CHAPTER-1

INTRODUCTION

1.1. BRIEF HISTORY

The idea of composite construction is not new and can be traced back over a period of seventy-five years. It is only within last twenty-five years or so that an intensive research program has been in progress in the field of steel composite construction.

Historically, the first analysis of a composite section was based on conventional assumptions of elastic theory. In recent years the concept of Ultimate load/Limit state design philosophy have been applied to composite action and experimental evidence has shown to be safe, economical to proportion composite section.

Scott(9) reported the first systematic approach in the field of composite construction in 1925 on testing of beams. The relevant design criteria for design of composite structures were established in 1930.Later, Newmark(10) and Viest(11) developed elastic approach to utilize full and partial composite action. Based on the above work, behaviour as well as flexural strength of steel- concrete composite construction was explained by Chapman(12) and Slutter(13) using ultimate strength approach.

1.2. GENERAL REMARKS

Materials of vastly different kinds are frequently brought together to form a single composite unit, the combined performance of which is superior to the sum of the performances of the separate elements. In the field of structural engineering an early example is the flitched timber beam in which steel plates were bolted or screwed to a timber beam to increase its strength and stiffness. Other examples are foamed sandwich construction, fiber reinforced plastics, and even reinforced and prestressed concretes. But in the context of this dissertation the term composite construction is restricted to describe structural elements in which reinforced concrete interacts with structural steel work.

The most important and frequently encountered combination of construction materials is that of steel and concrete, with applications in multistory commercial buildings, microwave towers, factories as well as bridges.

RCC slab is in compression and steel joist is in tension and hence most effective utilization of the material is possible as well as a lower structural steel section can be provided in composite construction compared to the section required for non composite construction .

Reduced beam depth reduces the storey height and consequently the cost of cladding in a building and lowers the cost of a building and lowers the cost of embankment in a flyover. Reduced depth also allows provision of lower cost for the fireproofing the beams exposed faces. Composite sections have higher stiffness than the corresponding steel section and thus deflection is lesser. Composite structures are best suited to resist repeated earthquake loadings, which require a high amount of resistance and ductility. Faster construction is possible due to the maximum utilization of rolled and/or fabricated components in case of composite construction.

In composite construction there are three principal elements those are the reinforced concrete, the structural steel work and the shear connector. Each of which makes its own contribution to the strength and stability of the structure.

Out of above mentioned three principal elements shear connector has its own importance in composite construction. Whatever its detailed makeup, it must perform two functions: to transfer shear between the steel and the concrete (i.e. to limit slip at the interface) and to prevent separation of the steel and the concrete at right angles to the interface i.e. to prevent uplift).

1.3 SCOPES & OBJECTIVES

The main objectives of the present study is to evaluate the behaviour of shear connectors in steel-concrete composite construction. From the literature three major areas were identified that need further investigation, which are; (i) Load-ship relationship (ii) stress-strain behaviour (iii) uplift of the concrete at the interface of composite construction. Standard push-out test recommended in CP: 117 carried out in the laboratory to evaluate all the behaviours mentioned above in the shear connectors. As to prepare the composite construction self-compacting concrete has used so to check its self-compactability slump, U-flow, V-flow, L-box tests also carried out. Finally to know the properties of concrete compressive strength` test, split tensile tests, UPV tests, Flexural strength tests also carried out. The test results of the present study and those identified in the literature were then used to get some useful conclusions.

1.4 ORGANIZATION OF CHAPTERS

This thesis organized in six chapters and the contents of each chapter and explained subsequently.

Chapter – 1 briefly introduces the topic and describes the scope and objectives of this research.

Chapter – 2 reviews the literature on steel-concrete composite construction and the requirements of shear connector in composite construction along with its; design specifications and various methods to find out the strengths, spacing and numbers. Also it covers different theories of composite action and methods of construction. Above all emphasize has given on the recent research works on the related topic in a summarized manner.

Chapter – 3 briefly describes the standard push-out test recommended by CP: 117, which has used in this experimental work to evaluate the behaviour of shear connector in steel-concrete composite construction.

In Chapter – 4 the constituent materials that are used, the mix design which adopted, fabrication of push-out specimen which prepared in the laboratory for the standard push-out test, the tests that were conducted on the fresh concrete during casting of specimen and the tests that were conducted on hard concrete elaborated as well as discussed briefly with their results.

Chapter - 5 deals with the experimental set up of the standard push out test along with their test results and analysis for different specimens.

Chapter – 6 summarizes the significant conclusion derived from this study along with the future scopes of work.

CHAPTER-2

LITERATURE REVIEW

GENERAL

This chapter deals with the basics of steel-concrete composite construction; different types of shear connectors along with their requirements/uses. It also covers different methods of composite constructions as well as the theoretical approaches in composite action and design specifications of composite constructions and shear connectors.

2.1. STEEL-CONCRETE COMPOSITE CONSTURUCTION

According to Chapman(12) the term 'composite construction in steel and concrete' is taken to include combinations of steel and concrete where interaction between the two elements can be taken into account in design, with the exception of those combinations which fall wholly within the generally accepted definitions of 'reinforced concrete' and 'prestressed concrete'. Thus, for example, the definition will include combinations of a rolled steel beam and a reinforced concrete slab, of a steel plate and reinforced or unreinforced slab, of a tubular or box column filled with concrete or of a latticed steel column embedded in concrete; it will exclude the interaction of steel rods and the surrounding concrete in reinforced concrete, of a precast beam and a reinforced concrete slab, or of one precast concrete element with another. The interaction of frames and walls is also excluded.

Commonly used forms of composite system are employing filler-joist bridge deck, solid web steel beams supporting concrete slabs. Supporting beam may be an open web joist, a castelled beam, a truss or a box beam. The concrete slab may partially or wholly enclose the beam. Also, steel tube filled with concrete is a composite construction for composite column. **The only requirement is that composite action must be ensured by means of suitable shear connectors** (Refresher course on composite construction, 2000).

Applicable codes in steel-concrete composite constructions in India are **IS: 11384-85** (superseded **IS: 3935-66** i. e. Code of Practice for Composition Construction) Code of Practice for Composite Construction in Structural Steel and Concrete (This code is restricted to buildings), **IRC: 22-86**, Section VI, Standard Specification and Code of Practice force Road Bridges-Composite Construction.

2.2. SHEAR CONNECTOR

According to Davis[21] shear connector that defines the various forms of steel structure parts connected to the upper segment of the steel beam most often by welding or pre-thermal stressed screws. The shear connectors are provided to ensure the integral action between the two structural units so that the two units act as composite structure.

According to IS: 11384-85, the shear connectors shall be of weldable steel and shall be end welded to the structural members. The capacity of the welds at permissible stress shall be not less then the shear resistance of the connectors. Welding shall be in accordance with the requirements of the relevant Indian Standard.

2.2.1. VARIOUS SECTIONAL FORMS OF SHEAR CONNECTOR

As per Davis[21] the transmission of shear forces and the intensity of stress in the steel beam, the weld that connects the shear connector to the flange of the beam, material of connector itself and the surrounding concrete of the slab, which all determines the strength, are highly dependant on the form of the shear connector. There are very different forms of means for composition that are used in practice.

1.Headed Stud.

2.Channel Connector.

3.Tee Connector.

4.Angle Connector.

5.Helical.

6.Bar.

7.Tension.

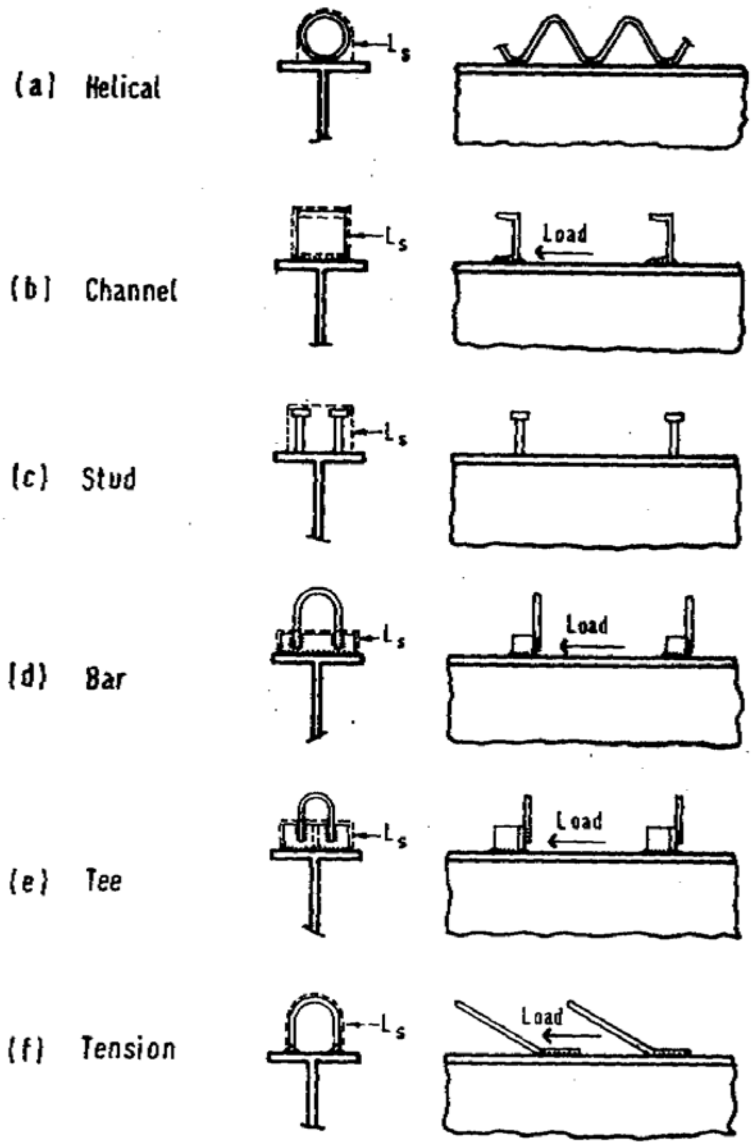


Fig. 1. Types of welded shear connector

2.2.2. TYPES OF SHEAR CONNECTORS

Shear connectors shall consist of any or a combination of the following types:

RIGID TYPE CONNECTORS

According to IRC:22-1986 these connectors derive their resistance to horizontal shear from the bearing pressure of concrete distributed evenly over the surface because of the stiffness of the connectors. Failure or slip is generally associated with the crushing of concrete. With a view to prevent the separation of the in-situ slab from the pre fabricated unit in the direction perpendicular to the contact surface, some mechanical device must be provided along with these connectors.

Rigid connectors consisting of short lengths of bars, angles, tees and channels welded on to the flange of the steel fabricated units as shown in Fig.2.

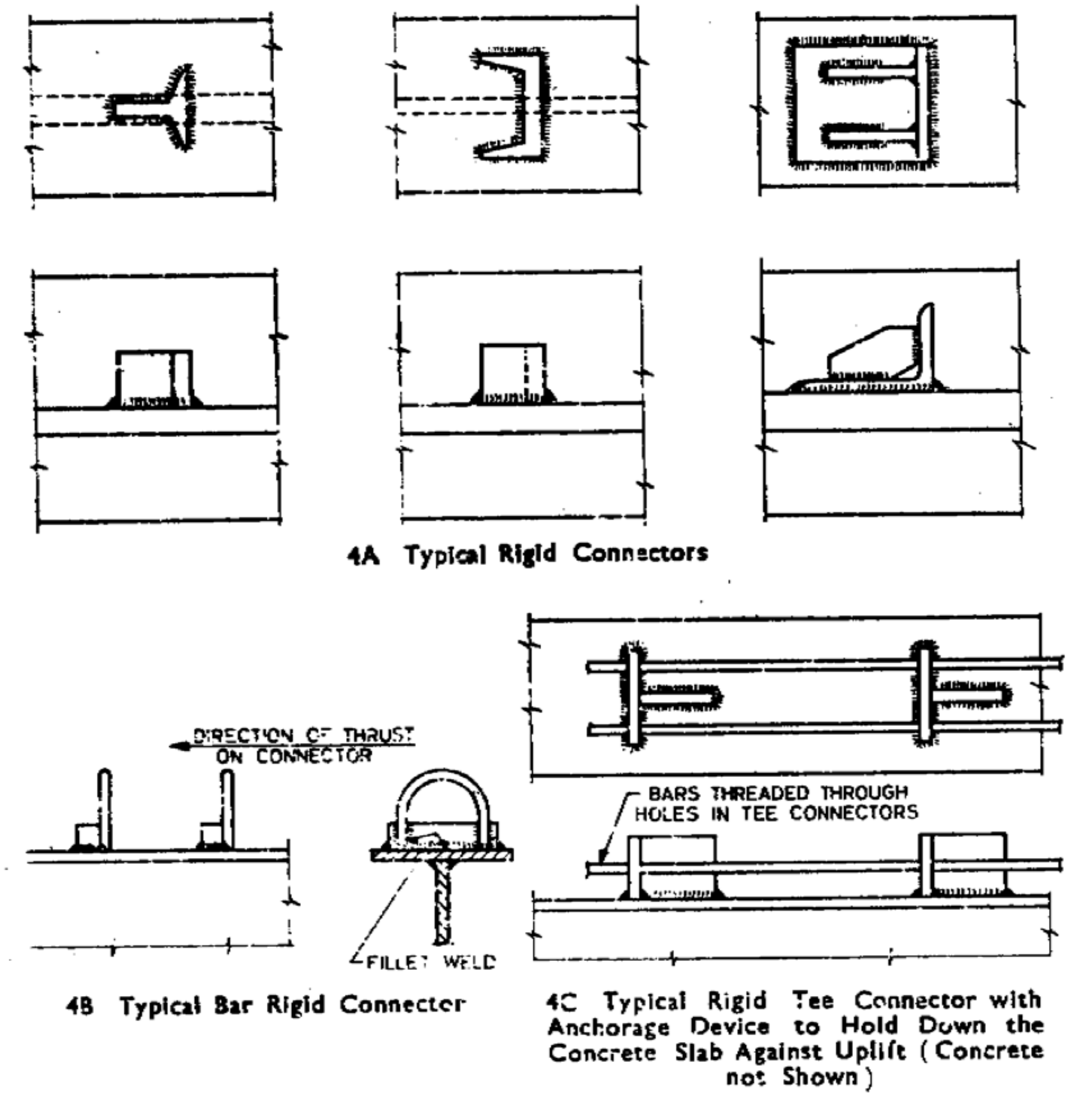


Fig.2.Typical rigid connectors.

FLEXIBLE TYPE CONNECTORS

They derive their resistance through the bending and normally failure occurs when the yield stress in the connector is exceeded resulting in slip between the steel beam and the connector slab (IS:11384-85).

Flexible connectors such as studs and channel connectors have shown in Fig.3 and Fig.4. In each of these cases the shear transfer occurs initially in the vicinity of the weld and the remainder of the connector (the head of the stud and the top flange of the channel) provides the anchorage against uplift. The welded stud connector is currently the most commonly used.

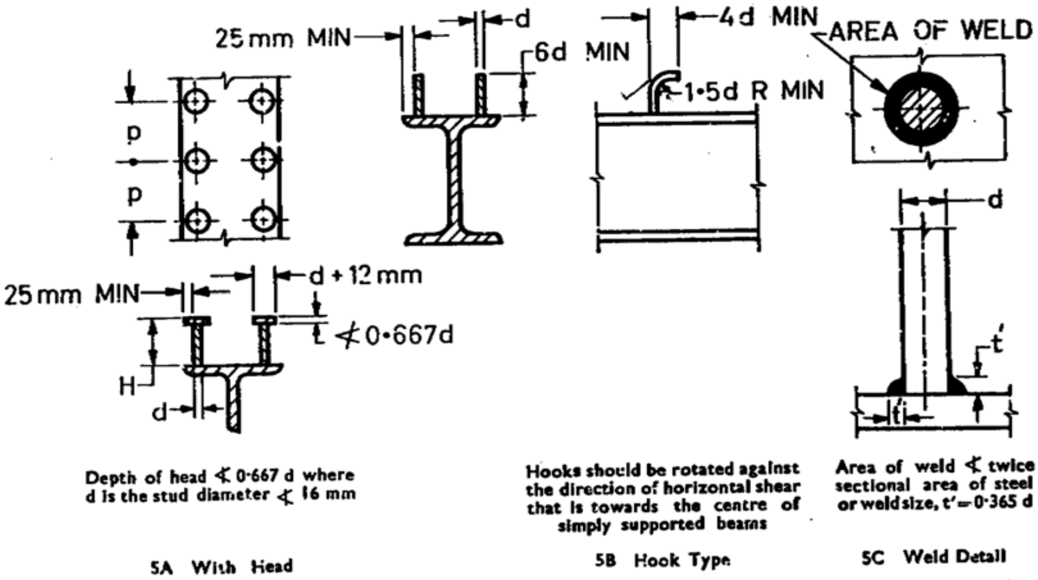


Fig.3. Typical Flexible Stud shear connectors.

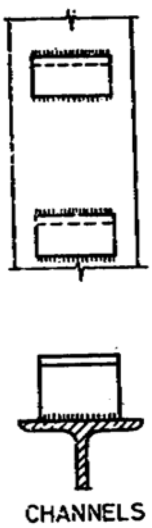


Fig.4.Typical Flexible channel connector.

BOND OR ANCHORAGE TYPE CONNECTORS

According to IS:11384-85,They derive their resistance through bond and/or anchorage action. They normally consist of the following and shown in Fig.5 and Fig.6.

1. Mild steel bars welded to the flange of the prefabricated unit in the form of vertical or inclined loop stirrups.
2. Inclined bars with one end welded to the flange of the steel unit and the other end suitably bent.
3. Bar stirrups welded to the flange of the steel in the form of helical shape.

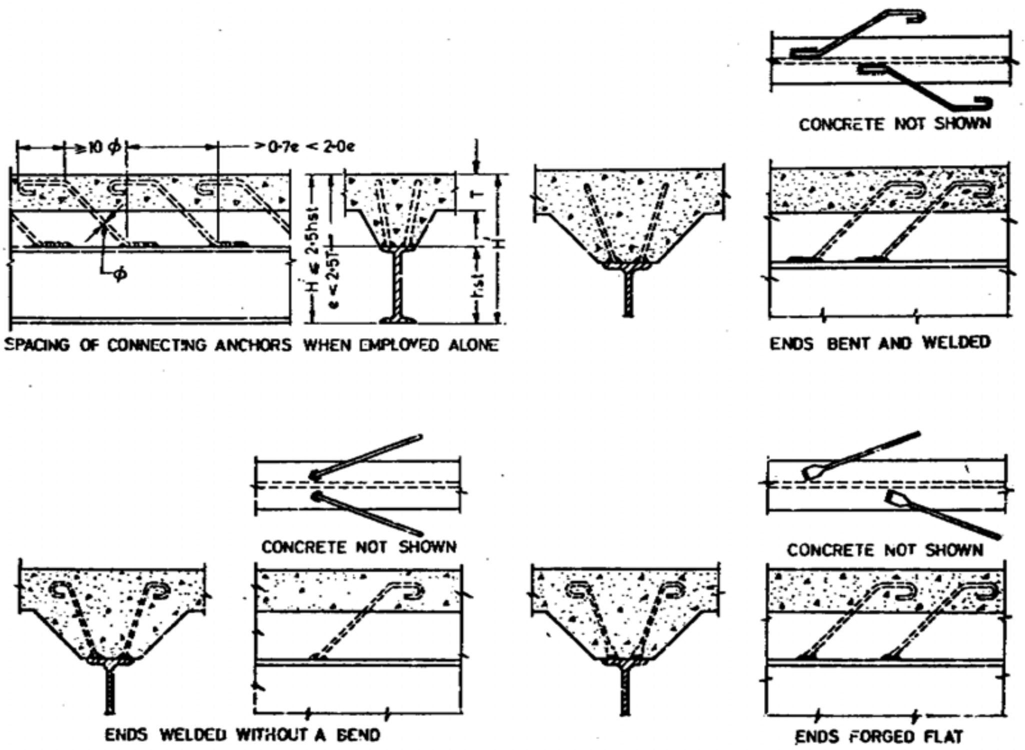


Fig.5. Typical Bond types connectors.

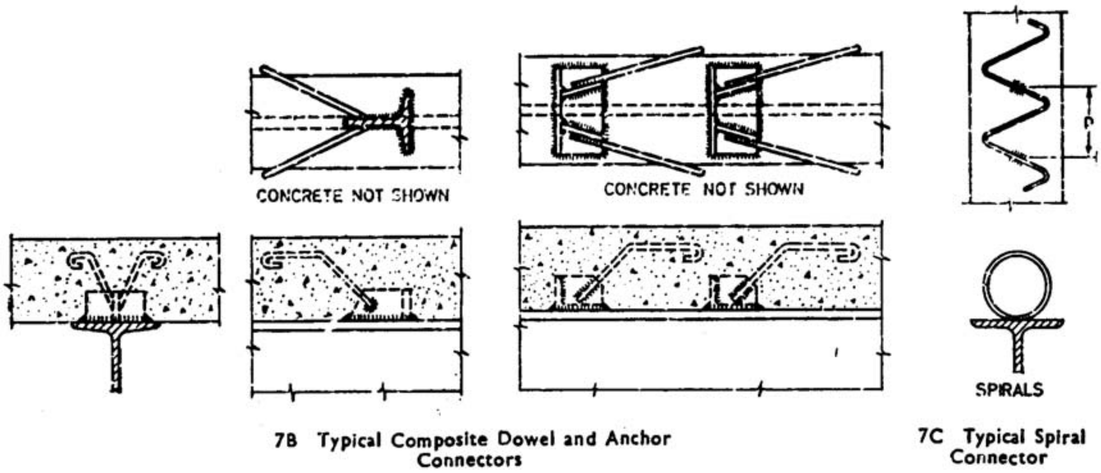


Fig.6. Bond or Anchorage type connectors.

2.3. INTERACTION OF SHEAR CONNECTOR

The degree of interaction deals with the compatibility of displacements. In general there are two types of interactions (Oehlers 1997).

COMPLETE INTERACTION

According to Devis[(21)], when a beam is subjected to bending, there is no longitudinal horizontal slip at the interface between the steel beam and the concrete slab then the interaction is said to be complete and the longitudinal strain due to flexure would very linearly and continuously over the entire depth of the composite beam section. Such an assumption can not be strictly true since however 'rigid' the shear connection between the steel and the concrete, some horizontal slip must take place at the interface, and in consequence the strain diagram is discontinuous at the level of the interface. However, for practical design purposes the assumption is not unreasonable and the error involved is normally small.

INCOMPLETE INTERACTION

According to Devis [(21)], due to the flexibility of the shear connectors and the compressibility of the concrete, horizontal slip at the interface cannot be completely prevented. The interaction between the steel and the concrete must, therefore, be incomplete and the effect of slip at the interface is to produce a discontinuity in the strain diagrams.

As per Deric J.Oehlers [29], for composite beams in buildings, where the axial strength of the concrete section is usually much larger than that of the steel section, incomplete interaction has virtually no effect on the strength. Conversely, incomplete interaction can reduce the strength of composite beams with very strong steel sections that is where the axial strength of the steel section is much greater than that of the concrete section.

2.4. DESIGN OF SHEAR CONNECTORS

According to Chapman[3],whether the composite section is designed on working stresses or on a load factor, it is rational to ensure that the shear connection should not be the cause of beam failure; failure of the shear connectors results in a sudden diminution in the carrying capacity of the beam, so that a catastrophic failure of the beam ensues, and this would not be a satisfactory basis for design.

It follows, therefore, that sufficient shear connectors should be provided to resist the total horizontal shear, which exists at failure of the beam. This force is very easily calculated for a rectangular stress distribution. For example, in a simply supported beam having the plastic neutral axis in the slab, the total horizontal shear force is equal to the yield stress multiplied by the area of the steel beam. If there is an end fixing moment, the horizontal shear is increased by the area of the effective slab reinforcement multiplied by its yield stress.

The determination of the shear distribution at failure is a lengthy process, but there is sufficient experimental evidence that flexible shear connectors perform satisfactorily up to beam failure if they are spaced uniformly along with length of the beam. This conclusion holds for central point loading (where the external shear is uniform) and for uniformly distributed loading (where the external shear varies linearly along the beam). A uniform spacing of connectors would not, however, be satisfactory where the bending moment is nearly uniform along the length of the beam. The redistribution of shear force that occurs with flexible connectors could not necessarily be relied upon in the case of very rigid connectors, or in the case of a deformed flange.

It seems preferable that the limiting load per connector (at beam failure) should be derived from test results, rather than from stress calculations based on an assumed pressure distribution. It is important that the degree of containment of the concrete surrounding the shear connectors should be correctly simulated in the test./

It is then necessary to decide whether the limiting load of the connector should be taken as some percentage of the ultimate load or as the load at which some specified slip occurs. The former alternative has the advantage of simplicity and is probably to be preferred for design purposes. A '100 per cent design is then one in which the shear connectors fail when the beam reaches its fully plastic moment, and it is reasonable that this should represent the theoretical minimum provision of shear connectors. If, for example, an 80 per cent design is adopted in practice, then the margin against shear failure is at once apparent.

The above argument is somewhat over simplified because in a 100 per cent design extensive slip occurs before the fully plastic moment is reached, and the stress distribution in the beam is considerably modified, so that the suggested basis for calculating the total horizontal shear is no longer valid. Tests have shown that yielding of the top flange does not then occur, but that the fully plastic moment is nevertheless reached by virtue of strain hardening in the bottom flange; the position of the resultant steel force is lowered and the ultimate moment is reached for a smaller horizontal shear than would occur if the stress distribution were rectangular. The stress distribution at failure in a particular beam will depend on the steel and concrete properties, the load-slip curves for the shear connectors, and on the geometry of the cross-section.

It can be said with some certainly that if sufficient shear connectors are provided to ensure that the fully plastic moment can be developed, then the slip at working load will not significantly modify the stress distribution. The exact nature of the stress distribution at failures probably not of direct importance to the designer.

2.4.1. DESIGN SPECIFICATIONS (According to IRC: 22-1986 & IS: 11384-85)

- The shear connectors shall be provided throughout the length of the beam and may be uniformly spaced between critical sections.
- Shear connectors may be either of M.S. or H.T.S. according to the material specifications of steel girders.

- The diameter of stud connectors welded to the flange plate shall not exceed twice the plate thickness.
- The height of the stud connectors shall not be less than four times their diameter or 100mm.
- The diameter of the head of the stud connectors shall not be less than one and half times the diameter of the stud as shown in Fig.7(a).
- The clear distance between the edge of a girder flange and the edge of the shear connectors shall not be less than 25mm as shown in Fig.7.
- The surface of a shear connector which resists the separation between the two units (i.e. the underside of the stud or the inner face of the top flange of a channel or the inside of a hoop) shall extend not less than 40mm into the compression zone of the concrete flange.
- In the case of studs, specialized fusion welding will be necessary and to permit satisfactory welding of studs, the gap between the heads of two adjacent connectors should not be less than 15mm.
- To ensure that the concrete slab is adequately tied down to the steel flange, the overall height of the shear connector (that is, the length of stud, diameter of the helix, height of the channel, hoop, etc) should not be less than 50mm nor project less than 25mm into the compression zone of the concrete slab. The thickness of the compression zone be that at the section of maximum bending moment.
- The leg length of the weld joining other types of connectors to the flange plate shall not exceed half the thickness of the flange plate.
- Channel and angle connectors shall have at least 6mm fillet welds placed along the heel and toe of the channels / angles as shown in Fig.7©.
- Channels and tees should be preferred to angles.
- The diameter of the spiral bars shall preferably be between 12 and 20mm.
- In all composite beams the spiral shall extend at least halfway in the slab.
- The ratio of the pitch of the spiral to the diameter shall be between 0.5 and 4.0.
- The developed length of the spiral per pitch shall not be less than 20 times the diameter of the bar.
- From fabrication consideration, the spiral pitch shall be within the limits of 100mm to 400mm.

- The overall height of a connector including any hoop which is an integral part of the connector shall be at least 100mm with a clear cover of 25mm as shown in Fig.7(c).
- The clear depth of concrete cover over the top of the shear connectors shall not be less than 25mm.
- Horizontal clear concrete cover to any shear connector shall not be less than 50mm as shown in Fig.8.

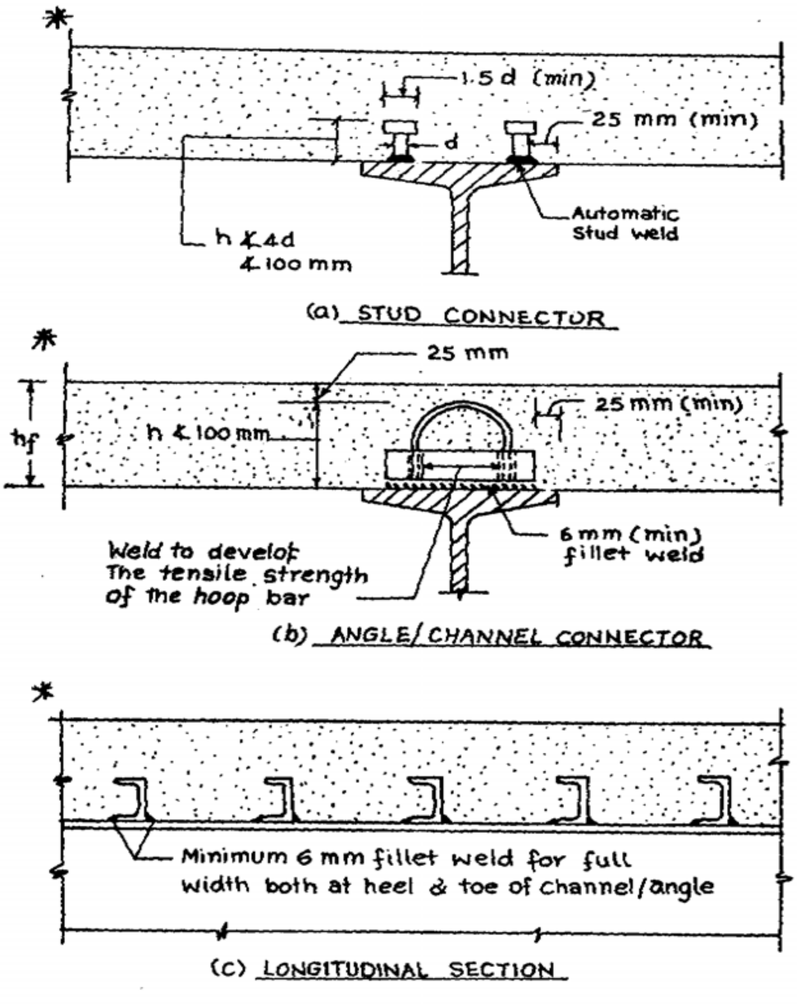


Fig.7. Details of connectors on steel girders.

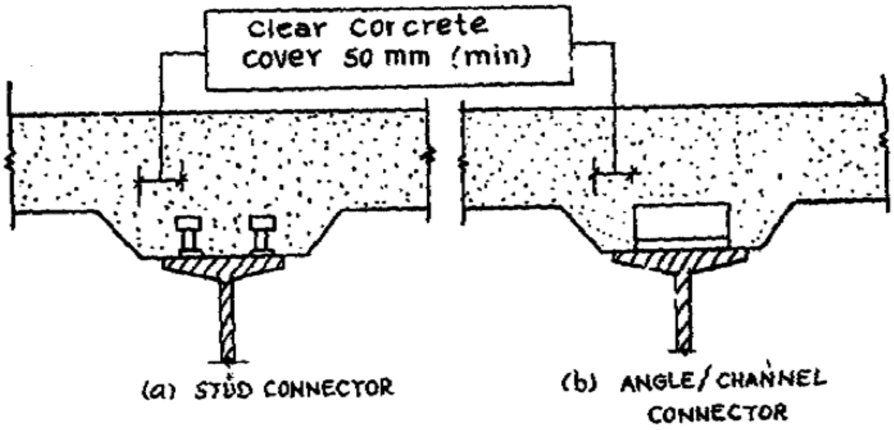


Fig. 8. Cover to shear connectors

2.4.2. CALCULATION OF THE LONGITUDINAL SHEAR

For the limit state design, the longitudinal shear is calculated from the dimensions of the composite beam required for the maximum sagging bending moment according to IS: 11384-1985.At such cross section, it is equal to F_{CC} i.e. the force in concrete in compression.

When the plastic neutral axis lies in the concrete slab in compression,

$F_{CC}=0.36\ f_{ck}\ b\ X_u$

When the plastic neutral axis lies outside the concrete slab in compression,

$F_{CC}=0.36\ f_{ck}\ b\ d_s$

Where, b= effective width of the slab having composite action with the steel beam.

X_u = depth of the plastic neutral axis for the composite beam.

d_s = depth of the slab.

Clause 611.4.1.2 of IRC: 22 gives ultimate strength design of shear connectors of high tensile steel complying with IS: 961 conforming to Fe 540 W117 (with latest amendment).

The maximum horizontal force is given by:

$H_1=A_{st}\ f_y\ .\ 10^{-3}$

$H_2=0.85\ f_{ck}\ b_f\ h_f\ 10^{-3}$

Where,

H_1, H_2= Horizontal force (kN)

A_{st} = Area of tensile steel (mm^2) in longitudinal direction

F_y = Yield stress of steel (Mpa)

F_{ck} = Cube (characteristic) strength of concrete (Mpa)

B_f = Effective width of flange of in-situ slab

H_f = Thickness of in-situ slab

The ultimate flexural strength of any composite section will be governed by either of the aforesaid equations depending upon whether the steel section or the concrete section is large. Therefore, the maximum possible compressive force in the slab would be the smaller of H_1 or H_2 and sufficient connectors should be provided to resist the horizontal force H_1 or H_2

Which ever is the lesser after calculating the ultimate strength of shear connectors from clause 611.4.1.2.2.

Clause 611.4.1.3.3 of IRC: 22 give the longitudinal shear per unit length at the interface of the prefabricated unit and in-situ unit shall be evaluated from the expression given below:

V_L= V.A_C.Y/I

Where,

V=Vertical shear due to dead load placed after composite section is effective and working live load with impact.

V_L =Longitudinal strength per unit length

A_C=Area of transformed section on one side of interface.

Y = Distance of the centroid of the area under consideration from the neutral axis of the composite section.

2.4.3. STRENGTH OF A SHEAR CONNECTOR

(I) IN ELASTIC DESIGN

As per IRC: 22,shear strength for welded channel/angle/tee connector made of mild steel with minimum ultimate strength of 420 to 500 Mpa, yield point of 230Mpa and elongation 21%, is given by:

$Q=107(h+0.5t)\ L\sqrt{f_{ck}}$

Where,

Q= The safe shear resistance in Newton of one shear connector

h= The maximum thickness of flange measured at the face of the web in mm.

t= Thickness of the web of shear connector in mm

L=Length of the shear connector in mm

f_{ck}=Characteristic compressive strength of concrete in Mpa.

For design of M.S. Stud connectors according to IRC: 22-1986

The safe shear for each shear connector shall be calculated as below:

For welded stud connector of steel with minimum ultimate strength of 460 Mpa, and yield point of 350 Mpa and elongation of 20 percent

(i) For a ratio of h/d less than 4.2

$Q=48\ h\ d\sqrt{f_{ck}}$

(ii) For a ratio of h/d equal to or greater than 4.2

$Q=196d2\sqrt{fck}$

Where,

H =Height of stud in mm

d =Dia of stud in mm

Q =The safe shear resistance in Newton of one shear connector.

(II) IN LIMIT STATE/ULTIMATE LOAD DESIGN

Shear strength P_e (viz M20, M30and M40) is given in Table 1 of IS: 11384-1985. For any change in length of the connector or grade of the concrete, interpolation or extrapolation can be suitably done.

As per clause, 611.4.1.2.2. of IRC: 22-1966,ultimate strength are given by the following expressions:

(i) H.T. Stud connectors

$Q_u=0.5A\sqrt{(f_{ck}.E_c)}.\ 10^{-3}$

(ii) H.T. Channel/Angle/Tee connectors

$Q_u=45(h+0.5h_1)L\sqrt{f_{ck}}.10^{-3}$

Where,

Q_u =Ultimate shear resisting capacity of one shear connector (kN)

A = Area of stud connector (mm^2)

h = Average thickness of channel/angle /tee flange (mm)

f_{ck} =Cube strength of concrete (MPa)

E_c =Modulus of elasticity of concrete (MPa)

2.4.4. SPACING OF SHEAR CONNECTORS

According to IRC: 22-1986, the shear connectors shall be provided throughout the length of the beam and may be uniformly spaced between critical sections.

FOR STEEL GIRDER

1. According to IRC: 22-1986, the maximum spacing of shear connectors in the longitudinal direction shall be limited to 600mm or three times the thickness of the slab or four times the height of the connector (including any hoop which is an integral part of the connector) whichever the least.

2.According to IRC: 22-1986, the spacing of the stud connectors in any direction shall not be less than 75mm.

FOR CONCRETE GIRDER

According to IRC: 22-1986, the spacing of the anchor or bond shear connectors shall not be less than 0.7 times the depth of the slab and shall not be greater than two times the depth of the slab.

1.According, to IS: 3935-1966, spacing of anchors when associated with rigid connectors shall not be greater than two and a half times the depth of slab.

COMPUTING SPACING OF CONNECTORS IN ELASTIC DESIGN

Spacing/pitch of shear connector shall be determined from the formula:

$P=Q$ (or) $\Sigma Q/V_L$

Where,

V_L=The longitudinal shear per unit length at the contact surface of the steel beam and the insitu concrete slab at the cross section under consideration.

Q =Safe shear resistance of each shear connector at a cross section.

ΣQ = Sum total of shear resistance of two or more shear connectors at a cross section in the case of stud connectors.

2.4.5. NUMBER OF SHEAR CONNECTORS

The number “n” of the connectors is given by the relation:

$\mathbf{n=F_{cc}/P_e}$

The shear connectors should be equally distributed between the sections of maximum bending moment.

2.5. METHODS OF CONSTRUCTION

Propped construction generally provides greatest reduction in steel weight. This method of construction involves supporting the steelwork from below with suitable props so that the props, which remain in position until the concrete has cured, take the whole of the dead load of steel and concrete. In this way full composite action will be available for dead load (Refresher course,2000[27]) .

Unpropped construction in which dead weight of steel beam and concrete slab is taken by steel beam alone. For bridges, propping may be difficult or impossible. In building prop may be a hindrance to operations such as screeding and finishing which can take place soon after the floor slab is cast if the floor area is free of obstruction (Refresher course,2000[27]).

2.6. DESIGN OF THE COMPOISTE SECTION

According to Chapman [3], the composite beam can be designed satisfactorily on the basis of a transformed section, using an appropriate value of modular ratio; the ratio will depend on the nature of the loading clearly a higher value should be taken for sustained than for transient loading (unless creep is to be separately considered).

To prevent the formation of a longitudinal crack on the line of shear connectors, sufficient transverse reinforcement must be provided to resist horizontal shear, transverse direct stresses and local stresses in the region of the shear connectors, as well as transverse bending stresses.

With certain restrictions it would appear that an ultimate load basis of design could be adopted for composite beams. There is ample experimental evidence that the calculated fully plastic moment is developed when the plastic neutral axis occurs well within the slab thickness, and this is the condition frequently encountered in buildings. When the plastic neutral axis lies close to the interface, it might be necessary to place a restriction on the slab dimensions to ensure that full plasticity of the steel is attained before spalling occurs. When the steel beam is very deep compared to the slab thickness, so that the plastic neutral axis falls well within the beam, flange yielding can again occur before spalling of the slab. In such a case, however, the shape factor would be rather small.

Restrictions may also be necessary to ensure that tensile slab bending stresses do not seriously reduce the effectiveness of the slab, and that the assumed support moments are not inconsistent with the rotational ductility at mid-span. It seems likely that in regions of negative moment of the slab reinforcement can be taken as contributing to the plastic moment, provided that a suitable effective breadth is taken for this purpose.

In the case of simple composite beams without slab haunching or bottom flange plates the adoption of a load factor of 1.75 would lead to considerably higher stresses at working load than are adopted in elastic design, owing to the large shape factor of such a section. In this way proper advantage could be taken of composite action. A similar effect would result from the use of higher working stresses in elastic design, but in that case different working stresses would have to be specified according to the shape factor of the particular composite section used.

2.7. THEORIES OF COMPOSITE ACTION

Historically, the first analysis of a composite section is based on conventional assumptions of elastic theory, which limit the stresses in the component materials to a certain proportion of their 'failure' stresses (yield in the case of steel and curing in the case of concrete). The assumptions inherent in the elastic method are similar to those for ordinary R.C.C. In recent years the concept of Ultimate load/Limit state design philosophy have been applied to composite action and experimental evidence has shown to be safe, economical to proportion composite section (Refresher course, 2000[(27)]).

2.7.1. ELASTIC THEORY

According to Devis [(21)], stresses in the beam must no where exceed specified working stress values based on yield stress of steel and crushing stress of concrete reduced by a suitable factor of safety.

COMPLETE INTERACTION

CP:117, Part 1 does in fact permit composite beams to be designed according to the elastic theory assuming complete interaction. On the basis of complete interaction the analysis and design are approached by the method of transformed areas.

According to Devis [(21)], the equations for the section properties of composite beam depend upon the location of the elastic neutral axis. There are two cases to be considered: (a) elastic neutral axis in concrete slab and (b) elastic neutral axis in steel beam.

Taking moments of the areas of the transformed section about the interface.

If $A_s(d_s - d) > (b.d/m).\ (d/2)$ then the elastic neutral axis is in the steel beam.

If $A_s(d_s - d) < (b.d/m).\ (d/2)$ then the elastic neutral axis is in the concrete slab.

<u>Case (I) – Elastic neutral axis in concrete slab (Ref. Fig.9)</u>

Taking moments of areas of the transformed section about the top surface of the concrete slab we get:

$d_n = A_s.m/b\ [\sqrt{(2bd_s/mA_s + 1)} - 1]$

The gross moment of inertia of the composite section in steel units is then:

$I_{comp} = b/m.\ d_n^3/3 + I_s + A_s(d_s - d_n)^2$

To obtain the value of the gross moment of inertia of the section in equivalent 'concrete' units the above expression for I_{comp} should be multiplied by the modular ratio m.

The two equations (upper and lower fibres) for section moduli are then:

For concrete in compression $Z_c = m.I_{comp}/d_n$

For concrete in tension, $Z_s = I_{comp}/(D + d - d_n)$

The extreme fibre stresses in the concrete and the steel due to bending will

$f_c = M/Z_c$

$f_s = M/Z_s$

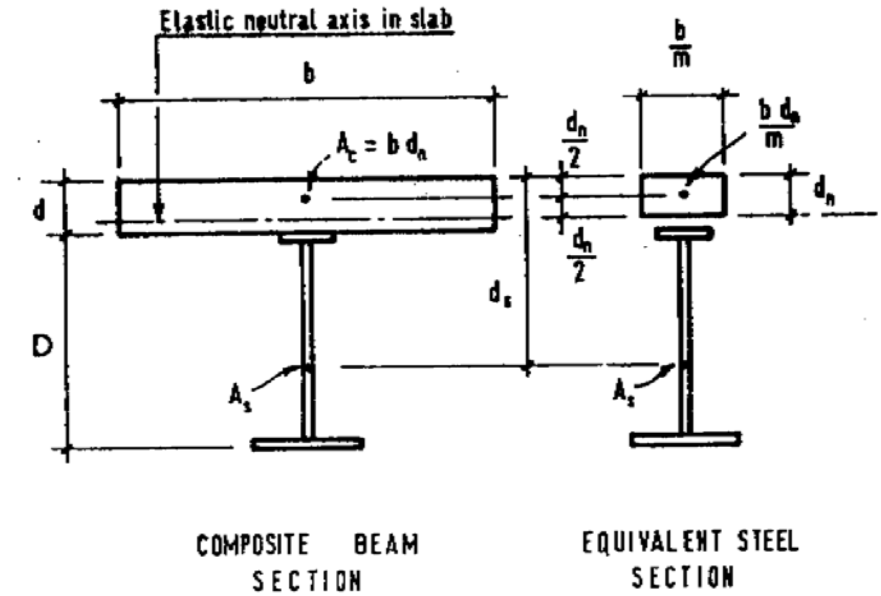


Fig. 9. Case (I) - Composite T - beam with elastic neutral axis in slab.

Case (II) – Elastic neutral axis in steel beam (Ref. Fig.10)

Depth of neutral axis, $(d_n) = (b.d/m\ .\ d/2 + A_s.\ d_s)/(b.d/m + A_s)$

Gross M.I., $(I_{comp}) = b/m\ .\ d^3/12 + b.d/m\ (dn - d/2)^2 + I_s + A_s(d_s - d_n)^2$

The equations for section moduli and extreme fibre stresses are as for Case (a) above.

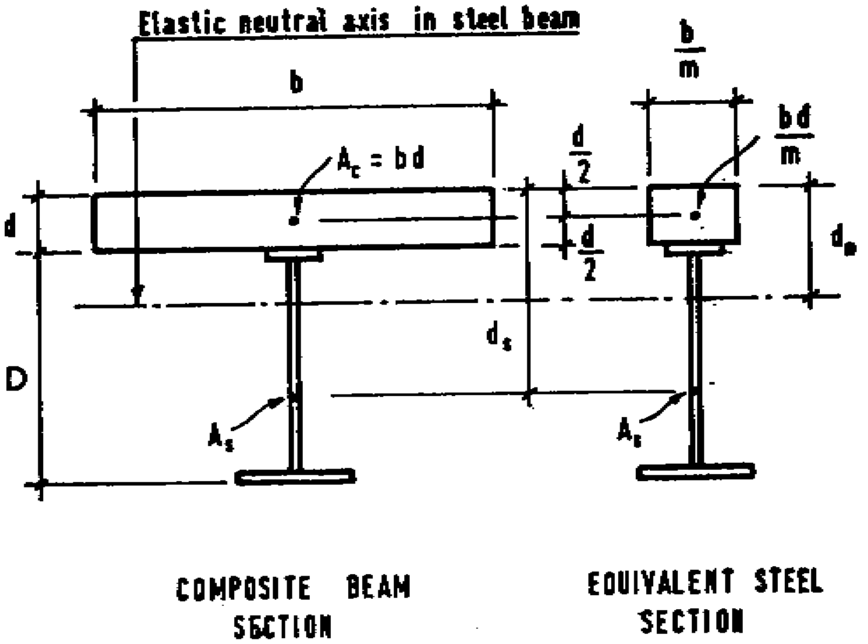


Fig.10. Case (II) - Composite T - beam with elastic neutral axis in steel beam.

INCOMPLETE INTERACTION

To take account of the loss of interaction within the elastic range a theory was developed in the early 1950s by Newmark(10),the theory assumes that:

a) The shear connection between the slab and the beam is continuous and uniform along the entire length of the beam

b) The load/slip relationship for the shear connection is linear.

c) The distribution of strains throughout the depth of the concrete and steel is linear, and

d) The beam and slab are assumed to deflect equal amounts at all points along their length at all times.

Although equations were developed for flexural stresses, horizontal shear force, horizontal slips and vertical deflections, in general they are too complex for practical design use.

DEFLECTION

In addition to the need to satisfy flexural strength criteria, it is also important, for functional reasons that the deflection of a beam in service is not excessive.

The Newmark theory, which allows for the effect of horizontal slips on vertical deflection, gives slightly greater deflections for the same conditions than the complete interaction theory. The effects of interface slip on vertical deflection are normally relatively small, and the theory of complete interaction can be applied to deflection calculations with confidence.

In service defection is given by:

$\delta = 5/384.L^{3}/E_s (W_d/I''_{comp} + W_l/I'_{comp})$

Where L =simply supported span

E_s = Young's modulus for steel beam

I''_{comp} = moment of inertia of transformed section in 'steel' units for modular ratio of 30

I'_{comp} = moment of inertia of transformed section in 'steel' units for modular ratio of 15

2.7.2. ULTIMATE LOAD THEORY

In fact the principles of ultimate load design for steel-concrete composite beams are very closely related to the ultimate load analyses applied to the separate materials of structural steelwork and reinforced concrete. (Devis [21])

Whatever the position of the plastic neutral axis under ultimate load conditions, whether it be in the slab or in the steel beam, there are three principle modes of failure:

i) Failure by flexure
ii) Failure of shear connection
iii) Failure by longitudinal splitting of the slab.

(I) FLEXURAL FAILURE

According to Devis [21], in ultimate load analysis for bending it is assumed that:

a) The tensile stress/strain curve for the steel beam has a definite yield plateau, and

b) The concrete slab below the plastic neutral axis is cracked and therefore unstressed.

In Appendix A4 of CP 117, formula are quoted for the depth d_n to the plastic neutral axis and the corresponding ultimate moment of resistance. It is necessary to consider three cases:

<u>Case (I)-Plastic neutral axis in concrete slab (Ref. Fig.11)</u>

Equating ultimate longitudinal force in steel and concrete, we get:

$d_n = 9/4.(Y_s.A_s/U_w.b) = \alpha.A_s/b$

Where $\alpha = 9Y_S/4U_W$

Then Mr= Fst X (lever arm)

Then $M_r = A_s.Y_s(d_c+d-d_n/2)$.

Alternatively:

$M_r = F_{cc}$ x (lever arm)

Then , $M_r = 4/9.(u_w.b.d_n)\ (d_c+d-d_n/2)$

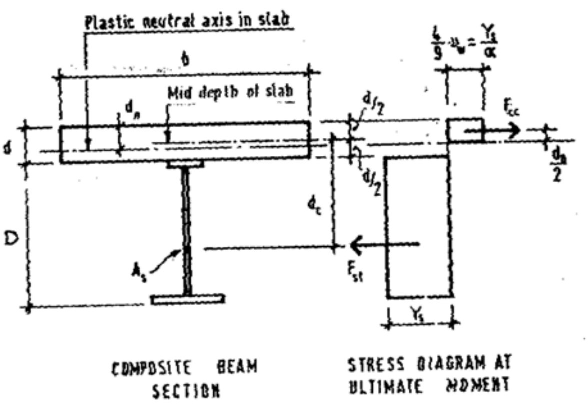


Fig. 11. Case I - Plastic neutral axis in concrete slab

Case II (i)-Plastic neutral axis in top flange of steel beam (Ref. Fig.12)

Equating the total force in concrete & equivalent tensile force in the steel , we get;

Depth of plastic neutral axis, $(d_n) = d+(\alpha.A_s\text{-}b.d)/2.\alpha.b_f$.

Moment of resistance $(M_r) = Y_s\ (A_s\ d_c\text{-}b_f(d_n\text{-}d).d_n$.

Case II (ii) -plastic neutral axis in web of steel beam. (Ref. Fig.12)

Equating the total force in concrete & equivalent tensile force in the steel, we get;

Depth of plastic neutral axis, $(d_n)= (d+t_f+\ \alpha\ (A_s\ \text{-}2.A_f)\text{-}b.d)/(2.\alpha.t_w)$

Moment of resistance, $(M_r)= Y_s\ [(A_sd_c\text{-}A_f(d+t_f)\text{-}t_w(d_n\text{-}d\text{-}t_f)(\ d_n+\ t_f)]$.

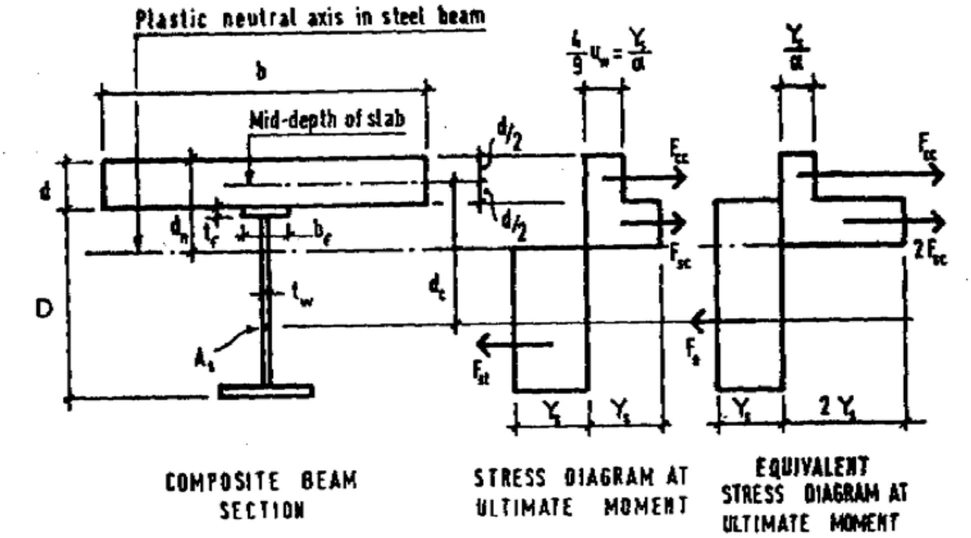


Fig. 12. Case II (i) & (ii) - Plastic neutral axis in steel beam.

(II) SHEAR CONNECTOR FAILURE

Case (I) Plastic neutral axis in slab (Ref. Fig.13)

If P_c is the design value of one shear connector then the number of shear connectors required between the point of zero moment (support) and maximum moment (load - point) is given by:

$N=F_{cc}/P_c= F_{st}/P_c= A_s.Y_s/P_c.$

<u>Case (II) Plastic neutral axis in steel beam</u>

The total numbers of shear connectors each side of the section of ultimate moment is:

$N= F_{cc}/P_c=4/9. (U_w .b.d).$

Once again this applies whether the beam is carrying a single point load or a uniformly distributed load.

For combination of distributed and heavy concentrated loads.

The number of connectors N should be distributed between the section of maximum and zero moment

$N=n_1+n_2+\ldots\ldots\ldots$

Then $n_1 = N [(a_1)/(a_1+a_2+\ldots\ldots\ldots)]$.

$n_2 =[N (a_2)/(a_1+a_2+\ldots\ldots\ldots)]$.

Where n_1 = number of connection in length 1_1

n_2 = number of connection in length1_2

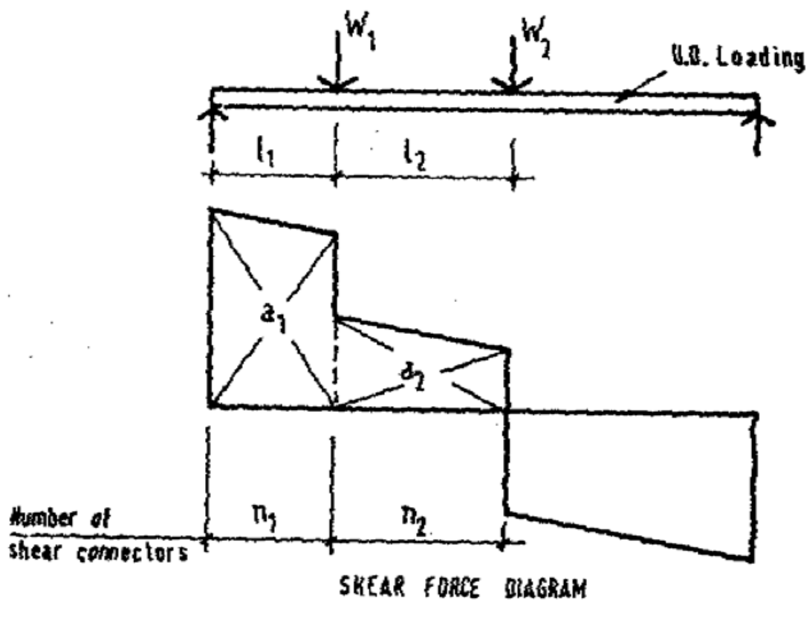


Fig. 13. Simply supported beam with heavy concentrated loads

(III) LONGITUDINAL SPLITTING OF SLAB

According to Devis [21], if the full theoretical moment capacity of a composite beam is to be realized, then not only must there be adequate shear connection, but in addition there must also be adequate transverse reinforcement in the slab in order to resist longitudinal shear and to prevent longitudinal splitting of the concrete. Inadequate transverse slab reinforcement may permit a crack to develop in the concrete slab along the line of the shear connectors, and this could lead to premature failure of the beam prior to the development of the ultimate moment of resistance of the composite section.

<u>Concrete parameters</u>

Length of shear surface at shear connectors $=L_s$

Concrete cube strength $=U_w$

<u>Reinforcement parameters</u>

Area per unit length of beam of transverse reinforcement

In bottom of slab $=A_t$

Yield stress of reinforcement $= F_y$

The total resistance of the slab to splitting may be expressed as the sum of the concrete and reinforcement contributions per unit length of beam as follows:

$\beta.L_s+\alpha.A_t$

Where $\beta = F\ (U_w)$ and $\alpha = \varnothing(f_y)$

If the number of connectors a cross section is N_c and the ultimate design load (or force) per connector is P_c then the shear force per unit length of beam is:

$Q = N_c.P_c/P$.

Where p is the longitudinal pitch of the shear connectors. So that if the connectors are to achieve their design load prior to longitudinal splitting, then:

$\beta.\ L_s + \gamma.\ A_t > N_e.\ P_c/P.$

CHAPTER-3

CHOICE OF TEST SPECIMEN FOR SHEAR CONNECTOR

According to C. Devis[21],the push-out specimen is a convenient and economic method of ascertaining and comparing the load/slip characteristics and ultimate load capacities of shear connectors. Push-out specimens have been used, and indeed still are being used; to determine both static and fatigue properties of shear connectors.

As per Colin Davis[4] ,the 'standard' specimen recommended by the CP:117 gives the highest ultimate load among all the stud patterns, as well as the studs of the 'standard' specimens exhibit superior performance throughout their loading.

The standard push-out specimen recommended by CP 117 consists of a 10″× 5 3/4″ UB × 29 Ib steel section (now 254 × 146 UB × 43) (ISMB 250) with two insitu reinforced concrete slabs as illustrated in Figure 3.04.

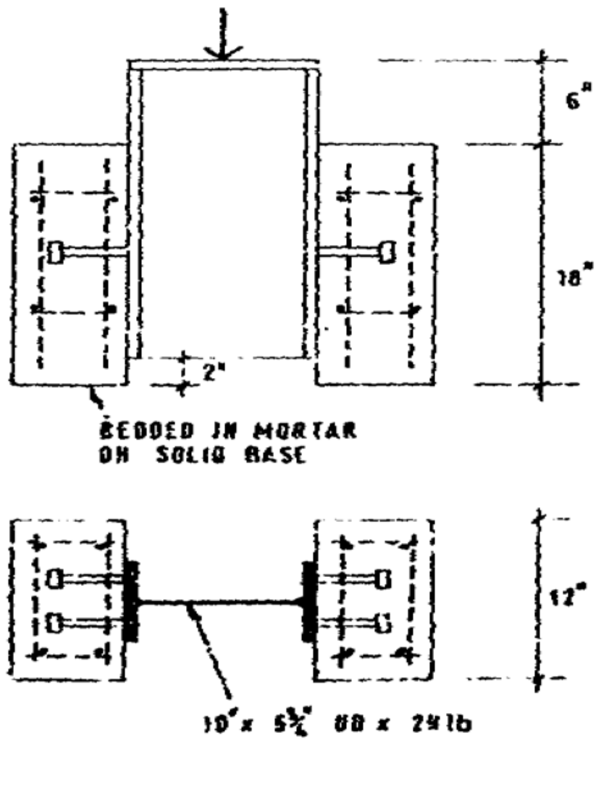


Fig. 14. Test piece for shear connector.

When casting the concrete blocks, the steel-concrete bond at the interfaces should be prevented by greasing the steel flanges over the area of contact, care being taken to ensure that fillets of grease are not deposited around the root welds of the connectors as this would adversely affect the performance of the connectors.

According to CP:117 , the standard push-out specimen illustrates two-welded stud connectors arranged as a pair across each flange. As per Colin Davis(4), the 'standard' push-out specimen which has two studs per flange arranged at right angles to the direction of the load give an ultimate load value about 25% greater, if the studs, were arranged in the direction of the load. When there are more than two studs per flange arranged longitudinally, the ultimate strength per stud is more sensitive to variations in spacing than when there are only two studs per flange.

Push-out tests may be carried out in standard laboratory testing machines fitted with compression testing facilities. When setting up a push-out specimen for testing, care should be taken to ensure that the compression load is applied concentrically; this may be done by bedding the specimen down on the lower platen of the machine using a neat high-alumina cement paste or mortar (say 4 mm to 6 mm thick) and applying the load centrally to the top cover plate through a steel ball.

If care is taken to ensure concentric loading, then the ultimate capacity of the connector under test may be taken simply as the maximum load on the push-out specimen divided by the number of connectors applied. Since a rapid rate of testing is likely to give high results, CP: 117 recommends that a push-out test should take not less than ten minutes.

CP:117 uses the results of standard push-out tests in order to arrive at the recommended ultimate design values quoted in the code. If on a particular project shear connectors are being used which are not of the size or kind listed in the Code, then it is recommended that the appropriate ultimate design value should be taken as 80 per cent of the lowest ultimate capacity of not less than three push-out tests. If shear connectors are to be used in slabs having haunches then the push-out specimen should include the proposed haunch and reinforcement.

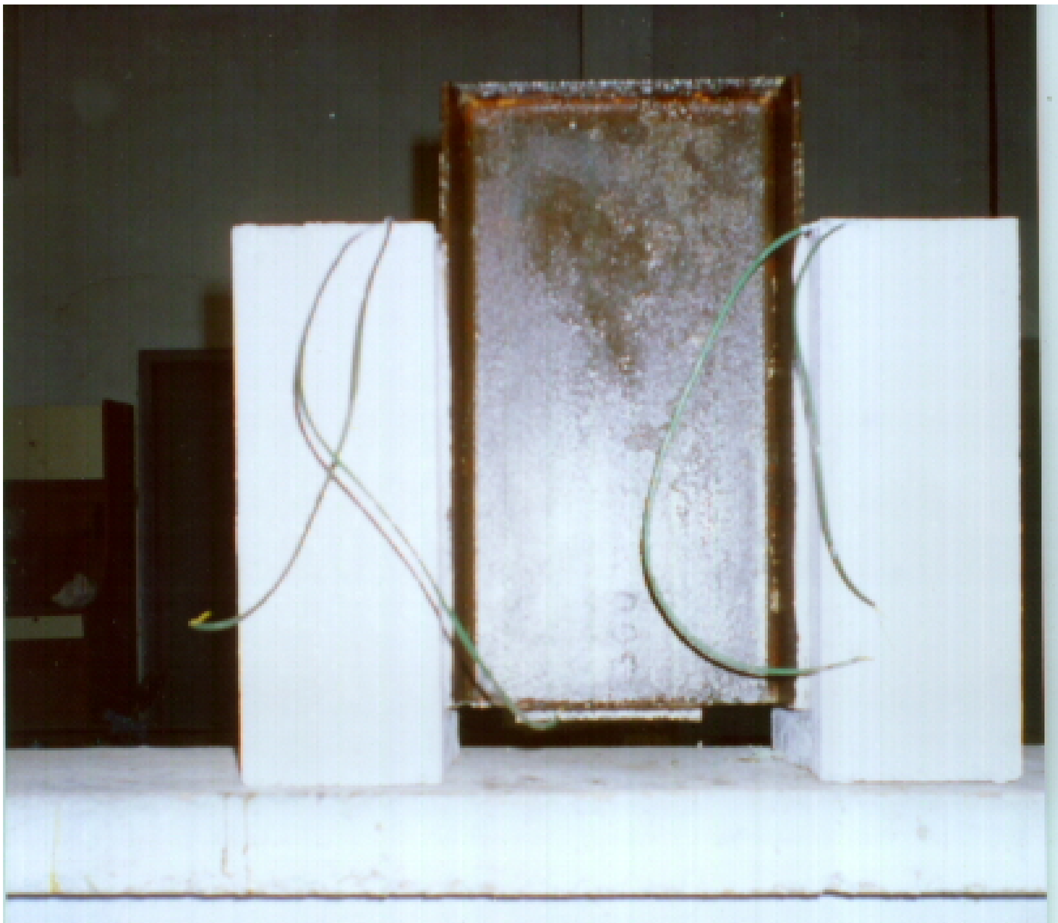


Fig.15. Standard steel-concrete composite push-out specimen.

CHAPTER-4

EXPERIMENTAL STUDY

GENERAL

This chapter presents details of materials used in the experimental work on steel-concrete composite construction along with their properties and the mix proportion of the SCC used for composite specimens. It covers the procedure of fabrication of composite push-out specimens in detail. It also covers all the tests conducted on fresh concrete during casting of the specimen and on hardened concrete.

4.1. CONSTITUENT MATERIALS

The various constituent materials used in the experimental work were infabricated steel beams, different types of shear connectors, cement, fine aggregate, coarse aggregate, fly ash, chemical admixtures and water.

4.1.1. INFABRICATED STEEL GIRDER

Built up Mild Steel girder (ISMB300) is used as the structural steel element of steel-concrete composite construction, which is symmetrical about both the principal axis. Throughout this thesis work, the properties of steel sections have been taken from steel table in SI units.

PROPERTIES OF (ISMB300)

Yield Stress (f_y)=250 N/mm^2

Young's Modulus (E)=2x10^5 N/mm^2

Sectional Area (a)=5626mm^2

Depth of Section (h)= 300mm

Width of Flange (b_f)=140mm

Thickness of Flange (t_f)=12.4mm

Thickness of Web (t_w)=7.5mm

I_{xx}=8603.6x10^{4}mm^4

I_{yy}=453.9x10^{4}mm^4

4.1.2. SHEAR CONNECTORS

The shear connectors are required to transmit horizontal shear between the beam and slab, and also to prevent the slab lifting away from the beam. It is one of the most important elements in composite construction.There are in general three types of shear connectors (Rigid, Flexible, Bond or Anchorage Connector). Each type again contains various types of shear connectors.

To evaluate the behaviour of shear connector, this thesis work emphasized on the most commonly used shear connectors.The connectors selected for test are "**Bar Connector, Headed Stud, Stud without Head & Channel Connector"** attached by 'Fillet weld' running all around the connector..Sectional properties of the connectors are given in Table.1.

Table.1: Sectional properties of shear connectors

Type of Connector	Material	C/S Area (mm^2)	Dia. (mm)	Length Along the flange (mm)	Height (mm)	b_f (mm)	t_f (mm)	t_w (mm)
[RIGID CONNECTOR]								
Bar Connector								
Bar (42.5x42.5) mm	Mild Steel	1806.25	---	75	42.5	---	---	---
Loop (20mmΦ)	Tor Steel	314.16	20	---	100	---	---	---
[FLEXIBLE CONNECTOR]								
Channel (ISMC 100)	Mild Steel	1170	---	75	100	50	75	4.7
Headed Stud	Mild Steel	314.16	20 mm (ϕ of shank)	20 mm	100	---	---	---
Stud Without Head	Tor Steel	314.16	20 mm	20 mm	100	---	---	---

4.1.3. SELF COMPACTING CONCRETE

The self compacted concrete (SCC) was first proposed by Okamura[17] in 1986 in Japan and developed in 1988 by using materials that are available in the market. It can be defined as a new category of high performance concrete that has excellent deformability and high resistance to segregation in fresh state. It can be placed and compacted under its self-weight without applying vibration.

SCC is independent of the quality of formwork and assures complete filling of complicated formwork, congested reinforced structural elements and hard to reach areas by means of its self-weight.

Following were the main reasons for the use of SCC in this dissertation work:

1. It shortens the construction period,
2. It ensures compaction in the complicated formwork where vibrating compaction is difficult.

The constituent materials used for self-compacting concrete have given below along with their properties.

4.1.3.1. CEMENT

The cement used in this experiment was of L&T 53 grade and the various properties of the cement are shown in Table.2. The main function of cement is to act as a binding material in between the aggregate and induce workability in fresh state.

The setting implies solidification of the plastic cement paste. The beginning of solidification was called 'Initial Set' that marks the point in time when the paste has become unworkable where as the time take to completely solidify, marks the 'Final Set'. Initial setting time of the cement was 180 min that is greater than permitted value of 30 minutes, may be due to high fineness of cement and Final setting times was 340 min. According to the code this value should not be more than 600 minute so, the test results are within the limit.

The term stiffening was defined as the loss of consistency by the plastic cement paste and was associated with the slump loss phenomenon in concrete. The consistency of the cement was 32%, which is within the limits.

The selection of the strength will depend upon the overall requirements for the concrete, such as strength and durability etc. The cement used in this experiment, was confirmed to the standards of IS: 8112. The above parameters were found by using Vicat Apparatus.

The typical content of cement was 300-450 kg/m^3, but as per IS: 456-2000, cement content more than 450 kg/m^3 may increase the risk of shrinkage and less than 300 kg/m^3 may only suitable with the inclusion of other filler materials such as fly ash, stone powder etc. but, this may not come under structural concrete. The cement used in both the mixes was within the above mentioned limits so no shrinkage was observed.

Table. 2 : Properties of cement

Test Parameter	**Value**
Specific Gravity	3.12
Blaine Fineness (m^2/kg)	309
Normal Consistency (%)	32
Initial Setting Time (min)	180
Final Setting Time (min)	340
Compressive Strength (MPa)	
7 days	41
28 days	48

4.1.3.2. FINE AGGREGATE

After cement, Sand is the second most important material that influences the workability of fresh concrete. Since the success or failure of concrete depends very narrowly, so it is important to study the sand, like the percentage of finer material, fineness modulus etc. The sand used in this experiment was crushed stone and of siliceous type. It was also found from literature study that the quantity of sand required as percentage of total aggregate, to produce satisfactory SCC with less quantity of superplasticizer is around 50%. The sieve analysis of fine aggregate has given in Table.3 and properties are given in Table.4.

The sand absorption value was 1.6%. The fineness modulus of the given sand was 2.2, which is well within the limits given by code 2 to 2.4, it should be kept in mind that sand having fineness modulus more than 3.2 should not be suitable for making satisfactory concrete. As per Abrams any sieve analysis curve of aggregate that will give the same fineness modulus will require the same quantity water to produce a mix of the same plasticity and gives concrete of the same strength, so long as it is not too coarse. The sand used in this experiment was comes under zone-III as per the IS: 383-1970, which means that this sand was finer than the standard sand.

Table. 3 : Sieve analysis of fine aggregate

Sieve Size (mm)	**Passing (%)**	**(%) Passing zones as per IS: 383-1970**		
		1	**2**	**3**
10	100	100	100	100
4.75	98.5	90-100	90-100	90-100
2.36	95.5	60-95	75-100	85-100
1.18	87.5	30-70	55-90	75-100
0.600	68	15-34	35-59	60-79
0.300	7	5-20	8-30	12-40
0.150	3.75	0-10	0-10	0-10

Table. 4 : Properties of fine aggregate

Properties	Values	Permissible Range
Fineness Modulus	2.2	2-2.4
Specific Gravity	2.6	-
Water Absorption (%)	1.6	-
Moisture Content (%)	1	-

4.1.3.3. COARSE AGGREGATE

Regarding the characteristics of different types of aggregates the angular aggregate tend to improve the strength because of better interlocking of angular particles, whilst rounded aggregates improve the flow because of lower internal friction. Gap graded aggregates are frequently better than those continuously graded which might experience greater internal friction and give reduced flow.

It is well-established fact that as the grade of concrete increases the maximum size of the aggregate gets reduces, since more the size of the aggregate more complex will be the transition zone. Considering the above aspect the maximum size of the aggregate was kept as 20 mm and like sand, these were also siliceous and crushed type. The sieve analysis data of both 20 mm and 10 mm aggregate is shown in (Table), the amount of sand contained in 10 mm aggregate was around 18%. The absorption of the given aggregate was 0.3%. The fineness modulus of the coarse aggregate was 6.7.

Table.5 : Sieve analysis of coarse aggregate

Sieve Size (mm)	Percent Passing (20 mm Nominal Size)	Percent Passing (12.5 mm Nominal Size)
40	100	100
20	100	100
12.5	N.A	96
10	10	84
4.75	3.5	18
2.36	N.A	4.5

Table.6 : Properties of coarse aggregate

Parameters	Values
Specific Gravity	2.69
Bulk Density (kg/m^3)	1610
Water Absorption (%)	0.3
Fineness Modulus	6.7

4.1.3.4. POZZOLANIC MATERIAL

The pozzolanic material used in this experiment was fly ash which was a fine inorganic material brought from the Dadri power plant.

The main purpose of the fly ash in SCC was to increase the stability of the mix in green state or it can also be said that for a given quantity, as the percentage of fly ash increases the quantity of the VMA reduces, to produce satisfactory SCC mix. The main mechanism involved is, that it attaches the ingredients with certain force, which does not allow the ingredients to easily separate.

This was collected from top portion of the electrostatic precipitator. The fly ash was classified as per ASTM as Class-F. The percentage of material passing through 75 μm sieve was 92% and the percentage of material passing through 25 μm sieve was around 86% which means 8% of fly ash particle size was similar to sand and 6% of fly ash was similar to size of silt and 86% of the fly ash particles were similar to size of clay particles.

4.1.3.5. SUPERPLASTICIZER

The superplasticizer used in this experiment was GLENIUM-51, which was based on a unique carboxylic ether polymer with long lateral chains that will generate a steric hindrance, which stabilizes the cement particle capacity to separate and disperse. It can reduce water demand up to 30-45% that is why it is also known as High Range Water Reduce (HRWR). The sulphonic groups of the polymer chains increase the negative charge on the surface of the cement particle and dispersion of the cement occurs by electrostatic repulsion.

The following are the main functions of the superplasticizer:

- Dispersion of Cement
- Reduction with by products of hydrating cements
- Alternating the normal trend of hydration particularly C_3 S

The followings are the some of the benefits associated with the usage of HRWR:

- Concrete with the lowest water cement ratio without segregation or bleeding
- Allows reduction in curing cycles
- Flowable Possibility of elimination of steam curing
- Less vibration required even in case of congested reinforcement locations
- Improve the concrete surface finish
- Compared to traditional superplasticizers the additional of Glenium improves the physical properties and thus the durability of concrete

<u>Glenium-51 increases:</u>

- Modulus of elasticity
- Early and ultimate compressive strength
- Early and ultimate flexural and tensile strength
- Adhesion to reinforcement and prestressed steel
- Resistance to aggressive atmospheric condition
- Resistance to carbonation and chloride ion attack of concrete

<u>Gelinium-51 decreases:</u>

- Risk of shrinkage
- Effects of creep

It should be kept in mind that always the superplasticizer should be added a little later after the addition of water to the mix. This process will fully utilize the action of the superplasticizer but, too much dosage of this could leads to delayed setting of cement, which in turn reflects on the removal of form work.

4.1.3.6. VISCOSITY MODIFYING AGENT (VMA)

It has found that, the addition of VMA reduces the variation in self-compactability when water content fluctuates. This admixture helps to provide very good homogeneity and reduces the tendency to segregation. The VMA used in this experiment was *Pozzolith* a product of Degussa. The method of using the viscosity modifying agent can increase the deformation of fresh concrete without segregation and then lead to achieve high optimum selfcompactability, there by widening the range of variation over which good selfcompactability was ensured. The use of VMA imparts cohesiveness to the mix and does not allow the mix to bleed, there by it gives stability to the mix, which is an important parameter of SCC mix particularly when it is used in heavily reinforced section.

The use of VMA reduces the resistance to segregation, which means that in increased amount of superplasticizer further increases the deformation of the fresh concrete without segregation. Such a synergic action on these performance leads to higher self-compactibility, which widens the range of variability in which high self-compactibility was ensured.

The VMA can be added in variety of ways but to get optimum quantity it should be added only after addition of superplasticizer depending upon the stability of fresh concrete. Prier to addition with the mix it should be mixed with water and uniformly applied through out the mix.

4.2. MIX DESIGN

This experiment contained SCC for the preparation of steel-concrete composite specimen. The mix details of SCC are shown in Table. The total quantity of the powder in SCC was 540kg/m^3. The concrete used in this experiment was of structural grade whose characteristic compressive strength is 40 MPa. No trial mixes were casted because the mix details of the M40 were available, so they were straight away taken. High deformability and high resistance to segregation are important properties of SCC. Since these two properties are opposite in nature, they tend to be sensitive to quality fluctuations of materials compacted to conventional concrete. The mix proportioning is based upon creating a high-degree of flowability while maintaining a low water-to-cementitious materials ratio, W/Cm (<0.40). This is achieved using high-range water reducing (HRWR) admixture combined with stabilizing agents to ensure homogeneity of the mixture. For better structural performance and durability, it is imperative to proportion SCC with high stability defined by its resistance to blockage upon spreading through closely spaced obstacles, bleeding and segregation. So concrete without proper stability due to insufficient cohesiveness between cement paste and aggregate could lead to *local separation* of aggregate upon spreading through obstacles and can exhibit anisotropy in the direction of casting that weakens the cement paste aggregate and bond.

Okamura(17) and Ozawa suggested the simplest method of mixture proportioning for self-compacting concrete. In this method:

- Coarse aggregate content is fixed at 50% of the solid volume.
- Fine aggregate is placed at 40% of the mortar fraction volume.
- Water-to-cementitious materials ratio in volume is assured as 0.9 to 1.0 depending on properties of the cement.

Superplasticizer dosage and the final W/Cm are determined so as to ensure the self-compactability. In the event that satisfactory performance can not be obtained, then consideration should be given to fundamental redesign of the mix. Depending on the apparent problem, the following course of action might be appropriate.

- The use of additional or different types filler
- Modify proportions of the sand or the coarse aggregate
- The use of viscosity modifying agent, if not already included in the mix
- Adjust the dosage of the superplasticizer and/or the viscosity modifying agent

- The use of alternative types superplasticizer, which may be more compactable with local materials.

- Different dosage rates of admixture to modify the water content, and hence the water/powder ratio.

The sequence of efficiently mix design was shown below

- Design of desired air content
- Determination of coarse aggregate volume
- Determination of sand content
- Design of paste composition
- Determination of optimum water to powder ratio and superplasticizer dosage
- Finally the concrete properties are assessed by standard tests.

Table.7 : Details of mix proportion of SCC

Particulars	Quantity
Cement (Kg/m^3)	410
Sand (Kg/m^3)	825
Coarse Aggregate (Kg/m^3)	825
Water (lt/m^3)	190
Fly Ash (Kg/m^3)	130
HRWR (lt/m^3)	3.7
VMA (lt/m^3)	1.15
W/Cm	0.35

4.3. FABRCATION OF PUSH-OUT SPECIMEN

The basic specimen used in most of the Push-Out Tests is illustrated in Figure.15 and differs from the specimen recommended in CP 117 for Static Tests mainly by having ISMB300 instead of ISMB250. The entire laboratory work involved in the fabrication of the specimen has been elaborated briefly below.

4.3.1. FABRICATION OF MOULD

Although idea of composite construction is not new, still it has been gaining popularity in India in recent years only and intensive research programs are going on in different Research Institutions. So the standard mould for the standard push-out specimen as per CP: 117 is not available in the market. Due to the non availability of standard mould the required mould was prepared in the Workshop of CRRI, New Delhi.

The standard mould (150x450x300mm) consisting of five plates (One base plate & four side plates) has fabricated by using 20mm thick plywood as shown in Fig.16.A portion having dimension of the width of the I-section(140mm) and the length (400mm) has removed from one side plate out of the four side plates, to support the flange of the ISMB300 as well as to keep the inside surface of the mould and outside surface of the flange of the I-section in one plane. The top portion was open to facilitate the pouring of concrete.To prepare a standard Push-Out specimen two moulds are required that for the left and right concrete blocks. To prepare four specimens eight numbers of moulds fabricated.

Fig. 16. Standard mould for composite push-out specimen.

4.3.2. PREPARATIN OF STEEL WORK

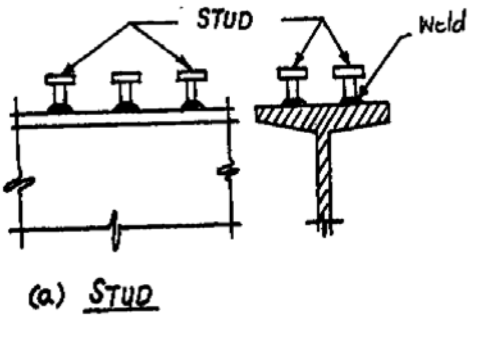
The area of the steel work to which shear connectors to be welded, cleaned as possible and freed from mill-scale, heavy rust, dirt, galvanizing, paint etc. by using wire-brush, sand paper and the chemical such as benzene. Then light grinding also done with a carborundum wheel in a hand-held power tool.

4.3.3. WELDING OF SHEAR CONNECTOR

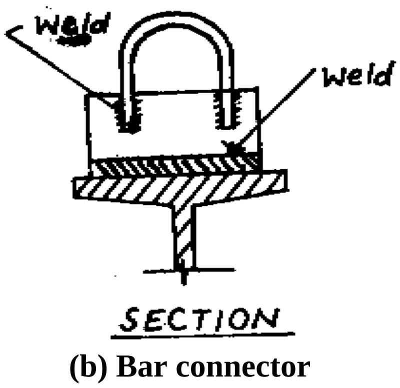
Concave fillet welds are favoured in general because it offer a smoother path for the flow of stresses and most suitable under alternating stresses but, experience has shown that single pass fillet welds of this shape has a greater tendency to crack upon cooling, which outweighs the effect of improved stress distribution.

From the various research works it has already been proved that the convex fillet weld actually stronger and gives better performance result under static load condition. As the above experimental work mainly concentrating on the static load instead of cyclic loads, it has been concluded to use the convex fillet weld.

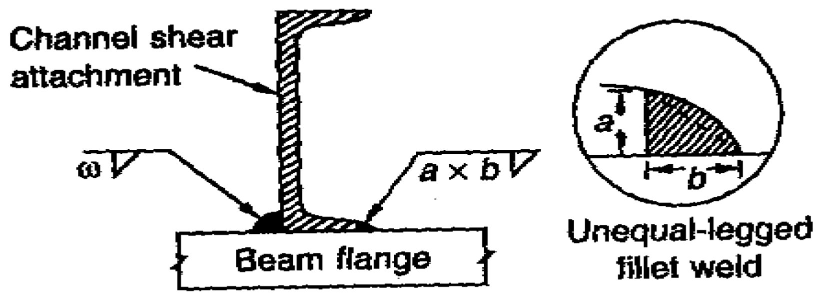
All the shear connectors (Headed Stud, Std without Head, Channel, Bar) were welded to the flanges of I-section by 6mm convex fillet welds running all around the connector at the workshop of CRRI, New Delhi, as shown in Fig.17.



(a) Stud connector



(b) Bar connector



(c) Channel connector

Fig. 17. Welding of shear connectors.

4.3.4. INSTALLATION OF STRAINGAUGES

As the strain is one of the most important aspects in case of steel so to measure the variation of strain in different shear connectors used due to static loading, the electrical resistance strain gauges were installed on the surface of the shear connectors opposite to the direction of the load.

PROPERTIES OF STRAIN GAUGES

Overall Size=7mmx5mm

Gauge Length=3mm

Resistance=120±0.2Ω

Prior to the installation of the strain gauges the surface of the shear connector at which the strain gauge is to be install, grinded by using a carborundum wheel in a hand-held power tool to make the surface of the shear connectors plane and uniform.

Efficiency of the strain gauges are tested by using the Multimeter and after getting the satisfactory result, it installed on the grinded surface of the shear connector by using Elphy.

To protect the surface of strain gauge from the wet concrete, waterproofing has done by using insulation tap and waterproofing has done only bon that portion where the strain gauges installed. All the strain gauges installed were at a distance of 1cm from the surface of the weld.

4.3.5. GREESNG OF THE FLANGES:

Satisfying the guidelines given by the CP 117, to prevent the steel-concrete bond during casting of the composite specimen, greased properly as shown in Fig. 18. over the area of contact of the flanges of the steel section. Special care has been taken to ensure that fillets of grease were not deposited around the root welds of the connectors as this adversely affects the performance of the connectors.

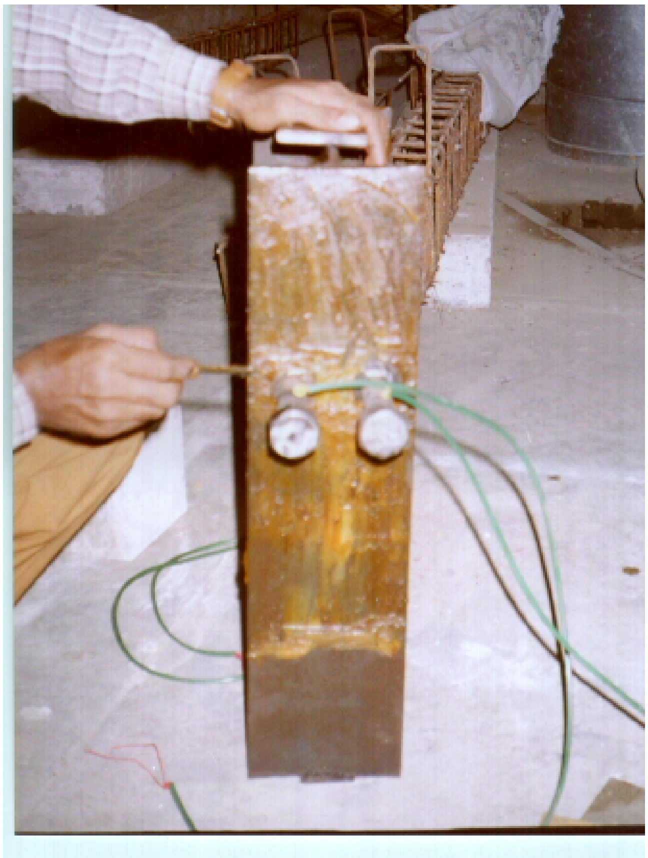


Fig.18. Greasing of flanges of steel beam.

4.3.6. CASTING OF SPECIMEN

The slabs were cast horizontally as would be the case in a beam, although this necessitated casting the two slabs simultaneously.

A base prepared for each of the specimen to keep the ISMB 300 at the level of requirement. Then the 'Standard' moulds having the nominal reinforcement attached with both the flanges of the I-section

To prevent the flow of concrete through the gap between the edges of the mould and the flanges of the I-section, plaster of paris applied on it. After hardening the plaster of paris oiling has been done to facilitate the removal of formwork after hardening of concrete.

Fig.19. Arrangement of the specimen prior to casting.

After the arrangement of the specimen some external supports were provided surrounding the specimen to prevent the movement of I-section and also to resist the load of wet concrete on the moulds.

As it is quite tedious to vibrate the specimen on the vibrating table, self-compacting concrete has used which is meant for such types of complicated work.

After performing all the tests on the fresh concrete, fresh mix just poured in the moulds through the opening part from 300 mm height. Pouring has done simultaneously in both the mould instead of one by one.

All the specimens casted during the thesis work were of the same dimensions an the concrete slabs were reinforced identifically as shown in Fig.20. The specimens were kept in damping place and left for 24 hours.

Fig. 20. Casting of steel concrete composite specimen

6 numbers of cylinders, 3 numbers of cubes, 3 numbers of prisms also casted with each of the specimen, (on the some day of the casting of the specimen using the same concrete) to find out modulus of elasticity, tensile strength, compressive strength, and bending stress of the concrete used for that specimen. In total 24 numbers of cylinders, 12 numbers of cubes and 12 numbers of prisms casted for all the four specimens.

4.3.7. CURING OF SPECIMEN

The rate of hydration, the rate of development of strength, reduces with time, so it is not worthwhile to cure for the full period of 28 days. IS: 456-2000 stipulates a minimum of 7-day moist curing, while IS: 7861 (part I)-1975 stipulates minimum of 10 days under hot weather condition. Also it has been seen that to develop the design strength, it is better to cure the concrete up to 28 days. So all the specimens were cured up to 28 days.

All the specimens were cured by using the wet hessian, just after the removal of the mould plates till the end of the curing period.All the cubes, cylinder, prisms cured in the curing tank for a period of 28 days same as that of the specimen.

4.4.TESTS ON THE FRESH CONCRETE DURING CASTING OF SPECIMEN

GENERAL

The essential characteristic of self-consolidating concrete lies in their behavior at fresh state. The fresh SCC should possess three essential properties that is , filling ability, resistance to segregation, and passing ability. A number of self-compacting test methods such as slump/flow (slump test), U-flow test, V-flow time, L-box test, etc, were in use for the evaluation of self-compacting properties of the concrete. Self-compacting concrete test methods have two main purposes. One is to judge whether the concrete is self-compactable or not and the other is to evaluate deformability or viscosity for estimating proper mixture proportionality if the concrete does not have sufficient self- compatibility. The most commonly used methods for this purpose are discussed in brief in the following sections.

4.4.1. SLUMP FLOW TEST

Slump flow test is the simplest and most commonly adopted test method for evaluating self-compactability quality of self-compacting concrete. An ordinary Abram's slump cone is filled with concrete without any tampering. The cone is lifted and the diameter of the concrete after the flow supported was measured, which is shown in Fig.21 was taken as the slump flow.

Self-compacting concrete was characterized by a slump flow of 660 to 720 mm(26 to 28 inches). Measurements of slump indicate the flowability of self-compacting concrete and determine the consistency and cohesiveness of the concrete. Sumps flow test judge the capability of concrete to deform under its own weight against the friction on the surface on the base plate with no other external resistance present. According to Nagataki[18] and Fujiwara[18] a slump/flow ranging from 500 to 700(20 to 28 inches) is considered as proper slumps required for a concrete to qualify for self-compacting concrete. At more than 700 mm, the concrete might segregate and at less than 500 mm the concrete is considered to have insufficient flow to pass thorough highly congested reinforcement. According to Bartos the slump flow test can give an indication of filling ability and susceptibility of segregation of the self-compacting concrete. The whole test was conducted within 5 minuets. The passing ability of concrete was not indicated by this test. Flowing time from the initial diameter of 200 mm to 500 mm i. e. T_{50} is one time used for a secondary indication of flow. A time of 3-7 seconds is acceptable for civil engineering applications and 2-5 seconds for housing applications. However, this test was not sensitive enough to distinguish between self-compacting concrete mixtures and superplasticized concrete.Slump flow test results are tabulate in (table) below

Open time: The time during which SCC maintain its desired rheological properties is very important to obtain good result in concrete placing. The time can suitably adjusted by choosing the right type of superplsticizer. Different admixtures have different effect on open time; they can be used according to the type of cement and the time of transport and placing of SCC.

Table. 8 : Results of slump flow test

Types of Shear Connectors used for the specimen	**Result of slump Flow Test**			**Remarks**
	T_{50} (sec)	**T_f (sec)**	**d (mm)**	**(Condition of fresh concrete)**
Headed stud	4.04	28.2	681.0	Self compactable and thixotropic
Stud without head	4.0	27.6	682.5	Self compactable and thixotropic
Channel Connector	4.01	28.0	680.0	Self compactable and thixotropic
Base Connector	4.06	29.2	679.5	Self compactable and thixotropic

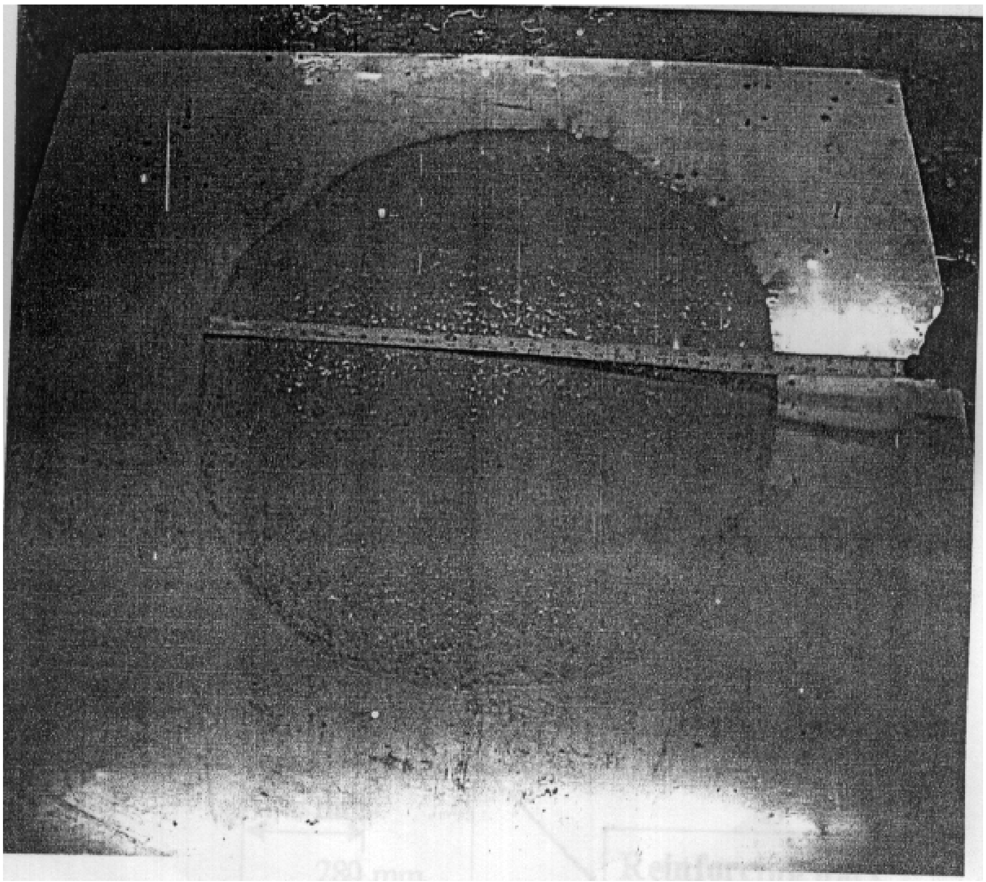


Fig. 21. Slump flow of SCC

4.4.2. U-FLOW TEST

This test examines the behavior of the concrete in a simulated field conditions. It is the most widely adopted test method for characterization of self-compacting concrete. It provides a good direct assessment of filling ability. This test is also known as 'box - shaped'. This text simulates the flow of concrete through a volume containing reinforcing steel. This test was considered more appropriate for characterizing self-compactability of concrete. In this text, the degree of compactability can be indicated by the height that the concrete reaches after flowing through an obstacle was shown in Fig.22.

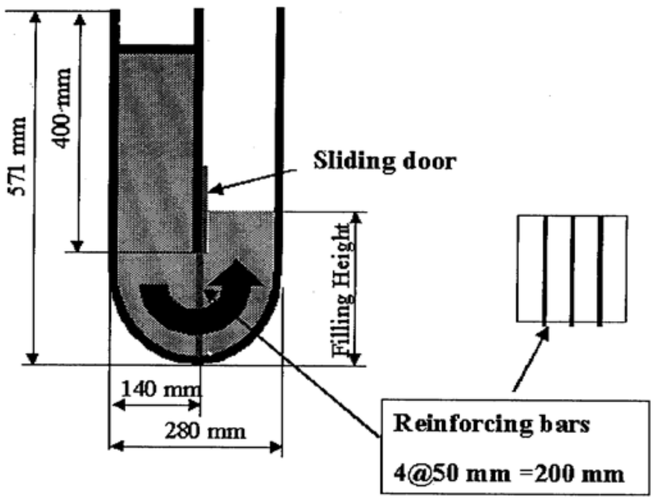


Fig. 22. U flow test

This test was performed by first completely filled the left chamber of the U - Flow device, while the sliding doors between the two chambers is closed. The door is then opened and the concrete flows past the rebars into the right chamber. The whole test was conducted within 5 minutes. Self-compacting concrete for use in highly congested areas should flow to about the same height in the two chambers. If the filing height is at least 70% of the maximum height possible, then the concrete is considered self - compacting. The selection of this percentage was arbitrary and higher value might be considered more conservative. In the U - flow device, the maximum height was 285.5 mm, half of 571 mm i.e, the total height. Therefore, a concrete with a final height of more than 200 mm was considered self - compacting concrete. This test measures filling, passing, and segregation properties of self - compacting concrete. U flow test results are tabulated in Table.9 below.

Table. 9 : Results of U flow test

Shear Connectors used for the specimen	(Results of U flow test) Final height (mm)	Remarks (Condition of the fresh concrete)
Headed steel	271	Self compactable and thixotropic
Stud without head	266	Self compactable and thixotropic
Channel Connector	268	Self compactable and thixotropic
Base Connector	273	Self compactable and thixotropic

4.4.3. L-BOX TEST

The L - box test method uses a test apparatus consisting of a vertical section and a horizontal section, which shown in Fig. 23.

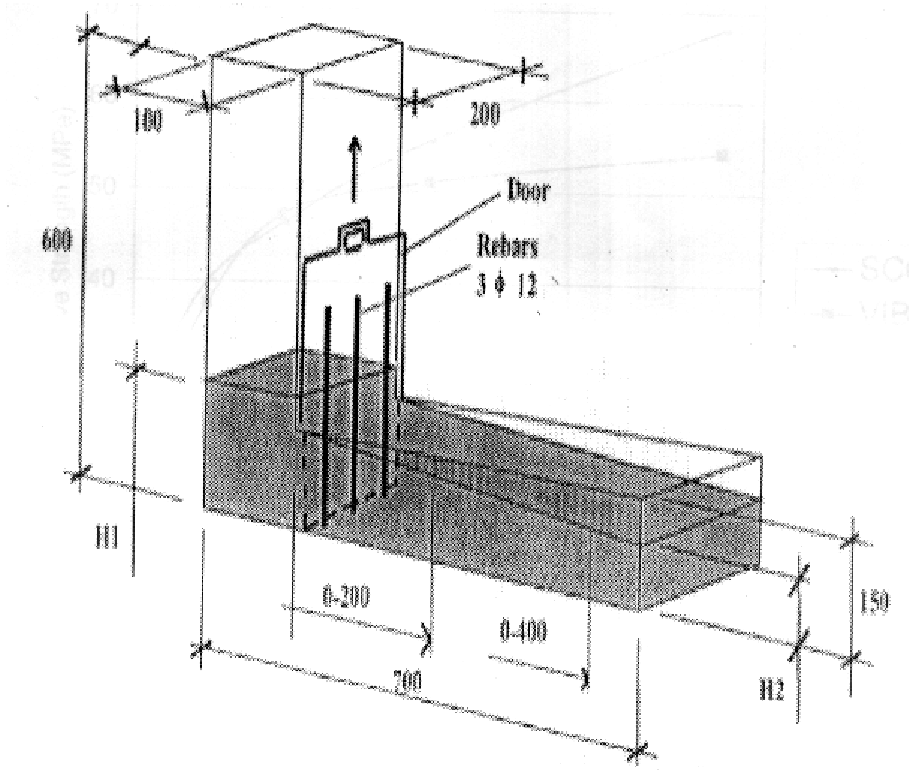


Fig. 23. L-box test

Reinforcing bars were placed at the intersection of the two areas of the apparatus. The vertical part of the box was filled with 12.7 liters of concrete and left to rest for one minute in order to allow any segregation to occur. The gap between the reinforcing bars is generally kept 55 mm for 20-mm coarse aggregate. The time taken by the concrete to flow a distances of 200mm (T_{20}) and 400mm, (T_{40}) in the horizontal section of apparatus, after the opening of the gate form the vertical section, was measured. The whole test was conducted within 5 minutes. These are an indication of filling ability. The heights of concrete at both ends of the apparatus (H_1 and H_2) are also measured to determine L-box results. This test gives an indication of the filling, passing, and segregation ability of the concrete. The ratio of H_2/H_1 was knows as "blocking ratio" which indicates the slope of the concrete when at rest, which is an indication of passing ability or the degree to which the passage of concrete through the bars was restricted. Test results of L-box test are tabulated below in Table. 10.

Table. 10 : Results of L-box test

Shear connectors used for the specimen	**Result of L-box test of Fresh concrete**			**Remarks (Condition of the fresh concrete)**
	T_{20} (sec)	T_{40} (sec)	Blocking ratio H_2/H_1	
Headed stud	3.0	6.9	0.82	Self compactable and thixotropic
Stud without head	2.9	6.7	0.79	Self compactable and thixotropic
Channel Connector	2.9	6.8	0.80	Self compactable and thixotropic
Box Connector	3.0	7.1	0.83	Self compactable and thixotropic

4.4.4. J-RING TEST

J - Ring tests is another type of method for the study of the blocking behavior of self-compacting concrete. The apparatus consists of re-bars surrounding the Abram’s cone in a slump flow test.

The spacing between the rebars is generally kept three times of the maximum size of the aggregate for normal reinforcement consideration. The concrete flows between the re-bars after thee cone is lifted and thus the blocking behavior / passing ability of concrete can be observed. Ferrari reported that a concrete mixture qualify for self-campactability based on slump/spread but could fail on the basis of V - flow and U - flow test results. Bouzoubaa and Lachemi also reported similar results for slump spread and V - flow test.

4.4.5. V-FLOW TEST

Another type of test, which is frequently adopted, was the V - flow test. It consists of a funnel with a rectangular cross section. The top dimension is 495 mm by 75 mm and bottom openings is 75 mm by 75 mm. The test is designed to measure the flowability; the result may be affected by concrete properties other than flow. The inverted cone shape will cause liability of the concrete to block reflected in the result. For example if there is too much coarse aggregate then high flow time can also be associated with low deformability due to a high paste viscosity and high inter particle friction. While the apparatus is simple, the effect of the angle of the funnel and the wall effect on the flow of concrete is not clear. The total height is 572 mm with a 150 mm long straight section. The concrete is poured into the funnel with a gate blocking the bottom opening. When the funnel is completely filled, the bottom gate is opened and the time for the concrete to flow the funnel is noted. As it measures the ease of flow of concrete, shorter flow time indicates greater flowability and prolonged time is the indication of the susceptibility of the mix to blocking. This is called the V-flow time. A flow time less than 10 seconds is recommended for a concrete to qualify for a self-compacting concrete.

4.4.6. ORIMET TEST

In this project this test has not conducted due to non-availability of the apparatus. However the test method and its significance are given below. This is a method for assessment of highly workable, flowing fresh concrete mixes on construction sites. This test is based on the principle of orifice rheometer. The Orimet consists of vertical casting pipe fitted with a changeable inverted cone-shaped orifice at this lower (discharge) end with a quick release trap door to close the orifice. Usually the orifice has an 80 mm internal diameter which is appropriate for assessment of concrete mixes of aggregate size not exceeding 20 mm. Orifice of other sizes, usually from 70 mm to 90 mm in diameter, can be fitted instead. Operation consists of filling the orimet with 8 liters of concrete without compacting or tapping. Then within 10 seconds after filling the trap door should be opened and allowed the concrete to flow out under gravity. The time taken for the complete discharge that is known as "the flow time", is to be noted down.

This test is able to simulate the flow of fresh concrete during actual placing on sites. It is a rapid test and equipment is simple and easily maintained. The test has the useful characteristic of being capable of differentiation between highly workable, flowing mixes. Therefore it is useful for compliance testing of successive loads at site.

This test measures the ease of flow of the concrete Shorter flow times indicate greater flowability. For SCC a flow time of 5 seconds or less is considered appropriate. The inverted cone shape at the orifice restricts flow and prolonged flow times may give some indication of the susceptibility of the mix to blocking and/or segregation.

4.5. TESTS ON HARDENED CONCRETE

GENERAL

Tests on hardened concrete has done to evaluate the strength in compression as well in tension, bending stress and modulus of elasticity of concrete, which gives an overall picture of the quality of the concrete used. Various test conducted on hardened concrete which are given below with their procedure and the test results.

4.5.1. CUBE COMPRESSIVE STRENGTH TEST

Strength is a measure of the amount of stress required to fail the material. The strength of the concrete is the property most valued by designers and quality control engineers. Since concrete is strong in compression, the compressive strength of the concrete was evaluated. Majority of concrete elements are designed to take advantage of higher compressive strength of the material. Compared to most other properties, testing of strength is relatively easy. Further more, many properties of concrete, such as elastic Modulus, water tightness and resistance to weathering are directly related to the strength.

In order to know the performance of SCC used for casting of the composite specimen total 12 cubes were casted. The size of the ‘Standard Mould’ was 15x15x15 cm. The fresh SCC mix is just poured in the moulds from 300 mm height without any vibration. The specimens were kept in damping place and left for 24 hours and then the moulds were removed immediately then the specimens were kept in curing tank for 28 days. Three specimens were tested each time for each specimen and the average value was reported. The compressive strength at the end was found and the result shows in Table.11.It has been observed that,most of the cases the failure of specimens were cup and cone type.

Table. 11 : Compressive strength results

Specimen	Shear Connector	Concrete	Days	Cube Compressive Strength (MPa)
I	Headed Stud Connector	SCC	28	47.10
II	Without Head Stud Connector	SCC	28	47.85
III	Bar Connector	SCC	28	45.14
IV	Channel Connector	SCC	28	48.21

4.5.2. SPLIT TENSILE STRENGTH TEST

Direct tensile strength test of concrete were seldom carried out because, the specimen-holding device introduce secondary stress that cannot be ignored. The most commonly used tests for estimating the tensile strength of concrete were “split tensile strength test” and the “Third point flexural loading test”. The split tensile test on cylinder was also known as "Brazilian Test".

Placing the cylindrical specimen horizontally between the loading surfaces of compression testing machine carried out the test and the load was applied until failure of the cylinder taken place along the vertical diameter. This loading condition produces a high compressive stress immediately below the two generators to which the load is applied but the larger portion of corresponding depth was subjected to uniform tensile stress acting for about 1/6th of the diameter and the remaining 5/6th depth was subjected to compression. The main advantage of this test was both cube compression test and split tensile strength test were carried out on the same machine.

The split tension test was simple and gives more uniform results than other tension tests and strength determined in the splitting test was believed to be closer to the true tensile strength of concrete than modulus of rupture. Split tensile strength gives about 10 to 15% higher values than the direct tensile strength.

The split tensile strength test was carried at the age of 28 days and the results were shown in Table.12. Total 24 specimens were casted with an average of 6 for each specimen.

Table. 12 : Split tensile strength results

Specimen	Shear Connector	Concrete	Days	Split Tensile Strength (MPa)
I	Headed Stud Connector	SCC	28	3.42
II	Without Head Stud Connector	SCC	28	3.61
III	Bar Connector	SCC	28	3.18
IV	Channel Connector	SCC	28	3.67

4.5.3. MODULUS OF RUPTURE TEST

The value of the modulus of rupture (extreme fiber stress in bending) depends upon the dimension of the beam and the manner of loading. The modulus of rupture was higher than the split tensile strength. The system of loading used to find the flexural tension was two point load test where the maximum bending moment was constant between the middle third points so the failure may be any where in between the middle third points. It is found that the two points loading will yield a lower value of the modulus of rupture than the center point loading. The standard size of the specimen were 10x10x50 cm. This is based upon the maximum size of aggregate of 20 mm; if the size of aggregate exceeds 20 mm then the standard size of specimens will be 15x15x75 cm. The test on the beam and size of the specimens were according to IS: 516-1959. In this case also the SCC beams were just filled by pouring mix from a height of 300 mm. The specimens were demoulded at the end of 24 hours and continuously kept in curing tank up to 28 days. The rate of loading that the failure of the prism is within the middle third. The results are shown in Table.13.

Table. 13 : Flexural strength results

Specimen	Shear Connector	Concrete	Days	Flexural Strength (MPa)
I	Headed Stud Connector	SCC	28	5.78
II	Without Head Stud Connector	SCC	28	5.82
III	Bar Connector	SCC	28	5.61
IV	Channel Connector	SCC	28	6.16

4.5.4. MODULUS OF ELASTICITY TEST

The modulus of elasticity of the concrete is an important parameter in the structural design of RCC members. Which assumes that a perfect bond exists between the steel and reinforcement. So the stress in concrete is 'm' times the corresponding stress in concrete at the location, where 'm' is known as the modular ratio which is defined as the ratio of modulus of elasticity of steel to modulus of elasticity of concrete. The accuracy of design depends upon the correct estimation of modulus of elasticity of concrete since the modulus of steel was more or less constant. The modulus of elasticity of concrete was determined by conducting compression test on 150 mm diameter cylinder by noting the stress versus strain values using dial gauges. The short term modulus of elasticity given by IS: 456- 2000 was based upon ACI value.

CHAPTER 5

RESULTS AND ANALYSIS

GENERAL

This chapter contains the results of push-out test conducted on all steel-concrete composite specimens. Relationship developed between shear load and slip, Shear stress and strain, ultimate strength of specimen and cube compressive strength of concrete also given in this chapter along with the uplift and failure modes of composite specimen.

5.1. TESTS ON PUSH-OUT SPECIMEN

The “standard push-out” test has been used in this project work to evaluate the shear strength, load-slip and load-strain characteristics of shear connectors under static load as recommended by CP : 117.

All composite specimens were of the same dimensions having identical reinforced concrete slabs and ISMB 300 connected by means of four different types of shear connectors. Those are :

SPECIMEN	SHEAR CONNECTOR
I	Headed Stud Connector(Diameter-20mm)
II	Stud Without Head Connector(Diameter-20mm)
III	Bar Connector(42.5x42.5mm)
IV	Channel Connector(ISMC100)

5.1.1. TEST SET UP

All push-out tests were carried out in a universal-testing machine (Capacity 200tf).In Fig. 24 a typical specimen is shown in the machine ready for testing. Each specimen was bedded down on the lower platen using elastomeric bearing. Before testing and special care was taken to ensure concentric loading

The instrumentation for measuring the relative slip between the steel beam and the concrete slab consisted of two deflectometer of least count 0.0025 mm attached to flange of the steel joist and top portion of the concrete slab. The strain variation in these shear connectors during the test was monitored through a strain indicator as shown in Fig.24.

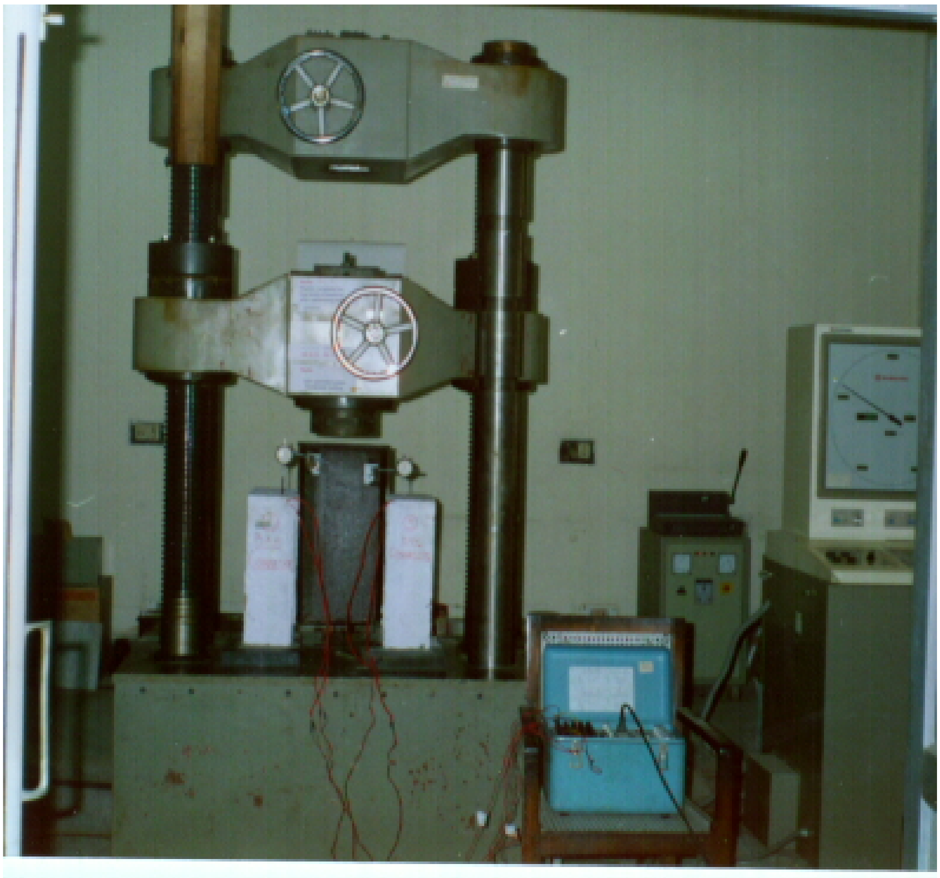


Fig. 24. Typical push-out specimen in UTM.

5.1.2. TESTING PROCEDURE

The increment of load was applied to the specimens through the 200-ton Universal Testing Machine. The load was applied in the increment of 10kN at the initial stage and later on it was reduced to 5kN after the onset of the initial cracking. The loads were gradually applied and data of the slip and the variation of the strain at each increment was recorded till the complete failure of the specimen.

5.1.3. INTERPRETATION

It was assumed that the shear load per shear connector was the total load applied to the specimen divided by total numbers of shear connectors. This assumption was shown to be reasonable during testing by the very small differences between the two slip gauge readings. In plotting the results the mean of the two readings was used.

5.2. TEST RESULTS AND OBSERVATIONS

5.2.1. SPECIMEN I

In this steel-concrete composite specimen headed stud shear connector (20mmΦ,100mm hight) has used, which is one of the most commonly used flexible shear connector now a days. The test results and observations on different aspects such as slip, uplift, strain and failure modes are given below.

SLIP

During testing difference between the two slip gauges readings was very small. So the mean magnitude of slip was calculated. The calculated average vertical slip and the corresponding shear load per connector is tabulated as given in Table.14.

Table. 14 : Shear load-vertical slip values of specimen I

SL NO	Shear Load (kN)	Shear Load Per Connector (kN)	Mean Slip (mm)	Remarks.
1	0	0	0	
2	20	5	0.097	
3	40	10	0.146	
4	60	15	0.215	
5	80	20	0.276	
6	100	25	0.365	
7	120	30	0.415	
8	140	35	0.495	
9	**148.5**	**37.125**	**0.530**	Initial cracking load
10	160	40	0.585	
11	180	45	0.749	
12	200	50	1.037	
13	220	55	1.352	
14	240	60	1.985	
15	260	65	2.846	
16	280	70	4.315	
17	**291.5**	**72.875**	**5.700**	Ultimate load

The observed static shear load are plotted against the mean vertical slip of the specimen in Fig.25.

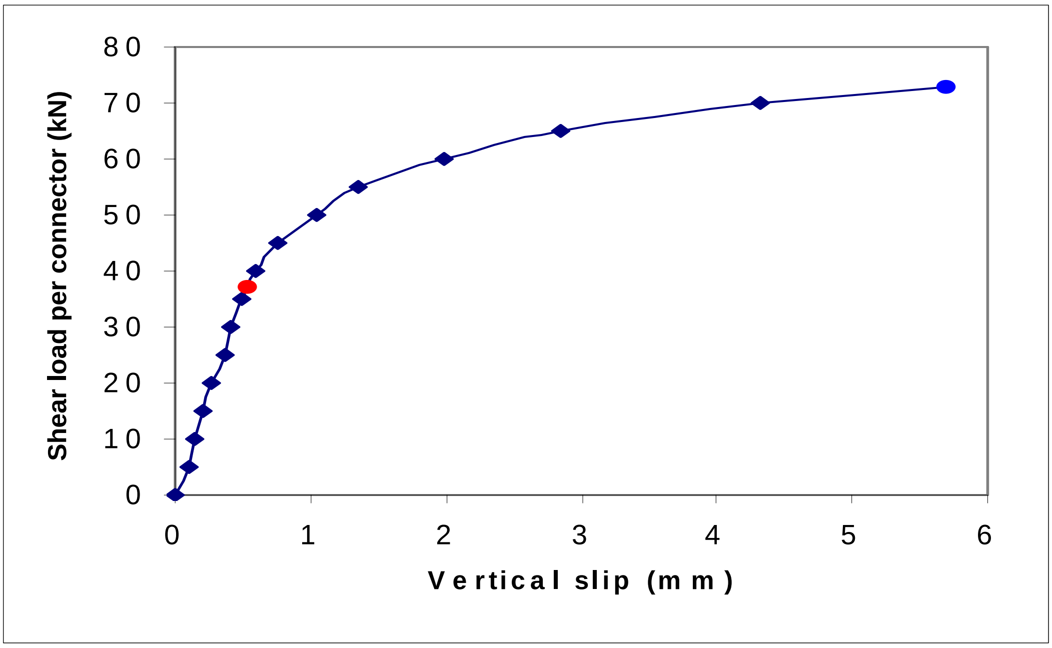


Fig. 25. Shear load-vertical slip relationship of specimen I

OBSERVATIONS

- It was noticed that the slip was not prominent till the occurrence of the initial crack. The magnitude of the recorded slip was only 0.53 mm at an applied load of 148.5kN.
- Load-slip relationship curve is linear till the initial cracking of concrete.
- After the development of initial crack the magnitude of slip increased at a faster rate as the rate of increment of shear load further increased.
- After the point of initial crack, the rate of slip increased at a faster rate with the standard increment of load
- The maximum mean slip recorded in the specimen was 5.70 mm at an ultimate load of 291.5 kN.
- Slip occurred partly due to delamination of concrete in the region of shear connector and partly due to deformation of shear connector.

STRAIN

During testing the strain value in all the studs due to shear load recorded. The recorded strain corresponding to the shear stress are tabulated in Table.15.

Table. 15 : Shear stress – strain values of specimen I

SL No	Shear Load Per Connector (N)	Shear resisting area (mm^2)	Shears stress of each connector (N/mm^2)	Strain measured (Micro Strain)			
				S_1	S_2	S_3	S_4
1	0	314.16	0	0	0	0	0
2	5000	314.16	15.915	387	380	100	100
3	10000	314.16	31.831	562	542	100	155
4	15000	314.16	47.746	1482	1352	120	185
5	20000	314.16	63.662	2617	1549	175	235
6	25000	314.16	79.577		1967	185	275
7	30000	314.16	95.493		4642	230	335
8	35000	314.16	111.408			302	380
9	**37125**	**314.16**	**118.170**			**405**	**386**
10	40000	314.16	127.324				446
11	45000	314.16	143.240				500
12	50000	314.16	159.154				490
13	55000	314.16	175.070				408
14	60000	314.16	190.985				360
15	65000	314.16	206.901				160
16	70000	314.16	222.816				-535
17	**72875**	**314.16**	**231.968**				**-**

The observed strain in stud 1, stud 2, stud 3, stud 4, plotted against the shear stress per connector of the specimen in Fig. 26.

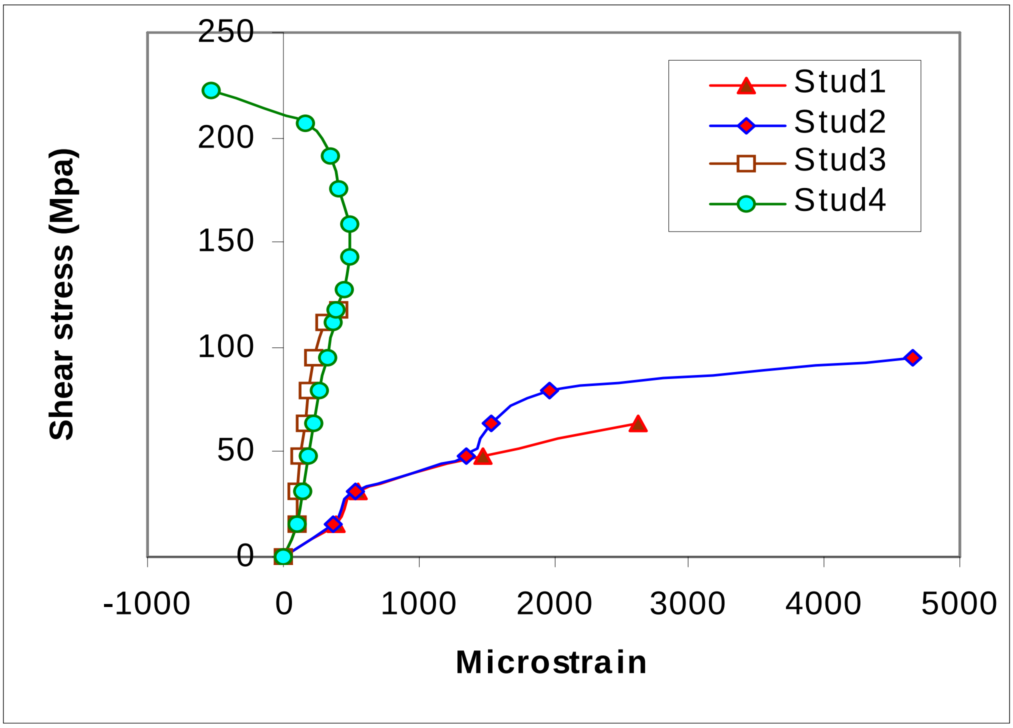


Fig. 26. Shear stress – strain relationship of specimen I

OBSERVATIONS

- The strain in studs increased with the increment of stress. Strain gauge of stud 1 failed at the load of 20kN and strain was 2617 micro strain. From Fig.26 it has observed that the stud 1 was completely in tension till the failure of strain gauge.
- Strain gauge of stud 2 failed at the load of 30KN and strain was 4642 micro-strain. From Fig.26 it has observed that although stud 2 also was completely in tension still the deviation in increment of strain was prominent and was too high from 20kN to 30kN.
- Strain gauge of stud 3 failed at an applied load at which initial crack occurred i/e. 37.125kN and the strain was 405 micro-strain. It has observed that stress-strain relationship of stud 3 was almost linier till the failure of strain gauge and was completely in tension.

- Strain gauge of stud 4 was consistent till the ultimate load. It has observed from the graph that strain increased proportionate to the stress till 45kN and was in tension but after 45kN increment rate of strain decreased and finally reached negative value (-535 micro-strain) which implies that the stud was in compression at the ultimate load of 75.875 kN.

UPLIFT

Measurable uplift occurred at the initial stage of applied load although it was not that much prominent and accelerated as the test progressed. It was found that the uplift of the slab was first induced in the lower half of the slab which than propagated in the upper half where it became visible.

Uplift of the concrete slab at the ultimate load was more at the bottom as compare to the top. At top it was 8mm at RHS and 7mm at LHS where as at bottom it was 12mm at RHS and 10mm at LHS.

It was noticed that the uplift was least at the area surrounding the shear connector and that was due to the head of the stud connector which hold down the concrete slab and did not permit the uplift.

FAILURE MODE

For the headed stud yield accompanied by considerable cracking and local crushing of the concrete due to high bearing stress near the root of the stud.

Initial crack occurred in the specimen at an applied load of 148.5 KN i.e. 50.94 percent of ultimate load. Initially cracks developed near the interface of steel and concrete of the composite specimen and later on it spread towards the side faces of the concrete blocks.

At the ultimate load of 291.5 kN, diagonal cracks leading from the stud to the sides of the concrete slab and forming an angle of between 40º – 60º to the line of the studs were also visible in the concrete slabs. A typical set of specimen after testing has shown in Fig.27.

Fig. 27. Photograph of specimen I after testing.

5.2.2. SPECIMEN II

In this steel-concrete composite specimen stud without head shear connector (20mm dia & 10mm height) has used which is one of the most commonly used flexible shear connectors. Test results and observations are given below.

SLIP

Due to very small differences between two slip gauge readings mean vertical slip calculated. The calculated average vertical slip and the shear load per connector tabulated in Table. 16.

Table. 16 : Shear load – vertical slip values of Specimen II

SL NO	Shear Load (kN)	Shear Load Per Connector (kN)	Mean Slip (mm)	Remarks.
1	0	0	0	
2	20	5	0.037	
3	40	10	0.080	
4	60	15	0.140	
5	80	20	0.215	
6	100	25	0.297	
7	120	30	0.382	
8	140	35	0.550	
9	**140.5**	**35.125**	**0.558**	Initial cracking load
10	160	40	0.764	
11	180	45	1.022	
12	200	50	1.312	
13	220	55	1.728	
14	240	60	2.0375	
15	260	65	2.744	
16	280	70	4.025	
17	**280.5**	**70.125**	**4.045**	Ultimate load

The observed static shear load are plotted against the mean vertical slip of the specimen in Fig. 28.

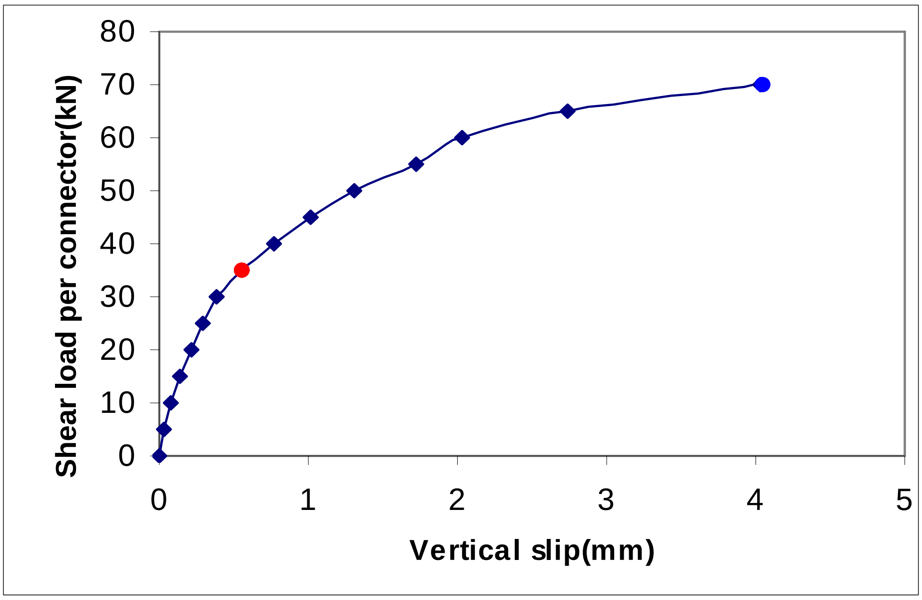


Fig. 28. Shear load – vertical slip relationship of Specimen II

OBSERVATIONS

- Initially slip was very less and the magnitude was only 0.56 mm at an applied load of 140.5 kN at which initial crack occurred in the specimen. Slip at initial cracking load was only 13.84 percent of the final slip.
- Load-slip relationship curve was almost linear upto the initial cracking load.
- After the occurrence of initial crack the rate of slip increased at a faster rate with the standard rate of increment of shear load.
- The increment of slip was maximum between 260kN to 280.5kN which was ultimate stage of the loading.
- Final mean slip recorded in the specimen was 4.045 mm at an ultimate load of 280.5 kN.

STRAIN

During testing strain in all the four studs due to the applied shear load recorded. The recorded strain corresponding to the calculated shear stress per connector tabulated in Table. 17.

Table 17. : Shear stress – strain relationship values of Specimen II

SL No	Shear Load Per Connector (N)	Shear resisting area (mm^2)	Shears stress of each connector (N/mm^2)	Strain measured (Micro Strain)			
				S_1	S_2	S_3	S_4
1	0	314.16	0	0	0	0	0
2	5000	314.16	15.915	-10	55	175	90
3	10000	314.16	31.831	-90	110	545	165
4	15000	314.16	47.746	10	180	1070	210
5	20000	314.16	63.662	20	260	1775	230
6	25000	314.16	79.577	-1370	395	2790	325
7	30000	314.16	95.493	-1425	376	4275	410
8	35000	314.16	111.408	-2755	430	8325	460

9	**35125**	**314.16**	**111.806**	**-2740**	**425**	**8350**	**475**
10	40000	314.16	127.324	-2620	330		
11	45000	314.16	143.240	-4770	415		
12	50000	314.16	159.154				
13	55000	314.16	175.070				
14	60000	314.16	190.985				
15	65000	314.16	206.901				
16	70000	314.16	222.816				
17	**70125**	**314.16**	**223.214**				

The recorded strain in all the studs (Stud 1, Stud 2, Stud 3, Stud 4) plotted against the shear stress per connector of the specimen in Fig. 29.

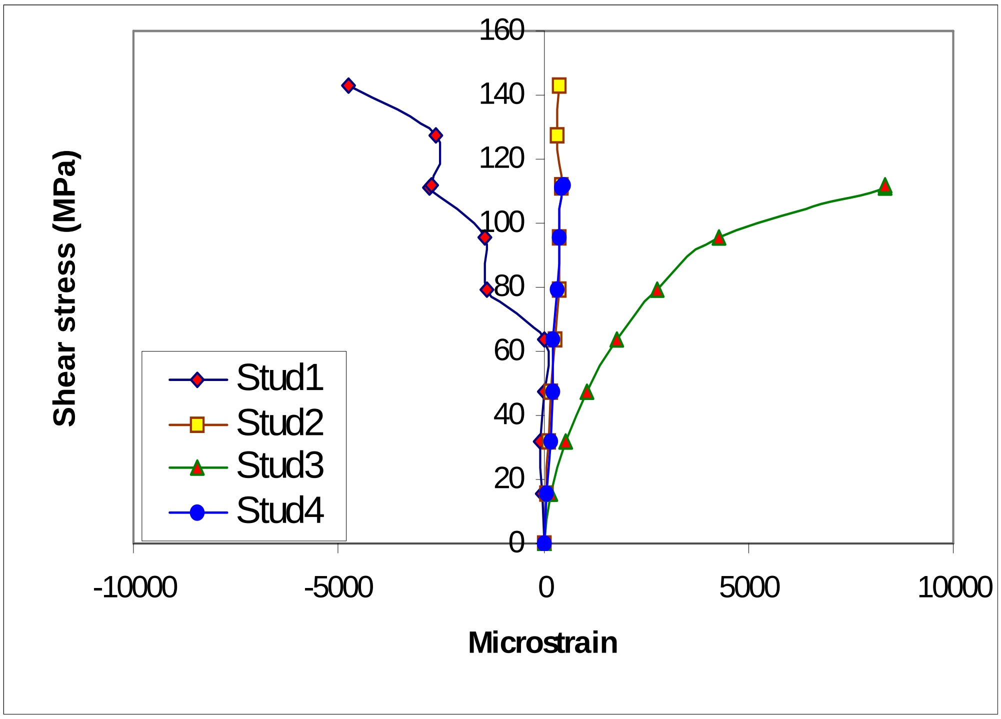


Fig. 29. Shear stress – strain relationship of Specimen II

OBSERVATIONS

- Strain gauge of Stud 1 failed at an applied load of 45kN and the strain was – 4770 micro strain. From the result it has noticed that stud 1 was in compression up to 10kN then tensile forces acted upon it and it was in tension from 10kN to 20kN. Then again after 20kN compressive forces acted upon it and was in compression from 20kN to 45kN till the failure of strain gauge.
- Strain gauge of stud 2 failed at the load of 45kN and the strain was 415 micro-strain. Fig. 29 shows that stud 2 was completely in tension till the failure of strain gauge and the stress relationship was almost linear. Strain was continuously increasing with the increment of load till 25kN then onwards it decreased and again it increased from 30kN till the initial cracking load of 35.125kN. Again the value decreased and increased after 40kN till the end.
- Strain gauge of stud 3 failed at an applied load of 35.125kN (initial cracking load). It has noticed that the stud 3 was completely in tension till the failure of strain gauge and increment of stain was maximum near the initial cracking load.
- Strain gauge of stud 4 also failed at an applied load of 35.125kN. Stud 4 was completely in tension till the failure of strain gauge. Increment of strain was maximum in between 20 to 25kN.

UPLIFT

Uplift of the slab was not prominent at initial stage of loading. From the final uplift measurement it has noticed that uplift of the concrete slab was more at bottom as compare to the top. At top it was 10 mm in RHS and 9 mm at LHS where as at bottom it was 14 mm at RHS and 12 mm at LHS.

It was noticed after comparison with specimen I that, the uplift in case of specimen I was 50 percent of the uplift in case of specimen II at the area surrounding shear connector and it was due to the lack of holding capacity of the without headed shear connector.

FAILURE MODE

Failure mode of the specimen II was due to the crushing of concrete and was due to the high bearing stress near the root of the stud.

Initially cracks developed near the interface of steel and concrete of the composite specimen leading from the studs. Initial crack occurred in the specimen at an applied shear load of 140.5kN i/e. 50.10 percent of ultimate load. Cracks developed at the sides of the concrete slab at an applied shear load of 171.90kN.

All the cracks were diagonal in nature. No cracks were found on the outer surfaces of the concrete slabs. A typical set of specimen after testing has shown in Fig. 30.

Fig. 30. photograph of specimen II after testing.

5.2.3. SPECIMEN III

In this steel-concrete composite specimen Bar Connector (C/S of 42.5 x 42.5 mm) has used as shear connector, which is one of the most commonly used rigid shear connector. The test results and observations on different aspects such as slip, uplift, strain and failure modes are given below.

SLIP

The calculated average vertical slip and the corresponding shear load per connector tabulated in Table. 18.

Table. 18 : Shear load – vertical slip values of Specimen III

SL NO	Shear Load (kN)	Shear Load Per Connector (kN)	Mean Slip (mm)	Remarks.
1	0	0	0	
2	20	10	0.032	
3	40	20	0.062	
4	60	30	0.092	
5	80	40	0.126	
6	100	50	0.175	
7	120	60	0.214	
8	140	70	0.269	
9	160	80	0.325	
10	180	90	0.385	
11	200	100	0.467	
12	220	110	0.537	
13	**220.02**	**110.01**	**0.537**	Initial cracking load
14	240	120	0.867	
15	260	130	1.300	
16	280	140	2.280	
17	300	150	2.875	
18	320	160	3.560	
19	**331.8**	**165.9**	**4.330**	Ultimate load

The observed static shear load are plotted against the mean vertical slip of the specimen in Fig. 31.

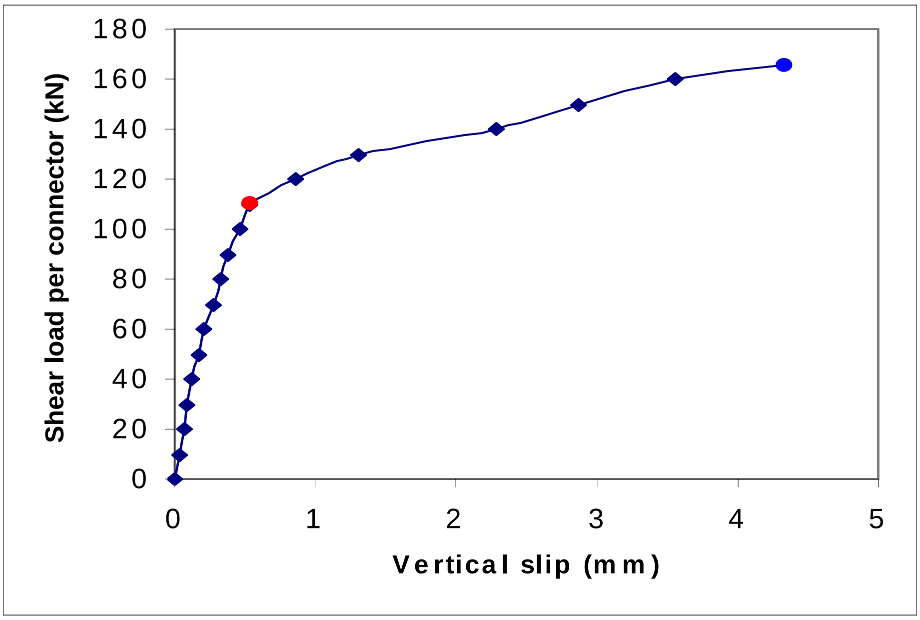


Fig. 31. Shear load – vertical slip relationship III.

OBSERVATIONS

- It has been noticed that the increment of slip was not prominent till the propagation of initial crack. The magnitude was only 0.54 mm i.e. 12.47 percent of the maximum slip at an applied load of 220 KN.
- Fig 31 also shows that the load-slip relationship was linear up to the occurrence of initial crack.
- The rate of increment of slip increased with the increment of applied load at a faster rate after the occurrence of initial crack.
- The magnitude of maximum mean slip recorded at an ultimate load of 331.8 KN was 4.33 mm.

STRAIN

During testing strain in all the loops connected to the bar connector due to shear load recorded. The recorded strain corresponding to the shear stress tabulated in Table. 19.

Table. 19 : Shear stress – strain values of Specimen III

SL No	Shear Load Per connector (N)	Shear resisting area (mm^2)	Shears stress of each connector (N/mm^2)	Strain measured (Micro Strain)			
				S_1	S_2	S_3	S_4
1	0	3187.5	0	0	0	0	0
2	10000	3187.5	3.137	130	155	315	205
3	20000	3187.5	6.275	255	290	425	315
4	30000	3187.5	9.142	355	420	510	415
5	40000	3187.5	12.550	495	620	690	555
6	50000	3187.5	15.686	655	810	845	710
7	60000	3187.5	18.824	790	1000	980	860
8	70000	3187.5	21.961	945	1225	1135	1020
9	80000	3187.5	25.098	1070	1450	1310	1190
10	90000	3187.5	28.235	1220	1639	1495	1370
11	100000	3187.5	31.372	1450	2095	1659	1895
12	110000	3187.5	34.510	1695	2380	1790	2385
13	110100	3187.5	34.541	1697.45	2383	1792	2390
14	120000	3187.5	37.647	28.90	3725		2824
15	130000	3187.5	40.784	4115			3375
16	140000	3187.5	43.921				3920
17	150000	3187.5	47.059				4515
18	160000	3187.5	50.196				5110
19	165900	3187.5	52.047				5765

The observed strain in strain gauge 1, strain gauge 2, strain gauge 3, strain gauge 4 plotted against the shear stress of the specimen in Fig. 32.

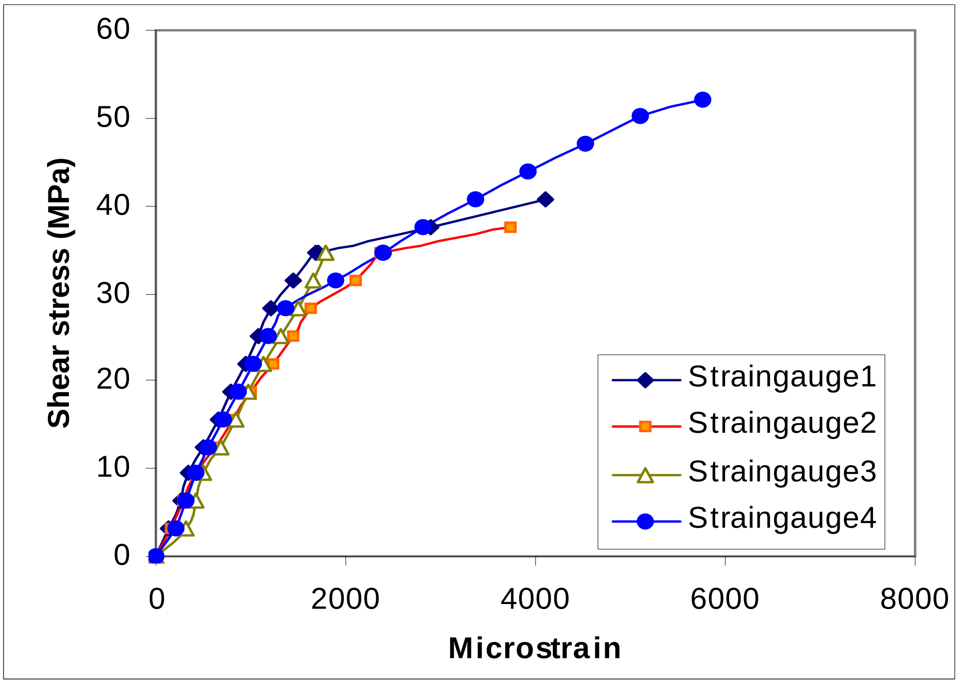


Fig. 32. Shear stress – strain relationship of Specimen III

OBSERVATIONS

- Strain gauge 1 and strain gauge 2 were installed at right side bar connector. Strain gauge 1 failed at an applied load of 130kN and the strain was 4115 micro-strain where as strain gauge 2 failed at an applied load of 120kN and the strain was 3725 micro-strain. After observing the stress-strain relationship of strain gauge 1, strain gauge 2 from Fig. 32 it was clear that, right hand side bar was completely in tension till the failure of strain gauges.

- Strain gauge 3 and strain gauge 4 were installed on left side bar connector. Strain gauge 3 failed at the initial cracking load of 110.10kN and the strain was 1792 micro-strain where as strain gauge 4 was in working condition till the ultimate load and the strain was 5765 micro strain. As there is minor differences in the strain values of strain gauge 3 and strain gauge 4 so to get the clear idea till the ultimate load strain gauge 4 readings were taken into account. From the measured strain values of strain gauge 4 it has noticed that left side bar connector was also completely in tension till the complete failure of the specimen.

UPLIFT

Although uplift of the slab was not prominent at the initial stage of loading still the uplift was to high at the ultimate load and right side concrete slab detached from the steel section. From the measurement of ultimate loading it has noticed that uplift at top was 14 mm, and 24 mm at bottom of the left side concrete slab. It was observed that the bar connectors have poor resistively towards the uplift.

FAILURE MODE

The concrete failure observed in tests on bar connector was the shearing of a wedge of concrete immediately in front of the connector and the mode of failure was the failure of concrete.

Initial crack occurred in the specimen at an applied shear load of 220.22kN which is 66.31 percent of the ultimate shear load. Initial cracks occurred at the side faces of the concrete slab at an applied shear load of 241.40 kN. It has noticed that cracks occurred at the outer faces of the concrete slab at ultimate load of 331.8 kN.

It was observed that all cracks developed near the interface of steel and concrete of the composite specimen leading from the bar connector and later on proceeded towards the sides and bottom part of the specimen with the increment in application of load. A typical set of specimen III after testing has shown in Fig.33.

Fig. 33. photograph of specimen III after testing.

5.2.4. SPECIMEN IV

Channel connector (ISMC 100) has used as shear connector in this composite specimen which is a flexible shear connector. The test results and observations on different aspects such as slip, uplift, strain and failure modes are given below.

SLIP

Due to the minor differences between the two slip gauges readings mean slip has calculated. The calculated mean slip and the corresponding shear load per connector tabulated in Table. 20.

Table. 20 : Shear load – vertical slip values of Specimen IV

SL NO	Shear Load (kN)	Shear Load Per Connector (kN)	Mean Slip (mm)	Remarks.
1	0	0	0	
2	20	10	0.145	
3	40	20	0.301	
4	60	30	0.445	
5	80	40	0.557	
6	100	50	0.700	
7	**100.80**	**50.40**	**0.705**	Initial cracking load
8	120	60	1.020	
9	140	70	1.320	
10	160	80	1.920	
11	180	90	2.732	
12	200	100	3.320	
13	220	110	4.082	
14	240	120	5.545	
15	260	130	8.107	
16	**269.4**	**134.7**	**9.310**	Ultimate Load

The observed static shear load are plotted against the mean vertical slip of the specimen in Fig. 34.

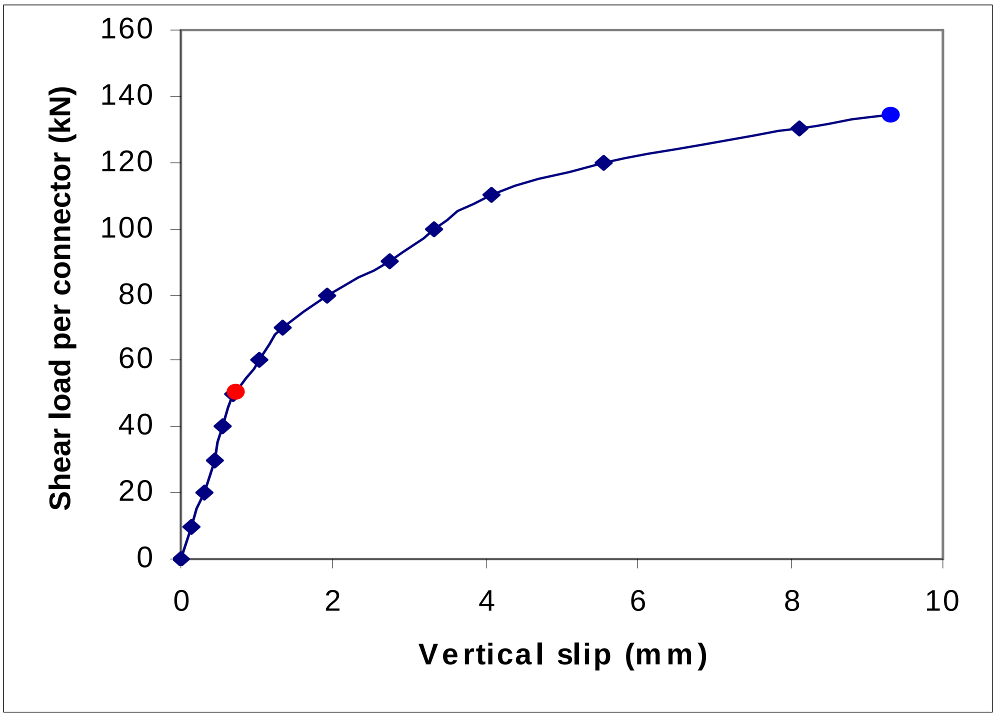


Fig. 34. Shear load – vertical slip relationship of Specimen IV.

OBSERVATIONS

Slip was not measurable at the initial stage of application of load. It was very small till the initial cracking occurred. The magnitude was only 0.705 mm which is only 7.57 percent of the final slip where as the load at 100.80 kN. Although magnitude of slip was very less still it was the maximum comparing to the other specimens.

From Fig. 34 it was clear that the load-slip relationship curve was linear till the occurrence of initial crack in the concrete slab. After the initial crack the rate of slip increased at a faster rate with the standard increment of loading.

The maximum mean slip recorded in the specimen was 9.31 mm at an ultimate load of 269.4 kN. The magnitude of maximum mean slip value was maximum among all the specimen although ultimate load was the least among all which concludes the prior resistivity of slip of the channel connector.

STRAIN

During testing strain in both the channel connectors due to the applied shear load recorded. The recorded strain corresponding to the calculated shear stress per connector tabulated in Table. 21.

Table. 21 : Shear stress – strain values of specimen IV

SL No	Shear Load Per Connector (N)	Shear resisting area (mm^2)	Shears stress of each connector (N/mm^2)	Strain Measured (Micro- strain)	
				S_1	S_2
1	0	3750	0	0	-1589
2	10000	3750	2.67	-100	-1514
3	20000	3750	5.33	-350	-1539
4	30000	3750	8.00	-430	-1519
5	40000	3750	10.67	-530	-1609
6	50000	3750	13.33	-650	-1634
7	**50400**	3750	**13.44**	**-655**	**-2234**
8	60000	3750	16.00	-605	-3024
9	70000	3750	18.67	-800	-4224
10	80000	3750	21.33	-880	-5909
11	90000	3750	24.00	-550	-7759
12	100000	3750	26.67	345	-10099
13	110000	3750	29.33	1080	-12294
14	120000	3750	32.00		
15	130000	3750	34.67		
16	**134700**	3750	**35.92**		

The recorded strain in both channel connectors (Channel 1, Channel 2) plotted against the shear stress per connector of the specimen in Fig. 35.

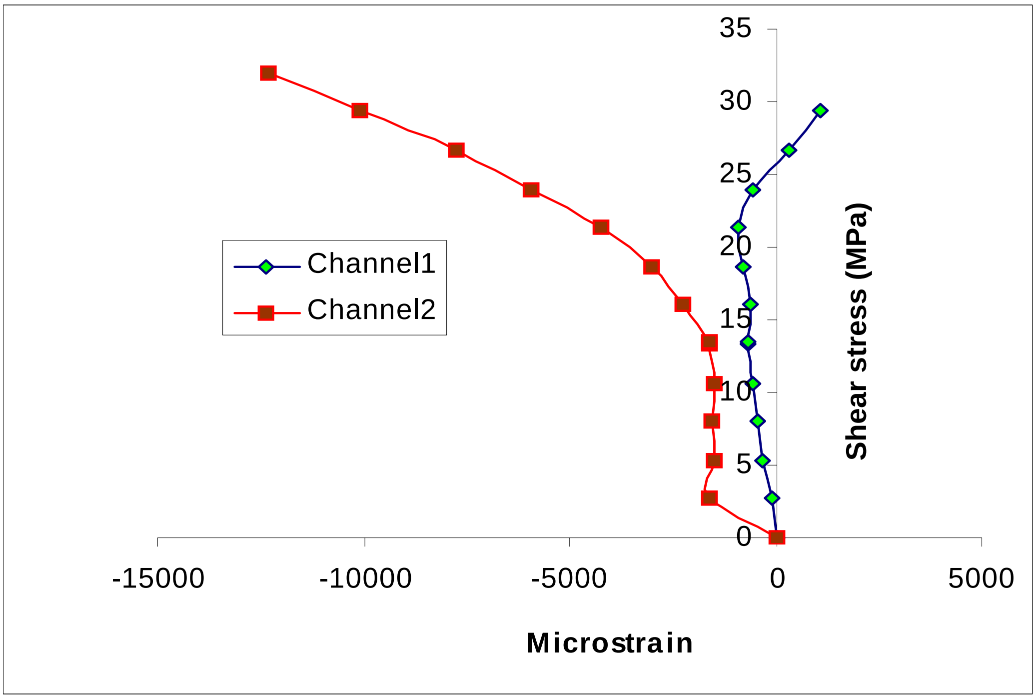


Fig. 35. shear stress – strain relationship of specimen IV.

OBSERVATIONS

Strain gauge 1 failed at an applied shear load of 110 kN and the strain was 1085 micro-strain. From Fig. 35 and Table. 21 it has observed that channel 1 was in compression till 90 kN and then tension forces developed on it till the failure of the strain gauge.

Strain gauge 2 failed at an applied shear load of 120 kN and the strain was – 12294 micro-strain. From Fig. 35 it has noticed that upto 10 kN strain increased and the channel was in compression. In between 10 to 20 kN strain decreased and after 20 kN again strain increased and after 30 kN again strain decreased finally from the load of 40 kN strain increased till the failure of strain gauge and through the entire loading strain was negative which was indicating that the channel was in compression throughout the test.

UPLIFT

It has noticed that uplift was very less and was first induced in the lower half of the slab and then propagated in the upper half where it became visible. Uplift at top of left side slab was less then the bottom and was least at the line of connector. It was 6 mm at top, 9 mm at bottom and 4 mm at the line of shear connector. From the observation it has noticed that the uplift was least at the line of shear connector and that was due to the flanges of channel connector which hold down the concrete slab and did not permit the uplift.

FAILURE MODE

Failure mode was the failure of shear connector (Channel Connector). In the case of channel connector the outstanding flanges yield, thereby throwing more pressure on to the spine and root of the connector.

Initial crack occurred in the specimen at an applied shear load of 100.80 kN which is 37.42 percent of ultimate load.

Initially cracks occurred at the interface of steel and concrete leading from the channel connector then it spread towards the side faces.

Finally the channel connector failed at an ultimate load of 269.4 kN. A typical set of Specimen IV after testing has shown in Fig. 36.

Fig. 36. Concrete slab of specimen IV after testing

5.3. DISCUSSIONS OF RESULTS

UPLIFT

Table. 22 : Comparison of uplift among all the Specimens

Specimen	Shear Connector	Uplift at Ultimate load (mm)				
		RHS		LHS		Line of Shear Connector
		Top	Bottom	Top	Bottom	
I	Headed stud	8	12	7	10	4
II	Stud without head	10	14	9	12	8
III	Bar	-	-	16	24	18
IV	Channel	-	-	6	9	4

OBSERVATIONS

- It has noticed that in case of bar connector the right side slab detached from the flange of steel beam and the uplift of left side slab is maximum (24 mm) among all the specimen which indicates it's poor resistance to uplift.
- In case of channel connector still from the measured value of uplift of left side concrete slab it has noticed that the uplift at top is least (6 mm) as well as uplift at the line of shear connector is also least (4 mm) among all the specimens.
- From the Table. 22 it is clear that flexible connectors such as channel and headed stud provides better resistance to uplift.

FAILURE MODE

Table. 23 : Failure mode of all the specimens

Specimen	Shear Connection	Initial Cracking Load. (KN)	Ultimate load. (KN)	Mode of Failure
I	Headed Stud	148.5	291.5	Concrete
II	Stud without head	140.5	280.5	Concrete
III	Bar	220.02	331.8	Concrete
IV	Channel	100.80	269.4	Channel

OBSERVATION

It has noticed that failure of all the connectors were due to crushing of concrete except channel connector which failed due to yielding of the channel.

SLIP

The calculated mean slip of all the specimen and the corresponding shear stress tabulated in Table. 24.

Table. 24 : Shear stress – mean slip values of all the specimen

SL No	Specimen I		Specimen II		Specimen III		Specimen IV	
	Shear stress (MPa)	Slip (mm)	Shear stress (MPa)	Slip (mm)	Shear stress (MPa)	Slip (mm)	Shear stress (MPa)	Slip (mm)
1	0	0	0	0	0	0	0	0
2	15.915	0.097	15.915	0.0375	3.137	0.033	2.67	0.145
3	31.831	0.146	31.831	0.085	6.275	0.062	5.33	0.301
4	47.746	0.215	47.746	0.146	9.412	0.093	8.00	0.445
5	63.662	0.276	63.662	0.215	12.550	0.126	10.67	0.558
6	79.577	0.365	79.577	0.297	15.686	0.175	13.33	0.700
7	95.493	0.415	95.493	0.382	18.824	0.214	13.44	0.705
8	111.408	0.495	111.408	0.550	21.961	0.269	16.00	1.020
9	118.17	0.53	111.806	0.558	25.098	0.325	18.67	1.320
10	127.324	0.585	127.324	0.764	28.235	0.385	21.33	1.920
11	143.240	0.149	143.240	1.023	31.372	0.468	24.00	2.733
12	159.154	1.037	159.154	1.313	34.510	0.538	26.67	3.320
13	175.070	1.352	175.070	1.728	34.541	0.538	29.33	4.083
14	190.985	1.985	190.985	2.038	37.647	0.868	32.00	5.545
15	206.901	2.846	206.901	2.744	40.784	1.300	34.67	8.108
16	222.816	4.315	222.816	4.025	43.921	2.280	35.92	9.310
17	231.968	5.70	223.214	4.045	47.059	2.875		
18					50.196	3.560		
19					52.047	4.335		

The calculated shear stress are plotted against the mean vertical slip of all the specimen in Fig. 37.

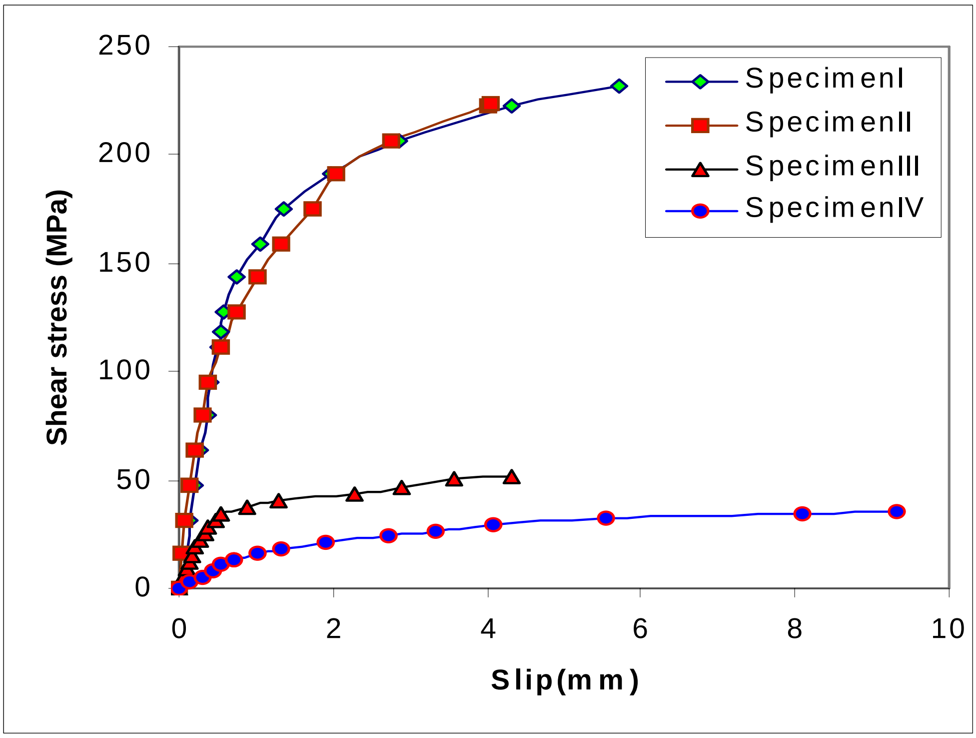


Fig. 37. Shear stress mean slip relationship of all the specimen.

OBSERVATIONS

- From the Fig. 37 it is clear that stress-slip relationship is linear till the initial crack develops and the rate of increment of slip proportionately increases with the increment of stress.
- After the occurrence of initial crock rate of slip accelerated with the standard increment of stress.
- It has observed that the magnitude of slip is maximum (9.31 mm) in case of specimen IV where as the stress is least (35.92 MPa) among all.
- Fig. 37 makes it clear that the magnitude of slip is least among all the specimen (4.045 mm) in case of without head stud connector at the stress of 223.214 MPa and in the increment of slip in case of both the connectors (Headed stud & stud without head) is almost same. It indicates that the head of connector although play a important role in case of uplift still it does not affect the slip.

- In case of bar connector it has noticed that although maximum slip was almost nearer (4.33 mm) to that of specimen I (4.045 mm) but the stress was too low (52.047 MPa) as compare to the specimen I (223.214 MPa).

COMPRESSIVE STRENGTH VS ULTIMATE STRENGTH OF SPECIMEN.

Table. 25 : Compressive strength – ultimate strength relationship of all specimen

Specimen	Shear Connector	Cube compressive strength of concrete (MPa)	Splitting tensile strength of concrete (MPa)	Ultimate Load of specimen. (KN)
I	Headed Stud	47.10	3.42	291.5
II	Stud without head	27.85	3.61	280.5
III	Bar	45.14	3.18	331.8
IV	Channel	48.21	3.67	269.4

OBSERVATIONS

- In case of specimen III compressive strength of concrete was least among all still the specimen could resist maximum load among all the specimen it concludes the better shear load resisting capacity of bar connector.
- In case of specimen IV compressive strength was maximum among all although the shear load resisting capacity was least which makes clear about the poor shear load resisting capacity of channel connector.

WELD STRENGTH VS ULTIMATE STRENGTH

Strength of the weld in shear (P) = Lt τ_{vf}

L = Effective length of the weld (mm)

t = Throat thickness in (mm) (i.e. 6 mm)

τ_{vf} = Shear stress in the weld in N/mm^2 (i.e. 108 N/mm^2)

P = Strength of the weld (N).

Table. 26 : Weld strength Vs Ultimate load of all specimen

Specimen	Shear Connector	Strength of weld per connector (kN)	Ultimate Load per connector (kN)
I	Heated Stud	23.06	72.87
II	Stud without head	23.06	70.125
III	Bar	101.15	165.9
IV	Channel	107.96	134.7

CHAPTER - 6

CONCLUSIONS

GENERAL

This chapter covers the conclusions of the present experimental research work and the scope for further work related to it.

6.1. CONLUSIONS

Based upon the result shown in relevant chapters, it can be concluded that.

1. Stud without head shear connector may be capable of transmit the shear load but lack of holding down the concrete slab and leads to uplift of the slab.
2. In case of headed stud connector the root of the studs transmits the horizontal shear while the head provides anchorage against uplift so gives better shear resistance capacity as well as better resistance to uplift.
3. Flexible connectors such as studs and channel connectors, in each of these cases the shear transfer occurs initially in the vicinity of the weld and the remainder of the connector (the head of the stud and the top flange of the channel) provides anchorage against uplift.
4. Flexible connectors derive their resistance through the bending and normally failure occurs when the yield stress in the concrete is exceeded resulting in slip between the steel beam and the concrete slab.
5. In case of bar connector (rigid connector) slips were considerably smaller and there was a much greater range of concrete strength.
6. In case of bar connector (rigid connector) failure or slip is generally associated with the crushing of concrete and to prevent the separation of the in-situ slab from the prefabricated unit in the direction perpendicular to the contact (uplift), some mechanical devices must be provided along with these connectors.
7. Slip in composite specimen occurs partly due to deformation of concrete in the region of shear connector and partly due to deformation of shear connector.

6.2. FUTURE SCOPE OF WORK

Future scope of work covers :

1. A range of steel-concrete composite beams connected by means of three major types of shear connectors i.e. Rigid, Flexible, Bond or Anchorage Connectors.
2. Effect of fatigue loading on composite specimen.
3. Behaviour of composite columns.

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