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Research Article

Strengthening of RCC Beams in Shear by Using SBR Polymer-Modified Ferrocement Jacketing Technique

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There is a common phenomenon of shear failure in RCC beams, especially in old buildings and bridges. Any possible strengthening of such beams is needed to be explored that could strengthen and make them fit for serviceable conditions. The present research has been made to determine the performance of predamaged beams strengthened with three-layered wire mesh polymer-modified ferrocement (PMF) with 15% styrene-butadiene-rubber latex (SBR) polymer. Forty-eight shear-designed and shear-deficient real-size beams were used in this experimental work. Ultimate shear load-carrying capacity of control beams was found at two different shear-span (a/d) ratios 1 and 3. The sets of remaining beams were loaded with different predetermined damage levels of 45%, 75%, and 95% of the ultimate load values and then strengthened with 20 mm thick PMF. The strengthened beams were then again tested for ultimate load-carrying capacity by conducting the shear load test at $a/d = 1$ and 3. As a result, the PMF-strengthened beams showed restoration and enhancement of ultimate shear load-carrying capacity by 5.90% to 12.03%. The ductility of strengthened beams was improved, and hence, the corresponding deflections were prolonged. On the other hand, the cracking pattern of PMF-strengthened beams was also improved remarkably.

1. Introduction

With the passage of time, many of 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 strength and the 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 technique depends on the type, cause, and nature of distress to be addressed.

All RCC elements are designed to fail in a ductile manner by making suitable detailing of reinforcement. During an earthquake, a sudden catastrophic failure can occur due to increased shear loads [1]. In reinforced concrete beams, the shear deficiencies pop up due to many reasons such as insufficient shear reinforcement, reduction in the steel area, increasing service loads, poor workmanship, and design faults. Shear strength of many existing structures might be deficient for present needs and requires strengthening to sustain and to 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 of developing an economical strengthening solution, which helps to restore the desired shear strength of beams.

In the recent years, the usage of different advanced materials such as ferrocement, glass fibre-reinforced polymer (GFRP), fibre-reinforced polymer (FRP), carbon fibre-reinforced polymer (CFRP), and steel plate jacketing has increased for retrofitting and strengthening of concrete structures. These materials have excellent properties such as high strength, light weight, and corrosion resistance abilities. Some researchers have explored the effect of various advanced composite bonding materials as

well as their orientation on the flexure and shear strength properties of retrofitted beams [2–15]. Many of these materials showed some flaws or shortcomings in terms of strength, cost, availability, and applicability. Hence, out of these available material options, the ferrocement has gained popularity and becomes the major structural material for strengthening and retrofitting as well for construction, especially in earthquake-prone areas because of its excellent ductility, toughness, availability, and other properties. Researchers have come to the conclusion that ferrocement is quite compatible with the existing concrete structures; apart from this, it is easy to apply [16–23].

Conventional ferrocement made with cement mortar matrix showed some deficiencies and got cracks under loads even much smaller than the ultimate loads, leading to reduced life [24–27]. The corrosion of wire mesh is also one of the primary reasons for the deterioration of ferrocement which is attributed to the permeability of cement mortar [28, 29]. To overcome these deficiencies of conventional ferrocement; it is necessary to enhance 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 [30–39]. It further imparts some outstanding properties like resistance to corrosion, higher strength-to-weight ratio, better ductility, and tensile strength as compared to conventional ferrocement [40–44]. There are various types of polymers available worldwide, but the studies have shown that the addition of styrene-butadiene-rubber latex (SBR) polymer is very efficient in improving the properties of conventional mortar and ferrocement [45–49]. Polymer-modified ferrocement is gaining popularity in developing countries because of its excellent properties and as an economical repair alternative to the expensive process of reconstruction. Details of PMF are simple to follow and easy to execute even by local skilled workers and the ingredients being readily available [50–54]. Further developments in the field of polymer-modified ferrocement (PMF) can make a drastic improvement in the area of composite rehabilitation and strengthening of existing structures.

In the past, a few experimental studies had been carried out to gauge the effect of the high-performance ferrocement strengthening technique on RCC beams. Kumar and Vidivelli [50, 51] investigated the use of acrylic latex and styrene-butadiene-rubber latex polymer-modified ferrocement to strengthen the RCC beams in flexure. Their test results showed that the strengthened beams exhibited 79% to 85% enhancement in their general performance after the usage of polymer ferrocement jacketing, having 15% of polymer and 5% volume of wire mesh. A similar study was made by Liao and Fang [52], and they found that RC beams strengthened with high-performance ferrocement showed higher ultimate load-carrying capacities and minor cracks as compared to controlled beams. Hughes and Evbuomwan [53] also used the polymer-modified ferrocement under the soffit of beams to strengthen the beams in flexure. They resulted that the ductility and ultimate flexural load-carrying capacity of beams were increased after strengthening, without any bond failure. Zhao et al. [54] used the polymer mortar and steel wire to strengthen the beams in shear. These strengthened beams showed the delay in crack development and improvement in the ultimate load capacity by 57% as compared to controlled beams. They also concluded that polymer mortar showed good bonding properties with the concrete members.

It is concluded that the application of polymer-modified ferrocement as an outer strengthening material is a viable technology for improving the structural performance of RCC beams in flexure. At present, there is no such research work recorded to study the effects of polymer-modified ferrocement to strengthen the beams in shear. It is the need of the hour to explore the utility of PMF as a strengthening material in the specified domain of shear. Many factors impart shear strength of RCC beams like a/d ratio, spacing of stirrups, and grade of concrete. In this experimental work, two variable factors as a/d ratio and spacing of stirrups are chosen to find their effect on strength, deflection, and cracking pattern of predamaged beams strengthened with PMF. Another study done by the authors showed that the PMF modified with 15% of SBR and three layers of square woven steel wire mesh had better flexural and tensile strength properties as compared to other compositions [45]. Therefore, three-layered wire mesh polymer-modified ferrocement with an optimum percentage of 15% SBR has been used in this experimental work to strengthen the RCC beams having three different levels of predamages.

2. Materials and Methods

A preliminary study has been done to determine the properties of ingredients required for this experimental work.

2.1. Materials. Portland pozzolana cement (PPC) with a 28-day compressive strength of 34.2 N/mm^2, specific gravity 2.9, fineness 2.1%, consistency 34%, initial setting time 98 minutes, final setting time 240 minutes, and soundness of 1 mm conforming to IS 1489-Part 1 [55] was used in the mortar and concrete. Natural sand (FA-1) was used to prepare the mortar mix for ferrocement, and riverbed sand (FA-2) was used for concrete mix, as per IS 383 [56] specifications. Two types of coarse aggregates with nominal size of 20 mm and 12.5 mm in a ratio of 60 : 40 were used to prepare the concrete mix as per IS 2386-Part 3 [57] specifications. Detail of test results of fine and coarse aggregates is given in Table 1.

The thermomechanical-treated (TMT) 12 mm diameter bars with an ultimate tensile strength of 710 N/mm^2 were used as tensile reinforcement, and 8 mm diameter TMT bars which have an ultimate tensile strength of 697.5 N/mm^2 were used as compressive reinforcement. Plain mild steel (MS) 6 mm diameter reinforcement bars with an ultimate tensile strength of 491.5 N/mm^2 were used as shear reinforcement. Galvanised square woven wire mesh of 0.49 mm diameter with centre-to-centre spacing of 8 mm and having an ultimate tensile strength of 950 N/mm^2 was

Table 1: Properties of fine and coarse aggregates.

Material description	Fineness modulus	Specific gravity	Water absorption (%)	Moisture content (%)	Grading zone
Fine aggregate (FA-1)	2.24	2.67	1.9	0.22	3
Fine aggregate (FA-2)	2.65	2.675	1.35	0.16	2
Coarse aggregate (CA-1) 20 mm	6.69	2.69	1.18	Nil	All-in-aggregate
Coarse aggregate (CA-2) 12.5 mm	6.11	2.685	1.11	Nil	All-in-aggregate

used in polymer-modified ferrocement as per ACI 549.1R guidelines [58]. Details of test results are given in Table 2.

Commercially available Sika® Latex Power [59] SBR-based polymer in liquid form having 45% solid contents, pH value of 8.50 at 25°, and specific gravity of 1.01 was used to modify the current mortar matrix of ferrocement. A 0.7% silicon solid by weight of the SBR polymer was used as an antifoaming agent in the PMF matrix [60]. Potable water was used for mixing and curing purposes.

2.1.1. Concrete Mix. Concrete of grade M20 with C : FA : CA in a ratio of 1 : 2.1 : 3.4 was designed as per IS 10262 [61] guidelines. The water-cement ratio of the concrete mix was 0.55 and having an ultimate compressive strength of 26.72 N/mm^2. The slump value of the mix was kept as 75 mm–100 mm. This concrete mix proportion was used to cast all the beam specimens. Detail of quantities required per m^3 of concrete is given in Table 3.

2.1.2. Polymer-Modified Ferrocement (PMF). Polymer-modified mortar (PMM) with a cement (C) to sand (FA-1) ratio of 1 : 2 and having 15% of SBR was used to develop the polymer-modified ferrocement (PMF). The water-cement ratio of mortar was found as 0.56 for a flow value of 105 ± 5% [62]. The 28-day compressive strength, flexural strength, and Young's modulus of elasticity of PMM were 31.53 N/mm^2, 8.52 N/mm^2, and 13.15×10^3 N/mm^2, respectively [45]. This polymer-modified mortar was further used to modify the three-layered square woven steel wire mesh ferrocement. The 28-day flexural and tensile strengths of polymer-modified ferrocement were 17.01 N/mm^2 and 6.12 N/mm^2, respectively [45]. This PMF is used as an encasing material in this experimental work to strengthen the predamaged RCC beams.

2.2. Experimental Program and Methods. This experimental study was conducted on 48 full-size (127 mm × 229 mm × 2700 mm) RCC beams. Out of these 48 beams, 24 were designed for shear-designed beams (DBs), and 18 stirrups of 6 mm diameter were provided at a spacing of 150 mm c/c. The rest 24 beams were designed as shear-deficient beams (SDBs), and 7 stirrups of 6 mm diameter were provided at a spacing of 450 mm c/c. All the beams were confined with 2–12 mm diameter bars on tensile face and 2–8 mm diameter bars on compression face. Beam section and reinforcement details of both types of beams are shown in Figure 1. For casting beam specimens, the reinforcement was correctly placed in the formwork with specified cover and then poured with concrete. A steel formwork was used, and IS 456 [63] specifications were strictly followed. The detail of beams along with their designation is given in Table 4.

Table 2: Properties of reinforcement steel bars.

Diameter (mm)	Yield stress (MPa)	Ultimate stress (MPa)	Elongation (%)
12	556.5	710.0	22.0
8	548.5	679.5	18.4
6	465.0	491.5	5.0
0.49 (square woven wire mesh)	665.0	950.0	18.2

Table 3: Detail of concrete mix design.

Cement (C)	Fine aggregate (FA-2)	Coarse aggregate (CA-1 : CA-2) (60 : 40)	Water (W)
340	714.0	1156	187

All quantities are in Kg.

2.2.1. Testing of Controlled Beams. After 28 days of curing, the shear-designed and shear-deficient controlled beams were tested over a loading frame fixed with a hydraulic jack. The load test on all the beams was performed for two different shear-span (a/d) ratios 1 and 3 over a simply supported effective span of 2500 mm as shown in Figure 2. A standard set of 3 beams was taken for each load test. Load-deflection curves were observed, and the ultimate shear load-carrying capacities of controlled beams (DBs and SDBs) were worked out 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 = 198 mm" denotes the effective depth of the beam. For a/d = 1.0, the value of "a = 198 mm" was kept, and for a/d = 3.0, the value of "a = 594 mm" was kept. These control beams were tested up to an ultimate failure to work out their average elastic, elastoplastic, and plastic load values corresponding to 45%, 75%, and 95% of ultimate load values, respectively. Details of test results are given in Table 5. Deflection of these beams was also determined and recorded by using LVDTs (linear variable displacement transducers) with a least count 0.001 mm. The LVDTs were placed under the soffit of beams which were further attached electronically with the computerised system. The location of these LVDTs was fixed according to the load position; that is, the 1st LVDT was placed under the loading point "P," the 2nd was under the midpoint of the beam, and the 3rd was exactly at the same distance from opposite support just mirrored to the

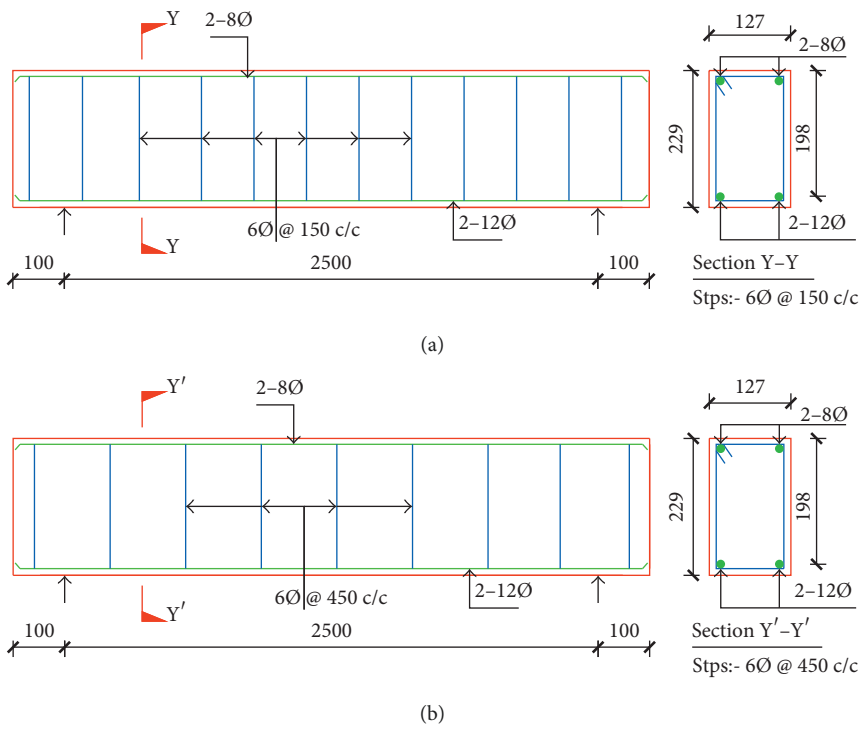


Figure 1: Beam section and reinforcement details: (a) shear-designed beams (DBs); (b) shear-deficient beams (SDBs).

Table 4: Shear-designed and shear-deficient RCC beam designation detail.

Designation of beams	Loading description	Shear-span ratio (a/d)	No. of samples
DB-ad-1	Shear-designed controlled beams	1.0	3
RDB45-ad-1	45% predamage + strengthening	1.0	3
RDB75-ad-1	75% predamage + strengthening	1.0	3
RDB95-ad-1	95% predamage + strengthening	1.0	3
DB-ad-3	Shear-designed controlled beams	3.0	3
RDB45-ad-3	45% predamage + strengthening	3.0	3
RDB75-ad-3	75% predamage + strengthening	3.0	3
RDB95-ad-3	95% predamage + strengthening	3.0	3
SDB-ad-1	Shear-deficient controlled beams	1.0	3
RSDB45-ad-1	45% predamage + strengthening	1.0	3
RSDB75-ad-1	75% predamage + strengthening	1.0	3
RSDB95-ad-1	95% predamage + strengthening	1.0	3
SDB-ad-3	Shear-deficient controlled beams	3.0	3
RSDB45-ad-3	45% predamage + strengthening	3.0	3
RSDB75-ad-3	75% predamage + strengthening	3.0	3
RSDB95-ad-3	95% predamage + strengthening	3.0	3

DB: shear-designed beam; RDB: strengthened shear-designed beam; SDB: shear-deficient beam; RSDB: strengthened shear-deficient beam.

1st LVDT (refer Figure 2). The load versus deflection curves of the 1st LVDT are only presented in this article for comparative study.

2.2.2. Predamaging of Beams. According to the test results of controlled beams, the other sets of beams were loaded on the same setup and predamaged for three different levels of initial stresses corresponding to 45%, 75%, and 95% of the ultimate load. The DB45 and SDB45 beams were initially loaded for 45% level of damage. Similarly, DB75 and SDB75 and DB95 and SDB95 were loaded for 75% and 95% levels of damage, respectively. These damaged beams were then unloaded and strengthened with 20 mm thick U-shaped

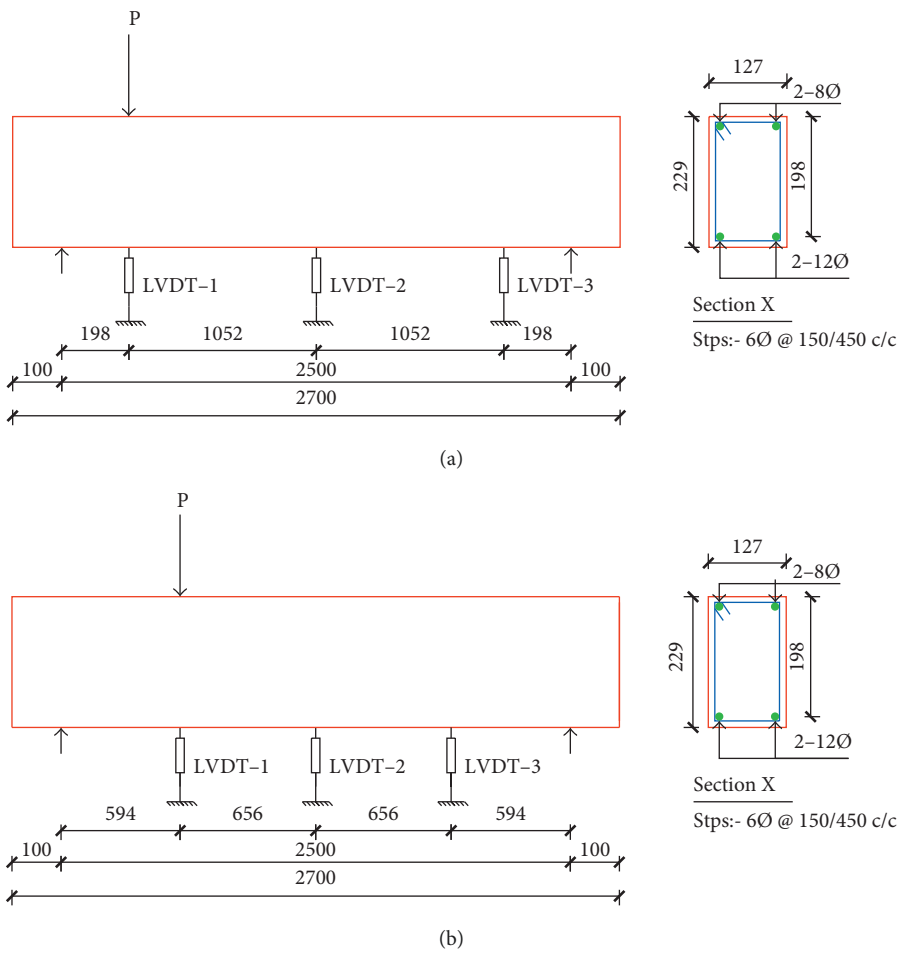


Figure 2: Schematic diagram of the test setup: (a) beams tested at $a/d = 1$; (b) beams tested at $a/d = 3$.

Table 5: Test results of controlled DB and SDB.

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	11.674	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

polymer-modified ferrocement jacketing which contained 15% SBR latex and three layers of square woven steel wire mesh. The detail of strengthening is given in the subsequent section.

2.2.3. Strengthening Procedure. 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 curing compounds. An improper preparation of the surface can result in debonding of PMF jacketing. Before the strengthening procedure, the beams were turned upside down to expose their soffit. The particular portion of all the predamaged beams was cleaned with a wire brush at their soffit and side faces where the jacketing is supposed to be applied. The surface was cleaned with such a way to expose the aggregates and to make the surface sufficiently rough for application of repairing mortar. Water was sprayed on the prepared surface to make it wet.

The another study done by the authors resulted that the three-layered square woven steel wire mesh PMF having an optimum percentage of 15% SBR showed better strength

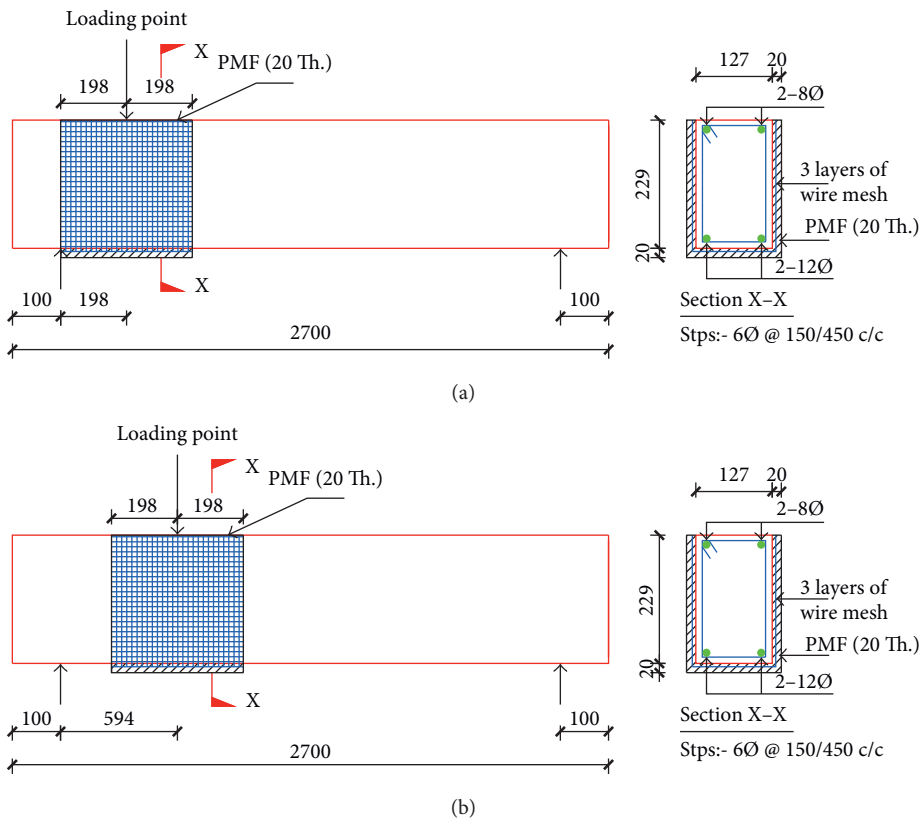


Figure 3: Detail of PMF strengthening: (a) beams tested at $a/d = 1$; (b) beams tested at $a/d = 3$.

properties and hence adopted in this present investigation to strengthen the predamaged RCC beams. The polymer-modified repairing mortar was constituted with cement, sand, SBR, and water in the ratio of 1 : 2 : 0.15 : 0.35 [45]. A silicon-based antifoaming agent was also added to the polymer-modified mortar matrix. Cement slurry was used as a bonding agent. First of all, three layers of wire mesh were affixed over the specified length of beams, and then, SBR-modified mortar was applied. A total of 20 mm thick PMF jacketing was applied under the soffit and side faces of the beams in a length of "$2d = 396$ mm", centred on the loading point, for both DB- and SDB-type beams, irrespective of a/d ratios. Different sets of beam specimens were prepared for each level of initial damage. The repair material was accurately positioned with the help of wooden forms. After 24 hours of application of PMF jacketing, the strengthened portion of the beams was cured with gunny bags for further 28 days. The details of PMF-strengthened beams are shown in Figure 3 and Plate 1.

2.2.4. Reloading of Predamaged Strengthened Beams. The PMF-strengthened beams were again placed on the same loading frame setups (as specified in Figure 2) to find their ultimate shear load-carrying capacities, deflection, and cracking pattern. The locations of LVDTs were kept the same as in the case of controlled beams. The load-deflection behaviour of all sets of tested beams was recorded for conclusive study and shown in Figures 4–7. The detail of observed loads, deflections, and failure modes is given in Table 6.

Plate 1: Application of PMF jacketing on the soffit and side faces of the beams (by turning the beams upside down).

3. Results

In this current experimental program, a total of 48 beams were tested. The testing was aimed to achieve many objectives by comparing the behaviours of these beams. The controlled and strengthened beams were loaded up to ultimate failure. Most of the beams showed diagonal cracking patterns and the shear mode of failure. The effects of different levels of initial stresses, a/d ratios, and spacing of shear stirrups on the strength, failure modes, cracking pattern,

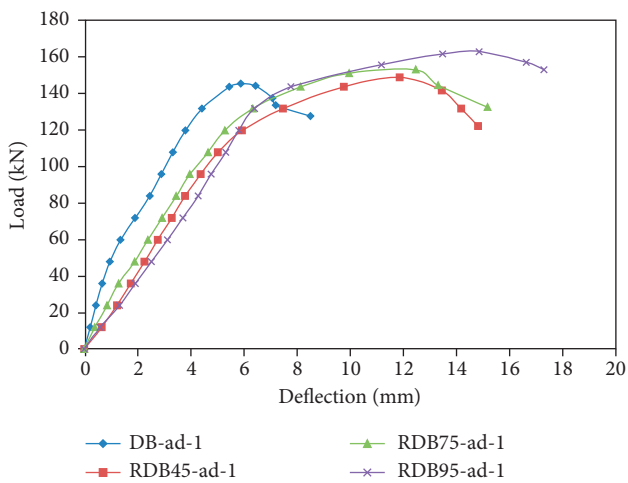


Figure 4: Load-deflection plot of shear-designed beams tested at a/d ratio 1.

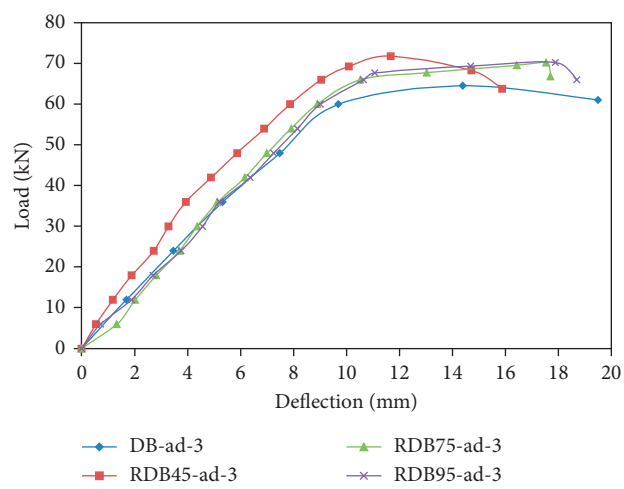


Figure 5: Load-deflection plot of shear-designed beams tested at a/d ratio 3.

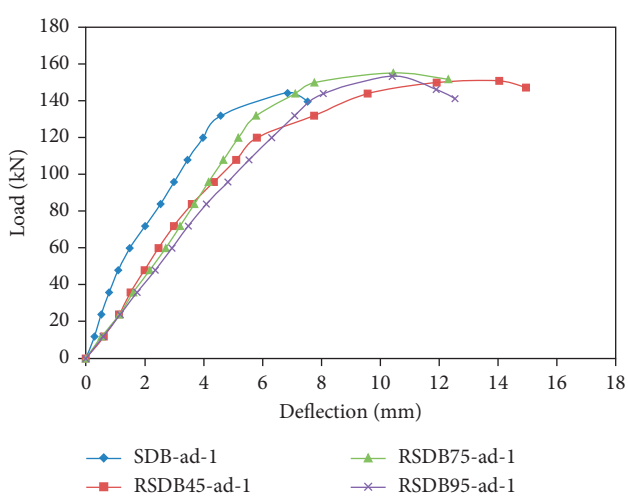


Figure 6: Load-deflection plot of shear-deficient beams tested at a/d ratio 1.

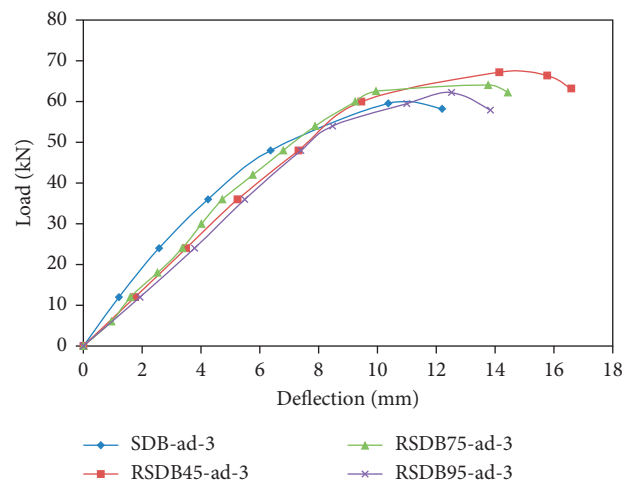


Figure 7: Load-deflection plot of shear-deficient beams tested at a/d ratio 3.

ductility, and deflection of strengthened beams are discussed in the subsequent sections.

3.1. Comparisons between Controlled and Strengthened Beams

3.1.1. Effect of Different Levels of Initial Stresses. All the strengthened beams which were initially damaged with 45%, 75%, and 95% of the ultimate load had showed a complete restoration and further enhancement of the original strength up to 12.03% after strengthening. The deflection behaviour of these beams was also changed after strengthening, and the beams exhibited more ductility as compared to controlled 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) showed the maximum improvement in their ultimate load value by 12.03%. The results of strengthened beams with an initial damage of 45% (RDB45-ad-1) and 75% (RDB-75-ad-1) also showed a similar trend of improvement in their ultimate load values by 2.38% and 5.33%, respectively, over the controlled beam DB-ad-1. The observed figures of load values were quite less for the beams tested at $a/d = 3.0$, but these beams showed 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 the improvement in the ultimate load by 11.19%, 8.82%, and 8.85%, respectively, with respect to the controlled beam DB-ad-3. Load-deflection comparative curves of controlled and strengthened shear-designed beams with different levels of initial damages, when tested at $a/d = 1$ and 3, are shown in Figures 4 and 5.

The strengthened shear-deficient beams having initial damages of 45%, 75%, and 95% were also tested for different a/d ratios. The strengthened RSDB75-ad-1 beams showed maximum improvement in their ultimate load by 7.57%, when compared with the controlled beam SDB-ad-1. The RSDB45-ad-1 and RSDB95-ad-1 beams also showed an improvement in their ultimate load values by 4.60% and

Table 6: Reloading test results of predamaged strengthened beams.

Beam description	Ultimate load (kN)	Percentage increase in ultimate load	Deflection at ultimate load (mm)	Failure mode
DB-ad-1	145.68	—	5.899	Shear
RDB45-ad-1	149.14	2.38%	11.891	Shear
RDB75-ad-1	153.45	5.33%	12.494	Shear flexural
RDB95-ad-1	163.20	12.03%	14.882	Shear flexural
DB-ad-3	64.53	—	11.674	Shear flexural
RDB45-ad-3	71.75	11.19%	14.564	Shear-flexural
RDB75-ad-3	70.22	8.82%	17.534	Shear
RDB95-ad-3	70.24	8.85%	17.887	Shear flexural
SDB-ad-1	144.28	—	6.857	Shear
RSDB45-ad-1	154.60	4.60%	14.048	Shear flexural
RSDB75-ad-1	161.87	7.57%	10.450	Shear flexural
RSDB95-ad-1	155.96	6.29%	10.412	Shear flexural
SDB-ad-3	59.55	—	10.371	Shear
RSDB45-ad-3	67.20	12.85%	14.146	Shear
RSDB75-ad-3	64.05	7.56%	13.763	Shear
RSDB95-ad-3	62.24	4.52%	12.523	Shear

6.29%, respectively, as compared to the controlled beam SDB-ad-1. At a higher a/d ratio = 3.0, the ultimate loads also exhibited enhancement, and the maximum improvement in the load was recorded for RSDB45-ad-3 beams by 12.85% over the controlled beam SDB-ad-3. The load-carrying capacities of RSDB75-ad-3 and RSDB 95-ad-3 beams were also improved by 7.56% and 4.52%, respectively. Load-deflection comparative curves of controlled and strengthened shear-deficient beams with different levels of initial stresses, when tested at $a/d = 1$ and 3, are shown in Figures 6 and 7.

Test results showed that the performance of PMF jacketing is very consistent for both shear-deficient and shear-designed beams at a/d ratios 1 and 3. Application of PMF jacketing filled the cracks which were developed during the initial damaging of beams and also helped to arrest the formation of such deformities during retesting of damaged strengthened beams. The PMF jacketing helped to enhance the ductility and caused to delay the shear failure of beams by resisting and distributing the applied loads. The proportional deflection of strengthened beams was also increased irrespective of their level of initial damages.

3.1.2. Effect of a/d Ratios. The controlled as well as strengthened shear-designed (DB) and shear-deficient (SDB) beams were tested for two different a/d ratios 1 and 3 to study the effect of shear span "a" on the performance of these beams. From the test results, it was found that the a/d ratio played a crucial role. The strengthened beams tested at $a/d = 1.0$ showed 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 stirrup spacing.

The controlled 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 was 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 8.

A similar trend was recorded for controlled shear-deficient SDB-ad-1 beams, and 142.28% higher load-carrying capacity was observed in comparison with SDB-ad-3 beams. The strengthened shear-deficient beams also behaved in a similar manner, and the ultimate load values of RSDB95-ad-1 beams (tested at $a/d = 1$) were improved to 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. Graphical comparisons of ultimate loads at different a/d ratios are shown in Figure 9.

At an 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 was shifted to a shear-flexure mode of failure. Shear forces were dominating in the case of beams tested at $a/d = 1.0$. All the controlled 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 the load point from the support was increased, the relative deflection was also increased. This is due to the parabolic shape of the 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 a higher a/d ratio of 3. This improved

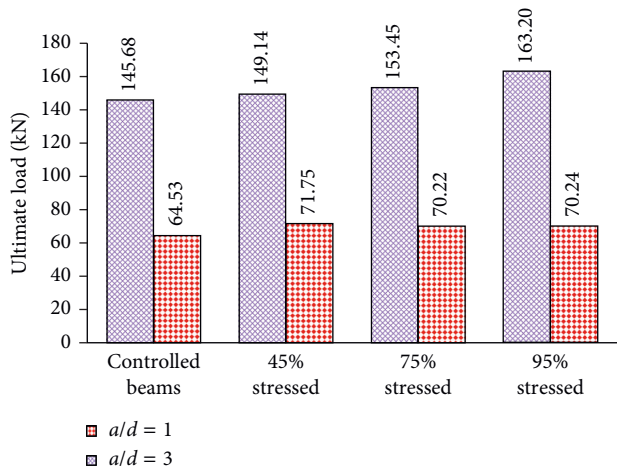


Figure 8: Comparison of ultimate loads of shear-designed beams tested at a/d ratios 1 and 3.

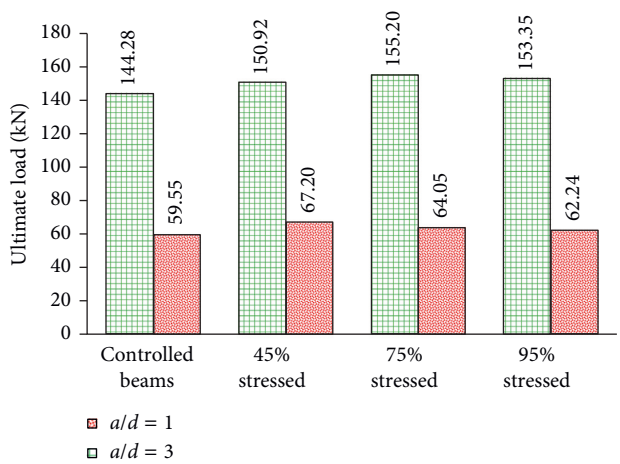


Figure 9: Comparison of ultimate loads of shear-deficient beams tested at a/d ratios 1 and 3.

ductility further imparts to increase the deflection of these beams.

It is 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.

3.1.3. Effect of Beam Type. The beams having stirrups at a spacing of 150 mm c/c were designated as shear-designed beams (DBs), and the beams have stirrups at a spacing of 450 mm c/c were designated as shear-deficient beams (SDBs). Both the controlled and strengthened beams were tested for two different a/d ratios. The controlled shear-deficient beams (SDB-ad-1) when tested at a/d ratio = 1.0 showed almost similar strength values as compared to controlled shear-designed beams (DB-ad-1) 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 the 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 $a/d = 1$, and the load values firstly improved for the beams with 45% and 75% initial damages and then decreased for beams having 95% of the initial damage. The RSDB95-ad-1 beams showed 6.04% lesser load-carrying capacities as compared to 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 is attributable to the arch action of the concrete element and the efficiency of PMF jacketing. 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 showed a lesser load-carrying capacity as compared to shear-designed beams for a particular damage

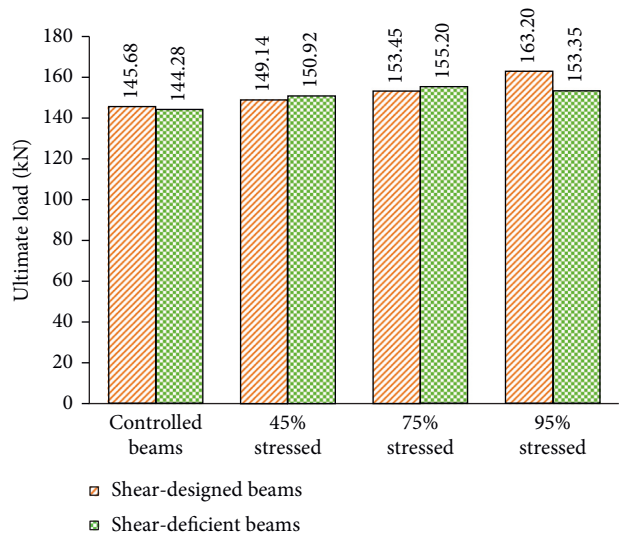


Figure 10: Comparison of ultimate loads of shear-designed and shear-deficient beams tested at a/d 1.

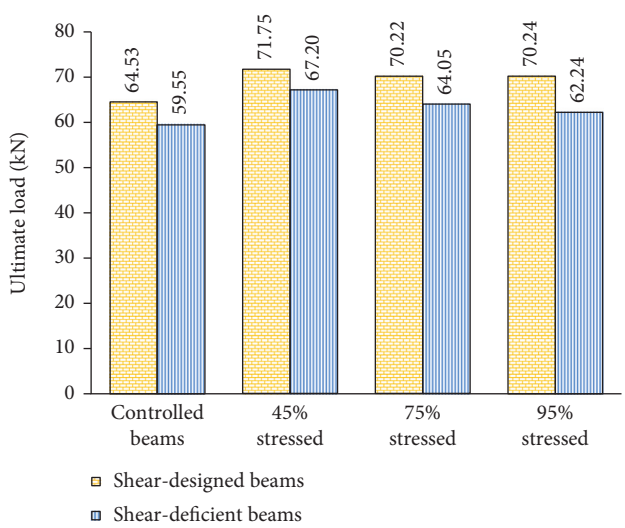


Figure 11: Comparison of ultimate loads of shear-designed and shear-deficient beams tested at a/d 3.

level. Deflection behaviour was also inconsistent in both the cases. Graphical comparison of ultimate loads of beams tested at a/d ratio 1 is shown in Figure 10.

The shear-deficient strengthened beams (RSDB45-ad-1 and RSDB75-ad-1) with initial stresses of 45% and 75% showed a higher strength of 1.19% and 1.14% over the shear-designed strengthened beams (RDB45-ad-1 and RDB75-ad-1), respectively, when tested at $a/d = 1$. However, at the higher initial damage level of 95%, strengthened RSDB95-ad-1 beams reflected 6.04% lesser strength as compared to the controlled RDB95-ad-1 beams. It is clear from this observation that the PMF jacketing technique is more effective in the case of shear-deficient beams. Apparently, PMF enhanced the shear strength of concrete near the support by means of transferring the increased load to the stirrups and converting the brittle concrete failure into ductile. At $a/d = 3$, the strengthened shear-deficient beams (RSDB45-ad-3, RSDB75-ad-3, and RSDB95-ad-3) showed lesser strength values by 6.34%, 8.79%, and 11.39% as compared to strengthened shear-designed beams (RDB45-ad-3, RDB75-ad-3, and RDB95-ad-3). Graphical comparison of ultimate loads of beams tested at a/d ratio 3 is shown in Figure 11.

The PMF jacketing is fully effective to restore and enhance the original strength of both types of initially damaged beams. Furthermore, 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 450 mm 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 inertia and ductility of the 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 similar elastic behaviour as the strengthened shear-designed beams,

and the beams were observed to fail in shear compression and diagonal tension.

4. Discussion

4.1. Comparison of Failure Load of Specimens. Failure load detail for all the tested beam specimens is given in Table 6. Almost shear mode of failures was observed in all the beam specimens. No debonding of jacketing appeared when the PMF-strengthened beams were tested up to an 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 considerably improved as compared to controlled beams at both a/d ratios 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. As the initial stress level of strengthened beams increased, the percentage improvement in the strength was also increased. Load enhancement behaviour was also observed for the strengthened beams tested at $a/d = 3.0$, and the ultimate load values of RDB-45-ad-3, RDB-75-ad-3, and RDB95-ad-3 beams were 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 predamages showed almost an equal improvement of shear load-carrying capacities. Graphical comparison of ultimate load values of shear-designed beams tested at a/d ratios 1 and 3 is shown in Figure 12.

The failure loads of strengthened shear-deficient beams were also considerably improved. This improvement was found to be lesser for the beams with a higher initial damage level of 95%. The initially stressed shear-deficient beams almost got damaged at 95% stress level, but the strengthening technique showed its worth to restore and enhance the total load-carrying capacity of such beams. The percentage improvement in the 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 controlled beam SDB-ad-1. The strengthened beams RSDB45-ad-3, RSDB75-ad-3, and RSDB95-ad-3 also showed the improvement in ultimate load values by 12.85%, 7.56%, and 4.52%, respectively, as compared to the controlled beam 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 if compared with beams tested at $a/d = 1.0$. It is concluded that the PMF jacketing restores and improves the original strength of predamaged beams. Graphical comparison of ultimate load values of shear-deficient beams tested at a/d ratios 1 and 3 is shown in Figure 13.

However, it was observed that some cracks appeared even through jacketing, which indicates a good bond of PMF jacketing with the existing concrete structure. However, in some cases, no cracks developed in the strengthened length where the jacketing was applied, and the failure cracks developed only in the vicinity of the jacketing edges. It reflected that the PMF jacketing behaved monolithically with the concrete specimens and helped to improve the ductility of beams after strengthening.

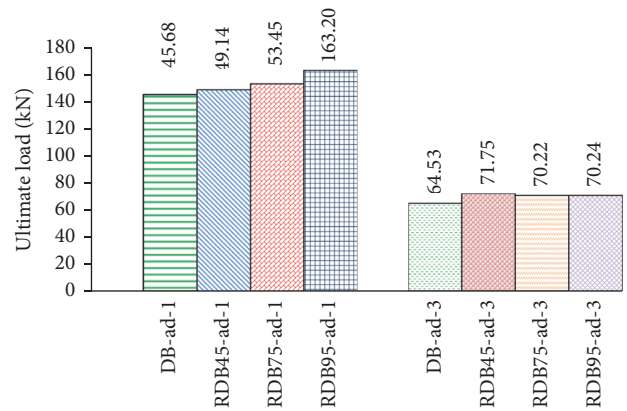


Figure 12: Ultimate loads of shear-designed beams tested at a/d ratios 1 and 3.

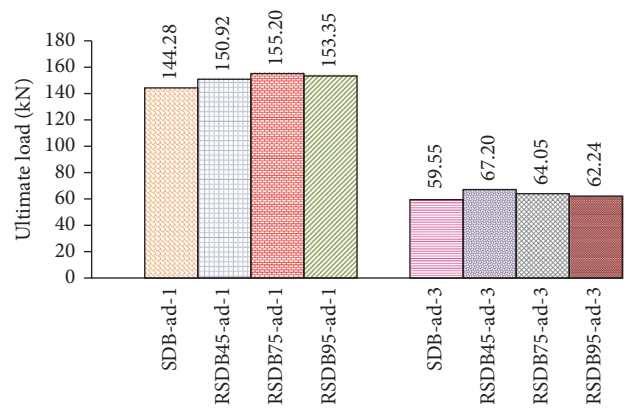


Figure 13: Ultimate loads of shear-deficient beams tested at a/d ratios 1 and 3.

4.2. Ductility and Deflection Behaviour 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 Figures 4–7. The ductility of both types of beams was improved after strengthening irrespective of whether the beams were tested at a/d 1.0 or 3.0. The main reason for the increase of ductility is the strengthening technique only. Strengthening of beams improved the section modulus of beams and made the beams more ductile to resist deflection even after initial damages, and hence, the beams showed higher deflection values as compared to controlled beams at any particular load level. It is concluded that the PMF increases the ductility of predamaged strengthened beams which further imparts to sustain higher deflection and applied loads.

4.3. Cracking Pattern and Failure Mechanism. A few number of cracks of varying width and spacing were observed in

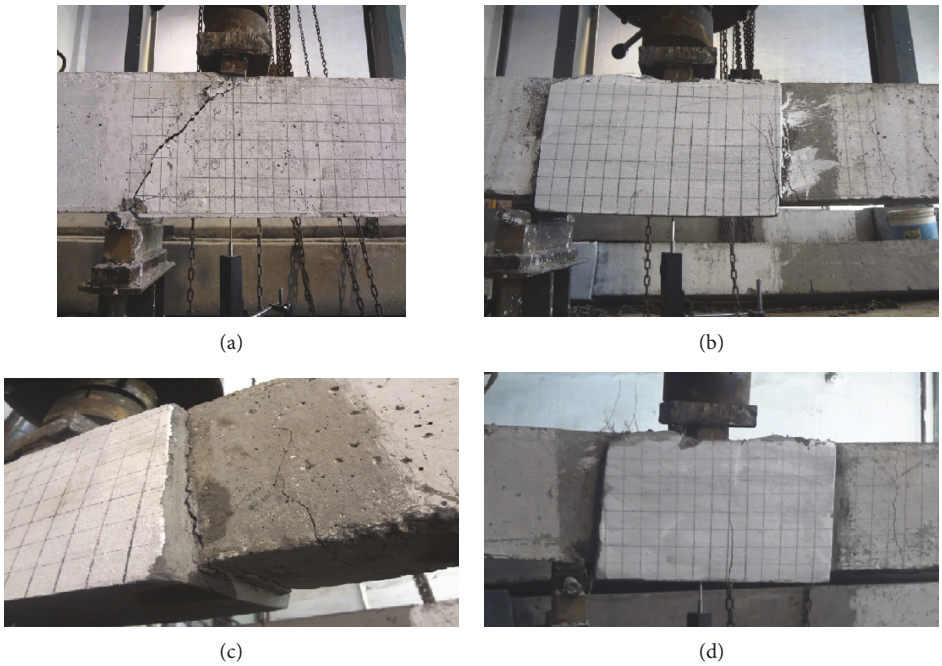


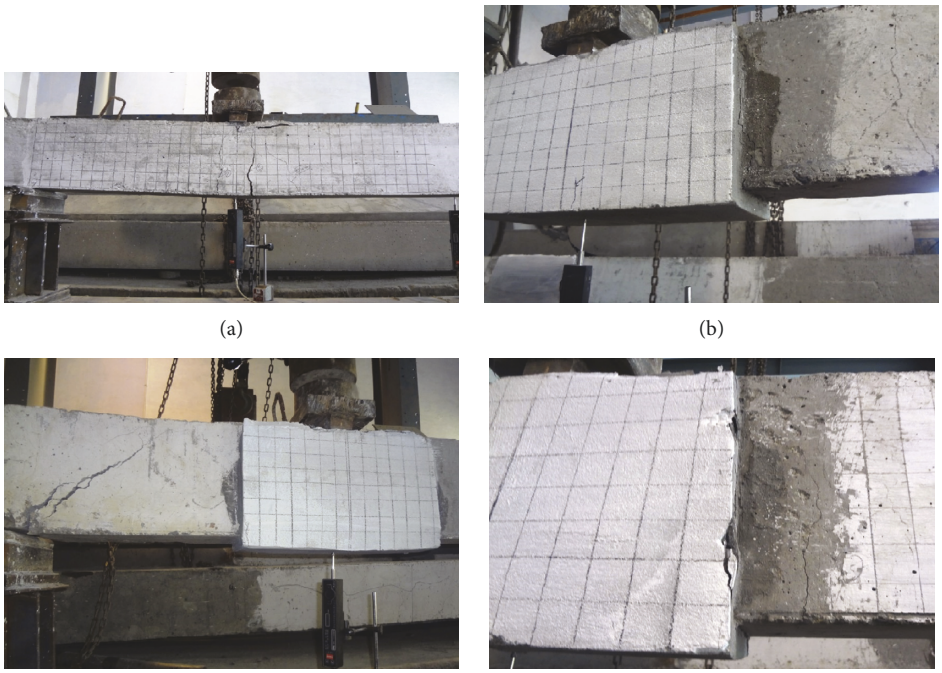
Plate 2: Shear-designed beams tested at $a/d = 1$: (a) DB-ad-1; (b) RDB45-ad-1; (c) RDB75-ad-1; (d) RDB95-ad-1.

both types of control beams. The beams ultimately failed because of the widening of any of these cracks. On the other hand, the predamaged 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 2–5 show the cracking pattern of shear-designed and shear-deficient controlled and strengthened beams tested at $a/d = 1$ and 3.

The tested beams got cracked, apparently due to two different conditions. In some cases, the cracks appeared diagonally near the support 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 the crack (refer Plates 2(a) and 4(a)). With the increase of the a/d ratio to 3.0, the observed crack pattern was of the other type which results due to a combination of shear force and flexure moment (refer Plates 3(a) and 5(a)).

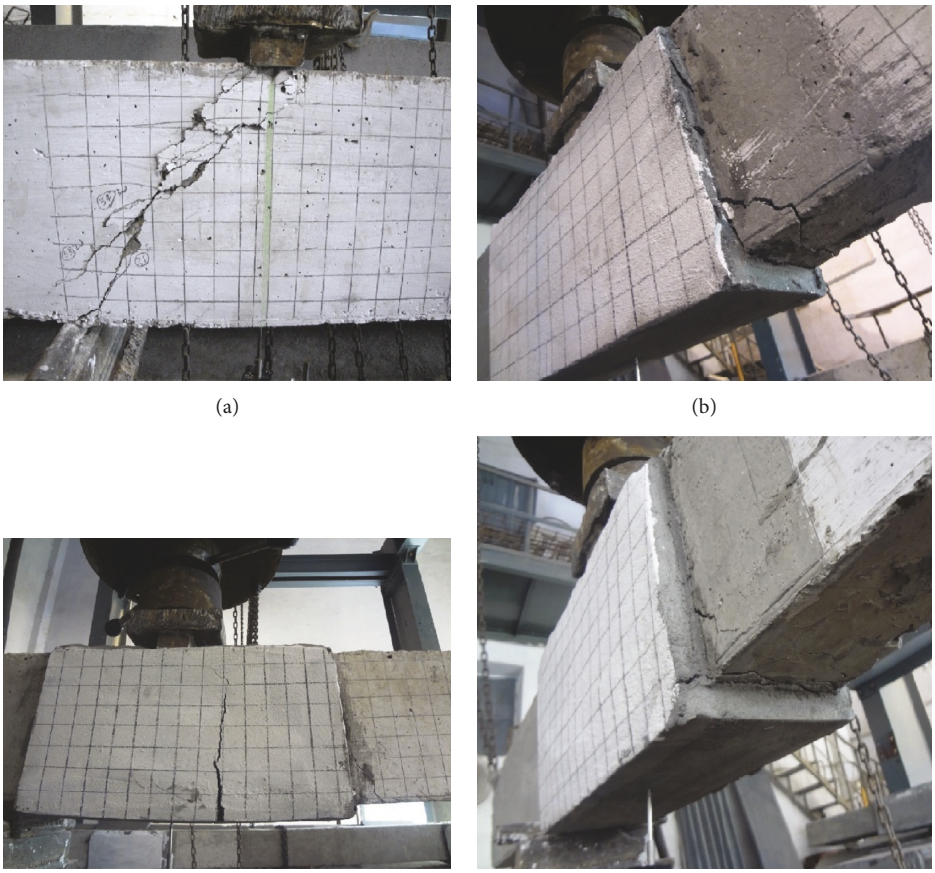
In the case of both shear-designed and shear-deficient beams when tested at $a/d = 1.0$, the first crack appeared almost in the region where the shear force was maximum and the bending moment was negligible, that is, in a shear span "a" which is equal to "$d = 198$ mm" for a/d ratio 1.0. The cracks aligned with one another and inclined to the axis of the beam at an angle between 30° and 45°. It was observed that these cracks belonged to the first type, that is, mostly due to shear forces. As the load increased, further cracks appeared in the zone of lesser shear stresses, that is, towards the centre of beams, and nonlinearity of the load-deflection behaviour increased. With the further increase of the load, more and more cracks 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 nonlinear.

The shear mode of failure was observed in the tested beams, and most of the cracks developed near the support and loading point (refer Plates 2–5). 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 controlled beams failed in a typical shear mode when tested at $a/d = 1.0$. The shear-designed controlled beam DB-ad-3 was observed to behave like DB-ad-1, but in this case, the cracks were not only in the shear zone, but some hairline cracks also developed towards the centre of the beam. These cracks were in the region of bending moment, and as such, these beams were in the 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 $a/d = 3.0$. Aggregate interlocking, dowel action of longitudinal bars, and bond strength of 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 3: 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 4: Shear-deficient beams tested at $a/d = 1$: (a) SDB-ad-1; (b) RSDB45-ad-1; (c) RSDB75-ad-1; (d) RSDB95-ad-1.

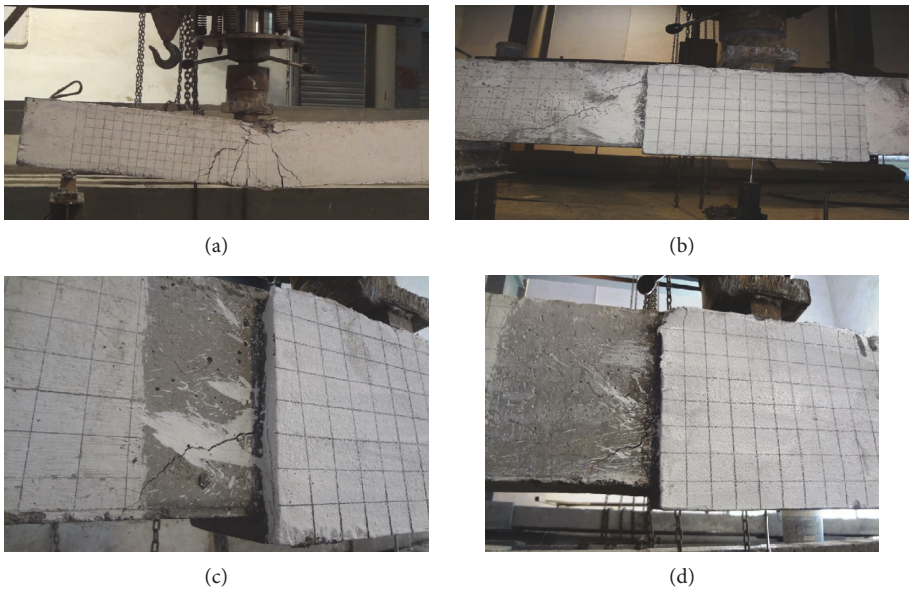


Plate 5: Shear-deficient beams tested at $a/d = 3$: (a) SDB-ad-3; (b) RSDB45-ad-3; (c) RSDB75-ad-3; (d) RSDB95-ad-3.

The behaviour of all the strengthened beams with different levels of initial damages was 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 causes to delay the formation of cracks. As a result, the figurative deflection of predamaged strengthened beams also enhanced. Spalling of concrete in the vicinity of the support point also reduced because of strengthening. PMF jacketing acted monolithically with the RCC beam specimens, and no bond failure was observed during the testing process. However, 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 unjacketed sections.

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

5. Conclusions

The following conclusions have been drawn from this experimental study:

(1) PMF is fully effective to restore and enhance the original strength of initially stressed beams even after 95% damage. All the strengthened beams showed a complete restoration of original strength irrespective of stirrup spacing, the level of initial damage, and a/d ratios. Maximum enhancement in the ultimate load value was observed as 12.03% for RDB95-ad-1 beams.

(2) PMF jacketing causes to delay the direct shear failure of beams and apparently increases the contribution of stirrups to resist more loads. This technique is found to be more efficient in case of shear-deficient beams.

(3) The PMF strengthening technique increased the ductility of predamaged beams and caused to delay the shear failure by resisting and distributing the applied loads. The beams behaved more elastically, and deflection behaviour also improved after strengthening.

(4) The rate of crack development was also reduced, and the strengthened beams displayed a less number of cracks as compared to the corresponding beams without strengthening.

(5) No bond failure of jacketing was observed which reflects the proper bonding and compatibility of polymer-modified ferrocement with the concrete structures.

Conflicts of Interest

The authors declare no conflicts of interest.

Acknowledgments

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