EXPERIMENTAL EVALUATION OF THE LATERAL STRENGTH OF MASONRY INFILLED REINFORCED CONCRETE FRAME RETROFITTED WITH FERROCEMENT

A Project
by

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It is hereby declared that this Project or any part of it has not been submitted elsewhere for the award of any degree or diploma.

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ABSTRACT

An experimental study has been performed to investigate the in-plane strength of masonry infilled reinforced concrete frame retrofitted with ferrocement. A half scale masonry infill panel was constructed and tested in the laboratory for this purpose. The load was applied monotonically at the top of the frame till the ultimate capacity was reached accompanied by substantial formation and propagation of cracks. Then both the infill and the frame was repaired by ferrocement coating. After rehabilitation, the frame was tested in the laboratory following the same procedure as for the original frame. The masonry infill frame retrofitted with ferrocement showed significant improvement in performance and more than original strength was achieved. This reveals the potential use of ferrocement as a retrofitting material of existing infilled frame.

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INTRODUCTION

1.1 GENERAL

Reinforced concrete structural components are often found to exhibit distress, even before their service period is over due to several causes. Such unserviceable structures require immediate attention, inquiry into the cause of distress and suitable remedial measure, so as to bring the structure into it's functional use again. In recent years repair and rehabilitation of existing structures have become one of the most challenging problem in civil engineering. In the last few decades several attempts have been made in various countries to study these problems and to increase the life of the structure by suitable strengthening measures.

Defects, failure and general distress in the structure could be the result of the structural deficiency caused by erroneous design, poor workmanship or overloading of the structure. It could be due to impact of explosive loading during the construction or service life of the structures. Another source of damage could be the physical damage due to the misuse of the structure. Also individual member or integrated structural unit may be over stressed and may need elaborate strengthening measures. Moreover, structure may suffer deterioration of concrete as a result of inadequate cover to the reinforcement, presence of chloride or poor quality concrete. Fire damage may also weaken the structure as a whole as well as individual concrete member.

The astronomical increase in the cost of construction has forced the engineers to look for economical and better methods for the repairs of damaged or distressed structures. A wide range of rehabilitation schemes are used for a variety of structures. In recent years, ferrocement is used more and more in the field of rehabilitation and strengthening of structures.

The rehabilitation and strengthening of structural elements applying ferrocement is relatively a new aspect of its use. It is now well established that ferrocement, when used for infrastructure rehabilitation has inherent and unique advantages. Ferrocement posses a degree of toughness, ductility and crack resistance that is considerable greater than that found in other form of concrete construction. All these properties are achieved within a thickness of about 25 mm.

A damaged or distressed structure can be repaired or renovated to a satisfactory level of performance by different methods. One of the cheapest and most widely used methods of strengthening is ferrocement coating in the distressed elements. 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 diameter wire mesh. It is considered to be an extension of reinforced concrete technology with relatively better mechanical properties and durability than ordinary reinforced concrete. Within certain loading limits, it behaves as a homogeneous elastic material and these limits are wider than for normal concrete. The uniform distribution and high surface area to volume ratio of its reinforcement results in better crack arrest mechanism, i.e. the propagation of cracks are arrested resulting in high tensile strength of the materials (ACI Committee 549,1993). Due to this high resistance to crack, ferrocement can be used as a layer of low permeable cover for reinforced concrete. This protective layer may be precast and can be used as a permanent formwork to the concrete element. Ferrocement as a permanent formwork looks like an economically viable alternative. Ferrocement as a permanent formwork not only can replace the conventional timber and steel shuttering but also improves the serviceability of the structural element through better crack resistance and less deterioration of reinforcement due to corrosion in aggressive environment.

1.2 FERROCEMENT-AN OVERVIEW

Ferrocement is a highly versatile form of reinforced concrete, constructed of hydraulic cement mortar reinforced with closely spaced layers of continuous and relatively small diameter wire mesh. The mesh may be made of metallic or other suitable material. Ferrocement primarily differs from conventional reinforced or prestressed concrete by the manner in which the reinforcing elements are dispersed and arranged. In this regard the American Concrete Institute (ACI) Committee 549 put forward the definition of ferrocement as follows:

"Ferrocement is a type of thin wall reinforced concrete construction where usually a hydraulic cement is reinforced with layers of continuous and relatively small diameter mesh. Mesh may be made of metallic or other material."

Ferrocement has gained widespread popularity in the developing nations. It has certain inherent advantages and has been accepted as a suitable technology for developing countries for the following reasons:

(a) Its basic raw materials are readily available in most countries.

(b) It can be fabricated into any desired shape.

(c) The skills for ferrocement construction can be acquired easily.

(d) Heavy plants and machinery are not involved in ferrocement construction.

(e) In case of damage, it can be repaired easily.

(f) Being labour intensive, it is relatively expensive in developing countries.

1.3 OBJECTIVES OF THE PRESENT STUDY

The objectives of the present study are as follows:

(i) To study the performance of ferrocement overlay as a repairing material for distressed masonry infilled RC frame under lateral loading condition.

(ii) Comparative study of the behaviour of masonry infilled RC frame and that of distressed masonry infilled RC frame repaired by ferrocement overlay.

1.4 SCOPE /LIMITATION OF THE STUDY

The present research topic has been under taken to study the behaviour of distressed infilled RC frame retrofitted by ferrocement overlay. To study this effect, one reinforced concrete frame was constructed in the laboratory. The features are pointed below:

(i) One frame was constructed due to the limitation in the project budget.

(ii) The whole experiment was carried out in the extension of BUET concrete laboratory. So, half scale model was constructed due to cater for height limitation.

(iii) Only monotonic load is used. Cyclic loading could not be applied due to the scarcity of proper instruments.

(iv) There are many repairing methods. In this experiment only ferrocement repairing technology is used under lateral loading condition.

Chapter-2

REPAIR OF DAMAGE USING FERROCEMENT

2.1 INTRODUCTION

To study the effect of ferrocement overlay on reinforced concrete elements, first it is necessary to understand the behaviour of ferrocement under different conditions. In this regard, it is necessary to identify the parameters affecting the properties of ferrocement and review relevant literature in this field. This chapter presents a brief literature review on the properties of ferrocement and review relevant literature in this field. Experimental investigations carried out by several researchers on the behaviour of a reinforced concrete beam with ferrocement overlay are also included.

2.2 CONSTITUENTS OF FERROCEMENT

The constituent materials of ferrocement are cement, sand, water and reinforcing mesh.

2.2.1 Cement

The cement is to be an ordinary Portland cement of type 1 and shall be conforming to ASTM standard.

2.2.2 Sand

Sand should be obtained from a reliable source and should comply with ASTM C33 for aggregates. It should be clean, hard, strong, and free of organic impurities and deleterious substances. The fineness of sand should be such that 100% of it passes through standard sieve no. 4(2.36mm)

2.2.3 Water

Water used in the mixing should be free from any organic and harmful solution, which lead to be the deterioration of the properties of mortar. In any cases saline water should not be used. Any water with a pH (degree of acidity) of 6.0 to 8.0 that does not taste saline is suitable for use.

2.2.4 Reinforcing Mesh

The most essential component of ferrocement is steel wire mesh. Different types of wire meshes are available in Bangladesh. Wire mesh generally consists of thin wires either woven or welded in to mesh.

2.3 PROPERTIES OF FERROCEMENT

Ferrocement, considered to be an extension of reinforced concrete technology, has relatively better mechanical properties and durability than ordinary reinforced concrete. Within certain loading limits, it behaves like as a homogeneous elastic material and these limits are wider than for normal concrete. The uniform distribution and better crack arrest mechanism arrests propagation of cracks and results in high tensile strength of materials.

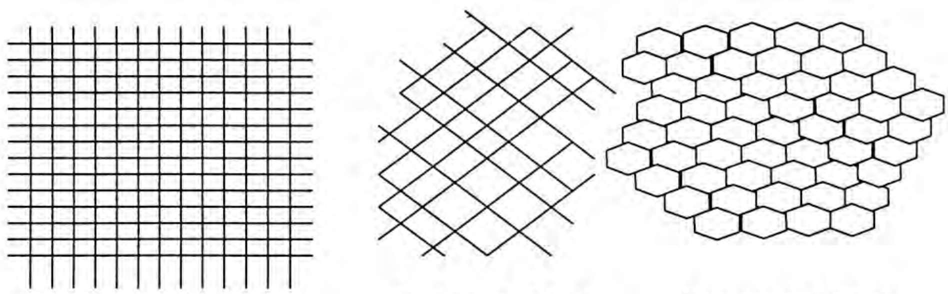
Many of the properties unique to ferrocement derive from the relatively large amount of two-way reinforcement made up of relatively small elements with much higher surface area than conventional reinforcement. In the words of Nervi, who first used the term ferrocement, its most notable characteristic is "greater elasticity and resistance to the cement mortar by the extreme subdivision and distribution of reinforcement". Therefore, the recognition of parameters defining the subdivision and distribution of the reinforcement is fundamental in understanding many of the properties of ferrocement.

2.3.1 Reinforcing Mesh

In 1993 ACI Committee 549 reported that one of the essential components of ferrocement is wire mesh. Different types of wire meshes are available everywhere. Common wire meshes have hexagonal or square openings. Meshes with hexagonal openings are some times referred to as chicken wire mesh or aviary mesh. They are not structurally as efficient as meshes with square openings because the wires are not always oriented in the directions of the principal (maximum) stresses. However, they are very flexible and can be used in doubly curved elements.

Meshes with square openings are available in welded or woven form. Welded wire mesh is made of straight wires in both the longitudinal and transverse direction. Thus welded mesh thickness is equal to two wire diameters. Woven mesh is made of longitudinal wires woven around straight transverse wires.

Depending on the tightness of the weve, woven mesh thickness may be up to three wire diameters. Welded wire meshes have a higher modulus and hence higher stiffness than woven meshes; they lead to smaller crack width in the initial portion of the load- deformation curve.



Square Mesh **Expanded Mesh** **Hexagonal Mesh**

Fig.2.1 Different types of wire mesh used in ferrocement, (ACI Committee 549, 1993).

Woven wire meshes are more flexible and easier to work with than welded meshes. Again, welding anneals the wire and reduces its tensile strength, (ACI Committee 549,1993).

Three parameters, commonly used in characterizing the reinforcement in ferrocement applications are as follows:

- The volume fraction
- The specific surface of reinforcement and
- The effective modulus of the reinforcement

Volume Fraction of Reinforcement

Volume fraction *(V_f)* is the total volume of reinforcement divided by the volume of composite (reinforcement and matrix). For a composite reinforced with meshes with square opening, V_f is equally divided in to V_{fl} and V_{ft} for the longitudinal and transverse directions, respectively. For other types of reinforcement V_{fl} and V_{ft} may be unequal. For ferrocement reinforced with square or rectangular the volume fraction of mesh reinforcement can be calculated from the following relation.

$$V_f = \frac{N\pi \ d_h^2}{4h}\left(\frac{1}{Dl}+\frac{1}{Dt}\right) \qquad 2.1$$

Where, N = number of layer of mesh reinforcement

d_b = diameter of wire mesh

h = thickness of ferrocement

D_l = center to center spacing of wires aligned longitudinally in reinforcing mesh

D_t = center to center spacing of wires aligned transversely in reinforcing mesh

Specific Surface of Reinforcement

Specific surface, S_r is the total bonded area of reinforcement (interface area or area of the steel that comes in contact with the mortar) divided by the volume of composite. S_r is not to be confused with the surface area of reinforcement divided by the volume of reinforcement. For a composite using square meshes, S_r is divided equally in to S_{rl} and S_{rt} in the longitudinal and transverse directions, respectively.

For a ferrocement plate of width b and depth h, the specific surface of reinforcement can be computed from (Rahman, 2002).

$$S_r = \frac{\sum o}{bh} \qquad 2.2$$

In which $\sum 0$ is the total surface area of bonded reinforcement per unit length.

For square and rectangular mesh the expression for S_r will become

$$S_r = \frac{\pi d_b(D_l + D_t)}{D_l D_t h} \qquad 2.3$$

Relation between S_r and V_f

The relation between S_r and V_f when square grid wire meshes are used is

$$S_r = \frac{4V_f}{d_b} \tag{2.4}$$

Where d_b is the diameter of the wire. For other types of wire meshes, S_{rl} and S_{rt} may be unequal,(Rahman,2002).

Effective Modulus of Reinforcement

Rahman (2002) describes that although most ferrocement properties have similar definition as those of reinforced concrete, one property is defined differently. It is the effective modulus of the reinforcing system *Er*. This is because the elastic modulus of the mesh (steel or other) is not necessary the same as the elastic modulus of the filament (wire or other) from which it is made. In a woven steel mesh, weaving imparts an undulating profile to the wires. Hence, the woven mesh behaves as if it has a lower elastic modulus than that of the steel wires from which it is made.

2.3.2 Effective Area of Reinforcement

The area of reinforcement per layer of mesh considered effective to resist tensile stresses in cracked ferrocement section can be determined as follows (Rahman, 2002)

$$A_{si} = \eta V_{fi} A_c \tag{2.5}$$

Where, A_{si} = effective area of reinforcement for mesh layer i

η = global efficiency factor of mesh reinforcement in the loading direction considered.

V_{fi} = volume fraction of reinforcement for mesh layer i

A_c = gross cross sectional area of mortar (concrete) section.

The global efficiency factor when multiplied by the volume fraction of reinforcement, gives the equivalent volume fraction (or equivalent reinforcement ratio) in the loading direction considered. In effect, it leads to an equivalent (effective) area of the reinforcemen per layer of mesh in that loading direction. For square mesh, η is equal to 0.5 when loading is applied in one of the principal directions. For a reinforcing bar loaded along its axis ($\eta = 1$), (Rahman, 2002).

2.4 STRENGTH PROPERTIES

The strength of ferrocement, as in ordinary concrete, is commonly considered as the most valuable property, although in many practical cases other characteristics, such as durability and permeability may in fact be more important. Nevertheless, strength always gives an overall picture of the quality of ferrocement, as strength is directly related to the properties of the hardness of cement paste and reinforcement.

2.4.1 Tensile Strength

The basis of the structural design is the knowledge of the material properties. The tensile characteristics of ferrocement have not yet been fully defined and standardized. In tension the load is essentially independent of specimen thickness because the matrix cracks well before failure and does mot contribute directly to composite strength. Naaman and Shah (1971) have studied the influence of types, sizes and volumes of wire meshes on elastic cracking and ultimate behaviour of ferrocement in uniaxial tension. They observed that the ultimate tensile strength of ferrocement is the same as that of mesh alone while its modulus of elasticity can be predicted from those of mortar and mesh, (Naaman and Shah, 1971, Johnston and Mattar, 1976 and Pama *et al.* 1974). The specific surface of the reinforcement strongly influenced the cracking behaviour of ferrocement. In general, the optimal choice of reinforcement for ferrocement strength in tension depends on whether the loading is essentially uniaxial or significantly biaxial. Expanded metal in its normal orientation is more suitable than other reinforcing meshes for uniaxial loading because a higher proportion of the total steel is effective in the direction of applied stresses, (Johnston and Mattar, 1976). For biaxial loading, square mesh is more effecive because the steel is equally distributed in the two perpendicular directions, although the weakness in the 45-degree direction may govern in this case.

2.4.2 Compressive Strength:

In this mode, unlike tension, the matrix contributes directly to ferrocement strength in proportion to its cross sectional area. Compressive strength of ferrocement (regardless of the amount of mesh reinforcement) seems to be much the same as that of mortar alone. The experimental results (Pama *et al.* 1974) showed that under compression the ultimate strength is lower than that of equivalent pure mortar. The compressive strength at ultimate condition is assumed to be 0.85 f'_c where f'_c is the ultimate compression strength of the mortar. An investigation in to the behaviour of ferrocement specimen in direct compression has been discussed by Rao (1969). Conclusions were drawn with respect to the effect of percentage of reinforcement and the size of reinforcement on the behaviour of ferrocement. Provision of reinforcement in excess of about 2 to 2.5% is uneconomical in ferrocement as the proportional increase in strength is not achieved (ACI committee 549). Smaller diameter wire mesh would be preferable to use as this gives higher elasticity and higher ultimate compressive strength for the same percentage of reinforcement, all other factors remaining essentially the same. When mesh reinforcement is arranged parallel to the applied in one plane only (as opposite to closed peripheral arrangement), no improvement in strength is observed, (Pama *et al.* 1974). The only forms of reinforcement likely to result in significant strength gains in compression are square mesh reinforcement (ACI Committee Report 549) fabricated in closed box or cylindrical arrangements which results in the matrix, thus forcing it to adopt the triaxial stress condition associate with higher strength.

2.4.3 First Cracking Strength of Ferrocement

The term first cracking strength or its equivalent appears frequently in literature on the behaviour of ferrocement under tension and flexure, but its use without qualification is unfortunate because it can be defined in various ways and therefore can mean different things to different people. In a comprehensive discussion of this problem. by Walkus (1975), it is noted that micro cracks are inherent in the mortar matrix even before it is loaded, and that as the micro cracks widen, propagate and progressively join together under load, they are detected by some means, visual or other wise, and termed "first crack". However, in the various Polish and Russian studies, by Walkus (1975) "first cracking is defined by a crack width ranging from 2×10^{-4} in. to a value visible to the naked eye, 1.2 - 3.3 $\times 10^{-3}$ in. (0.03 - 0.1 mm). In other studies, first cracking is defined as the first deviation from linearity of the load elongation function in tension (900-1500 micro strain) or the corresponding deviation of the load-deflection curve in flexure, also as a crack width under flexural loading of 0.003 in (0.075 mm), as the point at which the matrix at the tension face of flexural specimen reaches a strain equal to the cracking strain of the unreinforced matrix, or simply as the first visible crack (Johnston *et al.*

1976). Therefore, it is necessary to perform experimental investigations for accurate prediction of the first cracking stress of ferrocement in direct tension and in flexure.

Research studies have showed that crack width in reinforced concrete structure can be reduced by increasing the bond between the reinforcement and the concrete, by increasing the distribution of the reinforcement and by reducing the thickness of the cover. All these factors are favourable for ferrocement. Crack width is nearly zero at the interface between the steel and the mortar and increases from the interface towards the surface. Therefore the smaller the distance between the interface and the surface of the structure, i.e., the cover, the smaller the crack width. Specific surface and volume of the reinforcement and their influences are studied by several researchers and empirical relation ships between these parameters and the first cracking stress is proposed.

Experimental evidence from the work of Kumar and Sharma (1976) showed that the first crack stress in bending is a linear function of the percentage of steel reinforcement and they suggested an empirical relation,

$$f_{rf} = 284\rho + 483 \text{ psi} \qquad 2.6a$$

$$f_{rf} = 1.96\rho + 3.33 \text{ MPa} \qquad 2.6b$$

Where, ρ = percentage of reinforcement by volume or the volume fraction of the reinforcing mesh.

On the other hand, the crack stress in bending is a linear function of the specific surface and they presented an empirical equation relating these two parameters as follows:

$$f_{rf} = 1600\ S_{ri} + f_m \text{ psi} \qquad 2.7a$$

$$f_{rf} = 280.2\ S_{ri} + f_m \text{ MPa} \qquad 2.7b$$

Where, S_{ri} = the specific surface of the reinforcement in loading direction and f_m = modulus of rupture of plain mortar.

2.5 DEFORMATION CHARACTERISTICS

Following the consideration given to ultimate and cracking strength, it is appropriate to examine the overall loading deformation behavior of ferrocement under various form of loading, in particular its modulus of elasticity, which historically has been identified as one of its major attributes.

2.5.1 Load Deformation Behavior in Tension

For square mesh reinforcements, the load-elongation behavior of reinforcement has been characterized in three stages, (Naaman and Shah, 1971, Pama, *et al.* 1974). In the initial stages the matrix and reinforcement act as a continuum having a composite elastic modulus about equal to that predicted from the volumetric law of mixtures of the longitudinal reinforcement and the matrix, (Naaman and Shah, 1971). The second stage associated with a fully cracked matrix, is also linear. Its modulus is somewhat greater than the product of the volume fraction and the modulus of the longitudinal reinforcement, (Naaman and Shah, 1971 and Pama, *et al.* 1974) thus supporting the view that the, mortar and the lateral reinforcement continue to play an active role after first cracking, either individually or in combination. In the third stage, the matrix ceases to play role. Failure corresponds to the yielding of the reinforcement.

2.5.2 Load Deformation Behavior in Compression

When the reinforcement is in one plane only, it has a minimal effect on the load deformation relationship, and the associated elastic modulus remains virtually the same as that for mortar matrix, (Pama, *et al.* 1974). When present in closed peripheral form, the load-deformation relationship is curvilinear with the initial tangent modulus increasing gradually with the amount of reinforcement. The initial elastic modulus can be predicted quite accurately and conservatively on the basis of the volumetric influence of the two material components acting together. The values of the elastic modulus are slightly higher for specimens reinforced with welded mesh than for their equivalents with expanded metal, (Johnston and Mattar, 1976). The experimental results obtained by various investigators, (Lee and Pama, 1972 and Rao, 1969) show that the modulus of elasticity in direct compression increases proportionately with the increase in steel content.

Studies on mechanical properties of ferrocement have been made since the last decade but studies of formulation of these properties based on fundamental material properties has begun only recently. Some of its mechanical properties have not been sufficiently investigated yet and not enough technical information are available to suggest acceptable formula for design. In some other cases enough research information are available that can be used as assumption to set up a tentative design approach.

2.6 EXPERIMENTAL INVESTIGATION ON REHABILITATION OF STRUCTURES

Research and development work on ferrocement has progressed at a tremendous pace during recent years and a variety of structures using innovative design and construction techniques have been built worldwide. As a result, a large volume of technical information is now available on various aspects of ferrocement design, construction, maintenance and repair. Increasing popularity and growing public acceptance have made it necessary to formulate design and working guidelines by collecting the available information. Efforts have also been made in recent years to improve the performance of reinforced concrete elements by applying ferrocement overlay. The concept has been intuitively applied for repair and strengthening of distressed elements.

Anwar *et al.* (1991) investigated the rehabilitation technique for reinforced concrete structural beam elements using ferrocement. The technique involved strengthening of reinforced concrete beams by application of hexagonal chicken wire mesh and skeletal steel combined with ordinary plastering. The test result is in good compliance with the original design capacity of the beams. From the test result obtained a design chart has been developed to determine the parameters for rehabilitation of the beam elements.

Lub and Wanroji(1988) reported that strengthening of existing beams in reinforced concrete building structures by means of shotcrete ferrocement. It was found that the mesh is fully effective and a monolithic condition of shotcrete layer and original concrete beam attained. The wire mesh was found to act as excellent shear reinforcement.

Rosenthal and Bljuger (1985) studied the flexural behaviour of ferrocement reinforced concrete composite beams in the serviceability and ultimate limit states. The flexural behaviour of four rectangular composite beams made of low strength ferrocement, was compared with four reference beams in the serviceability and ultimate limit states. In doing so, special deformational and crack formation properties of the encasing elements (reinforced with wire meshes) were exploited, resulting in hair cracks which appear in the beams under service load, rather than regular width cracks. Cracking moments of the composite beams were 11% and 13% higher than those of reference beams due to additional flexural tensile strength contributed by the elements. Crack in the composite beams have only reached, at failure, a width of 0.4 mm to 0.5 mm, as compared to twice as much in the reference beams. Composite action between the skin and core components was fully obtained until crack appearance. Beyond that stage and up to failure, a partial separation might have happened, according to somewhat different crack patterns of the reference and composite beams.

Kaushik and Dubey (1994) studied the performance of RC ferrocement composite beams through experimental investigation on RC beam cast on ferrocement and distressed beams rehabilitated by ferrocement jacking. They reported that the increase in ultimate strength compared to RC beams was 44% for composite beams and 39% for rehabilitated beams. This showed that composite beams and rehabilitated beams are capable of performing equally well. Moreover, the ultimate strength and stiffness of RC beam can be significantly increased by strengthening with precast ferrocement plates in the shear failure zone. Therefore, ferrocement can satisfactorily be used as the precast part of the composite in which RC beam is cast.

An experimental investigation was carried by Kadir *et al.* (1997) to study the ultimate load, flexural behavior and mode of failure at collapse of reinforced concrete beams using ferrocement permanent formwork (composite beams). The linkage between the two materials was achieved by placing shear connectors along the length of the beam. Test result showed that the reinforced concrete beams with ferrocement permanent formwork failed by flexure. The composite beam with shear connectors carried about 12% higher load and 10% reserved flexural strength and showed lower deflection when subjected to reinforced concrete beam without shear connectors. The ferrocement formwork with and without shear connectors contributed about 21%-75% and 16%-50% to the flexural strength respectively.

Afsarudding and Hoque (1998) performed an experimental research work on reinforced concrete beams with ferrocement overlay in the concrete laboratory, BUET. They investigated the possibility of using ferrocement as a permanent formwork for reinforced concrete beams. A total of twelve beams were constructed and tested in the investigation. Eight ferrocement beam formworks were made having different sizes. All of them were filled with reinforced concrete. Four reinforced concrete beams and eight reinforced concrete coated with ferrocement formwork contaning single layer wire mesh were cast to compare the behaviour of ferrocement formwork reinforced concrete beam with the normal concrete beam. The study demonstrates that the use of ferrocement as a permanent formwork increase the cracking load and ultimate load of the composite system compared to normal RC beams. The number of cracks and width of cracks have been found to have reduced considerably due to the provision of ferrocement layer used as formwork. From the study it appears that permanent precast ferrocement formwork could become a reliable alternative to wooden formwork in the construction of reinforced concrete beams.

The ability of ferrocement to fit snugly into curved surface make it and ideal material for the rehabilitation of dome and shells. An example of such rehabilitation is the restoration of domes in the Widmill theatre in UK (1988) (Rahman, 2002).

Sharma *et al.* (1984) rehabilitated an overhead circular water tank of 210000-litre capacity using ferrocement. The superior crack resistance properties made it suitable for water retaining satisfactorily. The tank was put out of service due to heavy leakage soon after its construction. The inspection of tank revealed the presence of a large cracked and honeycombed area in the center of tank wall which was all along the wall periphery. At some point only coase aggregate was deposited with no fine aggregate making it the major source of water leakage through the voids in such an area. After repairing by using ferrocement no leakage was observed and the tank seemed to be performing with full efficiency. The rehabilitated tank is currently under service.

Trikha et al. (1988) reported the process of repairing of damaged steel water tank using ferrocement. Steel water storage tank are widely used in every part of the world. One predominant problem associated with steel tank is that of corroison. Due to the corrosion of steel tank have to replaced after an extra period of time. A common remedical measure is to patch up to the corroded portion with new plates welded in place. But this option is not economical. The rehabilitation using ferrocement is quite economical and simple. The process consists of using the existing steel tank as a formwork while a new ferrocement inner lining is provided at hold water. The steel tank at student hostel of Roorkee University has been successfully rehabilitated using ferrocement inner linings, (Rahman, 2002).

The process of sanitary sewer relining using ferrocement to rehabilitate the sewer has already gained wide acceptance in UK. It now being more commonly used in other countries as well. In sewer application it is important to pay attention to the type of and porosity of the mortar matrix used. This should pose no problems since there are a variety of formulations that provide adequate resistance to chemical attack. The cement used for mortar should be sulphate resistant.

Ioms (1987) studied the performance of ferrocement in construction of and repair of boats trawlers. They showed that in fact, the most successful and convincing application of ferrocement has been in construction and repair of boats. Iom suggests and open mold system to be used for better repair of boat. Instead of using no form the wire mesh layer is used directly.

Reinhorn and Prawel (1988) successfully used a thin ferrocement coating on the sides of the unreinforced masonry wall that need enhanced in plane and out of plane strength and ductility. Ferrocement coating was mounted on the two sides of the wall with tension ties provided through the masonry. The result of the test showed the suitability of ferrocement as a retrofit (strengthening) material with a doubling of the wall.

Singh *et al.* (1988) suggest a simple procedure for the strengthening of brick masonry columns using ferrocement. Brick masonry column in old structure and are usually used for low-rise structures. Although the performance of masonry columns under axial loads may be satisfactory, they posses a limited moment carrying capacity. Improving a moment carrying capacity become vital if structure is subjected to modifications resulting in eccentric loads to be transferred to the columns. Ferrocement encasement of masonry column can considerably increase its capacity to resist axial loads and moments. Applying the ferrocement encasement, (Singh *et. al.* 1988) report the failure loads to be double that of uncased columns. Failure is due to failure in casing under combined bending and tension under lateral loads.

For many repair and renovation programs of civil engineering structures, Chowdhury and Robles-Austriaco (1986) cite the suitability of ferrocement because of

1. Better cracking behavior.
2. Capacity of improving some of the mechanical properties of the treated structures.

3. Further modification or repair of ferrocement treatment is not difficult.
4. Imposition of little additional dead load requiring no adjustment of the supporting structures.
5. Ability to withstand thermal changes very efficiently.
6. Ability of achieving waterproofing property without providing any surface treatment.
7. Readily available constituent materials.
8. No need for special equipment.
9. Ability to be used in repair program with no distortion or down grading of architectural concept of the structures.
10. Flexibility of further modification.

2.7 EXISTING METHODS OF REPAIRING RC STRUCTURE

There are various methods for repairing and rehabilitation of distressed Reinforced concrete Structural elements some of which are given below

2.7.1 Cement Grout

Rahman (2002) reported that cracks wider than about 1 mm in the uper surfaces of beams, slabs etc can often be sealed by brushing in dry cement followed by light spraying with water. This treatment will seal the upper part of a crack against ingress of moisture and carbon dioxide. It will not hide the cracks completely. For cracks wider than about 2 mm it may be preferable to use a cement water grout.

2.7.2 Bonding with Epoxies

Cracks in concrete may be bonded by the injection of epoxy bonding compounds under pressure. Usual Practice is to drill in to the crack from face of the concrete at several locations, inject water or a solvent to flush out the defect, allow the surface to dry (using a hot air, if ineeded). Surface-seal the cracks between the injection points and inject the epoxy until it flows out of the adjacent sections if the cracks or begins to bulge out the surface seals, Just as in pressure grouting.

The epoxy is injected through the holes about 19mm diameter and 19mm at 152mm to 152mm; centers (The smaller spacing are used for fine cracks). Injection is made through five-valve stems fastened in the drilled holes with an epoxy bonding. Compound and injection process proceeds from point to point, all values in the circuit being capped except the one being injected the injection pressure should be maintained for several minutes to the epoxy back in to the firer parts of the cracks.

The injection of epoxies into cracks in concrete as a mean of for bonding the broken surface represents an application for which there is no real alternate procedure. However unless the crack is dormant (or the cause of cracking is removed there by making the crack dormant) it will probably recur, possibly some where else in the structure. Also the technique is applicable if the defects are actively leaking to the extent that they cannot be dried out or where the cracks are numerous.

2.7.3 Grouting and Sealing

This method involves enlarging the crack along it-exposed face and filling and sealing it with a suitable material. This is the simplest and most common technique for sealing cracks and is applicable for sealing both fine pattern cracks and larger isolated defects.

The operation consists of following along the crack with a concrete saw or hand or pneumatic tools. A minimum surface width of 6.35mm is desirable smaller opening are difficult to work on. The surface of the routed joint should be clean and permitted to dry before placing the sealant. The methods used for placing the sealant depends on the material to be used and follows standard techniques.

2.7.4 External stressing

Development of cracking in concrete is due to tensile stress and can be arrested by removing these stresses. Further inducing a compression force sufficient to overcome the tension and provide a residual compression can close the cracks. The compressive force is applied by use of the usual prestressing wires or rods.

2.7.5 Blanketing

Blanketing is similar to routing and dealing but is used on a large scale and is applicable for sealing active as well as dormant and joints.

2.7.6 Overlays

Overlays are used for repairing spelled and disintegrated concrete. It is also used to seal cracks and are very useful and desirable where a large number of cracks and treatment of each individual defects would be expensive.

2.7.7 Autogenous Healing

The inherent ability of concrete to heal cracks within itself is termed 'autogenous healing' and is a phenomenon, which has been known for some time. It has practical application for sealing dormant cracks developed during the driving of precast piling, sealing cracks in water tanks. The effect also provides some increase instant of concrete damaged by vibration during set and of concrete disrupted by the effects of freezing and thawing.

2.7.8 Ferrocement Overlay

Ferrocement jacketing can repair RC beams, slabs, columns and many other concrete structures. In this study repairing methodology by ferrocement overlay is applied for strengthening of distressed RC frames with infill.

2.8 GENERAL REHABILITATION PROCEDURE FOR RC FRAMES WITH INFILL

Rehabilitation or repair of Rc frames with infill structure by application of ferrocement layers will consist of choosing proper materials, preparation of affected surface, providing bonding medium and application of ferrocement. The general procedures are detailed below-

2.8.1 Preparation of surface

Rahman (2002) reported that the execution of the repair work the most essential requirement is to remove all deteriorated or damaged concrete from the structures. The affected area should be thoroughly scrubbed and cleaned from all grease, dirt and grit, roughened by chiseling or wire brush, washed with water and air blown to remove any loose materials. The preparation for the repair of the structural elements showing spelling of concrete, all unsound damaged, fouled, porous or otherwise undesirable concrete should be removed, where.

It is not deemed necessary encase the bars fully, more than half the circumstances of the bars shall be exposed. But the corner bars should be fully exposed. In case of exposing the bars of the beams and columns of any structure for the repair works temporary propping should always be provided beforehand. For that the propping system should be designed accordingly.

2.8.2 Cleaning the surface

The surface of the existing concrete, which is to be bonded to the new work, should be cleaned and moistened just prior to the placement of ferrocement.

Following a preliminary rinsing, cleaning by use of some blasting in preference to wire brushing should be made. The cleaning operation should be performed just prior to placing the fibrocement intruder to avoid fouling of the surface by wind blown dust or debris.

2.8.3 Moistening the surface

After clearing the surface should be saturated with water and then allowed to approach dryness just before placing the ferocement. The surfaces should keep moist for several hours to assure saturation.

2.8.4 Bonding layer:

To ensure sufficient bonding layer should be provided. It consists of cement slurry with 0.5 parts and 1 part by weight of cement by weight of water. Cement slurry should be applied on exposed surfaces and the cement sand mortar in the ratio of 1:2 by volume should be trowel led into the groves and surfaces, so that a flat rough surface is obtained. This bonding level should be thin film usually 4 – 6 mm thick.

2.9 REHABILITATION OF DISTRESSED RC FRAME

Rahman (2002) reported for the spelled concrete over the column surfaces, partly or totally or around the cross section a procedure similar to the state above should be followed. The procedure for placing the fibrocement is stated below:

Step -1:

The exposed sides of the main reinforcement and ties towards the core section should be filled micro concrete to rebuild column section up to the ties.

Step -2:

The first wire mesh should be laid all around the column section. After fixing the wire mesh with galvanized wires to the nails the wire mesh should be covered with 6-8 mm thick cement sand mortar of proportion 1:2.5 by volume.

Step -3:

After about 12 hours of mortar application as in step 2, the surface should be roughened the second layer of wire mesh should be laid and covered with 6-8 mm thick mortar as in step 2.

Step -4:

After finishing the surface to the desired level curing should be done by covering the rehabilitated areas with hessian and keeping it constantly wet for about 14 days.

2.10 REHABILITATION OF DISTRESSED INFILL

Step -1:

The plaster of the exposed wall is to be removed. The existing surface should be cleaned and moistened.

Step -2:

A single layer wire mesh should be laid all around the infill. After fixing the wire mesh with galvanized wire to the nails. The wire mesh should be covered with 19 mm thick cement sand mortar of proportion 1:2.5 by volume.

Step -3:

After finishing the surface to the desired level curing should be done by covering the rehabilitated areas with hessain and keeping it constantly wet for about 14 days.

2.11 REMARKS

The properties of ferrocement discussed in this chapter was used as suggestion for the process of repairing the distresses reinforced concrete infilled frame by using ferrocement overlay. This methodology provides an alternative in rehabilitation and repairing of structural elements using ferocement. It can effectively control the different distressed such as creaking, spelling and can be also strengthen the structure.

Chapter 3

EXPERIMENTAL SETUP AND RESULTS

3.1 INTRODUCTION

The experiment performed on original masonry infilled reinforced concrete frame and rehabilitated frame has been described in this chapter .One half scale masonry infilled reinforced concrete frame was prepared for this experimental programme. The frame was subjected to one point lateral load by hydraulic jack in the laboratory up to failure. The distressed frame was rehabilitated by using ferrocement overlay. The experiments were performed to investigate the performance of ferrocement overlay in strengthening of distressed masonry infilled reinforced concrete frame subjected to lateral load. For each test initial cracking load, failure load, crack pattern and deflection were observed.

3.2 EXPERIMENTAL SETUP

There was no scope of testing RC frame with existing lab facility. Therefore it was necessary to build a test rig to perform the experiment. The rig has to be built in such a way it can fulfill the objective of the research and can easily be removed after testing is completed. The following subsections describe the details of the test arrangement.

3.2.1 Reaction Frame

The sketch of reaction frame diagram is shown in Fig. 3.1, which consist of base BC and two columns AB and CD. The base must be stiff enough to withstand any bending failure during testing of the reinforced concrete infill frame. Some pictures of placement of reinforcement in to the base, reaction frame and casting are shown in Fig.3.2, Fig.3.3 and Fig.3.4.The properties of concrete and compressive test result are shown in appendix-A. The tensile test results of reinforcement are shown in appendix-B. Hydraulic jacks are mounted at the top of column, which is supported by column cantilever element projected from top of the column shown in Fig.3.5.

Detail design of the reaction frame is shown in Fig.3.6. The bending resistance of the column is not sufficient to withstand the reaction of the hydraulic jack. Therefore a pair of tie rods on each side of the frame is used to tie the column top to the farthest point of the base as shown in Fig.3.5 and Fig.3.7 To prevent the bending of the tie anchor, (25mm bar put across the column) additional clamping was arranged which could be seen in Fig.3.8.

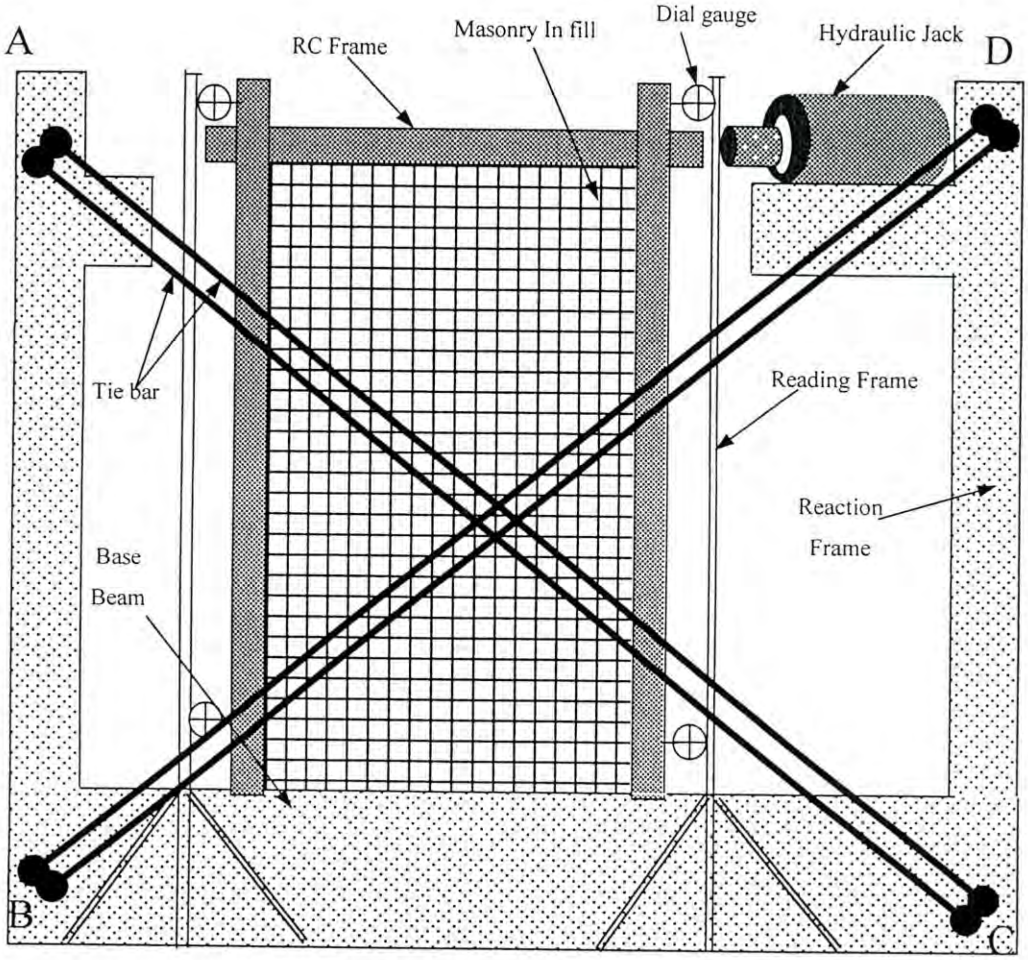


Figure: 3.1 Schematic elevation of Masonry infilled RC Frame and test setup.

3.2.2 The Infill Frame

Masonry infilled reinforced concrete frame is constructed monolithically with the reaction frame base BC that is shown in Fig.3.1 and Fig.3.6. We know that an

Fig.: 3.2 Construction of the reaction frame - before casting.

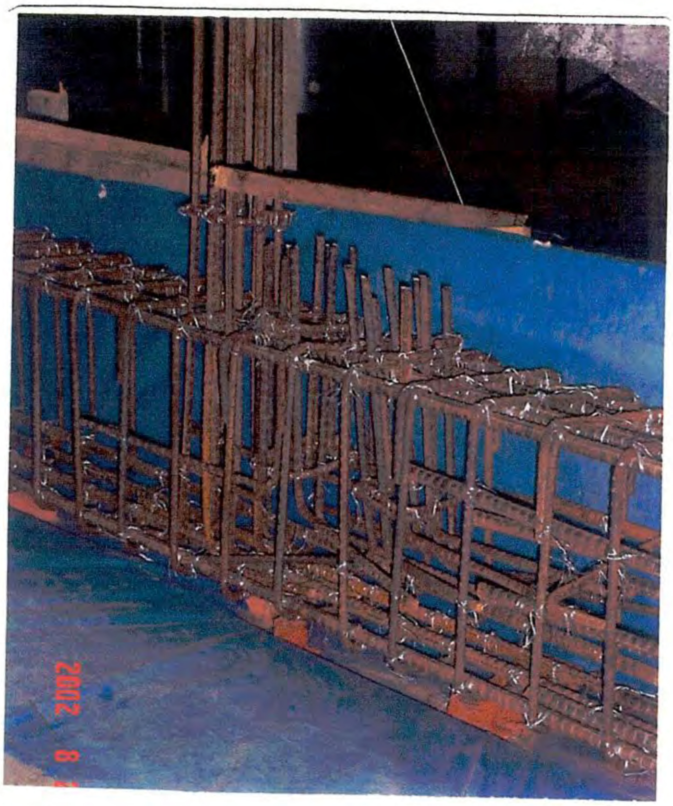


Fig.: 3.3 Close-up view of base reinforcement.

Fig.: 3.4 Construction of the test rig - casting of column in progress.

ordinary building having column and beam size of 300mmx300mm and floor height 3.049m has been modelled in half scale. So for the experimental purpose column and beam has been taken as 150mmx150mm and a height 1.5245m has been chosen. Again masonry infill size 110 mm × 55 mm × 35 mm was chosen. The wooden formworks are made up of as per the size of beam and columns. The frame was cast in the concrete laboratory of BUET. The concrete was made with ordinary Portland cement conforming to the ACI code, crushed coarse aggregate from first class picket and natural river sand as is fine aggregate, firmly in this proportion water cement ratio, reinforcing and other parameters used preparations of frame specimens are mentioned below. Finally, curing of the RC frame was done by covering with hessians and keeping them moist. For gaining the required strength proper curing is essential. So, the curing was carried out 21 days. The reinforcement detail of beam and column are shown in Fig.3.6.And some photograph of progressive casting and curing works are shown in Fig.3.9, 3.10 and 3.11.

3.2.3 The Infill material

The infill material is half scale model. Therefore half scale masonry elements are used in testing. The commonly used size of brick mass is 250mm×125mm×81mm. Therefore half scale brick specimens are obtaining by slicing a normal size bricks using a diamond saw, the photograph is shown in Fig. 3.12. Five half scale bricks high stack bonded prisms with plywood capping at top and bottom were loaded in uniaxial compression perpendicular and parallel to the joint to failure. The test results are shown in appendix-F.

3.2.4 Location of the dial gauges

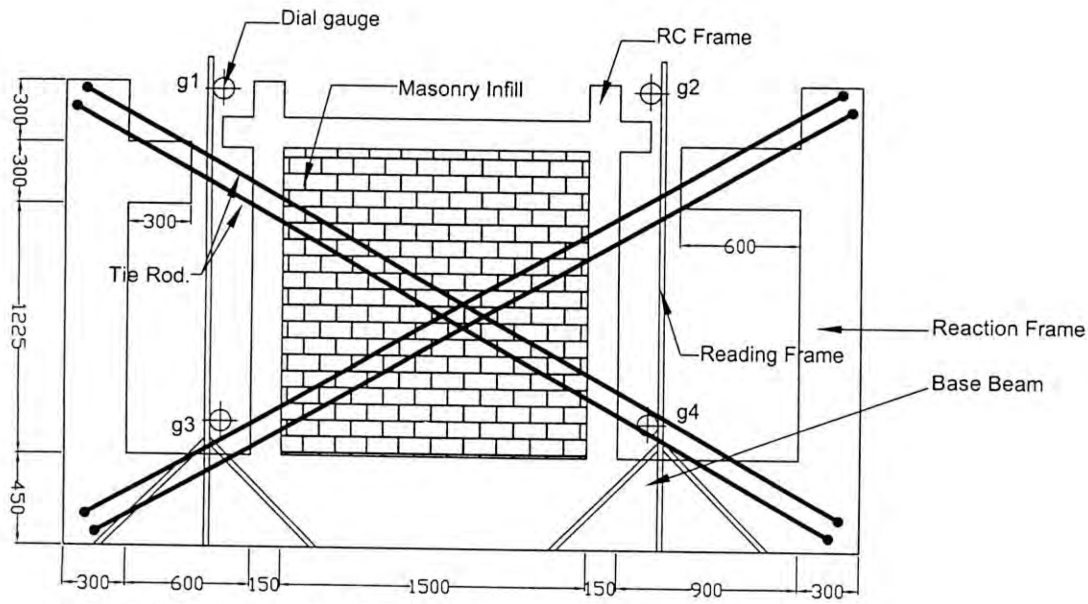
Four dial gauges were used to measure the deflection of the infilled frame. Two were towards the loaded side and others were on the leeward side. These are used to measure the top and the bottom deflection from both sides, shown in Fig.3.5 and Fig.3.13. One small division in dial gauge is 0.001inch.So,the conversion of dial gauge is 0.001 x 25.4mm= 0.0254mm.

3.2.5 Mounting of the dial gauges

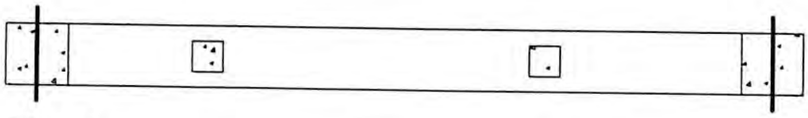
While measuring the dial reading, it was ensured that movement of the test frame does not affect the reading and that all dial readings were taken with respect to a stationary reference which is in this case the floor of the lab. To accomplish this, two sets of mounting frame made of 25mm dia rods were constructed and are shown in Fig.3.13.Four dial gauges marked g1, g2, g3 and g4 (see Fig.3.5) had been attached on the mounting frame.

3.2.6 Mounting of the hydraulic jack

To apply the lateral load towards the experimental frame the hydraulic jack was mounted on the cantilever portion of the reaction frame, shown in the Fig.3.14.The jack was attached to the cantilever potion of the reaction column by the U clamp, as shown in Fig.3.14. To prevent the jack reaction excess diagonal reinforcement was



FRONT ELEVATION.



PLAN.

Figure: 3.5 Elevation, Plan and Test setup of Masonry infilled RC Frame.

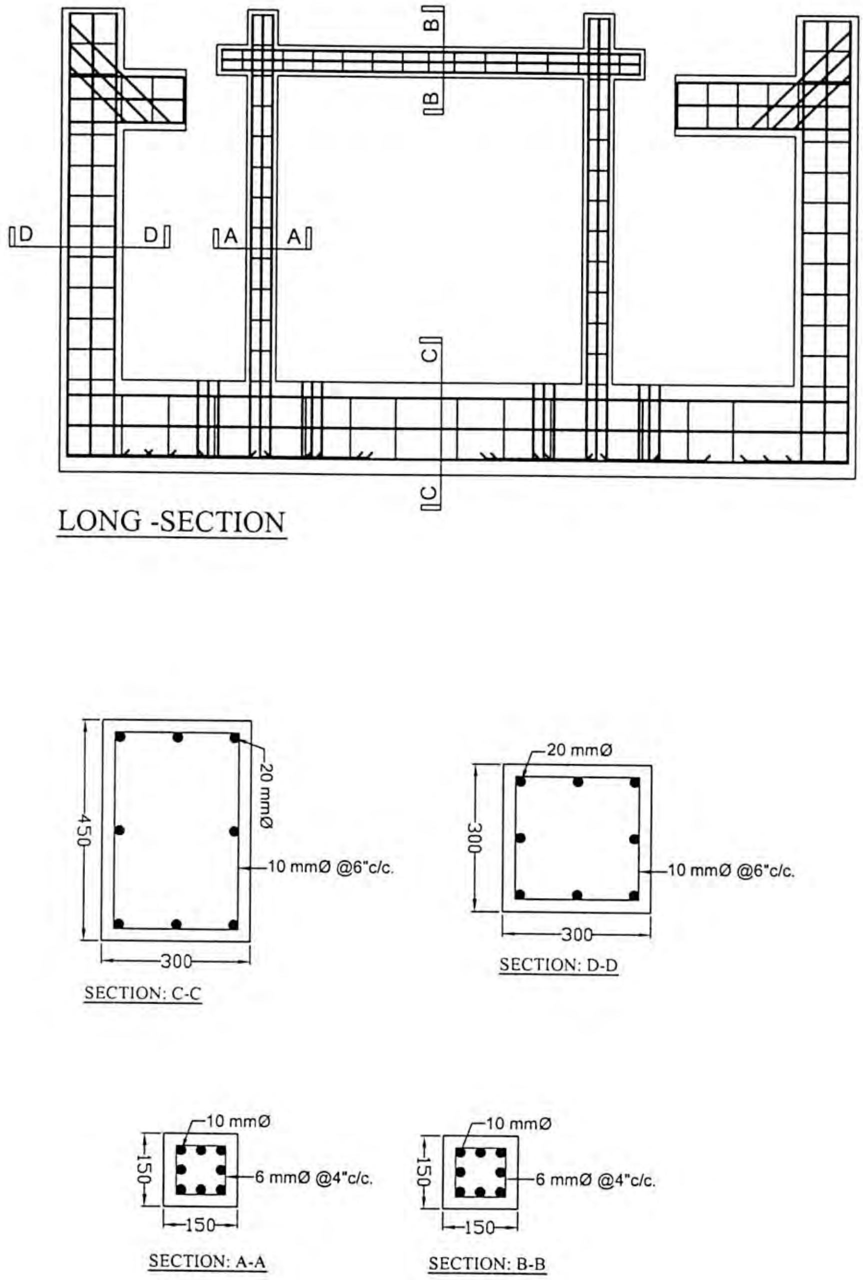


Figure: 3.6 Long and Cross- Section of Masonry infilled
Reinforced Concrete Frame.

Fig.: 3.7 Test frame with diagonal tie rods attached.

used, shown in Fig.3.15. The calibration of hydraulic jack machine is shown in appendix-G.

3.3 TESTING OF ORIGINAL FRAME

The half scale Masonry infilled RC frame was tested by hydraulic jack machine (compression) of capacity 50 ton (50,000kg) or 500 kN in the laboratory. Applying lateral load at the top of the RC column up to initiation of failure. The surface of masonry infill (both sides) and the all sides of column were white and yellow colored to facilitate the viewing of cracks. The infilled RC frame was subjected to one point loading by hydraulic jack (compression) testing machine at the top of the column to (see Fig.3.13). Loads were increased at uniform rate of 1000kg (or 1Ton) or 10kN until the failure occurred. Applied loads were recorded using machine dial gauges. To measure corresponding deflection of frame four dial gauges, two are at the top of the frame and other two are at bottom of both left and right side of the frame were placed the cracking load, failure load, failure pattern and deflection at top and bottom of the frame for each incremental load have been observed and recorded during testing. The load-deflection curves are shown in Fig.3.16, 3.17,3.18 and 3.19 after the completion of testing test setup is removed and ready for repairing are shown in Fig.3.20.

3.4 REPAIRING OF DISTRESSED INFILLED RC FRAME

After testing of infilled RC frame to failure, the frame was ready for repairing by ferro wrapping. Before any mesh was applied to the infill and frame it was carefully dismantled of plaster and of infill and the column (section), as shown in Fig.3.21

Fig.: 3.8 Arrangement of additional clamping to prevent the bending of the tie anchor.

Fig: 3.9 Close up view of the reinforcement in column of the infilled frame.

Fig.: 3.10 Construction of the test rig-mid way through the casting.

Fig.: 3.11 Construction of the test rig-reinforcement arrangement of the beams.

Fig.: 3.12 Samples of half scale masonry brick sliced from full size brick.

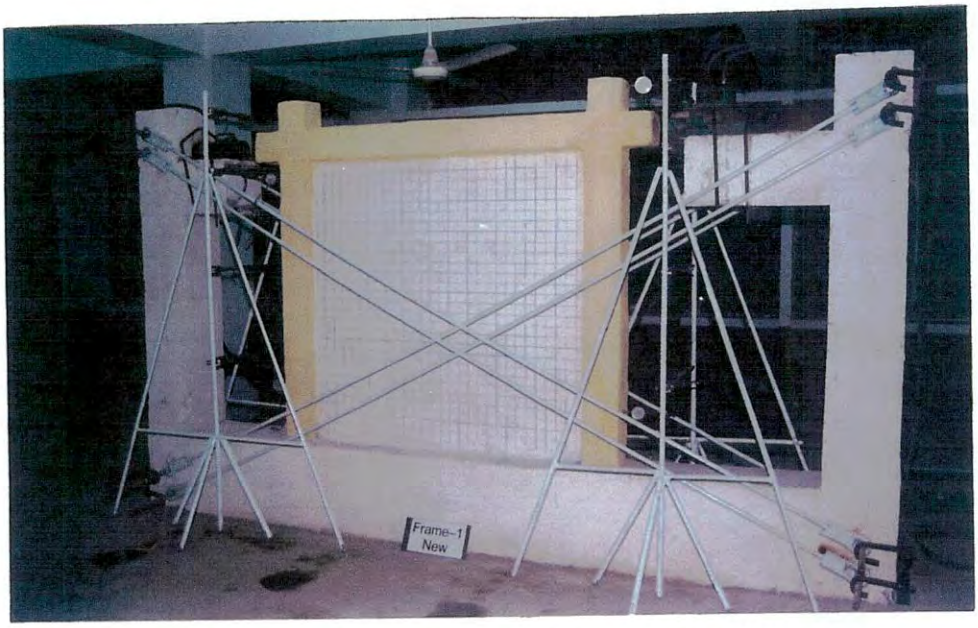


Fig: 3.13 Test setup with mounting frame to mount dial gauges.

Fig.: 3.14 Hydraulic jack mounted on the cantilever portion of the reaction frame.

Fig.: 3.15 Additional diagonal reinforcement used to prevent failure against reaction of hydraulic jack.

remove laitance and any loose concrete cased by the damaging process. It was then washed with water to ensure all dust was removed. The following subsection describes the details of the repairing of distressed infilled RC Frame.

3.4.1 Material used for Repairing

(i) Cement

Ordinary Portland cement of type 1 as per classification of ASTM was used in concrete for frame and mortar of ferrocement.

(ii) Sand

Natural river sand was used as fine aggregate passing through no. 4 seive (2.36mm). fineness modulus of sand was 2.65.

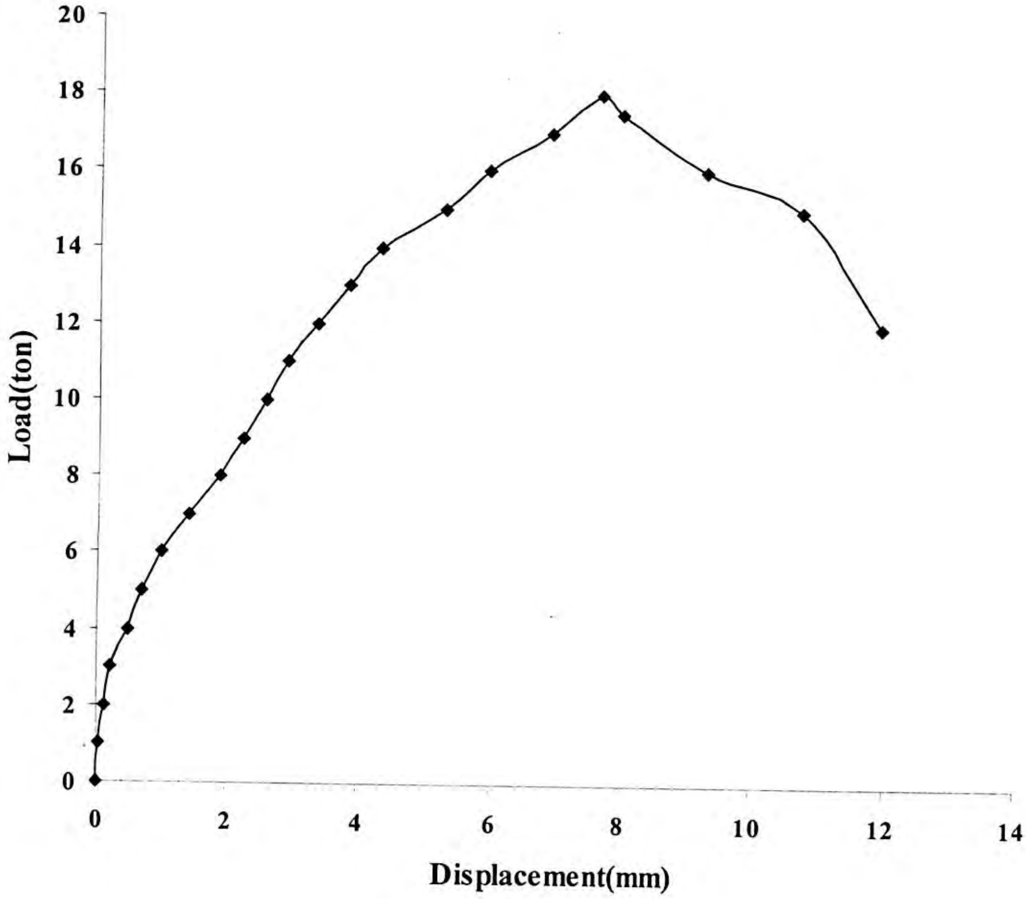


Fig-3.16 Load Vs Deflection at top of the right corner of the original frame.

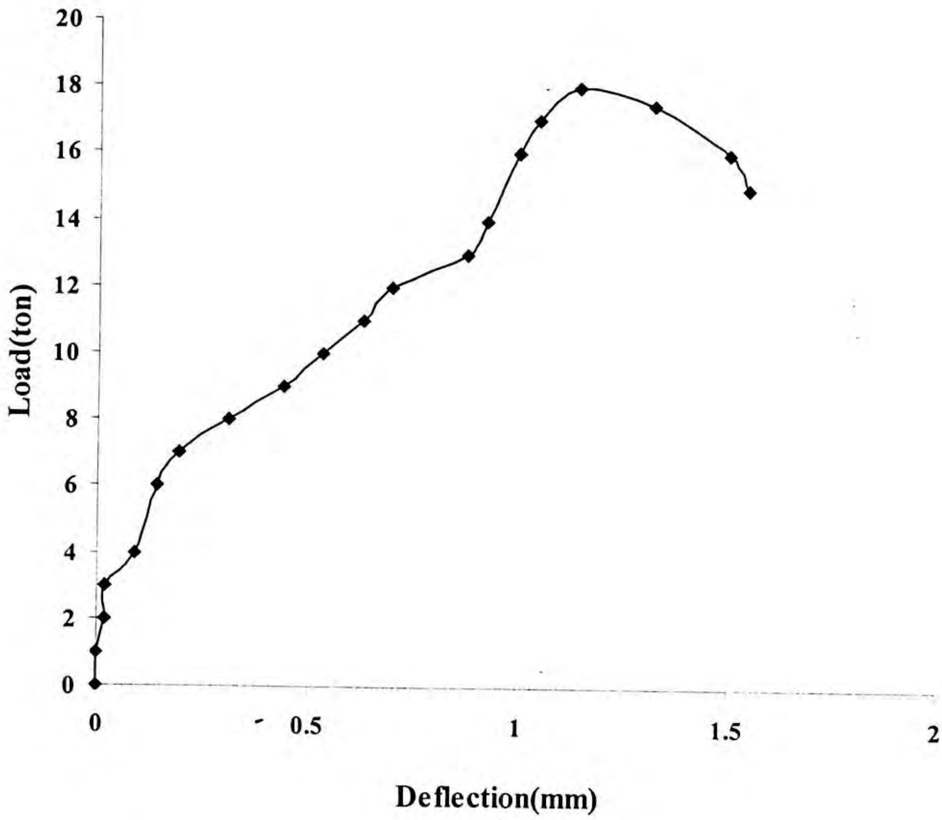


Fig-3.17 Load Vs Deflection at bottom of the right corner of original frame.

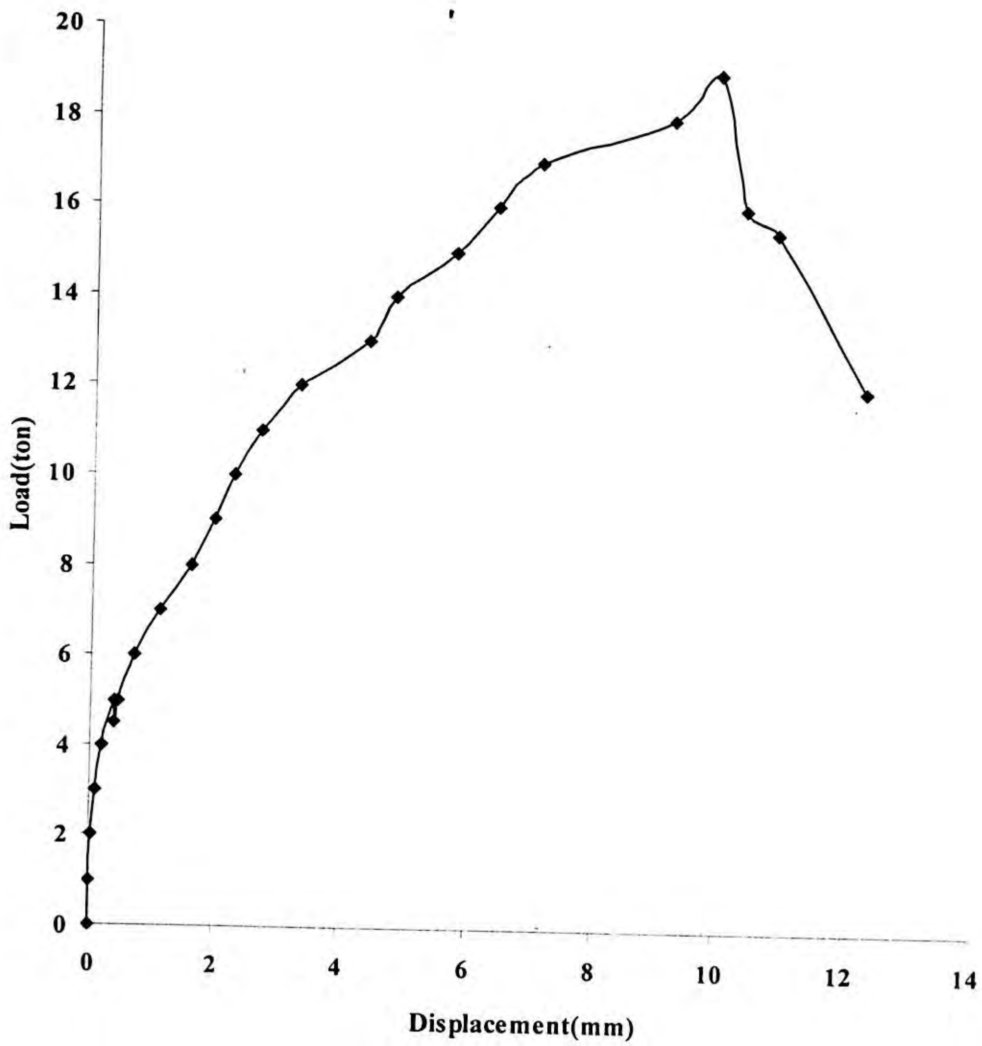


Fig-3.18 Load Vs Deflection at top of the left corner of the original frame

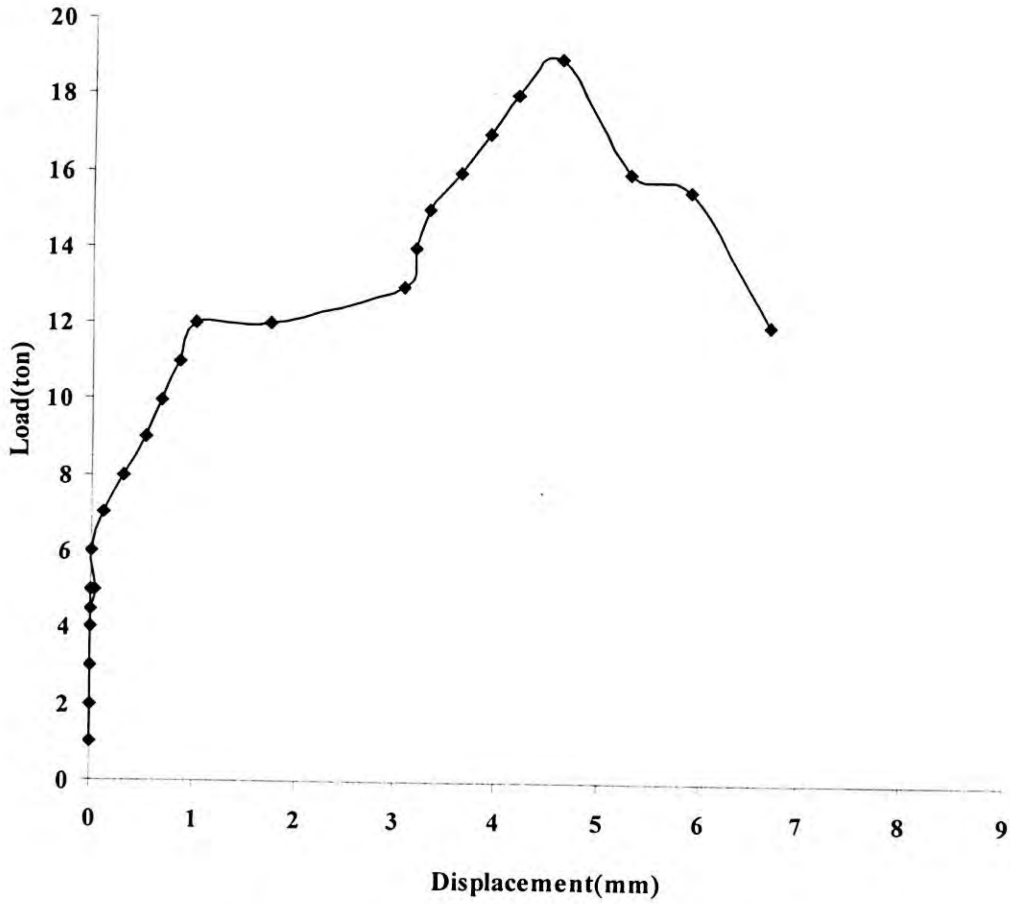


Fig-3.19 Load Vs Deflection curve at bottom of the left corner of the original frame.

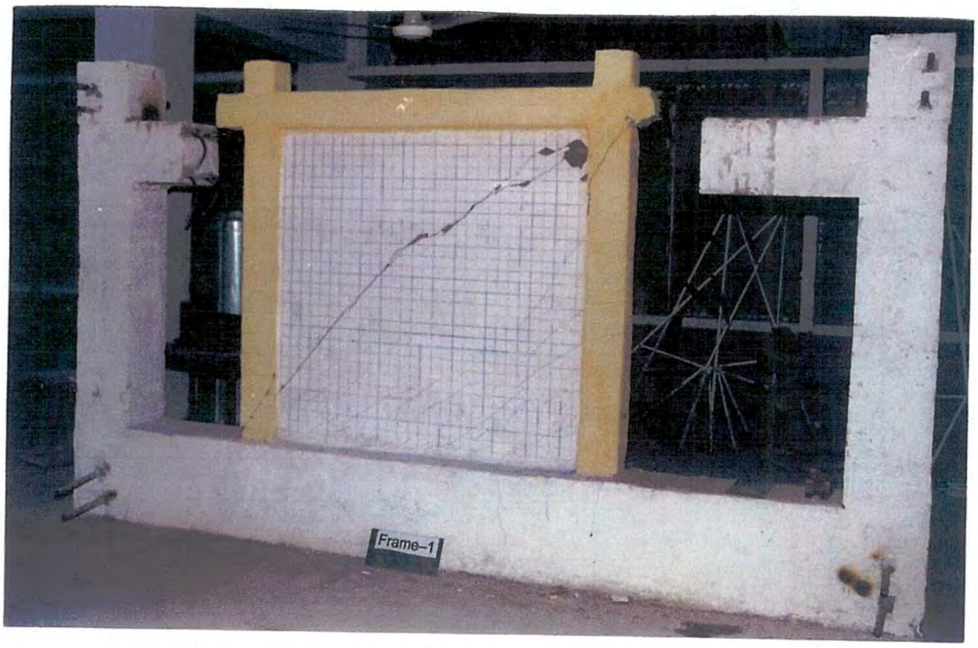


Fig.: 3.20 The infilled frame after failure.

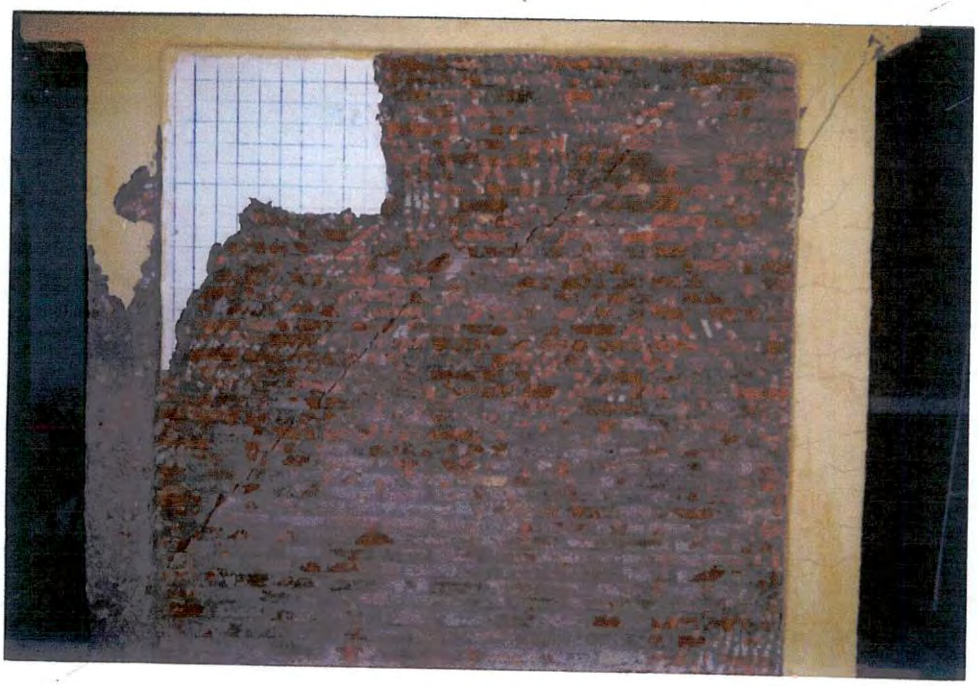


Fig.: 3.21 Diagonal crack through masonry infill after the plaster is removed.

(iii) Water

Tap water available in laboratory which is suitable for drinking was used in Preparation of concrete and mortar and also for curing.

(iv) Mortar

Properties of mortar used in ferrocement overlay for repairing the distressed RC frame and infill is given below:

Mixing Proportion of mortar	1:2.5
Water Cement Ratio	0.45
FM of Sand	2.65

7 days crushing strength of Mortar (50mm mortar cube) 32.22MPa(Appendix-C).

(v) Wire mesh

Reinforcing wire mesh of BWG 18 with 12mm square openings of woven type having mesh thickness 1.41mm were used in this study.

3.4.2 Repairing Procedure of distressed frame

Ferrocement technique was adopted to repair the distressed RC frame and masonry infill wall. First the concrete cover was chipped off up to 20mm depth. The chipping process continued from either side of the crack. After chipping the surface was cleaned thoroughly and washed with a water jet to remove dust and loose debris of concrete. In order to ensure proper bond between crack surfaces cement salary was sprayed over the crack. There after a thin layer of mortar was apply all over the surface of the column and masonry. Afterwards two layers of wire mesh place on column and one layer mesh was placed on the infill on both side. Which were held in place by the previously affixed nails. Thereafter with the help of a trowel mortar was applied on the wire mesh. This mortar penetrated through the openings of the mesh and came in contact with the previous applied mortar thus securing the mesh in place with proper bonding.

Finishing of the surface then performed using a trowel and following the usual practice. Finally, curing of the repaired frame was done by covering the rehabilitated region with Hessians and keeping them moist. For gaining the required strength proper curing is essential. So, the curing was carried out for 28 days.

3.5 TESTING OF REPAIRED MASONRY INFILLED RC FRAME

The distressed masonry infilled RC frame was repaired as described above and the surface of masonry infill (both side) and the all sides of column were white and yellow colored to facilitate the viewing of cracks. The repaired frame was again tested using hydraulic jack in the laboratory with the previous test setup, shown in Fig.3.22.The hydraulic jack was placed in testing frame in such way that it will act lateral load on the infilled RC frame. One point loading was applied only over the top of the frame and was recorded using machine dial gauge and the deflections were measured using dial gauge. Four of dial gauge used for recording the top and bottom deflection. The load-deflection curves are shown in Fig.3.23, 3.24,3.25, and 3.26.

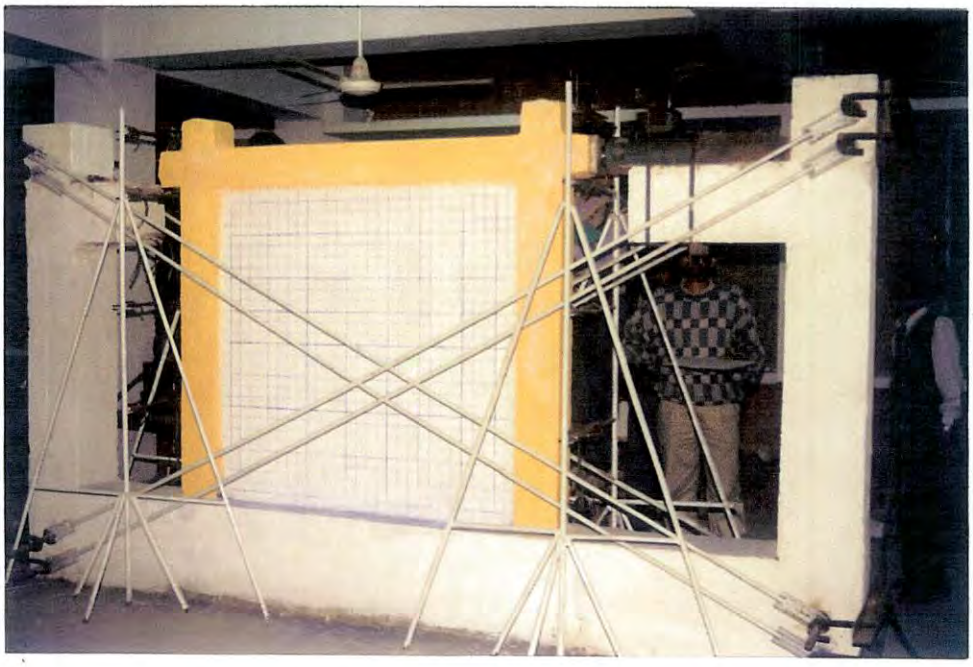


Fig.: 3.22 The repaired frame ready for testing again.

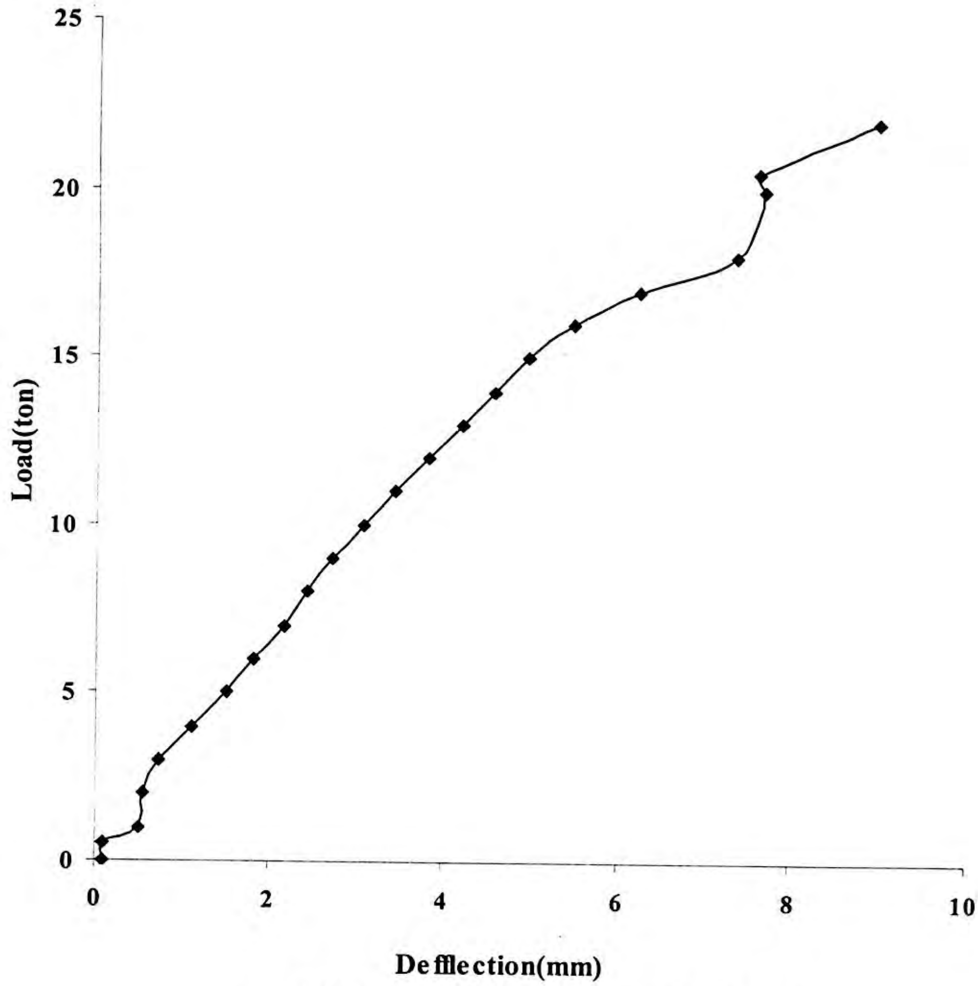


Fig-3.23 Load Vs Deflection at top of the right corner of the repaired frame.

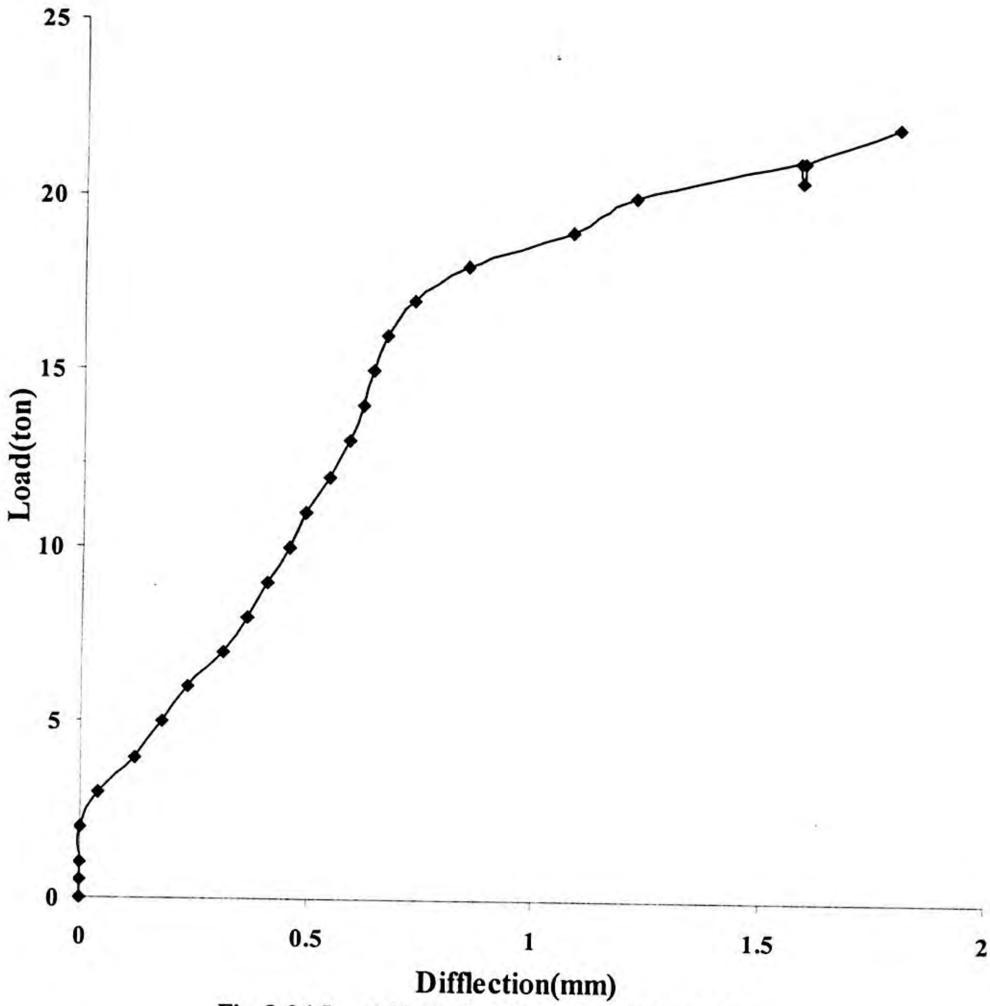


Fig-3.24 Load Vs Deflection at bottom of the right corner of the repaired frame.

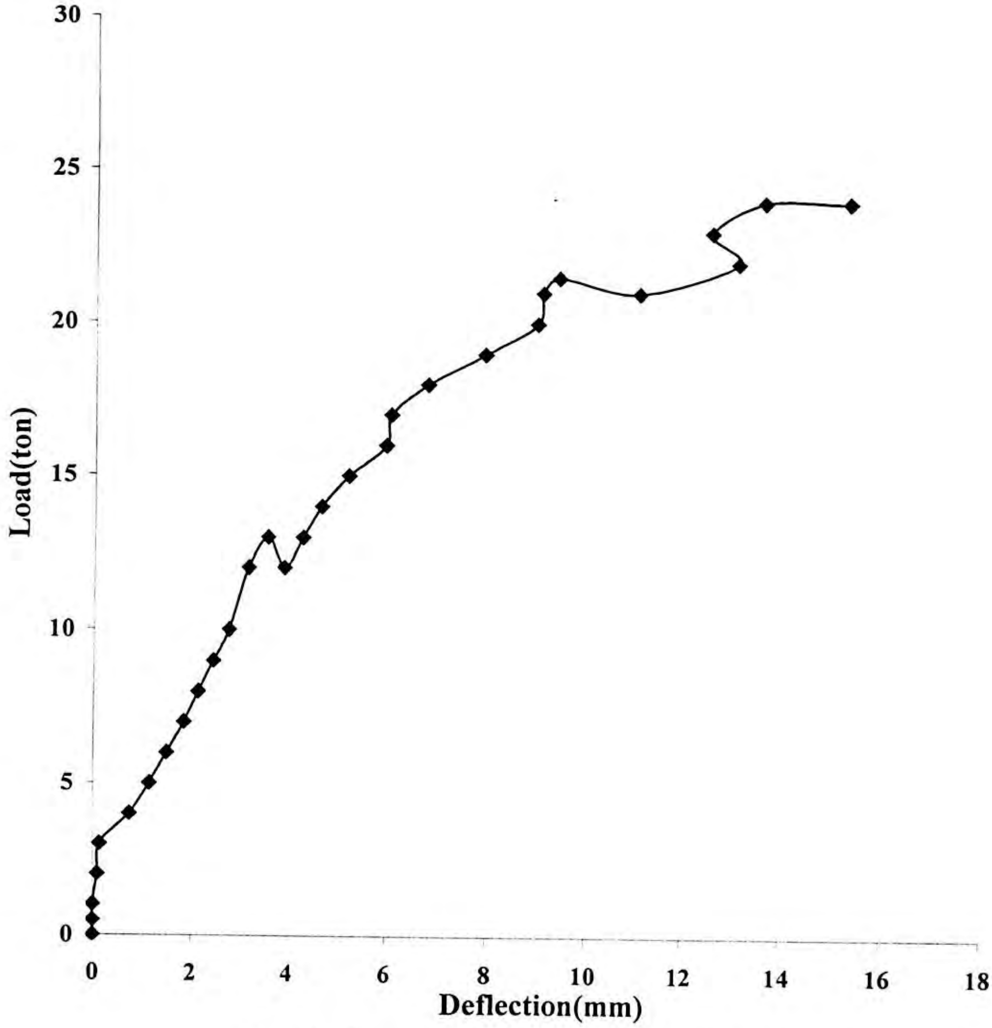


Fig-3.25 Load Vs Deflection at Top of the Left corner of the repaired frame.

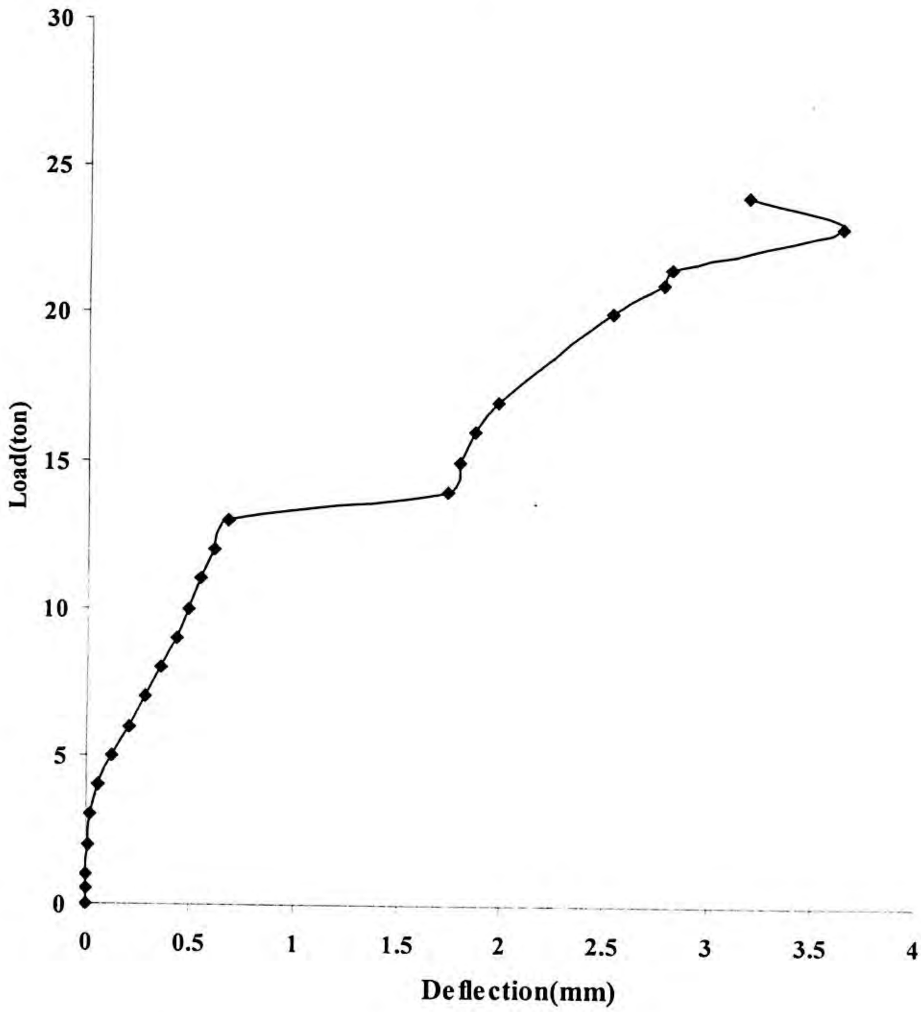


Fig-3.26 Load Vs Deflection at bottom of the left corner of the repaired frame.

3.6 RESULTS

As mentioned earlier, one infilled RC frame was tested under lateral load. The frame was allowed to form crack under lateral loading. During testing deflections were measured by dial gauges and cracks were observed with naked eyes. Failure loads, crack pattern and deflections were observed for original as well as for the repaired frame. These are described below:

3.6.1 Results from the original Frame

The load was applied laterally with the piston of the hydraulic jack at the top of the frame. The load was applied incrementally with an increment of 1 ton. At one stage of loading, approximately at 12ton,first crack appeared along bottom right corner of masonry infill as shown in Fig.3.27.

The crack propagated diagonally upward as load was gradually increased and the number of cracks were also increased with the increase of load. All the cracks propagated through infill and column towards upward direction, which is shown in Fig.3.28 and 3.29. Both faces of the infilled RC frame (top to bottom) showed similar crack pattern, which is shown in Fig.3.30. At a stage of load of 19ton the infill frame was failed with a major diagonal crack. The diagonal crack was also observed at top of the column at loaded side and at bottom of leeward side, which is shown in Fig.3.31. Crack went through the masonry, which means that brick failure occurred. A close up view of this is shown in Fig.3.32. After the full failure of the frame and infill the loading was stopped and frame was kept for repairing.

3.6.2 Results from the repaired Frame

The distressed masonry infilled RC frame was repaired by using ferrocement overlay with double layer wire mesh in columns and single layer for in infill wire mesh. The repairing was performed on both sides. The frame was tested after repairing in a similar way as the original frame. The first crack in this frame was formed at the bottom at a load of 16ton. The corresponding deflections at the top and bottom were 5.99mm and1.87mm which is less then these of original frame respectively (6.38mm and 3.63mm respectively).

These cracks propagated vertically upward as the load was gradually increased, which is shown in Fig.3.33. At a stage of loading of 23 ton, the repaired frame was failed as shown in Fig.3.34. Again the diagonal cracks were observed at top of the column at loaded side and at bottom of column at leeward side also, which is shown in Fig.3.35 and Fig.3.36.

As mentioned earlier, the same procedure was adopted for original and rehabilitated infilled RC frame. During testing of original frame as well-rehabilitated frame, the deflection was measured at top and bottom of the frame by setting four different dial gauges. Comparisons of deflection of different locations of both frames are shown in Appendix-E. Load deflection curve at top and the bottom for original frame and rehabilitated frame are shown in Fig.3.16 to 3.19 and Fig.3.23 to 3.26. From tests result it was observed that higher deflection were found at the top than that at bottom. It is also found that the deflections of rehabilitated frame are less than original frame, (shown in appendix-E).

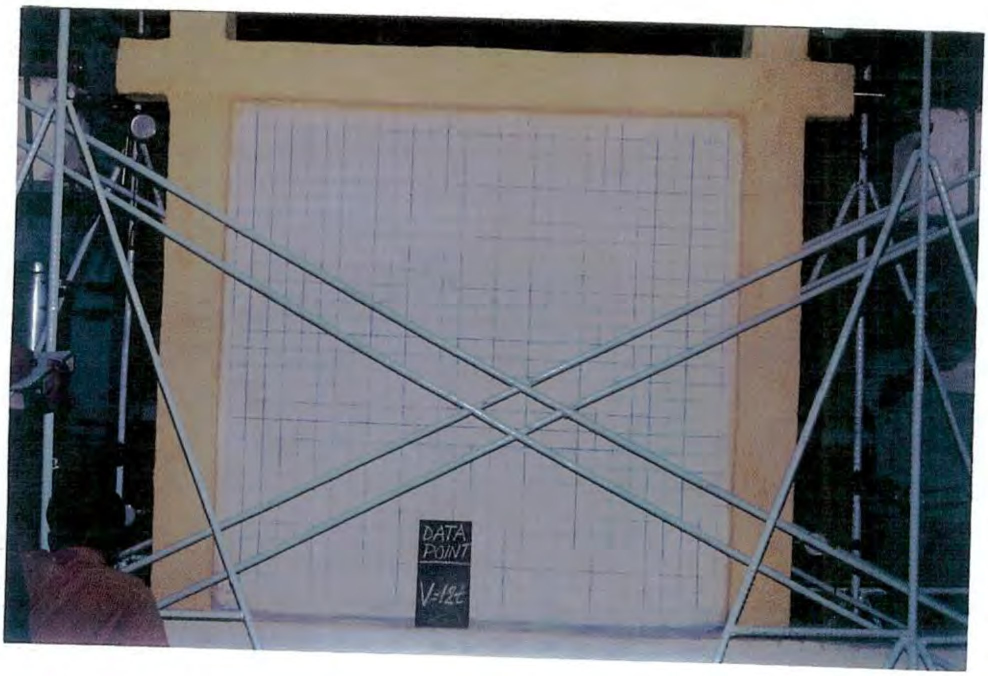


Fig.: 3.27 Initiation of visible crack in the original frame at 12 Ton load.

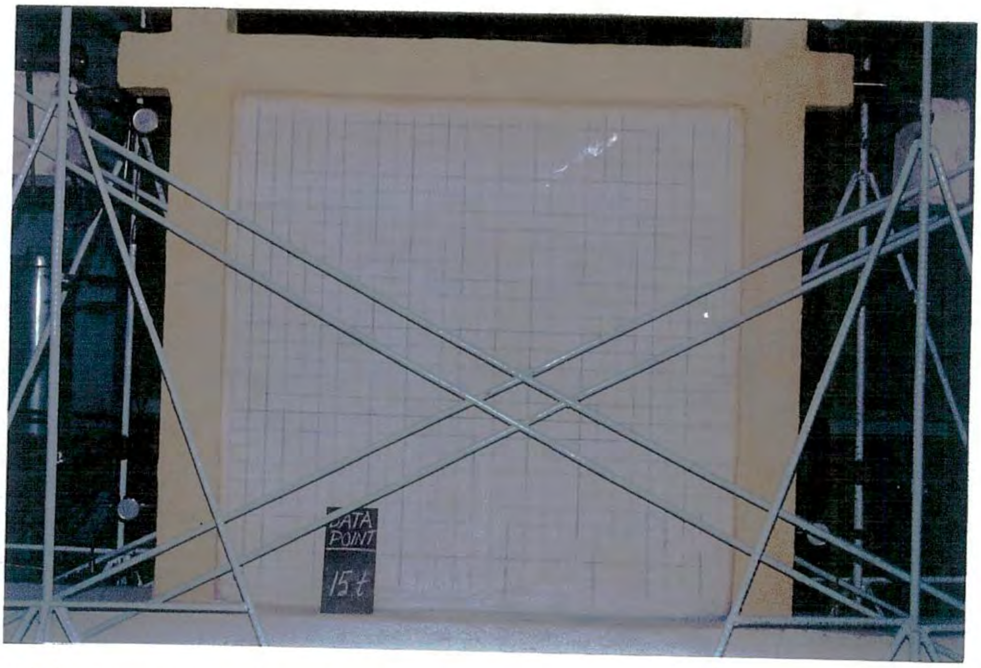


Fig.: 3.28 Propagation of cracks in the diagonal direction in the original frame at 15 Ton load.

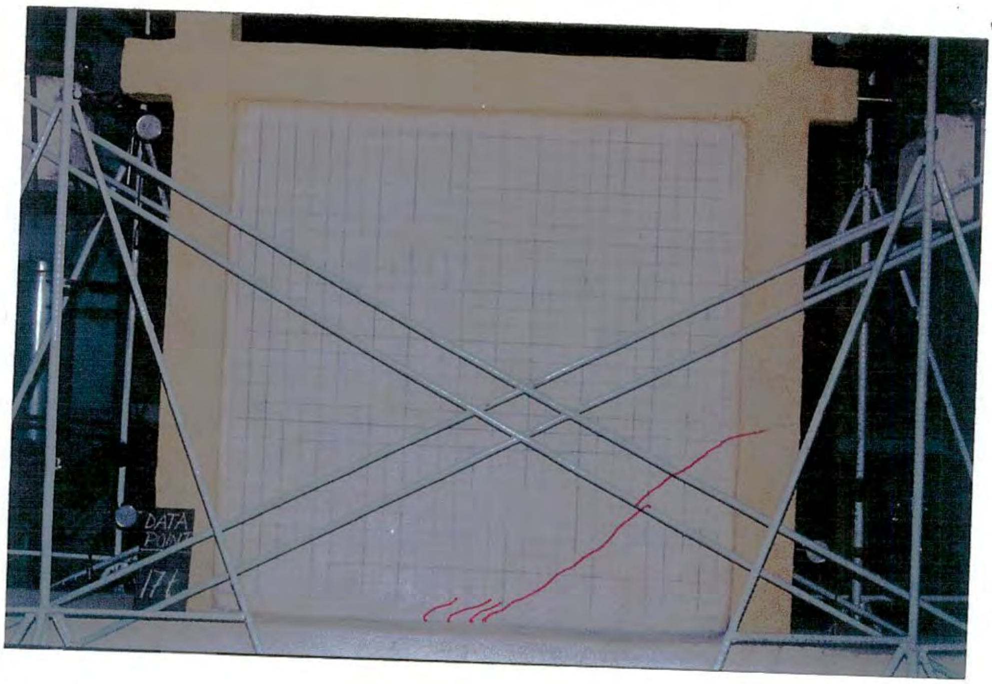


Fig.: 3.29 Propagation of cracks in the diagonal direction in the original frame at 17 Ton load.

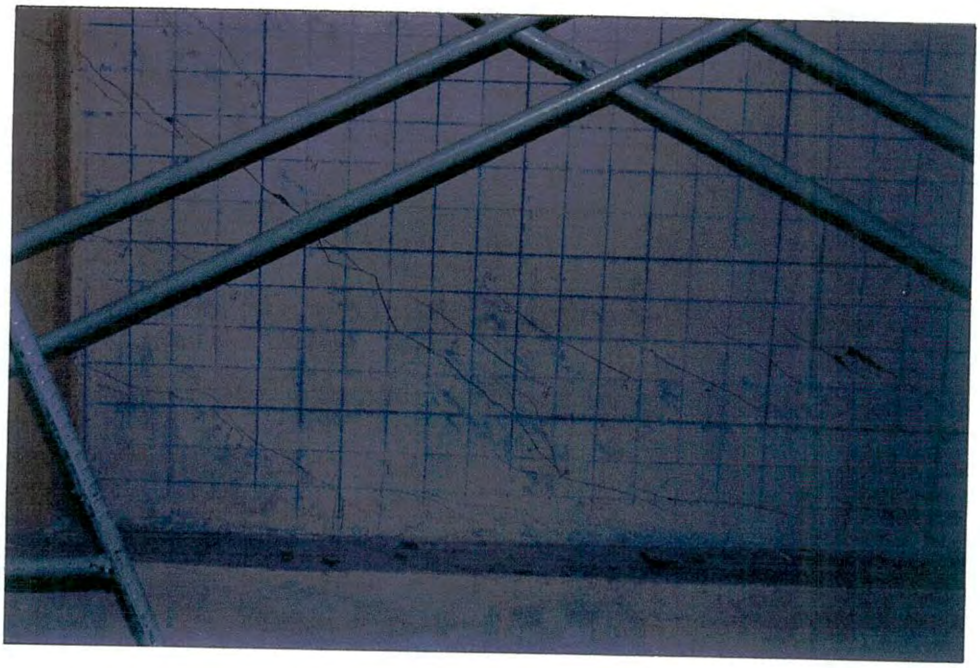


Fig.: 3.30 Crack Propagation continue in the opposite face of the original frame.

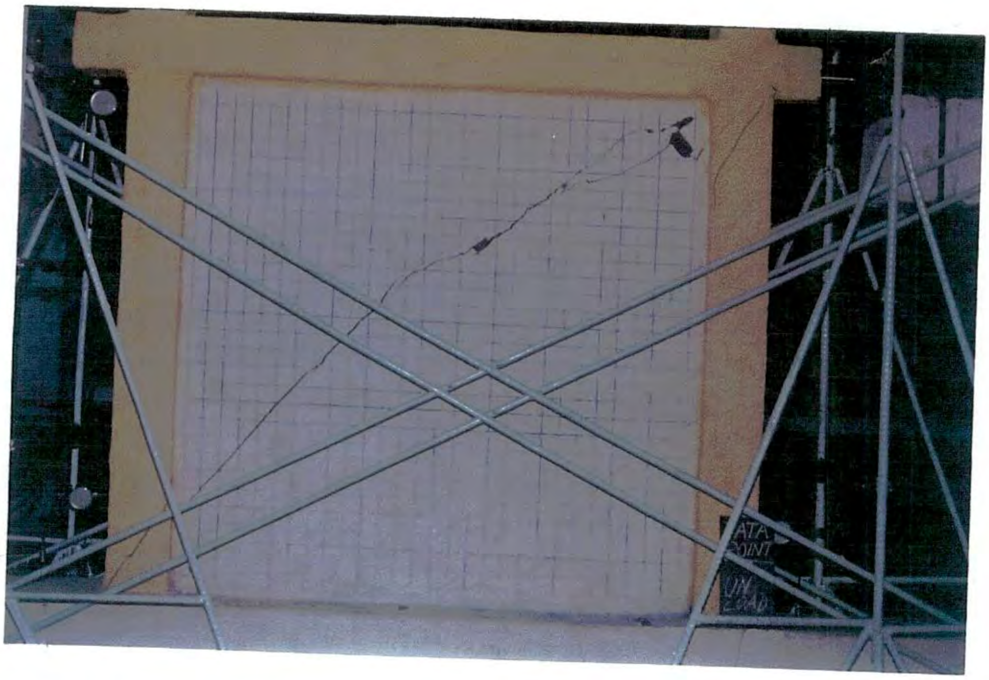


Fig.: 3.31 Complete diagonal failure of the original frame at a load of 19 Ton.

Fig.: 3.32 The close up view of crack through the masonry infill.

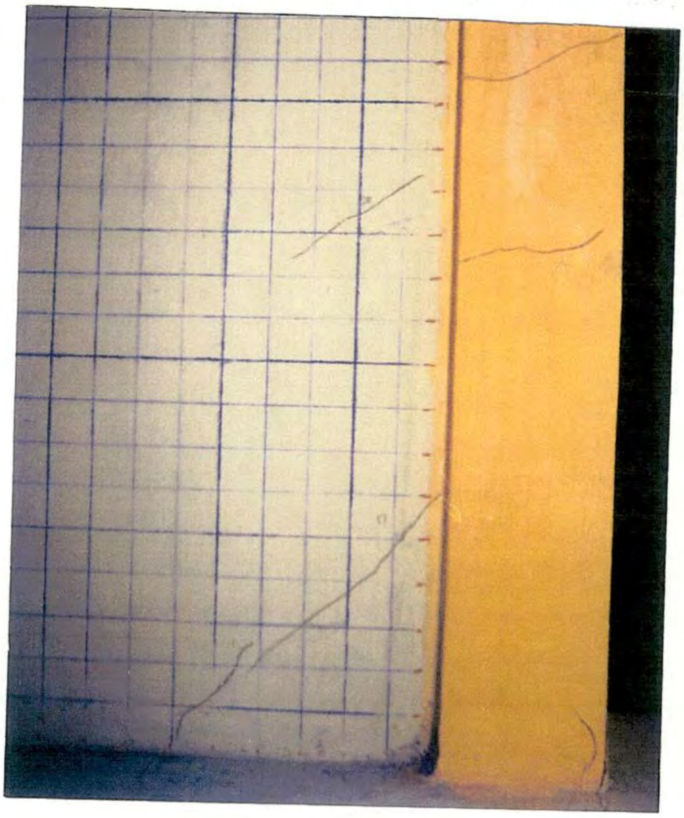


Fig.: 3.33 Tension cracks in the column of loading side.

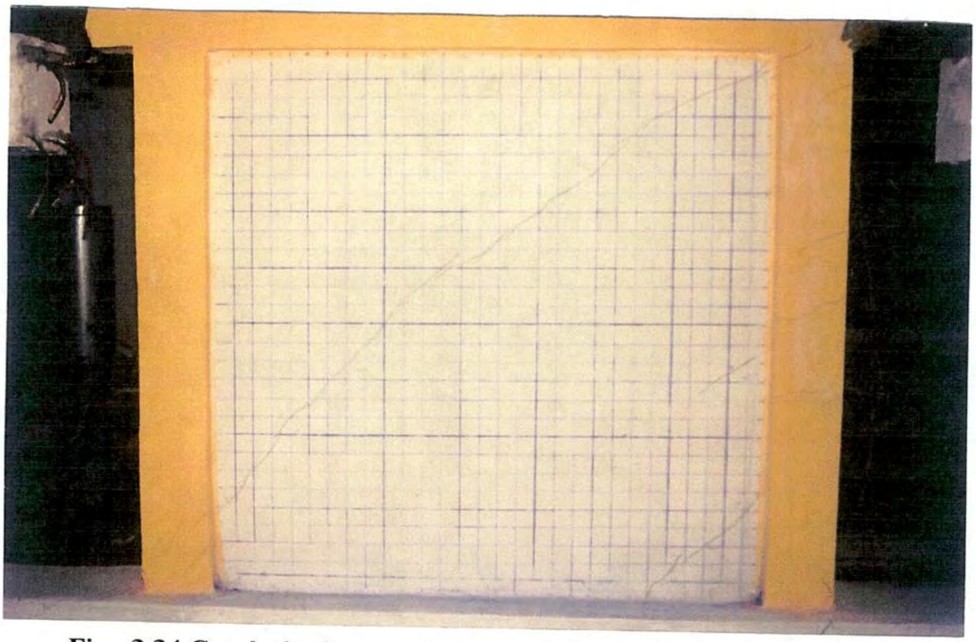


Fig.: 3.34 Cracks in the repaired frame at ultimate load of 23 Ton.

Fig.: 3.35 Shear cracks at the bottom of the column on the leeward side.

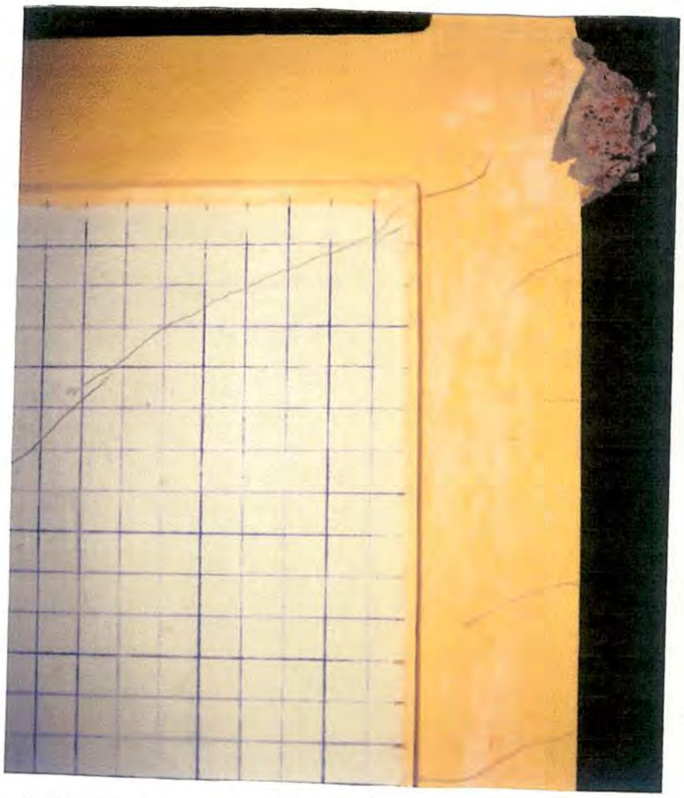


Fig.: 3.36 Tension cracks in the column on the loading side.

3.7 DISCUSSION

3.7.1 Crack growth

When load was gradually applied to the original frame the frame deflected at top, thereby causing shear deformation in the infill panel. The first visible crack appeared near the bottom of infill at the loading side at a load of 12ton. The corresponding top deflection was approximately 4mm.The cracks were diagonal in nature as seen in Fig.3.27.The corresponding shear strain can be calculated as 0.00262 radian. Thus we can get an idea of the maximum shear strain that may be developed in infill to produce visible cracks. This first diagonal crack appeared at the furthest corner from the loaded compression diagonal. This indicates that the infill was substantially bonded with the base as well as with the column. When load was increased to 15ton, several cracks parallel to the 1st one was observed, supporting this fact as seen in

Fig.3.28. However, at this stage horizontal cracks were developed in the beam at loading side. These cracks are developed due to the tension developed in the column. Development of such crack continued till 18ton load. When the load reached 19ton the frame with infill failed with a major diagonal crack as seen in Fig.3.31. Diagonal cracks were also observed at at top of the column on the loading side and bottom of the column on the leeward side. This indicates that the columns were weaker in shear.

Observing the major diagonal cracks it can be said that the infill failed basically due to diagonal tension, which develop in the direction approximately normal to the loaded diagonal. Also, the more or less straight nature of the crack revels that the crack passed through brick infills. This is clearly visible in Fig.3.32, which shows the damaged infill after the plaster was removed. The reasons behind failure of bricks may be attributed to the fact that the motar was probably stronger than the bricks.

When the frame was repaired, the dimension of the sections of infill and the beam and columns were slightly changed due to the addition of ferrocement coating. In the original frame the plaster was about 12.5mm thick everywhere. In the repaired frame this thickness was about 19mm. When the repaired frame was subjected to loading, cracks started appearing at 16ton load which is higher then the corresponding load for original frame. The failure load for the repaired frame was 23ton at which major diagonal crack occurred at the same location as was in the original frame. This is due to the fact that the cracks in the original frame were not repaired, instead, the whole frame was coated with ferrocement. This left the crack zones weaker than the other parts of the frame. As a result cracks in the repaired frame occurred at the same location, as the cracks appeared in the original frame. It was however observed that the amount of crack opening (width of crack opening) was significantly smaller than the cracks of original frame. This reveals the superior capability of ferrocement in protecting the damaged structure from environment. Fig.3.34 shows the repaired frame after failure in which the smaller width of cracks are clearly visible when compared with Fig.3.31 which is the failed original frame.

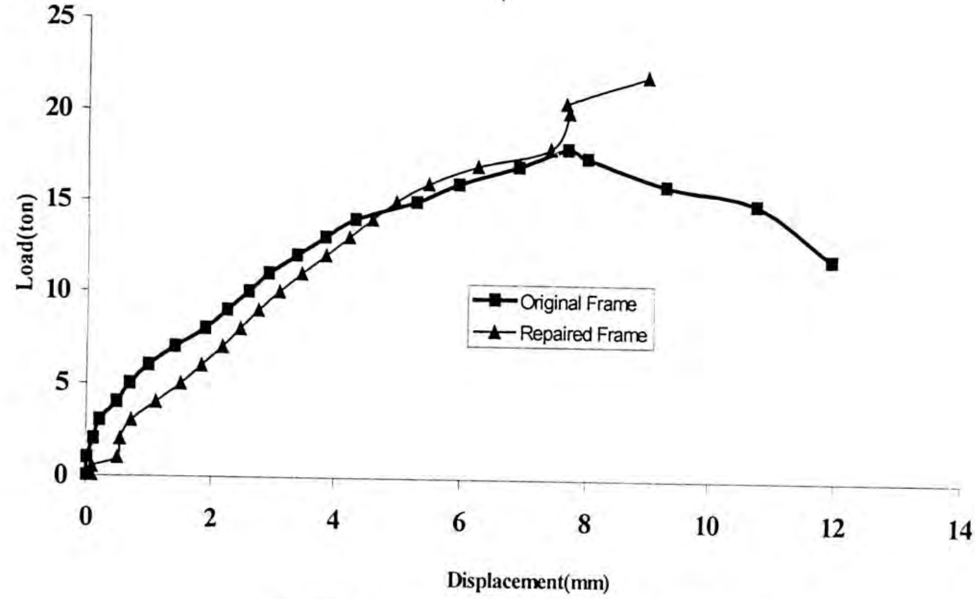


Fig-3.37 Load Vs Deflection at Top of the Right corner of original and repaired Frame

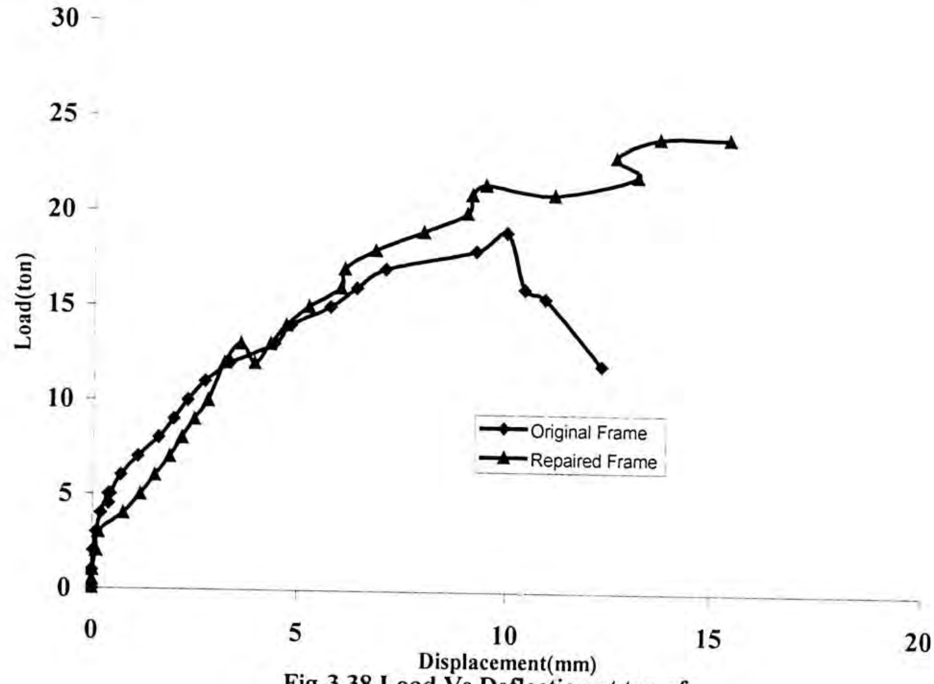


Fig-3.38 Load Vs Deflection at top of the left corner before repair

3.7.2 Load Capacity

Several analytical methods are available in the literature to predict the load carrying capacity of infill. For example the method proposed by Smith and Coull (1991), Saneinejad and Hobbs (1995) etc. Among these methods, the method of Saneinejad and Hobbs (1995) is chosen here to analytically compute the maximum capacity of infill due to its relative simplicity but comprehensive characteristics. This analytical model of infill has been successfully applied by other researcher (Madan *et al.*1997) who demonstrated its capability to reasonably simulate test result. According to this analytical model, the maximum load can be calculated to be 15ton and the corresponding top deflection can be calculated as 5.4mm(Appendix-D). While the actual load at failure was 19ton and the top deflection was 7.68mm. Thus it can be said that the analytical model of Saneinejad and Hobbs (1995) gives conservative results.

After the frame was repaired using ferrocement overlay the tested capacity was even higher (23ton). Comparison of the load Vs displacement characteristics of the original frame and repaired frame are shown in Fig.3.37 and Fig.3.38.These two figures clearly demonstrate the effectiveness of the repair methodology using ferrocement overlay. Science the failure of the repaired frame was diagonal tensile failure it can be inferred that the capacity of repaired frame depended on the tensile strength of the ferrocement. It can be thus said that the combined capacity of the two layers of ferrocement was higher than the masonry. If before applying ferrocement coating, the cracked masonry and column were repaired using epoxy grouting then these components could contribute to the load carrying capacity of the ferrocement. That would have resulted in a much higher load capacity. It can, thus be said that, if ferrocement overlay is applied to any existing undistressed infill then the capacity of the infilled frame will be significantly improved.

Chapter-4

CONCLUSIONS AND RECOMMENDATIONS

4.1 CONCLUSIONS

From the results of the test it can be summarized that ferrocement overlay is a highly effective method of strengthening /repairing distressed reinforced concrete frame with masonry infill. Tested capacity of the repaired frame was more than the capacity of the original frame. Thus it is quit logical to say that if ferrocement overlay is apply to any existing undistressed infill, The lateral load capacity of the frame would significantly be increased. However it should be kept in mind that due to limited scope, only a single frame was tested. In order to generalize more tests are needed to be done. Therefore the findings of the present study should be interpreted with due considerations given to the limited nature of the investigation.

4.2 RECOMMENDATIONS FOR FUTHER STUDY

1. Due to the budget limitation, only one frame was constructed and tested. More frames should be tested to gain more confidence on the findings.
2. Only brick infill was used. Other types of infill may be tested.
3. Only GI wire mesh was used in the ferrocement. Other types of meshes may also be tested.

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APPENDIX-A

Properties of Concrete (used in casting of columns & beam)

Coarse aggregate: 20 mm downgraded brick chips

Making ratio (by volume) = C: FA: CA = 1:2:4

W/C ratio = 0.5

Compressive test Result of Concrete Cylinder (at 7 days):

Specimen No	Cylinder No	Specimen Area (cm)	Maximum loud (kg)	Compressive Strength (MPa)	Avg. Comp Strength (MPa)
1	1	81.0	$14x10^3$	17.22	
	2	81.0	$13x10^3$	16	16.40
	3	81.0	$13x10^3$	16	

N.B: 10.034 kg per sq.cm = 1 MPa

APPENDIX-B

Tensile test Result of Reinforcement (used in beam & column)

SL. No.	Dia. of Bar (mm)	Area of Bar (cm)	Yield load (kg)	Yield Strength (MPa)	Ultimate load (kg)	Tensile Strength (MPa)	Elongation
1	16	2.0	8863.63	441.68	11227.27	559.46	18%
2	10	0.785	2500	317.39	3454.54	438.57	18%
3	6	0.2827	1954.54	689	2545.45	897.35	21%

NB: 10.034 kg per sq cm = 1 MPa

APPENDIX-C

Properties of Cement Mortar Used in Ferrocement:

Type of Cement	:	Ordinary Portland Cement of type 1
Mixing Proportion	:	1: 2.5
Water Cement Ratio	:	0.45
Fineness modulus of Sand	:	2.65

Compressive Test Result for Mortar Cube:

SL. No.	Date of Casting	Age (days)	Specimen Area (sq.cm)	Max^{m} load (kg)	Compressive Strength (MPa)	Avg. Comp Strength (MPa)
1	22.11.02	7	25	8839	35.24	33.81
2	22.11.02	7	25	8259	32.92	
3	22.11.02	7	25	8348	33.28	

N.B: 10.034 kg per sq.cm =1 MPa

APPENDIX-D

Input data for analytical model of Saneinejad and Hobbs(1995):

$h=1575$
$l=1575$
$h'=1500$
$l'=1500$
$\varphi=0.65$
$f_m'=14$
$t=75$
$\mu=0.65$
$\beta_o=0.2$
$M_{pj}=1.9800E+06$
$M_{pc}=9.9000E+06$
$M_{pb}=9.9000E+06$
$\varepsilon'_m=0.002$
$v=6$
And from the formula we have
The maximum capacity, Vm=132052.2 N $\cong$ 15Ton
The top deflection, Um=5.292855mm.

APPENDIX-E

Comparison of Deflection at Different location of Masonry infilled RC frame:

Load applied (KN)	Deflection at original Masonry infilled frame (Dial Gauge Reading) (mm)				Deflection after repairing (Dial Gauge Reading) (mm)			
	At Top		At Bottom		At Top		At Bottom	
	Right	Left	Right	Left	Right	Left	Right	Left
5	0.68	0.37	0.18	0	1.5	1.14	0.18	0.1225
10	2.59	2.28	0.53	0.85	3.1	2.77	0.455	0.49
15	5.29	5.75	1.04	3.63	4.97	5.23	0.64	1.79
18	7.68	9.22	1.14	4.61	7.4	6.82	0.85	1.14
21					7.65	9.1	1.58	2.78

APPENDIX-F

<u>Compression test of Brick Prism</u>

Load Perpendicular to the Joint:

SP. No.	Size	X-section area (cm.sq.)	Maximum load (kg)	Compressive Strength (MPa)	Avg. Strength (MPa)
1	12x6x20	72	12xID^3	16.61	
2	12x6x20	72	$15x10^3$	20.76	18
3	-	-	-	16.61	

Load Paralled to the Joint:

SP No.	Size (cm)	X-area (cm^2)	Maximum load (kg)	Compressive Strength (MPa)	Average Strength (MPa)
1	-	72	$12x10^3$	16.61	-
2	-	72	$10x10^3$	13.84	14
3	-	-	-	13.84	-

APPENDIX-G

Calibration of Hydraulic Jack Machine:

Pump (Ton) gauge	Deflection of load column (mm)	Dial of UTM (kip)	Deflection of load column (mm)	Dial of UTM (kip)
0	0	0		
5	98	8.2	100	8.6
10	227	19.5	232	19.8
15	359	30.8	266	31.5
20	495	42.0	504	42.5
25	627	53.5	640	53.5
30	754	64	663	64
35	882	75.5		
40				
45				

From Column Table:

KN.	Deflection
50	134.6
60	161.1
80	215.6
100	268.3
150	402.0
200	536.4