoated columns (control specimen), six brick columns coated with unreinforced plaster and fifty one columns coated with layer of ferrocement. Test results revealed that the application of ferrocement coating on bare masonry columns significantly enhances the compressive strength. Ferrocement coated brick columns showed 138 per cent increase in failure load compared to control specimen. The cracking resistance and stable crack growth mechanism is also improved.

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Chapter 1

INTRODUCTION

1.1 GENERAL

Brick masonry construction is in vogue since time immemorial (Singh et al. 1988). In Pakistan we find most of the old structures including some bridges are made of brick masonry columns. Moreover, these brick masonry columns are very common in low and medium rise masonry buildings. Due to its low ductility they are more vulnerable to earthquakes. These structures have also lost their durability as regard to safety due to continuous environmental effects and also fail to meet latest ACI code provisions. Therefore, in specific case of strengthening / retrofitting of columns in masonry structures in historical urban and rural nuclei, a compromise between the requirement of structural engineering and conservation of historic monument should be developed based on the use of traditional building materials (Tomazevic 1999).

In recent years, there has been an increased emphasis on the repair and rehabilitation of all types of structures, in preference to demolition and rebuilding. As a result; there is an increasing demand for repair materials. The selection of repair materials is a predictive effort to maximize future performance of durability. Therefore, selection must be based on the knowledge of the physical and chemical properties and the nature of the environment in which they will be placed.

Several studies have been carried out to increase the life of the distressed buildings by suitable retrofit and strengthening measures. Steel plates have been used with various bonding techniques on the sides of distressed members. Surface treatment is a common method, which has largely developed through experience. Surface treatment incorporates different techniques such as ferrocement, reinforced plaster, and shotcrete. By nature this treatment covers the masonry exterior and affects the architectural or historical appearance of the structure. Grout injection is a popular strengthening technique, as it does not alter the aesthetic and architectural features of the existing buildings (Elgawady et al. 2000).

This research project is aimed to study use of ferrocement around brick masonry columns and its strength enhancement effect.

1.2 PROBLEM STATEMENT

Brick masonry columns are very common in rural and urban areas of Pakistan. Many old masonry buildings, especially in earthquake prone regions without any provision for earthquake loading are a serious hazard. This makes brick masonry structures especially columns, unsafe and requires economical, safe and easy remedial measures.

Ferrocement is a type of thin wall reinforced concrete commonly constructed of hydraulic cement mortar reinforced with closely spaced layers of continuous and relatively small size wire mesh (ACI 549R - 97). In its role as a thin reinforced concrete product and as laminated cement base composite, ferrocement has found itself in numerous applications especially in repair and rehabilitation of existing structures. Ahmed et al. (1994) found that ferrocement can be used effectively for retrofitting of masonry columns.

This research study scrutinize the strengthening of unreinforced brick masonry columns with one layer of ferrocement, using different types of meshes, coated with Ultra Latex as bonding agent. It will make the process of retrofitting effective, economical and easy for application.

1.3 OBJECTIVES

The objectives of this research study are as follows:

- To evaluate the increase in compressive strength with application of ferrocement coating of commonly available wire meshes.
- To evaluate the crack resistance and crack growth mechanism.
- To draw a comparison between unreinforced plaster coating and ferrocement coating.

- To compare performance of ferrocoated columns using different type of meshes.

1.4 SCOPE AND LIMITATIONS

The research study is divided into three parts. The first part deals with masonry columns without ferrocement coating and plaster. In the second part, only cement plaster of 1:5 and 1:2 is applied on the masonry columns. The third part deals with the application of ferrocement coating using different types of wire meshes bonded with Ultra Latex as bonding agent and nails. The following parameters are kept constant in the research study:

- Water cement ratio equal to 0.4.
- Mortar ratio for brick masonry work equal to 1:4.
- Ferrocement mortar ratio equal to 1:2.
- Size of brick masonry column equal to 9 in. x 9 in. x 32 in.

Chapter 2

LITERATURE REVIEW

2.1 FERROCEMENT

Ferrocement consist of closely spaced multiple layers of hardware wire mesh of reinforcement ratio of 3-8 percent completely embedded in a high strength cement mortar layer. The mortar is toweled on through the mesh with covering thickness of 0.04-0.2 inch (1-5 mm). The mechanical properties of ferrocement depend on mesh properties. However, typical mortar mix consists of 1 part cement, 1.5-2 parts sand with approximately 0.4 w/c ratio (Paul and Pama 1978). Moreover, in order to improve the properties of matrix and thus the performance of the composite, such as its cracking behavior and shear resistance, short discontinuous fibers can be added to the matrix (Naaman 1999). In order to reduce the mortar cost, it is possible to use pozzolana cement as compared to Portland cement (Paul and Pama 1978).

Ferrocement is ideal for low cost housing since it is cheep and can be done with unskilled workers. It improves both in plane and out of plane behavior. The mesh helps to continue the masonry units after cracking and thus improves in plane inelastic deformation capacity (Elgawady et al. 2004).

2.2 CONSITUENT MATERIALS OF FERROCEMENT

Ferrocement consists of layers of wire mesh impregnated with a very rich mix (high ratio of cement to sand). A broader definition of ferrocement would include the use of skeletal steel in addition to mesh system, and a cementitious matrix with and without short discontinuous fibers. A brief description of the materials used for ferrocement is given below:

- Cement Mortar Matrix includes portland cement, fine aggregate (sand), water and various admixtures. The materials should satisfy standards similar to those used for quality reinforced concrete construction.
- Skeletal Steel is often used in the form of welded wire fabric, or simply as a grid of steel wires, rods or strands. The use of skeletal steel, when the thickness of ferrocement allows it, can be very cost effective. It acts as a spacer, leading to savings in mesh layers. It also adds significantly to the tensile and punching shear resistance of ferrocement while it does not add very much to the specific surface of reinforcement (important for crack width control), it can contribute significant resistance to bending although less effective because it is generally placed in the middle of the section. The properties of the skeletal reinforcement are typically those of standard reinforcing bars or prestressing strands used in reinforced or prestressed structure.
- Mesh Reinforcement includes square woven or welded meshes, chicken (aviary) wire mesh of hexagonal shape, and expanded metal lath or sheet similar to those used in plaster and stucco applications.

2.3 USE OF FERROCEMENT AS A REHABILITATION MATERIAL

Rehabilitation is defined as an act or process of making possible a compatible use for a property through repair, alterations and additions while preserving those portion or feature which convey its historical, cultural, or architectural values. Ferrocement is an ideal material for rehabilitation and strengthening of structures. It is used as a thin wall liner for rehabilitation. The advantages of ferrocement as a rehabilitation material are as following (Naaman 1999):

- It is made of materials (cements mortar and meshes) which are readily available in most of countries.

- It requires a low level of technology and common labor skills, because it is relatively light weight, it does not requires heavy construction equipment or plants.
- It can be fabricated into any shape.
- It can achieve water proofing property with out providing any surface treatment.
- It is very cost effective.
- It has tensile/flexural strength, fracture properties, toughness, and impact resistance.
- It provides a better cracking behavior.

2.4 PREVIOUS RESEARCH

Nayak and Jain (1988) performed various tests on composite brick masonry structures. The various parameters involved in this composite construction were the thickness of masonry, the thickness of ferrocement layer, the number and type of wire meshes used and type of loading on the composite. For a range of these parameters, test samples in form of columns, beams, wall, slabs and pipe were made with control test specimens of constituent materials such as cement mortar, brick masonry, wire mesh and ferrocement. The strength and stiffness properties were ascertained at various stages of loading to give the performance of composite. Based on test results a theory of bending for composite was established.

Singh et al. (1988) recommended a simple method for strengthening of brick masonry column using ferrocement. Method consisted of wrapping a few layers of steel wire mesh, U nailed or tied around the column. The ferrocement encased columns were tested under axial loads and crack formation, failure mode and failure load were determined. It was concluded that ferrocement was effective in increasing the failure load.

Ahmed et al. (1994) investigated the possibility of using ferrocement as a retrofit material for masonry columns. Uniaxial compression tests were performed on uncoated brick columns, brick columns coated with unreinforced plaster and columns coated with ferrocement. The study demonstrated that the use of ferrocement coating strengthened brick columns significantly. Moreover, the cracking resistance and stable crack growth mechanism of bare masonry columns was improved quite significantly due to provision of ferrocement coating. There was no appreciable increase of load carrying capacity of brick masonry columns due to the application of rich mortar coating.

Oliveira and Hanai (1998) presented a discussion on application of ferrocement coating on masonry walls. Axial compression load test was carried on wall without overlay, undamaged wall with reinforced mortar overlay and damaged wall strengthened with reinforced mortar overlay. The use of resistant overlays of reinforced mortar in structural walls increased the compressive strength and stiffness. The test results also revealed that application of high performance mortar overlay gave masonry structures special characteristics of performance such as waterproofing, flexure strength, impact resistance and vibration.

Elgawady et al. (2004) reviewed and discussed the state-of-art on seismic retrofitting of masonry walls and columns with emphasis on conventional techniques. The techniques discussed were surface treatment (ferrocement, reinforced plaster, and shotcrete), grout and epoxy injection, confining unreinforced masonry using reinforced concrete tie columns, and post tensioning. Each technique was elaborated in a very comprehensive manner. The efficiency, advantages, and disadvantages of each technique were also summarized.

Chapter3

EXPERIMENTAL INVESTIGATION

3.1 MATERIALS

The materials along with specifications, which were used for this experimental program, are summarized below.

3.1.1 Bricks

All the bricks were obtained from Peshawar. Tests were carried according to ASTM C 67 - 03a to determine the compressive strength and percentage of water absorption. Details of compressive strength and percentage of water absorption test are tabulated in Table 3.1 and Table 3.2 (Appendix I).

3.1.2 Fine Aggregate

Dry Kabul river sand was used for mixing all samples. The sieve analysis was determined in accordance with ASTM C 136 - 04.

The results of sieve analysis of fine aggregate as compared with the requirement of ASTM C 33 - 04 are tabulated in Table 3.3 (Appendix I) and particle size distribution illustrated in Fig. 3.1 (Appendix I).

3.1.3 Cement

In this research work locally manufactured OPC (Askari Cement) conforming to ASTM specification C 150 (Type-I) was used. The physical and chemical properties of cement are tabulated in Table 3.4 and Table 3.5 (Appendix I).

3.1.4 Wire Mesh

Commercially available expanded metal and square wire meshes of welded type were used. Meshes were differentiated with respect to type, diameter of wire, and size of opening. The values of yield strength were determined according to ACI 549-R 05 and tabulated in Table 3.6 (Appendix I).

3.1.5 Bonding Agent

Ultra Latex conforming to ACI 503.5R - 92 was used as bonding agent.

3.1.6 Mixing Water

Ordinary tap water from Nowshera was used for the entire experimental work.

3.2 SPECIMEN DESIGNATION

Three types of designations were used in this experimental study. The bare brick masonry columns were designated as control specimen (CM). The brick masonry columns which were plastered with mortar ratio of 1:5 and 1:2 were designated as 1:5CM and 1:2CM. The ferrocoated columns were designated as E0.03D 0.63S. The first letter represents the type of mesh, expanded or square wire mesh (E or W), 0.03D represents the diameter of wire used in the mesh, and 0.63S represents the opening/spacing between wires. This particular designation E0.03D 0.63S represents expanded metal mesh with 0.03in. diameter and 0.63in. opening/spacing between wires.

3.3 EXPERIMENTAL PROGRAM

The masonry columns were prepared under laboratory conditions having mix proportion of 1:4 (cement: sand). Water cement ratio was kept constant as 0.4. The size of the masonry columns was kept as 9 in. x 9 in. x 30 in. (0.2286 mm x 0.2286 mm x 0.7672 mm). All the specimens were moist cured and air dried in laboratory for two and three weeks. The testing was carried out after five weeks of casting of specimens. The details of the experiment matrix are shown in Table 3.7 (Appendix I).

3.4 TEST PROCEDURE

The load was applied in increment of 1 ton until the failure of the specimen occurred. Vertical strains were measured at each load increment by two strains gauges attached to the specimen as shown in Fig. 3.2 (Appendix I). During testing, observations regarding cracking load, failure load, cracking pattern, and stress strain behavior were recorded.

Chapter 4

TEST RESULTS AND ANALYSIS

4.1 TEST RESULTS AND ANAYLSIS OF SPECIMENS

The cracking loads, ultimate loads, stress and strain readings of the specimens tested, are tabulated in Tables 4.1 to 4.80 (Appendix II). The stress-strain plots are graphically illustrated in Fig. 4.1 to 4.20 (Appendix II). Combined stress-strain plot of all specimens is shown in Fig. 4.21 (Appendix II).

4.1.1 Analysis of Specimen CM

All three specimens showed brittle failure. The average failure load came out to be 14.4 ton. Major cracks were vertical, however, few horizontal cracks were also observed. The vertical cracks passed through the joints and bricks, while horizontal cracks were through joints only. The cracks widening and propagation was quite fast after their first appearance. Near failure, the specimen demonstrated out ward bulging on all four faces.

4.1.2 Analysis of Specimen 1:5CM

The behavior of specimens of this group was brittle. The average failure load came out to be 17.9 ton, which is 24 per cent higher than CM group. Higher failure load is probably due to the additional strength provided by the plaster. The cracks widening and propagation was quite fast after their first appearance. Near failure, spalling of cement-sand plaster was observed. Generally, the cracking pattern was similar to CM group.

4.1.3 Analysis of Specimen 1:2CM

As expected, these specimens exhibited fairly good results in terms of failure load. The increase in average failure load was 71 per cent more than the CM and 47 per cent more than specimen group 1:5CM. This substantiates the fact that the mix ratio used for plastering also plays a significant role in strengthening of brick masonry columns. The failure mechanism was similar to that of 1:5CM.

4.1.4 Analysis of Specimen E0.027D 0.87S

The confinement provided by layer of ferrocement increased the average failure load to 19.6 ton, which is 35 per cent higher than CM specimens. However, the average failure load of these specimens was observed to be 36 per cent less than 1:2CM group. This is due to the fact that, specimens reinforced with ferrocement layer, showed premature bond failure. The bond failure occurred at the edges of mesh-column interface.

4.1.5 Analysis of Specimen E0.028D 0.48S

The average failure load of this group came out to be 22.4 ton, which is 55 per cent higher than CM group and 20 per cent higher than E0.027D0.87S group. However, average failure load was still 16 per cent lower than 1:2CM group. This is again due to the fact that, specimens reinforced with ferrocement layer, showed premature bond failure. Failure mechanism resembled to that of E0.027D 0.87S.

4.1.6 Analysis of Specimen E0.02D 0.3S

The average failure load for this group of specimens was recorded to be 19.7 ton, which is 36 per cent higher than the CM group. The increase in average failure load is not significant. This is due to the fact that the diameter of the wire mesh used was less than the minimum prescribed limit of 0.023 in. (ACI 549R - 05).

4.1.7 Analysis of Specimen E0.03D 0.62S

The average failure load was recorded as 27.2 ton, which is 88 per cent higher than that of CM group and 17 per cent higher than 1:2CM. Specimens in this group also showed bond failure.

4.1.8 Analysis of Specimen E0.03D 0.33S

This group exhibited the highest average failure load of 34.4 ton, which is 138 per cent higher than that of CM group. There was no bond failure. Crack propagation after the initiation of first crack was slow and stable.

4.1.9 Analysis of Specimen E0.048D 0.92S

The average failure load of these specimens was found to be 29.03 ton, which is 100 per cent higher than CM group and 38 per cent lower than that of E0.03D 0.33S. It was observed that as the net area of reinforcement increases beyond 0.006 sq in. (E0.03D 0.33S), the failure load decreases and then remains stable. Failure mechanism of this group was observed to be quite similar to that of E0.03D 0.62S group.

4.1.10 Analysis of Specimen E0.05D 0.63S

The average failure load of these specimens was found to be 26.9 ton, which is 86 per cent higher than CM group and 52 per cent lower than that of E0.03D 0.33S. The above statement about the net area of reinforcement is also valid for this case. This also confirms why ACI 549R – 05, puts a limit on the use of diameter of wire mesh, that is, the diameter of wire mesh should not be greater than 0.4 in. All the specimens experienced bond failure. Although the first crack initiated quite late, yet their propagation was rapid.

4.1.11 Analysis of Specimen E0.064D 0.79S

The average failure load of these specimens was found to be 27.9 ton, which is 93 per cent higher than CM group and 45 per cent lower than that of E0.03D 0.33S. Two out of three specimens exhibited loss of bond. In general, the failure mechanism was similar to that of E0.05 0.63S, however, cracking was observed to initiate at an early stage as compare to E0.05 0.63S group.

4.1.12 Analysis of Specimen E0.075D 0.95S

The average failure load of these specimens was found to be 26.9 ton, which is 86 per cent higher than CM group and 52 per cent lower than that of E0.03D 0.33S. All the specimens exhibited bond failure. Crack propagation was slow in the beginning; however, near failure it became rapid.

4.1.13 Analysis of Specimen E0.083D 0.92S

The average failure load of these specimens was found to be 26.9 ton (same as of E0.075D 0.95S), which is 86 per cent higher than CM group and 52 per cent lower than that of E0.03D 0.33S. In comparison to E0.075D 0.95S, minimum cracks were observed at failure and no bond failure occurred, yet the failure load was less. It is thought to be due to the internal damage in the form of micro cracking. Cracks developed vertically, throughout the length of specimens till the complete failure. One of three specimens showed spalling of the cover.

4.1.14 Analysis of Specimen E0.04D 0.62S

The average failure load of these specimens was found to be 22.2 ton, which is 27 per cent higher than CM group and 111 per cent lower than that of E0.03D 0.33S. The reading of failure load of three specimens came out to be 18.4, 22.8 and 25.6 ton respectively. The difference in the reading of failure load of three specimens, from the average reading was found to be 17.4, 2.6 and 13.3 per cent respectively. It is thought that this deviation is the result of faulty construction of the individual specimens; hence the recoded data is considered as biased. Mode of failure was exactly the same as in case of E0.083D 0.92S.

4.1.15 Analysis of Specimen E0.075D 0.68S

The average failure load of these specimens was found to be 25.2 ton, which is 74 per cent higher than CM group and 64 per cent lower than that of E0.03D 0.33S. These specimens failed in similar fashion as of E0.083D 0.92S group.

4.1.16 Analysis of Specimen E0.055D 0.95S

The average failure load of these specimens was found to be 31.8 ton, which is 120 per cent higher than CM group and 18 per cent lower than that of E0.03D 0.33S. This group exhibited bond failure and that is why the strength of this group was lower than E0.03D 0.33S group, however, the ductility of this group was higher. The failure mechanism was observed to be similar to that of E0.083D 0.92S group.

4.1.17 Analysis of Specimen W0.05D 0.49S

This group exhibited the highest average failure load in square mesh group. The average failure load of these specimens was found to be 30 ton, which is 107 per cent higher than CM group. There was no bond failure. Crack propagation after the initiation of first crack was slow and stable.

4.1.18 Analysis of Specimen W0.091D 0.5S

The average failure load of these specimens was found to be 29.5 ton, which is 103 per cent higher than CM group and almost equal to average failure load of W0.05D 0.49S. However, spalling of surface cover was observed to occur at failure, in all three specimens.

4.1.19 Analysis of Specimen W0.095D 0.62S

The average failure load of these specimens was found to be 29.9 ton, which is 106 per cent higher than CM group and almost same as of that of W0.05D 0.49S. Specimens of this group, behaved exactly similar in manner as the specimens of W0.091D 0.5S group. Spalling of surface cover was observed near failure.

4.1.20 Analysis of SpecimenW0.098D 0.59S

The average failure load of these specimens was found to be 28 ton, which is 93 per cent higher than CM group and 9 per cent lower than W0.05D 0.49S. These specimens showed least failure load among its category (square wire mesh). High spalling of outer cover and bond loss was observed in all specimens.

4.2 DISCUSSION ON TEST RESULTS

4.2.1 Effect of Net Area of Reinforcement on Failure Load and Strain

Due to faulty construction, specimens of E0.04D 0.62S group were not considered in the analysis. The readings of net area of reinforcement, cracking load, failure load, percentage increase in failure load with respect to CM and strains are tabulated in Table 4.81 (Appendix II). Fig. 4.22 to Fig. 4.25 (Appendix II) graphically illustrates the effect of net area of reinforcement on failure load and strain.

For expanded wire mesh group, it was observed that initially with the increase in net reinforcement area the failure load increased. However, as the net area of reinforcement increased beyond 0.006 sq in., the failure load decreased to an average value of 29.03 ton, and then onward approximately remained stable.

For square wire mesh group, it was observed that initially with the increase in net reinforcement area the failure load increased. However, as the net area of reinforcement increased beyond 0.085 sq in., the failure load decreased. Strains were also found to remain stable after this value of net area of reinforcement.

4.2.2 Effect of Diameter of Wire Mesh on Failure Load and Strain

Due to faulty construction, specimens of E0.04D 0.62S group were not considered in the analysis. The readings of diameter of wire mesh, cracking load, failure load, percentage increase in failure load with respect to CM and strains are tabulated in Table 4.82 (Appendix II). Fig. 4.26 to Fig. 4.29 (Appendix II) graphically illustrates the effect of diameter of wire mesh on failure load and strain.

For expanded wire mesh group, it was observed that, with the increase in diameter of the wire mesh, the failure load kept on increasing. However, as the diameter of wire mesh increased beyond 0.03 in., the failure load decreased to an average value of 27.2 ton and approximately remained stable.

For square wire mesh group, it was observed that, with the increase in diameter of wire mesh, the failure load approximately kept on increasing. However, as the diameter of wire mesh increased beyond 0.095 in., the failure load decreased. Strains were also found to remain stable after this value of diameter of wire mesh.

4.2.3 Effect of Spacing of Wire Mesh on Failure Load and Strain

Due to faulty construction, specimens of E0.04D 0.62S group were not considered in the analysis. The readings of spacing of wire mesh, cracking load, failure load, percentage increase in failure load with respect to CM and strains are tabulated in Table 4.83 (Appendix II). Fig. 4.30 to Fig. 4.33 (Appendix II) graphically illustrates the effect of spacing of wire mesh on failure load and strain.

For expanded and square wire mesh, no proper relationship was observed between spacing of wire mesh and failure load. Strains were also found to fluctuate with the increase in the spacing of wire mesh.

Chapter 5

CONCLUSIONS AND RECOMMENDATIONS

5.1 CONCLUSIONS

Based on the observations made during the course of this research study following conclusions can be drawn.

- An appreciable increase in load carrying capacity of brick masonry columns due to the application of 2 different ratios of mortar was observed. The brick masonry columns coated with mortar 1:5 showed an increase of 23 per cent where as the specimen coated with rich mortar 1:2 showed 71 per cent increase in failure load.
- The application of ferrocement coating on brick masonry column enhances the compressive strength quite significantly.
- CM, 1:5CM and 1:2CM showed brittle failure. However, no such phenomenon was observed in case of ferrocement coated specimens.
- For expanded wire mesh group, it was observed that, with the increase in net reinforcement area, the failure load kept on increasing. However, as the net area of reinforcement increased beyond 0.006 sq in., the failure load decreased to an average value of 29.03 ton and subsequently remained stable.
- For square wire mesh group, it was observed that, with the increase in net reinforcement area, the failure load approximately kept on increasing. However, as the net area of reinforcement increased beyond 0.085 sq in., the failure load decreased. Strains were also found to remain stable after this value of net area of reinforcement.
- For expanded wire mesh group, it was observed that, with the increase in diameter of the wire mesh, the failure load kept on increasing.

However, as the diameter of wire mesh increased beyond 0.03 in., the failure load decreased to an average value of 27.2 ton and subsequently remained stable. No proper relationship was observed between diameter of wire mesh and strains of expanded wire mesh group.

- For square wire mesh group, it was observed that, with the increase in diameter of wire mesh, the failure load approximately kept on increasing. However, as the diameter of wire mesh increased beyond 0.095 in., the failure load decreased. Strains were also observed to remain stable after this value of wire mesh diameter.
- Out of the two types of the wire meshes, that is, square mesh and expanded metal mesh, later was found to be more economical than the former. In addition, it was relatively difficult to wrap square wire mesh around brick masonry column.

5.2 RECOMMENDATIONS

The research work highlights further work to be done in following areas:

- To investigate the effect of different types of bonding agents on the strength enhancement of brick masonry columns.
- To investigate the combined behavior of compressive and lateral load on ferrocement coated brick masonry columns.
- To develop the finite element model of ferrocement coated brick masonry columns and compare the results with test results obtained in this experimental investigation.
- To investigate the feasibility of strength enhancement of brick walls with ferrocement.

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