

STRENGTHENING OF SHEAR DEFICIENT REINFORCED CONCRETE BEAMS USING POLYMER MODIFIED FERROCEMENT (PMF)

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Submitted in fulfilment of the requirement
for the award of the degree of*

DOCTOR OF PHILOSOPHY IN CIVIL ENGINEERING

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January-2020**

Dedicated
to
My Grandparents

CERTIFICATE

Certified that the work presented in the thesis entitled “**STRENGTHENING OF SHEAR DEFICIENT REINFORCED CONCRETE BEAMS USING POLYMER MODIFIED FERROCEMENT (PMF)**” which is being submitted by Mr. Rajinder Ghai (Registration No. 951002004) in fulfilment of the requirements for the award of the degree of Doctor of Philosophy in Civil Engineering of Thapar Institute of Engineering and Technology, (Deemed to be University), Patiala, is an authentic record of the candidate’s own independent research work carried out by him under our supervision and guidance. The matter embodied in this thesis has not been submitted in part or full to any other university or institute for the award of any other degree.



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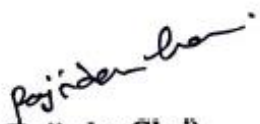


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DECLARATION

I hereby certify that the work which is being presented in this thesis, entitled **“STRENGTHENING OF SHEAR DEFICIENT REINFORCED CONCRETE BEAMS USING POLYMER MODIFIED FERROCEMENT (PMF)”**, in fulfilment of the requirements for the award of degree of ‘Doctor of Philosophy’ in Civil Engineering submitted to Civil Engineering Department of Thapar Institute of Engineering and Technology, (Deemed to be University), Patiala is an authentic record of my own work carried out under the supervision of Dr. Prem Pal Bansal and Dr. Maneek Kumar, and wherever other research work is referred it is indexed in the list of references.

The matter presented in this thesis has not been submitted in part or full to any other university or institute for the award of any degree in India or abroad.


(Rajinder Ghai)

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(Rajinder Ghai)

ABSTRACT

There is a common phenomenon of shear failure in RCC beams, especially in old buildings and bridges. Any possible strengthening of such beams needs to be explored that could strengthen and make them fit to fulfil the strength criteria. The present research has been carried out to determine the performance of polymer modified ferrocement (PMF) used to strengthen the pre-damaged RCC beams.

Ferrocement has become a universally acceptable thin composite material in the field of construction technology. This is primarily due to its better strength to thickness ratio. In the present research work, the polymer modified ferrocement has been used to strengthen the beams as it had been come of age in the last two decades or so and has found that the conventional ferrocement made with cement mortar matrix does not exhibit significant durability. Hence, two types of polymers were chosen to add in the mortar matrix to improve the properties of traditional ferrocement. As a pilot study, the effects of addition of Vinyl-Acetate-Ethylene (VAE) polymer and Styrene-Butadiene-Rubber latex (SBR) polymer on the properties of mortar and ferrocement have been studied. The polymer-modified mortar (PMM) matrices were prepared by adding different percentages of polymers varied from 0% to 20%. Cement-sand ratio for mortar was fixed as 1: 2 and water-cement ratios have been also worked out for a specified flow value of $105\pm 5\%$. The compressive strength, workability, flexural strength, air content and Young's modulus of elasticity of PMMs was worked out by conducting the various tests. Test results showed that the mortars modified with polymers showed better performance performed as compared to un-modified mortars.

Further, the polymer modified ferrocement (PMF) elements were cast with different volume fraction of reinforcement consisting of 2 and 3 layers of wire mesh. Same polymer modified mortar composition as of pilot study, was used to cast the PMF specimens. The overall test results reflect that the performance of SBR modified ferrocement is better as compared to VAE modified ferrocement. Flexural strength, tensile strength and elongation of three-layered PMF on addition of 15% SBR, showed an enhancement by 48.04%, 39.41% and 17.59% respectively, as compared to conventional ferrocement. This is attributable to the development of a polymeric layer, in which the polymer elements merge, a polymeric compound is propagated and an interlocked mesh composition is formed. Hence, based upon these test results, the 15% SBR modified mortar in three-layered wire mesh ferrocement elements was used to strengthen the RCC beam specimens for better durability and strength.

Forty-eight shear designed and shear deficient real size beams have been cast and tested to achieve the objective of present work. Ultimate shear load carrying capacity of control beams were found for shear-span (a/d) ratios of 1.0 and 3.0. The shear designed beams contained stirrups spacing of 150mm c/c and to make the beams deficient in shear, these stirrups were spaced at 450mm c/c. Based upon the test results of control beams, three zones (elastic, elasto-plastic and plastic zones) were located on the load v/s deflection curve. The pre-damaging load for beams has been worked out corresponding to the load level of 45%, 75%, and 95% of ultimate load, respectively. After that the sets of remaining beams were loaded with different pre-determined damage levels and then strengthened with 20mm thick PMF. U-shaped jacketing methodology was made to strengthen the beams for a length equal to '2d', irrespective of the a/d ratio, with distance 'd' on each side from load point. The ultimate load-carrying capacity of the strengthened beams was found out by conducting the re-load test for a/d ratio 1 and 3. The results indicate that the PMF strengthened beams show full restoration and also enhancement of ultimate shear load carrying capacity by 5.90% to 12.03%. The ductility and stiffness of the strengthened beams were improved; hence the corresponding deflections were prolonged. On the other hand, the cracking pattern of PMF strengthened beams was also improved remarkably. All the test results have been discussed and analysed in term of loads, deflections, effect of spacing of stirrups, effect of shear span ratios, modes of failure and cracking behaviour. Finite element modelling of control as well as strengthened beams was made on ATENA-3D software to validate the test results. The FE model also shows similar type of behaviour to experimentally tested control and strengthened beams with an error of $\pm 8\%$ for both a/d ratios of 1 and 3.

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1.1 GENERAL

Reinforced cement concrete (RCC) material is being used in the construction industry for more than a period of 100 years now. Concrete used universally and plays an important role in the development of modern society, may it be road pavements or high rise building structures. A large amount of concrete is used worldwide for the custom-made jobs by using local materials. The durability of concrete is a major issue even being the oldest construction material. *IS 456:2000* code also considers the importance of durability for concrete structures. Generally, it is believed that concrete requires very less repair and a durable material. However, this concept has changed now, and different researches have reflected that the concrete does not perform well as expected in different situations. Deterioration and cracking of concrete occur due to many factors like; construction errors, design errors, scaling and disintegration, reinforcement corrosion, spalling and pop-outs of concrete.

Furthermore, the structural elements of RCC structures reveal distress, even before achieving their designed life. At the initial stage, the RCC structures deteriorate at a prolonged rate due to variation in temperature, additional loading, other physical activities and chemical actions. In addition to the corrosion, sulphate attack, carbonation, shrinkage, thermal expansion and contraction, cracking and settlement of foundation are other various factors which reduces the strength of existing structures. At later stage, if proper care is not provided, this deterioration speeds up at a very high rate and reduces the designed service life of the structures. Moreover, concrete is weak in tension, and the chances of partial failure of concrete structures might be there as well, even when small deterioration takes place. Drying-out of the cement matrix due to long-term ageing effect would also results in strength reduction of the concrete. Dry and wet shrinkage cracks may further lead to failure of the structure. Sometimes, during construction work, there could be other factors which might have been ignored and can be responsible for the deterioration of these structures. If proper repair and maintenance is not done on time, this could leads to lesser designed life and increased cost of repair.

Thus, proper repair and maintenance are the imperatives to keep these structures in serviceable conditions for which purpose they are constructed and this also helps to enhancing the lifespan of these assets. As per a study was done by *Hing (2007)*, it was

concluded that 32% of concrete problems are due to improper curing and 20% problems are due to earlier removal of formwork owing to the time constraint. Overloading and shrinkage are also the reasons which cause problems, and contribute around 33%. Generally, the overloading happens during construction stage because of the storage of material at different floor levels.

The deterioration of RCC structures, which take place with the passage of time, needs special attention and a strengthening approach to restore their original strength and designed life. There's a need to investigate the causes of distress and types of unserviceable structures and a suitable repairing approach to bring them in serviceable condition again. To repair such type of partially-damaged structures is termed as strengthening. Many researchers have worked on the development of advance strengthening materials for strengthening, repairing and retrofitting of those structures which shows lesser durability or wear and tear with time. Hence, the strengthening can be considered as a cost effective alternative to avoid the replacement of existing deteriorated structures or structural elements and is often times the only feasible solution.

1.2 RCC BEAM COMPONENT

The beam components are generally designed for vertical loads along with horizontal loads which are produced due to earthquake and wind forces. These loads induce different type of stresses in the beams such as; compressive, tensile, flexural, shear and torsional stresses. The vertical gravity load induced tensile stresses in the bottom part and compressive stresses in the top of beams. This condition is vice-versa at the edge where the tensile stresses are produced at the top of the beams. The shear forces always remain throughout the whole length of the beam and maximum near the edges, under gravity as well as under horizontal (lateral) seismic or wind loads. Seismic activities also induced shear and flexural stresses in the structural frame due to its inertia. So, it is essential that a structure should behave elastically as well as in a ductile manner without any failure. Hence, if the beams are found deficient in shear, there are the chances of partial or a complete failure of the whole structure. Thus, a possible strengthening technique is required which would ensure sustenance of the present loading over the structure without failure and preserve the design lifespan of the structure.

1.2.1 Shear Behaviour of RCC Beams

The behaviour of RCC beams in shear depends on many factors, out of which shear-span to effective depth ratio (a/d ratio), plays a crucial role and is the governing factor in

design for shear. The beams can broadly be classified as deep beams and slender/short beams, based upon their behaviour and failure mode conditions (*ASCE-ACI Task Committee 426 report, 1973*).

MacGregor (1988) classified the beams as per their shear span to effective depth ratios. In case of the beam having $a/d < 1$ (very short), the beams fail in shear due to inclined cracks along the line which joins the support and loading point. Further, the beams having a/d ratio 1 to 3 can take additional loads. This attributes to internal redistribution of the loads and arch action. These beams may fail due to the crushing of concrete or due to shear-tension. In this case, horizontal cracks appeared in the bending zone near the bottom of the beam which further attributes to a loss of bond strength. As the a/d ratio increases, the shear failure transferred into flexural failure. In the case of slender beams (a/d ratio between 2.5 to 6) the shear-compression failure takes place due to the crushing of concrete near the loading point. The equilibrium of inclined cracks is generally disturbed in this case. The beam tends to fail in flexure in case of a/d ratio more than 6 (very slender). Flexural failure take place in these cases as the shear span is large and no shear crack is formed before flexure failure.

1.2.2 Different Shear Failure Modes

Shear failure of RCC beams may occur if the a/d ratio is small or if the shear reinforcement is not adequate. The failure due to shear is sudden in nature as compared to flexural failure. Following are the different shear failure modes that may occur in RCC beams: -

a. Diagonal tension failure

The diagonal cracks appear near the support of the beam approximately at 45° to the horizontal in this failure mode. These types of cracks occur where the shear force is as large as compared to the bending moment.

b. Flexure shear failure

In this mode of failure, the cracks usually appear at 90° with the horizontal. Flexural shear failure occurs where the bending moment is comparatively large with respect to the shear forces.

c. Diagonal compression failure

Diagonal compression failure occurs due to the crushing of concrete. This type of failure occurs within the compression zone. Herein, the inclined cracks form near the loading point and penetrate in that zone. This type of failure mainly occurs due to the excess percentage of reinforcement in the shear zone.

1.3 MECHANISM OF SHEAR RESISTANCE

Ritter (1899) initially enunciated the truss analogy model to establish the mechanism of shear resistance. This truss model is very useful for envisaging the concrete forces (diagonal compression), stirrups forces (tension), which further simplifies the method and concept to design the RCC beams in shear (*Wang and Salmon, 1992*). According to this truss model at 45° the yielding of stirrups represents the shear capacity corresponding to the shear stresses.

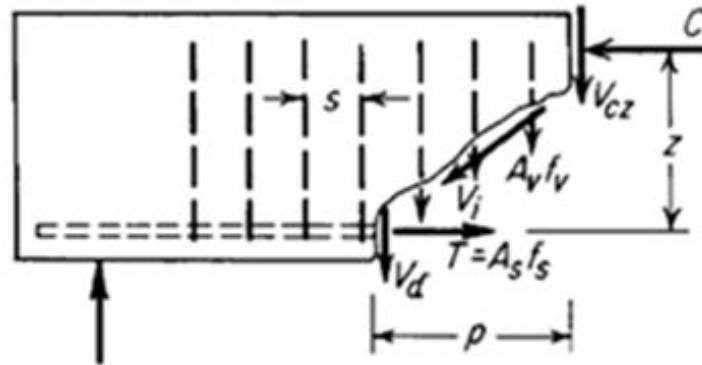


Fig. 1.1: General Mechanism of Shear Resistance (*Wang and Salmon, 1992*)

Generally, the main factors which are responsible for the mechanism of shear resistance of RCC beams (refer Figure 1.1) are detailed below: -

- i. Size of the beam (a/d ratio)
- ii. Concrete grade (concrete shear stress in compression zone)
- iii. Amount of shear reinforcement (spacing and size of stirrups)
- iv. Dowel action;

However, it is very complicated to calculate the amount of shear resistant of concrete only. Consequently, due to this difficulty, different researches had been done and different models have been proposed to calculate the cracking shear force of concrete beams with no shear reinforcement (*Niwa et al., 1986; Okamura et al., 1980*). Hence, *IS 456:2000* gives mathematical equations to calculate the amount and spacing of shear reinforcement as below:

-

$$V_{us} = \frac{0.87 f_y A_{sv} d}{s_v} \quad \dots (1.1)$$

IS 456:2000 gives the shear strength value of concrete mixes ' τ_c ' on the basis of the amount of longitudinal reinforcement and does not consider the effect of a/d ratio and shear reinforcement.

From above, it is concluded that the shear resistance mechanism of RCC beam is not merely the sum of shear resistance by concrete ($V_c = \tau_c bd$) and stirrups (V_{us}), but it should be the interaction between them which further depends upon the amount of longitudinal steel. Hence, concrete grade, shear stirrups spacing and the percentage of longitudinal steel are the major factors responsible to resist the shear forces induced in RCC beams.

1.4 PARAMETERS INFLUENCING THE SHEAR RESISTANCE OF BEAMS

In this section, the fundamental background is discussed related to shear behaviour of RCC beams. According to this discussion, a/d ratios have been fixed for this experimental program.

1.4.1 Factors Affecting the Shear Behaviour of RCC Beams

The shear failure of beams is catastrophic, brittle and contradictory to bending failure as the ductility governs the design. Hence, shear strength dominates the design of RCC beams, as shear failure may occur before the flexural failure. *MacGregor, 1988* explained that the behaviour and design of shear deficient concrete beams and beams having on shear reinforcement is primarily depends on the a/d ratio (size effect), compressive strength of the concrete, amount of longitudinal reinforcement and other axial forces.

1.4.2 Factors Affecting the Shear Capacity of RCC Beams

The cracking load governs the shear capacity of RCC beams having deficient or no shear reinforcement. However, in case of shear efficient beams, the beams tend to resist more shear load even after the cracking due to tensile stresses of concrete and it was *Collins et al. (1996)* who explored this fact of increased capacity. The beams without shear reinforcement and having 0.75 to 2.5% of flexural reinforcement were observed to have failed in shear. The beams having small amount of flexural reinforcement ratio are tend to fail in flexural even before attaining the maximum shear load level. Further, the load-carrying capacity of beams in shear also increased due to the axial compression produced due to the application of loads. However, this fact had been opposed by *Wight and MacGregor (2005)*, who stated that the shear capacity of beams decreased due to axial tension.

1.4.3 Effect of Shear Reinforcements

The failure RCC beam in flexure is not sudden and it shows some initial warning of distress before the failure takes place. Opposite to this, the failure due to shear is sudden, devastating and catastrophic. Hence, shear reinforcement is provided to overcome any such

kind of sudden failure. The shear reinforcement prevents the failure due to shear and transfers it to a flexure failure. The spacing of shear reinforcement is decided based upon the shear forces acting on the beam. The shear stirrups come into play only after the development of diagonal cracks either in their vicinity or while crossing them.

The stirrups resist the shear stresses and further control the crack propagation and penetration. *Nilson et al. (2004)* has also explained that the stirrups oppose the widening of cracks within the concrete and are responsible for the aggregates interlocking. Additionally, the stirrups tied the longitudinal bars which further help to confine the splitting effect of concrete and increase the cracking resistance. The resistance to shear offered by concrete and stirrups is mainly dependent on the parameters those were adopted during the design procedure. The stirrups with sufficient spacing help to confine the concrete within its region and thus increase the resistance of beams in shear. The shear reinforcement conserved the concrete contribution to resisting shear stresses which further allows the development of additional shear capacity.

1.5 SHEAR FAILURE OF RCC BEAMS

The RCC beams fails in shear if the induced shear stresses exceed the shear resistance capacity of beams. The failure due to shear is brittle in nature and may occur without any warning signs. The study related to the behaviour of RCC beams in shear has been going on since a century. Initially, it was the general view of researchers that the shear problem is not complex in nature and can be resolved in easy manners. However, after the incident of the year 1955 at Wilkins Air Force Depot, Ohio, where the beams failed in shear (Plate 1.1) (*Anderson, 1957; Cuenca, 2014*), the researchers realised that traditional design of beams in shear needs revision (*Dhir and Jones, 1999*).



Plate 1.1: Failure of Beams in Shear at Wilkins Air Force Depot, Ohio (*Anderson, 1957*)

There are many other examples of failure due to shear, and one of the great examples is during the Hanshin earthquake in the year 1995 (*JSCE, 1995*). As reported by JSCE, many

of the viaducts rigid framed structures were damaged and collapsed as shown in Plate 1.2. After the detailed study, it was found and reported that the amount of shear reinforcement used for these structures was less. Hence, the question was raised by many researchers that the shear strength of these structures was miscalculated?



Plate 1.2: Failure of Viaduct Rigid Frame Structure during the Hanshin Earthquake
(JSCE, 1995)

Another major example of shear failure was the collapse of the floor slab of a car parking at Wolverhampton, England. This car parking building was built in the year 1965. The top floor of this building weighing around 120 tonnes collapsed on 20th March 1997. It was identified that punching shear is the reason for this collapse (Plate 1.3). Degradation of concrete with time is also the other reason for the progression of collapse. The rigorous analysis was also made by Finite Element Analysis using ANSYS software by simulating the actual site conditions (Wood, 1997).



Plate 1.3: Floor Slab Collapsed at Wolverhampton (Wood, 1997)

As discussed earlier and also reported by *Talbot (1909)*, the shear strength of beams depends upon many factors like; beam stiffness, size of beam and dowel action due to longitudinal reinforcement. Concrete itself is also to be considered to resist shear if there is no web reinforcement. In the last four decades, the researchers have worked on the globally accepted fundamental design related to shear and many theories were established which describe the shear behaviour of RCC beams. However, it is still not exactly clear regarding the specific contribution of each element to resist shear force.

1.6 NEED FOR STRENGTHENING

The strengthening of concrete structures is a specialised field and a very important component of the structures maintenance industry. With the passage of time, many of the existing RCC structures deteriorates due to increase in service loads, corrosion of reinforcement and poor ductile detailing, which further results in loss of strength, cracking and spalling of the structural components. Such structural elements need special attention and must be retrofitted using suitable strengthening techniques to restore their strength and design life. Many researchers have worked on the development of various materials and techniques for repairing, retrofitting and strengthening of such structural elements. The selection of a particular strengthening material and method is governed by the cause, type, and nature of distress which is responsible for the degradation of structural components and needs to be addressed.

Moreover, worldwide the existing structures were designed and constructed by considering old codal provisions and can be considered as unsafe as per new design codal provisions. These structures cannot be replaced due to financial involvement and the importance of the building. Hence, strengthening has also become an acceptable solution to make these structures serviceable by enhancing their load-carrying capacity. Premature deterioration of buildings causes decay of the infrastructure, which leads to the exploration of new and advanced technologies for the repairing and strengthening.

Sometimes, proper design standards are not followed, and the construction work is accelerated to meet the desired goals on time. Such factors also contribute to the deterioration of the concrete structure along with the factors mentioned above. The information of all these factors is considered important to decide a particular solution to strengthen the damaged concrete structure to meet with current loading requirements.

The RCC structural elements might fail basically due to shear and flexure stresses. Most of the existing structures were designed and detailed for gravity (dead and live) loads.

These structures might be under-designed for lateral loads which could be smaller than the values specified by the standard codes. These lateral loads further produce shear stresses in the structural elements, and hence there is a common problem of shear failure in RCC beams. There are many factors such as concrete grade, detailing and amount of reinforcement, loading conditions etc. which are responsible for strength to the RCC beams in shear. Many of the existing RCC framed structures are not designed or constructed following the specific seismic codal provisions, and some of those structures might be located in high seismic zones. The susceptibility of the inadequate design reflects the seismic risk to the community, and due to this cause, there is a necessity to strengthen the existing structures and to match with the present seismic code provisions as well. Moreover, deterioration of these structures can take place with the passage of time, which needs special attention and a strengthening approach to make them fit for present serviceability conditions.

All RCC elements are designed by providing suitable reinforcement detailing so that they tend to fail in a ductile manner. During an earthquake, a sudden catastrophic failure can occur due to increased shear loads (*Rafeeqi et al., 2005*). The shear deficiencies of RCC beams can pop up due to many reasons such as; deficient shear reinforcement, increased service loads, reduction in the steel area due to corrosion, faulty design and poor workmanship. Shear strength of many existing structures might be deficient for present needs and require strengthening to sustain and satisfy current codal requirements. The engineers have experienced that shear failure of RCC beams, especially in older or heritage structures, buildings, and bridges is a serious problem which necessitates dismantling and replacement of the structures. Hence, there is a need for developing an economical strengthening solution, which helps to restore the desired strength of beams in shear.

The primary objective to strengthen the beams in shear is to eliminate the sudden brittle failure and to prolong the deformations due to lateral loads without failure during severe seismic demand. Hence, it is a need of the hour to produce a cost-effective and practical strengthening solution for the existing RCC beams in shear. It was found that ferrocement is a very effective material for strengthening the structural elements and to improve the performance of structures under severe loading conditions. The use of ferrocement is a cost-effective and simple approach to enhance the load-carrying ability of the structure. Moreover, the polymer-modified ferrocement is found to be more effective for strengthening of structures. Hence, in the present work, a sincere effort has been made by considering the strengthening of RCC beams in shear as the primary objective. The step by step procedure for strengthening is explained in subsequent sub-sections.

1.6.1 Evaluation of the Existing Structure

First and foremost, the present condition of a structure needs to be evaluated before deciding any strengthening approach. The conditions of the structure are assessed in terms of; disintegration of structure, the magnitude of deterioration, the area of distress, dampness, corrosion, the location of cracks, the dimension of cracks and the honeycombing etc. The structure is also needed to be monitored and inspected regularly. A record of such inspections is also one of the critical factors in deciding for any repair, retrofitting and strengthening approach. After detailed inspection and evaluation of the existing conditions of the structure, if any deficiency or risk is found then an immediate strengthening of the structure has to be carried out. A record of such type of repair and strengthening work for future references is also be maintained.

1.6.2 Selection of Strengthening Approach and Material

After detailed evaluation of the existing structure, it is found that the strengthening of partially damaged structure is needed to restore its designed life. Based upon the present needs and serviceability conditions; a suitable strengthening method is adopted. There are many options available to resolve a particular problem, but the choice of a specific approach depends upon the actual structural condition, requirements and compatibility of strengthening material with the existing structure.

There are some major factors which help in deciding the use of a specific approach for strengthening a deteriorated structure. The chosen method should help in detecting the cracks, to restore the structural integrity and original shape, control further deterioration, meet the present loading conditions and serviceability requirements, restore water integrity and aesthetic requirements. Hence, the strengthening approach should be so chosen that it is readily available in the local market, easy to apply and should be economical (*Kosednar and Mailvaganam, 2005*).

Besides the strengthening approach, the strengthening material also plays an important role. The selection of material also depends upon the; material composition, method of application, bonding, finishing, the coefficient of thermal expansion, compatibility with the existing material and its fresh and hardened properties (*Baker et al., 1992; Kosednar and Mailvaganam, 2005*). Hence, it is important to decide wisely before selecting a particular approach and material for the strengthening of a structural element or the structure as a whole.

1.6.3 Surface Preparation and Application of Strengthening Material

The efficiency of any strengthening material is dependent upon the surface preparation of the existing structure. Hence, after assessment of the existing conditions of a deteriorated structure and selecting a suitable strengthening approach and material, the existing surface of structure elements needs to be prepared before the application of strengthening material. The surface should be uncontaminated and free from any loose dirt. The loose dirt can be removed effectively with the oil-free compressed air or with the vacuum cleaning (*Kuhlmann, 1990*). Cleaning and surface preparation are two different terms needs to be understood. Cleaning refers to the process of removing dust, laitance or dirt by any means as discussed above, whereas, on the other hand, the process of removing loose layer of concrete involves the application of a bonding agent, termed as surface preparation.

Hence, the surface should be cleaned and prepared properly before the application of any repairing or strengthening material. For effective results, the strengthening material must be applied according to the guidelines and specifications.

1.7 VARIOUS TECHNIQUES OF STRENGTHENING

The RCC beams need strengthening when the existing reinforcement either in shear or bending is not sufficient for the present needs or the applied load on the specific structural component has increased (*Ong et al., 1992*). Following are the different solutions which are generally used for strengthening of the various structural elements: -

a. Addition of reinforcement bars

This strengthening approach is used when the amount of reinforcement is less than that required for applied loads and stresses. Further, this method is also used when there is corrosion of reinforcement.

b. Enlargement of cross-section

When the concrete and steel both are not capable of resisting the applied loads then enlargement of the cross-section could be a viable solution. This method is most suitable when there are chances of increase applied loads on the structure.

c. Shortening of span

Shortening of span is also one of the methods which can be used for the safety of the structure. The span shortening can be done by installing additional supports under the beams if they can resist the moment after redistribution. Appropriate arrangements like; the casting of RCC columns, placing steel columns etc. can be used for span shortening as per site requirements.

d. Sprayed concrete

This is most commonly used method and oldest technique for repairing and strengthening of concrete structures. This technique is chosen when the reinforcing bars are exposed to the atmosphere, or the concrete cover is damaged.

e. Steel plate bonding

The steel plates of the required thickness and size are designed and bonded to the members to resist excess applied moment or shear stress. These are attached to the existing structural members by efficient bonding.

f. Ferrocement

This is one of the most popular and economical technique for the strengthening of structures. Ferrocement laminates are used on the soffit of beam section for flexural strengthening and used on the three exposed faces for shear strengthening. Preparation of the surface is one of the key factors which should be ensured before applying ferrocement.

g. Strengthening by using Fiber Reinforced Polymer (FRP)

Fiber Reinforced Polymer (FRP) is also an effective solution for the strengthening of reinforced concrete structures because of their high strength to weight ratio. FRP materials have superior properties as compared to steel in terms of fatigue and tensile strength. Glass fiber-reinforced polymers (GFRP) and Carbon fiber-reinforced polymers (CFRP) are generally used under a wide range of temperatures for the strengthening of structures.

1.8 SHORTCOMINGS OF AVAILABLE TECHNIQUES

Rehabilitation industry has grown at a very fast rate in the recent years. The use of various advanced materials and techniques like; ferrocement, fibre reinforcement polymers (FRP), glass fibre reinforced polymers (GFRP), carbon fibre reinforced polymers (CFRP), and steel plate jacketing has increased, for strengthening and retrofitting of concrete structures. These specific materials have excellent properties such as lightweight, high strength; can be applied on irregular or curved surface and have abilities to resist corrosion. Researchers have explored the effect of various advanced composite bonding materials as well as their orientation on flexure and shear strength properties of retrofitted beams (*Khalifa, 1998; Sheikh et al., 2002; Bansal et al., 2006; Bansal et al., 2008; Jumaat and Alam, 2008; Wang and Hsu, 2009; Kothandaraman and Vasudevan, 2010; Kim and Shin, 2011; Obaidat et al., 2011; Ahmad et al., 2012; Ronagh and Eslami, 2012; Sheela and Geetha, 2012; Singh, 2013; Alferjani et al., 2014; Ezz-Eldeen, 2015*). Many of these materials have shown some flaws or shortcomings in term of strength, cost, availability, and applicability. The

conventional strengthening methods such as enlargement of member size, steel plate bonding, internal or external post-tensioning are very expensive and require extensive equipment, skilled labour, time and special resources.

Out of the available material options, the use of ferrocement has gained popularity and it has evolved as a major structural material for retrofitting, strengthening as well for new construction purposes, especially, in earthquake-prone areas because of its excellent toughness, ductility, availability, and other properties. Researchers have come to the conclusion that ferrocement is quite compatible with the existing concrete structures when used as strengthening material, apart from this; it is also easy to apply (*Kumar et al., 2005; Jumaat and Alam, 2006; Reddy and Maheshwar, 2011; Patil et al., 2012; Khan et al., 2013; Eswaran et al., 2016; Karunya et al., 2016; Dakhane et al., 2016*).

It is also further found that the use of polymer-modified mortar instead of cement mortar improved the properties of conventional ferrocement. The polymer modified mortar and ferrocement are discussed briefly in the succeeding sections.

1.9 INTRODUCTION TO FERROCEMENT

Ferrocement consists of cement, sand, wire mesh and is a versatile form of reinforced cement concrete. It can be shaped into any form like box-beam, I-beam, column, slab and other structural elements to construct low-cost houses, sheds and buildings etc. Ferrocement is widely used for the construction of water tanks, boats and ships.

1.9.1 Definition of Ferrocement

As per American Concrete Institute (*ACI 549R, 1997*) “Ferrocement is the reinforced concrete made of cement mortar and closely spaced layers of small diameter wire mesh. The mesh may be made of metallic or other suitable materials.”

According to *Paul and Pama (1978)* “A ferrocement panel is usually a thin section, consisting of layers of wire mesh impregnated with a very rich mix (high ratio of cement to sand) of cement mortar and cured for a specified period”. *Naaman and Shah (1971)* also defined Ferrocement as “a type of thin-walled construction where instead of reinforcing bars, the larger amount of smaller diameter wire meshes are used uniformly throughout the cross-section, and instead of concrete, the mortar is used.”

From the above definitions, it can be concluded that the material which has a thickness less than the concrete elements and is reinforced with some layers of wire mesh

instead of reinforcing bars is called ferrocement. The wire mesh used for ferrocement may be of various suitable shapes and detailed in the following section: -

1.9.2 Reinforcing Mesh Used in Ferrocement

Reinforcing mesh is one of the most important parts of ferrocement. Different types of wire meshes can be used to reinforce ferrocement, and those are available locally as well. The primary requirement of wire mesh is that it should be flexible enough to be easily bent in any shape especially around corners and should be easy to handle. The primary function of reinforcing mesh is to resist the tensile forces as the mortar is weak in tension. The structure tends to crack, twist and bend which results in cracking and fracture of the structure and hence, a sufficient amount of reinforcement is required to resist those stresses (*Paul and Pama, 1978*).

Reinforcing mesh usually consists of thin wires which might be welded or woven into the mesh. The overall strength and mechanical properties of ferrocement elements mainly depend upon the type, amount, strength and orientation of the wire mesh. The commonly used wire meshes are shown in Figure 1.2 (*Paul and Pama, 1978*).

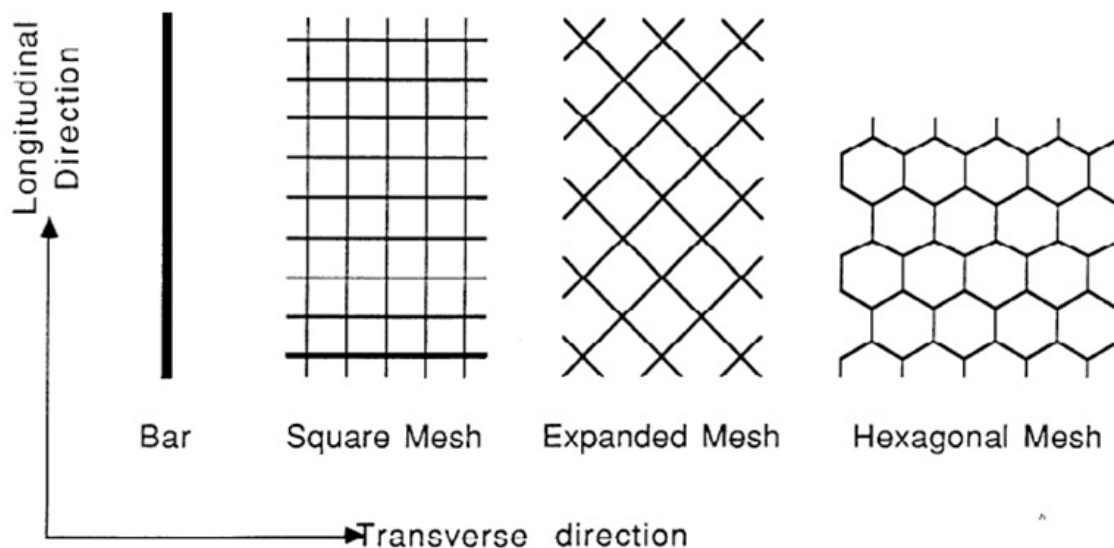


Fig. 1.2: Types of Reinforcing Wire Mesh for Ferrocement
(Retrieved: *ACI Committee 549.1R, 1993*)

1.9.3 Advantages of Ferrocement

Ferrocement has many advantages over reinforced concrete which are detailed below. Due to these advantages, the use of ferrocement has increased in developing countries.

1. The constituent material for ferrocement is readily available in the local market in most of the countries and is very cost-effective.

2. No highly skilled labour and machinery is required, and it can be formed in the desired shape with minimum formwork.
3. The strength to weight ratio of ferrocement is higher in comparison with other materials of a similar class. Due to its high impact strength, it is used to coat nuclear plants as a protective layer.
4. It is used for the housing system which is located in more prone to earthquake area due to its low self-weight and ease of erection.
5. It is used as storage bins and also used for the construction of sedimentation tanks, septic tanks and water tanks.
6. Ferrocement is quite durable and effectively used for strengthening and retrofitting of old structures.

Primarily due to these benefits, ferrocement has developed as a promising material capable of addressing all the social, environmental and economic issues.

1.10 POLYMER MODIFIED MORTAR (PMM)

Normal cement concrete mortars are used worldwide as a construction material from more than 200 years and more. With the passage of time, some deficiencies have been found in conventional cementitious materials such as; lower tensile strength, less resistance to chemical attacks and more drying shrinkage. In order to overcome these deficiencies of conventional mortar and concrete, a polymer was introduced into the cement matrix. The concrete and mortar which modified with polymers are so-called as polymer-modified-concrete (PMC) and polymer-modified-mortar (PMM). With the addition of polymer in the mortar matrix, a monolithic layer of polymer and cement is developed. This layer improves the properties of conventional cement mortar by binding the aggregate particles together (*Ohama, 1995*). The polymers have received considerable attention in the construction industry from the last 25 years and can be used as a cementitious material in many ways. Out of these available options, the latex is widely used to modify the mortar and concrete.

The polymers consist of a long chain of molecules which comprise of repetitive units of the alike structure. The polymers are generally categorised into two types; homopolymers and copolymers. Homopolymers are the monomers of one type and copolymers are the monomers of more than a single type. Polymer can be added to the mortar matrix either in the form of powder or liquid depending upon their compatibility with the cement mortar. Generally, monomers, latex re-dispersible polymer powder, water-soluble polymers and

liquid resins are used to modify the traditional mortar. The latex-polymers are commonly used to modify the cement mortar matrix (*Ohama, 1995*).

The choice of a specific kind of polymer depends upon its compatibility with the conventional cement mortar. There are various types of polymers available worldwide and it was found from the literature review that addition of vinyl-acetate-ethylene (VAE) polymers and styrene-butadiene-rubber latex (SBR) polymers are very efficient in improving the properties of conventional mortar, concrete and ferrocement (*Barluenga and Hernández, 2004; Medeiros et al., 2009; Wang and Wang, 2011; Chen et al., 2011; Wu et al., 2011; Wang et al., 2011; Ali et al., 2012; Singh et al., 2013; Ghai et al., 2016*).

Hence, these two types of polymers which are readily available in the market are adopted in this experimental work. As a pilot study, an optimum quantity of each type of polymer is worked out to modify the conventional cement mortar matrix.

The polymer structures are mainly of three types; crosslinked polymers, branched polymers and linear polymers. Linear polymer comprises of a long chain of monomer structures which are linked with each other. The primary examples of this type of polymers are polyvinyl chloride (PVC), polyvinyl alcohol and polyethylene. Branch polymers are constituted by the bonding of another chain of monomers with each other. The other chain is mostly available from another reactive group of monomers. When two or more chains of monomers are linked with another short-chain then they are known as crosslinked polymers. These chains also create a three-dimensional structure when linked with a more complex structure of monomers. The SBR polymer and VAE polymer used in this study have a branched type and linear type of structures, respectively, as shown in Figure 1.3. The SBR in liquid form and VAE in powder form are also shown in Plate 1.4.



Plate 1.4: SBR Polymer in Liquid Form and VAE Polymer in Powdered Form

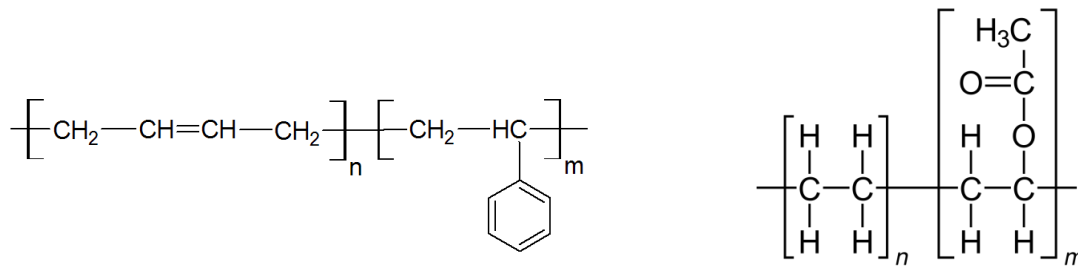


Fig. 1.3: Structure of SBR and VAE Polymer

1.11 POLYMER MODIFIED FERROCEMENT (PMF)

Conventional ferrocement made with the cement mortar matrix has better strength properties; however, it has shown some deficiencies and cracks under loads even much smaller than the ultimate loads, leading to its reduced life (*Rengaswamy et al., 1922; Naaman et al., 2000; Roxas et al., 2013; Lalaj et al., 2015*). The primary reason for the deterioration of ferrocement is the corrosion of wire mesh which is attributed by the permeability of cement mortar (*Andal et al., 2008; Kong et al., 2013*). To overcome these deficiencies of conventional ferrocement; it is essential to improve the properties of its mortar matrix with different types of additives. The addition of polymers to the conventional mortar is found to be very effective in such a case scenario (*Ohama, 1995; Vektaris et al., 2003; Chew et al., 2004; Žižková et al., 2011; Marceau et al., 2012; Ramli and Tabassi, 2012; Wang et al., 2012; Zhang et al., 2012*). It further imparts some more outstanding properties such as; higher strength to weight ratio, resistance to corrosion, improved tensile strength and ductility (*Bhikshma et al., 2011; Sheela and Ajith, 2012; Cheah and Ramli, 2012; Ramli and Tabassi, 2012; Sheela and Ganesan, 2014*).

Polymer modified ferrocement (PMF) is gaining popularity in developing countries because of its excellent properties and as an economical repair alternative to the expensive process of re-construction. Details of PMF are simple to follow and easy to execute even by local skilled workers and the ingredients being readily available (*Hughes and Evbuomwan, 1993; Kumar and Vidivelli, 2010a; Kumar and Vidivelli, 2010b; Liao and Fang, 2010; Zhao et al., 2012*). Furthermore, developments in the field of polymer-modified ferrocement (PMF) have a potential to make a drastic improvement in the area of composite strengthening and rehabilitation of old structures. The ferrocement modified with polymer is light in weight with high stiffness, strength and durability which makes it better to use for strengthening as compared to other composites of the same class. It can be firmly said that the application of polymer modified ferrocement as an outer strengthening material is a viable technology for improving the performance of RCC structures. The various RCC structural elements such as

columns, beams, slabs and joints can be strengthened by using PMF (*Khalifa, 1998*). Some studies have been made in recent years related to the strengthening of structural elements in flexure by using PMF. However, very few or no study has been reported on shear strengthening.

From the above discussion, it can be concluded that the application of PMF as an outer strengthening material is a viable technology for improving the structural performance of RCC beams in flexure. At present, there is not much research work which is recorded on the effects of polymer-modified ferrocement used for shear strengthening of RCC beams. Hence, it is the need of the hour to explore the utility of PMF as a strengthening material in the specified domain of shear. There are many factors which impart shear strength to RCC beams like concrete grade, stirrups spacing and shear span-to-depth ratios. In this experimental work, two variable factors as stirrups spacing and shear span-to-depth ratios are chosen to find their effect on strength, deflection, and cracking pattern of pre-damaged beams strengthened with PMF. An initial pilot study was proposed to evaluate the optimum percentage addition of polymer and the number of wire mesh layers for ferrocement to attain better flexural and tensile strength properties. The PMF so obtained is further proposed to be used to strengthen the RCC beams having three different levels of pre-damages.

1.12 INTRODUCTION TO FINITE ELEMENT (FE) APPROACH

Finite Element (FE) is a numerical approach used to analyse typical problems related to engineering. Manual calculations are quite complicated and time-consuming, and hence the application of FE based software has been increased to validating and verifying the experimental data. Several FE software packages are available which are specific for a particular domain application. FE technique helps engineers to optimise and refine the designs before construction. It is a revolution in the engineering industry which significantly improved the methodology and standards of engineering designs. The primary advantage of FE based software is that it gives precise results without consuming much time through effective simulations.

1.12.1 Introduction to ATENA-3D

ATENA is FE based software which is able to solve the complex engineering problems. It is effectively used for the analysis of stresses, fluid flows, heat transfer, soil mechanics, biomechanics, acoustic and electromagnetic fields. This is quite a helpful tool in predicting the behaviour of design, calculating safety factor and to find design deficiencies

and enable the user to ascertain the optimal design with confidence. ATENA software enables the users to choose various materials and solution approaches from the in-built library of this software. This software consist of 2-D and 3-D user interfaces and the 3-D user interface has been used for this study. ATENA is working on an incremental algorithm for an iterative solution.

1.13 SELECTION OF VARIABLES FOR THIS STUDY

Numbers of investigations have been done by various researchers to find the exact shear behaviour of RCC beams, but still there is no integrated work is reported /available in the literature pertaining to the use of PMF as strengthening material for beams in shear. Hence, based upon these studies, the two variables as spacing of stirrups and shear span-to-depth ratio have been adopted for the present study. Later, the effect of PMF strengthening on partially damaged RCC beams in shear has also been investigated via this study. The properties of polymer-modified mortars and ferrocement have been worked out in the pilot study by using different percentage additions varied from 5% to 20% of SBR and VAE polymers. The effect of two and three number of wire mesh layers on the properties of polymer modified ferrocement has also been studied.

1.14 RESEARCH SIGNIFICANCE

Shear strength of RCC beams is considered a difficult problem due to redistribution of the stresses. There are various factors which affect the strength of RCC beams in shear like; the size of the beam (i.e. a/d ratio), grade of concrete, anchorage and amount of flexure and shear reinforcement. These factors are responsible to change the behaviour of RCC beams from ductile to brittle. Further, the partially stressed/damaged beams in shear are also need to be addressed with a strengthening approach. Hence, the core objective of this research work is to investigate the use of polymer modified ferrocement (PMF) as shear strengthening material for RCC beams. The aim of strengthening scheme is to provide measurable repair and to enhance the designed service life of deteriorated and deficient RCC beams. This research work specifically intends to create a polymer-modified ferrocement having better properties and explore its usage for repairing and strengthening of RCC members.

1.15 METHODOLOGY

The present research work will be executed as detailed below: -

- (i) **Development of Polymer Modified Ferrocement (PMF): -**

Initially, the mechanical properties of polymer modified mortar have been investigated by adding different percentages of SBR latex and VAE polymer. Various properties such as; air content, flexural strength, workability, compressive strength and modulus of elasticity of mortars have been evaluated with respect to relevant standards. Further, this modified mortar has been employed to form the polymer-modified ferrocement, apart from this, the flexural and tensile strength of PMF has also been evaluated by incorporating both 2 and 3 layers of steel woven wire-mesh in the mortar matrix. From the pilot study, the best performer PMF is finalized and used for strengthening of beams.

(ii) Testing of beams: -

Full scale 48 numbers shear designed (DB) and shear deficient beams (SDB) RCC beams have been cast and tested for different shear span-to-depth ratios of 1 & 3. Based upon the test results of control beams, the other set of beams have been loaded to three different stress levels I, II & III to induce partial damage in the beam elements.

(iii) Strengthening and re-testing of beams: -

The pre-damaged RCC beams are then strengthened with PMF over the specified length & location and then re-tested till failure. The behaviour of the strengthened beams has been studied w.r.t. the parameters like; load v/s deflection curves, cracking behaviour, crack pattern, collapse load and modes of failure.

(iv) Validation of test results: -

Finite element (FE) based ATENA-3D software is used to validate the experimental test results.

1.16 ORGANIZATION OF THESIS

The present research work is organised in 7 chapters of the thesis as given below: -

Chapter - 1 introduces the mechanism of shear resistance of RCC beams, need of strengthening, various methods used for strengthening, introduction to ferrocement, polymer-modified mortar (PMM) and polymer-modified ferrocement (PMF), and introduction to ATENA-3D. The significance and methodology for this research work are given at the end of this chapter.

Chapter - 2 on literature review related to work done by different researchers in the area of ferrocement, polymer-modified mortar and polymer-modified ferrocement and further their use as a strengthening material. The application of various other materials to strengthen the RCC beams is also reviewed. The literature related to Finite Element modelling of

strengthened RCC beams with different materials is also presented. Effect of various parameters on the shear strength of beams is also discussed. The gaps and objectives of this research work are given at the end of this chapter.

Chapter - 3 details the materials used, casting and testing of polymer-modified mortar and polymer-modified ferrocement for the pilot studies undertaken to finalization the optimum percentage addition of a particular type of polymer and wire-mesh layers to further adopt for the strengthen of pre-damaged RCC beams. The details about their results and conclusions are also given.

Chapter - 4 details the materials used, techniques adopted for casting, curing, testing, details the scheme of experimentation, variables involved, testing of control beams, pre-damaging of RCC beams and strengthening of these beams using Polymer Modified Ferrocement (PMF) jacketing.

Chapter - 5 deals with the analysis and discussion of experimental results of control and strengthened RCC beams. These results are given both in graphical as well as tabular form.

Chapter - 6 Finite Element models have been developed by using ATENA-3D and validation of these models with the experimental test results is presented in this chapter.

Chapter - 7 presents the conclusions obtained from this research work and the future scope of work.

List of research publications from this work and the list of references, referred in the work are presented after Chapter-7.

CHAPTER – 2

LITERATURE REVIEW

2.1 GENERAL

With the passage of time, the existing RCC structures deteriorate due to increases in service loads, corrosion of reinforcement and poor ductile detailing, which results in loss of strength, cracking and spalling of the structural components. Such structural elements need special attention and must be retrofitted using suitable strengthening techniques to restore their original strength and design life. Any possible strengthening of such members needs to be explored, which could strengthen and make them fit for the present serviceable conditions. Many researchers have already worked on the development of various materials and techniques for repairing, retrofitting and strengthening of such structural elements. The selection of particular strengthening material and the specific technique depends upon the type, cause and nature of distress to be addressed.

Polymer modified ferrocement (PMF), has been used as a strengthening material in this study. The polymers are added in the mortar matrix of conventional ferrocement to improve its tensile strength, flexural strength and durability. Vinyl-acetate-ethylene (VAE) polymers and styrene-butadiene-rubber latex (SBR) polymers have been used to modify the conventional cement mortar. In the process, these polymers improve the durability, drying shrinkage, flexural strength of mortar. However, due to the absence of design codes, poor understanding of issues related to composite construction, lack of industry standardization, cost analysis and lack of designers experience with polymer composite materials, place these materials on a weak front when compared to other conventional materials. In light of the above issues, this study enlightens on the effects of application of polymers in the strengthening and retrofitting domains. As we know that strengthening and retrofitting is a cost-effective alternative to mere replacement of existing deteriorated structures and is often the only feasible solution.

In spite of all these above properties of PMF, still, there are very limited research work references to it. Till date, the experimental research work by the other researchers using PMF is limited to flexure only and almost no data or record of its experimental work has been penned out with respect to strengthening of RCC beams in shear by using PMF. The available literature pertaining to polymer modified ferrocement and mortar is reviewed in this chapter.

It is also necessary to initially explore brief literature related to conventional ferrocement and hence it is also discussed in the succeeding sections.

2.2 FERROCEMENT

High growth rate in urban areas of the world, powered by economic progress of its population has led to significant progress in the construction systems and practices. In this changing scenario, providing thin structural elements with higher strengths have become the need of the hour, not only from economic considerations but also from the aesthetical point of view. Ferrocement, a composite comprising of wire mesh and cement mortar has now become a readily acceptable thin material in the field of construction technology. Many researchers have investigated various properties of metallic-wire mesh reinforced ferrocement and have found it to be a good thin structural composite alternative to earlier non-metallic mesh reinforced ferrocement technologies in the same class (*ACI Committee 549R-97; Xiong and Singh, 1997; Arif et al., 1999; Al-Rifai and Al-Shukur, 2001; Montesinos and Naaman, 2004; Austriaco, 2006; Rao et al., 2006*). Some of the conventional meshes like polypropylene, woven alkali-resistant glass and organic fabrics from jute and bamboo fibers. Application of the ferrocement increases worldwide due to its better properties such as higher strength to weight ratio, resistance to cracking and impact, better ductility and greater environmental sustainability compared to the conventional reinforced concrete (*ACI Committee 549R-97*). Moreover, ferrocement can be cast in different shapes, without even requirements of extensive formwork. Its constituents are easily available and that too at a cheaper cost. The research on flexural strength, tensile strength, shear strength, compressive strength, impact resistance, durability, thermal/sound conductivity, usage in strengthening and confinement and fire resistance of ferrocement has already been carried out throughout the world. The properties of ferrocement have been investigated worldwide and some of these are discussed in succeeding paragraphs.

Wafa and Fukuzawa (2010) studied the various effects of reinforcing meshes on the flexural strength of thin composite ferrocement elements. They have used E-fiberglass meshes and stainless steel meshes as reinforcement with a different number of layers, diameters and opening sizes. They have used the conventional cement mortar and the polymer-modified mortar as constituents for ferrocement elements. They have studied the effect of these variables on various parameters like; the bending stiffness, first crack load, crack characteristics, the load–deflection behaviour, ultimate flexural load capacity, energy absorption and ductility as well. This experimental test was conducted on 24 simply

supported ferrocement specimens of size 520mm × 40mm × 10mm as shown in Figure 2.1 & 2.2.

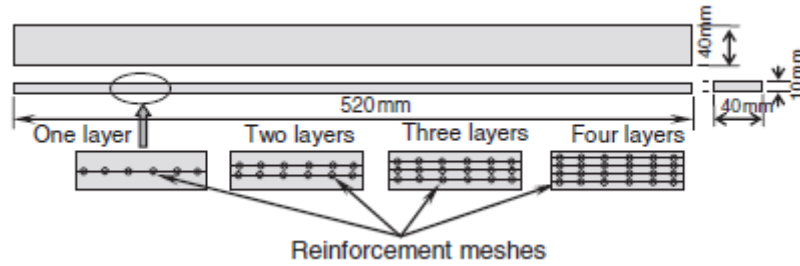


Fig. 2.1: Specimen Details

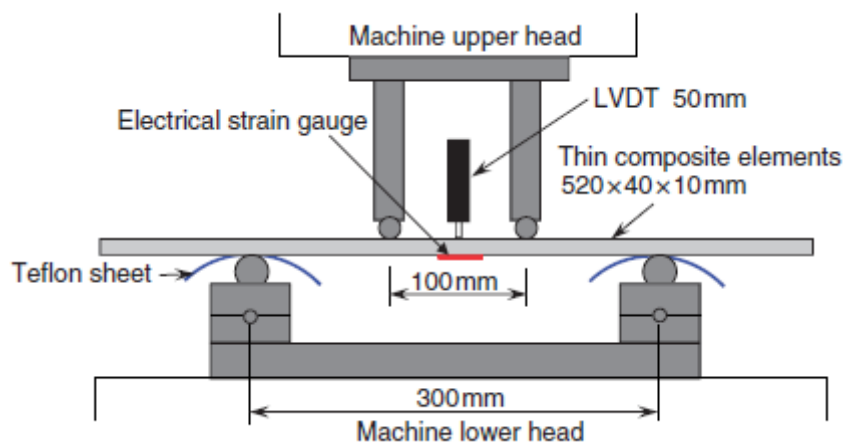
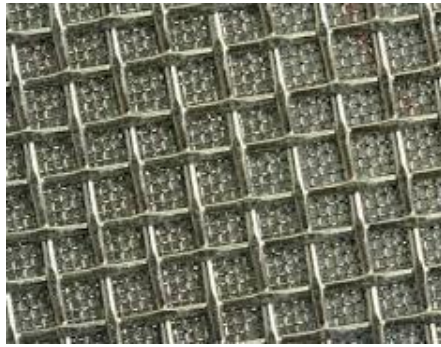


Fig. 2.2: Flexural Load Test Set-Up

The test values showed that the specimens with steel wire meshes performed better in terms of ductility, bending and cracking pattern as compared to specimens having fibreglass meshes. The increased volume fraction of wire mesh can also significantly improve the properties of ferrocement. They have also studied the effect of variation in the size of the mesh opening and diameter of the wire. The results also validated the *ACI Building Code 318* provisions of calculating the moment carrying capacity of ferrocement element having steel and glass fibre meshes.

Lalaj et al. (2015) reviewed the recent perspectives for ferrocement. The aim of their study was to review the existing literature and to discuss new uses of ferrocement in future. In their paper, a review on the use and properties of ferrocement has been presented. They discussed the properties of ferrocement w.r.t. RCC and concluded that the ferrocement has excellent flexural, tensile and ductile strength properties. Low-cost and low-rise, earthquake-resistant houses have been constructed using ferrocement and the performance of ferrocement houses against earthquake has been proven during high magnitude earthquake which

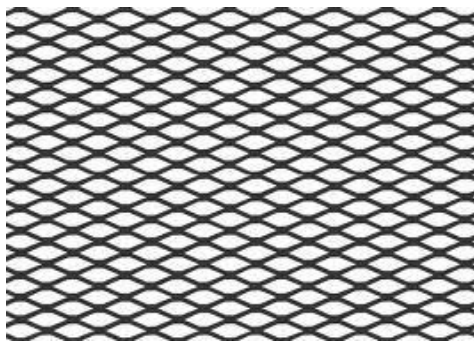
occurred in Chile on 27th February 2010. All the properties of ferrocement have also been proven experimentally, analytically and practically by many researchers. However, due to its less thickness, it is not durable as compared to RCC. Ferrocement is mostly used for water tanks, boats, roofs and silos. In the last three decades, new perspectives have also appeared on the usages of ferrocement.



(a)



(b)



(c)



(d)

Fig. 2.3: Wire Meshes for Ferrocement; (a) Woven Wire; (b) Welded Wire; (c); Stretched Metal; (d) Hexagonal Wire

This study concluded that ferrocement is a suitable strengthening material for structural elements like columns, walls, beams and slabs. It was reported that when ferrocement used as strengthening material it leads to increase in strength, durability, stiffness and cracking behaviour of not only the structural elements but also the structure as a whole. Figure 2.3 shows the various types of meshes used for ferrocement. The properties of ferrocement also get affected by the orientation and type of mesh used as reinforcement. They have studied the effects of orientation and type of meshes on ferrocement. *ACI 549R-97* standard code presents the guidelines on good practices for use of ferrocement; however, the guidelines for its use as a strengthening material are not given in this standard code or any

other code. They concluded that the ferrocement can be a good option for the strengthening of structures and more needs to explore in this regard.

In the prior studies the ferrocement is used as the strengthening material. Many researchers (*Kumar et. al., 2005; Anugeetha and Sheela, 2009; Khan et. al., 2013; Paranik et. al., 2014; Bansal et. al., 2016; Eswaran et. al., 2016; Karunyal et. al., 2016;*) have studied the effects of various variables such as; orientation of wire mesh, type of wire mesh, different volume fractions of reinforcement etc. where ferrocement is used as strengthening and retrofitting material for RCC beams and columns against static and dynamic/seismic loadings in shear and flexure. Studies reveal that use of ferrocement is a viable and economical solution to strengthen and retrofit the existing and partial damaged structures to preserve their designed life and sustainability.

Rafeeqi et al. (2005) had studied the performance of strengthened RCC beams in shear by ferrocement. Five full sizes (100mm × 200mm × 915mm) beams were cast in a ratio 1: 2: 4 as concrete with w/c ratio of 0.55 and compressive strength of 22 N/mm². All those beams were reinforced on tension face with 2-12mm diameter bars and on compression face with 2-6mm diameter bars. Shear stirrups of 6mm diameter bars were given c/c spacing of 50mm. All those beam specimens were tested with third-point loading method. The control beam was loaded till failure. The other set of beams was initially loaded up to 70 kN and then was unloaded. Later it was strengthened with ferrocement wraps and strips with 1 and 2 layers of wire mesh, respectively (refer Plate 2.1 and 2.2). The ferrocement with a cement-sand ratio of 1:2 and w/c ratio of 0.45 was reinforced with 18 gauge woven square wire meshes of diameter 0.9mm with c/c spacing 4.6mm. The tensile and compressive strength of ferrocement mortar was 3.5 N/mm² and 25 N/mm², respectively.



Plate 2.1: Detail of Beam Strengthened by Ferrocement Strips in Shear Span

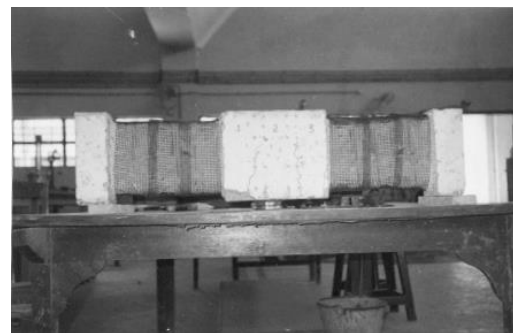


Plate 2.2: Detail of Beam Strengthened by Ferrocement Wraps in Shear Span

Results revealed the increase in shear capacity from 1.5% to 5.8% when strengthened by wrapping two-layered wire mesh ferrocement. The failure mode of beams also transfers from brittle to ductile. During the strengthening with ferrocement, there was a delay in the failure in spite of the ample warning. The study concluded that the over-reinforcing in a shear zone by external means of confining can cause beam failure in a ductile manner. The test results would have been better if the beams were strengthened with wraps as compared with the strips. The deflection and width of cracks also got reduced with these strengthening techniques. Further, it was concluded that this strengthening technique is very simple to understand and apply. In order to achieve maximum results, it can also be used in rural construction.

Anugeetha and Sheela (2009) investigated the effect of ferrocement and glass fibre reinforced polymer (GFRP) sheets used as strengthening material for RCC beams. The effect of the number of wire mesh layers was also studied. The experiment was performed on full-sized M20 grade RCC beams of 120mm × 200mm × 1200mm size. The beams were reinforced with 2-8mm diameter bars on tension and compression face and with 2-legged 6mm diameter shear stirrups @ 200mm c/c. The ultimate load-carrying capacity of control beams was found out by loading the beams till failure. The remaining beams were then pre-loaded to 60% of the ultimate load of control beams. Different sets of beams were then retrofitted with ferrocement and GFRP sheets. The U-shaped 15mm thick ferrocement was applied on three sides of the damaged beams. One, two and three layers of 6×22 gauge wire mesh was used in ferrocement. The cement, sand and water were kept in the ratio of 1: 2: 0.5 for mortar mix. Another set of pre-damaged beams were strengthened with GFRP sheets having one, two and three layers. Detail of the beam specimens and their strengthening with ferrocement is shown in Figure 2.4.

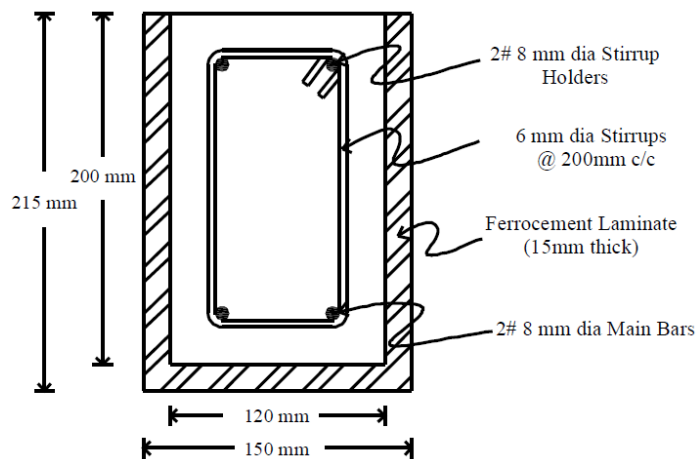


Fig. 2.4: Detail of Beam Specimen and Strengthening with Ferrocement

From the test results, it was found that the ultimate load-carrying capacity of beams strengthened with 1, 2 and 3 layered ferrocement increased by 6.25%, 50% and 81.25%, respectively. On the other hand, the improvement in the capacity by 50%, 68.75% and 81.25% with the use of 1, 2 and three layers of GFRP sheets was also noticed. It was concluded that the beams retrofitted in the flexural zone with one layer of GFRP showed a higher strength-to-cost ratio.

Miah et al. (2019) studied the efficacy of ferrocement technique in improving the performance of reinforced concrete beams made in low-strength concrete. This study was carried out on RC un-retrofitted and retrofitted beams of size (width 230 mm, height 230 mm, and length 2135 mm) made with unconventional concrete (i.e., very low strength concrete of 12.5-13.0 MPa) under four-point incremental static loading. In order to gain a deeper understanding of the role of loading positions on the flexural performance of the beams, numerical simulations had been performed via the finite element software ABAQUS. Four-point flexural tests are performed experimentally and numerically on both retrofitted and un-retrofitted beams with different loading positions: (i) 2 loads at $L/3$ (Reference case), (ii) 2 loads close to the support (Shear case), (iii) 2 loads close to the mid-span (Bending case), and (iv) 1 load next to the support and 1 load on the mid-span (Combined case).

It was concluded that the beams tested under unsymmetrical loading showed a worse mechanical performance due to shear action. Among four cases, poor performance of RC beam was observed in shear case which was associated to the formation of major crack at the end of the beam (shear failure). Further, the ductile mode of failure was observed for the reference and bending cases in retrofitted beams, while a brittle failure had seen for the shear case due to severe cracking with damage localization occurred during the pre-cracking phase on the un-retrofitted beam. Numerical results of un-retrofitted and retrofitted tests were in quite good agreement with the experimental results. It was recommended to use ferrocement technique to strengthen existing structures made with low strength concrete to avoid sudden collapse due to extreme loading, such as earthquake.

Apart from above, *Kaish et al. (2018)* and *Sonam and Mohan (2018)* reported that the use of ferrocement composite is also an effective technique to strengthen the RCC columns.

From the literature, it can be concluded that the use of ferrocement is an economical and a sustainable construction and strengthening material in the present construction scenario. It was also revealed (*Rengaswamy et al., 1992*) that the conventional ferrocement showed some deficiencies and did show cracks under the load much smaller than ultimate load, which may further cause a reduction in the overall life span of ferrocement. *Andal et al. (2008)*

reported that the wire mesh of ferrocement matrix is also susceptible to corrosion due to higher the permeability of mortar and it is one of the main reason for its deterioration.

Hence, to overcome these deficiencies of traditional ferrocement, it is essential to improve the properties of its conventional mortar matrix by using some additives. Many researchers (*Ohama, 1995; Vektaris et al., 2003; Chew et al., 2004; Chen et al., 2011; Wu et al., 2011; Žižková et al., 2011; Marceau et al., 2012; Ramli and Tabassi, 2012; Wang et al., 2012; Zhang et al., 2012*) found that the addition of polymers to the conventional mortar is very effective to improve its properties. The polymers addition in the cement mortar matrix of ferrocement further imparts some outstanding properties like resistance to corrosion, higher strength to weight ratio, tensile strength and better ductility as compared to conventional ferrocement. The effect of the addition of various polymers on the properties of mortar and ferrocement is reviewed in detail in succeeding sections.

2.3 POLYMER MODIFIED SYSTEMS

2.3.1 Polymer Modified Mortar (PMM)

The prior studies revealed that the physical and mechanical properties of fresh and hardened mortars can remarkably improve with the addition of various types of polymers in its matrix. A vast range of polymers are available those can be added into the mortar matrix to improve its properties. Dispersive polymers like; styrene-butadiene-rubber latex polymer (SBR), vinyl acetate and ethylene polymer (VAE) and PAE have been found to perform better as compared to other dissolvable polymers. *Sakai et al. (1995), Afridi (1995), Ohama (1995), Barluenga and Olivares (2004)* and *Wu (2011)* were concluded that the polymer phase and monolithic phase was developed in the mortar matrix due to the addition of polymers. These phases interpenetrate and unite the aggregates of PMM. The development of this co-matrix improved the properties of mortar such as; workability, adhesion, durability and strength as compared to conventional mortar.

Barluenga and Olivares (2004) studied the effect of SBR polymer and w/c ratio on the properties of Latex Modified Mortar (LMM). SBR latex with a solid content of 38% and a pH of 8 was used. Cement to the sand ratio of 1: 3 was fixed for this study. They studied the consistency, setting time, flexural strength, compressive strength, density, ultrasonic modulus and porosity for different ages (1, 7, 28 and 56 days). Different percentage viz. 0%, 5%, 10%, 15%, 20% and 25% of the SBR latex w.r.t. the weight of cement was added and w/c ratios were found corresponding to the flow table diameter of 180, 200 and 220 mm. No plasticizer was added in the mortar matrix. For a flow table diameter of 180 ± 10 mm, the required

water/cement ratio was observed as 0.55, 0.53, 0.475, 0.425, 0.4 and 0.375, respectively, corresponding to the polymer addition of 0%, 5%, 10%, 15%, 20% and 25%. It was found that the polymer addition improves the consistency and decreases the requirement of water in order to obtain fixed flow value, due to its plasticizer effect. Reduction in the setting time of LMM was also observed at 15% addition level of latex. To study the flexural strength of LMM, prismatic specimens of size 40mm × 40mm × 160mm were cast. The overall test results revealed that the polymer addition improved the fresh as well as hardened properties of mortar. It was found that, for an addition level of below 15%, cement hydration governed the properties and above 15%, the polymer film formation governed the properties of modified mortar. Hence, 15% is the optimum percentage in addition to get the best results.

Wang et al. (2005) and *Wang et al. (2011)* experimentally studied the effect on properties of styrene–butadiene rubber (SD622S), with effective content of 58.3%, on the emulsion modified cement mortars. The experiments were done by keeping the cement-sand ratio of 1:3 and water/cement (w/c) ratio was varied from 0.28 to 0.40. Various combinations were established by adding the polymers (percentages varied from 0 to 25%) in the mortar matrix. Compressive strength and flexural strength tests were conducted after curing ages of 7 and 28 days. Scanning Electron Microscope (SEM) was also conducted to analyse the microstructure of polymer-modified mortar. Specimens of size 40mm × 40mm × 160mm were casted for the flexural strength test. The studies concluded that the toughness of polymer modified mortar improved up to an additional level of 10%. The flexural strength improved by 58% and 47% after 7 and 28 days, respectively, with the addition of 25% SBR. The compressive strength of mortars also increased up to polymer addition of 10%. Studies concluded that the addition of polymer emulsion caused the hydration of cement and impetration of polymers in one another. This process helped to form a continuous net structure and led to the reduction in micro-cracks and contributed to improving the flexural strength toughness.

In different studies, *Wang and Wang (2011)* and *Wang et al. (2012)* used re-dispersible polymer powder of vinyl acetate and versatate copolymer (VA/VeoVa) to modify the matrix of conventional cement mortar. Test results are showing that the addition of VA/VeoVa caused an improvement in the properties of mortar by exhibiting water-retention and water-reduction effects in conventional mortar. The flexural strength and toughness were also improved at 8% addition level of polymer; however, the compressive strength did not improve. The air-entrainment effect of VA/VeoVA caused a decrease in the bulk density and

increased the air content of fresh mortar. VA/VeoVa powder also helped reduce the shrinkage rate and improve water impermeability and hydrophobicity.

Chen et al. (2011) investigated the effect of VAE latex addition on the properties of cement mortar. The bond strength between the mortar and modified mortar block was measured with the bent bond strength tests. The cement/sand ratio was kept as 1:3 and the water/cement ratio was measured for a flow value of 160 ± 5 mm. The reasonable defoaming technique was also used to solve the foaming problem of VAE latex modified cement mortar. The mortar block of size 40mm×40mm×160mm was made and cut from the middle into two blocks. Out of these two pieces, one block was modelled and built from the polymer-modified mortar. The newly prepared sample was kept for curing for a specified number of days and tested on a flexural testing machine to find the bond strength. The cracking resistance and adhesive strength of mortars were found to improve maximum with a 12% polymer addition.

Hitachi Ltd., Japan, investigated the microstructure of unmodified and polymer-modified specimens. Test results showed that the anti-permeability and bond strength initially increased and decreased after-wards with the addition of different percentages of polymers. The anti-permeability pressure attained a maximum value of 1.3MPa at 10% addition level. Test results of SEM also showed that the polymer-modified samples had a much more compacted matrix structure as compared to the unmodified samples. It was concluded that the polymeric film wraps the cement particles and fills the cavity between sand and cement which further enhanced the interfacial bond. The other study made by *Wu et al. (2011)* also concluded that the addition of VAE polymers coating compacted the cement mortar matrix, wherein the polymer coating covers the cement particles helped to fill the voids between cement and sand and caused an increase in the bond strength. *Ali et al. (2012)* also found that the addition of SBR polymer is quite effective in improving the properties like workability, flexural strength, toughness and impermeability of conventional cement mortar (CCM).

Ramli and Tabassi (2012) studied the effects of SBR, PAE, PVE polymers and different curing conditions on the properties of mortar. Test results showed that the pore structure of the mortar matrix was enhanced with the addition of polymers which has improved the waterproofness. The impermeability of latex modified mortar has increased 4 to 5 times as compared to unmodified cement mortar at an addition level of 15%. Further, the addition of polymers has also caused the reduction in water requirement of mortar mix.

Singh et al. (2013) reviewed past research on the properties of polymer concrete. They mainly reviewed the effect of polymer type, fibers, curing conditions and processing

parameters on the properties of concrete modified with polymers. They corroborated the brief findings of all the major researches carried out in this area. From the review data; they concluded that 10% to 20% polymer addition is the optimum value to modify concrete with polymers. The compressive strength has increased up to 30% with the 15% addition of polymers. The review study also reveals that the toughness and strength of polymer concrete also improved with the addition of glass fibers.

Çavdar (2014) studied the effect of different curing conditions on the properties of mortars modified with polymers. He used Polycarboxylate modified Styrene Butadiene Rubber (PSBR), Styrene-Butadiene Rubber (SBR), Modified Acrylic Dispersion (MAD) polymers, in five different ratios (0%, 5%, 10%, 15%, 20%), by volume to modify the cement mortar matrix. To investigate the physical and mechanical properties of modified mortars, the mixes were cured for three different curing conditions of; (i) 16 hours in the water - 8 hours out (SCC), (ii) watering twice a day (FCC), (iii) one day in the water-one day out (TCC). The cement: sand: water ratio of the mixes was kept as 1:3.3:0.5. The study showed that the flexural strength increased by 24% and 29%, respectively, with the addition of 20% PSBR and MAD polymers for FCC curing condition. The flexural strength of SBR modified mortar increased by 24% at an addition level of 15% under FCC curing conditions. The compressive strength of polymer modified mortar decreased with used of all type of polymers and decreased by 23%, 42% and 60% with 5% addition of SBR, PSBR and MAD, respectively. The compressive strength at an addition level of 10%, 15% and 20% did not decrease in the case of SBR modified mortar under TCC condition. The best curing condition reported was FCC. It was also reported that with the addition of polymers in the mortar matrix, the unit mass is decreased by 10 to 15% but at the same time, consistency and workability increased with the addition of polymers.

Tukimat (2017) reviewed the fresh and hardened state of polymer-modified mortars and concrete. He has discussed the promising and excellent benefits of this advanced long-serving material of Civil Engineering. In his study, he mentions that the different types of polymers such as latex, epoxy and emulsions can be used to improve the workability and strength properties of mortar and concrete. He has also mentioned that the characteristics of polymer modified mortar and concrete such as adhesion, performance, multi-functional and local availability makes this material more favourable in the construction industry. He has concluded that addition of polymers is a viable approach to enhance the overall properties of conventional cement mortar and concrete.

Kumar and Vidivelli (2017) also evaluated the performance of polymer modified mortar and ferrocement and recommended their application for repair and strengthening. They have also studied the properties of ferrocement with the different volume fractions of reinforcement. They concluded that the optimum percentage of SBR polymer addition is 15% and the water absorption of mortar reduced up to 82% with the addition of 15% SBR.

Sun et al. (2019) reported that styrene-butadiene rubber (SBR) latex is valuable to the workability of cement-based materials, but the mechanism remains uncertain. This study aims to examine the consequence of SBR latex on the rheological property and pore structure of cement paste. The results showed that the fluidity of the modified cement-based material was enhanced, and an important linear relation was obtained among the air content and the fluidity of the cement paste. The yield stress and viscosity of the paste decreased with increasing SBR latex content, and the modified cement paste exhibited shear-thinning behaviour. In addition, the SBR latex can deteriorate the pore structure of the hardened cement paste at 3 days owing to the hydrophobic groups easily trapping the air into the paste. Compared with that of the ordinary Portland cement paste, the number of pores (size range of 0.1 μm -1 μm and 10 μm -200 μm) in the SBR-modified cement paste was clearly improved. However, the lowest porosity was achieved at 28 days, when 2% SBR was used owing to the filling consequence of the hydration products and polymer particles. The throat/pore ratio and fractal dimension augmented with the SBR latex content, illustrating the escalating of ink-bottle pores with a rough surface.

2.3.2 Polymer Modified Mortar (PMM) Used as Strengthening Material

Many researchers like *Mirza et al. (2002)*, *Pattanaik (2009)*, *Park et al. (2009)*, *Jummat et al. (2010)* and *Wu et al. (2011)* evaluated the performance of polymer-modified mortar as repairing and strengthening material. Different studies have been made by varying the different parameters and variety of polymers. These studies have finally come to the conclusion that the polymer modified mortar is a better alternative material for repair and strengthening purposes and is capable to restore the originally designed strength of structural members.

Medeiros et al. (2009) studied that the use of acrylate polymers and ethylene-vinyl-acetate copolymer (EVA) in the matrix of mortar and experimentally studied its utility for repairing of concrete structures. The mix proportion of 1: 2.24: 3.12 (C: FA: CA) with water/cement ratio of 0.60 was adopted for the reference concrete mix. The mortar used for repairing was made of two types of cement named; filler-modified Portland cement and early

high strength Portland cement, by keeping the cement-sand ratio as 1:3. Superplasticizer of 2.4% was also used to maintain a flow of 200 ± 10 mm. Polymers were added with the ratio of 10% and w/c ratio was kept between 0.40 to 0.43. Test results showed that the tensile strength of mortar significantly increased as compared to compressive strength with the addition of polymers. The direct shear bond strength has also improved. The compressive strength, tensile strength and bond strength of repairing mortars have increased by 10-20%, 30-40% and 40-50%, respectively with the addition of 10% EVA and acrylate polymers.

Ahmad et al. (2012) studied the use of locally available polymer-modified mortar for the strengthening of existing cracked RCC beams. They performed the experiment on twelve full-scale beams of size $152.4\text{mm} \times 304.8\text{mm} \times 3353\text{mm}$. Out of these twelve beams, six beams were cast with flexural reinforcement only and named as HB and six beams were cast with both flexural and shear reinforcement and named as GB. Detail of beam specimens and loading arrangement is shown in Figure 2.5 and 2.6. Initially, all the beams were loaded to pre-damage for a limiting value of crack width of 1mm. The pre-damaged beams were then strengthened by applying polymer-modified mortar. The strengthened beams were then again loaded till failure after a curing age of 3 days. Test results revealed that the load-carrying capacity of beams increased up to 36% after strengthening. It indicates that the application of PMM is a viable technology for the strengthening of damaged structures.

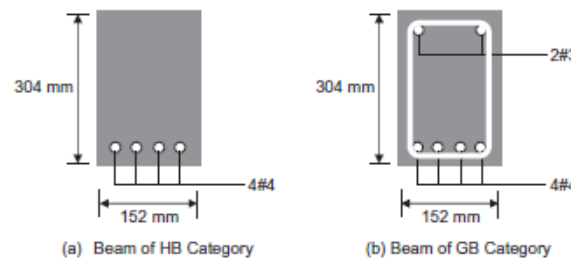


Fig. 2.5: Detail of Beam Specimens

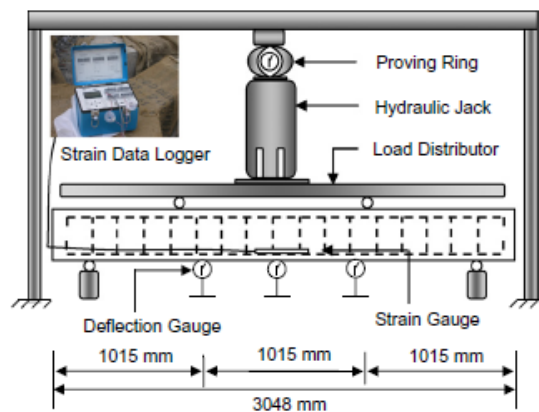


Fig. 2.6: Loading Arrangements for Testing

2.3.3 Polymer Modified Ferrocement (PMF)

The conventional ferrocement made with ordinary cement mortar has showed some deficiency in terms of durability, earlier cracking load, deformation and lesser flexural and tensile strength. It was the need of the hour to improve the properties of ferrocement with some additives and make it more effective for use in the field of construction and retrofitting industry. Hence, the properties of ferrocement modified with latex, resin and impregnation polymers have been explored in this study. Existing studies showed that with the addition of polymers, the properties of ferrocement have shown a marked improvement in their flexural behaviour, durability, tensile strength and deformation properties as compared to unmodified ferrocement (UMF).

Neelamegam et al. (1984) studied the flexural behaviour of polymer-ferrocement using latex-ferrocement (LFC), polymer-impregnated ferrocement (PIFC), resin-ferrocement (RFC) specimens and compared the properties with conventional ferrocement (FC) specimens. Reinforcement consisting of crimped wire cloths of galvanized wires (1.5 mm dia.) and welded wire fabrics of un-galvanized wires (2.6 mm diameter) with five types of variation in layers was used to prepare the specimen of polymer ferrocement. OPC and river sand of size 1.2 mm or finer with a ratio of 1:3 (by weight), the water-cement ratio of 40.0%, the polymer-cement ratio of 20% and a flow value of 172 was adopted for this study. Unsaturated polyester resin (UP) and styrene-butadiene latex (total solids of 43.7%) used as a liquid resin and styrene (S_t) was employed for impregnation. Latex-ferrocement specimens of size 550mm $\times$ 250mm $\times$ 20mm and polymer modified mortar cylindrical specimens of size 75mm $\times$ 150mm were cast for this experimental study. From the test results, it was concluded that the flexural toughness and ultimate moment carrying capacity of polymer-modified ferrocement improved markedly with the addition of polymers as compared to the unmodified ferrocement. Increase in reinforcement layers also improved the properties of ferrocement.

Ohama (1995) concluded via a study that ferrocement modified with latex showed an improvement in flexural behaviour and durability as compared to conventional ferrocement. The use of EVA and SBR polymers in the mortar were found to be very effective in improving the drying shrinkage, impact resistance, flexural behaviour and durability of ferrocement. He also concluded that the use of steel and carbon fibers to modify the mortar matrix is also a viable option to further improve the flexural behaviour of ferrocement.

Xiong et al. (2004) investigated the behaviour of polymer-modified ferrocement under sustained flexural load by using soluble glass-polyvinyl acetate hybrid polymer. To simulate

better site conditions, the test was performed for flowing sulphuric acid solution. OPC, sand and fly ash (class II) were used in this experimental work in accordance with Chinese standards. For creating both flowing and stagnant corrosion environments the sulphuric acid solution (1% concentration) was used in this work. Four kinds of mortars of ration 1:2 (with six mixture proportions by adding 18% polymer and 15% of fly ash) were chosen. The cubes of size 100mm × 100mm × 100mm and prism of size 40mm × 40mm × 100mm were tested after adopting different conditions. Ferrocement specimens of size 350mm × 125mm × 30mm were used for this work. Eight layers of galvanized squared weld wire mesh of diameter 0.8mm and having a size of 10.16mm × 10.16mm with a cover of 5mm was used in ferrocement. Eight layers of wire mesh were provided in the stress direction with total reinforcement percentage of 1.287%. Results showed that the mass loss of various mortars was about 3.36-4.53 times higher in flowing sulphuric acid solution as compared to the stagnant specimens. The polymer-modified mortar showed significantly higher resistance to flowing sulphuric acid condition as compared with unmodified mortar. No corrosion of steel was observed in the case of polymer-modified ferrocement when tested under sustained flexural load. However, the unmodified ferrocement showed corrosion of steel which further cause to decrease the moment carrying capacity. Test results showed that the behaviour and properties of modified ferrocement were remarkably better as compared to unmodified ferrocement for under sustained loading and flowing sulphuric acid solution conditions.

Zulkarnain and Suleiman (2008) studied the effect on the durability of ferrocement with the addition of polymers specifically when cured with salty water to match with the field conditions of severe environmental exposure. Cement-sand mortar of ratio 1:3 with w/c ratio of 0.45 was used in this study. Well-graded river sand fine aggregates used with maximum size 5mm. Slump value of 130-150 mm was kept for all the mixes irrespective to the final w/c ratio used. Three different liquid polymers named; natural rubber latex (NR), styrene-butadiene rubber latex (SBR) and epoxy resin (ER) were used with 10% addition level to modify the mortar matrix and ferrocement. The flexural and compressive strength of mortars for curing age of 28 days was evaluated by conducting the test on mortar prisms of size 100mm × 100mm × 500mm and mortar cubes of size 100mm × 100mm × 100mm, respectively. These tests were conducted according to BS 1881: Part 118: 1983 and BS 1881: part 116: 1983, respectively.

The ferrocement specimens of size 350mm × 125mm × 30mm cased with 3 layers of square welded wire mesh of opening 12mm × 12mm, the diameter of 1mm and volume fraction of 0.65% (refer to Figure 2.7). To find the flexural strength of modified ferrocement,

a four-point load test was performed over a simply supported span of 300mm as shown in Figure 2.8 and Plate 2.3. The tests on ferrocement were performed for the age of 30, 90, 180 and 365 days.

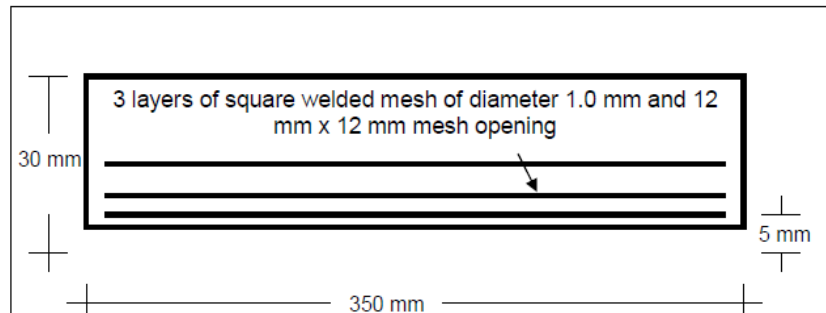


Fig. 2.7: Detail of Ferrocement Specimens

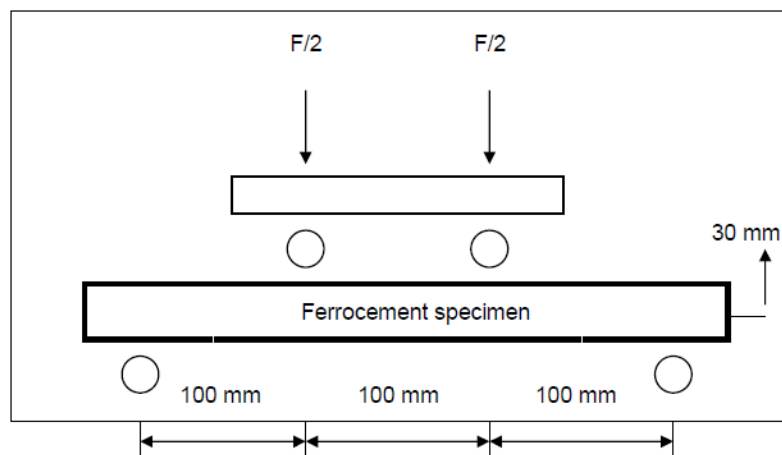


Fig. 2.8: Flexural Strength Test Set-Up for Ferrocement



Plate 2.3: Four-Point Load Flexural Strength Test under Progress

The test results were showing the improvement in flexural strength of mortar with the addition of polymer in its matrix. Due to this increase of flexural strength, the load value of polymer-modified ferrocement at first crack was increased as compared to unmodified ferrocement. This mainly happens due to the addition of polymers which further form the polymeric film and caused to binds the cement particles and aggregates and results in a durable mortar matrix. From this study, it was concluded that the addition of polymers improved the first crack load, deflection characteristics and maximum load values of ferrocement and further keep on improving with the age.

Cheah and Ramli (2012) studied the properties of ferrocement modified with high calcium wood ash (HCWA) and densified silica fume (DSF). This experiment was conducted on the total of 21 ferrocement specimens of size 350mm × 125mm × 30mm having five layers of square steel wire mesh of diameter 1.05mm and opening 13mm with a volume fraction of 1.93% (refer Figure 2.9).

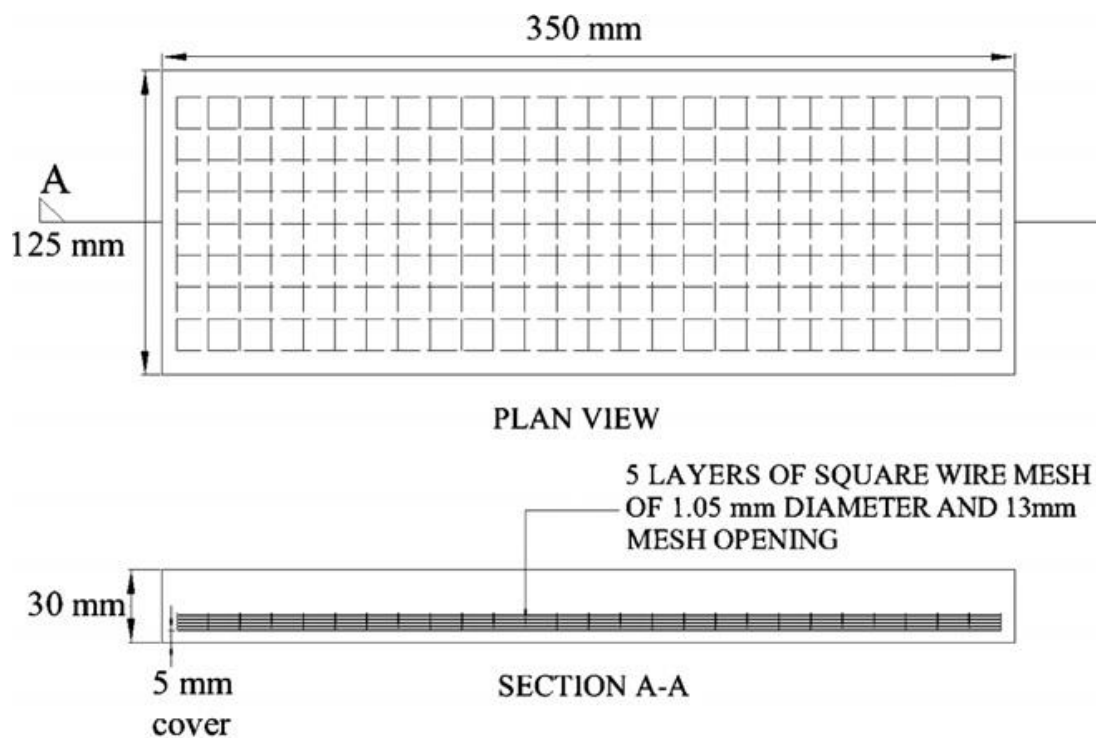


Fig. 2.9: Specification and Reinforcement Details of Ferrocement Panels

Four-point load flexural test (as shown in Figure 2.10) was conducted to access the properties of high strength mortar ferrocement. Cement to the sand ratio of 1:2.25 and water to cement (w/c) ratio of 0.32 was kept. The HCWA was added in the mortar matrix of ferrocement by replacing the cement in the ratio of 2%, 4%, 6%, 8% and 10%. With the same constitutes, the constant percentage of 7.5% of DSF was also added. Experimental results

concluded that the first crack load, flexural strength, compressive strength, ultimate failure load, bond strength, ductility, Young's modulus of elasticity and bulk density of modified ferrocement has been improved remarkably with 6% addition of HCWA.

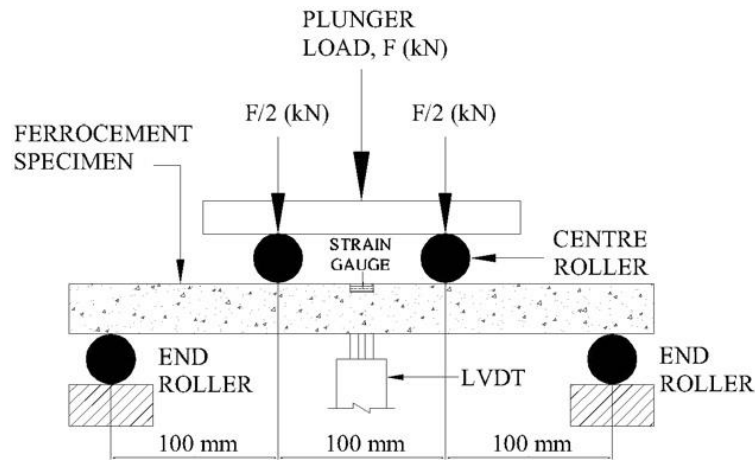


Fig. 2.10: Support and Load Arrangement to Test Ferrocement Panels

Bhikshma et al. (2011) studied the durability properties of ferrocement modified with fly ash and polymer. The fly ash was used by replacing the cement with ratio of 0%, 5%, 10%, 15%, 20%, 25% & 30% and polymer with ratio of with 0%, 2.5%, 5%, 7.5%, 10% & 12.5%. The permeability of cement mortar was found out by casting cylindrical specimens of size 100mm $\times$ 50 mm. The corrosion performance was also studied on ferrocement prism specimens of size 300mm $\times$ 50mm $\times$ 25 mm and by varying the volume fraction of reinforcement. The number of wire mesh layers was varied by 4, 6 and 8 layers to cast ferrocement specimens. Half specimens were prepared by replacing the cement by fly ash and the half by replacing the cement by the polymer. Results showed that the coefficient of permeability decreased with the increase in polymer addition and maximum decreased at a 12.5% polymer replacement and 30% fly ash replacement level. Corrosion-preventing property of polymer and fly ash modified ferrocement was considerably improved with an increase of polymer addition from 0% to 12.5% and fly ash addition from 0% to 30%, respectively.

Ramli and Tabassi (2012) evaluated the properties of poly-acrylic-ester (PAE), vinyl acetate-ethylene (VAE) and styrene-butadiene rubber (SBR) polymer-modified ferrocement (PMF) panels by adding 15% extra polymer by weight of cement and compared the results with unmodified ferrocement elements. Two types of curing conditions were adopted to conduct the tests as cured in saltwater exposure conditions and cured in air. Mix proportion of 1:3 was used in this study. The water-cement ratio was kept 0.40 for unmodified ferrocement

elements. The water-cement ratio of modified ferrocement was adjusted from 0.272 to 0.320 to obtain a designed slump value of 130 to 150mm. The superplasticizer (SP) was also used as and when necessary. Mortar specimens of size 100mm × 100mm × 50mm were cast in accordance with standard specifications to find the flexural strength and tensile strength. The ferrocement specimens of size 500mm × 100mm × 25 mm and reinforced with a volume fraction of 0.91% (with three layers of square welded wire mesh having the diameter of 2.2mm) were cast. The four-point load test was performed on the polymer-modified ferrocement to evaluate its flexural strength, crack width, crack spacing and deflection properties for different curing conditions. The test results showed that with the addition of polymers, the properties of ferrocement remarkably improved as compared to unmodified ferrocement even in salt water exposed environmental conditions.

Sheela and Ganesan (2014) evaluated the cracking characteristics of polymer modified ferrocement (PMF) flexural elements with two and the four layers of wire mesh having size of 6 × 22 and 4 × 20 gauge. They also investigated the strength properties of PMF with different surface shapes. Cement-sand ratio of 1:2 and w/c ratio of 0.50 was kept for all mixes. The polymer addition level was also varied from 0% to 4%. The different layers of wire mesh (varied from 2 to 4) were used for different sets of the specimens. It was concluded that the polymer increased the ductility of ferrocement and number of cracks were developed at failure level. The ultimate strength of PMF was also increased as compared to normal ferrocement specimens. They had studied the three different shapes as; channel, trapezoidal and corrugated and found that the corrugated shape of ferrocement performed better and showed the crack propagation among all the three shapes. In the case of corrugated shape, the widening of cracks was also slower and was increased gradually.

Kumar and Vidivelli (2017) also evaluated the performance of polymer modified mortar and ferrocement and recommended their application for repair and strengthening of RCC structural members. Cement to the sand ratio of 1:2 was taken in this experimental investigation work. Water to cement ratio of 0.40 was taken for control mortar specimens and 0.30 was taken for polymer-modified specimens. Polymer modified ferrocement specimens were prepared by using wire mesh with three different volume fractions of 3.55%, 5% and 6.43%. The mortar matrix of ferrocement was modified with 5%, 10 %, 15% and 20% addition of SBR polymers by a solid mass of polymer to the cement mass. Test results showed that the 1% addition of SBR polymer is the optimum value to get better results. The water absorption of polymer modified mortar after 24 hours was also reduced up to 82% with an addition of 15% SBR. The tensile strength of mortar increased with the increase in

addition of polymer percentage, which further improved the strength properties of ferrocement. Properties of PMF also varied with the variation of volume fraction of wire mesh and the PMF showed the best performance when the percentage of reinforcement varied from 5% to 6.43%. On the basis of this research work, they finally expected that the polymer-modified ferrocement would be the best and economical solution to strengthen the RCC structural elements.

It is clear from the above literature that better tensile strength, flexural strength, durability and better deformation properties of polymer modified ferrocement can be achieved by selecting the appropriate type of polymer, the water-cement ratio of mortar for ferrocement, type of wire mesh and number of wire mesh layers.

2.3.4 Polymer Modified Ferrocement (PMF) Used as Strengthening Material

In the past, a few experimental studies had been carried out to gauge the effect of high-performance ferrocement strengthening technique on RCC beams. The researches were carried out in past are on strengthening the beams in flexural only and very few or no literature is available on shear strengthening of beams with PMF.

Hughes and Evbuomwan (1993) used the polymer-modified ferrocement under the soffit of beams to strengthen the beams in flexure. A total of six RCC beams were used in this study. Out of which five beams were strengthened with the different volume fraction of wire mesh reinforcement. One beam was tested as a control beam. The repairing material was constituted with cement, sand, pulverized fly ash (PFA), polyvinyl alcohol (PVA), micro-silica and water in a ratio of 1: 2.13: 0.33: 0.0533: 0.05: 0.33. Three combinations of wire mesh reinforcement were used as; (i) two layers of steel mesh of 1.6mm diameter and 13mm spacing, (ii) five layers of polypropylene fabric mesh with a weft of 222 tex and 11.6% elongation at break and (iii) four 1mm glass fibre rods at 16mm spacing. Detail of ferrocement specimens, strengthening and testing is shown in Figure 2.11. Test results of the strengthened beams were compared with control beams and it was concluded that the ultimate flexural load carrying capacity and ductility of beams increased after strengthening and without any bond failure. The cracking load, ductility and ultimate load values increased in the range of 17 to 50%, 30 to 104% and 27 to 55%, respectively. The beams strengthened with polypropylene mesh exhibited a significant increase in deflection values and hence the ductility was also increased over 100% as compared to control beam specimen. This was mainly due to the high yield strain of the polypropylene mesh. The stiffness of beams was also enhanced with the application of PMF.

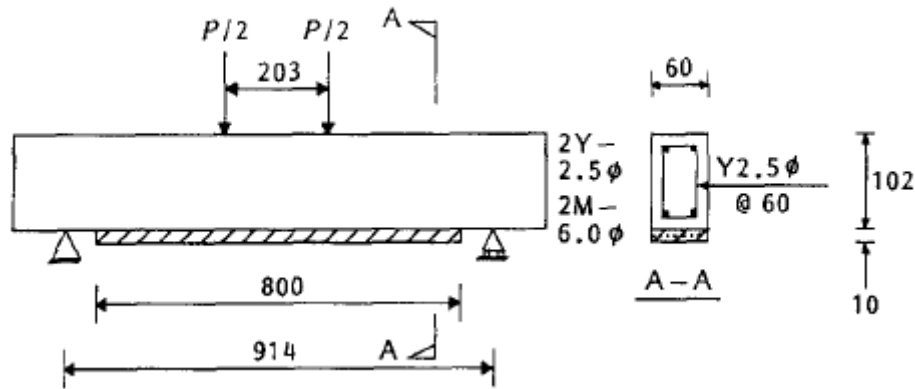


Fig. 2.11: Detail of Ferrocement Specimens, Strengthening and Testing

Rajkumar and Vidivelli (2010a) used the polymer modified ferrocement (PMF) to strengthen the flexural deficient RCC beams. Firstly, the mechanical properties of polymer modified ferrocement were investigated having different reinforcement volume fraction (V_r) of 3.55%, 5% and 6.43%. Styrene-butadiene rubber latex (SBR) polymer was used to modify the mortar matrix of ferrocement. The cement-sand ratio was fixed as 1:2 and polymer addition levels of 5%, 10 %, 15% and 20% (by mass of solid phase of SBR polymer) was adopted for this study. A total of 8 full-sized (125 mm $\times$ 250mm $\times$ 3200mm) RCC beams were cast to gauge the effect of PMF as a strengthening material. These beams were tested under two-point loading for static conditions. A typical set of two beams was used and initially, the control beams were tested to find out the ultimate load-carrying capacity. On the basis of test results of control beams, the remaining beams were pre-damaged for two different load levels. The typical setup for testing of control as well as strengthened beams is shown in Figure 2.12. These pre-damaged beams were then retrofitted with polymer-modified ferrocement having different reinforcement volume fractions as adopted during the initial investigation. Test results showed that the application of polymer modified ferrocement as a strengthening material caused to reduce the width and spacing of tensile cracks. A typical cracking pattern of control and strengthened beams is shown in Plate 2.4. The performance of PMF found better at 15% addition level of polymer with 5% volume fraction of wire mesh reinforcement. The deformation, crack width and rebar strain of strengthened beams reduced considerably as compared to control beams. The overall performance of beams was increased by 85% after application of PMF as a strengthening material.

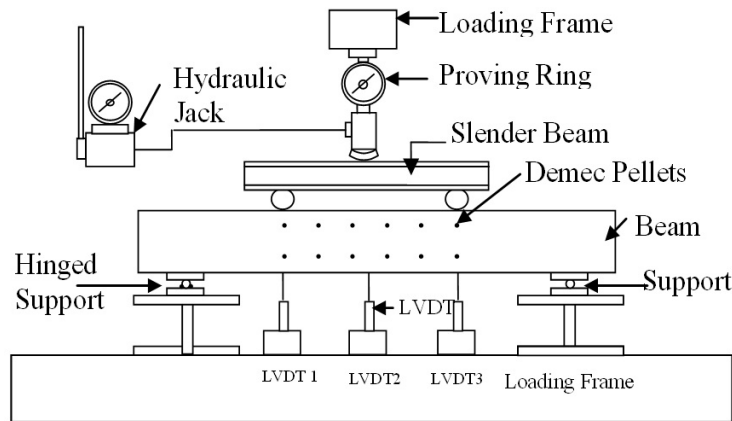
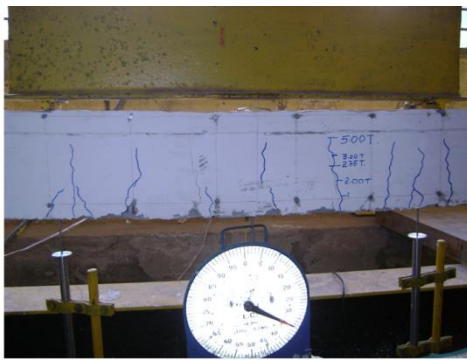


Fig: 2.12: Typical Setup for Testing of Beams



(a)



(b)

Plate 2.4: Typical Cracking Pattern; (a) Control Beam, (b) Strengthened Beam

Rajkumar and Vidivelli (2010b) have also studied the use of acrylic rubber latex to modify the ferrocement mortar matrix by choosing the similar experimental setup as previously discussed in the above-referred paper. They found that the ferrocement modified with acrylic rubber latex polymer is also a possible alternative retrofitting material for RCC structural elements. The polymer-modified ferrocement has shown the best test results at 15% polymer addition and 5% volume fraction of wire mesh reinforcement. The deformation properties, crack width and rebar strains of PMF strengthened beams showed an improvement of 79% in its overall performance as compared to the control beams.

Liao and Fang (2011) studied the flexural behaviours of RCC beams strengthened with high-performance ferrocement. Five full-size RCC beams (200mm × 400mm × 3800mm) were cast with M15 grade of concrete and reinforced with different percentage of tensile reinforcement. Out of these five beams, two beams were tested as control beams and three beams were strengthened by ferrocement. The beams were strengthened with U-shaped ferrocement which put on the soffit and two side faces with a thickness of 30mm and 25mm, respectively. The detail of strengthening is given in Figure 2.13. The high-performance

ferrocement constituted with mortar having a compressive strength of 36.9, 42.3 and 40.5 MPa and reinforced with 5 and 6 diameter bars at different spacing. They concluded that the beams strengthened with ferrocement showed higher initial stiffness as compared to the control beams. The strengthened beams exhibited higher crack load, yield load and ultimate load than control beams. The width of cracks was also reduced with the application of ferrocement.

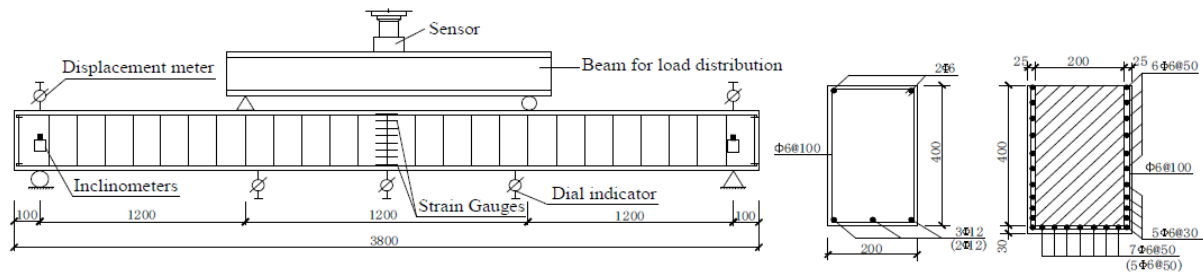


Fig: 2.13: Detail of Testing, Beam Size and Reinforcement

Zhao *et al.* (2012) studied the performance of RCC beams in shear after strengthened with steel wire polymer mortar. They conduct the experiment on three full-sized RCC beams grade C20 and size 200 mm × 300 mm × 2300 mm. The beams were reinforced with 2-25mm diameter bars at the bottom and 2-18mm diameter at the top. Shear reinforcement of 6mm diameter was provided at a spacing of 100mm c/c. The BS-1 was designated for control beam and BS-2 and BS-3 for strengthened beams. The BS-2 and BS-3 beams were strengthened with 25mm thick U-shaped steel wire polymer mortar. The steel wire mesh of 3.05mm diameter and c/c spacing of 30mm with tensile strength and MoE of 1870 MPa and 1.34×10^5 MPa, respectively was used for ferrocement. The polymer used for ferrocement having a compressive strength of 24MPa and tensile strength of 4.8MPa. The a/d ratio was kept as 1.87 for beam BS-1 and BS-2 and 2.28 for BS-3. The detail of beam specimens along with loading arrangement is given in Figure 2.14. To validate the test results, the beams were also modelled with finite element software named ANSYS. The FE model developed with ANSYS is also presented in Figure 2.15.

Experimental results showed that the shear capacity of BS-2 and BS-3 beams increased by 57% and 39.1%, respectively as compared to control beam BS-1. The beams tested for shorter a/d ratio showed shear failure, however, the beam tested at higher a/d showed shear-flexural failure. The strengthened beams did show a delay in the development of cracks as compared to the control beams. The cracks developed in strengthened beams were smaller and dense as compared to BS-1 beam. Plate 2.5 shows the cracking pattern of

BS-1, 2 and 3 beams. Analytical test results obtained from ANSYS were also showed good agreement with experimental test values.

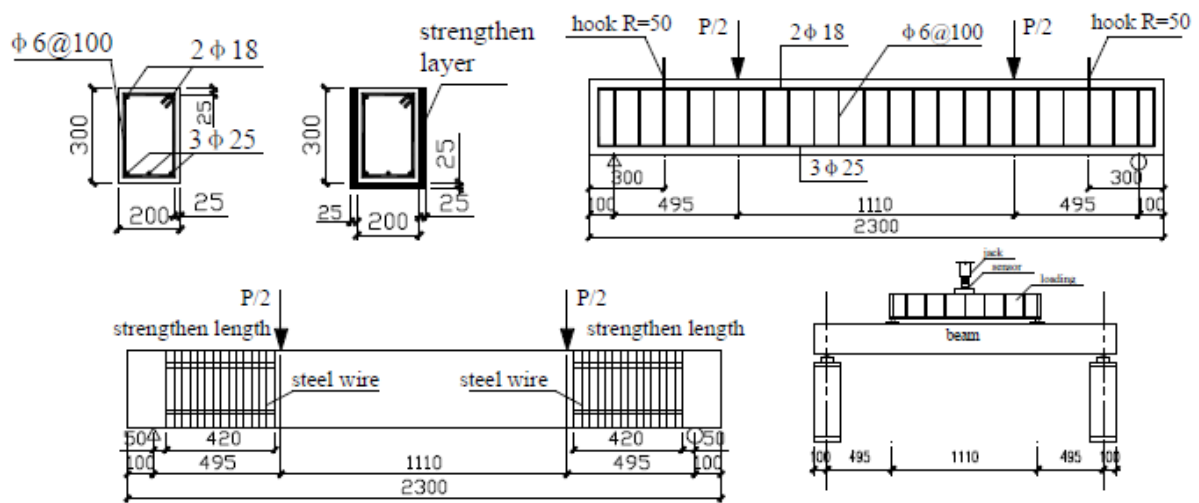


Fig. 2.14: Detail of Beam Specimens and Loading Arrangement

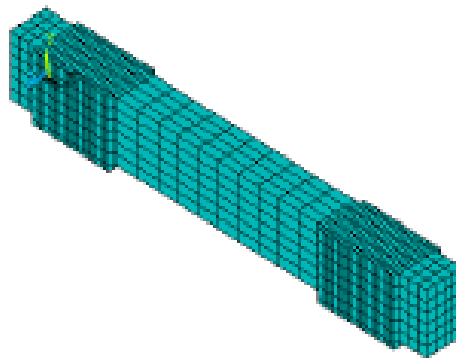


Fig. 2.15: Finite Element Modelling of RCC Strengthened Beam



Plate 2.5: Cracking Pattern of BS-1, BS-2 and BS-3 Beams

Ruben (2018) studied the flexural behaviour of reinforced concrete beams when strengthened by using polymer-modified ferrocement laminates. Styrene-butadiene-rubber (SBR) polymer was used to modify the conventional ferrocement matrix. The experimental results were reported and discussed w.r.t. strength, deflection, moment and curvature characteristic of the strengthened specimen. Eight beams of size 3200mm in length, 125mm

width and 250mm depth were cast. Out of those eight beams, two beams were considered as control specimen and tested for ultimate load by applying two-point loading. The remaining six beams were strengthened by ferrocement laminates with different volume fraction ($V_r = 3.554\%, 4.991\%, 6.429\%$).

It was concluded that the compressive strength and tensile strength of polymer-modified mortar was increased with the increase of SBR addition when compared with ordinary mortar. Ferrocement specimens having volume fraction of 3.554% shows good load-deflection behaviour as compared to 4.99% and 6.429% of volume fraction. The theoretical and experimental moments-curvature behaviour for control specimen compared with ferrocement laminated beams. The rehabilitation methods showed 16% strength improvement when compared to perfect beams.

Manoharan et al. (2019) also studied the behaviour of reinforced concrete beam damaged due to overloading and strengthened by using polymer ferrocement laminates. The laminates were directly glued into the fractured tension face of the beam by epoxy adhesives. Beams of size 3200 mm length, 125 mm width and 250 mm depth were casted and tested under two-point loading situation. The beams were subjected to overloading by applying 70% ultimate load for four beams and 80% ultimate load for two beams. The beams subjected to collapse situation were strengthened by polymer ferrocement laminates with two different volume fractions of 4.94 and 7.41%. It was found that the use of polymer-ferrocement laminates is an effective approach to strengthen the damaged beams. The theoretical and experimental moment-curvature activities observed in the case of control samples were compared with polymer ferrocement laminate beams, and the results were validated using ANSYS software.

2.4 STRENGTHENING OF RCC BEAMS IN SHEAR WITH OTHER MATERIALS

Worldwide researches are going on to explore the application of various materials such as; FRP, CFRP, GFRP to strengthen the RCC structural components in shear and bending. Some of the literature related to the use of these materials to strengthen the RCC beams in shear has been discussed below: -

Sheikh et al. (2002) studied the use of carbon and glass fiber-reinforced polymers (CFRP and GFRP) for retrofitting concrete structures for shear and flexure. They simulate and cast the RCC beam on a 5/6th scale. Slab and wall specimens were cast on a full scale. Different reinforcement details were adopted for different structural elements. Ready-mix

concrete of grade M30 was used in this work. The RCC beams were pre-loaded to damage these beams for shear and flexure, respectively. The flexure damaged beams were retrofitted with CFRP and GFRP for the full length of the beam and the shear damaged beams were retrofitted for a '2d' length of beams, respectively. Test results of retrofitted specimens showed that both FRPs are effective to strengthen the structure in flexure as well as in shear. The shear strengthening resulted to increase the ultimate displacement by more than 10 times and the toughness by more than 26 times. They also validate their results with mathematical modelling.

Taljsten et al. (2006) used CFRP (Carbon Fibre Reinforced Polymer) to strengthen the concrete beams in shear. They also investigated the effect of mineral-based composites as bonding agent instead of epoxy resins. The epoxy resins have disadvantages such as; thermal compatibility, diffusion and the temperature of assemble etc. Hence, the use of mineral based bonding agent such as; polymer-modified mortar was used to overcome these problems. The polymer-based material has similar properties of parent concrete and has more compatibility and environmental friendly. Therefore, the epoxy interface was replaced with polymer-based bonding agent. Five full-length RCC beams were cast and tested. One beam was considered as reference beam while the other four were strengthened with CFRP grids. All the beams were designed to fail in shear by providing sufficient flexural reinforcement. Test results showed that all the beams failed in shear and the load-carrying capacity of strengthened beams increased by 40 to 100%.

Bhuvaneshwari and Mohan (2015) studied the efficacy of glass fiber strips and cement-based polymer composite materials when used to strengthen the pre-damaged RCC shear deficient beams. The test was performed on 6 full size 100mm ×150mm ×1500mm RCC beam of M30 grade. The beams were reinforced with 2-10mm diameter bars at top and bottom. The three beams were considered as control beams (RB) and reinforced with shear stirrups at c/c spacing of 100mm. And three beams were considered as shear deficient beams (SD) and stirrups were provided at a spacing of 400mm. The control beams were tested under four-point bending up to ultimate failure and the behaviour of these beams was noted. The SD beams were loaded to simulate pre-damage in beams before retrofitting. The cracks of the damaged beams were repaired with SBR polymer-based mix having cement to the sand ratio of 1:3, w/c ratio 0.44 and polymer content of 20%. After the application of polymer composite, the beams were strengthened with glass fiber strips. Non-linear FE analysis was made by using ANSYS to validate the test results. Test results showed that the load-carrying capacity of shear deficient retrofitted beams increased by 18.63% as compared to reference

beams. The ANSYS modelling also resulted in enhancement of strength by 21.62%. Ductile failure of beams was observed with more cracks shifted in the flexural mid-span region. The polymer composite mortar also proved its effectiveness to arrest the shear cracks. The deflection values at the ultimate failure load were also reduced by 11.1% after strengthening. ANSYS results also showed good agreement with the experimental values and showed the improvement in deflection by 12.43%. The ultimate moment and stiffness improved by 13.51% and 31.35%, respectively. It was concluded that the cement-polymer binder and glass sheet composite enhanced the strength and stiffness of damaged beams even after deficiency of shear reinforcement. The numerical analysis also validates the test results.

Roy *et al.* (2015) also evaluated the performance of glass fiber-reinforced polymer (GFRP) laminates used to strengthen the heat damaged T-beams. They cast 21 number RCC beams of length 1400mm. 2-12mm diameter bars were used as tension reinforcement and 4-8mm diameter bars were used as compressive reinforcement. The detail of beam specimens is shown in Figure 2.16.

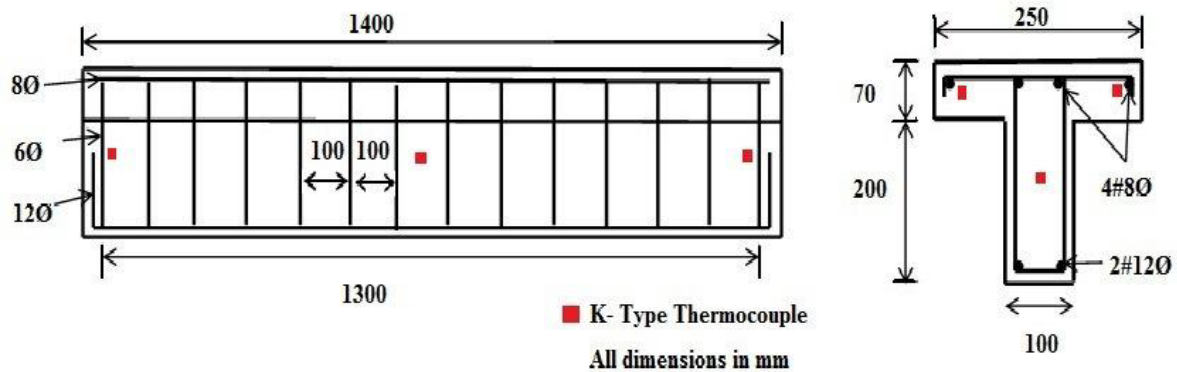


Fig. 2.16: Detail of T-beams

These beams were heated to 600°C and 900°C with the help of electric furnace after the age of 90 days. Initially, the controlled beams were tested at room temperature and other sets of beams were damaged with heat and then retrofitted with different schemes of FRP laminates. The pattern adopted to strengthen the beams is shown in Figure 2.17.

The test results concluded that the ductility was decreased with the heat damage of 600°C and 900°C by 4% and 32%, respectively. The load-carrying capacity was also decreased by 14% and 61%. The application of GFRP improved the load-carrying capacity of beams damaged with 600°C heat; however, the 900°C heat-damaged beams showed lesser strength results.

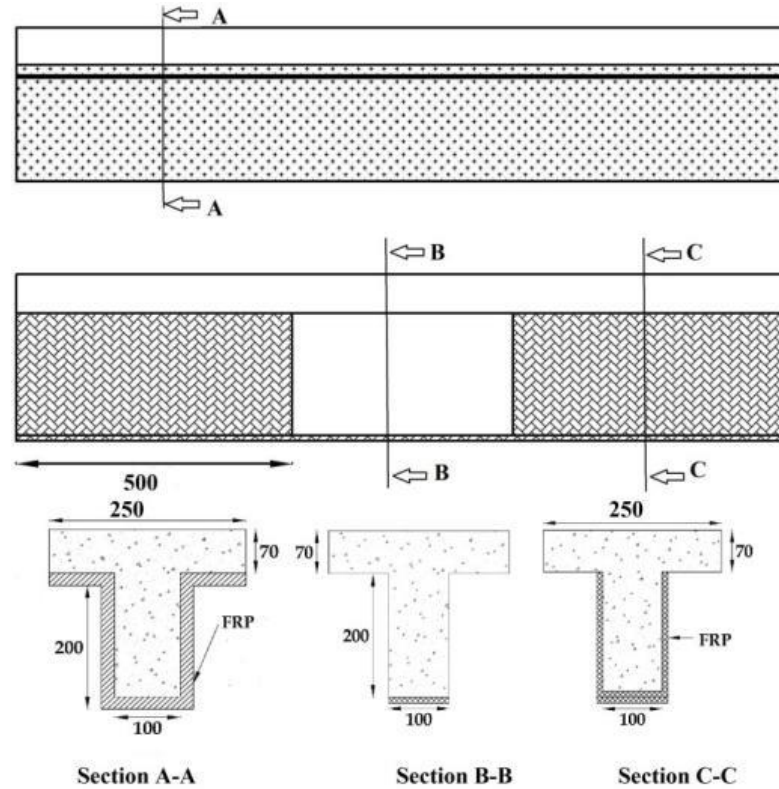


Fig. 2.17: Pattern of Strengthening

On a similar lines, the research work has been done by *Santhakumar and Chandrasekaran (2004)*, *Jayaprakash (2008)*, *Jayaprakash (2010)*, *Bukhari (2010)*, *Singh (2013)* on CFRP and by *Saafan (2006)*, *Sundarraja (2009)* and *Kim (2015)* on GFRP and by *Mosallam (2007)* and *Kim (2008)* on FRP when used to strengthen the RCC beams in shear and also studied the other variable parameters and the orientation of strengthening material.

Carlioni and Subramaniam (2012) have analysed the bond characteristics of FRP when used for strengthening, repairing and rehabilitation of reinforced concrete structural members. The debonding of FRP is one of the critical concerns when its used to strengthen the members suspected for fatigue loading conditions. In this study, the FRP-concrete shear debonding has been idealized as fracture problem along with the bi-material interface. This study was also correlated with the other experimental work done by the authors. Figure 2.18 shows a typical load versus global slip response obtained from the direct shear test. This study concluded that the FRP-concrete width ratio was not fully understood. Further, the fracture energy cannot be directly correlated to the experimental load-carrying capacity of the interface.

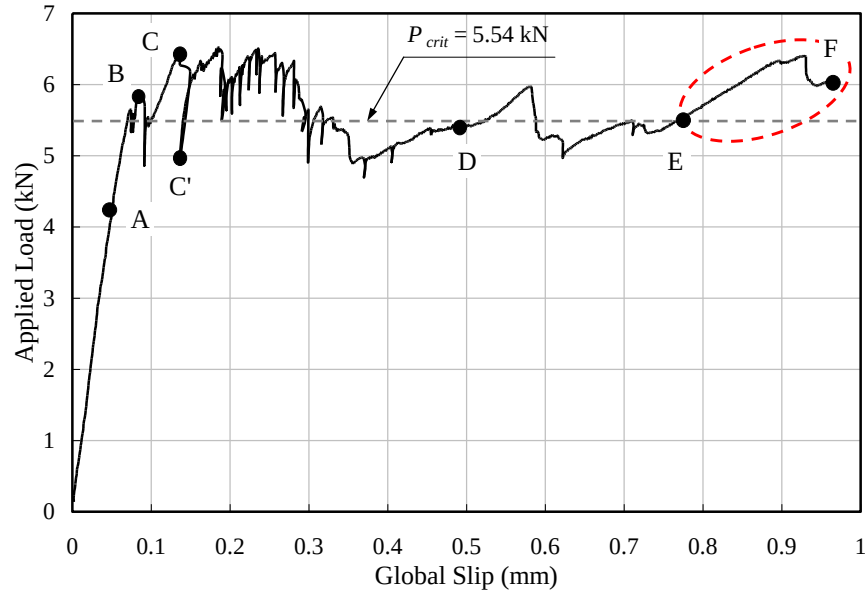


Fig: 2.18: Load versus Global Slip Response

Sarsam *et al.* (2017) presented an experimental investigation of structural behaviour of reinforced concrete deep beams strengthened in shear by CFRP strips. The experimental program consisted of fabricating, casting and testing of nine identical porcelaine lightweight aggregate reinforced concrete deep beams. Three of the tested deep beams were unstrengthened to serve as reference beams, while the remaining beams were tested after being strengthened using CFRP strips in two different orientations (vertical and horizontal). The locally available natural porcelanite aggregate was used to produce lightweight aggregate concrete. The beams were designed to satisfy the requirements of ACI 318M-14 building code. In order to insure shear failure modes, adequate flexural steel reinforcement were provided. Effect of three different values of shear-span to effective-depth ratio ($a/d = 1.0, 0.8, 1.2$) were selected. All beams have been tested as a simply supported beams subjected to two concentrated points loading. The beam specimens were tested up to failure under monotonic loads. The experimental work showed that the failure load increases as the shear-span to effective-depth ratio deceases. As the shear-span to effective-depth ratio decreased from 1.0 to 0.8, the percentage of increase in the ultimate load was about 24%. In addition, the diagonal compression strut crack of unstrengthened control beams was changed to several diagonal cracks at mid depth within the shear span of the strengthened beams and exhibited more ductile failure mode. They also conclude that the use of CFRP strips in the strengthening lightweight aggregate deep beams reduces deflections by about 50% and increases the load carrying capacity by about 45%. Cracks were smaller and more distributed in the strengthened beams compared with their controls ones. The load-displacement

response of the tested deep beams strengthened with CFRP strips was influenced by changing the a/d ratio in such a manner that more deformability was achieved with the increasing of a/d ratio, which was due to the reduction of arch action effect. CFRP composite materials help to reduce the growth of diagonal cracks and further reducing their progression into the compression zone. This phenomenon provides more uncracked concrete section at the head of the crack in the compression zone which enhances the shear resistance contributed by concrete. There was no debonding failure of CFRP was observed.

Mishra and Jain (2019) studied the behaviour of RCC beams strengthened with waste flex banner and steel wire mesh. Total 12 number of RCC beam specimen were prepared and tested under flexure. Commonly available steel wire mesh - chicken mesh and weld mesh were bonded on the surfaces of different beams using carpet/tile adhesive. Beams other than control beams were finally wrapped and pasted with waste PVC flex banners. Load carrying capacity of different beam specimen was compared. It was found that the load carrying capacity of beams were increased by 6% when wrapping and pasting PVC flex banner, by 21% when chicken mesh and flex banner were provided on beam and by 147% when weld mesh and flex banner were provided on beam. It was concluded that by using commonly available material and simple techniques, the load carrying capacity of RCC beams can be significantly increased with as economical way. Waste flex banner wrapping adds to the durability of the RCC beams besides helping in plastic waste management.

All these strengthening materials showed the better performance to restore the original strength of the structural components. However, some deficiencies like; higher cost, required skill labour, thermal compatibility of epoxy and minimum temperature of assembling, diffusion closeness, working environment etc. have been observed from the literature review, which makes their application on the back foot. Hence, the application of ferrocement and advanced ferrocement modified with different polymers is the next viable option as strengthening material. It needs lesser or no skilled labour and the overall application cost is also very less as compared to the other high-end materials.

2.5 FINITE ELEMENT (FE) MODELLING OF RCC BEAMS

Worldwide, many researchers have validated their research work by using various finite element based software such as; ANSYS, ABAQUS, SAP2000, LUSAS and ATENA etc. Past researches concluded that the FE modelling showed good agreement with the experimental test results with small variations. Some of the work related to the application of different FE softwares for different works have been studied and detailed as below: -

Kachlakev et al. (2001) reported the work related to finite element modelling of reinforced concrete structures strengthened with FRP laminates. ANSYS software was used for this work. The full-size control and retrofitted beams were modelled. The study concluded that the load-deflection behaviour at mid-span of the beam showed good agreement with the test data. However, the FE modelling showed slightly more stiffness as compared to the actual test data. The bond-slip and micro-cracks effect has been ignored in the FE model which causes an increase of stiffness. The load-strain graphs also showed similar behaviour as experimentally obtained. The mode of failure and crack pattern of FE modelled beams was also observed and found similar to the experimental beams.

Ibrahim and Mahmood (2009) also used ANSYS to model RCC beams strengthened in shear with CFRP and GFRP. They create six FE models for different conditions. The RCC beams of size $150\text{mm} \times 250\text{mm} \times 3620\text{mm}$ were modelled for shear loading as shown in Figure 2.19. The beams B-1 and B-2 were designated as control beams; beam B1C-90 and B2C-90 designated as strengthened beams with one and two layers of CFRP, respectively and beam B1G-90 and B2G-90 designated for beams strengthened with one and two layers of GFRP, respectively (refer Figure 2.19 and 2.20). The FE element models were developed by using three-dimensional layered elements for the FRP composites and smeared cracking approach for concrete (refer Figure 2.21). The Solid65 element was used to model the concrete, Link8 was used for reinforcement and Solid46 for FRP layered composite, respectively. The experimental load-deflection curves at mid-span and failure loads were compared with the FE analysis results. The FE analysis showed good agreement with the experimental test results. The load-deflection curves of FE analysis were almost overlapping with the experimental curves when compared in a linear zone, however, there was a slight variation in a non-linear zone and FE analysis showed stiffer results as compared to experimental results. The maximum variations in the results were in the range of 7.8% for all the cases.

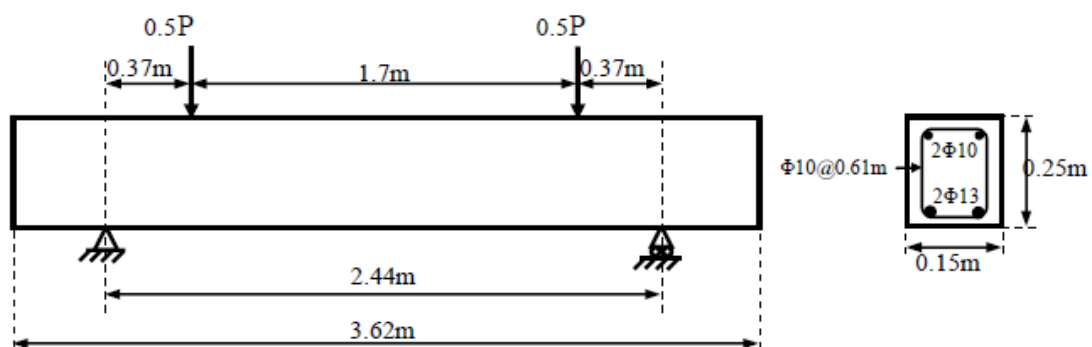


Fig. 2.19: Dimension and Reinforcement of Beam B1

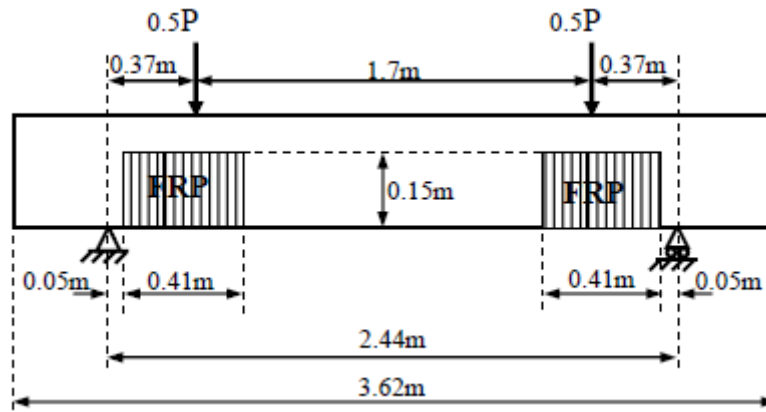


Fig. 2.20: Shear Strengthening Details for Beams B1C-90 and B1G-90

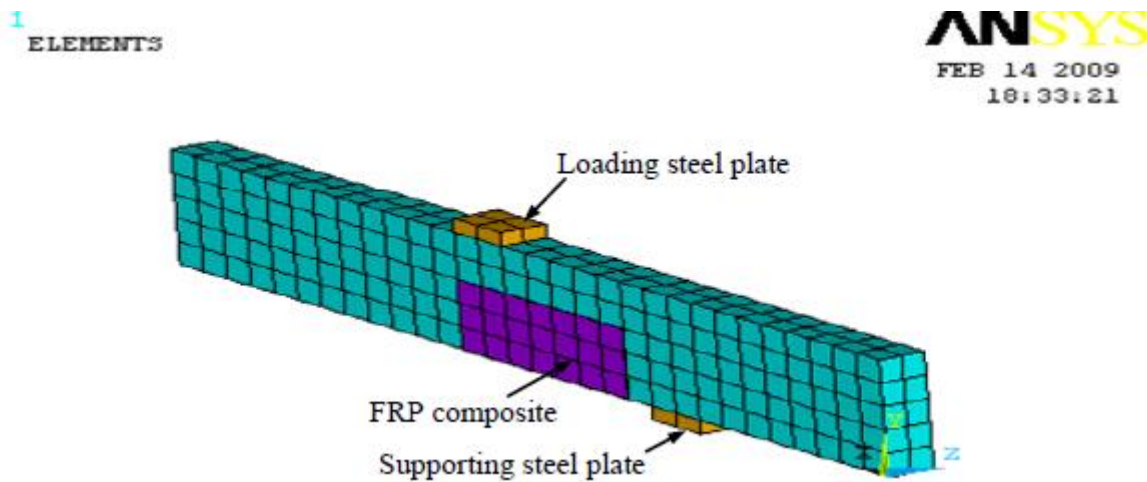


Fig. 2.21: Finite Element Modelling for B1C-90 and B1G-90

Büyükkaragöz (2010) used ANSYS to model the beams strengthened with a prefabricated concrete plate (Figure 2.22). The results obtained by FE analysis were compared with the experimentally obtained results. It was found that the FE model gave similar results as observed experimentally. He proposed to deploy this modelling to save time and money and use it to predict the actual behaviour of specimens without experimental testing.

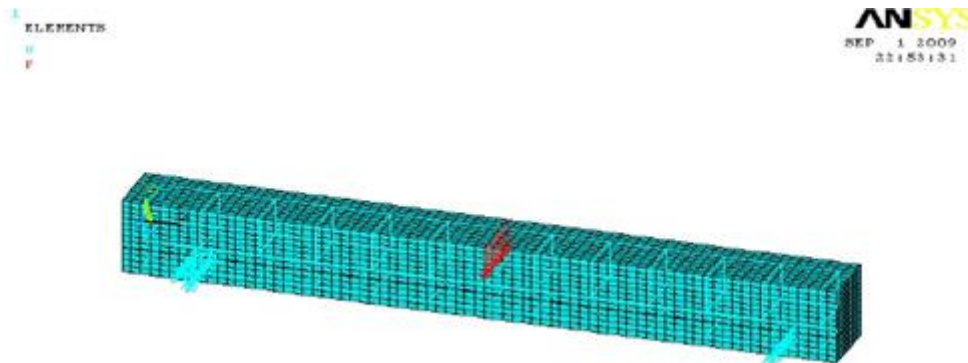


Fig. 2.22: Loads and Supports of Strengthened Specimen

Biscaia et al. (2014) experimentally tested the RCC T-beams when retrofitted with GFRP and then three dimensional smeared crack models were created in ATENA software. The FE model of T-beam having length 3m depth 0.28m, flange width 0.4m, flange thickness 0.08m, web thickness 0.15m was created. A total of 22 computational analysis were created on FE software (refer Figure 2.23).

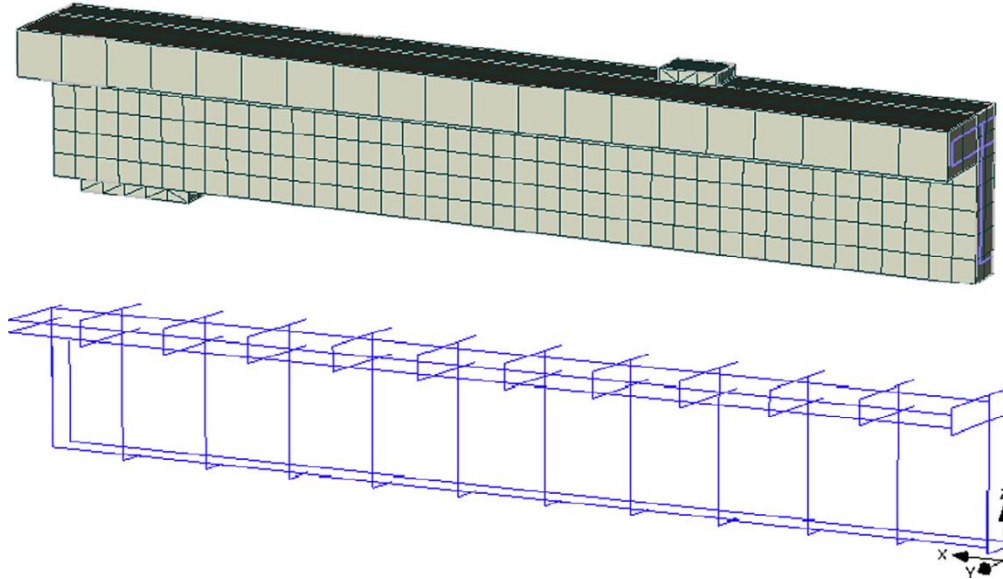


Fig. 2.23: Standard Mesh Discretization and Steel Reinforcement of the RC T-Beams

These models were made with Mohr-Coulomb failure criteria and the interface between concrete and FRP was defined by considering different strength types. The retrofitted T-beams were showing an increase of flexural strength as compared to the unretrofitted beams. The FE model has also confirmed the effective approach to strengthening the RCC beams. The rigid interface approach made the results output slightly higher and reflects the beams results as stiffer. They also compared the results by considering rotated and fixed crack approach and it was found that the predicted results of T-beams rupture showed improvement when the model was made with the rotated crack approach. It was concluded that the FE analysis predicted good results of externally bonded GFRP composites T-beams.

Jongvivatsakul et al. (2019) investigate the performances of reinforced concrete (RC) beams strengthened in shear with steel fiber-reinforced concrete (SFRC) panels are through experiment, analytical computation, and numerical analysis. An experimental program of RC beams strengthened by using SFRC panels, which were attached to both sides of the beams, was carried out to investigate the effects of fiber volume fraction, connection type, and number and diameter of bolts on the structural responses of the retrofitted beams. The current

shear resisting model was also employed to discuss the test data considering shear contribution of SFRC panels. The experimental results indicated that the shear effectiveness of the beams strengthened by using SFRC panels is significantly improved. A three-dimensional (3D) nonlinear finite element (FE) analysis adopting ABAQUS was also conducted to simulate the beams strengthened in shear with SFRC panels. They conclude that the shear capacity of RC beams significantly increased when the RC beams were strengthened using SFRC panels. This investigation revealed good agreement between the experimental and analytical results in terms of the mechanical behaviours.

2.6 EFFECT OF VARIOUS PARAMETERS ON THE SHEAR STRENGTH OF RCC BEAMS

The shear behaviour of RCC beams mainly depends upon the shear stress of uncracked concrete, arch action and dowel action, shear transfer interface, shear span to depth ratio (a/d) and effective span to depth ratio. Out of these, the a/d ratio is one of the important governing factor which is responsible for the shear behaviour of RCC beams. Literature pertaining to the effect of a/d ratio and other variables on the shear strength of RCC beams is presented below: -

Tan et al. (1995) studied the behaviour of high-strength concrete deep beams with effective span and shear span variations of nineteen deep beams with compressive strengths in the range of 41 MPa to 59 MPa. The beams were tested under two-point loading. All the beams were singly reinforced with main steel percentage of 1.23 percent and with nominal percentage of shear reinforcement of 0.48 percent. The beams were tested for seven shear span-depth ratios (a/d), ranging from 0.27 to 2.70, and four effective span-depth ratios (l_e/d) ranging from 2.15 to 5.38. Test results indicated that l_e/d has little influence on the magnitude of the failure load. But for beams with a/d greater than or equal to 1.00, the flexural failure mode becomes dominant with increasing l_e/d .

Fathifazl et al. (2009) investigate the shear performance of reinforced concrete (RC) beams made with coarse recycled concrete aggregate (RCA). In this work, RCA was treated as a two-phase material comprising mortar and natural aggregate therefore to proportion the concrete mixture with RCA, the relative amount and properties of each phase were considered separately. Several beams were designed and tested to study the effect of a number of parameters including the shear span-to-depth (a/d) ratio and beam size on the serviceability and strength of RCA concrete beams without shear reinforcement. For each beam, its load-deflection curve, shear deformations, diagonal cracking load, crack pattern,

ultimate shear strength and failure mode were determined. The results showed that the shear performance of RC beams made with RCA can be comparable, or even superior, to that of beams made entirely with natural aggregates at both serviceability and ultimate limit states. Furthermore, the simplified methods of ACI and CSA standards as well as Eurocode 2 were found applicable to all reinforced RCA-concrete beams. It was concluded that the shear strength of RRC beams increased as the a/d ratio decreased, irrespective of the source, mainly owing to the higher contribution of the arch mechanism to the shear resistance at lower a/d ratios. For the same a/d ratio, concrete compressive strength and beam depth, the effect of aggregate type (RCA against natural aggregate) on the shear strength of RRC beams was found to be negligible.

Reddy et al. (2011) studied the improvement of shear strength of high strength concrete beams (70 MPa) with different shear span to depth ratios ($a/d = 1, 2, 3$, and 4) and various dosages of fibers (0.4%, 0.8%, and 1.2% by volume of concrete), without shear reinforcement. The experimental work revealed that steel fiber volume has different influence at different shear span to depth ratios (a/d). The test results indicated an increase in the cracking shear resistance noticeably and ultimate shear strength moderately. It was concluded that the first crack load and the ultimate load of fiber reinforced concrete elements is considerable at lower a/d ratios, i.e. at a/d less than two. At higher a/d ratios, the influence of fiber on shear capacity of fiber reinforced elements was small. It was also reported that the shear capacity of beams was maximum increased at 0.8% volume fraction of fibers for all a/d ratios.

Naik and Kute (2013) predicted the shear strength of high-strength steel fiber-reinforced concrete deep beams. They studied the effect of clear span-to-overall depth ratio on shear capacity of steel fiber high-strength deep beams using artificial neural network (ANN8). The three-layered model was developed having eight input nodes which represent width, effective depth, volume fraction, fiber aspect ratio and shear span-to-depth ratio, longitudinal steel, compressive strength of concrete, and clear span-to-overall depth ratio. The model predicts the shear strength of high-strength steel fiber deep beams to be reasonably good when compared with the results of proposed equations by researchers as well as the results obtained by neural network (ANN7) which was developed for seven inputs excluding span-to-depth ratio. The developed neural network ANN8 proves the versatility of artificial neural networks to establish the relations between various parameters affecting complex behaviour of steel fiber-reinforced concrete deep beams and costly experimental processes. They conclude that the size effect is predominant in decreasing the shear strength

and must be taken into consideration as an important factor in analysis and design of high-strength steel fiber-reinforced concrete (SFRC) deep beams. The result predicted by developed ANN8 resembles with the findings of previous researchers. They also conclude that the various parameters affecting the shear strength of SFRC deep beams and make the analysis more complex and there is need to study the behaviour of high-strength SFRC deep beams by suitable combinations of the parameters.

Alhamad et al. (2017) investigate the shear behaviour of deep concrete beams reinforced with basalt fiber reinforced polymer (BFRP) bars for flexure without web reinforcements. The experimental testing performed on 4 short beams, three of which were reinforced with BFRP and one beam was reinforced with steel bars as counterpart for reference. The primary test variable was the shear-span-to-effective-depth ratio (a/d) and its influence on the beams' mid-span deflections, shear capacity, load-deformation relationships and the failure modes. It was found that all beams failed by a diagonal crack that extended from the support to nearest loading point. Also, higher ductility was noted near failure for beams that showed top concrete crushing in addition to typical diagonal shear crack. The overall BFRP-RC beams exhibited higher mid-span deflections and less stiffness compared to their steel counterparts. This was attributed to the low modulus of elasticity of the BFRP bars in comparison to steel bars. Increasing the a/d ratio increased the mid-span deflection due to the tendency of beams with large a/d ratios to behave as a flexure-critical member rather than a shear-critical one. Moreover, a linear correlation between the shear capacities and the cubic root of a/d ratio was evident from experimental result which was agreed with the CSA S806-12 code. This correlation was attributed to the inclination of the angle between the diagonal compression struts and the horizontal tie which resulted in improving the beam's arch-action mechanism.

Hu and Wu (2018) studied that the additive model of shear strength of reinforced concrete (RC) members, i.e. shear strength (V) is equal to the sum of the contribution from concrete (V_c) and that from transverse reinforcement (V_s), has been widely accepted in the literature and engineering practice. Shear span-to-depth ratio (a/d) is known to be a significant factor affecting V of RC members. However, very few quantitative studies on the influence of a/d on V_c and V_s have been reported in the literature. Another issue is related to the controversial relationship between the shear force at first diagonal cracking (V_{cr}) and V_c , for which different guidelines are given in ACI, AASHTO LRFD and CSA codes. They have tested 11 RC beams to directly measure the V_c and V_s , and to provide experimental evidence for these issues. The experimental results showed that V_c can be very different

from V_{cr} . At small shear span-to-depth ratio (a/d), V_c was much larger than V_{cr} , while it was opposite for the beams with a large a/d value. All the stirrups were not crossed the critical shear crack yield at ultimate shear strength, and V_c as well as V_s are not constant under increasing member deformation. They also conclude that the design codes given a conservative prediction of V , as un-conservative value of V_c was predicted at large a/d .

2.7 SUMMARY

Thus, this could be concluded that the application of polymer modified ferrocement as an outer strengthening material was a viable technology for improving the structural performance of RCC beams in flexure. At present, no such research work is available related to study the effects of polymer-modified ferrocement used to strengthen the RCC beams in shear. It is therefore desirable to explore the utility of PMF as a strengthening material in the specific domain of shear deficient beams. Many factors impart shear strength to RCC beams like; a/d ratio, spacing of stirrups and grade of concrete etc. In this experimental work, two variable factors viz. a/d ratio and spacing of stirrups are chosen to study their effect on strength, deflection and cracking pattern of pre-damaged beams strengthened with PMF. SBR and VAE polymers are selected to modify the properties of ferrocement and mortar. It is also decided to study the effect of two-layer and three-layer square woven steel wire mesh on the properties of polymer modified ferrocement. The best performing ferrocement is then further proposed to be used to strengthen the pre-damaged RCC beams deficient in shear. FE analysis is to be carried out to validate the test results.

2.8 GAPS IN PRESENT STUDY

All the structural elements are designed and constructed to resist a particular type of loading condition and to withstand different environmental conditions. Still, many structural elements like beams, columns etc. often require repairs to restore their structural integrity and also to protect them from severe weathering condition. In recent years, the premature deterioration of RCC structures has become a major problem. Hence, it is necessary to develop efficient repair and strengthening systems from the viewpoint of sustaining the structural performance during their designed service life.

Several types of materials and techniques are available for repairing and strengthening of existing deteriorated reinforced concrete members, but compatibility and strength properties need to be studied for a particular application of these materials. From the literature review, it is summarized that the use of polymer-based mortars as strengthening and

retrofitting material for RCC beams has performed in a better way as compared to other materials of the same class. It can be further said that the use of polymer modified ferrocement (PMF) is a viable approach to sustain and increase the flexural and shear strength of deficient RCC structural elements.

The use of PMF as strengthening material for RCC beams has been investigated for various parametric variations in flexure, but considerable investigations for shear have not been carried out in the past. So, it is the need of the hour to develop an advanced strengthening material which performs better in shear. Hence, it is proposed to explore the use of polymer modified ferrocement (PMF) to strengthen the shear deficient reinforced concrete beams. FEM modelling is also proposed to validate the experimental test results so obtained.

2.9 RESEARCH OBJECTIVES

This study intends to investigate the performance of polymer modified ferrocement (PMF) when used to strengthen the RCC beams in shear. It is found from the literature review that uses of vinyl-acetate-ethylene (VAE) and styrene-butadiene-rubber Latex (SBR) polymers are very effective to improve the properties of conventional ferrocement. Thus, polymer modified ferrocement (PMF) has been developed by replacing the conventional cement-sand mortar with polymer modified mortar (PMM).

On the basis of critical literature review and the available gaps, the main objectives of the present investigation are set as below:-

- i. To evaluate the salient properties of the selected polymer (VAE vinyl acetate and ethylene polymer, SBR styrene-butadiene rubber Latex) and evaluate its suitability for use as a constituent of retrofitting material for RCC structural elements.
- ii. To evaluate the physical properties of Polymer Modified Ferrocement (PMF).
- iii. To study the structural response of RCC beams retrofitted using PMF, designed for different modes of failure (in shear).
- iv. To develop a mathematical model for retrofit using PMF.

PILOT STUDY FOR POLYMER MODIFIED FERROCEMENT (PMF)

3.1 GENERAL

There is not much literature available, particularly in the area related to properties of polymer modified ferrocement (PMF) and polymer modified mortar (PMM) and its application for the strengthening of RCC beams in shear. Hence, it was important to evaluate the properties of PMF initially because these materials have to be used further for strengthening purpose of RCC beams. Thus, a pilot study was designed to finalize the parameters for the final study. The work is divided into two phases, in the first phase; two types of commercially available polymers named; Styrene-Butadiene-Rubber (SBR) and Vinyl-Acetate-Ethylene (VAE) were used to evaluate the properties of mortars modified with different percentages of polymers. In the second phase, properties of polymer modified ferrocement having different percentages of polymers, with two and three layers of steel woven wire mesh were evaluated. The ferrocement composition which showed the best results with the optimum percentage of a particular polymer type and an optimum number of wire mesh layers has then subsequently been used for strengthening of beams in the main phase of the study.

It has been observed from the literature that SBR and VAE polymers show best results when used to modify the mortar matrix and hence, have been chosen to modify the matrix of mortar and ferrocement in the present experimental work. The properties of conventional cement mortar and ferrocement were observed to have improved with the addition of both SBR and VAE polymers as evident from performed work. The optimum percentage of each type of polymer has also been worked out in the pilot study. Initially, the basic properties of all materials like cement, sand and aggregates were found out experimentally, and then the main testing work commenced as detailed in the succeeding sections.

Thus, as could be seen from the earlier reported works, the polymer modified ferrocement were found to perform better as compared to ordinary ferrocement. In order to study the efficacy of polymers for modification of ferrocement mortar matrix, an experimental determination of the flexural and tensile strengths of PMF elements was felt necessary and thus, it became the aim of the present pilot study. With this as a backdrop, investigations towards the effect of the addition of SBR and VAE polymers on the physical

and mechanical properties of mortar and ferrocement were carried out. As reported by *Bhattacharyya et al. (2003)*, the flexural moment capacity of ferrocement elements also increases with increase in the volume fraction of reinforcement, ferrocement elements with two and three-layered wire mesh were tested. The number of wire mesh layers has been restricted to three by keeping in view the practicability of use of PMF as strengthening material in structural elements in the field.

3.2 MATERIALS

The basic properties of materials used in this experimental work were found out by conducting tests as per the standard codes of practice and are described in detail.

3.2.1 Cement

Portland Pozzolana Cement (PCC) brand name ACC has been used in this experimental work. All major properties of cement have been found in the laboratory as per *IS:1489 (Part-1):1991* specifications and procedures. The detail of the test results is given in Table 3.1.

Table 3.1: Physical Properties of Portland Pozzolana Cement

S. No.	Characteristics	Value specified by <i>IS:1489 (Part-1):1991</i>	Test value
1.	Standard consistency (%)	-	34
2.	Fineness of cement as retained on 90 μ m sieve (%)	< 10	2.1
3.	Setting time (minutes)		
	Initial	30	98
	Final	600	240
4.	Soundness (mm) (Le-Chatelier's expansion)	< 10 (Fresh Cement) < 5 (Old Cement)	1.0
5.	Specific Gravity (Specific gravity bottle)	-	3.08
6.	Compressive Strength (MPa)		
	7 days (168 $\pm$ 2 hour)	22.0	23.1
	28 days (672 $\pm$ 4 hour)	33.0	34.2

3.2.2 Fine Aggregate (FA-1)

Natural river bed sand (FA-1) of fineness 2.24 has been used to prepare mortar mixes and ferrocement. The properties of sand have been worked out in the laboratory as per *IS 383:1970* specifications and are detailed in Table 3.2 below: -

Table 3.2: Physical Properties of Sand (FA-1)

S. No.	Characteristic	Value
1.	Type	Natural Sand
2.	Specific Gravity (oven dry basis)	2.67
3.	Fineness Modules	2.24
4.	Water absorption (%)	1.9
5.	Moisture content (%)	0.22
6.	Grading Zone	III

3.2.3 Steel Wire Mesh

Galvanised square woven wire mesh of 0.49mm diameter with a centre-to-centre spacing of 8mm having an ultimate tensile strength of 950 N/mm² has been used in the polymer modified ferrocement as per *ACI 549.1R* guidelines.

3.2.4 Water

Clean and fresh, drinking quality potable water has been used in the present study as per Indian standard *IS 456-2000*. It has been used for casting and curing of the specimens, for both mortar as well as concrete.

3.2.5 Polymers

Two types of commercially available polymers termed; (a) Styrene-butadiene-rubber (SBR) base “Sika[®] Latex Power” in liquid form having 45% solid contents and (b) Vinyl-acetate-ethylene (VAE) polymer powder Wacker 5010N (*VINNAPAS[®]*) VAE having 98-100% solid contents have been used to modify the properties of conventional cement mortar and ferrocement. The polymers have been added in different percentage dosages of 0%, 5%, 10%, 15% and 20% by weight of in the mortar matrices. The detailed properties of SBR and VAE polymers are listed in Tables 3.3 & 3.4. The images of polymers are shown in Plate 3.1 and 3.2. An anti-foaming agent, of 0.7% silicon solid, by weight of polymers, was also used in polymer modified mortar (PMM) and polymer modified ferrocement (PMF) matrices (*Afridi et al., 1995*).

Table 3.3: Properties of SBR Sika Latex (supplied by manufacturer)

S. No.	Specification data	Value
1.	Appearance	Milky white pourable liquid
2.	Total solids (%)	45.0±1.0
3.	pH value at 25°C	8.50±0.5
4.	Odour	Slight
5.	Specific Gravity (kg/l)	1.010-1.022

Table 3.4: Properties of VAE Wacker Polymer Powder (supplied by manufacturer)

S. No.	Specification data	Value
1.	Solids content (%)	98-100
2.	Ash Content (%)	9-13
3.	Bulk density (kg/m ³)	490-590
4.	Appearance	White to pale yellow powder
5.	Protective colloid / emulsifier system	Polyvinyl alcohol
6.	Particle size % (over 400µm)	Maximum 4
7.	Predominant particle size re-dispersion (µm)	0.5-8
8.	Minimum film forming temperature (°C)	4
9.	Film properties of the re-dispersion	cloudy, tough-elastic



Plate 3.1: SBR Latex in Liquid Form



Plate 3.2: VAE in Powder Form

3.3 MORTAR MIX FOR POLYMER MODIFIED FERROCEMENT (PMF)

3.3.1 Mix Details

In the first phase of the study, Polymer Modified Mortar (PMM) was initially prepared by adding different percentages of polymers to the conventional cement mortar (CCM) matrix. To prepare CCM matrix, cement and fine aggregates (FA-1) were mixed in the proportion of 1:2 by weight. The CCM was modified by addition of 5%, 10%, 15% and 20% (by weight of cement) of two different types of polymers i.e. SBR and VAE polymers. In the first case, the water/cement (w/c) ratio of CCM was worked out as 0.56 for flow value of $105\pm5\%$, when tested as per *ASTM C 1437-07*. The same water-cement ratio of 0.56 was kept for other mixes and the mixes having constant w/c ratio of 0.56 and modified with different percentage addition of SBR and VAE polymers are designated as S11, S21, S31, S41 and V11, V21, V31, V41, respectively. The detail of mixes is given in Table 3.5.

Table 3.5: Mix Details and w/c Ratios of Mortar Mixes

Description	Details	Control	SBR				VAE			
	Polymer Content (%age)	0	5	10	15	20	5	10	15	20
For constant w/c ratio of 0.56	w/c ratio	0.56	0.56	0.56	0.56	0.56	0.56	0.56	0.56	0.56
	Mix designation at constant w/c ratio	CON	S11	S21	S31	S41	V11	V21	V31	V41
For constant flow value of $105\pm5\%$	w/c ratio	0.56	0.39	0.37	0.35	0.32	0.51	0.48	0.46	0.43
	Mix designation at adjusted w/c ratios	CON	S1	S2	S3	S4	V1	V2	V3	V4

In the second case, the flow value of $105\pm5\%$ was kept constant for mortar mixes, and the w/c ratios of PMMs were adjusted for different percentages of polymer additions. The adjusted w/c ratio values of modified mortar mixes are given in Table 3.5. The mixes having constant flow value of $105\pm5\%$ and modified with different percentage addition of SBR and VAE polymer are designated as S1, S2, S3, S4 and V1, V2, V3, V4, respectively. The effect of the addition of polymers on the workability of PMM was studied by monitoring the flow values. The air content percentage of SBR and VAE polymer modified mortars was also measured as per *ASTM C 185-08* after adding the anti-foaming agent.

3.3.2 Workability of Mortar by Flow Table Test

Flow table test is a method to evaluate the workability of cement mortars and flow value is defined as the percentage increase of the base diameter of a mortar mass after the standard procedure of vibration on a flow table, w.r.t. the initial base diameter of mortar mass. In this experimental work, the effect of two different polymer additions on the workability of mortar has been studied. The flow table values have been worked out for mixes having constant w/c ratio and the mixes having constant flow value of $105 \pm 5\%$ at varying w/c ratios.

First of all, the constituents of mortar were thoroughly mixed, and a uniform mix was prepared. The standard mould, open from bottom and top, having a base diameter of 100mm and an upper diameter of 70mm with a height of 50mm was used for this test. The mix was poured in the mould in two layers. Each layer was compacted with 20 numbers of blows with a standard mild steel rod having a diameter of 25mm. The pressure on the tampering rod was just sufficient for the proper filling of material in the mould. After filling the mould on a standard flow table, it was removed gently by pulling it upward, and the initial diameter of the mortar sample was measured (d_i). The test was conducted by vibrating the table with 25 number blows and free falling of 12.5mm. The standard duration for these 25 number blows was 15 seconds. After following the test procedure, the final diameter (d_f) was measured and recorded. Plate 3.3 shows the flow table apparatus and the measurement of the flow value of mortar.

The flow value was worked out as: -

$$\text{Flow Value} = \frac{\text{Final Diameter } (d_f) - \text{Initial Diameter } (d_i)}{\text{Initial Diameter } (d_i)} \times 100 \quad \dots (3.1)$$



Plate 3.3: Flow Table and Measurements of Flow Value of Mortar

3.3.3 Effect of Addition of Polymer on Workability of Mortar

The workability of the mortar mixes was measured in terms of flow values as per *ASTM C 1437-07*. As a first case, the flow values were worked out for a constant w/c ratio of 0.56 (as required for the control mix) with variable addition of SBR and VAE polymers. Subsequently, in the second case, the w/c ratios were found out for a fixed flow value of $105\pm 5\%$ for the mixes containing different percentage addition of polymers. The mortar mixes were designated w.r.t. the type and amount of polymer added. The prefix ‘S’ and ‘V’ denote the SBR and VAE polymer addition, respectively. Mix designation along with w/c ratio and flow table test results are given in Tables 3.6 and 3.7.

Table 3.6: Flow Table Values of Polymer-Modified Mortars at w/c Ratio of 0.56

S. No.	Mix designation	p/c ratio (%)	w/c ratio	Flow value (%)
1.	CON	0	0.56	106
2.	S11	5	0.56	139
3.	S21	10	0.56	176
4.	S31	15	0.56	196
5.	S41	20	0.56	Expanded
6.	V11	5	0.56	130
7.	V21	10	0.56	158
8.	V31	15	0.56	166
9.	V41	20	0.56	172

Table 3.7: Flow Table Values of Polymer-Modified Mortars at Adjusted w/c Ratio for a Fixed Flow Value of $105\pm 5\%$

S. No.	Mix designation	p/c ratio (%)	w/c ratio	Flow value (%)
1.	CON	0	0.56	106
2.	S1	5	0.39	105
3.	S2	10	0.37	106
4.	S3	15	0.35	110
5.	S4	20	0.32	105
6.	V1	5	0.51	108
7.	V2	10	0.48	108
8.	V3	15	0.46	105
9.	V4	20	0.43	104

The flow values of mixes S11, S21 and S31 containing SBR polymer were found to increase by 29.91%, 64.49% and 83.18%, respectively, as compared to CON mix. On the other hand, the flow of S41 mix expanded more than 100% and thus was not measurable on the flow table. The mixes containing VAE polymers were also found to show an increase of

flow value. The flow values of V11, V21, V31 and V41 mixes increased by 22.64%, 49.06%, 56.60% and 62.26%, respectively, as compared to the CON mix. However, this increase was much less than that observed for the SBR polymer-modified mortars.

In the second case, w/c ratios were worked out for a constant flow value of $105 \pm 5\%$. Both the SBR and VAE modified mortars were observed to show a reduction in the w/c ratios as compared to the CON mix. The w/c ratios of SBR and VAE modified mortars reduced from 0.56 to 0.32 and from 0.56 to 0.43, respectively, as the polymers percentage addition increased from 0 to 20%. The w/c ratio requirement thus, decreased by about 42% and 23% in the case of 20% addition of SBR and VAE polymers, respectively, for maintaining a flow value of $105 \pm 5\%$. As seen from the results, the SBR and VAE polymer-modified mortars show a remarkable improvement in water reduction, comparatively more in the case of SBR modified mortars. Figures 3.1 and 3.2 show the results of the flow table test and w/c requirement of polymer-modified mortars.

This substantial improvement in the workability is attributable mainly to the spherical supporting properties of polymers, dispersing effect of surfactants in the latexes and entrained air in the modified mortar matrix (*Ohama, 1995*) which further helps in accelerating the hydration process and an increase of flow value. The surface tension of polymer molecules further eases the flow value of the mortar mix for the same w/c ratio (*Aggarwal et al., 2007*).

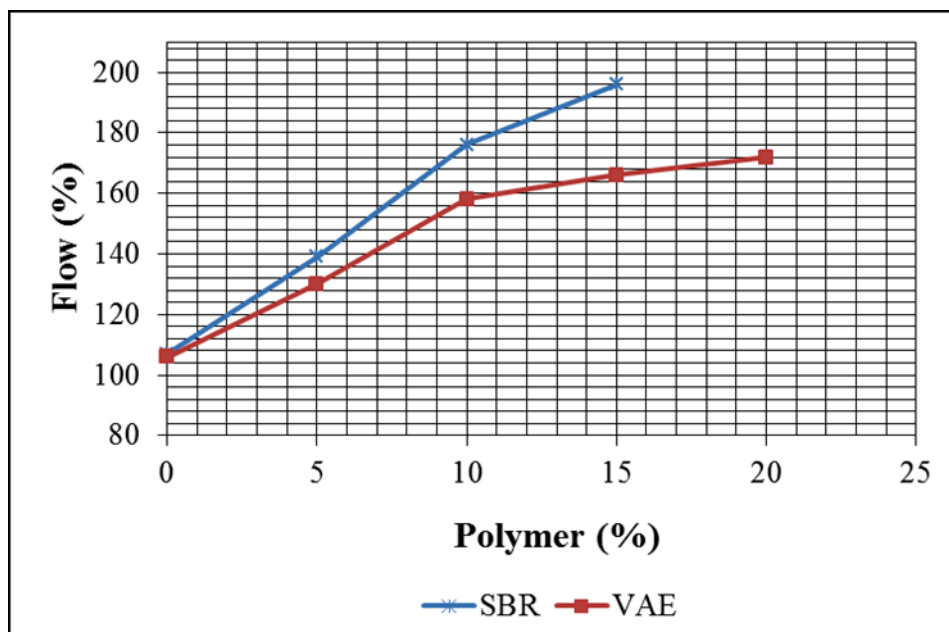


Fig. 3.1: Flow Values v/s Polymer Percentage at a Constant w/c Ratio of 0.56

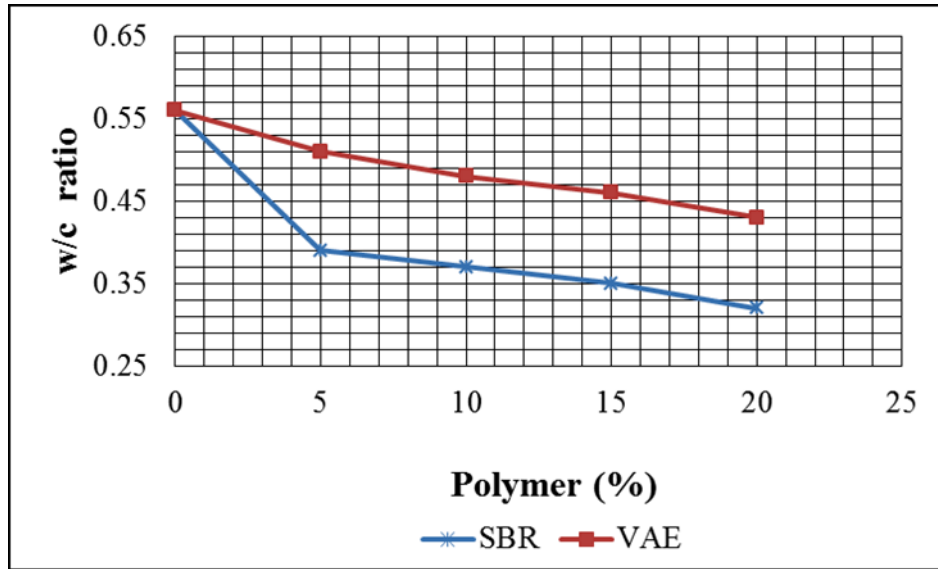


Fig. 3.2: Adjusted w/c Ratios v/s Polymer Percentage for Constant Flow Value of $105 \pm 5\%$

3.4 EFFECT OF POLYMER ADDITION ON THE PROPERTIES OF MORTAR

The properties of cement mortars modified with different percentage additions of SBR and VAE polymers were also evaluated. The obtained test results have been studied in terms of, compressive strength, flexural strength, modulus of elasticity and air content. These polymer modified mortars are subsequently, further used to develop the Polymer Modified Ferrocement (PMF). The effects of polymer addition on the properties of mortar are detailed and discussed in the succeeding sub-sections.

3.4.1 Casting and Testing the Specimens

The polymer modified mortar (PMM) samples for different tests and different curing ages were cast, to study the effect on various properties, with the addition of different percentages of SBR and VAE polymers. The detail of the specimens is given in Table 3.8.

All the specimens of specified sizes were cast and demoulded after 24 hours and kept for curing for a specified number of days (7 and 28 days). The curing of all specimens was done in a water tank at a temperature of $27 \pm 2^\circ\text{C}$. After attaining the requisite curing age, these specimens were taken out from the curing tank for 20 to 30 minutes before testing. All the specimens were visually checked for any discrepancy before testing. Detailed test procedures have been adopted as per various standard codes to evaluate the properties of polymer modified mortar. The detail of mixes is provided in Table 3.5. The detail of each test procedure is described in succeeding sections. Figure 3.3 shows the flow diagram of the test program for polymer modified mortar (PMM).

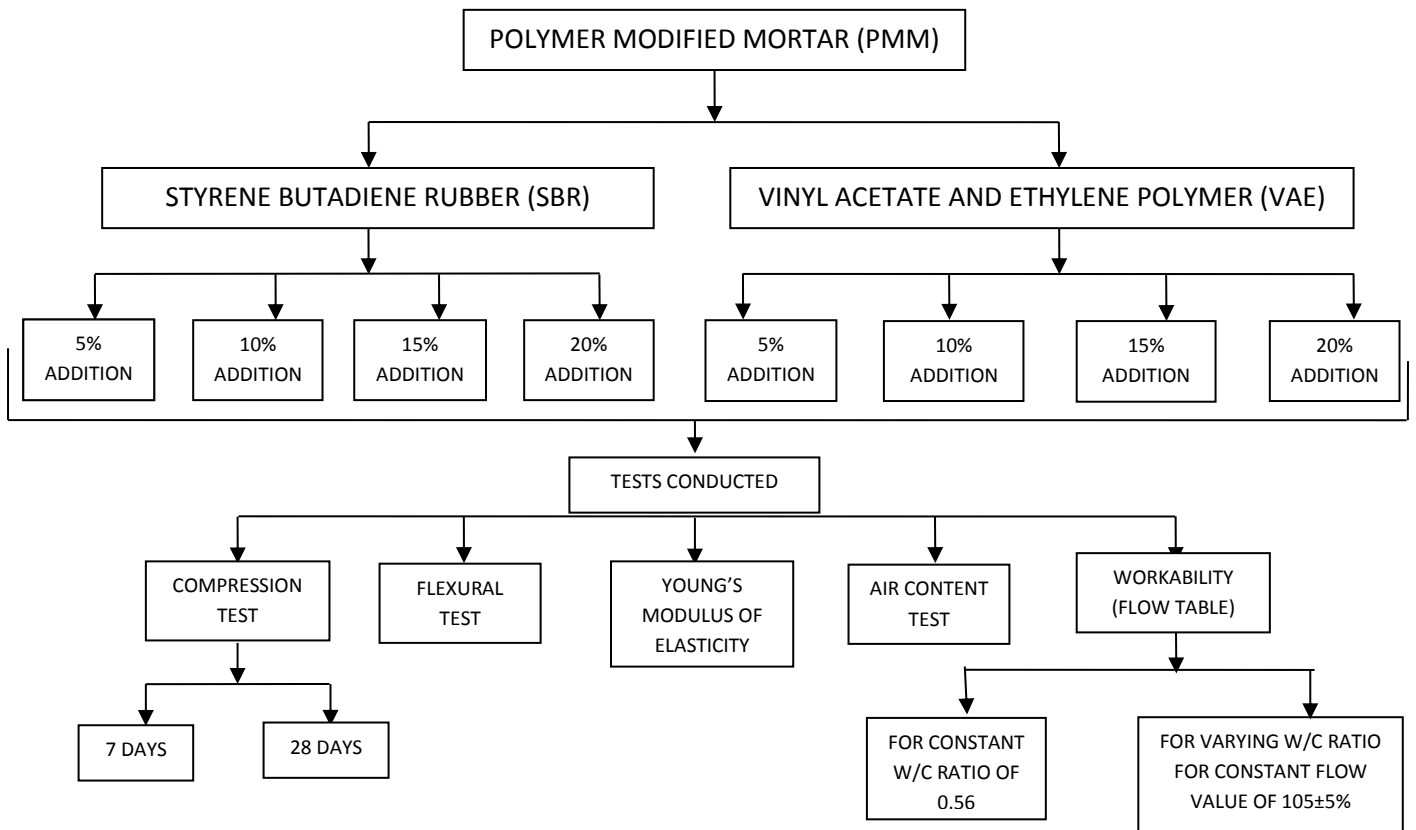


Fig. 3.3: Test Scheme for Polymer Modified Mortars (PMM)

3.5 COMPRESSIVE STRENGTH TEST OF MORTAR

The compressive strength test, on the various mortar specimens, was conducted as per *ASTM C 109-93* specifications on specimens of standard size 70.6mm × 70.6mm × 70.6mm. A total number of 102 cubical specimens of SBR and VAE polymer modified mortars were cast. The polymers were added in the range of 0 to 20%, by weight of cement, with 5% step increment. These samples were cast with constant and adjusted water/cement (w/c) ratios and tested for curing periods of 7 days and 28 days. Detail of samples along with their designation names have been given in Tables 3.6, 3.7 and 3.8. All the specimens were adequately checked against any kind of discrepancy before testing. The specimens were tested on a universal testing machine (UTM) by applying gradual (without shock) loading at a constant rate of 70kN/minute till the failure. The mortar cubes after casting are shown in Plate 3.4. The progression compressive strength test on mortar specimen is shown in Plate 3.5. For calculation of compressive strength, the formula is: -

$$f = \frac{P}{A}$$

.... (3.2)

Table 3.8: Sampling Details of Mortar Specimens

Description	Size	Testing age	Control	VAE				SBR				Total	G. Total
				5	10	15	20	5	10	15	20		
Polymer %age			0	5	10	15	20	5	10	15	20		
Cubes specimens at constant w/c ratio of 0.56 for compressive test	70.6mm× 70.6mm×70.6mm	7	3	3	3	3	3	3	3	3	3	27	102
Cubes specimens at constant flow table value of 105±5% for compressive test	70.6mm× 70.6mm×70.6mm	7		3	3	3	3	3	3	3	3	24	
Cubes specimens at constant w/c ratio of 0.56 for compressive test	70.6mm× 70.6mm×70.6mm	28	3	3	3	3	3	3	3	3	3	27	
Cubes specimens at constant flow table value of 105±5% for compressive test	70.6mm× 70.6mm×70.6mm	28		3	3	3	3	3	3	3	3	24	
Beam specimens for flexural strength	40mm×40mm×160mm (length)	28	3	3	3	3	3	3	3	3	3	27	27
Cylindrical specimens for MoE	100mm(diameter) × 200mm(height)	28	3	3	3	3	3	3	3	3	3	27	27



Plate 3.4: Mortar Cubes after Casting



Plate 3.5: Compressive Strength Test on Mortar Specimens in Progress

Table 3.9: Compressive Strength of Polymer Modified Mortars at a Constant w/c Ratio (0.56)

S. No.	Constant w/c ratio (0.56)					
	Mix description	Comp. strength (N/mm ²)		Mix description	Comp. strength (N/mm ²)	
		After 7 days of curing	After 28 days of curing		After 7 days of curing	After 28 days of curing
1	CON	20.41	29.34	CON	20.41	29.34
2	S1	18.15	27.53	V1	17.22	24.56
3	S2	17.24	22.88	V2	15.19	19.88
4	S3	13.72	19.17	V3	12.97	18.02
5	S4	11.16	16.41	V4	10.28	15.54

3.5.1 Effect of Polymer Addition on Compressive Strength of Mortar

In this investigation, the compressive strength of control as well as of polymer modified mortars was worked out for different variable parameters. The test was performed

for constant and varying w/c ratio mortars having 7 and 28 days age of curing. The compressive strength test results at constant and adjusted w/c ratio for varying polymer percentage additions in the mortar matrix are detailed in Tables 3.9 & 3.10 and shown in Figures 3.4 to 3.7, respectively.

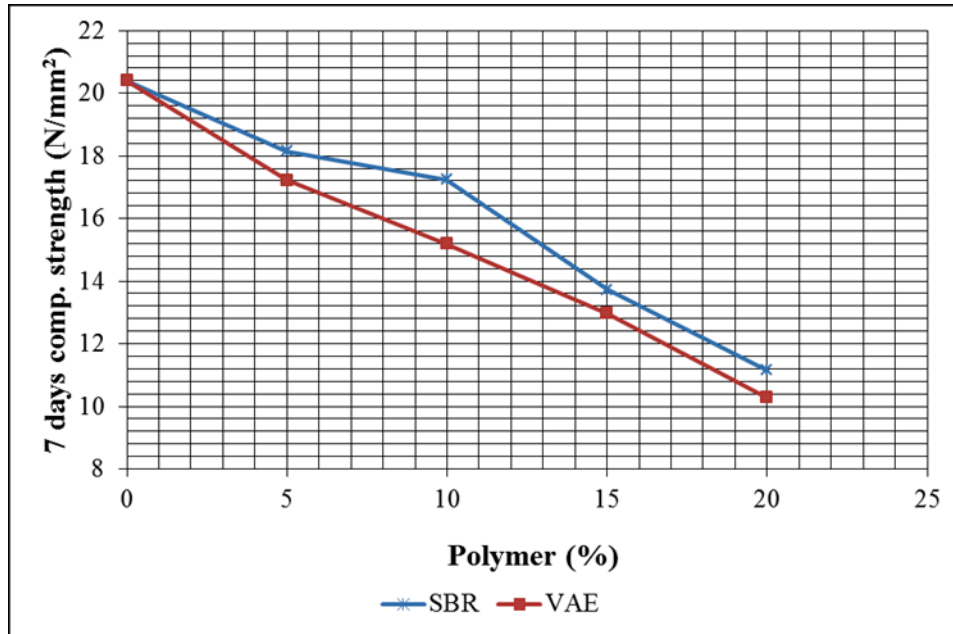


Fig. 3.4: Compressive Strength of the Samples (cured for 7 days) v/s Polymer Percentage at a Constant w/c Ratio of 0.56

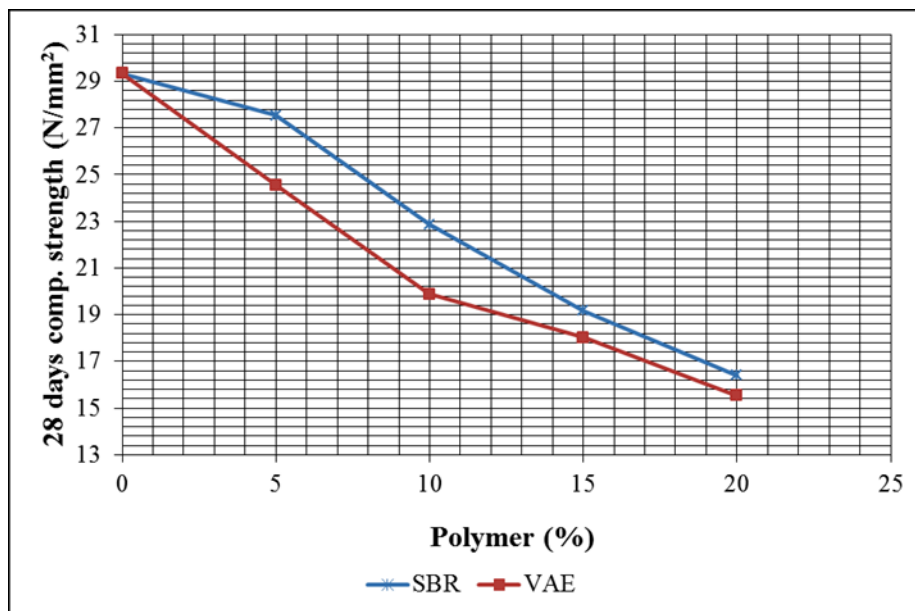


Fig. 3.5: Compressive Strength of the Samples (cured for 28 days) v/s Polymer Percentage at a Constant w/c Ratio of 0.56

As it can be observed from the Table 3.9 and Figures 3.4 & 3.5, the compressive strength after 7 days of curing (at constant w/c ratio) in case of S11, S21, S31 and S41

mixes was observed to decrease by 11.07%, 15.53%, 32.78% and 45.32% whereas, in the case of V11, V21, V31 and V41 mixes the compressive strength decreased by 15.63%, 25.58%, 36.45% and 49.63%, respectively, as compared to the CON mix strength. The compressive strength after 28 days of curing also followed the same reduction pattern for both the polymer-modified mortars. Thus, the compressive strength of both types of modified mortars decreased with the increase in percent addition of polymer for the constant w/c ratio 0.56. The degree of reduction in the compressive strength after 7 days of curing is however observed to be higher than that after 28 days of curing.

Table 3.10: Compressive Strength of Polymer Modified Mortars at Varying w/c Ratio for a Constant Flow Value of 105±5%

S. No.	Adjusted w/c ratio for flow value of 105±5%					
	Mix description	Comp. strength (N/mm ²)		Mix description	Comp. strength (N/mm ²)	
		After 7 days of curing	After 28 days of curing		After 7 days of curing	After 28 days of curing
1	CON	20.41	29.34	CON	20.41	29.34
2	S1	21.32	30.64	V1	20.07	26.93
3	S2	21.65	30.67	V2	16.46	25.12
4	S3	21.72	31.53	V3	14.72	22.78
5	S4	19.69	29.09	V4	14.29	21.89

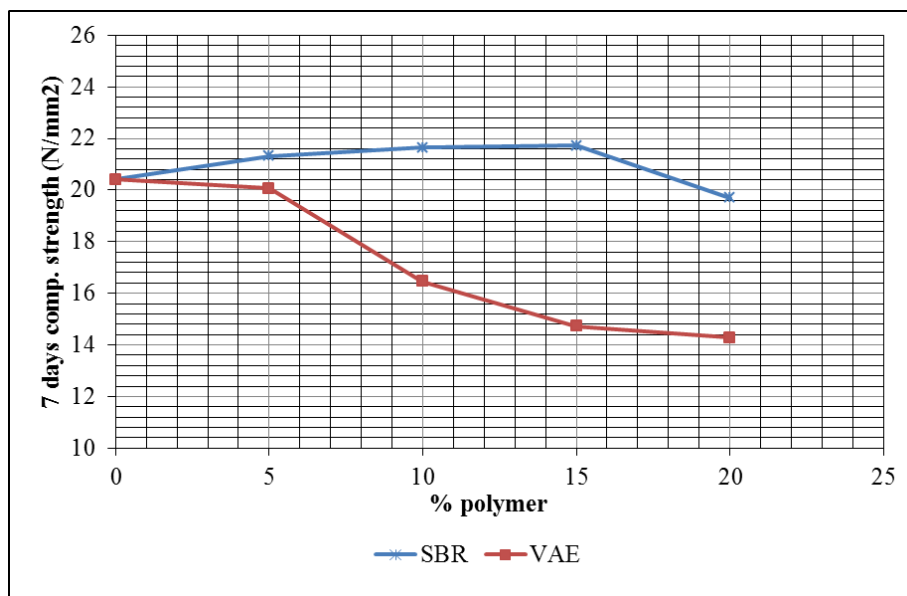


Fig. 3.6: Compressive Strength of the Samples (cured for 7 days) v/s Polymer Percentage for Flow Value of 105±5%

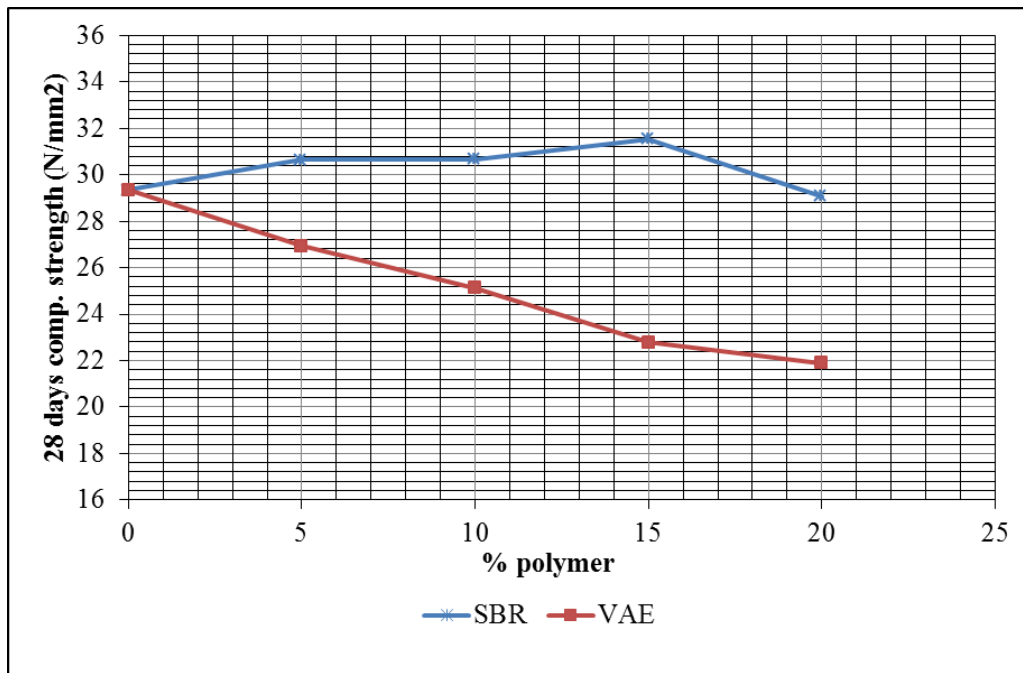


Fig. 3.7: Compressive Strength of the Samples (cured for 28 days) v/s Polymer Percentage for Flow Value of $105 \pm 5\%$

As can be observed from Table 3.10 and Figures 3.6 & 3.7, at the adjusted w/c ratios for a constant flow value of $105 \pm 5\%$ and curing age of 28 days, the compressive strength of S1, S2 and S3 mixes showed an increase of 4.43%, 4.53% and 7.46%, respectively, but for the S4 mix, the strength was almost the same as was achieved for the CON mix. On the other hand, not much of a difference has been observed for 7 days compressive strength of SBR modified mortar and the strength values were observed almost the same as that for CON mix. The 28 days compressive strength of V1, V2, V3 and V4 mixes, however, showed a decline of the order of 8.21%, 14.38%, 22.36% and 25.39%, respectively, as compared to the CON mix. A similar decline pattern of strength was also observed for 7 days compressive strength of VAE modified mortar.

As can be observed, for a constant flow value and adjusted w/c ratios, the compressive strength of SBR modified mortars first increased and then decreased with increase in percentage addition of SBR polymer and the maximum increase was observed at 15% addition of polymer which is 7.46%. Thus, it can be concluded that the polymer addition of optimum quantity is responsible for increasing the strength of the mortar at later days. In the SBR latex modified mortar, the polymeric film helps in retaining the internal moisture content which leads to the reduction of voids thereby improving the compressive strength. In the polymer modified mortar (PMM), two types of bonds namely cement-cement bond and cement-polymer bond exist. The cement-polymer bonds are weaker in

compression as compared to cement-cement bond, hence, an increase in percentage addition of SBR in PMM beyond 15%, results in a decrease in the compressive strength. Further SBR modified mortars had more rigid wedges in molecule chain and show better compatibility with cement mortar which also tends to increase the compressive strength, at 7 and 28 days of curing, though not at a very higher rate.

The VAE film may have a good interactive effect, but the VAE modified mortars co-matrix reflected anti-interactive effect. This decreased the compatibility of VAE polymer with cement mortar, thereby causing a reduction in the compressive strength of VAE modified mortars at both 7 and 28 days curing ages. The entrapped air volume is also found more in VAE modified mortars as compared to SBR modified mortars. This increase in air content due to the surface activity of VAE was also responsible for a reduction in compressive strength of PMM in this case.

A very small increment of compressive strength has been observed in the case of SBR polymer modified mortar. However, on the other hand, the compressive strength of VAE modified mortar has been decreased with the increase in percentage addition of polymer. As also observed from the literature (*Ohama, 1995; Çavdar, 2014*) that the compressive strength of polymer modified mortar is not only depended upon a single factor and there are many other factors which affect the strength properties of mortar. The water/cement ratio, types of polymers used, percentage addition of polymers, properties of sand and cement, curing conditions, air content etc. are the main factors which are responsible for the strength properties of mortar. Hence, it is difficult to find out the exact parameters involved behind the variation in compressive strength of PMM.

For an equal addition of polymer percentage and other similar parameters, the mortars modified with SBR showed an improvement in compressive strength as compared to the mortars modified with VAE. Thus, it can also be concluded that the compressive strength of SBR modified mortars is maximised at a constant flow value of $105 \pm 5\%$ with a polymer addition of 15%.

3.6 FLEXURAL STRENGTH TEST OF MORTAR

The flexural strength of material indicates its resistance against bending. The flexural strength is also known as bend strength, modulus of rupture or fracture strength. It is the measure of peak stress induced within the material at its fracture. In this experimental investigation, the central point load flexural strength test was performed as per *ASTM C 348-02* specification. A total of 27 beam specimens of mortar of size 40mm × 40mm ×

160mm were cast to investigate the flexural strength. These specimens were cast by adding different percentages of SBR and VAE polymers varying from 0% to 20% for a curing age of 28 days. The flexural strength test was performed only on the mortar mixes having a constant flow value of $105 \pm 5\%$ at adjusted w/c ratios. The detail of the sampling is given in Table 3.8. The effective length of specimens was kept as 120mm and test was performed on UTM at a fixed rate of loading of 2.65kN/minute. Figure 3.8 shows the schematic diagram of test and Plate 3.6 shows the progression of flexure strength test on the mortar.

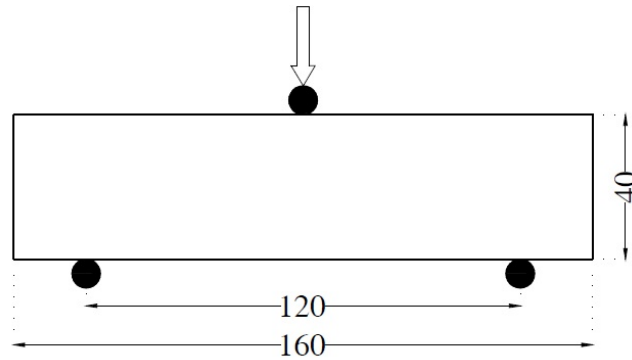


Fig. 3.8: Schematic Diagram of the Central Point Loading Test to find the flexural Strength of Mortar



Plate 3.6: Flexure Strength Test of Mortar in Progress

The bending equation for central point loading to find the flexural strength of specimen is as below: -

$$S_f = \frac{3PL}{2BD^2} = 0.0028P$$

.... (3.3)

3.6.1 Effect of Polymer Addition on Flexural Strength of Mortar

The flexural strength of polymer modified mortars (PMM) after curing of 28 days was investigated for adjusted w/c ratios for a constant flow value of $105 \pm 5\%$ and the values are presented in Figure 3.9 and Table 3.11. With 15% addition of polymer by weight of cement, the flexural strength of SBR and VAE modified mortars was observed to increase by 44.89% and 23.46%, respectively. With any further addition of these polymers, the flexural strength was found to reduce. The 28 days flexural strength of polymer modified mortars at adjusted w/c ratio was found to be increased by 26.87%, 35.88%, 44.89%, 38.27% and 12.41%, 16.50%, 23.46%, 11.22%, respectively, with the addition of 5%, 10%, 15% and 20% SBR and VAE polymers.

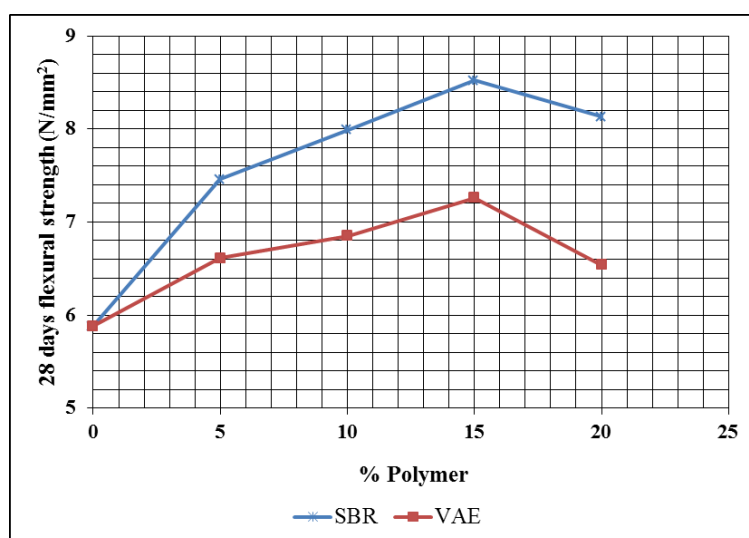


Fig. 3.9: Flexural Strength of the Samples (cured for 28 days) v/s Polymer Percentage for Constant Flow Value of $105 \pm 5\%$

Table 3.11: Flexural Strength of Polymer Modified Mortars at Varying w/c Ratio for a Constant Flow Value of $105 \pm 5\%$

S. No.	Adjusted w/c ratio for flow value of $105 \pm 5\%$			
	Mix description	Flexural strength (N/mm ²) after 28 days of curing	Mix description	Flexural strength (N/mm ²) after 28 days of curing
1	CON	5.88	CON	5.88
2	S1	7.46	V1	6.61
3	S2	7.99	V2	6.85
4	S3	8.52	V3	7.26
5	S4	8.13	V4	6.54

It can be seen from the test results that with the addition of both SBR and VAE polymers, the flexural strength increases considerably. With the addition of these polymers in the mortar matrix, a polymeric film is formed which retards cement hydration process and contributes to the strengthening of microstructure and interconnectivity of the mortar mix, thus influencing the flexural strength. A higher increase in the percentage of polymer addition (beyond 15%), however, causes a discontinuity in microstructure hence indicating a reduction in the flexural strength values. As such, the addition of polymers reduced the water requirement of the modified mix, thereby reducing the permeability and enhancing the strength properties.

In comparison, it is further seen that the SBR modified mortar show a higher percentage increase in flexure strength as compared to VAE modified mortar. This is attributed to the stability of bonds of the aromatic ring in the microstructure of SBR.

3.7 YOUNG'S MODULUS OF ELASTICITY (MoE) OF MORTAR

The modulus of elasticity or elastic modulus is defined as the tendency of the material to deform elastically after the application of load on it. The modulus of elasticity is defined as stress to strain ratio (*ASTM C 469-02*). MoE is one of the major property of any material as it is used in the structural design of any member with or without reinforcement. The stress with respect to any particular strain value can be evaluated with MoE. The modulus of elasticity (Young's modulus) of a construction material describes its reversible deformation due to an applied stress. If the deformation returns to its initial state when the stress is removed, this is described as linear or ideal elastic behaviour. The gradient of the lines in the stress-strain curve in the elastic deformation region gives these material properties.

$$E = \frac{\Delta\sigma}{\Delta\epsilon} \quad \dots (3.4)$$

E = Young's modulus [MPa, N/mm²]

$\Delta\sigma$ = Difference in stress between the initial state and the applied stress [N/mm²]

$\Delta\epsilon$ = Resulting elastic strain for the observed stress state [%]

A total of 27 cylindrical specimens of size 100mm×200mm were cast to investigate the Young's modulus of the polymer-modified mortar. The specimens for Young's modulus test were cast for SBR and VAE polymer-modified mortars by varying the polymer content from 0 to 20% for the adjusted w/c ratios with a constant flow value of 105±5%. The detail

of the specimens is provided in Table 3.8. The specimens were tested on compression testing machine as per procedure laid down by *ASTM C 469-02* and is shown in Plate 3.7. The surface of cylindrical specimens was checked for smoothness, and if not found flat then these were made smooth with a grinder before testing. The stress-strain curves have been obtained for each specimen.

As per *ASTM C-469-02*, the modulus of elasticity, to the nearest 50 000 psi (344.74 MPa) is calculated as follows:

$$E = (S_2 - S_1)/(\epsilon_2 - 0.000050) \quad \dots (3.5)$$

E = chord modulus of elasticity, psi,

S_2 = stress corresponding to 40 % of ultimate load,

S_1 = stress corresponding to a longitudinal strain, ϵ_1 , of 50 millionths, psi, and

ϵ_2 = longitudinal strain produced by stress S_2 .



Plate 3.7: Mortar Cylinder during Testing for Modulus of Elasticity

3.7.1 Effect of Polymer Addition on Young's Modulus of Elasticity of Mortar

The modulus of elasticity values for polymer modified mortars (PMM) after curing of 28 days were investigated for adjusted w/c ratios for a constant flow value of $105 \pm 5\%$ and are represented in Table 3.12 and Figure 3.10. On analysing the results, it was found that MoE of SBR modified mortars increased by 12.45% with 10% polymer addition. Further addition of polymer resulted in a reduction of MoE values. However, the elasticity of SBR modified mortar remained higher than that of the control mix at all addition levels. The MoE of SBR was found to increase by 4.87%, 12.45%, 9.01% and 4.70% at 5%, 10%, 15% and 20% addition levels, respectively. On the other hand, the modulus of elasticity of VAE modified mortars decreased at a higher rate as compared to the control mortar mix and

decreased up to 44.68% at 20% polymer addition. The modulus of elasticity (MoE) of polymer modified mortar is generally lower than unmodified mortar since cement-polymer bonds have generally lesser elasticity as compared to cement-cement bonds. Thus, the strength and deformation characteristics of polymer modified mortar may differ from the control mortar mix.

Table 3.12: Young's Modulus of Elasticity (MoE) of Polymer Modified Mortars at varying w/c Ratio for a Flow Value of 105±5%

S. No.	Adjusted w/c ratio for flow value of 105±5%			
	Mix description	MoE (N/mm ²) x 10 ³ after 28 days of curing	Mix description	MoE (N/mm ²) x 10 ³ after 28 days of curing
1	CON	12.065	CON	12.065
2	S1	12.652	V1	11.998
3	S2	13.567	V2	10.198
4	S3	13.152	V3	7.957
5	S4	12.632	V4	6.674

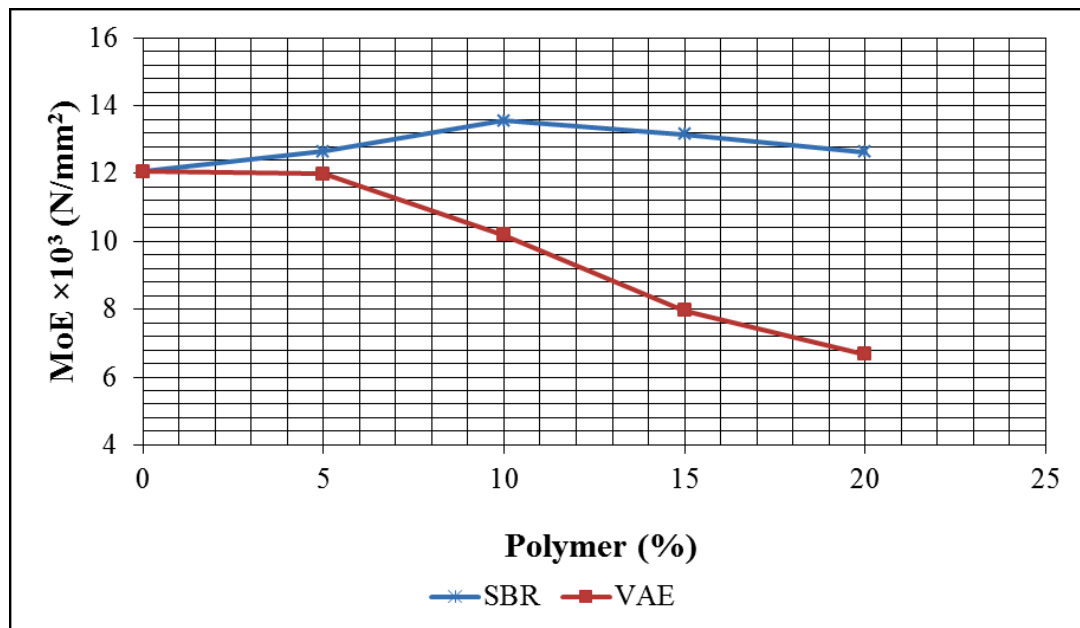


Fig. 3.10: MoE of Samples (cured for 28 days) v/s Polymer Percentage for Value Flow of 105±5

3.8 AIR CONTENT OF MORTAR

The air content of mortars has been measured as per the standard procedure laid down in *ASTM C 185-08*. The test was conducted to find the percentage air content present in the mortar matrices having different percentages of SBR and VAE polymers. This test

was conducted for mortar having adjusted w/c ratio for a constant flow value of 105±5%. The cement sand mortar mixes were prepared with different percentage addition of polymers. The mortar mixes were then compacted into the measure of known volume, and its mass was determined. The air content of mixes was calculated as per ASTM equation, as given below: -

$$\text{Air content, volume \%} = 100 - W \left[\frac{(182.7 + P)}{(2000 + 4P)} \right] \quad \dots (3.6)$$

W = mass of 400 mL of mortar, g, and

P = percentage of mixing water, based on mass of cement used

3.8.1 Effect of Polymer Addition on Air Content of Mortar

The air content of mortars was measured as per standard procedure laid down in *ASTM C 185-08*. The percentage air content present is measured for mortar matrices having different percentages of SBR and VAE polymers. This test was conducted for mortar having adjusted w/c ratio for a constant flow value of 105±5%. Table 3.13 and Figures 3.11 show the variation of air content percentage with the addition of polymers.

Table 3.13: Polymer-Modified Mortars at Adjusted w/c Ratio for Flow Value of 105±5%

S. No.	Mix designation	p/c ratio (%)	w/c ratio	Flow value (%)	Mass of 400mL mortar (gm)	Air content (%)
1.	CON	0	0.56	106	915.6	16.20
2.	S1	5	0.39	105	910.4	16.72
3.	S2	10	0.37	106	911.2	16.65
4.	S3	15	0.35	110	914.4	16.37
5.	S4	20	0.32	105	912.4	16.56
6.	V1	5	0.51	108	881.6	19.32
7.	V2	10	0.48	108	890.8	18.49
8.	V3	15	0.46	105	904.4	17.25
9.	V4	20	0.43	104	902.4	17.44

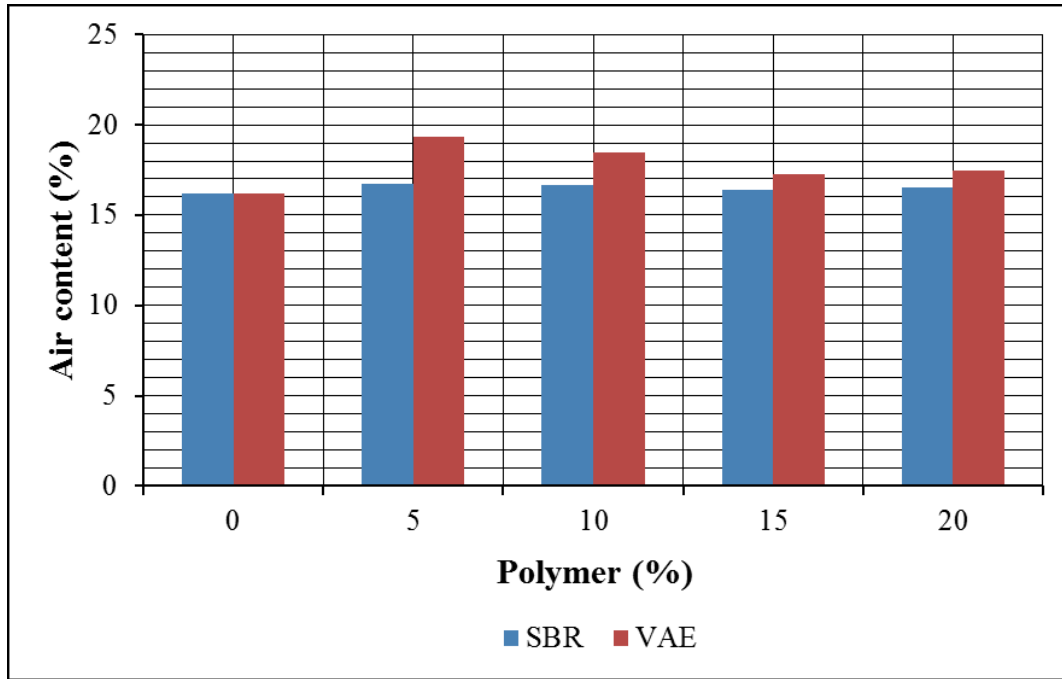


Fig. 3.11: Air Content v/s Polymer at Adjusted w/c Ratio for Flow Value of 105±5%

It can be observed from the test results that the percentage air content of SBR and VAE modified mortars was lying in the range of 16.37-16.72% and 17.25-19.32%, respectively, with the use of antifoaming agents, as compared to 16.20% for the CON mix. The result also shows that the use VAE polymer in the mortar entraps more air content as compared to SBR modified mortar for the same percentage addition level. The anti-foaming agent was also used to make the PMM equally dense as the control mortar mix and to control the percentage of air content. The percentage of air content drastically increased at 5% VAE polymer addition level. As the percentage addition of VAE increased, the percentage air content of the mortar matrix was decreased. A similar pattern was also observed for SBR polymer-modified mortar, but the percentage air was found less in this case as compared to VAE modified mortar matrices. At 15% addition level, both SBR and VAE polymer-modified mortars show a lower amount of air content of 16.37% and 17.25%, respectively, as compared to other percentage addition levels. Hence, 15% addition ratio of the polymer is the optimum percentage to modify the mortar matrix. Thus, it can be concluded that the addition of polymers increased the percentage air content of the mortar matrix and it was higher with the use of VAE polymers as compared to SBR polymers.

As reported by *Ohama (1995)*, *Wang et al. (2011)* and *Marceau et al. (2012)*, the porosity of mortars increased with the addition of any polymer due to their air-entraining effect. Presence of air in the fresh mortar matrix helps to improve its consistency and

freezing-thawing resistance of hardened mortar, but at the same time, the mechanical strength properties of modified mortars also reduce due to this porosity. To control the porosity and to enhance the strength properties of modified mortars, the anti-foaming agents are added. This is due to the fact that the wet particle displacement is caused by the dispensing of active components of the anti-foaming agent with air bubbles. This results in the subversion of the thin layer of an air bubble (*Piekarczyk and Szwabowski, 2012*). This was found adaptable, and the anti-foaming agent proved its utility in the results of the present experiment as well.

3.9 DEVELOPMENT OF POLYMER MODIFIED FERROCEMENT (PMF)

From the literature review, it was concluded and further decided to use the Polymer Modified Ferrocement (PMF) to strengthen the RCC beams in shear as to fulfil the primary objective of this study. Hence, after finding the effects of different percentage addition of polymers on the properties of mortars, a pilot study was made to find out the optimum percentage of polymer and numbers of wire mesh layers for which the PMF performs best in terms of tensile and flexural strengths. The effect of the addition of different percentages of SBR and VAE polymers on the properties of PMF having two and three-layers of steel woven wire mesh was investigated in this study.

The polymer-modified ferrocement (PMF) specimens were prepared by using polymer modified mortar (PMM) having cement to sand (FA-1) ratio of 1:2 by weight (as developed in Section 3.3) and having 0%, 5%, 10%, 15% and 20% addition of SBR and VAE polymers. The adjusted water-cement ratios (as found in Section 3.3.1) for a constant flow constant value of $105 \pm 5\%$ were adopted to prepare the mortar mixes for ferrocement.

A total of 108 samples of SBR and VAE polymer modified ferrocement (PMF) elements of size $500\text{mm} \times 100\text{mm} \times 20\text{mm}$ at adjusted w/c ratios for a flow value of $105 \pm 5\%$ were cast to investigate the flexural and tensile strengths of specimens, as per *ACI 549.1R-93* specifications. The mortar matrix of ferrocement was modified with different percentage addition of both polymers with variation from 0% to 20%. The sand to cement ratio of the mortar matrix was kept as 1:2. The PMF elements were reinforced with two and three layers of galvanized square steel woven wire mesh with a volume fraction of 0.463% and 0.695%, respectively, as shown in Figure 3.12 and Plates 3.8 & 3.9. The details of these specimens are given in Table 3.14. The flow diagram of the test program for polymer modified ferrocement (PMF) is shown in Figure 3.13.

Table 3.14: Detail of Specimens for Flexural and Tensile Strength Test of PMF

	Control		SBR								VAE								Total
Polymer Addition (%)	0		5		10		15		20		5		10		15		20		
No. of wire mesh layers	2	3	2	3	2	3	2	3	2	3	2	3	2	3	2	3	2	3	
No. of samples for flexural strength test	3	3	3	3	3	3	3	3	3	3	3	3	3	3	3	3	3	3	54
No. of samples for tensile test	3	3	3	3	3	3	3	3	3	3	3	3	3	3	3	3	3	3	54

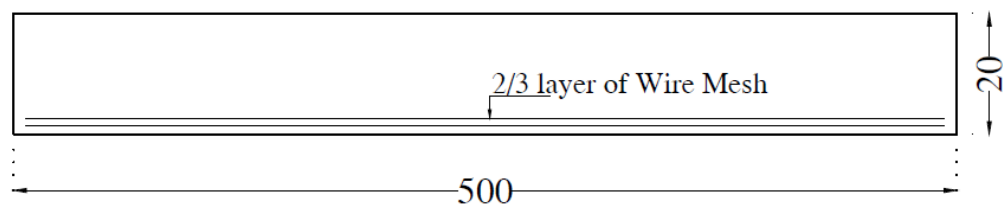


Fig. 3.12: Detail of 2 and 3 Layered Wire Mesh Polymer-Modified Ferrocement Specimens



Plate 3.8: Polymer-Modified Ferrocement Specimens under Casting

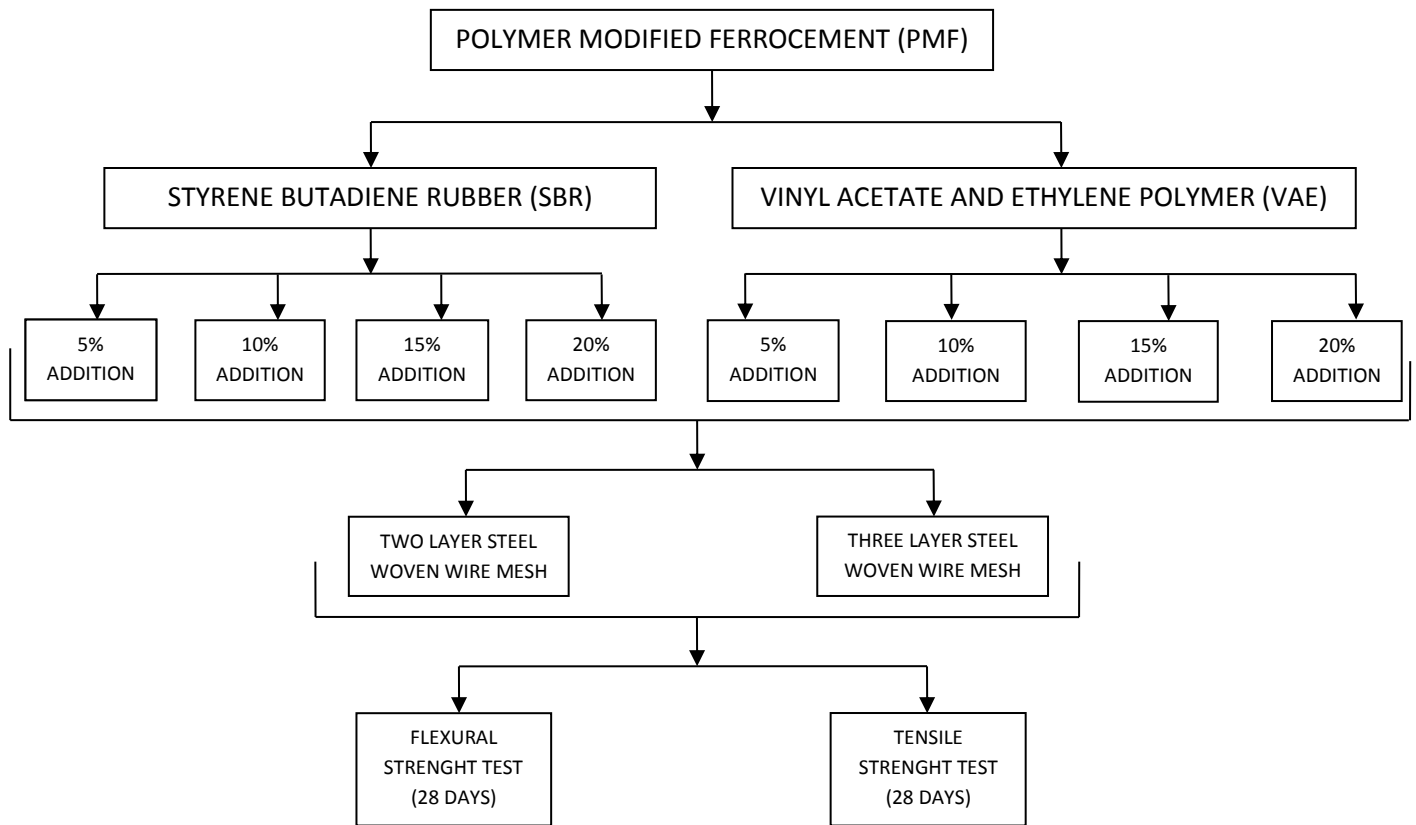


Fig. 3.13: Test Scheme for Polymer Modified Ferrocement (PMF)



Plate 3.9: Polymer Modified Ferrocement Specimens after Demoulding (before testing)

3.10 FLEXURAL STRENGTH TEST OF FERROCEMENT

Flexural strength is the measure of resistance provided by a material against bending. The bending strength of simply supported elements is directly proportioned to the deflection. The 54 number of casted beam specimens were demoulded after 24 hours and kept for curing for a specified number of days (7 and 28 days). The curing of all specimens was done in a water tank at a temperature of $27 \pm 2^\circ\text{C}$. After attaining the specified curing ages, these specimens were taken out from the curing tank for 20 to 30 minutes before testing. All the specimens were visually checked for any discrepancy before testing.

The flexural test in terms of load and deflection was performed on PMF elements under a third point load by taking 400mm as the effective span. The test procedure has been adopted as per *ACI 549.1R-93* specifications to evaluate the flexural strength of polymer modified ferrocement. The schematic diagram of the loading point and span of elements under flexural strength test is shown in Figure 3.14. The polymer modified ferrocement specimens were tested under UTM by applying gradual load as shown in Plate 3.10. The deflection at the central point w.r.t. the test loading was observed by a dial gauge having a least count of 0.01 mm.

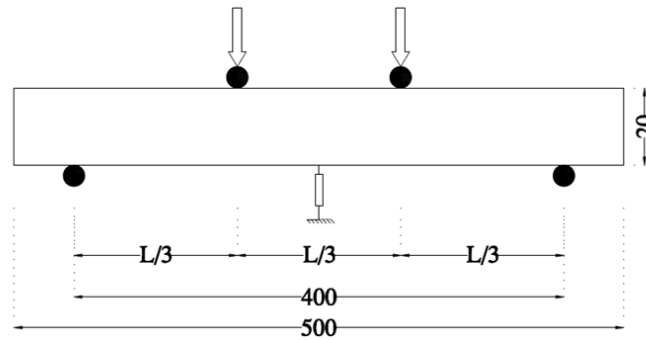


Fig. 3.14: Schematic Diagram of Third Point Loading Test to Find the Flexural Strength of Ferrocement



Plate 3.10: Flexural Strength Test of Ferrocement in Progress

3.10.1 Effect of Polymer Addition on the Flexural Strength of Ferrocement

Flexural strength and tensile strength of polymer modified ferrocement were found out for each of the 2 and 3 layered wire mesh elements, modified with different percentages of SBR and VAE polymers. The SBR and VAE polymers were added in percentages of 5%, 10%, 15% and 20%, respectively. The w/c ratio of mortar was kept as required to maintain a constant flow value of $105 \pm 5\%$ as detailed in Table 3.5. Based upon these test results, the better performing PMF is adopted for the strengthening of RCC beams in the next phase of the study.

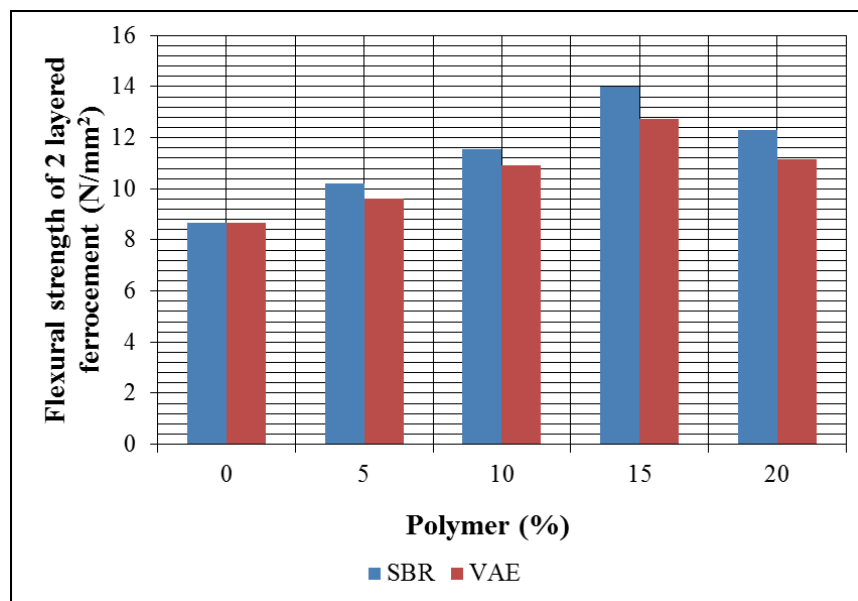


Fig. 3.15: Flexural Strength of 2 Layered Ferrocement (cured for 28 days) v/s Polymer Percentage for Flow Value of $105 \pm 5\%$

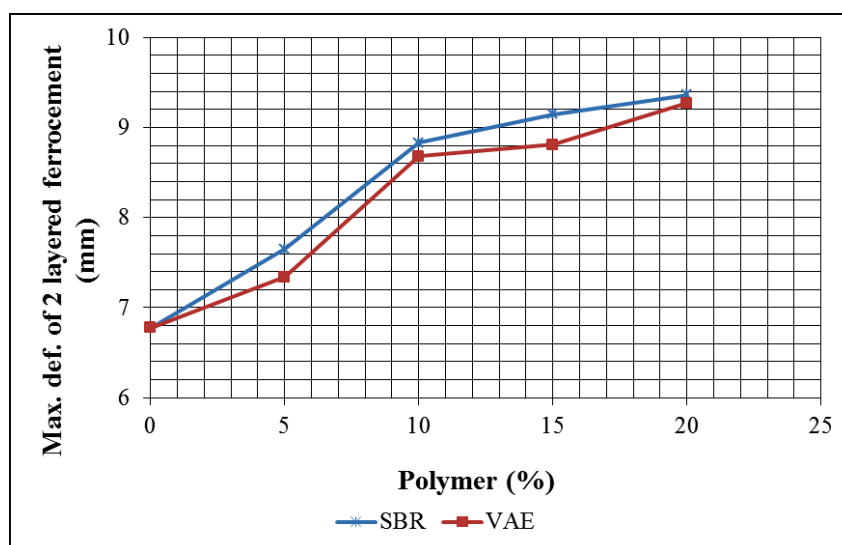


Fig. 3.16: Maximum Deflection of 2 Layered Ferrocement (cured for 28 days) v/s Polymer Percentage for Flow Value of $105 \pm 5\%$

Figures 3.15 & 3.16 and Table 3.15 show the variation in flexural strength and maximum deflection values of polymer modified ferrocement (PMF) having two-layers of wire mesh tested after 28 days of curing. It can be seen from these figures that with 15% addition of both SBR and VAE polymers, the flexural strength of two-layered PMF increased by 61.25% and 46.94%, while the maximum deflection of PMF exhibited an increase of 35% and 30%, respectively as compared to the control specimen. A similar pattern was observed for 5% 10% and 20% SBR and VAE polymer modified ferrocement, and the flexural strength increased in order of 17.53%, 33.33%, 41.75% and 10.96%, 25.72%, 28.84%, respectively. Similarly, the maximum deflection of SBR and VAE ferrocement is also observed to increase by 12.83%, 30.24%, 38.05% and 8.26%, 28.02%, 36.73%, respectively.

Table 3.15: Flexural Strength and Deflection of 2 and 3 Layered Polymers Modified Ferrocement after 28 Days of Curing

S. No.	Polymer %	2 layered				3 layered			
		SBR		VAE		SBR		VAE	
		Flexural st. (N/mm ²)	Max. Def. (mm)	Flexural st. (N/mm ²)	Max. Def. (mm)	Flexural st. (N/mm ²)	Max. Def. (mm)	Flexural st. (N/mm ²)	Max. Def. (mm)
1	0	8.67	6.78	8.67	6.78	11.49	7.35	11.49	7.35
2	5	10.19	7.65	9.62	7.34	14.86	8.99	13.67	9.03
3	10	11.56	8.83	10.9	8.68	16.08	9.72	15.05	9.49
4	15	13.98	9.15	12.74	8.81	17.01	10.58	15.49	10.19
5	20	12.29	9.36	11.17	9.27	16.47	10.87	15.21	10.54

A similar improvement in strength and deflection values was also seen in the case of three-layered SBR and VAE polymer modified ferrocement, in which the flexural strength increased by 48.04% and 34.81% and maximum deflection increased by 44% and 38.64%, respectively, at 15% polymer addition. However, with an increase in the polymer percentage addition from 15% to 20%, the strength characteristics of both types of PMF showed a declining pattern, although their deflection characteristics did show improvement. The flexural strength of SBR and VAE polymer modified ferrocement was observed higher by 29.33%, 39.95%, 43.34% and 18.97%, 30.98%, 32.38%, respectively with the addition of 5%, 10% and 20% polymers. The maximum deflection values also increased by 22.31%, 32.24%, 47.89% and 22.86%, 29.12%, 43.40%, respectively, for SBR and VAE polymer modified ferrocement. Flexural strength results of three-layered wire mesh PMF are presented in Figures 3.17 & 3.18 and Table 3.16. It was also observed during the testing that

most of the cracks were generated near the centre, although, some of the specimens did show horizontal cracks and spalling of the bottom cover. A random Plate 3.11 shows the cracking pattern of PMF failure specimens.

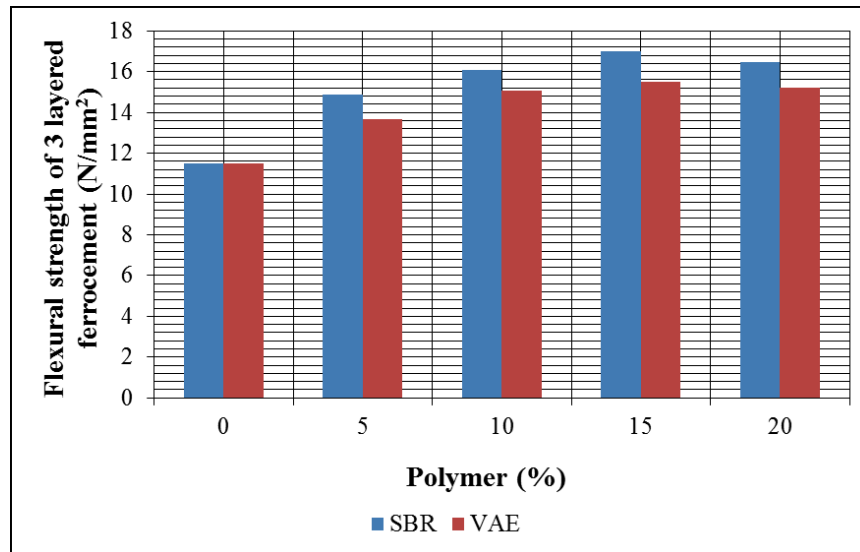


Fig. 3.17: Flexural Strength of 3 Layered Ferrocement (cured for 28 days) v/s Polymer Percentage for Flow Value of $105 \pm 5\%$

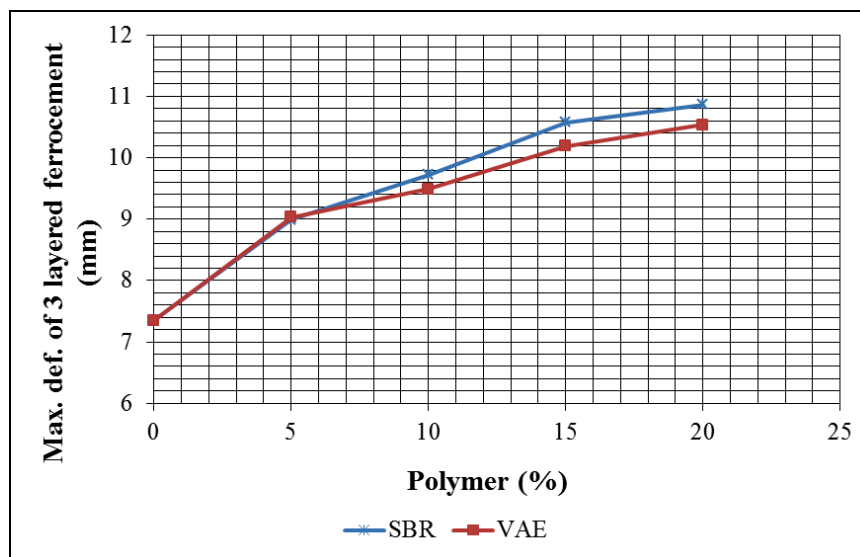


Fig. 3.18: Maximum Deflection of 3 Layered Ferrocement (cured for 28 days) v/s Polymer Percentage for Flow Value of $105 \pm 5\%$



Plate 3.11: Flexural Failure of Ferrocement Elements

Due to the formation of the polymeric layer in the modified mortar matrix, the cement-sand particle packing improved and a more homogeneous microstructure developed. This contributed to a higher flexural strength of PMF. Also, the polymeric interlocked mesh composition imparts flexibility to the otherwise rigid matrix thereby absorbing and transferring the forces efficiently. Further, a higher percentage addition of polymers, from 15% to 20%, causes the formation of a larger cement-polymer phase which further resulted in discontinuity of microstructure and thus resulting in a reduction of strength to some extent.

The flexural moment capacity of PMF elements also increases with an increase in the volume fraction of reinforcement. The increase in the volume fraction of reinforcement also enhances the deflection resistance of ferrocement. The flexural strength of two-layered PMF was drastically improved with the addition of polymers as compared to the control specimens (without polymer). With the addition of polymers, the deflection values of specimens have also been increased. Hence, from the load-deflection response, it could be concluded that the addition of polymers makes the ferrocement more ductile to resist flexural loads. Reinforcement content in the ferrocement matrix improves the toughness of the material which further improves the strength properties. In case of higher volume fraction of steel, a high workable mortar is required for proper compatibility of reinforcement and mortar. With an increase in the percentage of polymer content, the workability and strength of mortar are also increased, which further enhanced the strength properties of PMF. The reason for this increased strength of PMF is due to flexibility extended by the well-distributed polymer particles in the modified matrix.

As compared to flexural strength, the deflection characteristics improve at a higher pace with an increase in reinforcement volume for the same polymer percentage.

3.11 TENSILE STRENGTH TEST OF FERROCEMENT

The axial pull force required to break material is defined as the tensile strength of the material. The tensile strength test on PMF elements was performed on 54 specimens for 7 and 28 days of curing. The tensile strength test was performed on the universal testing machine (UTM) by applying direct tensile load over a gauge length of 280mm to find the tensile strength of PMF elements as per *ACI 549.1R-93* specifications. The load and elongation values were observed during the test. The PMF specimen was put into the jaw of UTM for proper gripping against sliding. The load/force was applied axially through the UTM. Elongation corresponding to axial load was observed till failure of the specimen.

The schematic diagram of the tensile test is shown in Figure 3.19, and the test under progress is shown in Plate 3.12.

3.11.1 Effect of Polymer Addition on the Tensile Strength of Ferrocement

The results showing variation of the tensile strength of two-layered wire mesh reinforced PMF, cured for 28 days, with increase in the percentage of polymer addition, are presented in Figures 3.20 & 3.21 and Table 3.16. It can be observed from the results that the tensile strength of two-layered wire mesh SBR and VAE polymer modified ferrocement increased in the order of 20.16%, 27.32%, 33.95%, 35.01% and 11.94%, 22.28%, 29.18%, 27.85%, respectively with addition of 5%, 10%, 15% and 20% polymers. In the same way, the maximum elongation values of PMF also show an improvement of 6.70%, 12.95%, 16.52%, 18.45% and 3.57%, 6.39%, 10.55%, 14.12%, respectively. The test results showed that the flexural strength properties of PMF enhanced remarkably with 15% to 20% addition of any polymer.

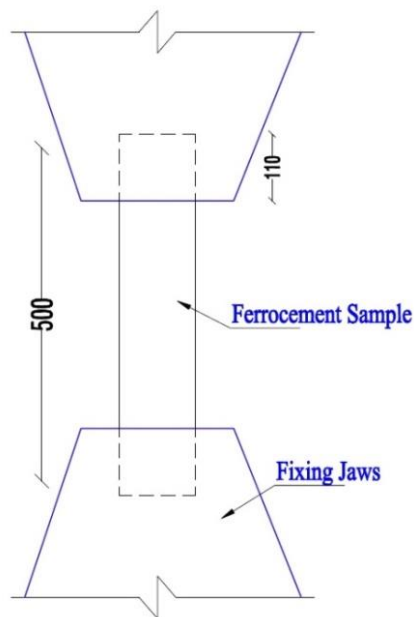


Fig. 3.19: Schematic Diagram of Axial Pull to Find Tensile Strength of Ferrocement



Plate 3.12: Tensile Strength Test of Ferrocement in Progress

The tensile strength and elongation of three-layered PMF (tested after 28 days of curing) were also observed to show an upward trend with the increase in polymer percentage. However, a slight decrease in the tensile strength was observed at 20% polymer addition, but the elongation increased continuously. The results of variation of tensile strength values and elongation values are shown in Figures 3.22 & 3.23 and Table 3.16. The three-layered SBR-modified ferrocement exhibited the best results as the tensile strength

increased by 39.41% and the elongation increased by 17.59% with 15% addition of polymer.

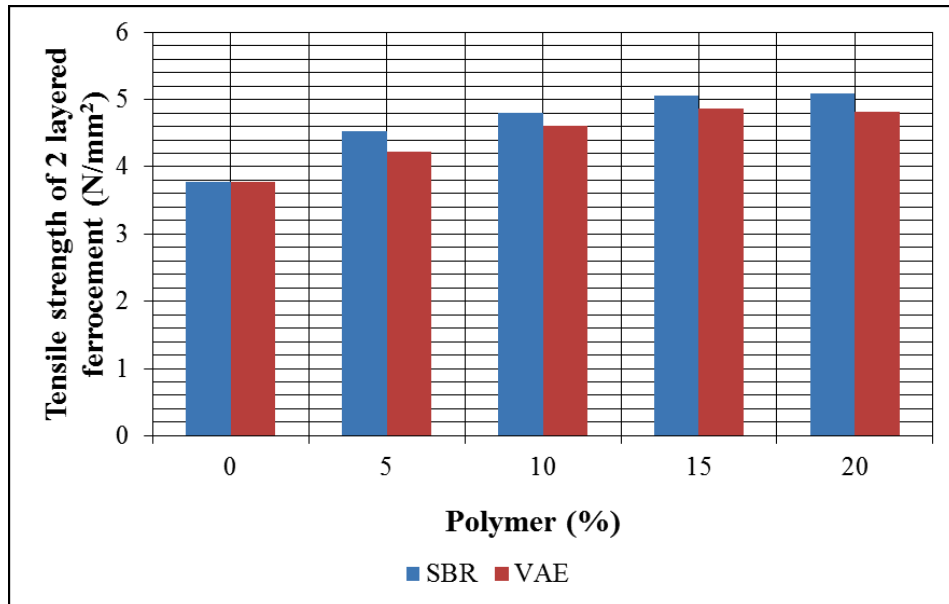


Fig. 3.20: Tensile Strength of 2 Layered Ferrocement (cured for 28 days) v/s Polymer Percentage for Flow Value of 105±5%

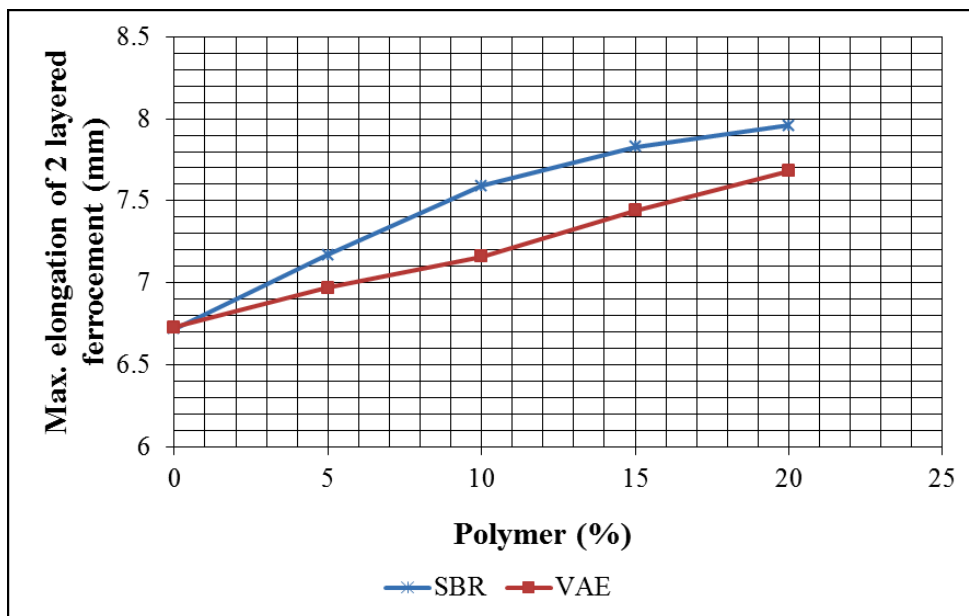


Fig. 3.21: Maximum Elongation of 2 Layered PMF (cured for 28 days) v/s Polymer Percentage for Flow Value of 105±5%

Table 3.16: Tensile Strength-Elongation of 2 and 3 Layered Polymer Modified Ferrocement after 28 Days of Curing

S. No.	Polymer %	2 layered				3 layered			
		SBR		VAE		SBR		VAE	
		Tensile st. (N/mm ²)	Max. Elon. (mm)	Tensile st. (N/mm ²)	Max. Elon. (mm)	Tensile st. (N/mm ²)	Max. Elon. (mm)	Tensile st. (N/mm ²)	Max. Elon. (mm)
1	0	3.77	6.72	3.77	6.73	4.39	6.88	4.39	6.88
2	5	4.53	7.17	4.22	6.97	5.32	7.20	5.07	7.11
3	10	4.80	7.59	4.61	7.16	5.84	7.66	5.51	7.39
4	15	5.05	7.83	4.87	7.44	6.12	8.09	5.65	7.72
5	20	5.09	7.96	4.82	7.68	6.06	8.22	5.81	7.99

The tensile strength and deformational behaviour of PMF show considerable improvement in comparison with unmodified ferrocement (UMF). This is mainly attributed to the formation of a strong polymeric layer within the mortar mix. An increase in percent reinforcement volume also contributes to the enhanced tensile strength and elongation properties of PMF. The improved mechanical properties of PMF are attributable to the refinement of the transition zone and arresting of micro-cracks at the initial stage of loading and their further propagation. The improved workability, dispersion and homogeneity of PMM inter alia improve the tensile strength of PMF.

Investigations on the elongation results show that the elongation was more in SBR modified ferrocement specimen as compared to VAE modified ferrocement specimens. It reflects that the SBR modified ferrocement exhibits more ductility as compared to VAE modified ferrocement.



Plate 3.13: Tensile Failure of Ferrocement Elements

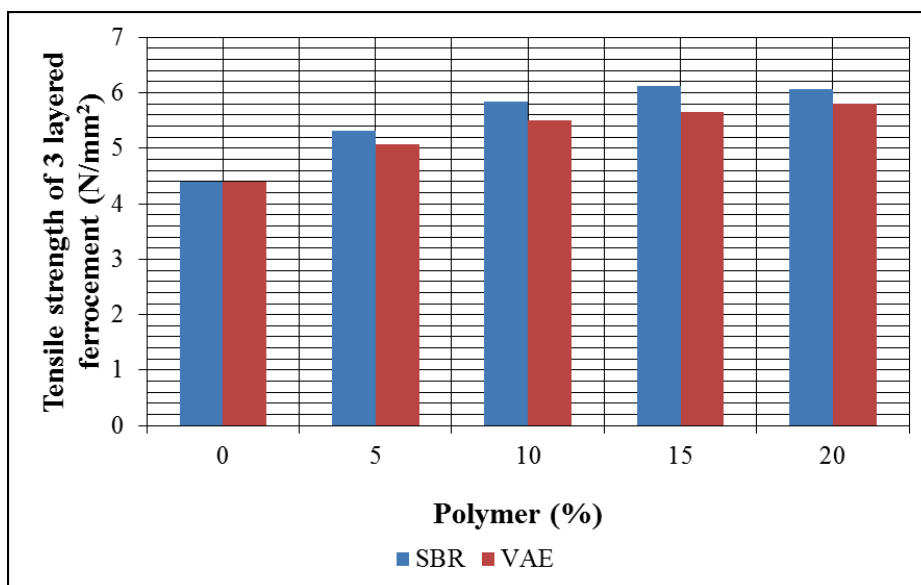


Fig. 3.22: Tensile Strength of 3 Layered Ferrocement (cured for 28 days) v/s Polymer Percentage for Flow Value of 105±5%

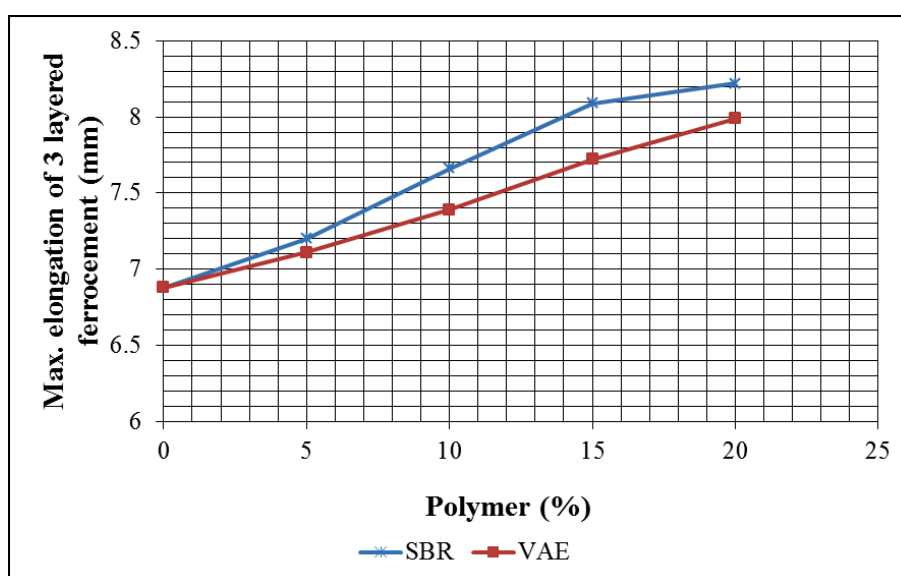


Fig. 3.23: Maximum Elongation of 3 Layered Ferrocement (cured for 28 days) v/s Polymer Percentage for Flow Value of 105±5%

A similar improvement was also seen in the case of VAE modified ferrocement, which exhibited an increase in tensile strength and elongation by 32.35% and 16.13%, respectively with 20% addition. With the increase in the volume fraction of reinforcement, the strength and elongation properties also increased by 5 to 10% for the same percentage addition of both SBR and VAE polymers. The tensile strength of three-layered PMF was observed to increase with addition of 5%, 10%, 15%, 20% SBR and VAE polymers by 21.18%, 33.03%, 39.41%, 38.04% and 15.49%, 25.51%, 28.70%, 32.35%, respectively. The maximum elongation also increased by 4.65%, 11.34%, 17.59%, 19.48% and 3.34%,

7.41%, 12.21%, 16.13%, respectively. Hence, it is concluded that the addition of both types of polymers increased the elongation of specimens which further attributed to improve in ductility. The cracking pattern of random specimens of PMF is shown in Plate 3.13.

3.12 CONCLUSION FROM PILOT STUDY

Based upon the experimental results of the pilot study, the major observations and conclusions are provided as below: -

3.12.1 Polymer Modified Mortar (PMM)

1. The addition of both SBR and VAE polymers enhance the workability of modified mortars as compared to the control mix. Polymer modified mortars (PMMs) show less requirement of water as compared to the control mix for maintaining the same flow value of $105 \pm 5\%$. The w/c ratio requirement decreased by about 42% and 23% in case of 20% addition of SBR and VAE polymers, respectively.
2. The test results exhibit higher strength values with the addition of emulsion-based polymer (SBR) as compared to the powder based polymer (VAE).
3. The compressive strength of VAE polymer modified mortar decreases at constant and adjusted w/c ratio. The compressive strength of SBR modified mortar is though almost the same as compared to the control mix. However, results show a fractional increase of compressive strength up to 7.46% at adjusted w/c ratio for the fixed flow of $105 \pm 5\%$, after 28 days of curing and 15% addition of SBR polymer. At varied water-cement ratios, the compressive strength of VAE modified mortars decreases but the compressive strength of SBR modified mortars increases both after 7 and 28 days.
4. The compressive and flexural strengths of modified mortar peaked at 15% addition of SBR polymer for a constant flow value of $105 \pm 5\%$.
5. The flexural strength of both SBR and VAE modified mortars increases by 30.99% and 23.58%, respectively, with 15% addition, but further addition shows a reduction in strength.
6. It can be concluded that the addition of polymers increased the percentage air content of the mortar matrix and hence the anti-foaming agents need to be used to make the PMM equally dense as the control mortar mix. It can be concluded that the addition of polymers increased the percentage air content of the mortar matrix and it was more so with the use of VAE polymers as compared to SBR polymers.

7. The modulus of elasticity of SBR modified mortars increased with increase in polymer content up to 10 % and then starts decreasing, but modulus of elasticity of VAE modified mortars decreased with increase in polymer content in a mortar.
8. The addition of antifoaming agents maintains the density of polymer-modified mortars equivalent to that of the control mortar mix, which further helped to maintain consistency for comparison of various properties of unmodified and modified mortars.

3.14.2 Polymer Modified Ferrocement (PMF)

1. For obtaining maximum flexural and tensile strength of both two and three-layered polymers modified ferrocement (PMF), the required quantity of either SBR or VAE polymers addition in the mortar matrix should be of the order of 15%. Thus, it is concluded that to obtain better strength results of PMF, a 15% addition of SBR polymer is the optimum percentage addition in the mortar matrix.
2. SBR and VAE modified ferrocement showed an improvement in flexural strength and deformation properties. Two-layered and three-layered SBR modified ferrocement elements show remarkable enhancement in flexural strength of 61.25% and 48.04%, respectively, with 15% addition of polymer as compared to control ferrocement. However, the figurative strength value of three-layered PMF, which is 17.01 N/mm^2 , is higher than that of two-layered PMF, which is 13.98 N/mm^2 .
3. The tensile strength of both VAE and SBR modified ferrocement increases with increase in polymer addition. Strength and elongation of the three-layered polymer modified ferrocement were found higher as compared to two-layered PMF.
4. The SBR modified ferrocement provides better ductility as compared to the VAE modified ferrocement.
5. It could be concluded that the addition of polymers makes the ferrocement more ductile to resist flexural loads.

The optimum percentage of best-performing polymer has been evaluated for different layers of wire mesh. Subsequently, from the obtained results of this pilot study, the best performing three-layered wire mesh polymer modified ferrocement, with the optimum percentage of 15% SBR (*Ghai et al., 2016*) has been further used in this experimental work to strengthen the RCC beams having three different pre-defined pre-damaged levels (45%, 75% and 95%). The number of wire mesh layers has been restricted to three keeping in view the practicability of use of PMF as strengthening material in structural elements in the field.

EXPERIMENTAL PROGRAM AND TEST PROCEDURE

4.1 GENERAL

The experimental program was designed to study the effect of various parameters such as; the spacing of shear stirrups and shear-span to effective depth (a/d) ratios, on the strength properties of reinforced concrete beams, initially stressed in shear to predetermined levels, and subsequently strengthened with polymer modified ferrocement jacket. A total of 48 number of prototype RCC beams of size 127mm $\times$ 229mm $\times$ 2700mm were cast. Out of these 48 beams, the 24 beams were designed for shear (DB), and the rest 24 beams were designed as shear deficient (SDB). A typical sample set of 3 numbers of beams was adopted to investigate the desired purpose of this study.

The control specimens of each category were tested initially to evaluate the behaviour in terms of deflection, cracking, mode of failure and ultimate strength. The results so obtained were used further to fix the limiting load values for instilling partial damages to other sets of beams. Based upon the load-deflection behaviour of control specimens, three different loads limits, as 45%, 75% and 95% of the ultimate load value were fixed. After partially damaging the beams, these were strengthened with polymer modified ferrocement (PMF) for a specified length and re-tested to evaluate their behaviour. The PMF having the best results in terms of flexural and tensile strength has been adopted for the strengthening of these partially damaged RCC beams. These experimental results so obtained were further validated with the help of FEM based software ATENA-3D and is explained in Chapter 6.

4.2 MATERIALS

The designed concrete mix of grade M20 was used to cast the RCC beam specimens. All relevant tests were conducted to find the basic properties of all constituent materials of concrete as per Indian Standard (IS) specifications. The same type and grade of cement was used for developing concrete-mixes and polymer modified ferrocement (PMF) like the ones used in the pilot study. The properties of sand (FA-1) used for PMF are detailed in Section 3.2.2. The properties of other materials used for the concrete mix like; sand (FA-2), coarse aggregates (CA-1, CA-2 and CA-3) are described as below: -

4.2.1 Fine Aggregate (FA-2)

Natural river bed sand (FA-2) of fineness 2.65 is used for developing the concrete mix. The properties of sand have been worked out in the laboratory as per *IS 383:1970* specifications and detailed in Table 4.1 below: -

Table 4.1: Physical Properties of Sand (FA-2)

S. No.	Characteristic	Value
1.	Type	Crushed Sand
2.	Specific Gravity (oven dry basis)	2.675
3.	Fineness Modules	2.65
4.	Water absorption (%)	1.35
5.	Moisture content (%)	0.16
6.	Grading Zone	II

4.2.2 Coarse Aggregate

Two types of coarse aggregates, with a nominal size of 20mm (CA-1) and 12.5mm (CA-2), respectively, mixed in 60:40 ratio, was used to prepare the concrete mix as per *IS: 383-1970 & IS 2386-Part 3* specifications. The details of individual test results of coarse aggregates (CA-1, CA-2) are given in Tables 4.2 and 4.3. The properties of mixed aggregates (CA-3) in the ratio of 60:40 (CA-1: CA-2) were also evaluated and are given in Tables 4.4 below: -

Table 4.2: Physical Properties of Coarse Aggregates (CA-1)

S. No.	Characteristic	Value
1.	Type	Crushed
2.	Specific Gravity (oven dry basis)	2.69
3.	Fineness Modules	6.69
4.	Water Absorption (%)	1.18
5.	Moisture Content (%)	Nil
6.	Size (mm)	20

Table 4.3: Physical Properties of Coarse Aggregates (CA-2)

S. No.	Characteristic	Value
1.	Type	Crushed
2.	Specific Gravity (oven dry basis)	2.685
3.	Fineness Modules	6.11
4.	Water Absorption (%)	1.11
5.	Moisture Content (%)	Nil
6.	Size (mm)	12.5

Table 4.4: Physical Properties of Mixed Coarse Aggregates (CA-3)

S. No.	Characteristic	Value
1.	Type	Crushed
2.	Specific Gravity	2.687
3.	Fineness Modules	6.54
4.	Water Absorption (%)	1.12
5.	Moisture Content (%)	Nil
6.	Size (mm)	Mix aggregates

4.2.3 Concrete Mix

Different trials were made to evaluate the proportion of ingredients of concrete. The concrete mix of grade M20 with C: FA: CA in a ratio of 1: 2.1: 3.4 was designed as per *IS 10262:2009* specifications. The water-cement ratio of the concrete mix was 0.55 and having an ultimate compressive strength after 28 days of curing as 26.72 N/mm^2 . The slump value of mix was kept between 75mm-100mm range of medium workability. The designed concrete mix proportion was used to cast the beam specimens. The quantities of various ingredients required per m³ of concrete are given in Table 4.5.

Table 4.5: Detail of Concrete Mix Design (all quantities are in kg/m³)

Cement (C)	Fine Aggregate (FA-2)	Mixed Coarse Aggregates (CA-3)	Water (W)
340	714.0	1156	187
1	2.1	3.4	0.55

4.2.4 Reinforcement

The thermo-mechanical-treated (TMT) 12mm diameter bars having an ultimate tensile strength of 710 N/mm^2 were used as tensile reinforcement and 8mm diameter bars having an ultimate tensile strength of 697.5 N/mm^2 were used as compressive reinforcement. Plain mild steel (MS) 6mm diameter reinforcement bars of ultimate tensile strength of 591.5 N/mm^2 were used as shear reinforcement. The ultimate tensile strengths of these reinforcement bars have been found out in the laboratory as per *IS 1786:1985* specifications. The tensile strength test conducted on reinforcing bars is shown in Plate 4.1. The properties of the reinforcement used are given in Table 4.6.

Table 4.6: Properties of Reinforcement Steel Bars

Diameter (mm)	Yield Stress (MPa)	Ultimate Stress (MPa)	Elongation (%)
12	556.5	710.0	22.0
8	548.5	697.5	18.4
6	445.0	591.5	25.0



Plate 4.1: Testing of Reinforcement Bars

4.2.5 Polymer Modified Ferrocement (PMF)

The properties of polymer modified mortar and ferrocement were investigated in the pilot study work as detailed in Chapter-3. From the study, it was concluded that the polymer modified ferrocement having three-layers of steel woven wire mesh (volume fraction = 0.695%) and 15% of SBR addition performed better in terms of flexural strength, tensile strength, elongation and deflection. Hence, three-layered PMF with 15% SBR, having a flexural strength of 17.01 N/mm^2 and tensile strength of 6.12 N/mm^2 is being adopted for use in strengthening the pre-damaged RCC beams.

4.3 CASTING OF REINFORCED CONCRETE BEAMS

A total of 48 number prototype beams of size $127\text{mm} \times 229\text{mm} \times 2500\text{mm}$ were cast with M20 grade concrete for the experimental study. All the beams were designed as balanced sections as per *IS 456:2000* (limit state approach) and reinforced with two bars of 12mm diameter on the tensile face and two bars of 8mm diameter on the compression face.

The grade of concrete, the spacing of shear stirrups and the amount of longitudinal reinforcement are the key factors which are responsible for resisting the shear forces in RCC beams. In the present investigation, the grade of concrete and amount of longitudinal reinforcement is kept the same for the entire group of specimens and the only variable is the spacing of shear stirrups. The spacing of stirrups being the key variable is decided based upon the Indian Standard (IS) code specifications. As per clause no. 6.3.5 of *IS 13920:1993*, the spacing of hoops over a length of $2d$ ($198 \times 2 = 396\text{mm}$) at either end of a beam should

not be more than 100mm. However, it was considered that the existing RCC structures might not have been properly detailed in shear and the spacing of stirrups may vary from the one which is actually required as per the design. Hence, in the present study, the effect of different spacing of stirrups on two different shear-span (a/d) ratios was studied.

Therefore, 24 beams were considered to be designed as shear efficient beams (DB) and 18 stirrups of 6mm diameter were provided at a spacing of 150mm c/c. The rest 24 beams were considered to be designed as shear deficient beams (SDB) and 7 stirrups of 6mm diameter were provided at a spacing of 450mm c/c. The reinforcement and sectional details of both types of beams are shown in Figure 4.1.

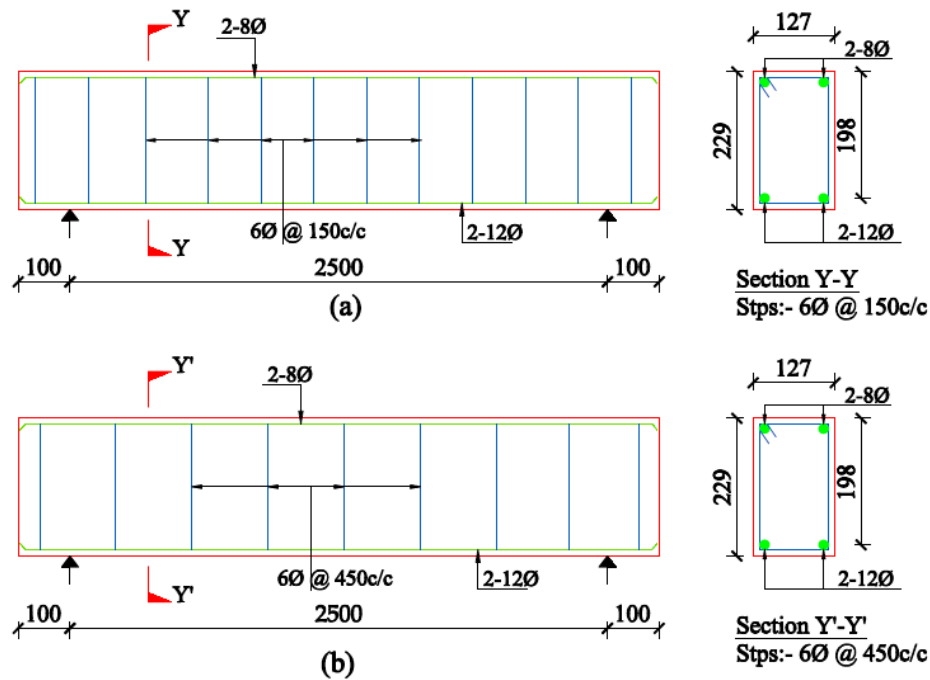


Fig. 4.1: Beam Section and Reinforcement Details
(a) Shear Designed Beams (DB); (b) Shear Deficient Beams (SDB)

Steel formwork/mould of desired dimensions was prepared and used for casting of beam specimens. The *IS 456:2000* specifications were strictly followed. The steel rebars were cut and reinforcement casings for two different types of beams i.e. shear designed (DB) and shear deficient (SDB) beams were bent as detailed in Figure 4.1. The formwork was properly lubricated before casting of beams for ease in the demoulding process. The reinforcement case was correctly placed in the formwork with specified cover and then poured with concrete. Concrete cover blocks of 25mm size were provided under the reinforcement to maintain uniformity. The concrete mix of grade M20 with the ratio of C: FA-2: CA-3 (1: 2.1: 3.4) was used to cast all the beam specimens. Workability of concrete

was kept constant (75mm-100mm) with water/cement (w/c) ratio of 0.55. The electrical pan mixture was used to prepare the concrete mix as shown in Plate 4.3 (a). Nozzle vibrator was additionally used for proper compaction of concrete. Plate 4.2 and 4.3 shows the casting of beams in progress. The beams were demoulded after 24 hours and cured with gunny bags for 28 days before testing of beams. Plate 4.4 shows the beam specimens during curing. Figure 4.2 shows the flow chart of the experimental program for the strengthening of RCC beams using PMF.



Plate 4.2: Preparation of RCC Beam Specimen
(a) Preparation of Steel Formwork; (b) Placement of Reinforcement in the Mould



Plate 4.3: Concreting the RCC Beam Specimen
(a) The Concrete Pan Mixer used for Preparing Concrete Mix; (b) Pouring and Vibrating the Concrete



Plate 4.4: Curing of RCC Beam Samples

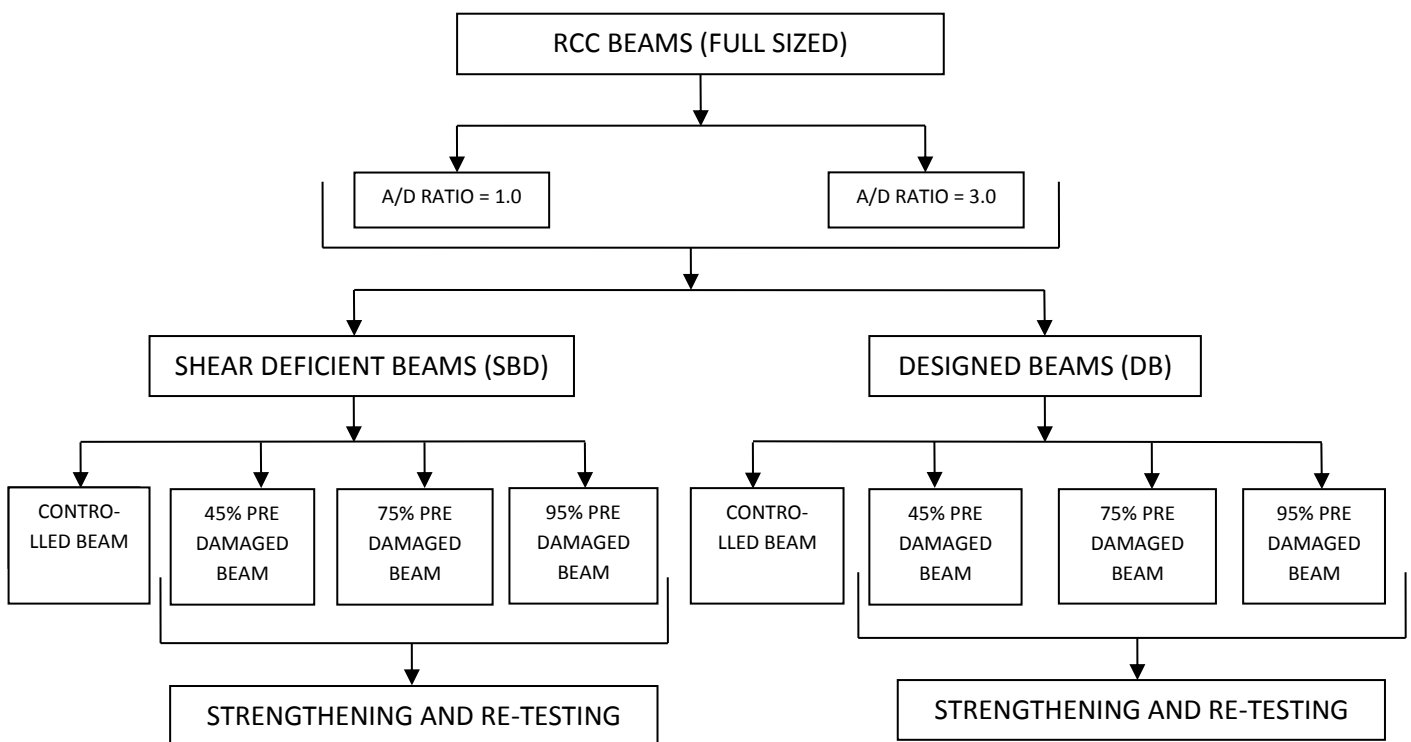


Fig. 4.2: Test Scheme for RCC Beams

4.4 TESTING OF CONTROL RCC BEAM SPECIMENS

In the envisaged experimental program, a typical set of 3 beams was used to evaluate the behaviour of control beams. A total of 12 full-size beams were tested as control beams. Out of these 12 beams, 6 beams of each type i.e. shear designed or shear efficient (DB) and shear deficient (SDB) RCC beams were tested for two different a/d ratios of 1.0 and 3.0. The beam specimens were tested after 28 days of curing over a loading frame fixed with a hydraulic jack. The loading frame and the assembly for testing of RCC beams is shown in Figure 4.3.

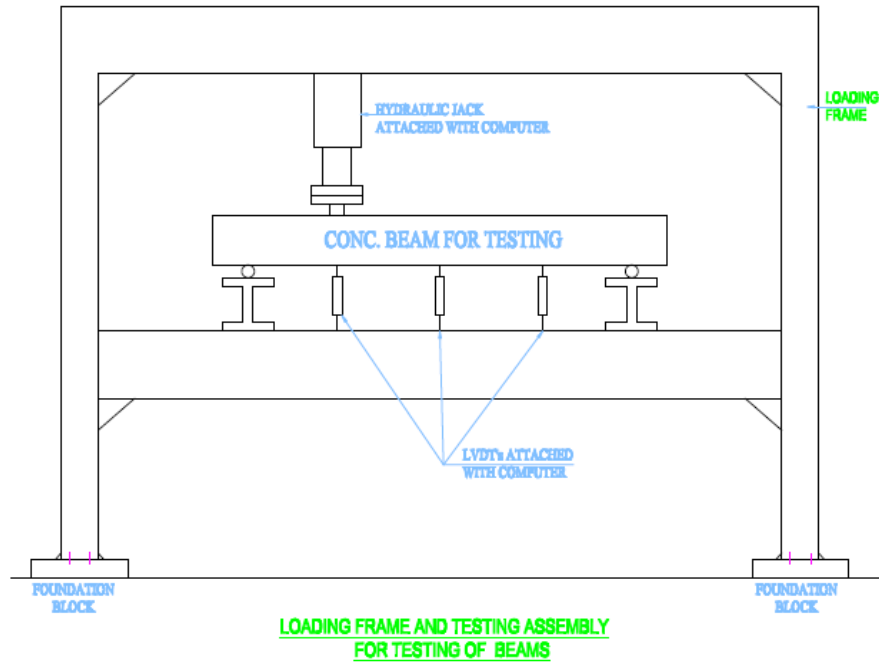


Fig. 4.3: Loading Frame Setup

The test was performed on beams with two different shear-span (a/d) ratios of 1 & 3, having a simply supported effective span of 2500mm as detailed in Figure 4.4. A standard set of 3 beams were taken for each load test. Ultimate shear load carrying capacities and load-deflection curves of DB and SDB control beams were plotted by applying single point load 'P' at two different shear-span ratios (a/d) = 1.0 and 3.0, respectively. Where 'a' denotes the shear span of the beam and 'd = 198mm' denotes the effective depth. For a/d ratio = 1.0, the value of 'a = 198mm' and for a/d = 3.0, the value of 'a = 594mm'; respectively, was kept for the load test. These control beam specimens were tested up to ultimate failure, and subsequently, 45%, 75% and 95% of the ultimate load values were worked out corresponding to their average elastic, elastoplastic and plastic limits. Plate 4.5 shows the process of shifting and placing of RCC beam specimens on the loading frame. Plate 4.6 and 4.7 shows the 'testing in progress' of beams under a/d ratio 1 & 3, respectively.

The deflections of these beams were recorded electronically by using LVDTs (linear variable displacement transducers) having a least count of 0.001mm. These LVDTs were placed under the soffit of beams and further attached with the computerised system. The locations of LVDTs were fixed according to the load position, i.e. the 1st LVDT was placed under the loading point 'P'; the 2nd LVDT was placed under the midpoint of the beam and the 3rd was exactly at the same distance from opposite support just a mirror

image to the 1st LVDT (refer Figure 4.4). The cracking pattern, failure modes, deflections and ultimate strength of beams were observed/ recorded during the test. The load v/s deflection curves of 1st LVDT are only discussed in this research work for comparative study.

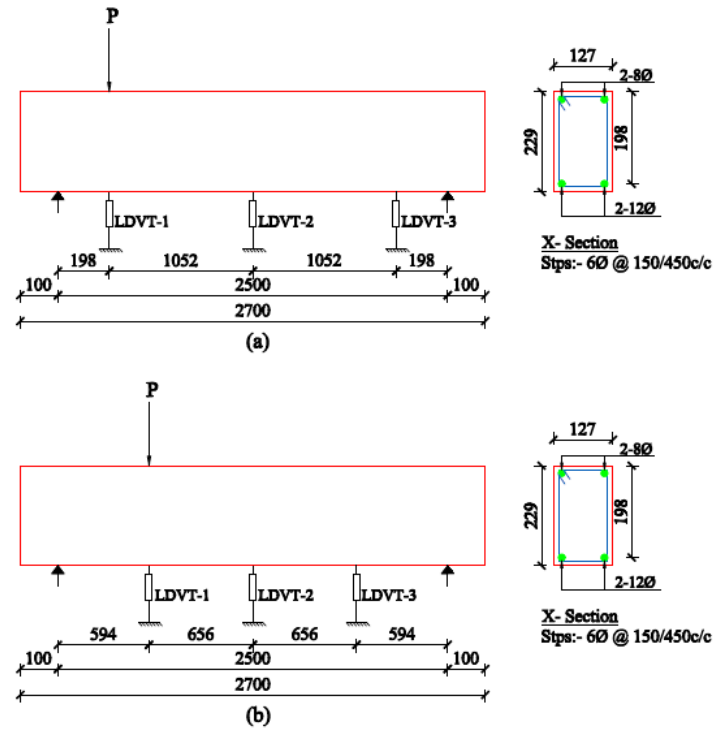


Fig. 4.4: Schematic Diagram of the Test Setup
 (a) Beams Tested at $a/d = 1$; (b) Beams Tested at $a/d = 3$



Plate 4.5: Shifting and Placing of Beams on the Loading Frame

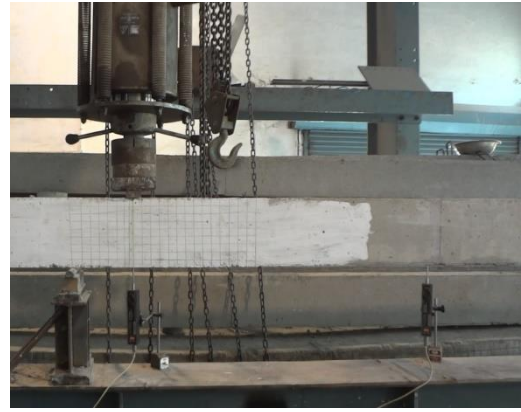
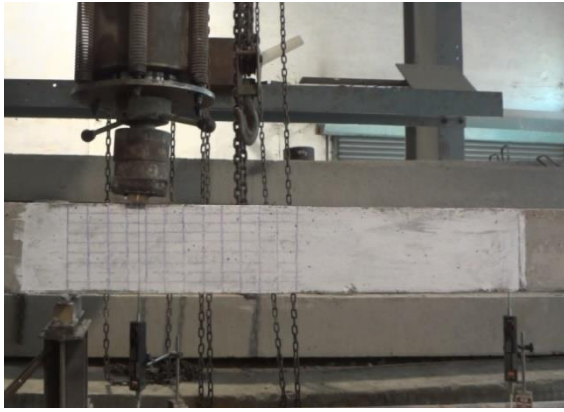


Plate 4.6: Testing of RCC Beams in Progress ($a/d = 1.0$)



Plate 4.7: Testing of RCC Beams in Progress ($a/d = 3.0$)

4.4.1 Control Shear Designed (DB) Beams

Tables 4.7 and 4.8 show the load values and deflection values of control shear designed (DB) beams at all LVDTs points for a/d ratios 1 and 3, respectively. Figures 4.5 and 4.6 show the graphical representation of load-deflection curves for these beams. Plate 4.8 shows the cracking pattern of DB.



(a)



(b)

Plate 4.8: Shear Designed Beams tested at: (a) $a/d = 1$; (b) $a/d = 3$

Table 4.7: Control Shear Designed Beams (DB) Tested at a/d Ratio 1.0

S. No.	Beam 1				Beam 2				Beam 3			
	Load (kN)	Deflection (mm) at			Load (kN)	Deflection (mm) at			Load (kN)	Deflection (mm) at		
		A	B	C		A	B	C		A	B	C
1.	0	0	0	0	0	0	0	0	0	0	0	0
2.	12	0.246	0.428	0.006	12	0.202	0.356	0.058	12	0.234	0.392	0.059
3.	24	0.446	0.978	0.166	24	0.444	0.764	0.118	24	0.445	0.871	0.142
4.	36	0.634	1.426	0.304	36	0.72	1.196	0.178	36	0.68	1.311	0.241
5.	48	0.914	2.11	0.52	48	1.042	1.772	0.256	48	0.95	1.941	0.388
6.	60	1.24	2.842	0.73	60	1.504	2.574	0.278	60	1.356	2.708	0.504
7.	72	1.654	3.624	0.794	72	2.19	3.346	0.432	72	1.89	3.485	0.613
8.	84	2.256	4.454	1.018	84	2.688	4.286	0.586	84	2.468	4.37	0.802
9.	96	2.646	5.3	1.19	96	3.172	5.11	0.64	96	2.899	5.205	0.915
10.	108	3.042	6.022	1.376	108	3.656	5.954	0.804	108	3.325	5.988	1.09
11.	120	3.496	6.946	1.548	120	4.164	6.802	0.874	120	3.785	6.874	1.211
12.	132	4.05	8.094	1.828	132	4.812	7.784	1.066	132	4.455	7.939	1.447
13.	144	4.67	9.034	2.01	144	6.174	9.03	1.262	144	5.56	9.032	1.636
14.	147.84	5.602	9.882	2.19	145.2	6.536	9.232	1.282	138	6.254	9.338	2.524
15.	145.5	6.646	10.441	2.27	144	7.176	9.536	1.288	134	6.452	9.368	2.28
16.	135.45	7.264	10.65	2.294	139.68	7.774	9.74	1.288				
17.					133.74	7.982	9.77	1.288				
18.					127.86	8.526	9.852	1.288				

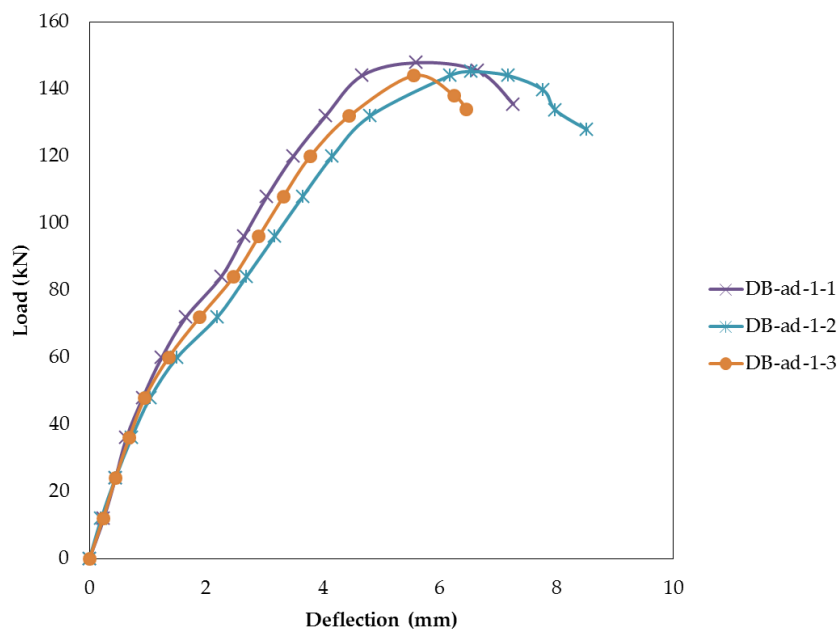


Fig. 4.5: Load-Deflection Curves of Control Shear Designed (DB) Beams Tested at a/d = 1.0

Table 4.8: Control Shear Designed Beams (DB) Tested at a/d ratio 3.0

S. No.	Beam 1				Beam 2				Beam 3			
	Load (kN)	Deflection (mm) at			Load (kN)	Deflection (mm) at			Load (kN)	Deflection (mm) at		
		A	B	C		A	B	C		A	B	C
1.	0	0	0	0	0	0	0	0	0	0	0	0
2.	6	0.73	0.912	0.66	6	0.834	0.9	0.294	6	1.023	1.235	0.587
3.	12	1.648	1.98	1.322	12	1.342	1.594	0.812	12	2.108	2.148	1.009
4.	18	2.77	3.252	2.158	18	2.07	2.488	1.414	18	2.896	2.886	1.528
5.	24	3.844	4.516	2.97	24	2.99	3.544	2.13	24	3.564	3.846	2.112
6.	30	4.824	5.782	3.604	30	3.736	4.36	2.452	30	4.211	4.441	2.457
7.	36	5.998	7.17	4.362	36	4.672	5.424	3.218	36	5.316	5.128	3.229
8.	42	7.156	8.16	5.14	42	5.728	6.612	3.888	42	6.726	6.634	3.896
9.	48	8.194	9.798	5.904	48	6.724	7.826	4.646	48	7.542	7.742	4.874
10.	54	9.344	11.174	6.592	54	7.71	8.872	5.166	54	8.556	8.869	5.011
11.	60	10.598	12.456	7.288	60	8.96	9.984	5.914	60	9.52	9.982	6.235
12.	61.62	14.472	15.332	8.582	66	10.982	11.638	6.800	64.62	13.282	12.724	8.242
13.	58.93	17.41	17.59	9.614	67.35	15.424	15.804	9.012	60.39	20.632	18.676	10.176
14.	56.72	19.194	18.946	10.238	63.81	20.456	16.945	10.656				

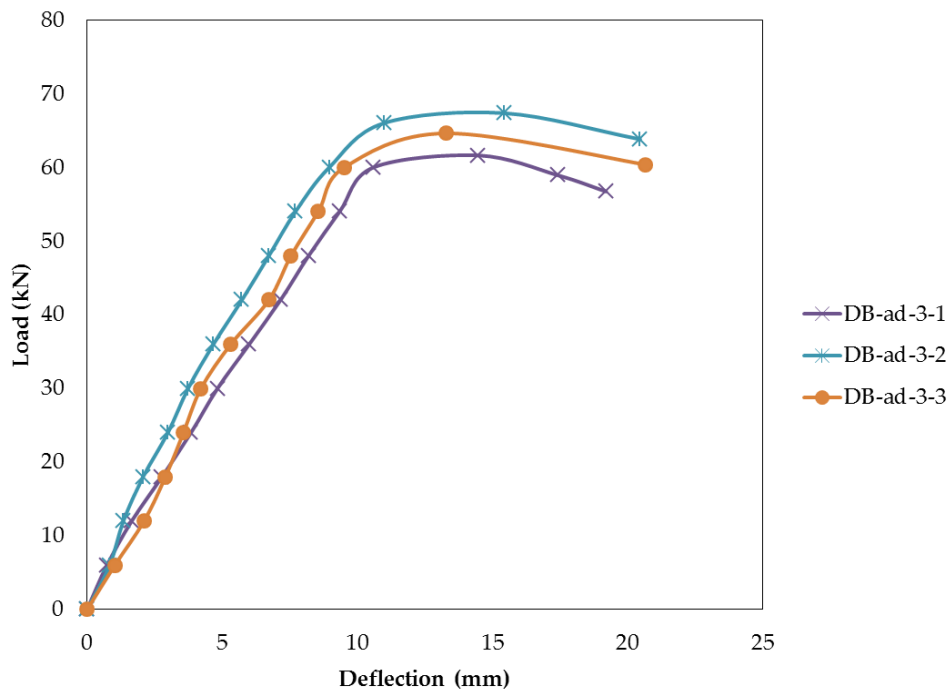


Fig. 4.6: Load-Deflection Curves of Control Shear Designed (DB) Beams Tested at a/d = 3.0

Table 4.9: Control Shear Deficient Beams (SDB) Tested at a/d Ratio 1.0

S. No.	Beam 1				Beam 2				Beam 3			
	Load (kN)	Deflection (mm) at			Load (kN)	Deflection (mm) at			Load (kN)	Deflection (mm) at		
		A	B	C		A	B	C		A	B	C
1.	0	0	0	0	0	0	0	0	0	0	0	0
2.	12	0.448	0.456	0.086	12	0.202	0.356	0.058	12	0.246	0.428	0.006
3.	24	0.696	0.942	0.188	24	0.444	0.764	0.118	24	0.446	0.978	0.166
4.	36	1.034	1.522	0.294	36	0.72	1.196	0.178	36	0.634	1.426	0.304
5.	48	1.362	2.326	-0.05	48	1.042	1.772	0.256	48	0.914	2.11	0.52
6.	60	1.748	3.014	0.064	60	1.504	2.574	0.278	60	1.24	2.842	0.73
7.	72	2.312	3.886	0.26	72	2.09	3.346	0.432	72	1.654	3.624	0.794
8.	84	2.792	4.774	0.466	84	2.588	4.286	0.586	84	2.256	4.454	1.018
9.	96	3.282	5.742	0.67	96	3.072	5.11	0.64	96	2.646	5.3	1.19
10.	108	3.784	6.676	0.874	108	3.556	5.954	0.804	108	3.042	6.022	1.376
11.	120	4.388	7.452	1.008	120	4.064	6.802	0.874	120	3.496	6.946	1.548
12.	132	4.986	8.19	1.172	132	4.712	7.784	1.066	132	4.05	8.094	1.828
13.	140.28	6.03	9.272	1.402	144	7.014	10.27	1.576	144	4.67	9.034	2.01
14.	135.89	6.862	10.571	1.408	146.01	9.084	11.784	1.898	146.55	5.456	8.872	1.804
15.	132.82	7.172	11.185	1.764	142	9.756	12.166	2.162	141.96	6.596	8.912	1.802
16.	128.11	7.861	12.747	2.132					137.86	7.132	9.544	2.121

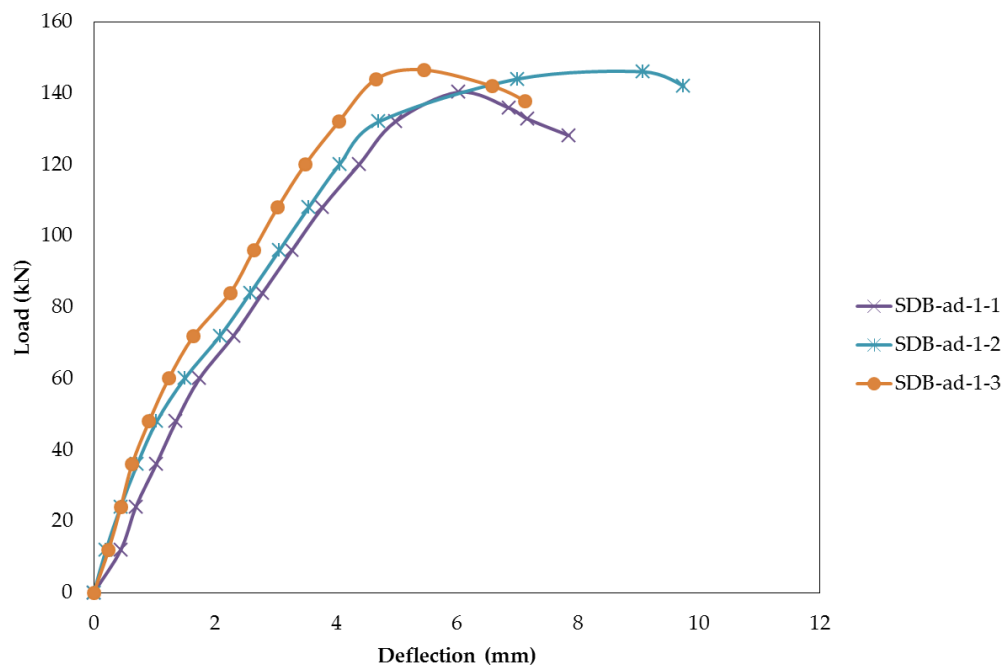


Fig. 4.7: Load-Deflection Curves of Control Shear Deficient (SDB) Beams Tested at a/d = 1.0

Table 4.10: Control Shear Deficient Beams (SDB) Tested at a/d Ratio 3.0

S. No.	Beam 1				Beam 2				Beam 3			
	Load (kN)	Deflection (mm) at			Load (kN)	Deflection (mm) at			Load (kN)	Deflection (mm) at		
		A	B	C		A	B	C		A	B	C
1.	0	0	0	0	0	0	0	0	0	0	0	0
2.	6	0.301	0.451	0.3	6	0.546	0.757	0.412	6	0.95	1.068	0.526
3.	12	0.901	0.932	0.631	12	1.21	1.358	0.784	12	1.518	1.972	0.942
4.	18	1.503	1.653	1.172	18	1.856	2.14	1.324	18	2.238	2.63	1.484
5.	24	2.135	2.706	1.744	24	2.598	3.083	1.89	24	2.998	3.48	2.04
6.	30	2.946	3.608	2.225	30	3.31	3.922	2.275	30	3.672	4.24	2.33
7.	36	4.012	4.885	2.934	36	4.311	5.011	2.892	36	4.402	5.11	2.846
8.	42	5.051	6.163	3.623	42	5.242	6.22	3.564	42	5.474	6.256	3.484
9.	48	6.25	7.366	4.314	48	6.35	7.41	4.201	48	6.496	7.462	4.092
10.	54	7.967	7.989	5.066	54	7.9	8.815	4.903	54	7.638	8.644	4.744
11.	54.06	7.972	8.808	5.080	60	9.842	10.574	5.814	60	10.09	10.754	5.892
12.	53.26	8.211	8.922	5.116	63	11.35	11.627	6.008	61.59	11.79	11.994	6.442
13.	51.37	8.984	9.372	5.342	61.08	13.37	13.98	6.782	59.63	14.168	14.506	7.624
14.	49.22	9.486	10.028	5.934	60.37	14.327	14.584	6.942	56.37	15.232	16.237	8.458

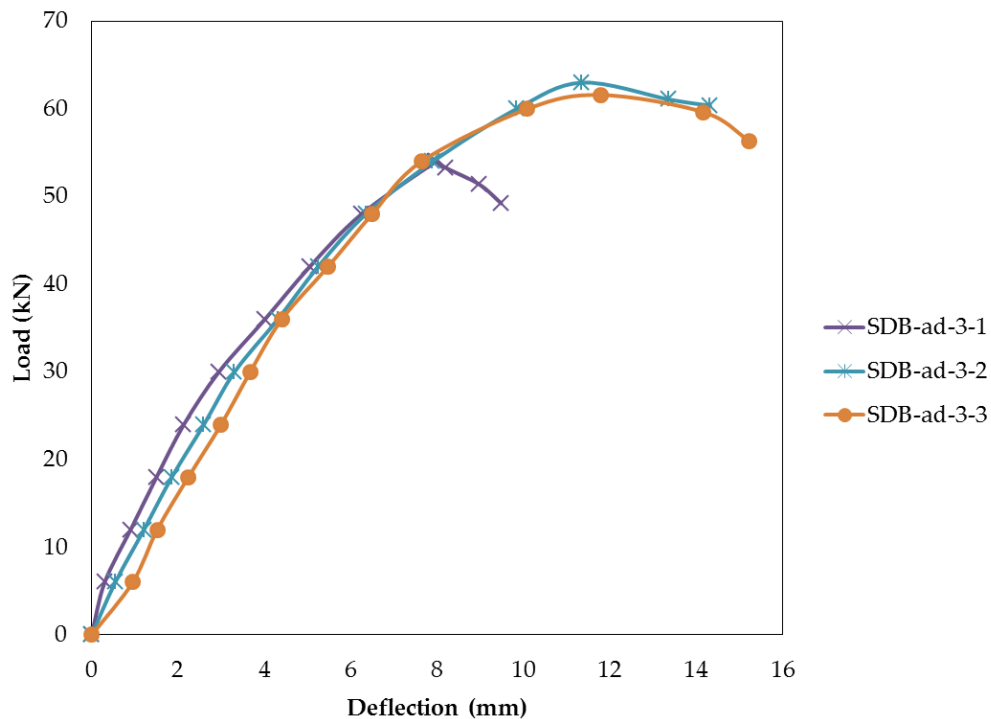


Fig. 4.8: Load-Deflection Curves of Control Shear Deficient (SDB) Beams Tested at $a/d = 3.0$

4.4.2 Control Shear Deficient (SDB) Beams

Tables 4.9 and 4.10 show the load and deflection values of control shear deficient (SDB) beams at all LVDTs points for a/d ratio 1 and 3, respectively. Figures 4.7 and 4.8 show the graphical representation of load-deflection curves for these beams. Plate 4.9 shows the cracking pattern of SDB.

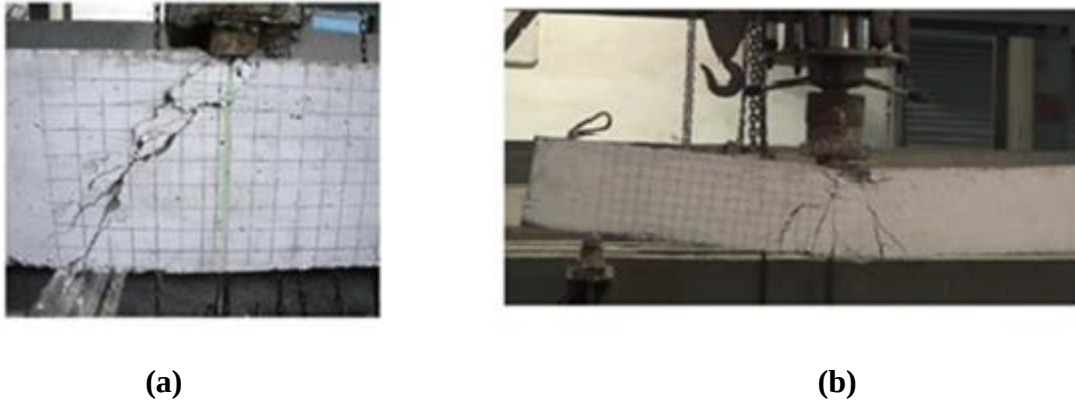


Plate 4.9: Shear Deficient Beams Tested at: (a) $a/d = 1$; (b) $a/d = 3$

4.5 TEST RESULTS OF CONTROL BEAM SPECIMENS

The control beams were loaded up to the ultimate failure point. Most of the beams showed diagonal cracking patterns and a shear mode of failure. The ultimate load of beams tested at a/d 3.0 showed almost 3 times lesser values as compared to the RCC beams tested at a/d of 1.0, irrespective of the stirrups spacing. Further, the cracking and failure behaviour of DB beams show a linear pattern at both a/d ratios as compared to SDB beams. In case of shear deficient beams, the beams tested at $a/d = 1.0$ show cracking pattern and behaviour somehow similar to shear designed beams but SDB beams tested at $a/d = 3.0$ showed sudden failure with wider cracks. This is mainly attributed to the fact that the spacing of shear stirrups is more and in many cases the reinforcement might not have been encountered in the path of crack, thus leading to sudden failure of beams. At a/d ratio = 1.0, the point load tends to transfer the load directly to the support with strut/corbel action and apparently decreased the contribution of stirrups to resist the total shear force, thereby making the stirrups less effective. Hence, in such a case, the total experimental shear strength of beams may be primarily due to the strength of concrete.

The average ultimate load values and deflection under the point load (at the 1st LVDT position) of both types of control beams are given in Table 4.11 and shown in Figure 4.9. From the test results of control beams, the load-deflection curves were plotted for this position for comparison purposes and also to affix the partial failure loads, needed for

setting the pre-damaging limits for other beam specimens. Based upon the load-deflection curves, three zones viz. elastic, elasto-plastic and plastic have been identified and the load values corresponding to 45%, 75% and 95% were worked out accordingly (refer Table 4.11). These calculated load values have been further used for inducing pre-damage to the beams. The other sets of 3 beams were loaded to the affixed pre-damaged levels for particular fixed sets of parameters (i.e. a/d ratio and stirrups spacing). These beams were partially damaged w.r.t. pre-fixed load for different zones as discussed above. The load-deflection behaviour and cracking pattern of these beams were observed and is discussed in Chapter-5.

Table 4.11: Detail of Pre-Damaging Fix Load Test Results of Control DB and SDB Beams

Designation of Beams	a/d Ratio	Avg. Ultimate Load Value (kN)	Avg. Deflection at Ultimate Load (mm)	Calculated Load Value for Different Stress Levels (kN)		
				45%	75%	95%
DB-ad-1	1	145.68	5.899	65.56	109.26	138.40
DB-ad-3	3	64.53	14.393	29.04	48.40	61.30
SDB-ad-1	1	144.28	6.857	64.93	108.21	137.07
SDB-ad-3	3	59.55	10.371	26.80	44.66	56.57

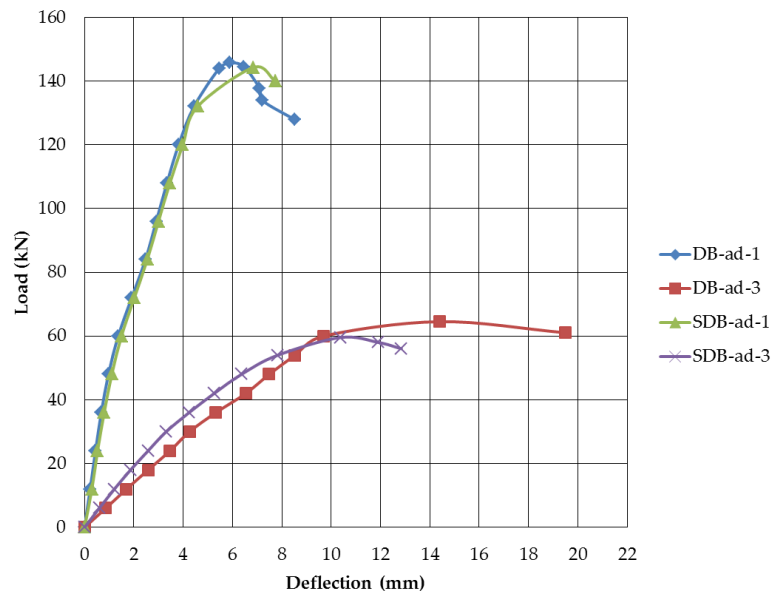


Fig. 4.9: Load-Deflection Test Results of Control DB and SDB Beams

The beams were designated as per the level of initial stresses as 45, 75, 95 and a/d ratios 1 and 3. The prefix 'R' is also used to designate the strengthened beams. The detail of the beams designations is given in Table 4.12. The DB45 & SDB45 beams were initially

loaded to a 45% level of damage. Similarly, DB75 & SDB75 and DB95 & SDB95 were loaded to 75% and 95% level of damage, respectively.

Table 4.12: Designation Details of Shears Designed and Shear Deficient RCC Beams

Designation of Beams	Loading Description	Shear-Span Ratio (a/d)	No. of Samples
DB-ad-1	Shear designed control beams	1.0	3
RDB45-ad-1	45% pre-damage + strengthening	1.0	3
RDB75-ad-1	75% pre-damage + strengthening	1.0	3
RDB95-ad-1	95% pre-damage + strengthening	1.0	3
DB-ad-3	Shear designed control beams	3.0	3
RDB45-ad-3	45% pre-damage + strengthening	3.0	3
RDB75-ad-3	75% pre-damage + strengthening	3.0	3
RDB95-ad-3	95% pre-damage + strengthening	3.0	3
SDB-ad-1	Shear deficient control beams	1.0	3
RSDB45-ad-1	45% pre-damage + strengthening	1.0	3
RSDB75-ad-1	75% pre-damage + strengthening	1.0	3
RSDB95-ad-1	95% pre-damage + strengthening	1.0	3
SDB-ad-3	Shear deficient control beams	3.0	3
RSDB45-ad-3	45% pre-damage + strengthening	3.0	3
RSDB75-ad-3	75% pre-damage + strengthening	3.0	3
RSDB95-ad-3	95% pre-damage + strengthening	3.0	3

DB: Shear designed beam; RDB: Strengthened shear designed beam; SDB: Shear deficient beam; RSDB: Strengthened shear deficient beam

4.6 PRE-DAMAGING OF BEAM SPECIMENS

Based upon the ultimate load test values, the other sets of typical 3 number of beams were loaded for different load values based upon the three zones located as the per load-deflection behaviour curves obtained for control beams. The first zone was located in the elastic zone, the second was located in the elasto-plastic zone, and the third one was located in the pure plastic zone. The corresponding load values were worked out according to 45%, 75% and 95% level of ultimate load values, respectively. The typical set of beams was pre-damaged as per above-mentioned load/stress levels. The study was conducted for both a/d ratios, i.e. 1.0 & 3.0.

The beams are designated according to their pre-damaging level and a/d ratio and detailed in Table 4.12. The detail of the load values calculated corresponding to percentage of damages levels are given in Table 4.11. The beams were placed on the same loading setup as used for control beams (refer to Figure 4.3 and 4.4). Different sets were loaded to different damaged pre-defined damaged levels at a/d ratio 1.0 and 3.0, as shown in Plate

4.10 & 4.11. These damaged beams were then unloaded and shifted for re-strengthening. The strengthening of these partially damaged beams was done with 20mm thick U-shaped polymer modified ferrocement jacketing containing 15% SBR latex and three-layers of the square woven steel wire mesh. The detail of strengthening is given in subsequent sections. The load-deflection behaviour, mode of failures and cracking pattern of these beams were observed during testing and discussed in Chapter-5.

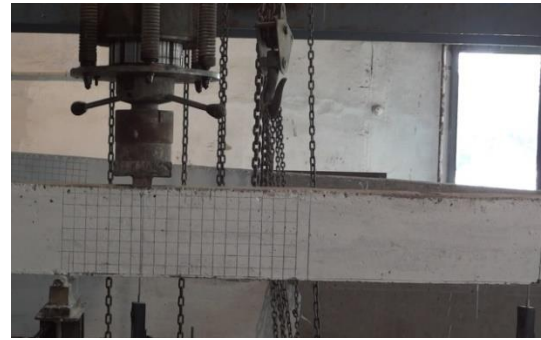


Plate 4.10: Pre-Damaging of RCC Beams at $a/d = 1.0$

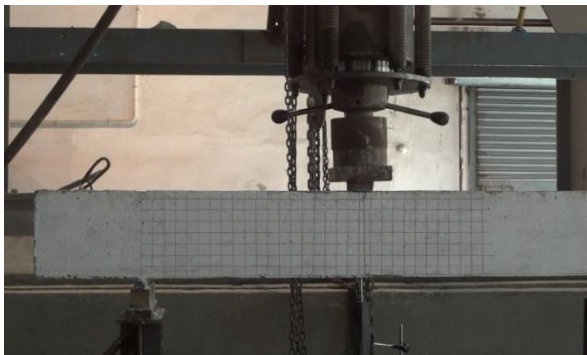


Plate 4.11: Pre-Damaging of RCC Beams at $a/d = 3.0$

4.7 STRENGTHENING OF PRE-DAMAGED BEAM SPECIMENS

4.7.1 Cleaning and Preparing Surface

The behaviour of strengthened beams is highly dependent upon the surface preparation and application of the strengthening material. The repaired surface of the beams should be free from dirt, oil, dust, existing matter, and all curing compounds. Improper preparation of surface can result in debonding of PMF jacketing. Hence, the surface should be cleaned and prepared properly before application of any repairing and strengthening material.

Before the strengthening procedure, the beams were turned upside down to expose their soffit. The particular portion of all the pre-damaged beams was cleaned with a wire

brush at their soffit and side faces where the jacketing is supposed to be applied (Plate 4.12). From the reviewed literature (Sheikh *et al.*, 2002; Mosallam and Banerjee, 2007; Ibrahim and Mahmood, 2009; Zhao *et al.*, 2012), it was decided to strengthen the beams for a length of '2d,' i.e. for 396mm. The surface was cleaned in such a way so as to expose the aggregates and make the surface sufficiently rough for application of repairing mortar. Water was sprayed on the prepared surface to make it moist.

As earlier concluded from the pilot study, it was decided to use the three-layer square woven steel wire mesh PMF having an optimum percentage of 15% SBR, as it showed better strength properties and hence, was adopted to strengthen the pre-damaged RCC beams. The polymer modified repairing mortar constituted of cement, sand, SBR and water in the ratio of 1: 2: 0.15: 0.35. A silicon-based anti-foaming agent was also added to the polymer modified mortar matrix.



Plate 4.12: Preparation of Surface before Strengthening

4.7.2 Strengthening Procedure

First of all, the three layers of wire mesh were cut and affixed over the specified length of beams and then SBR modified mortar was applied. The cement slurry was used as a bonding agent. A total of 20mm thick U-shaped PMF jacketing was applied under the soffit and side faces of the beams in a length of ' $2d = 396\text{mm}$ ', centred on the loading point, for both DB and SDB type beams, irrespective of a/d ratios. The repair material was accurately positioned with the help of wooden forms. The strengthened portion of the beams was cured with gunny bags for further 28 days. The curing process started 24 hours after the application of PMF. The details of PMF strengthened beams are shown in Figure 4.10 and Plate 4.13.

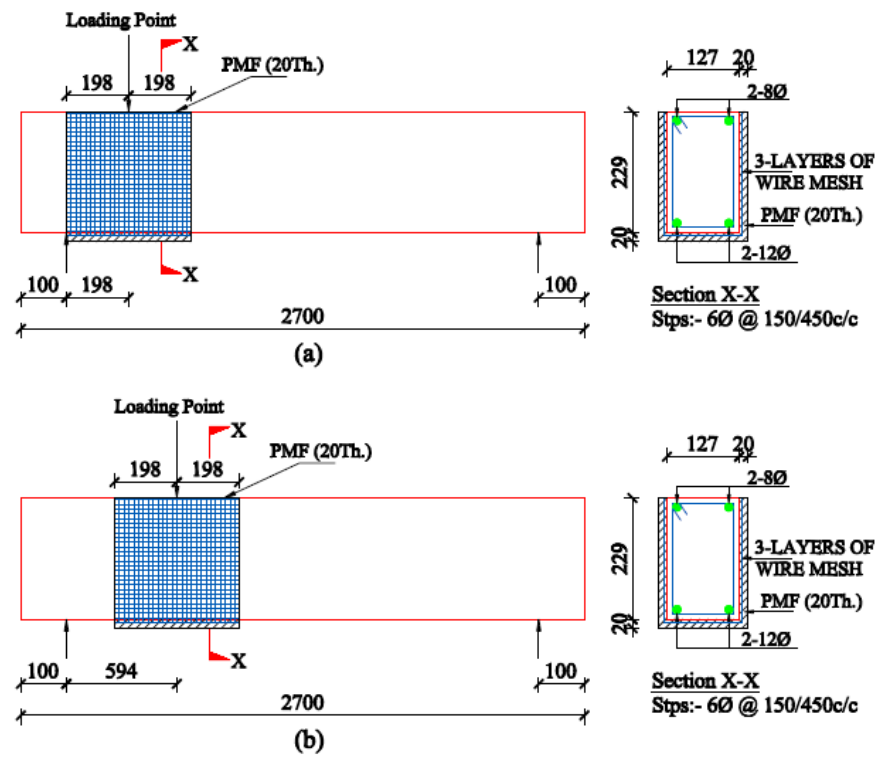


Fig. 4.10: Detail of PMF Strengthening
 (a) Beams Tested at $a/d = 1$; (b) Beams Tested at $a/d = 3$





**Plate 4.13: Application of PMF Jacketing on the Soffit and Side Faces of the Beams
(by turning the beams upside down)**

4.8 RE-TESTING OF PRE-DAMAGED STRENGTHENED BEAMS

The pre-damaged RCC beam specimens strengthened with polymer modified ferrocement (PMF) jacketing were again placed on the same loading frame setup and tested as per the earlier laid down procedure (Figure 4.4) to find their ultimate shear load carrying capacities, deflections, cracking pattern and failure modes. The locations of LVDTs were kept the same as in the case of control beams. The first LVDT (A) was placed just below the loading point, i.e. at a distance of ‘ $a=198\text{mm}$ ’ for a/d ratio 1.0 and at a distance ‘ $a=594\text{mm}$ ’ for a/d ratio 3.0. The second LVDT (B) was placed in the middle of beam specimen, i.e. at a distance of $l/2$ from the support. The third LVDT (C) was placed a mirror image opposite to first LVDT (A) at the same distance from opposite support (refer Fig. 4.3 and 4.4). The load-deflection behaviour of all sets of tested beams was recorded for conclusive study. The results obtained from the detailed experiments have been discussed in Chapter-5. The Plates 4.14 and 4.15 show the progression of test on strengthened RCC beam specimens under a/d ratio 1 and 3.



Plate 4.14: Testing of Strengthened RCC Beams in Progress ($a/d = 1.0$)

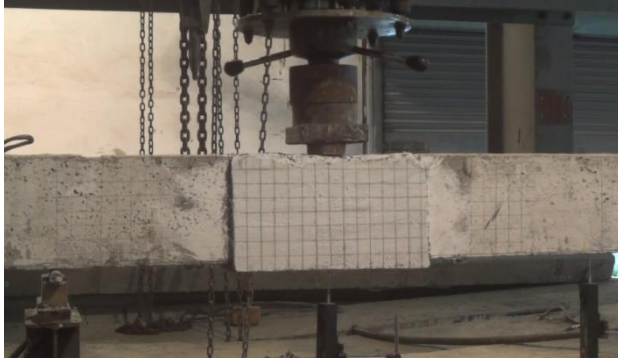


Plate 4.15: Testing of Strengthened RCC Beams in Progress ($a/d = 3.0$)

CHAPTER – 5

RESULTS AND DISCUSSION

5.1 GENERAL

In this chapter, the test results of partial damaged PMF strengthened beams have been discussed. The partially damaged beams unloaded from the loading frame and strengthened with PMF as detailed in Chapter 4. After strengthening, these beams were again loaded for ultimate failure. The load-deflection behaviour, modes of failure and the cracking pattern were observed for these strengthened beams. The load deflection values of all the three positioned LVDTs for strengthened beams having pre-damaging of 45%, 75% and 95% at different a/d ratios and stirrups spacing are detailed in subsequent sections. The test results of strengthened beams have been compared with corresponding control beams and the effect due to variation in parameters such as; initial damage level, a/d ratio and spacing of stirrups of these beams after strengthening is discussed.

5.2 TEST RESULTS OF STRENGTHENED SHEAR DESIGNED (RDB) RCC BEAMS

Pre-damaged PMF strengthened shear designed (RDB) beams were tested for two different a/d ratios as 1.0 and 3.0 as control beams. A typical set of 3 beams was used to evaluate the behaviour of strengthened beams. The test setup along with locations of LVDTs has been detailed in Chapter 4. The deflection at all the three points corresponding to load values has been given in Table 5.1 to 5.6 for beams tested at a/d ratio 1 and 3. For comparison purpose, the load-deflection curve corresponding to 1st LVDT position is only discussed in subsequent sections. Figure 5.1 to 5.6 shows the Load-Deflection plot of control and strengthened shear designed beams initially loaded for different stress levels and tested at a/d ratio 1 and 3.

Table 5.1: 45% Pre-Damaged Strengthened Shear Designed Beams (RDB45-ad-1) at a/d Ratio 1.0

S. No.	Beam 1				Beam 2				Beam 3			
	Load (kN)	Deflection (mm) at			Load (kN)	Deflection (mm) at			Load (kN)	Deflection (mm) at		
		A	B	C		A	B	C		A	B	C
1.	0	0	0	0	0	0	0	0	0	0	0	0
2.	12	0.674	1.186	0.284	12	0.638	0.916	0.206	12	0.664	1.023	0.274
3.	24	1.304	2.18	0.528	24	1.136	1.762	0.398	24	1.269	2.047	0.517
4.	36	1.876	3.022	0.716	36	1.612	2.506	0.592	36	1.778	3.08	0.722
5.	48	2.416	3.986	0.92	48	2.09	3.28	0.81	48	2.317	3.865	0.947
6.	60	2.962	4.784	1.134	60	2.502	3.996	0.976	60	2.847	4.632	1.157
7.	72	3.472	5.75	1.206	72	3.062	4.816	1.178	72	3.364	5.655	1.289
8.	84	3.982	6.692	1.422	84	3.55	5.646	1.382	84	3.871	6.7	1.4
9.	96	4.548	7.708	1.676	96	4.176	6.824	1.47	96	4.446	7.636	1.632
10.	108	5.172	8.792	1.878	108	4.87	7.724	1.584	108	5.07	8.667	1.947
11.	120	6.006	9.588	2.034	120	5.698	8.858	1.79	120	6.127	9.462	2.012
12.	132	7.886	11.76	2.336	132	7.238	10.762	2.24	132	7.342	11.022	2.228
13.	144	10.78	15.012	2.782	144	9.125	12.856	2.66	144	9.445	13.248	2.847
14.	148.38	12.8	16.38	2.956	149.33	10.618	13.472	2.742	149.72	12.254	13.847	2.994
15.	139.38	15.002	18.214	3.278	142.71	11.62	13.99	2.724	143.72	13.8	14.1	3.112
16.	136.41	15.314	18.412	3.244	125.66	12.486	14.088	2.732	134	14.82	15.623	3.442
17.	135.44	15.344	18.412	3.244	109.71	13.626	14.188	2.738	122	15.56	16.874	3.685

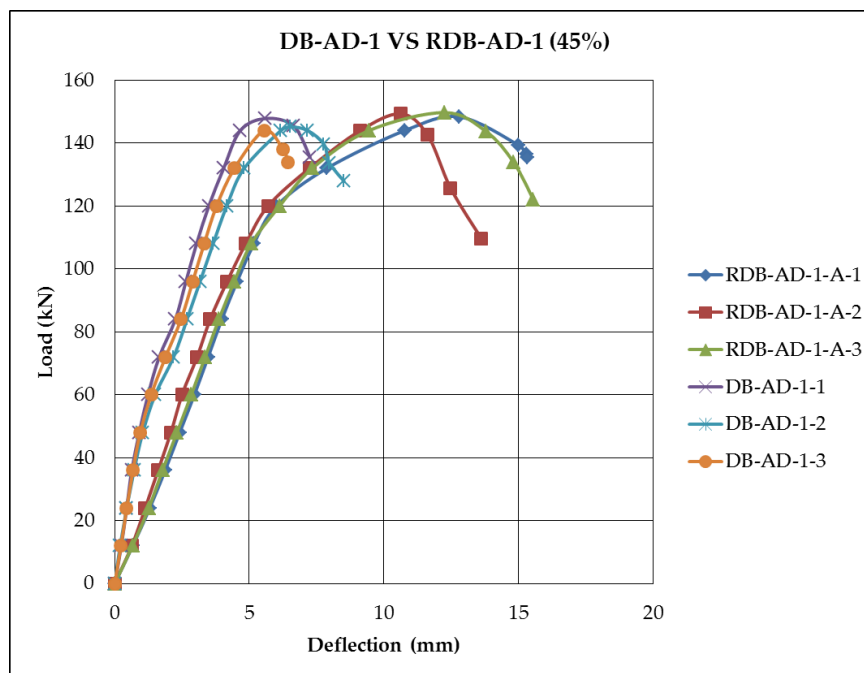


Fig. 5.1: Load-Deflection Plot of Control and Strengthened Shear Designed Beams (45% initially loaded) Tested at a/d ratio 1

Table 5.2: 75% Pre-Damaged Strengthened Shear Designed Beams (RDB75-ad-1) at a/d Ratio 1.0

S. No.	Beam 1				Beam 2				Beam 3			
	Load (kN)	Deflection (mm) at			Load (kN)	Deflection (mm) at			Load (kN)	Deflection (mm) at		
		A	B	C		A	B	C		A	B	C
1.	0	0	0	0	0	0	0	0	0	0	0	0
2.	12	0.384	0.758	0.096	12	0.416	0.524	0.146	12	0.392	0.655	0.086
3.	24	0.868	1.724	0.34	24	0.858	1.402	0.362	24	0.859	1.679	0.368
4.	36	1.28	2.525	0.546	36	1.288	2.194	0.538	36	1.312	2.002	0.55
5.	48	1.868	3.514	0.802	48	1.87	3.054	0.744	48	1.963	3.416	0.792
6.	60	2.384	4.324	1.024	60	2.37	3.866	0.938	60	2.458	4.118	1
7.	72	2.916	5.194	1.248	72	2.884	4.716	1.128	72	3.012	4.996	1.135
8.	84	3.436	5.916	1.43	84	3.388	5.544	1.318	84	3.578	5.928	1.44
9.	96	3.988	6.764	1.582	96	3.92	6.428	1.5	96	4.02	6.75	1.596
10.	108	4.562	7.614	1.774	108	4.732	7.432	1.724	108	4.698	7.532	1.794
11.	120	5.16	8.612	1.872	120	5.272	8.326	1.936	120	5.468	8.511	1.95
12.	132	6.542	10.31	2.178	132	5.944	9.256	2.16	132	6.578	10.12	2.472
13.	144	8.75	12.892	2.47	144	7.37	10.678	2.418	144	8.345	12.821	2.647
14.	156	12.18	15.956	2.894	152.06	9.9	11.98	2.738	150.76	10.06	13.98	2.96
15.	156.27	14.472	17.818	3.678	153.33	12.95	13.963	3.413	147	11.326	14.524	3.235
16.	137.49	15.178	18.2	3.876	150	13.5	15.011	3.587	139	13.942	16.738	3.667
17.	119.61	15.734	18.248	4.108	140	15.92	17.92	3.982				

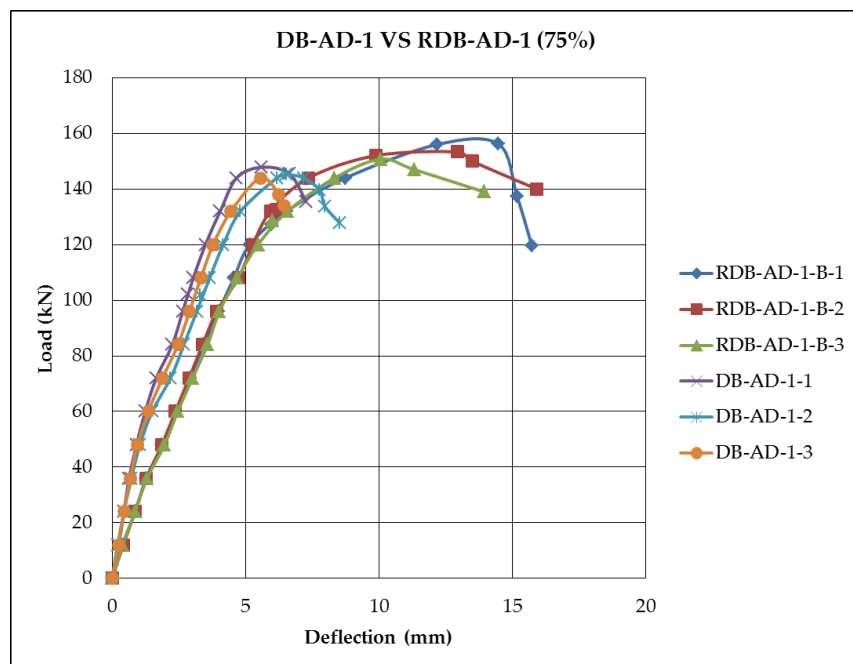


Fig. 5.2: Load-Deflection Plot of Control and Strengthened Shear Designed Beams (75% initially loaded) Tested at a/d Ratio 1

Table 5.3: 95% Pre-Damaged Strengthened Shear Designed Beams (RDB95-ad-1) at a/d Ratio 1.0

S. No.	Beam 1				Beam 2				Beam 3			
	Load (kN)	Deflection (mm) at			Load (kN)	Deflection (mm) at			Load (kN)	Deflection (mm) at		
		A	B	C		A	B	C		A	B	C
1.	0	0	0	0	0	0	0	0	0	0	0	0
2.	12	0.43	1.17	-0.17	12	1.099	1.12	-0.34	12	0.68	1.08	-0.177
3.	24	0.88	2.354	0.118	24	1.775	2.794	-0.10	24	1.342	2.454	0.112
4.	36	1.582	3.364	0.368	36	2.413	4.09	-0.31	36	1.787	3.576	0.462
5.	48	2.128	4.262	0.628	48	3.015	5.256	-0.08	48	2.445	4.678	0.667
6.	60	2.736	5.186	0.924	60	3.662	6.332	0.02	60	3.012	5.08	0.976
7.	72	3.26	6.146	1.244	72	4.117	7.506	0.316	72	3.778	6.118	1.34
8.	84	3.806	7.008	1.644	84	4.762	8.662	0.688	84	4.325	7.05	1.997
9.	96	4.344	8.096	2.052	96	5.212	9.676	1.59	96	4.789	8.324	2.361
10.	108	4.898	8.926	2.412	108	5.778	10.586	2.826	108	5.328	9.267	3.06
11.	120	5.388	9.78	2.646	120	6.247	12.698	3.064	120	5.84	10.736	4.574
12.	132	5.888	10.712	2.9	132	7.012	13.83	4.716	132	6.358	11.22	4.821
13.	144	6.966	12.246	3.298	144	8.98	14.52	5.122	144	7.425	11.952	5.607
14.	156	8.78	15.094	3.97	156	12.872	17.022	6.058	156	11.95	14.74	5.902
15.	162	12.694	16.326	4.252	162	14.322	19.58	6.882	159.92	13.463	15.48	6.342
16.	165.22	15.33	18.338	4.64	164.45	15.854	20.88	7.143	156	15	17.782	6.708
17.	158	17.474	20.02	4.97	158	17.5	21.057	7.528	153	15.92	18.328	7.114
18.	152	18.242	20.426	5.012	155	17.8	22.82	7.847				

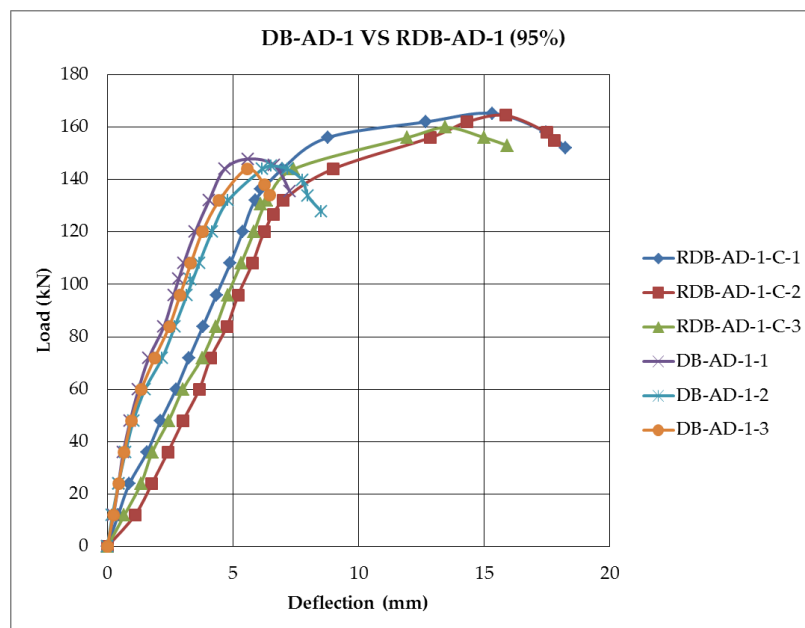


Fig. 5.3: Load-Deflection Plot of Control and Strengthened Shear Designed Beams (95% initially loaded) Tested at a/d Ratio 1

Table 5.4: 45% Pre-Damaged Strengthened Shear Designed Beams (RDB45-ad-3) at a/d Ratio 3.0

S. No.	Beam 1				Beam 2				Beam 3			
	Load (kN)	Deflection (mm) at			Load (kN)	Deflection (mm) at			Load (kN)	Deflection (mm) at		
		A	B	C		A	B	C		A	B	C
1.	0	0	0	0	0	0	0	0	0	0	0	0
2.	6	0.602	0.868	0.552	6	0.474	0.66	0.406	6	0.554	0.779	0.502
3.	12	1.214	1.682	1.044	12	1.144	1.58	1.05	12	1.185	1.623	0.966
4.	18	1.918	2.652	1.588	18	1.862	2.518	1.676	18	1.887	2.584	1.684
5.	24	2.766	3.674	2.112	24	2.672	3.528	2.258	24	2.741	3.684	2.367
6.	30	3.26	4.272	2.472	30	3.29	4.304	2.705	30	3.3	4.292	2.556
7.	36	4.216	5.353	3.248	36	3.728	4.836	3.066	36	3.875	5.162	3.132
8.	42	5.274	6.646	4.038	42	4.818	6.104	3.868	42	4.578	6.485	3.972
9.	48	6.162	7.634	4.742	48	5.686	7.074	4.42	48	5.788	7.421	4.568
10.	54	7.104	8.784	5.458	54	6.614	8.212	5.054	54	6.972	8.596	5.224
11.	60	8.034	9.806	6.058	60	7.702	9.386	5.708	60	7.882	9.742	6.008
12.	66	9.122	11.002	6.756	66	8.875	12.034	7.08	66	9.155	11.854	6.847
13.	70.2	10.368	13.504	8.03	68.28	9.48	13.854	7.992	69.32	10.43	13.252	7.412
14.	73.35	12.39	16.858	9.606	70.23	10.346	16.328	9.176	71.68	12.287	15.972	8.712
15.	69.73	14.689	17.008	9.684	68.22	13.828	16.65	9.27	67	15.63	17.646	9.216
16.	67.17	15.312	19.137	10.104	61.77	15.712	18.675	9.668	62.37	16.622	19.892	10.218

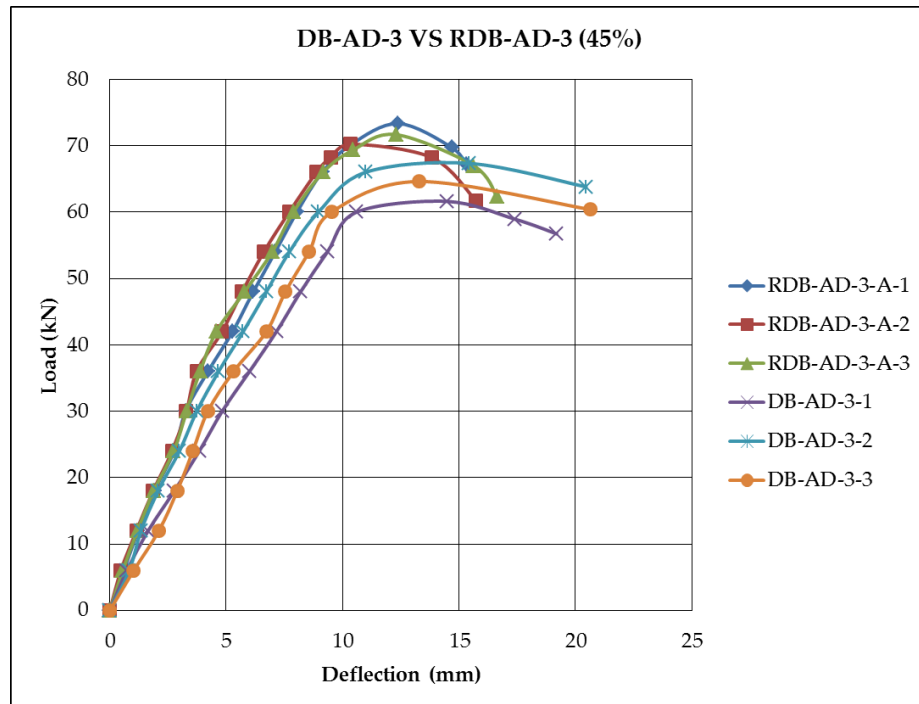


Fig. 5.4: Load-Deflection Plot of Control and Strengthened Shear Designed Beams (45% initially loaded) Tested at a/d Ratio 3

Table 5.5: 75% Pre-Damaged Strengthened Shear Designed Beams (RDB75-ad-3) at a/d Ratio 3.0

S. No.	Beam 1				Beam 2				Beam 3			
	Load (kN)	Deflection (mm) at			Load (kN)	Deflection (mm) at			Load (kN)	Deflection (mm) at		
		A	B	C		A	B	C		A	B	C
1.	0	0	0	0	0	0	0	0	0	0	0	0
2.	6	1.992	0.95	-0.41	6	0.42	0.516	0.288	6	1.574	1.324	0.444
3.	12	2.626	1.84	0.092	12	1.242	1.324	0.674	12	2.2	2.146	0.982
4.	18	3.338	2.794	0.774	18	2.036	2.38	1.41	18	3.07	3.25	1.688
5.	24	4.37	3.818	1.456	24	2.802	3.388	2.076	24	3.912	4.234	2.312
6.	30	5.058	4.584	1.95	30	3.562	4.302	2.544	30	4.484	4.828	2.644
7.	36	5.702	5.372	2.408	36	4.532	5.382	3.224	36	5.16	5.55	3.11
8.	42	6.792	6.628	3.136	42	5.366	6.356	3.818	42	6.34	6.922	3.926
9.	48	7.552	7.526	3.75	48	6.204	7.358	4.458	48	7.228	7.929	4.514
10.	54	8.494	8.552	4.312	54	7.12	8.426	5.09	54	8.138	9.006	5.114
11.	60	9.504	9.576	4.98	60	8.064	9.484	5.816	60	9.136	10.12	5.86
12.	66	10.654	10.85	5.72	66	9.594	10.654	6.364	66	11.318	12.324	7.004
13.	67.74	14.69	14.116	7.262	66.96	10.422	11.926	7.09	68.52	13.984	14.914	8.342
14.	69.24	18.654	17.95	9.116	69.48	12.796	14.166	8.208	69.9	17.838	17.928	9.8
15.	64.38	18.896	18.106	9.202	71.52	16.11	17.19	9.716	68.25	17.894	17.944	9.802
16.					67.77	16.318	17.332	9.678	66.78	17.894	17.944	9.802

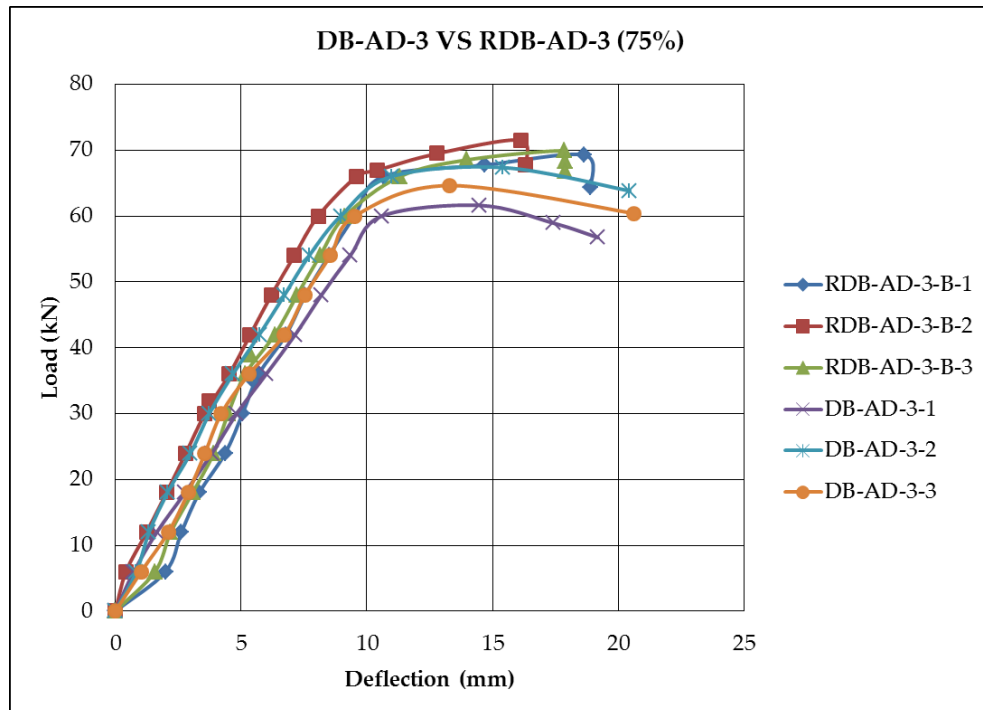


Fig. 5.5: Load-Deflection Plot of Control and Strengthened Shear Designed Beams (75% initially loaded) Tested at a/d Ratio 3

Table 5.6: 95% Pre-Damaged Strengthened Shear Designed Beams (RDB95-ad-3) at a/d Ratio 3.0

S. No.	Beam 1				Beam 2				Beam 3			
	Load (kN)	Deflection (mm) at			Load (kN)	Deflection (mm) at			Load (kN)	Deflection (mm) at		
		A	B	C		A	B	C		A	B	C
1.	0	0	0	0	0	0	0	0	0	0	0	0
2.	6	1.21	0.292	0.154	6	0.506	0.682	0.448	6	0.448	0.528	0.578
3.	12	2.928	1.952	0.986	12	1.42	1.838	1.22	12	1.325	1.784	1.364
4.	18	3.704	2.922	1.638	18	2.282	2.962	1.958	18	2.128	2.996	1.847
5.	24	4.65	4.008	2.304	24	3.316	4.218	2.702	24	3.306	3.785	2.714
6.	30	5.576	4.966	2.9	30	4.038	5.078	3.214	30	4.108	4.963	3.118
7.	36	6.478	5.948	3.53	36	4.612	5.73	3.648	36	4.559	5.847	3.627
8.	42	7.41	7.02	4.152	42	5.83	7.146	4.496	42	5.867	6.972	4.489
9.	48	8.182	7.902	4.71	48	6.738	8.138	5.102	48	6.845	8.018	5.007
10.	54	9	8.918	5.344	54	7.726	9.234	5.71	54	7.718	9.118	5.831
11.	60	9.812	9.866	5.87	60	8.616	10.236	6.308	60	8.621	10.363	6.423
12.	66	10.822	10.966	6.528	66	11.692	13.278	7.85	66	9.425	11.358	6.971
13.	67.29	11.782	11.692	6.946	69.81	16.638	17.868	10.002	68.08	10.358	11.894	7.27
14.	69.36	17.83	16.434	9.136	66.66	17.216	18.28	10.206	69.37	11.548	17.037	9.568
15.	70.8	19.698	17.796	9.316					70.12	17.326	18.114	10.86
16.	63.93	20.056	18.02	10.21					67.35	18.82	19.764	11.008

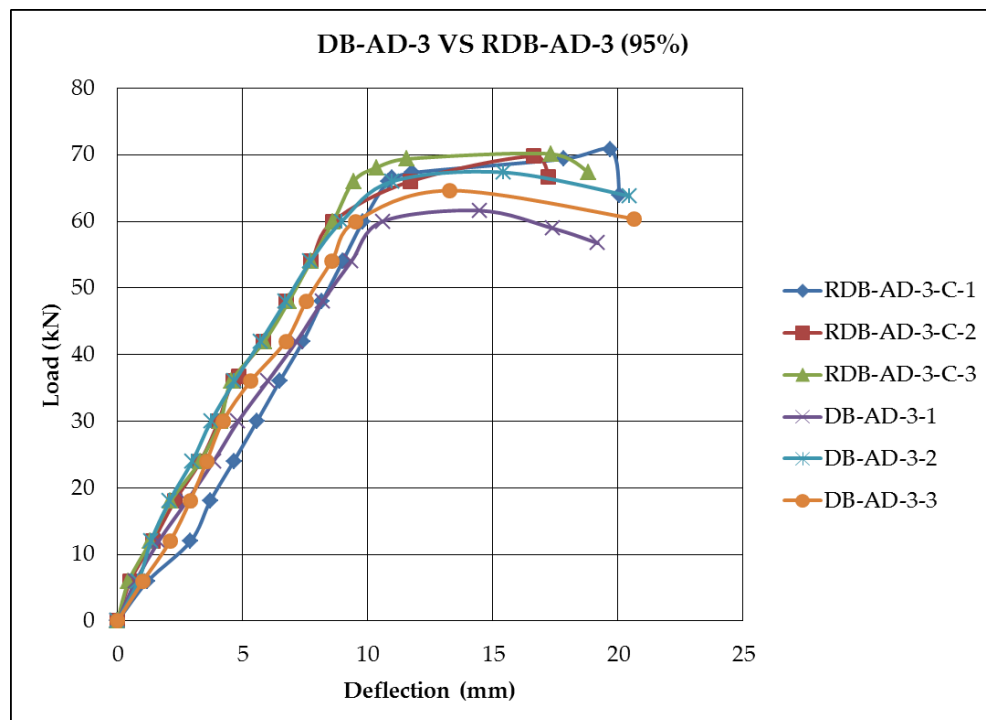


Fig. 5.6: Load-Deflection Plot of Control and Strengthened Shear Designed Beams (95% initially loaded) Tested at a/d Ratio 3

5.3 COMPARISONS BETWEEN CONTROL AND STRENGTHENED SHEAR DESIGNED (RDB) BEAMS

5.3.1 Effect of Different Levels of Initial Stresses

All the strengthened beams which were initially damaged with 45%, 75% and 95% of the ultimate load have shown a complete strength restoration and also a further enhancement of the original strength up to 12.03% after strengthening. The deflection behaviour of these beams is also observed to change after strengthening, and the beams exhibited more ductility as compared to control beams. Hence, higher deflection values were observed for all the strengthened beams at the ultimate failure level.

The strengthened shear designed beams when tested at $a/d = 1.0$ and having the initial stress level of 95% (RDB95-ad-1) have shown maximum improvement in their ultimate load value, by 12.03%. The results of strengthened beams with initial damage of 45% (RDB45-ad-1) and 75% (RDB75-ad-1) also show a similar trend of improvement in their ultimate load values by 2.38% and 5.33%, respectively, as compared to control beams DB-ad-1. Similarly, the deflection behaviour of strengthened beams was also improved and RDB45-ad-1, RDB75-ad-1 and RDB95-ad-1 beams shows increase in deflection values at ultimate load level by 101.58%, 111.80% and 152.28%, respectively, as compared to control beams DB-ad-1. The observed figures of load values were quite less for the beams tested at $a/d = 3.0$, but these beams also did show a similarly improved trend after strengthening. The strengthened RDB45-ad-3, RDB75-ad-3, and RDB-95-ad-3 beams tested at $a/d = 3.0$, demonstrated an improvement in ultimate load by 11.19%, 8.82%, and 8.85%, respectively w.r.t. control beams DB-ad-3. The deflection of RDB45-ad-3, RDB75-ad-3, and RDB-95-ad-3 beams changed by -18.88%, 21.82% and 24.28%, respectively, as compared to control beams DB-ad-3. Load-deflection comparative curves of control and strengthened shear designed beams with different levels of initial damages, when tested at $a/d = 1$ & 3, are detailed in Table 5.7 & 5.8 and shown in Figure 5.7 & 5.8.

Table 5.7: Shear Designed Beams at a/d Ratio 1.0

S. No.	Control Beams (DB-ad-1)		45% pre-damaged strengthened beams (RDB45-ad-1)		75% pre-damaged strengthened beams (RDB75-ad-1)		95% pre-damaged strengthened beams (RDB95-ad-1)	
	Load (kN)	Def. (mm)	Load (kN)	Def. (mm)	Load (kN)	Def. (mm)	Load (kN)	Def. (mm)
1.	0	0.000	0	0.000	0	0.000	0	0.000
2.	12	0.227	12	0.659	12	0.397	12	0.593
3.	24	0.445	24	1.236	24	0.862	24	1.332
4.	36	0.678	36	1.755	36	1.293	36	1.927
5.	48	0.969	48	2.274	48	1.900	48	2.529
6.	60	1.367	60	2.770	60	2.404	60	3.137
7.	72	1.911	72	3.299	72	2.937	72	3.718
8.	84	2.471	84	3.801	84	3.467	84	4.298
9.	96	2.906	96	4.390	96	3.976	96	4.782
10.	108	3.341	108	5.037	108	4.664	108	5.335
11.	120	3.815	120	5.944	120	5.300	120	5.825
12.	132	4.439	132	7.489	132	6.355	132	6.419
13.	144	5.468	144	9.783	144	8.155	144	7.790
14.	145.68	5.899	149.14	11.891	151.41	9.980	156	11.201
15.	144.5	6.461	141.94	13.474	153.45	12.494	162	13.508
16.	137.71	7.097	132.02	14.207	144.83	13.335	163.2	14.882
17.	133.87	7.217	122.38	14.843	132.87	15.199	157.33	16.658
19.	127.86	8.526					153.33	17.321

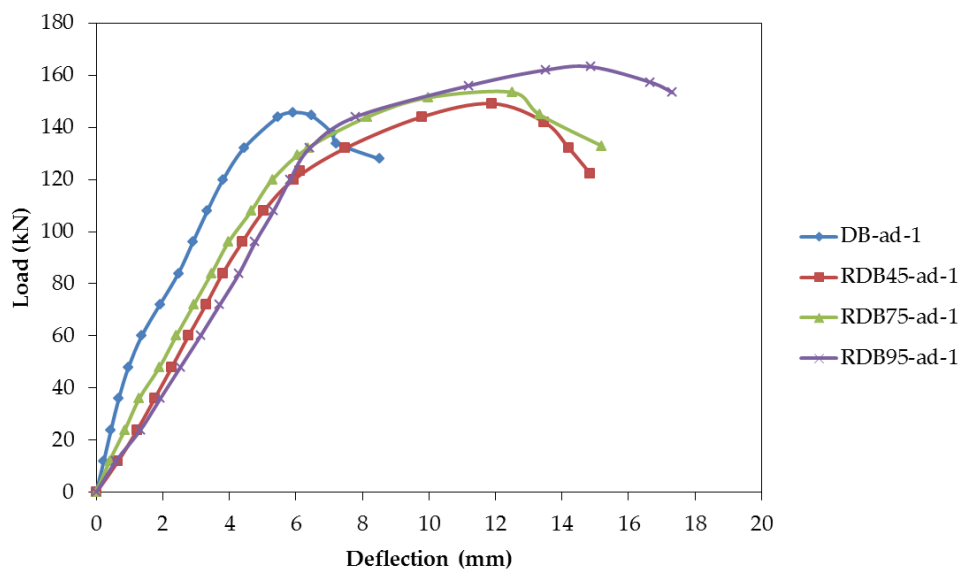


Fig. 5.7: Load-Deflection Plot of Shear Designed Beams Tested at a/d Ratio 1

Table 5.8: Shear Designed Beams at a/d Ratio 3.0

S. No.	Control Beams (RDB-ad-3)		45% pre-damaged strengthened beams (RDB45-ad-3)		75% pre-damaged strengthened beams (RDB75-ad-3)		95% pre-damaged strengthened beams (RDB95-ad-3)	
	Load (kN)	Def. (mm)	Load (kN)	Def. (mm)	Load (kN)	Def. (mm)	Load (kN)	Def. (mm)
1.	0	0.000	0	0.000	0	0.000	0	0.000
2.	6	0.862	6	0.543	6	1.329	6	0.721
3.	12	1.699	12	1.181	12	2.023	12	1.891
4.	18	2.579	18	1.889	18	2.815	18	2.705
5.	24	3.466	24	2.726	24	3.695	24	3.757
6.	30	4.257	30	3.283	30	4.368	30	4.574
7.	36	5.329	36	3.940	36	5.131	36	5.216
8.	42	6.537	42	4.890	42	6.166	42	6.369
9.	48	7.487	48	5.879	48	6.995	48	7.255
10.	54	8.537	54	6.897	54	7.917	54	8.148
11.	60	9.693	60	7.873	60	8.901	60	9.016
12.	64.53	14.393	66	9.051	66	10.522	66	10.646
13.	61.04	19.499	69.27	10.093	67.74	13.032	67.69	11.070
14.			71.75	11.674	69.54	16.429	69.37	14.689
15.			68.32	14.716	70.22	17.534	70.24	17.887
16.			63.77	15.882	66.8	17.703	65.98	18.697

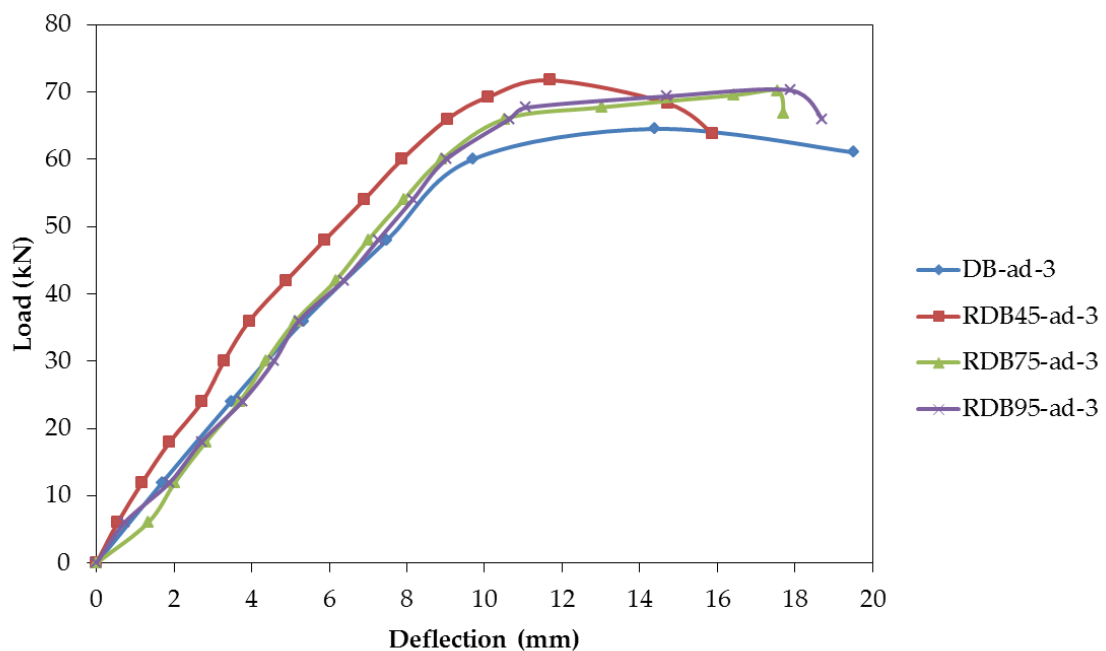


Fig. 5.8: Load-Deflection Plot of Shear Designed Beams Tested at a/d Ratio 3

5.3.2 Effect of Shear Span-to-Depth (a/d) Ratios

The control as well as strengthened shear designed (DB) and shear deficient (SDB) beams were tested for two different a/d ratios 1 & 3 to study the effect of shear span 'a' on the performance of these beams. From the test results, it was found that a/d ratio played a crucial role. The strengthened beams tested at a/d = 1.0, have shown more improvement in their ultimate strength, cracking pattern, ductility, and deflection behaviour as compared to beams which were tested at a/d = 3.0, irrespective of stirrups spacing.

The control shear designed beams tested at a/d = 1.0 (DB-ad-1) showed a higher load carrying capacity of 125.76% as compared to DB-ad-3 beams tested at a/d = 3. The observed pattern for strengthened beams was also similar, and the shear load carrying capacity of RDB45-ad-1, RDB75-ad-1, and RDB95-ad-1 beams increased by 107.86%, 118.53%, and 132.35%, respectively, when compared with RDB45-ad-3, RDB75-ad-3, and RDB95-ad-3 beams. Graphical comparisons of ultimate loads at different a/d ratios are shown in Figure 5.9.

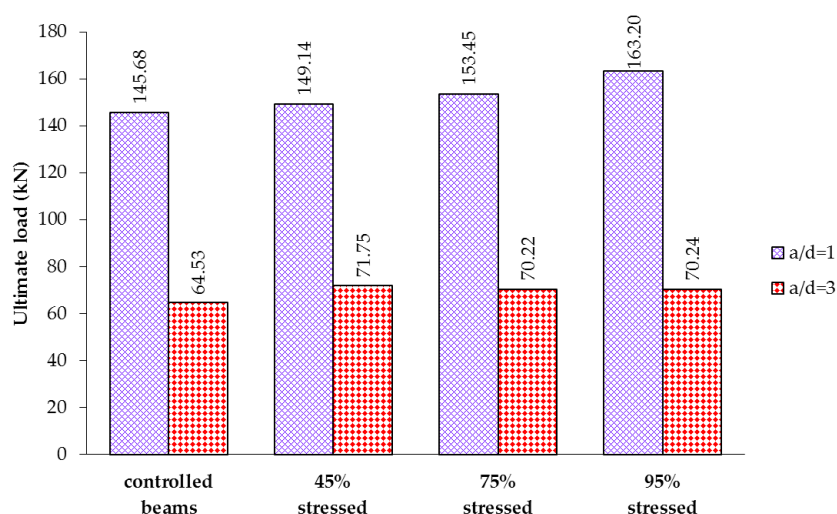


Fig. 5.9: Comparison of Ultimate Loads of Shear Designed Beams Tested at a/d Ratio 1 & 3

5.4 TEST RESULTS OF STRENGTHENED SHEAR DEFICIENT (RSDB) RCC BEAMS

Pre-damaged PMF strengthened shear deficient (RSDB) beams were tested for two different a/d ratios as 1.0 and 3.0 as control beams. A typical set of 3 beams was used to evaluate the behaviour of strengthened beams. The test setup along with locations of LVTDs has been detailed in Chapter 4. The deflection at all the three points corresponding to load values has been given in Table 5.9 to 5.14 for beams tested at a/d ratio 1 and 3. For comparison purpose, the load-deflection curve corresponding to 1st LVDT is only discussed

in subsequent sections. Figure 5.10 to 5.15 shows the Load-Deflection plot of control and strengthened shear deficient beams initially loaded for different stress levels and tested at a/d ratio 1 and 3.

Table 5.9: 45% Pre-Damaged Strengthened Shear Deficient Beams (RSDB45-ad-1) at a/d Ratio 1.0

S. No.	Beam 1				Beam 2				Beam 3			
	Load (kN)	Deflection (mm) at			Load (kN)	Deflection (mm) at			Load (kN)	Deflection (mm) at		
		A	B	C		A	B	C		A	B	C
1.	0	0	0	0	0	0	0	0	0	0	0	0
2.	12	0.462	0.792	0.168	12	0.53	0.824	0.122	12	0.864	1.136	0.09
3.	24	0.96	1.604	0.4	24	1.05	1.766	0.368	24	1.378	2.068	0.146
4.	36	1.344	2.262	0.584	36	1.414	2.382	0.546	36	1.818	2.928	0.234
5.	48	1.85	3.034	0.72	48	1.914	3.294	0.792	48	2.244	3.696	0.246
6.	60	2.326	3.834	0.938	60	2.416	4.05	1.03	60	2.678	4.478	0.244
7.	72	2.84	4.722	1.178	72	2.95	4.97	1.254	72	3.2	5.296	0.258
8.	84	3.398	5.598	1.418	84	3.714	6.032	1.52	84	3.702	6.304	0.296
9.	96	3.972	6.534	1.66	96	4.548	7.292	1.714	96	4.538	7.634	0.3
10.	108	4.784	7.62	1.858	108	5.208	8.342	1.97	108	5.35	8.85	1.03
11.	120	5.468	8.67	2.124	120	5.904	9.416	2.224	120	6.066	9.94	1.03
12.	132	6.22	9.758	2.402	132	8.32	11.606	2.606	132	8.738	12.482	2.062
13.	144	7.792	11.612	2.788	141.69	13.558	15.762	3.11	144	11.368	14.254	2.687
14.	150	10.81	14.078	3.158	138.33	14.22	16.192	3.198	150	13.035	15.375	2.985
15.	156	12.038	15.142	3.338					151.02	14.623	16.72	3.442
16.	160.05	13.964	16.702	3.602					147	15.722	17.982	3.875
17.	156.48	14.93	17.294	3.716					144	16.072	19.012	4.246
18.	151.98	15.272	17.368	3.732								

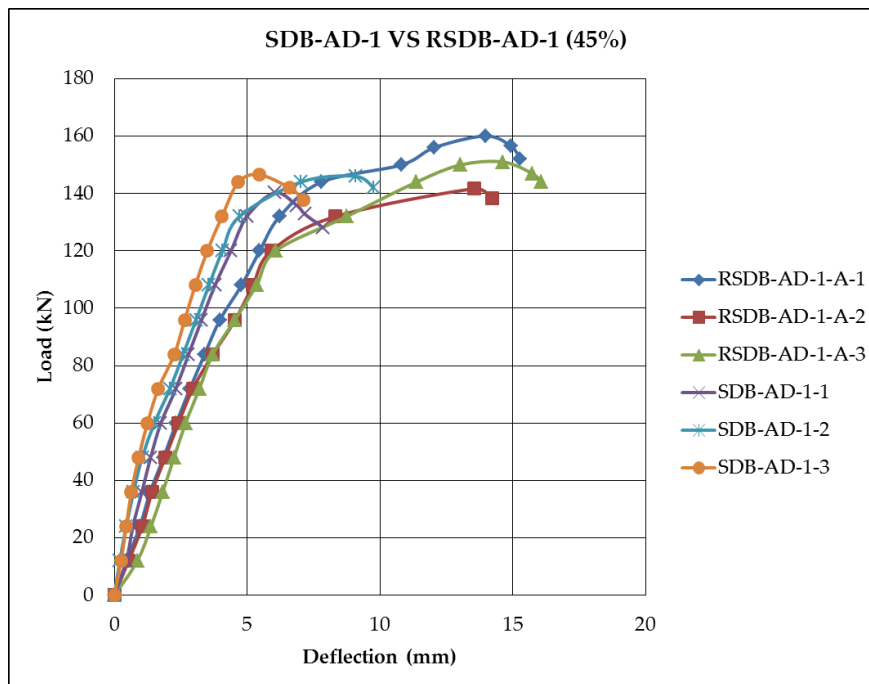


Fig. 5.10: Load-Deflection Plot of Control and Strengthened Shear Deficient Beams (45% initially loaded) Tested at a/d Ratio 1

Table 5.10: 75% Pre-Damaged Strengthened Shear Deficient Beams (RSDB75-ad-1) at a/d Ratio 1.0

S. No.	Beam 1				Beam 2				Beam 3			
	Load (kN)	Deflection (mm) at			Load (kN)	Deflection (mm) at			Load (kN)	Deflection (mm) at		
		A	B	C		A	B	C		A	B	C
1.	0	0	0	0	0	0	0	0	0	0	0	0
2.	12	0.662	1.06	0.284	12	0.47	0.782	0.11	12	0.472	0.88	0.222
3.	24	1.458	2.19	0.63	24	0.992	1.672	0.25	24	0.984	1.912	0.558
4.	36	1.964	3.094	0.822	36	1.442	2.464	0.44	36	1.45	2.802	0.846
5.	48	2.51	4.046	1.03	48	1.976	3.204	0.578	48	2.042	3.712	1.162
6.	60	3.012	4.932	1.23	60	2.522	4.066	0.678	60	2.622	4.738	1.412
7.	72	3.458	5.792	1.4	72	2.98	4.86	0.8	72	3.19	5.562	1.564
8.	84	3.916	6.692	1.568	84	3.44	5.532	1.048	84	3.714	6.4	1.75
9.	96	4.38	7.554	1.836	96	3.892	6.368	1.8	96	4.24	7.234	1.98
10.	108	4.84	8.394	2.04	108	4.384	7.12	1.27	108	4.806	8.198	2.196
11.	120	5.308	9.232	2.18	120	4.898	8.034	1.414	120	5.348	9.116	2.438
12.	132	5.814	10.13	2.37	132	5.412	8.8	1.538	132	6.144	10.306	2.712
13.	144	6.43	11.152	2.568	144	6.018	9.896	1.952	144	8.91	13.102	3.272
14.	150	7.972	12.568	2.752	150	7.568	10.069	2.048	146.19	9.98	13.942	3.456
15.	156	9.62	14.316	2.956	155.22	9.289	12.863	2.179	144.63	11.348	14.818	3.658
16.	162	10.91	15.718	3.188	150.31	11.362	16.272	3.011	136.86	12.508	15.294	3.714
17.	164.19	12.08	16.77	3.366								
18.	160.56	14.246	18.196	3.68								

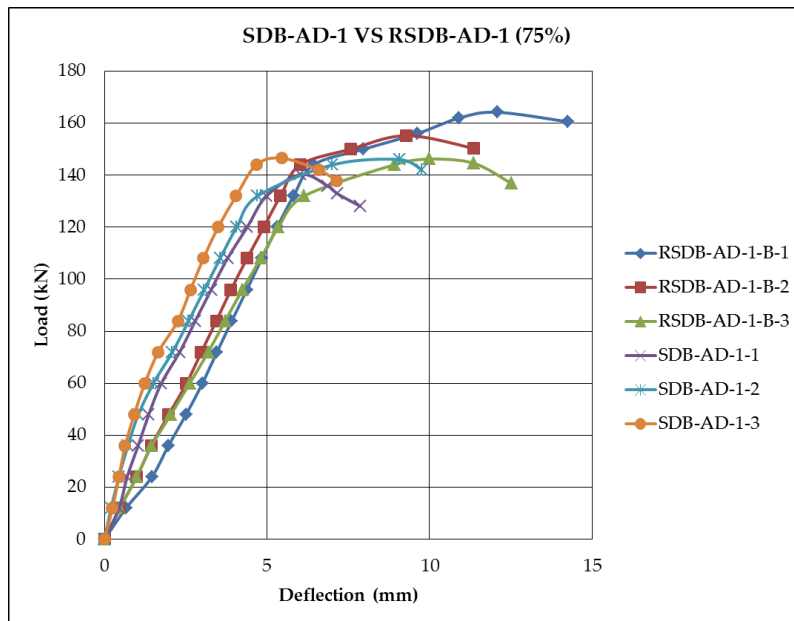


Fig. 5.11: Load-Deflection Plot of Control and Strengthened Shear Deficient Beams (75% initially loaded) Tested at a/d Ratio 1

Table 5.11: 95% Pre-Damaged Strengthened Shear Deficient Beams (RSDB95-ad-1) at a/d Ratio 1.0

S. No.	Beam 1				Beam 2				Beam 3			
	Load (kN)	Deflection (mm) at			Load (kN)	Deflection (mm) at			Load (kN)	Deflection (mm) at		
		A	B	C		A	B	C		A	B	C
1.	0	0	0	0	0	0	0	0	0	0	0	0
2.	12	0.574	1.232	0.114	12	0.436	0.754	0.188	12	0.766	1.168	-0.026
3.	24	1.034	2.208	0.494	24	1.146	1.828	0.42	24	1.338	2.106	0.48
4.	36	1.524	3.23	0.806	36	1.674	2.648	0.608	36	2.044	3.162	0.738
5.	48	2.096	4.144	1.106	48	2.31	3.596	0.814	48	2.682	4.028	0.84
6.	60	2.584	5.058	1.352	60	2.96	4.496	0.942	60	3.246	4.976	1.042
7.	72	3.094	5.888	1.536	72	3.528	5.446	1.124	72	3.832	5.946	1.23
8.	84	3.606	6.834	1.792	84	4.14	6.428	1.332	84	4.536	6.966	0.43
9.	96	4.13	7.712	1.96	96	4.987	7.29	0.526	96	5.376	7.72	0.684
10.	108	4.626	8.502	2.194	108	5.734	8.178	0.858	108	6.282	8.556	1.022
11.	120	5.16	9.26	2.284	120	6.582	8.92	1.19	120	7.215	9.328	1.325
12.	132	5.72	10.1	2.544	132	7.388	9.956	1.518	132	8.194	9.874	1.587
13.	144	6.43	11.01	2.79	144	8.662	11.066	1.788	144	9.115	12.226	2.895
14.	156	7.852	12.254	3.114	148.95	10.924	12.894	2.134	153.42	11.022	14.02	3.012
15.	157.68	9.29	13.662	3.502	147.72	11.308	13.146	2.238	150.14	14.41	16.12	3.13
16.	140.88	9.998	13.908	3.608	144.48	11.802	13.314	2.316	147.63	15.394	16.514	3.162
17.	131.64	10.438	13.88	3.168	139.62	14.37	15.108	2.69				

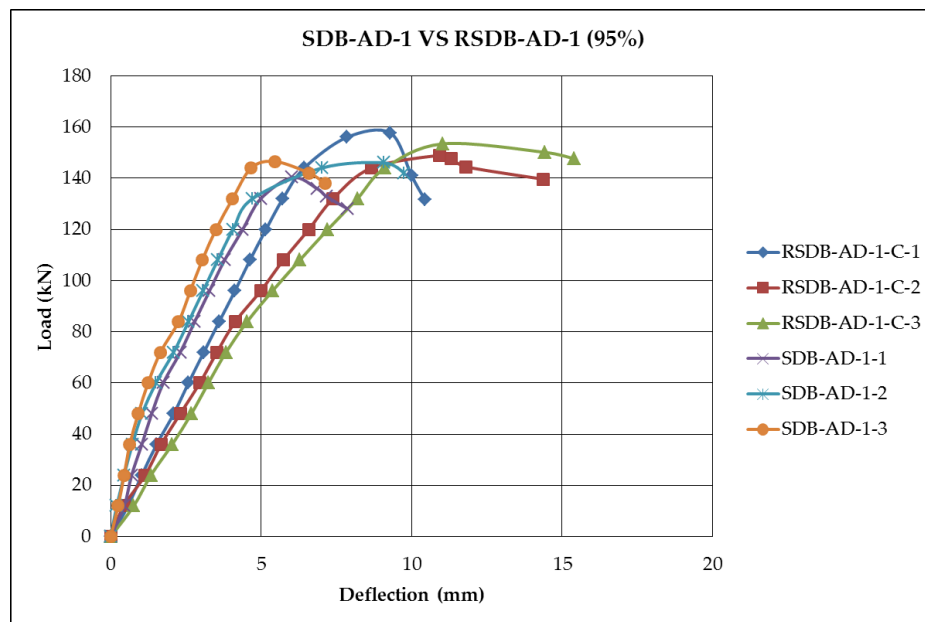


Fig. 5.12: Load-Deflection Plot of Control and Strengthened Shear Deficient Beams (95% initially loaded) Tested at a/d Ratio 1

Table 5.12: 45% Pre-Damaged Strengthened Shear Deficient Beams (RSDB45-ad-3) at a/d Ratio 3.0

S. No.	Beam 1				Beam 2				Beam 3			
	Load (kN)	Deflection (mm) at			Load (kN)	Deflection (mm) at			Load (kN)	Deflection (mm) at		
		A	B	C		A	B	C		A	B	C
1.	0	0	0	0	0	0	0	0	0	0	0	0
2.	6	0.972	1.25	0.689	6	0.846	0.952	0.458	6	1.188	1.148	0.514
3.	12	2.014	2.526	1.025	12	1.32	1.68	0.932	12	1.952	1.794	0.94
4.	18	3.17	3.01	1.482	18	2.022	2.676	1.544	18	2.774	2.91	1.72
5.	24	4.132	3.902	2.132	24	2.712	3.598	2.116	24	3.638	3.938	2.462
6.	30	5.082	4.346	4.432	30	3.218	4.126	2.512	30	4.322	4.66	2.8
7.	36	6.256	5.704	3.358	36	4.184	5.274	3.246	36	5.292	5.824	3.492
8.	42	7.31	6.872	3.972	42	5.182	6.39	3.878	42	6.484	7.176	4.274
9.	48	8.378	8.104	4.616	48	6.006	7.346	4.488	48	7.57	8.38	5.11
10.	54	9.492	9.33	5.408	54	7.01	8.452	5.196	54	8.618	9.55	5.738
11.	60	10.588	9.858	5.568	60	8.022	9.506	5.882	60	9.75	10.784	6.448
12.	66	12.332	13.922	6.608	66	10.102	10.588	6.356	66	11.42	12.43	7.422
13.	66.49	14.172	15.292	7.894	67.76	13.556	14.134	7.262	67.34	14.71	15.562	8.928
14.	65.63	16.321	17.342	8.972	67.72	15.454	16.574	9.36	65.85	15.544	16.112	9.19
15.	64.67	17.382	19.896	10.012	64.37	16.292	17.462	9.766	60.54	16.094	16.204	9.176

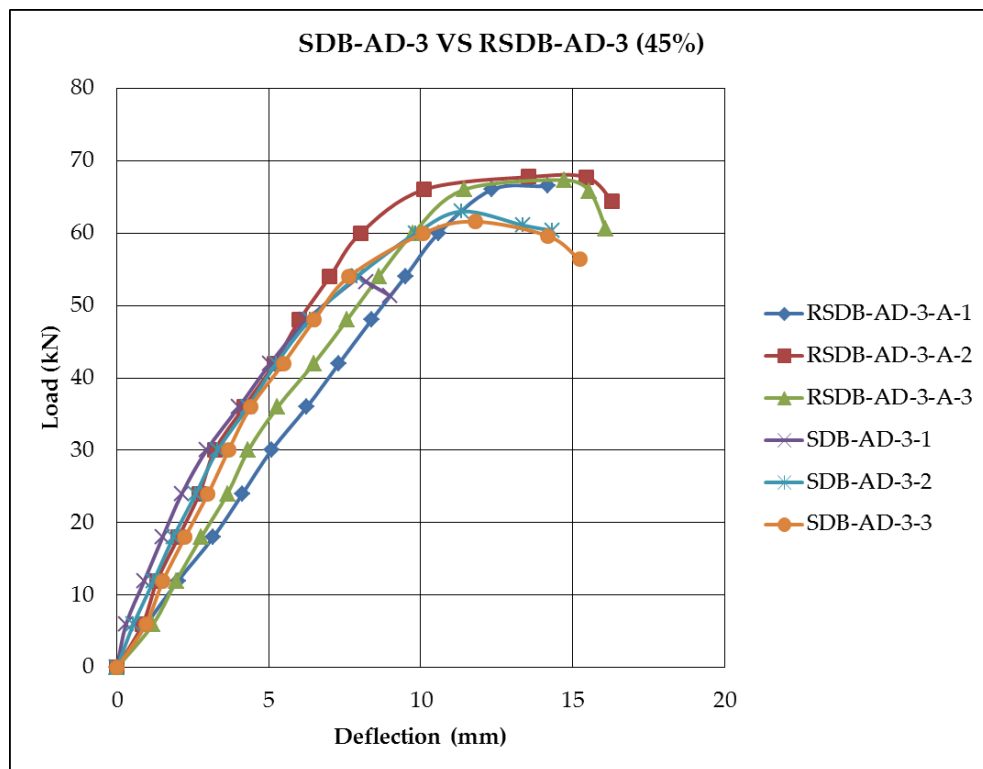


Fig. 5.13: Load-Deflection Plot of Control and Strengthened Shear Deficient Beams (45% initially loaded) Tested at a/d Ratio 3

Table 5.13: 75% Pre-Damaged Strengthened Shear Deficient Beams (RSDB75-ad-3) at a/d Ratio 3.0

S. No.	Beam 1				Beam 2				Beam 3			
	Load (kN)	Deflection (mm) at			Load (kN)	Deflection (mm) at			Load (kN)	Deflection (mm) at		
		A	B	C		A	B	C		A	B	C
1.	0	0	0	0	0	0	0	0	0	0	0	0
2.	6	1.356	1.156	0.502	6	0.532	0.784	0.498	6	0.96	1.414	0.91
3.	12	2.136	2.134	1.2	12	1.036	1.478	0.93	12	1.624	2.338	1.54
4.	18	3.058	3.29	1.914	18	2.04	2.654	1.684	18	2.454	3.368	2.136
5.	24	3.944	4.298	2.498	24	2.854	3.672	2.26	24	3.304	4.34	2.784
6.	30	4.436	4.858	2.94	30	3.46	4.322	2.69	30	4.148	4.746	2.972
7.	36	5.216	5.77	3.416	36	4.034	5.028	3.096	36	4.934	6.186	3.83
8.	42	6.146	6.926	4.106	42	5.34	6.504	4.032	42	5.798	7.12	4.38
9.	48	7.508	8.37	4.96	48	6.19	7.486	4.618	48	6.696	8.064	5.002
10.	54	8.64	9.548	5.638	54	7.19	8.684	5.244	54	7.814	9.2	5.628
11.	60	9.914	10.748	6.35	60	8.712	10.358	6.31	60	9.114	10.422	6.336
12.	63.3	10.492	11.712	6.804	63.99	15.538	16.206	8.986	61.8	9.412	10.916	6.598
13.	64.62	14.792	14.628	8.228	61.77	15.818	16.39	9.086	63.54	10.958	12.102	7.24
14.	63	15.022	14.63	8.224								

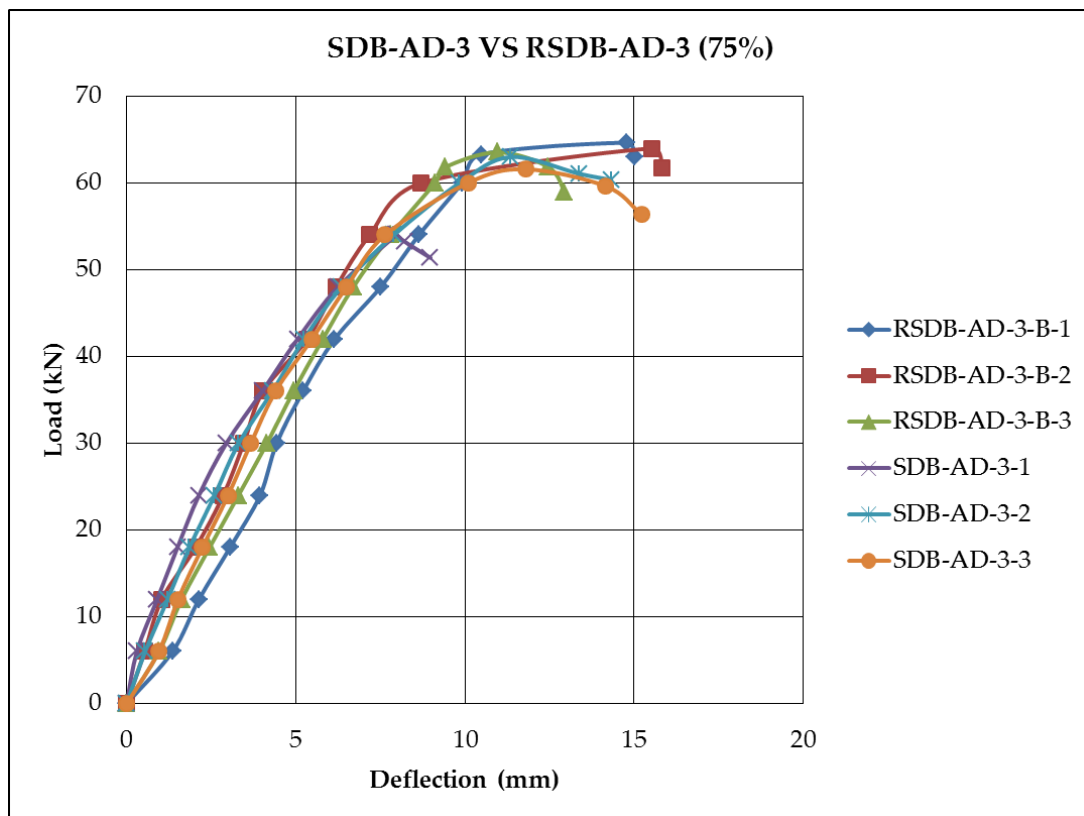


Fig. 5.14: Load-Deflection Plot of Control and Strengthened Shear Deficient Beams (75% initially loaded) Tested at a/d Ratio 3

Table 5.14: 95% Pre-Damaged Strengthened Shear Deficient Beams (RSDB95-ad-3) at a/d Ratio 3.0

S. No.	Beam 1				Beam 2				Beam 3			
	Load (kN)	Deflection (mm) at			Load (kN)	Deflection (mm) at			Load (kN)	Deflection (mm) at		
		A	B	C		A	B	C		A	B	C
1.	0	0	0	0	0	0	0	0	0	0	0	0
2.	6	1.14	1.188	0.646	6	1.376	1.122	0.358	6	0.86	1.278	0.742
3.	12	1.985	2.114	1.223	12	2.242	2.198	1.068	12	1.556	2.224	1.336
4.	18	2.891	3.346	1.854	18	3.156	3.326	1.778	18	2.5	3.368	1.97
5.	24	3.782	4.452	2.604	24	4.146	4.41	2.514	24	3.418	4.462	2.604
6.	30	4.415	4.986	2.882	30	4.778	5.044	2.844	30	3.982	4.86	2.858
7.	36	5.448	6.38	3.672	36	5.746	6.37	3.628	36	5.266	6.396	3.668
8.	42	6.253	7.401	4.276	42	6.712	7.314	4.282	42	6.142	7.422	4.298
9.	48	7.448	8.468	4.887	48	7.61	8.408	4.858	48	7.132	8.526	4.964
10.	54	8.547	9.658	5.549	54	8.718	9.642	5.578	54	8.156	9.696	5.528
11.	59.85	11.72	12.36	7.23	60	9.842	11.034	6.34	58.65	11.426	12.132	6.854
12.	54.22	13.462	14.672	8.432	66	11.804	12.998	7.434	52.77	11.572	12.236	6.904
13.					68.22	14.422	14.926	8.266	51.87	11.904	12.34	6.918
14.					66.78	16.494	16.52	9.012				
15.					65.7	16.618	15.9	8.958				

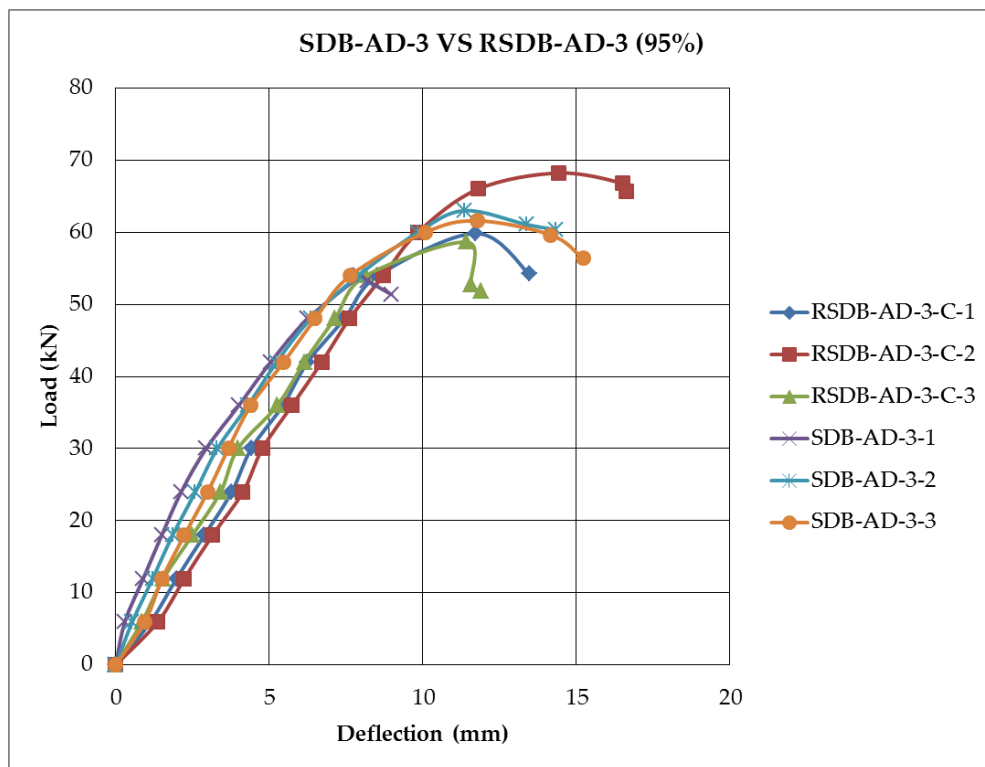


Fig. 5.15: Load-Deflection Plot of Control and Strengthened Shear Deficient Beams (95% initially loaded) Tested at a/d Ratio 3

5.5 COMPARISONS BETWEEN CONTROL AND STRENGTHENED SHEAR DEFICIENT (RSDB) BEAMS

5.5.1 Effect of Different Levels of Initial Stresses

The strengthened shear deficient beams having initial pre-damages of 45%, 75% and 95% were also tested for different a/d ratios. The strengthened RSDB75-ad-1 beams have shown maximum improvement in their ultimate load by 7.57% as compared to control beams SDB-ad-1. The RSDB45-ad-1 and RSDB95-ad-1 beams also showed an improvement in their ultimate load values by 4.60% & 6.29%, respectively, as compared to control beams SDB-ad-1. Similarly, the deflection behaviour of strengthened beams was also improved and RSDB45-ad-1, RSDB75-ad-1 and RSDB95-ad-1 beams show an increase in deflection values at ultimate load level by 104.87%, 52.40% and 51.84%, respectively, as compared to control beams SDB-ad-1. At a higher a/d ratio = 3.0, the ultimate loads also exhibited enhancement and the maximum improvement in load was recorded for RSDB45-ad-3 beams by 12.85% over the control beams SDB-ad-3. The load carrying capacities of RSDB75-ad-3 and RSDB 95-ad-3 beams have also improved by 7.56% and 4.52%, respectively. The deflection of RSDB45-ad-3, RSDB75-ad-3, and RSDB-95-ad-3 beams improved by 36.40%, 32.71% and 20.75%, respectively, as compared to control beams SDB-ad-3.

Thus, it can be said that the performance of PMF jacketing is very consistent for both shear deficient and shear designed beams at a/d ratios of 1 & 3. Application of PMF jacketing filled the cracks which developed during the initial damaging of beams and also helped to arrests the formation of such deformities during re-testing of pre-damaged strengthened beams. The PMF jacketing helped enhance the ductility and caused a delay in the shear failure of beams by resisting and distributing the applied loads. The proportional deflection of strengthened beams also increased irrespective of their level of initial damages.

Load-deflection comparative curves of control and strengthened shear deficient beams with different levels of initial damages, when tested at a/d = 1 & 3, are detailed in Table 5.15 & 5.16 and shown in Figure 5.16 & 5.17.

Table 5.15: Shear Deficient Beams at a/d Ratio 1.0

S. No.	Control Beams (SDB-ad-1)		45% pre-damaged strengthened beams (RSDB45-ad-1)		75% pre-damaged strengthened beams (RSDB75-ad-1)		95% pre-damaged strengthened beams (RSDB95-ad-1)	
	Load (kN)	Def. (mm)	Load (kN)	Def. (mm)	Load (kN)	Def. (mm)	Load (kN)	Def. (mm)
1.	0	0.000	0	0.000	0	0.000	0	0.000
2.	12	0.299	12	0.619	12	0.535	12	0.592
3.	24	0.529	24	1.129	24	1.145	24	1.173
4.	36	0.796	36	1.525	36	1.619	36	1.747
5.	48	1.106	48	2.003	48	2.176	48	2.363
6.	60	1.497	60	2.473	60	2.719	60	2.930
7.	72	2.019	72	2.997	72	3.209	72	3.485
8.	84	2.545	84	3.605	84	3.690	84	4.094
9.	96	3.000	96	4.353	96	4.171	96	4.831
10.	108	3.461	108	5.114	108	4.677	108	5.547
11.	120	3.983	120	5.813	120	5.185	120	6.319
12.	132	4.583	132	7.759	132	5.790	132	7.101
13.	144.28	6.857	144	9.580	144	7.119	144	8.069
14.	139.62	7.738	150	11.923	150	7.770	153.35	10.412
15.			150.92	14.048	155.20	10.450	146.25	11.905
16.			147.27	14.957	151.83	12.319	141.25	12.545

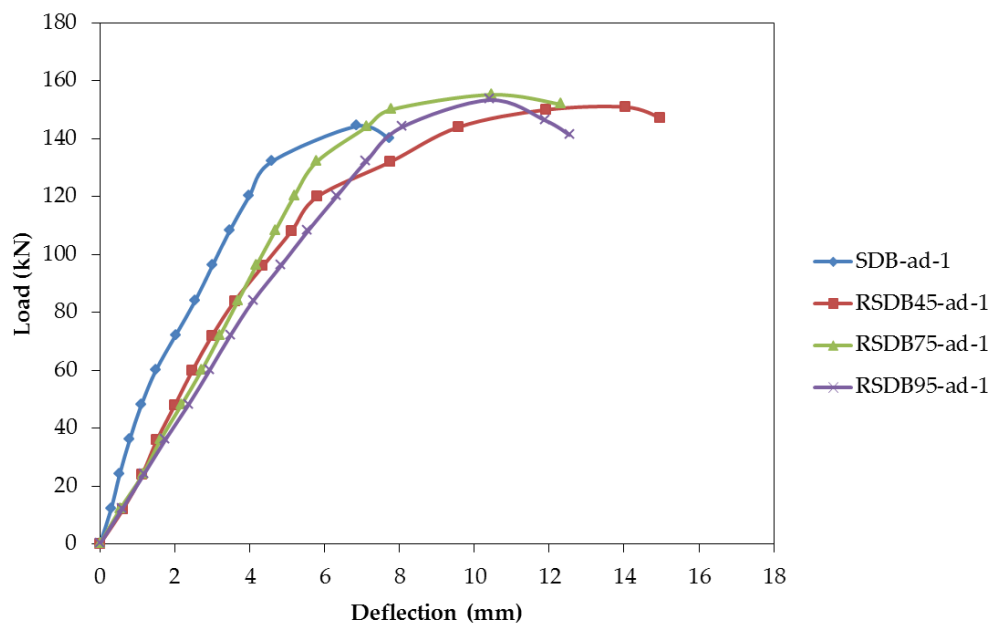


Fig. 5.16: Load-Deflection Plot of Shear Deficient Beams Tested at a/d Ratio 1

Table 5.16: Shear Deficient Beams at a/d Ratio 3.0

S. No.	Control Beams (RSDB-ad-3)		45% pre-damaged strengthened beams (RSDB45-ad-3)		75% pre-damaged strengthened beams (RSDB75-ad-3)		95% pre-damaged strengthened beams (RSDB95-ad-3)	
	Load (kN)	Def. (mm)	Load (kN)	Def. (mm)	Load (kN)	Def. (mm)	Load (kN)	Def. (mm)
1.	0	0.000	0	0.000	0	0.000	0	0.000
2.	6	0.599	6	1.002	6	0.949	6	1.125
3.	12	1.210	12	1.762	12	1.599	12	1.928
4.	18	1.866	18	2.655	18	2.517	18	2.849
5.	24	2.577	24	3.494	24	3.367	24	3.782
6.	30	3.309	30	4.207	30	4.015	30	4.392
7.	36	4.242	36	5.244	36	4.728	36	5.487
8.	42	5.256	42	6.325	42	5.761	42	6.369
9.	48	6.365	48	7.318	48	6.798	48	7.397
10.	54	7.835	54	8.373	54	7.881	54	8.474
11.	59.55	10.371	60	9.453	60	9.247	59.50	10.996
12.	57.99	11.916	66	11.285	62.55	9.952	62.24	12.523
13.	56.04	12.848	67.20	14.146	64.05	13.763	57.92	13.843
14.			66.40	15.773	62.20	14.437		
15.			63.19	16.589				

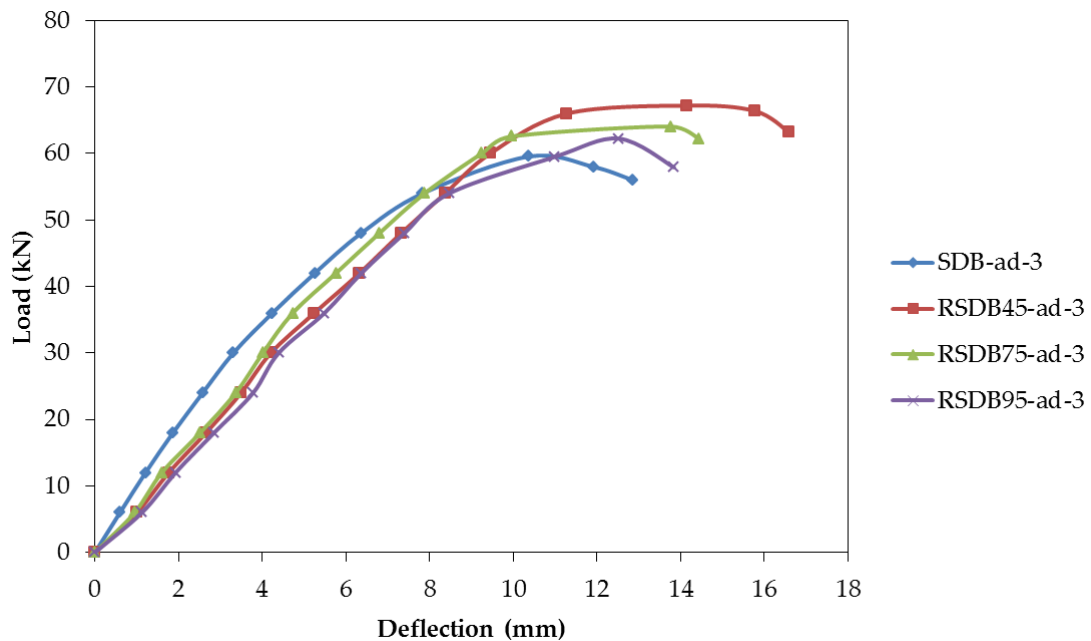


Fig. 5.17: Load-Deflection Plot for Shear Deficient Beams Tested at a/d Ratio 3

5.5.2 Effect of Shear Span-to-Depth (a/d) Ratios

A similar trend was recorded for control shear deficient (SDB-ad-1) beams, wherein a 142.28% higher load carrying capacity was observed in comparison to SDB-ad-3 beams. The strengthened shear deficient beams also behaved in a similar manner, wherein the ultimate load values of RSDB95-ad-1 beams (tested at $a/d = 1$) improved by a maximum of 146.38%. The RSDB45-ad-1 and RSDB75-ad-1 beams showed improvement in their ultimate strength by 124.58% and 142.31% respectively, as compared to RSDB75-ad-3 and RSDB95-ad-3 beams. The graphical comparison of ultimate loads at different a/d ratios is shown in Figure 5.18.

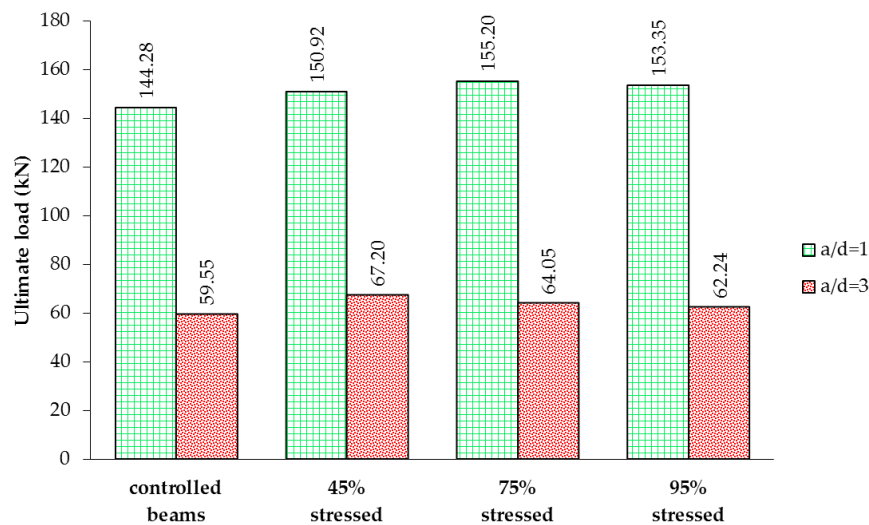


Fig. 5.18: Comparison of Ultimate Loads of Shear Deficient Beams Tested at a/d Ratio 1 & 3

It is also observed that at a/d ratio of 1.0, the beams tend to fail in shear only, and as the a/d ratio increased, the behaviour of beams shifted to a shear-flexure mode of failure. The shear forces were more dominant in the case of beams tested at $a/d=1.0$. For higher a/d ratio, the flexure strength governs the failure criteria of beams. All the control and strengthened beams showed more figurative strength values and a comparatively lesser deflection at a particular load level when tested at a smaller a/d ratio of 1.0 as compared to the beams tested at a higher a/d ratio of 3.0. As the distance of load point from the support was increased, the relative deflection also increased. This is due to the parabolic shape of deflection curve, and hence, the observed deflection values were smaller for the beams tested near the support as compared to the deflection values of the beams tested at '3d' away from the support. The observed behaviour of strengthened beams was more ductile when tested at higher a/d ratio of 3. This ductility improvement is due to change in failure mode. Thus it could be concluded that the PMF jacketing technique contributes more

towards improving the shear resistance of beams and further enhances the ultimate load values when the beams were tested at $a/d = 1$. Hence, PMF jacketing delays the direct shear failure and it apparently increases the contribution of stirrups to resist higher loads.

5.6 COMPARATIVE ANALYSIS AND DISCUSSIONS

5.6.1 Ultimate Shear Strength of Specimens

The failure load detail for all the tested beam specimens is produced in Table 5.17. No debonding of jacketing appeared when the PMF strengthened beams were tested up to ultimate failure level. These strengthened beams showed a significant improvement in their load carrying capacities, ductility behaviours, and cracking patterns.

The ultimate load carrying capacity of all the strengthened beams was found to have considerably improved as compared to control beams at both a/d ratios of 1.0 and 3.0. The percentage increase in ultimate load values of shear designed beams RDB-45-ad-1, RDB-75-ad-1 and RDB-95-ad-1 tested at $a/d = 1.0$, was 2.38%, 5.33% and 12.03%, respectively. It could be seen that as the initial stress level of strengthened beams increased, the percentage improvement in the strength also increased. A similar load enhancement behaviour was also observed for the strengthened beams tested at $a/d = 3.0$, wherein the ultimate load values of RDB-45-ad-3, RDB-75-ad-3 and RDB95-ad-3 beams increased by 11.19%, 8.82% and 8.85%, respectively. At higher $a/d = 3.0$, the percentage increase of load was observed to be more consistent, and all the shear designed beams with different levels of pre-damages did show an almost equal improvement of shear load carrying capacities. The graphical comparison of ultimate load values of shear designed beams tested at a/d ratio 1 & 3 is shown in Figure 5.19.

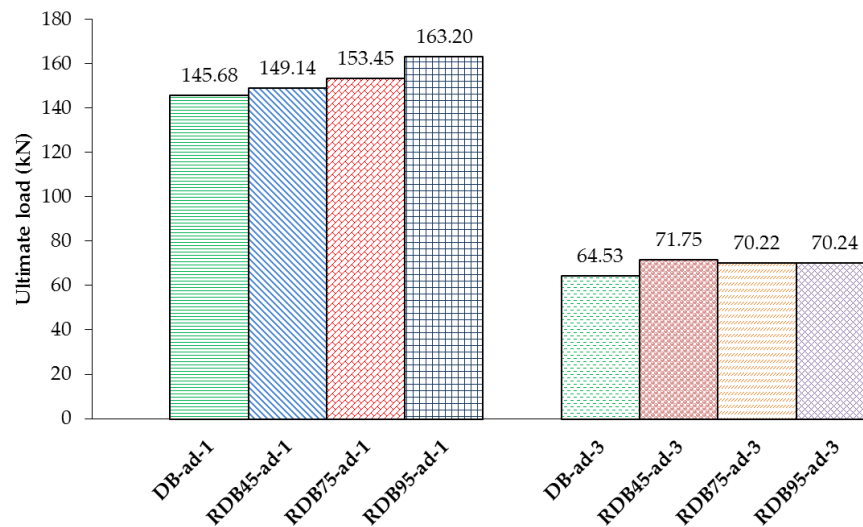


Fig. 5.19: Ultimate Loads of Shear Designed Beams Tested at a/d Ratio 1 & 3

Table 5.17: Reloading Test Results of Pre-Damaged Strengthened Beams

Beam Description	Average Ultimate Load (kN)	Percentage Increase in Ultimate Load	Average Deflection of 1st LVDT at Ultimate Load (mm)	Percentage Increase in Def. at Ultimate Load	Failure Mode
DB-ad-1	145.68	---	5.899	---	Shear
RDB45-ad-1	149.14	2.38%	11.891	101.58%	Shear
RDB75-ad-1	153.45	5.33%	12.494	111.80%	Shear-flexural
RDB95-ad-1	163.20	12.03%	14.882	152.28%	Shear-flexural
DB-ad-3	64.53	---	14.393	---	Shear-flexural
RDB45-ad-3	71.75	11.19%	11.674	-18.88%	Shear-flexural
RDB75-ad-3	70.22	8.82%	17.534	21.82%	Shear
RDB95-ad-3	70.24	8.85%	17.887	24.28%	Shear-flexural
SDB-ad-1	144.28	---	6.857	---	Shear
RSDB45-ad-1	150.92	4.60%	14.048	104.87%	Shear-flexural
RSDB75-ad-1	155.20	7.57%	10.450	52.40%	Shear-flexural
RSDB95-ad-1	153.35	6.29%	10.412	51.84%	Shear-flexural
SDB-ad-3	59.55	---	10.371	---	Shear
RSDB45-ad-3	67.20	12.85%	14.146	36.40%	Shear
RSDB75-ad-3	64.05	7.56%	13.763	32.71%	Shear
RSDB95-ad-3	62.24	4.52%	12.523	20.75%	Shear

It was observed that the failure loads of strengthened shear deficient beams also improved. This improvement was found lesser for the beams with a higher initial damage level of 95%. The shear deficient beams stressed to an initial level of 95% almost got fully damaged, but the strengthening technique showed its worth to restore and enhance the total load carrying capacity of such beams. The percentage improvement in failure load of beams RSDB45-ad-1, RSDB75-ad-1 and RSDB95-ad-1 tested at $a/d = 1.0$, was 4.60%, 7.57% and 6.29%, respectively as compared to the control beams SDB-ad-1. The strengthened beams RSDB45-ad-3, RSDB75-ad-3 and RSDB95-ad-3 also showed improvement in ultimate load values by 12.85%, 7.56% and 4.52%, respectively, as compared to the control beams SDB-ad-3 when tested at a/d ratio 3.0. The strengthened shear deficient beams tested at a/d 3.0 showed more percentage improvement in ultimate loads as compared to beams tested at a/d

= 1.0. Thus, it could be concluded that the PMF jacketing does not restore only but also improves the original strength of pre-damaged beams. The graphical comparison of ultimate load values of shear deficient beams tested at a/d ratio 1 & 3 is shown in Figure 5.20.

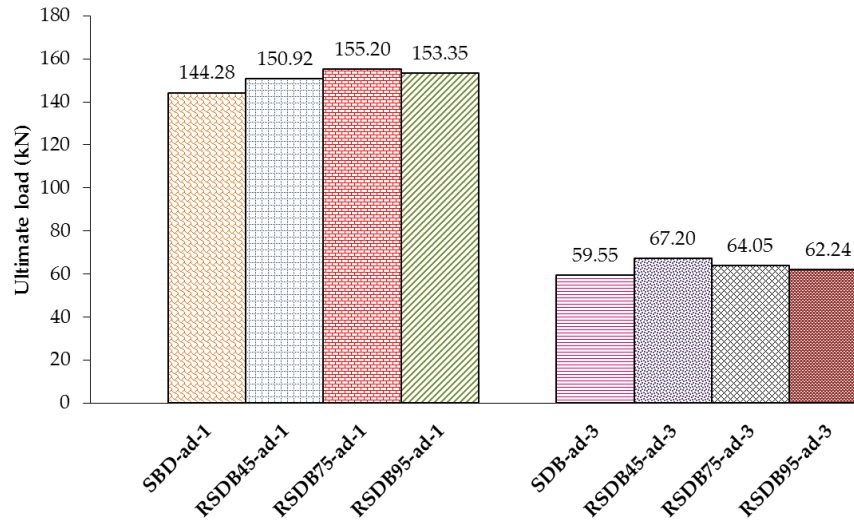


Fig. 5.20: Ultimate Loads of Shear Deficient Beams Tested at a/d Ratio 1 & 3

However, it was observed that some cracks did appear even through the jacketing, which indicates a good bond of PMF jacketing with the existing concrete structure. Although in some cases, no cracks emerged in the strengthened length where jacketing was applied, rather the failure cracks developed only in the vicinity of jacketing edges. Thus, it could be said that the PMF jacketing behaved monolithically with the concrete specimens and helped improve the ductility too, of the beams after strengthening.

For a/d ratio = 3, the ultimate strength of 75% pre-damaged strengthened beams is less as compared to 45% pre-damaged beams. However, when the beams were tested for a/d = 1, there is slightly increase of strength by 2 to 3%. This can be attributed to the fact that the shear resistance of beams mainly depends upon multiple factor such as: -

- Shear stress of un-cracked concrete,
- Arch action and dowel action,
- Shear transfer interface

Hence, there is little difference of 2 to 3% in the strength values for 75% pre-damaged strengthen beams as compared to 45% pre-damaged strengthened beams.

Further the flexural and tensile strength of retrofitting material is more than the parent material (concrete), which also contributes to increase in load carrying capacity of beams when tested at smaller a/d ratio.

As per clause 40.5.1 of IS 456, if the angle of failure plane is forced to be inclined more steeply than 30° , the shear force required to produce failure is increased. The enhancement of shear strength may be taken into account in the design of sections near a support by increasing design shear strength of concrete to $2d \tau_c/a_v$ provided that design shear stress at the face of the support remains less than the values given in Table 20 ($\tau_{c \max} = 3.24 \text{ N/mm}^2$). The value of τ_c to be taken as maximum only when the a/d ratio is less than 2. As per clause 40.5.3, for a/d ratio more than 2.0, the value of τ_c shall be taken as per Table 19.

At lesser a/d ratio, the major shear is resisted by concrete and contribution of stirrups is very small. Hence, the shear resistance contribution of concrete is decreased as the a/d ratio increased. Further, when the beams were tested for a/d ratio of 3, then the mode of failure was changed from shear to flexure. Hence, the effect of PMF to improve shear strength is not very significant. The shear strength of RCC beams is reduced as a/d ratio increased from 1 to 3 (*Tan et al., 1995; Fathifazl et al., 2009; Reddy et al., 2011; Naik and Kute, 2013; Alhamad et al., 2017; Hu and Wu, 2018; Sarsam et al., 2017*).

5.6.2 Effect of Spacing of Shear Stirrups

The beams having stirrups at a spacing of 150mm c/c were designated as shear designed beams (DB), and the beams with stirrups at a spacing of 450mm c/c were designated as shear deficient beams (SDB). Both the control and strengthened beams were tested for two different a/d ratios. The control shear deficient beams (SDB-ad-1) when tested at a/d ratio = 1.0, showed almost similar strength values as compared to control shear designed (DB-ad-1) beams with a negligible decrement of 0.96% only. A similar trend was observed at $a/d = 3.0$ and the SDB-ad-3 beams showed 7.72% lesser ultimate load as compared to DB-ad-3 beams. With the increase of initial damage level, the percentage decrease in loads showed an upward trend when the beams were tested at a/d ratio 3. The observed behaviour was, however, different in the case of beams tested at $a/d = 1$. The strengthened shear deficient beams (RSDB45-ad-1 & RSDB75-ad-1) with initial stresses of 45% and 75% showed a very minor increase in strength of about 1% only over the strengthened shear designed beams (RDB45-ad-1 & RDB75-ad-1). However, at the higher initial damage level of 95%, strengthened RSDB95-ad-1 beams reflected 6.04% lesser strength as compared to the strengthened shear designed beams RDB95-ad-1 beams. At $a/d = 1.0$ and for a particular initial damage level, the strengthened shear designed and shear deficient beams showed almost equal strength with a minor variation of 1% to 6% only.

This can be attributed to the arch action of the concrete element and the efficiency of PMF jacketing.

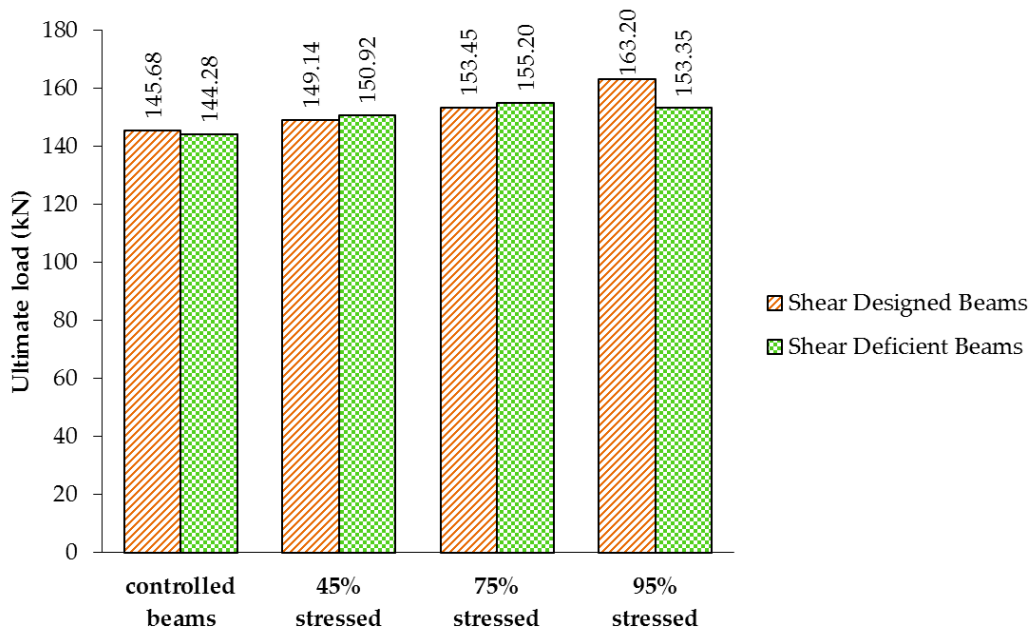


Fig. 5.21: Comparison of Ultimate Loads of Shear Designed and Shear Deficient Beams Tested at a/d 1

The strengthening technique is observed to be more efficient in case of shear deficient beams when tested at a/d ratio 1. In the event of beams tested at a/d = 3.0, a different type of behaviour was observed, and the strengthened shear deficient beams also showed lesser load carrying capacities as compared to shear designed beams for a particular damage level. Deflection behaviour was also inconsistent in both cases. Apparently, PMF enhanced the shear strength of concrete near the support by means of transferring the increased load to the stirrups and convert the brittle concrete failure into ductile. A graphical comparison of ultimate loads of beams tested at a/d ratio 1 is shown in Figure 5.21.

At a/d = 3, the strengthened shear deficient beams (RSDB45-ad-3, RSDB75-ad-3 & RSDB95-ad-3) showed lesser strength values by 6.34%, 8.79% & 11.39%, respectively, as compared to strengthened shear designed beams (RDB45-ad-3, RDB75-ad-3 & RDB95-ad-3). A graphical comparison of ultimate loads of beams tested at a/d ratio 3 is shown in Figure 5.22.

The PMF jacketing is fully effective to restore and enhance the original strength of both types of initially damaged beams. Further, in the case of shear deficient beams, it is observed that the cracks had developed at an angle of 30° to 45°. As the stirrups were at

spaced as far as 450mm c/c, many of these cracks were not interfered by the stirrups and therefore caused failure of beams in shear. The use of PMF tends to enhance the second moment of area and ductility of beam section in such cases and is thus observed to cause a delay in beam failure. Hence, the observed experimental load and deflection values are higher due to the combined strength of concrete and ferrocement jacketing. The strengthened shear deficient beams exhibited equally elastic behaviour as the shear designed beams, and the beams were observed to fail in shear-compression and diagonal-tension.

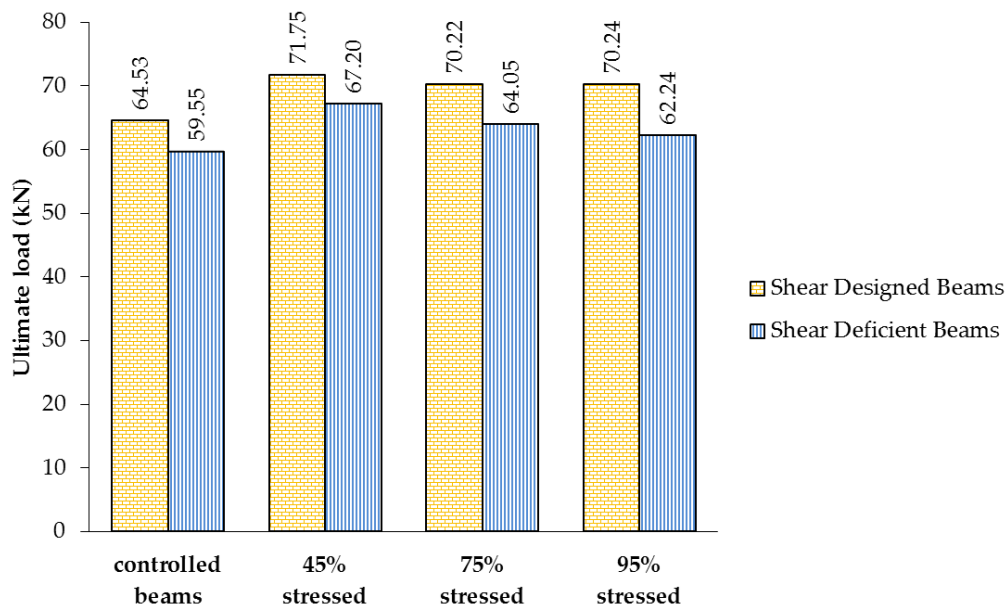


Fig. 5.22: Comparison of Ultimate Loads of Shear Designed and Shear Deficient Beams Tested at a/d 3

5.6.3 Mode of Failures

The control and strengthened beams were loaded and tested until ultimate failure. The modes of failures of the beams were observed at the ultimate level for comparative study. Table 5.17 gives the details of ultimate load values, deflection, the percentage increase in ultimate load and observed mode of failures. It has been observed that the failure modes basically depend upon the a/d ratios and spacing of stirrups. Mostly shear mode of failure was observed for shear deficient beams having a lesser amount of shear reinforcement as compared to the shear designed beams having sufficient shear reinforcement. The shear-flexural type was also observed for strengthened beams tested at a/d ratio 1.0 irrespective to their stirrups spacing. At a/d ratio 3.0, almost shear type mode of failure was observed for both SDB and RSDB beams. The control DB and SDB beams failed in shear except for the DB-ad-3 beam which failed in shear-flexure mode. The main

reason for the shear-flexure failure of this beam is the sufficient amount of shear reinforcement which shifts the failure into flexural mode and secondly, the a/d ratio of 3.0 which contributed to flexure type failure of the beam.

The pre-damaged strengthened beams tested at a/d 1.0 were found to resist more shear loads due to strengthening by PMF and tends to transfer the mode of failure from shear to shear-flexure. The shear deficient strengthened beams RSDB-ad-3 tested at a/d 3.0 showed shear failure even after strengthening. This is mainly due to less amount of shear reinforcement. The shear deficient strengthened beam RSDB-ad-1 showed a shear-flexural failure when tested at a/d 1.0. This shear-flexural mode of failure is due to the dowel action which helps to resist the induced shear stresses near the support. From the overall behaviour of beams, it can be concluded that PMF jacketing helps to resist the shear stresses better and transfers the pure shear failure mode into shear-flexural failure.

5.6.4 Load-Deflection Response of Specimens

For comparative study, the deflection values under the load point are only discussed in this article. The load-deflection plots for all the beam specimens are shown in Figure 5.1 to 5.17. It was observed that the ductility of both types of beams showed improvement after strengthening irrespective of beams tested at a/d 1.0 or 3.0. The main reason for the increase of ductility could be the strengthening of beams made with PMF. The section modulus of beams improved after strengthening and the beams start behaving with a more ductile manner which further helps to resist the deflection even after initial damages. Hence, the strengthened beams show higher deflection values as compared to the control beams at the ultimate failure load level. However, the comparative deflection values of strengthened beams at any particular load level seem to be higher as compared to the controlled beams. This behaviour is mainly due to an increase in the ductility of beams after strengthening. It can be concluded that the PMF increases the ductility of pre-damaged strengthened beams which further imparts to sustain higher deflection and ultimate loads. The deflection of strengthened beams was further prolonged after attaining ultimate failure level.

The deflection values of strengthened shear designed beams RDB45-ad-1, RDB75-ad-1 and RDB95-ad-1 was increased by 101.58%, 111.80% and 152.28%, respectively, when compared with deflection of controlled beams DB-ad-1 at ultimate failure level. A similar type of improvement was also observed in the case of shear deficient beams when tested at a/d ratio 1. The deflection values of beams RSDB45-ad-1, RSDB75-ad-1 and RSDB95-ad-1 was increased by 104.87%, 52.40% and 51.84%, respectively, as compared

to SDB-ad-1 beams. At a/d ratio 3, the overall deflection behaviour of beams was also improved; however, this improvement was comparatively lesser as compared to the beams tested at a/d ratio 1. The deflection of RDB75-ad-3 and RDB95-ad-3 was improved by 21.82% and 24.28%, respectively, as compared to DB-ad-3. The deflection value of strengthened shear designed beams (RDB45-ad-3) having 45% of initial damage was observed lesser by 18.88% at ultimate failure level. It reflects that the PMF imparts more stiffness in this case and cause to resist more deformation even at failure level. The deformation properties of shear deficient beams were also improved by a similar pattern and the deflection values of RSDB45-ad-3, RSDB-75-ad-3 and RSDB95-ad-3 beams at ultimate failure level was improved by 36.40%, 32.71% and 20.75%, respectively, as compared to SDB-ad-3 beams.

The pure shear failure of control beams was observed when tested at a/d ratio 1, irrespective to the spacing of stirrups. The beams tend to fail in shear mode when tested at smaller a/d ratio. The test results indicated that at smaller a/d ratio, the PMF strengthening technique enhanced the stiffness of beams to resist deformation at the initial stage and further impart ductility at a later stage which causes the transformation of shear failure into shear-flexural failure. Hence, with the application of PMF, the deflection further prolonged and higher deflection values were observed. It can be concluded that the deflection properties of beams were overall improved after strengthening and the comparative improvement is more in case of beams tested at a/d ratio 1.

5.6.5 Cracking Pattern and Failure Mechanism

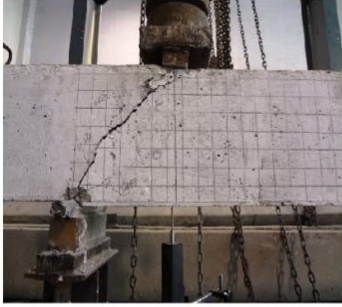
A few numbers of cracks of varying width and spacing were observed in both types of control beams. The beams ultimately failed because of the widening of any of these cracks. On the other hand, the pre-damaged strengthened beams showed fewer hairline cracks during testing. The strengthened beams ultimately failed due to the formation of inclined shear cracks. These beams showed minor cracks when compared with the control beams of that particular segment. Plates 5.1 to 5.4 shows the cracking pattern of shear designed and shear deficient control and strengthened beams tested at a/d = 1 & 3.

The tested beams got cracked, apparently due to two different conditions. In some cases, the cracks appeared diagonally near the support and mostly because of shear forces and in others; the cracks appeared towards centre due to shear forces together with some flexure moment. Most of the beams tested at a/d ratio 1.0, showed cracks of the first type which are described as shear compressive and diagonal tension cracks. In this mode of

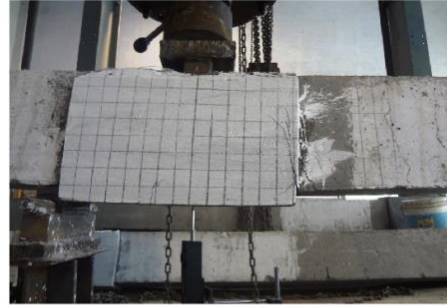
failure, the inclined cracks propagate rapidly due to inadequate shear reinforcement and cause the failure of concrete from the edge of crack (refer Plate 5.1.a, 5.3.a).). With the increase in a/d ratio from 1.0 to 3.0, the observed crack pattern was of the second type which is due to the combined action of shear force and flexure moment (refer Plate 5.2.a, 5.4.a).

In the case of both shear designed and shear deficient beams when tested at $a/d=1.0$, the first crack appeared almost in the region where the shear force was maximum and the bending moment was negligible, i.e. in a shear span 'a' which is equal to ' $d = 198\text{mm}$ ' for a/d ratio 1.0. The cracks were aligned with one another and inclined to the axis of the beam at an angle between 30° to 45° . It was observed that these cracks belonged to the first type, i.e.; mostly due to shear forces. As the load increased, further cracks appeared in the zone of lesser shear stresses, i.e. towards the centre of beams, and non-linearity of the load-deflection behaviour was increased. With the further increase of load, more and more cracks were developed and a dominant crack propagated towards the point of loading, resulting in ultimate failure. From the load-deflection plots, it is observed that the behaviour of beams remained linear up to a certain point and after that, it changed into non-linear.

Shear mode of failure was observed in the tested beams, and most of the cracks were developed near the support and loading point (refer Plate 5.1 to 5.4). The cracking pattern of shear designed and shear deficient beams was changed as the a/d ratio changed from 1.0 to 3.0. It was observed that the shear deficient control beams were failed in a typical shear mode when tested at $a/d = 1.0$. The shear designed control beams DB-ad-3 were observed to behave like DB-ad-1, but in this case, the cracks were not only in the shear zone, but some hairline cracks were also developed towards the centre of the beam. These cracks were in the region of bending moment, and as such, these beams were in combined mode condition. These cracks further grew and followed a bent path indicating a valid contribution of flexure moment in such cases. Further increasing of the load caused the development of a dominant crack towards the reduced compression zone. The response of shear designed beams was generally ductile when tested at $a/d = 3.0$. This is due to flexural effect of beams at higher a/d ratios. Aggregate interlocking, dowel action of longitudinal bars and bond strength of the concrete also impart to resist the stresses which were further induced due to the widening of cracks. However, it is difficult to estimate the exact contribution of these different components.



(a)



(b)



(c)



(d)

Plate 5.1: Shear Designed Beams tested at $a/d = 1$: (a) DB-ad-1; (b) RDB45-ad-1; (c) RDB75-ad-1; (d) RDB95-ad-1



(a)



(b)

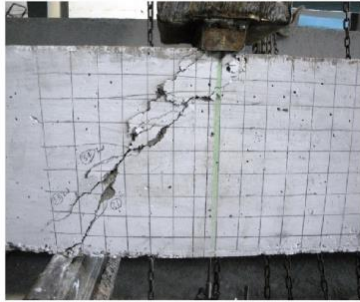


(c)



(d)

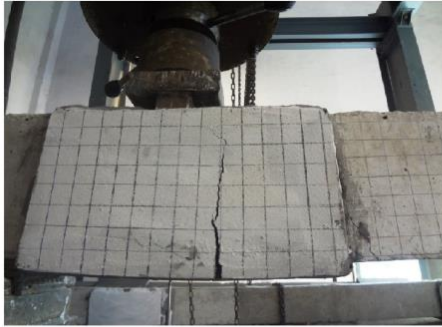
Plate 5.2: Shear Designed Beams tested at $a/d = 3$: (a) DB-ad-3; (b) RDB45-ad-3; (c) RDB75-ad-3; (d) RDB95-ad-3



(a)



(b)

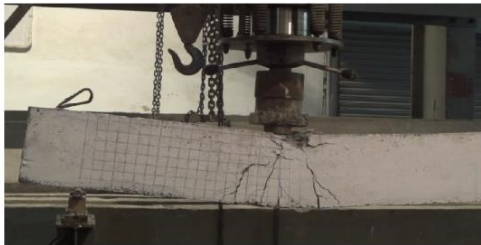


(c)

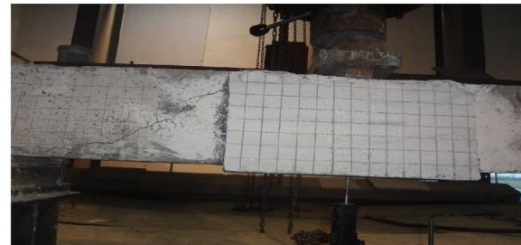


(d)

Plate 5.3: Shear Deficient Beams tested at $a/d = 1$: (a) SDB-ad-1; (b) RSDB45-ad-1; (c) RSDB75-ad-1; (d) RSDB95-ad-1



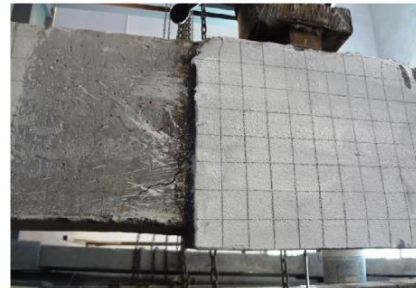
(a)



(b)



(c)



(c)

Plate 5.4: Shear Deficient Beams tested at $a/d = 3$: (a) SDB-ad-3; (b) RSDB45-ad-3; (c) RSDB75-ad-3; (d) RSDB95-ad-3

The behaviour of all the strengthened beams those have different levels of initial damages were observed experimentally, and it was found that the jacketing neutralised the opening of cracks. The strengthened beams behaved stiffer as compared to the corresponding beams without strengthening, and the rate of crack development was also reduced. PMF strengthened beams further displayed a lesser number of cracks when loaded to failure. This strengthening technique helps to improve the ductility of beams and cause to delay the formation of cracks. As a result, the figurative deflection of pre-damaged strengthened beams was also enhanced. Spalling of concrete in the vicinity of support point was also reduced because of strengthening. PMF jacketing acted monolithically with the RCC beam specimens, and no bond failure was observed during the testing process. Though, some cracks were found near the edge of jacketing which are attributable to the differential characteristics of strengthened beams at the newly created intersection of jacketed and un-jacketed sections.

The shear failure mechanism of RCC beams is a complicated phenomenon because of the interlinking of many factors such as loading pattern, shear-span ratio, beam section, strength of concrete and the quantity of shear & bending reinforcement. The characteristic of shear failure is abrupt as compared to flexural failure. However, the PMF jacketing technique leads to achieving the ductile failure of strengthened beam specimens. From the above experiments; it is concluded that the PMF strengthening technique has improved the deformation behaviour, cracking pattern and the ultimate shear load carrying capacity of the initially damaged beams.

6.1 GENERAL

The experimental test results of control and strengthened beam specimens have been validated in this chapter for their strength and ultimate load enhancement by using Finite Element Method (FEM) based software ATENA-3D. The FE models of the tested RCC beams were created and run on the designated software for analysis. The actual laboratory conditions were simulated using the software and the controls as well as strengthened beams were modelled accordingly. In the first phase of modelling, the control beams were modelled and the results have been compared with the actual experimental values. In the second phase, the beams were modelled for initial partial load values of, 45%, 75% and 95% of the ultimate values of control beams. FE models of these partially damaged beams were then developed with polymer modified ferrocement (PMF) jacketing, which was used for strengthening of beams in the specified length. The strength and other properties of PMF were assigned as per actual experimental result values. In this chapter, the detailed step-by-step procedure, highlighting the modelling and analysis of FE model for control as well as for strengthened beams. The stepwise procedure adopted for pre-processing, analysis and post-processing is explained with the help of flow chart in Figure 6.1. A total of 16 FE models were created and run with this software. Out of these 16 models, 4 models were developed for shear designed and shear deficient control beams for a/d ratio 1 & 3. The remaining 12 FE models were created for shear designed and shear deficient beams which were further simulated for pre-damage with partial load of 45%, 75% and 95% and then modelled as strengthened beams with three-layered PMF. Non-linear FE analysis was carried out under static load conditions.

6.2 FINITE ELEMENT MODELLING OF CONTROL RCC BEAM SPECIMENS IN ATENA-3D**6.2.1 Assumptions for Finite Element Modelling**

The followings are the assumptions for FE modelling of control specimens: -

1. Steel reinforcement and concrete are considered as homogeneous and isotropic materials.
2. Maximum compressive strain in concrete is assumed as 0.0035mm/mm as per Indian standard code IS 456:2000.

3. Perfect bond is considered between concrete and reinforcement.
4. Tensile strength of concrete below the N.A. of the beam was not considered and ignored for analysis.
5. Reinforcement steel is assumed as elastic-perfectly-plastic material with identical properties in compression and tension.
6. It is assumed that the plane section remains plane before and after loading i.e. strain in steel and concrete is proportional to its distance from N.A.

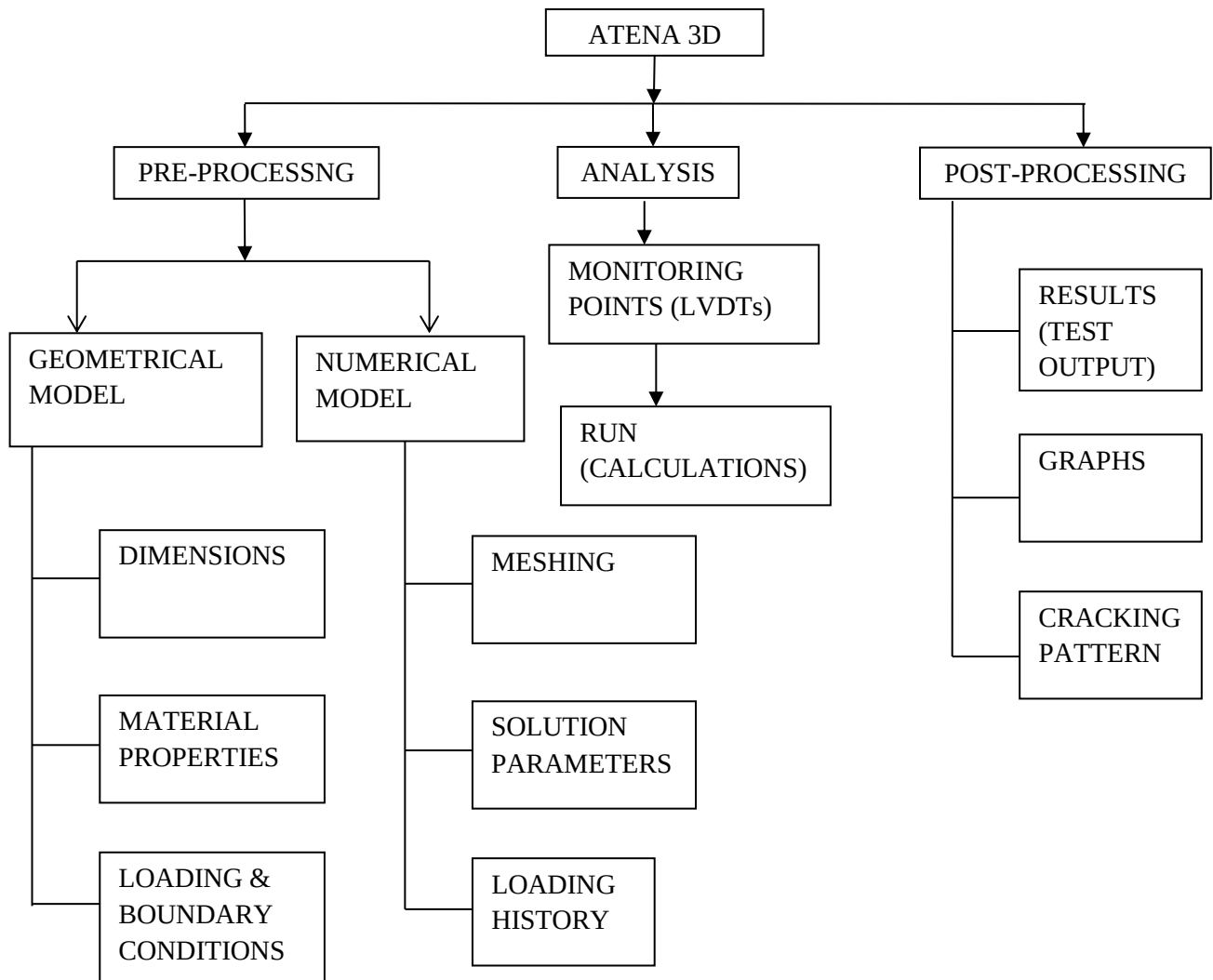


Fig. 6.1: Steps for ATENA Modelling and Analysis

6.2.2 Criteria for Failure of Concrete

As per *Willam and Warnke (1975)*, the model is capable of predicting failure for concrete materials. Both cracking and crushing failure modes are accounted for. The two input strength parameters – i.e., ultimate uniaxial tensile and compressive strengths - are

needed to define a failure surface for the concrete. Consequently, a criterion for failure of the concrete due to a multiaxial stress state can be calculated.

A three-dimensional failure surface for concrete, as considered by *Kachlakev et al. (2001)*, is shown in Figure 6.2. The most significant nonzero principal stresses are in the x and y directions, represented by σ_{xp} and σ_{yp} , respectively. Three failure surfaces are shown as projections on the σ_{xp} - σ_{yp} plane. The mode of failure is a function of the sign of σ_{zp} (principal stress in the z direction). For example, if σ_{xp} and σ_{yp} are both negative (compressive) and σ_{zp} is slightly positive (tensile), cracking would be predicted in a direction perpendicular to σ_{zp} . However, if σ_{zp} is zero or slightly negative, the material is assumed to crush.

In the FE modelling, it was found that if the crushing capability of the concrete is turned on, the finite element beam models fail prematurely. Crushing of the concrete started to develop in elements located directly under the loads. Subsequently, adjacent concrete elements crushed within several load steps as well, significantly reducing the local stiffness. Finally, the model showed a large displacement, and the solution diverged. A pure “compression” failure of concrete is unlikely. In a compression test, the specimen is subjected to a uniaxial compressive load. Secondary tensile strains induced by Poisson’s effect occurs perpendicular to the load. Because concrete is relatively weak in tension, these actually cause cracking and the eventual failure. Therefore, in this study, the crushing capability was turned off and cracking of the concrete controlled the failure of the finite element models (*Kachlakev et al., 2001; Wolanski, 2004; Ibrahim and Mahmood, 2009*).

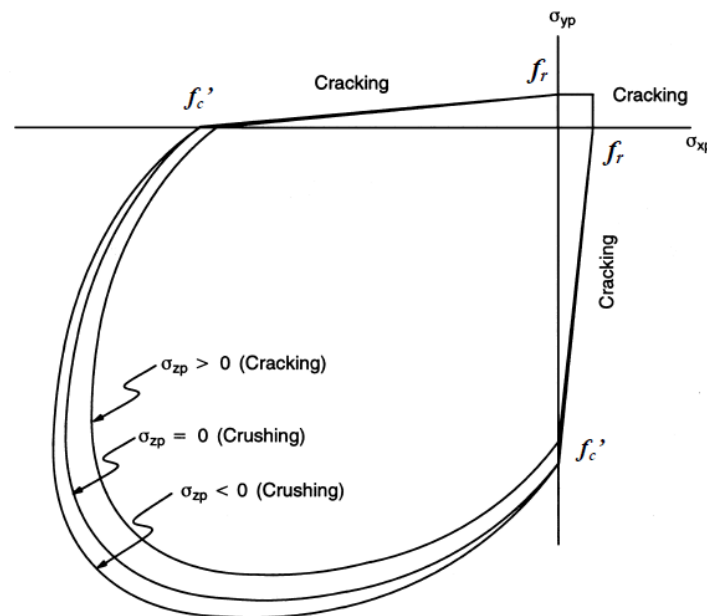


Fig. 6.2: 3-D Failure Surface for Concrete (*Kachlakev et al., 2001*)

6.2.3 Pre-processing

The first step of pre-processing is to define the geometrical dimensions of the specimen. The input data of joints in term of coordinates is to be entered. These specified joints are then joined with the geometrical lines to create the whole specimen. After defining the geometry, the next step is to define the regions and elements of these connected lines. For the same, the material properties of each element also needed to be defined and the detail is described in the following sections.

6.2.4 Type of Elements for Finite Element Modelling

Many types of material elements can be modelled in ATENA 3-D and there are a variety of in-built materials available in the software library. In the present modelling, the concrete and steel are the main elements which are considered as per ATENA 3D Theory Manual (ATENA software manual) and are detailed below: -

a. Concrete Element

Concrete is modelled as a 3D solid brick element in this model with 8 to 20 nodes as shown in Figure 6.3. There are three degrees of freedom in x, y and z-direction at each node of this 3D brick element with non-linear material properties. The concrete is modelled as an isoparametric material element, which is incorporated using the Gauss integration. This type of element is able to deform plastically, and crushed and crack in three orthogonal directions.

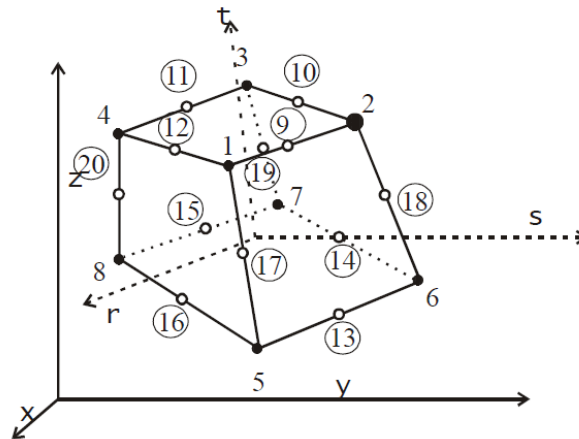


Fig. 6.3: Input Geometry of Concrete Element (ATENA Manual)

b. Reinforcement-bar Element

Reinforcement-bar needs to define in the next step of FE modelling. ATENA also enabled the users to define the reinforcement bar element at any time during the modelling

work and before defining the macro-elements and mesh generation. The reinforcement element can be modelled as smeared or discrete in ATENA 3-D. In the present work, the reinforcement has been modelled as a discrete element. This bar element has three-dimensional freedom at each node with uniaxial tensile and compressive properties. The uniform stresses have been considered for the whole length of the bar element. The creep, plasticity, stress stiffening and large deflection are also incorporated in the element properties. The reinforcing bar element is shown in Figure 6.4.

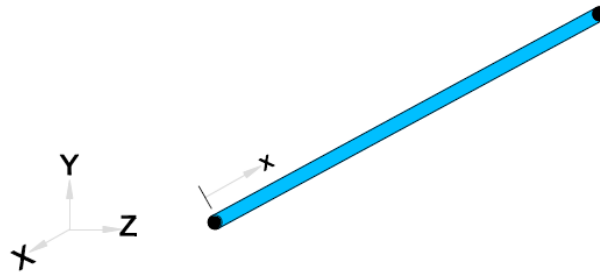


Fig. 6.4: Geometry of Steel Reinforcement Element

c. Steel Plates

The load on the beam specimens was applied by means of steel bars in the laboratory and the supporting system is also made of steel members. Hence, to simulate the actual laboratory conditions, a homogeneous structural solid element was assigned at the vertical supports and at the loading point. The steel plate element was assigned with the same properties as reinforcing bars with the yield strength of Fe250. The steel plate element is shown in Figure 6.5.

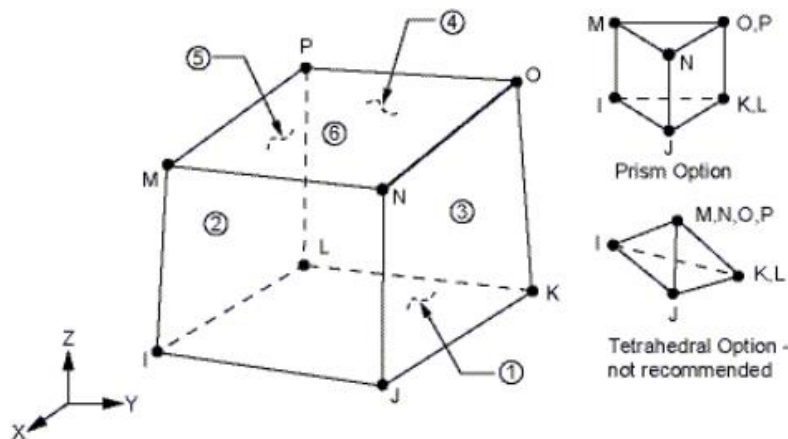


Fig. 6.5: Geometry of Steel Plate Element

6.2.5 Description of Material Properties

After defining the element type, the material properties were assigned in the modelling of RCC beams. The concrete of grade M-20 (with laboratory strength of 26.72

N/mm²), main bottom reinforcement of diameter 12mm grade Fe-500 (with laboratory yield strength of 556.5 N/mm²), top reinforcement of diameter 8mm grade Fe-500 (with laboratory yield strength of 548.5 N/mm²) and shear stirrups reinforcement of grade Fe-250 (with the laboratory yield strength of 445 N/mm²) were modelled. The material specifications used in FE modelling are described as below: -

1) Concrete

The concrete of RCC beam specimens was modelled as ‘3D nonlinear cementitious2’ element. The input data properties of concrete for FE modelling are described in Table 6.1.

2) Reinforcement Bars

The 12mm diameter and 8mm diameter HYSD bars of grade Fe-500 were used as the main bottom and top reinforcement, respectively. The 6mm diameter mild steel bars of grade Fe-250 was used as shear reinforcement. The input data properties of reinforcement for FE modelling are described in Table 6.2.

3) Steel Plate

The loading point and vertical supports are modelled as steel plates elements and the same is described as mild steel plate having yield strength of Fe250.

Table 6.1: Concrete Input Data Properties for FE Modelling

S. No.	Description	Values
1.	Compressive Strength (f_{ck}) (N/mm ²)	26.72
2.	Tensile Strength (f_{st}) (N/mm ²)	3.618
3.	Modulus of Elasticity (E_c) = $5000\sqrt{f_{ck}}$ (N/mm ²)	25845.7
4.	Poisson's Ratio (μ)	0.17
5.	Specific weight (γ) (kN/m ³)	24000
6.	Co-efficient of Thermal Expansion (α_c)	1.2×10^{-5} 1/K
7.	Fixed Crack Model co-efficient	1.0
8.	Ultimate strain in bending (ϵ)	0.0035

Table 6.2: Reinforcement-Bars Input Data Properties for FE Modelling

S. No.	Description	Values
1.	Yield Strength (f_y) (N/mm ²)	500
2.	Modulus of Elasticity (E) (N/mm ²)	2.1×10^5
3.	Specific weight (γ) (N/m ³)	78500
4.	Co-efficient of Thermal Expansion (α_t)	1.1×10^{-5} 1/K

6.2.6 Modelling

The shear designed and shear deficient RCC beams were modelled as per actual dimensions of the beam (refer Fig. 4.1 of Chapter 4) as below and a combined volume was created in ATENA-3D as shown in Figure 6.6.

Total length (L) of RCC beams	=	2700mm
Effective length (l_{eff}) of RCC beams	=	2500mm
Width (b) of beams	=	127mm
Total Depth (D) of beam	=	229mm
Clear cover of concrete	=	25mm
Effective cover of concrete	=	31mm
Effective depth (d) of beam	=	198mm
For a/d ratio 1.0, the value of 'a'	=	198mm
For a/d ratio 3.0, the value of 'a'	=	594mm
Steel plate for vertical support	=	25mm × 90mm × 127mm
Steel plate for loading point	=	50mm × 25mm × 127mm

The four types of control specimen's models were generated in ATENA-3D and detailed as in Table 6.3.

Table 6.3: Description of Control Specimens of RCC Beams

S. No.	Designation	Tested at a/d ratio	Description of Control Beam	Spacing of shear stirrups	Point of Load application from left support
1.	DB-ad-1	1.0	Shear Designed Beams	150 mm c/c	198mm
2.	DB-ad-3	3.0	Shear Designed Beams	150 mm c/c	594mm
3.	SDB-ad-1	1.0	Shear Deficient Beams	450 mm c/c	198mm
4.	SDB-ad-3	3.0	Shear Deficient Beams	450 mm c/c	594mm

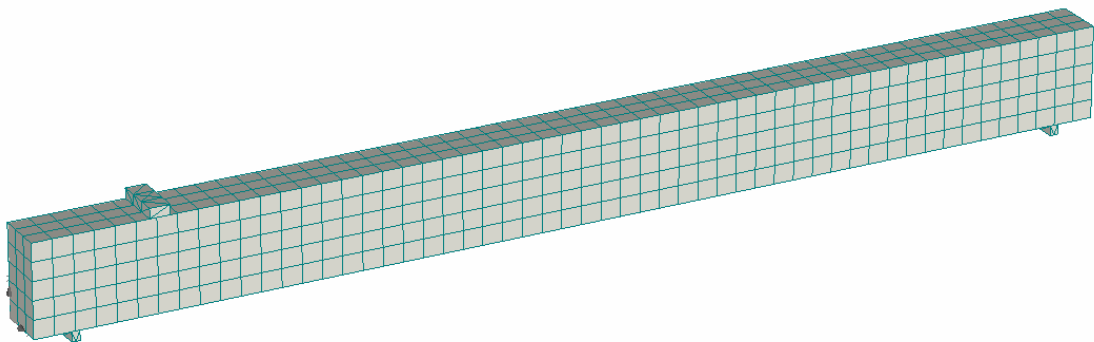


Fig. 6.6: Combined Volume Created in ATENA-3D for the Control Specimen

6.2.7 Mesh Generation

After defining the macro-elements, the mesh can be generated. The mesh is generated automatically in the ATENA-3D software. The mesh is generated by pressing the button mesh generation. The finite element mesh has been generated automatically as per the size of the element. The mesh shape can be defined by the user and the rectangular mesh elements have been created in this modelling, since the models created with rectangular elements have shown better results (*Wolanski, 2004; Kachlakev et al., 2001*). The mesh generated in ATENA-3D is shown in Figure 6.7. The reinforcement arrangement model for both shear designed and shear deficient beam specimens are also shown in Figures 6.8 (a) & 6.8 (b).

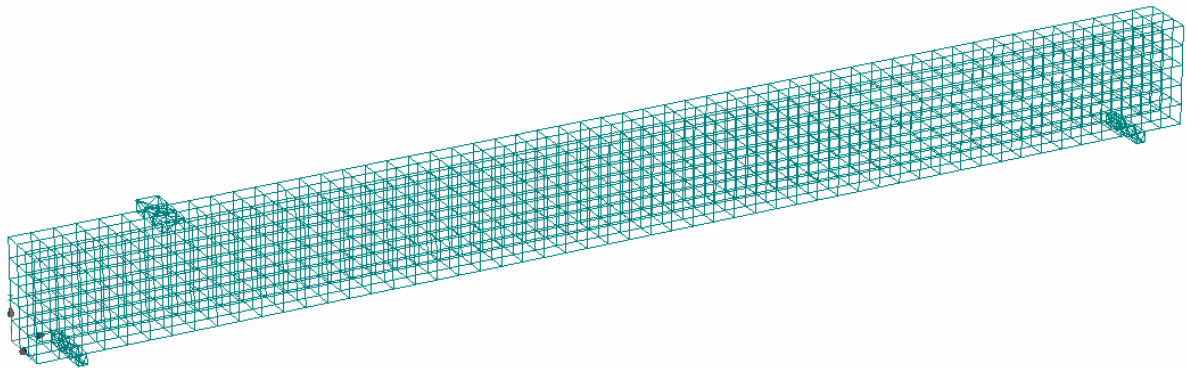


Fig. 6.7: Mesh Created in ATENA-3D for the Control Specimen

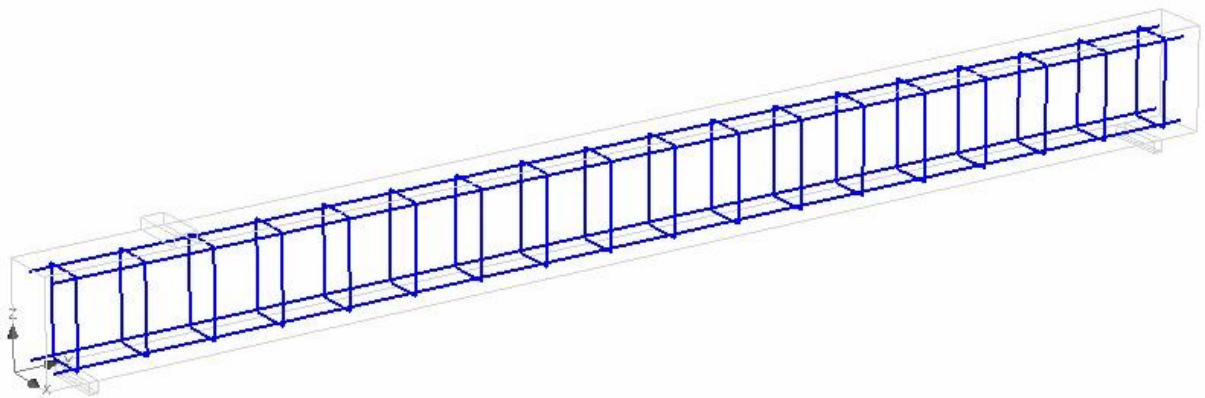


Fig. 6.8 (a): Reinforcement Arrangement Modelled in ATENA-3D for the Control Specimen (shear designed beams – DB)

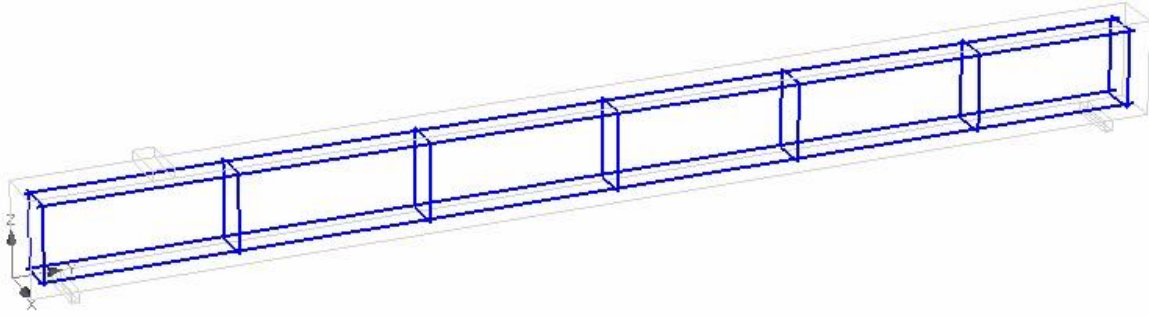


Fig. 6.8 (b): Reinforcement Arrangement Modelled in ATENA-3D for the Control Specimen (shear deficient beams – SDB)

6.2.8 Support Conditions and Loading

This section defines the physical constraints (supports/boundary conditions) and the loadings (actions) of the beam model. The RCC beam specimen is supported by vertical supports at the bottom which is described with the help of steel plate element in the FE modelling. The beam specimen is loaded by a point load at top, which is also defined with the steel plate element. By FE modelling, we are validating and determining not only the maximum load-carrying capacity of the specimen but also the structural response. For the same, vertical load is applied, which is further increased in each step. In order to make the FE model as a displacement load control, the displacements were restrained in term of boundary conditions by inputting the initial deformation value as zero. The boundary conditions of the RCC beam specimens are shown in Figure 6.9 (a) & (b). The load is applied at distance of 198mm and 594mm for two different a/d ratios of 1.0 and 3.0, respectively (refer Table 6.3 for detail). The application of load at a/d ratio 1.0 and 3.0 is also shown in Figure 6.10 (a) & 6.10 (b), respectively.

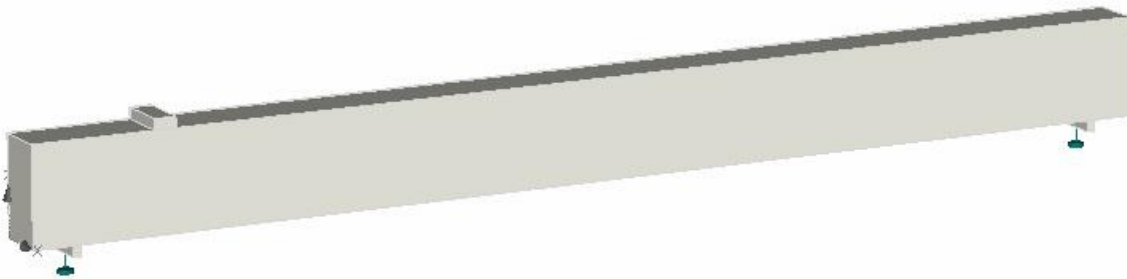


Fig. 6.9 (a): Boundary Conditions for the Control Specimen ($a/d = 1.0$)

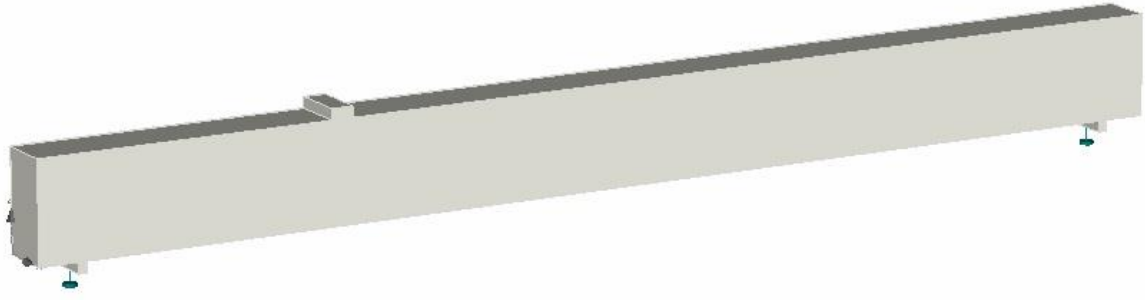


Fig. 6.9 (b): Boundary Conditions for the Control Specimen ($a/d = 3.0$)

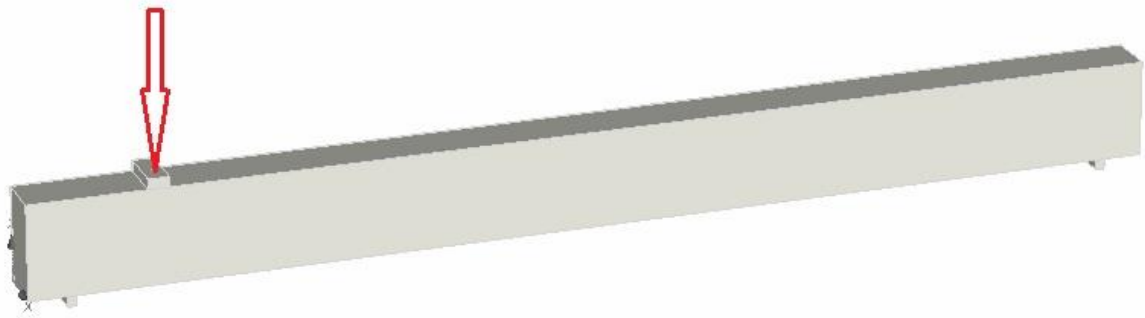


Fig. 6.10 (a): Loading Conditions for the Control Specimen ($a/d = 1.0$)

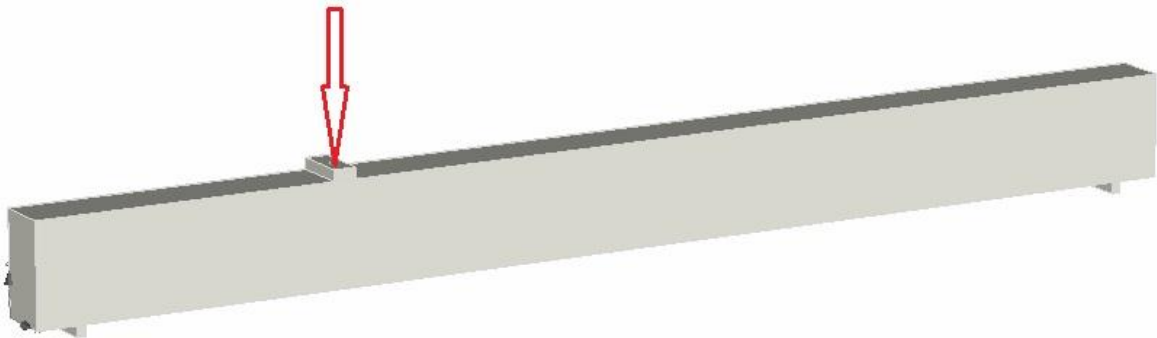


Fig. 6.10 (b): Loading Conditions for the Control Specimen ($a/d = 3.0$)

6.2.9 Monitoring Points

It is useful to monitor the displacements, loads and stresses in the FE model during a non-linear analysis. This data provides information about the structural state during testing. Though this, it is possible to find out whether the maximum load or deflection was reached or not. In this FE model, the first monitoring point is added near the left support and just below the loading point where the prescribed displacement is applied i.e. at a distance of 198mm for a/d ratio 1.0 and at a distance of 594mm for a/d ratio 3.0, respectively. The

second monitoring point is located at the centre of beam specimen near its bottom surface, as the largest deflection is expected at this monitoring point. The third point is fixed as per the actual laboratory conditions i.e. just mirrored opposite to the first point. These monitoring points are at the same location where the LVDTs were setup to determine the deflection in the laboratory experiment. For details about the location of LVDTs, refer Section 4.4 of Chapter 4. With the help of these monitoring points, the load-displacement curve was monitored during the analysis. The changes in load and displacement were observed at each load step. The changes in the values could also be observed for each iteration. Figure 6.11 (a) & 6.11 (b) shows the monitoring points in FE modelling for beams tested at a/d ratio 1.0 and 3.0, respectively.

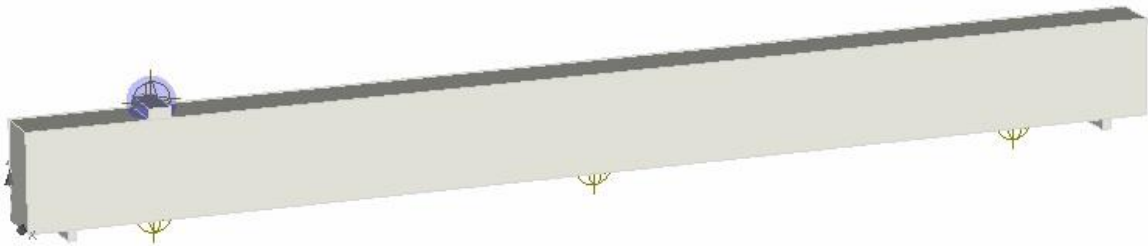


Fig. 6.11 (a): Monitoring Points in FE Modelling for Beams tested at $a/d = 1.0$

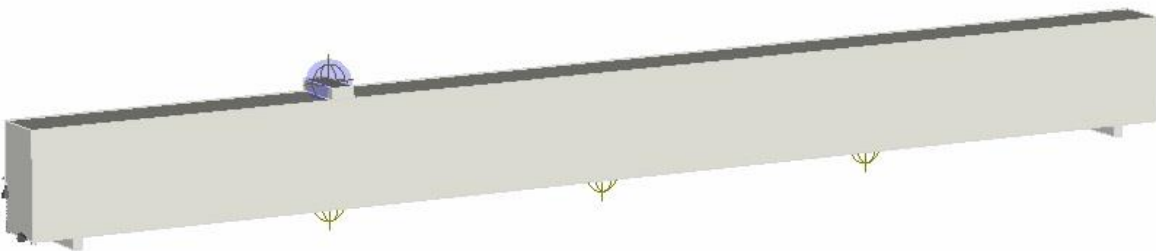


Fig. 6.11 (b): Monitoring Points in FE Modelling for Beams tested at $a/d = 3.0$

6.2.10 Finite Element Non-Linear Analysis Process

a. Run Analysis

After the pre-processing, as detailed above, non-linear FE analysis was ‘run’ to get the final results. The progress of analysis could be monitored in the interactive window. New load steps could also be added after the completion of initial described steps. Results were saved after completion of the analysis process for further study.

b. Post-processing

After the completion of FE analysis or termination of analysis, the software automatically entered into post-processing window. We could also use the post-processing command to get the final results. This software enables the user to get the post-processing output data in the form of; load-deflection curve, internal stress-strain diagrams, crack pattern, crack propagation and text output and analysis log files. This output data of FE analysis was printed online. In the present study, the load-deflection behaviour of first monitoring point is only presented here for comparative study.

6.3 FINITE ELEMENT MODELLING OF RCC STRENGTHENED BEAMS WITH PMF

Based upon the test results of control beams specimens, the three zones of pre-damaging were initially fixed at 45%, 74% and 95% of ultimate load values. These pre-damaged beams were then strengthened with polymer modified ferrocement for a specified length of the beam. The detail of strengthening is given in section 4.7 of chapter 4. These strengthened beams are modelled using ATENA-3D software by simulating the actual laboratory conditions. The analysis was carried out for these strengthened beams and the results so obtained have been compared with the actual laboratory obtained values.

6.3.1 Assumptions for Finite Element Modelling of Strengthened Beam Specimens

The followings are the assumptions made for modelling of strengthened RCC beam specimens: -

1. Steel reinforcement and concrete are considered as homogeneous and isotropic materials.
2. Reinforcement steel is assumed an elastic-perfectly plastic material with identical properties in compression and tension.
3. It is assumed that the ferrocement behave linear-elastic in tension till failure and stresses carried by ferrocement are along its axis only.
4. It is assumed that the plane section remains plane before and after loading i.e. strain in steel and concrete is proportional to its distance from N.A.
5. Maximum compressive strain in concrete is assumed to be 0.0035mm/mm as per Indian standard code *IS 456:2000*.
6. Tensile strength of concrete below the N.A. of the beam was not considered and ignored for analysis.

7. Perfect bond is considered between concrete and reinforcement and between concrete and strengthening jacket.

6.3.2 Element Types for Ferrocement used for Strengthening

Concrete, reinforcement and steel plates are modelled the same way as for control beams. The only change made in the FE modelling of RCC strengthened beams is to add the ferrocement element. The ferrocement element is modelled as a brick element in the ATENA-3D modelling. This is an isometric brick element which is composed of 8 nodes (as shown in Figure 6.12).

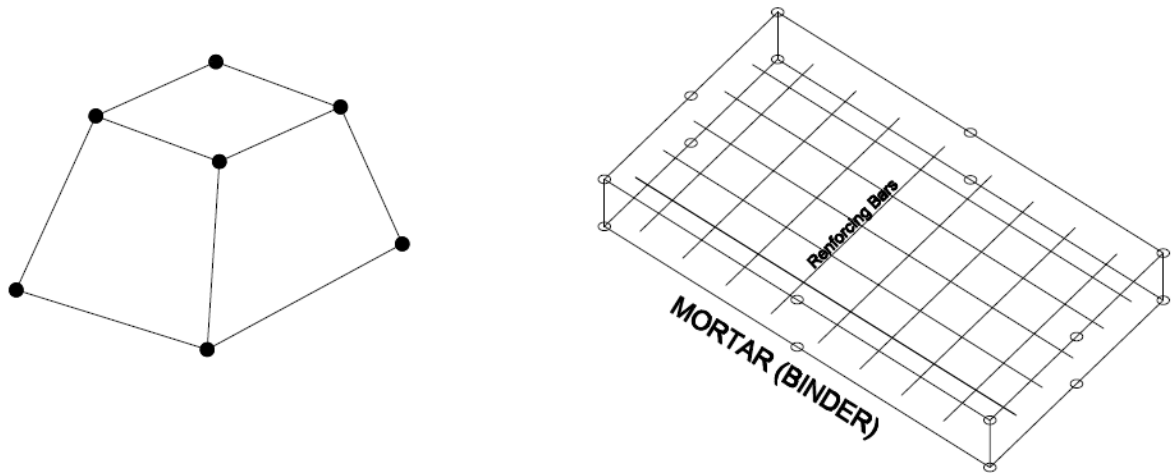


Fig. 6.12: Eight Nodes Brick Element Model for Concrete and 20 Nodes Shell Element Model for Ferrocement Composite

6.3.3 Material Properties

Concrete, reinforcement-bars, steel plates and polymer modified ferrocement containing three layers of steel woven square wire mesh, have been modelled for RCC strengthened beam specimens. The properties and specifications of these materials are detailed below: -

- 1. Concrete:** As explained earlier, concrete has been modelled as '3D nonlinear cementitious- 2' element in ATENA-3D.
- 2. Steel Plate:** The vertical supports and the point load are modelled as steel plates, the same as modelled for the control beam specimens.
- 3. Reinforcement-bars:** The reinforcement- bars are also modelled the same way as for the control specimens.

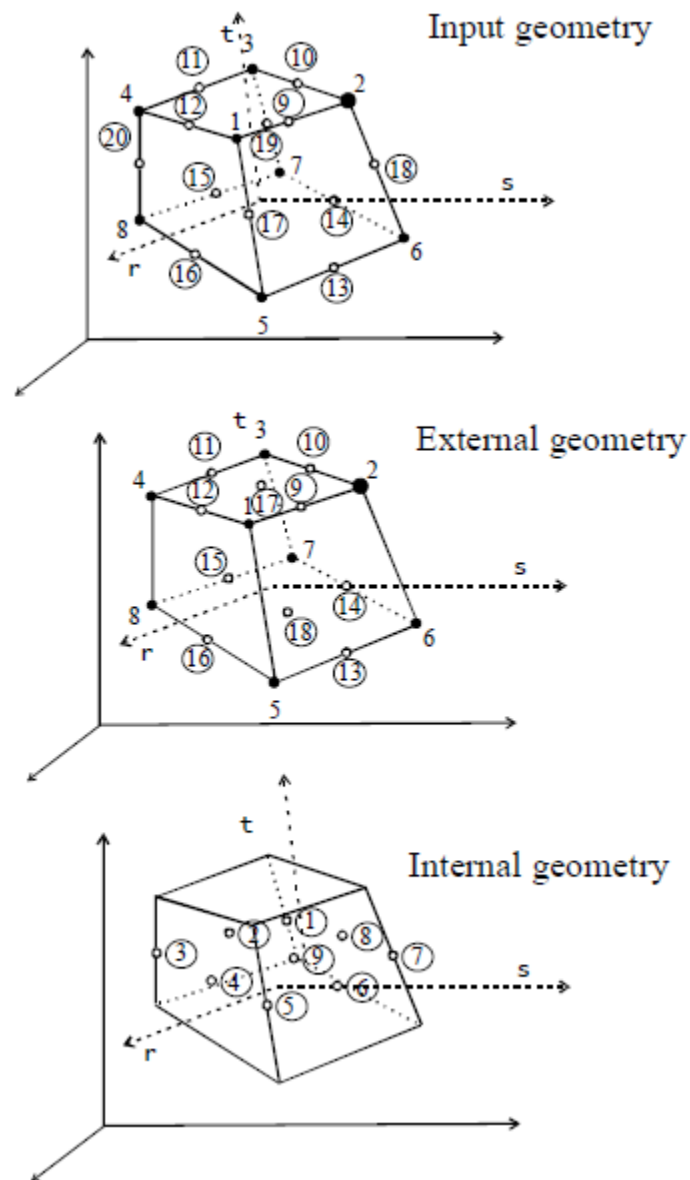


Figure 6.13: Geometry of the Ferrocement (ATENA Manual)

4. Ferrocement: The polymer modified ferrocement, used as strengthening material of 20mm thickness, is modelled as ‘3D nonlinear cementitious- 2’ (refer Figure 6.13). The properties of polymer modified ferrocement are given in Table 6.4.

Table 6.4: Properties of Polymer Modified Ferrocement for FE Modelling

S. No.	Description	Values
1.	Elastic Modulus (E) MPa	1.315×10^4
2.	Poisson's Ratio (μ)	0.150
3.	Tensile Strength (f_t) MPa	6.12
4.	Compressive Strength (f_c) MPa	31.53
5.	Specific Material Weight (γ) MN/m ³	2.286×10^{-2}
6.	Coefficient of Thermal Expansion (α)	1.200×10^{-5} 1/K

5. Wire Mesh: Galvanised square woven wire mesh of 0.49mm diameter with the centre to centre spacing of 8mm and having an ultimate tensile strength of 950 N/mm^2 was used as the reinforcement component in the polymer modified ferrocement for the strengthening of pre-damaged RCC beams. The properties of wire mesh used for FE modelling are given in Table 6.5.

Table 6.5: Properties of Wire Mesh for FE Modelling

S. No.	Description	Values
1.	Elastic Modulus (E) MPa	2.1×10^5
2.	Tensile Strength (f_t) MPa	950
3.	Coefficient of Thermal Expansion (α)	$1.200 \times 10^{-5} \text{ 1/K}$
4.	Specific Material Weight (γ) MN/m ³	7.850×10^{-2}

6.3.4 Modelling of Ferrocement Retrofitting in ATENA 3D

The strengthening component can be modelled by two different methods in ATENA-3D modelling. In the first method, the model could be created and run by creating two different construction cases as case 1 and case 2. The contribution of strengthening material has been ignored when the analysis was run for case 1. The case 1 has been specified for different loads corresponding to different pre-damaged levels of 45%, 75% and 95%. The construction case 2 was activated where the pre-described load value for case 1 had been attained. In case 2, the full strength of strengthening material was considered and analysis was made.

In the second method, the material properties are to be assigned corresponding to specific Damage-Index and the analysis could be made. In the present work, the first method was adopted, wherein; two different construction cases were made accordingly, for three different levels of pre-damaging stresses. The strengthening approach which has been adopted is presented in Figure 6.14. The combined volumes created in ATENA-3D for strengthened beam specimen is shown in Figure 6.15.

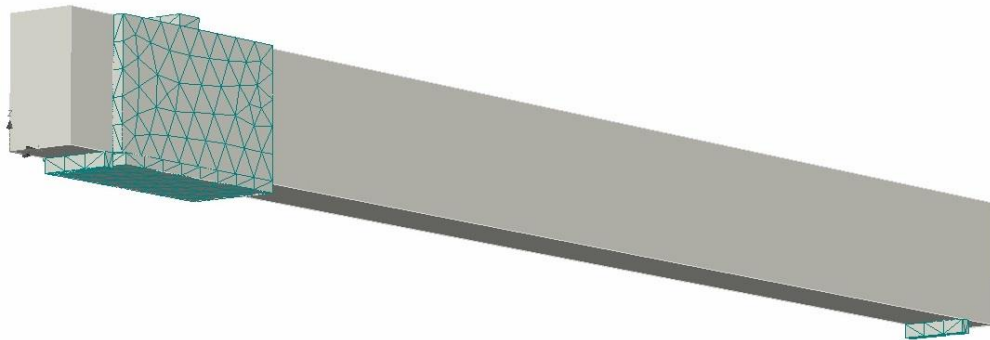


Fig. 6.14: Scheme of Modelling for Ferrocement for Strengthening of Beams

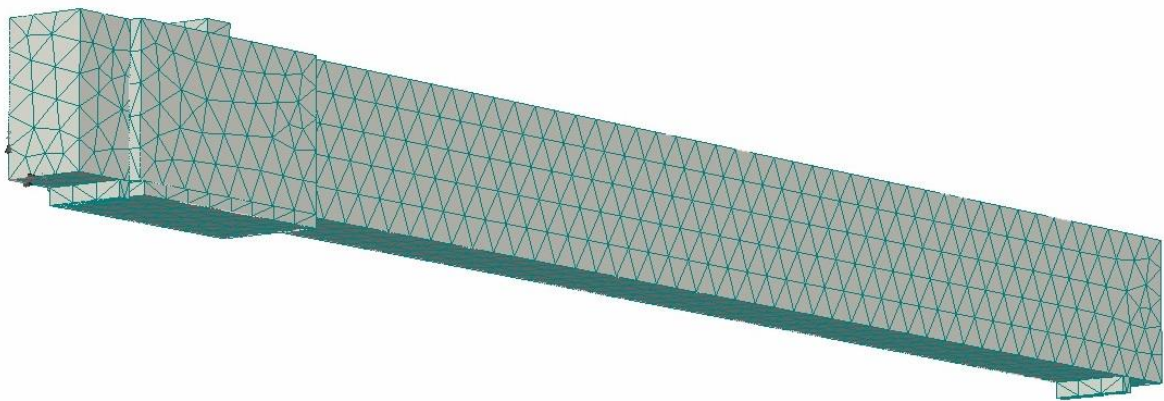


Fig. 6.15: Combined Volumes Created in ATENA-3D for Strengthened Specimen

6.3.5 Mesh Generation

The generation of FE mesh of RCC strengthened beams specimens has been done the same way as was done for control beam specimens. The only difference herein is that the mesh was also generated for ferrocement brick element. The nodes of ferrocement elements have been connected with the adjacent concrete elements to create a perfect bond between them. Figure 6.16 (a) and 6.16 (b) show the mesh generated for RCC strengthened beam elements for a/d ratio 1.0 and 3.0, respectively. Ferrocement layers meshed as brick elements in such a way that the nodes of the ferrocement layered solid elements are connected to those of adjacent concrete solid elements in order to satisfy the perfect bond assumption.

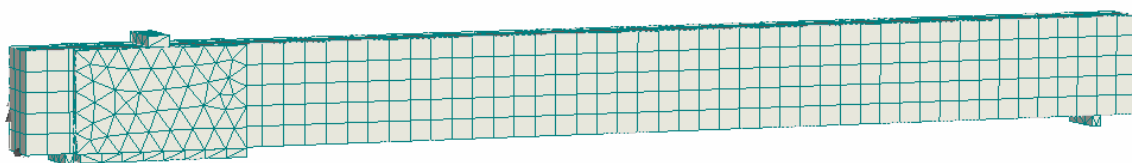


Fig. 6.16 (a): Mesh generated for RCC strengthened beam elements for a/d ratio 1.0

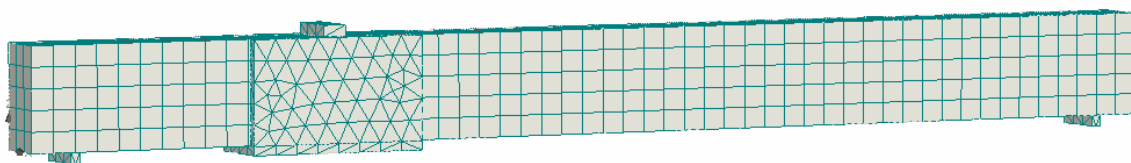


Fig. 6.16 (b): Mesh generated for RCC strengthened beam elements for a/d ratio 3.0

6.3.6 Support Conditions and Loading

Similar boundary conditions were applied for these strengthened beam specimens as were for the control specimens. The location of point load was also fixed at 198mm for $a/d = 1.0$ and at 594mm for $a/d = 3.0$. The only difference in this FE modelling as compared to control beam specimens is that the pre-application of load command, via construction case 1, was given for different per-damaged levels of beams as 45%, 75% and 95%, respectively. The support conditions of FE modelled strengthened beam specimens tested at a/d ratio 1.0 and 3.0 are shown in Figure 6.17 (a) and 6.17 (b), respectively.



Fig. 6.17 (a): Support Conditions of FE Modelled Strengthened Beam Specimens (tested at $a/d = 1.0$)



Fig. 6.17 (b): Support Conditions of FE Modelled Strengthened Beam Specimens (tested at $a/d = 3.0$)

6.3.7 Monitoring Points

The analysis of strengthened beams was monitored via the same points as for the control specimens for comparison purpose. The load is also applied through the same point and the location of deformation monitoring points was also kept the same. The only change which was made is that the monitoring point which represented the first LVDT (under load point) is located 20mm below the beam bottom surface to compensate for the thickness of ferrocement. Figure 6.18 (a) & 6.18 (b) shows the monitoring points for strengthened beams tested at $a/d = 1.0$ and 3.0, respectively.

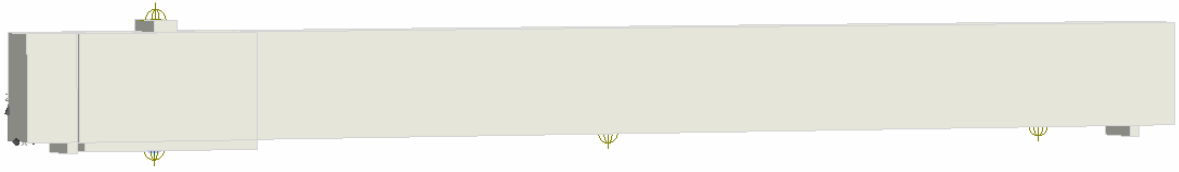


Fig. 6.18 (a): Monitoring Points of Strengthened Beam Specimens in ATENA 3D tested at $a/d = 1.0$



Fig. 6.18 (b): Monitoring Points of Strengthened Beam Specimens in ATENA 3D tested at $a/d = 3.0$

6.3.8 Finite Element Non-Linear Analysis Process for Strengthened Beams

a. Run Analysis

After pre-processing and creation of geometry for the strengthened RCC beams, the next step is to run the analysis for getting final results. The FE analysis is run in a similar manner as for the control specimens as discussed in section 6.2.10.

b. Application of Pre-damaged Load (Loading History and Solution Parameters)

Initially, the pre-defined pre-damaged load was applied over the beam specimens by defining the construction case 1. Subsequently, this case was combined with construction case 2 to analyse the beam structure. For explanation, two load cases were defined; one containing the vertical supports, and second with the prescribed deformations at the top steel plate. This section describes the definition of loading history for the analysis of shear beam. The loading history consists of load steps. Each load step is defined as a combination of load cases, which had been defined previously. Each load step contains a definition of solution parameters, which define solution methods that are to be used during the load steps. ATENA 3D contains a standard set of solution parameters. The standard solution parameters can be examined in the table of solution parameters. This table appears in the table window after highlighting the solution parameters item in the data access tab.

c. Post-processing

After Run Analysis, the next step is to post-process the model to get the load-deflection curves, internal stress-strain diagrams, crack pattern, crack propagation, text output and analysis log files. This output data of FE analysis was printed online in the same

way as was done for control specimens. The load-deflection curve for LVDT 1 (which is located under the point load) is only presented here for comparative study.

6.4 VALIDATION OF FINITE ELEMENT MODEL RESULTS

As detailed above, the RCC beams have been modelled under static load conditions and a non-linear response was observed. The FE analysis under the static incremental loads has been performed using ATENA 3D software. The analytical results for the beam specimens obtained from ATENA are compared with the experimental results. Load-deflection and cracking behaviour results were obtained from the FE analysis. The load in terms of prescribed deflection was applied on the beam specimens for a/d ratios of 1.0 & 3.0, respectively. The load-deflection behaviour for these analyses have been recorded for every step and detailed in Tables 6.6 to 6.9. The crack propagation and pattern was also studied. The FE results so obtained were compared with the experimental test results and are detailed in subsequent sections.

6.4.1 Control and Strengthened Shear Designed Beam Specimens Tested at a/d = 1.0 (DB-ad-1)

The control and strengthened beam specimens were modelled in ATENA 3D software. The load-deflection values were observed as per procedure detailed in preceding sections. Figure 6.19 to 6.22 and Table 6.6 shows the comparison of experimental v/s analytical load-deflection behaviour of shear designed beams tested at a/d ratio 1.0. Figure 6.23 show the analytical cracks and stresses in control beam specimen (DB-ad-1) at ultimate load level. From the figures and table, it can be observed that the load values obtained from ATENA show only 3.36% less value as compared to the experimental values indicating similarity in the two behaviours. The deflection values at ultimate load level obtained from FE analysis were found to be 30% less as compared with the test values. However, the overall trend of the load-deflection curves of both experimental and analytical analysis was found similar.

The analytical behaviour of strengthened beams was also compared with the tested beams and the variation in the load values were found within $\pm 5.00\%$. This shows the compatibility of analytical results with the test results. On the other hand, ATENA analysis shows lesser deflection values for the strengthened beams irrespective of initial stress level of 45%, 75% and 95%. However, the analytical load-deflection trend is found to be in line with experimental trend.

Table 6.6: Comparison of Experimental and Analytical (ATENA) Results of Shear Designed Beams Tested at a/d Ratio 1.0

S. No.	Control Beams (DB-ad-1)				45% (RDB45-ad-1)				75% (RDB75-ad-1)				95% (RDB95-ad-1)			
	Experimental		ATENA		Experimental		Analytical		Experimental		Analytical		Experimental		Analytical	
	Load (kN)	Def (mm)	Load (kN)	Def (mm)	Load (kN)	Def (mm)	Load (kN)	Def (mm)	Load (kN)	Def (mm)	Load (kN)	Def (mm)	Load (kN)	Def (mm)	Load (kN)	Def (mm)
1.	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
2.	12.00	0.23	11.70	0.09	12.00	0.66	11.64	0.39	12.00	0.40	11.64	0.39	12.00	0.74	11.64	0.47
3.	24.00	0.45	23.38	0.19	24.00	1.24	23.27	0.72	24.00	0.86	23.27	0.72	24.00	1.33	23.27	0.81
4.	36.00	0.68	37.95	0.42	36.00	1.76	36.88	1.18	36.00	1.29	36.88	1.18	36.00	1.93	36.88	1.28
5.	48.00	0.97	46.53	0.61	48.00	2.27	49.86	1.67	48.00	1.90	49.86	1.67	48.00	2.53	49.86	1.76
6.	60.00	1.37	61.20	0.98	60.00	2.77	61.68	2.14	60.00	2.40	61.68	2.14	60.00	3.14	61.68	2.23
7.	72.00	1.91	72.09	1.26	72.00	3.30	70.99	2.49	72.00	2.94	72.79	2.58	72.00	3.72	72.79	2.58
8.	84.00	2.47	85.04	1.62	84.00	3.80	84.93	2.98	84.00	3.47	84.02	2.98	84.00	4.30	84.02	2.98
9.	96.00	2.91	98.27	1.98	96.00	4.39	98.56	3.46	96.00	3.98	98.54	3.52	96.00	4.78	98.54	3.52
10.	108.00	3.34	111.70	2.36	108.00	5.04	107.10	3.76	108.00	4.66	109.10	3.83	108.00	5.33	109.10	3.86
11.	120.00	3.82	124.40	2.74	120.00	5.94	121.10	4.26	120.00	5.30	121.00	4.22	120.00	5.83	122.70	4.32
12.	132.00	4.44	135.40	3.14	132.00	7.49	131.20	4.61	132.00	6.35	133.50	4.73	132.00	6.42	136.00	4.74
13.	144.00	5.47	139.10	3.56	144.00	9.78	143.40	5.11	144.00	8.16	144.50	5.23	144.00	7.79	148.40	5.22
14.	145.68	5.90	140.80	3.93	149.14	11.89	154.00	5.73	151.41	9.98	154.00	5.87	156.00	11.20	154.30	6.15
15.	144.50	6.46	139.90	3.99	141.94	13.47	155.00	6.02	153.45	12.49	156.40	6.37	162.00	13.51	155.60	6.65
16.	137.71	7.10	137.50	4.12	132.02	14.21	154.60	6.67	144.83	13.33	153.40	6.99	163.20	14.88	155.10	6.78
17.	133.87	7.22	134.52	4.35	122.38	14.84	152.66	7.08	132.87	15.20	150.12	7.58	157.33	16.66	153.67	7.18
18.	127.86	8.53	130.00	4.89			148.37	7.62			147.70	8.18	153.33	17.32	151.47	7.48
19.							145.00	8.67			145.82	8.79			148.08	8.47
20.															146.37	8.93

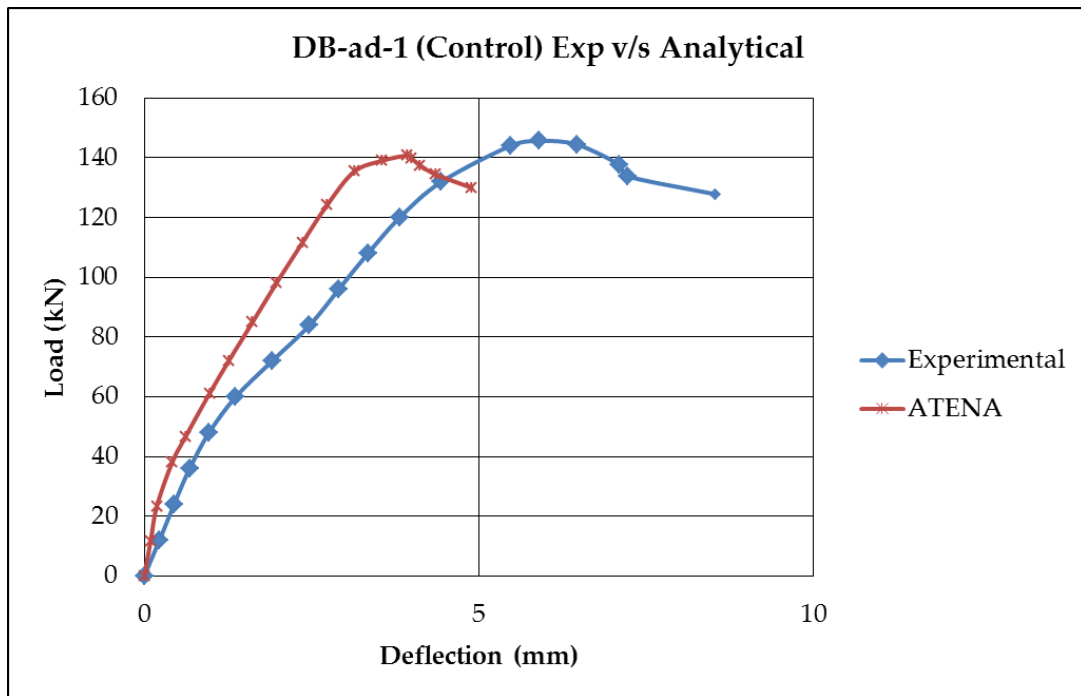


Fig. 6.19: Comparison of Experimental and ATENA 3D Load v/s Deflection Results of Shear Designed Control Beam Specimen Tested at $a/d = 1.0$ (DB-ad-1)

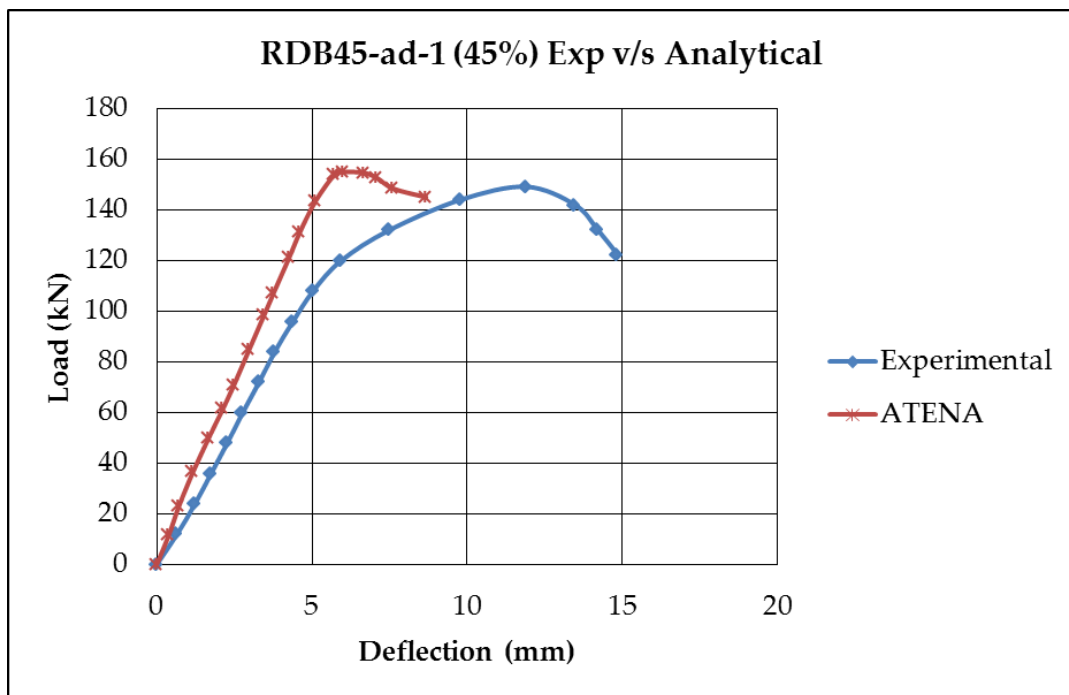


Fig. 6.20: Comparison of Experimental and ATENA 3D Load v/s Deflection Results of 45% Pre-Damaged Strengthened Shear Designed Beam Specimen Tested at $a/d = 1.0$ (RDB45-ad-1)

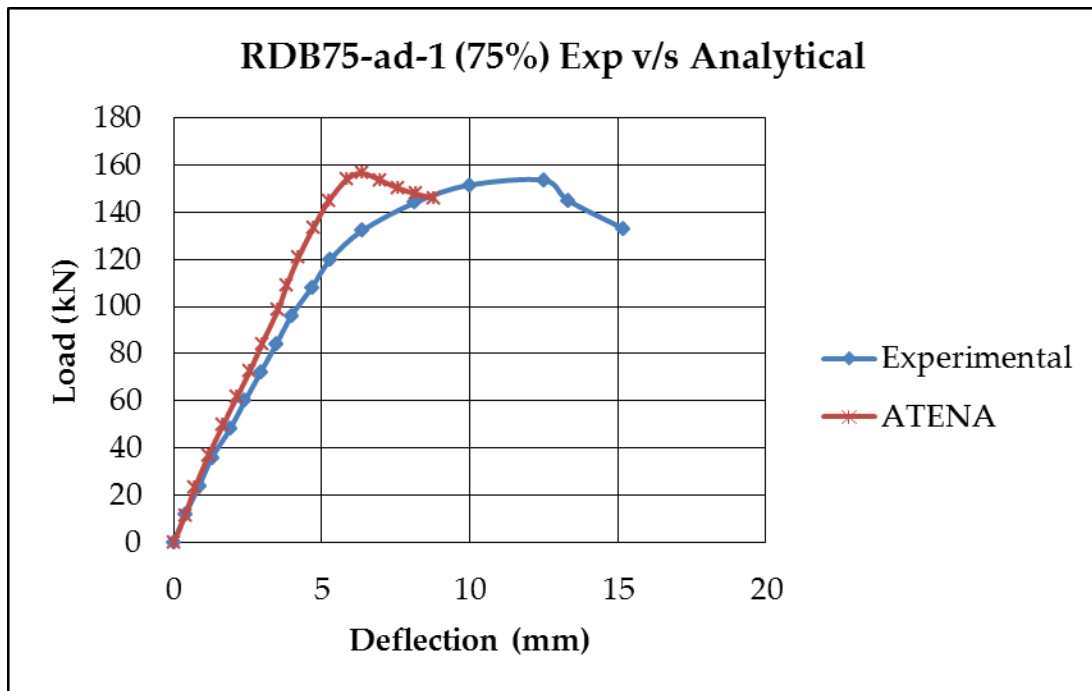


Fig. 6.21: Comparison of Experimental and ATENA 3D Load v/s Deflection Results of 75% Pre-Damaged Strengthened Shear Designed Beam Specimen Tested at $a/d = 1.0$ (RDB75-ad-1)

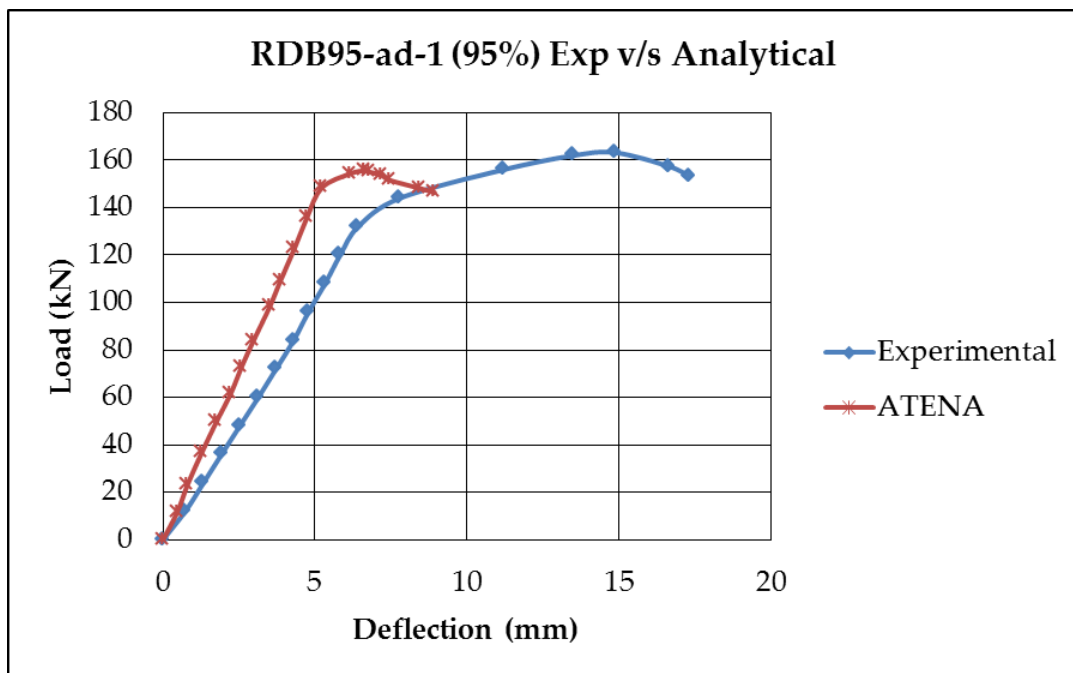


Fig. 6.22: Comparison of Experimental and ATENA 3D Load v/s Deflection Results of 95% Pre-Damaged Strengthened Shear Designed Beam Specimen Tested at $a/d = 1.0$ (RDB95-ad-1)

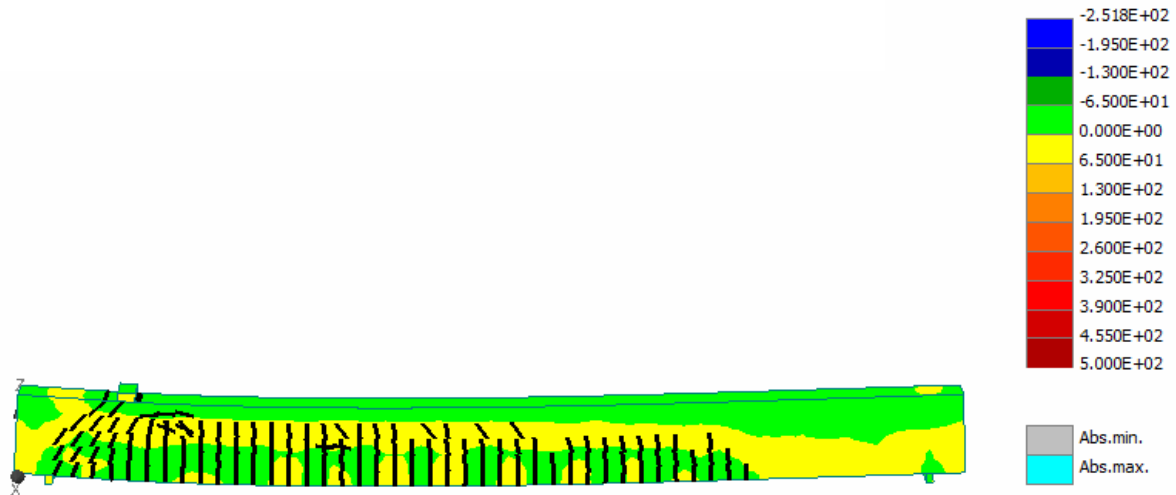


Fig. 6.23: Cracks and Stresses in Control Beam at Ultimate Load (DB-ad-1)

6.4.2 Control and Strengthened Shear Designed Beam Specimens Tested at $a/d = 3.0$ (DB-ad-3)

The test results of control shear designed beam specimens tested at a/d ratio 3.0 (DB-ad-3) were compared with the ATENA software analytical results. Figure 6.24 to 6.27 and Table 6.7 shows the comparison of experimental v/s analytical load-deflection behaviour of shear designed beams tested at a/d ratio 3.0. Figure 6.28 show the analytical cracks and stresses in control beam specimen (DB-ad-3) at ultimate load level.

From the figures and table it can be observed that the value of the ultimate load obtained by FE analysis is only 4.26% less as compared with the experimental value. The deflection values at ultimate load level obtained from FE analysis were found to be only 2.62% lower as compared with the test values, which reflect similar behaviour of control beam as observed from experimental as well as analytical results. The overall trend of the load-deflection curve of both experimental and FE analysis overlap with each other.

The variation in the analytical load values was found within $\pm 8.00\%$ when compared with the test values. This shows higher compatibility of test results with the FE analysis. The deflection values observed from ATENA analysis are lesser by $\pm 26.00\%$ as compared with test values for all initial stress levels of 45%, 75% and 95%. However, the final deflection values of FE analysis are almost equal to the experimental values. A similar trend of the load-deflection curves was found for analytical and experimental behaviour.

Table 6.7: Comparison of Experimental and Analytical (ATENA) Results of Shear Designed Beams Tested at a/d Ratio 3.0

S. No.	Control Beams (DB-ad-3)				45% (RDB45-ad-3)				75% (RDB75-ad-3)				95% (RDB-95-ad-3)			
	Experimental		ATENA		Experimental		Analytical		Experimental		Analytical		Experimental		Analytical	
	Load (kN)	Def (mm)	Load (kN)	Def (mm)	Load (kN)	Def (mm)	Load (kN)	Def (mm)	Load (kN)	Def (mm)	Load (kN)	Def (mm)	Load (kN)	Def (mm)	Load (kN)	Def (mm)
1.	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
2.	6.00	0.86	6.71	0.50	6.00	0.54	6.53	0.39	6.00	1.33	6.53	0.39	6.00	0.72	6.53	0.39
3.	12.00	1.70	15.69	1.29	12.00	1.18	13.00	1.03	12.00	2.02	12.48	1.01	12.00	1.89	12.48	1.01
4.	18.00	2.58	19.35	1.68	18.00	1.89	18.47	1.59	18.00	2.81	18.02	1.52	18.00	2.70	18.33	1.59
5.	24.00	3.47	23.25	2.21	24.00	2.73	24.15	2.12	24.00	3.69	24.68	2.18	24.00	3.76	24.11	2.16
6.	30.00	4.26	30.34	3.09	30.00	3.28	30.18	2.77	30.00	4.37	30.04	2.78	30.00	4.57	30.43	2.88
7.	36.00	5.33	37.16	4.17	36.00	3.94	36.38	3.47	36.00	5.13	36.66	3.68	36.00	5.22	36.21	3.58
8.	42.00	6.54	43.75	5.24	42.00	4.89	42.44	4.37	42.00	6.17	42.45	4.57	42.00	6.37	42.40	4.49
9.	48.00	7.49	50.13	6.33	48.00	5.88	48.38	5.26	48.00	6.99	48.33	5.36	48.00	7.26	48.33	5.36
10.	54.00	8.54	56.38	7.56	54.00	6.90	54.26	6.16	54.00	7.92	54.18	6.16	54.00	8.15	54.23	6.22
11.	60.00	9.69	59.48	9.18	60.00	7.87	60.49	7.15	60.00	8.90	60.14	7.15	60.00	9.02	60.22	7.23
12.	64.53	14.39	61.78	11.37	66.00	9.05	63.32	8.64	66.00	10.52	63.56	9.03	66.00	10.65	63.06	8.54
13.	61.04	19.50	61.39	13.97	69.27	10.09	65.00	10.52	67.74	13.03	64.63	10.62	67.69	11.07	64.56	10.53
14.			60.12	15.96	71.75	11.67	66.01	12.35	69.54	16.43	65.20	11.70	69.37	14.69	65.33	11.71
15.			58.06	17.94	68.32	14.72	65.40	13.99	70.22	17.53	65.22	12.00	70.24	17.89	65.48	12.30
16.					63.77	15.88	64.59	14.37	66.80	17.70	65.23	12.89	65.98	18.70	65.52	13.19
17.							63.03	15.06			65.00	13.77			64.44	14.18
18.							62.10	15.45			64.60	14.27			63.71	15.06
19.											64.01	14.96			63.45	15.26
20.											63.50	15.42			62.87	15.53

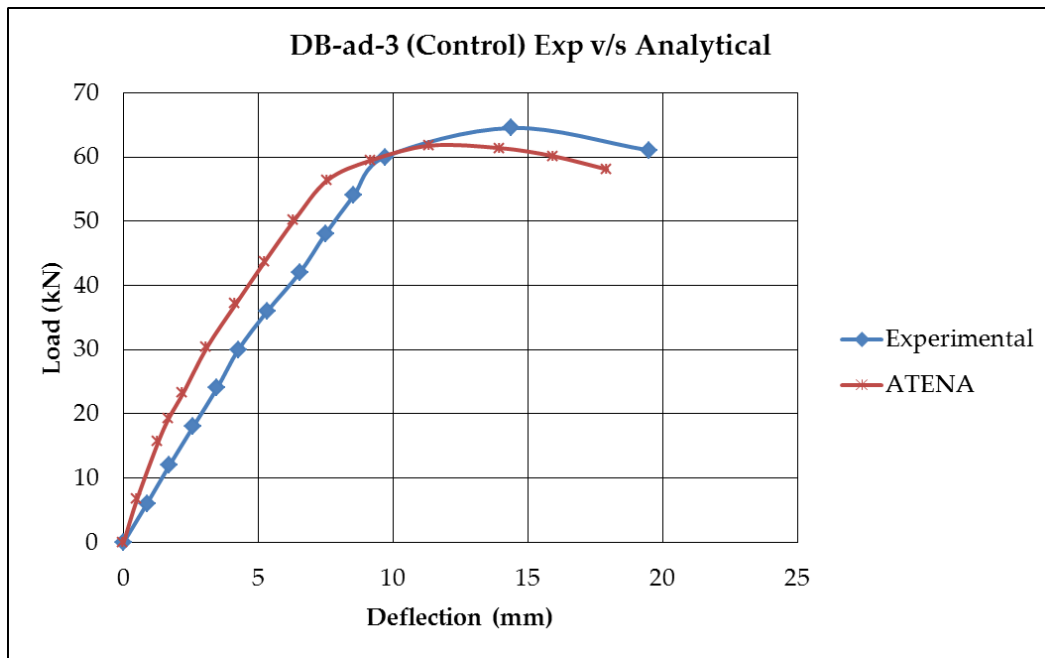


Fig. 6.24: Comparison of Experimental and ATENA 3D Load v/s Deflection Results of Shear Designed Control Beam Specimen Tested at $a/d = 3.0$ (DB-ad-3)

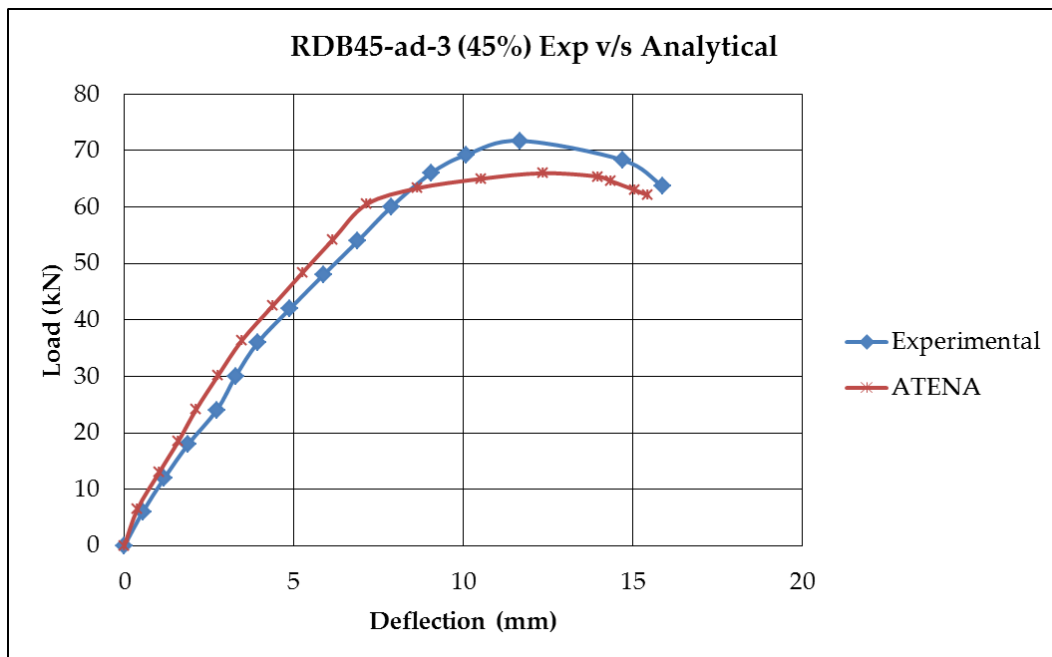


Fig. 6.25: Comparison of Experimental and ATENA 3D Load v/s Deflection Results of 45% Pre-Damaged Strengthened Shear Designed Beam Specimen Tested at $a/d = 3.0$ (RDB45-ad-3)

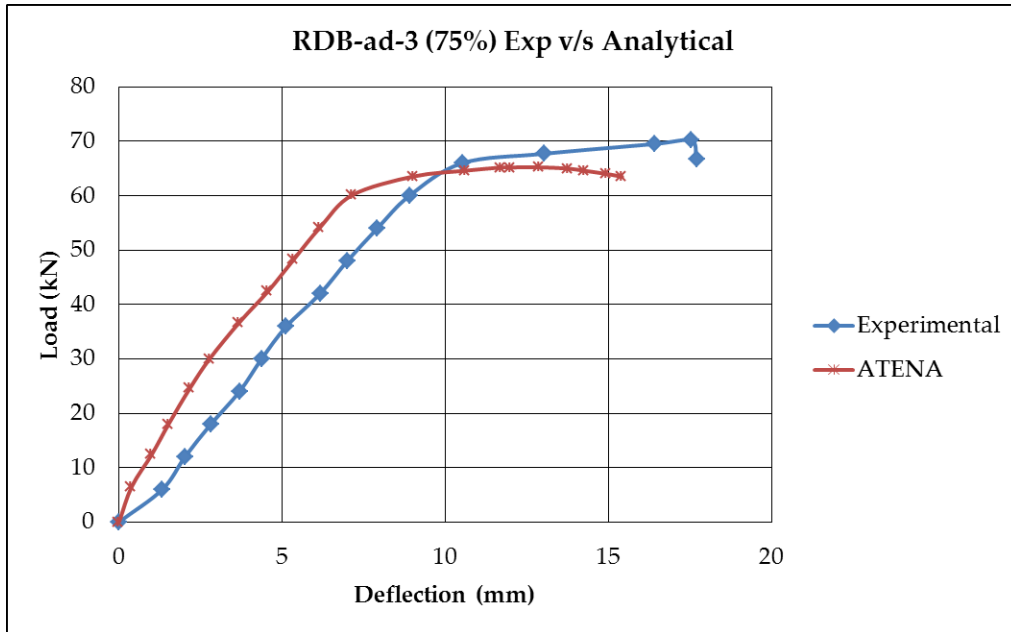


Fig. 6.26: Comparison of Experimental and ATENA 3D Load v/s Deflection Results of 75% Pre-Damaged Strengthened Shear Designed Beam Specimen Tested at $a/d = 3.0$ (RDB75-ad-3)

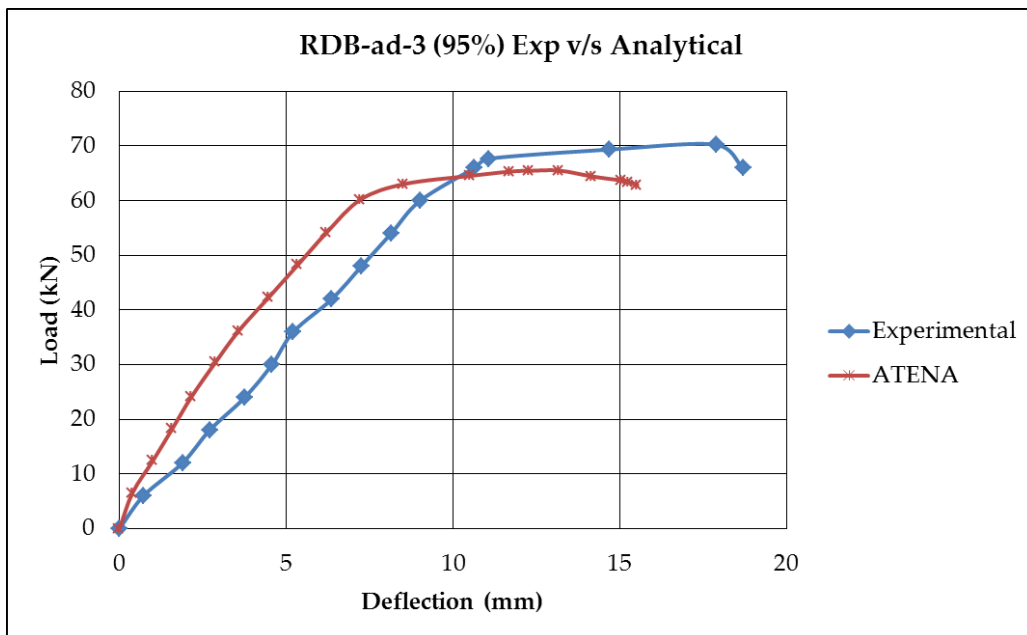


Fig. 6.27: Comparison of Experimental and ATENA 3D Load v/s Deflection Results of 95% Pre-Damaged Strengthened Shear Designed Beam Specimen Tested at $a/d = 3.0$ (RDB95-ad-3)

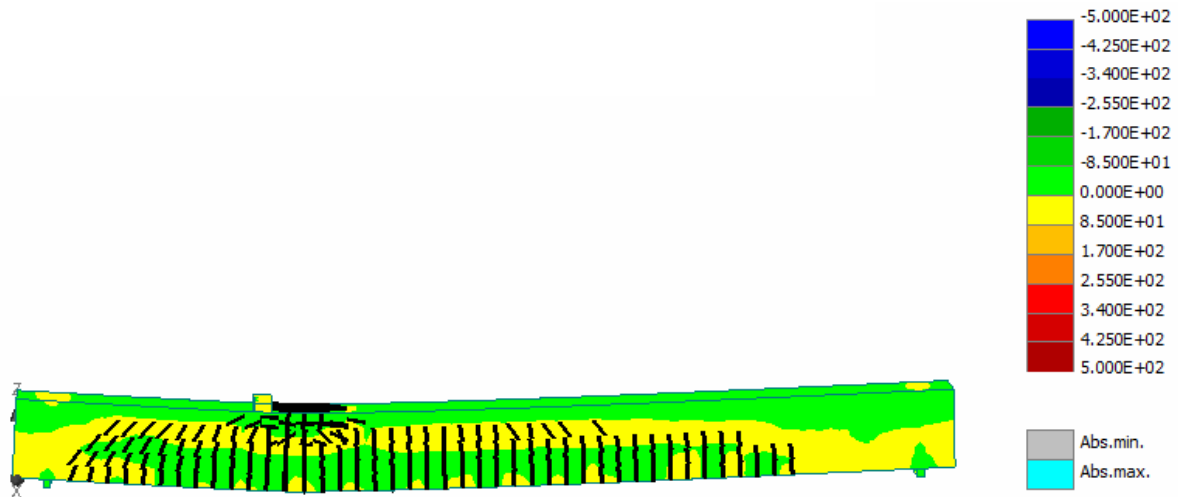


Fig. 6.28: Cracks and Stresses in Control Beam at Ultimate Load (DB-ad-3)

6.4.3 Control and Strengthened Shear Deficient Beam Specimens Tested at $a/d = 1.0$ (SDB-ad-1)

The shear deficient beam specimens were also modelled on ATENA and the results were compared with actual test results. Figure 6.29 to 6.32 and Table 6.8 shows the comparison of experimental v/s analytical load-deflection behaviour of shear deficient beams tested at a/d ratio 1.0. Figure 6.33 show the analytical cracks and stresses in control beam specimen (SDB-ad-1) at ultimate load level.

From the figures and table it can be observed that the FE analysis results of SDB-ad-1 control shear deficient beams show a similar type of variation and trend as observed for DB-ad-1 beam specimens. The load values obtained from ATENA are showing very less variation of 1.23% when compared with experimental value and the deflection values at the ultimate load level obtained from FE analysis were found to be 12.50% less as compared with the test values. The trend of the load-deflection curve is found to be in similar line with experimental curve.

The analytical result values of strengthened beams also varied in the range of $\pm 3.70\%$ which attributes the compatibility of the FE model with the actual laboratory results. However, there is a large variation in the deflection values was observed when the results of FE analysis were compared with actual test results. The actual behaviour of pre-damaged strengthened beams was compared with analytical results and it was concluded that the observed ultimate load values are almost equal and with the same trending cure, but there is variation in the deflection values at ultimate load level.

Table 6.8: Comparison of Experimental and Analytical (ATENA) Results of Shear Deficient Beams Tested at a/d Ratio 1.0

S. No.	Control Beams (SDB-ad-1)				45% (RSBD45-ad-1)				75% (RSDB75-ad-1)				95% (RSDB95-ad-1)			
	Experimental		ATENA		Experimental		Analytical		Experimental		Analytical		Experimental		Analytical	
	Load (kN)	Def (mm)	Load (kN)	Def (mm)	Load (kN)	Def (mm)	Load (kN)	Def (mm)	Load (kN)	Def (mm)	Load (kN)	Def (mm)	Load (kN)	Def (mm)	Load (kN)	Def (mm)
1.	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
2.	12.00	0.30	11.69	0.29	12.00	0.62	11.64	0.38	12.00	0.53	11.64	0.38	12.00	0.59	11.64	0.38
3.	24.00	0.53	23.36	0.49	24.00	1.13	23.26	0.69	24.00	1.14	23.26	0.73	24.00	1.17	23.26	0.76
4.	36.00	0.80	36.50	0.68	36.00	1.53	37.33	1.22	36.00	1.62	36.78	1.24	36.00	1.75	37.33	1.28
5.	48.00	1.11	49.18	0.97	48.00	2.00	48.15	1.58	48.00	2.18	50.86	1.77	48.00	2.36	48.15	1.76
6.	60.00	1.50	60.40	1.26	60.00	2.47	63.26	2.09	60.00	2.72	60.63	2.12	60.00	2.93	58.30	2.18
7.	72.00	2.02	71.58	1.55	72.00	3.00	72.50	2.42	72.00	3.21	74.76	2.58	72.00	3.48	72.50	2.74
8.	84.00	2.55	86.10	1.94	84.00	3.60	85.20	2.92	84.00	3.69	84.10	2.88	84.00	4.09	85.20	3.19
9.	96.00	3.00	96.40	2.23	96.00	4.35	99.56	3.51	96.00	4.17	96.10	3.29	96.00	4.83	95.65	3.61
10.	108.00	3.46	109.90	2.62	108.00	5.11	110.30	3.94	108.00	4.68	109.80	3.74	108.00	5.55	110.30	4.11
11.	120.00	3.98	119.80	2.91	120.00	5.81	120.30	4.32	120.00	5.18	120.10	4.11	120.00	6.32	120.30	4.49
12.	132.00	4.58	132.80	3.29	132.00	7.76	135.90	4.96	132.00	5.79	133.90	4.62	132.00	7.10	135.90	5.04
13.	144.28	6.86	139.40	3.96	144.00	9.58	147.80	5.75	144.00	7.12	143.80	5.02	144.00	8.07	143.70	5.44
14.	139.95	7.74	142.00	4.81	150.00	11.92	155.50	6.78	150.00	7.77	154.00	5.76	153.35	10.41	154.00	6.28
15.			142.50	6.00	150.92	14.05	156.50	7.24	155.20	10.45	156.20	6.26	146.25	11.91	156.50	7.41
16.			140.37	6.42	147.27	14.96	154.20	8.11	151.83	12.32	156.60	7.28	141.25	12.54	154.20	7.92
17.			138.35	7.20			150.10	8.87			153.10	8.09			149.60	8.63
18.			136.72	7.79			143.70	9.83			150.30	9.00			147.90	9.44
19.							138.30	10.94			147.80	9.71			143.20	10.73
20.							135.60	12.03			143.50	10.51			137.50	11.62
21.															136.10	12.39

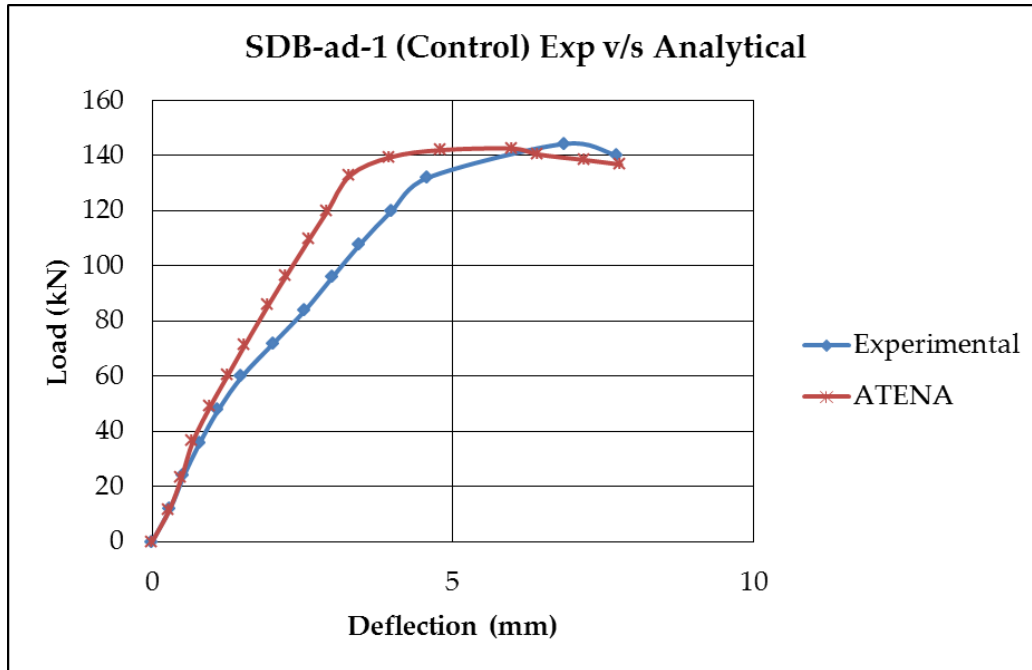


Fig. 6.29: Comparison of Experimental and ATENA 3D Load v/s Deflection Results of Shear Deficient Control Beam Specimen Tested at $a/d = 1.0$ (SDB-ad-1)

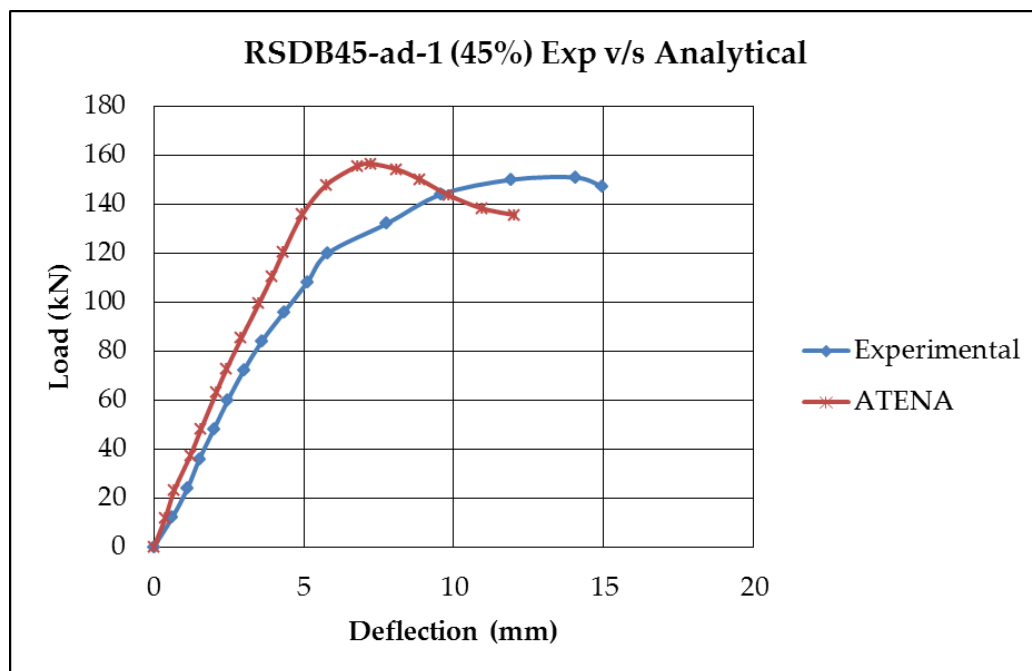


Fig. 6.30: Comparison of Experimental and ATENA 3D Load v/s Deflection Results of 45% Pre-Damaged Strengthened Shear Deficient Beam Specimen Tested at $a/d = 1.0$ (RSDB45-ad-1)

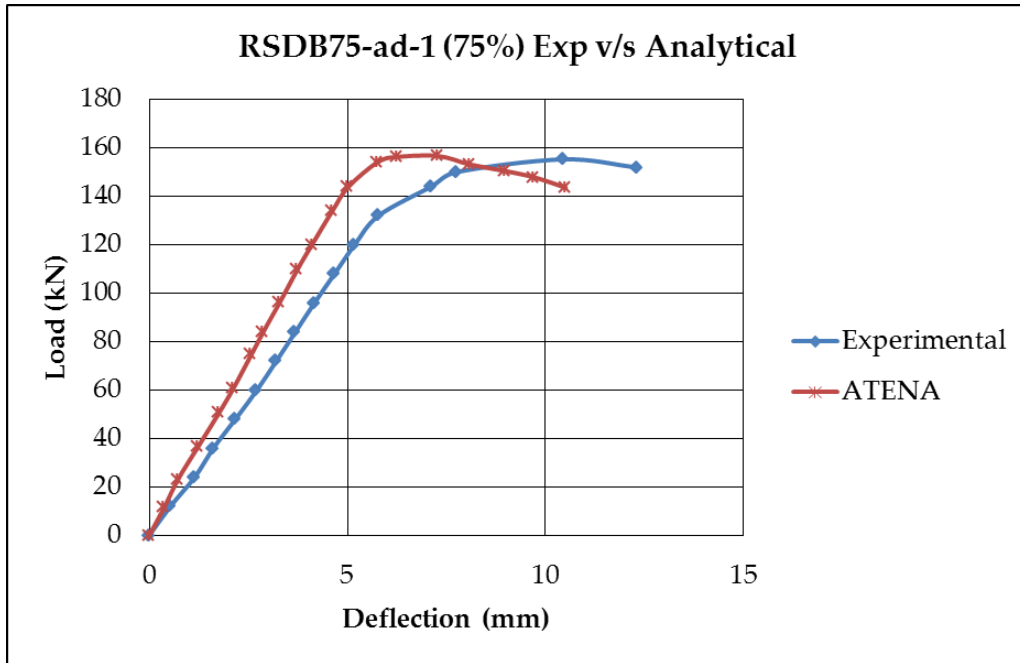


Fig. 6.31: Comparison of Experimental and ATENA 3D Load v/s Deflection Results of 75% Pre-Damaged Strengthened Shear Deficient Beam Specimen Tested at $a/d = 1.0$ (RSDB75-ad-1)

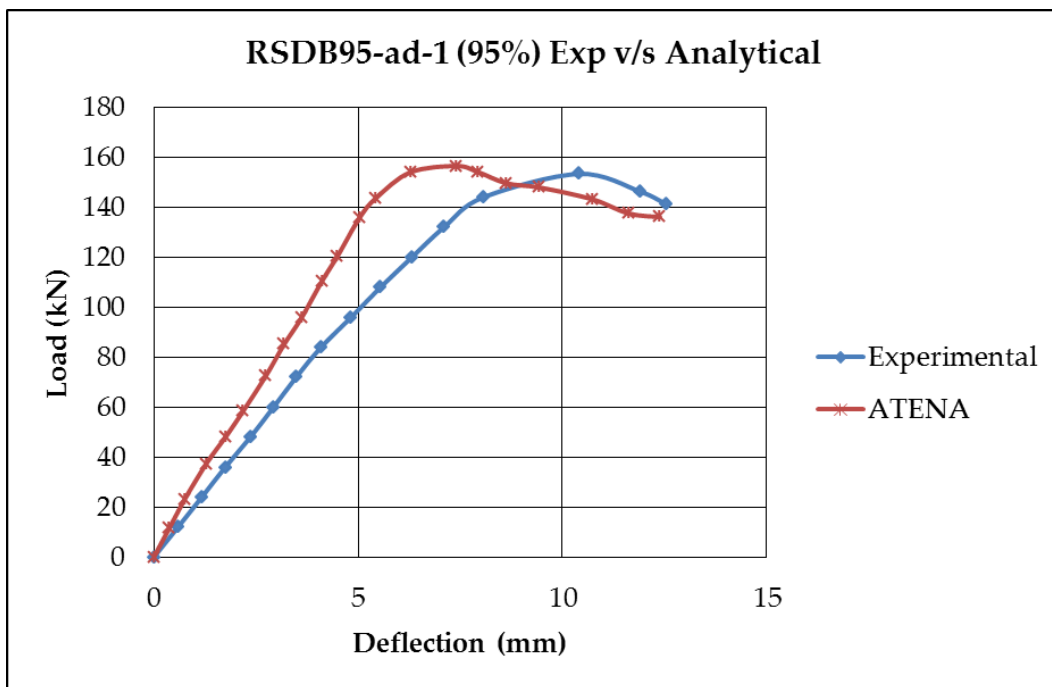


Fig. 6.32: Comparison of Experimental and ATENA 3D Load v/s Deflection Results of 95% Pre-Damaged Strengthened Shear Deficient Beam Specimen Tested at $a/d = 1.0$ (RSDB95-ad-1)

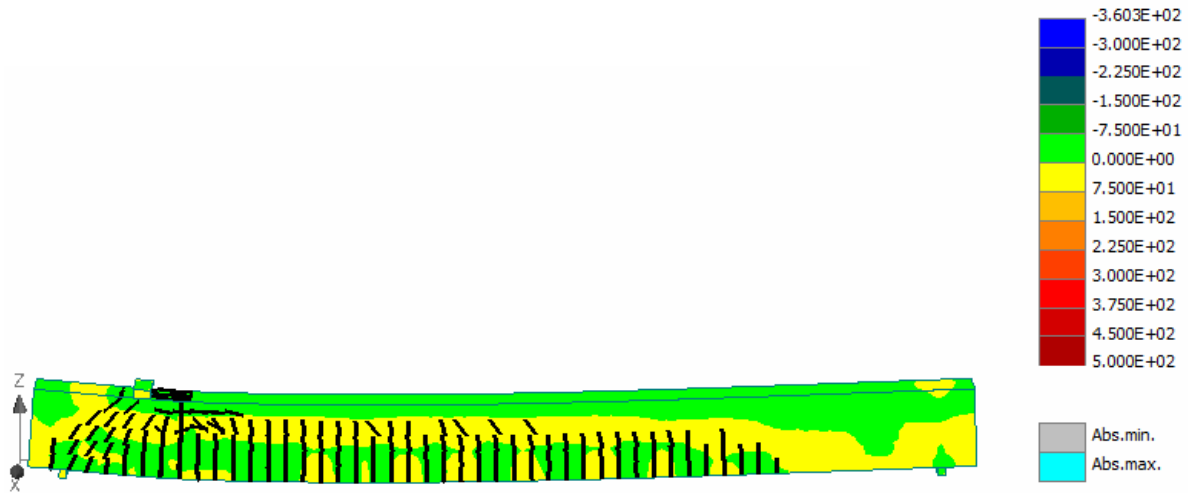


Fig. 6.33: Cracks and Stresses in Control Beam at Ultimate Load (SDB-ad-1)

6.4.4 Control and Strengthened Shear Deficient Beam Specimens Tested at $a/d = 3.0$ (SDB-ad-3)

The load-deflection curves for control shear deficient beam specimens tested at a/d ratio 3.0 (SDB-ad-3) were obtained from ATENA software analytical results and compared with experimental result values. Figure 6.34 to 6.37 and Table 6.9 shows the comparison of experimental v/s analytical load-deflection behaviour of shear deficient beams tested at a/d ratio 3.0. Figure 6.38 show the analytical cracks and stresses in control beam specimen (SDB-ad-3) at ultimate load level.

From the figures and table it was observed that there is a minor variation of 1.16% in ultimate load values. In this case, a higher deflection value of 5.87% was also observed as compared to the test values. The test results of the FE model was found similar with experimental test values and the load-deflection curves are also overlap with each other.

The strengthened beam results were also showing a similar trend with a variation of $\pm 4.9\%$ in ultimate load values. The variation in deflection value is also found less as $\pm 15.9\%$ when compared to the test results of strengthened beam specimens. The final deflection observed from ATENA analysis is found almost equal as compared with the experimental values. It reflects the compatibility of finite element analysis with the actual experimental results. The trend of the load-deflection curves of both experimental and analytical analysis was also observed similar for the strengthened beam specimens.

Table 6.9: Comparison of Experimental and Analytical (ATENA) Results of Shear Deficient Beams Tested at a/d Ratio 3.0

S. No.	Control Beams (RSDB-ad-3)				45% (RSDB45-ad-3)				75% (RSDB75-ad-3)				95% (RSDB95-ad-3)			
	Experimental		ATENA		Experimental		Analytical		Experimental		Analytical		Experimental		Analytical	
	Load (kN)	Def (mm)	Load (kN)	Def (mm)	Load (kN)	Def (mm)	Load (kN)	Def (mm)	Load (kN)	Def (mm)	Load (kN)	Def (mm)	Load (kN)	Def (mm)	Load (kN)	Def (mm)
1.	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
2.	6.00	0.60	7.71	0.46	6.00	1.00	6.53	0.43	6.00	0.95	6.53	0.43	6.00	1.13	6.53	0.43
3.	12.00	1.21	15.67	1.25	12.00	1.76	13.00	1.09	12.00	1.60	12.48	1.06	12.00	1.93	12.48	1.06
4.	18.00	1.87	18.67	1.56	18.00	2.66	18.49	1.68	18.00	2.52	18.01	1.58	18.00	2.85	18.34	1.68
5.	24.00	2.58	23.19	2.10	24.00	3.49	24.15	2.37	24.00	3.37	24.59	2.47	24.00	3.78	24.93	2.48
6.	30.00	3.31	30.36	3.00	30.00	4.21	30.24	3.37	30.00	4.01	30.78	3.27	30.00	4.39	30.33	3.37
7.	36.00	4.24	37.15	4.00	36.00	5.24	36.64	4.27	36.00	4.73	36.54	4.17	36.00	5.49	36.40	4.27
8.	42.00	5.26	43.71	5.00	42.00	6.33	42.46	5.16	42.00	5.76	42.26	5.06	42.00	6.37	42.42	5.22
9.	48.00	6.37	50.08	6.00	48.00	7.32	48.40	6.06	48.00	6.80	49.40	6.23	48.00	7.40	48.27	6.06
10.	54.00	7.84	56.33	7.00	54.00	8.37	54.29	7.05	54.00	7.88	55.08	7.06	54.00	8.47	54.16	7.03
11.	59.55	10.37	58.31	7.99	60.00	9.45	60.46	8.04	60.00	9.25	61.57	8.22	59.50	11.00	60.36	8.34
12.	57.99	11.92	59.95	9.98	66.00	11.28	62.60	9.03	62.55	9.95	63.60	9.13	62.24	12.52	62.98	9.82
13.	56.04	12.85	60.24	10.98	67.20	14.15	63.77	10.42	64.05	13.76	64.54	10.52	57.92	13.84	64.34	10.91
14.			59.24	11.22	66.40	15.77	65.00	11.02	62.20	14.44	65.10	11.22			65.28	11.95
15.			58.97	11.69	63.19	16.59	65.78	11.90			65.67	11.92			64.93	12.40
16.							65.39	12.18			64.42	12.53			63.64	12.92
17.							64.82	12.74			63.71	12.92			62.81	13.44
18.							62.86	13.37			61.76	13.65			61.47	14.09
19.																
20.																

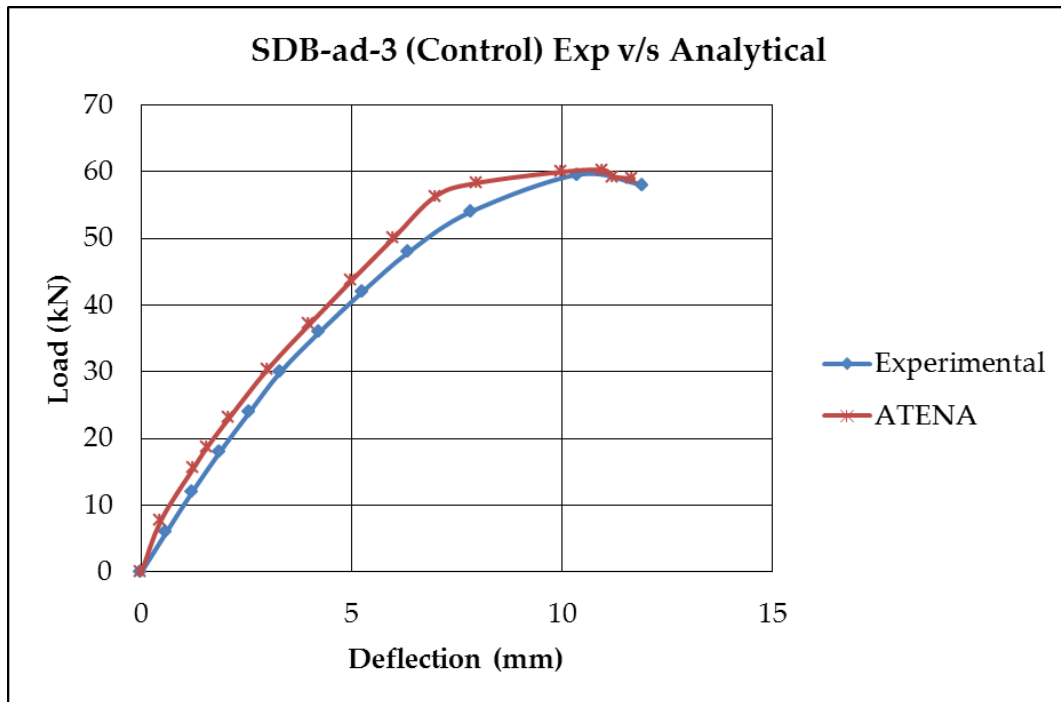


Fig. 6.34: Comparison of Experimental and ATENA 3D Load v/s Deflection Results of Shear Deficient Control Beam Specimen Tested at $a/d = 3.0$ (SDB-ad-3)

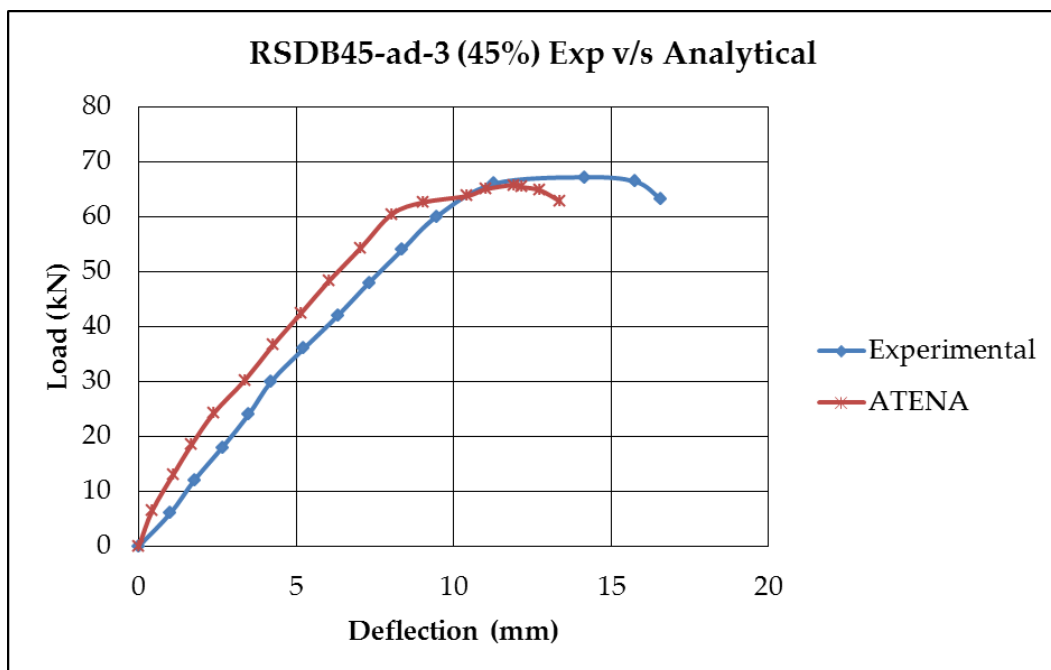


Fig. 6.35: Comparison of Experimental and ATENA 3D Load v/s Deflection Results of 45% Pre-Damaged Strengthened Shear Deficient Beam Specimen Tested at $a/d = 3.0$ (RSDB45-ad-3)

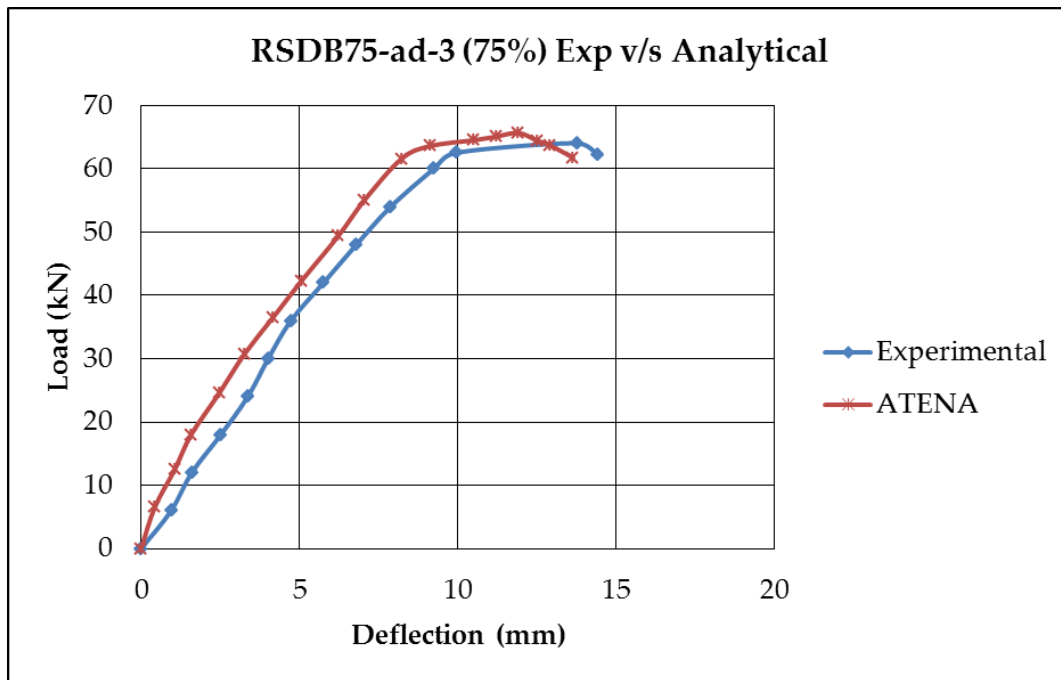


Fig. 6.36: Comparison of Experimental and ATENA 3D Load v/s Deflection Results of 75% Pre-Damaged Strengthened Shear Deficient Beam Specimen Tested at $a/d = 3.0$ (RSDB75-ad-3)

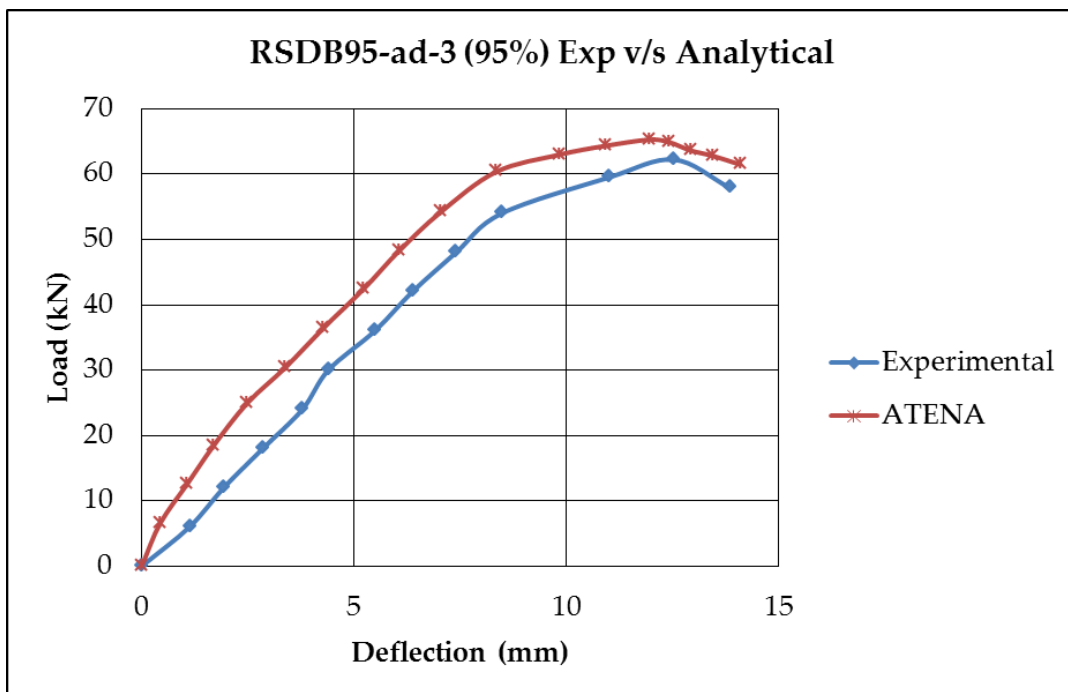


Fig. 6.37: Comparison of Experimental and ATENA 3D Load v/s Deflection Results of 95% Pre-Damaged Strengthened Shear Deficient Beam Specimen Tested at $a/d = 3.0$ (RSDB95-ad-3)

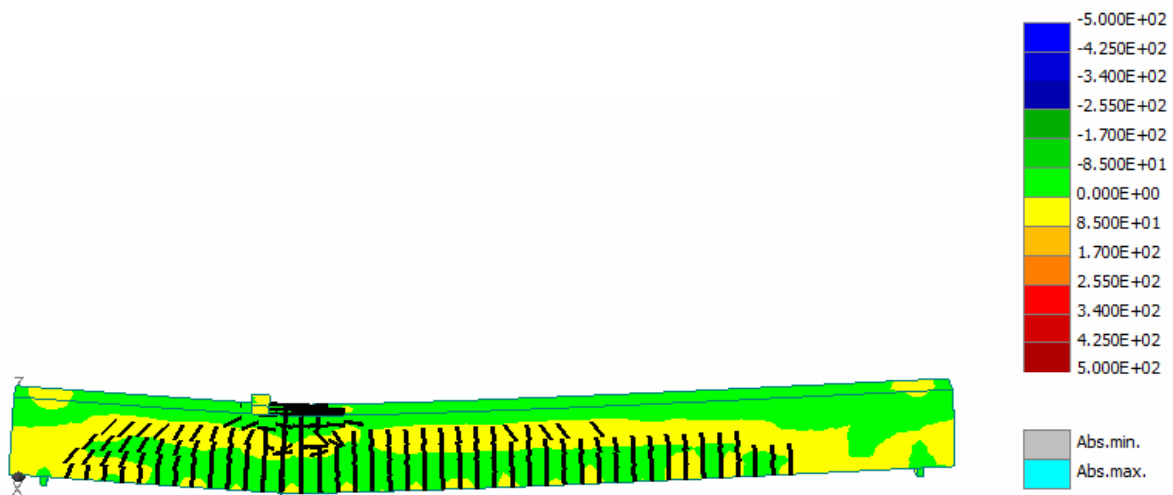


Fig. 6.38: Cracks and Stresses in Control Beam at Ultimate Load (SDB-ad-3)

6.5 SUMMARY

The test results obtained experimentally have been compared analytically by using Finite Element based software ATENA 3D and detailed in Table 6.10. The ultimate load results of both control as well as strengthened beams have been compared for both a/d ratios of 1.0 and 3.0. A total of 16 FE models have been developed for different variables as detailed in previous sections and a summary of the reloading test results is presented in Table 6.10. The load-deflection curves have been compared and it is found that the results obtained by ATENA follow a similar trend to that of the experimental values. As can be observed from Table 6.10, the ultimate load values of control as well as strengthened beams obtained by ATENA for both a/d ratios are within $\pm 8\%$ which reflects the compatibility of FE software with the actual experimental results. The load-deflection curves obtained from FE analysis for shear designed and shear deficient beams follow a trend which is very similar to the experimental curves when tested at a/d ratio of 3.0. However, in the case of beams tested at a/d ratio 1.0, there was variation in the curves. This can be attributed to higher stiffness in case of FE analysis as compared to the experimental analysis.

The crack pattern of beam specimens obtained from FE analysis was also compared with the actual pattern and it is found that the cracking initiated in a manner similar to the experimentally tested beams and the location of cracks also coincided with the actual trend.

Thus, it can be summarized and concluded that the finite element modelling results validate not only the experimental result values but also the load-deflection behaviour of control as well as strengthened beam specimens.

Several factors may cause the higher stiffness in the finite element models. The bond between the concrete and steel reinforcing is assumed to be perfect (no slip) in the finite element analyses, but for the actual beams the assumption would not be true, as slip does occur, therefore the composite action between the concrete and steel reinforcement is lost in the actual beams. Also the microcracks produced by drying shrinkage and handling are present in the concrete to some degree. These would reduce the stiffness of the actual beams, while the finite element models does not include microcracks due to factors that are not incorporated into the models. After the initiation of cracks, the beam stiffness was reduced and the linear load-deflection behaviour ended when the internal steel reinforcement began to yield.

The second reason is due to low value of fracture energy of concrete used in the testing than that the standard values used in the software. Also, the difference is due to non-adjustment in the seating load that was applied to the samples during the experimental study.

Table 6.10: Reloading Test Results of Pre-Damaged Strengthened Beams

Beam Description	Exp. Ultimate Load (kN)	ATENA Ultimate Load (kN)	Percentage Difference in Ultimate Load	Exp. Deflection of 1st LVDT at Ultimate Load (mm)	ATENA Deflection of 1st LVDT at Ultimate Load (mm)	Percentage Difference in Deflection at Ultimate Load Level
DB-ad-1	145.68	140.80	-3.36%	5.899	3.93	-33.38%
RDB45-ad-1	149.14	155.00	+3.93%	11.891	6.02	-49.37%
RDB75-ad-1	153.45	156.40	+1.92%	12.494	6.37	-49.02%
RDB95-ad-1	163.20	155.60	-4.66%	14.882	6.65	-55.32%
DB-ad-3	64.53	61.78	-4.26%	11.674	11.37	-2.62%
RDB45-ad-3	71.75	66.01	-8.00%	14.564	12.35	-15.20%
RDB75-ad-3	70.22	65.23	-7.11%	17.534	12.89	-26.49%
RDB95-ad-3	70.24	65.52	-6.72%	17.887	13.19	-26.26%
SDB-ad-1	144.28	142.50	-1.23%	6.857	6.00	-12.50%
RSDB45-ad-1	150.92	156.50	+3.70%	14.048	7.24	-48.46%
RSDB75-ad-1	155.20	156.60	+0.90%	10.450	7.28	-30.33%
RSDB95-ad-1	153.35	156.50	+2.05%	10.412	7.41	-28.83%
SDB-ad-3	59.55	60.24	+1.16%	10.371	10.98	5.87%
RSDB45-ad-3	67.20	65.78	-2.11%	14.146	11.90	-15.88%
RSDB75-ad-3	64.05	65.67	+2.53%	13.763	11.92	-13.39%
RSDB95-ad-3	62.24	65.28	+4.88%	12.523	11.95	-4.58%

7.1 GENERAL

In the research work carried out and presented herein, firstly, a pilot study was conducted to obtain the optimum percentage of polymer addition for mortar matrix of ferrocement to get the maximum strength results. Further, this polymer modified mortar (PMM) was used as the mortar matrix of ferrocement and the effect of two and three layers of wire mesh on the properties of polymer-modified ferrocement (PMF) was gauged. From the test results of the pilot study, the best performing PMF, in terms of strength and other properties was further used to strengthen the pre-damaged RCC beams. The three-layered polymer modified ferrocement (PMF) with 15% SBR polymer was used to strengthen the pre-damaged shear designed and shear deficient beams. The effect of this strengthening technique on the various parameters such as; deflection, cracking, mode of failure and ultimate strength were obtained and analysed. Finite Element (FE) models were also developed using ATENA 3D software, for control as well as for PMF strengthened beams to validate the experimental results. All the research objectives have been achieved by this study. Based on the experimental and analytical results, the major conclusions are provided below: -

7.2 RESULTS OF PILOT STUDY

1. Polymer modified mortars (PMMs) shows less requirement of water as compared to control mix to maintain the fixed flow value of $105 \pm 5\%$. Thus it is concluded that addition of both types of polymers enhance the workability of modified mortars.
2. The compressive and flexural strength values of polymer modified mortar peaked with 15% addition of SBR polymer, for a constant flow value.
3. The flexural strength and tensile strength of both two and three-layered polymer modified ferrocement (PMF) increases with increase in percentage of polymer addition and attained maximum values with 15% addition of SBR or VAE polymers. However, the emulsion based polymer (SBR) exhibit higher strength values as compared to the powder based polymer (VAE), indicating that SBR is better suited for use where higher strength is the criteria.
4. Deflection and elongation test results show that the addition of polymers enhances the ductility of ferrocement as compared to unmodified ferrocement.

5. Strength, deflection and elongation properties of 3 layered polymer modified ferrocement were found to be higher as compared to the 2 layered PMF.

Thus, from the pilot study it can be concluded that 15% SBR addition is the optimum percentage level of polymer, with three layers wire mesh, to attain a higher efficient PMF and could be used to strengthen the pre-damaged RCC beams. The numbers of wire mesh layers were also restricted to three keeping in view the practical usage of PMF as strengthening material for structural elements.

7.3 RESULTS OF EXPERIMENTAL WORK

1. PMF is fully effective to restore and enhance the original strength of initially stressed beams even after 95% damage.
2. All the strengthened beams did show a complete restoration of the original strength irrespective of stirrups spacing, the level of initial damage, and a/d ratios.
3. PMF jacketing causes delay in the direct shear failure of beams and apparently increase the contribution of stirrups to resist more loads. This technique is found to be more efficient in case of shear deficient beams.
4. The PMF strengthening technique increases the ductility of pre-damaged beams and caused delay in the shear failure by resisting and distributing the applied loads. The beams behaved more elastically, and deflection behaviour also improved after strengthening.
5. The deflection of beams improves with PMF jacketing and the beams showed lesser deformation at any particular load level. Further, the deformation of strengthened beams was also prolonged at ultimate failure level and the strengthened beams shows higher deflection values as compared to the control beams at ultimate failure level.
6. The deflection properties of beams were improved after strengthening and the comparative improvement is more in case of beams tested at a/d ratio 1.
7. The mode of failure shifted for shear failure to flexural-shear failure after the application of PMF jacketing, indicating better performance.
8. The strengthened beams displays less number of cracks as compared to the corresponding beams without strengthening, as the rate of crack development reduced due to PMF strengthening.
9. No bond failure of jacketing was observed, which reflects proper bonding and compatibility of polymer modified ferrocement with the concrete structures.

10. The optimum addition level of SBR polymers was found to be 15% with three layers wire mesh and the same has been effectively used to strengthen the pre-damaged RCC beams.

7.4 VALIDATION OF TEST RESULTS WITH ATENA-3D

1. The ultimate load values of controlled as well as strengthened beams obtained using ATENA software FE analysis, for both a/d ratios, are within $\pm 8\%$.
2. The cracking pattern of beam specimens obtained from FE analysis showed that the beams cracked in a similar manner and the location of cracks also coincided with the actual trend obtained experimentally.
3. The load-deflection curves obtained from FE analysis for shear designed and shear deficient beams follow a trend which is very similar to the experimental curves when tested at a/d ratio of 1.0. However, in the case of beams tested at a/d ratio 3.0, there was variation in the curves which could be attributed to higher stiffness values when the analysis was made using ATENA software, which further reduced the analytical deflection values.

Thus, from the FE analysis results it can be concluded that the finite element modelling results validate the experimental result values and behaviour of the control as well as of the strengthened RCC beams.

7.5 FUTURE SCOPE OF WORK

1. Analytical derivations and analytical evaluation of the shear behaviour, coupled to the ferrocement needs to be explored and design procedure needs to develop.
2. The measurements of strain fields needs to explore to predict more accurate behaviour of beams in shear.
3. Different bonding agents or other effective way of strengthening may be studied.
4. In this study, only a length of '2d' was strengthened with PMF. Study can be proposed by increasing the strengthen length.
5. In some beams all sides may not be exposed due to monolithic casting with slab. Extent of increase in strength in such cases may be found in further study. Similarly, optimum size of wire mesh may be found in further study.
6. The interfacial bond behaviour between concrete substrate and strengthening materials may be studied.

RESEARCH PUBLICATIONS FROM THIS WORK

SCI Journal Publications

1. Ghai, R., Bansal, P. P. and Kumar, M., (2016), “Mechanical properties of styrene-butadiene-rubber latex (SBR) and vinyl-acetate-ethylene (VAE) polymer modified ferrocement (PMF),” *Journal of Polymer Materials (IF 0.409)*, vol. 33, no. 1, pp. 111–126.
2. Ghai, R., Bansal, P. P. and Kumar, M., (2018), “Strengthening of RCC Beams in Shear by Using SBR Polymer-Modified Ferrocement Jacketing Technique” *Advances in Civil Engineering, Hindawi (IF 1.104)*, Vol. 2018, Article ID 4028186, 16 pages <https://doi.org/10.1155/2018/4028186>.

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