Study of Various Repair Methods for Reinforced Concrete Flexural Members

by

Ahmad Abdul-Hamid Sallam Gubati

A Thesis Presented to the

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In Partial Fulfillment of the
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In

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Gubati, Ahmad Abdul-Hamid Sallam, M.S.

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U·M·I
300 N. Zeeb Rd.
Ann Arbor, MI 48106

STUDY OF VARIOUS REPAIR METHODS FOR REINFORCED CONCRETE FLEXURAL MEMBERS

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This thesis, written by AHMAD ABDUL-HAMID SALLAM GUBATI under the direction of his Thesis Advisor, and approved by his Thesis Committee, has been presented to and accepted by the Dean of the College of Graduate Studies, in partial fulfillment of the requirements for the degree of MASTER OF SCIENCE IN CIVIL ENGINEERING.

Thesis Committee

Dr. I. A. Basunbul (Chairman)

Dr. G. J. Al-Sulaimani (Co-Chairman)

Dr. M. H. Baluch (Member)

Chairman, Civil Eng. Department

Dean, College of Graduate Studies

Date July 9, 1988

بسم الله الرحمن الرحيم

الاهداء الى الأحبــــــة :

الـوالـدين العزيزيـــن

الأخوة الأعـــــــزاء

الـزوجة والأولاد الأعزاء

الـذيـن شـاركوني في الـمعـانـاة والـتضحية خلال دراستي الـجامعية .

This thesis is dedicated to :

my beloved parents, brothers, wife and children

who sacrificed during my study at KFUPM.

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THESIS ABSTRACT

Name : AHMAD ABDUL-HAMID SALLAM GUBATI

Title of Study : STUDY OF VARIOUS REPAIR METHODS FOR REINFORCED CONCRETE FLEXURAL MEMBERS

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One of the major problems of reinforced concrete structures is cracking due to so many reasons, which helps in creating additional problems such as reinforcement corrosion. Reinforced concrete structures, in Saudi Arabia and Gulf region, have shown signs of deterioration due to the harsh environment, poor quality of aggregate, poor construction practice and the change of function of the structure or faulty design and detailing as a result of the rapid construction in the last few years.

A comparison between repair methods for reinforced concrete members is the aim of this research. Three different repair methods, ferrocement, epoxy resin injection, and steel plate bonding, were studied in the laboratory. A combined method of epoxy and ferrocement methods was also studied. In this research, 30 beams were tested in flexure by applying two points load up to different deflections. Then, these beams were repaired and re-tested up to failure in the same way as the the original ones. During the test, load, deflection, and crack width were recorded.

The comparison of the results obtained from the different methods of repair showed that ferrocement and steel plate bonding gave higher increase in the ultimate load than other methods, while their ductility was low. In the case of epoxy repair method, there was no increase in the ultimate load, but the ductility was the highest among the above three methods. The combined method showed increase in the ultimate load as well as the ductility.

MASTER OF SCIENCE DEGREE

KING FAHD UNIVERSITY OF PETROLEUM AND MINERALS
Dhahran, Saudi Arabia

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خلاصة الرسالة

الاســـــــــــم : أحمد عبد الحميد سلام قباطي

عنوان الدراســـة : دراسة طرق مختلفة لاصلاح أعضاء الخرسانة المسلحـة ذات التصرف العزمي

التخصـــــــص : هندسة مدنية (انشاءات)

تاريخ الشهـــادة : ٢١ يونيو ١٩٨٨م .

من أضخم المشاكل التي تواجهها الخرسانة المسلحة هو تصدعها وفـي المملكة العربية السعودية ودول الخليج المجاورة لها أظهرت بعض المبانـي علامات التدهور السابقة لأوانها ويرجع ذلك الى عدة عوامل منها قسوة الجو ، المواد المستخدمة في الخلط ، أخطاء في التصميم والوصف الهندسي أو تغييـر طبيعة استخدام المباني وقد ساعد على تدهور الخرسانة عدم وجود مواصفـات بناء خاص بهذه المنطقة .

الهدف من القيام بهذه الدراسة هو عمل مقارنة بين طرق مختلفة لاصـلاح الخرسانة المسلحة ، هذه الطرق هي الاصلاح بالاسمنت الحديدي ، الحقن بمـادة الأيبوكسي ، والربط بقطعة حديد بواسطة مادة الأيبوكسي . كذلك طريقة مركبـة من مادة الأيبوكسي ومادة الاسمنت الحديدي قد أستخدمت في هذه الرسالة .

هذا وقد تم اختبار ٣٠ جسرا بطريقة التصرف العزمي تحت قوتين متساويتين عند نهاية الثلث الوسط من الجسر في كل اتجاه . وقد قسمت هذه الجسور الـى ثلاث مجموعات للاختبار ١٠ ، ١٥ مم والنهاية القصوى للانعطاف . وقد تم اصـلاح هذه الجسور بعد تصدعها بأستخدام الطرق المختلفة واختبارها الى النهايـة القصوى للأنعطاف .

من الدراسة والمقارنة تبين أن مادتي الاسمنت الحديدي وربط قطعة الحديد أظهرتا أرتفاع في النهاية القصوى للحمل مع قلة المرونة . كذلك تبيـن أن مادة الأيبوكسي لم تعط أي أرتفاع في الحمل النهائي مع الاحتفاظ بمرونـة عالية غير أن الطريقة المركبة للاصلاح أظهرت أرتفاع في الحمل النهائـي مع الابقاء على مرونة عالية مقارنة بالطرق الأخرى .

درجة الماجستير في العلوم
جامعة الملك فهد للبترول والمعادن
الظهران ، المملكة العربية السعودية

يونيـو ١٩٨٨م

Chapter 1

INTRODUCTION

1.1 General

Building construction has grown very rapidly in Saudi Arabia and Gulf countries in the last two decades and many problems have grown with it. It is known that reinforced concrete is a highly durable material under normal conditions, but concrete structures in the Gulf area have shown earlier signs of distress and deterioration. Some of these problems, in which structures have shown signs of distress within a short time of completion, are caused by impact and dynamic loading, static overloading, shrinkage, creep or thermal gradients, and corrosion of reinforcement and/or sulfate attack. Most of these problems have resulted from the absence of a well defined code of practice which should take into consideration all problems related to local materials and environment. The construction environment in Saudi Arabia is unfavorable for durable concrete structures due to the prevailing severe environmental conditions such as high humidity, high ambient temperature, the poor quality of aggregates and poor construction practice.

At some stages of their lives, concrete structures will show minor and/or major cracks. Minor cracks can be ignored except when the structure is exposed to aggressive conditions which will result in corrosion of reinforcement. But major cracks are indicators of structural problems. So cracks can be classified into three different types :

- Dormant cracks : which are stable and remain as they are, e.g. cracks resulting from shrinkage and drying.

- Active cracks : which are growing in width, e.g. cracks resulting from corrosion of reinforcement or differential settlement.

- Live cracks : which close and open because of loading and unloading. e.g. cracks occuring in a bridge deck or structural members.

The repair of damaged concrete structures has become a challenge to civil engineers. Different repair companies have brought to the market different types of repairing methods and materials, which have not been evaluated in this harsh and arid environmental conditions.

Repair materials and methods may have worked well in the western countries, however they may not be suitable for direct application in regions with poor quality materials and a very aggressive environment like the Gulf region.

In order that new techniques and materials be effective to provide the answer to these types of problems, it is important to know the causes of these structural problems.

The significance of cracks differs from one structure to another and it depends on the type of structure and the nature of the cracks themselves. For the most efficient way of repair, understanding the main causes of the cracks is the first step. The second step is the choice of the method of repair and the last is the selection of the material to be used for that purpose. Otherwise, the repair will have no meaning and it will only be temporary, or may cause further problems.

In this study, three different repair techniques will be investigated as a treatment for cracks which are induced in the reinforced concrete beams. These cracks can be produced by overloading or faulty detailing. The problem of overloading was chosen because it is considered to be one of the problems that concrete structures can be subjected to due to changes of the function of the structure and lack of regulations.

It will be very useful to make a comparison between different repair methods, especially in this area of the world where durability of reinforced concrete structures due to weather conditions and poor quality materials is a major issue. The three different methods of repair studied in this research are ferrocement, epoxy resin injection, and steel plate bonding. Besides, epoxy injection and ferrocement techniques are used as a combined repair method to repair some beams to see the efficiency of combining two repair methods. Now, it remains to define what is ferrocement and what is epoxy.

Ferrocement (FC) : is a type of reinforced concrete in which wire mesh is used instead of rebars and sand rather than sand and aggregate as in the ordinary concrete mix. The term ferrocement implies the combination of a ferrous product with cement. It combines with cement to form a ferrocement. Ferrocement gives high durability, stiffness and strength in spite of its small thickness. Ferrocement often acts more like steel than reinforced concrete. It neither needs skilled workers nor sophisticated equipments for applications.

Epoxy resin : is usually defined as a molecule containing two or more ethylene oxide terminal groups that are capable of polymerization which is achieved by mixing of at least two compounds together; the resin and the hardener. Epoxy resin is a thermosetting plastic, when cured it does not melt but it loses its stiffness at higher temperatures and properties change adversely. Epoxy can withstand a wide range of commonly used chemicals and undergo a considerable deformation before reaching its elastic limit. In terms of cost, epoxies themselves are somewhat expensive, but their total cost often becomes minimal when the quality of repair achievable is high in comparison to other materials or to the cost of new construction.

The results obtained in this research gave good indications of the effectiveness and performance of each method.

1.2 Scope and Objectives

It is known that the replacement of concrete buildings, bridges, dams and other structures is becoming more and more difficult because of time consumption and escalating high costs. Researchers started to think for a solution of this problem in an economical and efficient way. Repairing reinforced concrete beam as a member of the structure is the aim of restoring the integrity of the structure.

The main objectives of this study are as follows:

1- This study will include the evaluation of three different methods of repairing reinforced concrete beams. These methods are epoxy injection, ferrocement, and steel plate bonding in addition to a combined method of epoxy injection and ferrocement.

2- To cast reinforced concrete beams for a general study of repairing reinforced concrete beams.

3- To divide the beams into three groups and test them in flexure under two points load, up to 10,15 mm and ultimate mid-span deflections.

4- To make a comparison for each method of repair at different deflections as mentioned above.

5- To compare all methods in terms of economics, availability in the market, the ease of use, stiffness, cracking and ultimate loads.

6- The overall objective of this study is to include the evaluation of the different methods of repair in terms of strength and ductility for durability problems.

Chapter 2

LITERATURE REVIEW

2.1 General

In the developed countries, construction has been grown step by step based on experience gained from the use of local materials, under the prevailing climatic conditions, and technical expertise. On the contrary, in Saudi Arabia and its neighboring Gulf states, due to the big amount of revenues which came from the increase in the price of oil during the last few years. A rapid construction of buildings, bridges, dams, and other structures has been erected quickly and gave no little chance for gradual evolution to occur in the field of construction. As a result of the ungradual development and absence of codes and specifications of construction, most of the concrete technology and specifications had to be transfered totally through the American, European, Korean, Japanese and Chinese companies which have built most of the largest projects in these countries. The most common causes of concrete cracking encountered in the coastal regions of Saudi Arabia and Gulf states are the internal and surface volume changes due to thermal stresses, sulfate attack, alkali-aggregate reaction, corrosion of steel reinforcement, and moisture movement [1,2].

In addition to the severe conditions in this area, poor control of concrete operations, lack of experience, and lack of local regulations contributed to concrete failures as a result of severe cracking [2].

Condition surveys on structures located in the Eastern province of Saudi Arabia were carried out at the KFUPM. The results of the collected data showed an alarming degree of deterioration with a short span of 10 to 15 years. This deterioration is caused by corrosion of reinforcement, sulfate attack, and environmental cracking [3].

The condition surveys indicated that corrosion of the reinfocement is the most prevalent form of this deterioration [4].

Under such constraints, quality assurance(QA) and quality control(QC) should be expected to play an important role in concrete construction.

Daoud [5] has proved in a project research in the university of Kuwait that an effective QA/QC program is both possible and cost effective in the Arabian Gulf region. He added that in the Gulf region, the traditional conflicts between owners, engineers and contractors are magnified due to cultural and communication problems resulting in a poor quality of construction.

Oliver [6] believes that hot weather concreting is fundamentally a management problem.

For the existing concrete structures which have been damaged due to factors mentioned above, a need for maintenance has become a matter of serious concern in the construction industry in Saudi Arabia and the Gulf states. Since many companies have brought new and different materials for repairing concrete structures to the market without doing any research on the effect of these materials. These materials, even if they worked well in the western countries, may not be useful for concrete repair in the Gulf region. In this case, a research should be carried out to check these materials on the concrete structures with local materials under environmental conditions. Before proceeding to repairs, a feasibility study should be done. When deciding on repairing, four basic steps should be followed to reach a successful concrete repair. These steps are the evaluation of the causes, extent, and consequences of deterioration; selection of repair material; preparation for repair and placing of repair material [7].

Warner [8], after studying some of the material properties, suggested that the ease of application, cost, availability of labor skills and equipments should be considered before the final decision of choosing a material repair.

2.2 Concrete Cracking

2.2.1 Introduction

Concrete cracking seems to be a universal characteristic. They have many causes. In most cases, cracks are unsightly and unharmful and their present has no objection. In other cases, cracking does reduce the usefulness of the structure, and in a few cases it even might require to stop using it or lead to its demolition for public safety reasons.

The most common causes of concrete cracking are the high w/c ratio, high cement content, plastic and drying shrinkage, thermal stresses, sulfate attack, alkali-aggregate reactions, corrosion of steel reinforcement, overloading, moisture movement and differential settlement or expansive soils. Unfortunately, most of the causes mentioned above,if not all, are there in the coastal region of Saudi Arabia [9,10].

2.2.2 Mechanism of cracking

Concrete is the most widely used material in construction but it does crack. Also, it is not very ductile material underload. When the tensile stresses present in concrete member exceed the tensile stress of concrete , then concrete starts to crack. The problem increases when several factors act together forcing concrete to crack or when one factor causes initial cracks that open the way for chemicals to penetrate and react on both faces of these cracks and produce further cracking [10].

2.3 Factors Cause Cracking

2.3.1 Drying shrinkage

Concrete cracking due to drying shrinkage is one of the most important problems encountered in the concrete structures in arid zones. Drying shrinkage is caused by the fast loss of moisture from the cement paste constituent. When concrete trying to shrink, the restraint will prevent it from doing so, resulting in developing tensile strains which will lead to concrete cracking (the restraint of the concrete usually provided by another part of the structure or by the subgrade soils). In the case of massive concrete structures, tensile stresses are caused by differential shrinkage between the surface and interior concrete. During continuous drying of the concrete surface cracks penetrate deeper into the concrete.

The amount of drying shrinkage is mainly affected by the amount of water content and aggregate in the mix. The amount of drying shrinkage comes from the higher amount of water content and/or the less amount of coarse aggregate in the mix. drying shrinkage can be reduced by using the maximum practical amount of aggregate in the mix and low w/c ratio.

Also, it can be controlled by using properly spaced contraction joints and proper steel detailing [11].

2.3.2 Settlement of fresh concrete

Cracks may develop in concrete just after finishing due to the restraint provided by the reinforcing steel bars during the consolidation of concrete. This local restraint may result in voids and/or cracks adjacent to the reinforcing steel. The tendency for settlement cracking to occur decreases with increasing cover, small bar size, and lower slump as shown in Fig. 2.1 [12].

2.3.3 Thermal stresses

Temperature differences within a concrete structure may be due to cement hydration (in mass concrete) or changes in ambient conditions (in any structure) or both. These temperature differences result in differential volume changes. The tensile stresses created by the expansion and contraction due to the change of temperature, from maximum in summer and minimum in winter or maximum in the day time and low at night, causes cracking of concrete especially when resulting thermal stresses exceeded the tensile strength of the concrete.

The wide variation in both temperature and humidity created by the climatic conditions in this region has

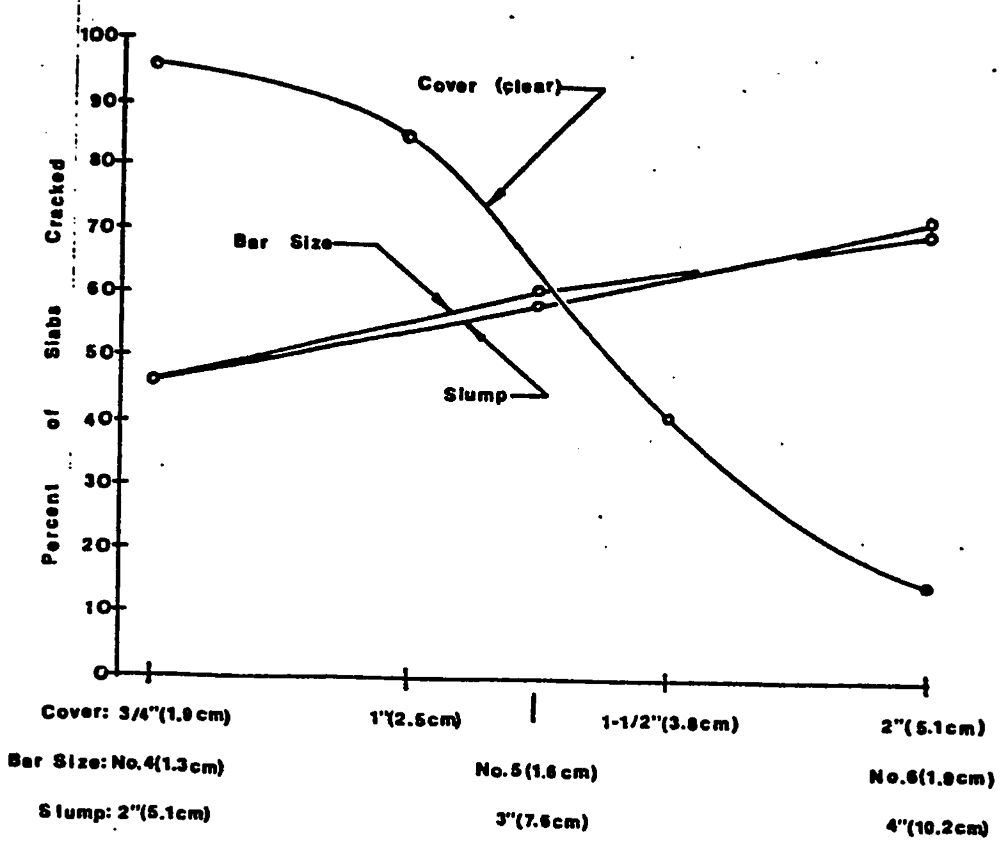

Fig. 2.1 : Cracking as a function of bar size, slump, and cover.

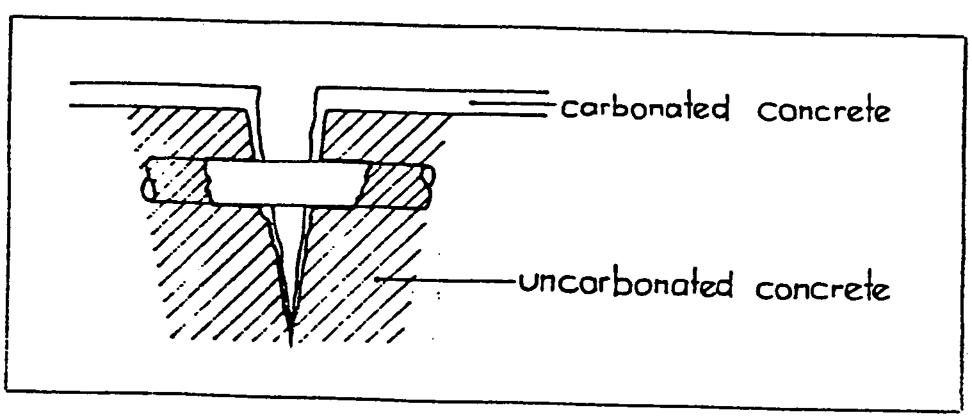

Fig. 2.2 : Carbonation in an open crack.

resulted in extra loading conditions than the normal design ones. Sharp thermal gradients are applied to concrete in the Gulf states due to the extremely hot ambient environment in the summer months on the outside surfaces and the local air conditioned environment on the inside. Also, due to the sharp and rapid daily fluctuation in temperature during summer days when external surfaces are subjected to maximum temperature at noon and a minimum temperature at night. The consequences of harsh climate in the Gulf region is more severe than elsewhere in the world [13].

2.3.4 Corrosion of reinforcement

Concrete normally provides reinforcing steel with excellent corrosion protection due to the natural alkalinity of concrete (PH = 12.5). At this PH, a protective film of gamma ferric oxide forms on the steel surface, effectively inhabitating the corrosion process. There are many factors influencing corrosion of reinforcement, but the two main causes of reinforcement corrosion are carbonation and the presence of chlorides.

For corrosion to occur either the alkalinity of the concrete must be reduced due to carbonation or the present of chloride ions. Since moisture is necessary for carbonation to occur, it is a problem more generally associated with humid climates. The most rapid occurance being where the humidity ranges between 50 and 75 % [14].

The passivity created by the highly alkaline environment in the cement paste can be destroyed by the carbonation of concrete. Atmospheric carbon dioxide reacts with the alkaline medium forming carbonates, thereby reducing the PH value as low as 8.5. With good quality of concrete and an adequate cover, the rate of carbonation is slow. However, for any reason, a crack opens in the concrete, the carbonation process starts again from the crack surfaces, and may reach a point near the steel, Fig. 2.2, [15]. At this instant, the alkalinity of the concrete is lowered and passivation is lost. In consequence, corrosion starts.

The chloride ion, being a specific destroyer of the protective film, is especially effective in eliminating passivity against corrosion. Corrosion caused by the presence of chlorides is much more insidious, because chloride ions are already there in the concrete (they may be in the aggregate or the water used for the mix) since the day of placing. This type of corrosion has affected buildings in the Middle East and some other arid areas in an alarming rate.

Cracks in the concrete will maximize the penetration of corrosion inducing agents such as oxygen, chloride ion, carbon dioxide, and water. When steel reinforcement corrodes, its volume increases and causes expansion of the concrete. This expansion will lead to concrete cracking, and the steel cross-section will decrease causing a reduction in the tensile capacity of the steel, which eventually lead to failure of the structure [16,17,18].

2.3.5 Chemical attack of concrete

Solid salts penetrate into concrete react with hardened concrete paste in the present of magnesium and calcium sulfates in solutions. The sulfate reacts with $Ca(OH)_2$, calcium hydroxide, and with calcium aluminate hydrate and starts attacking the concrete.

$Ca(OH)_2$ is abundantly present in hydrated portland cement owing to the hydration of its major components. The products of the reactions are gypsum and calcium sulfo-aluminate (ettringite) which occupy larger space than the chemicals that formed during the hydration process which leads to expansion and disruption of the concrete, that result in cracking. Also, $MgSo_4$, Magnesium sulfates, have less effect on the attack of concrete, because of the very low solubility, but under certain conditions the attack will be more and lead to serious deterioration of concrete [19].

2.3.6 Externally applied load

It is well known,in concrete design, that load-induced tensile stresses result in cracks in concrete members. The design procedures (ACI 318 and AASHTO Standard Specifications for Highway Bridges) use reinforcing steel, not only to carry the tensile forces, but to obtain both an adequate crack distribution and a reasonable limit on crack width. Crack patterns and crack widths have been investigated in detail for cracks associated with tensile and flexural stresses. However, shear and torsion may also cause significant cracking. Flexural and tensile crack widths can be expected to increase with time for members subjected to either sustained or repetitive loading.

Well- distributed reinforcing offers the best protection against load-induced cracks. The use of larger amount of steel will reduce the steel stress and the amount of cracking. While a reduction of cover will reduce the surface crack width but it will not be enough to protect the reinforcing bars from corrosion. Engineers should distinguish between longitudinal and perpendicular cracks to the reinforcing bars. Perpendicular cracks to reinforcing steel do not have a major effect on the corrosion of that steel, while longitudinal cracks are always dangerous as corrosion is concerned [20].

2.3.7 Construction overloads

Loads induced during construction can be far more severe than those experienced in service. Unfortunately, these conditions may occur at early ages when the concrete is most susceptible to damage and often result in permanent cracks. Precast members, such as beams and panels, are most frequently subjected to this abuse, but cast-in-place concrete can also be affected. Cast-in-place is subjected to this loads in cold climates when heaters are used to provide an elevated working temperature within a structure. If these heaters are of high volume and located near exterior concrete members, especially thin walls, an unacceptable high thermal gradient can result within the members. The interior of the wall will expand in relation to the exterior. To avoid this problem, heaters should be kept away from the exterior walls.

Storage of materials and the operation of equipment can easily result in loading conditions during construction far more severe than any load for which the structure was designed. Load limitations, should be given to the construction supervisors in order to prevent the problem from taking place (20).

Another cases of cracking are related to errors in design and detailing, and to poor construction practices. The importance of proper design and detailing depend on the particular structure and its loading. Special care is needed during the design and detailing of any structure to avoid major problems of cracking.

Also, conditions inspection is needed during all phases of construction as a check of the proper design and detailing. There are well known methods in the ACI manuals to prevent cracking of concrete structures due to poor construction practices, but they should be followed by both the contractor and the owner's representative with special attention in order to insure a proper execution.

2.4 Methods of Repair

2.4.1 Ferrocement

Ferrocement is a highly versatile form of reinforced concrete made of hydraulic cement-sand mortar matrix reinforced with closely spaced layers of continuous and relatively small wire diameter mesh, Fig. 2.3, [21].

Ferrocement exhibits a behavior very different from that of reinforced concrete in performance and strength due to the uniformly distribution of the wire mesh in the matrix throughout the thickness and all over the area. Ferrocement reinforcement can be assembled into its final designed shape and mortared or plastered directly in place.

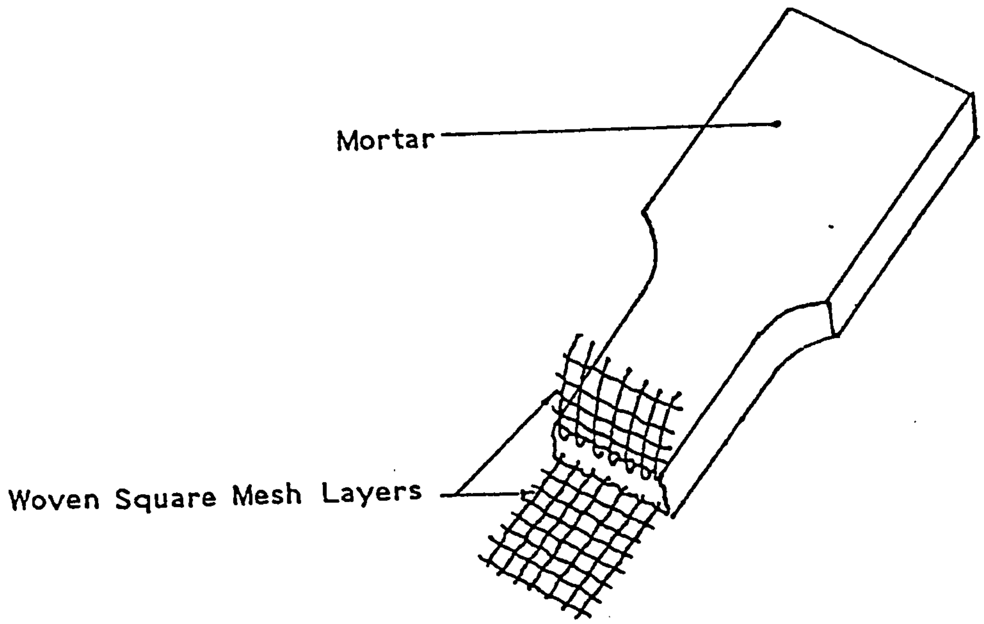

Fig. 2.3 : A typical ferrocement specimen reinforced with 2 layers of woven square mesh.

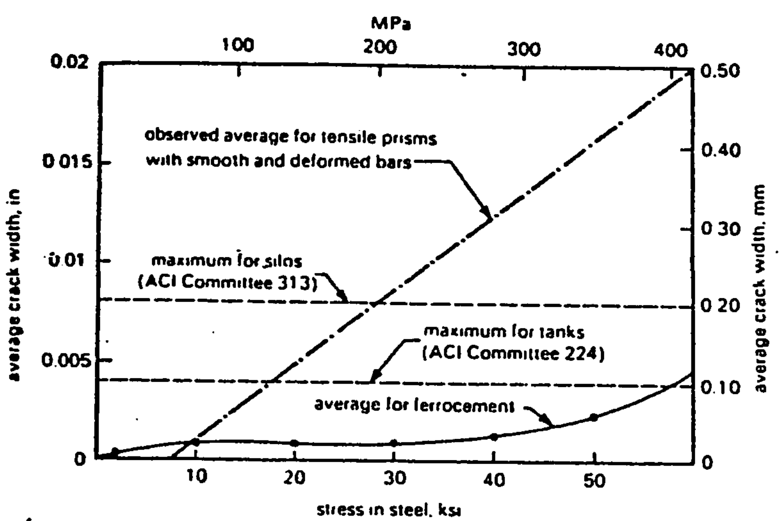

Fig. 2.4 : Comparison of average crack width verses steel stress of ferrocement and reinfoced concrete.

History of ferrocement

The earliest attempt of using ferrocement in construction was done by a French called Jean Louis Lambot. He constructed a boat in 1848 using ferrocement. But because of the time and effort required to construct ferrocement and where technology was very poor, to make mesh of thousands of wires, the concept of ferrocement was almost forgotten for more than one century. In the early 1940s, Pier Luigi Nervi, Italian engineer demonstrated the utility of ferrocement as a building material. Nervi built a small storehouse of ferrocement in 1947. In the early 1960s, ferrocement was accepted in the United Kingdom, Newzealand and Australia, as a boat-building material. However the universal availability of the basic component materials of the ferrocement and because of the unskilled labors used for construction, ferrocement got wide applications in the developing countries. In 1972, the National Academy of Sciences set up the Ad Hoc Panel on the utilization of ferrocement in developing countries. In 1974, the American Concrete Institute (ACI) had set up the committee 549, ferrocement, to review and report the factors affecting the use of ferrocement in construction. The committee state-of-the-Art report on ferrocement was published in 1982.

In 1976, International of ferrocement Information Center (IFIC) was found at the Asian Institute of Technology, Bangkok, Thailand and published the Journal of ferrocement every three months with the help of the New Zealand ferrocement Marine Association (NZFCMA). In the last two decades, ferrocement has become important and useful material in buildings, roofing system, tanks and irrigation structures for both the developing and developed countries.

In Saudi Arabia, in 1983, a group of researchers at KFUPM, carried out a research project on the mechanical properties of ferrocement which was supported by King Abdul-Aziz City for Science and Technology (KACST). The increase in the use of ferrocement in construction was because it has high durability, ductility, and strength if properly shaped in addition to the unrequirement of skilled labors and the availability of materials [21,22].

The Use of ferrocement in construction

Ferrocement is suggested by Romualdi [23] as a material of many paradoxes from the fact that ferrocement posses a degree of toughness, ductility, durability, strength and crack resistance that is considerably greater than that of reinforced concrete. Further more, these properties are achieved in structures with a thickness not exceeding 25 mm which is not a practical one in the use of ordinary concrete for construction. Another paradox of ferrocement is to be a forgiving material through the high levels of performance in ductility, strength, and other properties which can be achieved ever without a very good quality control.

It is known that good quality control leads to better quality and performance, but in ferrocement surprising good performance can be achieved with almost primitive field conditions and unskilled labors. Ferrocement has its roots in the developing countries of the world than in the developed countries as a new material for boats, roofs, tanks, and housings. The fact that relatively unskilled labor can construct very serviceable structures has given the material the reputation of being labor intensive which is not favorable in the industrialized countries. The main applications of ferrocement in construction are in boat buildings [24], water tanks [25, 26], and roofs[27].

Ferrocement construction can be divided into four phases:

* fabricating the skeletal framing system,
* applying rods and mesh,
* plastering, and
* curing. [28]

Experience has shown that the quality and application of mortar are very critical in the use of ferrocement in construction. Mortar can be applied by hand or by shotcreting. Ferrocement is very suitable for construction of curved surfaces such as domes [29], shells and free-form shapes, because of the unneed of formwork.

Ferrocement as a repair material

Ferrocement provides higher tensile and flexural strengths with numerous fine, cracks at an average width of less than 0.01 mm which is very small compared to the reinforced concrete which develop few but wider cracks, as shown in Fig. 2.4 .

Ferrocement can withstand thermal changes very efficiently and since it is made of the same material which makes up reinforced concrete, thus making it thermally compatible. Ferrocement can obtain very good bond when laid over surface of concrete or brickwork. Most of the ferrocement structures do not require a water proofing treatment, because of its highly crack resistance and imprevious surface.

Ferrocement provides a surface free from danger of cracking and offers a highly imprevious layer, it can be safely adopted for water proofing treatment of structures constructed with reinforced concrete. The use of ferrocement as a repair material in developing countries has been very limited to few jobs, because of the lack of industrial use, the lack of knowledge of ferrocement behavior and its reputation of being labor intensive.

The first reported user of ferrocement as a water proofing layer was Architect Jorn Utzn, many years ago, over reinforced cement concrete roof of New Sydnay Opera House in Australia. The Structural Engineering Research Center, SERC, Roorkee (India) carried out field and laboratory investigations and from these investigations and others, SERC recommended the use of ferrocement for water proofing of old and new structures.

More than a dozen buildings including factory, offices, and residences have been water proofed with ferrocement and since then all these structures have been performing very well. Also, ferrocement has been used, in India, for water-proofing treatment of roofs and basements [30,31].

Ferrocement is known as a material of high impermeability and ease of application on any surface. Due to such reasons a leaking 50,000 gallon reinforced concrete overhead tank was treated with ferrocement lining. This tank was constructed by the Military Engineering Services (MES), in 1971 at Roorkee Cantonment (India), and was abandoned after sometime due to the very heavy leakage through the wall of the tank.

In 1982, (SERC), Roorkee suggested to (MES) a treatment which could be applied to bring this tank back to its serviceability condition. After studying the problem in detail, the center suggested the use of ferrocement for lining the tank from inside after treating the cracks and honeycombed areas. Since then, the tank is still in use.

Ferrocement was suggested to be applicable for rehabilitation projects from water treatment plants to tunnel linings. In the United Kingdom, ferrocement has been used for relining a sewer system of 60 m in length, with 2.4 m high and 2.6 m wide. Although the ferrocement layer thickness was only 35 mm, it was claimed that the addition of lining was capable of supporting half of the loading from London traffic on the Counters Creek Sewer. The great advantage of using ferrocement for relining the sewer is the significant increase in the strength which can be achieved from using additional material provided that ferrocement is simpler to install than most other linings. Application of the mortar matrix was by shotcreting [32].

Recently, ferrocement has used in relining several deteriorated swimming pools. This is because it provides a tough, watertight surface in addition to being cost-competitive solution. The commercial viability of ferrocement for rehabilitating water containing structures rests upon several recent innovations. The most significant is a three-dimensional mesh that was originally developed in New Zealand to avoid the problem of using multiple layers of conventional mesh in a very thin layer of ferrocement see

Fig. 2.5, [33], which is a total of five thicknesses of wire from one side to the other. In case of the lining for a rehabilitated swimming pool or other water containing structures, where bending strength of secondary importance of toughness, durability and flexibility, only one layer of the three-dimensional mesh is necessary. To insure complete encapsulation of the mesh by the mortar matrix, however, the ,mesh must be separated by some small distance from the original pool surface. This is accomplished by means of round plastic disks about 1" (25 mm) in diameter and 1/4" (6 mm) thick that are glued to the original wall surface before installation of the single layer of mesh. The mesh is then fastened in place by means of powerdriven fasteners. The fastener is first placed through a washer and then shot through the mesh and through the plastic spacers.

In case of swimming pools, the final coating is often white cement with crushed aggregate. Painted surfaces are also common. The steps procedure are applied to the deteriorated swimming pool at the Veterans Administration Hospital in Pittsburgh, Pennsylvania and the municipal pool wall in Richmond Heights, Ohio, in the U.S.A. . The plastering of ferrocement can be done by hand, and a vibrating trowel is recommended especially for small projects. However, for large projects, shotcreting has been found to be effective. It was used for the relining of the

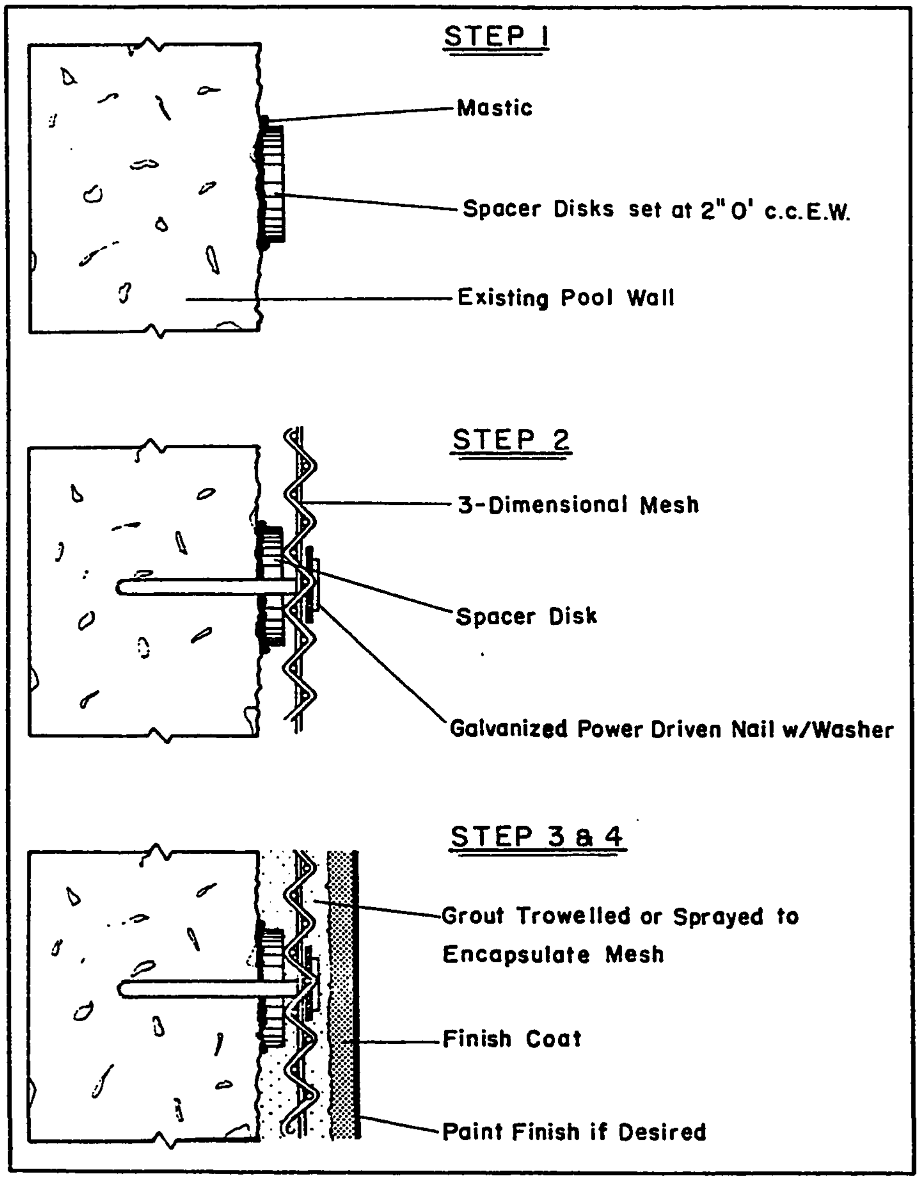

Fig. 2.5 : Steps in the application of a ferrocement liner.

municipal pool in Richmond Heights.

2.4.2 Epoxy resins

Introduction

Epoxy resins have been used very widely for many applications all over the world to repair buildings, dams and bridges for more than two decades. There are so many papers written on the use of epoxy resin for repairing, strengthening and rehabilitation of structures. Results showed a good performance of epoxy materials used to repair concrete structures.

History of epoxy

The word "epoxy" is a Greek description of the chemical symbol for the family of epoxies. The first practical application of epoxy resins took place in Germany and Switzerland in 1930s, with some experiments were done in the U.S. .

In 1939, Greenlee developed several basic epoxy systems, some of them still used until today. First interest in the use of epoxy in the construction industry as an adhesive was in 1948 and it showed satisfactory results in bonding two pieces of hardened concrete.

In the early 1950s, epoxy resin adhesives were available in the market for use. The U.S. Army Corps of Engineers published the first Federal Specification for an epoxy resin system in 1959. Since then, the use of epoxy resin adhesives expanded in many directions.

Today, the need of epoxy resins have been increased due to economy and performance of the material [34].

Selection of the material and material properties

Schutz [35], 1981 , listed the properties and specifications for epoxies used in concrete repair. It is just a summary of standard tests and specifications which are adopted by the American Society for Testing and Materials (ASTM).

These standard specifications were published in 1978 and they are :

- ASTM C881 on epoxy-resin-base bonding systems for concrete.

- ASTM C882 on bond strength of epoxy resin systems.

- ASTM C883 on effective shrinkage of epoxy resin systems used with concrete.

- ASTM C884 on thermal compatibility between concrete and epoxy resin overlay.

Fattuhi [36], in 1983, started a research, at the university of Kuwait, on proposing two simple techniques for testing the performance of repair materials for concrete cracks. In order to study the effect of resin injection of the cracked concrete. Steel moulds were prepared and steel plates were placed at the center of the mould to form a groove (simulated crack) in the beams for flexural test. Cracks were filled with resin injection and epoxy mortars and two beams were tested in flexure. Also, halved concrete cylinders were joined by repair material and tested in splitting tension. The beams and cylinders were subjected to different temperatures before testing. The results showed a reduction of stress in the concrete repair materials at high temperature.

Plecnik and a group of four researches [37] studied factors affecting the epoxy penetration (temperature of epoxy and concrete, epoxy injection pressure, viscosity of epoxy, wetability of epoxy and crack size. They found that viscosity, pot life and wetability were the most important properties which have a significant effect on the epoxy penetration. The results of the study, showed that epoxy adhesives with long pot life and low viscosity provided the optimum results for crack penetration and rebonding of reinforcing steel.

Also, Plecnik and two separate groups studied the behavior of epoxy repaired beams under fire.

The first group [38] discussed the effects of temperature and fire on epoxy repaired concrete beams. About 200 beams (both rectangular and T-sections with small-and-large scale) were prepared for the test. A concentrated load at mid-span was applied until failure, then repaired by epoxy resin injection, only the high penetration epoxy adhesives were used. The beams were then subjected to two standards fire exposures, the 2-hr ASTM E-119 or 1-hr Short Duration High Intensity (SDHI). The results showed that, for flexural type epoxy repaired cracks without compression zone failure, the ultimate residual strengths of the beams are not significantly affected by epoxy repair but the stiffness is greatly reduced due to temperature rises.

The second group [39] investigated only nine beams, specimens of rectangular and T-sections. The beams were subjected to the 2-hr ASTM E-119 or the 1-hr SDHI fire exposure after having beam repaired with the epoxy adhesives. The effectiveness of a plaster coating applied to epoxy repaired beams was determined. The results showed that the residual deflection is very much affected when using a one inch plaster coating but not the ultimate residual strength at initial failure cracks for beams exposed to fire.

The use of epoxy resins in the repair of the structure

Chung [40], 1975, carried out a research on epoxy repaired of reinforced concrete beams. Three reinforced concrete beams were used for the test, and two point loads were applied to the middle third of the beams, Fig. 2.6 . Loading was increased by increments until failure, then major cracks were repaired by epoxy resin injection. The behavior of the repaired beams was similar to that of the original ones, and the repaired cracks did not open. Instead, new cracks were formed and some of them were just adjacent to old cracks. Also, the ultimate load was greater in the case of repaired beams than the unrepaired ones, Fig. 2.7 .

Also, in 1977 , he and Lui [41] carried out another research on the use of epoxy to repair concrete joints. In this research, shear tests were carried out on concrete push-off specimens which were first tested until failure and then repaired by epoxy injection. The results of this research proved that the use of epoxy resins to repair concrete joints was very effective.

In 1985, Mansur and Ong [42] carried out a research on repairing reinforce concrete beams, each with a large transverse rectangular opening, using epoxy injection. The opening caused the beams to fail by crushing of the concrete

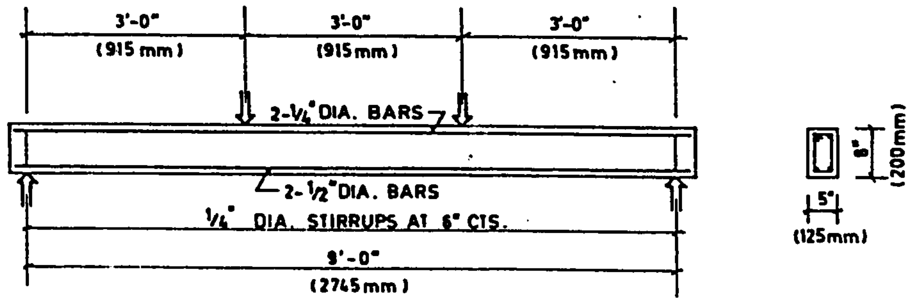

Fig. 2.6 : Details of Chung test specimens.

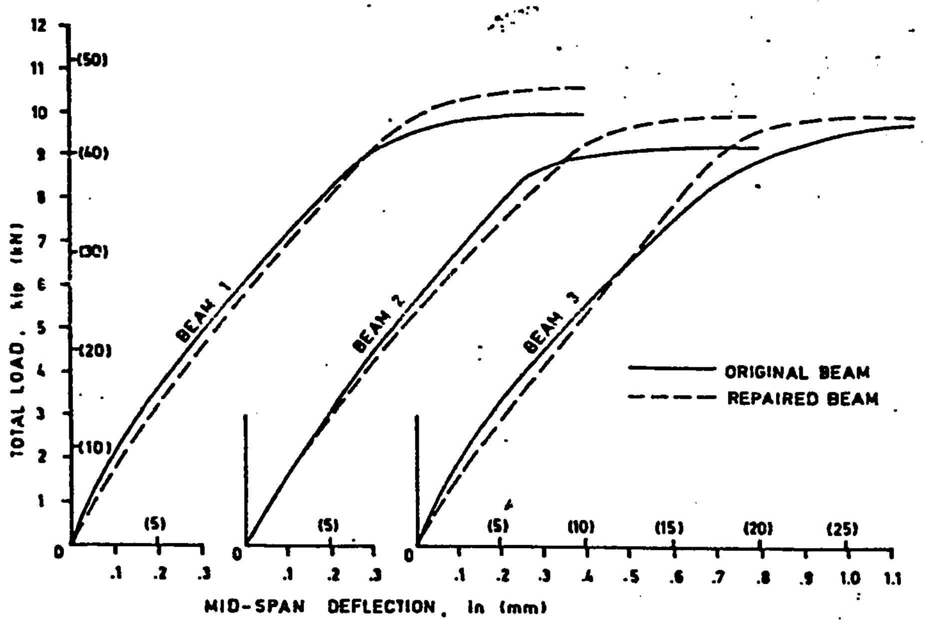

Fig. 2.7 : Load-deflection curves of original and epoxy repaired beams by Chung.

at the four corners of the opening. Six reinforced concrete beams were tested until failure under a static point load. The crushed concrete was replaced by epoxy mortar and the cracks of the beams were then repaired by epoxy injection. The repaired beams were tested the same as the original ones. Comparing the results of the test between the original and the repaired beams in terms of deflection, mode of failure, cracking and ultimate strength. The results showed the effectiveness of the repair technique.

In 1978, Chung and Lui [43] investigated the effect of repairing concrete joints under dynamic loads. Some shear tests using push-off specimens (two series of eight in each) were carried out in order to achieve that purpose using epoxy injection. The results showed that the repaired joints can reach a shear strength and absorb the same amount of impulse as the original ones.

In 1982, a research carried out by Hewlett and Morgan [44] on the static and cyclic response of reinforced concrete beams repaired by resin injection. Ten beams were used in this study. The beams were designed to fail both in tension and in shear. These beams were tested to failure then repaired and retested. A concentrated downward load at mid-span was applied. The results showed some limitations to the effectiveness of the epoxy materials due to the crack width. For the beams with diagonal shear cracks, the repaired beams were often stiffer and stronger than the original beams. For the beams with tension type, good results were achieved when maximum crack width was not greater than about 1 mm and the repaired beams exhibited a failure load but not the stiffness.

Amon and Snell [45] suggested the use of pulse velocity techniques to monitor and evaluate epoxy grout repair to concrete. This technique involves the use of a soniscope and two transducers, one transducer will send the wave through the concrete and the other one will receive it. Then the soniscope can determine the pulse velocity. If the pulse velocity is almost the same as that of the uncracked concrete, the injection is accepted; otherwise, the crack should be re-injected. This technique is very useful in replacing the need of coring for checking the effectiveness of the repair material.

2.4.3 Steel plate bonding

Introduction

Sometimes it is necessary to strengthen existing concrete structures due to the changing function of the structure or because of under strength due to design or constructional faults. In the past, concrete bridges were strengthened by additional beams or props. In recent years, new technique has been developed to increase the amount of reinforcement by bonding steel plates to the surface of the concrete using epoxy resin, which is very benefitable and it sometimes be an economically attractive solution. Advantages of this technique are the minimum reduction of the headroom and the minimum disruption of traffic. However, there are some disadvantages like the little advance warning of the sudden failure after the separation of the plate, and the required effort for the preparations to achieve a high quality bond between the steel and the concrete.

History of steel plate bonding

The use of bonded steel plates as additional external reinforcement to concrete structures was first tried in France and South Africa in the late 1960s and early 1970s. It has also been used in other countries including Switzerland, United Kingdom, and Japan where at least 240 bridges had been plated by 1975 [47,48].

The large number of under-strength bridges was mainly due to a big intensity of heavy truck traffic since the bridges were designed.

Research and applications of steel plate

Irwin [46], in 1975, studied the use of this technique for strengthening beams in flexural where he prepared two reinforced concrete beams, for the purpose of comparison,one with bonded steel plate and one without. After testing the beams up to failure, the results showed that the crack widths of the plated beam were significantly reduced to almost half of those on the unplated beam, while the moment capacity does not increase much, see Fig. 2.8 .

In 1978, Macdonald [47] studied the use of steel plates for strengthening four reinforced concrete beams. The plates were bonded to the tension flanges of the beams by epoxy-resin. This study covers the effects of the change of adhesives, joint in the plate, the change of plate thickness and load cycling. The results showed that the load to produce a plated beam was approximately double the load to produce the same size-crack in the unplated beam, but no much increase in load carrying capacity. From observation, in all cases, failure to the plated beam occured by horizontal shear in the concrete adjacent to the steel plate commencing at the free end. Also, after plate separation, subsequent failure usually occurs by compression of the concrete at a load similar to the failure load of an unplated beam.

In 1980, Raithy [48] carried out a research on the strengthening and load testing of four bridges at an interchange on the M5 motorway at Quinton, United Kingdom, and summeries relevant research tests being carried out at the Transport and Road Research Laboratory. Full-scale

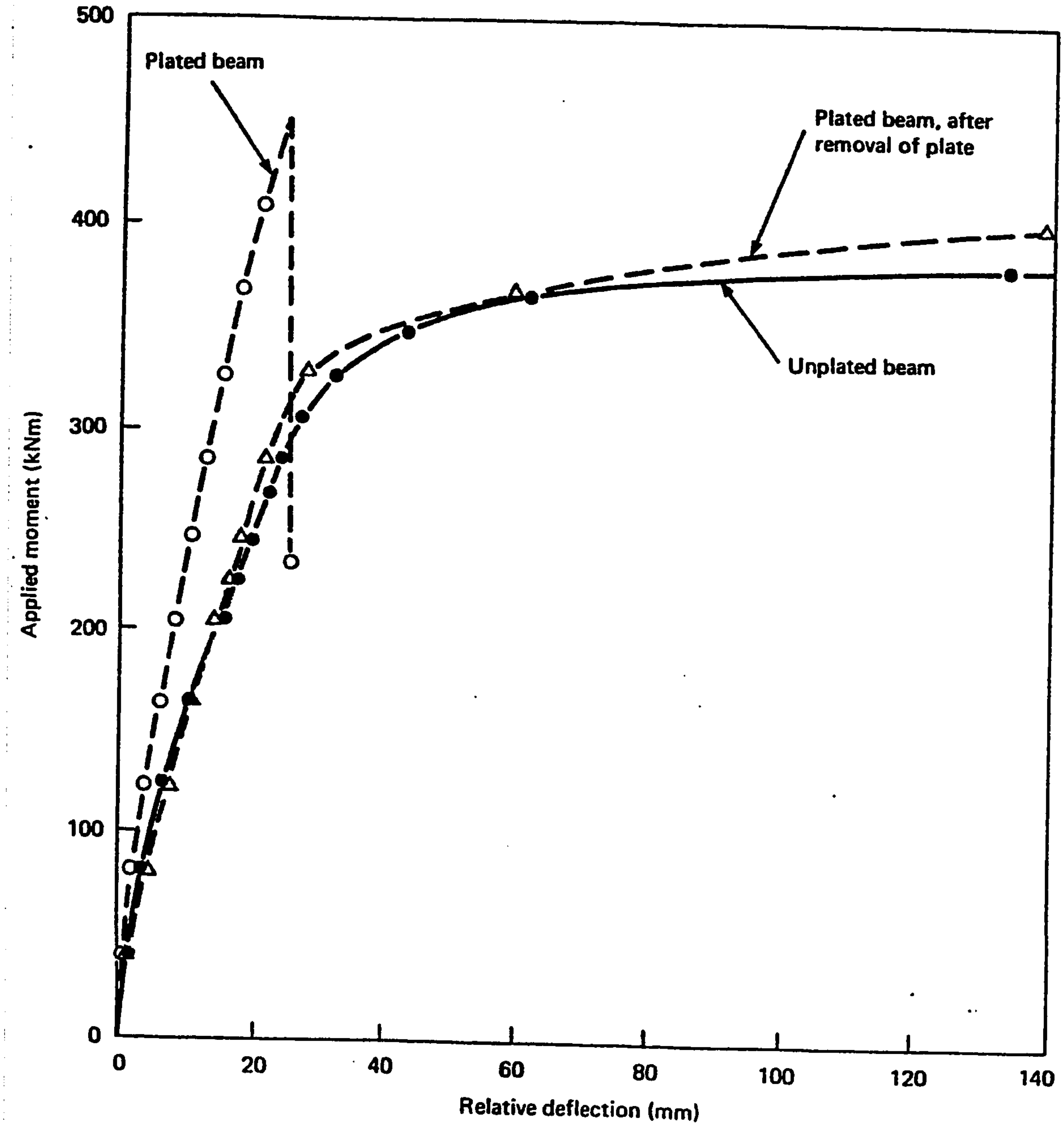

Fig. 2.8 : Plated and unplated beams; relative deflections.

loading tests on one of the bridges before and after strengthening showed that the required improvements had been achieved. Also, laboratory tests indicated that performance could be improved further by better detail design of the plates and further research is needed for future applications.

For a long-term behavior, a protective treatment of the steel surface against corrosion is very essential. For the objective of studying the long-term behavior of reinforced concrete structures strengthened with externally bonded reinforcement, the Swiss Federal Laboratories for Materials Testing and Research (EMPA) started, in 1973, a special program of testing reinforced concrete beams strengthened by externally bonded steel plates, with a planned observation period of at least 15 years.

Reference [49] has been provided by the initial information obtained after one, three and five years of this study. The different parameters have been studied in this research are the magnitude of the load, the type of weathering and the type of corrosion protection. The results showed that no significant damage has been occupied in comparison with the original conditions.

Chapter 3

EXPERIMENTAL PROGRAM

3.1 Preparation for Testing

Preliminary design of beams

First: 3 beams of 150 * 150 mm in cross section and 1000 mm long were prepared along with 3 cylinders of 75 * 150 mm. Beams and cylinders were tested after curing of 28 days. The cracks in the flexural zone were very fine at ultimate load, which would be difficult to repair by injection.

Second: the main steel reinforcement of the beam was reduced from 2-12-mm diameter bars to 2-10-mm diameter bars and the beams dimensions were kept the same. The beams of this group were tested after curing of 28 days. Two of these beams were repaired by epoxy injection and results were acceptable, but cracks at 10 and 15 mm deflection were still too fine to repair by epoxy injection.

Beams final design

The dimensions of the beam were fixed to 150 * 150 mm in cross section and 1250 mm long, with a shear span to depth ratio greater than 2.5 to assure wider flexural cracks at the central zone of the beam. The main reinforcement was

2-10-mm diameter reinforcing bars, with top reinforcement of 2-6-mm in diameter, 6-mm stirrups were used as shear reinforcement at 60 mm spacing all over the beam with 30 mm cover from all sides, as shown in Fig. 3.1 . The final design of the beam is given in Appendix-A.

Moulds

Six steel moulds were fixed to a large steel plate and prepared for casting at the company yard. Also, the preparation of steel stirrups tied to the reinforcement was made before the day of casting, as shown in plate 3.1 .

Materials

The mix design used was as follows :

cement	400 kg/cu. m	Saudi Bahrain Type V
Coarse aggregate		
(18.75 mm max)	1073 kg/cu. m	Abu Hadrihah
Fine aggregate	651 kg/cu. m	Local dune sand
Free water	192 kg/cu. m	Local
Air content	2.0 %	

Casting of beams

In order to get uniform supply of concrete mix for each group, the beams were cast in Al-Moraba Contracting and Construction Co. in Dammam in groups. Each consisted of 6 beams. Before casting, the plastic spacers of the

Reinforcement of the Beam

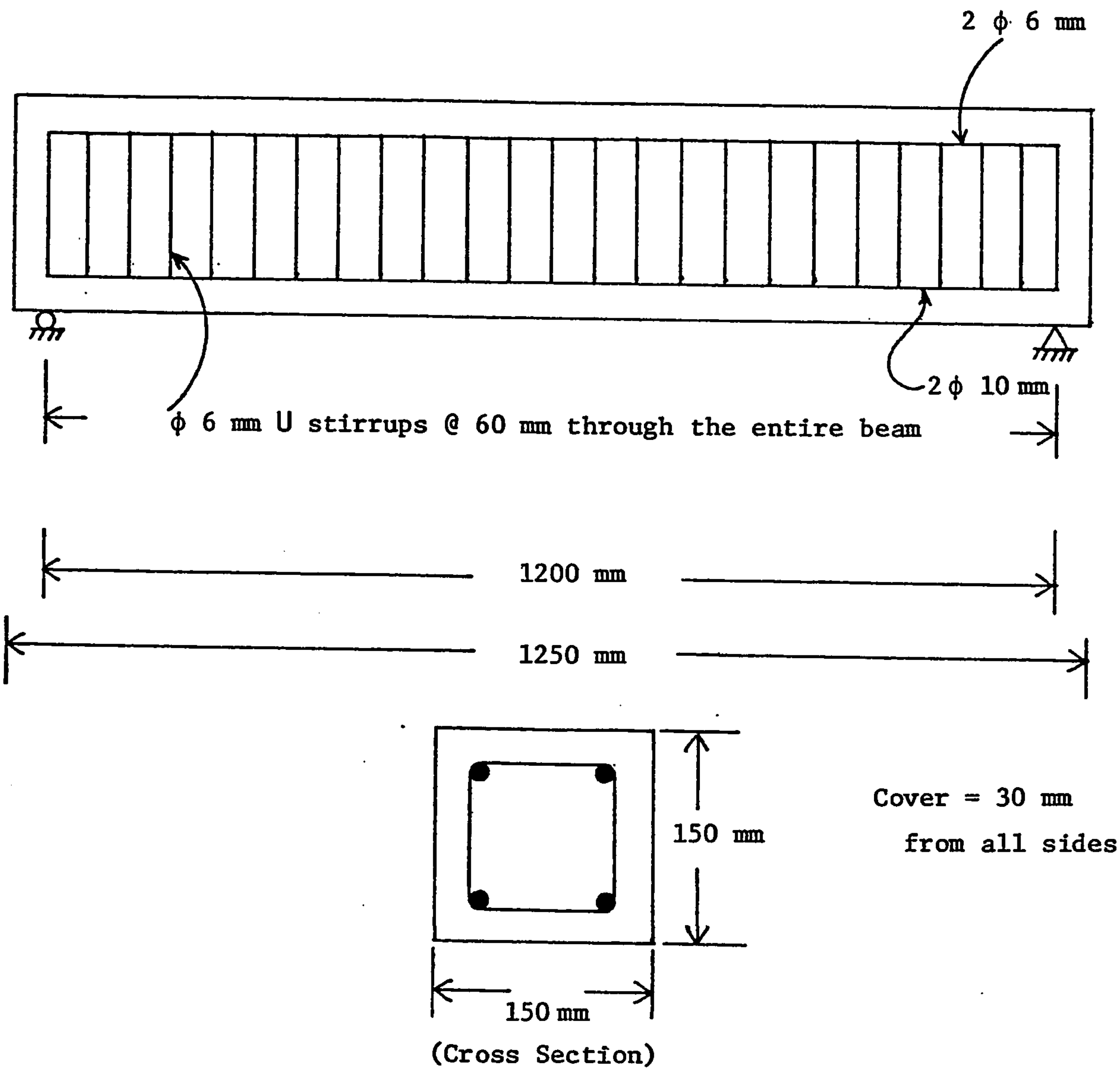

Fig. 3.1 Reinforcement Detail of the Beam

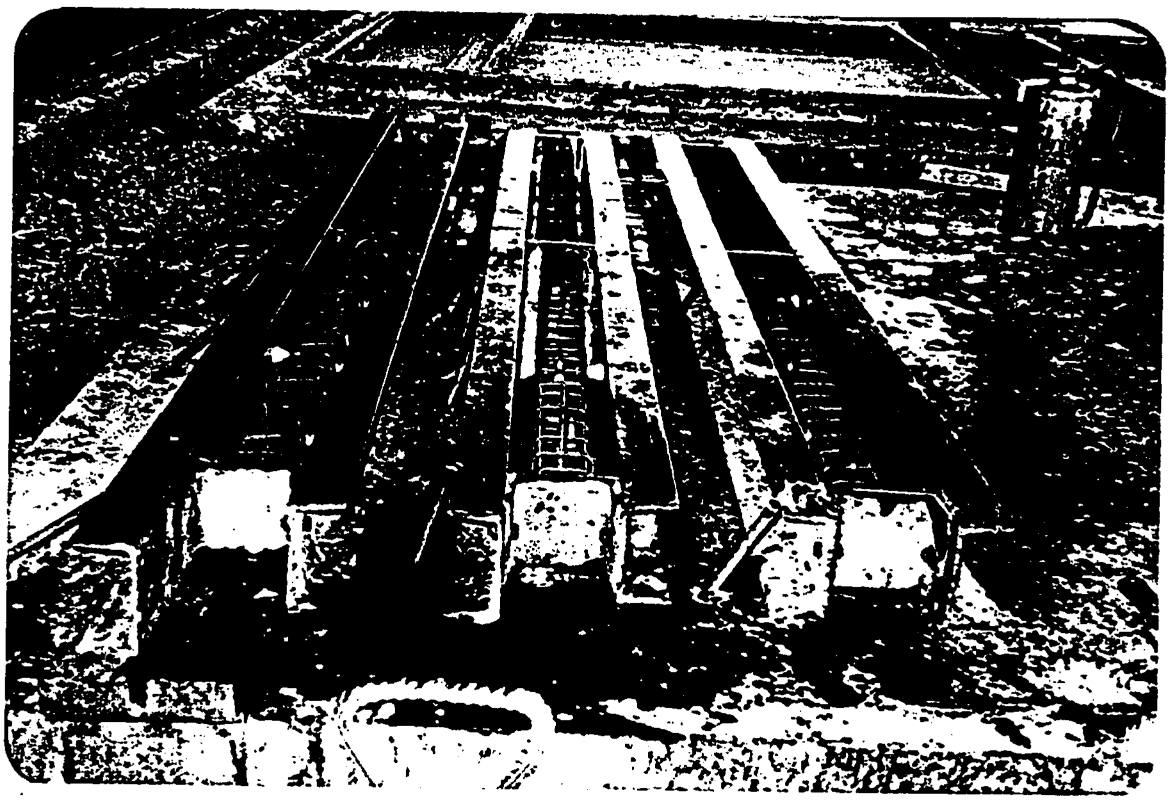

Plate 3.1 : Steel moulds for casting.

reinforcement were checked (30 mm from all sides), then a slump test was taken. Four cylinders of 150 * 300 mm were filled with concrete of the same mix in three layers according to the ASTM : C192 and prepared for compressive strength test, as shown in plate 3.2 . During casting, internal and external vibration were used. Preparation of steel reinforcement and demoulding of the beams were done one day before the new casting. Casting was finished in 2 weaks time. TONIPACT machine, of 3000 kN capacity, is used to test the 150 * 300 mm cylinders in compression.

Curing and transportation

All beams were cured at the company's yard where wet burlaps were kept saturated with water on the beams. Water was sprayed 3 times a day to all beams for 28 days, as shown in plate 3.3 .

After the curing period, all beams were transported together on a large truck to KFUPM, campus for testing and repair.

3.2 Testing Program

The aim of this research was stated earlier to investigate the different repair methods of concrete structures which were damaged. Therefore, the repair will take place, when cracks induced on a structural member were

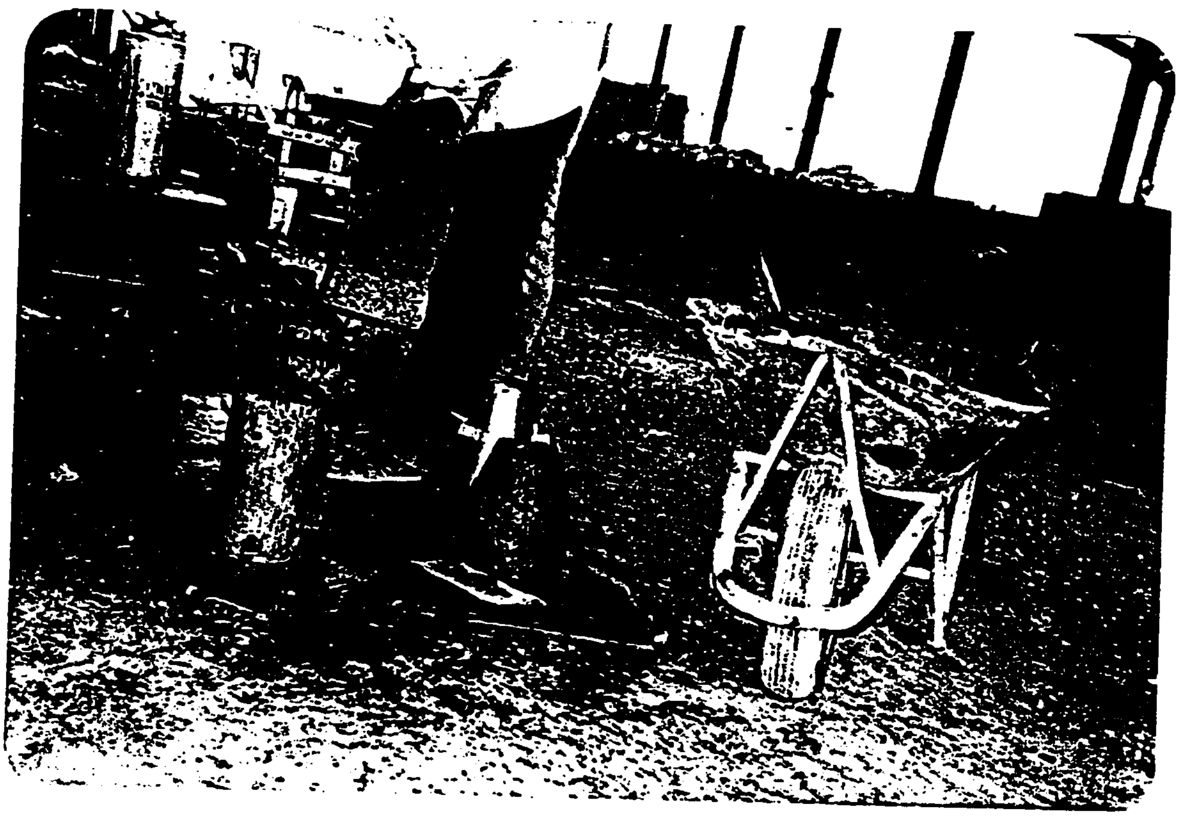

Plate 3.2 : Slump test before casting.

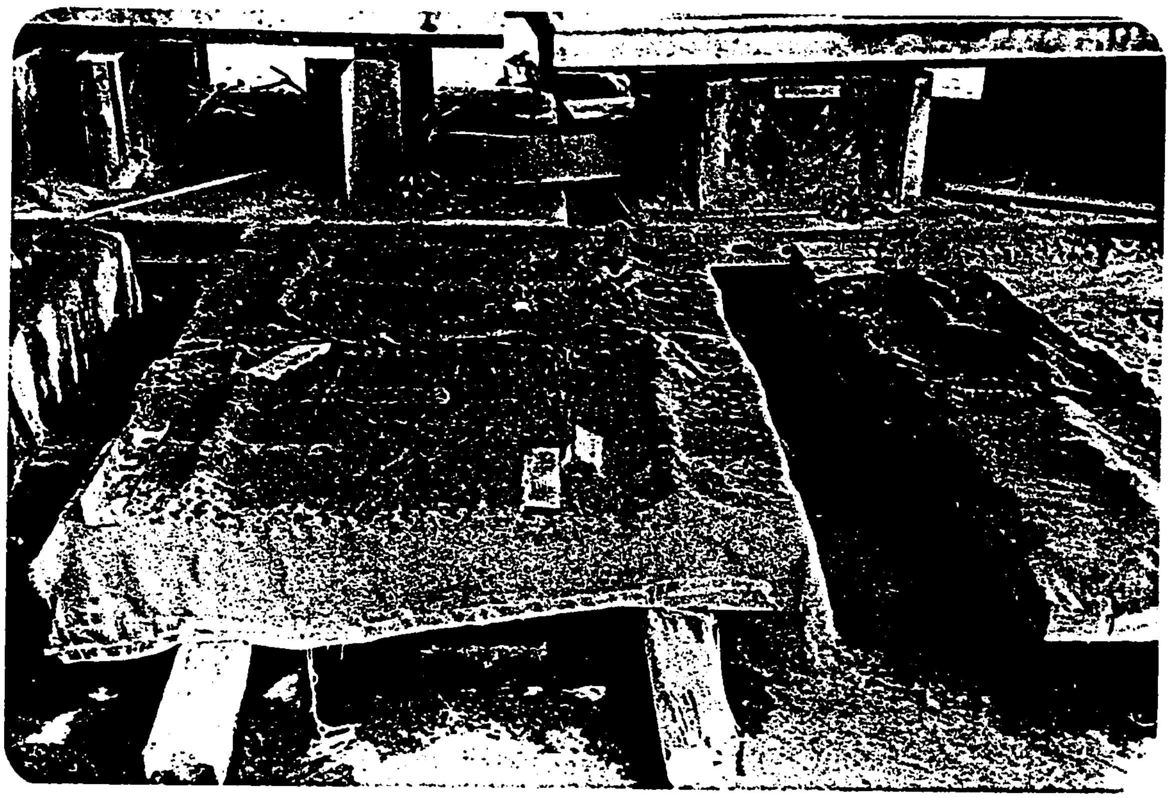

Plate 3.3 : Curimg the beams at the company's yard.

wide enough for repair. Concrete beams under two points load will crack when they are subjected to loads above the design service loads. The crack width will differ from beam to another under the same load intensity.

Therefore, in order to compare the efficiency of the repair methods, the beams were divided into three groups for testing :

* group of 10 mm central deflection 9 beams
* group of 15 mm central deflection 9 beams
* group of ultimate central deflection 12 beams

as shown in the testing program chart, Fig. 3.2 .

Marking the beams

The beams were marked at the third points where the 2 points load were applied. Also, the bottom side was marked under the loads and at the center of the beam for measuring the deflection at the three points using the Linear Variable Differential Transducer (LVDTs). The points of supports (Reactions) were marked at 600 mm on each side of the center.

Machine set up

A 25 tons capacity INSTRON machine was used for testing the beams in flexural. The load was applied through a small ball rested on a steel I-beam which is transfering the load

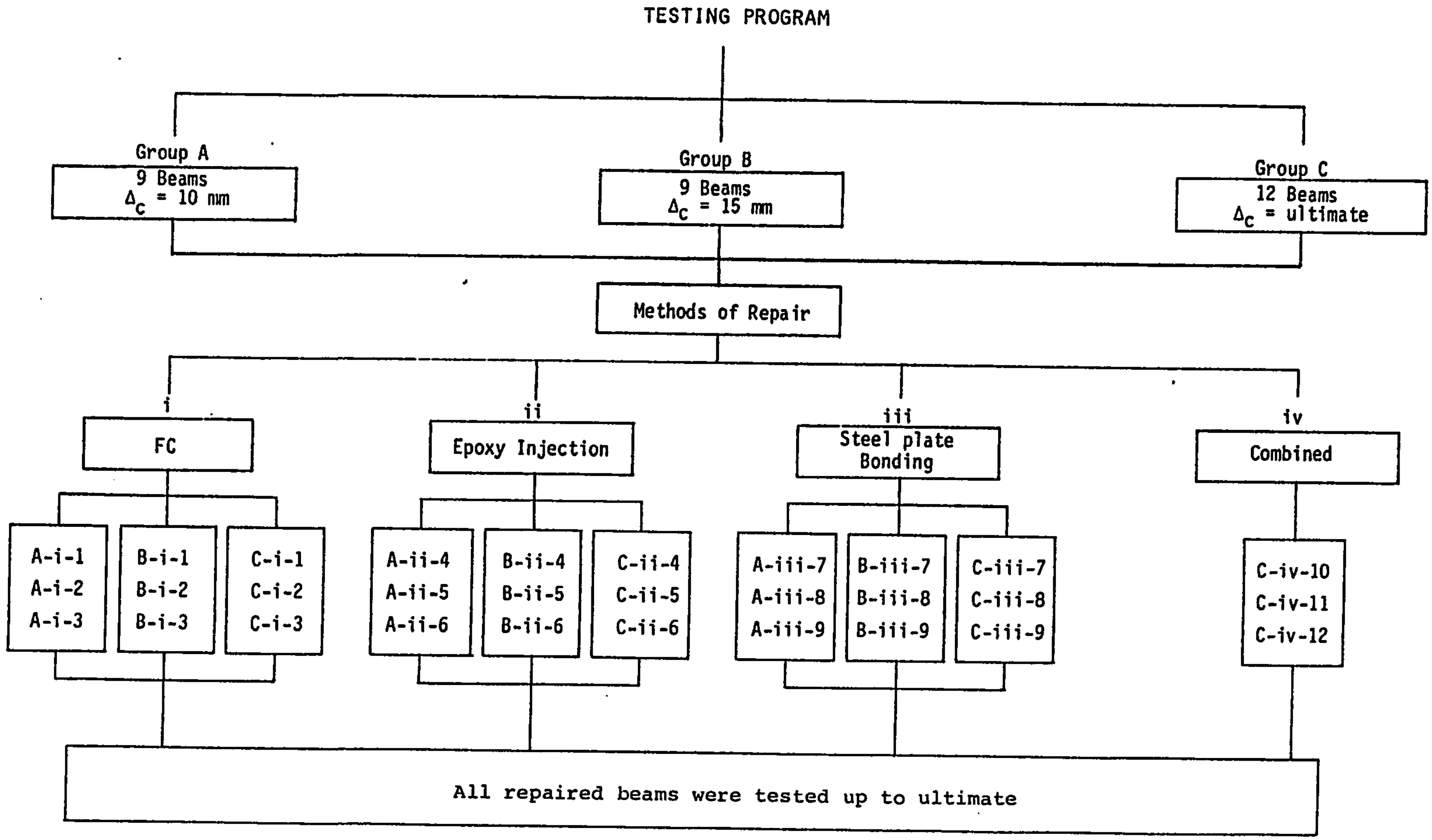

Fig. 3.2 Testing program chart

directly to the middle third points of the beam. The beam was supported by a semi-circular steel roller rests on a heavy steel I-beam. Three LVDTs and a crack width indicator were mounted on the beam to record deflection at the three points (at center and under the two points load) and crack width at the central zone of the beams from one side. The deflection and the crack width were automatically recorded by the data acquisition system along with the load, as shown in plate 3.4 .

The following machine parameters were set up before testing:

* Head speed of the INSTRON machine is kept at constant speed of 1 mm/second.
* Machine was set at a load capacity of 100 kN.
* The central deflection was monitored in order to be able to stop loading at any desirable deflection.
* Initialization of the system before loading was taken.

Testing procedure

Load started at speed of 1 mm/s and readings were taken every 5 kN until the appearance of the first crack in the central zone, then the load is stopped. The crack indicator was then installed on the crack from the side and it was initialized by taking the small nail out of its position very slowly. The first reading taken before loading started

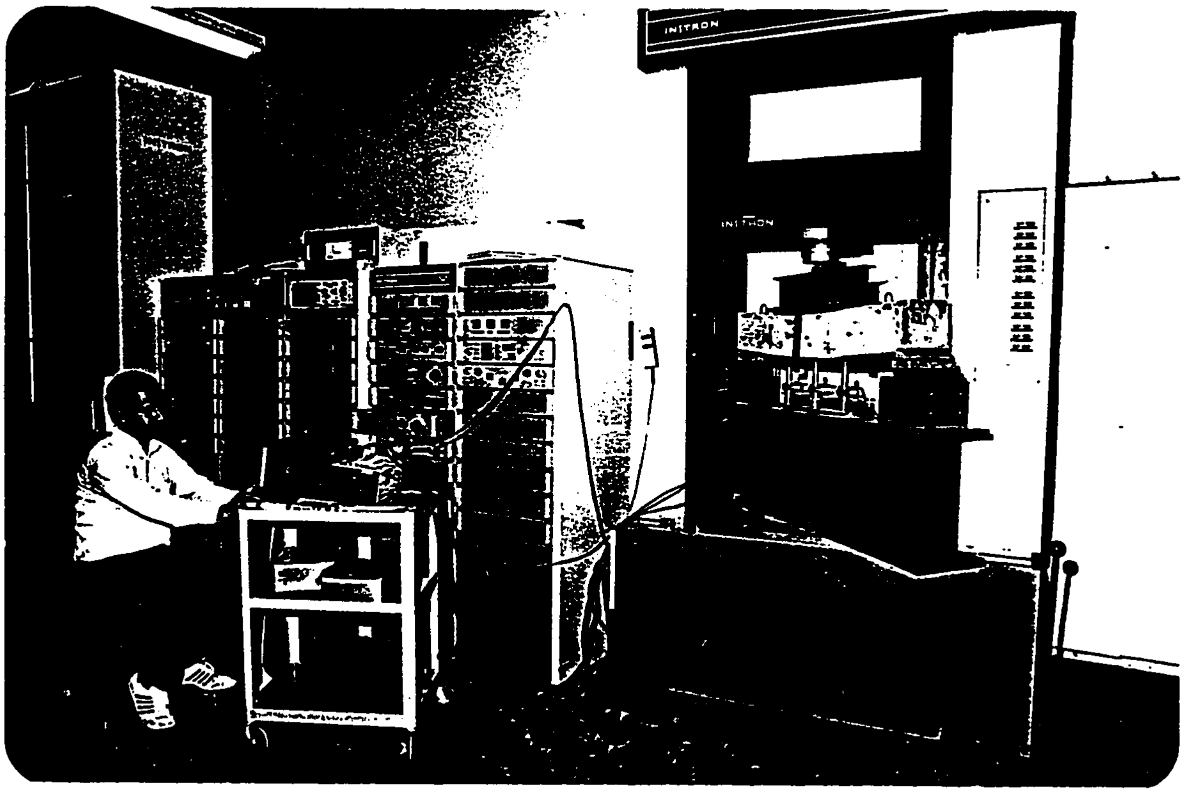

Plate 3.4 : INSTRON machine set up for testing.

again was considered to be as the crack initial value. Loading was continued and readings were taken every 3 kN until reaching the desirable deflection, where loading was stopped and unloading was taken back to zero in the same way as loading the beams.

Repaired beams testing

After the original testing the beams were repaired using different methods, epoxy injection, ferrocement, and steel plate. Each method of repair was applied to 9 beams.

Also, 3 beams of group C were repaired by epoxy injection and then ferrocement.

All repaired beams were tested, after curing according to each method, up to ultimate load, Which are summarized in Fig. 3.2 (testing program chart).

A typical flexural cracks resulted from the original testing of the beams for 10, 15 mm and ultimate central deflection are shown in plates 3.5 through 3.7 respectively.

Plate 3.5 : Typical 10 mm-deflection beams.

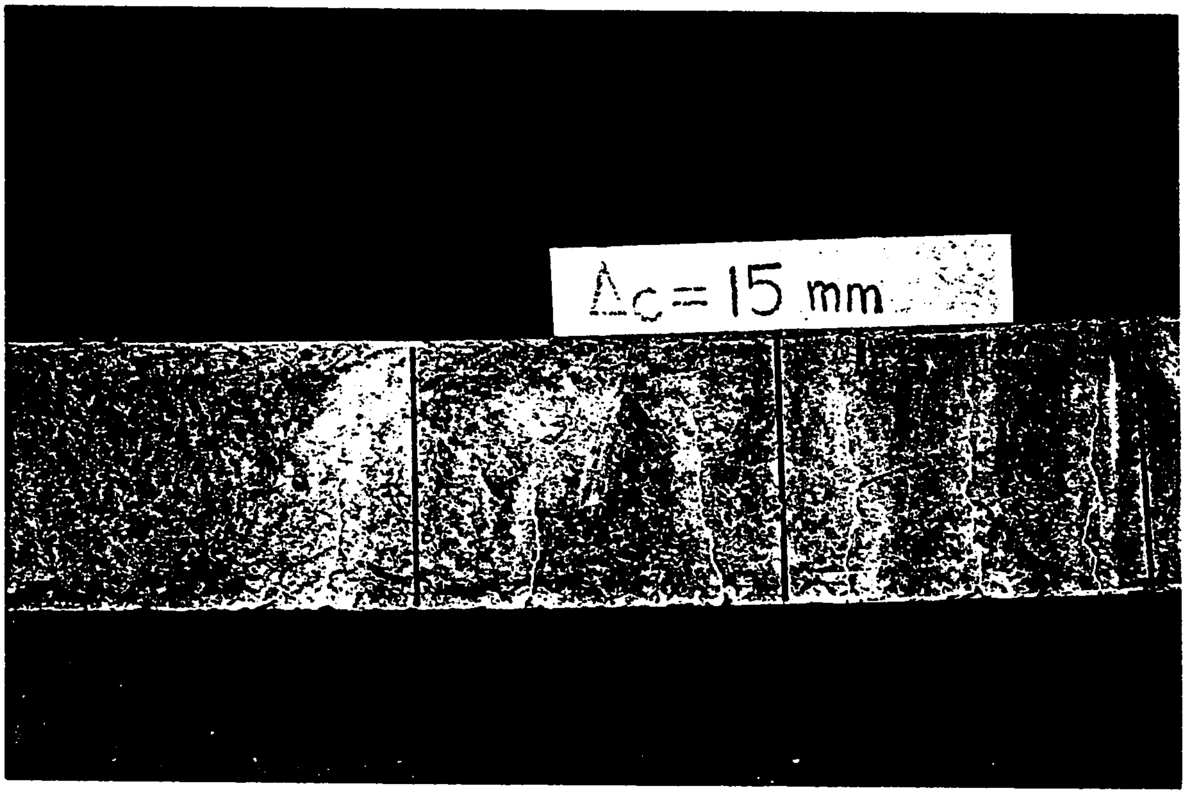

Plate 3.6 : Typical 15 mm-deflection beams.

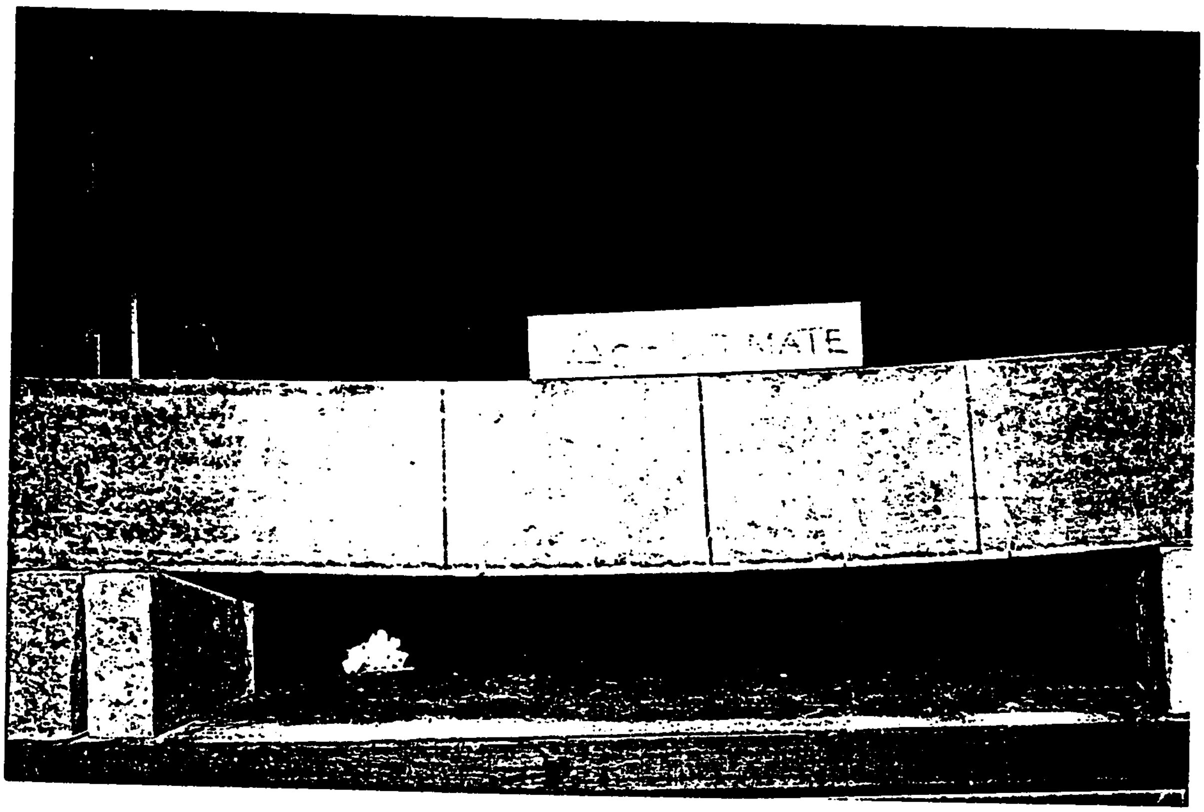

Plate 3.7 : Typical ultimate-deflection beams.

3.3 Repair Methods

3.3.1 Repair by ferrocement

Steps of repair

The basic steps used to repair a beam by ferrocement are as follows :

(1) Surface preparation

After testing the original beams, the preparation of beams' surface was started. For improving bond between the hardened concrete and the new mortar, three sides of each beam have to be roughened, bottom and other two parallel sides. Ordinary hammer-axe was used to roughen the surface, but using sandblasting will be more useful for saving time and effort, as shown in plate 3.8 . In this method, ferrocement only applied to three sides of the beam to simulate the actual beams in real structures.

(2) Cleaning

Using the air pressure, the surface of the beams were cleaned to remove dust and other materials in order to have a good bond between the mortar and the hardened concrete.

(3) Preparing the mix and wire mesh

The mix of the mortar used was:

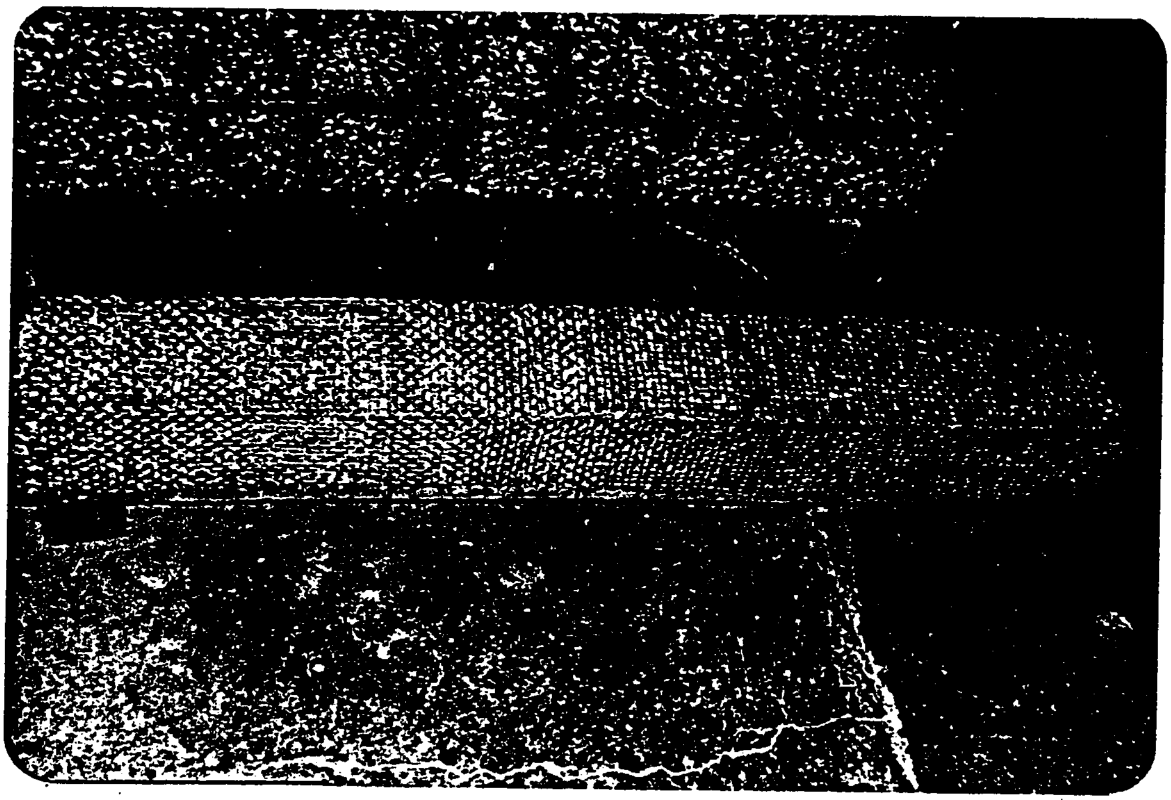

Plate 3.8 : Roughening the surface of the beams from three sides.

water/cement ratio = 0.40

cement/sand ratio = 1 : 2

Type I cement and dune sand were used for the mix.

Water content was adjusted to offset the absorption of water in sand which is equal to 3.33 % by weight of sand. The mortar for 3 beams was prepared at a time.

One layer of wire mesh was used, which was a two dimensional square mesh. The wire mesh used was 7.54 mm square opening, diameter of 0.9 mm and ultimate strength 414 MPa. The wire mesh was installed to the beam from three sides like a channel of 1100 mm long and 125 mm in depth.

(4) Mixing the mortar for ferrocement application

After preparation of water, cement and sand, start mixing and spray water on the roughened surface, as shown in plate 3.9 . Cover the surface with a thin layer of mortar, then put the wire mesh on top of that which will be applied from three sides, plate 3.10 . Tie the mesh to the beam using wires or a powerdriven nail to fasten the wire mesh to the hardened concrete. The final stage is to force the mortar through and on top of the wire mesh in order to have almost a 15 mm layer of repairing. Leveling the surface will be started after 30 minutes to get a smooth surface. Three cubes of 50 mm were filled with mortar to get the compressive strength of the mortar after 28 days curing.

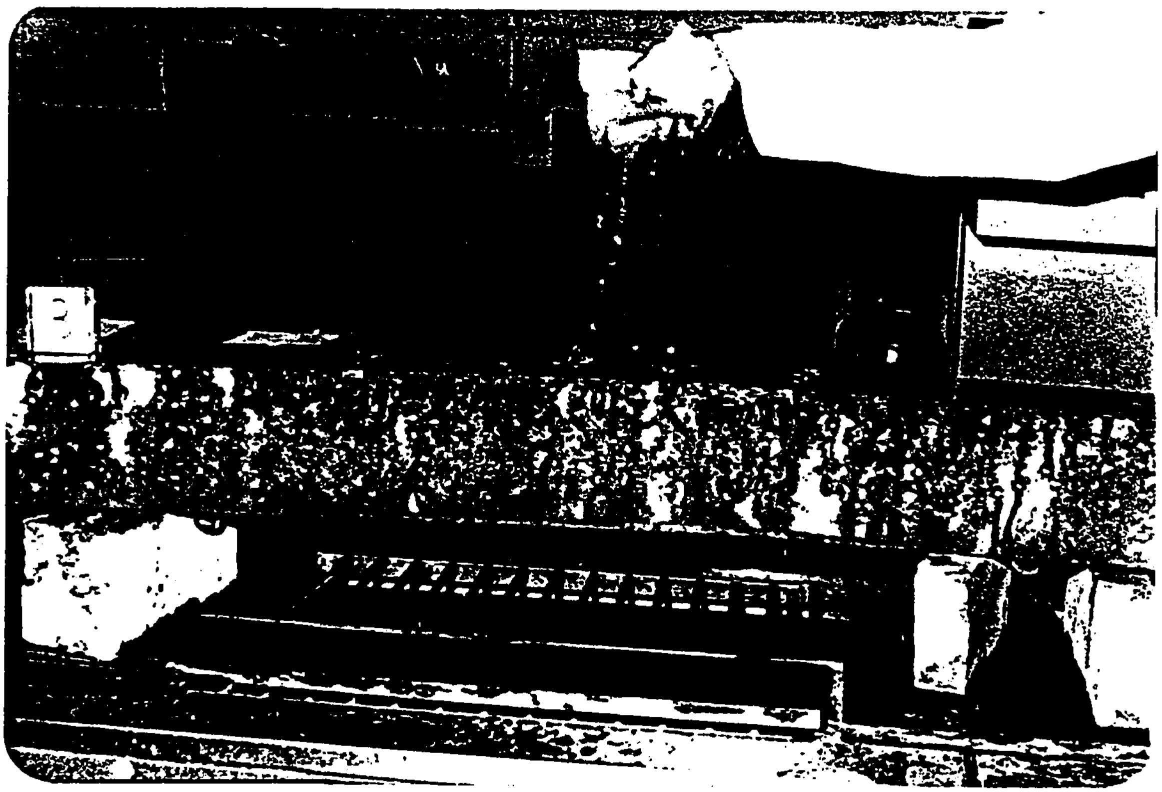

Plate 3.9 : Spraying water on the roughened surface.

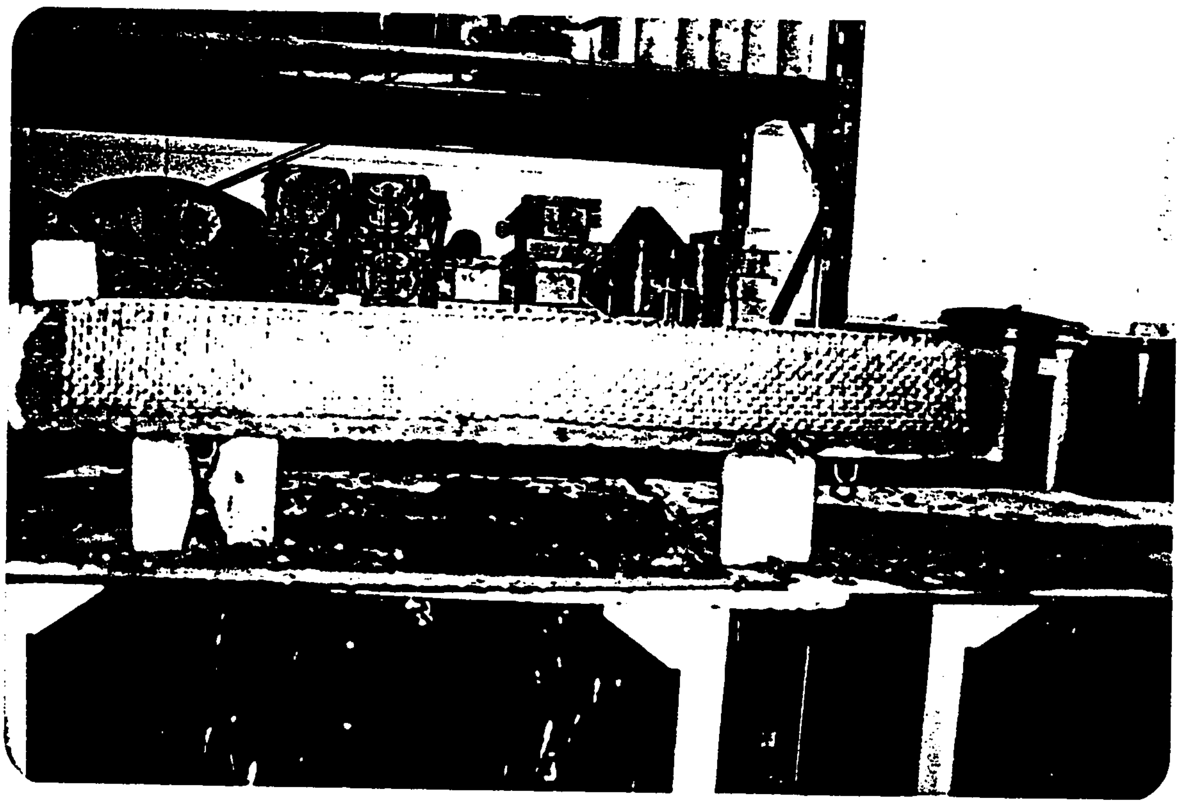

Plate 3.10 : Installing the wire mesh on top of small layer of mortar from three sides.

(5) Curing

Curing is very important for the hydration process of the mortar. The curing conditions used are:

(a) The beams were moist cured by covering them with wet burlap under plastic sheets for 14 days.

(b) Then, the beams were cured in a water tank for another 14 days.

Total curing days were 28 days for beams and cubes. An example of ferrocement repaired beams is shown in plate 3.11, just before testing.

Preparing specimens for permeability test

Ferrocement method of repair is believed to enhance the properties of the repaired beams among which is reduction of the permeability. Reducing permeability will reduce the harmful materials of getting into the already cracked beams and deteriorate the concrete and steel.

In order to asses the repair method using ferrocement with regard to permeability the following testing program has been devised :
Three cubes of 150 mm were filled with concrete of the same mix used for casting the beams. Also, another three cubes, of the same dimensions, were filled with concrete of the

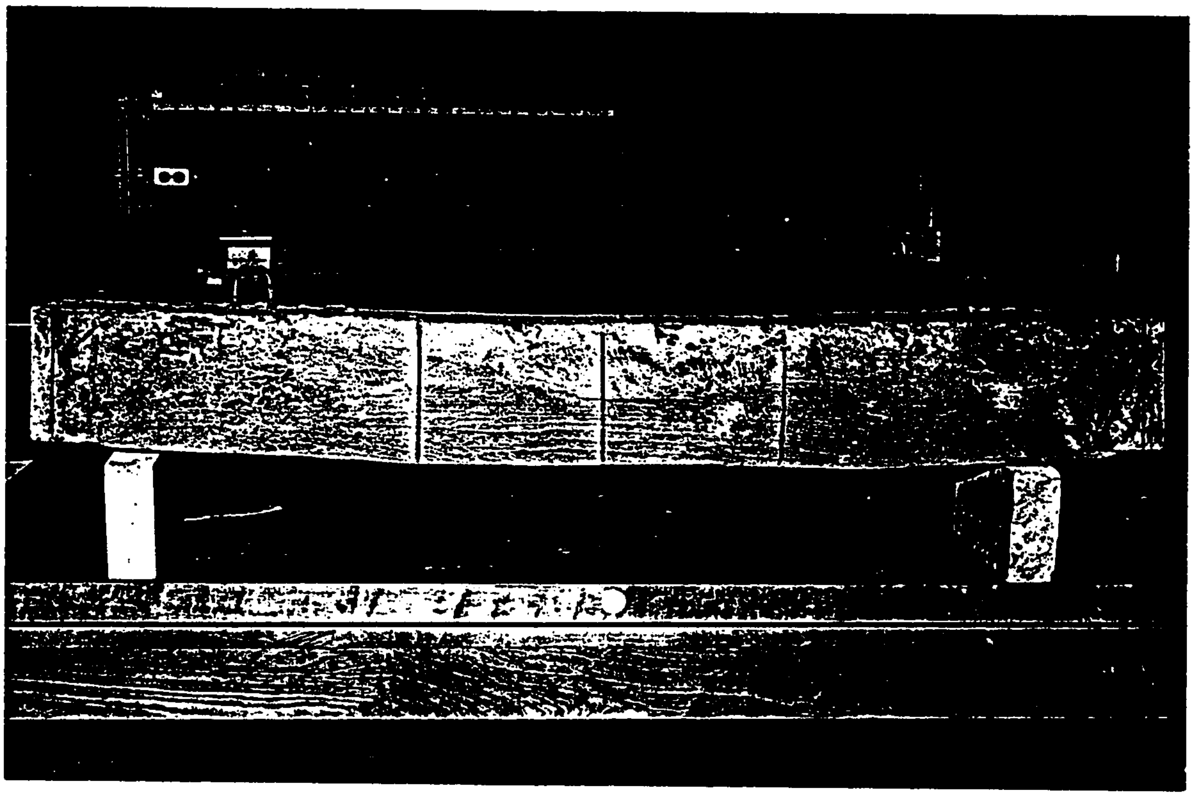

Plate 3.11 : Ferrocement repaired beam after curing.

same mix on top of 10 mm ferrocement layer. The 10 mm ferrocement layer was divided into two parts by a wire mesh. The mix of the mortar and the wire mesh were the same as in ferrocement repair method. The permeability test was carried out according to the German Industrial Standards (DIN 1045). After 28 days curing of concrete cubes, the permeability test applied water under fixed pressures with fixed time as follows:

1 bar for 48 hours.

3 bar for 24 hours.

7 bar for 24 hours.

Thereafter, all cubes were split into 2 parts to measure the maximum penetration depth in the concrete using TONIPACT machine.

3.3.2 Repair by epoxy injection

Epoxy injection can be used to restore structural soundness of buildings, bridges, and dams where cracks are dormant or can be prevented from moving further. This technique involves installing entry ports and injecting the epoxy through these entry ports (nipples) using an injection gun.

Three basic steps are needed for epoxy repair method :

(A) *Sealing the outer side of the crack*

Before sealing the cracks, The concrete being bonded must be dry, clean, and sound. Air pressure was used to clean the cracks from all traces of dirt, oil or laitence.

The steps to seal the surface are :

A surface seal is required to prevent liquid resin from leaking out of the crack before gelling, and entry ports can be used to allow injection through them.

(1) Mixing

- The sealant materials are :

EP-CA Resin (white) : 0.940 kg/0.61 liters

EP-CA Hardener (black) : 0.560 kg/0.34 liters

These materials are supplied by Ciba-Geigy

- Mixing ratio : 100 parts by weight resin

to 60 parts by weight hardener.

Mix one can of resin with one can of hardener. Stir the contents of each can separately, then pour the entire can of hardener into the can of resin. Stir the hardener into the resin until the mix is even gray colour throughout using the ordinary electric hand drills if possible.

(2) Install the entry ports

The entry ports was installed at lowest and highest points of the crack. After that, the process of installing the entry ports starts by covering the outer part of the entry ports (nipples) with the sealant material to fix them at the chosen position, plate 3.12 .

(3) Finishing the sealing process:

Just after the installation of the entry ports, the sealing of cracks should be started, using a scraper, within a pot life of this type of epoxy as indicated below :

60 minutes at 20°C.

20 minutes at 30°C.

(B) Injecting the epoxy into the crack

After curing of the sealant, which takes at least 6 hours, prepatation of epoxy materials can be started.

The epoxy materials are:

EP- IS Resin : 0.175 litre tube

EP- IS Hardener : 0.070 litre tube which are supplied also by Ciba-Geigy.

The equipments used are: Hoses, caps, nipples and injection gun. Hoses are used to connect the injection gun to the nipples.

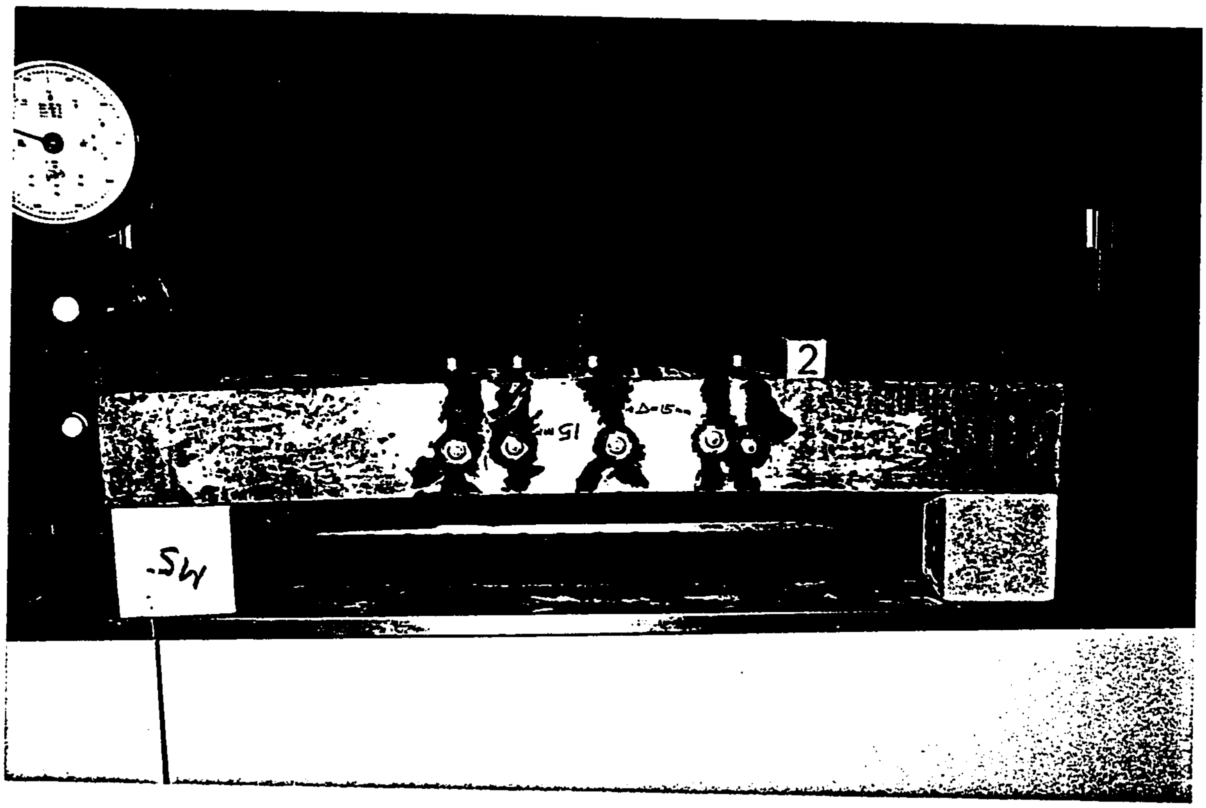

Plate 3.12 : Sealing the outer side of the crack by epoxy materials.

Mixing Procedure of Epoxy Materials

(1) Open the end of the hardener and resin tubes with a sharp pointed tool.

(2) Completely and slowly empty the tube of hardener into the tube of resin.

(3) Close the mix with a plastic cap and mix the contents by slowly inverting the mix in the tube almost 30 times. Do not shake very fast to avoid the air bubbles.

(4) Use the resin/hardener mix as soon as possible with the pot life indicated below :

50 minutes at 20°C.

25 minutes at 30°C.

(5) Use the injection gun to inject the epoxy into the entry ports from the lower entry ports for vertical cracks, until the epoxy runs out from the above entry port, then cap the lower injection port and repeat the process at successively higher ports until the crack has been completely filled, as shown in plate 3.13 .

(C) *Grinding process*

The removal of the excess sealant from the the beam surface should be started after curing of the epoxy injected into the cracks, which usually takes 7 days to reach full

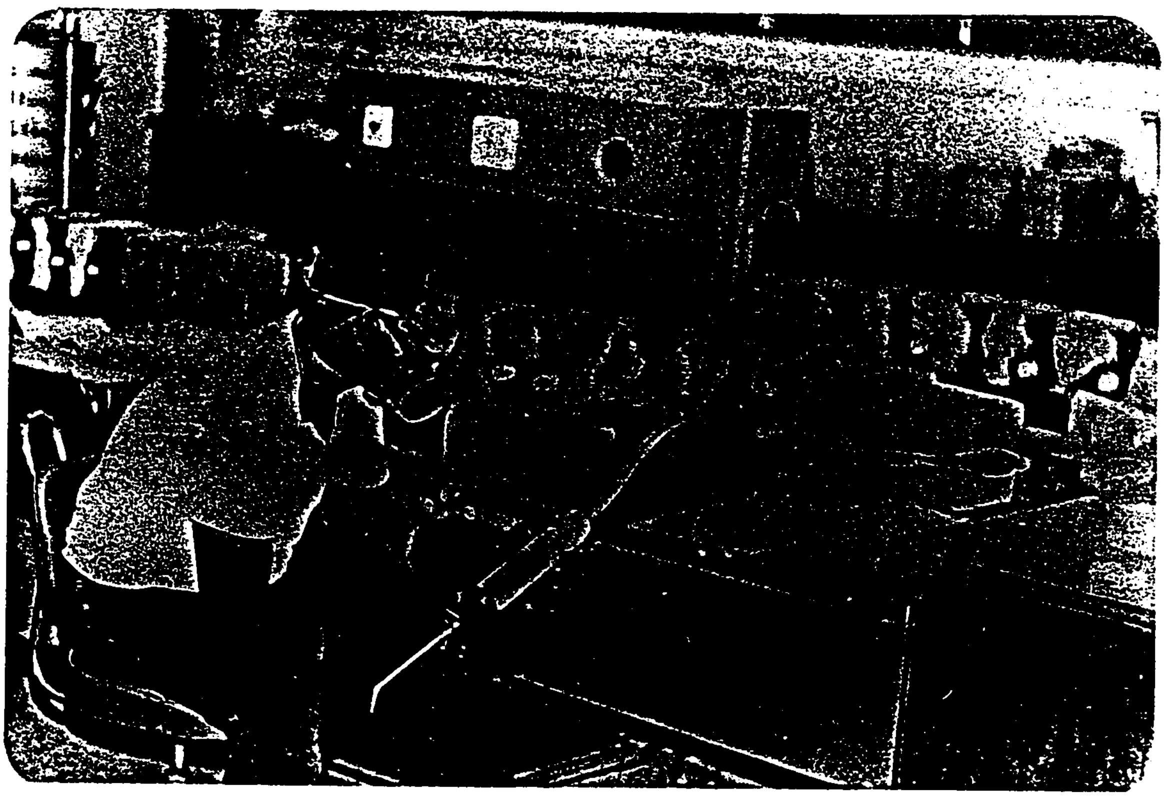

Plate 3.13 : Injecting the epoxy into cracks.

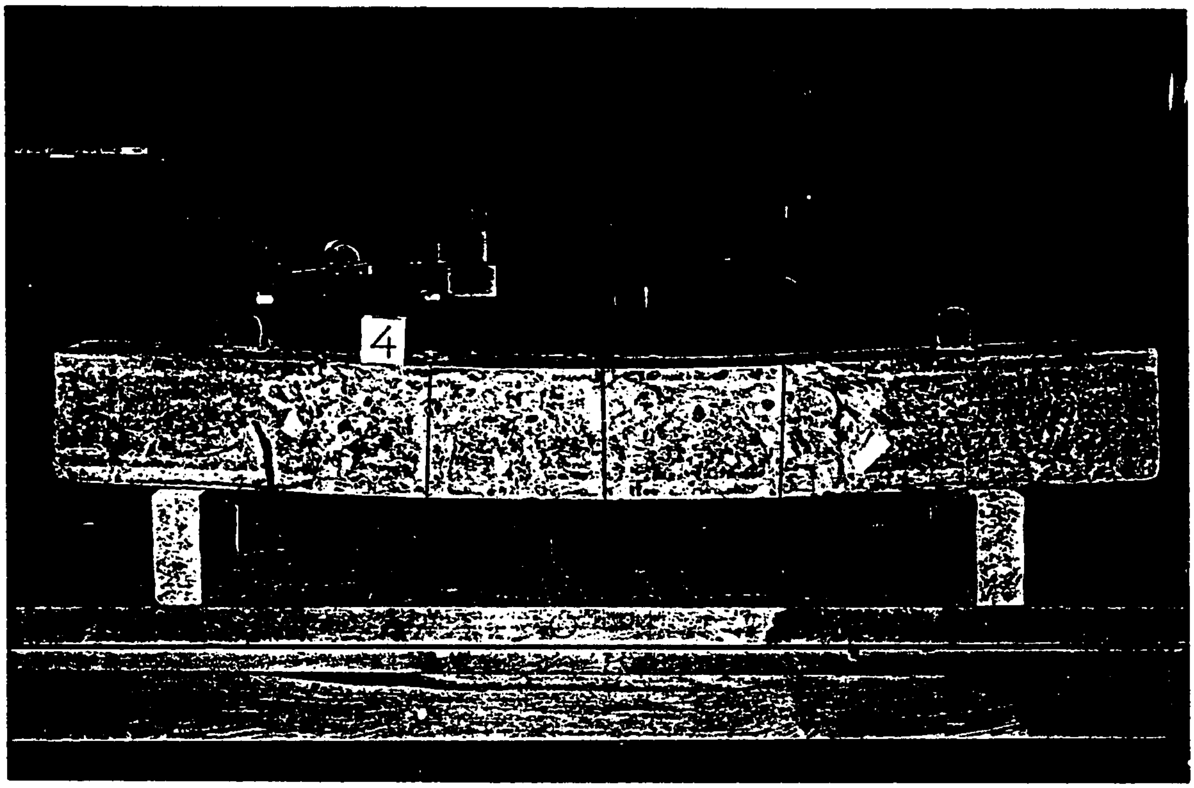

Plate 3.14 : Epoxy repaired beam after grinding.

strength. The grinding process can be done by using a grinder machine, plate 3.14 .

3.3.3 Repair by steel plate bonding

Steps of repair

The basic steps used to repair a beam by Steel plate are as follows :

(1) Surface and steel plate preparation

The bottom side of the beam was roughened in order to have good bond between the hardened concrete and the epoxy materials used for this purpose. The method to roughened the surface was the same as in the case of ferrocement repair and shown in plate 3.15 .

(2) Steel Plate Preparation

The steel plates were prepared from mild steel in the mechanical workshop at KFUPM. The dimensions of the steel plates were chosen having in mind the available clearance of the beam, and almost the same amount of steel used for the main reinforcement. so 1100 * 100 mm steel plates with 1 1/2 mm thickness were prepared to be used for strengthening the beam.

Before using the steel plate, it will be necessary to remove the surface contamination and laitence in order to

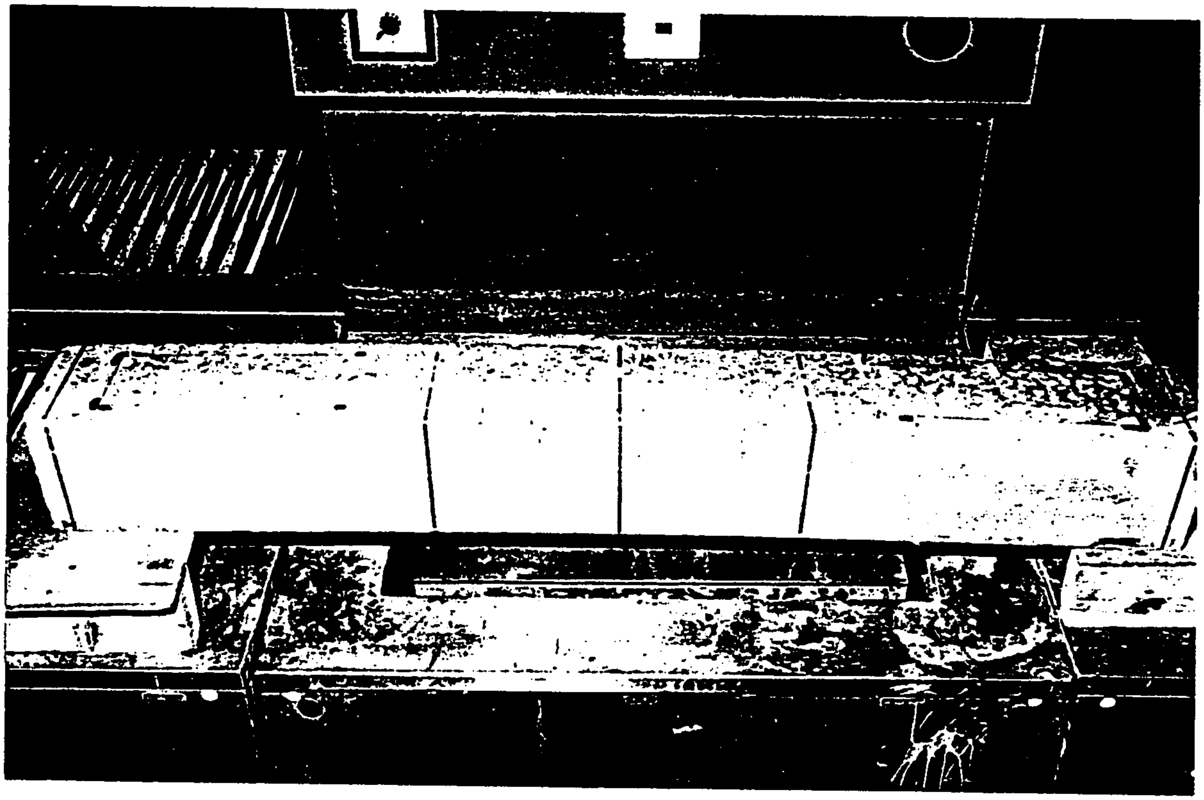

Plate 3.15 : Surface preparation from bottom side of the beam.

provide excellent adhesion and prevention of rust from occuring. Two chemical solvents were prepared in the environmental laboratory which were Hcl and Acidic Acid for the purpose of cleaning the plates, as shown in plate 3.16 . The two solvents are used one after the other and at least 30 minutes before the starting the application of the epoxy material between the concrete and the steel plates.

(3) Mixing epoxy materials

The epoxy materials used for this purpose are the EP-CA resin (white) and EP-CA hardener (black), which are the same materials used to seal the outer side of the cracks in the case of epoxy repair method. One can of the resin was mixed with another another of the hardener. First by stirring each can separately, followed by pouring the entire can of the hardener into the can of resin. Then, stir the mix until its colour becomes gray. The electric hand drill was used for mixing the epoxy materials, plate 3.17 .

(4) Applying the epoxy material

Before applying the epoxy material, the roughened side of the beam should be dry, clean and free of dust by using the air pressure. Start applying the epoxy to cover the marked area of the beam with a small layer of epoxy, plate 3.18. Also, cover the middle strip of the steel plate with epoxy and then directly force the steel plate on the epoxy layer.

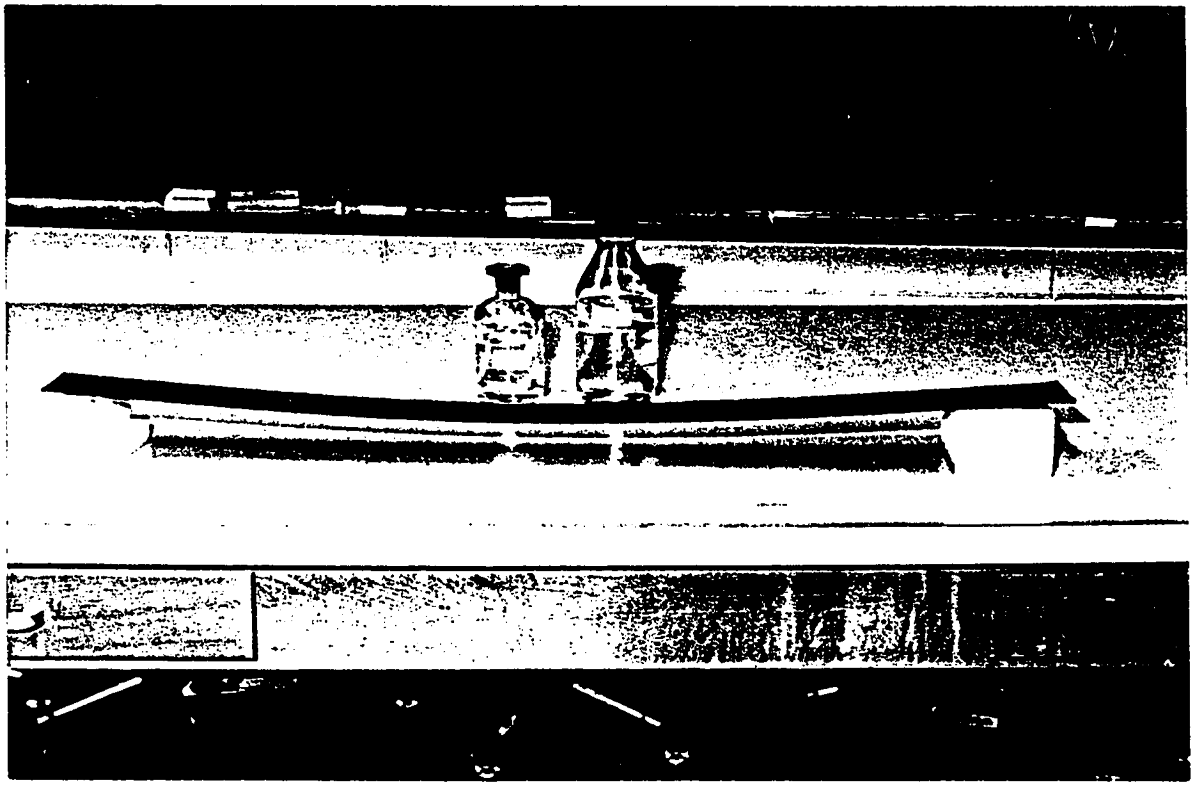

Plate 3.16 : Cleaning the steel plate by Hcl and Acidic Acid to avoid corrosion problems.

Plate 3.17 : Mixing the epoxy materials using the electric hand drills.

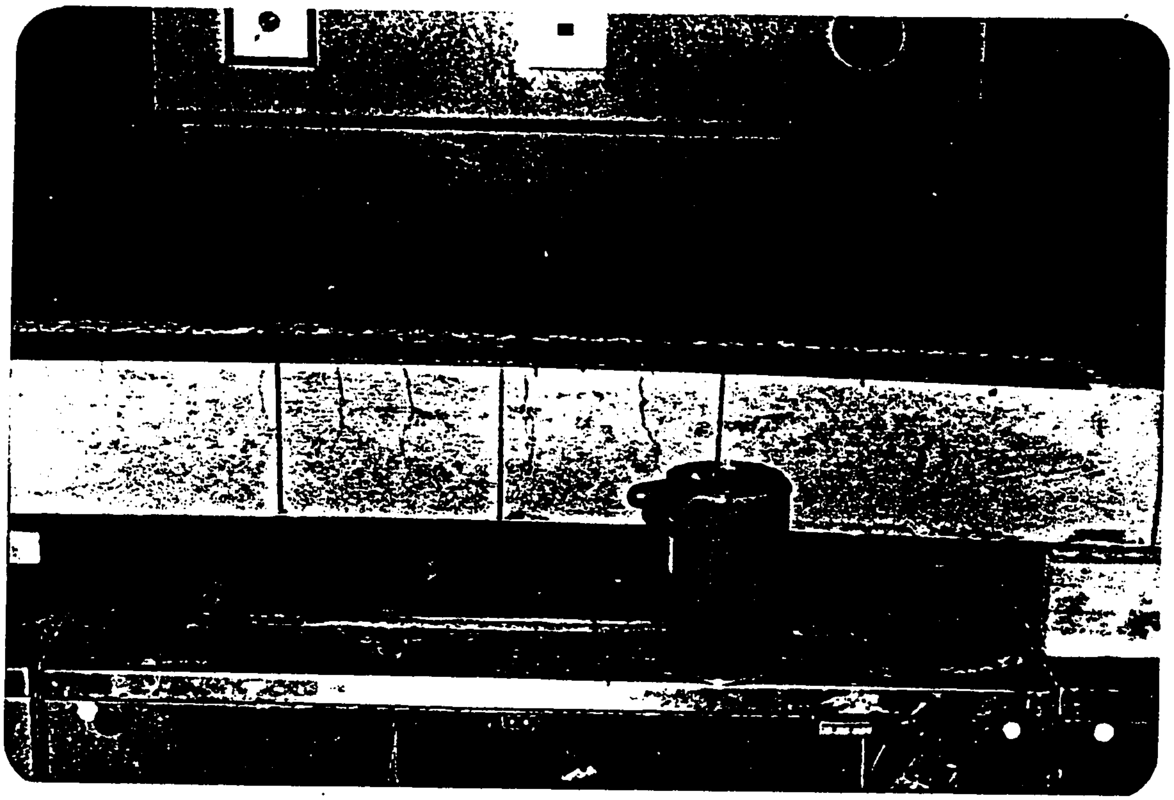

Plate 3.18 : Applying epoxy material between the concrete & the steel plate.

Remove all epoxy moving out of the plate periodically during the first few hours. The epoxy material should be used within its pot life as indicated in the supplier's instruction sheets.

(5) Curing

All beams were cured in the laboratory at room temperature. Nine concrete cylinders of 75 * 150 mm were distributed on each plate, with a load of 13.448 kg/m, as shown in plate 3.19 . The weight on the plate was removed after 7 days as a minimum and before testing.

The bond strength between epoxy & concrete

In order to determine the bond strength between the bonding steel plate and the cracked beam, which can be used in calculating the ultimate capacity of the repaired beam theoretically.

The following test program had been devised :
Three prisms of 90 * 90 mm in cross section and 150 mm depth were cast, with a 10 mm diameter bar at the center of the cross-sectional area going through the whole depth with a 400 mm length. These prisms were attached to plates (of the same type used for steel plate repair) from two sides by epoxy at a contact area of 20 * 25 mm and cured for 7 days. The specimens were tested in tension in order to asses the

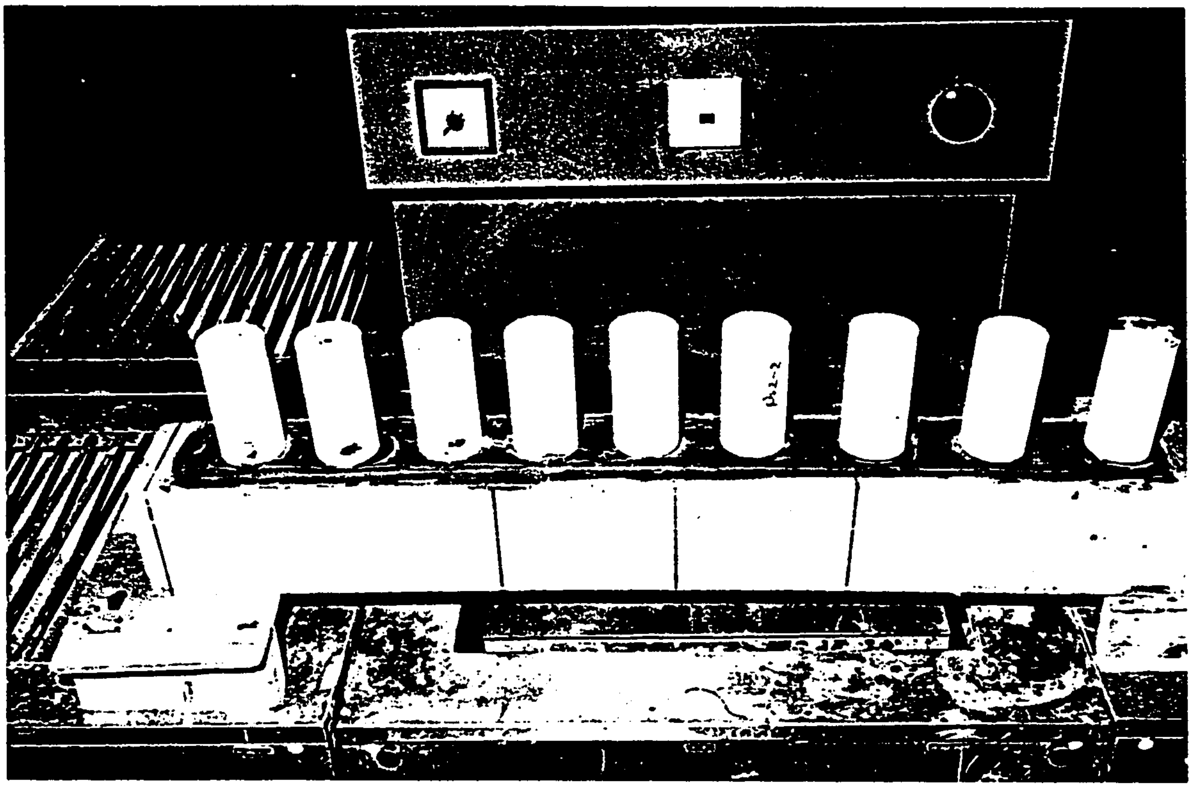

Plate 3.19 : Nine concrete cylinders were left on the plate for 7 days, before testing.

shear strength of the epoxy between concrete and steel plate, as shown in Fig. 3.3 . The FORNEY machine was used to test the three prisms in tension, (see plate 3.20).

3.3.4 Repair by the combined method

This method is just two separate methods applied to three of the ultimate group beams, one after the other. First, cracks were filled with epoxy, then after curing of epoxy, beams were strengthened by ferrocement. The process used to repair the beams using the combined method was the same as the two methods of repair mentioned above.

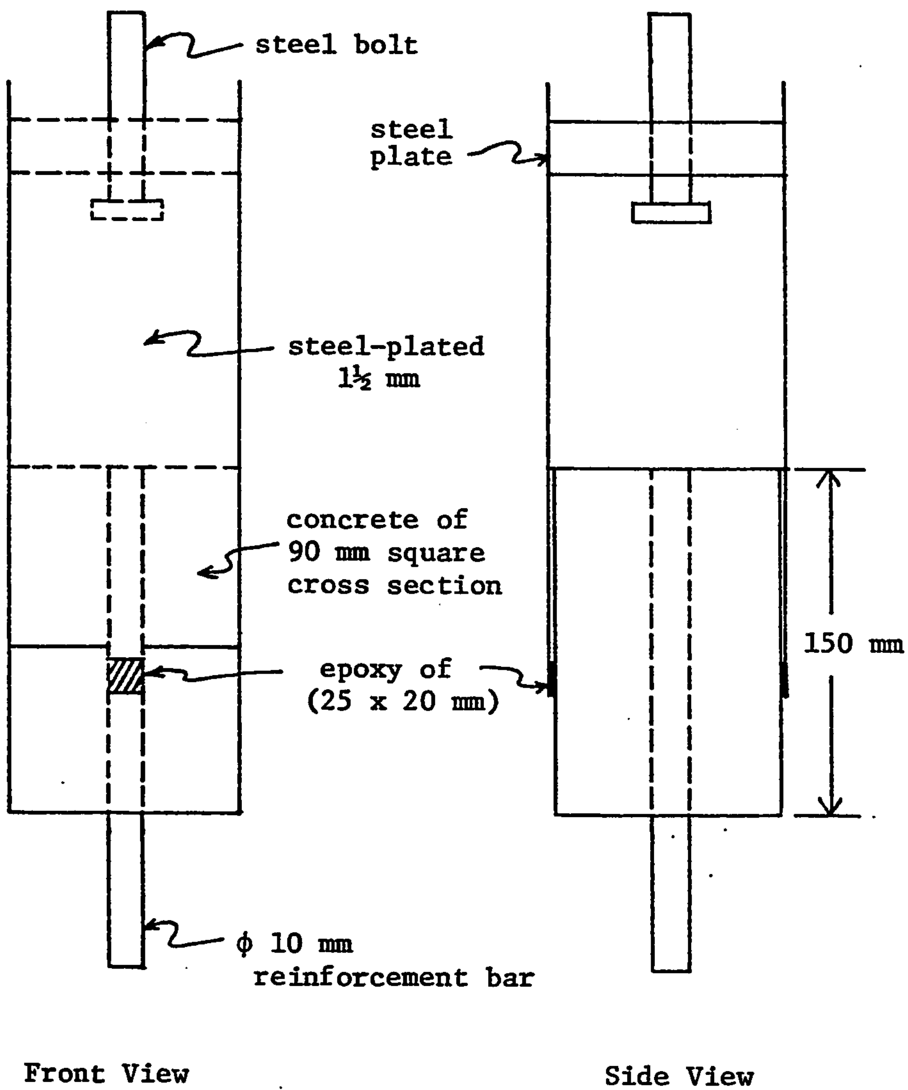

Fig. 3.3 Schematic test set up for determining the shear strength of epoxy between concrete and steel plate

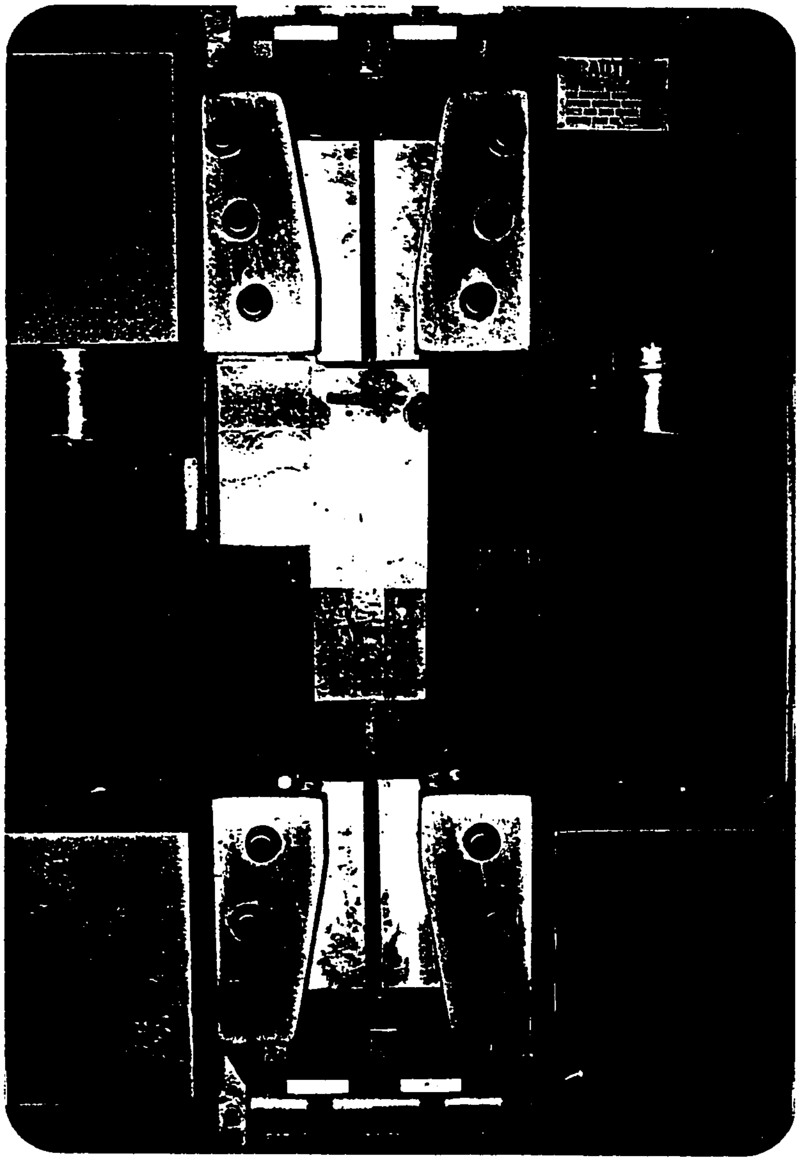

Plate 3.20 : Tension test using FORNEY machine.

Chapter 4

RESULTS AND DISCUSSION

This chapter includes the results of both the original and the repaired beams in terms of plotting the load vs mid-span deflection, moment vs curvature and load vs crack width for different repair methods. Also, it includes the discussion of the results of the ferrocement, epoxy injection and steel plate bonding repair methods, in addition to a combination of both epoxy and ferrocement methods. Also, it includes the comparison between all methods in terms of cost, strength and ductility.

The compressive strength of the mix design used to cast the original beams, after 28 days of curing in water, was as an average of 45.28 MPa (6565 psi) and the average of slump test also was 64.26 mm (2.53"). The design compressive strength used in designing the beam was 31 MPa (4500 psi).

4.1 Data Reduction

The data collected for each beam before and after repair at each load level P (kN) as follows :
mid-span deflection Δ_c, deflection under two points of loading Δ_1 and Δ_2 (mm) and crack width w (mm).

The load P (kN) vs mid-span deflection Δ_c (mm) and load P (kN) vs crack width w (mm) can be plotted directly from the original data, but the moment M (kN-m) and curvature φ (Rad./mm*10^{-6}) were calculated from the original data and prepared for plotting. For a simply supported beam loaded at the third points by two equal loads, the curvature φ, of the beam at the central zone is constant due to the constant moment at the same zone. Moment at the central zone is maximum and is given by M=a*P/2, where 'a' is the shear span.

From the analysis of a simply supported beam the following is true :

$$\frac{M}{EI}=\varphi=\frac{1}{\rho}$$

Let Δd = the difference between mid-span deflection and the average of the deflections under the two points load.

From Fig. 4.1

$$\Delta d = \Delta_c - \frac{\Delta_1+\Delta_2}{2}$$

and from Δ ABC

$$\rho^2 = (\frac{a}{2})^2 + (\rho - \Delta d)^2$$

$$\rho^2 = (\frac{a}{2})^2 + \rho^2 - 2\rho\Delta d + \Delta d^2$$

$$\text{or } \rho = \frac{(a/2)^2 + \Delta d^2}{2\Delta d}$$

where

ρ = radius of curvature

Therefore;

$$\varphi = \frac{1}{\rho} = \frac{2\Delta d}{(a/2)^2 + \Delta d^2}$$

For a = 400 mm (constant), the values of φ and M are calculated at every reading, before and after repair.

For every beam, two files of data were prepared, one file for the original beam and another for the repaired one. A small FORTRAN programs was used to find the moment and curvature of each reading, then the data was collected in another file, as shown in table 4.1 as a sample data. A second FORTRAN program was prepared to plot the data using Calcomp. Then, the plots were printed on laser using Waterloo Script. The FORTRAN program was designed to draw any two columns of the data. Three curves will be plotted for each beam with comparison of results, before and after repair.

The curves to be plotted are :

* Load vs mid-span deflection
* Moment vs curvature
* Load vs crack width

In case of steel plate bonding, the crack width was not monitored, because the original cracks were there as they were without being filled by any material. So, only load vs deflection and moment vs curvature were plotted.

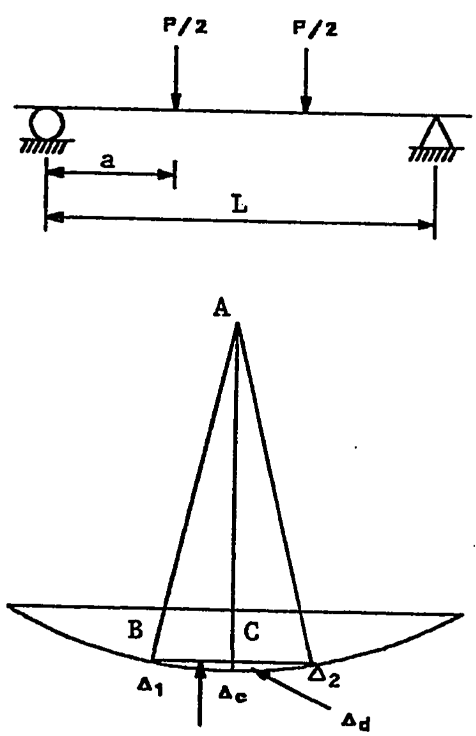

Fig. 4.1 The curvature of the beam.

TABLE 4.1 : Sample Data for Beam C-ii-4

P (kN)	Δ_c (mm)	Δ_1 (mm)	Δ_2 (mm)	w (mm)	M (kN-m)	$\varphi*10^{-6}$ (Rad/mm)
0.000	0.000	0.000	0.000	0.000	0.000	0.000
5.225	0.267	0.230	0.206	0.000	1.045	2.450
10.302	0.529	0.440	0.417	0.000	2.060	5.025
15.323	0.857	0.706	0.679	0.000	3.065	8.225
18.705	1.469	1.197	1.137	0.000	3.741	15.100
21.264	1.792	1.472	1.398	0.100	4.253	17.850
24.454	2.178	1.808	1.720	0.130	4.891	20.700
28.861	2.763	2.299	2.193	0.162	5.772	25.850
30.328	3.020	2.505	2.415	0.171	6.066	28.000
34.096	3.493	2.900	2.807	0.199	6.819	31.975
37.156	3.847	3.206	3.119	0.222	7.431	34.225
40.058	4.186	3.507	3.406	0.248	8.012	36.474
44.290	5.059	4.213	4.065	0.454	8.858	45.999
47.320	6.195	5.085	4.891	0.836	9.464	60.348
49.090	6.991	5.681	5.434	1.104	9.818	71.671
50.020	7.474	6.042	5.761	1.270	10.004	78.620
51.120	10.042	8.707	7.548	1.663	10.224	95.716
51.880	10.854	9.503	8.197	1.732	10.376	100.190
52.340	12.241	10.616	9.429	1.886	10.468	110.911
52.960	12.888	11.152	10.018	1.970	10.592	115.135
53.030	13.181	11.397	10.470	2.017	10.606	112.361
53.930	15.820	12.805	11.639	2.272	10.786	129.842
54.000	15.339	13.291	12.071	2.354	10.800	132.876
54.430	16.078	13.962	12.660	2.473	10.886	138.324
54.680	16.936	14.784	13.344	2.604	10.936	143.571
50.490	18.251	16.667	14.225	2.671	10.098	140.223
40.464	17.476	15.706	13.561	2.608	8.093	142.096
29.841	16.361	14.719	12.585	2.526	5.968	135.425
20.128	15.236	13.722	11.618	2.442	4.026	128.279
9.948	13.921	12.555	10.506	2.344	1.990	119.508
0.021	12.452	11.247	9.248	2.232	0.004	110.212

4.2 Ferrocement Repair Method

As described in chapter 3, three different stages of loading up to 10, 15 mm and ultimate deflections were applied to the nine reinforced concrete beams (three at each stage) for the purpose of ferrocement repair. Also, results of permeability test will be discussed.

4.2.1 Presenting the results of ferrocement

Load vs deflection

The load, P (kN), vs the central deflection, Δ_c(mm), are plotted for every beam before and after repair to compare the effect of repaired beams with the original ones. Figs. 4.2 through 4.4 show the plots of the load vs deflection curves of beams tested up to 10 mm mid-span deflection and repaired by ferrocement. While, Figs. 4.5 through 4.7 show the plots of the load vs deflection curves of beams tested up to 15 mm mid-span deflection and repaired by ferrocement. Also, Figs. 4.8 through 4.10 show the plots of the load vs deflection curves of beams tested up to ultimate mid-span deflection and repaired by ferrocement.

The plots of the load vs deflection of the original beams were separated from the plots of the repaired ones in order to get the initial and reduced stiffnesses in addition to the load and deflections at different stages. Table 4.2

contains the following :

P_{cr} (the cracking load), P_y (the yielding load), P_t (the tested load) or P_{ult} (the ultimate load) and Δ_{ult} (the ultimate deflection). Also, it contains K_i (the initial stiffness) and K_r (the reduced stiffness) in addition to ductility measurements.

The loads and deflections were found from the plots. The initial stiffness was found as the slope of the first linear portion of the P-Δ curve, and the slope of the line starting at the cracking point is called the reduced stiffness. The tested load was found only for beams tested up to 10 and 15 mm deflection. Also, the ultimate load, ultimate deflection and ductility were found for beams tested up to ultimate deflection. The same was repeated for all repaired beams. The ductility was calculated as the area under the P-Δ curves using planimeter.

A comparison between the parameters mentioned above for the original and ferrocement repaired beams was prepared from the load vs deflection curves as shown in table 4.2 .

Moment vs curvature

Figs. 4.11 through 4.13 represent the moment vs curvature curves for the beams tested up to 10 mm mid-span deflection and repaired by ferrocement. Figs. 4.14 through 4.16

Table 4.2 : Comparison of Original and Ferrocement Repaired Beams

Item		P_{cr} (kN)		K_i (kN/mm)		K_r (kN/mm)		P_y (kN)		P_t (kN)		$\Delta_{ult.}$ (mm)		Ductility (kN-mm)	
Δ_c	Beam#	Unrep	Rep.	Unrep	Rep.	Unrep	Rep.	Unrep	Rep.	Unrep	Rep.	Unrep	Rep.	Unrep	Rep.
10mm	A-1	14.76	17.47	13.77	20.80	7.35	11.27	34.85	52.00	44.47	56.98	-	10.40	-	380.10
	A-2	14.00	16.23	13.77	16.98	8.38	9.47	32.10	51.80	42.23	55.63	-	10.53	-	348.64
	A-3	15.28	19.05	12.51	16.06	7.63	10.32	32.30	51.40	42.40	55.99	-	11.64	-	406.46
Ave.		14.68	17.58	13.35	17.95	7.79	10.36	33.08	51.73	43.03	56.20	-	10.86	-	378.40
15mm	B-1	15.27	16.17	11.76	14.65	7.22	8.72	31.50	50.70	43.79	54.32	-	9.84	-	274.66
	B-2	15.81	16.95	15.56	19.08	6.71	9.97	35.00	51.90	44.86	56.18	-	9.79	-	291.67
	B-3	14.81	14.50	14.80	15.03	6.80	8.42	33.30	52.60	46.58	56.03	-	10.84	-	336.73
Ave.		15.30	15.87	14.04	16.25	6.91	9.04	33.27	51.73	45.08	55.51	-	10.16	-	301.02
Ult.	C-1	12.12	14.31	17.00	13.76	7.06	8.68	34.00	55.30	47.13	58.03	19.80	9.49	639.46	261.34
	C-2	13.21	21.10	18.53	13.08	7.15	8.87	35.20	56.50	49.09	63.50	27.26	8.09	1003.40	187.07
	C-3	14.60	15.10	14.46	16.61	7.43	8.98	34.70	59.30	49.06	60.50	27.86	7.24	727.89	146.26
Ave.		13.31	14.71	16.66	14.48	7.22	8.85	34.63	57.03	48.43	60.68	24.97	8.32	790.25	198.22

represent the moment vs curvature curves for the beams tested up to 15 mm mid-span deflection and repaired by ferrocement. Also, Figs. 4.17 through 4.19 represent the moment vs curvature curves for the beams tested up to ultimate mid-span deflection and repaired by ferrocement.

Load vs crack width

Figs. 4.20 through 4.22 represent the load vs crack width curves for the beams tested up to 10 mm mid-span deflection and repaired by ferrocement. Figs. 4.23 through 4.25 represent the load vs crack width curves for the beams tested up to 15 mm mid-span deflection and repaired by ferrocement. Also, Figs. 4.26 through 4.28 represent the load vs crack width curves for the beams tested up to ultimate mid-span deflection and repaired by ferrocement.

Permeability test results

The permeability test applied to the cubes was in accordance with the German Industrial Standards (DIN 1045). The test of the three cubes of concrete made from the same mix of the beams gave an average value of to 21.7 mm depth of the water from the bottom of the cube, while the test of the other three cubes made of concrete on top of a 10 mm ferrocement gave a result of an average of 5.67 mm depth of the water from the bottom of the cube.

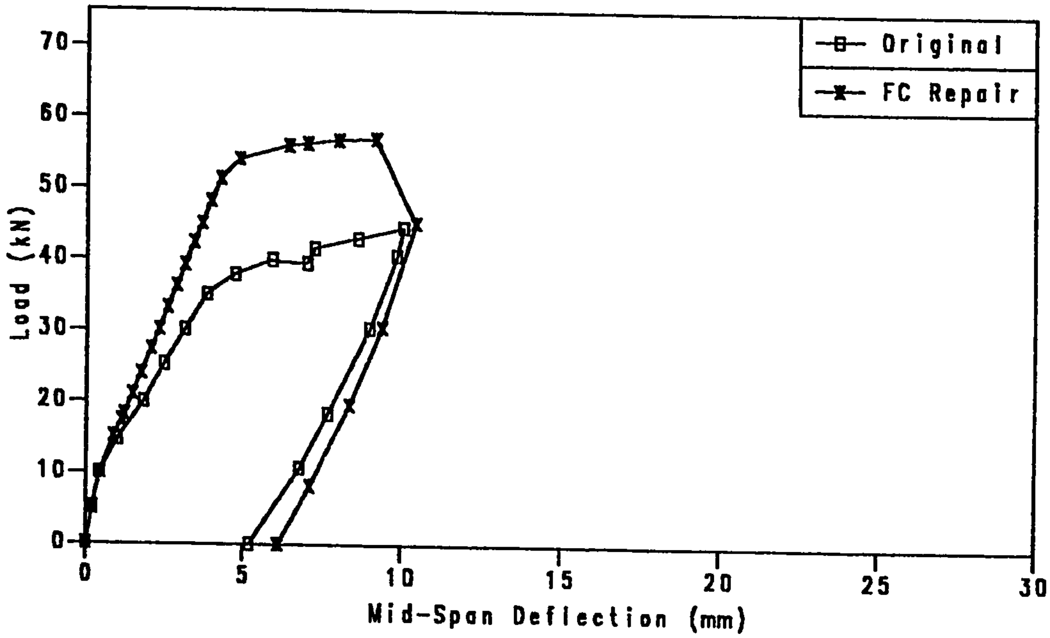

Fig. 4.2 Load Vs Deflection Curves for Beam A-i-1

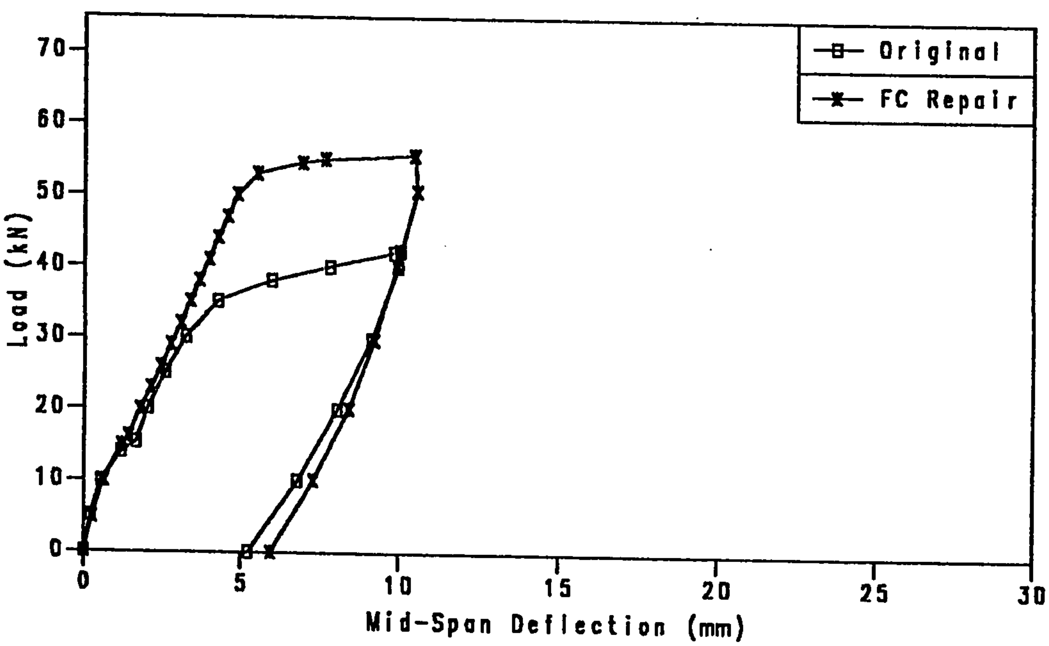

Fig. 4.3 Load Vs Deflection Curves for Beam A-i-2

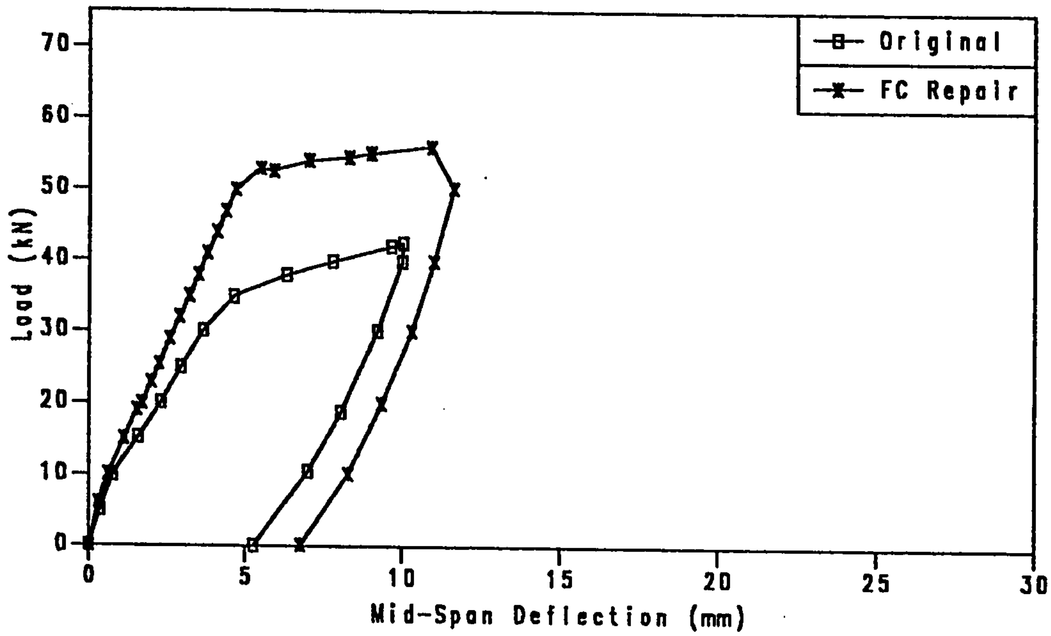

Fig. 4.4 Load Vs Deflection Curves for Beam A-i-3

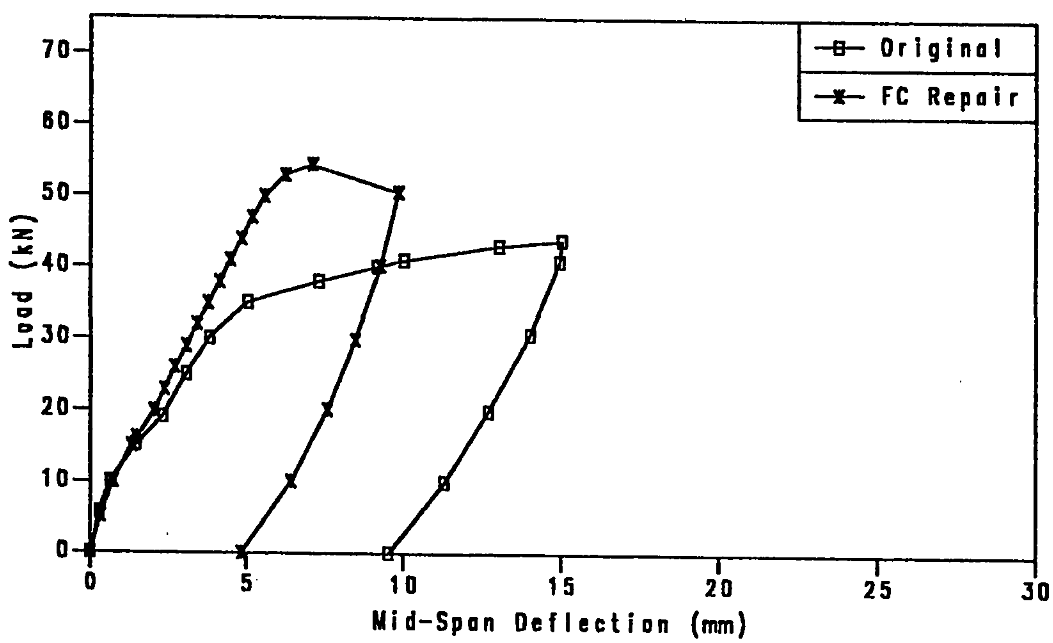

Fig. 4.5 Load Vs Deflection Curves for Beam B-i-1

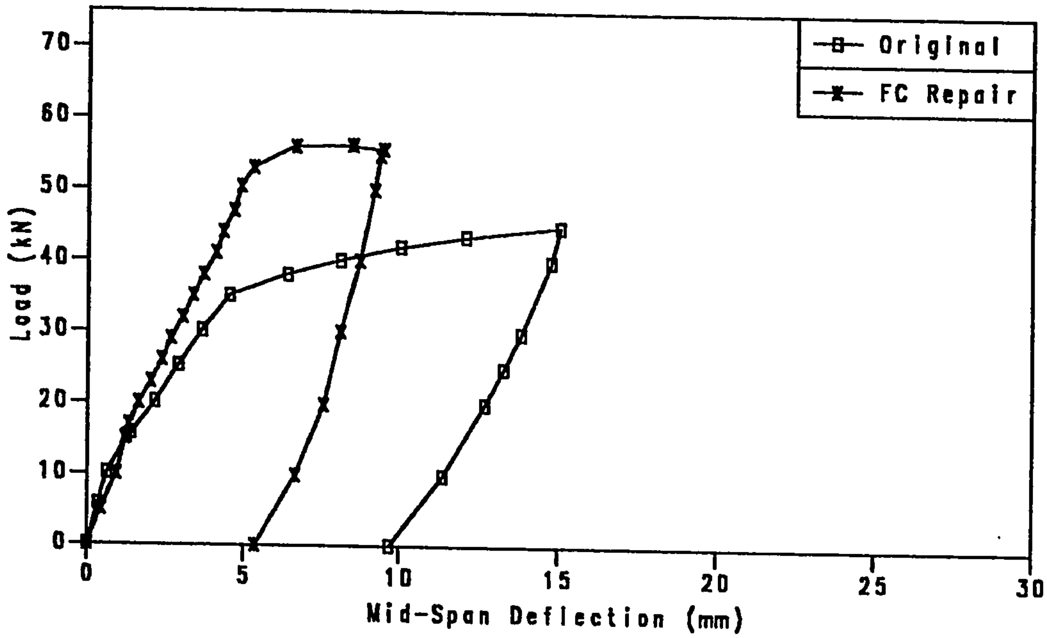

Fig. 4.6 Load Vs Deflection Curves for Beam B-i-2

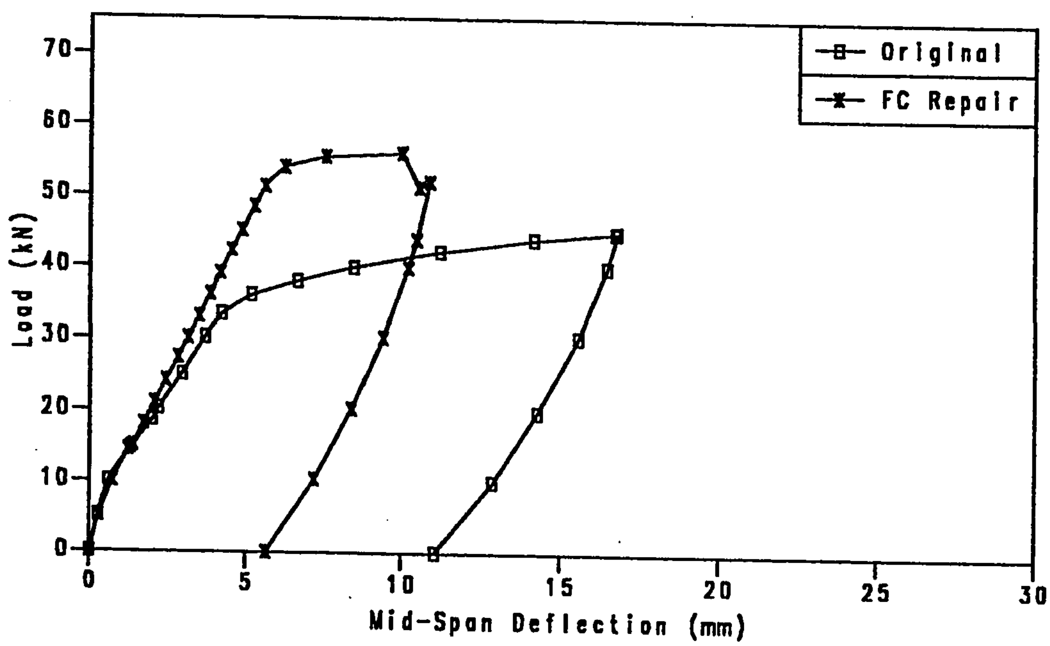

Fig. 4.7 Load Vs Deflection Curves for Beam B-i-3

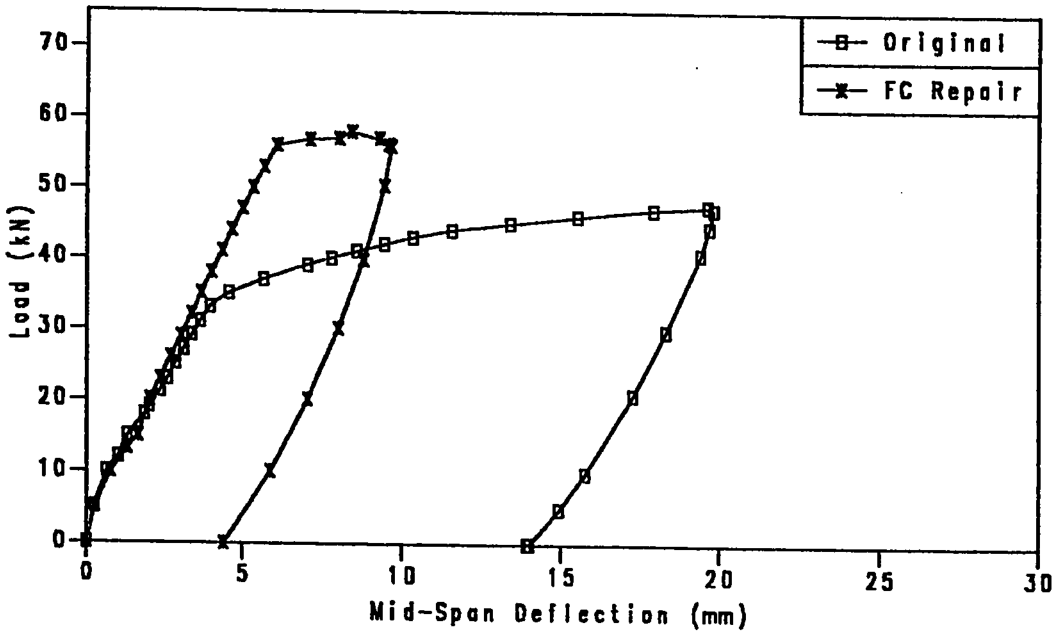

Fig. 4.8 Load Vs Deflection Curves for Beam C-i-1

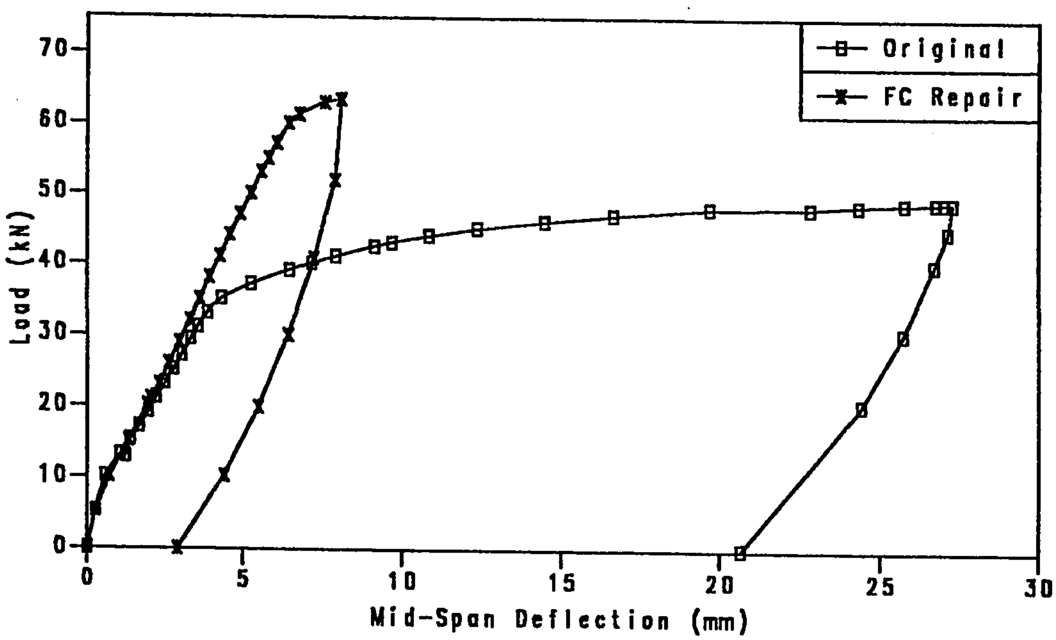

Fig. 4.9 Load Vs Deflection Curves for Beam C-i-2

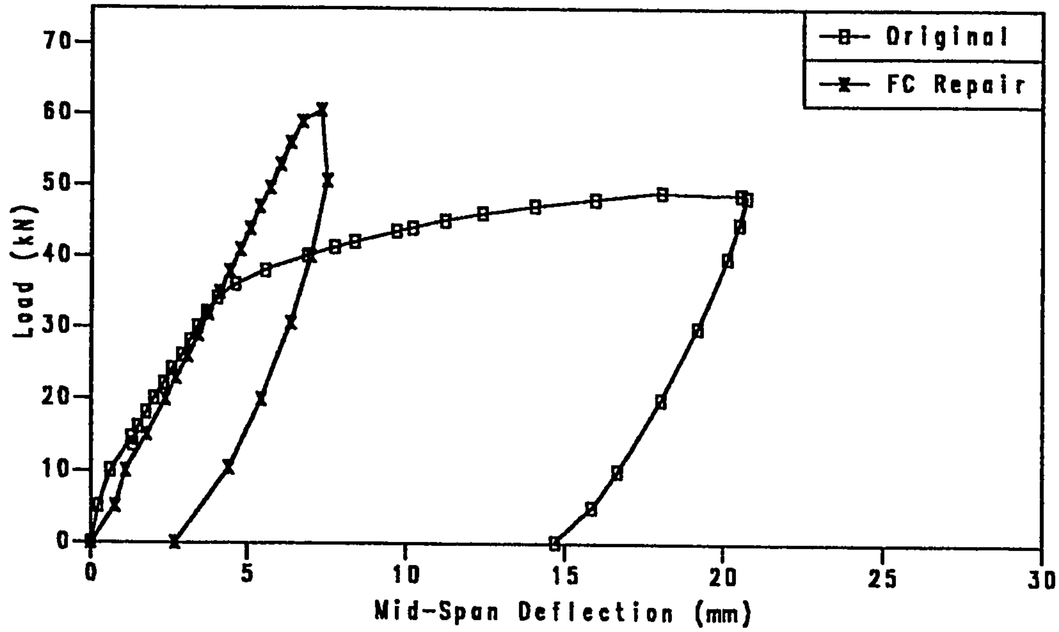

Fig. 4.10 Load Vs Deflection Curves for Beam C-i-3

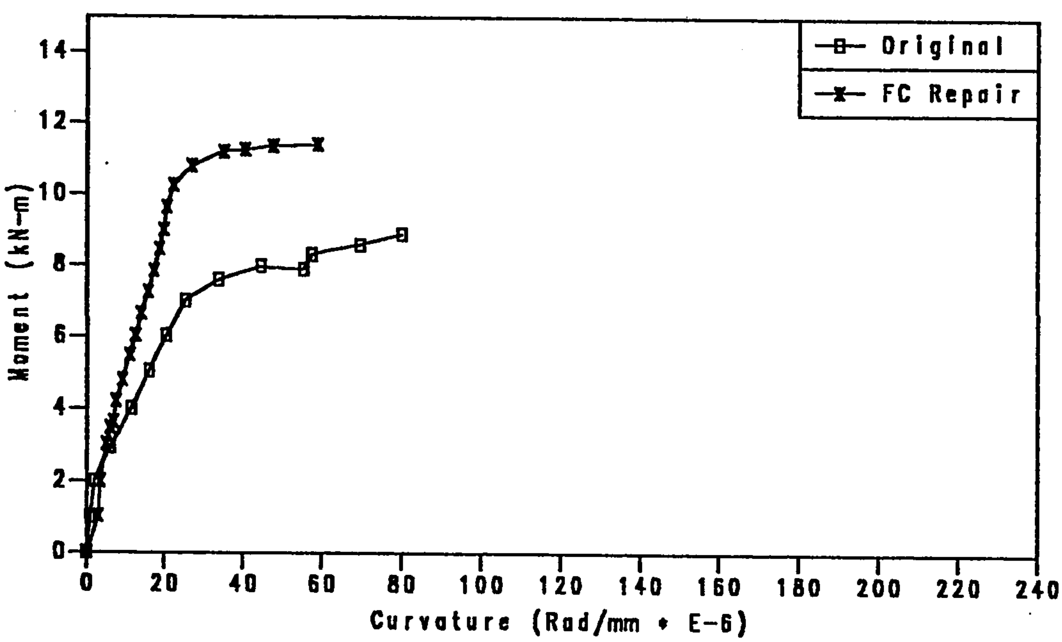

Fig. 4.11 Moment Vs Curvature Curves for Beam A-i-1

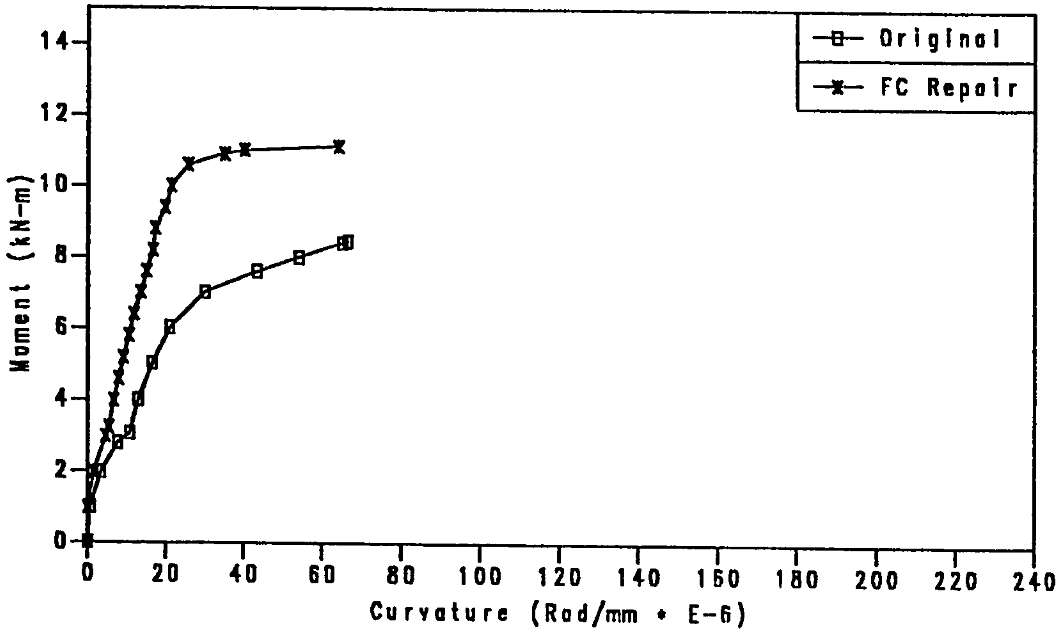

Fig. 4.12 Moment Vs Curvature Curves for Beam A-i-2

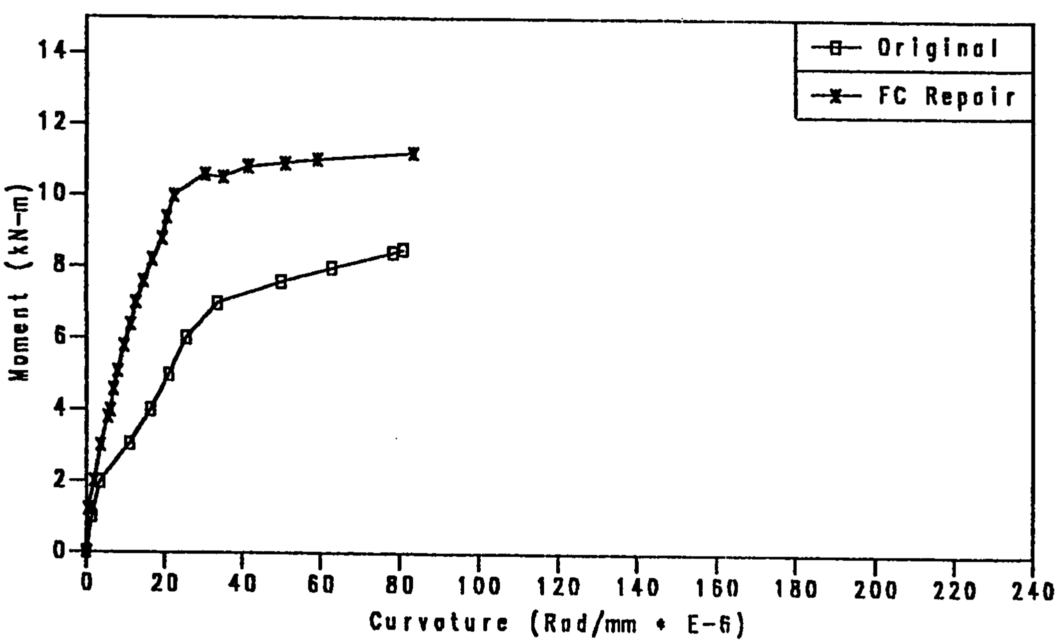

Fig. 4.13 Moment Vs Curvature Curves for Beam A-i-3

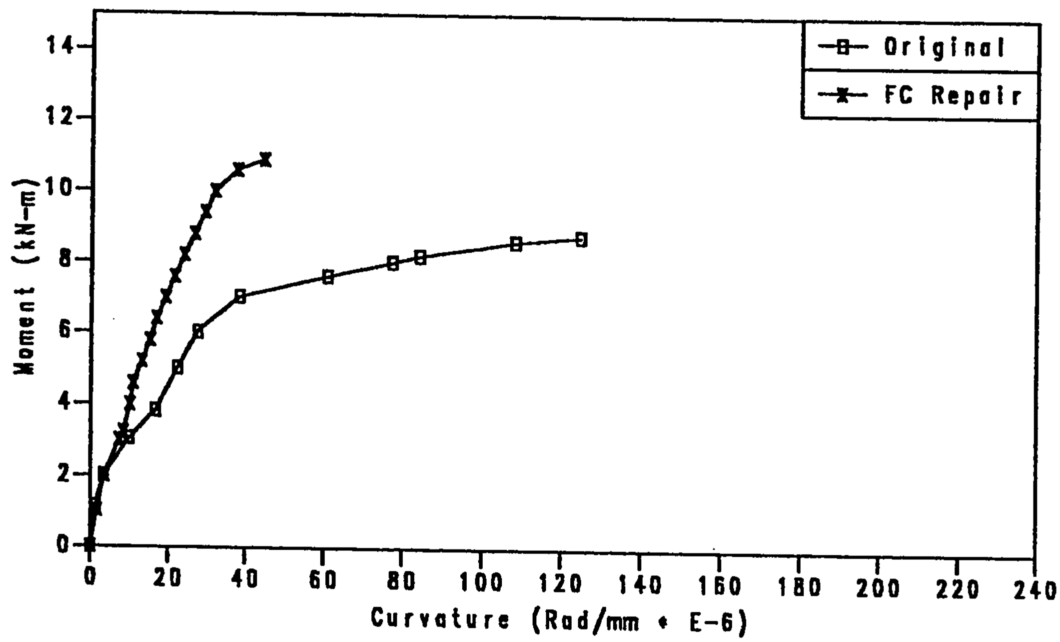

Fig. 4.14 Moment Vs Curvature Curves for Beam B-i-1

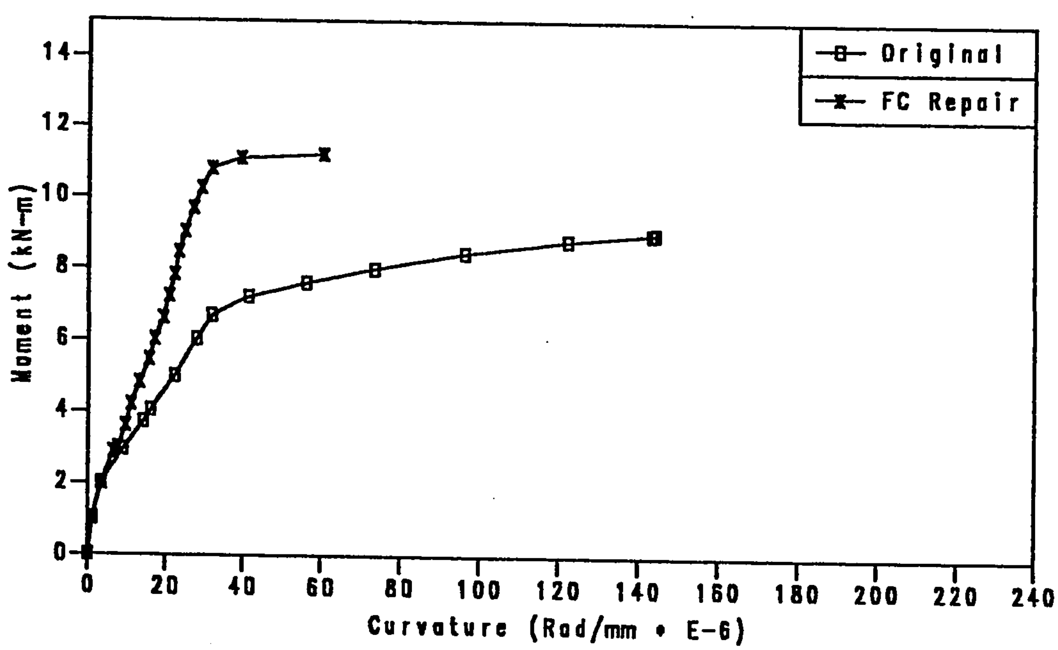

Fig. 4.15 Moment Vs Curvature Curves for Beam B-i-2

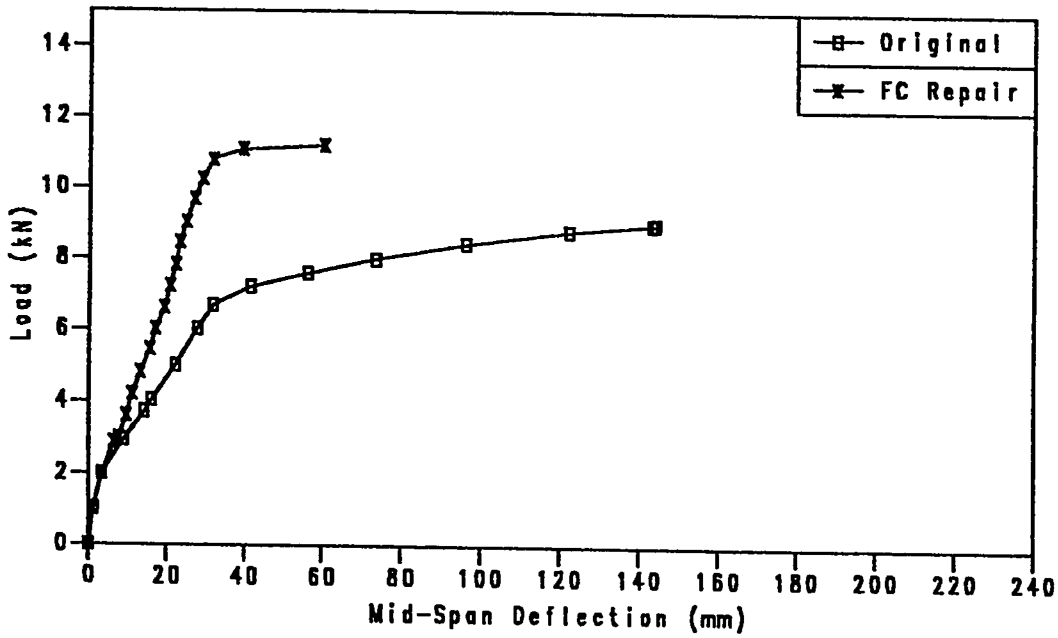

Fig. 4.16 Moment Vs Curvature Curves for Beam B-i-3

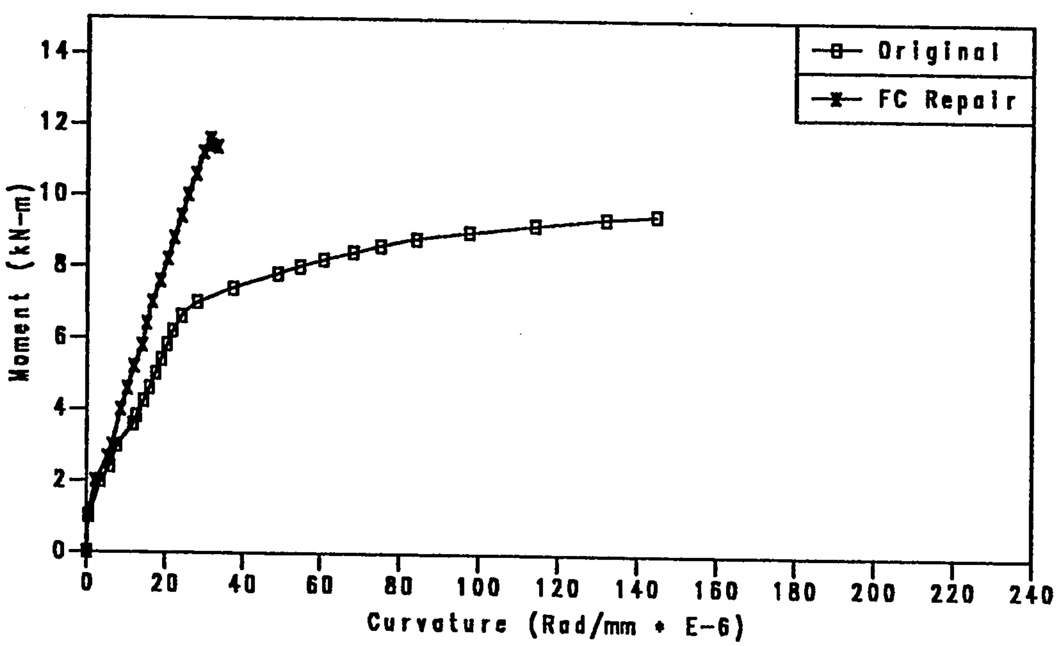

Fig. 4.17 Moment Vs Curvature Curves for Beam C-i-1

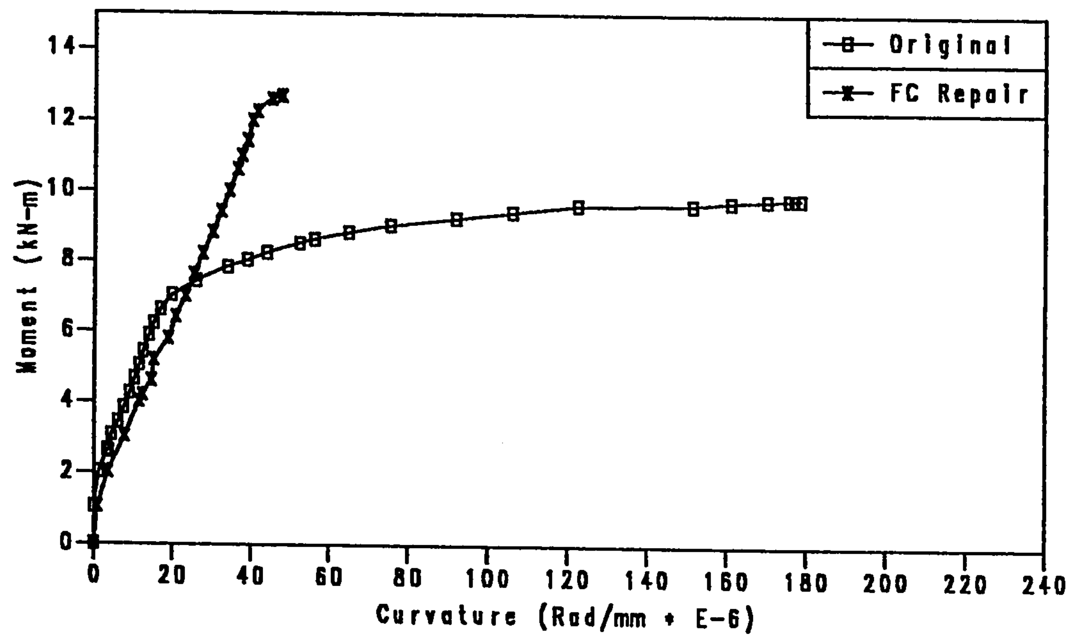

Fig. 4.18 Moment Vs Curvature Curves for Beam C-i-2

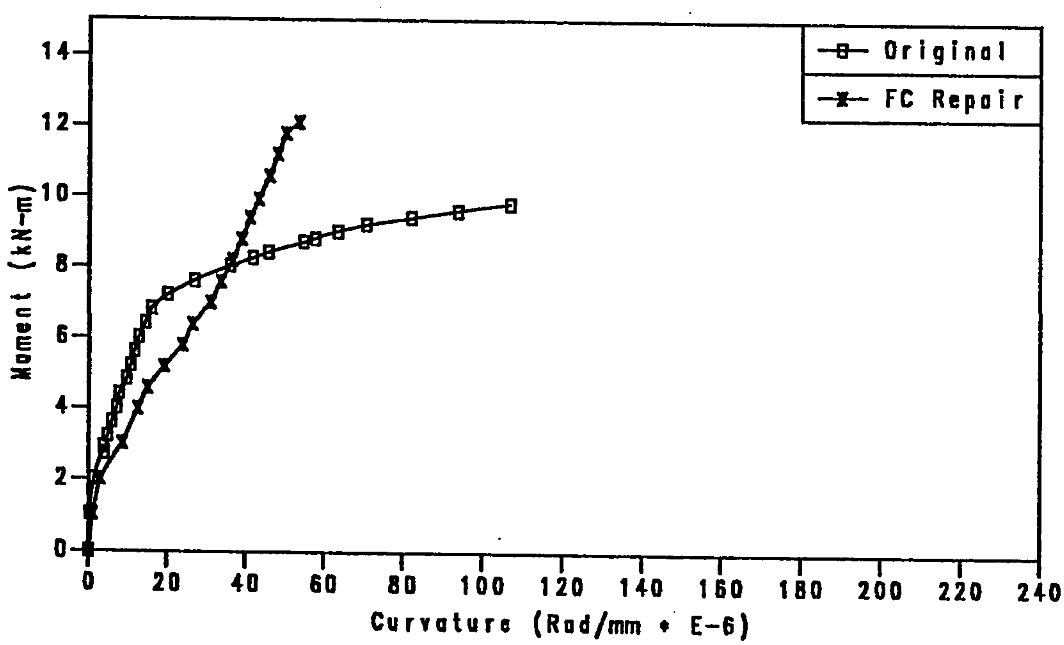

Fig. 4.19 Moment Vs Curvature Curves for Beam C-i-3

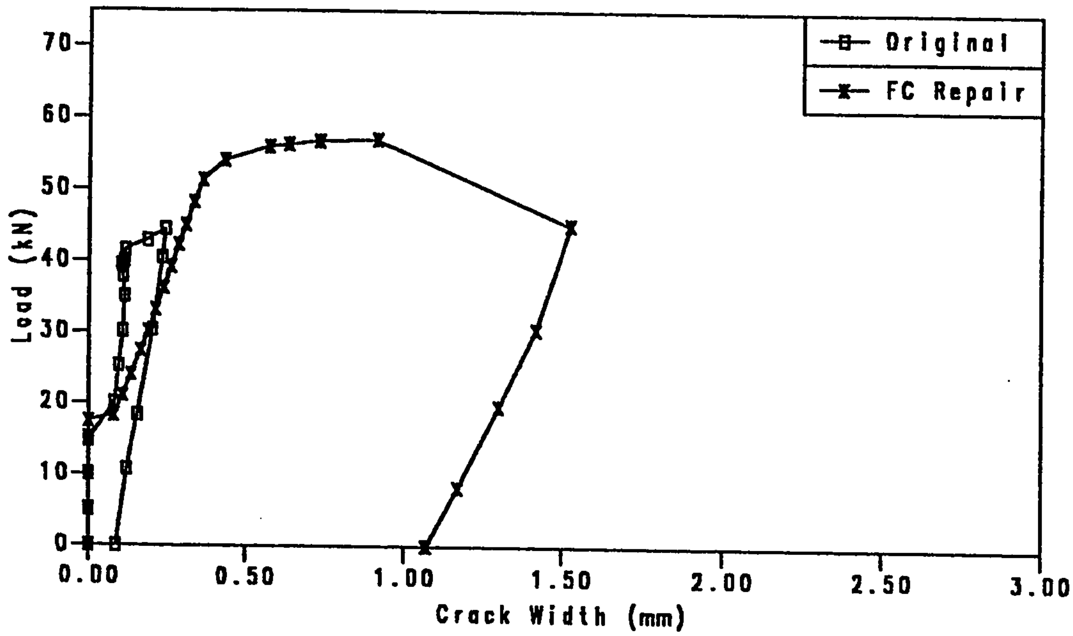

Fig. 4.20 Load Vs Crack Width Curves for Beam A-i-1

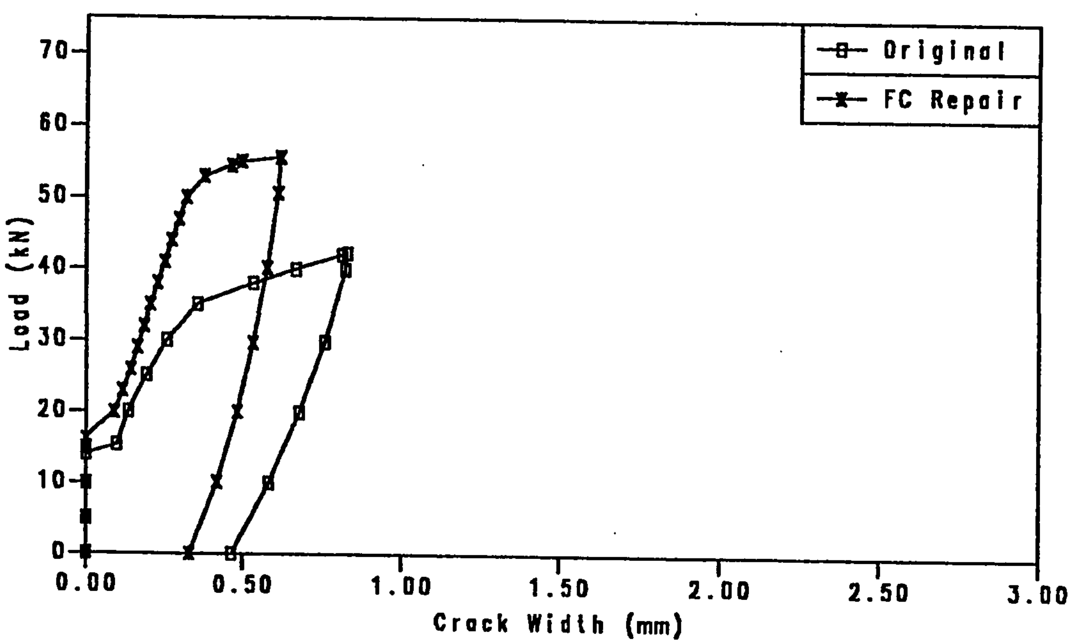

Fig. 4.21 Load Vs Crack Width Curves for Beam A-i-2

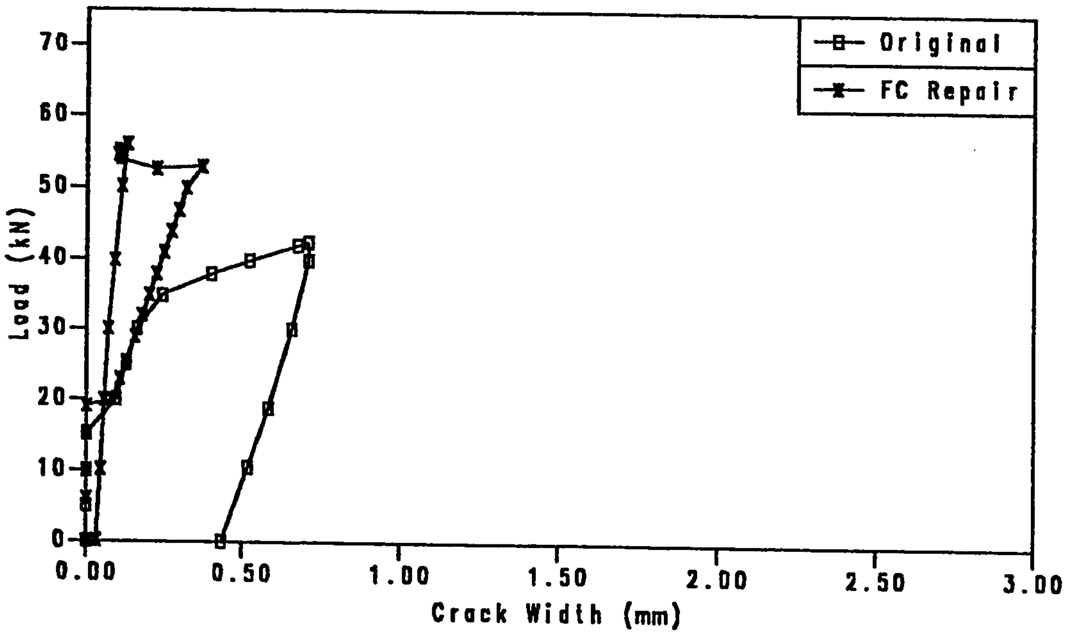

Fig. 4.22 Load Vs Crack Width Curves for Beam A-i-3

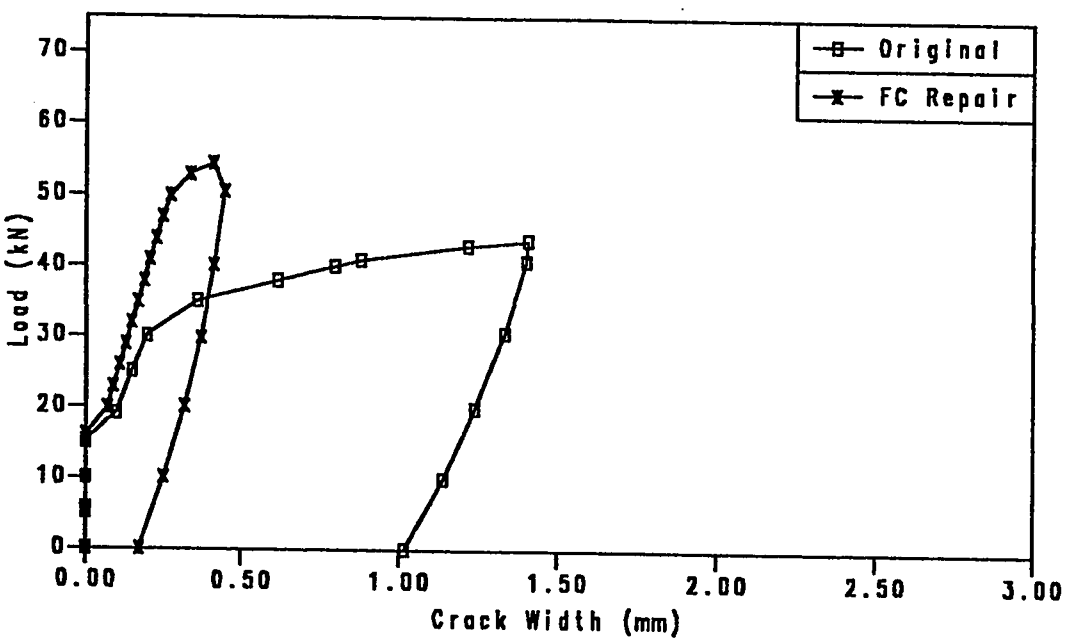

Fig. 4.23 Load Vs Crack Width Curves for Beam B-i-1

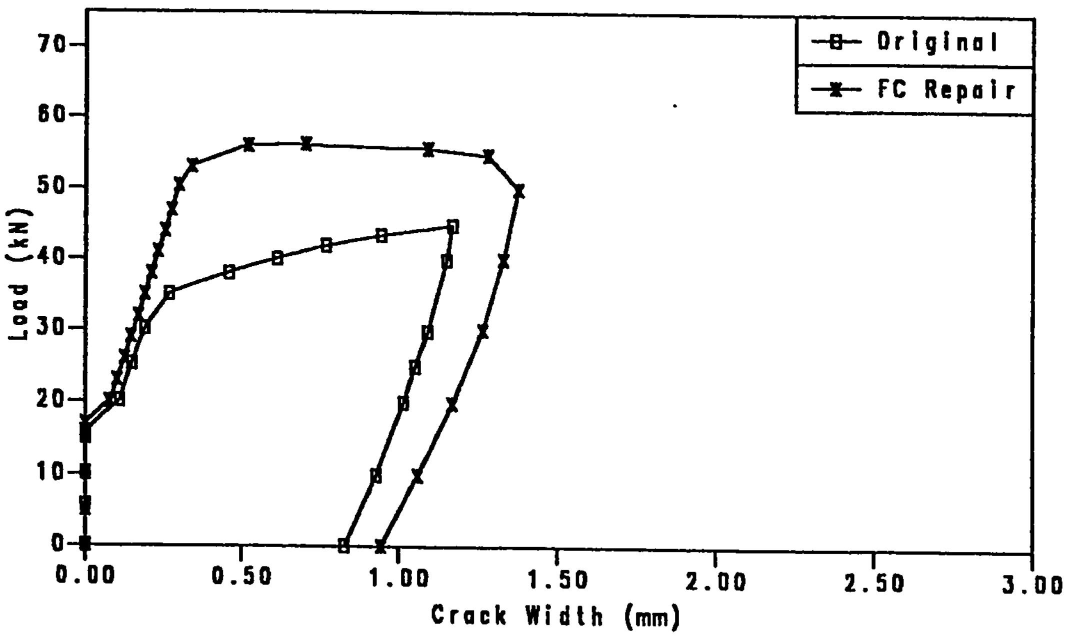

Fig. 4.24 Load Vs Crack Width Curves for Beam B-i-2

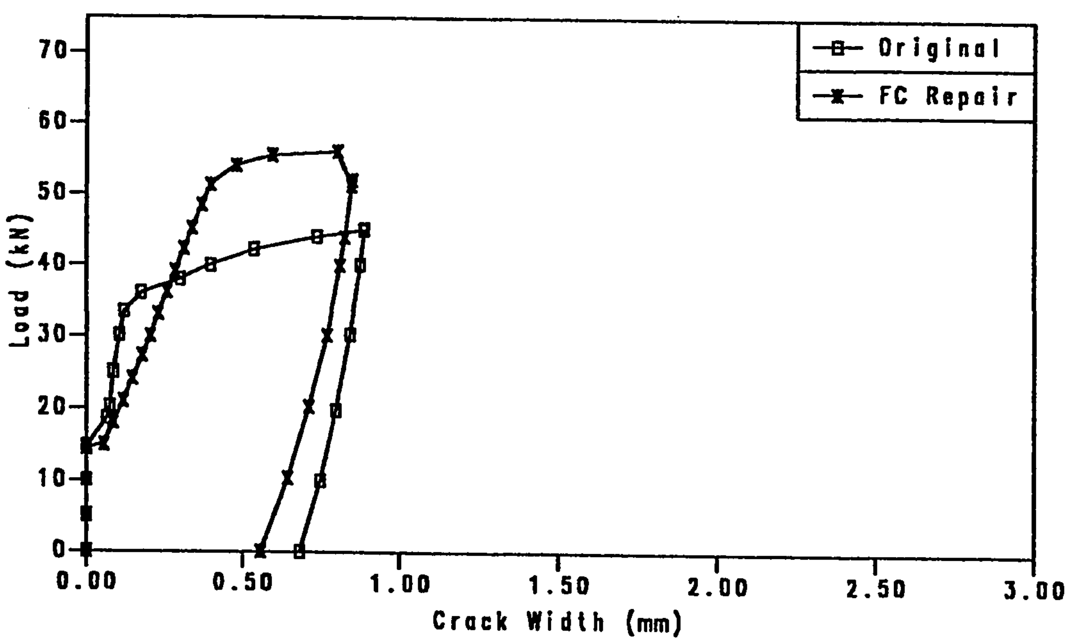

Fig. 4.25 Load Vs Crack Width Curves for Beam B-i-3

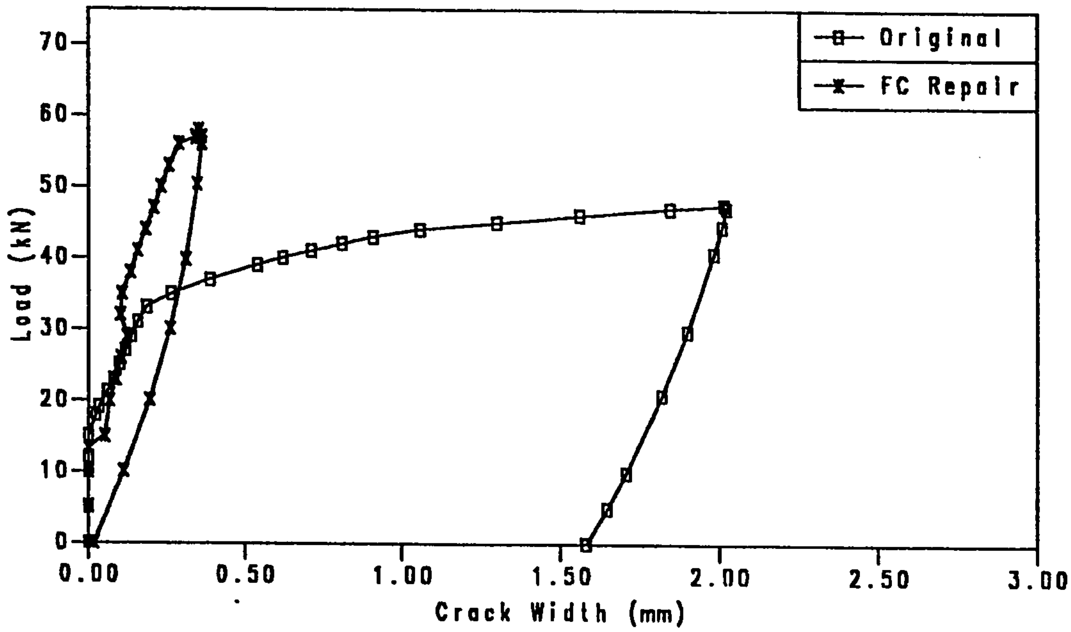

Fig. 4.26 Load Vs Crack Width Curves for Beam C-i-1

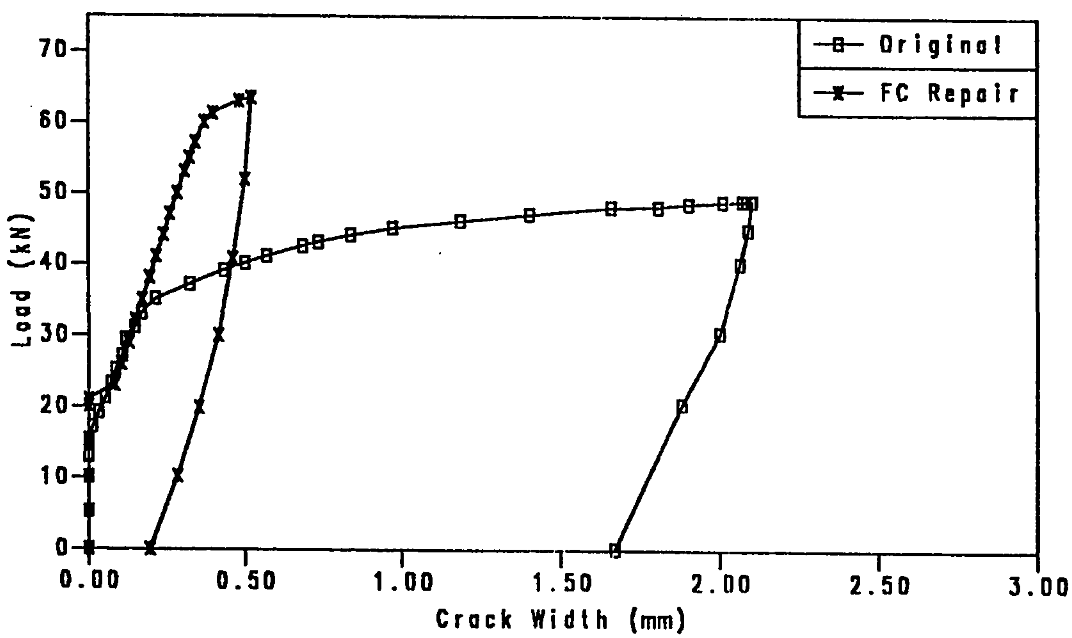

Fig. 4.27 Load Vs Crack Width Curves for Beam C-i-2

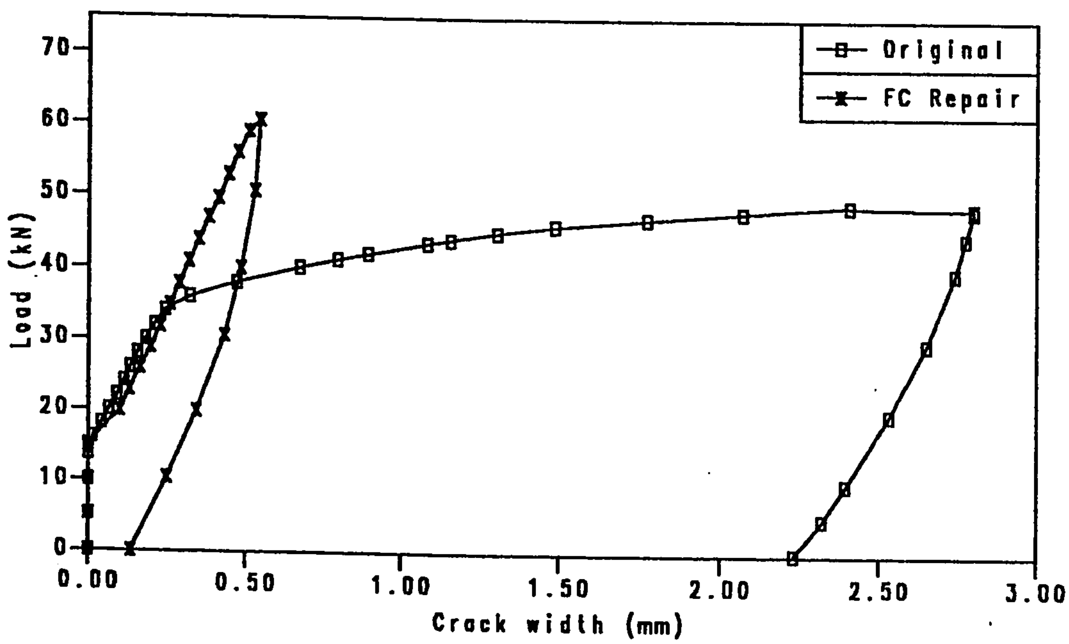

Fig. 4.28 Load Vs Crack Width Curves for Beam C-i-3

4.2.2 Discussion of ferrocement results

Calculating ultimate capacity

The load capacity of the beam was found to increase due to the additional layer of ferrocement from the bottom and on the two sides of the beam, the sides which are accessible in most structures.

Only one layer of wire mesh was used in the ferrocement repair method, which was a two dimensional square mesh of a 7.54 mm square opening, diameter of 0.9 mm and ultimate strength of 414 MPa. The wire mesh was applied to the beam from three sides like a channel of 1100 mm long and 125 mm in depth. Fig. 4.29 shows the ferrocement repaired beam cross section. For the analysis of ultimate load capacity of the ferrocement repaired beams, the tensile forces were calculated from Fig. 4.29, as follows:

The yielding stresses of the reinforcing bars and the wire meshes are = f_y = 414 MPa (60000 psi)

$T_1 = A_{s_1} f_{y_1}$

where

A_{s_1} = area of reinforcing rebars, and

f_{y_1} = the yielding stress of reinforcing rebars.

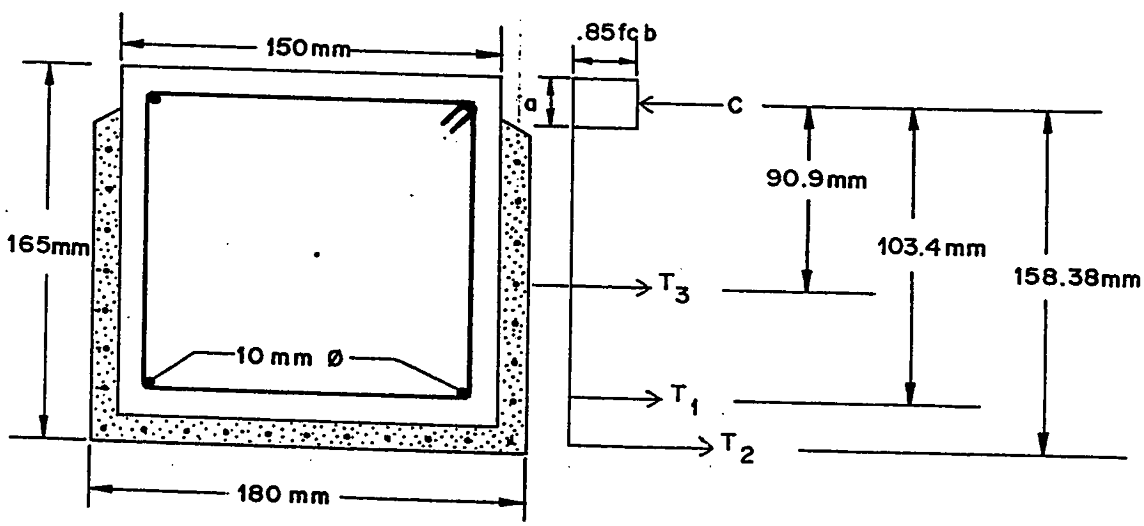

Fig. 4.29 : Ferrocement beam cross section.

$T_1 = [2*(10)^2\pi/4]*414*10^{-3} = 65.04kN$

T_2 = Area of all wires at bottom level * f_y of wiremesh

$T_2 = A_{s_2} f_{y_2}$

$A_{s_2} = (0.9)^2*\pi/4(1500/75.4) = 12.66\ mm^2$

$T_2 = 12.66*414*10^{-3} = 5.24\ kN$

T_3 = Area of all wires on both sides * f_y of wiremesh

$T_3 = 2 * A_{s_3} f_{y_3}$

$A_{s_3} = 2*(0.9)^2*\pi/4(1250/75.4) = 21.25\ mm^2$

$T_3 = 21.25*414*10^{-3} = 8.8\ kN$

$T=T_1+T_2+T_3 = 65.04+5.24+8.80 = 79.08\ kN$

$$T = C \rightarrow a = \frac{A_s\ f_y}{0.85f'_c b}$$

$$\text{Therefore}\quad a = \frac{79.08}{0.85*31*0.18} = 16.65\ mm$$

M_n = total nominal flexural strength.

Ignoring top reinforcement of the beam and taking moment around C,

$M_n = T_1$*moment arm + T_2*moment arm + T_3*moment arm

$M_n = T_1*103.4 + T_2*158.38 + T_3*90.9$

$M_n = 65.04*103.4 + 5.24*158.38 + 8.8*90.9 = 8.355\ kN\text{-}m$

$$M_n = \frac{P_n\ L}{6} + \frac{W_D L^2}{8}$$

$$M_n(DL) = \frac{M_u}{0.9} = \frac{W_D L^2 * 1.4}{0.9*8}$$

W_D = dead load of a beam repaired by ferrocement

$M_n(DL)$ = nominal moment due to dead load of beam and ferrocement

$$W_D = 24.0\ kN/m^3*(0.180)*(0.165)m^2 = 0.713\ kN/m$$

$$M_n(DL) = \frac{0.713*(1.20\ m)^2*1.4}{8*0.9} = 0.200\ kN\text{-}m$$

$$M_n = 8.355 = 0.200 + \frac{P_n*1.2}{6} \rightarrow P_n = 40.75\ kN$$

Therefore the nominal load $P_n = 40.75$ kN while the theoretical load capacity of the beam without ferrocement is 35.6 kN. Therefore the load capacity of the beam has been increased by 14.5 % by the addition of ferrocement layer.

Load vs deflection

The load vs deflection curves, for all beams of 10, 15 mm and ultimate deflections repaired by ferrocement, show increase in the results of the repaired beams when compared with the original ones. The ultimate load of the repaired beams if compared to the similar beams tested up to ultimate, the percentage increase in the ultimate strength varies between 15 to 25 % which has been supported by the theoretical calculations shown above due to the addition of ferrocement layer to the beam. Using the maximum load of the original beams, the corresponding deflections were reduced very significantly. This can be explained from the significant increase in the stiffness of the beam after repair by ferrocement. In other words, the increase in the ultimate load and stiffness came from the increase in the cross section of the beam and the amount of steel. But, there was a decrease in the ductility as shown in table 4.2 due to the presence of cracks, even after repair. Thus is also obvious from the decrease in the ultimate deflection, as shown in table 4.2.

Moment vs curvature

From the moment vs curvature curves, it can be concluded that all beams tested up to 10 and 15 mm deflections and repaired by ferrocement showed an increase in the moment capacity. The amount of increase is dependent on the level of deflection which the beam have been subjected to before repair. Also, the increase can be looking at it from the point view of cracking and cracking size, beams which have been tested to failure then repaired have shown up to 25% increase in the moment capacity in comparison to beams subjected to 10 and 15 mm deflection which have shown only 16 and 15 % increase in moment capacity respectively. The rigidity of the beams was increased very significantly and that can be clearly observed from Figs. 4.11 through 4.16 . But for beams tested up to ultimate deflection and repaired by ferrocement, no increase in rigidity is observed in spite of the increase in the moment capacity.

Crack patterns

The crack patterns of the repaired beams by ferrocement can be seen from plate 4.1 . It was very clear that beams repaired with ferrocement exhibited multiple of fine cracks at ultimate deflection in comparison with unrepaired beams and some of them tried to close up after relaxation, because of their fineness and stability.

It is also has been observed that most of the new cracks occured at the same place as in the original ones especially for the 10 mm deflection group where mortar can not fill the very fine cracks, which will be the first ones to show up. The cracking load was increased after repair as shown in table 4.2 .

From Figs. 4.20 through 4.28 depicting load vs crack width curves and table 4.2, the cracking load of ferrocement repaired beams decreased as the tested load of original beams increased and that is due to the decrease in the moment of inertia of the cracked beams. Also, from the same figures, the crack width of the ferrocement repaired beams

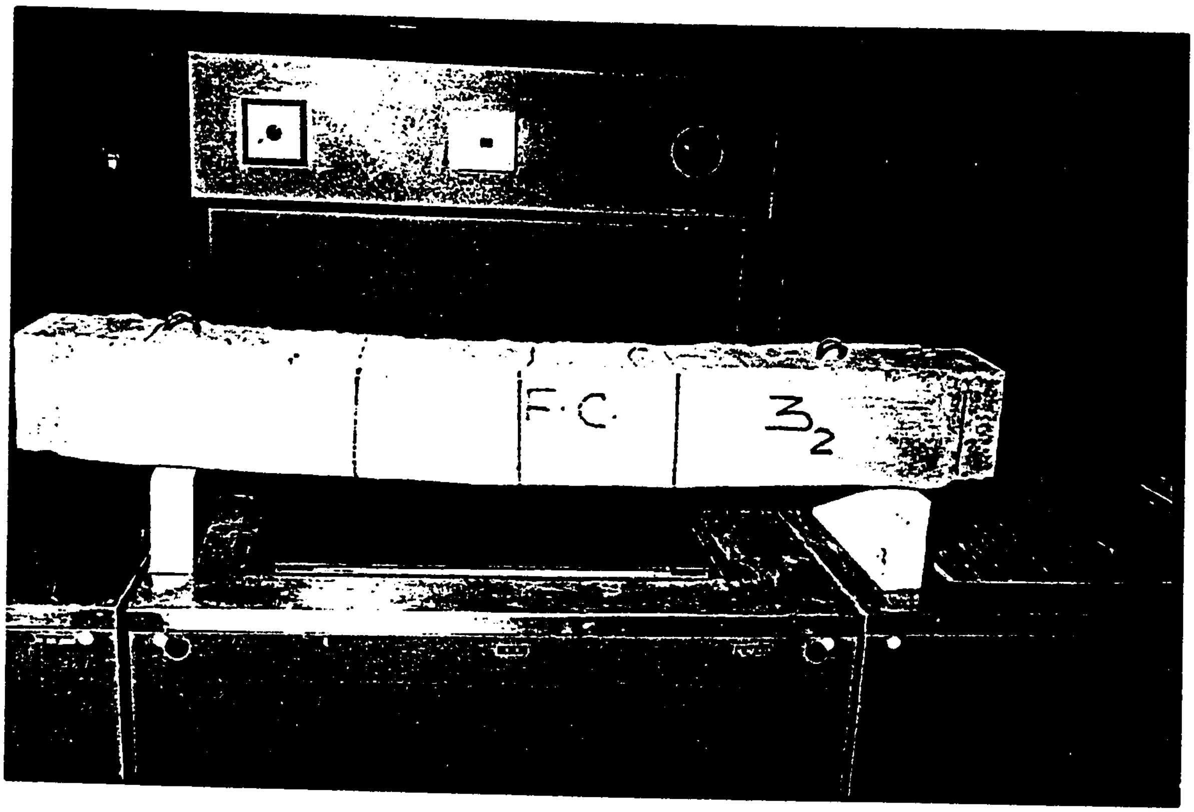

Plate 4.1 : Crack patterns of ferrocement repair method.

decreased compared to the original ones, especially for some of the beams having 10 and 15 mm deflections. But, for the beams with ultimate deflection there was a greater decrease in the crack width in comparison with the original beams. This is due to the filling of the wide cracks by the mortar.

Beams repaired with ferrocement at the same level of loading as that of unrepaired beams showed finer cracks and reduced deflections in comparison to the original ones.

Permeability test results

The permeability test applied to the cubes according to the DIN 1045 standards.

1- Cubes of regular mix :
After the test, the cubes were split into 2 parts and the average depth of the water in the concrete was found to be equal to 21.7 mm, as shown in plate 4.2 .

2- Cubes with a 10 mm ferrocement layer :
The average depth of the water in the concrete was observed to be equal to 5.67 mm, as shown in plate 4.3 .

Comparing the above two results, the depth of the water in the concrete was reduced by 74 % when using ferrocement, which means that the ferrocement used for the repair will reduce the permeability of concrete considerably. This will help in protecting the reinforcing bars and cement from

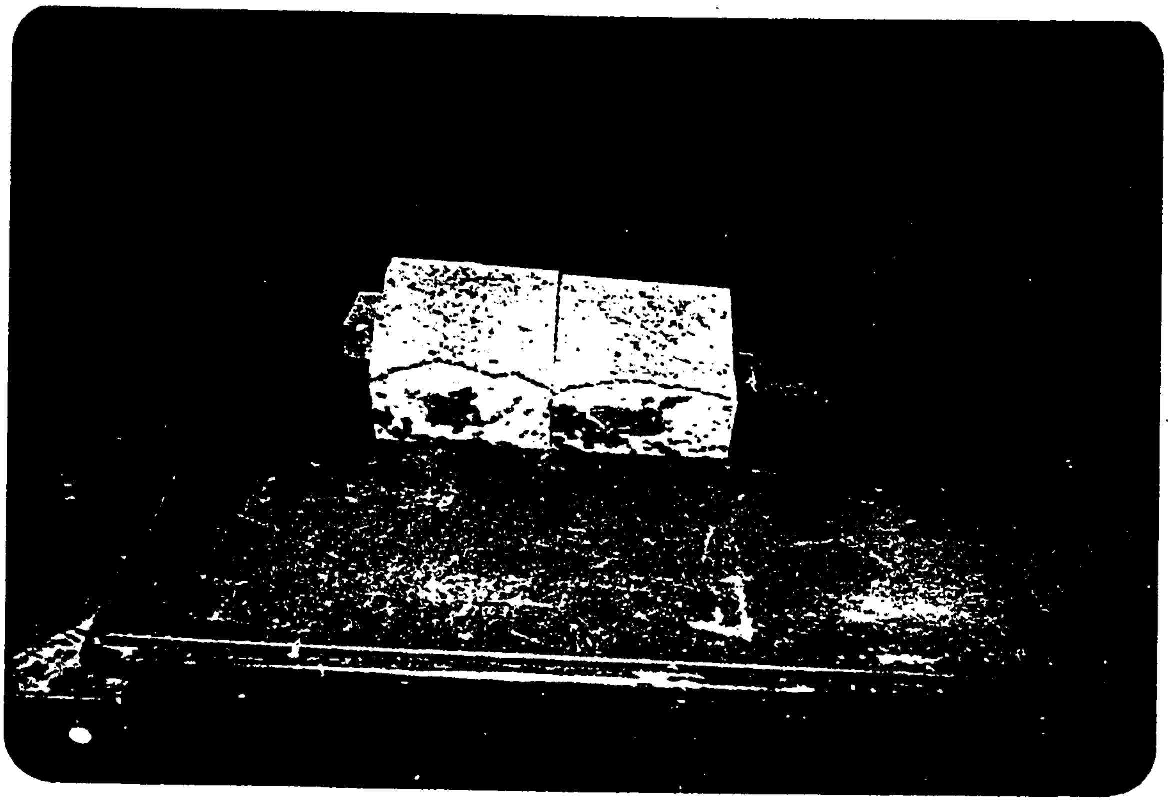

Plate 4.2 : Permeability test results for cubes of concrete mix design.

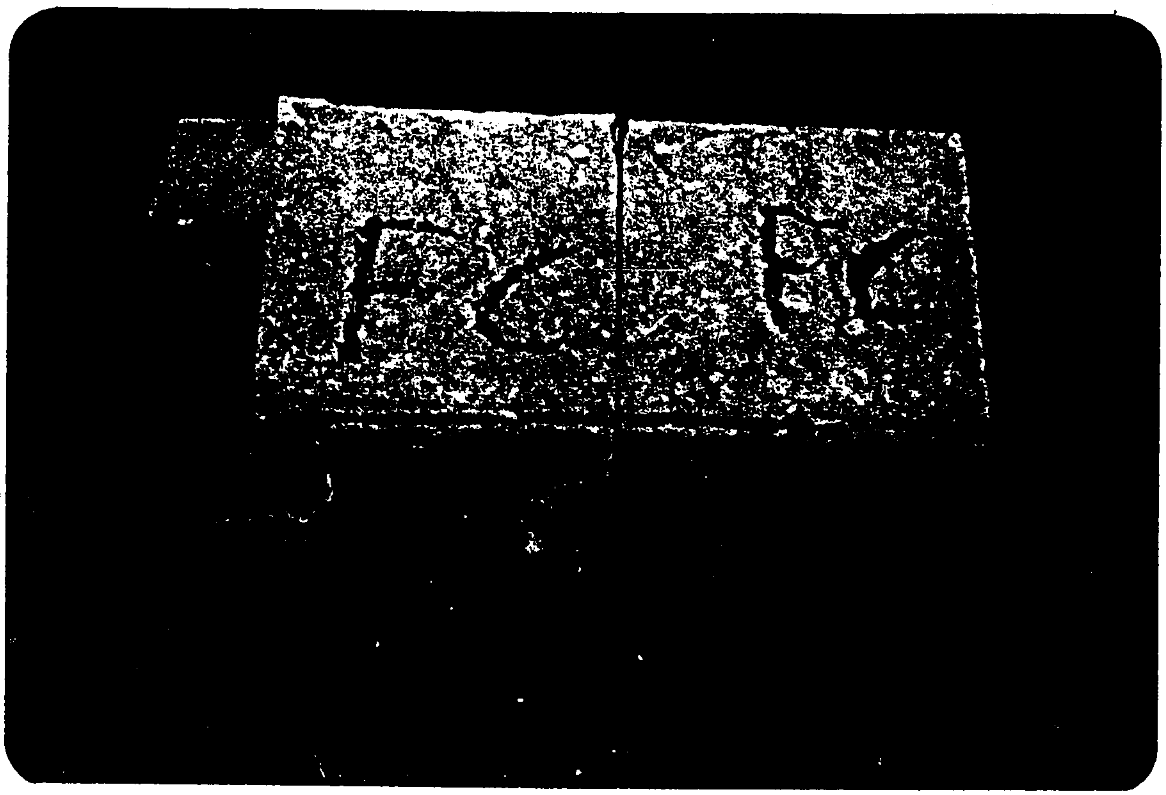

Plate 4.3 : Permeability test results of cubes with a 10 mm ferrocement layer.

corrosion and sulfate attack respectively.

4.3 Epoxy Repair Method

Epoxy injection of cracks is a common method for repairing concrete structures. In this study, three levels of cracks which were produced by 10, 15 mm and ultimate deflections were repaired by epoxy injection. For each level three beams were used to study the behavior of the repair method and repair materials.

4.3.1 Presenting the results of epoxy injection

Load vs deflection

Figs. 4.30 through 4.32 show the load vs deflection curves of beams which had been subjected to a load which produced 10 mm mid-span deflection and repaired by epoxy injection, while Figs. 4.33 through 4.35 show the load vs deflection curves of beams which had been subjected to a load which produced 15 mm mid-span deflection and repaired by epoxy injection. And, Figs. 4.36 through 4.38 show the load vs deflection curves of beams which had been subjected to a load which produced ultimate mid-span deflection and repaired by epoxy injection.

From these curves, the initial stiffness K_i, reduced stiffness K_r, the cracking load P_{cr}, yielding load P_y and testing load P_t were obtained. Also, the ductility represented by the area under the P-Δ curves and ultimate deflections were found for the ultimate group and all the repaired beams. The collected information is presented in table 4.3 for all beams before and after repair.

Moment vs curvature

Figs. 4.39 through 4.41 represent the moment vs curvature curves of beams which had been subjected to a load which produced 10 mm mid-span deflection and repaired by epoxy injection, while Figs. 4.42 through 4.44 represent the moment vs curvature curves of beams which had been subjected to a load which produced 15 mm mid-span deflection and repaired by epoxy injection. And, Figs. 4.45 through 4.47 represent the moment vs curvature curves of beams which had been subjected to a load which produced ultimate mid-span deflection and repaired by epoxy injection.

Moment and curvature readings for the original and repaired beams were calculated at every point of load recording and prepared for plotting as in section 4.1.

Load vs crack width

Figs. 4.48 through 4.50 represent the load vs crack width curves of beams which had been subjected to a load which produced 10 mm mid-span deflection and repaired by epoxy

Table 4.3 : Comparison of Original and Epoxy Repaired Beams

Item		P_{cr} (kN)		K_i (kN/mm)		K_r (kN/mm)		P_y (kN)		P_t (kN)		$\Delta_{ult.}$ (mm)		Ductility (kN-mm)	
Δ_c	Beam#	Unrep	Rep.	Unrep	Rep.	Unrep	Rep.	Unrep	Rep.	Unrep	Rep.	Unrep	Rep.	Unrep	Rep.
10mm	A-4	15.28	18.43	18.10	16.23	8.43	8.36	36.20	42.20	45.03	53.01	-	26.48	-	1095.24
	A-5	13.94	19.49	17.52	17.56	8.03	7.87	36.45	43.90	45.28	52.70	-	28.20	-	1151.36
	A-6	13.00	17.62	14.92	13.90	8.21	6.90	32.50	41.70	45.33	52.74	-	21.27	-	806.12
Ave.		14.07	18.03	16.85	15.90	8.22	7.71	35.05	42.60	45.21	52.82	-	25.32	-	1017.57
15mm	B-4	13.42	16.46	12.00	15.00	6.72	7.35	35.00	43.50	47.28	53.90	-	25.46	-	1023.81
	B-5	15.77	18.64	11.67	12.94	8.55	7.93	35.00	42.70	46.52	51.85	-	18.42	-	661.56
	B-6	16.67	17.47	16.76	16.30	7.77	6.92	35.20	44.00	46.49	52.64	-	22.34	-	869.05
Ave.		15.29	17.52	13.48	15.65	7.68	7.40	35.07	43.40	46.76	52.80	-	22.07	-	851.47
Ult.	C-4	13.50	18.71	22.15	15.40	7.48	7.64	36.00	43.00	51.10	54.68	27.38	18.25	1061.22	687.93
	C-5	14.19	16.07	16.33	15.85	8.20	7.27	33.90	42.00	51.93	51.01	26.97	17.04	1015.31	593.54
	C-6	14.23	19.05	14.35	13.11	8.69	8.15	33.00	41.20	51.01	50.01	27.86	12.78	1032.31	416.67
Ave.		13.97	17.94	17.61	15.63	8.13	7.69	34.30	42.07	51.38	52.23	27.40	16.02	1036.28	566.05

injection. Figs. 4.51 through 4.53 represent the load vs crack width curves of beams which had been subjected to a load which produced 15 mm mid-span deflection and repaired by epoxy injection. Also, Figs. 4.54 through 4.56 represent the load vs crack width curves of beams which had been subjected to a load which produced ultimate mid-span deflection and repaired by epoxy injection.

4.3.2 Discussion of epoxy injection results

Load vs deflection

The epoxy has been used to fill the cracks, which means that the cross sectional area of the beams unchanged after repair. Similarly other properties which depend on the cross sectional area would not change such as moment of inertia.

From the load-deflection curves the following observation can be drawn :

1) The cracking load (P_{cr}) for the repaired beams were increased by 29, 25 and 28 % respectively.
2) The initial stiffness (K_i) and the reduced stiffness (K_r) have been decreased for all levels of deflection.
3) The ultimate deflection for the repaired beams, have shown continuous decrease with increasing the amount of damage of the unrepaired beams. In other words, the

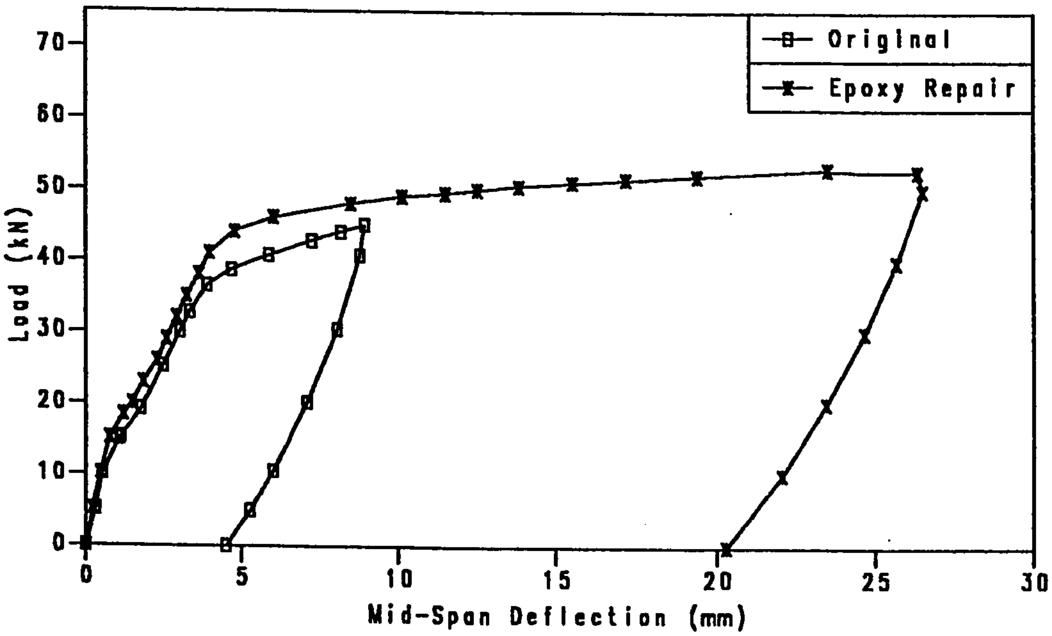

Fig. 4.30 Load Vs Deflection Curves for Beam A-ii-4

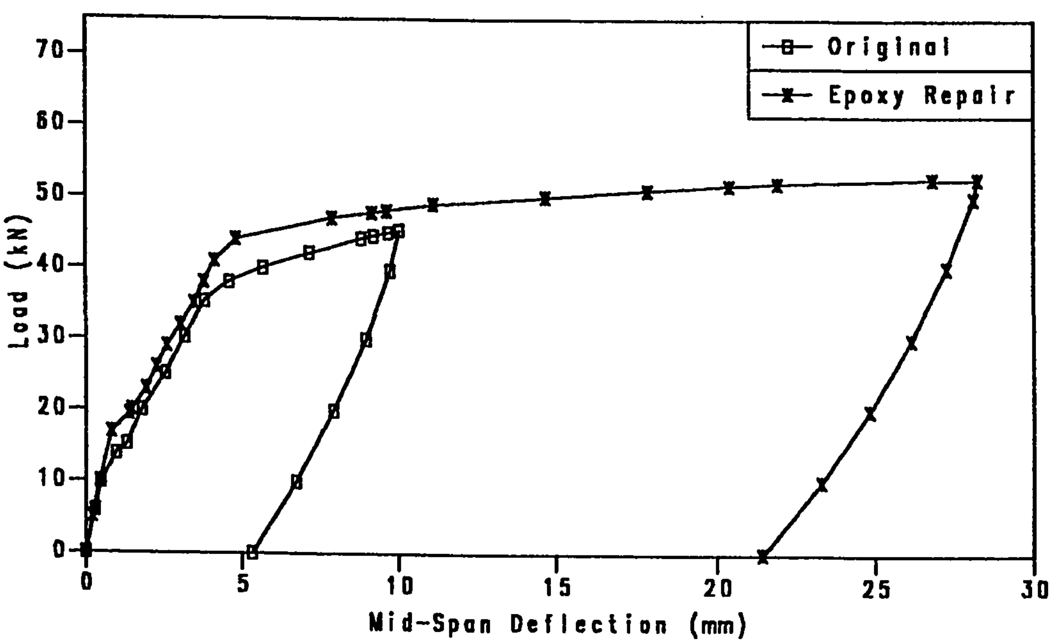

Fig. 4.31 Load Vs Deflection Curves for Beam A-ii-5

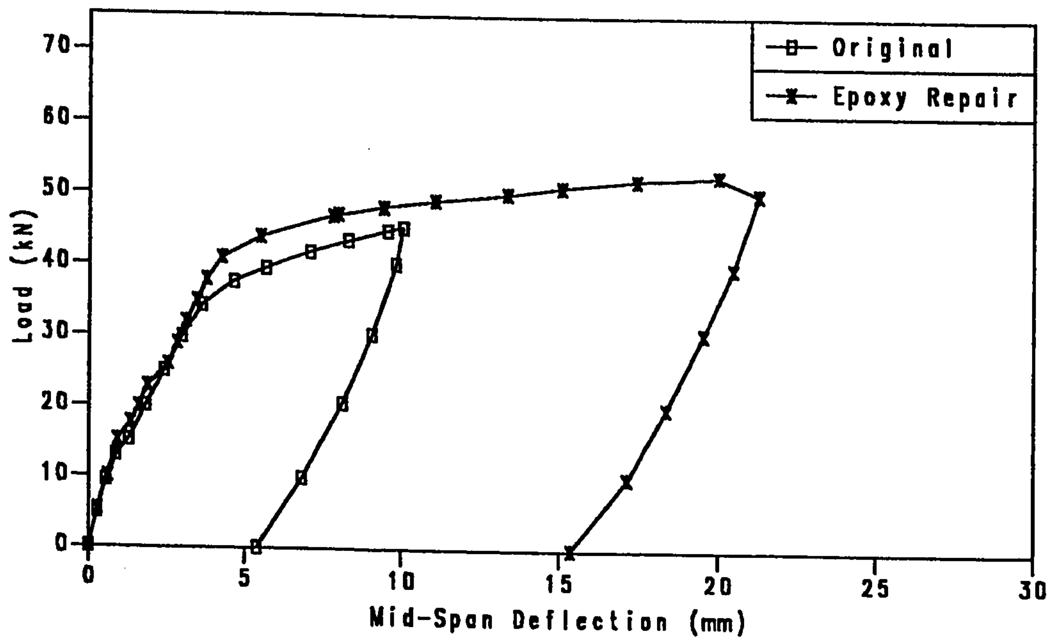

Fig. 4.32 Load Vs Deflection Curves for Beam A-ii-6

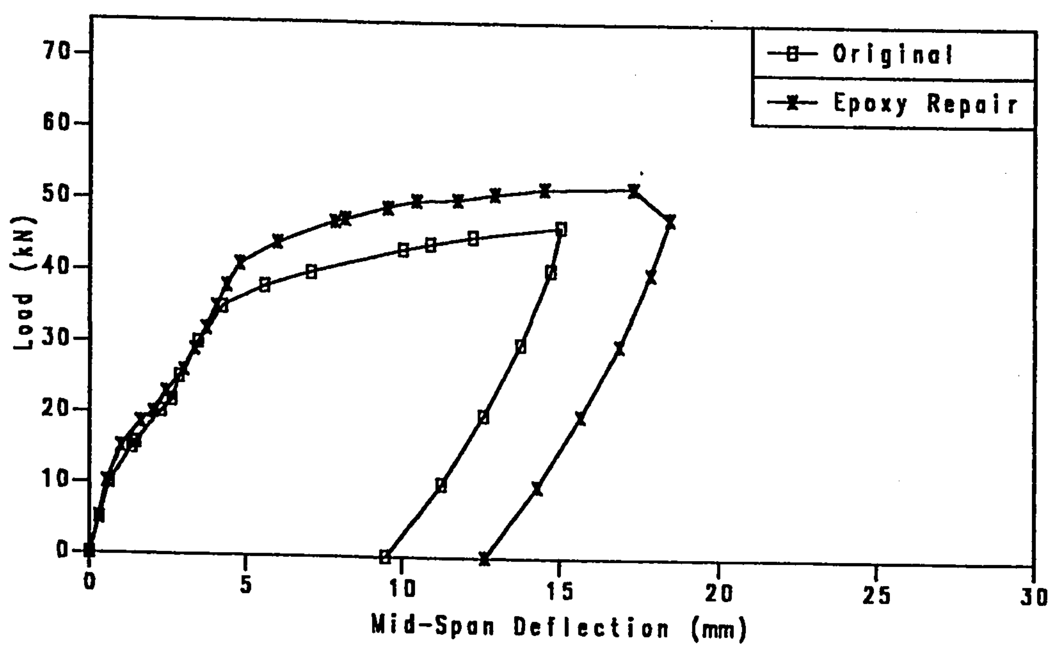

Fig. 4.33 Load Vs Deflection Curves for Beam B-ii-4

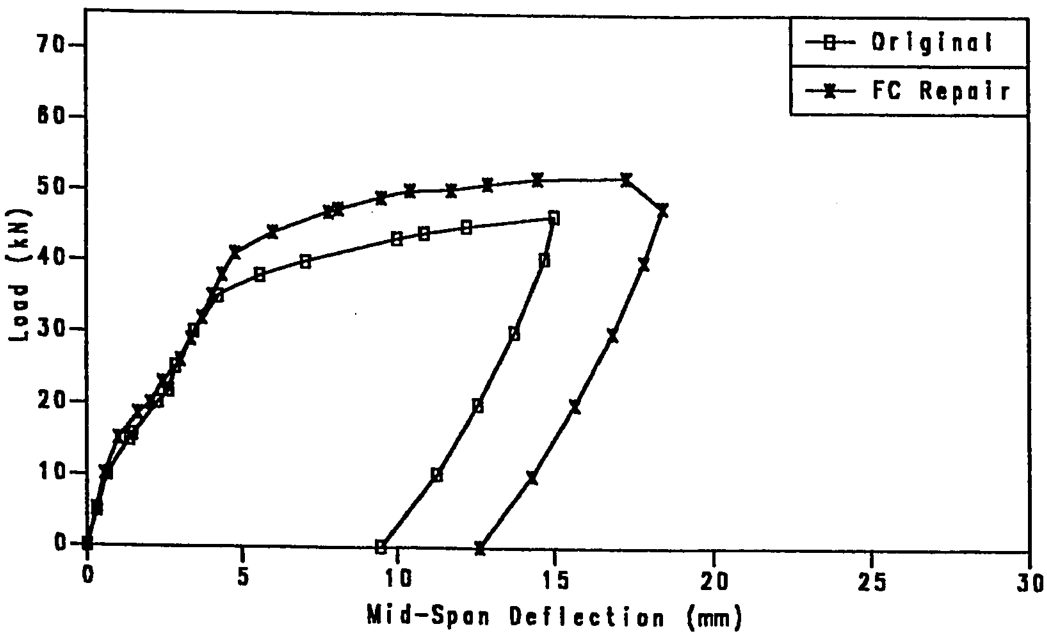

Fig. 4.34 Load Vs Deflection Curves for Beam B-ii-5

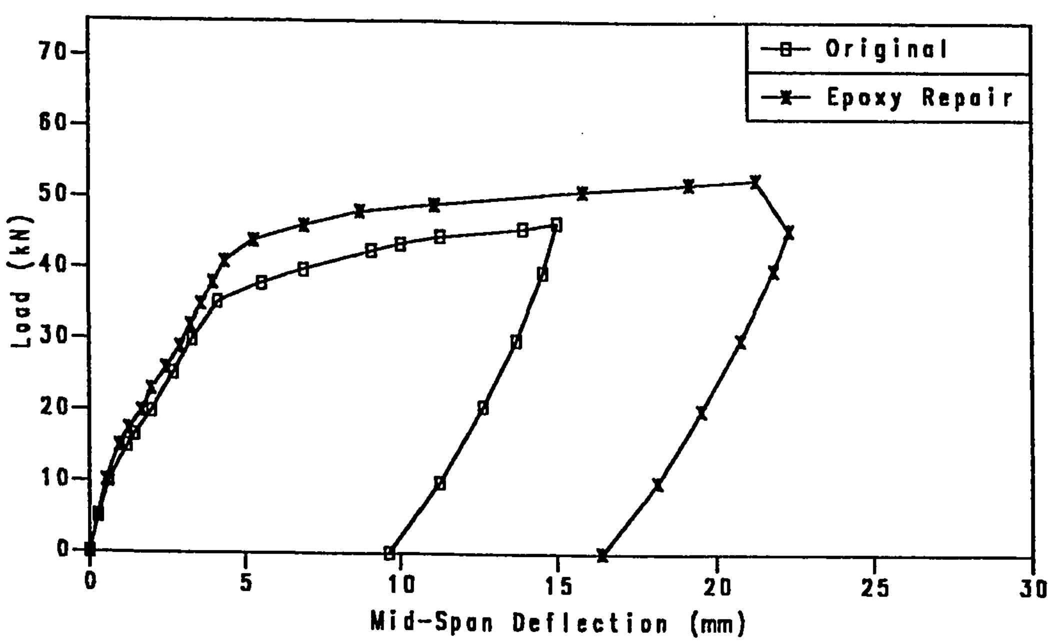

Fig. 4.35 Load Vs Deflection Curves for Beam B-ii-6

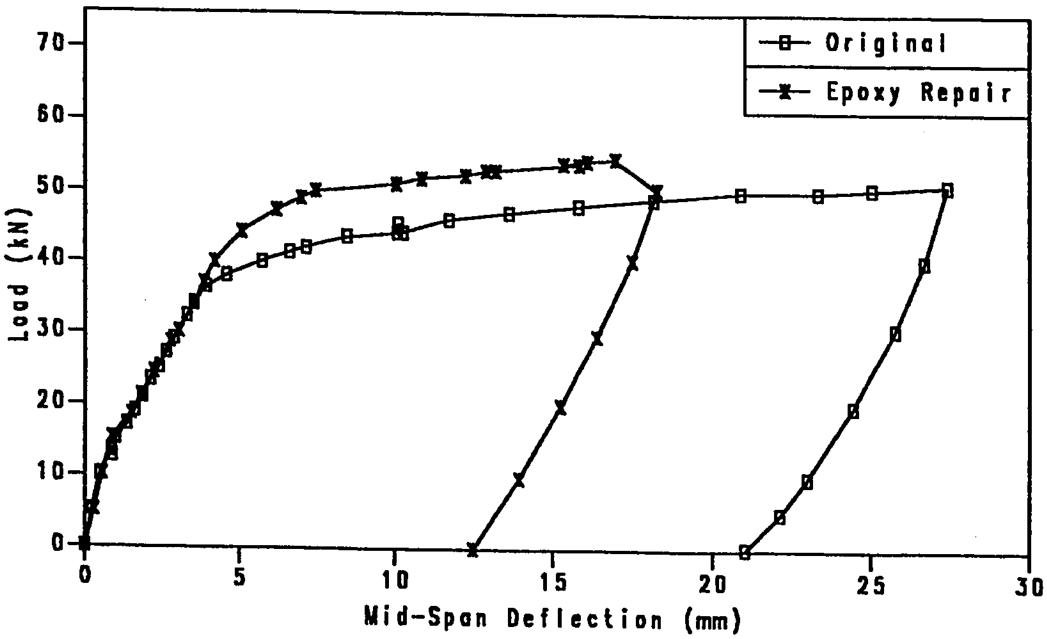

Fig. 4.36 Load Vs Deflection Curves for Beam C-ii-4

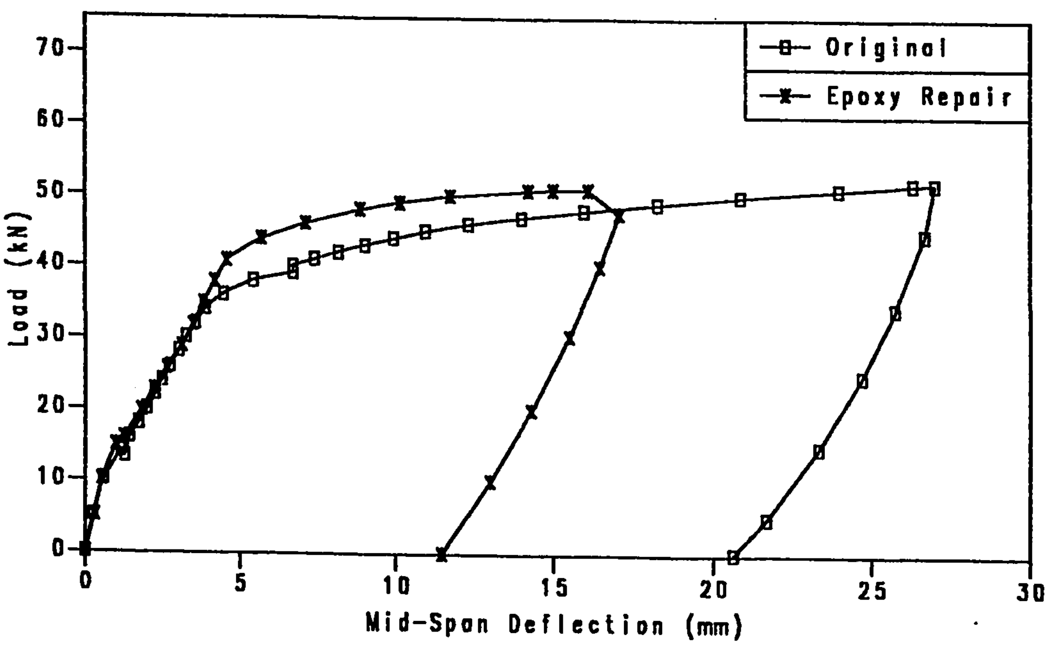

Fig. 4.37 Load Vs Deflection Curves for Beam C-ii-5

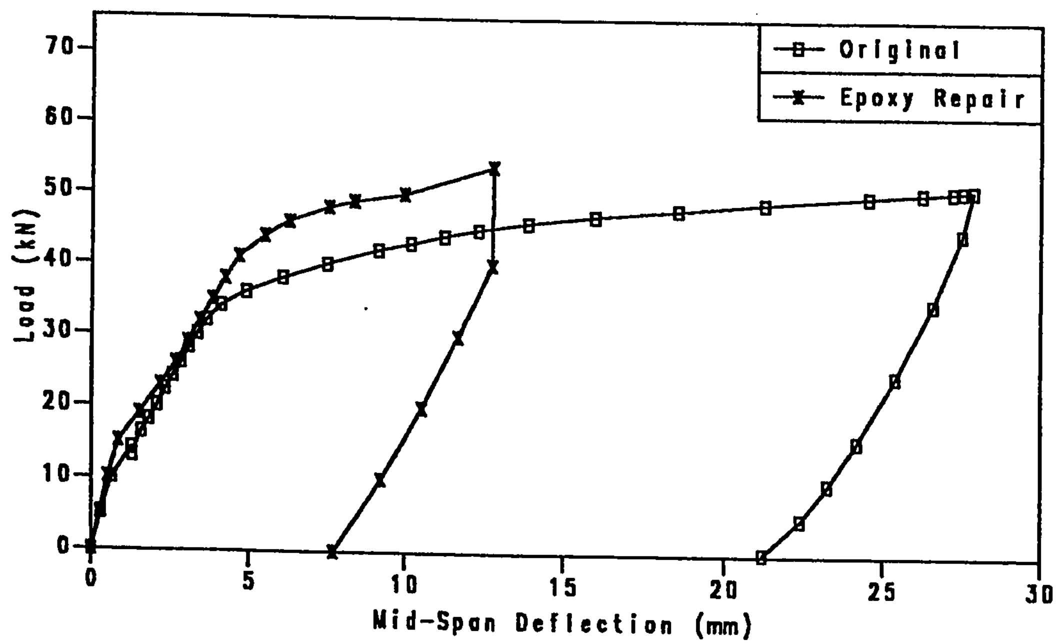

Fig. 4.38 Load Vs Deflection Curves for Beam C-ii-6

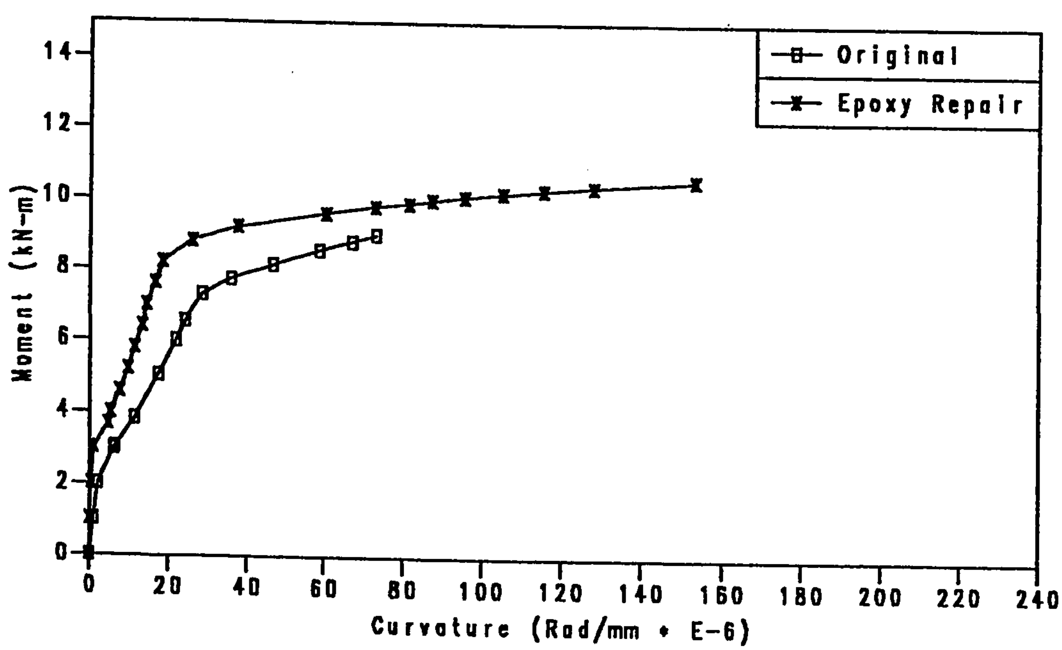

Fig. 4.39 Moment Vs Curvature Curves for Beam A-ii-4

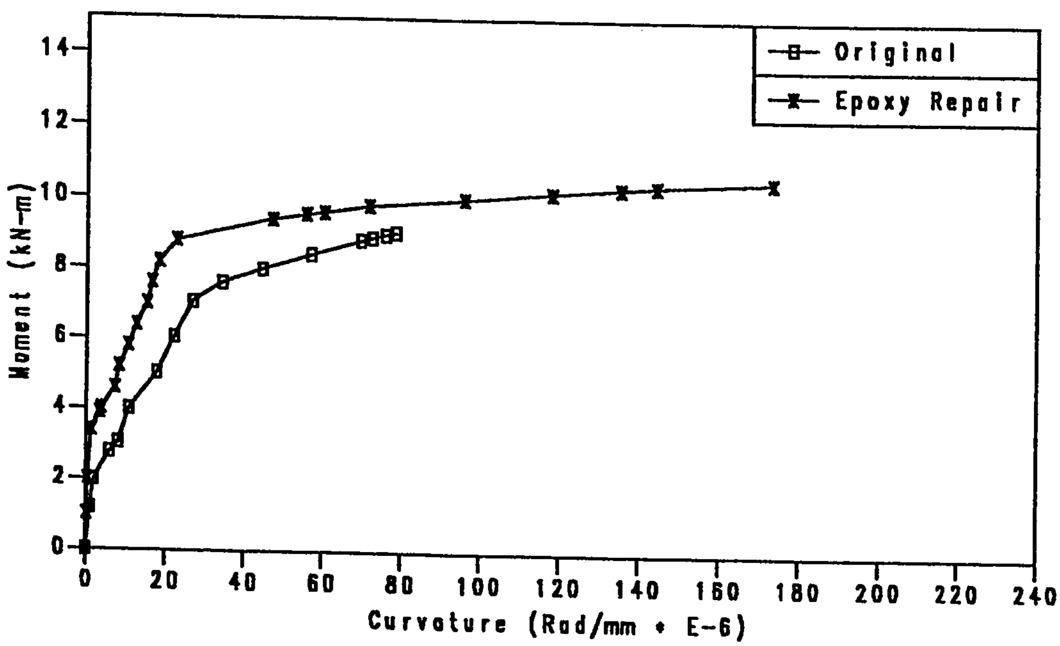

Fig. 4.40 Moment Vs Curvature Curves for Beam A-ii-5

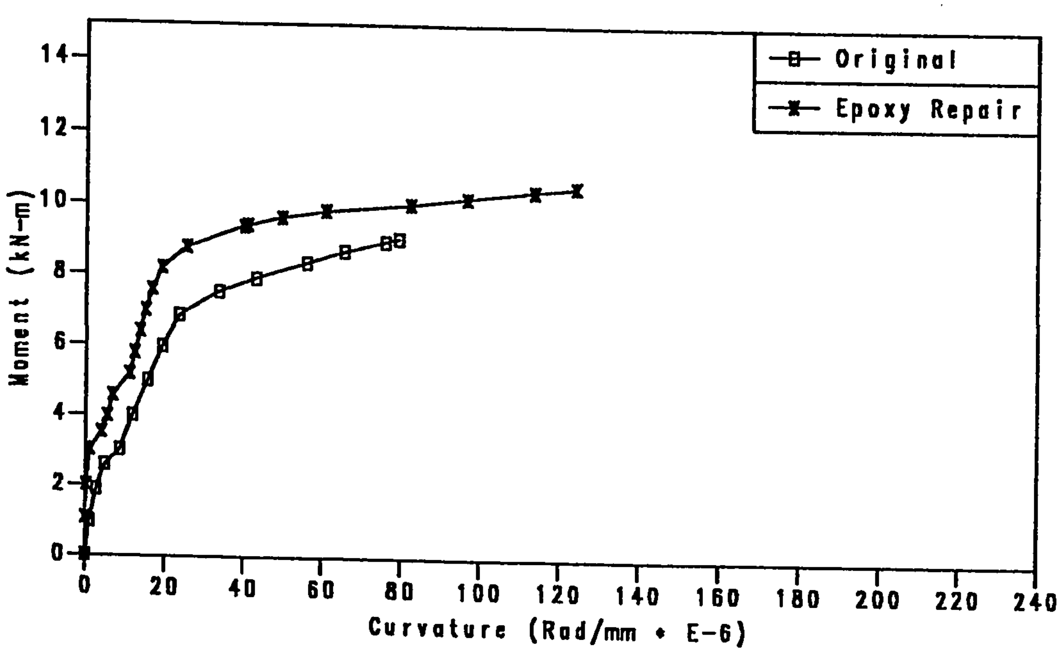

Fig. 4.41 Moment Vs Curvature Curves for Beam A-ii-6

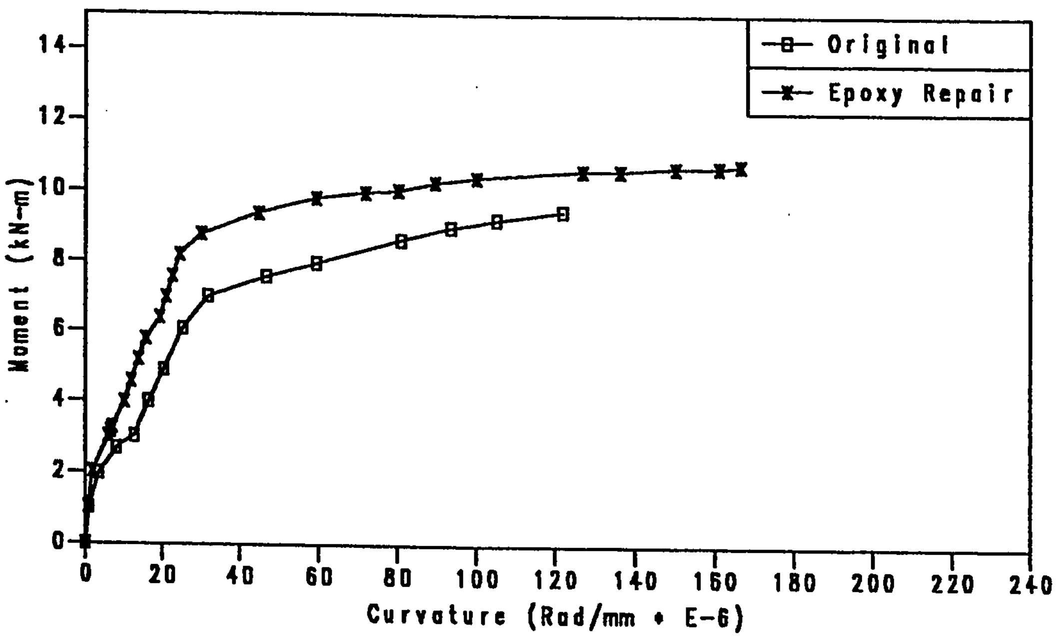

Fig. 4.42 Moment Vs Curvature Curves for Beam B-ii-4

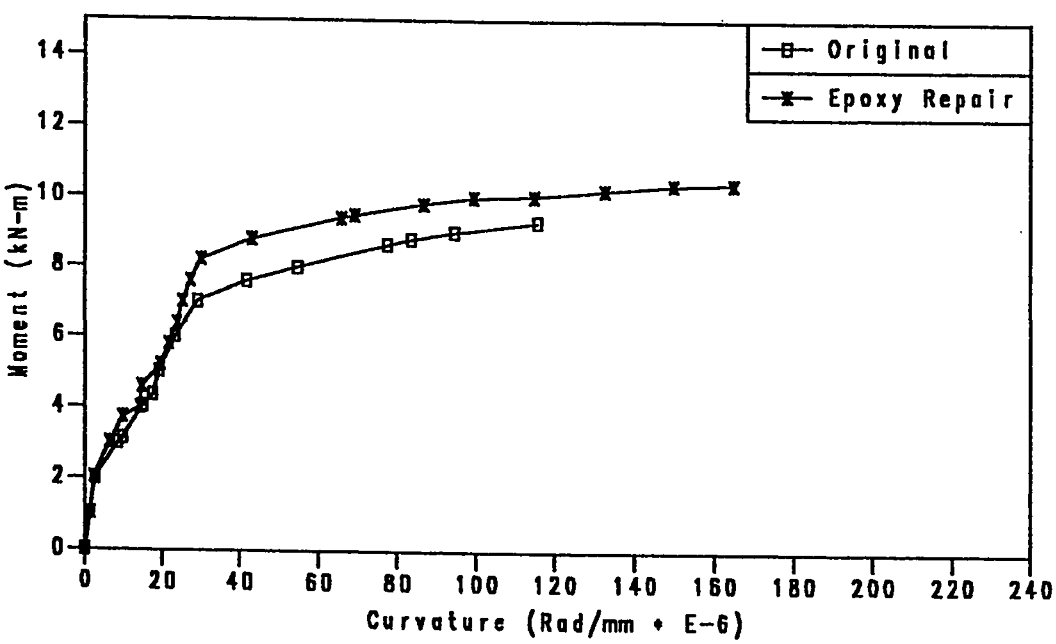

Fig. 4.43 Moment Vs Curvature Curves for Beam B-ii-5

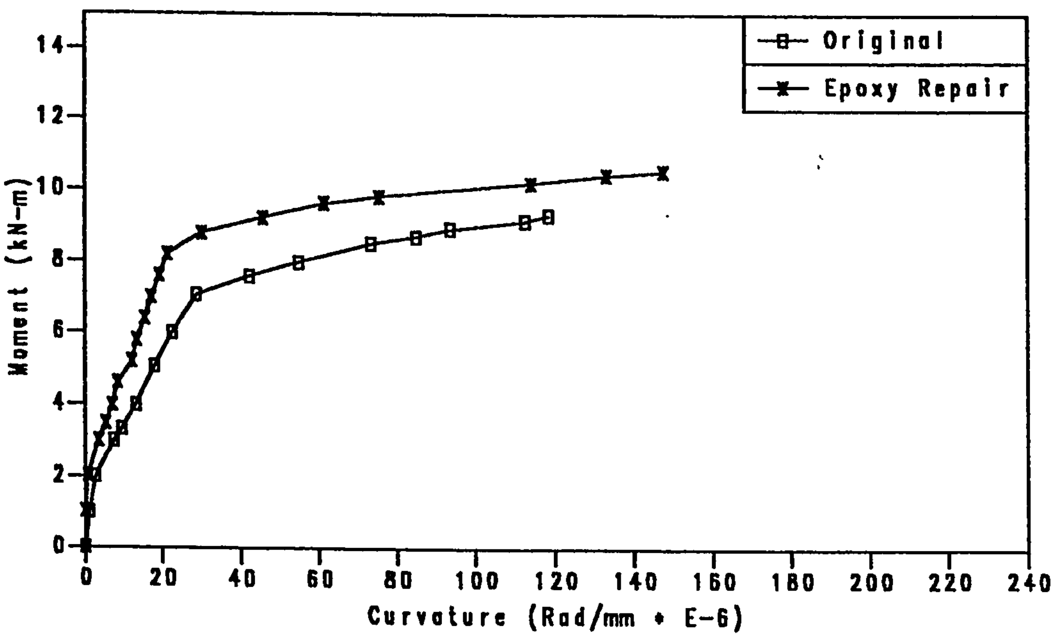

Fig. 4.44 Moment Vs Curvature Curves for Beam B-ii-6

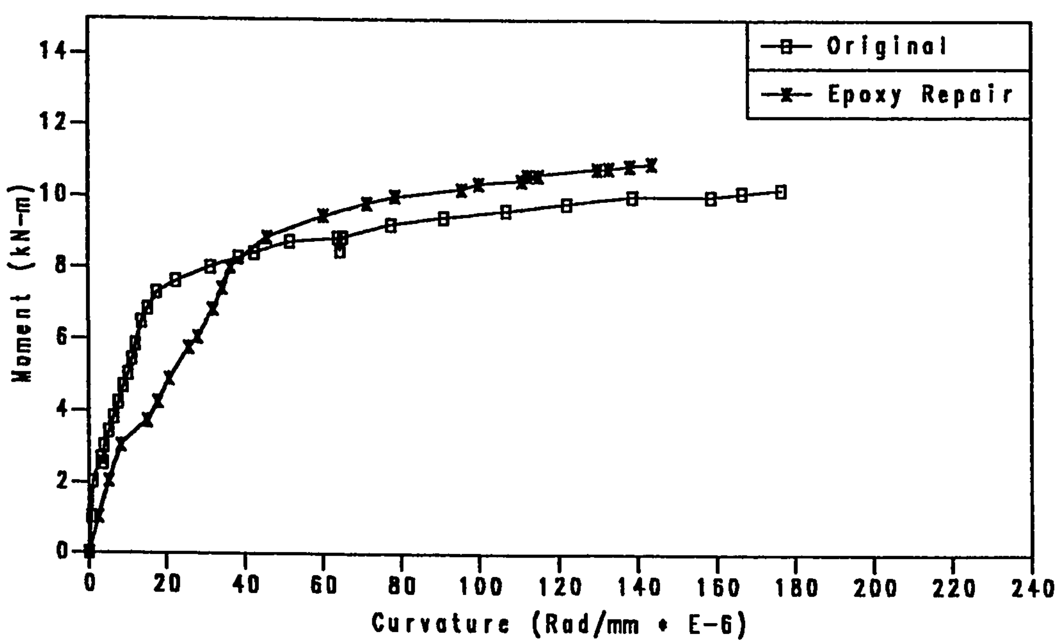

Fig. 4.45 Moment Vs Curvature Curves for Beam C-ii-4

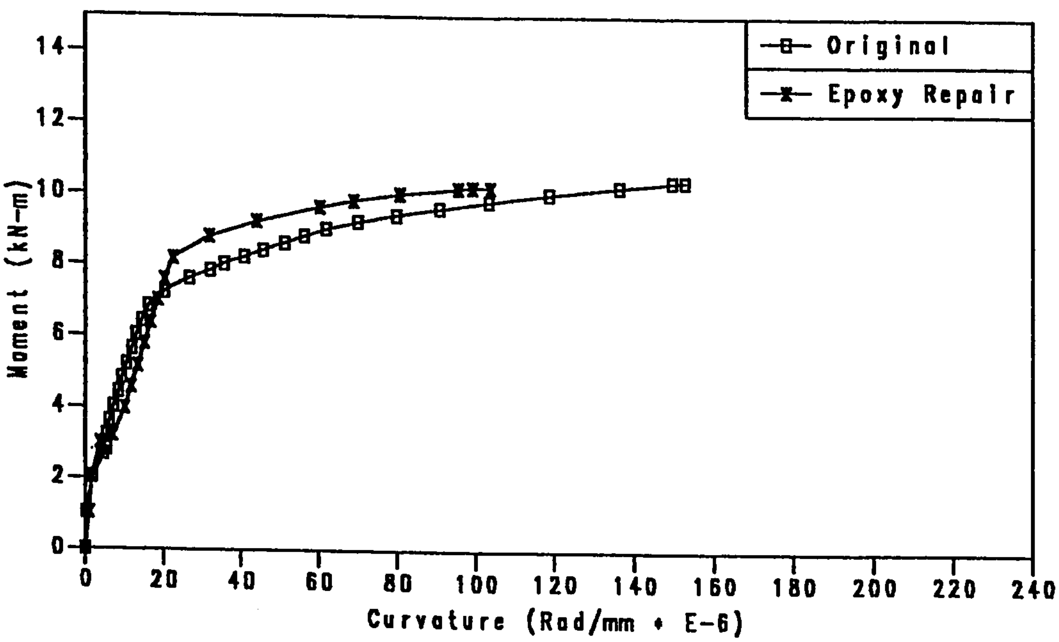

Fig. 4.46 Moment Vs Curvature Curves for Beam C-ii-5

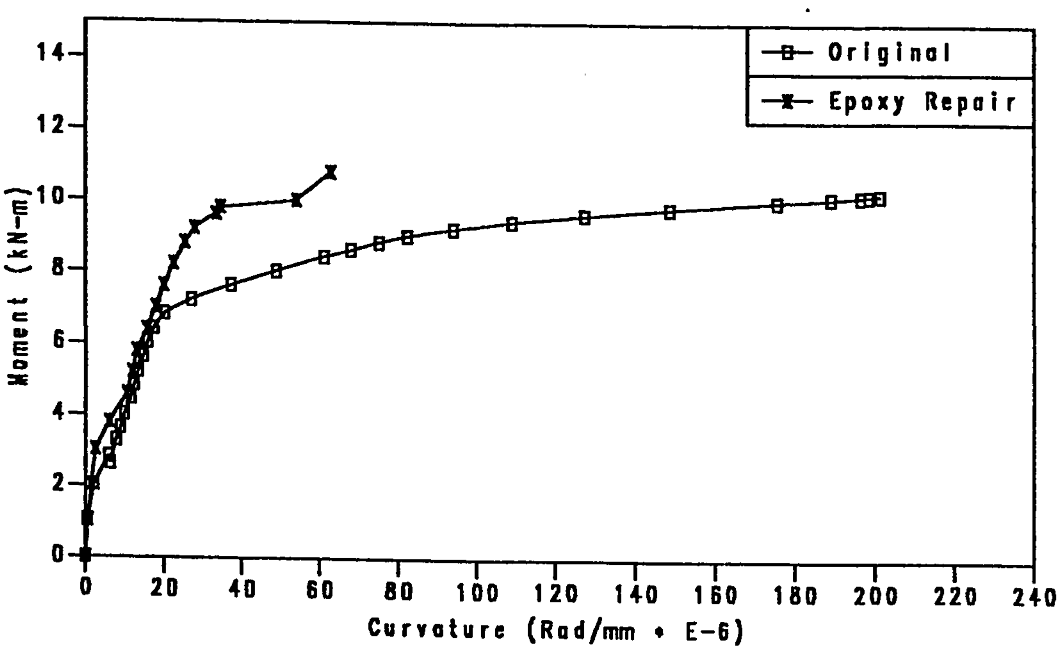

Fig. 4.47 Moment Vs Curvature Curves for Beam C-ii-6

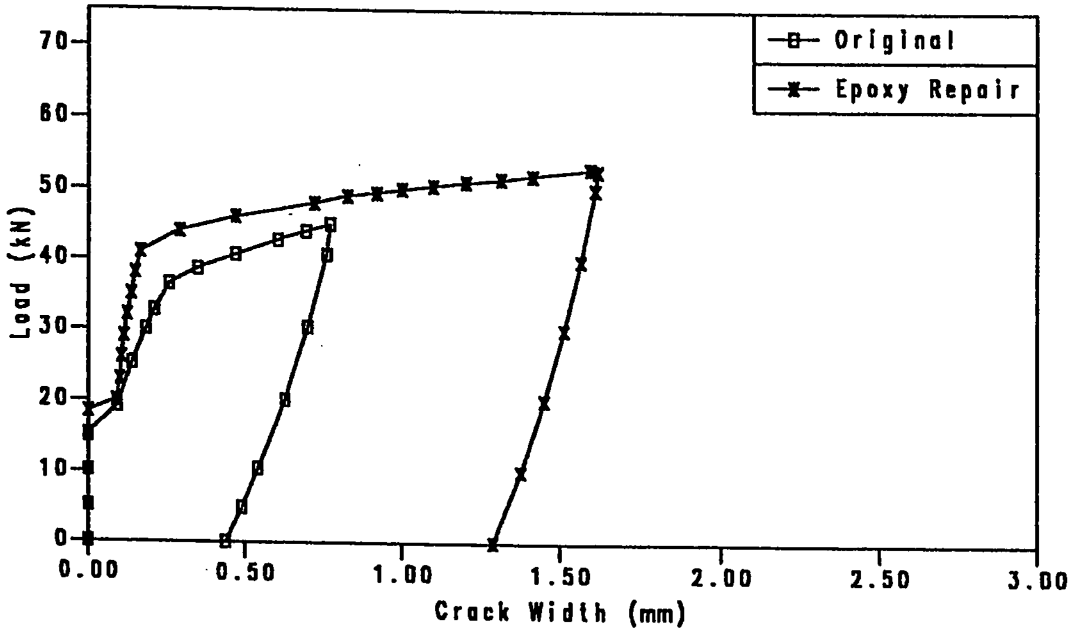

Fig. 4.48 Load Vs Crack Width Curves for Beam A-ii-4

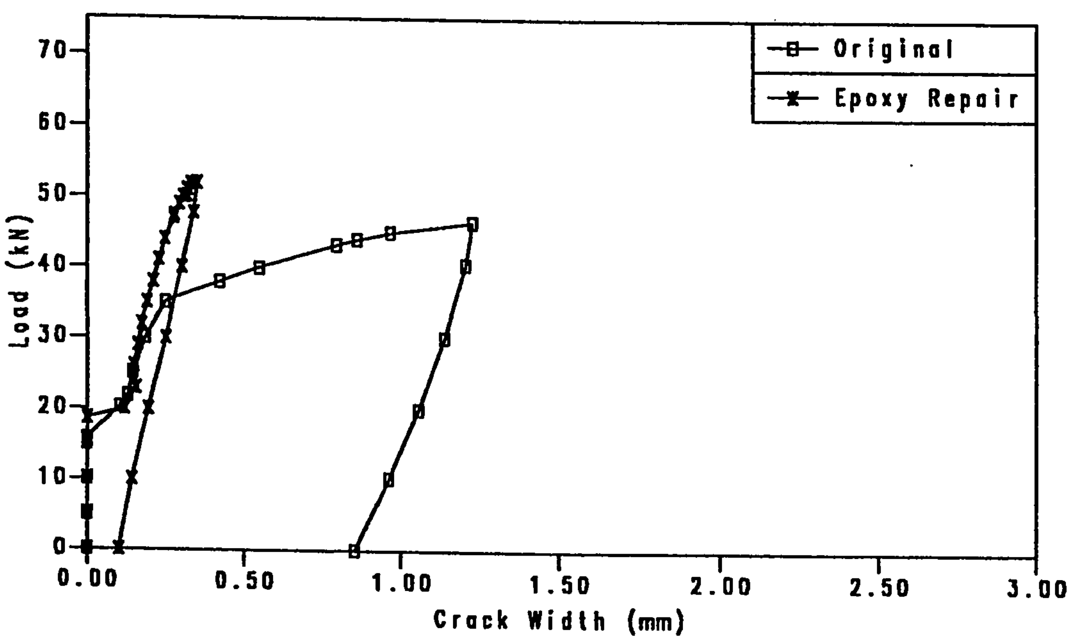

Fig. 4.49 Load Vs Crack Width Curves for Beam A-ii-5

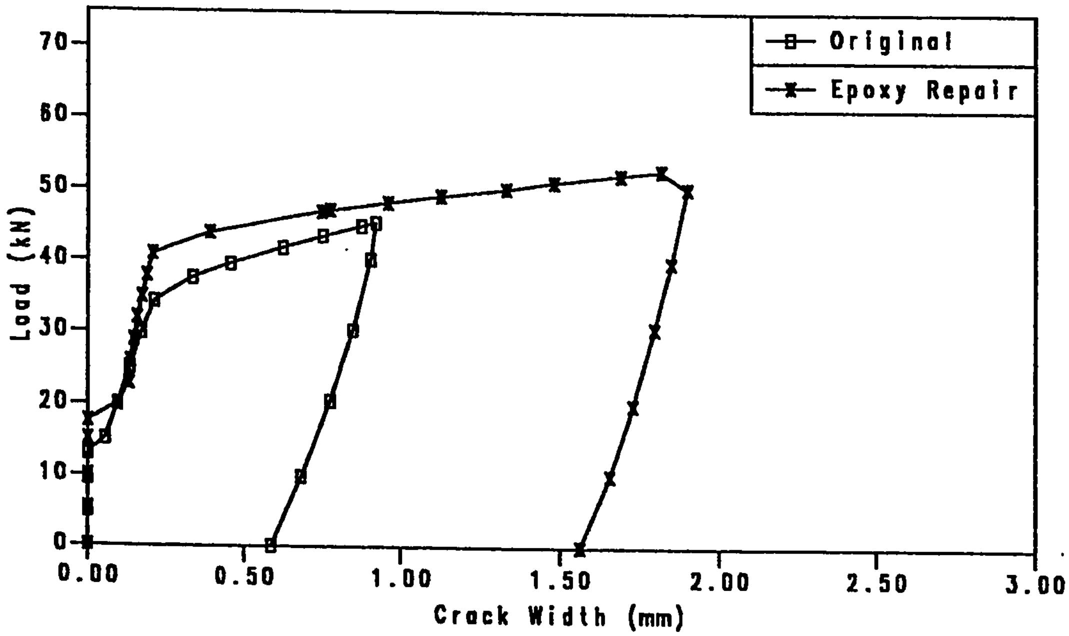

Fig. 4.50 Load Vs Crack Width Curves for Beam A-ii-6

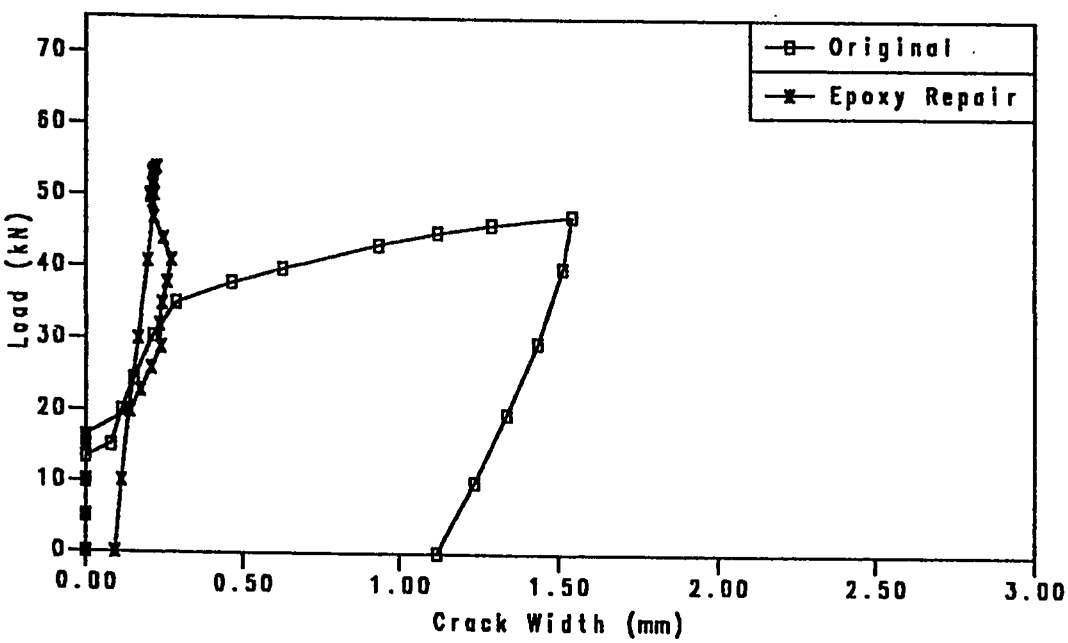

Fig. 4.51 Load Vs Crack Width Curves for Beam B-ii-4

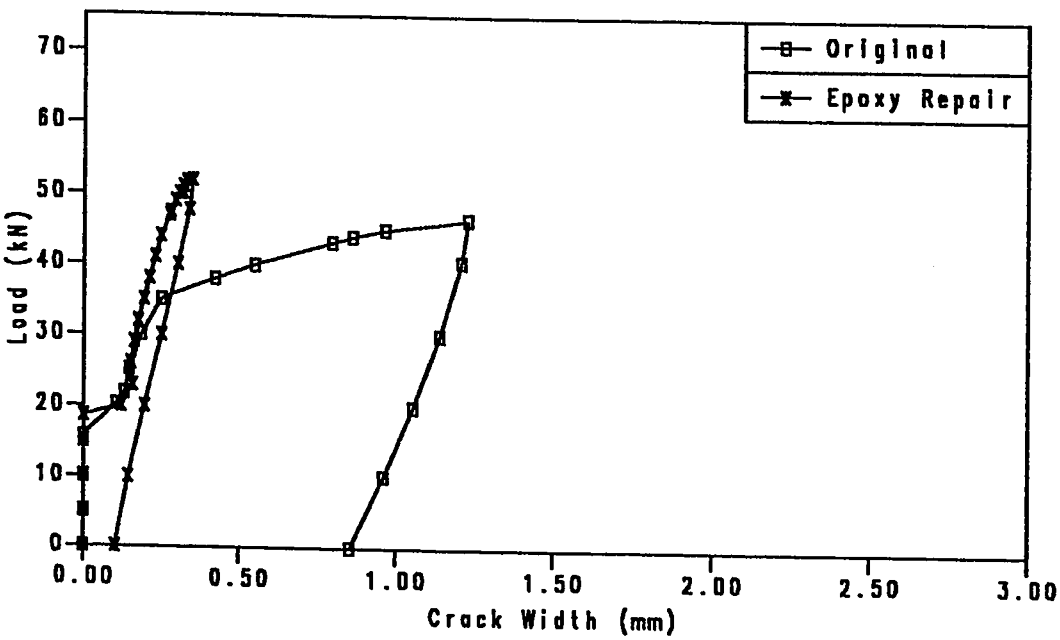

Fig. 4.52 Load Vs Crack Width Curves for Beam B-ii-5

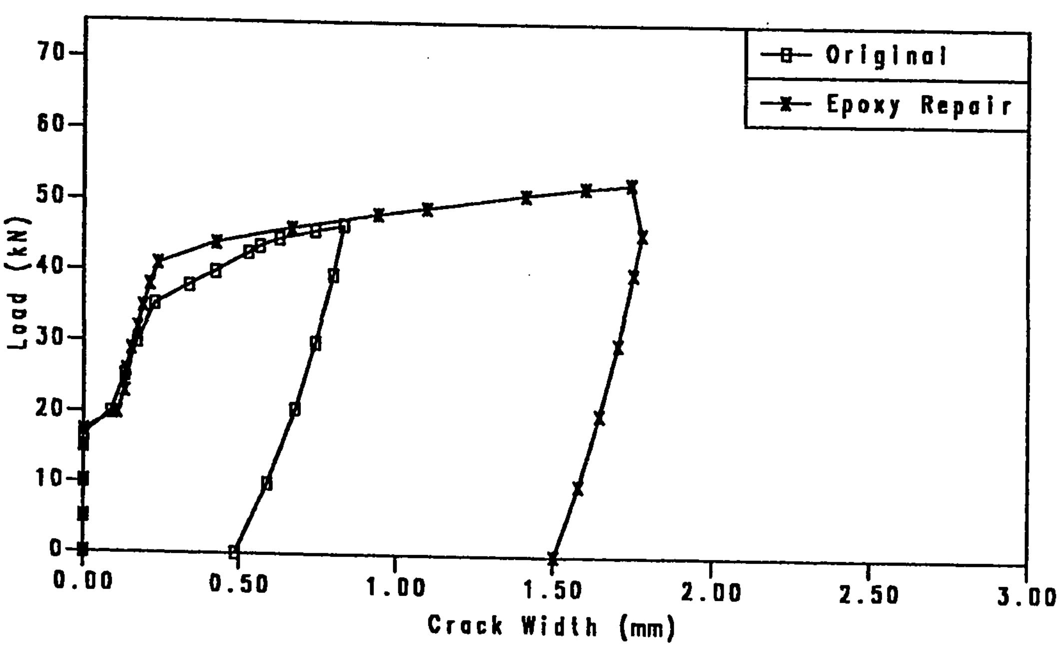

Fig. 4.53 Load Vs Crack Width Curves for Beam B-ii-6

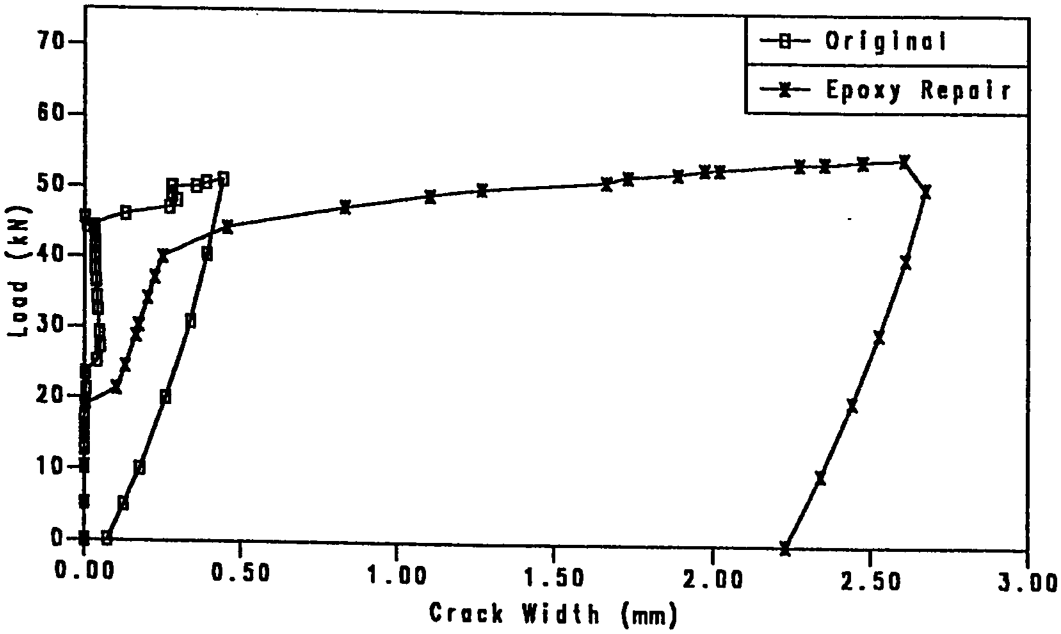

Fig. 4.54 Load Vs Crack Width Curves for Beam C-ii-4

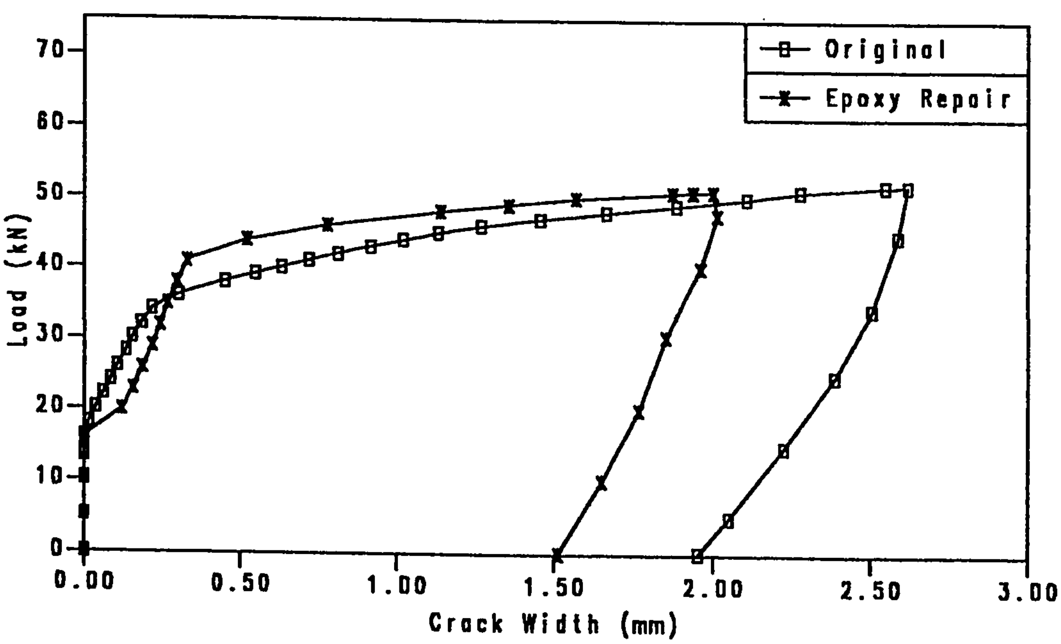

Fig. 4.55 Load Vs Crack Width Curves for Beam C-ii-5

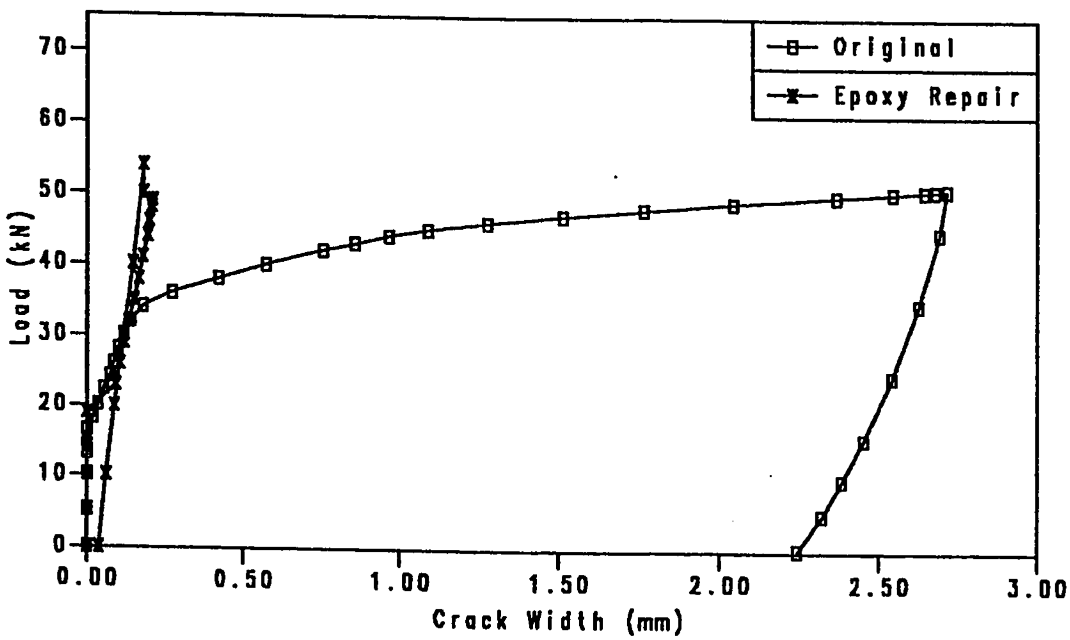

Fig. 4.56 Load Vs Crack Width Curves for Beam C-ii-6

beams loaded to ultimate have exhibited reduction in the ultimate deflection after repair in comparison to beams repaired after has been subjected to 10 mm deflection. This is due to the fact that beams which have been loaded to ultimate had more cracks which were too fine to be injected in the mean time have reduced the stiffness of the beams.

Moment vs curvature

From Figs. 4.39 through 4.47, which represent the moment vs curvature curves, it can be concluded that all beams tested up to 10 and 15 mm deflections showed no increase in the moment capacity inspite the increase in the rigidity of the beams. In the case of beams tested up to ultimate deflection, two beams showed decrease in the rigidity of the beams when repaired by epoxy, while one beam showed a very small increase in its rigidity after repair. Hence the moment is a function of the rigidity (EI) which is the property of the cross section, there will be no increase expected.

Load vs crack width

New cracks were developed when the repaired beams were tested up to ultimate deflection as shown very clearly in plate 4.4 . The new cracks occured at different locations than the original ones which indicates that the strength of

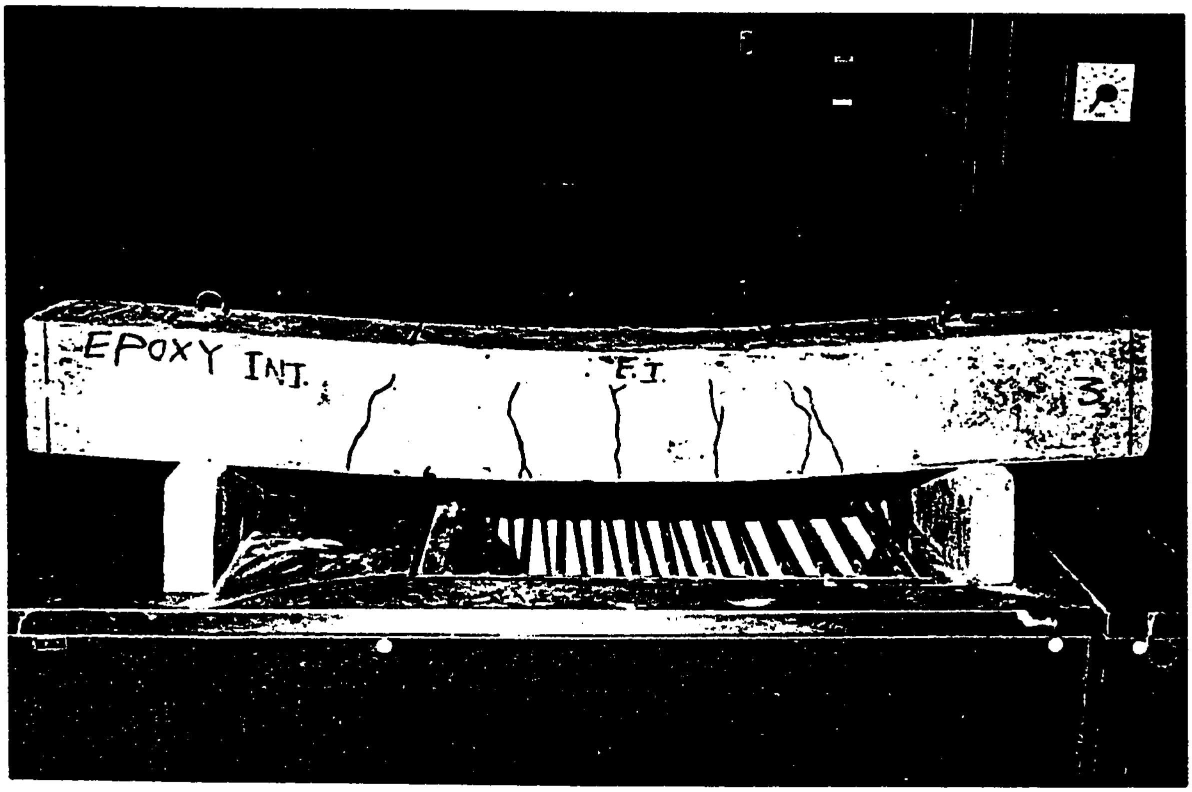

Plate 4.4 : Crack patterns of epoxy repair method.

the epoxy material is higher than that of the concrete used. The reason that cracks in the original beams which were less than 0.1 mm in width can not be filled with epoxy have opened again during the loading of the repaired beams. This has been visually observed during testing. From Figs. 4.48 through 4.56, the crack width does not show a significant decrease due to the opening of new cracks in the original concrete.

4.4 Steel plate bonding method

External reinforcement is sometimes needed to strengthen a structure. Bonding steel plate by epoxy material to a structure was used to strengthen the reinforced concrete beams in many bridges in the west. In this study, 9 beams were tested up to three different levels of deflections, each group consisting of 3 beams, as described in chapter 3 (testing program) . The steel plate bonding method was used to strengthen a reinforced concrete beams and compare their results with the unrepaired ones. Also, a study of the bond strength of the epoxy materials was investigated.

4.4.1 Presenting the results of steel plate bonding

The 9 beams strengthened by steel plates were tested up to ultimate deflection. The collected data was prepared according to the description of section 4.1 for each beam. Only 2 curves were plotted, load vs deflection and moment vs curvature for both the original and the strengthened beams. Crack width of the beams was not recorded, because cracks were still there after strengthening the beams and therefore no load vs crack width curves were plotted.

The thickness of the epoxy layer between the steel plate and the concrete beam was in the range of 1.5 to 2.0 mm.

Load vs deflection

Figs. 4.57 through 4.59 represent the load vs deflection curves of the beams tested up to 10 mm mid-span deflection and strengthened by steel plates. Figs. 4.60 through 4.62 represent the load vs deflection curves of the beams tested up to 15 mm mid-span deflection and strengthened by steel plates. Figs. 4.63 through 4.65 represent the load vs deflection curves of the beams tested up to ultimate mid-span deflection and strengthened by steel plates.

The curves were separated from each other to find the initial stiffness, yielding and tested loads and ultimate deflection were obtained from the curves. Also, the ductility represented by the area under the P-Δ curves was obtained for the ultimate group and all repaired beams. All data obtained from the curves was summarized in table 4.4 for both the original and strengthened beams.

Table 4.4 : Comparison of Original and Steel Plate Bonded Beams

Item		P_{cr} (kN)		K_i (kN/mm)		K_r (kN/mm)		P_y (kN)		P_t (kN)		$\Delta_{ult.}$ (mm)		Ductility (kN-mm)	
Δ_c	Beam#	Unrep	Rep.	Unrep	Rep.	Unrep	Rep.	Unrep	Rep.	Unrep	Rep.	Unrep	Rep.	Unrep	Rep.
10mm	A-7	14.35	-	15.76	16.26	8.27	-	33.40	60.00	43.86	63.97	-	4.92	-	149.66
	A-8	12.73	-	15.56	19.40	7.51	-	35.00	63.25	43.51	72.03	-	4.42	-	143.71
	A-9	12.35	-	15.44	20.08	7.18	-	34.75	57.00	43.08	60.03	-	3.04	-	68.03
Ave.		13.14	-	15.59	18.58	7.66	-	34.38	60.08	43.48	65.34	-	4.13	-	120.47
15mm	B-7	16.10	-	15.53	17.93	6.98	-	34.95	64.00	45.55	63.47	-	4.61	-	93.54
	B-8	14.41	-	15.81	17.92	7.28	-	34.00	63.00	45.30	64.90	-	3.54	-	96.94
	B-9	14.86	-	15.35	17.04	8.21	-	35.00	60.00	45.81	67.96	-	3.97	-	159.86
Ave.		15.12	-	15.57	17.63	7.49	-	34.65	62.33	45.55	65.44	-	4.04	-	116.78
Ult.	C-7	14.99	-	12.17	15.91	8.06	-	35.00	54.10	50.50	55.13	26.20	3.91	959.18	78.95
	C-8	13.25	-	17.86	15.40	8.22	-	35.00	44.80	51.06	45.03	27.08	2.92	1023.81	48.87
	C-9	13.05	-	18.42	16.50	7.86	-	35.00	60.00	50.75	62.98	27.13	4.02	1023.81	83.33
Ave.		13.76	-	16.15	15.94	8.05	-	35.00	52.97	50.77	59.06	26.80	3.62	1002.27	70.38

Moment vs curvature

The moment and curvature data of the beams were calculated from the collected data, during the tests of the beams as described in section 4.1.

Figs. 4.66 through 4.68 represent the moment vs curvature curves of the beams tested up to 10 mm mid-span deflection and strengthened by steel plates. Figs. 4.69 through 4.71 represent the moment vs curvature curves of the beams tested up to 15 mm mid-span deflection and strengthened by steel plates. Figs. 4.72 through 4.74 represent the moment vs curvature curves of the beams tested up to ultimate mid-span deflection and strengthened by steel plates.

Bond stress results

In order to assess the bonding strength of epoxy used to bond the steel plate to the concrete beams, three prisms of 90 mm square cross section and 150 mm depth with a 10 mm diameter bar at the center of the square were prepared with a contact area of 20 * 25 mm^2 of steel plates from two parallel sides of the prism using epoxy. The tension test was applied after 7 days of bonding the plates to the concrete using epoxy. The tension loads obtained from the test were 8, 8.1 and 7.23 kN with an average of 7.777 kN. Plate 4.5 shows the bond stress test result.

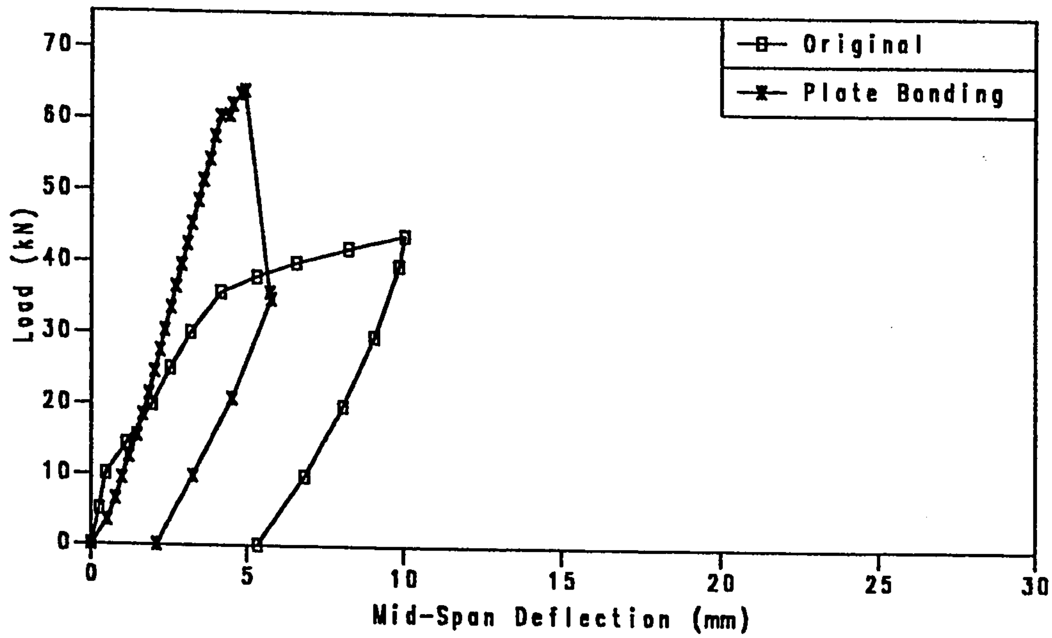

Fig. 4.57 Load Vs Deflection Curves for Beam A-iii-7

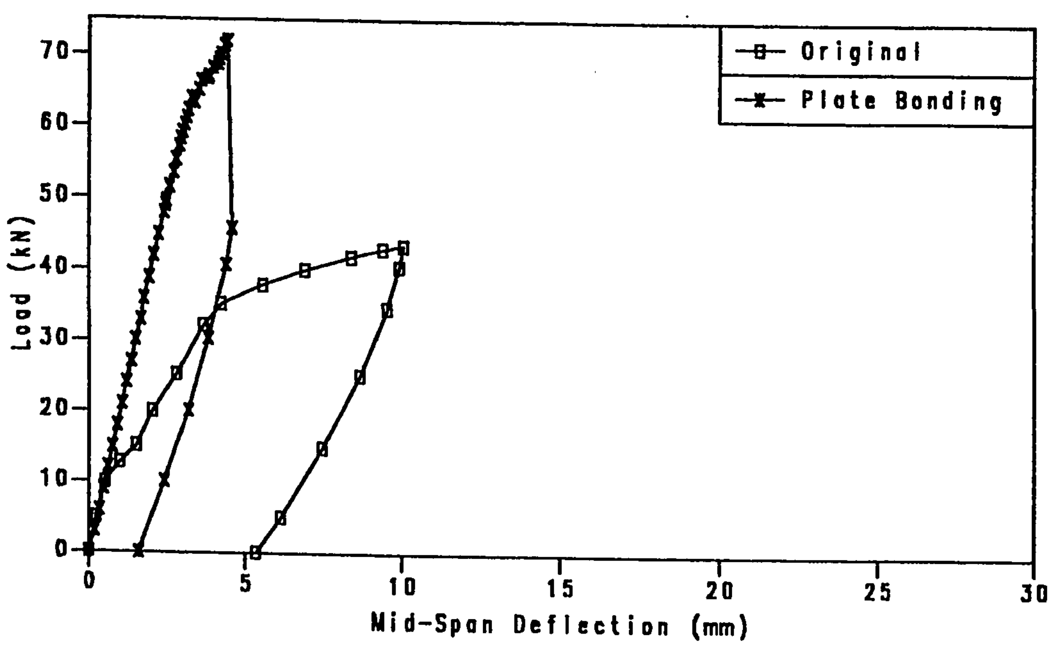

Fig. 4.58 Load Vs Deflection Curves for Beam A-iii-8

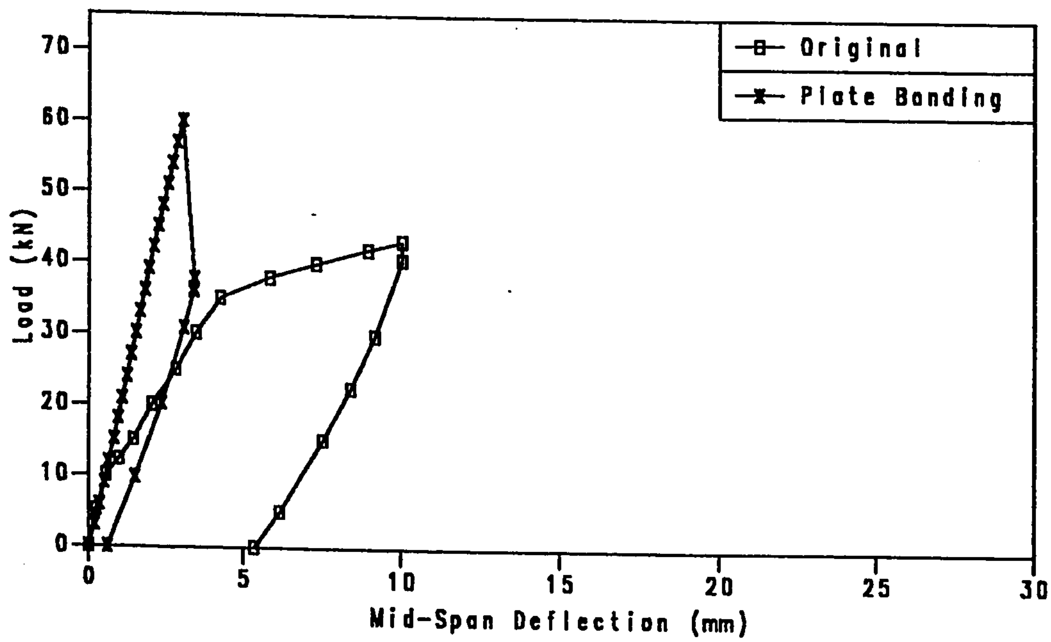

Fig. 4.59 Load Vs Deflection Curves for Beam A-iii-9

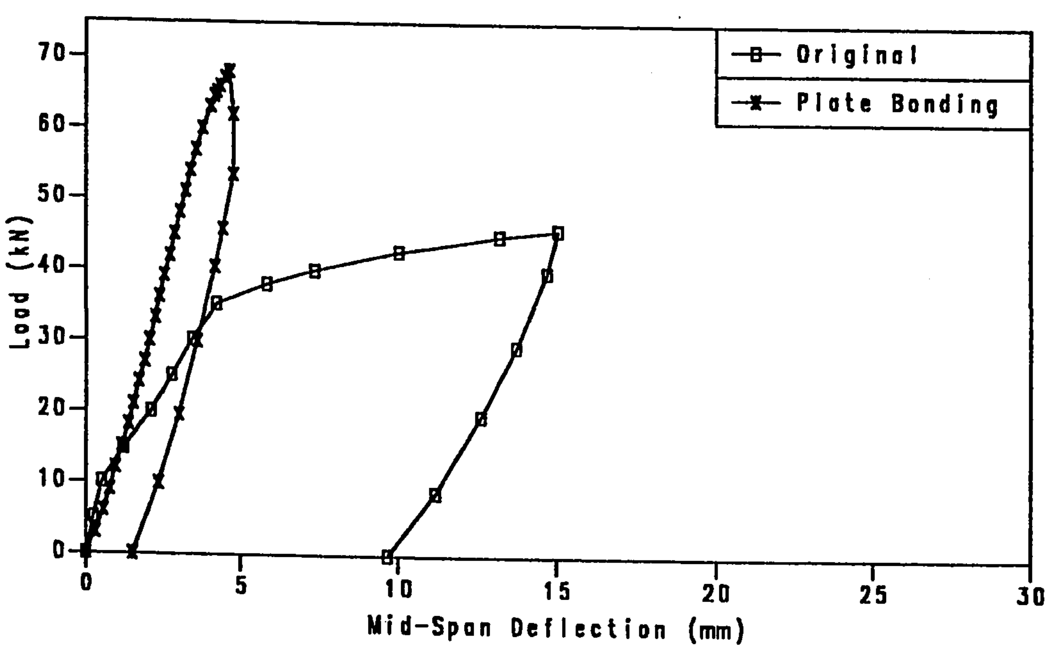

Fig. 4.60 Load Vs Deflection Curves for Beam B-iii-7

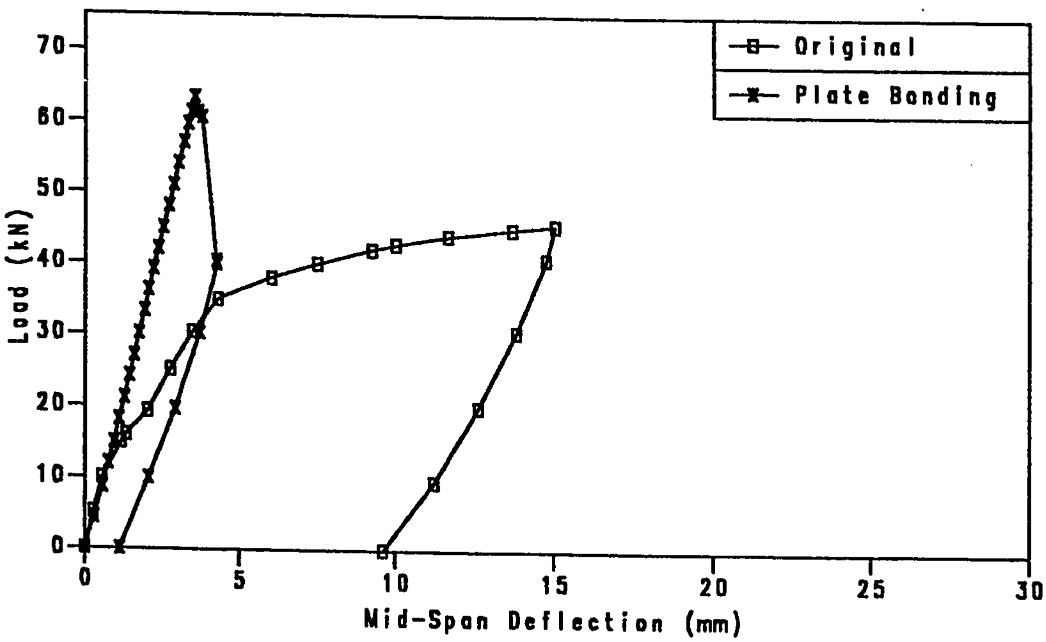

Fig. 4.61 Load Vs Deflection Curves for Beam B-iii-8

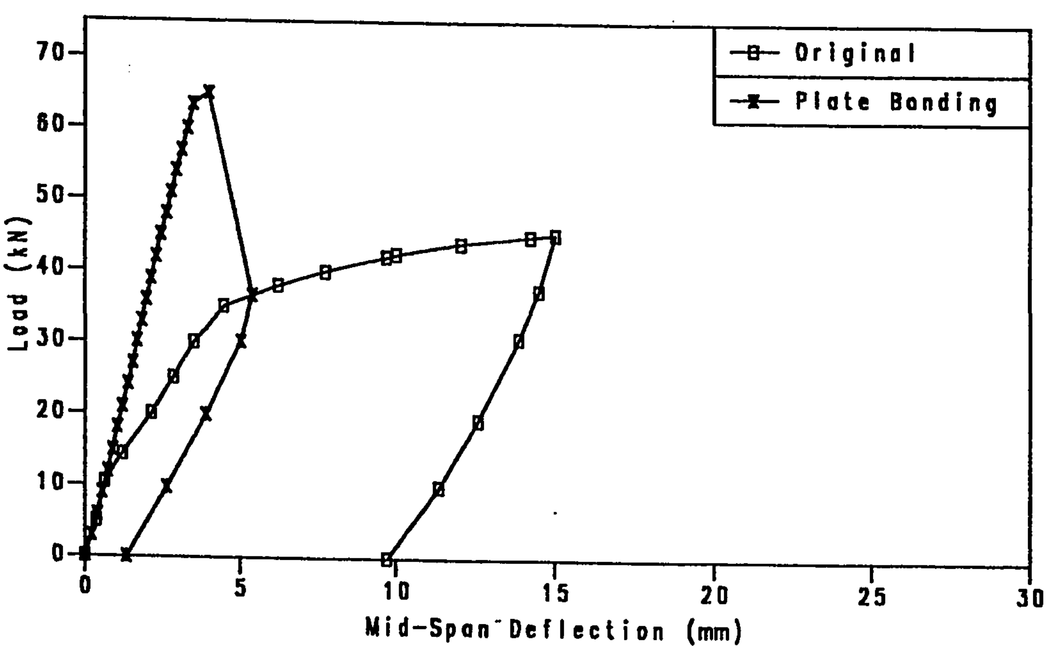

Fig. 4.62 Load Vs Deflection Curves for Beam B-iii-9

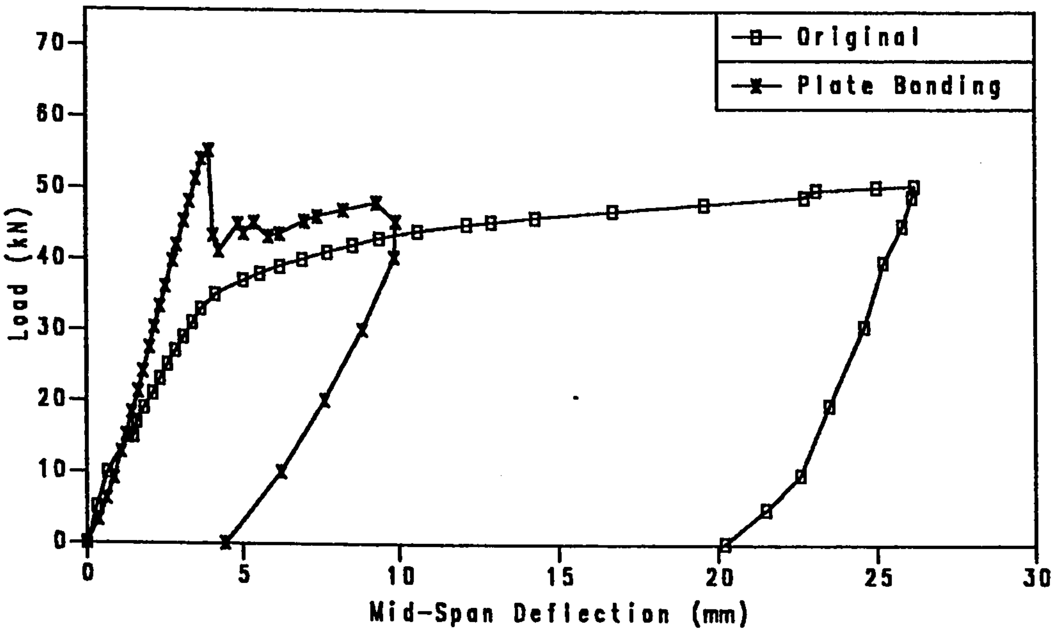

Fig. 4.63 Load Vs Deflection Curves for Beam C-iii-7

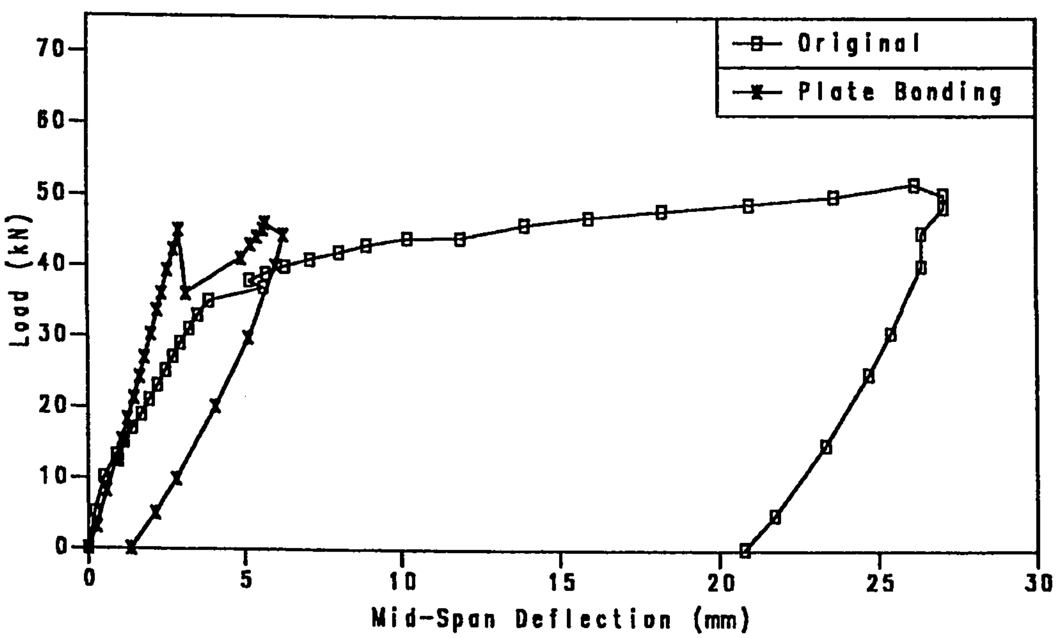

Fig. 4.64 Load Vs Deflection Curves for Beam C-iii-8

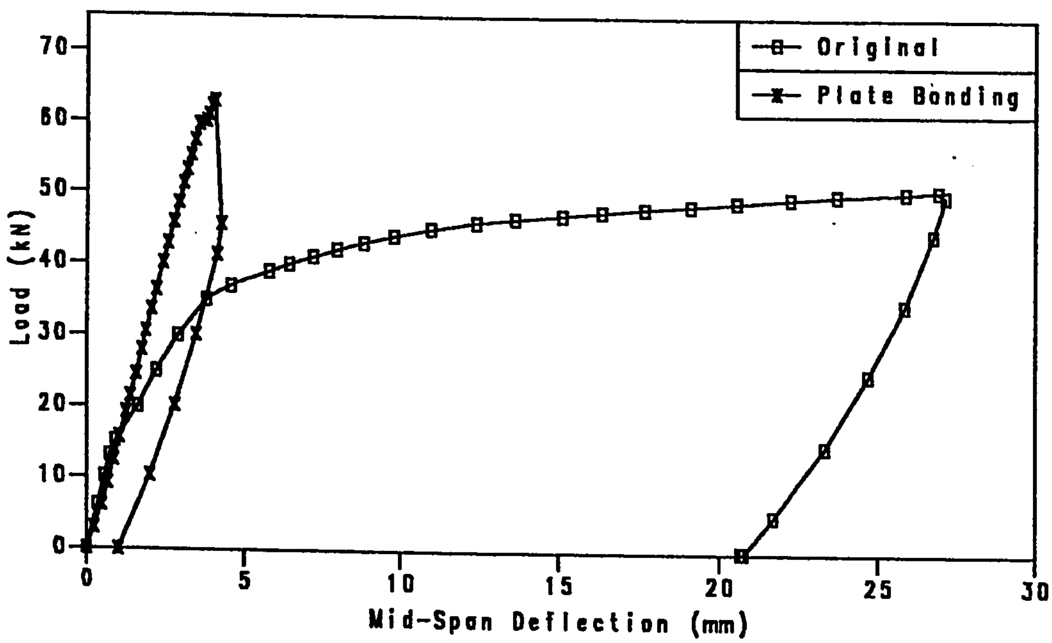

Fig. 4.65 Load Vs Deflection Curves for Beam C-iii-9

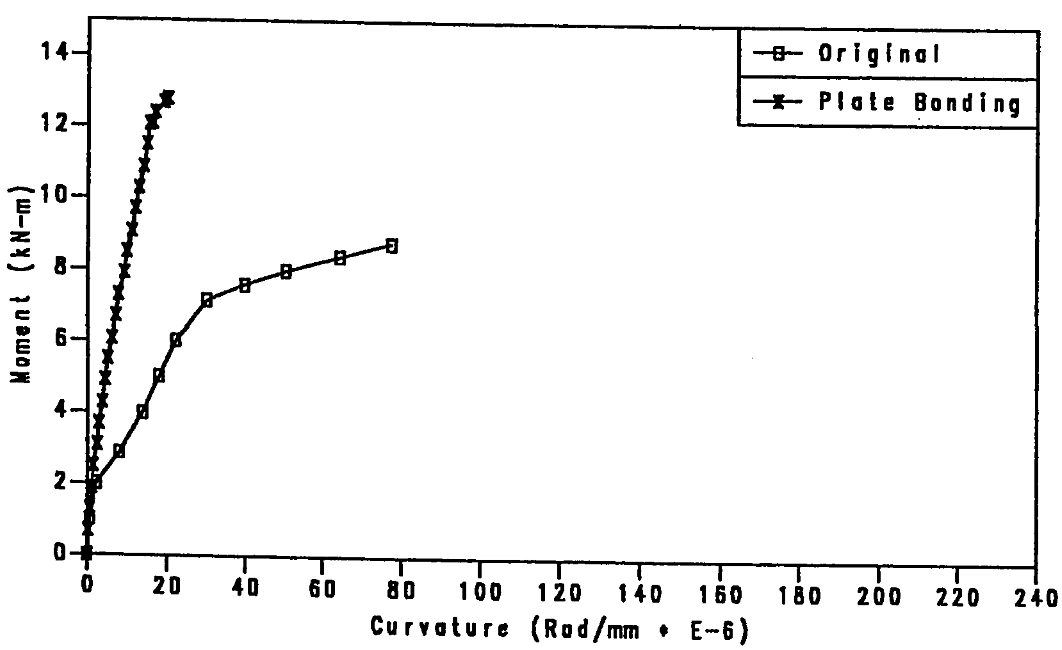

Fig. 4.66 Moment Vs Curvature Curves for Beam A-iii-7

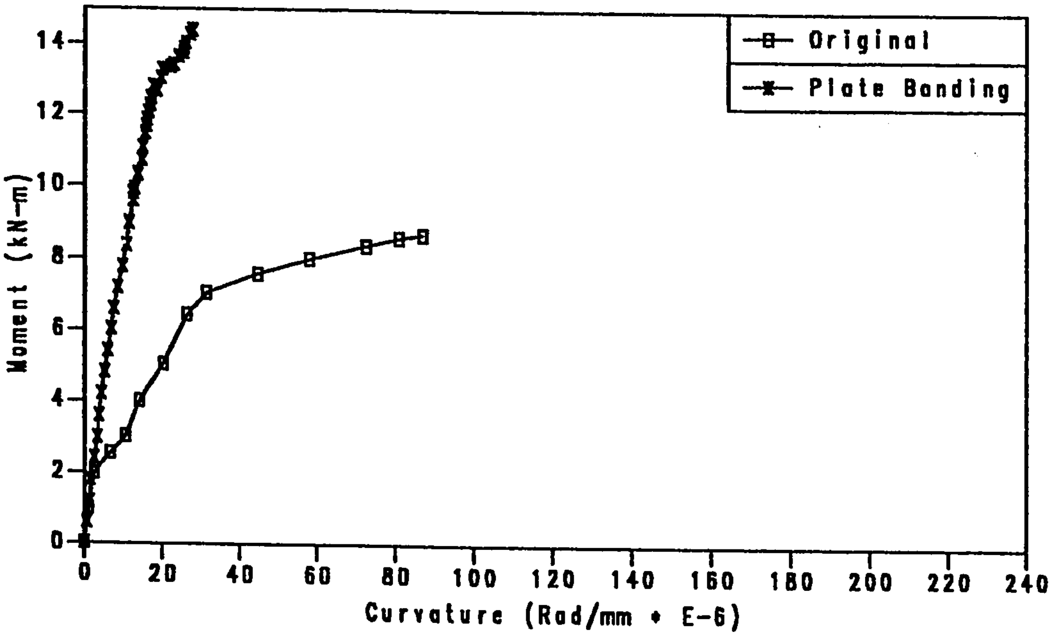

Fig. 4.67 Moment Vs Curvature Curves for Beam A-iii-8

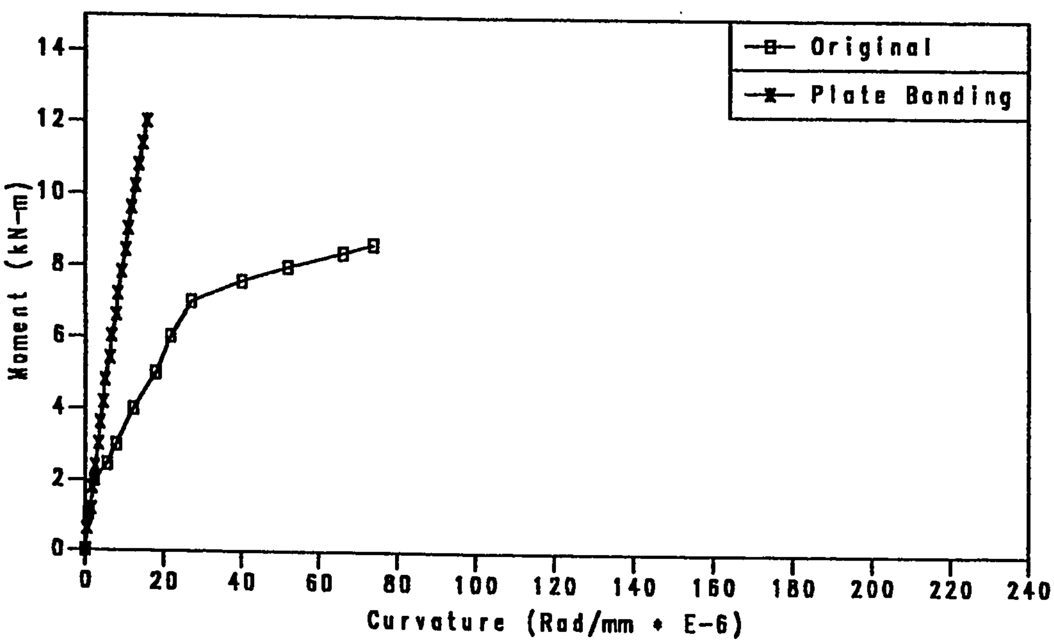

Fig. 4.68 Moment Vs Curvature Curves for Beam A-iii-9

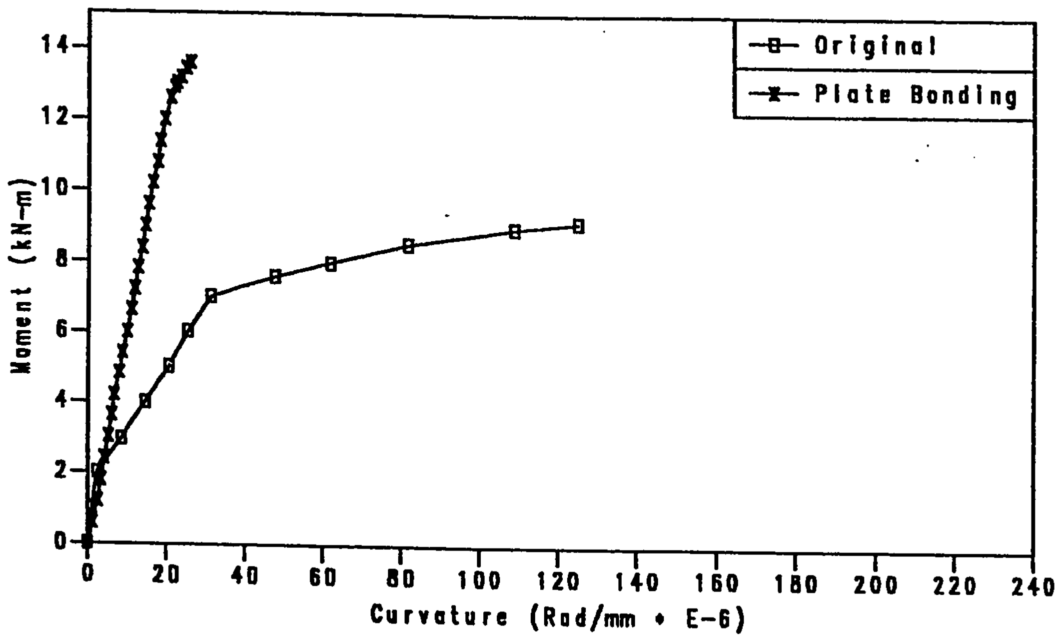

Fig. 4.69 Moment Vs Curvature Curves for Beam B-iii-7

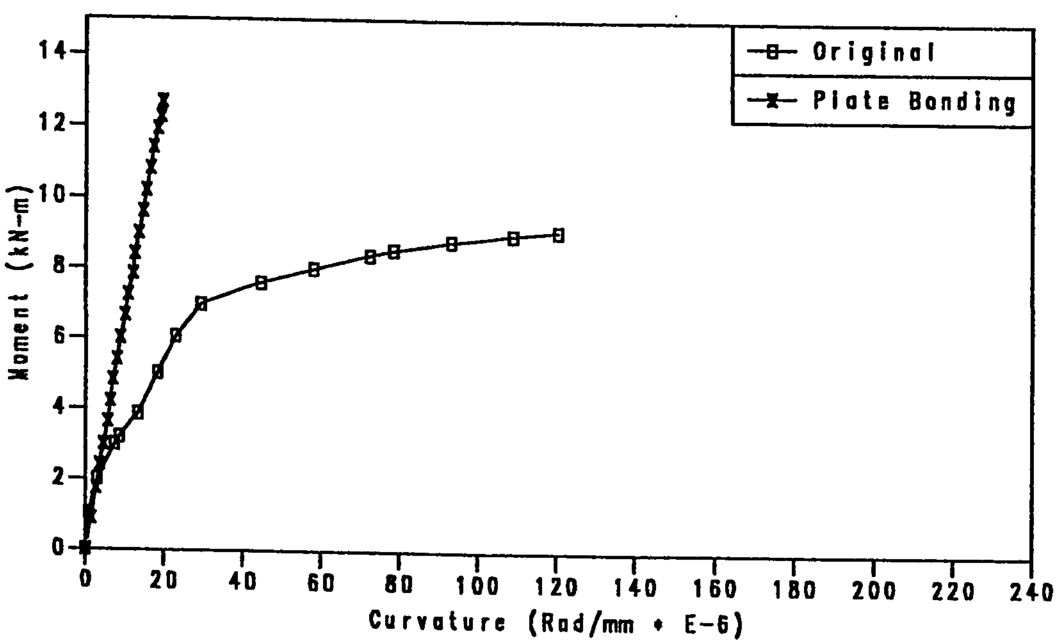

Fig. 4.70 Moment Vs Curvature Curves for Beam B-iii-8

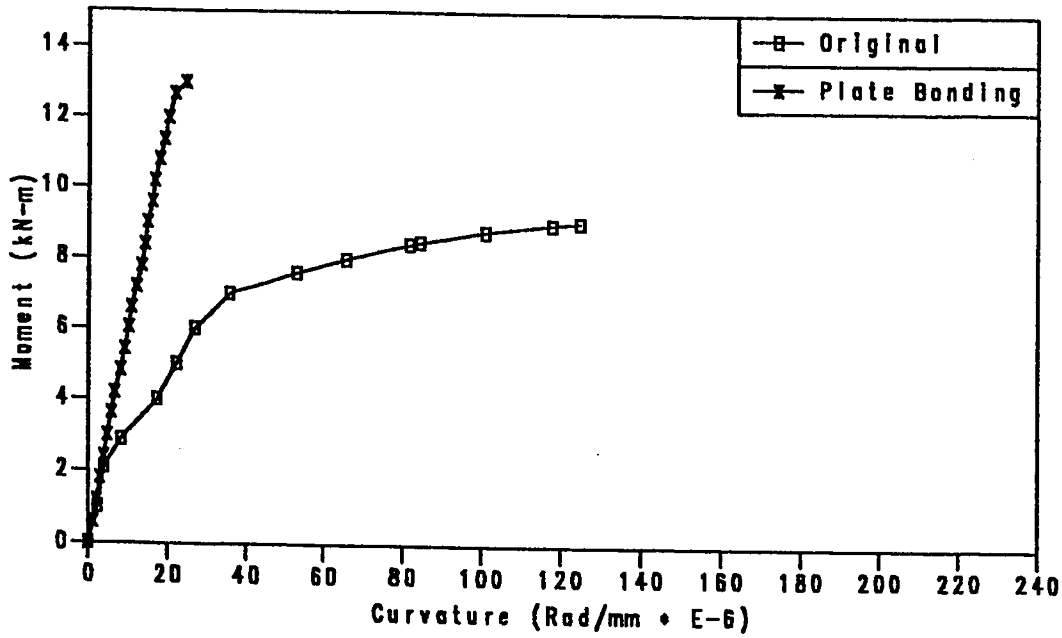

Fig. 4.71 Moment Vs Curvature Curves for Beam B-iii-9

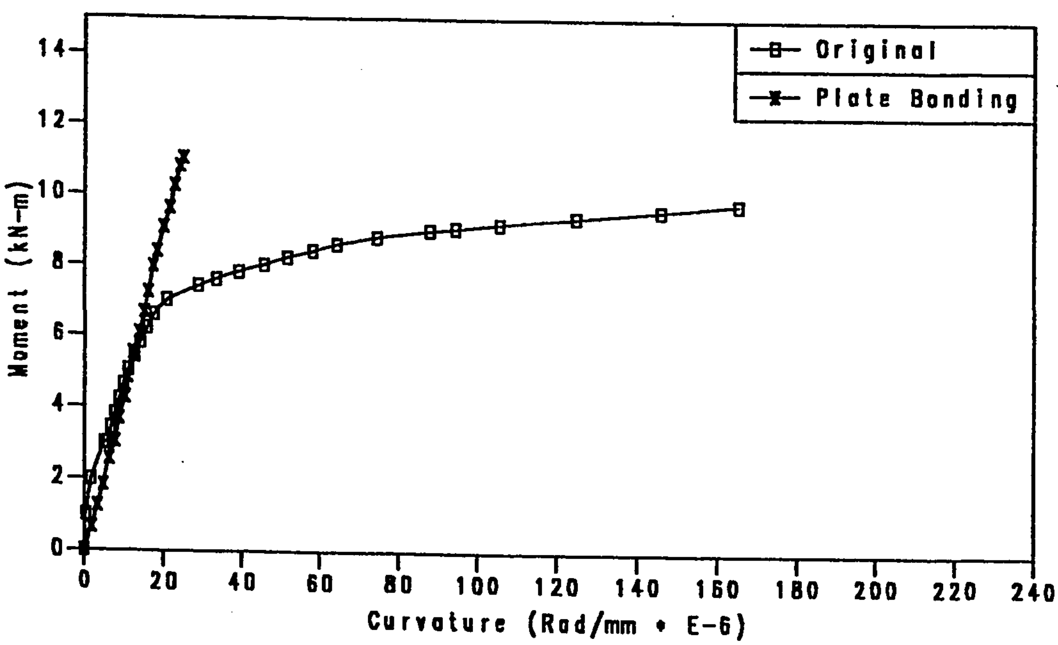

Fig. 4.72 Moment Vs Curvature Curves for Beam C-iii-7

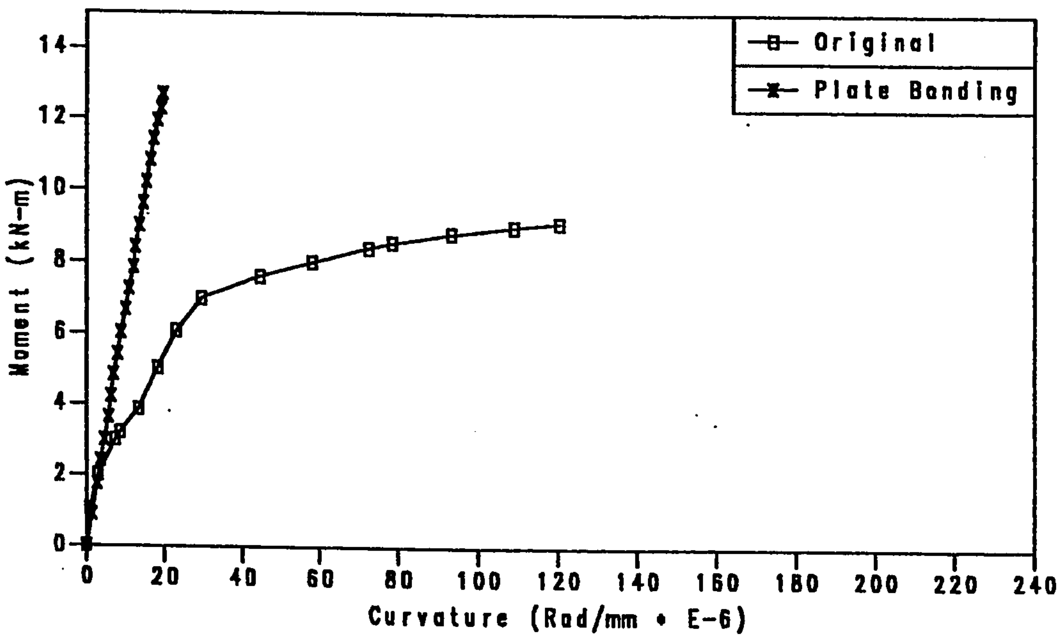

Fig. 4.73 Moment Vs Curvature Curves for Beam C-iii-8

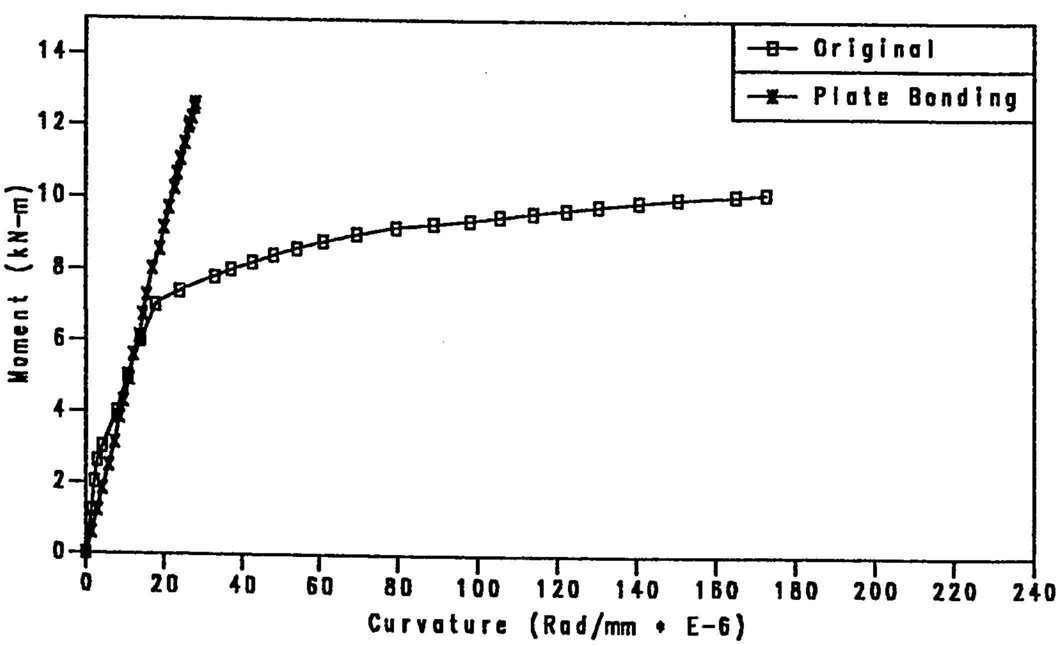

Fig. 4.74 Moment Vs Curvature Curves for Beam C-iii-9

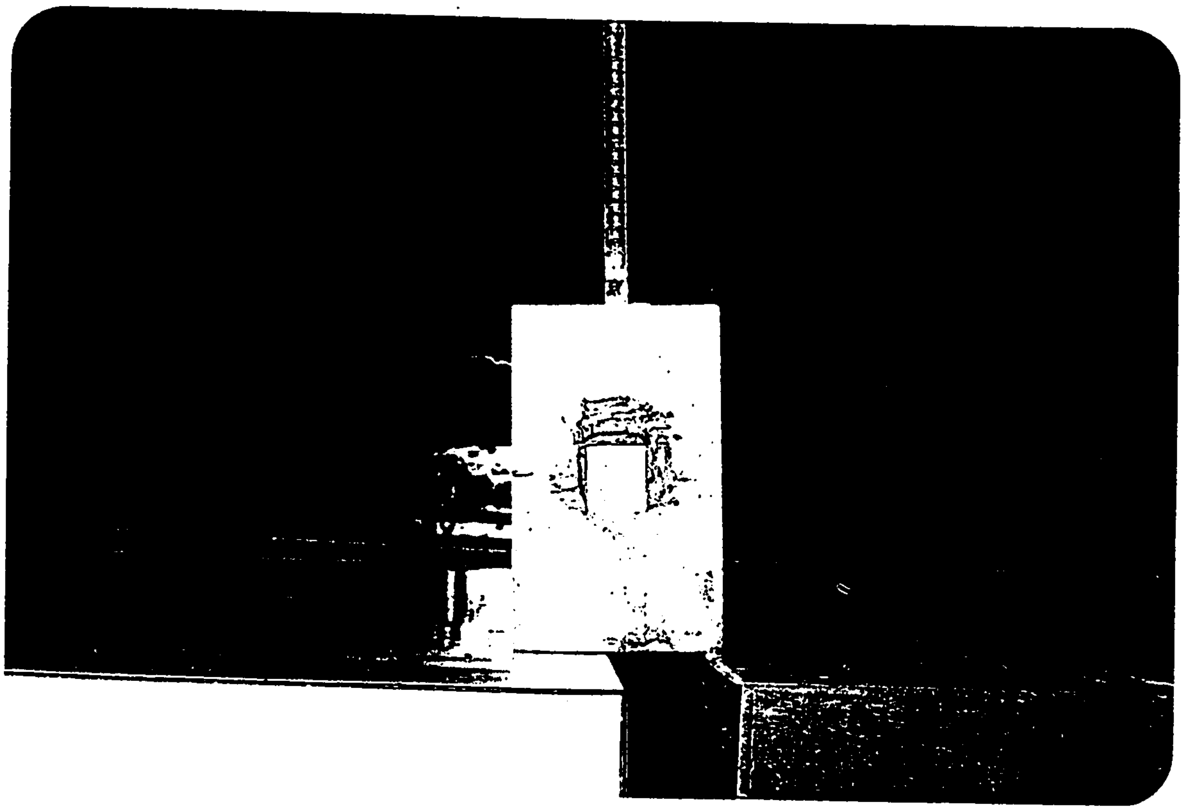

Plate 4.5 : Bond stress test results.

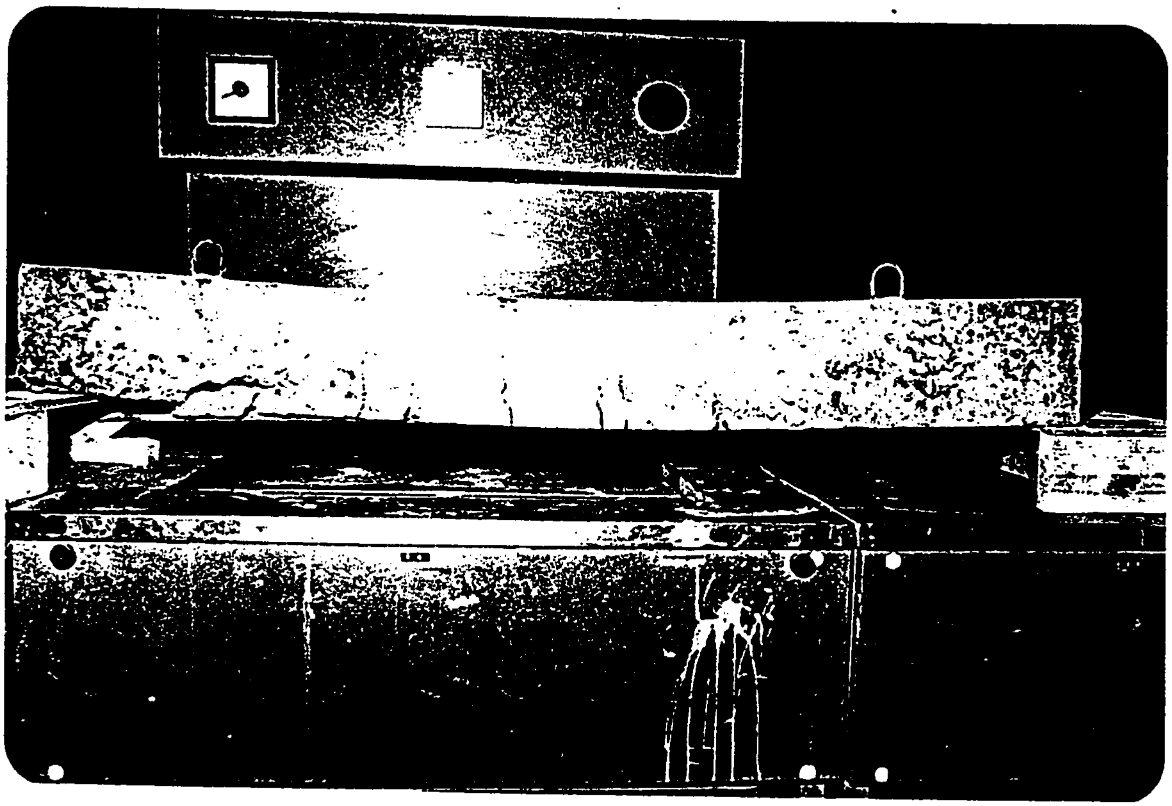

Plate 4.6 : Sample failure of steel plate bonding method.

4.4.2 Discussion of steel plate bonding results

For all beams tested after repair, load started to increase and went beyond the ultimate load of the original beams, but suddenly load reduced to almost half the load reached due to the sudden separation of the steel plate from the bottom of the beam which was caused by cracks at the shear zone. In other words, failure was caused by diagonal tension shear cracks near the supports (for all beams) which caused a separation of the steel plates as shown in plate 4.6 . But in case of 10 and 15 mm deflection groups, the failure occured due to the separation of the steel plate which reflected the weakness of the bond between the plate and concrete.

Calculating the load capacity

After strengthening the beams by Steel Plates, the load capacity of the beam increased due to the increase in the external reinforcement. The steel plate used was of mild steel and its yielding stress was (f_y) = 269 MPa (39,000 psi).

From Fig. 4.75 and using the ultimate stress method, the load capacity will be calculated as follows :

From conventional reinforced concrete theory, assuming that the reinforcing bars and steel plate yielded, the

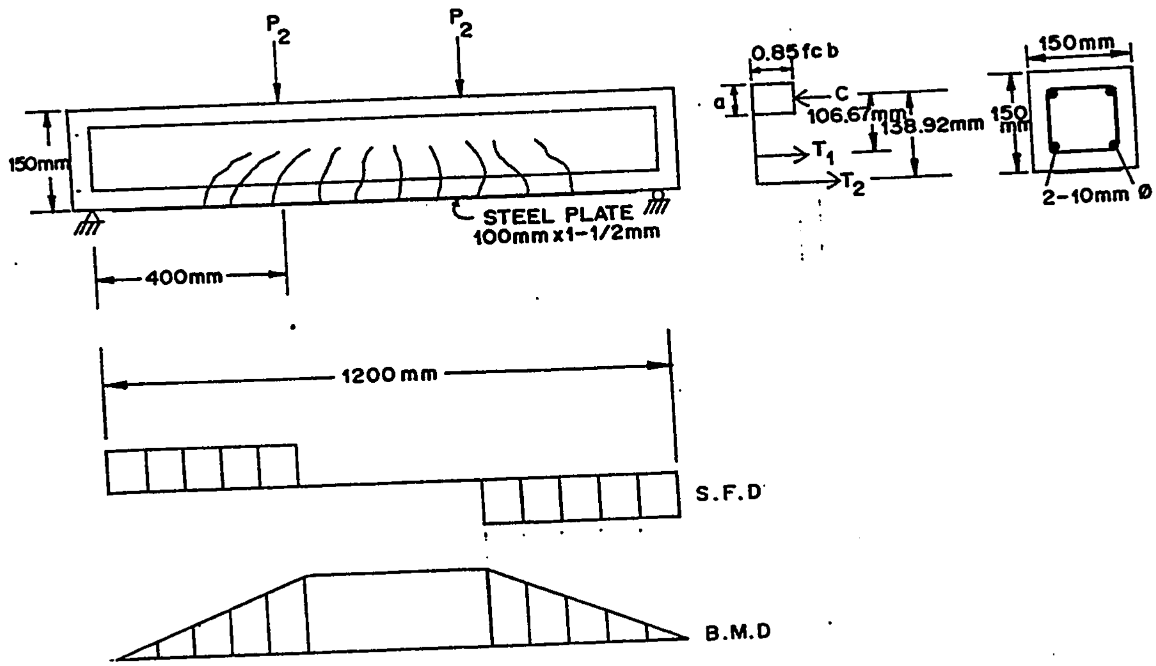

Fig. 4.75 : Steel plate bonding stresses.

reinforcing steel will contribute the following tensile forces :

T_1 = the tensile load capacity of the reinforcing bars.

$T_1 = A_{s_1} f_{y_1} = [2*(10)^2\pi/4]*414 = 65.03$ kN

where

A_{s_1} = area of reinforcing rebars

f_{y_1} = the yielding stress of reinforcing rebars

T_2 = the tensile load capacity of the steel plate.

$T_2 = A_{s_2} f_{y_2}$

A_{s_2} = area of bonded steel plate

f_{y_2} = the yielding stress of the bonded plate

$T_2 = 1.5$ mm * 100 mm * 269 MPa = 40.35 kN

T = Total tensile force in the beam

$T = T_1 + T_2 = 65.03 + 40.35 = 105.38$ kN

From equilibrium, the total force in the reinforcing bars and steel plate should be equal to the concrete compressive force.

or

$T = C\ 0.85f'_c ba$

Therefore $a = \dfrac{105.38}{0.85*31*0.15} = 26.66$ mm

Ignoring top reinforcement of the beam and taking moment around C, to determine the moment capacity.

M_n = Total nominal flexural strength

$M_n = T_1$ * moment arm + T_2 * moment arm

$$M_n = T_1*(120 - \frac{26.66}{2}) + T_2*(152.25 - \frac{26.66}{2})$$

$$= 65.03*106.67 + 40.35*138.92$$

$$= 12.542 \text{ kN-m}$$

M_{DL} = moment due to dead load of the beam

M_{DL} = 0.151 kN-m (from beam design)

$$M_n = 12.542 \text{ kN-m} = 0.151 + \frac{P_n*1.2}{6}$$

or

$$P_n = 61.96 \text{ kN}$$

The load capacity of the beam has been increased by 74 % due to the addition of external reinforcement. In fact, if the beam failed in flexure, the load P_n must be higher by at least 10 % contributed by convention steel. P_n must be in the range of 70 kN. Also, an analysis of the plate bonded beam using the working stress method is available in Appendix-A.

Load vs deflection

The load vs deflection curves of the beams strengthened by steel plate showed a significant increase in the ultimate load for all beams, but the increase was almost the same for the 10 and 15 mm deflection groups and less for the ultimate group. It can be seen from these curves that, there is no reduction in the initial stiffness and failure occured only due to the sudden separation of the plate, which was caused by large cracks at the shear zone near the supports. But, the ultimate deflection and ductility of the bonded beams were considerably reduced when compared with the unrepaired ones in the ultimate deflection of the unrepaired beams.

The cracking load was not recorded due to the existing cracks even after strengthening the beams. Table 4.4 shows the initial stiffness and the ultimate load for all beams.

Moment vs curvature

The moment vs curvature of the beams strengthened by steel plates are presented in Figs. 4.66 through 4.74. As can be seen in these figures, the curvature of the beams reduced very significantly but rigidity of the beams increased. Whereas the increase in the rigidity of the 10 and 15 mm deflection groups is higher when compared with the ultimate deflection group. This can be explained from the reduced moment of inertia of the latter, due to the presence of longer and wider cracks.

Bond stress

From Fig. 4.75 which shows the steel plate bonding stresses, the tensile forces will be obtained as follows:

T_2 = tension force due to steel plate

T_2 = area of the plate * f_y for mild steel

= 1.5 mm*100 mm*269 MPa = 40.35 kN

Therefore;

the shear force acting on the epoxy = 40.35 kN

The average experimental load to break the bond of epoxy between steel plate and concrete = 7777 N

The contact area from two sides, as shown in Fig. 3.3 = 2*(20*25) =1000 mm^2

$$\text{bond stress} = \frac{7777N}{1000}$$

$$= 7.777\ N/mm^2$$

bond force = bond stress * plate shear area

$$\text{Therfore the bond force} = 7.777\ N/mm^2 * 400mm * \frac{100mm}{1000}$$

$$= 311\ kN > 40.35\ kN$$

It can be concluded that after repair, failure occured due to the diagonal tension shear cracks and not due to the breaking of the bond, because the bond force is much greater than the shear force especially for the ultimate deflection group beams.

In summary, the addition of steel by bonding the steel plate to the beam had changed the behavior of the concrete beam. The load capacity increased, the ductility decreased and the mode of failure changed.

4.5 Combined Repair Method

In this study, epoxy injection was used as a repair method to restore the strength of the reinforced concrete beams, followed by strengthening the beams by a small layer of ferrocement to produce more durable beams. The two methods were combined as if they were one method. The reason for this study is to assess the durability of the concrete structure represented in terms of strength and ductility.

For the purpose of repair by the combined method, only 3 beams were tested up to ultimate deflection and repaired by this method. A comparison of the results of this method with other methods at ultimate deflection is also presented.

4.5.1 Presenting the results of the combined method

As mentioned above, only 3 beams were used in this method after testing them up to ultimate deflection. The 3 beams

were repaired by epoxy, followed by ferrocement, cured and then tested up to ultimate. From the collected data, the curves presented for each beam are as follows :
the load vs central deflection, moment vs curvature and load vs crack width.

Load vs deflection

Figs. 4.76 through 4.78 represent the load vs mid-span deflection for beams tested up to ultimate and repaired by epoxy and strengthened by ferrocement.

AS in all load vs deflection curves, table 4.5 was prepared for the original and repaired beams. The table includes the initial and reduced stiffnesses and the loads at cracking, yielding and ultimate points. Also, the ultimate deflection and ductility, area under the P-Δ curves, were prepared for all beams, before and after repair, as shown in table 4.5.

Moment vs curvature

The moment vs curvature curves were plotted from the data obtained during testing, as described in section 4.1.

Figs. 4.79 through 4.81 represent the moment vs curvature curves of the beams tested up to ultimate mid-span deflection, repaired by epoxy and strengthened by ferrocement method.

Table 4.5 : Comparison of Original and Combined Repaired Beams

Item		P_{cr} (kN)		K_i (kN/mm)		K_r (kN/mm)		P_y (kN)		P_t (kN)		$\Delta_{ult.}$ (mm)		Ductility (kN-mm)	
Δ_c	Beam#	Unrep	Rep.	Unrep	Rep.	Unrep	Rep.	Unrep	Rep.	Unrep	Rep.	Unrep	Rep.	Unrep	Rep.
	C-10	12.65	30.20	15.00	23.48	7.60	7.62	36.00	47.90	50.30	56.70	22.25	13.81	792.51	623.30
Ult.	C-11	13.02	27.70	14.35	22.22	5.80	6.58	36.00	48.00	46.15	52.06	24.82	8.80	857.14	758.50
	C-12	14.63	29.68	15.57	22.58	5.92	7.28	35.10	50.80	51.32	55.90	26.28	12.79	979.59	579.08
Ave.		13.43	29.19	14.97	22.76	6.77	7.16	34.70	48.90	49.26	54.89	24.46	11.80	876.42	653.63

Load vs crack width

Figs. 4.82 through 4.84 represent the load vs crack Width curves of the beams tested up to ultimate mid-span deflection, repaired by epoxy and strengthened by a layer of ferrocement.

4.5.2 Discussion of the combined repair results

As in the case of ferrocement method of repair, the load capacity of the beams has been increased by 15 %. And Failure of all beams occured because of crushing of concrete in the compression zone and due to development of large cracks in the flexural zone, as shown in plate 4.7 .

Load vs deflection

The load vs deflection curves of the beams repaired by epoxy and strengthened by ferrocement were presented in Figs. 4.76 through 4.78. The curves show a significant increase in the initial stiffness. Also, there is an increase in the reduced stiffness and the ultimate load. The average values of cracking and ultimate loads and the initial and reduced stiffnesses of the original and repaired beams were prepared, as shown in table 4.5 . As can be seen from this table that, the ultimate deflection and ductility have shown relatively small decrease in their values compared to the original ones, when compared with other

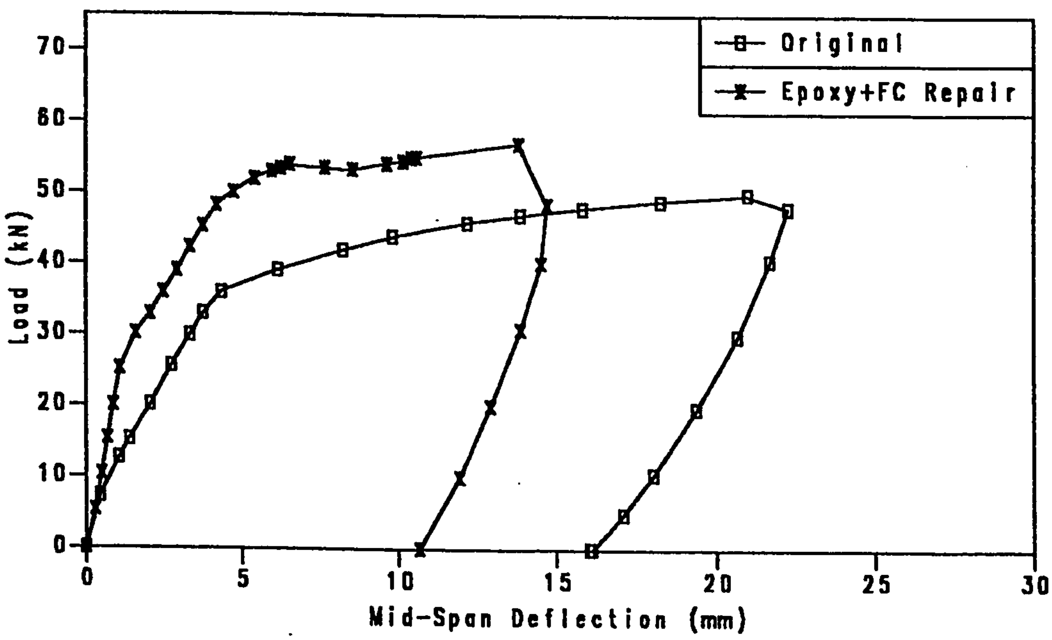

Fig. 4.76 Load Vs Deflection Curves for Beam C-iv-10

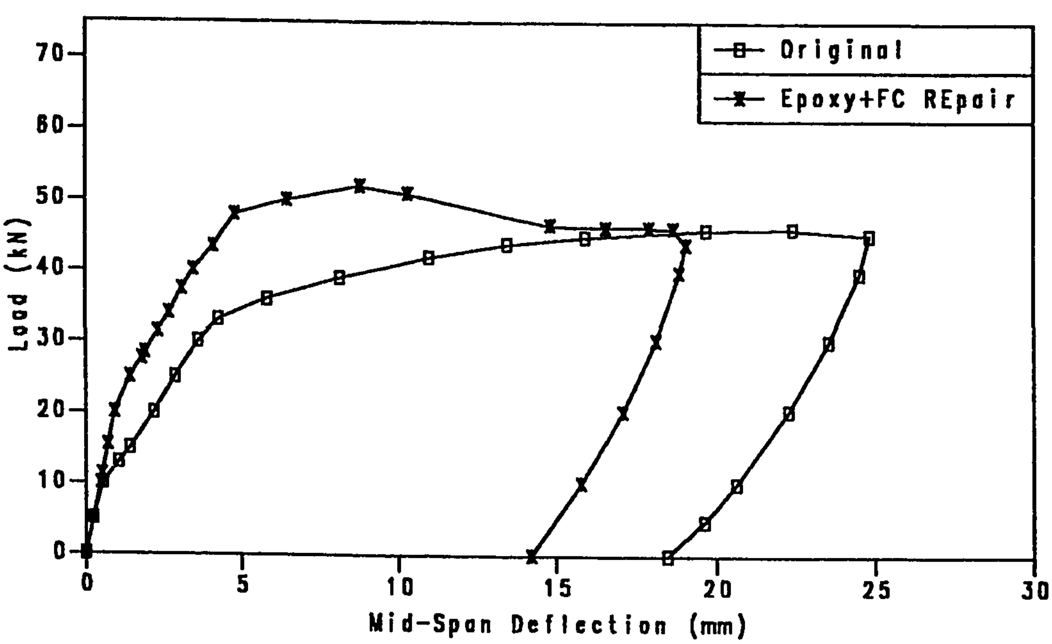

Fig. 4.77 Load Vs Deflection Curves for Beam C-iv-11

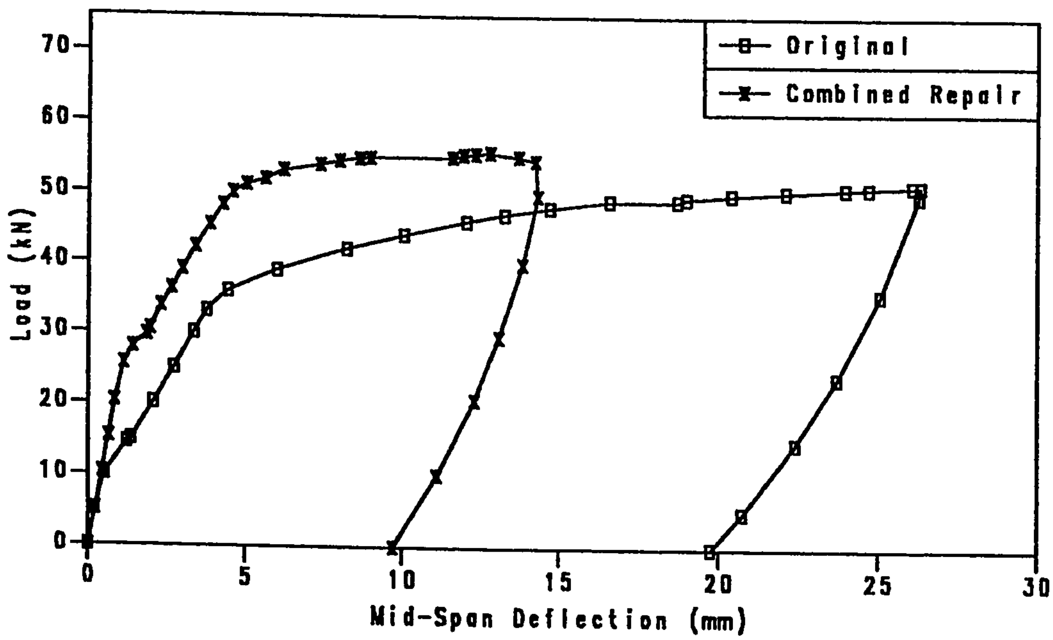

Fig. 4.78 Load Vs Deflection Curves for Beam C-iv-12

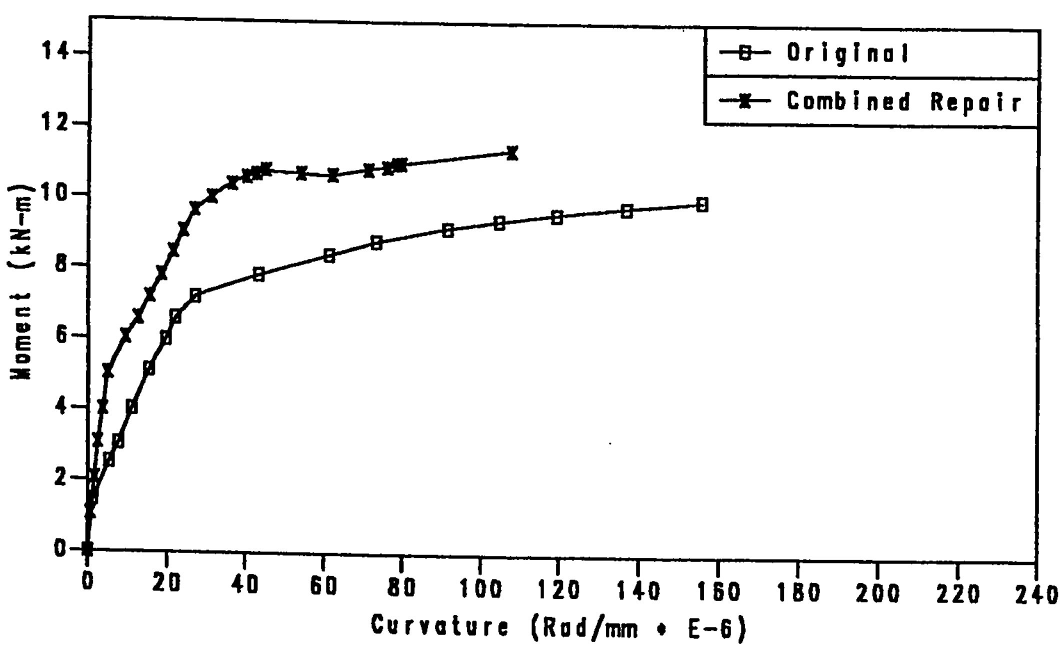

Fig. 4.79 Moment Vs Curvature Curves for Beam C-iv-10

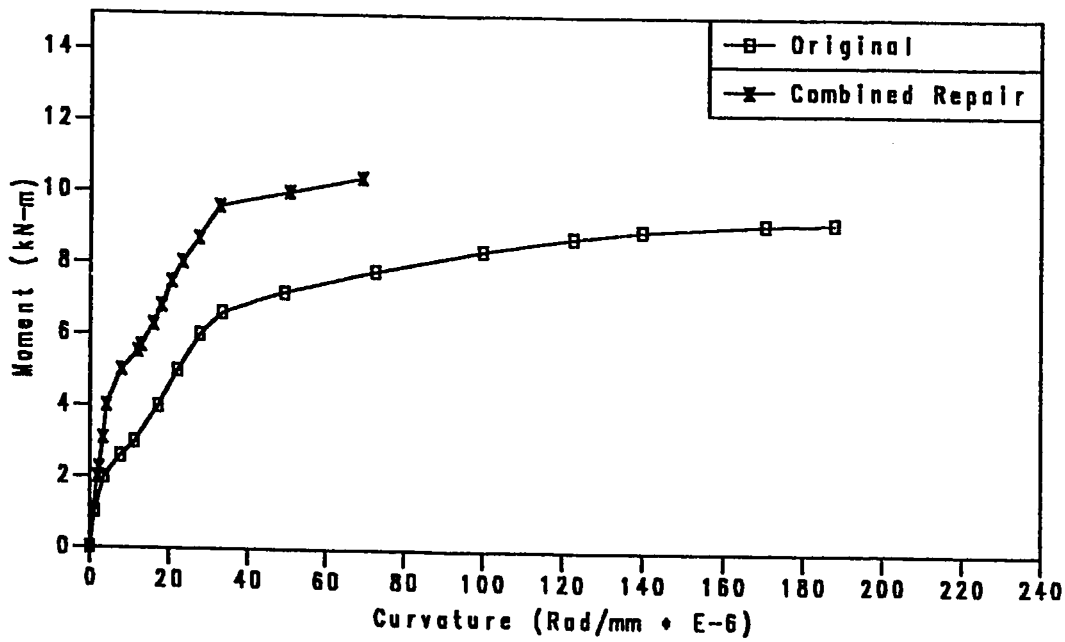

Fig. 4.80 Moment Vs Curvature Curves for Beam C-iv-11

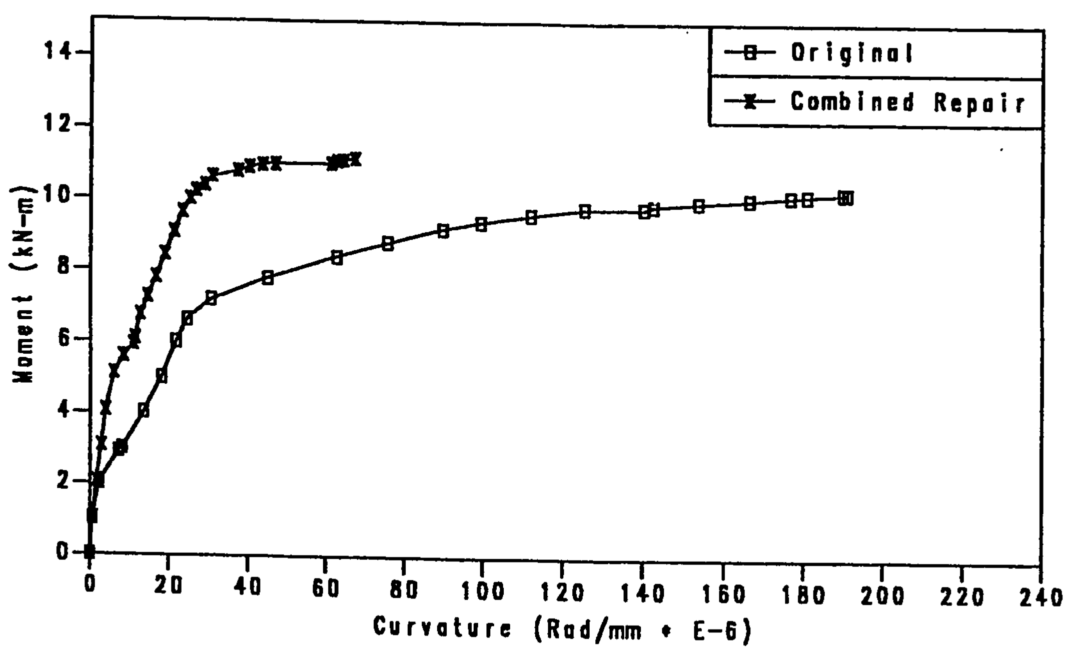

Fig. 4.81 Moment Vs Curvature Curves for Beam C-iv-12

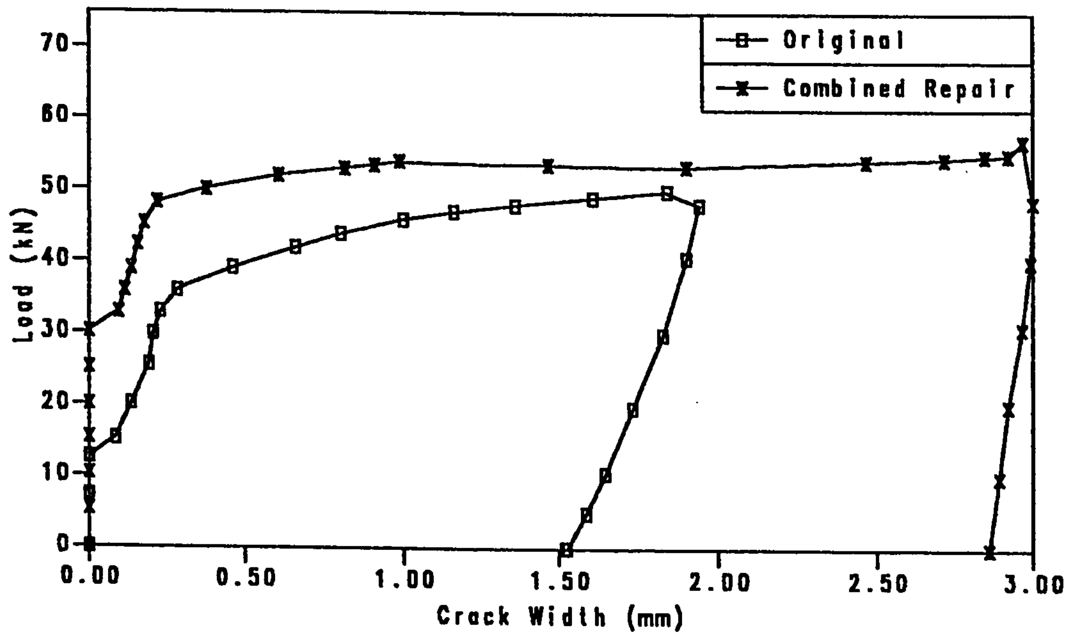

Fig. 4.82 Load Vs Crack Width Curves for Beam C-iv-10

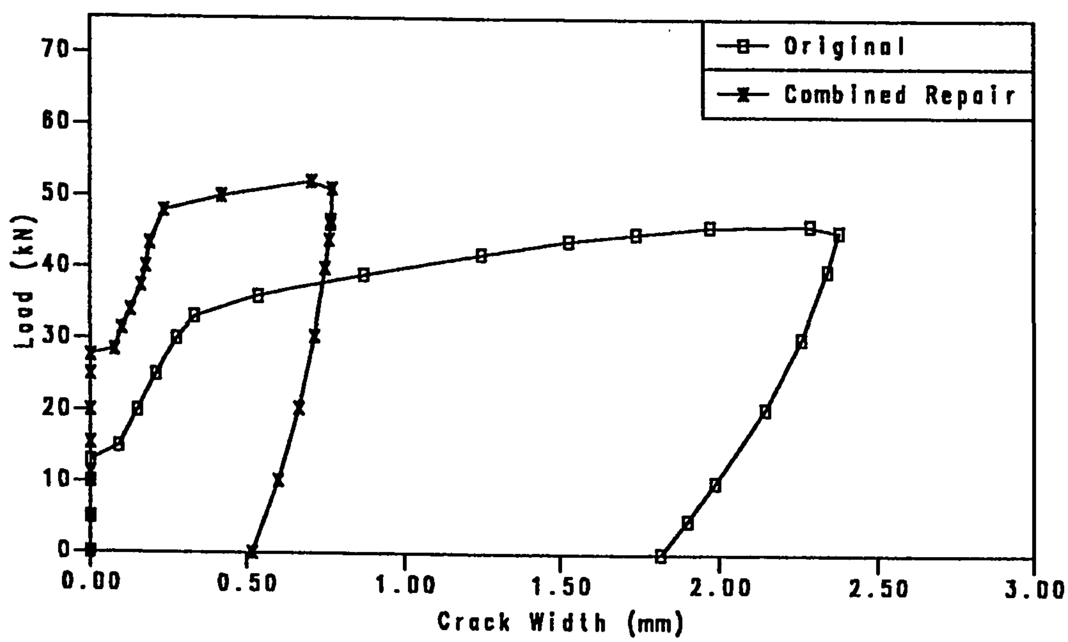

Fig. 4.83 Load Vs Crack Width Curves for Beam C-iv-11

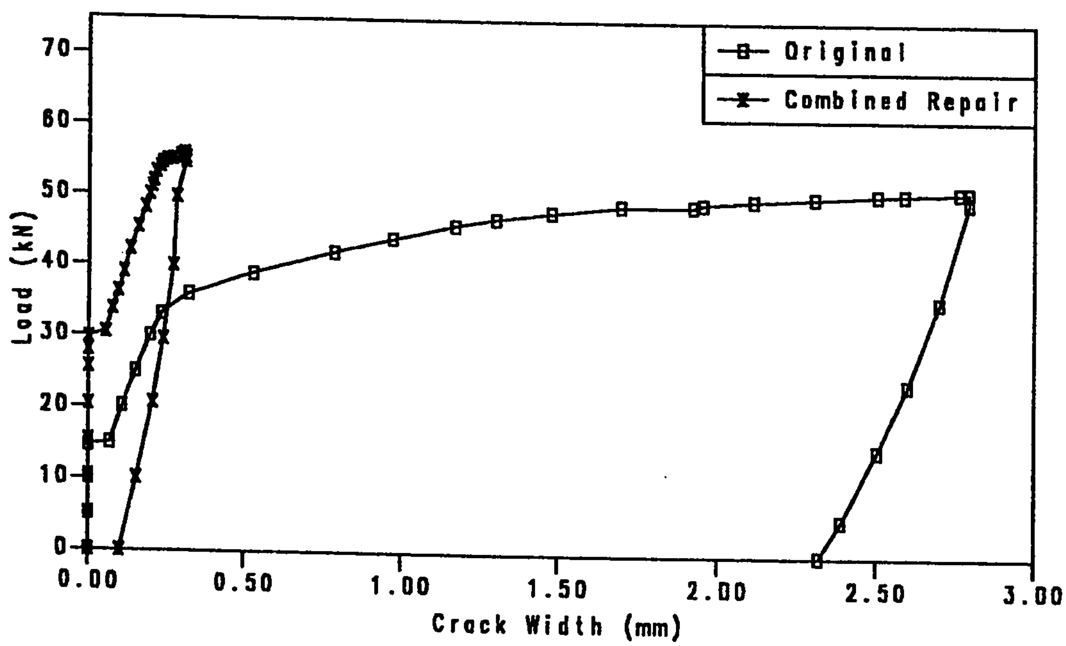

Fig. 4.84 Load Vs Crack Width Curves for Beam C-iv-12

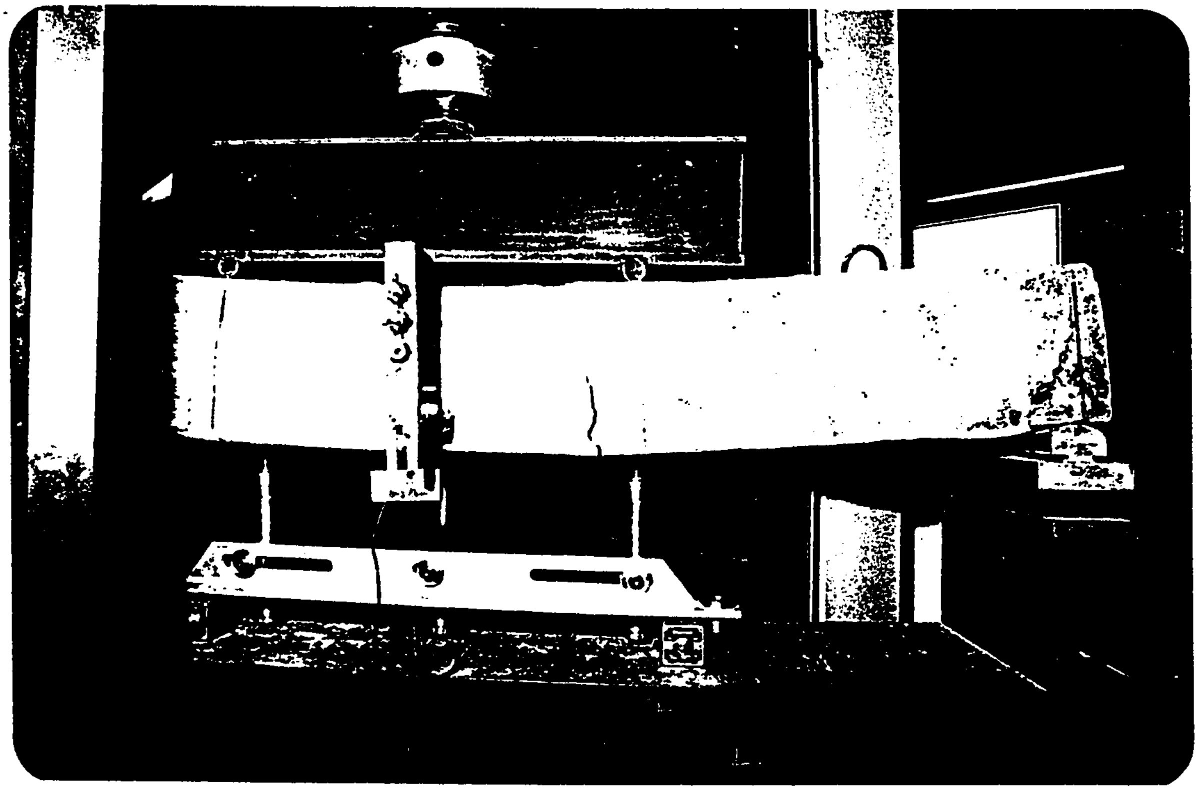

Plate 4.7 : Crack patterns of combined repair method.

methods of repair.

Moment vs curvature

The moment vs curvature curves of the beams repaired by epoxy and strengthened by a ferrocement layer are presented in Figs. 4.79 through 4.81 .

The curvature of the beams reduced very significantly after applying the combined method to the 3 beams, while the rigidity of the beams showed an increase after strengthening the beams. This can be explained from the property of epoxy material which filled the cracks and thereby restored the rigidity of the beams in addition to an increase in the cross section and the amount of reinforcement of the beams due to the addition of ferrocement layer.

Load vs crack width

The load vs crack width curves of the original and the repaired beams are shown in Figs. 4.82 through 4.84, which show a significant increase in the cracking load of the repaired and strengthened beams by the combined method. The average cracking load before and after repair can be obtained from table 4.5. As can be noticed from the load vs crack width curves that , the crack width of the repaired beams decreased significantly in comparison with the original beams and the reason for this decrease is due to the strengthening of the beams by the ferrocement layer.

Also, in this method, all cracks were observed to be away from the original ones, as in the case of epoxy repair method.

4.6 Comparison of test results between repair methods

4.6.1 Presenting the results of all repair methods

Load vs deflection

Figs. 4.85 through 4.87 and 4.88 through 4.90 represent the load vs deflection of the different repair methods (ferrocement, epoxy, steel plate bonding) of the 10 and 15 mm deflection groups respectively. Figs. 4.91 through 4.93 represent the load vs deflection of the different repair methods (ferrocement, epoxy, steel plate bonding and combined) of the ultimate deflection group.

Table 4.6 represents a comparison between the different repair methods (ferrocement, epoxy, steel plate bonding and combined) at different levels of deflection in terms of cracking, yielding and tested loads in addition to the initial and reduced stiffnesses, ultimate load, ultimate deflection and ductility.

Table 4.7 is a summary of table 4.6 which contains a

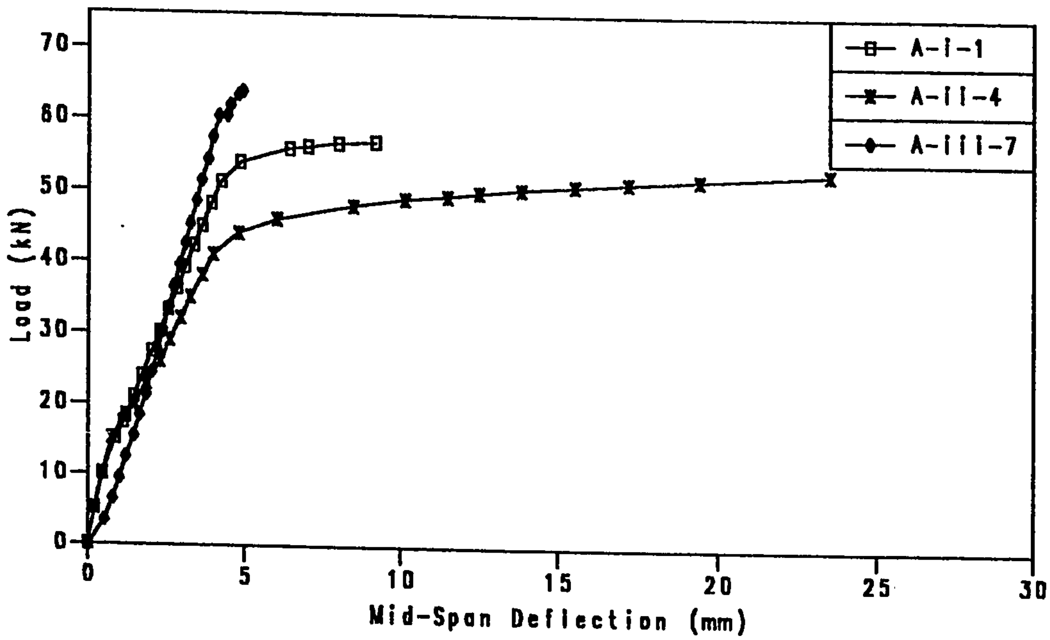

Fig. 4.85 Load Vs Deflection Curves for Various Repair Methods

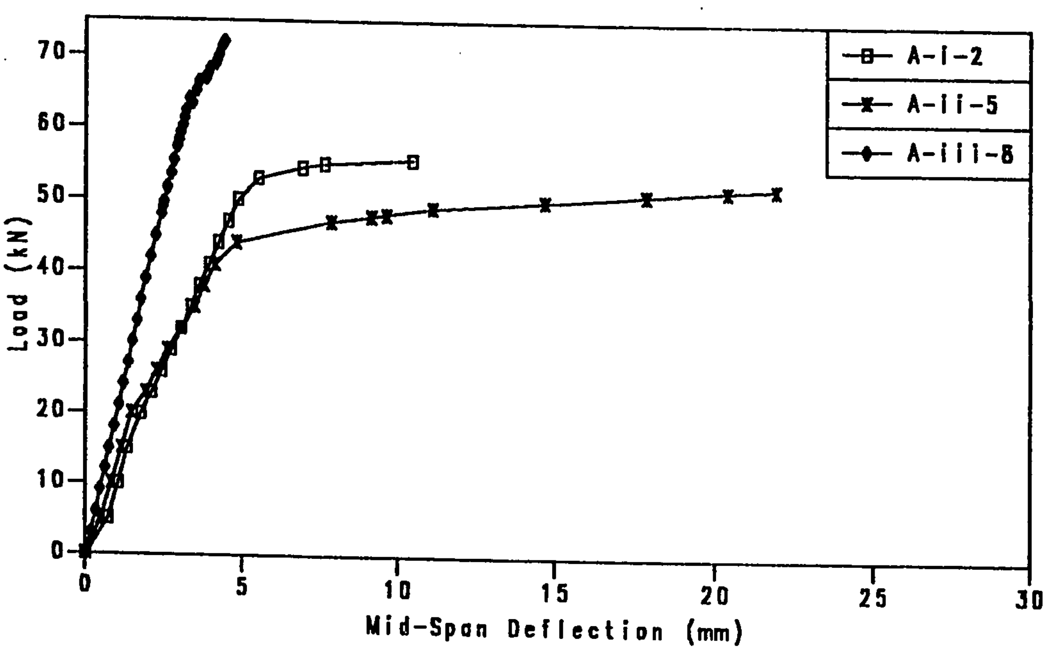

Fig. 4.86 Load Vs Deflection Curves for Various Repair Methods

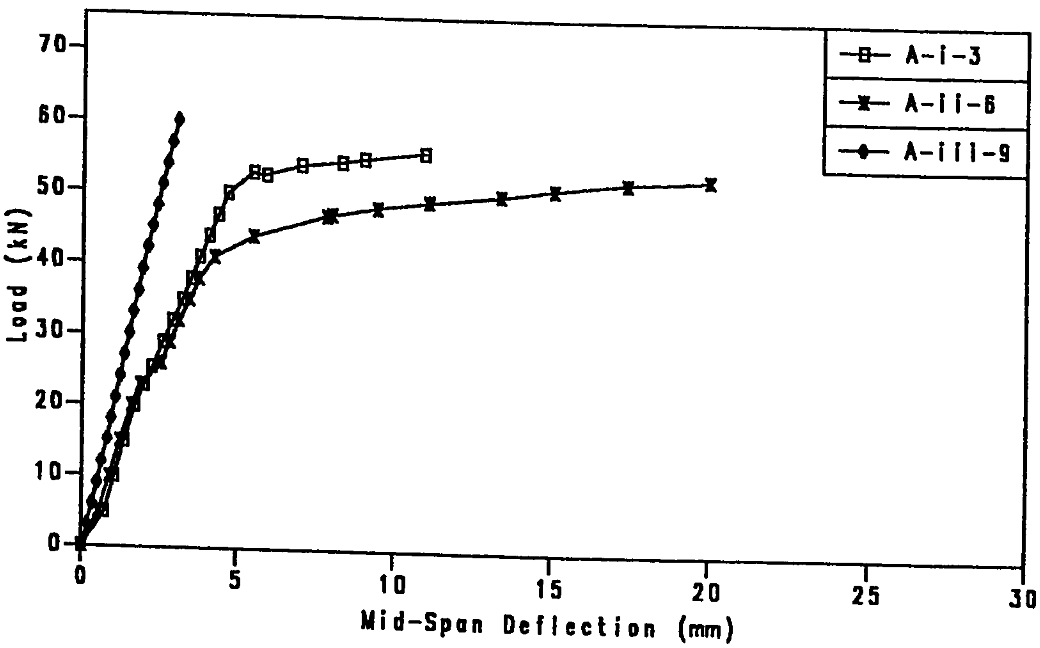

Fig. 4.87 Load Vs Deflection Curves for Various Repair Methods

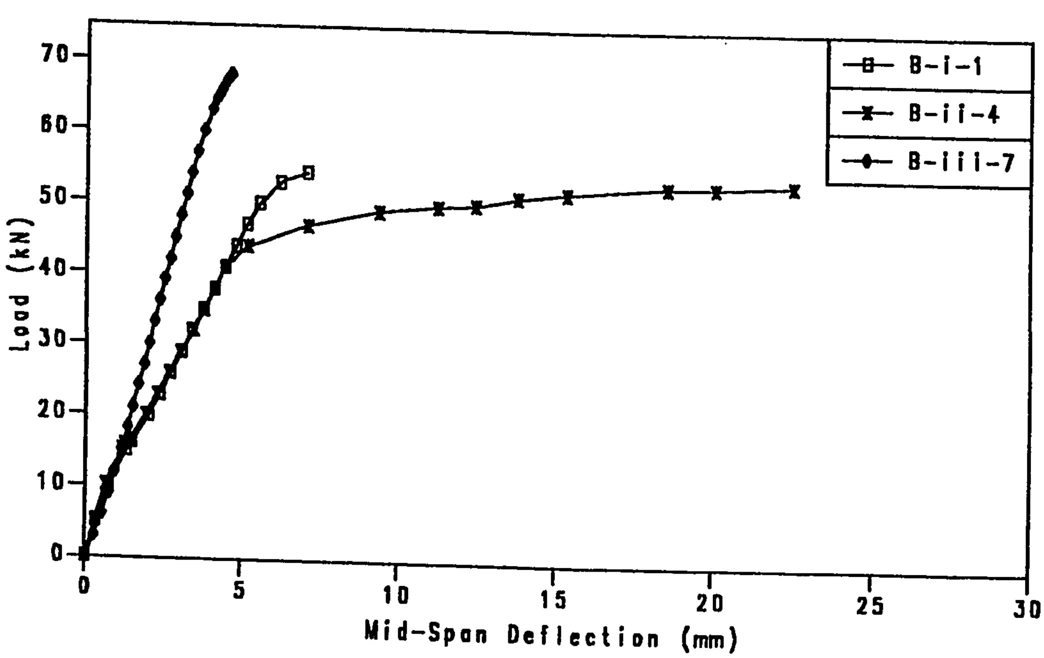

Fig. 4.88 Load Vs Deflection Curves for Various Repair Methods

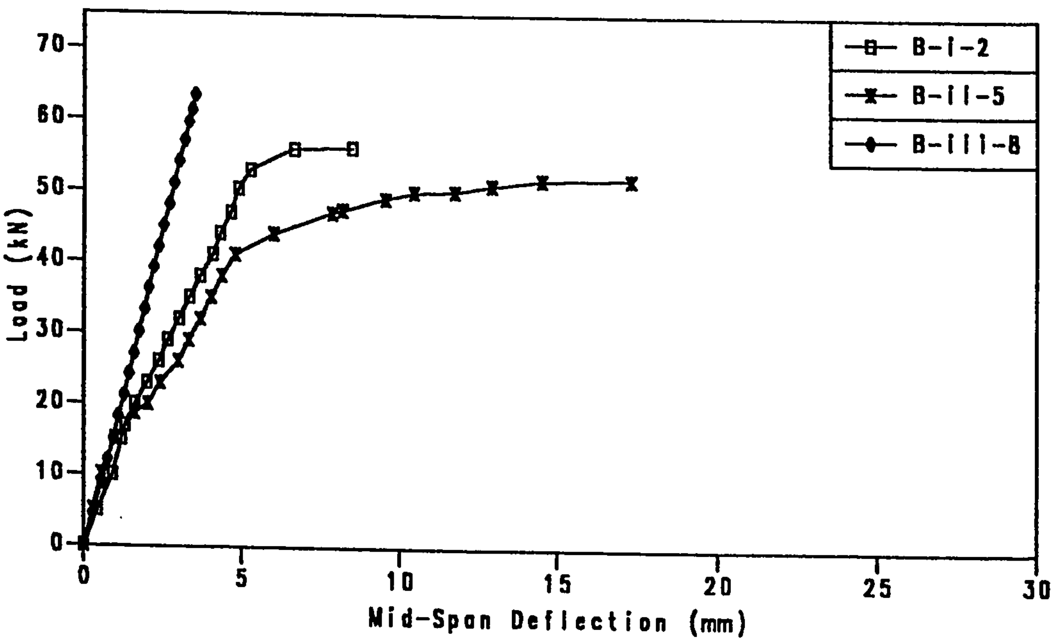

Fig. 4.89 Load Vs Deflection Curves for Various Repair Methods

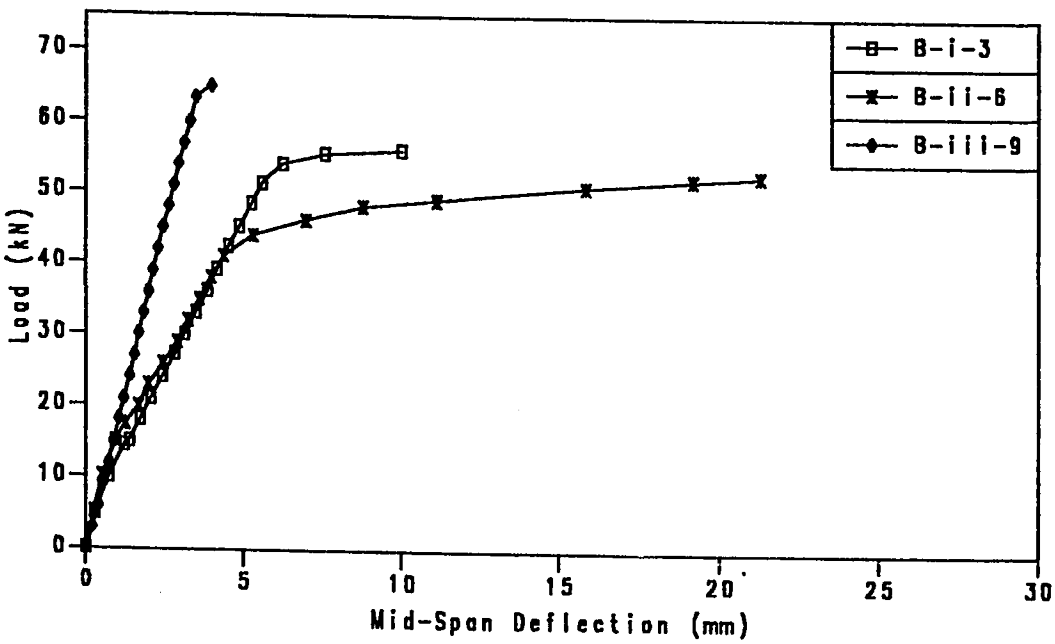

Fig. 4.90 Load Vs Deflection Curves for Various Repair Methods

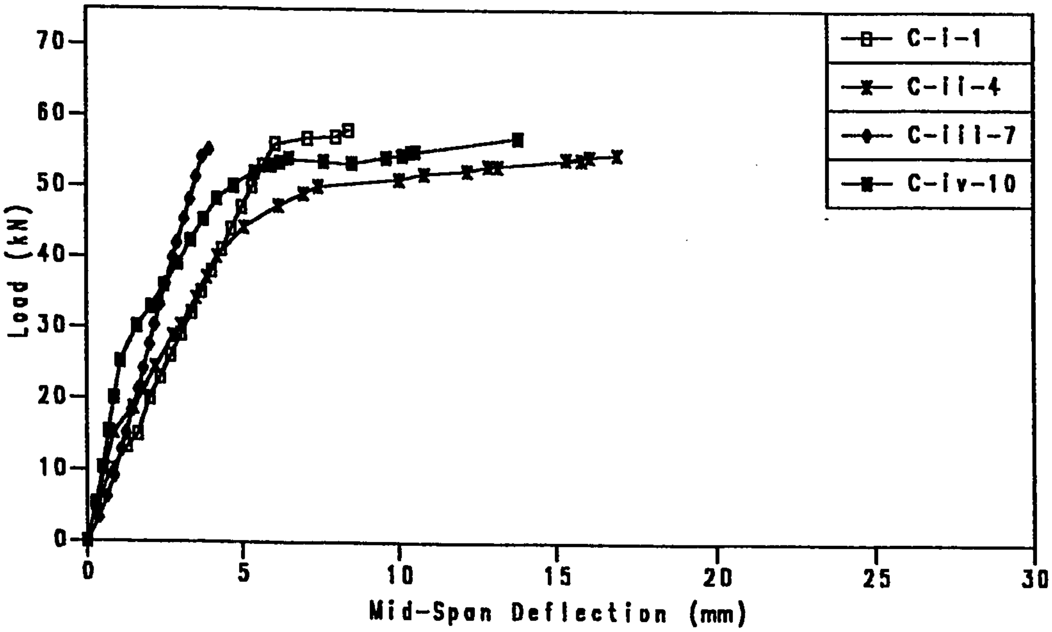

Fig. 4.91 Load Vs Deflection Curves for Various Repair Methods

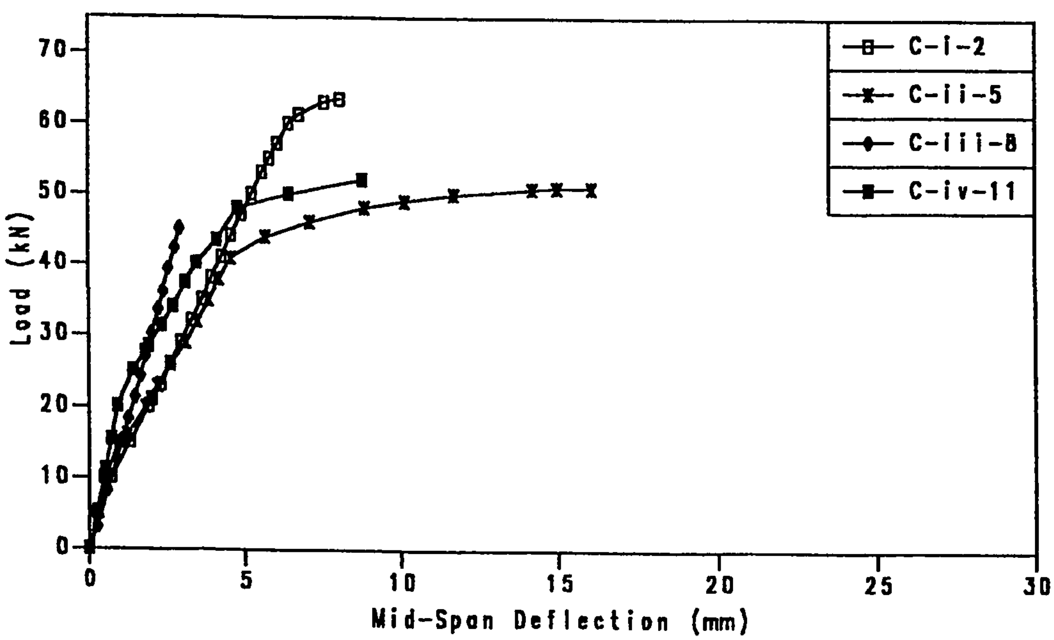

Fig. 4.92 Load Vs Deflection Curves for Various Repair Methods

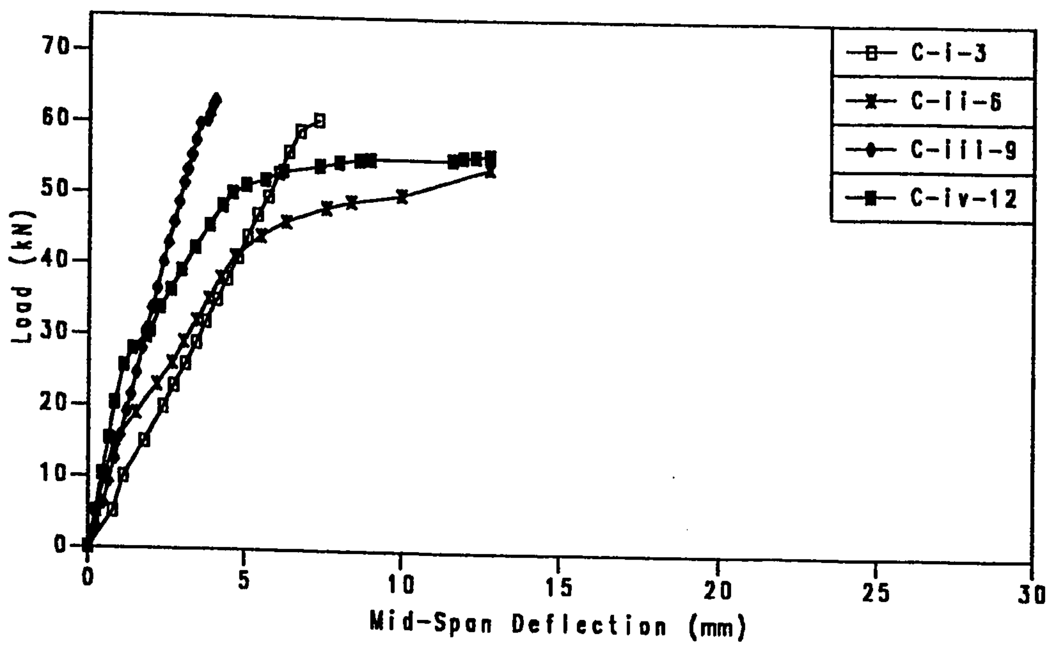

Fig. 4.93 Load Vs Crack Width Curves for Various Repair Methods

Table 4.6 : Comparison of Methods of Repairing Beams

Item		P_{cr} (kN)		K_i (kN/mm)		K_r (kN/mm)		P_y (kN)		P_t (kN)		$\Delta_{ult.}$ (mm)		Ductility (kN-mm)	
Type	Δ_c	Unrep	Rep.	Unrep	Rep.	Unrep	Rep.	Unrep	Rep.	Unrep	Rep	U nrp	Rep.	Unrep	Rep.
FC	10mm	14.68	17.58	13.35	17.95	7.79	10.36	33.08	51.73	43.03	56.20	-	10.86	-	378.40
	15mm	15.30	15.87	14.04	16.25	6.91	9.04	33.27	51.73	45.08	55.51	-	10.16	-	301.02
	Ult.	13.31	14.71	16.66	14.48	7.22	8.85	34.63	57.03	48.43	60.68	24.97	8.32	790.25	198.22
Epoxy	10mm	14.07	18.03	16.85	15.90	8.22	7.71	35.05	42.60	45.21	52.82	-	25.32	-	1017.57
	15mm	15.29	17.52	13.48	15.65	7.68	7.40	35.07	43.40	46.70	52.80	-	22.08	-	851.47
	Ult.	13.97	17.94	17.61	15.63	8.13	7.69	34.31	42.07	51.38	52.23	27.40	16.02	1036.28	566.05
Steel Plate	10mm	13.14	-	15.56	18.52	7.66	-	34.38	60.08	43.58	65.34	-	4.13	-	120.47
	15mm	15.12	-	15.57	17.63	7.49	-	34.65	62.33	45.55	65.44	-	4.04	-	116.78
	Ult.	13.76	-	16.15	15.94	8.05	-	35.00	52.97	50.77	59.06	26.80	3.62	1002.27	70.38
Com-bined	Ult.	13.43	29.19	14.97	22.76	6.77	7.16	34.70	48.90	49.25	54.89	24.46	11.80	876.42	663.63

Table 4.7 : Comparison of Average Results of Different Groups of Repair Methods (Load in kN and Stiffness in kN/mm)

Methods	Δ_c	P_{cr}	K_i	K_r	P_{ulti}		Δ_{ulti} (mm)	
					Unrep	Rep.	Unrep	Rep.
FC	10mm	17.584	17.949	10.356	-	56.20	-	10.86
	15mm	15.871	16.254	9.936	-	55.51	-	10.16
	Ult.	14.709	14.482	8.846	48.43	60.68	24.97	8.32
Epoxy	10mm	18.026	15.897	7.708	-	52.82	-	25.32
	15mm	17.521	15.650	7.401	-	52.80	-	22.08
	Ult.	17.941	15.625	7.689	51.38	52.23	27.40	16.02
Plate Bonding	10mm	-	18.581	-	-	65.34	-	4.13
	5mm	-	17.628	-	-	65.44	-	4.04
	Ult.	-	15.936	-	50.77	59.06	26.80	3.62
Combined	Ult.	29.194	22.760	7.16	49.25	54.89	24.46	11.80

comparison between the different repair methods.

Table 4.8 is a summary of the percentage increase in the stiffness, load and ultimate deflection of the beams after repair and was calculated from table 4.6 for all methods of repair comparing the results of the repaired beams to the results of the ultimate group of each method of repair.

Number of cracks and crack width

Table 4.9 is a summary of the average number of cracks at three levels of crack width and the average maximum crack width for all methods of repair for different levels of deflection. In this table, cracks were counted at different crack widths of 0.1, 0.3 and >0.3 mm. For the original beams, only cracks >0.3 mm were counted and cracks <0.1 mm were ignored. Also, the average maximum crack widths were measured and recorded for the original and repaired beams.

Strength and ductility ratios

The ductility or deformations capacity is the ability of a material to undergo large inelastic deformations without fracture, and it is important not only because it may serve as warning of impending failures, but it is also essential if the structure must resist large dynamic loads, such as, hurricane, earthquake, etc.

Table 4.8 : Percentage Increase in the Load, Deflection and Stiffness of the Repaired Beams Compared to the Ultimate Group of Repair Methods. (Load in kN and Stiffness in kN/mm).

Methods	Δ_c	P_{cr}	K_i	K_r	P_y	P_{ult}	$\Delta_{ult.}$
FC	10mm	32	8	44	49	16.0	-57
	15mm	19	-2	25	49	15.0	-59
	Ult.	11	-13	23	65	25.0	-67
Epoxy	10mm	29	-10	-5	24	2.8	- 8
	15mm	25	-11	-9	27	2.8	-19
	Ult.	28	-11	-5	23	1.7	-42
Plate Bonding	10mm	-	15	-	72	29.0	-85
	15mm	-	9	-	78	29.0	-85
	Ult.	-	-1	-	51	16.1	-87
Combined	Ult.	117	52	6	41	11.4	-52

Table 4.9 : The Average Number of Cracks and Average Maximum Crack Width (mm)

Methods	Δ_c	Original N_3	Repaired N_1	Repaired N_2	Repaired N_3	Max. Crack Width Original	Max. Crack Width Repaired
FC	10mm	5	1	3	3	0.67	1.10
	15mm	6	2	5	1	1.17	0.83
	Ult.	5	1	4	1	2.03	0.50
Epoxy	10mm	5	2	1	8	0.70	2.17
	15mm	6	2	2	7	1.10	2.00
	Ult.	6	1	2	4	2.50	2.67
Plate Bonding	10mm	5	-	-	-	0.63	-
	15mm	6	-	-	-	1.07	-
	Ult.	6	-	-	-	2.53	-
Combined	Ult.	6	2	1	3	2.33	4.57

* N_1 represent the average number of cracks <.1 mm.

* N_2 represent the average number of cracks ≤.3 mm but >.1 mm.

* N_3 represent the average number of cracks >.3 mm.

Table 4.10 represents a comparison between the strength and ductility ratios for all methods of repair at different levels of deflection. Also, Fig. 4.94 represents the ductility ratios for all methods of repair at different levels of deflection, while Fig. 4.95 represents the strength to ductility increase for all methods of repair at different levels of deflection. The strength ratio was calculated as the average ultimate load of the repaired beams to the average ultimate load of the beams of ultimate deflection group. And, the ductility ratio was calculated as the ductility of the repaired beams to the ductility of the ultimate deflection group beams.

Cost of materials and labours

Table 4.11 represent a comparison of the cost of man-hour and materials per beam between the various repair methods. Also, it includes the curing time, the need of skilled labours and the average increase in the ultimate load with respect to the repair cost.

4.6.2 Discussion of results of all repair methods

Load vs deflection

From Figs. 4.85 through 4.90 of the load vs deflection curves of the 10 and 15 mm deflection groups, the highest increase in the ultimate load capacity was obtained by the

Table 4.10: A Comparison of Strength Ratio (SR) to the Ductility Ratio (DR) for All Methods of Repair.

Methods	Δ_c	$P_{ultimate}$ (kN)			Ductilituy (kN-mm)		
		Original	Repaired	SR %	A_o	A_r	DR %
FC	10mm	-	56.200	116	-	378.40	47
	15mm	-	55.510	115	-	301.02	38
	Ult.	48.427	60.677	125	790.25	198.22	25
Epoxy	10mm	-	52.817	103	-	1017.57	98
	15mm	-	52.797	103	-	851.47	82
	Ult.	51.377	52.233	102	1036.28	566.05	55
Plate Bonding	10mm	-	65.340	129	-	120.47	12
	15mm	-	65.440	129	-	116.78	12
	Ult.	50.770	59.055	116	1002.27	70.38	7
Combined	Ult.	49.250	54.887	111	876.42	653.63	75

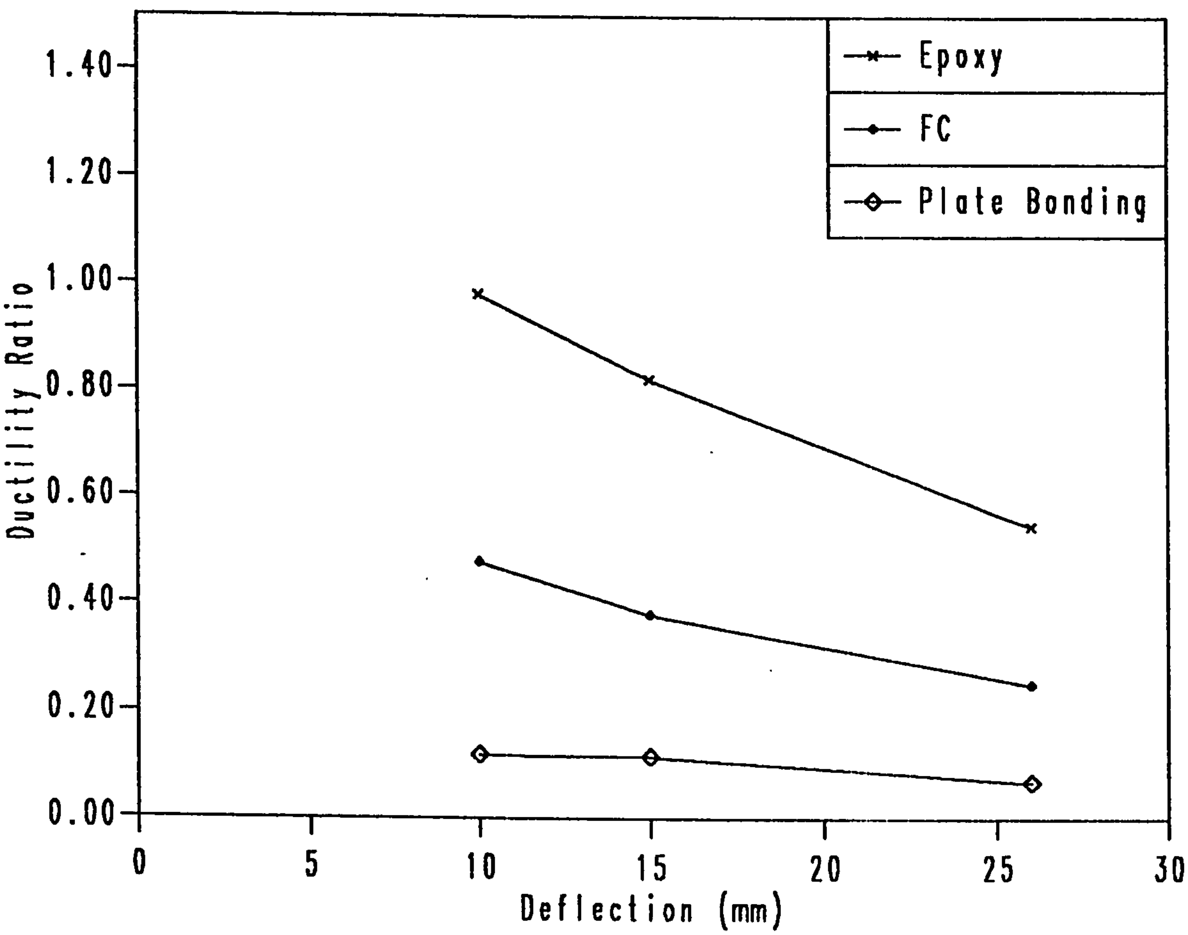

Fig. 4.94 Ductility ratio of various repair methods at different levels of deflection.

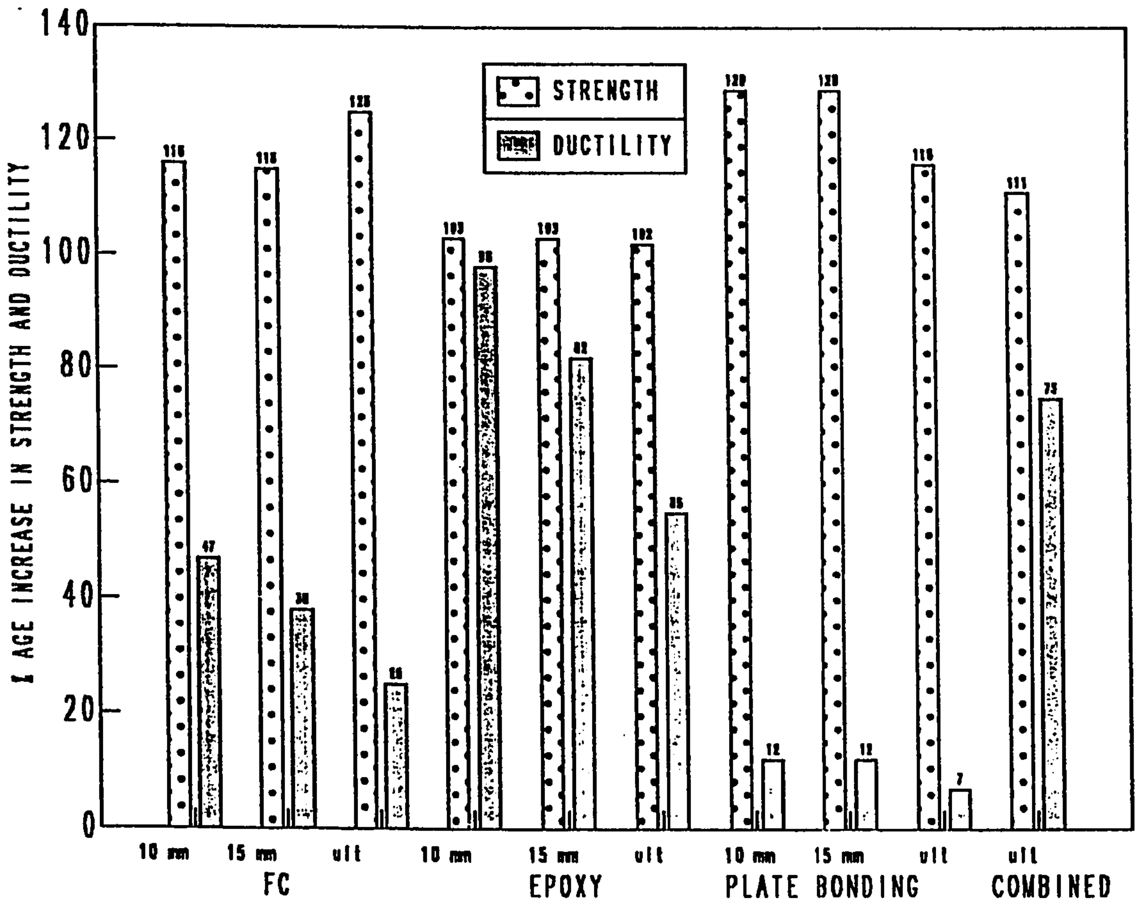

Fig. 4.95 Strength and ductility ratios for all repair methods at different levels of deflection.

TABLE 4.11 Comparison of Cost & Man-hour Per Beam Between Repair Methods

Item	Epoxy	Steel Plate Bonding	Ferrocement
Materials	Group of 12 Beams EP-CA (2 Cans) = SR 259 NITOKIT (one box) = SR 633 108(nipples+caps) = SR 432 Total = SR 1324 Ave. Cost Per Beam = SR 110	Group of 9 Beams EP-CA (2 Cans) = SR 259 9 Steel Plates = SR 45 Total = SR 304 Ave. Cost Per Beam = SR 34	Group of 3 Beams Cement 15 kg $=15\times\frac{10}{50}$ = SR 3 Sand 30 kg negligible Wire Mesh (13200 cmxcm) = SR 12 Total = SR 15 Ave. Cost Per Beam = SR 5
Average Man-hour Per Beam	• Surface preparation and mixing EP-CA $1\frac{1}{2}$ hr • Epoxy injection 1 hr • Grinding $\frac{1}{2}$ hr Total man-hour 3 hrs Cost = 3x30 = SR 90	• Roughing the surface (one side) $\frac{1}{4}$ hr • Steel plate preparation $\frac{1}{4}$ hr • Mixing and spreading the EP-CA $\frac{3}{4}$ hr Total man-hour $1\frac{1}{4}$hrs Cost = 1.25x20 =SR 25	• Roughing the surface (3 sides) $\frac{3}{4}$ hr • preparation of wire mesh and other materials $\frac{1}{4}$ hr • Mixing and placing FC layer 1 hr Total man-hours 2 hrs Cost = 2.0x20 = SR 40
Cost Per Beam	110+90 = SR 200	34+25 = SR 59	5+40 = SR 45

(Continue Table 4.11)

Item	Epoxy	Steel Plate Bonding	Ferrocement
Curing Time	12 hours, but it will reach full flexural strength and failure will occur in concrete when tested for flexural or shear strength after 7 days. The minimum curing of EP-CA is 6 hours before starting the Epoxy injection	12 hours, but it is recommended to be minimum of 7 days for full strength. During this period, some should be placed on the plate.	7 days but, for full strength, it requires 28 dys.
Equipments Used	Pins,injection gun,hoses, gloves,goggles,mask, scraper and cleaner.	Electrical mixer, gloves, goggles,mask,scraper and cleaner.	Trawler, mixer,balance, tray and scissors.
Availability of Materials	All materials should be imported.	Most of the materials to be imported.	Only wire mesh should be imported.
Need for Skilled Labors	There is a need for skilled labors.	There is no need for skilled labors.	No need for skilled labors.
Ave. Pult. for Original and Repaired Beams	51.377 KN 52.233 KN	50.770 KN 59.055 KN	48.427 KN 60.677 KN
%Ave. Increase in P Ultimate	2.8 %	16.0 %	25.3 %

steel plate bonding, then the ferrocement, while the epoxy repaired beams did not experience any increase in the load capacity. The beams strengthened by steel plate bonding method were stiffer than those of other methods of repair, while ductility of the beams was the highest for the epoxy repaired beams. The yielding load in the case of ferrocement beams is higher than in the epoxy repaired beams, while it is the highest for the steel plate bonded beams, because of the large stiffness of these beams.

From Figs. 4.91 through 4.93 of the load vs deflection curves of the ultimate deflection group for the ferrocement, epoxy, steel plate bonding and combined repair methods, the ductility of the epoxy is the highest followed by the combined method. Also, from the same figures, the initial stiffness of the combined method is the highest as well as the ultimate load of the ferrocement.

From table 4.6, for all methods, the cracking, yielding and tested loads were higher than those of the original beams. But for the combined repair method, the cracking load increased more than double because the cracks were filled with epoxy and the beam strengthened by a ferrocement layer which increased the stiffness of the damaged beams. Also, the yielding and ultimate loads increased very significantly in the ferrocement repair method, especially in the ultimate deflection beams where cracks were filled with mortar and gave more strength.

In case of ferrocement repair method, the initial stiffness decreased in according to the level of deflection used before repair such as, 10, 15 mm and ultimate deflections. This is because cracks were there even after repair and the stiffness decreased as the cracks became longer and wider. In other words, the 10 mm deflection beams gave the highest stiffness, followed by the 15 mm and ultimate deflection beams respectively. It was also the same for the initial stiffness in the steel plate bonding method which does not show a reduction in the stiffness. But for the epoxy method, the initial and reduced stiffnesses were the same after repair for all beams irrespective of their original deflections. This is due to the filling up of cracks by epoxy material, which restores the integrity of the beams.

The effect of levels of deflection

The moment of inertia of the unrepaired beams will be reduced when the beams crack and continue to decrease as the cracks progress due to the movement upward of the neutral axis. And since cracks were there, even after repair by ferrocement and plate bonding methods, the cracking load and stiffness were reduced as the original deflection increased. But the cracking load and stiffness were almost the same for all beams repaired by epoxy because cracks were filled by the epoxy material. Also, there is a large increase in the cracking load and initial stiffness for beams repaired by the combined method.

In case of the ultimate load and ultimate deflection there is no difference for the 10 and 15 mm deflection groups repaired by ferrocement and plate bonding methods, but they decreased for the ultimate group of the two methods, except that, the ultimate load of the ferrocement repair method was increased because the cracks were wide enough for the mortar to fill during repair which increased the ultimate load of the beams. But, the ultimate load was the same for all groups when using epoxy method due to filling of the cracks. In case of the combined method, the ultimate load was increased and the ultimate deflection decreased by almost 52 % as shown in table 4.7.

Percentage increase in the load and stiffness

From table 4.8, the following points can be concluded :

1- The increase in the cracking load is almost the same in case of epoxy repair method and it increased very significantly for the combined method. Also, the percentage increase in the cracking load decreased as the original deflection increases for the ferrocement repair method.

2- The initial stiffness reduced as the original deflection increased for all repair methods. But, it was increased very significantly in the case of the combined method.

3- There is a clear increase in the reduced stiffness in the ferrocement repair method compared to other methods.

4- The percentage increase of the ultimate load is approximately the same for the 10 and 15 mm deflection groups in each of the three repair methods, but the increase is less for all repair methods in case of ultimate deflection except for the ferrocement method where the increase was almost double that of the 10 and 15 mm deflection groups.

5- The ultimate deflection was decreased for all repair methods, but the decrease was very high in case of plate bonding method leading to a large reduction of ductility.

Number of cracks and crack width

From table 4.9 , one can conclude the following:

1- Beams repaired by ferrocement repair method showed very fine cracks and the average maximum crack width was reduced up to 75 % in case of ultimate deflection group, which is so because of the uniform distribution of the wire mesh in the matrix throughout the whole thickness of the ferrocement layer.

2- Using the epoxy repair method, cracks >0.3 mm were increased without any reduction in the average maximum crack width.

3- Using the combined method, a large single crack appeared in the flexural zone increasing the maximum crack width of this method of repair and failure occured due to the combination of the large crack width in the flexural zone and the crushing of the concrete in the compression zone.

Strength and ductility ratios

From figs. 4.94 and 4.95 and table 4.10 the following points can be drawn:

1- The strength ratio of the ferrocement repair method had been increased and was the highest for the ultimate deflection group, but there was not much increase in the ductility ratio when compared to the epoxy or the combined repair method.

2- There was no significant increase in the strength ratio (SR) using the epoxy repair method , but ductility was the highest and it decreased as the original deflection increased .

3- The strength ratio was the highest when using the steel plate bonding method, especially for the 10 and 15 mm deflection groups, but ductility was reduced very much

4- The strength ratio of the combined method increased by 11% and 75% of ductility was restored, which was the highest for the ultimate group beams in all methods.

Cost of materials and labours

As discussed in table 4.11, ferrocement is an easy method of repair as most of the materials needed are locally available and does not require skilled labours. Its cost is the least and it gives higher increase in the ultimate load for the ultimate deflection group compared to that of the other methods. Also, from the same table, the cost of the epoxy materials for one beam is three to four times higher than that of the bonded plate or the ferrocement repaired beams respectively with no significant increase in the ultimate load for the epoxy method. Also, there is a need to import the epoxy material used for epoxy and steel plate bonding methods. The cost of the combined method is the total cost of the epoxy and ferrocement methods with only 11.4 % increase in the ultimate load, which is higher than that of the epoxy. But this is less than that of the ferrocement due to the crushing of concrete in the compression zone, thus it can not withstand any more load than its capacity even after strengthening the beam in flexure. While, the combined method restored 75 % of its initial ductility, which can be adopted for durability requirements.

Chapter 5

CONCLUSIONS AND RECOMMENDATIONS

5.1 Conclusions

Based on the study, the following conclusions of the repair methods studied can be drawn :

Ferrocement repair method

(1) The mortar used to repair the reinforced concrete beams in the ferrocement method was very dense and has low permeablity than the original concrete which indicates that ferrocement can produce more durable structural members with high resistance to the ingression of harmful agents.

(2) Ferrocement is very easy to use and does not require advance equipments or material nor skilled labours. Besides, ferrocement materials are very cheap due to the availability of its basic ingredients locally.

(3) Using ferrocement as a repair method, most cracks were away from the original ones except in the case of 10 mm deflection where mortar can not fill the cracks.

(4) The ferrocement repair method gave high strength and reduced crack width, but it showed less ductility

compared to epoxy repair method.

(5) In casing the structural member with ferrocement layer will seal all cracks which will improve the durability of the structural member.

Epoxy repair method

(1) Most cracks were away from the original ones because epoxy is a liquid material and can fill the cracks very easily up to 0.1 mm crack width.

(2) There is no large increase in the ultimate load of the repaired beams because the load capacity does not increase by filling the cracks with epoxy.

(3) It requires specially trained technicians and its material and equipments have to be imported from abroad.

(4) The total cost of the material and labours is very high when calculated for the average cost of the beam and compared with other methods.

(5) Epoxy method gave the highest ductility because cracks were filled with epoxy. Thus, it can be used to close the cracks and restore the integrity of the structure.

(6) The disadvantage of this method is that, the new cracks will be more in number and larger in crack width.

Steel plate bonding method

[1) There is an average increase in the ultimate load equal to 16 % in case of ultimate group beams.

(2) Cost of materials and labours is reasonable, but epoxy materials which are needed for bonding the plate have to be imported from abroad.

(3) A sudden failure occured due to the separation of the plates, as a result of the shear failure in the case of ultimate group beams, without any warning.

(4) The bonded plate method gave less ductility and since failure occured suddenly, it may not be recommended without further study of the bond between the plate and concrete. Also, since cracks were there after strengthening, there is a large chance for corrosion of reinforcement to occur.

Combined method

(1) All cracks were away from the original ones, because cracks were filled with epoxy before applying ferrocement.

(2) Cracking load has been increased by 117 % of the original one and the initial stiffness by 52 %.

(3) The increase in the ultimate load after repair is 11.4%.

This increase is not very considerable when compared to ferrocement method alone or to its high cost and requirements of curing time which were equal to the total of both the epoxy and ferrocement methods.

(4) The combined method gave high strength and ductility which can be adopted for durability requirements, because major cracks were filled with epoxy and minor cracks covered by ferrocement repair. The only problem is that, it showed larger crack width than other methods, when tested up to ultimate.

5.2 Recommendations

For further study, the following points are suggested :

1- To study the effect of thermal gradients on the epoxy materials using different resins to account for the actual situation of temperature, humidity, etc., which may lead to major problems.

2- To study the increase in the ultimate load capacity due to the effect of increasing the number of wire mesh layers, in case of ferrocement. Also, to fill the cracks with epoxy and use ferrocement from bottom side only.

3- To study the improvement in the ductility of the plate bonded method of repair due to the effect of filling the cracks by epoxy or even simple mortar grouting before bonding the steel plate to the beam.

4- It is also recommended to investigate the effect of using different thicknesses of plate and different types of resins in case of steel plate bonding.

5- To carry out more investigation on the bond between the steel plate and concrete, and if possible of using new techniques rather than epoxy to avoid the sudden separation of the steel plate and the effect of thermal gradients on the epoxy materials.

6- To carry out more study on the effect of repair in handling the durability problems.

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APPENDIX-A

(1) Final Design of the Beam

All symbols are explained in detail in Ref. [50].

$b = 150$ mm (6") $f'_c = 31$ MPa (4,500 psi)

$d = 120$ mm (4.8") $f_y = 414$ MPa (60,000 psi)

For rectangular beam $M_n(\text{required}) = R_u\, b\, d^2$

$\rho_{max.} = 0.0233$ for $f'_c = 31$ MPa and $f_y = 414$ MPa

Adopt $\rho = 0.35\, \rho_{max.} = 0.00816 >$ minimum.

$R_u = 3.162$ (from tables)

$M_n(\text{required}) = R_u\, b\, d^2 = 3.162*150*120^2 = 6.83$ kN-m

$A_s(\text{required}) = 0.00816*150*120 = 146.8\ \text{mm}^2$

Choose 2 φ 10 mm$\rightarrow A_s = 2(10)^2*\pi/4 = 157.1\ \text{mm}^2 >$ required.

Check ultimate moment capacity

$$M_n = A_s\, f_y\, (d-a/2)$$

$$T = C \rightarrow a = \frac{A_s\, f_y}{.85 f'_c b}$$

$$A_s = 2\ \varphi\ 10\ \text{mm} = 2*(10)^2*\pi/4 = 157.1\ \text{mm}^2$$

$$\therefore a = \frac{157.1*414}{.85*31*150} = 16.45\ \text{mm}$$

Ignoring top reinforcement of the beam and taking moment around C,

$$M_n = 157.1*414(120-8.227)*10^{-6} = 7.27\ \text{kN-m}$$

$$M_n = \frac{P_n\ L}{6} + \frac{W_D L^2}{8}$$

$$M_n(DL) = \frac{M_u}{0.9} = \frac{W_D L^2 * 1.4}{0.9*8}$$

$$W_D = 24 kN/m^3 * 0.150^2 m^2 = 0.54\ kN/m$$

$$M_n(DL) = \frac{0.5475 kN/m * 1.2 m^2 * 1.4}{8*.9} = 0.151\ kN\text{-}m$$

$M_n = 7.27$ kN-m > required.

$$M_n = 7.27 = 0.151 + \frac{P_n * 1.2}{6} \rightarrow P_n = 35.6\ kN \leftarrow P_n(\text{flexural})$$

Design of shear reinforcement

$$V_n = P_n/2 = \frac{35.6}{2} = 17.8\ kN$$

$$V_c = \frac{1}{6}\sqrt{f'_c}\ b_w d$$

$$V_c = \frac{1}{6}\sqrt{31} * 1000 * 0.15 * 0.12 = 16.7\ kN$$

$$V_s = V_n - V_c = 17.8 - 16.7 = 1.1\ kN$$

max. spacing = $d/2 \rightarrow S = \frac{120}{2} = 60$ mm

$$A_v(\text{required}) = \frac{S * V_s}{f_y * d}$$

$$A_v(\text{required}) = \frac{60\ mm * 1.1\ kN}{414\ MPa * 120\ mm}$$

$$= 0.996\ mm^2$$

$$A_v = (6)^2 * \pi/4 = 0.28.27\ mm^2$$

The detail of the tension and shear reinforcement of the beam is shown in Fig. 3.1.

Check shear load capacity

$V_n = P_n/2 = \frac{35.6}{2} = 17.8$ kN

$V_c = \frac{1}{6}\sqrt{f'_c}\, b_w d$

$V_c = \frac{1}{6}\sqrt{31}*1000*0.15*0.12 = 16.7$ kN

max. spacing = $d/2 \rightarrow S = \frac{120}{2} = 60$ mm

$A_v = (6)^2*\Pi/4 = 0.28.27$ mm^2

$V_s = \frac{A_v f_y d}{S} = \frac{2.0*28.27*414*120}{60}$

$= 46.82$ kN

$V_c + V_s = 16.7 + 46.82 = 63.52$ kN

$\therefore V_n = V_c + V_s = 63.52$ kN

$\therefore P_n = V_n*2 = 127.0$ kN

(from shear point of view)

(2) Plate Bonded Beam Analysis

Using the **working stress method** :

$nA_{s_1} = 7.5 * 157 = 1178\ mm^2$, and

$nA_{s_2} = 7.5 * 1.5 * 100 = 1125\ mm^2$.

where n= modular ratio = 7.5 for $f'_c = 31$ MPa

From the stress distribution of the transformed section in Fig. 5.1,

$$\frac{150 * x^2}{2} = 1178(120-x) + 1125(152-x)$$

$$x^2 = 1884.8 - 15.71\ x + 2280.0 - 1500\ x$$

$$x^2 = 4164.8 - 30.71\ x$$

$$x^2 + 30.71\ x - 4164.8 = 0.0$$

$$x = 51.0\ mm$$

The maximum load obtained from the test was equals to 65.0 kN, and the maximum moment under two points load equals to 13.088 kN-m.

Also, from Fig. 5.1, the following are true : $\dfrac{f_c}{51} = \dfrac{f_{s_1}/n}{69}$

or

$f_{s_1} = 10.148\ f'_c$ and

$$\frac{f_c}{51} = \frac{f_{s_2}/n}{101}$$

or

$f_{s_2} = 14.850\ f'_c$

$a_1 = (120\text{-}x/3) = 103$ mm and

$a_2 = (152.25\text{-}x/3) = 135.25$ mm.

$T_1 = A_{s_1} * f_{s_1} = 157*1.353\ f_c$

$T_1 = 1593.91\ f_c$

$T_2 = A_{s_2} * f_{s_2} = 150*1.980\ f_c$

$T_2 = 2227.50\ f_c$

$M = T_1 * a_1 + T_2 * a_2$

or $1593.91 f_c * 103 + 2227.5 f_c * 135.25 = 13088000$ N-mm

$f_c = 28.12\ N/mm^2$

$T_1 = 1593.91*28.12 = 44.820$ kN

$T_2 = 2227.50*28.12 = 62.636$ kN

$C = \frac{1}{2} f_c bx$

$C = 0.5*28.12*150*51/1000 = 107.557$ kN

$f_{s_1} = 10.148 f_c = 10.148*28.12 = 285.36$ MPa

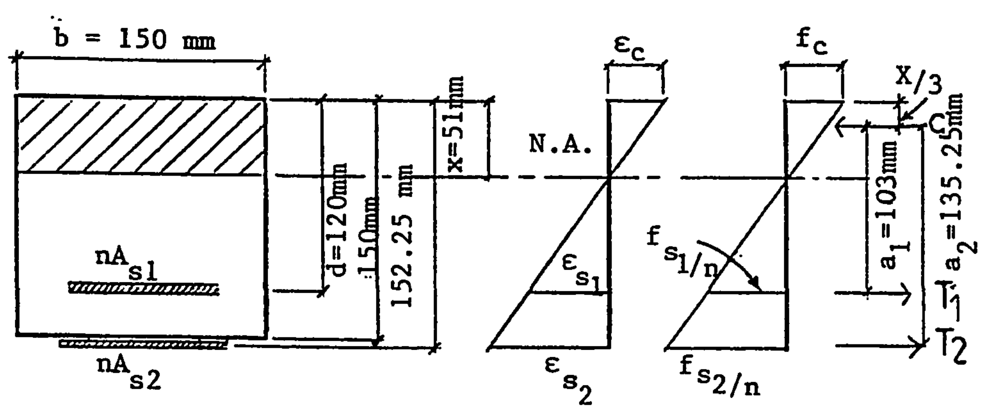
$f_{s_2} = 14.850 f_c = 14.850*28.12 = 417.58$ MPa

$$\varepsilon_{s_1} = \frac{f_{s_1}}{E_s}$$

$$\varepsilon_{s_1} = \frac{285.36}{200000.0} = 0.00143 < \varepsilon_y = 0.00207$$

$$\varepsilon_{s_2} = \frac{f_{s_2}}{E_s}$$

$$\varepsilon_{s_2} = \frac{417.58}{200000.0} = 0.00209 > \varepsilon_y = 0.00134 \text{ (steel plate yield)}$$



(a) Transformed Section (b) Strain (c) Stress

Fig. 5.1 Stresses of transformed bonded plate beam.