1. Introduction

This report deals with the tests conducted to study the shear strengthening mechanism in reinforced concrete beams strengthened with NSM steel strips and steel rods to the sides of the beam. The objective of the present study is to understand the effect on shear strength of strengthening the beam using the NSM steel strips and steel bars in RC beams. The main parameters varied were the NSM reinforcement material and the provision of the internal shear stirrups.

Earlier to this report the submitted minor project report, a literature study was carried out for various distresses to the structure, their causes and various methods of retrofitting the structures were described. And we have decided to use Near Surface Mounted (NSM) reinforcement as the retrofitting/strengthening technique for the beams casted in laboratory which were weak in shear as shear reinforcement was omitted from half of the beam in two specimens (S, S1) and was fully omitted in one beam (NS).

And in this report we have described about the various literature studies carried out on the NSM technique for strengthening the RC structures, preparation of test specimen, the test set-up and the properties of the constituent materials are described first. Secondly, the observations from each test are described individually. Thirdly, the inferences made from all the test results are discussed.

1.1 NSM Technique for strengthening the RC structures

Over a period of time, extensive research has been conducted on the strengthening of reinforced concrete (RC) structures using externally bonded metal plates(plate bonding), steel rods (RC Jacketing), fiber reinforced polymer (FRP) laminates (externally bonded reinforcement) etc; the technology has also been implemented in a large number of practical projects world-wide. More recently, near-surface mounted (NSM) reinforcement has attracted an increasing amount of research as well as practical application. In the NSM method, grooves are first cut into the concrete cover of an RC element and the reinforcement is bonded therein with appropriate groove filler (typically epoxy paste or cement grout or epoxy-sand mortar). Earlier the NSM reinforcement was also known as ‘‘grouted reinforcement’’, or ‘‘embedded reinforcement’’. [1]

The use of NSM steel rebars in Europe for the strengthening of RC structures date back to the early 1950s [1]. More recent applications of NSM stainless steel bars for the strengthening of masonry buildings and arch bridges have also been documented [2].The advantages of FRP versus steel as NSM reinforcement are better resistance to corrosion, increased ease and speed of installation due to its lightweight, and a reduced groove size due to the higher tensile strength and better corrosion resistance of FRP. But as FRP is costlier and availability is also concerned we decided to retrofit the beams with packing steel strips and steel rebars which are good in tensile strength and easily available.

The NSM system has a number of advantages some are listed as follows:

1. The amount of site installation work may be reduced, as surface preparation other than grooving is no longer required (e.g., plaster removal is not necessary; irregularities of the concrete surface can be more easily accommodated; removal of the weak laitance layer on the concrete surface is no longer needed);
2. NSM reinforcement as strips are less prone to debonding from the concrete substrate as they get a two sided bonded surface area which is double than as in external bonding of strips;
3. NSM bars can be more easily anchored into adjacent members to prevent debonding failures; this feature is particularly attractive in the flexural strengthening of beams and columns in rigidly-jointed frames, where the maximum moments typically occur at the ends of the member;
4. NSM reinforcement can be more easily pre-stressed;
5. NSM bars are protected by the concrete cover and so are less exposed to accidental impact and mechanical damage, fire, and vandalism; this aspect makes this technology particularly suitable for the strengthening of negative moment regions of beams/slabs;
6. The aesthetic of the strengthened structure is virtually unchanged.

Due to the above advantages, the NSM reinforcement method is in many cases superior to the externally bonded reinforcement method or can be used in combination with it, provided that the cover of the member is sufficiently thick for grooves of a desirable size to be accommodated. But as IS: 456 2000 edition has recently revised the cover thickness as earlier the cover thickness [3] was less hence this method has lot of scope in retrofitting industry in coming future.

Fig 1.1 The NSM system

The NSM system has following constituents as shown in fig 1.1 [8]

1.1.1 The NSM reinforcement:

Various types of reinforcements such as deformed steel bars, mild steel strips, CFRP, GFRP, and AFRP etc have been studied and used extensively as NSM reinforcement. Each type of bar has its own applications and benefits such as CFRP NSM reinforcement has been used to strengthen concrete structures. GFRP has been used in most applications of the NSM method to masonry and timber structures. The tensile strength and elastic modulus of CFRP are much higher than those of GFRP or steel, so for the same tensile capacity; a CFRP bar has a smaller cross-sectional area than a GFRP or steel bar and requires a smaller groove. This in turn leads to easier installation, with less risks of interfering with the internal steel reinforcement, and with savings in the groove-filling material.

The shape of the reinforcement can also be varied the NSM reinforcement may be round; square, rectangular and oval bars, as well as strips (reinforcement having thinner cross-section). Each shape has its own benefits such as square bars maximize the bar sectional area for a given size of square groove while round bars are more readily available and can be more easily anchored in pre-stressing operations. Narrow strips maximize the surface area-to-sectional area ratio for a given volume and thus minimize the risk of debonding, but require a thicker cover for a given cross-sectional area. Most recently a trapezoidal shaped bar was used in one of the studies as NSM. And it was concluded in the study that the trapezoidal CFRP bar increased both ductility and ultimate load carrying capacity of the beam [4].

NSM bars can also be with a variety of surface textures, which strongly affect their bond behavior as NSM reinforcement. Their surface can be smooth, sand-blasted, sand-coated, or roughened with a peel-ply surface treatment. Round bars can also be spirally wound with a fiber tow, or ribbed

1.1.2 The groove filler

The groove filler is the medium for the transfer of stresses between the NSM reinforcement and the concrete substrate. In terms of structural behaviors, its most relevant mechanical properties are the tensile and shear strengths. The tensile strength is especially important when the embedded bars have a deformed surface, which produces high circumferential tensile stresses in the cover formed by the groove filler as a result of the bond action. In addition, the shear strength is important when the bond capacity of the NSM reinforcement is controlled by cohesive shear failure of the groove filler.

The most common and best performing groove filler is a two-component epoxy which we have used in this project with a sand mix mortar. Low-viscosity epoxy can be easily poured into the grooves and hence is selected for strengthening in negative moment regions. For other cases, a high-viscosity epoxy is needed to avoid dripping or flowing-away. Fillers are used to improve properties of epoxy which is itself weak in shear and to stop the flow of the epoxy fluid. These can be of various as mineral fillers (calcium carbonates, magnesium carbonates), silica and silicates, glass fillers , metal oxide fillers metal powder fillers etc [5]. In our study we used standard sand of medium grade as filler in various proportions with epoxy. The addition of sand to epoxy can increase the volume, control the viscosity, lower the coefficient of thermal expansion, and raise the glass transition temperature. But this addition have unwanted reduction of the bond between the bar and mortar interface.

The use of cement paste or mortar in place of epoxy as a groove filler has recently been explored in an attempt to lower the material cost, reduce the hazard to workers, minimize the environmental impact, allow effective bonding to wet substrates, and achieve better resistance to high temperatures and improved thermal compatibility with the concrete substrate. However, cement mortar has inferior mechanical properties and durability, with a tensile strength an order of magnitude smaller than that of common epoxies. Results of bond tests and flexural tests [6] have identified some significant limitations of cement mortar as groove filler. Test has also been conducted to understand the effect of the type of adhesives on the bond strength. [7]

1.1.3 The Groove and its dimensions

The groove is cut using a cutter. It could be rectangular or square as per requirement. The groove dimension it mostly related to the dimensions of the NSM reinforcement in various studies. In the following figure 1.2 various configuration of the groove dimensions are shown.

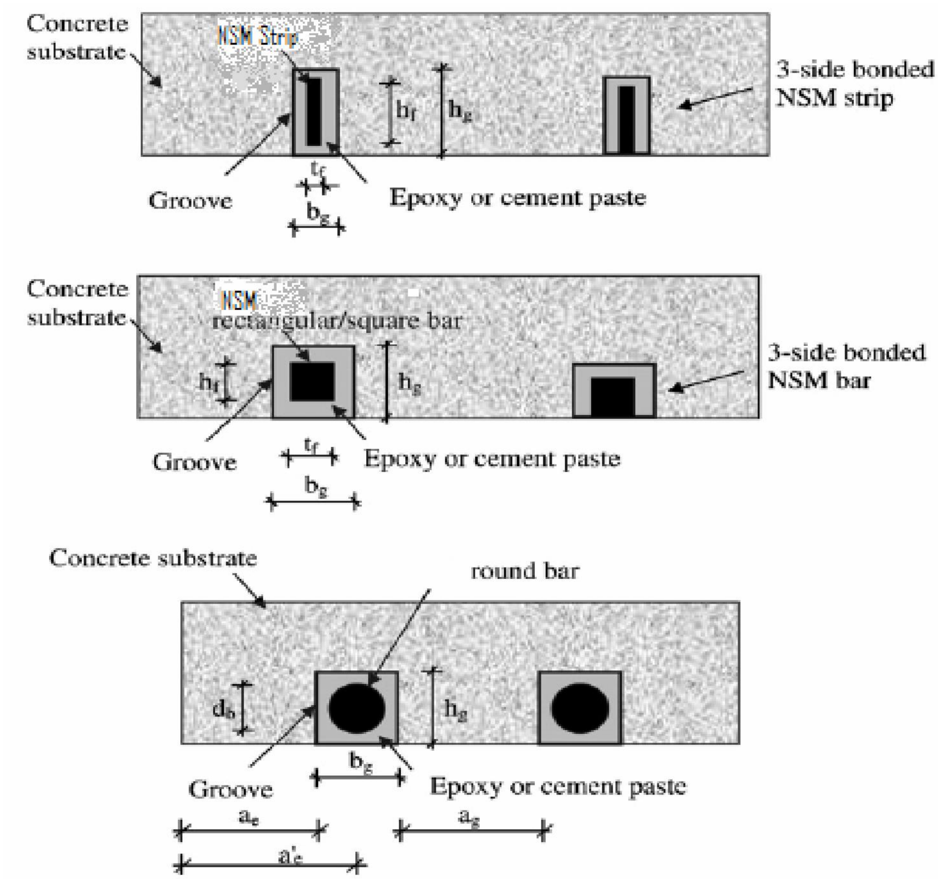


Fig 1.2 Different systems of NSM and its Nomenclature [1]

In the NSM strip system either can be a three side bond or four sided bond depending on the size of the NSM strip available and the thickness of the cover. Here t_f represents the thickness of the strip, b_f represents the width of the strip, b_g represents the width of the groove and h_g represents the depth of the groove. Various experimental studies have shown that the minimum width of the groove to implant NSM strip should be greater than three times the thickness of the strip. And depth should not be less than 1.5 times the width of the strip. i.e. $b_g \geq 3t_f$ and $h_g \geq 1.5b_f$. [1, 9]

In the NSM rectangular/square bar system h_f, h_g represents the height of the NSM bar and height of the groove respectively. And b_g and t_f represents the width of groove and thickness of the NSM bar. for the square groove $h_g = b_g$. And in circular NSM bar system d_b represents the diameter of the NSM bar and h_g represents the height of the groove and b_g represents the width of groove. The dimensions of the groove and NSM reinforcement have been related by De Lorenzis [1,8] based on bond test results by defining $k = b_g / d_b$ and proposed a minimum value for k as 1.5 for smooth or less roughened bars and 2 for deformed bars. It is suggested in various papers that in NSM bars system $b_g \geq 1.5d_b$ or 1.5 t_f and $h_g \geq 1.5d_b$ and 1.5 h_f. [1,9].

The positioning of the NSM bar with respect to distances from edges of member and spacing between the two NSM bars is also an important aspect in the designing of the NSM system. If there is only single NSM bar it should be placed centrally in the face being strengthened. Otherwise as suggested by Blaschko [10] from the test for bond by him the minimum edge distance a'_e should be ($a'_e = a_e + b_g/2$) should be about 20mm to avoid splitting failure of the concrete corner. He also suggested that a'_e should not be less than 30mm or the maximum size of the aggregate whichever is greater. Also the spacing of the groove "a_g" minimum 2- 4 times the bar diameter as suggested by Hassan and Rizkalla [11]. The edge distance and groove spacing greatly influence the mode of failure of the NSM system.

1.2 Failure modes and mechanism

The most important factor that affects the behavior of the NSM strengthening method is the bond between the NSM bar and the substrate material. The performance of the bond is dependent on various factors like

1. The groove and the bar dimensions;
2. The tensile and the shear strength of the groove filler and concrete;
3. The bar cross-sectional shape and surface configuration;
4. The degree of roughness of the groove surface;
5. The anchorage length of NSM;
6. The type of the groove filler etc.

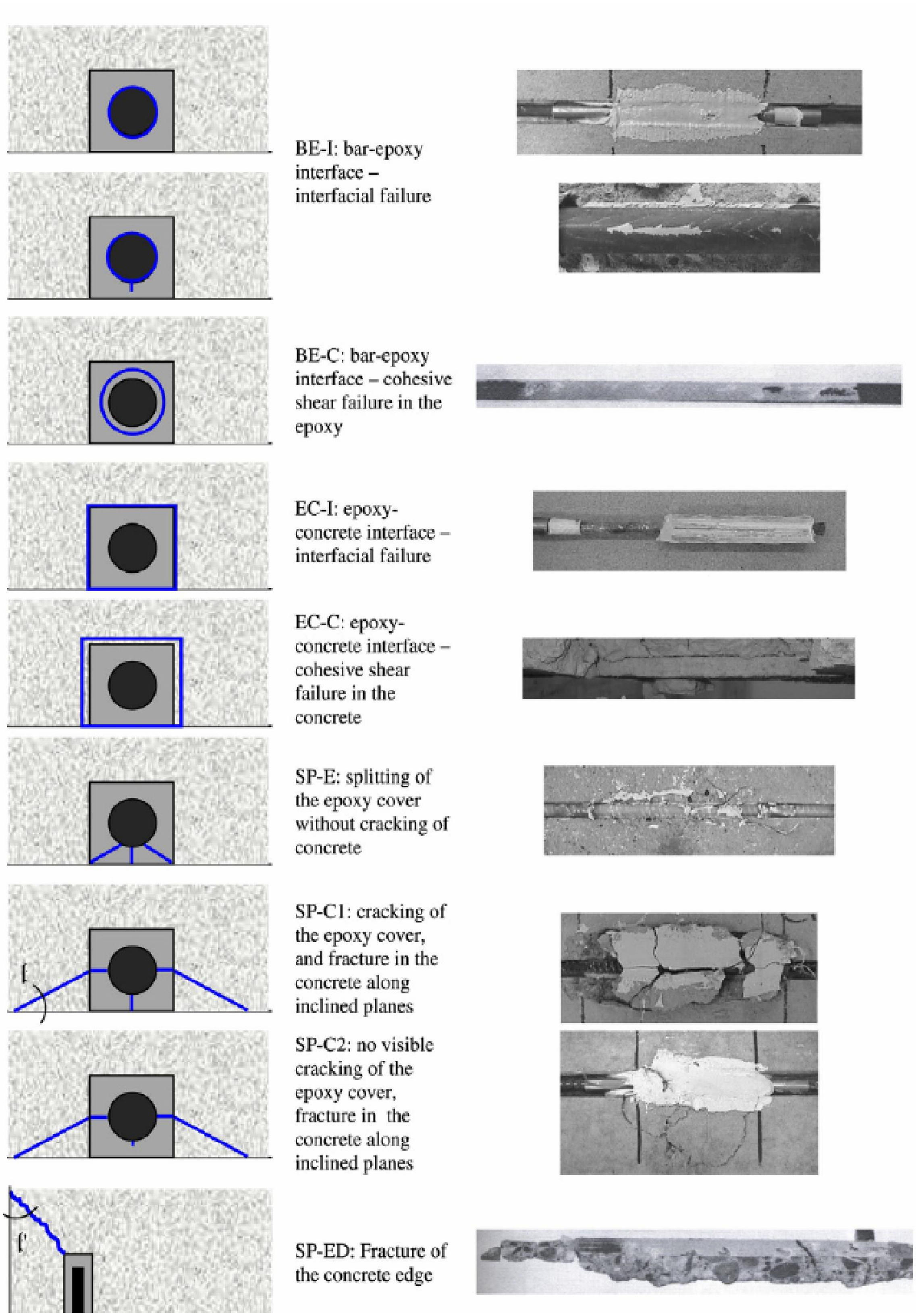


Fig 1.3 The various bond failure modes of NSM system observed in various bond tests[1]

Different types of bond failures are shown in figure 1.3

BE-I (B= bar, E= epoxy/ groove filler and I= pure interfacial failure) represents pure interfacial failure at the bar epoxy interface and this happens mostly when the bars are smooth or lightly sand blasted i.e. the degree of surface roughness is no sufficient to provide proper interlocking between the groove and the groove filler and as we know bond resistance primarily depends on the adhesion between bar and the groove filler.

BE-C (B= bar, E= epoxy/ groove filler and C= cohesive shear failure) represents cohesive shear failure in epoxy at the bar epoxy interface and this is mostly observed in NSM strips with roughened strip when the shear strength of the epoxy is exceeded.

EC-I (E= epoxy/ groove filler, C= concrete and I= pure interfacial failure) represents pure interfacial failure at the epoxy concrete interface and is critical for precast grooves or when the NSM spirally wounded bars or ribbed bars were with high rib protrusions. And when the ribs were with high protrusions this mode was critical k values were larger than a minimum value.

EC-C (E= epoxy/ groove filler, C= concrete and C= cohesive shear failure) represents cohesive shear failure in concrete which had never occurred in the bond test but has been observed in the bending test of the beams.

Cover splitting is described as the longitudinal cracking of the groove filler or the fracture of the surrounding concrete along the inclined planes and is represented as SP. These types of the bond failure occur when either of concrete or the groove filler reaches its limiting values.

SP-E represents the longitudinal splitting of epoxy cover without cracking of concrete which occur when the k ratio is very low and the failure is limited to the epoxy only.

SP-C1 represents the longitudinal splitting of the epoxy cover and fracture in the concrete along the inclined plane. This happens for the higher k values. Fracture in concrete cover starts as soon as the epoxy cover cracks and the tensile stresses are redistributed.

SP-C2 represents longitudinal splitting of concrete cover along the inclined plane with no visible cracking of the concrete cover. This happens when the depth of groove is too large or when the tensile strength ratio between concrete and epoxy is small.

SP-ED represents the splitting of edges of a concrete member when the NSM bars are close to the edges.

All the matter discussed above is about the average bond strength and which usually decreases with the increase in the bond length as a result of non-uniform distribution of bond stresses the local bond strength refers to the maximum value of bond stress that the interface can resist which is different from the overall bond strength which is the maximum transferable load of the joint. Bond stress is maximum at the loaded end and minimum at the free end as in figure 1.4.

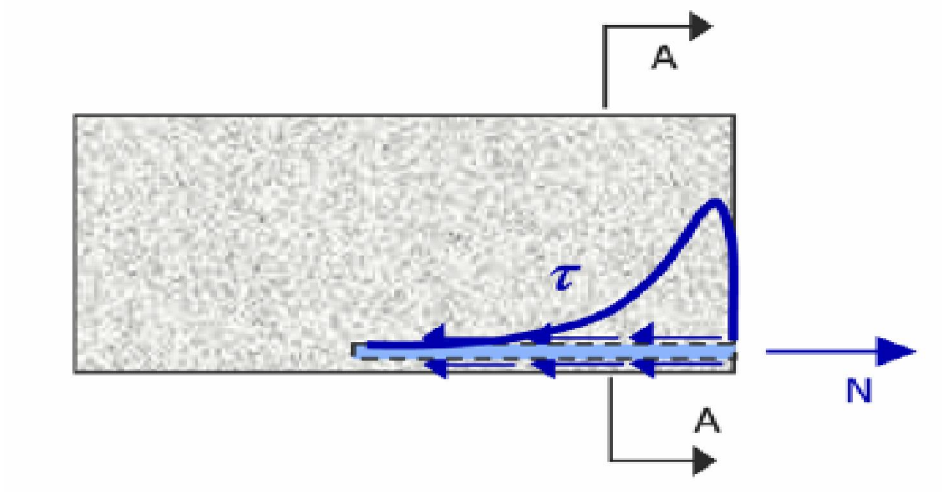


Fig 1.4 : The bond stresses in the longitudinal plane[1]

1.3 Strengthening applications of NSM

1.3.1 Flexural strengthening:

It is the most popular application of NSM reinforcement. Researches have been carried out with the variations in the various parameters to know the behavior of the flexural strengthened members using NSM. Various parameters studied were like shape of the reinforcement, embedment lengths, type of groove filler, steel ratio, and number of NSM bars, beam size etc. and the results observed and concluded of the researches were as the following.[1, 12]

1. Test beams with equivalent amount of NSM reinforcement as strips and round bars the beams with NSM strips performed better as they failed at higher strains by tensile rupture as a result of higher local bond stress.

2. In comparison to externally bonded reinforcement NSM behaved better and the debonding occurred at higher strains in NSM.

3. As for the embedment length the mode of failure was changed from concrete cover separation to concrete crushing followed by secondary cover separation.

The observations made in these researches regarding the failure mode of the flexural strengthened beams with NSM were mainly of two types which included crushing of surrounding concrete / NSM reinforcement rupture after yielding of internal steel or premature debonding failure due to loss of composite actions. The debonding failure could happen in any of the following ways

1.Bar – epoxy interfacial debonding;

2.Concrete cover separation;

3.Epoxy – concrete interfacial debonding

1.3.2 Shear strengthening:

NSM strengthening is also effective in shear strengthening of RC members. For this purpose the beam is embedded with NSM reinforcement into the groove cut on the sides of the member. The inclination of the NSM can be varied wrt. the beam axis. The studies carried out to find the behavior of the NSM in shear strengthening have studied shear strengthening wrt. the parameters like bar spacing, inclination angles and anchorage of the bars in the flange. There was a significant increase in the shear strength of the beam. All the results shows that the beam without internal shear reinforcement got failed in bending in due to NSM shear strengthening, instead of failing suddenly due to shear. [1, 13]

The failure modes observed were debonding of the NSM bars by splitting of epoxy cover and cracking of surrounding concrete along with diagonal tension failure of concrete. This failure was prevented by providing NSM reinforcement, at a proper spacing and inclination, which does not exert any restraining action on the longitudinal reinforcement subjected to dowel action. These forces combined with the normal pressure due to bond action of the tensile steel, create a lot of tensile stresses in the cover region and ultimately lead to cover separation failure. This application of shear strengthening of the beams weak in shear due to omission of internal shear stirrups using NSM steel strips and NSM round steel bars is the part of our report.

To find the contribution of the NSM reinforcement it has been given in the paper by De Lorezis [1] that the nominal shear strength of beams externally strengthened with FRP is given by

$$V_n = V_c + V_{us} + V_{FRP} \qquad \text{Equation (1)}$$

Where Vn is the shear strength of the member, Vc is the nominal shear strength of the concrete, Vs shear strength contribution of the shear stirrups, and V_{FRP} is the shear strength contribution of the NSM FRP reinforcement.

Here in our case we will be using term V_{FRP} as a generalized term for NSM reinforcement.

Here Vc can be calculated using

$$V_{uc} = \tau_{uc} b\, d \qquad \text{Equation (2)}$$

Where, "τ_{uc}" is the ultimate shear strength of concrete which depends on percentage of steel in beam section and characteristic strength (f_{ck}) of the concrete, "b" is the width of the member, and d is the effective depth of the member.{IS 456: 2000}[3]

Or it can be calculated by

$$V_c = 3.5\sqrt{f_c}\, b_w d \qquad \text{Equation (3)}$$

Where" fc" is the compressive strength of the concrete in psi (1MPa ≈ 145psi), "b_w" and "d" are the web width and depth in inches (1inch = 25.4mm), thus "Vc" is in kips (1kips ≈ 4.5 kN)[13, 14]

And Vus is given by

$$V_{us} = \frac{0.87\, f_y A_{sv} d}{S_v} \qquad \text{Equation (4)}$$

Where "f_y" is the yield strength of the steel in MPa, "Asv" is the cross-sectional area of shear stirrups in mm², "d" is the effective depth of the section in mm and "Sv" is the spacing of the shear stirrups in mm.[3]

For the calculation of the V_{FRP} the preliminary approach in the paper [13] includes two equations. The first equation computes the contribution of NSM shear strength related to the bond controlled failure "V_{F1}". The shear resisted by the NSM reinforcement may be computed as the sum of the forces resisted by a NSM rods intersected by the shear crack may be ideally divided in two parts at two sides of the crack. The force in each of these rods at the crack location can be calculated as the product of the average bond strength and the surface area of the shortest part and which is given by

$$V_{F1} = 2\sum A_i f_i = 2\pi . d_b . \tau_b . L_{tot\ min}$$ Equation (5)

Where "Ai" is the nominal cross-sectional area of the rods, and "fi" is the tensile stress in the rod at the crack location, and summation extended by all the rods crossed by a 45 crack, "$L_{tot\ min}$" is the sum of effective lengths of all the rods crossed by the cracks, "d_b" is the nominal diameter of the rod and "τ_b" bond strength [13]

The "$L_{tot\ min}$" depends on d_{net}, on spacing s of the NSM shear reinforcement, and on their inclination. And for vertical rods it is given by

$$L_{tot\ \min} = d_{net} - s \text{ , if } \frac{d_{net}}{3} < s < d_{net}$$ Equation (6)

$$L_{tot\ \min} = 2d_{net} - 4.s, \text{ if } \frac{d_{net}}{4} < s < \frac{d_{net}}{3}$$ Equation (7)

Where "s" is the spacing of the rods, and "d_{net}" is the reduced length of the NSM rods.

$$d_{net} = d_r + 2.c$$ Equation (8)

Where d_r is the height of the shear strengthened part of the cross-section and c is the concrete cover of the internal longitudinal reinforcement.

The second equation to calculate the shear contribution of the NSM rods calculates the shear resisted by the NSM rods when the maximum strain in the rods. Is equal to 4000με, and is indicated as V_{F2}. This limiting value was suggested by Khalifa [13, 15].

In this part of contribution by NSM in shear the effective length of the NSM rod corresponding to the strain of the 4000με, and to the average bond strength τ_b can be obtained by the equilibrium.

$$L_i = 0.001\frac{d_b.E_b}{\tau_b} \quad \text{Equation (9)}$$

Where d_b is the nominal bar diameter, E_b is the elastic modulus of NSM rods, And τ_b average bond stress.

The value of V_{F2} in different cases for different ranges of spacing is given as

For $d_{net}/2 < s < d_{net}$ and $L_i > d_{net} - s$ the bond failure is controlled by V_{F1} and occurs before maximum strain is4000με.

And if $L_i < d_{net} - s$ then the bond failure is controlled by V_{F2} and is given by

$$V_{F2} = 2\pi d_b.\tau_b.L_i \quad \text{Equation (10)}$$

For $d_{net}/3 < s < d_{net}/2$; V_{F1} controls if $L_i > s$ and if $L_i < s$ V_{F2} controls and is given by

$$V_{F2} = 2\pi d_b.\tau_b.(L_i + d_{net} - 2s) \quad \text{Equation (11)}$$

For $d_{net} - 2s < L_i < s$; if $L_i < d_{net} - 2s$

$$V_{F2} = 4\pi d_b.\tau_b.L_i \quad \text{Equation (12)}$$

For $d_{net}/4 < s < d_{net}/3$; V_{F1} controls if $L_i < d_{net} - 2s$. And if $L_i > d_{net} - 2s$ V_{F2} controls and is given

If $s < L_i < d_{net} - 2s$

$$V_{F2} = 2\pi d_b.\tau_b.(L_i + d_{net} - 2s) \quad \text{Equation (13)}$$

And if $d_{net} - 3s < L_i < s$

$$V_{F2} = 2\pi d_b.\tau_b.(2L_i + d_{net} - 3s) \quad \text{Equation (14)}$$

So from the above equations we can estimate the shear strength of the NSM strengthened beam with different variations in spacing. The shear strength contribution of the NSM reinforcement is given by the minimum of V_{F1} and V_{F2} our case study the beam was strengthened to have the shear bond failure and was controlled by V_{F1}.

2. Specimens

In this study, a total of four RC beams of size 150 mm x 250 mm x 2050 mm were cast in the laboratory and the reinforcement details of the beam are illustrated in Fig. 1, 2, 3. The internal shear stirrups were deliberately omitted in the $1/2^{th}$ portion of the beam for two beams (S, S1) (Fig. 2) and for one beam casting was carried out without providing any shear stirrups (NS) (Fig. 3). And the fourth beam was casted with shear reinforcement throughout the beam (R) (Fig. 1). Thus, a total of 4 tests were conducted using the four beams and they are designated as shown in Table 1. The main parameter varied in this study was the type of the NSM reinforcement i.e., steel strips and steel rods that were used to strengthened the beam on both faces of the no ring side of the beam. One beam with half side shear stirrups was strengthened with steel strips and the beam entirely without any shear reinforcement was strengthened with steel bars. The matrix material used in both the cases was 1:4 epoxy–sand mortar. The procedure of preparation of the test specimen is as following.

2.1 Preparation of test specimen

Initially four beams (R, S, S1, and NS) were casted in laboratory as shown in figures (2.12, 2.13, and 2.14). Each beam was casted with the variation of shear stirrups as shown in figures. The strain gauges were also fixed to the main reinforcement and shear stirrups at various locations to keep a record of the strain in different locations. Beam R was tested as was casted, no other additional reinforcement/strengthening was done on it.

2.1.1 Strengthening of the beam “S”:

Beam S was casted with half side shear stirrups omitted on purpose to achieve a shear failure. The beam was observed with a hairline crack at its mid span, it was vertical and all around the beam. The beam was earlier to be tested for shear failure without any kind of strengthening. But due to this crack which could have occurred during the time of lifting and placing during curing period, this beam was strengthened as follows.

1. The beam was overturned to work on its bottom face.
2. A groove of depth 15mm and 20mm wide was cut throughout the length along the centre of the bottom face using a cutter.
3. The groove was cleaned for any loose material with an iron wire brush and was blasted with compressed air to remove all the dust.
4. The groove and NSM steel bar was then primed using epoxy adhesive to wet the surface of the groove to impart good adhesion for the bonding between the interface of concrete-mortar and mortar-NSM.
5. 1:5 epoxy-sand mortar was prepared and was poured into the groove for small depth and the NSM steel rod of 10mm diameter was placed and pressed into it and was latter completely filled with mortar and its surface was leveled.
6. The web surface around the crack was cleaned and the crack was later covered with GFRP wrap around the crack using epoxy resin as adhesive.
7. The beam was thus strengthened for the crack as shown in fig. xx and was left to set till testing was done and before the testing the beam was painted to have good view of cracks and the test specimen RWS (reference beam without stirrups) was prepared by retrofitting beam “S” using NSM steel rod in flexure and GFRP wrap.

Fig 2.1 NSM Strengthened beam "S"

2.1.2 Strengthening of the beam "S1":

The beam S1 was casted in similar conditions as for beam S. It was also without shear stirrups on its half span. And it was also installed with strain gauges at the main reinforcement. But this beam was strengthened using the NSM steel strips. The beam was strengthened for testing as following.

1. The beam was laid on one of its side and assumed crack position was marked on it. The crack was assumed along the line joining the support line at support and assumed load position which was kept at 70mm from the support. The slope of the crack line was 19.8° to the horizontal.

2. After marking the crack position the position of the strip was marked perpendicular to the crack line at a spacing of 85mm centre to centre.

3. After marking a groove was cut along the strip line was widened using the cutter (fig. 2.2). Depth of cut was kept 20mm and the average width of the groove was 8mm. eleven such grooves were done on each side.

4. Mean while the NSM steel strips (Fig. 2.3) 18mm wide and 0.8mm thick was cut into equal length which was equal to 27mm the length of the groove. 22 such pieces were cut into the required length.

5. These NSM steel strips were then roughened using grinding stone to remove the external coating to have rough surface for good adhesion and the strip was later cleaned using benzene solution.

6. After cleaning the strip it was marked for its centre and stain gauge was fixed using glue at its centre and was later waterproofed.

7. The groove was cleaned for any loose material with an iron wire brush and was blasted with compressed air to remove all the dust (Fig. 2.6).

8. The groove and NSM steel strip was then primed using epoxy adhesive to wet the surface of the groove to impart good adhesion for the bonding between the interface of concrete-mortar and mortar-NSM strip.

9. The NSM strip was then placed in the centre of the groove and the spaces was filled using the 1:4 epoxy-sand mortar and the mortar was hand compacted using thin strips and was leveled with the concrete face (Fig. 2.4).

10. The beam was then kept in same position for minimum of 72 hours to give time to epoxy mortar to set. After setting of epoxy the beam was over tuned and other side was also strengthened for shear using NSM steel strips in the similar manner.

11. The beam was later grinded of the excessive epoxy on surface (Fig. 2.5) using grinder and was painted before testing to have good view of cracks and the test specimen B1-SS was prepared using NSM shear strengthening technique using NSM steel strips.

Following figures shows the steps and instrument for preparation of beam S1.

Fig. 2.2 The groove cutter and groove cutting

Fig. 2.3 The NSM strip

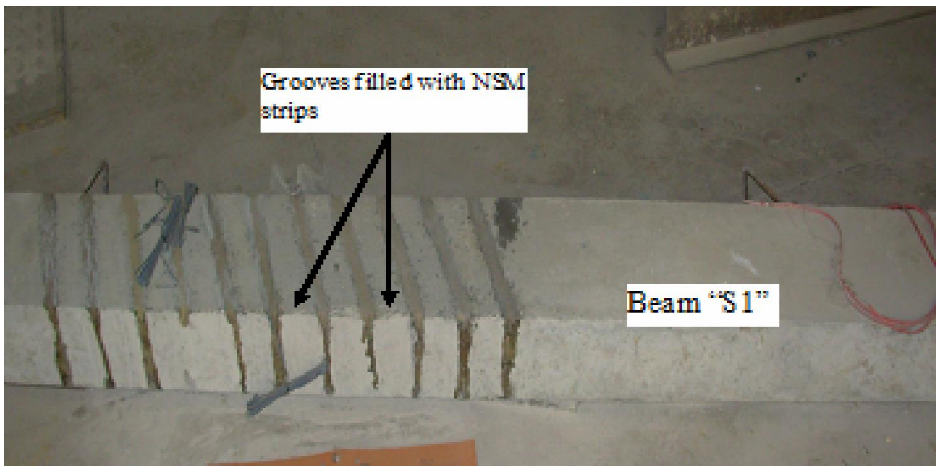


Fig. 2.4 Beam S1 strengthened with NSM strips

Fig 2.5 Grinding of beam surface at strengthened side to remove excessive mortar

Fig 2.6 Air compressor used for cleaning the grooved surface

2.1.3 Strengthening of the beam "NS"

The beam NS was casted without any shear reinforcement and was internally fixed with strain gauges at its mid span. This beam was casted for the purpose of strengthening the beam with external shear reinforcement using NSM steel rods of 10mm diameter. The NSM steel rods were also fixed with strain gauges to record the strain during the test. Following are the steps performed to prepare the specimen.

1. The beam was laid on one of its side and assumed crack position was marked on it. The crack was assumed along the line joining the support line at support and assumed load position which was kept at 70mm from the support. The slope of the crack line was 19.8° to the horizontal.

2. After marking the crack position the position of the strip was marked perpendicular to the crack line at a spacing of 100mm centre to centre.

3. After marking a groove was cut along the strip line was widened using the cutter. Depth of cut was kept 20mm and the average width of the groove was 20mm, sixteen such grooves were done on each side (Fig. 2.7).

4. Meanwhile the NSM steel rod 20mm diameter cut into equal length which was equal to 27mm the length of the groove, using hand operated saw-toothed blade. 32 such pieces were cut into the required length.

5. These NSM steel rods were then roughened using sand-paper to remove the rust and dirt to have clean rough surface for good adhesion and the strip was later cleaned using benzene solution.

6. After cleaning the rod it was grinded with the grinder at its centre to make a plane seat for fixing the strain gauge and the stain gauge was fixed using glue at the seat at centre of rod and was later waterproofed using the same glue (Fig. 2.8).

7. The groove was cleaned for any loose material with an iron-wire brush and was blasted with compressed air to remove all the dust.

8. The groove and NSM steel rod was then primed (Fig. 2.9) using epoxy adhesive to wet the surface of the groove to impart good adhesion for the bonding between the interface of concrete-mortar and mortar-NSM rod.

9. The NSM rod was then placed in the centre of the groove after filling the groove to its half and was pressed to have proper bond with 1:4 epoxy-sand mortar under it and then the spaces was filled using the epoxy-sand mortar and the mortar was hand compacted using thin strips and was leveled with the concrete face (Fig. 2.11).

10. The beam was then kept in same position for minimum of 72 hours to give time to epoxy mortar to set. After setting of epoxy the beam was over tuned and other side was also strengthened for shear using NSM steel rods in the similar manner.

11. During preparation of the specimen one of the internal wires connecting strain gauge was damaged and was repaired by cutting the concrete around the wire and was connected to new wire and the space was filled using epoxy-sand mortar (Fig. 2.10).

12. The beam was later painted before testing to have good view of cracks and the test specimen B2-SR was prepared from beam NS using NSM shear strengthening technique using NSM steel rods.

Following are the figures showing the steps for preparing the specimen.

Fig 2.7 Cutting of grooves

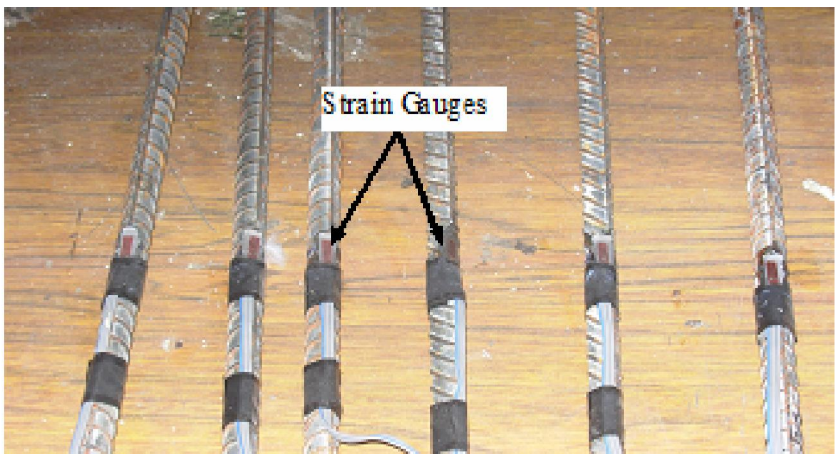


Fig 2.8 NSM rod fixed with strain gauges

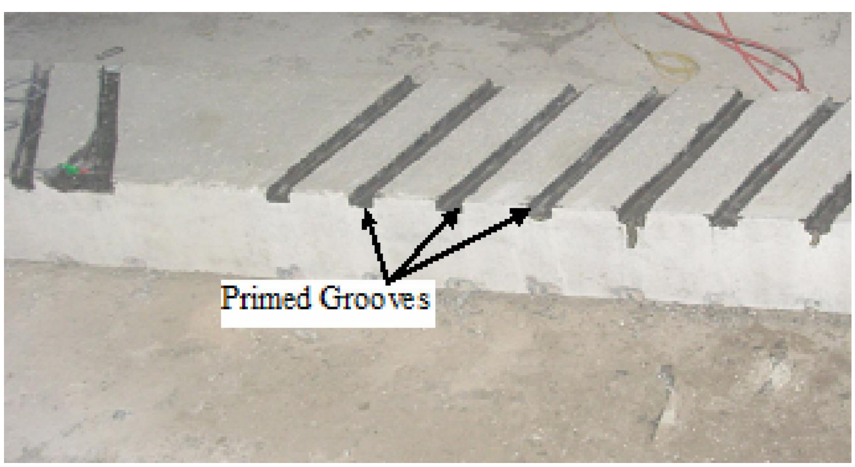


Fig 2.9 Primed grooves

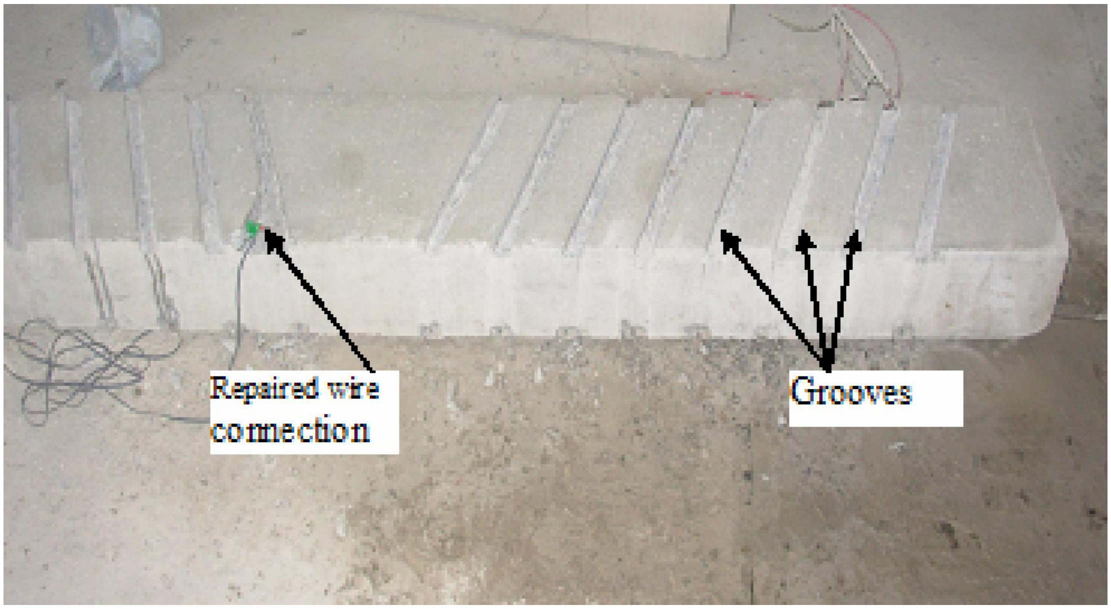


Fig 2.10 Grooved beam with repaired wire

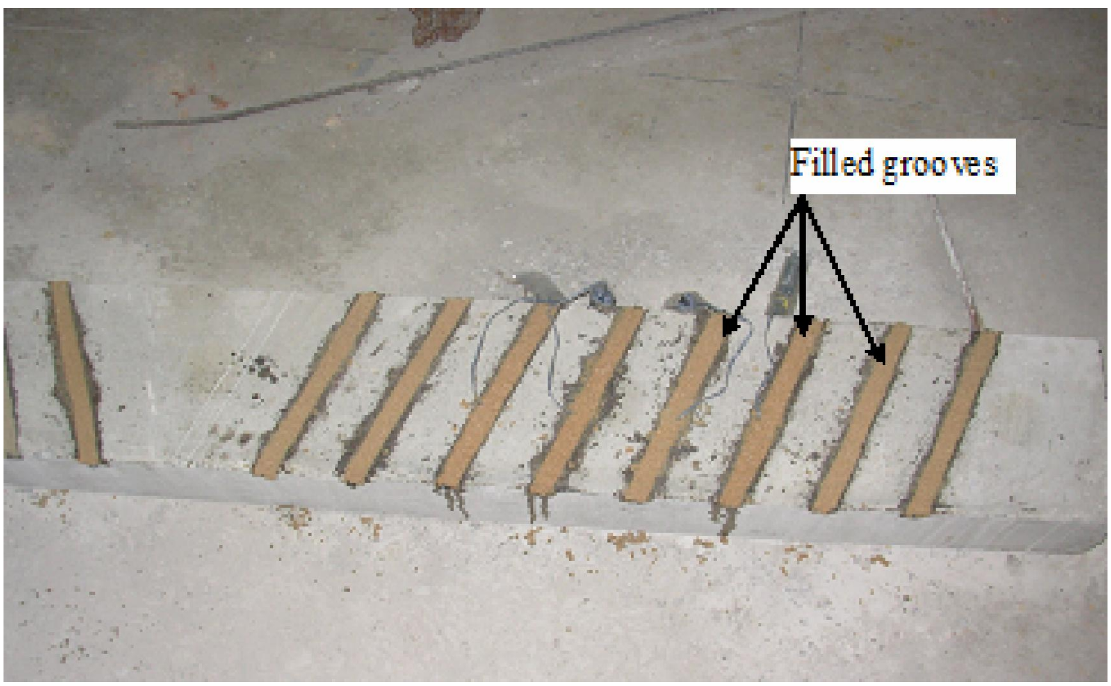


Fig 2.11 Grooves filled with NSM steel rods using epoxy-sand mortar

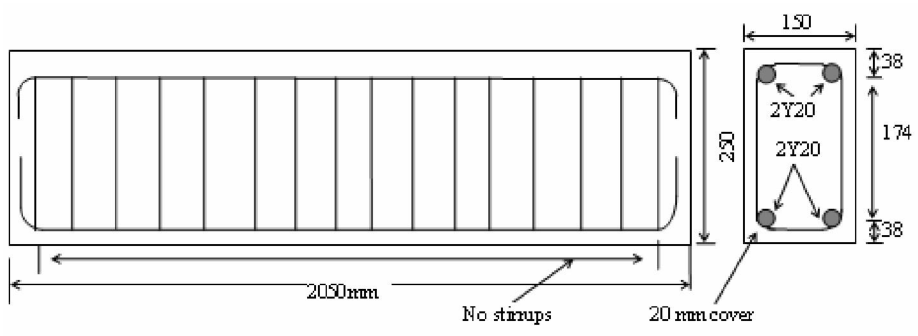


Fig 2.12 Details of reinforcement in reference specimen(R)

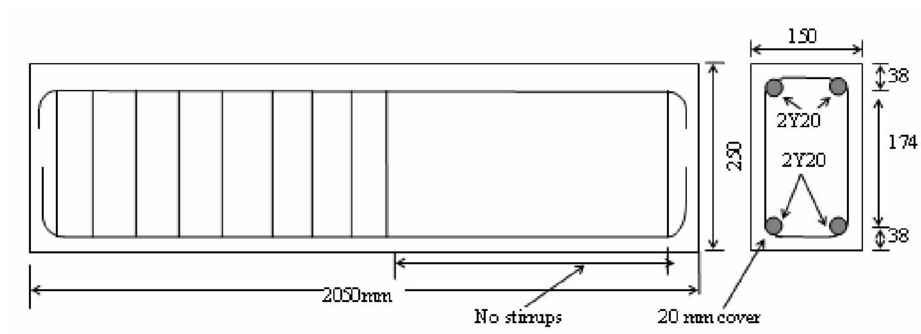


Fig 2.13 Details of reinforcement in specimen(S, S1)

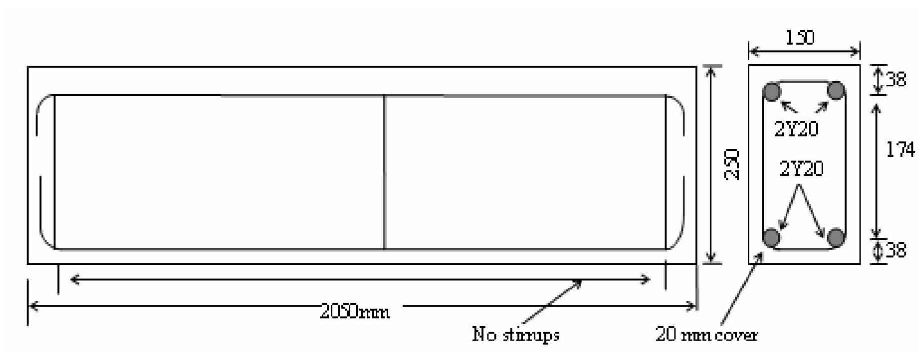


Fig 2.14 Details of reinforcement in specimen without shear reinforcement (NS)

Table 1 Details of test conducted

Sr. No	Test	NSM type	Beam Configuration	Stirrups provision	Span (mm)	Figure No.
1	RWS	Steel rod on bottom face	Reference beam	Provided on half side	1800	3.3
2	B1 – SS	Steel strips	Retrofitted beam (DSS)	Provided on half side	1800	3.1
3	B2 – SR	Steel rods	Retrofitted beam (DSR)	No	1800	3.2
4	BSR	No NSM	Reference beam	Yes	1800	3.4

Note: DSS stands for diagonal shear steel strips; and DSR stands for diagonal shear steel rods

3. Test set up

The casted beams (S1 and NS) were first strengthened with NSM steel strips (Fig. 3.1), with NSM steel rods (Fig. 3.2). And beam (S) was retrofitted with NMS steel bar at the tension face and with GFRP Wrap at the mid span (Fig. 3.3). Fourth beam was tested as it was casted (Fig 2.12). Then the test beams were tested simply supported over effective span 1800mm as shown in Figures 3.4, 3.5, 3.6, 3.7. And the load was applied on the top of the beam by a hand operated hydraulic jack through a load cell placed over the load plate or through a proving ring. The ratio of the shorter shear span (700 mm) to the effective depth of the beam (212 mm) was kept greater than 3 to avoid an increase in shear strength due to short span tied arching action and to develop a diagonal shear crack in beam web.

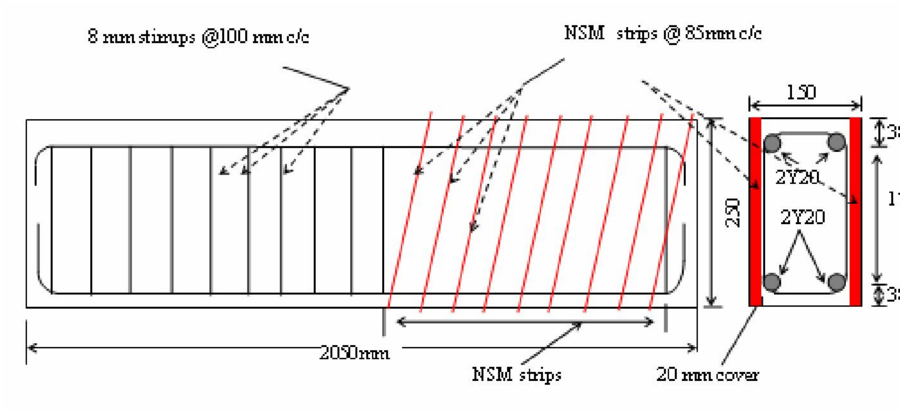


Fig 3.1 Arrangement of NSM steel strips in beam (S1)

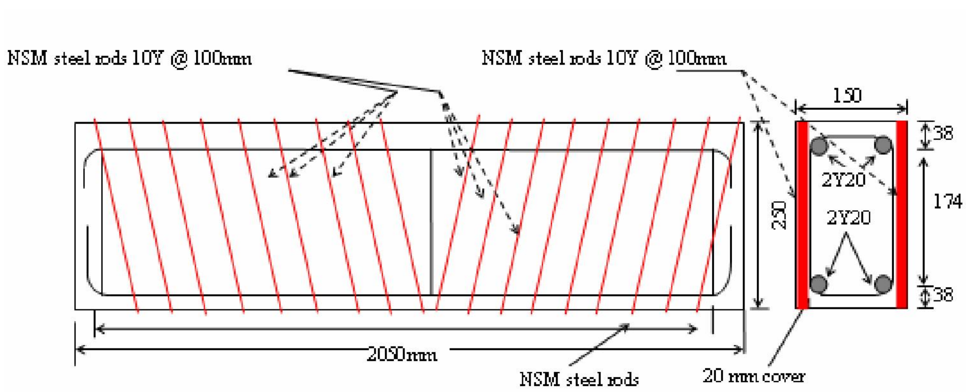


Fig 3.2 Arrangement of NSM steel rods in beam (NS)

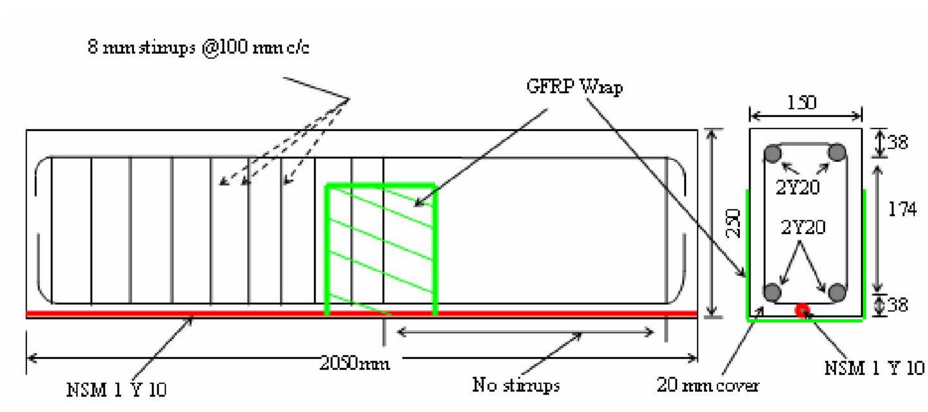


Fig 3.3 Arrangement of NSM rod and GFRP Wrap in retrofitted beam(S)

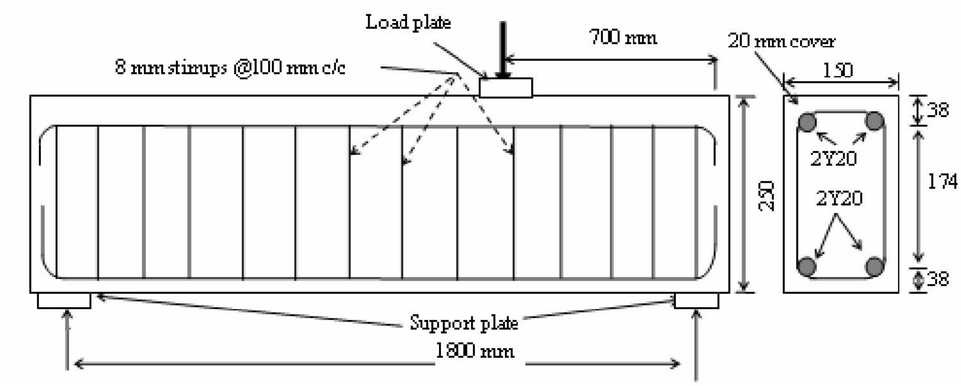


Fig 3.4 Loading arrangement of test reference beam (R)

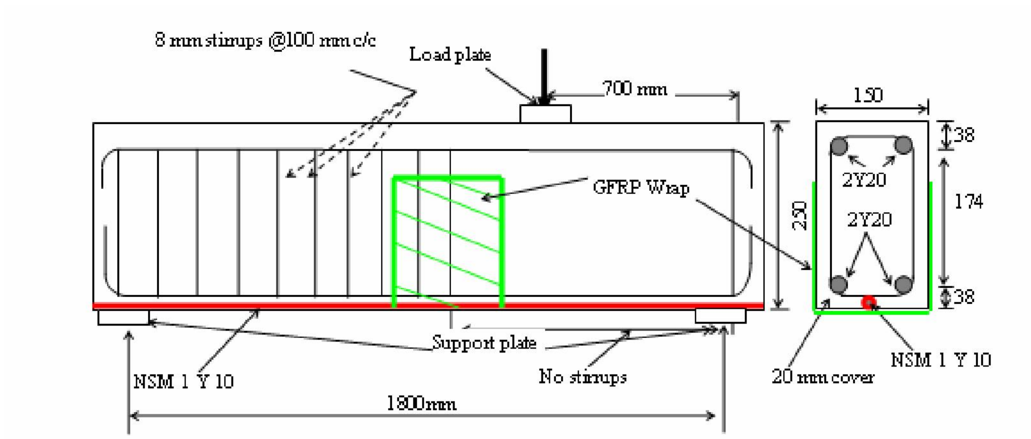


Fig 3.5 Loading arrangement of test reference without shear stirrups beam (RWS)

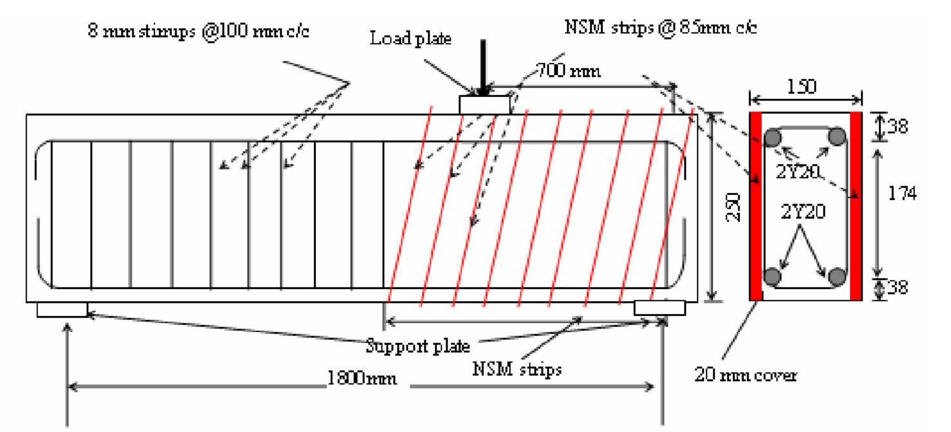


Fig 3.6 Loading arrangement of test beam with NSM steel strips (B1-SS)

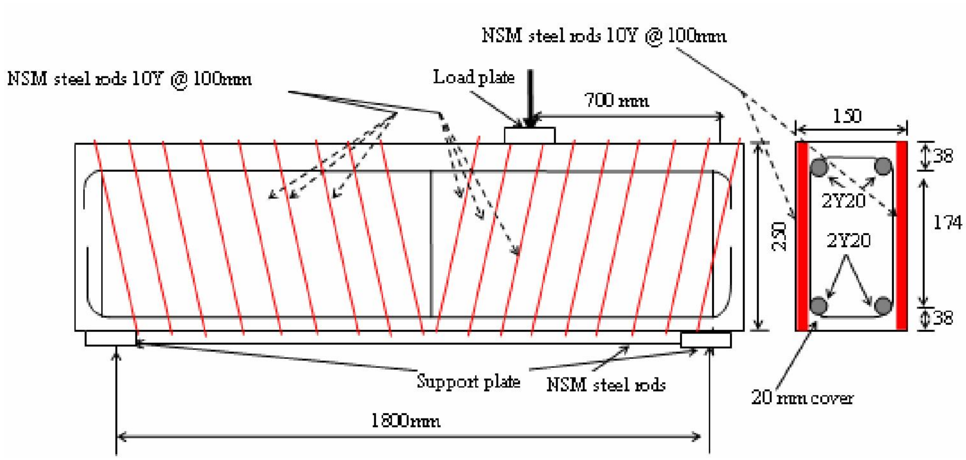


Fig 3.7 Loading arrangement of test beam with NSM steel rods (B2-SR)

4. **Instrumentation**

Electrical resistance strain gauges of resistance 120 ±0.2 ohms, grid length 2 mm with overall dimensions 5 x 3 mm were bonded to the longitudinal steel reinforcement and to the shear stirrups before casting of the beams to measure the magnitudes of the strain variations at various stages of the applied load. Before installing the strips and bars into the groove the strain gauges were also bonded to the strips and bar at the centre of their length in order to detect the debonding of the strips and rods at various sites. The arrangements of the strain gauges that were bonded to the steel reinforcement, NSM strips/rods and their numbering are shown in Figs. 4.1 to 4.6 for all the beams tested in the present study. The strain gauges are designated as SG-1, SG-2, and SG-3 etc., where SG stands for the strain gauge and the numerals after hyphen indicates the number of the gauge location. The beam deflection under the load point was monitored with a dial gauge.

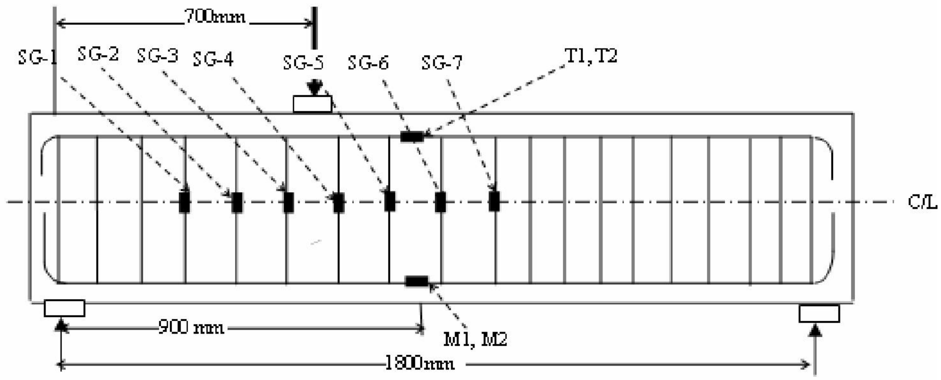


Fig 4.1 Arrangements of internal strain gauges in reference beam(R)

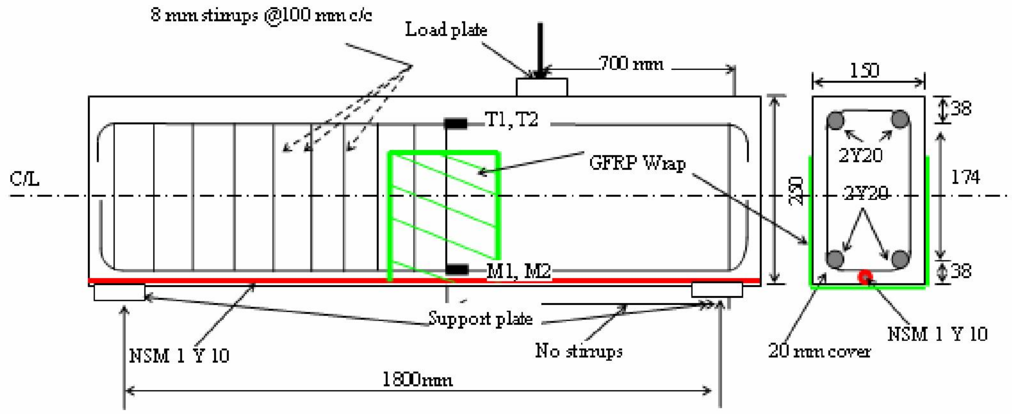


Fig 4.2 Arrangements of strain gauges in reference ring beam without shear stirrups (RSW)

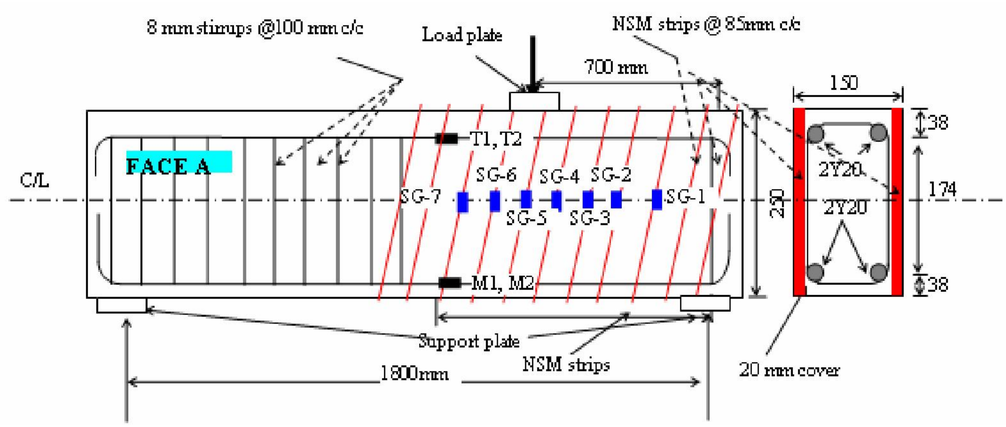


Fig 4.3 Arrangement of strain gauges on main bars and on NSM Steel Strips (Face A) (B1-SS)

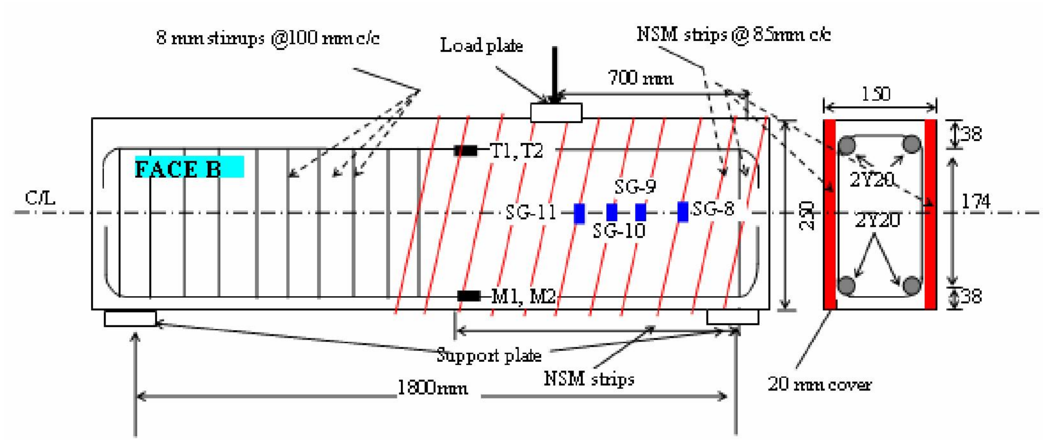


Fig 4.4 Arrangement of strain gauges on main bars and on NSM Steel Strips (Face B) (B1-SS)

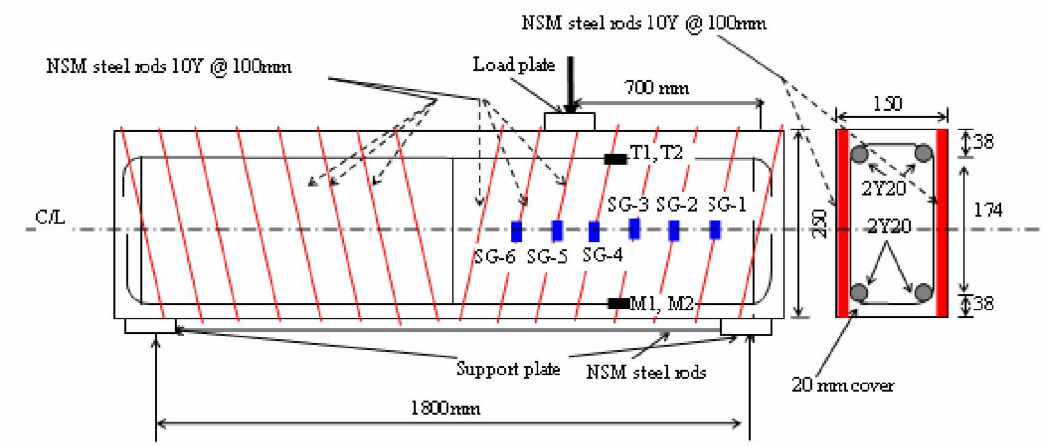


Fig 4.5 Arrangement of strain gauges on internal bars and NSM steel rods (Face A)

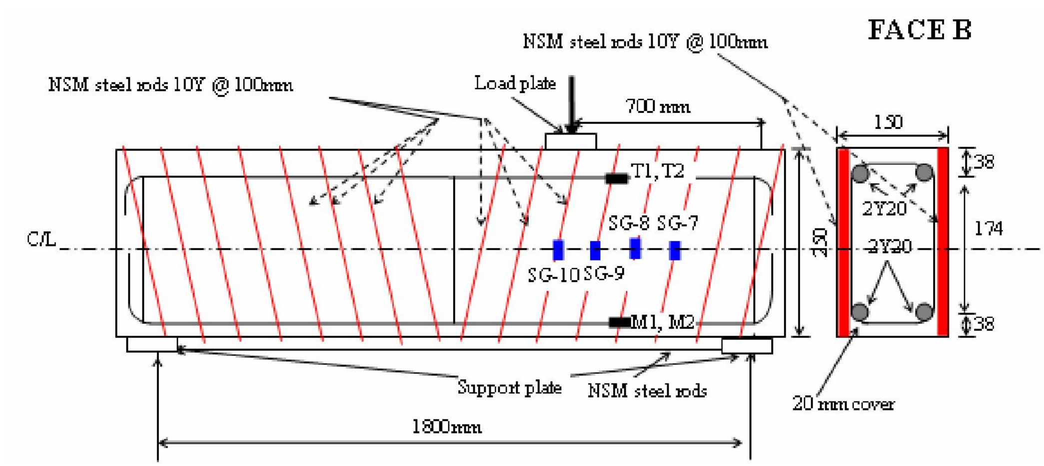


Fig 4.6 Arrangement of strain gauges on internal bars and NSM steel rods (Face B)

5. Material Properties

The beam specimens were cast using a concrete designed mix proportion of 1.0 (ordinary Portland cement): 1.44 (sand): 2.94 (coarse aggregate of 10 mm maximum size), all by weight, with a water-cement ratio of 0.422. (IS: 10262 -1982) [16]. The cube (150mm x150mm x 150mm) compressive strength (f_{cu}), Brazilian tensile strength (f_b) (cylinder of150 mm diameter and 300 mm length) and the elastic modulus (E_c)

5.1 Test for Compressive strength

The compressive strength of the concrete was tested as prescribed in test no. 5 in code IS: 516 – 1959 [17]. Three cubes of 150mm x150mm x 150mm size were casted in the laboratory for every beam casted. The cubes were casted at the time of casting beams and were tested under a compression testing machine on the day of testing to have the concrete strength at the day of testing. All the cubes were casted in the proportion of 1: 1.44: 2.94. Loading was done at the rate prescribed in the 140 kg/cm²/min i.e. 31.5 t/min. The setup is as shown in Fig 5.1. The crack pattern is as shown in Fig 5.2.

Fig 5.1 Test setup for compressive strength

Fig. 5.2 Compressive failure of cube

5.2 Test for tensile strength of concrete

For tensile strength of the concrete "Brazilian tensile strength" (split cylinder test) was conducted on the cylinders under a compression testing machine as prescribed in IS: 5816 – 1999 [18]. Three representative cylinders of diameter 15mm and height of 30mm was casted while the beams were casted. The cylinders were tested on the testing of beams. Loading was done at the rate prescribed in the 140 kg/cm²/min. i.e. 31.5 t/min. The cylinder is kept in laid position and load is applied along its diameter till the cylinder splits into two parts. The setup of the test, the crack, and the split cylinder are as shown in the figures 5.3 (a), 5.3(b), 5.3(c) and 5.4. Tensile strength (f_t) is given by the following

$$f_t = 2F / \pi L d \qquad \text{Equation (15)}$$

Where F is the load at which cylinder splits in Newton,

L is the length of cylinder and

d is the diameter of the cylinder

(a) (b) (c)

Fig 5.3 (a) Split cylinder test setup (b) crack along the diameter (c) split cylinder

Fig 5.4 Split surface of cylinder

5.3 Test for E (Modulus of Elasticity) of concrete

This test was conducted on a cylinder as described in the test no. 9 in code IS: 516 – 1959 [17]. The cylinder were tested for "E" of concrete and was later was used as the specimen for the split cylinder test. Test was conducted by means of an extensometer.

The test procedure is as the following:

1. First wet the cylinder for a while.
2. Then attach the extensometer parallel to its axis.
3. Now place the cylinder in the testing machine (Fig 5.5). The top and bottom surface of the cylinder should be plane.
4. Now the load is applied at a rate of 140 kg/cm²/min. (14 MPa), which in our case was 24.7 t/min.
5. First the load is applied upto (C+5) kg/cm² which in our case was 178.3kg/cm² i.e. 31.5 tonnes.
6. Now the load was reduced to 1.5 kg/cm² and the dial gauge reading attached to extensometer was recorded as the initial reading.
7. Now another cycle of the loading was done and load was raised to (C+1.5) kg/cm² i.e. 174.8 kg/cm² i.e. 30.9 tonnes and the reading at the dial gauge is recorded as the final reading.
8. Following the above cycle load is again reduced to 1.5 kg/cm² and another cycle of loading and unloading is carried out.
9. The dial gauge readings are recorded in 10 to 12 equal intervals.
10. And if the readings of 2nd and 3rd cycle are same graph between stress and strain is plotted and the slope of the graph is calculated.
11. The modulus of elasticity is calculated as following

$$E = \frac{stress}{strain} = \frac{w\,L}{A.\Delta L}$$ Equation (16)

Where w = load (C+1.5) – 1.5 kg/cm² and

$$C = \frac{f_{ck}}{3}$$ Equation (17)

Where, f_{ck} *is characteristic strength of the concrete.*

L = gauge length of the cylinder = 250mm in our case

ΔL = total deflection of the cylinder = final gauge reading – initial gauge reading in mm.

A = cross-sectional area of the cylinder mm²

All the values of the cube (150mm x150mm x 150mm) compressive strength (f_{cu}), Brazilian tensile strength (f_b) (cylinder of150 mm diameter and 300 mm length) and the elastic modulus (E_c) are listed in Table-2. All the beams were cast in the laboratory on different day. Material properties of the concrete used in the beams are also listed in Table-2. In this table, the individual values for each test are given under the "i" column and their mean value under the "m" column. Material properties of the NSM steel strips/rods, reinforcing bars and shear stirrups are shown in Table-3. The material properties of the epoxy glue (manufacturer's specifications) used for preparing epoxy-sand mortar as a groove filler is indicated in Table-4.

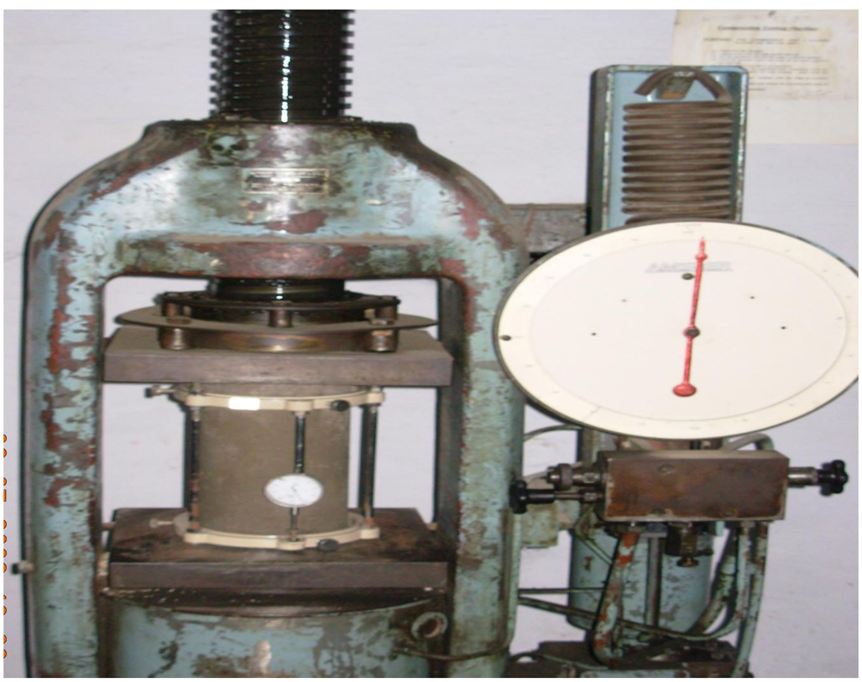


Fig 5.5 Test set up for the "E" of concrete using extensometer

Table-2: Material properties of the concrete

S.No.	Specimen Name	E_c (MPa)	f_b (MPa)		f_{cu} (MPa)	
		m	i	m	I	m
1.	RWS	38094	3.24	3.21	45.76	45.56
			3.18		45.36	
2.	B1-SS		3.04	3.11	41.77	42
			3.18		42.23	

Where, E_c= Elastic modulus of the concrete, f_{cu} = Cube compressive strength of the concrete, f_b = Brazilian tensile strength of the concrete

Table-3: Material properties of the glue Sikadur 32 (Sika hibond) (as per supplier)

Compressive strength 7days (ISO 604)	Elastic modulus in tension (ISO 527)	Tensile bond strength (ISO 527)	Maximum operating temperature
75 MPa	2354.4 MPa	>11 MPa	40^o C

Table-4: Material properties of the steel reinforcement and NSM

Item	f_y (MPa)			f_u (MPa)				E_p (GPa)
Sample No.	1	2	Mean	1	2	3	Mean	
Rebar Y20	415	414.8	415	598.0	580.0	-	589.0	200
Rebar Y10	-	-	415	667.8	661.3	665.0	665	200
NSM steel strips				566.7	567.3	-	567	200
NSM steel rods Y10			415	667.8	661.3	665.0	665	200
Shear stirrups Y8			415					200
GFRP Wrap 4mm				329	331	330	330	41

Where, f_y= yield strength of the steel reinforcement,

f_u= ultimate tensile strength of the steel/ FRP plate,

and E_p= elastic modulus of plate

6. Test Procedure

The tests were conducted on simply supported beams subjected to a single point concentrated load as shown in Figs. 3.4 to 3.7 and 4.1 to 4.6. For each test specimen, initially the load was applied in increments of 5 kN and the magnitude of the strain at different gauge locations, deflection under the load point corresponding to each increment of applied load were recorded. The propagation of the cracks was marked on the beams and the crack pattern was also photographically recorded. The load increment was kept constant at 5kN when a debonding crack was spotted and the same load increment was maintained till the complete debonding of the plate occurred.

The load positioning from the support was decided to be kept constant and was to be for ratio $a/d < 3$. Where "a" is the shear span the distance of load from the support and "d" is the effective depth of the specimen. This shear span represents the dominance of the shear over flexure. The ratio a/d is very important in terms of the type of the crack going to take place [19]. As $a = M/V$, where M = moment and V= shear, with its variation value of the M and V can be varied. Also diagonal tension is a function of the ratio a/d.

As for large values of a/d ($a/d > 6$); i.e. ($M/Vd > 6$) [19], the moment is predominant and the failure essentially occurs due to flexure (starting with yielding of main steel and finally by the crushing of the concrete) and formation of vertical cracks on tension side in the region of the maximum bending moment. This type of failure depends completely on the maximum bending moment.

For intermediate values of a/d ($3 < a/d < 6$), the failure takes place due to the combined effect of shear and bending. The initiation of crack is due to flexure and the crack is vertical and then it become inclined at higher load and the crack is known as flexure – shear crack or the diagonal crack. And on further increase in the load the crack become unstable and propagate in the compression zone to the load and finally the specimen splits into two segments. This mode of failure is specially known as diagonal – tension failure or shear – flexure failure.

For values of $a/d < 3$, the shear forces are predominant, the diagonal crack is formed in the concrete which is plain and is enclosed between longitudinal reinforcement. Beam generally remain stable after this cracking but on increasing the load further the cracking leads to shear failure in one of the following mode;

1. For value a/d lower but closer to 3 the diagonal crack instead of moving upward toward load stops propagating at some point and the crack propagate along the longitudinal reinforcement on the tension side towards the support. This results in failure of beam by splitting of the concrete along the reinforcement and pulling out of the main steel. This type of failure is known as shear – tension failure or shear – bond failure as the debonding of the bond between the longitudinal reinforcement and concrete takes place. The ultimate load causing this failure is not much higher than causing the diagonal crack. This kind of failure is prevented by providing the adequate anchorage and minimum steel beyond the support.

2. For values of a/d much lower than 3 the diagonal crack penetrates into the compression region at load higher than the load causing diagonal crack. The failure occurs by the crushing of the concrete which may be sometime explosive. This mode of failure is called shear – compression failure. The load at failure is almost 2 to 4 times the load causing diagonal crack. This kind of failure is prevented by limiting the maximum shear at the section.

To avoid the shear failure shear reinforcement is provided by transverse reinforcement. These may be provided in any of the following forms;

1. Vertical stirrups, or links,
2. Inclined stirrups making an angle greater than or equal to 45°, with the longitudinal steel.
3. Inclined bent-up bars obtain by bending the main steel into an angle of 45° or more with respect to the axis of the longitudinal steel can also serve the purpose.
4. Or the combination of the 1,2and 3 can be provided to serve the purpose.
5. Or in or case we have used NSM Steel Strips and NSM Steel rods.

Many more methods are there for retrofitting the beam for shear and have been discussed in the submitted minor report.

7. Test Results

The observations from all the tests conducted in this study are described in detail in the following sections. In the following discussion, the shear force *V* in the shear span being tested is used for better clarity and the total applied load *P* is given in the brackets.

7.1 Test RWS (reference without shear reinforcement, retrofitted with NSM bar and GFRF wrap)

As indicated in Table 1, that half the shear span of RWS was not provided with internal shear stirrups and was retrofitted with NSM steel rod Y10 at the tension face and with GFRP Wrap on the mid span on three faces of the beam. The retrofitting was done to repair the crack on the mid span of the beam which might had occurred during curing stage due to mishandling of the beam. The purpose of this test was to determine the shear strength of the concrete beam without shear stirrups. The test results serve as a datum for studying the shear strength and behavior of the NSM strengthened beams. Test RWS was carried out on the shear span without shear stirrups and the shear force V in the span was 0.61 P kN as shown in Fig. 17. The test setup was as shown in Fig .18

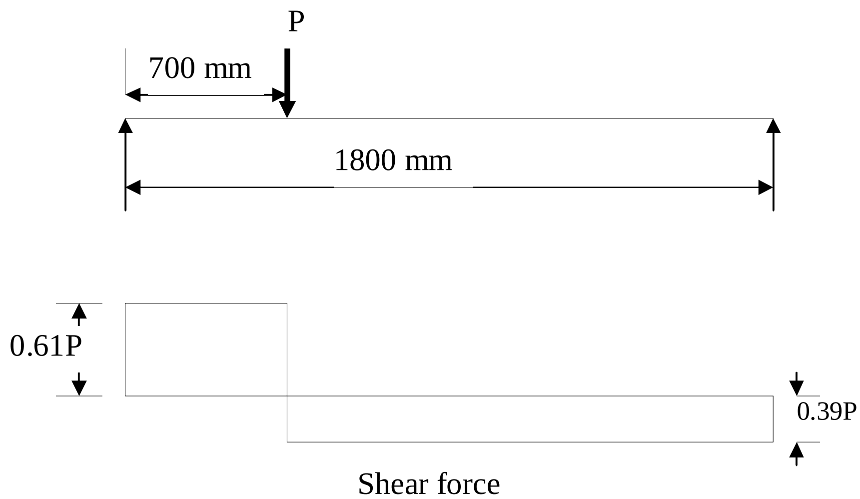


Fig 7.1 Test RWS loading arrangement

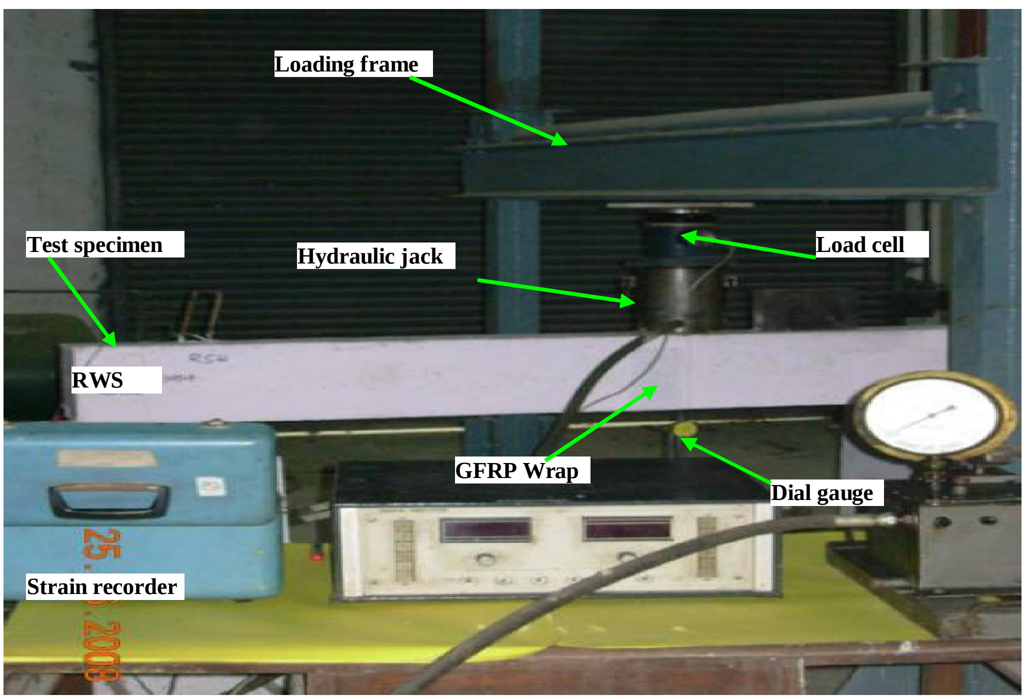


Fig 7.2 Test set up for beam RWS

7.1.1 Crack Propagation and Failure Mode

At an applied shear load V of 36.6 kN (P=60 kN), first hair line flexural crack at near GFRP Wrap occurred as seen from Fig. 7.3. On applying further load to V of 39.65 kN (P=65 kN), the existing crack was enlarged and new crack also appeared nearly 4cm away from the load line; this phenomenon was observed up to a shear load *V* of 42.7 kN (*P*=70 kN) as in Fig 7.4. At further increase of load first web shear crack appeared. As the applied shear load *V* was further increased to 45.75 kN (*P*=75 kN), a critical diagonal shear crack suddenly developed and the load was reduced to (P=72.8 kN); the root of this crack was at a distance of 300 mm from the support and height of 50mm from bottom face and it extended towards the outer edge of the load plate (Fig. 7.5). On further increase in load the existing shear crack widened and propagated towards the load point and support. The inclination of crack was also tending to become horizontal as to move along the longitudinal reinforcement. Also new shear crack were developed which originated from the initial shear crack as its branches (Fig. 7.6). The beam was stable upto the shear load V of 51.85 kN (P=85 kN). When the applied load *V* rose to 54.9 kN (P=90 kN), the beam the crack touches the load and support and crack widened and the beam was not able to sustain it for more than 2 minutes and failed in shear as shown in Fig. 7.7 and 7.8 with a breaking sound and the shear load *V* dropped abruptly to 39.65 kN (*P*= 65 kN). The test was terminated at this stage of loading.

At bottom face the cracks transverse to the length appeared were seen as shown in Fig 7.10. First crack was seen at load 65 kN directly under the load also another crack was seen at a different location. At higher loads new cracks appeared were near the support.

Observation for GFRP Wrap:- Initially at 60 kN – 70 kN crack was seen near GFRP and extended vertically to a height of 5 cm but latter there was no other crack observed along the GFRP and no debonding or cracking of GFRP was observed.

Observation for NSM bar and Epoxy: - Till failure load there was no effect or crack was observed either in epoxy mortar–concrete interface or with in epoxy mortar or epoxy mortar–steel interface. But after the failure some crack was observed near the support in epoxy mortar–concrete interface and was parallel to NMS rod (Fig 7.9.).

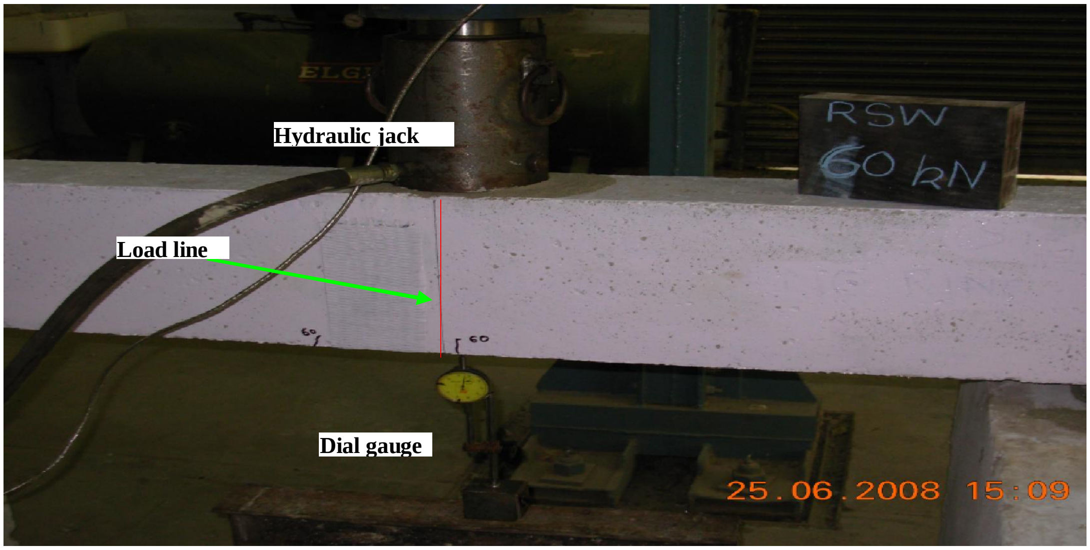


Fig 7.3 Test RWS crack pattern at 60 kN

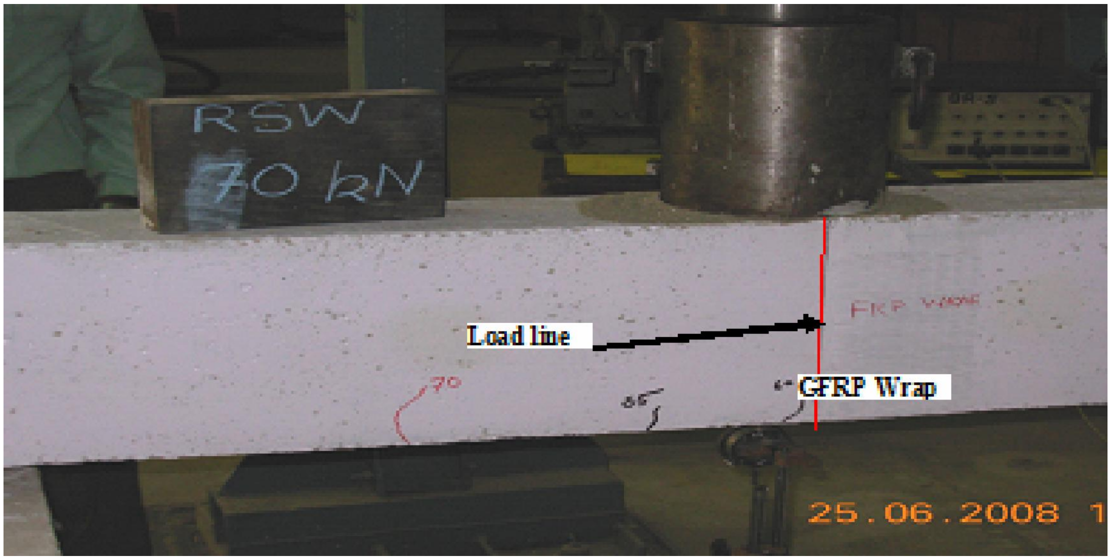


Fig 7.4 Test RWS crack pattern at 70 kN

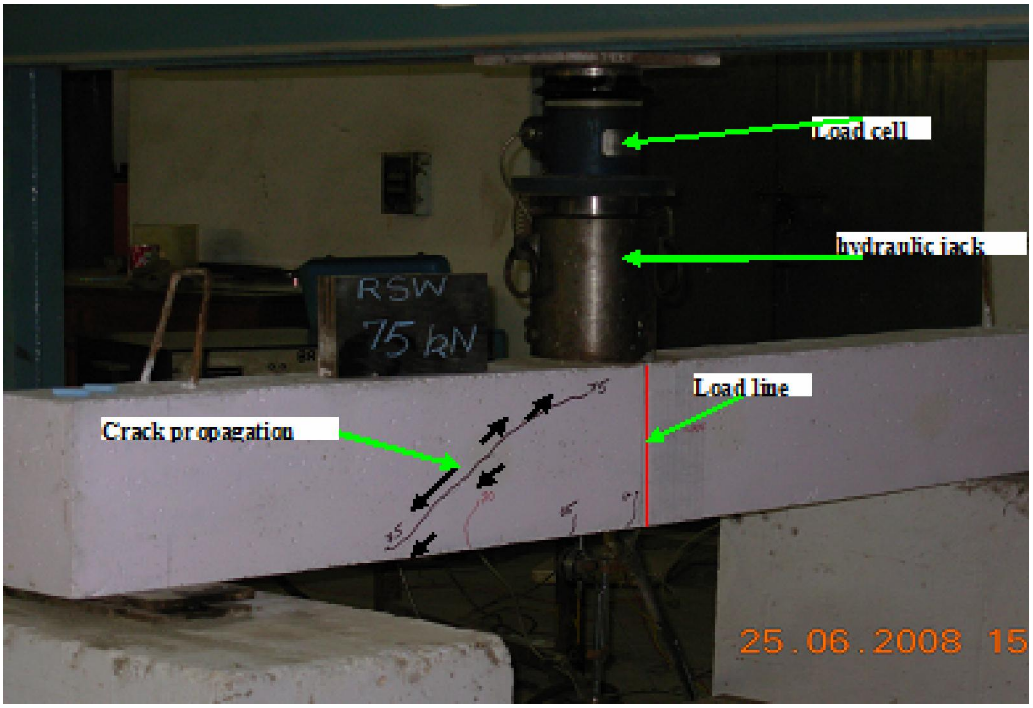


Fig 7.5 Test RWS crack pattern at 75 kN

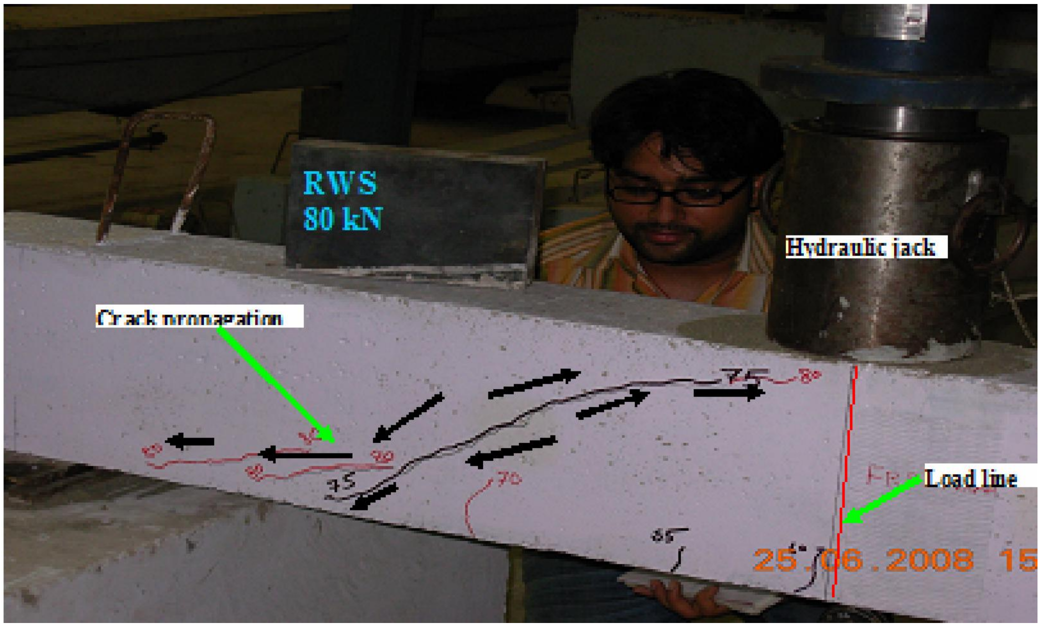


Fig 7.6 Test RWS crack pattern at 80 kN

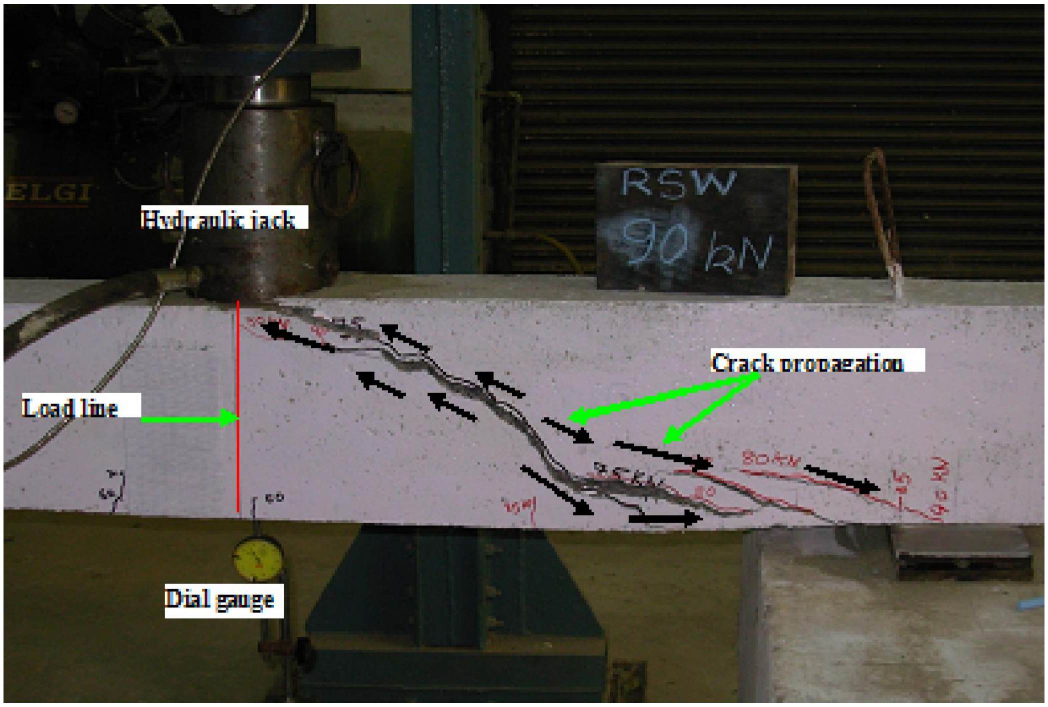


Fig 7.7 Test RWS final crack pattern at 90 kN failure load

(Face A)

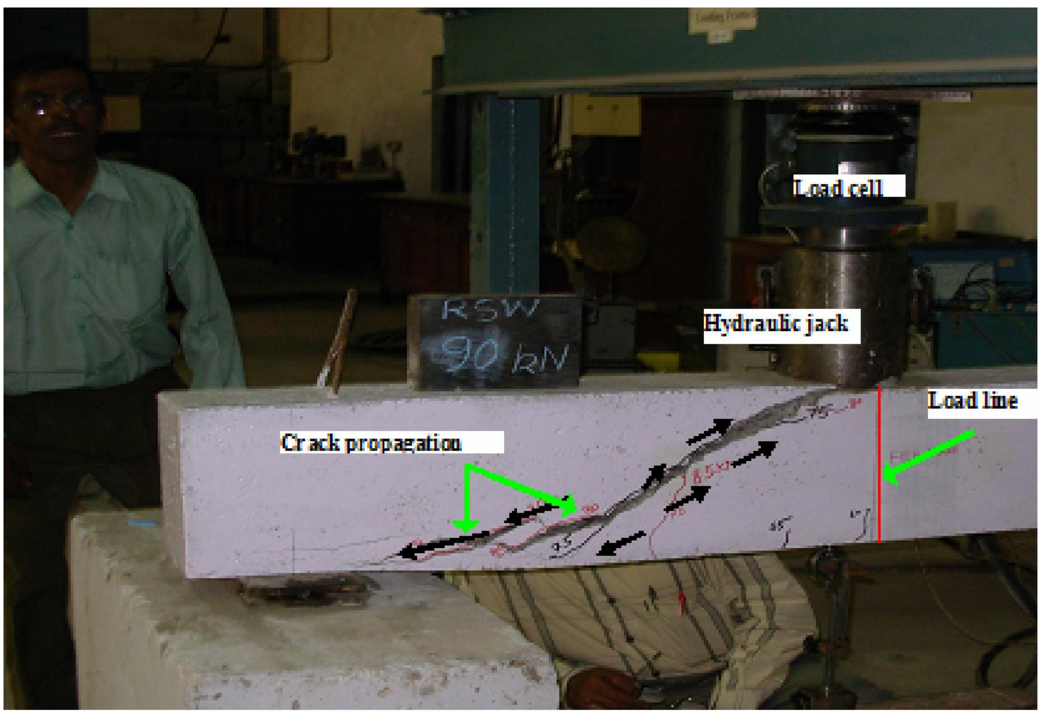


Fig 7.8 Test RWS final crack pattern at 90 kN failure load

(Face B)

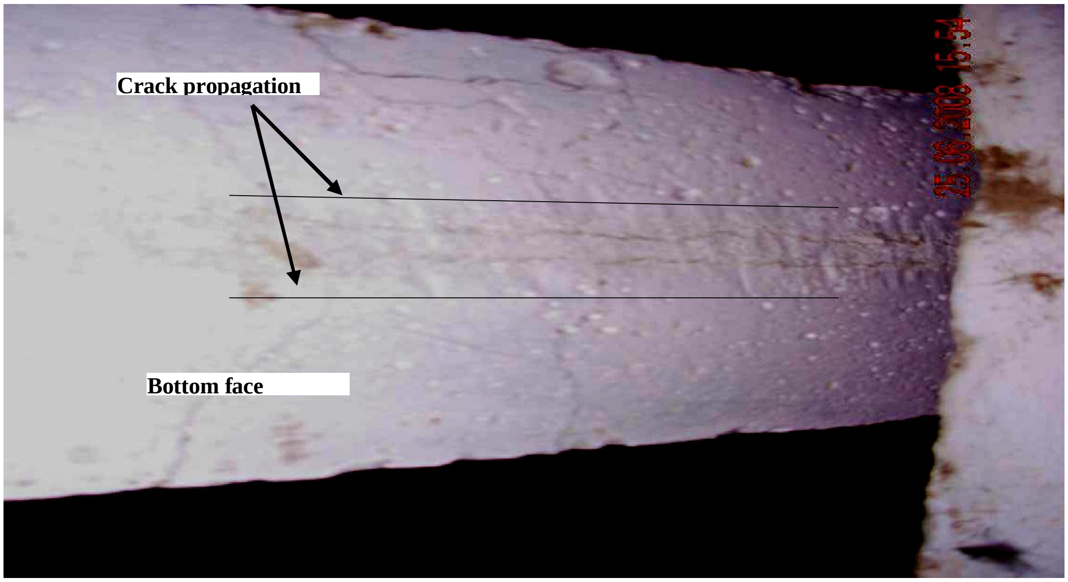


Fig 7.9 Test RWS crack at interface of epoxy mortar and concrete at bottom face

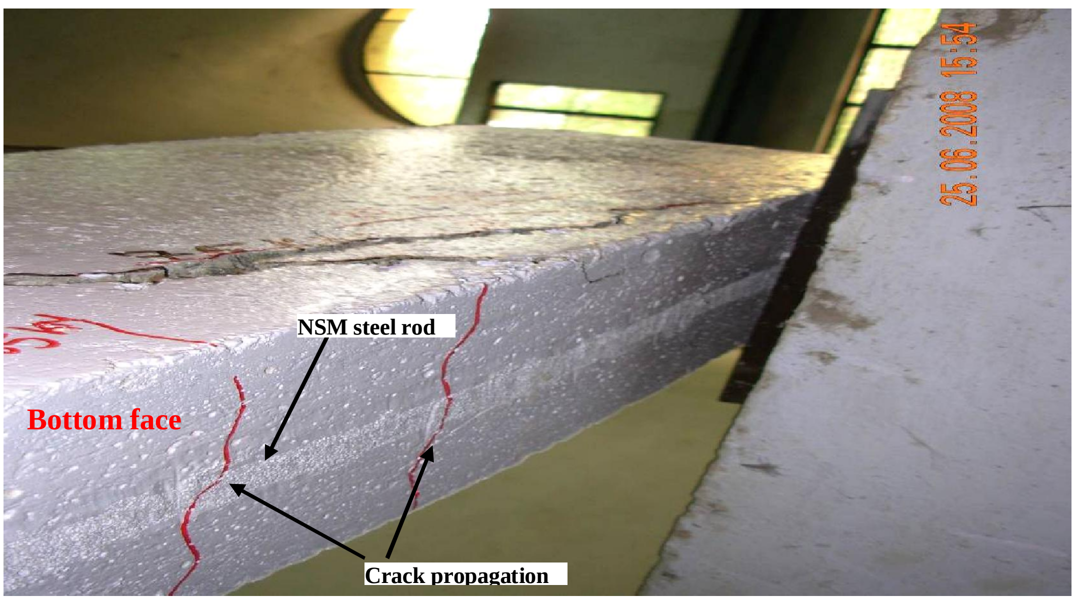


Fig 7.10 Test RWS transverse cracks at bottom face

7.1.2 Strain distribution in the longitudinal reinforcement

Figure 7.11 shows the variation of the strain magnitudes recorded at the gauge location as shown in (Fig. 4.2) with the maximum applied load. It can be seen that the strain magnitude increased linearly with the increased applied shear load until a maximum applied shear load *V* was 54.9 kN (*P*=90 kN).

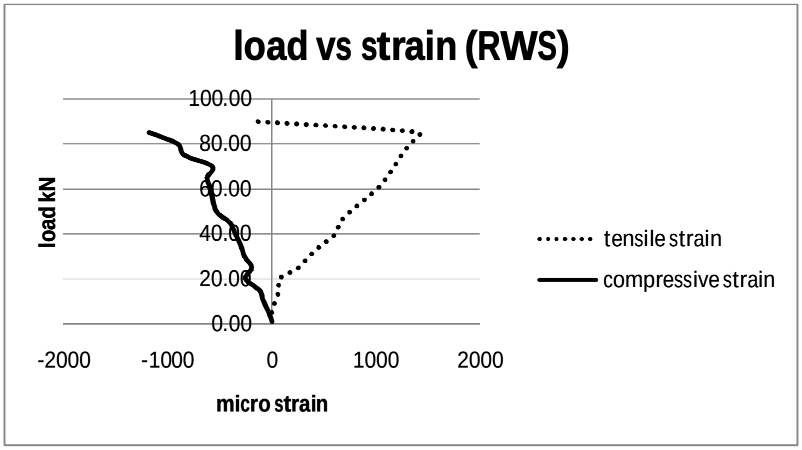


Fig 7.11 Strain variation in longitudinal reinforcement (RWS)

The magnitude of the maximum recorded strain in the reinforcement was 1408 micro strain on the occurrence of the diagonal shear crack that corresponds to an applied load P of 85 kN. This can also be evidenced from the failure of the beam that occurred at a load P=90 kN. The recorded maximum strain magnitude was less than the yield strain of the steel (2075 microstrain).

7.1.3 Deflection

The variation of the deflection under the load point with the maximum applied shear load for the beam RWS is shown in Fig. 7.12. The beam deflection was 5.3 mm at the applied load (*P*) of 75 kN, that is, when the diagonal shear crack occurred. The maximum deflection of the beam was 7.56 mm which was recorded at the instance of the termination of the test and the load of 90 kN as shown in Fig. 7.12.

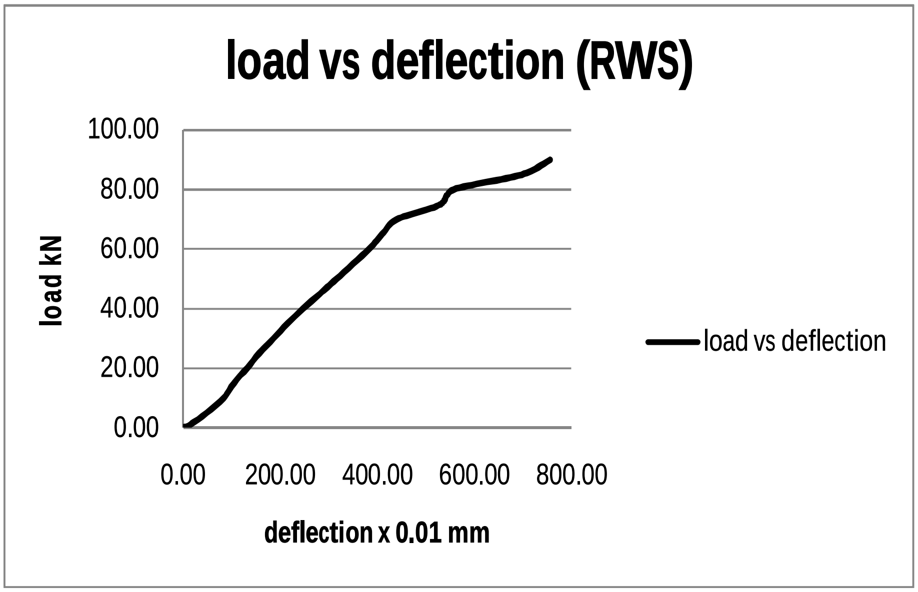


Fig 7.12 Test RWS Deflection of the beam

7.2 Test B1–SS (with NSM shear steel strips)

The objective of the test was to determine the effect on shear strength of the beam after strengthening it in shear with NSM steel strips. The internal reinforcement details, details of installed NSM steel strips, loading arrangement and strain gauge locations are as shown in Fig. 2.13, 3.1, 3.6, 4.3, and 4.4 respectively. For this purpose the beam was casted without shear reinforcement on half side and latter was strengthened with inclined NSM shear steel strips. NSM steel strips were installed by cutting grooves into side faces of beam and placing the strip and filling the groove with epoxy–sand mortar. The strips were 18mm wide and 0.8mm thick. Thus a total of 11 NSM strips were installed on both side with a spacing of 85mm centre–centre. Strain gauges were bonded to the tension and compression reinforcement internally and to the inclined NSM shear steel strips 7 strain gauges on NSM strips of one face and 4 strain gauges on the NSM strips of other face of the beam. The test was conducted by the application of single concentrated load in the shear span at a distance of 700 mm from the support so that the shear force in the tested shear span was 61.1 % of the applied load (*P*). The adjacent shear span was provided with internal shear strips. The test set up is as shown in Fig. 7.13

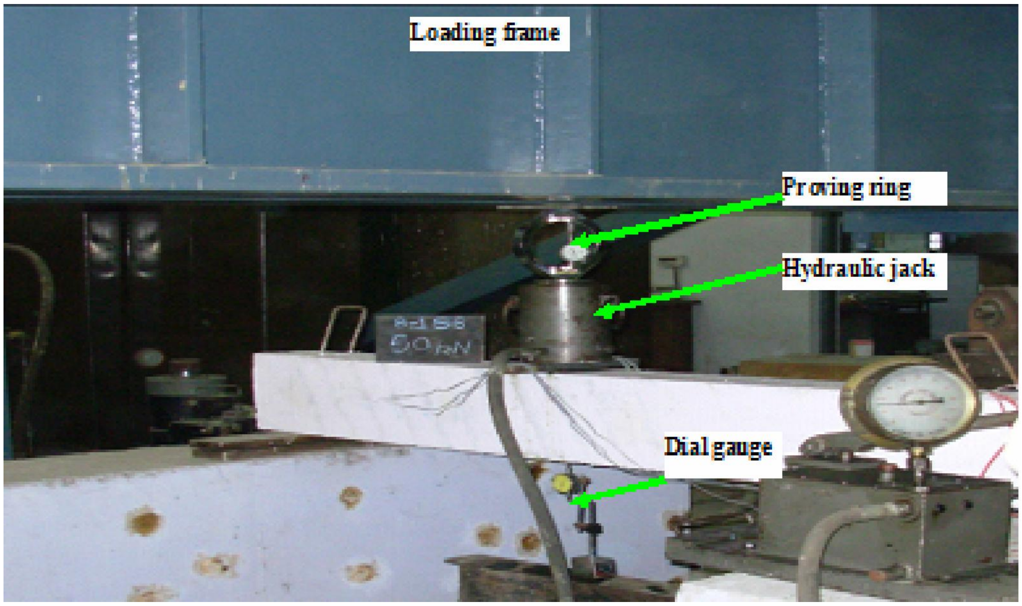


Fig 7.13 Test B1–SS setup

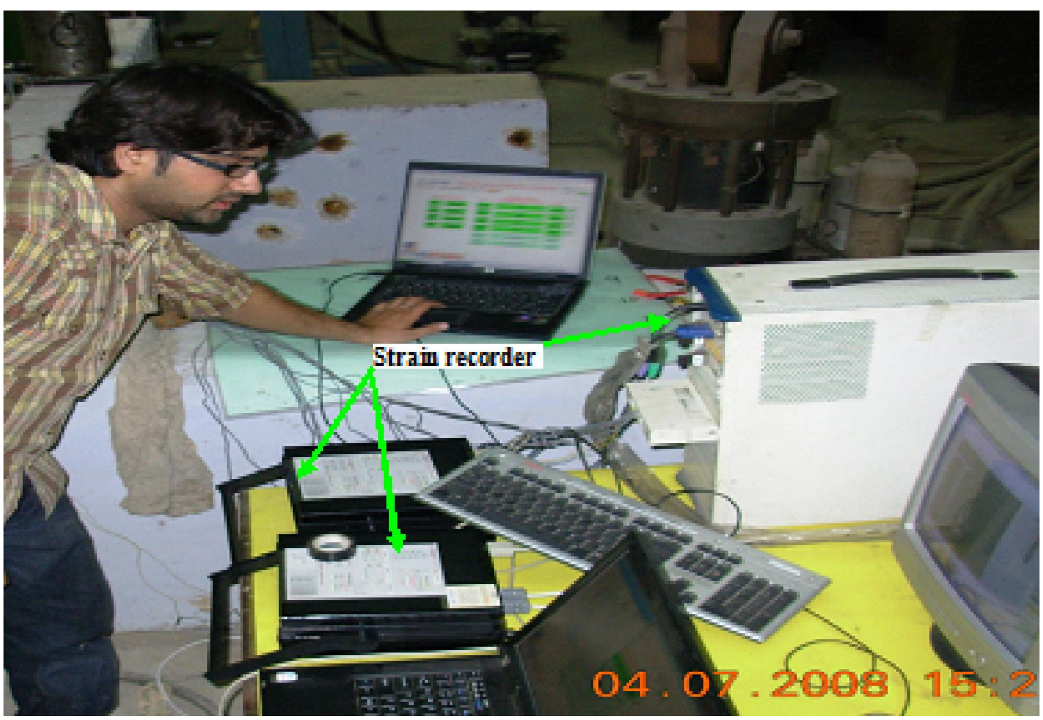


Fig 7.14 Arrangement for strain recoding

In this test load was measured and applied through 200 kN capacity proving ring. And the strain was recorded with the help of SCAD 500 strain measuring system a Pyro – Dynamic made (Fig. 7.14). Strain measurements were recorded per second by the system. And the measurements were also recoded at every load interval to keep a check.

7.2.1 Crack propagation and failure mode

Along face S–E: - The crack pattern at different stages of applied load at face S-E can be seen from Fig.7.15 to 7.19. It can be seen from figures that at an applied shear load (*V)* of 36.66 kN (***P***=60 kN), hairline cracks occurred along the 8th strip from the support extending upto the length of 5cm and the same crack extended and widened along the strip to a length of 13.5 cm length at shear load of 94.6 kN (P=155kN). On the same strip a crack in epoxy mortar–concrete interface was observed at shear load of 113 kN (P = 185kN) and this crack then extended into the concrete and become vertical at shear load 119 kN (P = 195 kN). The crack along the strip widened clearly showing that debonding at the steel – epoxy mortar interface had occurred.

On the 9th strip from the support similar crack developed at shear load 73.2 kN (P = 120 kN) and it kept on extending and widened on further increase of load till it reaches the level of top reinforcement at shear load 119 kN (P = 195 kN). A flexural crack was also seen in concrete between the 9th and 10th strip at the load (P = 195 kN) to a depth of 16 cm and was parallel to the strips. This clearly shoes the confining of concrete between the strips. Cracks along the 11th strip and concrete which are in shear stirrups region were also seen clearly showing the failure of beam in flexure. Cracks along the strip occurred along the strips nearer to the support as the load was increased. At loads nearer to the failure cracks in epoxy was seen which extended into the concrete.

First web shear crack occurred at shear load 94.7 kN (P = 155kN) in concrete between 4th and 5th strip and extended on both side at higher loads. On further increase of load more shear cracks occurred in the concrete between 3rd and 4th strip and 5th and 6th strip which are on both side of the first shear crack at shear load 100.8 kN (P = 165 kN). The shear crack between the 3rd and 4th strip which are nearer to the support extended the most and extended to 2^{nd} strip at shear load 110 kN (P = 180 kN) and to 7^{th} strip at shear load 119 kN (P = 195 kN).

Along face S–W: - The crack pattern at different stages of applied load at face S-W can be seen from Fig.7.20 to 7.23. As seen from figures the cracks along the 8^{th} and 9^{th} strips from the support which are directly under the load extending upto 3cm in 9^{th} strip and upto 9cm in 8^{th} strip at shear load of 55 kN (P = 90kN). This was the initiation of the interfacial crack between epoxy mortar–steel which is also known as debonding crack. These crack latter widened and extended towards the top steel at shear load of 104 kN (P = 170kN) and beam was failed due to debonding of the NSM strips at steel – epoxy mortar interface. The crack at the 8^{th} strip also enters into epoxy at height of 12 cm which further enters into concrete and become vertical and extended upto the level of compression steel at shear load of 119 kN (P = 195 kN). As was observed on S-E face similarly cracks along strip occurred along the strips nearer to the support as the load was increased.

First shear crack occurred in concrete between 4^{th} and 5^{th} strip at shear load 94.7 kN (P = 155kN) this crack did not extended to 6^{th} toward the load strip at shear load of 119 kN (P = 195kN). More shear cracks occurred in concrete between 3^{rd} and 4^{th} strip at shear load of 103.85 kN (P = 170kN) and in concrete between 6^{th} and 7^{th} strip at shear load of 107 kN (P = 175 kN).

At bottom face: - the crack pattern at the bottom face can be seen from figure 7.24. Three cracks at the tension face under the load occurred at an applied load 115 kN and further new transverse crack occurred on the tension side at applied load of 175 kN, 180 kN 190 kN. It was observed development of cracks was moving nearer to support as the load was increasing.

From the above crack profile study it was seen that with the strengthening of the beam with NSM strips the concrete between the strips got confined and beam was strengthened in shear and a brittle and sudden shear failure of beam without shear stirrups was converted into a ductile flexural failure which occurred due to debonding of the strips at the epoxy mortar and steel interface. Single diagonal shear crack which was as seen in beam without shear stirrups was converted into several shear cracks which initiates from the concrete confined between the shear strips and were provided with resistance to propagate by NSM strips.

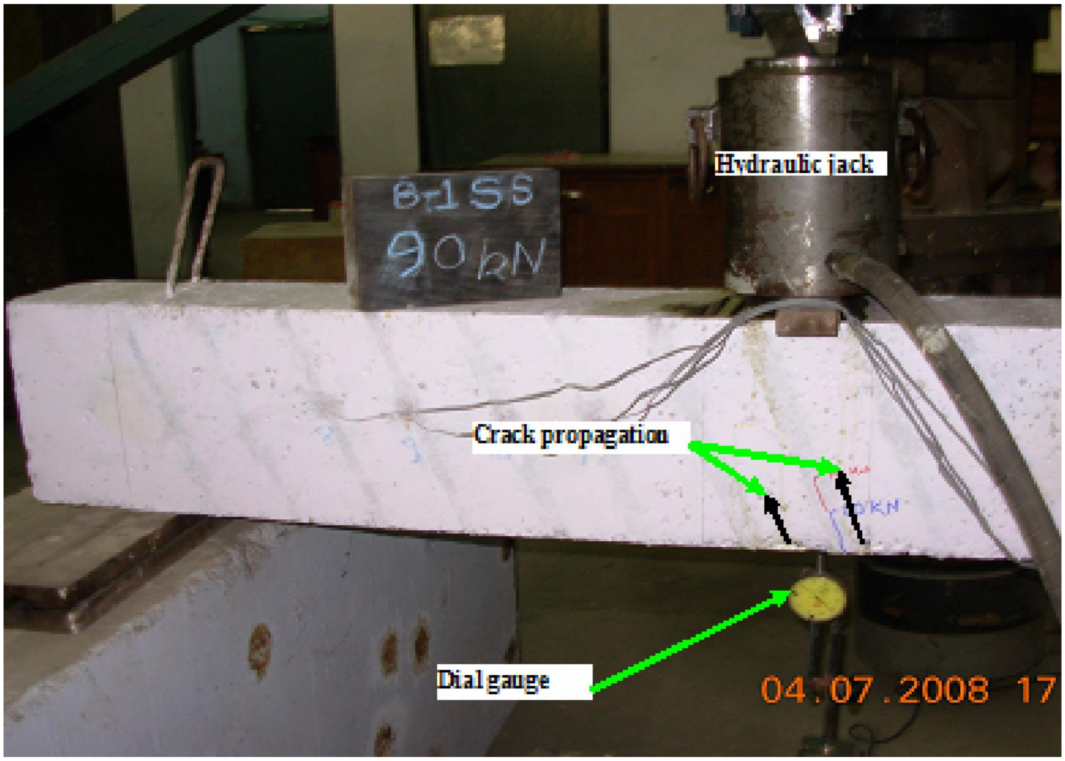


Fig 7.15 Test B1-SS crack pattern at 90 kN (Face SE)

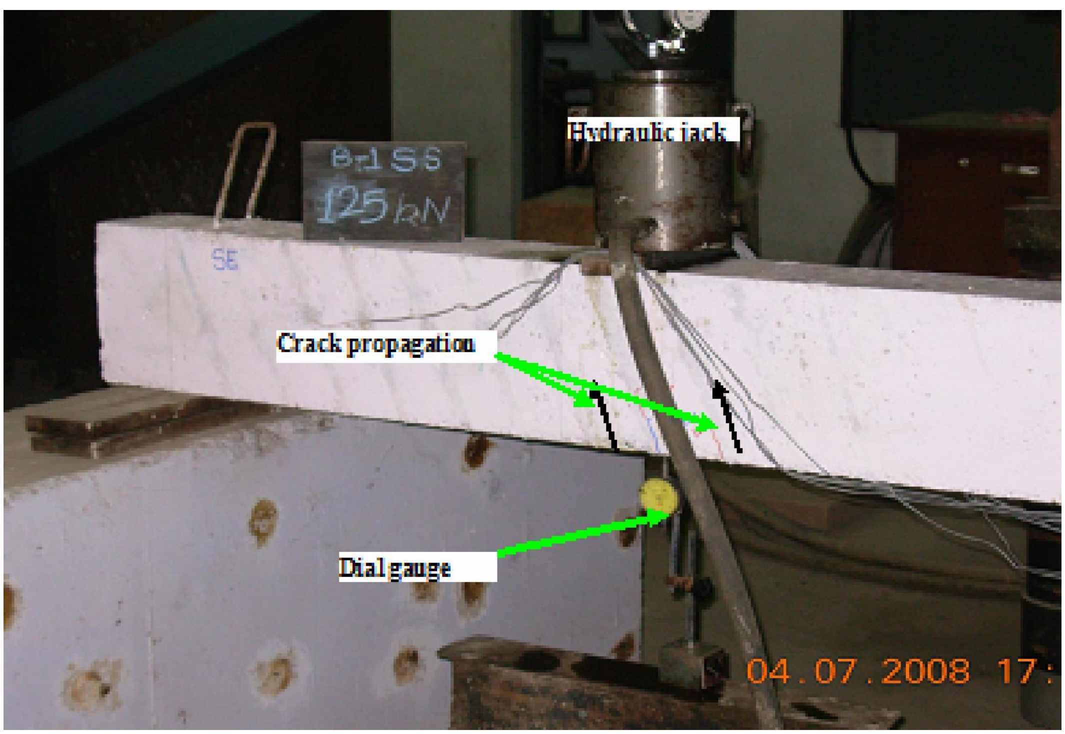


Fig 7.16 Test B1-SS crack pattern at 125 kN (Face SE)

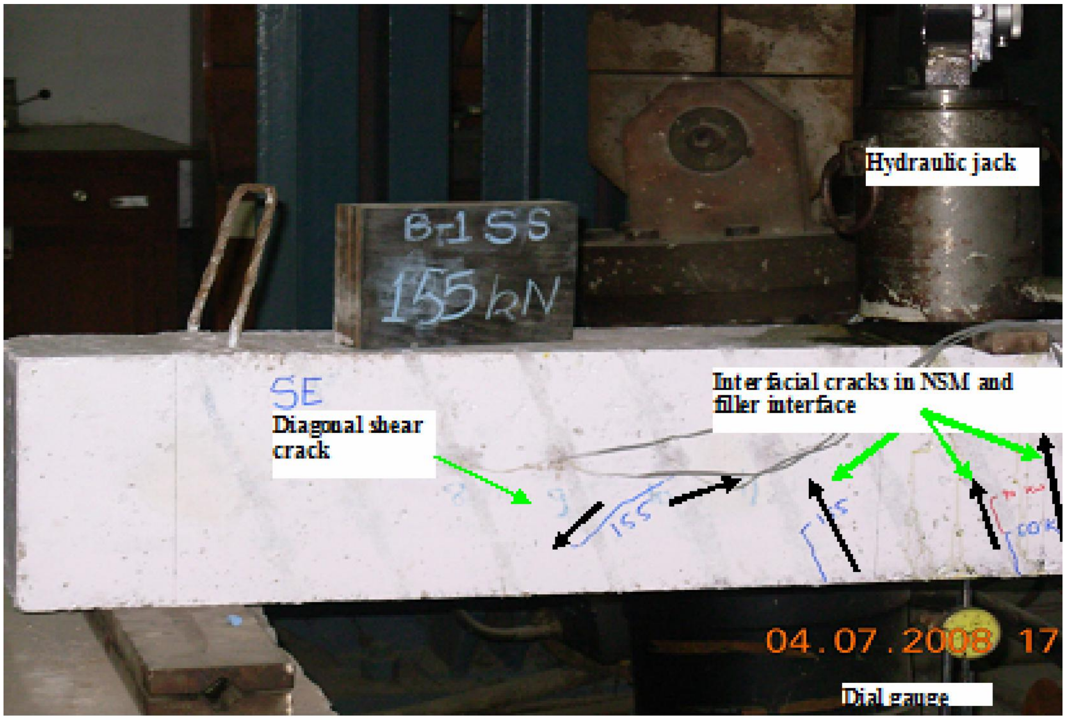


Fig 7.17 Test B1-SS crack pattern at 155 kN (Face SE)

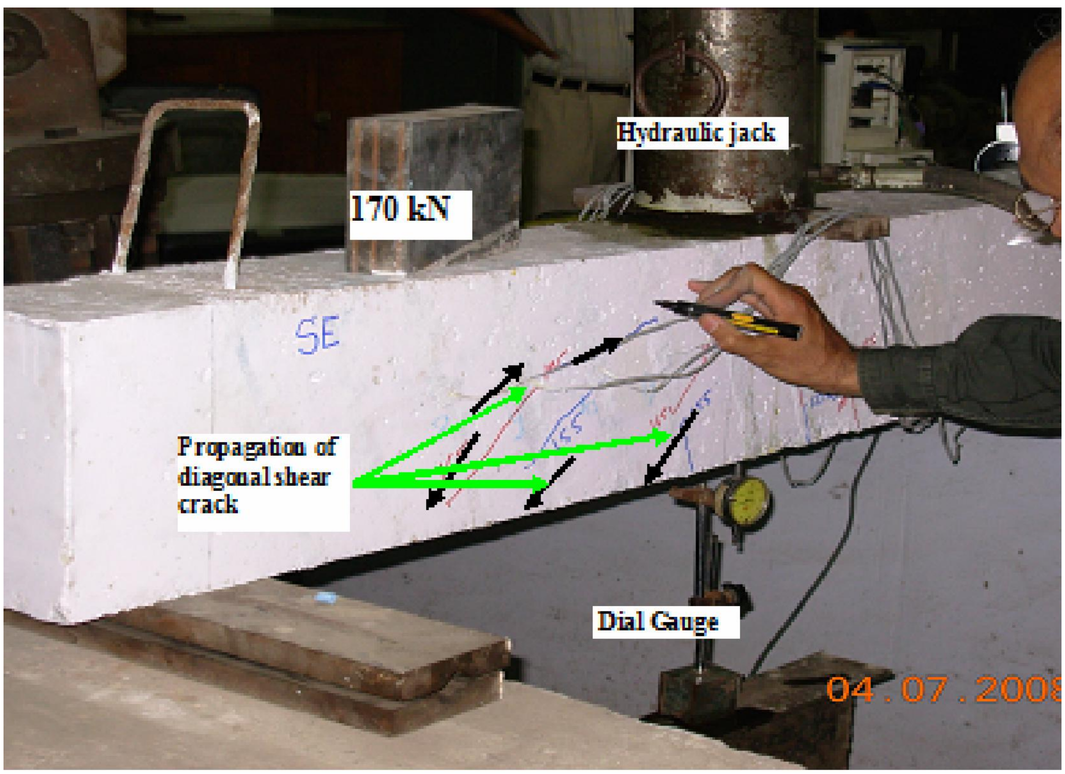


Fig 7.18 Test B1-SS crack pattern at 170 kN (Face SE)

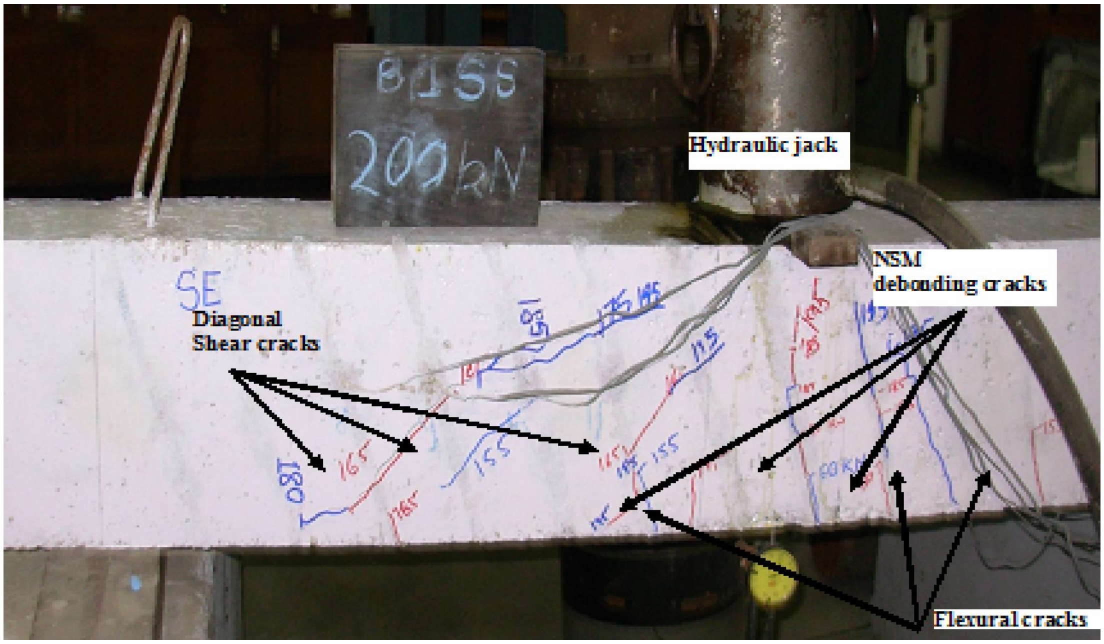


Fig 7.19 Test B1-SS final crack pattern at 200 kN (Face SE)

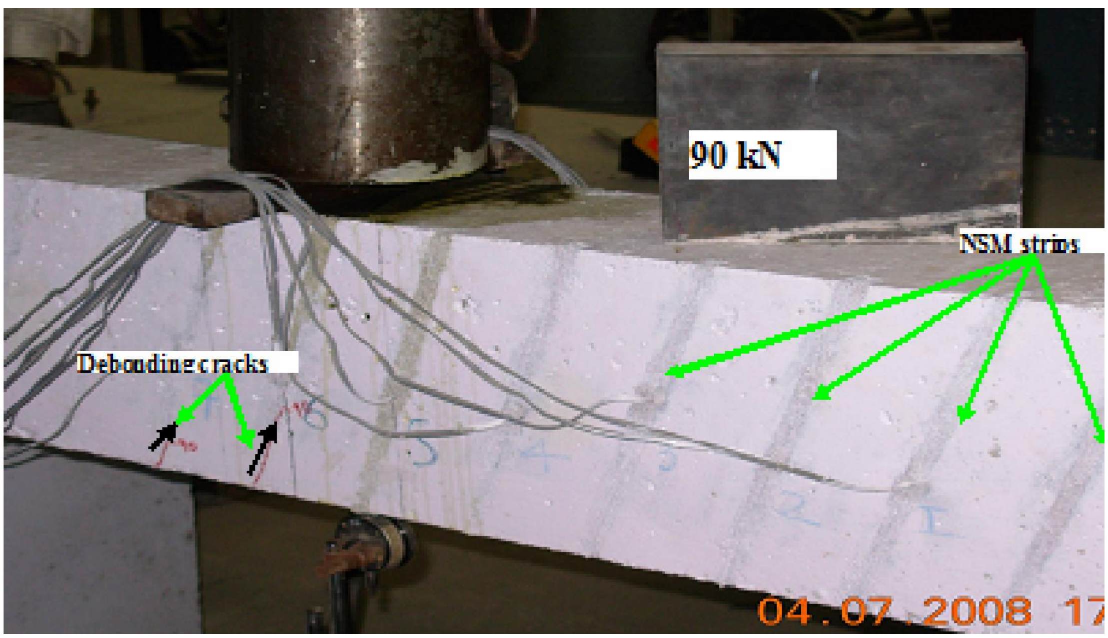


Fig 7.20 Test B1-SS crack pattern at 90 kN (Face SW)

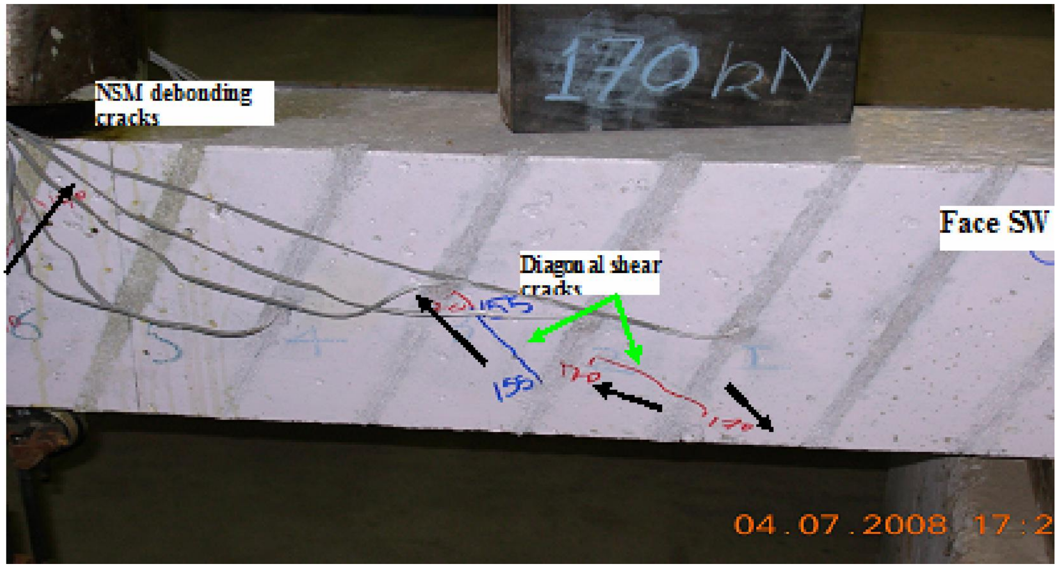


Fig 7.21 Test B1-SS crack pattern at 170 kN (Face SW)

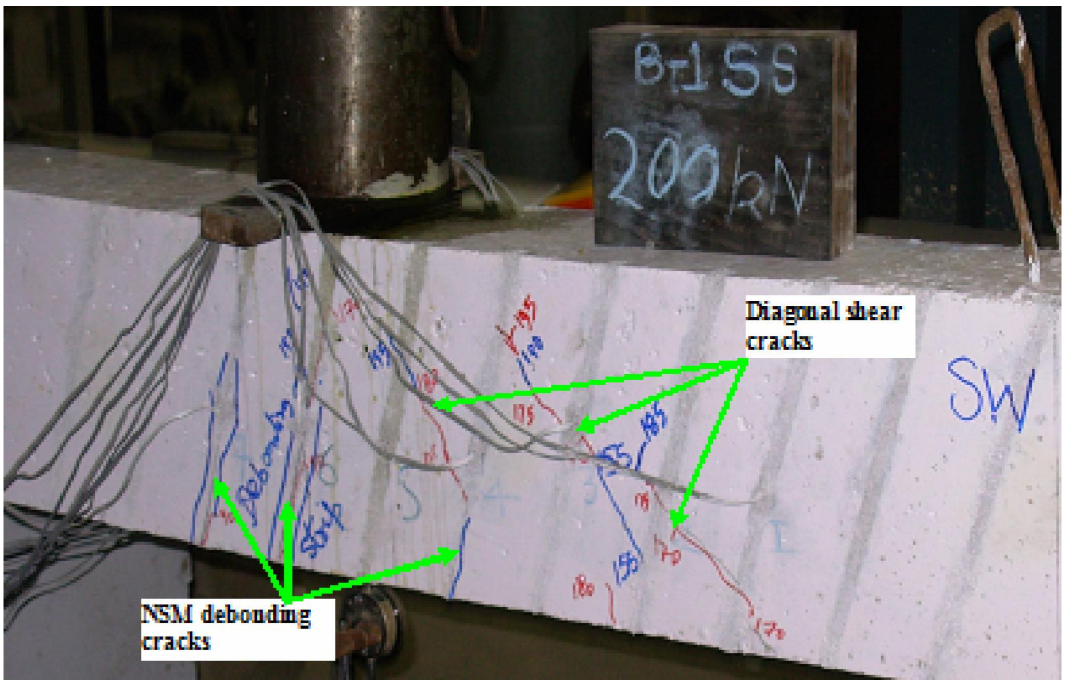


Fig 7.22 Test B1-SS final failure crack pattern at 200 kN (Face SW)

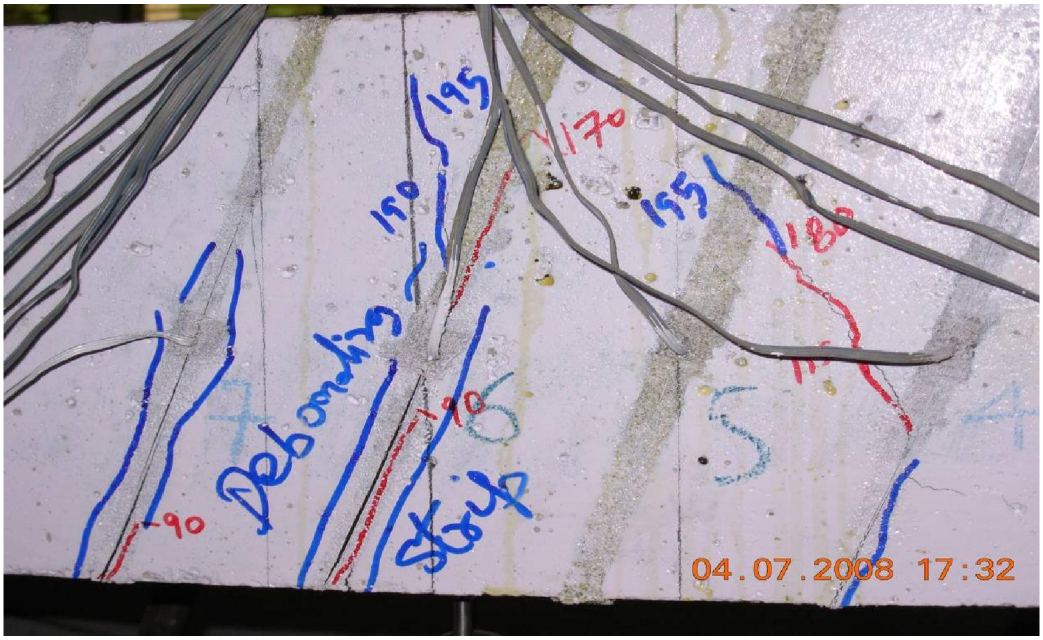


Fig 7.23 Close view of NSM strip debonding failure

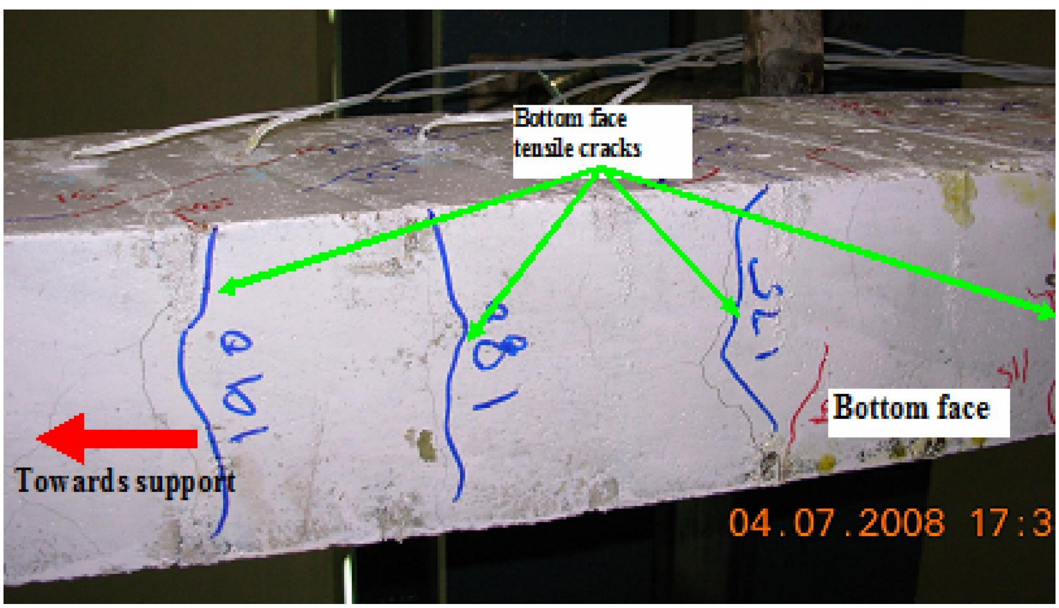


Fig 7.24 Test B1-SS final crack pattern at 200 kN (Bottom Face)

7.2.2 Strain distribution in longitudinal reinforcement

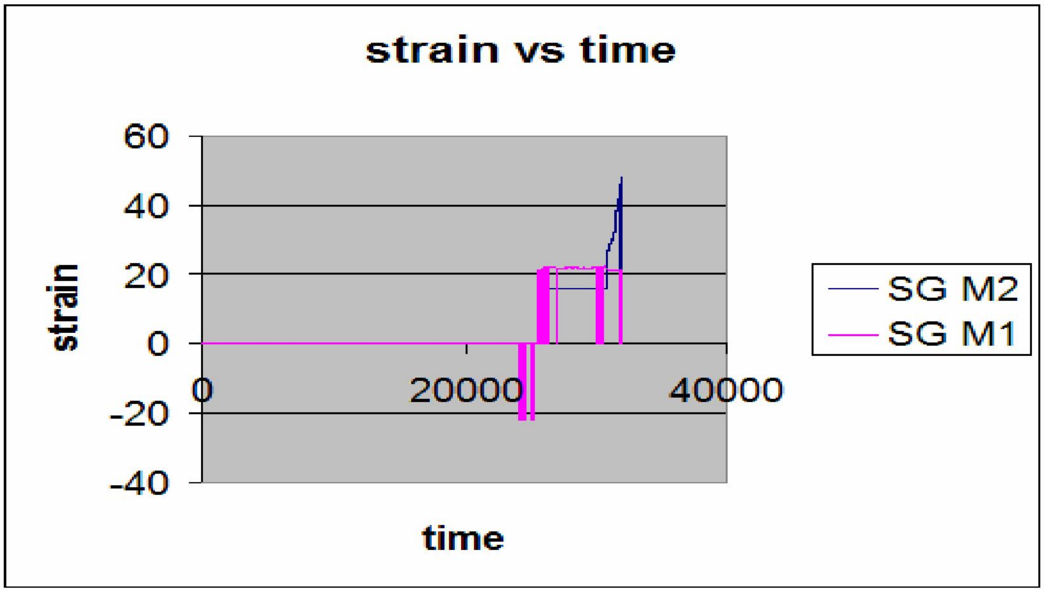


Fig 7.25 The mid span Strain variations (microstrain x 10^{-2}) in longitudinal bars wrt. Time

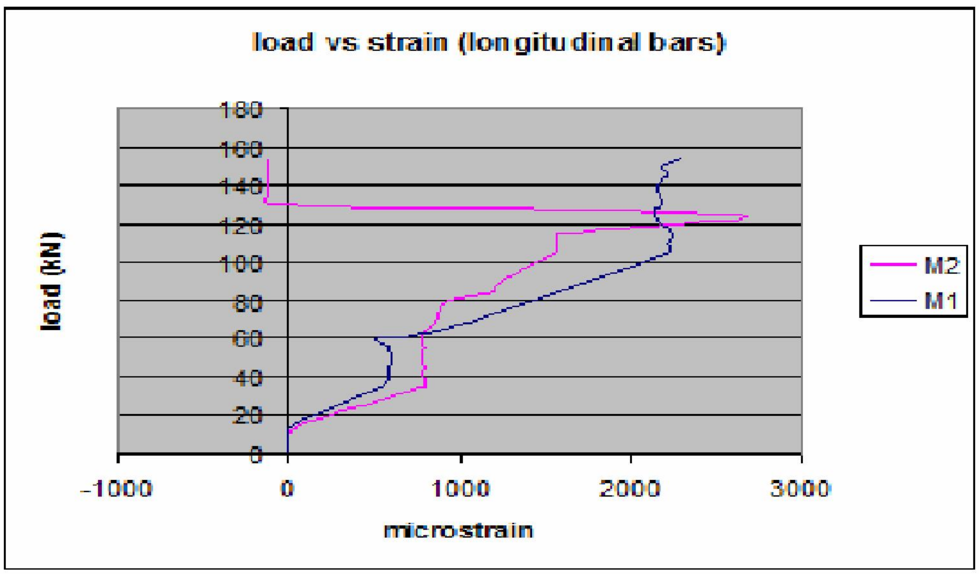


Fig 7.26 The mid span strain variations in longitudinal bars at mid span wrt. load

As can be seen from the figure(4.3and 4.4) strain gauges were fixed at mid span out of four T1, T2, M1, M2 only last two were in working condition. It can be seen from the Fig. 7.25 and 7.26 that in both M1 and M2 strain have increased simultaneously with every increment in load. Initially strain in M1 was greater than M2 but latter it was reversed for most of the time the strain in M1 was near 2000 micro-strain. But at final applied load, strain in M2 increased and was out of yield limit of strain in steel.

It can also be seen from the fig 7.26 as strain M2 after reaching maximum strain of about 2600micro-strain at load kN 125, reduced suddenly on increase of further load and was further constant at about -150micro-strain his shows that either debonding of main steel had occurred or the strain gauge had been damaged. Similarly in M1 strain after reaching about 2200microstrain at load about 105 kN it was constant on increase of further load.

7.2.3 Strain distribution in NSM steel strips

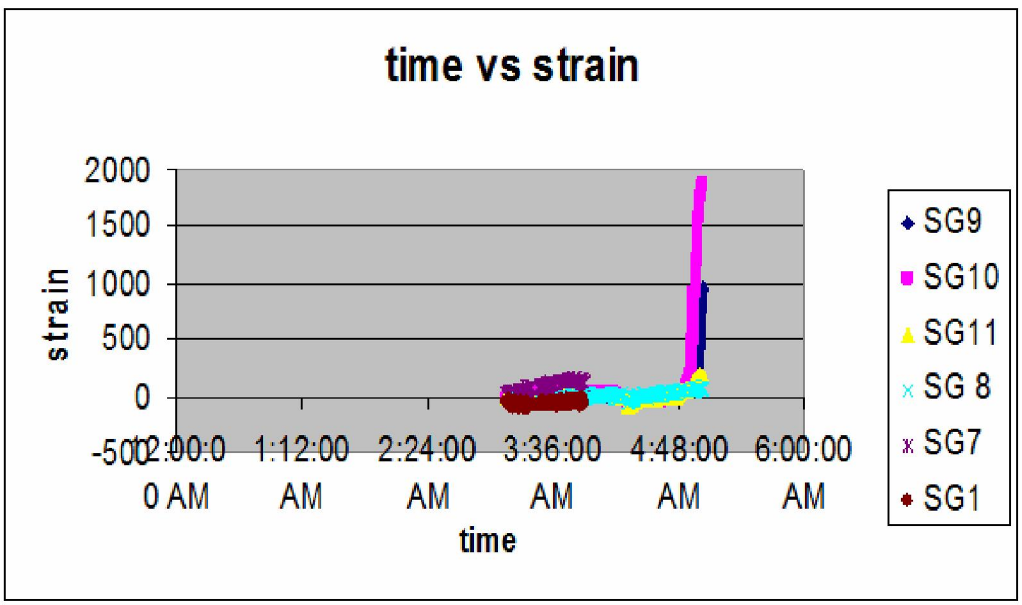


Fig 7.27 Graph showing strain (micro strain) in NSM steel strips vs Time

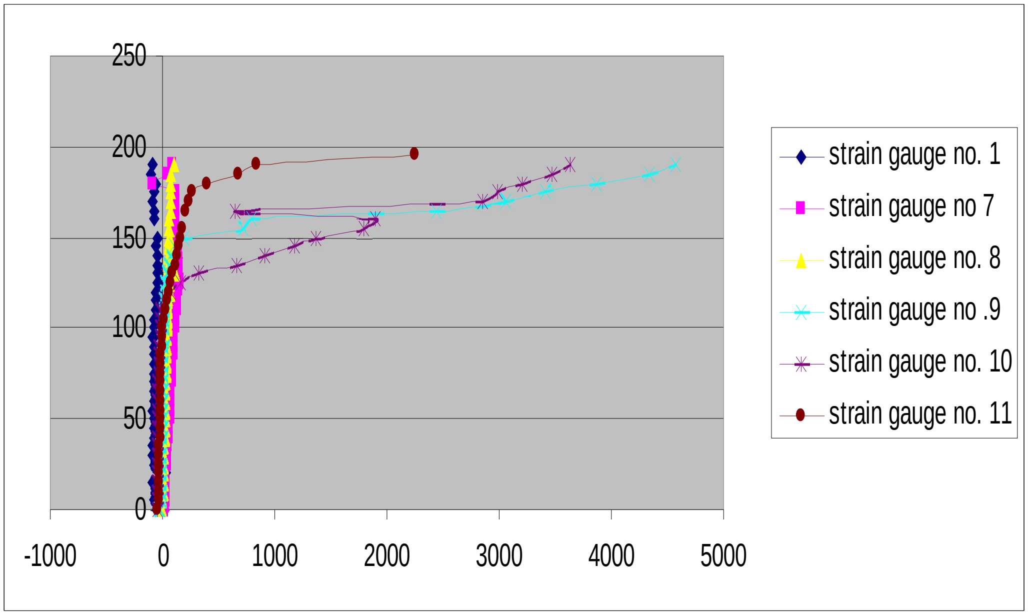


Fig 7.28 Graph showing strain (micro strain) in NSM steel strips vs load

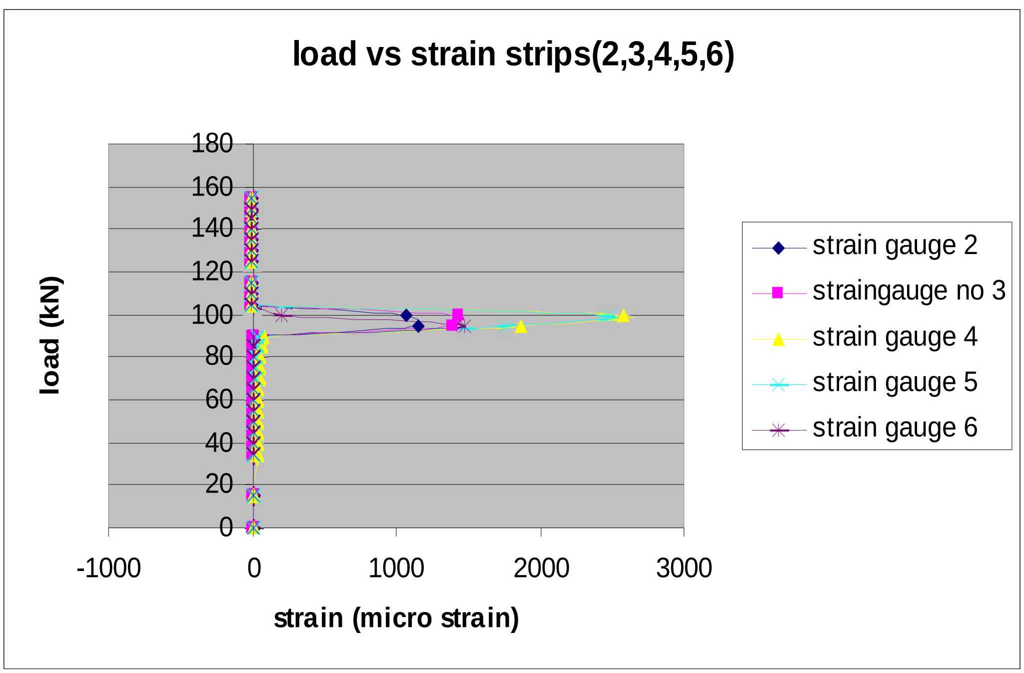


Fig 7.29 Graph showing strain (micro strain) in NSM steel strips vs load

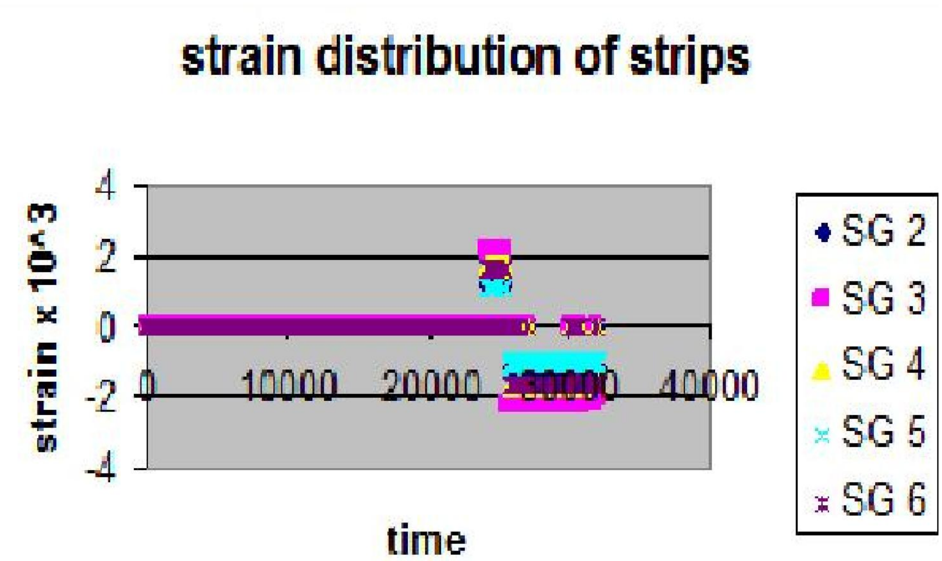


Fig 7.30 Graph showing strain (micro strain) in NSM shear strips vs time

The variation of strains in the NSM shear steel strips were recorded from strain gauges installed along with NSM strips is as shown in Fig. 4.3, 4.4. The strain variations at different stages of load application with the time are as shown in figure 7.27 and 7.30. It can be seen from the Fig. 7.27 and 7.30 that during the initial period the load was not taken by the NSM strips hence there was negligible strain. In strips with strain gauge no.9, 10, 11 strain increases abruptly to about 1000, 1900and 300 micro-strains respectively in the final stages of load application during the tests showing that these strips were taking part in shear strengthening and it can be seen that first shear crack also occurred around these NSM shear strips in Fig. 7.19. In NSM strips with strain gauge no. 7, 8, 11 debonding at the interface of NSM and epoxy mortar had occurred and hence these strips did not show any significant strain. It is also observed that the NSM strip with strain gauge no. 1 did not take any strain. This is also evident from Fig. 7.19. In Fig. 7.28, the above mentioned observations can be validated as it can be seen that the NSM strips with strain gauges no. 7, 8 take very less strain as were debonded earlier at the interface of NSM and epoxy-sand mortar and the strips with strain gauge no.9, 10, 11 shared the shear load which was transferred to them as they show a sudden increase in strain with increase in load after load (P = 125kN*). Strain in the NSM strip 4th and 5th from the support has crossed its yield limit at a load of (P=170kN*). These strip had crossed there ultimate limit of

(2600 micro-strain) to about 4500 and 3800 micro-strain respectively. The 6th strip from the support behaved in a proper manner and strain in it was with in the ultimate limit.

Similar observations can be made from the Fig. 7.29, in which it is evident that the strips with strain gauge no. 5, 6 i.e. 7th and 8th strips from the support shows the maximum strain and were debonded at an load of about 100kN. This can also be seen in Fig 7.23, where the debonding crack is more critical.

7.2.4 Deflection

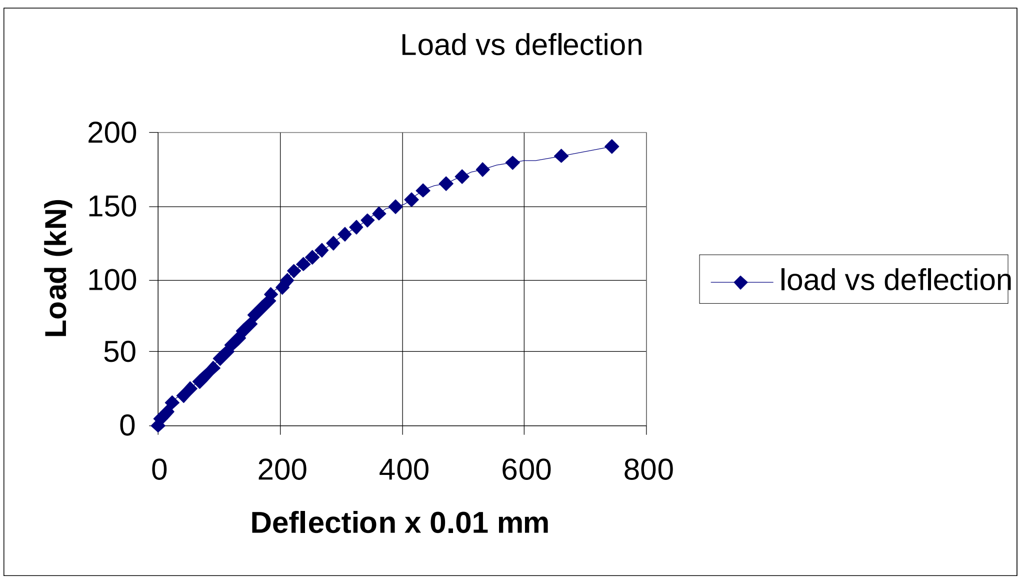
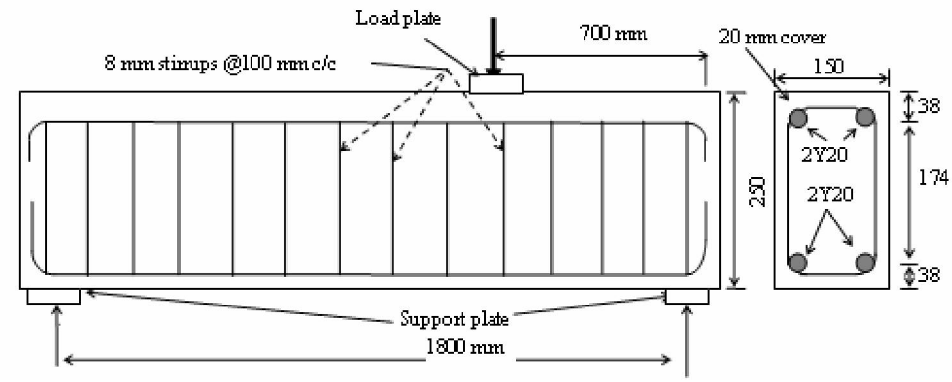


Fig.7.31 Test B1–SS: Deflection of the beam

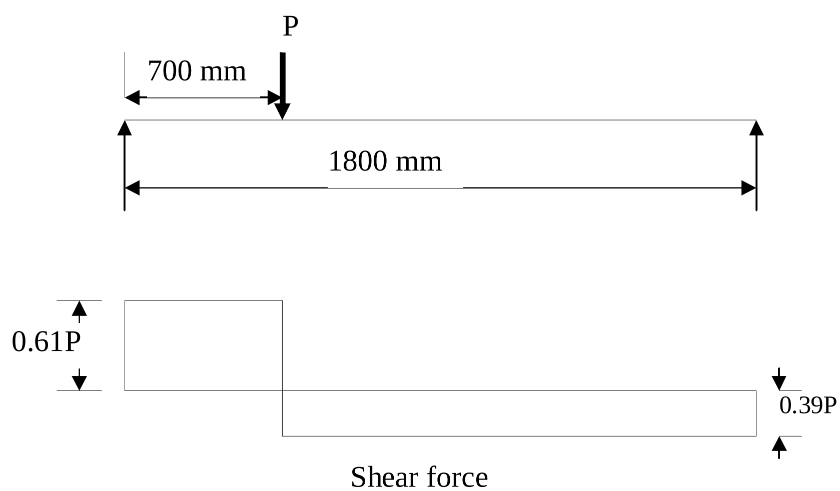
The variation of the deflection under the load point with the maximum applied shear load for the beam B1-SS is shown in Fig. 7.31. The maximum beam deflection was 742mm at the applied load (*P*) of 190 kN i.e., when the debonding of NSM strips had occurred. The deflection of the beam was almost varied linearly to about 250mm at load (P=125 kN*) which was recorded before the initiation of the shear crack (Fig. 7.16).

8. Analyses for Shear Strength

8.1 Beam BRS with Internal Shear Stirrups (R, BRS)



(a)



(b)

Fig 8.1 Loading and SFD diagram for beam (R, BRS)

As we know that

$$V_u = V_{uc} + V_{us}$$

Where "Vu" is the ultimate shear strength of the beam, "Vuc" is the ultimate shear strength contribution of the concrete member without shear reinforcement. And "Vus" is the contribution of shear strength by web steel.

Also "Vuc" is given by

$$V_{uc} = \tau_{uc} b\, d$$

Where, "τ_{uc}" is the ultimate shear strength of concrete which depends on percentage of steel in beam section and characteristic strength (fck) of the concrete, "b" is the width of the member, and d is the effective depth of the member.

For %Ast = 1.976% and *fck* = 40 MPa, τ_{uc} = 0.876MPa (IS: 456 2000) [3] and with b = 150mm and d = 212mm. We have Vuc= 27.86 kN.

This value represents the safe limit value. But for the ultimate shear strength there is a relation given in one paper taken from ACI 318 (13, 14) which is as follows

$$V_c = 3.5\sqrt{f_c} b_w d$$

Where" fc" is the compressive strength of the concrete in psi (1MPa ≈ 145psi), "b_w" and "d" are the web width and depth in inches(1inch = 25.4mm), thus "Vc" is in kips (1kips ≈ 4.5 kN)

Thus using above equation we have

Vc = 61.173 kN

And Vus as per IS: 456 2000 [3] is given as follows

$$V_{us} = \frac{0.87 f_y A_{sv} d}{S_v}$$

Where "f_y" is the yield strength of the steel in MPa, "Asv" is the cross-sectional area of shear stirrups in mm², "d" is the effective depth of the section in mm and "Sv" is the spacing of the shear stirrups in mm

Thus using the above formula for Vus we have

$$V_{us} = \frac{0.87 X 415(2 X \pi 4^2) X 212}{100} = 76.54 kN$$

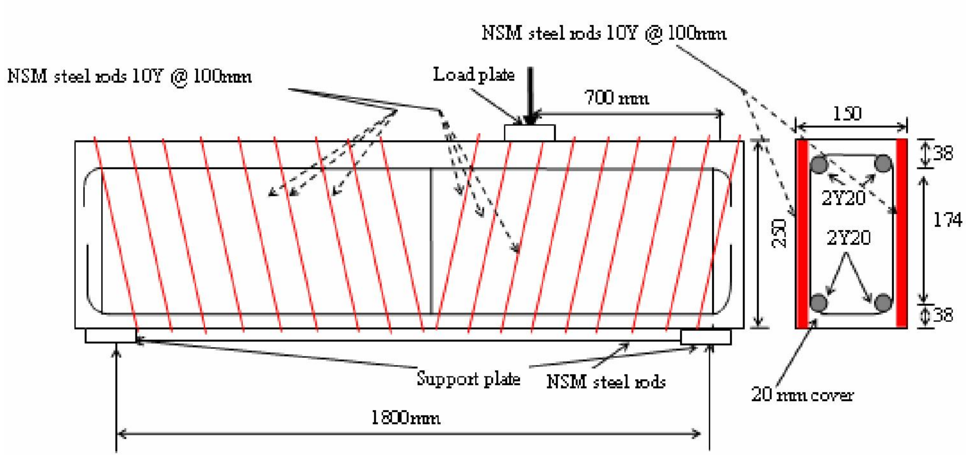
Thus we have Vu as

$$V_u = V_{uc} + V_{us} = 137.71\ kN$$

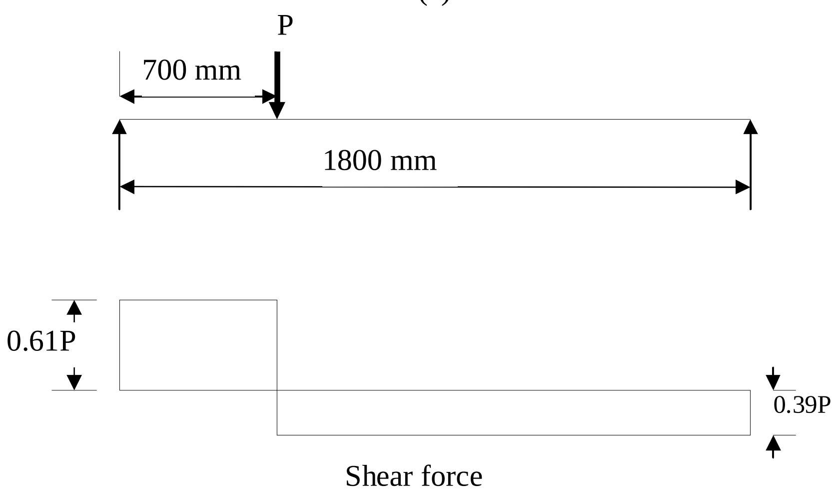
And from Fig. 8.1(b) showing SFD we have ultimate applied load Pu as

$$P_u = \frac{V_u}{0.61} = 225.75 kN$$

8.2 Beam B2-SR with NSM round bars (NS, B2-SR)



(a)



(b)

Fig 8.2 Loading and SFD diagram for beam (NS)

For inclined NSM shear reinforcement, assuming the crack to formed along the line joining the support and the load. The relation formulated by De Lorenzis the shear contribution of the NSM rod is given by

$$V_{1F} = 2\sum A_i f_i = 2\pi . d_b . \tau_b . L_{tot\,min}$$

Where "Ai" is the nominal cross-sectional area of the rods, and "fi" is the tensile stress in the rod at the crack location, and summation extended by all the rods *crossed by a 45 crack, "L_{tot}" is the sum of effective lengths of all the rods crossed by the cracks, "d_b" is the nominal diameter of the rod and "τ_b" bond strength.*

And the relations to find $L_{tot\ min.}$ is given as the following

$$L_{tot\ \min} = d_{net} - s \text{ , if } \frac{d_{net}}{3} < s < d_{net}$$

$$L_{tot\ \min} = 2d_{net} - 4.s, \text{ if } \frac{d_{net}}{4} < s < \frac{d_{net}}{3}$$

Where "s" is the spacing of the rods, and "d_{net}" is the reduced length of the NSM rods.

As per our dimensions $L_{tot\ min}$ is defined by first equation and is equal to 112mm. And hence we have

$$\boldsymbol{V_{1F} = 2 \times \pi \times 10 \times 11 \times 112 = 77.41\ kN}$$

The contribution of concrete in shear strength is similar as in first case and

$$\boldsymbol{Vc = 61.173kN}.$$

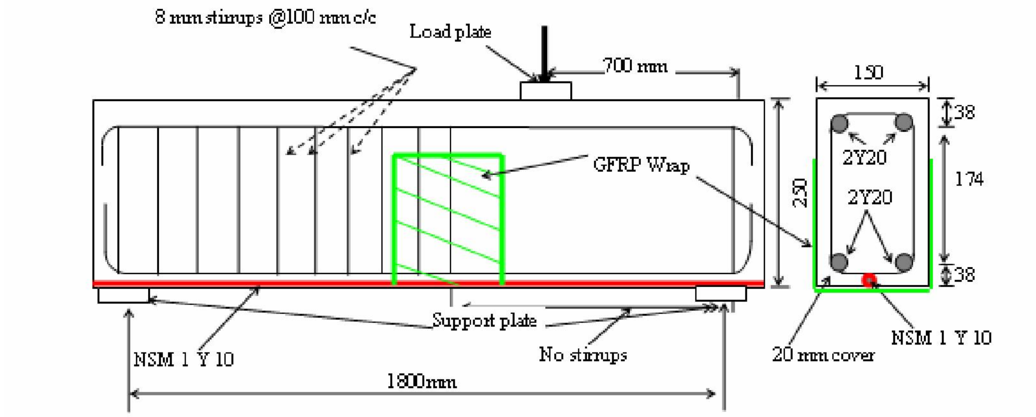
Hence,

$$\boldsymbol{Vu = Vc + V_{1F} = 71.41 + 61.173 = 132.583kN}$$

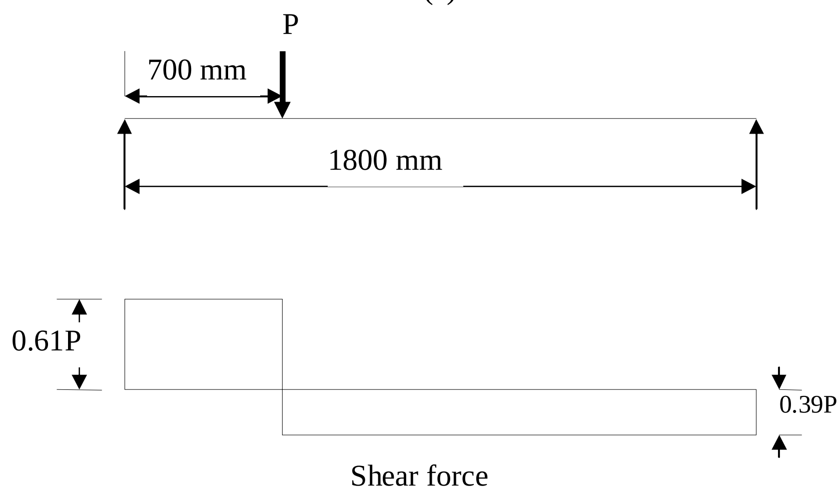
And from the SFD it is clear that

$$P_u = \frac{V_u}{0.61} = 217.4kN$$

8.3 Beam RWS without any shear reinforcement (S, RWS)



(a)



(b)

Fig 8.3 Loading and SFD diagram for beam (RWS)

As in above analysis we find the shear strength of the beam without any shear reinforcement by using equation given in ACI 318 as

$$V_c = 3.5\sqrt{f_c}\,b_w d$$

And we have

$$\boldsymbol{Vc = 64.5\ kN = Vu}$$

And the ultimate load Pu is given by

$$P_u = \frac{V_u}{0.61} = 105.7\,kN$$

8.4 Beam B1-SS strengthened with NSM steel strips (S1, B1-SS)

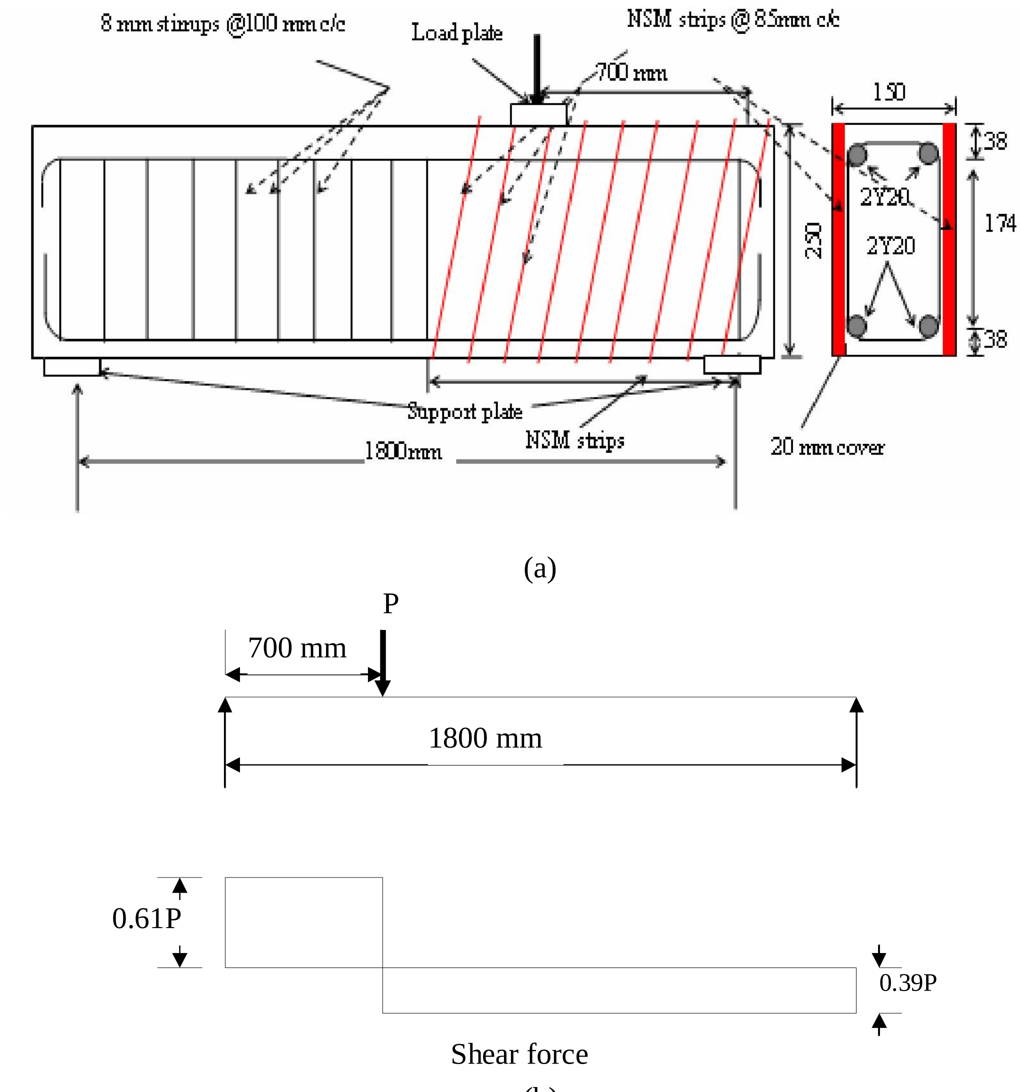


Fig 8.4 Loading and SFD diagram for beam (B1-SS)

Here also concrete of cube compressive strength of 40MPa is used.
Hence "Vuc" is similar as in the above cases

$$V_c = 61.173kN$$

And "Vus" is calculated from the same equation as given in case 2 but as the term $\pi.d_b.L_{tot\ min.}$ is the surface area of the NSM rod. The surface area of the NSM strip is given by $2.b.L_{tot\ min}$. where “b” = 16mm is the width of the strip. The shear contribution of the shear strip is thus given by

$$V_{us} = 2.(2.b.L_{tot\ \mathrm{min}})\tau_b = 88.7kN$$

Therefore

$$\boldsymbol{Vu = Vc + Vus = 149.88\ kN}$$

and the ultimate capacity of the beam is given by Pu as

$$P_u = \frac{V_u}{0.61} = 245.7kN$$

9. Discussion and comments on test results

9.1 Crack pattern

1. Beam RWS: The beam S casted without shear reinforcement on half side as in figure 2.13 and retrofitted as shown in figure 3.3 was as applied with load at 700mm from the support as in figure 3.5. The ratio of the shorter shear span (700 mm) to the effective depth of the beam (212 mm) was kept greater than 3 i.e. (a/d > 3) to avoid an increase in shear strength due to short span tied arching action and to develop a diagonal shear crack in beam web. The first crack initially initiated under the load was vertical and flexural. Latter on increasing load at a constant load interval few more cracks vertical cracks initiated and at a load of 75kN from the crack which was tending to get inclined a shear crack initiated and propagated both side towards the load and support. And at 90 kN the crack cracks the top surface under the load and the beam got failed in a sudden manner with a breaking sound as shown in figure 7.7 and 7.8. This kind of cracking pattern was expected from the kind of setup arranged. As the load was applied in intermediate range of a/d (3 < a/d < 6) the failure takes place due to the combined effect of shear and bending. This kind of sudden failure is exactly explained earlier in this report under the section (6) test results. Hence this mode of failure is known as diagonal – tension failure or shear – flexure failure.

This mode of failure is unacceptable and it should be avoided using shear reinforcement as the failure is sudden and brittle and does not give any warning prior to failure. As can be seen the ultimate failure shear load V= 54.9 kN (P= 90kN) was only at a shear load V= 9.15 kN (P= 15kN) more than the shear load V= 45.75kN (P= 75kN) at which first shear crack appeared. Thus the load increase from knowing the failure and to the failure is not too much and does not give enough warning time hence should be avoided. The purpose of the project was thus to avoid this kind of failure in a similar beam without any shear reinforcement on one half by strengthening the beam for shear using NSM steel strips and NSM steel rods and to observe their effect and failure behavior.

As the beam was retrofitted using NSM steel rod at the bottom face a debonding was seen near the support on beam failure as shown in fig 7.9 the cracking was at the interface of the concrete and epoxy mortar. Meanwhile the GFRP was unaffected by the failure but earlier the flexure crack occurred very near to it.

2. Beam B1-SS: The beam S1 casted without shear reinforcement on half side as in figure 2.13 and was strengthened using NSM steel strips of 16mm wide and 0.8mm thick as shown in figure 3.1 was as applied with load at 700mm from the support as in figure 3.6. The ratio of the shorter shear span (700 mm) to the effective depth of the beam (212 mm) was kept greater than 3 i.e. (a/d > 3) to avoid an increase in shear strength due to short span tied arching action and to develop a diagonal shear crack in beam web similar as in the first case to have a proper comparison of the crack pattern.

The types of cracking observed during the testing of the beam were debonding of the NSM strip at the interface of NSM and filler (epoxy-sand mortar). At few locations the cracks originated along the NSM enters into the concrete and propagated towards the load. At a shear load of 94.6 kN (P= 155kN) shear crack in the concrete between 4th and 5th strip was observed. Latter two more diagonal shear cracks both at different location in concrete between 3rd and 4th strip and between 5th and 6th strip occurred at higher load. Thus it shows that a single diagonal shear crack in beam without shear stirrups was converted into multiple diagonal shear cracks by using NSM inclined shear strip. Also the main mode of the failure was the debonding seen in the NSM strips at interface of NSM and filler (epoxy-sand mortar) under the load.

Other type of cracks observed were flexural cracks which first occur at a load of 115kN and than these cracks were very prominent at load more than 170kN. These cracks could be seen at the bottom face and at higher loads they were very closer to support.

Thus from comparing the cracking patterns of RWS beam and B1-SS beam the main observation were that a single diagonal shear crack in beam RWS without shear stirrups was converted into multiple diagonal shear cracks by using NSM inclined shear strip in beam B1-SS.thus the shear failure was distributed. The concrete between the strips was thus confined. Also it was seen that the major failure was due to debonding of the strip under the load. Thus the shear failure was avoided. At the loads near the termination of the test due to the capacity of the proving ring (200kN) flexural cracks at the bottom face were seen.

All the above discussion shows that the use of NSM shear strip has improved the mode of failure from a sudden, brittle, diagonal shear failure to a prolonged, ductile and debonding failure which could have led to a flexural failure at higher load than 200kN if the test had been continued. All these improvements in the shortcomings of the structure without shear stirrups are due to the use of NSM strips are similar to the improvements seen due to improvement due to internal stirrups. Hence NSM steel strips could be used as the shear strengthening scheme in retrofitting the concrete structures.

9.2 Load carrying capacity

The load carrying capacity of the beams can be compared at the loads when the first crack diagonal shear crack was seen. In RWS it happened at 75kN and in beam B1-SS it occurred at 155kN. Thus the increase was about 80kN (106.7%). On comparing the ultimate load carrying capacity of the beams, the increase in the shear capacity could have been more than 122.22% than Vuc, if the test was completed (compared with $Pu_{(exp)*}$ =200kN the value at debonding failure) .

On comparison of experimental values with the analytical values, the load carrying capacity of beam RWS $Pu_{(exp)}$ =90 kN which is very near to $Pu_{(ana)}$ = 105.7 kN. This indicates that it has achieved 85% of the analytical value ($Pu_{(ana)}$ = 105.7kN).

In beam B1-SS, the experimental value was not available at the ultimate failure as the test could not be conducted to its ultimate but it can be said that it would have failed at a load more than 200kN and taking it as the value for comparison hence $Pu_{(exp)*}$=200 kN the value of load at which debonding failure occurred. Thus $Pu_{(exp)*}$=200 kN and $Pu_{(ana)}$ =245.7 kN. $Pu_{(ana)} > Pu_{(exp)}$ by less than 45 kN or this could have been very near values. In this case $Pu_{(exp)*}/Pu_{(ana)}$ = 0.81 which is quiet near.

Now comparing the ultimate values of the beams obtained analytically, in RWS it was 105.7kN and in beam B1-SS it was 245.7kN. Thus showing an increase of 132.4% and which is comparable to the experimental results (an increase of more than 122.22%).

On comparing the analytical result for beam B1-SS (Pu = 245.7kN) and the beam BRS (Pu= 225.75kN) the beam with internal shear stirrups shows that the ultimate load carrying capacity is nearly same with an improvement of 8.84% in the beam with NSM strips.

On comparing the analytical result for the B2-SR (Pu = 217.4kN) the beam strengthened with NSM steel rods and the beam BRS (Pu = 225.75kN) the beam with internal shear stirrups shows that the ultimate load carrying capacity is nearly same with an reduction of 3.7% in the beam with NSM steel rods.

Comparing the analytical results of two different NSM materials it can be said that the strengthening the beam with NSM steel strips which gives an increase of 132.4% in comparison to Vuc is better than strengthening the beam with NSM steel rod which gives an increase of 105.67% in the load carrying capacity of the beam.

9.3 Deflection under load

As can be seen from the graph in figure 9.1 the deflection under the load in the B1-SS is less at a load than the deflection in the beam RWS at the same load. The maximum deflection at the failure in RWS was 7.56mm. The load vs deflection line is seen nearly linear upto the load of 75kN and deflection of 4.40mm and on further increase in load by 5kN to 80kN the deflection was increased by 0.90mm. And at 90kN the beam failed in shear in a brittle manner. The change in linearity was at the same load when the shear crack was seen during the test at 75kN.

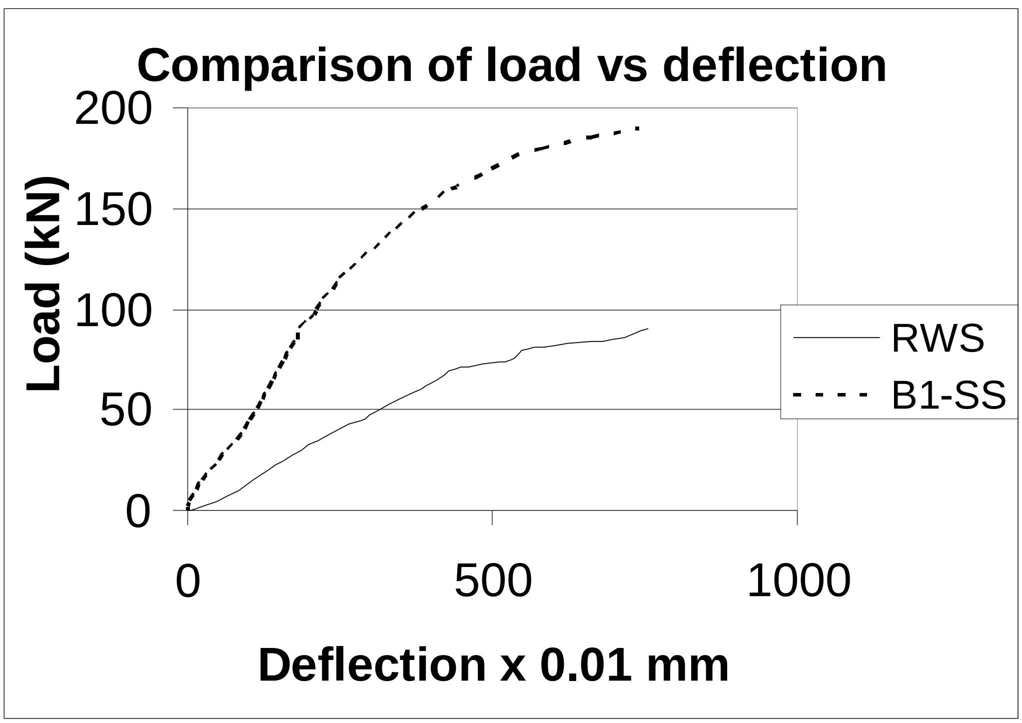


Fig 9.1 Comparison of load vs deflection

But in case of the B1-SS the maximum deflection was 7.42mm at the load 190kN (nearly same as in RWS at failure). Thus it can be seen that the load for same deflection has increased by almost 110% after strengthening the beam with NSM strips. The change in linearity was can be seen from the graph at 150kN which is almost at the same load when the shear crack was seen during the test at 155kN. The beam was tested till 200kN and still the beam was not failed completely but had some debonding of NSM along the strips. From the graph it is clear that the plastic flow had started after a load of about 175kN. Thus the failure which was sudden and brittle in RWS has been converted into prolonged and ductile failure by strengthening the beam using NSM steel strips.

Thus using NSM the stiffness of the beam was increased and more than double i.e. stiffness for RWS was 11.9kN/mm and for B1-SS stiffness is 25.61kN/mm an increase of about 115%.

9.4 Strain variations in Longitudinal steel reinforcement

As shown in the following figure 9.2

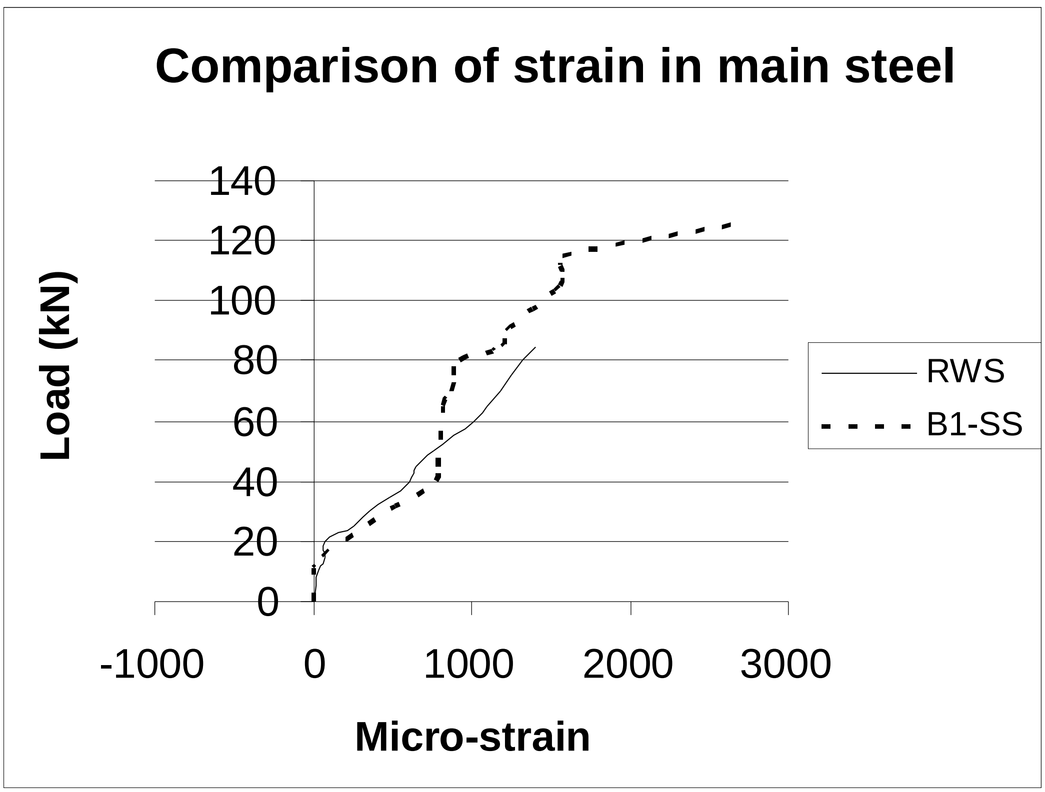


Fig 9.2 The strain variation in longitudinal steel wrt. load

In beam RWS, the magnitude of maximum strain was 1408 microstrain at an applied load of 85 kN and 2630.5 microstrains was recorded in B1-SS at an applied load of 125kN. The load vs strain curve for beam RWS is almost linear till failure clearly showing that the yield in longitudinal steel has not yet occurred.

But in beam B1-SS the load vs strain line is nearly linear upto the load of 115kN and the strain is 1573.3 microstrains. The strain was increased to 2630.5 microstrains at load 125kN clearly showing a plastic flow and thus yield has occurred in the longitudinal reinforcement. But since the failure in beam had not occurred till 200kN the beam had shown a prolonged and ductile behavior which is a characteristic of the shear strengthened beam with stirrups. Hence the beam B1-SS has been strengthened in shear by using NSM steel strips.

9.5 Strain variation in NSM strips at different distances from the support

As shown in figure 9.3 the strain variation vs position of NSM wrt. support at a load of 180 kN.

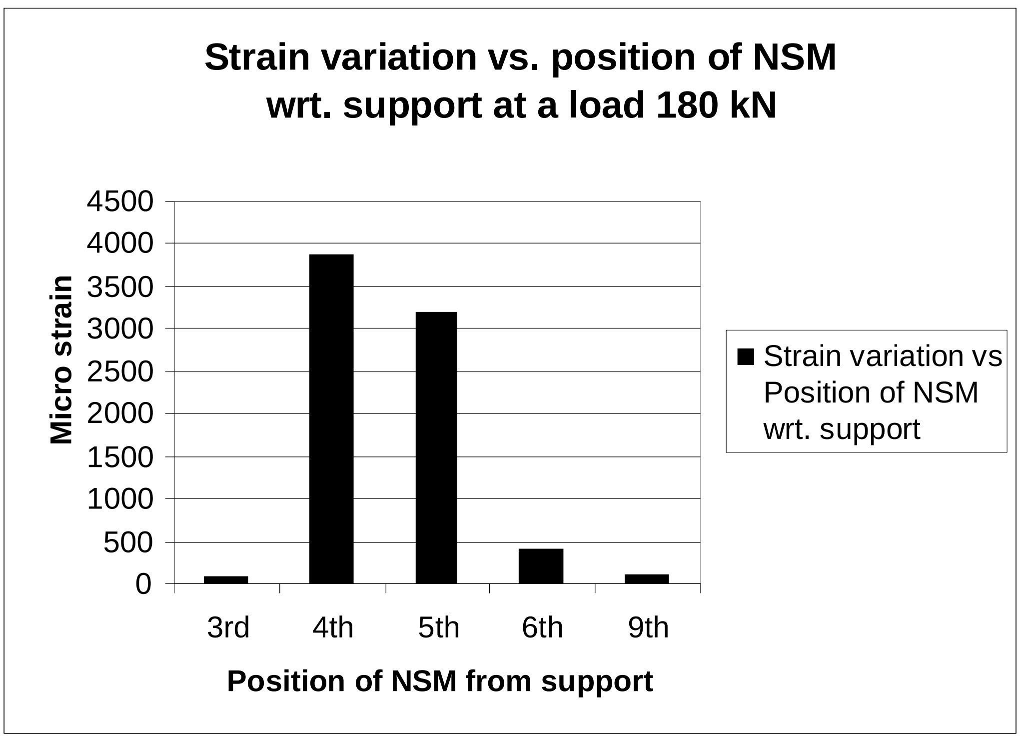


Fig 9.3 The strain variations in NSM strip at different location at 180kN

It is clear from the figure that the 4th and 5th strips from the support has participated the maximum in carrying the shear load. It can also be seen from the figure (7.19 and 7.29) that the initial shear crack had occurred in concrete between the 4th and 5th strip at a load of 155kN. The 3rd,6th and 9th strip from the support does not took part in sharing the shear load as there was debonding of NSM strips at the interface of strip and groove filler had occurred very soon as they started to take shear load. It can be seen from the figure 7.28 the strain in 5th strip was more than in 4th strip initially but near failure it was reversed. Also it can be seen from the figure 7.28 that the 6th strip later took the shear load when both 4th and 5th NSM strip had reached the yield strain of 2600 microstrain i.e. at a load more than 180 kN.

9.6 Steel requirement of various methods

Steel provided in the beams can be compared to each other in term of volume required. Volume of steel provided in beam B1-SS in form of NSM steel strip is 85536mm^3, and the surface area provided is 106920mm^2. The surface area calculated neglects the area of thinner sides of the strip. The ratio of surface area to volume of the NSM strips provided is 1.25.

Volume of steel provided in beam BRS; the beam with internal shear stirrups is 321699mm^3 and the surface area provided is 16085mm^2. The ratio of surface area to volume of internal shear stirrups provided is 0.05.

Volume of steel provided in beam B2-SR; the beam with internal shear stirrups is 339292mm^3 and the surface area provided is 16964.6mm^2. The ratio of surface area to volume of internal shear stirrups provided is 0.05.

The percentage saving of steel using NSM steel strips wrt. NSM steel rods is about 74.79%.

And the percentage saving of steel using NSM steel strips wrt. internal steel stirrups is about 73.41%. this kind of saving in steel is due to higher surface area provided by the NSM strip.

It must be noted from Section 9.2 that the percentage increase in load carrying capacity of beams BRS, B1-SS and B2-SR wrt. beam RWS; the beam without any kind of shear reinforcement are 113.57%, 132.4% and 105.67% respectively. Hence from this discussion it can be commented that NSM steel strips are effective in terms of load carrying capacity and cost.

10. Conclusion

As it is seen from the above discussions that the use of NSM steel strips has improved the mode of failure from a sudden, brittle, diagonal shear failure to a prolonged, ductile and debonding failure which could have led to a flexural failure at higher load than 200kN if the test had been continued. Also with use of NSM steel strip, the stiffness of the beam was increased and more than double i.e. stiffness for RWS was 11.9kN/mm and for B1-SS stiffness is 25.61kN/mm an increase of about 115%. Also with use of NSM steel strips for shear strengthening the experimental increase in the shear capacity could have been more than 122.22% than Vuc, if the test was completed (compared with $Pu_{(exp)*}$ =200kN the value at debonding failure and the last load applied). Also the percentage saving of steel using NSM steel strips wrt. NSM steel rod is about 74.79%. And the percentage saving of steel using NSM steel strips wrt. an internal steel stirrup is about 73.41%. and the increase in load carrying capacity(analytical) of beams; BRS, B1-SS and B2-SR wrt. beam RWS; the beam without any kind of shear reinforcement are 113.57%, 132.4% and 105.67% respectively are comparable. This kind of saving in steel and increase in load carrying capacity is due to higher surface area provided by the NSM strips for same volume of steel. Hence it can be concluded that NSM steel strips can be used to significantly increases the shear capacity of the RC beams by converting the mode of failure from a sudden, brittle, diagonal shear failure to a prolonged, ductile and debonding failure which could have led to a flexural failure at higher loads if test was completed. Also this method is cost effective in terms of savings in steel and also the operational cost is less than other retrofitting method.

In future other two beams (BRS and B2-SR), can be tested and their test results can be related to experimental results of the tested beams (RWS and B1-SS) and these test results can also be related to the analytical results.

Hence our scheme for shear strengthening of beams using NSM steel strips was successful. This technique needs more attention in research in various aspects as it possesses a lot of scope in future retrofitting industry.

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