SHEAR RESISTANCE OF REINFORCED CONCRETE BEAM; A CASE OF WELDED WIRE MESH AS STIRRUPS

By

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A thesis submitted to the Post-graduate School, Ahmadu Bello University, Zaria, in partial fulfilment of the requirements for the degree of M.Sc. Civil Engineering Construction Materials in the Department of Civil Engineering, Faculty of Engineering, Ahmadu Bello University, Zaria, Nigeria.

July, 1989

DECLARATION

I hereby declare that this thesis has been prepared by me and that it is a record of my own research work. It has not been presented in any previous application for a higher degree.

All quotations are indicated and sources of information are particularly acknowledged by means of references.

NNYAGU, Emmanuel Chinweuba

July, 1989.

CERTIFICATION

This thesis entitled:

SHEAR RESISTANCE OF REINFORCED CONCRETE BEAM; A CASE OF WELDED WIRE MESH AS STIRRUPS by NNYAGU, Emmanuel Chinweuba meets the regulation governing the award of the degree of M.Sc. Civil Engineering Construction Materials of Ahmadu Bello University, Zaria and is approved for its contribution to knowledge and literary presentation.

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DEDICATION

This thesis is dedicated to my mother, brothers and sisters for their moral and financial buttress over the period.

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ABSTRACT

The ordinary U-stirrups have always been used especially in beams as web reinforcement in most modern construction. As a probable substitute, here wire wesh of 3mm diameter is used to reinforce beams in shear. The investigation thus involves testing of beams reinforced in shear separately with the ordinary U-stirrups and with wire mesh. The beams are all the same with the only difference being in the web reinforcement. Apart from the mesh being easier to fabricate, the analysis of the results obtained shows that beam with the ordinary U-stirrups recorded more strains in the concrete than those with wire mesh. Moreover, crack control is better in the beams with mesh as their formation and propagation was more gradual leading to a gradual failure of the beams. However, it is recommended that a basic formular for calculating the theoritical shear resistance of beams involving mesh be deviced to include the influence of the horizontal members of the mesh on the shear resistance.

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LIST OF SYMBOLS

A_s	Area of tension steel
A_{sv}	Area of Stirrup bar
a	Shear span
BM1	First beam with mesh on grid 50mm x50mm
BM2	Second beam with mesh on grid 50mmx50mm
BM3	First beam with ordinary U-stirrups
BM4	Second beam with ordinary U-stirrups
BM5	First beam with mesh on grid 100mmx100mm
BM6	Second beam with mesh on grid 100mmx100mm
b	breadth of beam
d	Effective depth of beam
f_{cu}	Characteristic strength of concrete
f_y	Yield strength of tension steel
f_{yv}	Yield strength of stirrup
M	Bending moment
M_u	Maximum moment of resistance
P	Applied load
S_v	Spacing of stirrup
V	Shear force
V_{ay}	Interface shear force
V_{cz}	Shear carried by the Compression zone
V_d	Dowel Shear force
V_R	Shear resistance

V_s	Force in the Stirrup
V_T	Reaction at the Support
v	shear stress of beams
v_c	shear carried by concrete
v_{max}	Maximum shear stress in beams
W,W,Y&Z	Points of insertion of dial gauges for measuring deflection.

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CHAPTER ONE

1.0 INTRODUCTION

1.1 BACKGROUND INFORMATION

It has been the practice that in the design of concrete structures, an adequate margin of safety must be provided against any mode of failure that might occur under the forces that act upon the structure during its lifetime. One general type of failure that must be prevented is the shear failure. This is a failure under combined shearing forces and bending moment plus occasionally torsion and is characterised by small deflections and lack of ductility. In beams they exist where there is a change in bending moment along the length of the member. The shear force itself being equal to the rate of change of bending moment. Shear failures generally reduce the strength of structural elements below the flexural capacity and considerably reduce the durability of the elements. Especially for the later reason, shear failures are generally considered undesirable.

Shear failures of beams are usually characterised by the occurence of inclined cracks. In some cases, inclined cracking is immediately followed by member failures and in other cases, the inclined cracks stabilise and substantially more shear force may be applied before the member fails. Usually the formations of inclined cracks in the web of a reinforced concrete beam changes the beam into a highly complex structural member in which loads are carried to the supports by an action similar in many respect to that of a truss. In this action the stirrups act as the tensile members of the truss while the concrete between the cracks act as the inclined compressive struts. This, of course, is the classical conception of the behaviour of a beam having inclined cracks, but, althrough a great simplification of complex mechanism, it is sufficiently accurate to form a basis of an understanding of the problem. The conception suggests that there are two possible sources of primary shear failure: either the stirrups may cause failure, e.g. by reaching their yield stress or the concrete between the cracks may crush owing to the inclined compressive stress. This thesis is however concerned with the failure due to the type of stirrups and studies mainly the shear resistance of beam with welded wire mesh as web reinforcement, comparing it at the same time with the use of ordinary conventional U-stirrups.

Here effort was geared towards examining the strains in the concrete, the crack development and pattern as well as the deflection of the beam as the load increase progresses. This gives room for effective comparison between the two types of shear reinforcement.

The arrangement for investigating shear failure is as shown in Fig. 1.1 below. It has the advantage of combining two different test conditions namely; pure bending, that is bending without shear force as present between the two loads, P, and then constant shear force in the two end sections.

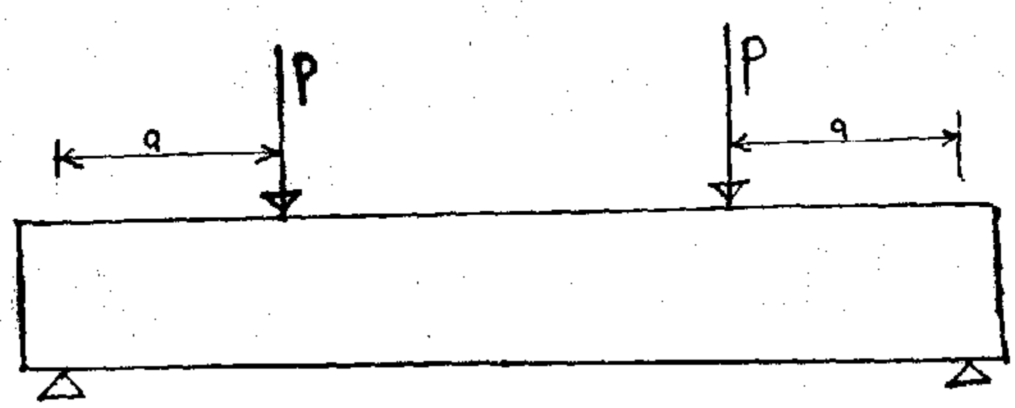


Fig. 1.1 : Arrangement for Investigating Shear Failure.

Since the beams will be slender, their behaviour will be same as the behaviour of a slender beam subjected to a gradually increasing load with the first crack appearing long before the allowable load is reached. These cracks are usually narrow. Under further loading, the cracks increase both in width and length indicating that the area of the compressive zone decreases. This effect is especially visible when the stress in the steel reaches and exceeds the yield point stress.

In the ensuring tests however, care will be exercised to keep the shear span constant throughout. This vividly affords a clear basis for comparison since the strains in the concrete as well as the cracks will all be measured within the shear span.

1.2 OBJECTIVE OF THIS REPORT

In almost all the concrete structures designed and constructed in the country today, the web reinforcement has been the U-type or rectangular stirrup which is either made of mild steel or high yield steel as the case may be. The cost of these steel in recent time has become alarming. The main aim of this report thus is to study the shear resistance of a beam using welded wire mesh as the web reinforcement and compare it with the use of ordinary conventional U-stirrups. This comparision will bring to light the one that offers better shear resistance and possibly a better and lasting substitute to the conventional U-stirrup.

1.3 SCOPE OF WORK

As aforementioned, this work involves the testing of beams reinforced in shear with ordinary U-stirrup and wire mesh respectively. The type one shear reinforcement is as shown in Fig. 1.2 and involves the orinary U-stirrups. This acts as the base sample. The second type of shear reinforcement is as shown in Fig. 1.3 and involves the use of wire mesh. Fig. 1.3a shows mesh of 3mm diameter and 50mm x 50mm spacing while Fig. 1.3b shows mesh of 3mm diameter on grid 100mm x 100mm.

In the two considerations, the following will be kept constant;

(i) Beam dimensions

(ii) The main bars

(iii) The concrete mix

(iv) The magnitude as well as rate of loading

(v) The shear span, a.

Thus, the only variable will be the shear reinforcement. While using ordinary U-stirrup in Type one, wire mesh will be used in Type two. Two sets of wire mesh of same diameter but different spacing will be used as indicated in Fig. 1.3. This helps to widen the validity of the results obtained and further helps to infer whether the spacing of the mesh has any influence on the result.

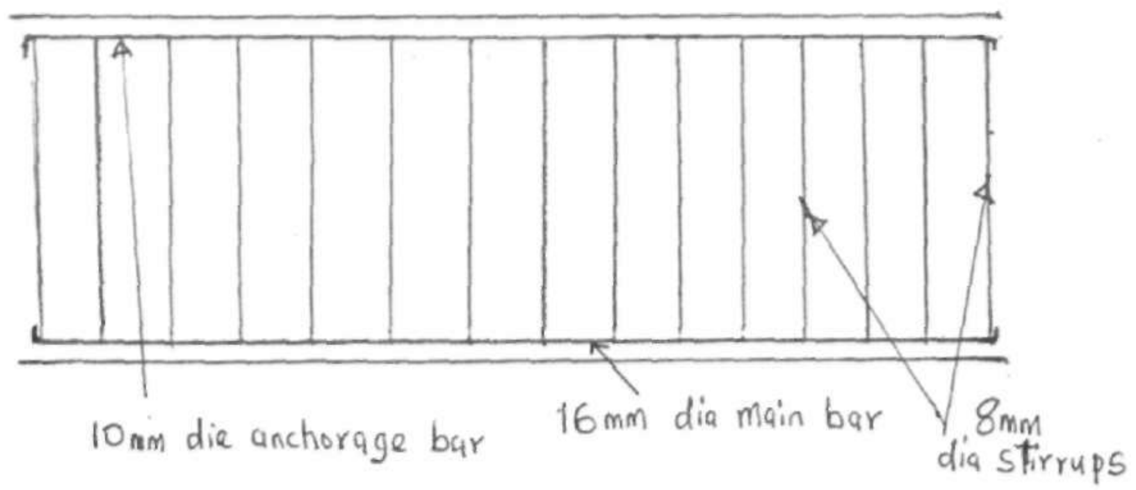
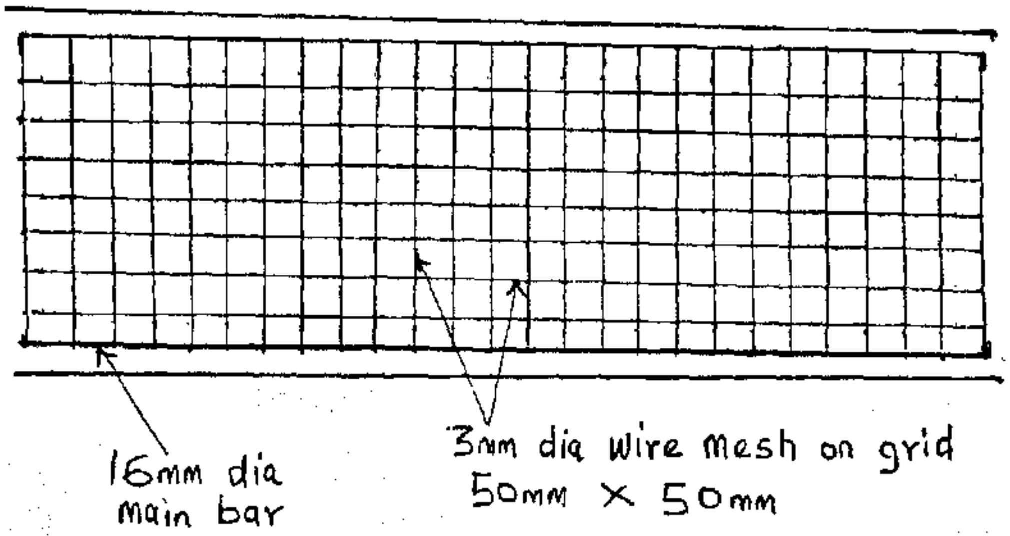
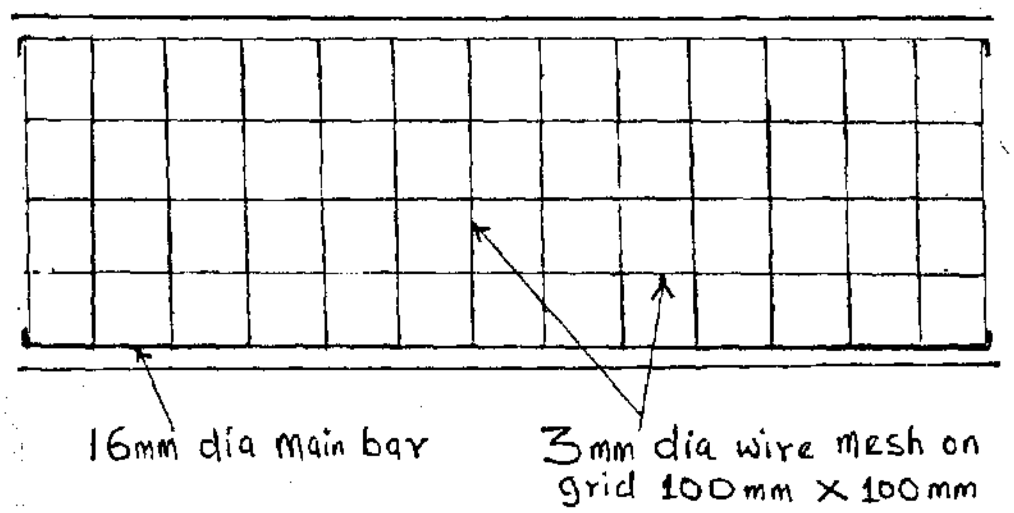


Fig.1.2: Type One Shear Reinforcement with U-stirrups.



(a) Shear reinforcement with 3mm diam mesh on grid 50mm x 50mm.



(b) Shear i reinforcement with 3mm dia mesh on grid 100mmx100mm

Fig. 1.3 Type two shear reinforcement with wire mesh

CHAPTER TWO

2.0 LITERATURE REVIEW

Shear failure has been reported in beams, columns, slabs, brackets and other members. In general, each type of member exhibits different modes of cracking and failure, although the mechanisms by which shear is transferred within the member may be similar.

Reinforced concrete and prestressed concrete beams of moderate slenderness (a/d or M/V_d = 2-6) develop inclined cracks due to the combination of shear and flexural stresses. Beams may exhibit a number of different modes of shear failure, the most common being the crushing or shearing of the compression flange over the inclined crack which is often accompanied by splitting along the tension reinforcement.

Shearing failures of deep beams, brackets and similar members differ considerably from those in normal beams. This largely due to the much steeper inclined cracks in deep members. This in turn changes the relative importance of the various shear transfer mechanisms as compared to normal beams.

Generally, the understanding and knowledge of the shear transfer mechanisms in various structural members has progressed significantly. In reinforced concrete members, shear has been discovered to be transmitted from one plane to another in various ways. The behaviour, including the failure modes depends on the method of shear transmission. Normally the main types of shear transfer include;

(a) Shear in the uncracked concrete

(b) Interface shear transfer (usually referred to as aggregate interlock).

(c) Dowel action

(d) Arch action and

(e) Shear reinforcement

These mechanisms occur to widely varying extents in various types of structural elements. Fig. 2.1 shows the forces in a reinforced concrete beam with diagonal crack. (Journal of Strucural Div. ASCE 1973)

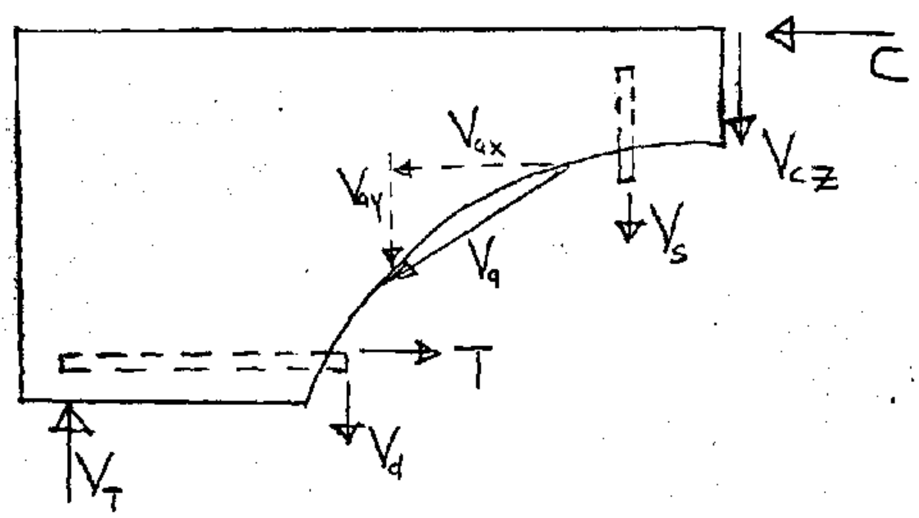


Fig. 2.1 Forces acting at Inclined Crack.

The longitudinal forces T and C, are related to the flexural resistance of the member. The forces along the diagonal tension crack, V_{ax} and V_{ay} are due to interface shear transfer. The V_s and V_d forces are the forces in the stirrups and the dowel shear carried by the longitudinal steel respectively and V_{cz} equals the shear carried by the compression zone. Thus;

$$V_T = V_s + V_{cz} + V_d + V_{ay}$$

The internal shearing force components at a crack such as the one shown in Fig. 2.1 are related to the applied shear in Fig. 2.2.

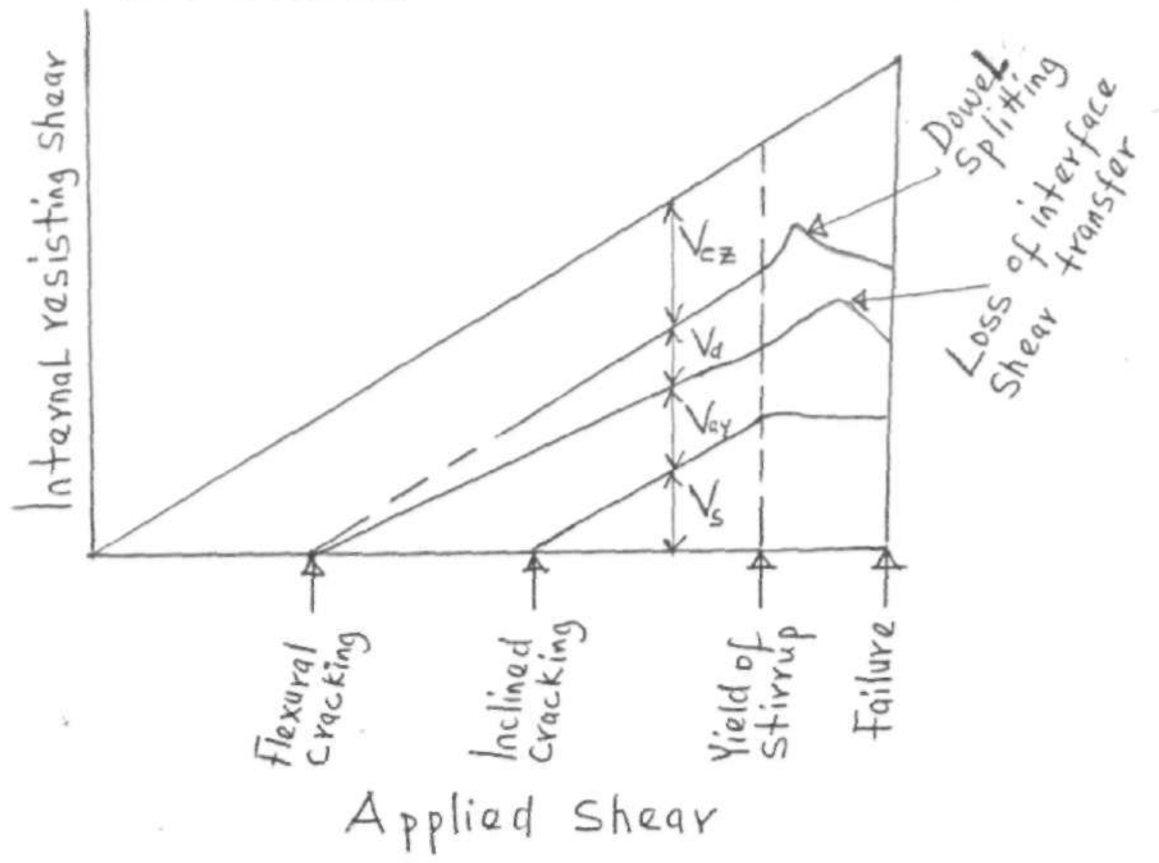


Fig. 2.2: Distribution of Internal shears in Beam with Web reinforcement.

Prior to flexural cracking all the shear is carried by the uncracked concrete. Between flexural and inclined cracking, the external shear is resisted by concrete, V_{cz}; the interface shear transfer, V_{ay}; and by dowel action, V_d. Following the formation of inclined cracks, a portion of the shear is carried by the web reinforcement, V_s. When the shear carried by the web reinforcement can no longer increase due to its yielding, any additional shear must be carried by V_{cz}, V_d and V_{ay}. As the inclined cracks widen the interface shear transfer, V_{ay}, decreases, forcing V_d and V_{cz} to increase at an accelerated rate until either splitting (dowel) failure occurs or the compression zone fails due to combined shear and compression.

As evident from the foregoing, shear failures of beams is characterised by the occurrence of inclined cracks. Inclined cracks in the web of a beam may develop either before a flexural crack occurs in their vicinity or as an extension of a previously developed flexural crack. This first type of crack is referred to as "web-shear crack" (Fig. 2.3a). The second type is often identified as a "flexure shear crack" and the flexural crack causing the inclined crack is referred to as the "initiating flexural crack (Fig.2.3b).

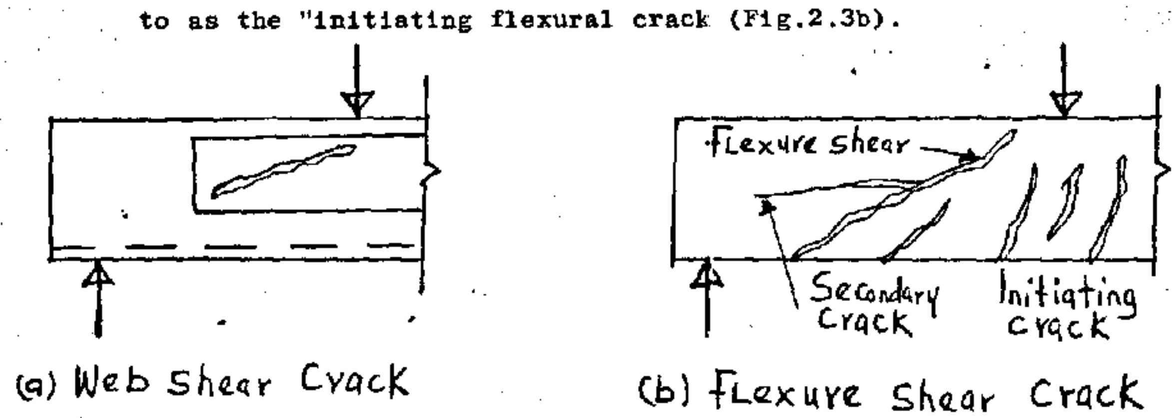


Fig. 2.3: Types of Inclined Cracks

In addition to the primary cracks, secondary cracks often result from splitting forces developed by the deformed bars when slip between concrete and steel reinforcement occurs, or from dowel action forces in the longitudinal bars transferring shear across the cracks. The manner in which cracks develop and grow and the type of failure that subsequently develops is strongly affected by the relative magnitudes of shearing stress, v, and flexural stress f_x. As a first approximation, these stresses are defined as

$$v = \alpha_1 \frac{V}{bwd} \quad \text{and} \quad f_x = \alpha_2 \frac{M}{bd^2}$$

where α_1 and α_2 are coefficients depending on several variables including the goemetry of the beam, the type of loading, the amount and arrangement of reinforcement, the type of steel and the interaction between steel and concrete. The values V and M are the shear and moment at a given section, respectively; b = width of the flange; bw = the width of the web; and d = the effective depth of the beam.

Flexure - shear crack of the type shown in Fig.2.3b form in a part of beam already cracked due to flexure. Although these cracks are the most common type of crack observed in reinforced and prestressed concrete beams, the mechanism by which they form is not yet entirely understood. In a beam containing flexural cracks, the variation in steel stress from crack to crack sets up forces on the cantilever elements or "teeth" between the flexural cracks which tend to cause bending and shearing deflections of the teeth. The internal mechanism of such a beam nearing flexural failure may be seen in Fig. 2.4. The formation of flexural cracks has transformed the reinforced concrete beam into a comb-like structure: the compression zone of the beam is the backbone of the comb, while in the tension zone the "concrete teeth" separated from each other by flexural cracks, represent the teeth of the comb. As the teeth deflect prior to inclined cracking, the dowelling forces and aggregate interlock stresses developed between the teeth tend to prevent relative deflections of the teeth, thus delaying, the formation of further cracks. The rate of propagation of the flexural cracks and the stage at which the inclined cracks develop in a given beam is a function of the magnitudes of the flexural and shearing stresses at the head of the crack. These in turn, are a function of the M/Vd, the height of the flexural crack and the amount of shear transferred by interface shear transfer and dowelling.

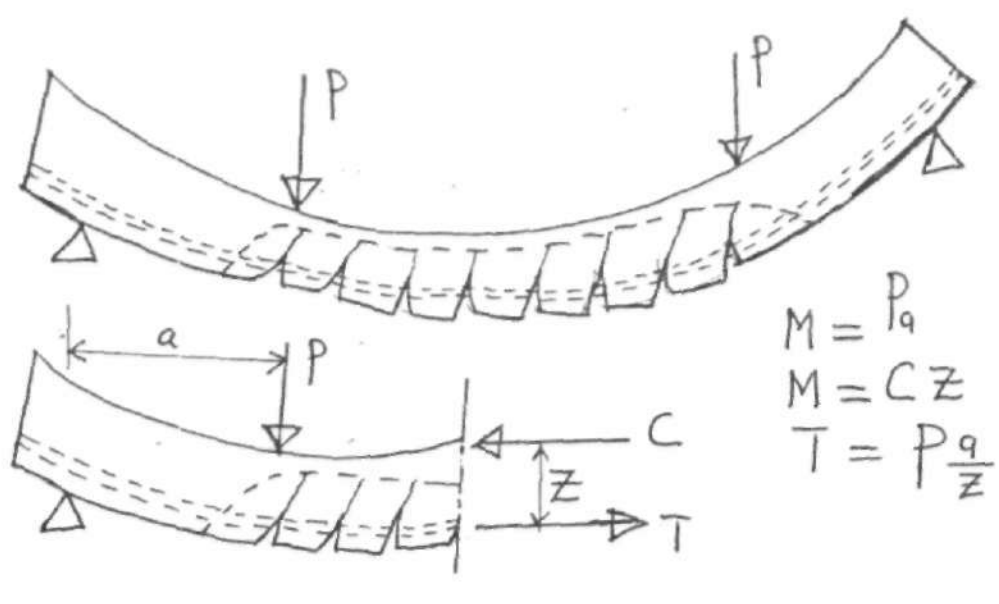


Fig.2.4: Mechanism of flexural failure

Tests of rectangular beams carried out by Taylor (1971) shows that approximately one third of the total applied shear was transfered by the compression zone above flexural cracks in the shear spans prior to the formation of inclined cracks. The balance was presumably transfered by interface shear transfer and dowel action. These measurements suggest that a significant shear stress exist at the upper ends of flexural cracks in the shear spans. Broms (1964) has shown too that the non-uniform stresses between cracks lead to secondary shear stress which are particularly severe near the top of flexural cracks in the shear spans. These secondary stresses combined with the stresses necessary to balance the applied loads and cause major diagonal principal tensile stresses at the head of the flexural cracks.

A number of experimental investigations have also been carried out in beams without stirrups to assess the relative magnitude of forces shown in Fig. 2.1. Achara and Kemp (1965) show that only 40% of the total shear on a section can be carried by the compression zone if the stress conditions at the head of the crack are to be compatible with an acceptable failure criterior for concrete subjected to shear and normal force.

Fenwick and Paulay (1968), Taylor (1970) and Mattock, et al (1972), among others carried out tests on interface shear. Particularly, Gergely (1969) conducted tests in which interface shear transfer was eliminated by performing smooth sided cracks in beams. These tests and those of

Taylor indicate that between 33% to 50% of the total force on a beam may be carried by interface shear transfer.

However, the ACI procedure for designing stirrups assumes that the shear not carried by concrete after inclined cracking is resisted by stirrups or bent bars acting essentially according to the Truss analogy. This anology implies that failure is prevented by the transverse reinforcement steel bars and the compression between the diagonal cracks form a truss which resists further loads. Failure occurs when the compression diagonals crush under the combination of axial and shear forces. It is thus on this vein that the ACI Building Code (1971) requires a minimum area of web reinforcement in most members when the shear strength of beams, v_u exceeds half the allowable shear carried by concrete v_c. Studies by Bresler and Scordelis (1964), Haddalin and Hong (1971) and Hernandex (1958) have shown that a small amount of web reinforcement have significantly larger effect on shear strength. Infact web reinforcement has three primary effects on the strength of a beam namely,

a) It carries part of the shear, v_s.

b) It resistricts the growth of diagonal tension crack width thereby helps in maintaining interface shear transfer, V_{ay}

c) It holds the longitudinal bars and increase their dowel capacity, V_d.

In addition to these, stirrups may transfer small force across the crack by dowel action and tend to enhance the strength of the compression zone by contining the concrete.

Research work by Kani and Gergely (1969) show that isolated stirrups at the bottom of diagonal cracks would prevent shear failure in most cases, but of course the position of the cracks is not known in design. However, their tests indicate that the primary role of web reinforcements is in the restriction of the width of diagonal cracks.

Tests by Leonhardt and Walther (1964) indicates that beams with bent up bars as shear reinforcement tended to have lower shear strength and much wider cracks than similar beams with stirrups for several reasons;

a) they tend to cause longitudinal cracking or crushing at the bend points

b) they do not confine the concrete in the shear region and

c) they are less efficient in tying the compression flange and web together.

Due to these, the CEB Recommendations (1970) require the use of vertical stirrups in addition to bent up bars. This problem is also discussed by Robinson (1965). The ACI Building Code (1971) also limits the shear that can be carried by bent bars.

It has also been established that major inclined cracks frequently develop near the ends of reinforcing bars cut-off in zone of tension. Adjacent to the cut-off point the stresses and deformation are decreasing in the cut-off bars and increasing in the remaining bars. This, combined with the eccentric pull in the cut-off bars lead to a state of high shearing and diagonal tensile stresses in the vicinity of the cut-off. Baron (1966) has showntthat changes in the moment arm in this region lead to increase shears in the cut off zone. Also tests by Fergusion and Matloob (1959), Baron (1965), Scordelis and Bresler (1964) show that;

(a) Cut off bars in the tension zone will lower the shear strength of beams

(b) Closely spaced stirrups in the cut off zone will prevent premature failure.

Some tests were also carried out to determine the effect of spacing and bar size on the shear strength of beams. In the tests carried out in Stutggart (1965), welded wire mesh was employed. The principal conclusions of this test were;

a) Mats of 50mm by 100mm spacing of stirrups bars were best with respect to crack width and compresses stresses in the web.

b) An equivalent yield strength of $420 Nmm^{-2}$ can be used for load factor design.

The commentary to the AC1 Building Code (1971) provides similar anchorage details for welded wire mesh stirrups.

In some cases, it is convenient to use pairs of U-shaped stirrups lapped in the web of the beam to form a rectangular stirrup. In such a stirrup, the lap occurs in a region of diagonal cracking. The tests on this type of stirrup carried out by Hanson and Hulsbos (1971) shows that the capacities of the stirrups are a function of the lap length, stirrup diameter and cover. Allen and Huggin (1964) suggested that a lapped stirrup will have the same capacity as a rectangular stirrup if the lap is long enough to develop the yield strength of the stirrup bars.

Generally, the foregoing review work have been confined to the shear theory and the various web reinforcements in concrete beams and the development of shears with the use of any particular web reinforcement. Comparison work (mainly between the ordinary U-stirrup and wire mesh) has not been done possibly to determine the one that offers greater shear resistance at the same loads. Both the U-stirrups and wire mesh are all web reinforcement materials as we know. A comparison of the shear resistance of each of them under the same load is therefore possible. Of course, this work provides a forum for such a comparison.

CHAPTER THREE

3.0 EXPERIMENTAL PREPARATIONS AND TESTING

This deals with the experimental investigations which covers the materials preparations as well as the testing itself.

3.1 MATERIALS

The materials referred here includes aggregates, cement, water and the reinforcements.

3.1.1 Aggregates

The aggregates used for this investigation comprised coarse and fine aggregates. The coarse aggregate used was got from a batch of crushed rock (gravel). Due to the congested nature of the reinforcements and to facilitate effective compaction, aggregates with a maximum nominal size of 15mm were used. Care was taken to eliminate silt from the coarse aggrates by sieving them out, but at the same time making sure that the very fine stones were not eliminated since an all-in-aggregate was required.

The fine aggregates were also sieved to eliminate those portions with nominal size more than 4.76mm.

In all, both the coarse and fine aggregates were sieved and stored separately in a dry cubicle within the concrete laboratory. This was done so as to afford them time to dry since being a bit wet will affect the already calculated water-cement ration.

3.1.2 CEMENT

The cement used in this investigation was a locally made ordinary portland cement. They were stored in a dry environment also within the concrete laboratory so that the qualities would remain intact. Owing to the fact that cube strength tests would be carried out, there thus became no need to carry out separate quality tests on the cement.

3.1.3 WATER

The water used here was tap water available in the laboratory. Naturally the water has been treated in the station and was consequently free from dirt and other contaminations.

3.1.4 REINFORCEMENT

The main bars were all 16mm diameter high yield bars while those used for the ordinary U-stirrup were mild steel bars of 8mm diameter. Both the 50mm x 50mm and 100mm x 100mm mesh were all 3mm in diameter. However, test carried out later indicated that the tensile strength of that on grid 100mm x 100mm was greater than that of 50mm x 50mm.

To check the characteristic strength used in calculation, all the reinforcement were subjected to tensile tests. For each particular reinforcement, three specimen of 500mm long were cut and tested to failure. The universal testing machine was used to this purpose. The tensile strength and ultimate strength for each bar were got from the yield load and ultimate failure load observed from this test. The tabulated results are as shown below.

Table 3.1 Results of the tensile strength of both the 16mm & 8mm diameter a bars

Nominal dia(mm)	Effective Cross-Sectional Area (mm^2)	Yield Load (kN)	Yield Stress (Nmm^{-2})	Ultimate Load (kN)	Ultimate Stress (Nmm^{-2})
16	201	84.5	420.1	114.0	567.2
16	201	84.0	417.6	110.0	547.3
16	201	81.0	403.0	110.0	547.3
8	50	15.5	310.0	22.5	450.0
8	50	15.2	304.0	22.1	442.0
8	50	14.9	298.0	22.1	442.0

Average Yield Stress (Nmm^{-2})

16	413.5
8	304.0

Table 3.2: *RESULTS OF THE TENSILE STRENGTH OF WIRE MESH*

FOR MESH ON GRID 50mm x 50mm

Nominal dia(mm)	Effective Cross-Sectional Area (mm^2)	Yield Load (kN)	Yield Stress (Nmm^{-2})	Ultimate Load (kN)	Ultimate Stress (Nmm^{-2})
3	7	2.5	357.1	4.5	642.9
3	7	2.5	357.1	5.2	742.9
3	7	3.0	428.6	5.3	757.1

Average Yield Stress = $380.9 Nmm^{-2}$

FOR MESH ON GRID 100mm x 100mm

3	7	12	1714.3	13.4	1914.3
3	7	10.2	1457.1	16.4	2342.9
3	7	12.5	1785.7	13.1	1871.4

Average Yield Stress = $1652.4 Nmm^{-2}$

3.2 DETAILS OF SPECIMEN

The beam section chosen for this test lies within the range of a normal beam. Incidentally at the time of preparation for this test such a formwork of dimensions 2750 x 160 x 275 mm^3 was available in the laboratory. Consequently, this was chosen as the section dimensions and used for all the beams cast.

The various bar sizes necessary for the specimens were measured and cut to the required lengths. The wire mesh which hitherto was still in sheet were then cut to the size of the beam and welded properly to the main bars. The other shear links were also cut into their respective shapes.

There were six beams in all. Two of which have the 8mm diameter link, another two have wire mesh of 50mm x 50mm grid as shear reinforcement while the rest two have wire mesh of 100mm x 100mm grid as the shear reinforcement. This was so in order to widen the validity of the results.

Thus, the various reinforcement cages for these six beams were assembled and kept ready for casting.

3.3 DESIGN OF THE TEST SPECIMEN REINFORCEMENT

The design of the reinforcement was done to CP110.

The dimensions of the section is 2750 x 160 x 275 mm^3

To avoid failure due to bending since failure due to pure shear is required, a shear span of 500mm at each end of the beam was chosen. This is as shown in Fig. 3.1 below.

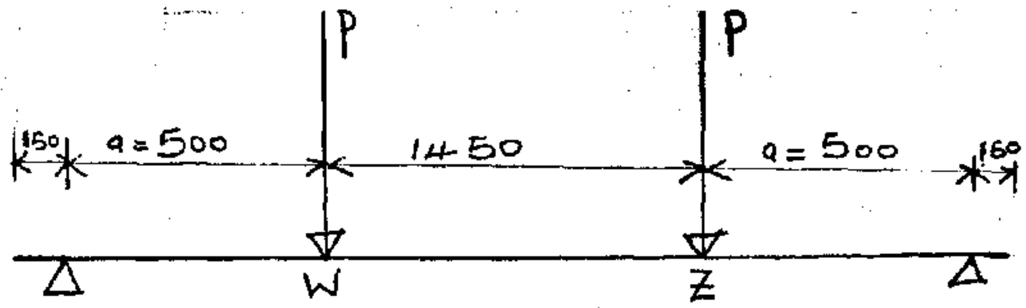


Fig. 3.1 Arrangement showing the shear span.

The concrete cover used in this investigation was 20mm, and the main bars were of 16mm dia.

Thus, the effective depth, d = 275 - 20 - 8

= 247mm.

$$\text{Shear Span/Effective depth} = \frac{500}{247} = 2.02$$

This ensures that there will be shear failure within the Shear Span.

3.3.1 DESIGN OF MAIN BARS

In the design of the main bars, the following were used;

Concrete cover = 20mm

Characteristic yield stress of steel, $f_y = 410Nmm^{-2}$

Characteristic concrete strength, $f_{cu} = 30Nmm^{-2}$

To compute the maximum moment of resistance of the section, M_u we make use of Equation 2 of CP110,

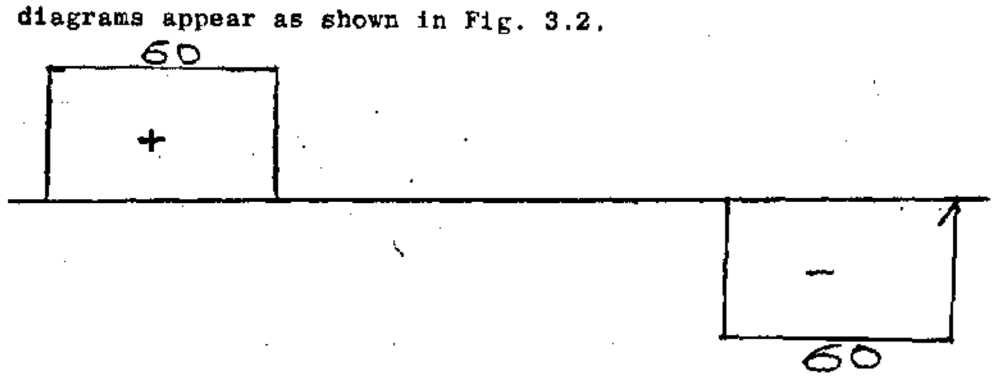
i.e $M_u = 0.15\ f_{cu}\ bd^2$ ... ... (1)

From equ. 1 therefore,

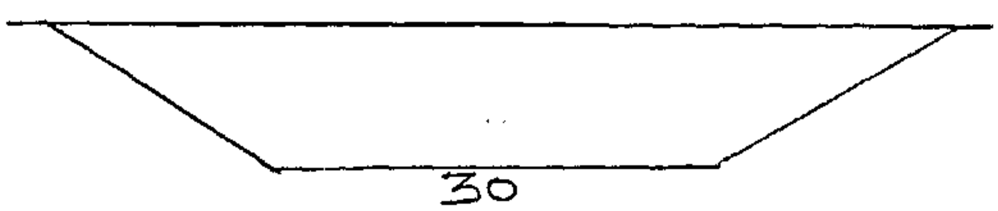
$M_u = 0.15 \times 30 \times 160 \times (247)^2 \times 10^{-6}$

$= 43.9$kNm.

Assuming a maximum force of 60kN each applied at points M and N as P in Fig. 3.1, then both the shear force and bending moment diagrams appear as shown in Fig. 3.2.



(a) Shear Force Diagram



(b) Bending Moment diagram

Fig. 3.2: Diagrams of Shear force and Bending Moment.

From Fig. 3.2,

the shear force = 60kN

and bending moment = 30kNm

But from equ. (1), M_u = 43.9kNm

Therefore beam will not fail by crushing of concrete.

Using a special case of Equ. 1 (CP110) with $Z = \frac{3d}{4}$ we have

$$M_u = (0.87\ f_y)\ A_s\ \frac{3.d}{4} \qquad \ldots \qquad (2)$$

This is to ensure the use of sufficient steel so that the beam will be able to resist a bending moment equal to M_u.

Re-arranging this, we have

$$A_s = \frac{M_u \times 4}{0.87\ f_y\ 3d} \qquad \ldots \qquad \ldots \qquad (3)$$

$$A_s = \frac{30 \times 10^6 \times 4}{0.87 \times 410 \times 3 \times 247}$$

$$= 454.00\text{mm}^2$$

Use 3 - 16mm diameter = 603mm^2 (ρ = 1.53%)

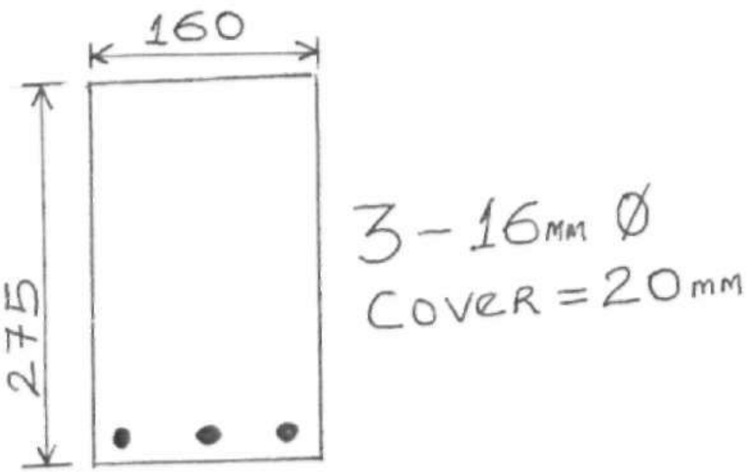


Fig. 3.3 Section showing the main bars

3.3.2 DESIGN OF SHEAR REINFORCEMENT.

(a) Design of the ordinary conventional U-Stirrups.

From the shear force diagram, V = 60kN.

$$\frac{100A_s}{bd} = \frac{100 \times 603}{160 \times 247} = 1.53\%$$

From table 5 (CP110), using a concrete grade, of 30Nmm^{-2}

$v_c = 0.80$

Also from Table 6 CP110

$v_{max} = 4.10N_{mm}^{-2}$

Using Equ. 8 (CP110),

$$v = \frac{V}{bd} \quad \ldots \quad \ldots \qquad (4)$$

Substituting in equ. 4 above

$$v = \frac{60 \times 10^3}{160 \times 247} = 1.52 Nmm^{-2}$$

$0.80 < 1.52 < 4.10$

Thus, since the shear stress, v, calculated from Equation 4 above is greater than the ultimate shear stress in concrete, v_c, but less than the maximum stress, v_{max}, then shear reinforcement will be provided.

Assuming 8mm diameter links, A_{sv} = $101mm^2$ for 2 legs and

f_{yv} = 250 Nmm^{-2},

Rearranging Equ. 9 (CP110), we have

$$S_v = \frac{A_{sv} \times 0.87 f_{yv}}{B(v - v_c)} \quad \ldots \quad \ldots \quad (5)$$

Substituting in Eqn. 5 above, we have;

$$S_v = \frac{101 \times 0.87 \times 250}{160 \ (1.52 - 0.80)}$$

$$= \frac{101 \times 0.87 \times 250}{160 \times 0.72}$$

$$= 190.69mm$$

Use 8mm diameter links @ 190mm pitch.

In order to facilitate proper anchorage of the reinforcement, 2 - 10mm diameter achorage bars were provided at the top of the beams. The arrangement is as shown in Fig. 3.4.

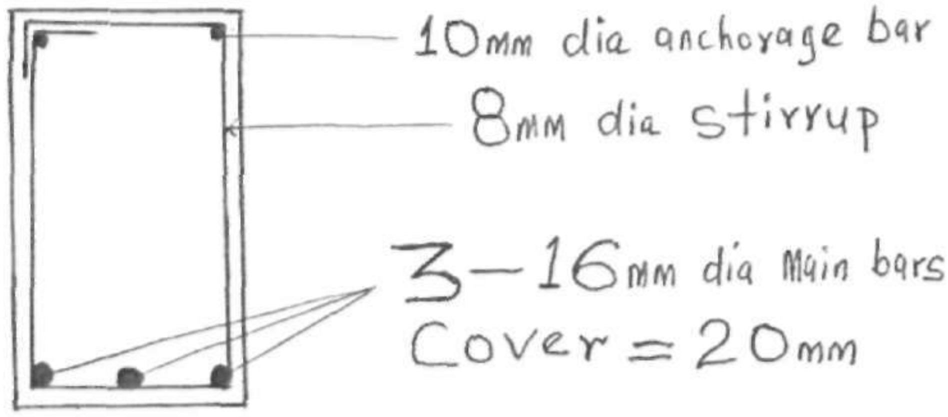


Fig. 3.4: Section showing the arrangement of links and main bars.

b) In case of the wire mesh, no actual design was done for them. This is due mainly to the nature of mesh since they are tiny and already formed into a grid. What was done was to cut them out from the sheet to the size of the beam and then properly welded to the main bars and anchorage bars as shown in the pictorial view evident in Fig. 1.3.

3.4 CALCULATION OF THE THEORETICAL SHEAR RESISTANCES OF THE BEAMS

Generally for a beam containing tension reinforcement as well as vertical links, the shear resistance is given by:

Total resistance $V_R = V_{concrete} + V_{links}$

$$\text{i.e. } V_R = V_c + V_s \qquad \ldots \quad \ldots \quad \ldots \qquad (6)$$

But $V_c = v_c bd$ and

$$V_s = \frac{d}{S_v} \cdot A_{sv} \cdot \frac{f_{yv}}{1.15}$$

Equation (6) becomes

$$V_R = v_c bd + \frac{d}{S_v} \cdot A_{sv} \cdot \frac{f_{yv}}{1.15} \qquad \ldots \quad (7)$$

(a) FOR THE BEAMS WITH ORDINARY U-STIRRUPS (i.e. BM3 and BM4).

$$\frac{100A_s}{bd} = \frac{100 \times 603}{160 \times 247} = 1.53$$

From Table 5(CP110) with concrete grade of 30, then

$v_c = 0.8$.

But from calculation using Equ. 5, $S_v = 190$mm
Also $A_{sv} = 101\text{mm}^2$ and $f_{yv} = 304\text{Nmm}^{-2}$

Thus, from Equ. 7, we have that

$$V_R = (0.8 \times 160 \times 247) + (\frac{247}{190} \times 101 \times \frac{304}{1.15})$$

$$= 31616 + 34708.9 = 66324.9\text{N}$$

Shear resistance, $V_R = 66.3$kN.

b) FOR THE BEAMS WITH MESH

In the calculation of the theoretical shear resistance of the beams with mesh as web reinforcement, only the vertical members of the mesh is considered here. This is because there is no laid down formular involving also the horizontal member of the mesh. Thus, here the horizontal distance between the vertical members is taken as the spacing of the links.

(i) FOR THE BEAMS WITH MESH ON GRID 50 x50mm (ie BM1 and BM2)

Diameter of mesh = 3mm

A_{sv} for 2 legs = 14.13mm^2

Here also, S_v = 50mm and f_{yv} = 380.9Nmm^{-2}

From Equ. (7) we have;

$$V_R = 31616 + (\frac{247}{50} \times 14.13 \times \frac{380.9}{1.15})$$

$$= 31616 + 23119.7$$

$$= 54735.7N$$

Shear resistance, V_R = 54.7kN.

ii) FOR THE BEAMS WITH MESH ON GRID 100x 100mm (ie BM5 and BM6)

Diameter of mesh = 3mm

A_{sv} for 2 legs = 14.13mm^2

But S_v = 100mm and f_{yv} = 1652.4 Nmm^{-2}

Substituting in Equ. 7, we have

$$V_R = 31616 + (\frac{247}{100} \times 14.13 \times \frac{1652.4}{1.15})$$

$$= 31616 + 50148.3$$

$$= 81764.3N$$

Shear resistance, V_R = 81.8kN

3.5 CONCRETE MIX

In this investigation, a standard normal mix of 1:2:4 was chosen. This is expected to achieve the required concrete strength of 30Nmm^{-2}, and is generally used in all normal constructions.

3.5.1 CALCULATION OF VARIOUS QUANTITIES FOR EACH BEAM

With this mix, the various quantities (cement, sand and stone) for each beam was calculated to ascertain their magnitude so as to avoid waste. Furthermore, for easier penetration and proper workability since a congested reinforcement especially in the beams with wire mesh is envisaged, a water-cement ratio of 0.6 was adopted. This will be ensured by carrying out some slump and compacting factor tests during the casting.

CALCULATION

Since the beam dimension is 2750 x 160 x 275

Volume of beam = 2750 x 160 x 27

$= 0.12m^3$

In each beam, five cubes and three cylinders will be cast to determine the concrete strength at 28 days.

Vol. of 5 cubes = 4 5 x 150 x 150 x 150

$= 0.01m^3$

Vol. of 3 cylinders $= 3 \times \pi \times (75)^2 \times 300$

$= 0.02m^3$

Total Vol. $= 0.15m^3$

Allow for bulk and waste 25%

$= \frac{25}{100} \times 0.15$

$= 0.04m^3$

Overall vol. = 0.15 + 0.04

$= 0.19m^3$

Since the mix proportion is 1:2:4, then,

Vol. of cement $= \frac{0.19}{7} \times 1$

$= 0.027m^3$

Vol. of sand $= \frac{0.19}{7} \times 2$

$= 0.054m^3$

Vol. of stone $= \frac{0.19}{7} \times 4$

$= 0.109m^3$

But the densities of cement, sand and stone are given as $3100kgm^{-3}$, $2650kgm^{-3}$ and $2650kgm^{-3}$ respectively.

wt. of cement = 0.027 x 3100 = 83.7kg

wt. of sand = 0.054 x 2650 = 143.1kg

wt. of stone = 0.109 x 2650 = 288.85kg

Since a w/c of 0.6 is to be adopted,

wt of water = 83.7 x 0.6 = 50.22kg

Summarily, the quantities are,

Cement	= 83.7kg
sand	= 143.1kg
stone	= 288.85 kg
water	= 50.22kg

3.6 CASTING OF THE SPECIMENS

Generally six beams were cast in all. Two of which represent one with the ordinary U-stirrup, the other two represent the one with wire mesh on grid 50mmx50mm while the remaining two represents the one with wire mesh on grid 100mm x 100mm. Each beam was cast with six cubes and three cylinders. In all only a beam and the corresponding cylinders and cubes were cast in a day. This is because only one formwork was available at the time of casting.

Each casting starts by cleaning the formwork and the moulds properly with mould oil so as to unenable the sticking of concrete to the walls of the formwork. The reinforcement cage was then placed in the formwork making sure that the reguired concrete cover is maintained. Thereafter, the formwork and the three cylinders and six cubes were placed on the vibrating table in order to ensure an even distribution of the concrete mass and consequently effect a sound vibration.

Owing to the fact that the volume of the mixing drum is less than the volume of each beam, the mixing and batching of each was done in three stages. The aggregates and cement were first weighed on the weighing balance. This was then transferred to the mixing machine for dry-mixing by an open rotating drum driven by an electric motor. The needed quantity of water was then, weighed and added. The mixing then continued until a uniform mix was got. Each mass of the uniformly mixed concrete was then placed in the formwork and cubes and cylinders making sure there is an even distribution each time. The pouring of each concrete batch was followed by the vibration of the mass by setting the vibration table on for about a minute. This affords adequate compaction, thereby reducing the air voids.

The specimens were cast in this order;

(i) BM.1 representing the first beam with mesh on grid 50mm x 50mm

(ii) BM. 2 representing the second beam with mesh on grid 50mm x 50mm.

(iii) BM.3 representing the first beam with ordinary U-stirrup.

(iv) BM.4 representing the second beam with ordinary U-stirrup.

(v) BM.5 representing the first beam with mesh on grid 100mm x 100mm.

(vi) BM.6 representing the second beam with mesh on grid 100mm x 100mm.

It must be noted that some slump tests and compacting factor tests were carried out during the casting of the specimens to ascertain the workability and compaction of the mix. The results are shown below.

TEST 1: CARRIED OUT DURING THE CASTING OF BM.1

The slump obtained was 110mm

Compacting factor equals 0.96

TEST 2: CARRIED OUT DURING THE CASTING OF BM.3

The results obtained were;

Slump = 105mm

Compacting factor = 0.96

TEST 3: CARRIED OUT DURING THE CASTING OF BM. 6

The tests also gave the following result;

Slump = 110mm

Compacting factor = 0.96

These tests carried out show that the concrete has a high degree of workability and consequently suits the congested reinforcement especially for the beams with wire mesh.

3.7 METHOD OF CURING

Each specimen after casting was left in the mould for a day before the forms were removed. The specimen was then transferred to the curing room within the concrete laboratory. All the test specimens and the accompanying cubes and cylinders were cured under the same condition. This is to facilitate uniform curing and ensure that properties obtained from the cubes were typically representative of the concrete of the specimen. This curing was achieved by spraying water on the specimen two times a day for 28 days. The curing was a surface curing.

3..8 PREPARATION OF THE BEAMS FOR TESTING

After curing the specimens for 28 days, they were brought out from the curing room and carefully painted with white paint. This was done so that the crack appearance and propagation will easily be seen and monitored.

Since straing gauges were not available at the time of execution of this work, griding of the shear spans was then done. This was done so that extensometer would be used to determine the strains in concrete within the shear spans. The griding was done taking the base of the extensometer into consideration and at the same time making sure that the envisaged lines of major cracks were covered. A 4 inches base extensometer was used here. Demec points were then placed at the various points of the grid using glue to fix them rigidly to the concrete. The griding on both the left and right side of the beams is as shown in fig. 3.5.

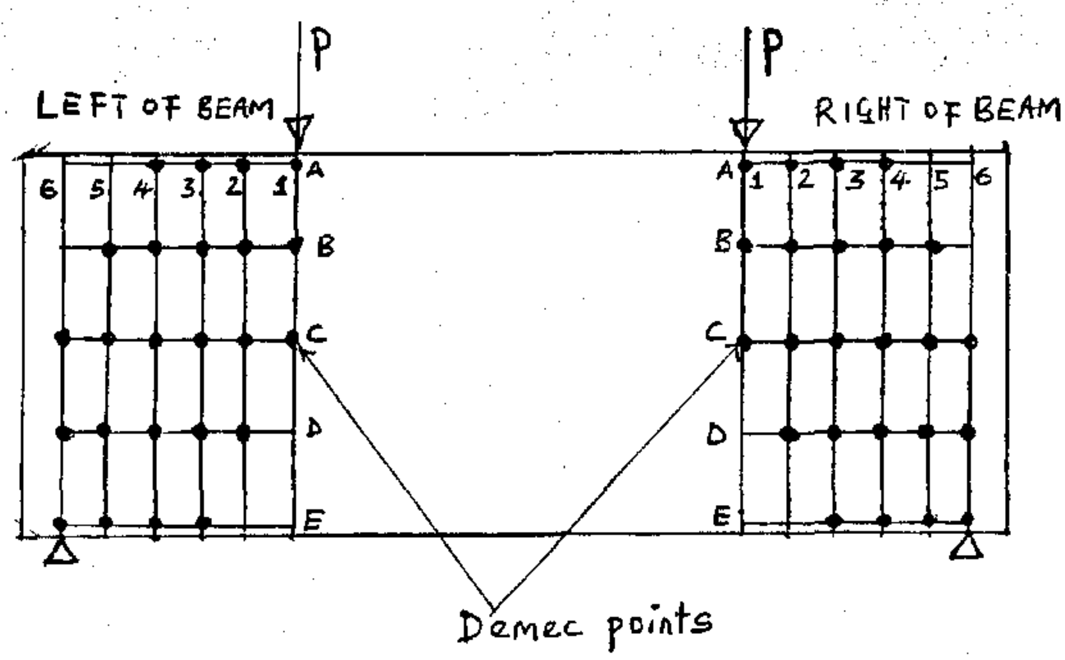


Fig. 3.5 Nature of grid points on the shear span.

In addition to the griding of the shear spans, loading points were also marked out on each beam together with the points where the dial gauges for measurement of deflections will be inserted. The points of insertion of the dial gauges are labelled as W, X, Y, Z as shown in fig. 3.6 below.

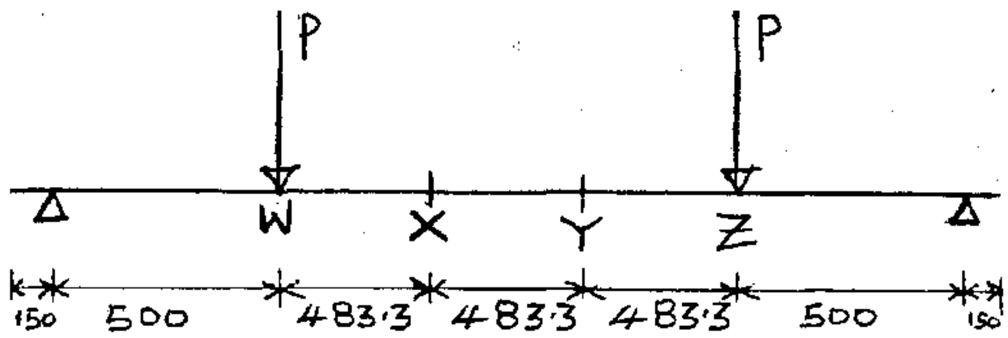


Fig. 3.6: Loading points and points of insertion of dial gauges.

As can be seen from the preceding figure, two dial gauges were placed at the points of application of load and then the remaining two placed at third points between the points of application of loads. These were placed so in order to monitor the deflection of the beam effectively.

3.9 TEST PROCEDURE

After marking out the loading points and points of placement of dial gauges together with the fixing of the Demec points within the grided shear spans on the left and right sides of the beams, each beam in turn was then mounted on the testing support above which hung a rectangular frame of heavy channel steel sections where either the loading cells or hydraulic jacks were fixed. Two loading cells of 250 kN and 500kN maximum capacity connected to pressure jacks operated manually were initially used. The use of this was discontinued however after testing two beams (BM1 and BM 3) when it was discovered that the pressure meters used in registering pressures from the jacks were faulty. This was obvious since the two beams failed quite below their design loads. The design load was 60kN while they failed at 40kN and 50kN respectively.

Consequent upon this, a new equipment was then set up which has two hydraulic jacks of 100kN maximum capacity each applied at the two loading points. This was then used in testing the remaining four beams which included BM2, BM4, BM5 and BM6.

In all, pressure was applied through the jacks in predetermined stages of 10kN increment. But before applying pressure, the initial readings of both the strains and deflections were taken. Thereafter, readings were then taken at the end of each loading stage. The strain readings were taken by the aid of the extensometer. This was done by placing it at the various Demec points and taking the reading at those points. During the readings, careful visual observations were carried out on each beam to monitor the appearance and propagation of cracks. The profiles of these cracks were then marked out on the face of each beam using a very dark pencil. Loading however was stopped in each case when the movement of the dial gauges for measuring deflectrions became so rapid such that further readings could not be taken, showing that the beam has failed.

3.10 TEST RESULTS

During the test, values of the strains at both the left and right hand sides of the shear spans of each beam at each loading stage were taken as mentioned in 3.8. The values of these strains are presented as shown in Tables 3.3-3.10. The values of the deflections for each beam are also shown in Tables 3.11 - 3.14. In case of the crushing load and stresses of cubes cast with each beam, the values are as presented in Table 3.15.

Table 3.3: Strain Readings at the Right hand side of BM2

	Load (kN)/Strain						
Grid	0	10	20	30	40	50	60
A-12	1613	1635	1628	1617	1615	1630	1642
A-23	1025	1020	1011	1005	1004	1006	1005
A-34	790	784	782	775	780	785	785
B-12	1898	1904	1900	1945	1968	1920	1918
B-23	720	718	713	711	723	772	790
B-34	791	790	789	788	786	778	778
B-45	773	774	771	772	771	768	775
C-12	1251	1250	1253	1250	1260	1257	1256
C-23	829	830	829	841	821	832	809
C-34	800	800	800	798	852	891	932
C-45	847	847	847	848	843	837	836
C-56	792	787	791	797	791	790	790
D-23	832	835	839	866	878	882	888
D-34	866	864	865	870	928	973	1016
D-45	1259	1262	1262	1260	1253	1247	1247
D-56	1223	1214	1213	1220	1224	1210	1204
E-34	769	844	845	850	862	911	936
E-45	825	828	830	830	820	847	880
E-56	830	831	835	839	847	824	823

Table 3.4: Strain Readings at the Left hand side of BM2

	Load (kN)/Strain						
Grid	0	10	20	30	40	50	60
A-12	863	916	884	887	840	881	888
A-23	1059	1084	1050	1070	1033	1052	1056
A-34	1534	1527	1521	1519	1522	1534	1511
B-12	962	991	911	971	933	944	953
B-23	1019	1005	1007	1003	1000	1038	1031
B-34	1392	1384	1411	1395	1400	1385	1376
B-45	897	789	790	796	795	785	784
C-12	1060	1062	1161	1061	1068	1070	1070
C-23	1200	1197	1207	1200	1202	1212	1207
C-34	1182	1181	1181	1184	1184	1259	1287
C-45	648	646	644	645	645	631	630
C-56	1033	1035	1034	1033	1033	1026	1025
D-23	790	780	779	806	790	824	824
D-34	691	692	692	691	693	674	670
D-45	806	808	807	810	810	908	945
D-56	996	994	991	993	994	981	978
E-34	1014	1014	1018	1021	1030	1055	1060
E-45	876	780	889	890	890	937	950
E-56	1317	1311	1311	1318	1320	1343	1368

Table 3.5: Strain Readings at the right hand side of BM4

	Load (kN)/Strain						
Grid	0	10	20	30	40	50	60
A-12	798	795	784	776	760	747	733
A-23	872	868	860	852	846	848	848
A-34	-	-	-	-	-	-	-
B-12	800	799	796	795	808	839	865
B-23	909	908	905	903	905	895	890
B-34	580	580	578	578	574	570	566
B-45	804	804	802	801	800	798	795
C-12	882	882	883	898	922	926	933
C-23	1408	1409	1409	1408	1436	1495	1570
C-34	802	802	802	802	805	794	786
C-45	1100	1099	1100	1098	1093	1090	1087
C-56	1001	1000	1001	1001	998	996	996
D-23	2455	2456	2455	2455	2455	2455	2454
D-34	1885	1885	1890	1895	1960	2024	2080
D-45	790	790	790	788	782	777	775
D-56	863	863	864	865	869	852	850
E-34	810	803	818	826	841	846	850
E-45	778	780	782	786	846	918	986
E-56	800	800	802	803	792	785	776

- No reading due to removal of Demec point.

Table 3.6: Strain Readings at the Left hand side of BM4

Grid	Load (kN)/Strain 0	10	20	30	40	50	60
A-12	1450	1458	1450	1440	1430	1421	1414
A-23	790	758	780	775	771	770	768
A-34	690	690	683	681	678	677	675
B-12	1190	1187	1184	1185	1198	1213	1227
B-23	768	766	762	761	768	754	750
B-34	1226	1225	1225	1224	1222	1221	1218
B-45	1490	1487	1483	1483	1480	1480	1478
C-12	800	785	786	800	780	778	778
C-23	963	965	966	980	1020	1160	1101
C-34	947	948	948	948	943	941	938
C-45	790	790	788	788	786	781	790
C-56	568	568	568	568	568	565	563
D-23	786	790	792	820	885	950	982
D-34	951	953	955	953	948	945	940
D-45	592	595	596	598	591	586	580
D-56	1150	1148	1147	1147	1145	1141	1138
E-34	800	802	811	838	915	93[illegible]	
E-45	1226	1230	1233	1236	1232	1224	1216
E-56	1158	1158	1158	1160	1156	1151	1139

TABLE 3.7: Strain Readings at the Righ hand side of BM5

Grid	Load (kN)/Strain 0	10	20	30	40	50	60
A-12	805	795	786	778	769	759	752
A-23	671	667	661	656	650	653	655
A-34	981	982	978	975	975	979	979
B-12	793	791	789	789	794	794	787
B-23	803	806	800	799	801	831	855
B-34	862	861	860	859	858	850	846
B-45	830	830	829	827	825	821	820
C-12	813	815	820	832	845	856	889
C-23	811	813	804	815	825	823	821
C-34	911	913	913	915	913	992	1022
C-45	1178	1179	1179	1178	1174	1168	1165
C-56	759	758	758	758	755	751	751
D-23	1173	1179	1191	1220	1255	1268	1311
D-34	842	844	847	853	889	978	1032
D-45	927	928	928	926	920	910	906
D-56	911	911	911	911	905	897	891
E-34	828	833	837	848	860	879	887
E-45	889	892	905	908	941	1010	1032
E-56	864	863	863	867	870	864	876

Table 3.8: Strain Readings at the Left hand side of BM5

Grid	Load (kN)/Strain 0	10	20	30	40	50	60
A- 12	911	908	905	897	890	882	879
A-23	971	968	965	962	961	961	960
A-34	996	991	989	988	987	989	988
B-12	1119	1118	1112	1120	1146	1179	1200
B-23	988	986	984	985	980	973	970
B-34	845	842	841	840	838	835	834
B-45	1152	1153	1152	1151	1150	1149	1148
C-12	1007	1010	1013	1015	1013	1010	1010
C-23	1169	1169	1170	1194	1240	1292	1349
C-34	777	779	779	777	775	770	770
C-45	809	801	808	809	806	803	806
C-56	740	740	739	740	739	736	736
D-23	830	831	845	884	958	1025	1054
D-34	845	847	845	843	840	835	885
D-45	941	943	944	943	938	931	928
D-56	780	781	783	786	786	779	776
E-34	876	865	872	904	980	1014	1061
E-45	803	808	804	820	812	798	793
E-56	971	973	870	978	974	956	950

Table 3.9: Strain Readings at the Right hand side of Bm6

	Load (kN)/Strain						
Grid	0	10	20	30	40	50	60
A-12	641	635	627	620	605	594	578
A-23	770	768	761	760	755	756	775
A-34	790	776	766	759	755	756	750
B-12	1213	1211	1210	1214	1229	1225	1234
B-23	700	695	690	700	694	714	740
B-34	728	728	728	728	726	722	711
B-45	791	796	780	791	781	784	784
C-12	730	731	730	726	721	718	720
C-23	803	809	808	829	881	930	989
C-45	803	804	803	806	798	780	786
C-56	792	795	792	798	797	789	786
D-23	812	816	825	858	881	888	908
D-34	874	880	874	893	930	1009	1050
D-45	883	886	887	890	880	875	878
D-56	2455	2458	2458	2460	2453	2449	2443
E-34	811	920	928	952	995	1009	1032
E-45	695	698	701	710	711	736	795
E-56	794	790	795	804	806	796	790

Table 3.10: Strain Readings at the left hand side of BM6

Grid	Load (kN)/Strain 0	10	20	30	40	50	60
A-12	803	798	791	785	775	768	761
A-23	724	719	713	708	707	704	700
A-34	608	604	601	598	598	595	595
B-12	835	832	832	832	838	855	876
B-23	1019	1015	1013	1013	1013	1011	1008
B-34	662	660	660	657	655	655	651
B-45	909	908	907	905	906	905	903
C-12	785	785	785	788	801	820	861
C-23	736	735	737	744	740	731	730
C-34	1019	1015	1011	1012	1016	1013	1010
C-45	1021	1020	1020	1020	1018	1015	1015
C-56	910	907	906	906	910	905	905
D-23	815	818	825	839	890	912	941
D-34	1079	1083	1084	1089	1086	1083	1080
D-45	584	584	588	586	583	580	574
D-56	715	713	712	711	710	706	704
E-34	838	841	848	858	854	848	845
E-45	762	763	766	770	768	761	755
E-56	918	911	916	913	911	903	895

TABLE 3.11 Deflection Readings for BM2

Load (kN)	Deflections at; W	X	Y	Z
0	000	000	000	000
10	108	129	160	102
20	210	263	300	202
30	307	397	420	301
40	404	540	580	412
50	561	734	760	562
60	671	924	940	670

Table 3.12 Deflection Readings for BM4

Load (kN)	Deflections at; W	X	Y	Z
0	000	000	000	000
10	96	140	200	80
20	206	290	340	186
30	317	434	600	290
40	445	598	640	415
50	574	763	800	539
60	710	947	1000	674

TABLE 3.13 Deflection Readings for BM5

Load (kN)	Deflections at; W	X	Y	Z
0	000	000	000	000
10	112	136	131	101
20	213	271	261	194
30	308	405	392	288
40	421	549	531	361
50	558	631	610	532
60	698	917	887	674

TABLE 3.14: Deflection Readings for BM 6

Load (kN)	Deflections at; W	X	Y	Z
0	000	000	000	000
10	59	143	120	105
20	174	293	260	206
30	290	445	420	309
40	418	606	560	426
50	549	776	680	558
60	695	950	960	725

Table 3.15: Crushing Load of Cubes Cast with beams

Beams	Cube No	Cube Mass (kg)	Crushing Load (kN)	Crushing Stress Nmm^{-2}	Average Crushing Stress (Nmm^{-2})
	1	8.1	920	40.89	
BM1	2	8.2	910	40.44	39.85
	3	8.0	860	38.22	
	1	8.1	850	37.78	
BM2	2	8.1	920	40.89	38.08
	3	8.1	800	35.56	
	1	8.1	950	42.22	
BM3	2	8.1	860	38.22	39.55
	3	8.0	860	38.22	
	1	8.2	760	33.78	
BM4	2	8.2	720	32.00	32.30
	3	8.1	700	31.11	
	1	8.1	800	35.56	
BM5	2	8.2	860	38.22	36.45
	3	8.2	800	35.56	
	1	8.1	820	36.44	
BM6	2	8.2	720	32.00	34.81
	3	8.2	810	36.00	

CHAPTER FOUR

4.0 ANALYSIS AND DISCUSSION OF TEST RESULTS

The various results obtained during the experimental investigations were shown in chapter three. These results include those of the tensile strengths of the reinforcing steel, the cube strength of concrete, the deflections of the beam as well as the strains measured. This chapter however deals with the analysis and discussion of these test results.

4.1 TENSILE STRENGTH OF REINFORCING STEEL

The yield and ultimate stresses of the various sizes of reinforcement used are shown in Tables 3.1 and 3.2. That of Table 3.1 shows the main bars and the 8mm diameter stirrups while Table 3.2 shows that of the mesh. These values were obtained from the yield load and ultimate failure load observed on test specimens. As mentioned before, the specimens were tested using the universal testing machine. The yield stress values were obtained by dividing the yield load by the effective cross-sectional area while the ultimate load by the effective sectional area. The average yield stress values for the sizes of reinforcement used appear at the lower part of the tables. The characteristic strength used during the calculation of the area of main bars and spacing of stirrups were $410Nmm^{-2}$ and $250Nmm^{-2}$ respectively. All the yield stress values got from this experiment are above these values taken in initial calculation; thus the average yeild stress values given at the lower part of Table 3.1 for each are acceptable for our test. In the case of the average yield stresses of the mesh in Table 3.2, that of mesh on 100mm x 100mm grid is about four times the one on 50mm x 50mm grid. Actually at the time of carrying out this report, the mesh on grid 50mm x 50mm was available in the market while in the case of 100mm x 100mm mesh, wires of 3mm diameter were bought and fabricated into that grid. On testing, it was then discovered that the strength of these wires was far greater than those of 50mm x50mm mesh.

4.2 CRUSHING STRESS OF CONCRETE

The crushing forces and stresses of cubes cast with each beam are shown in Table 3.21. These values are got by crushing the cubes on the universal testing machine. These cubes were all cured and tested after 28 days according to standard procedures. The characteristic concrete strength used for the calculation of the moment of resistant of the beams was $30Nmm^{-2}$. Thus, the average stress for the cubes for each beam shown in Table 3.21 is satisfactory for our test.

4.3 FAILURE LOADS OF THE BEAMS

From 3.8, it is evident that the first two beams tested (ie BM1 and BM3) failed quite below their design load due to faulty equipment as have been explained. This is further true owing to the fact that both their concrete strength and steel strength satisfied the values employed during the calculation. Thus, since the equipment was faulty, all the experimental results for these two beams will not be used in the analysis. For the remaining four beams consequently the failure loads recorded during test is as shown in Table 4.1. These loads actually represent the shear resistances of the various beams since they were loaded to fail in shear.

Table 4.1 Failure loads of the beams

Beam	Failure Load (kN)
BM2 (with mesh 50x50mm)	78.0
BM4 (with stirrups)	66.5
BM5 (with mesh 100mm x 100mm)	72.0
BM6(with mesh 100mm x 100mm)	72.5

From the values got from the calculation of theoretical shear resistance in 3.4, it is clear that the failure load of BM4 (with ordinary U-stirrups) corresponds with the theoritical shear resistance calculated due to the beam. In the case of beams with wire mesh, the difference between the theoritical and failure loads must have arisen from the influence of the horizontal members of the mesh on the shear resistance. Actually in the theoritical calculations, the horizontal members were not considered since there is no laid down formular involving them. Thus their contribution to shear resistance might have majorly accounted for

4.4 DEFLECTIONS OF THE BEAMS

During the test deflection readings were taken at points W, X, Y and Z as shown in Fig. 3.6. These readings are tabulated as shown in Tables 3.11 to 3.14. To obtain the real deflection values in millimeters, these readings are multiplied by the gauge factor which is 0.005mm. The values are therefore as shown in Tables 4.2 to 4.5.

Table 4.2 Deflection Values for BM 2

Load	Deflections (mm) at;			
(kN)	W	X	Y	Z
0	0.00	0.00	0.00	0.00
10	0.54	0.65	0.80	0.51
20	1.05	1.32	1.50	1.01
30	1.54	1.99	2.10	1.51
40	2.02	2.70	2.90	2.06
50	2.81	3.67	3.80	2.81
60	3.36	4.62	4.70	3.35

Table 4.3 Deflection values for BM4

Load (kN)	Deflections (mm) at; W	X	Y	Z
0	0.00	0.00	0.00	0.00
10	0.48	0.70	1.00	0.40
20	1.03	1.45	1.70	0.90
30	1.59	2.75	3.00	1.45
40	2.23	2.99	3.20	2.08
50	2.83	3.82	4.00	2.70
60	3.55	4.74	5.00	3.37

Table 4.4 Deflection Values for BM5

Load (kN)	Deflections (mm) at; W	X	Y	Z
0	0.00	0.00	0.00	0.00
10	0.56	0.68	0.66	0.51
20	1.07	1.37	1.31	0.97
30	1.54	2.03	1.96	1.44
40	2.11	2.75	2.66	1.81
50	2.79	3.16	3.05	2.66
60	3.49	4.59	4.45	3.37

Table 4.5 Deflection Values for BM6

Load (kN)	Deflections (mm) at; W	X	Y	Z
0	0.00	0.00	0.00	0.00
10	0.30	0.72	0.62	0.53
20	0.87	1.47	1.30	1.03
30	1.45	2.23	2.10	1.55
40	2.09	3.03	2.80	2.13
50	2.27	3.88	3.40	2.79
60	3.47	4.75	4.80	3.63

The values in these tables show that the deflections for all the beams are virtually the same. For instance, the deflections at the four points (W,X,Y and Z) at maximum load for all the beams are same. The web reinforcement therefore has no significant effect on the deflection of the beams.

4.5 STRAINS IN THE CONCRETE

In tables 3.3 to 3.10 are shown the strain readings recorded during the test at both the left and right hand sides of the beams. As said previously, these readings were recorded using the extensometer. In the reductions that now follow, Δl represents the difference between the initial extensometer readings when no load was applied and that due to applied load. For example, Δl_{10} means the difference between initial reading without load and that due to an increase in load of 10kN. This also applies to Δl_{20}, Δl_{30}, Δl_{40}, Δl_{50} and Δl_{60}. These reductions are shown in Tables 4.6 to 4.13. All the readings got are supposed to be multiplied by the instrument factor which is 1.62×10^{-5}. However for easier computation the values in these tables have only been multiplied by 1.62 and are therefore subject to further multiplication by 10^{-5} as indicated at the bottom of each table.

Furthermore, these values were taken to draw the graphs of strain within a grid at the various levels of the beams. This was done both for the left and right hand sides of the shear spans as shown in Figs. 4.1 to 4.8.

Table 4.6: Values of Strain on the right hand side of BM2

Strains due to increase in load

Grid	Δl_{10}	Δl_{20}	Δl_{30}	Δl_{40}	Δl_{50}	Δl_{60}
A-12	-35.64	-24.30	- 6.48	-3.24	-27.54	-46.98
A-23	8.10	22.68	32.40	34.02	30.78	32.40
A-34	9.72	12.96	24.30	16.20	8.10	8.10
B-12	- 9.72	- 3.24	-76.14	113.30	-35.64	-32.40
B-23	3.24	11.34	14.58	- 4.86	-48.60	-71.28
B-34	1,62	3.24	4.86	8.10	21.06	21.06
B-45	-1.62	3.24	1.62	3.24	8.10	-3.24
C-12	1.62	-3.24	1.62	-14.58	-9.72	-8.10
C-23	-1.62	0.00	-19.44	12.96	-4.86	32.40
C-34	0.00	0.00	3.24	-84.24	-147.42	243.84
C-45	0.00	0.00	-1.62	6.48	16.20	17.82
C-56	8.10	1.62	-8.10	1.62	3.24	3.24
D-23	- 4.86	- 11.34	-55.08	-74.52	-81.00	-90.72
D-34	3.24	1.62	- 6.48	-100.44	-173,34	-243.00
D-45	-4.86	-4.86	- 1.62	9.72	19.44	27.54
D-56	14.58	16.20	4.86	-1.62	21.06	30.78
E-34	-121.50	-123.86	-132.22	-150.66	-230.04	-270.54
E45	- 4.96	-8.10	- 8.10	8.10	-35.64	- 89.10
E-56	-1.62	-8.10	-14.58	-27.54	9.72	11.34

Values in this table to be multiplied by 10^{-5}

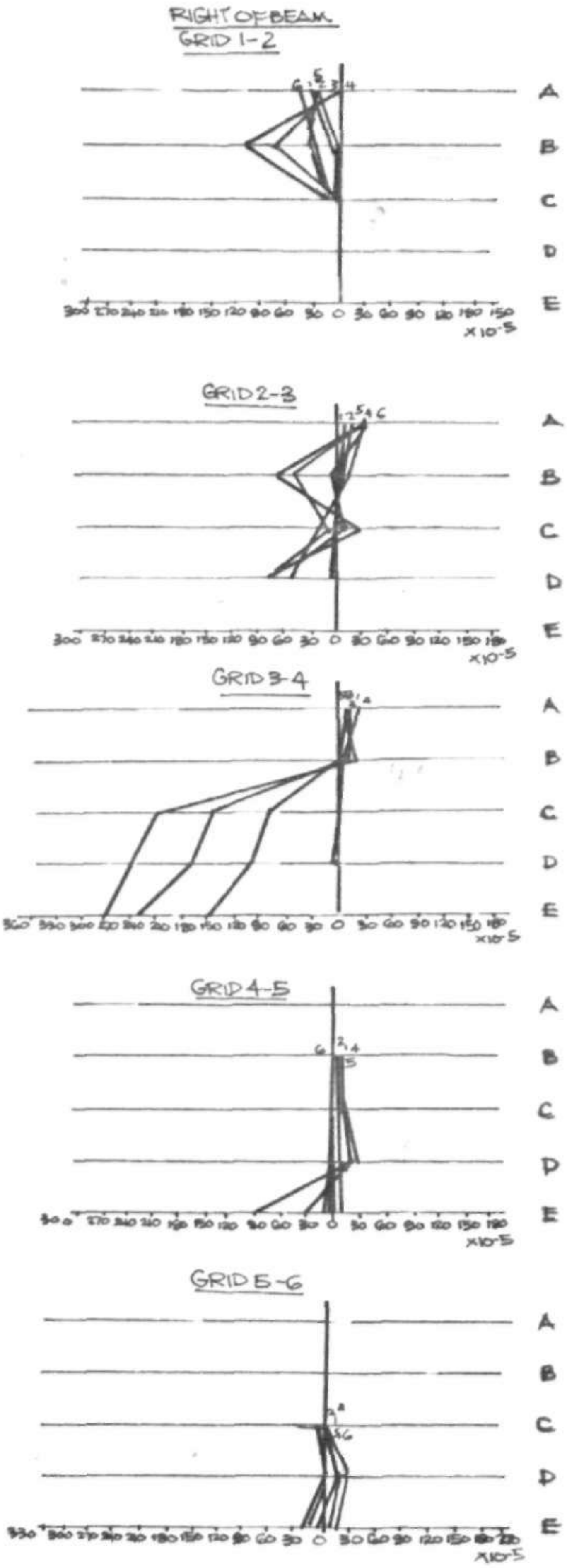


FIG. 4.1 Graphs of strain within the Various grids at the right hand side of BM2.

Table 4.7: Values of Strain on the left hand side of BM2

Strains due to increase in load

Grid	Δl_{10}	Δl_{20}	Δl_{30}	Δl_{40}	Δl_{50}	Δl_{60}
A-12	-85.86	-34.02	-38.88	37.26	-29.16	-40.50
A-23	-40.50	14.58	-17.82	42.12	11.34	4.86
A-34	11.34	21.06	24.30	19.44	0.00	37.26
B-12	-46.98	82.62	-14.58	46.98	-51.84	14.58
B-23	22.68	19.44	25.92	30.78	-30.78	-19.44
B-34	12.96	-30.78	-4.86	-12.96	11.34	25.92
B-45	174.96	173.34	163.62	165.24	181.44	183.06
C-12	-3.24	-1.62	- 1.62	-12.96	-16.20	-16.20
C-23	4.86	-11.34	0,00	-3.24	-19.44	-11.34
C-34	1.62	1.62	-3.24	-3.24	-124.74	-170.10
C-45	3.24	6.48	4.86	4.86	27.54	29.16
C-56	0.00	1.62	0.00	0.00	11.34	12.96
D-23	16.20	17.82	-25.92	0.00	-55.08	-55.08
D-34	-1.62	-1.62	0.00	-3.24	27.54	34.02
D-45	-3.24	-1.62	-6.48	-6.48	-162.24	-225.18
D-56	3.24	8.10	4.86	3.24	24.30	29.16
E-34	0.00	-6.48	-11.34	-25.92	-66.42	-74.52
E45	-6.48	-21.06	-22.68	-22.68	-98.82	-119.88
E-56	9.72	9.72	-1.62	-4.86	-42.12	-82.62

Values in this table to be multiplied by 10^{-5}

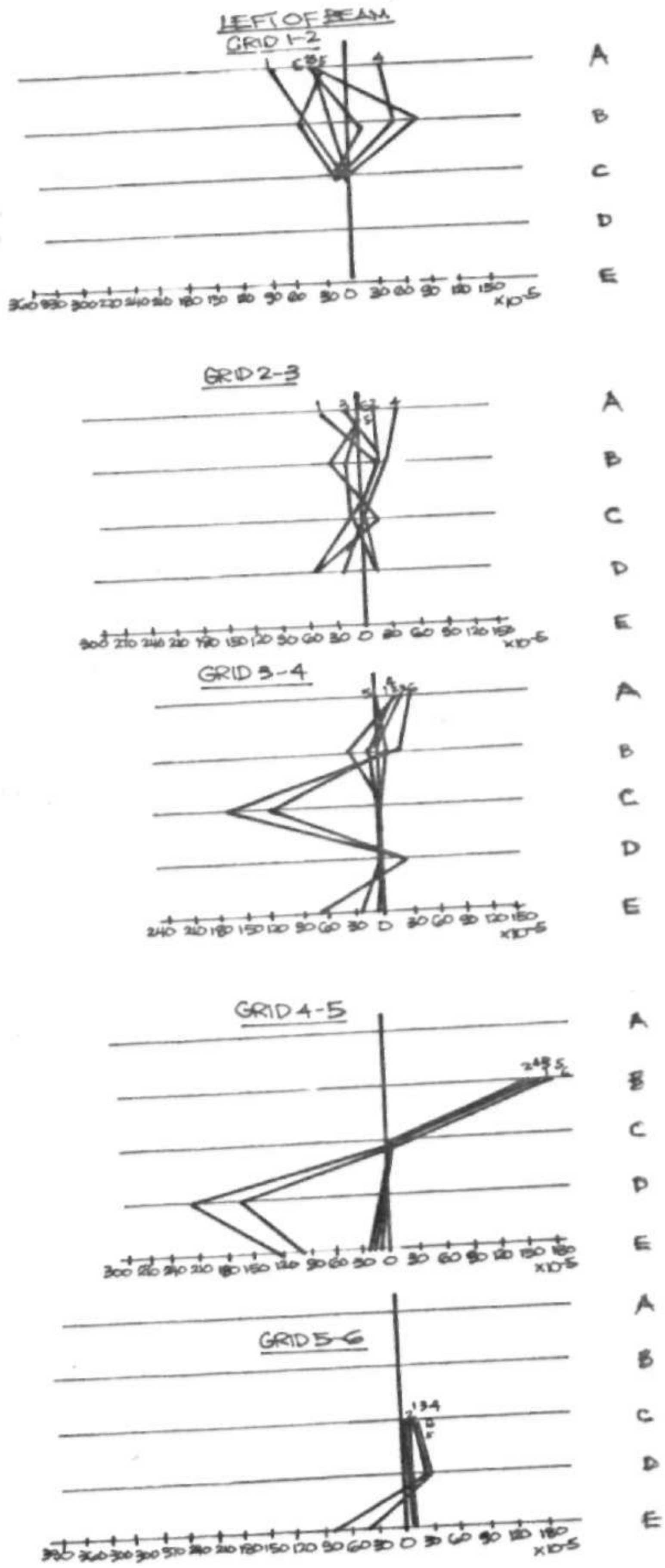


FIG. 4·2: Graphs of strain within the various grids at the left hand side of BM2

Table 4.8: Values of Strain on the Right hand side of BM4

Strains due to increase in load

Grid	Δl_{10}	Δl_{20}	Δl_{30}	Δl_{40}	Δl_{50}	Δl_{60}
A-12	4.86	22.68	35.64	61.58	82.62	105.30
A-23	6.48	19.44	32.40	42.12	38.88	38.88
A-34	**	**	**	**	**	**
B-12	1.62	6.48	8.10	-12.96	-63.18	-105.30
B-23	1.62	6.48	9.72	6.48	22.68	30.78
B-34	0.00	3.24	3.24	9.72	16.20	22.68
B-45	0.00	3.24	4.86	6.48	9.72	14.58
C-12	0.00	-1.62	-11.34	-64.80	-71.28	-82.63
C-23	-1.62	-1.62	0.00	-45.36	-140.98	-221.94
C-34	0.00	0.00	0.00	-4.86	12.96	25.92
C-45	1.62	0.00	3.24	11.34	16.20	21.06
C-56	1.62	0.00	0.00	4.86	8.10	8.10
D-23	-1.62	0.00	0.00	0.00	0.00	1.62
D-34	0.00	-8.10	-16.20	-121.50	-225.18	-277.02
D-45	0.00	0.00	3 .24	12.96	21.06	24.30
D-56	0.00	-1.62	-3.24	- 9.72	17.82	21.06
E-34	11.34	-12.96	-25.92	-50.22	-58.32	-64.80
E45	-3.24	-6.48	-12.96	-110.16	-226.80	-289.98
E-56	0.00	-3.24	-4.86	12.96	24.30	38.88

Values in this table to be multiplied by 10^{-5}

** Indicates no values due to removal of Demec point.

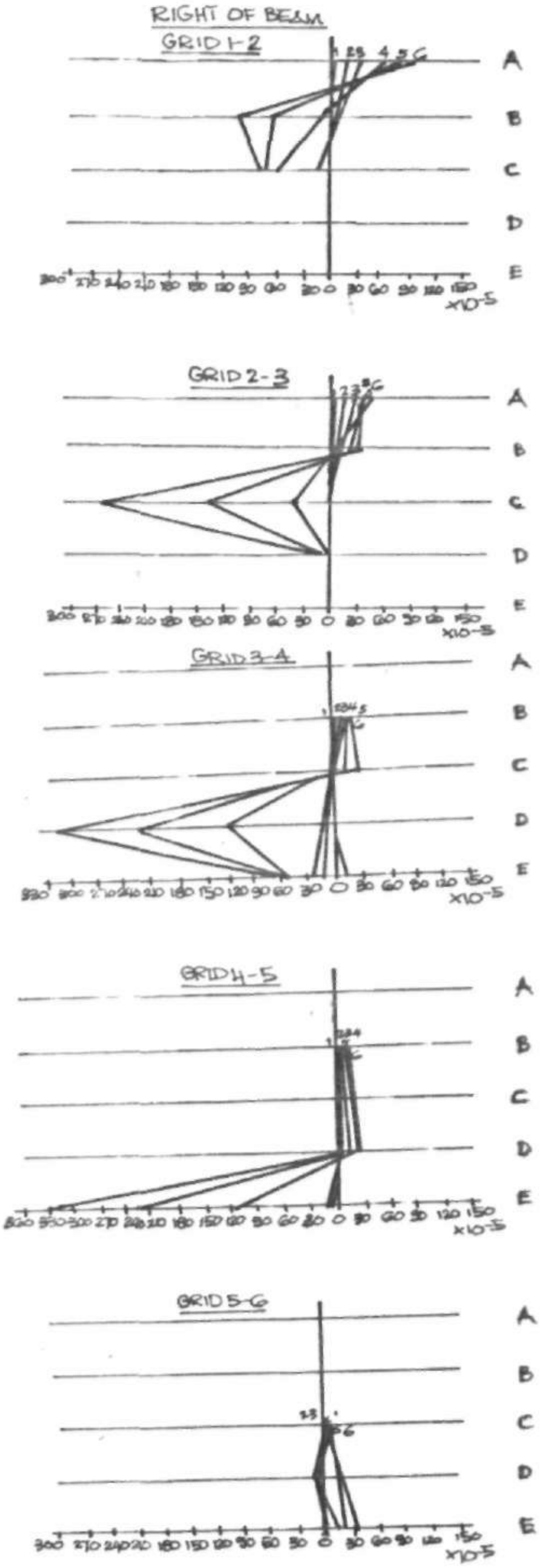


FIG. 4.3: Graphs of strain within the Various grids at the right hand side of BM. 4

Table 4.9: Strain values on the left hand side of BM4

Grid	Strains due to increase in load					
	Δl_{10}	Δl_{20}	Δl_{30}	Δl_{40}	Δl_{50}	Δl_{60}
A-12	-12.96	0.00	16.20	32.00	46.98	58.32
A-23	51.84	16.20	24.30	30.78	32.40	35.64
A-34	0.00	11.34	14.58	19.44	21.06	24.30
B-12	4.86	9.72	8.10	-12.96	-37.26	-59.94
B-23	3.24	9.72	11.34	0.00	22.68	29.16
B-34	1.62	1.62	3.24	6.48	8.10	12.96
B-45	4.86	11.34	11.34	16.20	16.20	19.44
C-12	24.30	22.68	0.00	32.40	35.64	35.64
C-23	-3.24	-4.86	-27.54	-92.34	-157.14	-223.56
C-34	-1.62	-1.62	-1.67	6.48	9.72	14.58
C-45	0.00	3.24	3.24	6.48	14.58	0.00
C-56	0.00	0.00	0.00	0.00	4.86	8.10
D-23	-6.48	-9.72	-55.08	-160.38	-265.68	-317.52
D-34	-3.24	-6.48	-3.24	4.86	9.72	17.82
D-45	-4.86	-6.48	-9.72	1.62	9.72	19.44
D-56	3.24	4.86	4.86	8.10	14.58	19.44
E-34	-3.24	17.82	-61.56	-186.63	-218.70	-315.90
E45	-6.48	-11.34	16.20	- 9.72	3.24	16.20
E-56	6.00	0.00	-3.24	3.24	11.34	30.78

Values in this table to be multiplied by 10^{-5}

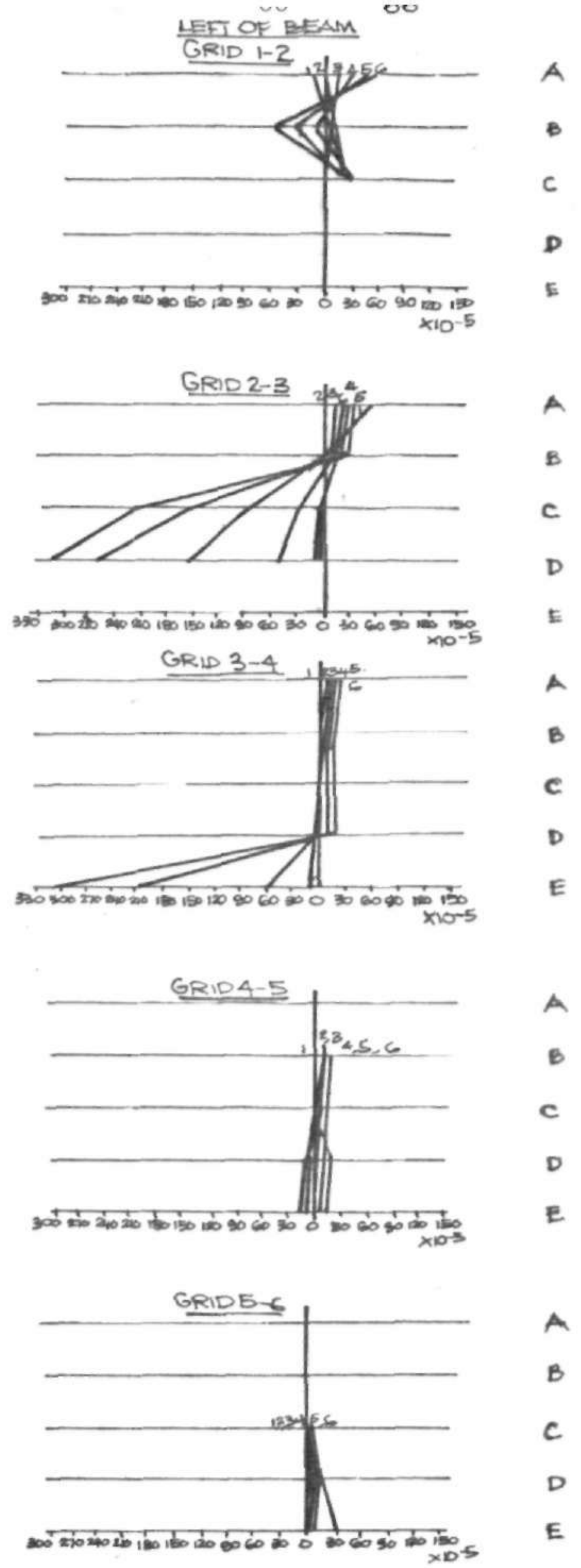


FIG. 4·4: Graphs of strain within the various grids at the left hand side of BM4

Table 4.10: Values of Strain on the Right hand side of BM5

Strains due to increase in load

Grid	Δl_{10}	Δl_{20}	Δl_{30}	Δl_{40}	Δl_{50}	Δl_{60}
A-12	16.20	30.78	43.74	58.32	74.52	85.86
A-23	6.48	16.20	24.30	34.02	29.16	25.92
A-34	-1.62	4.86	9.72	9.72	3.24	3.24
B-12	3.24	6.48	6.48	- 1.62	-1.62	-6.48
B-23	-4.86	4.86	6.48	3.24	-45.36	-84.24
B-34	1.62	3.24	4.86	6.48	19.44	25.92
B-45	0.00	1.62	4.86	8.10	14.58	16.20
C-12	-3.24	-11.34	-30.78	-51.84	-69.66	-123.12
C-23	-3.24	11.34	- 6.48	-26.68	-19.44	- 16.20
C-34	-3.24	-3.24	-6.48	-3.24	-131.22	179.82
C-45	-1.62	-1.62	0.00	6.48	16.20	21.06
C-56	1.62	1.62	1.62	6.48	12.96	12.96
D-23	-9.72	-29.16	-76.14	-132.84	-153.90	-223.56
D-34	-3.24	-8.10	-17.82	-76.14	-220.32	-307.80
D-45	-1.62	-1.62	1.62	11.34	27.54	34.02
D-56	0.00	0.00	0.00	-6.48	22.68	32.40
E-34	-8.10	-14.58	-32.40	-51.84	-82.62	-95.58
E45	-4.86	-25.92	-30.78	-84.24	-196.02	-231.66
E-56	1.62	1.62	-4.86	-9.72	0.00	-19.44

Values in this table to be multiplied by 10^{-5}

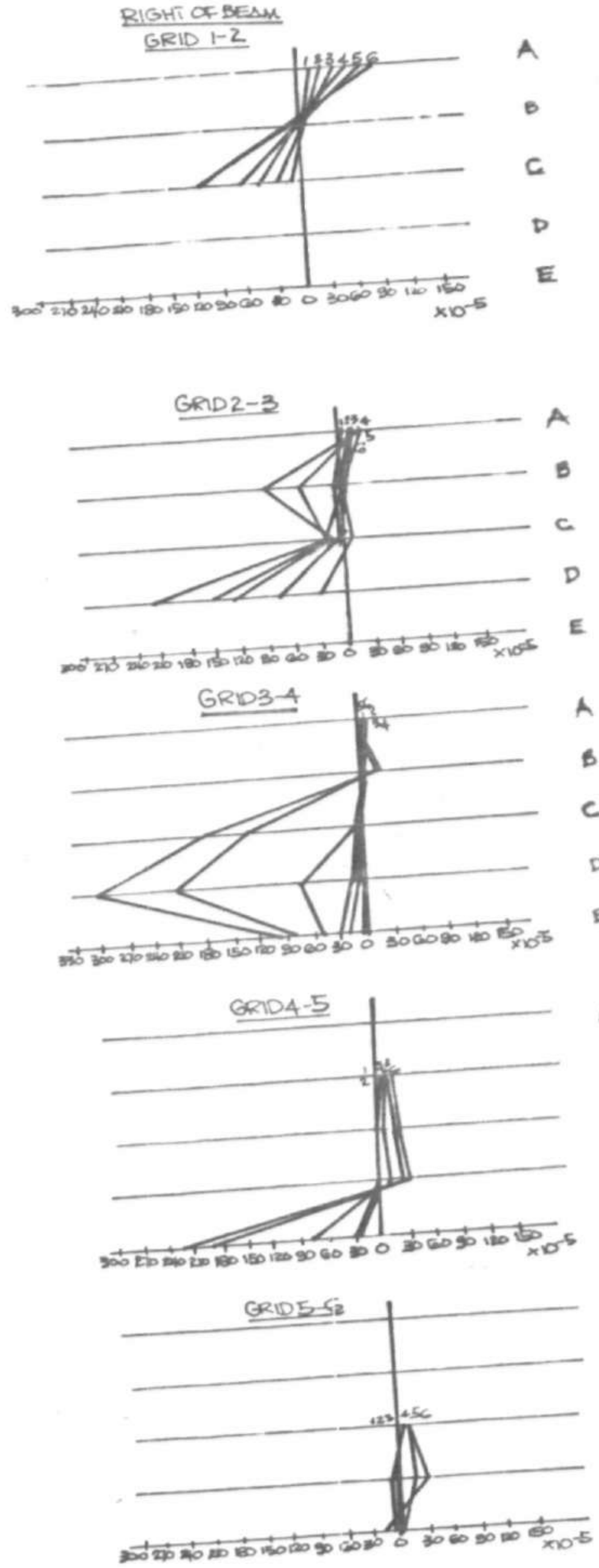


FIG. 4.5: Graphs of strain within the various grids at the right hand side of BM5

Table 4.11: Values of Strain on the left hand side of BM5

Grid	Strains due to increase in load					
	Δl_{10}	Δl_{20}	Δl_{30}	Δl_{40}	Δl_{50}	Δl_{60}
A-12	4.86	9.72	22.68	34.02	46.98	58.32
A-23	4.86	9.72	14.58	16.20	16.20	16.20
A-34	8.10	11.34	12.96	14.58	11.34	11.34
B-12	1.62	11.34	-1.62	-43.74	-97.20	-131.22
B-23	3.24	6.48	4.86	12.96	24.30	30.78
B-34	4.86	6.48	8.10	11.34	8.10	19.44
B-45	-1.62	0.00	1.62	3.24	4.86	8.10
C-12	-4.86	-9.72	-12.96	-9.72	-4.86	-6.48
C-23	0.00	-1.62	-40.50	-115.02	-199.26	-322.38
C-34	-3.24	-3.24	0.00	3.24	11.34	14.58
C-45	12.96	1.62	0.00	4.86	9.72	14.58
C-56	0.00	1.62	0.00	1.62	6.48	8.10
D-23	-1.62	-24.30	-87.48	-207.36	-315.90	-366.12
D-34	-3.24	0.00	3.24	8.10	16.20	19.44
D-45	-3.24	-4.86	-3.24	4.86	16.20	25.92
D-56	-1.62	-3.24	-9.72	-9.72	1.62	8.10
E-34	17.82	6.48	-45.36	-168.48	-223.56	-304.56
E45	-8.10	-1.62	-27.54	-14.58	8.10	22.68
E-56	-3.24	1.62	-11.34	-4.86	24.30	42.12

Values in this table to be multiplied by 10^{-5}

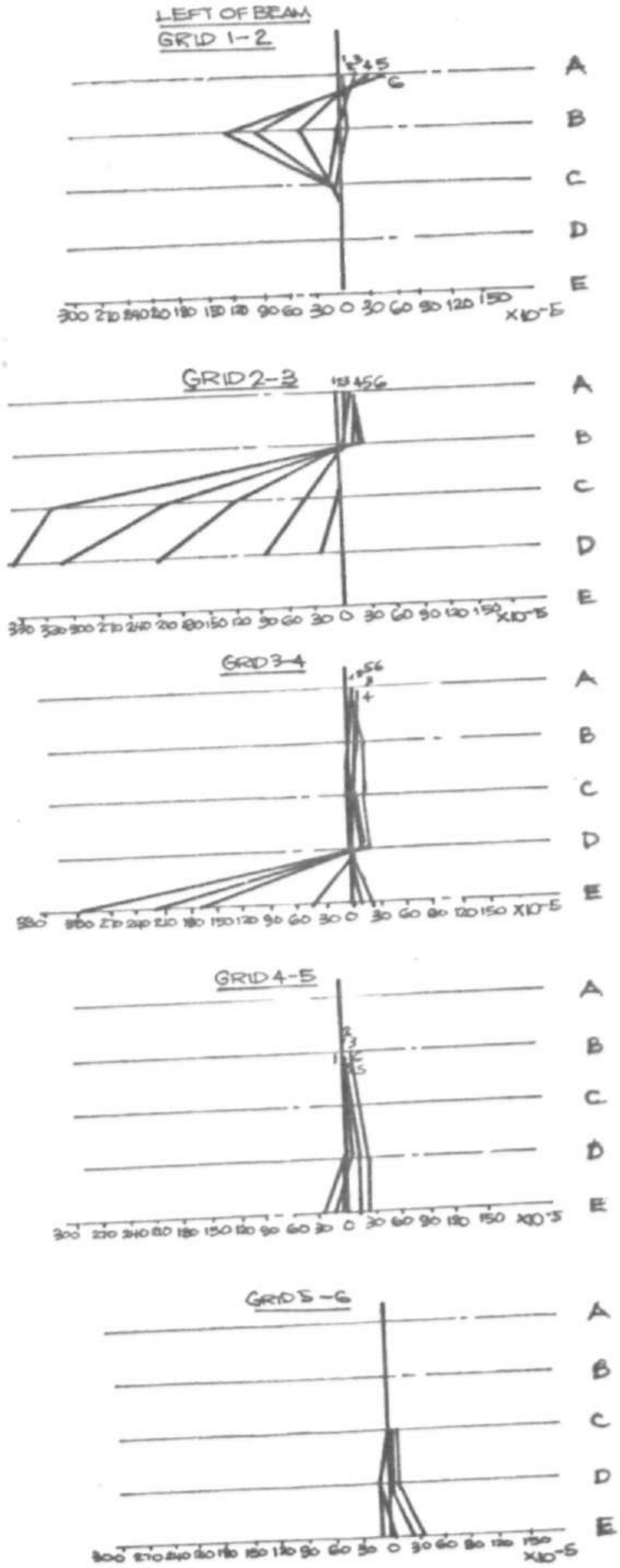


FIG. 4.6: Graphs of strain with in the various grids at the left hand side of BM5

Table 4.12: Values of strain on the right hand side of BM6

Grid	Δl_{10}	Δl_{20}	Δl_{30}	Δl_{40}	Δl_{50}	Δl_{60}
	Strains due to increase in load					
A-12	9.66	22.54	33.81	57.96	75.67	101.43
A-23	3.22	14.49	16.20	24.15	22.54	- 8.05
A-34	22.54	38.64	49.91	56.35	54.74	64.40
B-12	3.22	4.83	-1.62	-25.76	-19.32	-33.81
B-23	8.10	16.20	0.00	9.72	-22.68	-64.80
B-34	0.00	0.00	0.00	3.24	9.72	27.54
B-45	-8.10	17.82	0.00	16.20	11.34	11.34
C-12	-1.62	0.00	6.48	14.58	19.44	16.20
C-23	-9.72	-8.10	-42.12	-126.36	-205.74	-301.32
C-34	1.62	0.00	0.00	8.10	19.44	27.54
C-45	-1.62	0.00	-4.86	8.10	37.26	27.54
C-56	-4.86	0.00	-9.72	-8.10	4.86	9.72
D-23	-6.48	-21.06	-74.52	-111.78	-123.12	-155.52
D-34	-9.72	0.00	-30.78	- 90.72	-218.70	-285.12
D-45	-4.86	-6.48	-11.34	4.86	12.96	8.10
D-56	-4.86	-4.86	- 8.10	3.24	9.72	19.44
E-34	-176.58	-189.54	-228.42	-298.08	-320.75	-358.02
E45	- 4.86	- 9.72	-24.10	-25.92	-66.42	-162.00
E-56	6.48	- 1.62	-16.20	-19.44	- 3.24	6.48

Values in this table to be multiplied by 10^{-5}

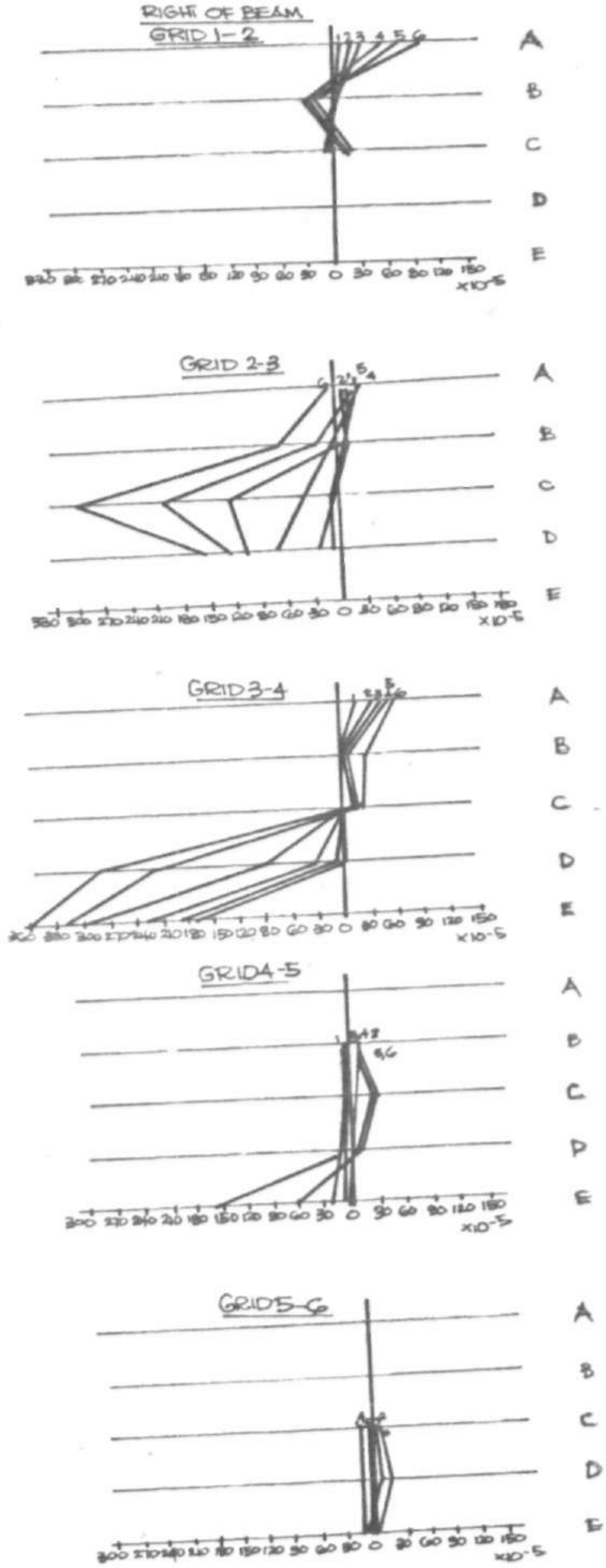


Fig. 4.7: Graphs of strain within the various grids at the right hand side of BMG

Table 4.13: Values of Strain on the left hand side of BM6

Grid	Strains due to increase in load					
	Δl_{10}	Δl_{20}	Δl_{30}	Δl_{40}	Δl_{50}	Δl_{60}
A-12	8.10	19.44	29.16	45.36	56.70	64.04
A-23	8.10	17.82	25.92	27.54	32.40	38.88
A-34	6.48	11.34	16.20	16.20	21.06	21.06
B-12	4.86	4.86	4.86	-4.86	-32.40	-66.42
B-23	6.48	9.72	9.72	9.72	12.96	17.82
B-34	3.24	3.24	8.10	11.34	11.34	17.82
B-45	1.62	3.24	6.48	4.86	6.48	9.72
C-12	0.00	0.00	-4.86	-25.92	-56.70	-123.12
C-23	1.62	-1.62	-12.96	-6.48	8.10	9.72
C-34	6.48	12.96	11.34	4.86	9.72	14.58
C-45	1.62	1.62	1.62	4.86	9.72	9.72
C-56	4.86	6.48	6.48	0.00	8.10	8.10
D-23	-4.86	-16.20	-38.88	-121.50	-157.14	-204.12
D-34	-6.48	-8.10	-16.20	-11.34	-6.48	1.62
D-45	0.00	-6.48	-3.24	1.62	6.48	14.58
D-56	3.24	4.86	6.48	8.10	14.58	17.82
E-34	-4.86	-16.20	-32.40	-25.92	-16.20	-11.34
E45	-1.62	-6.48	-12.96	-9.72	1.62	11.34
E-56	11.34	3.24	8.10	11.34	24.30	37.26

Values in this table to be multiplied by 10^{-5}

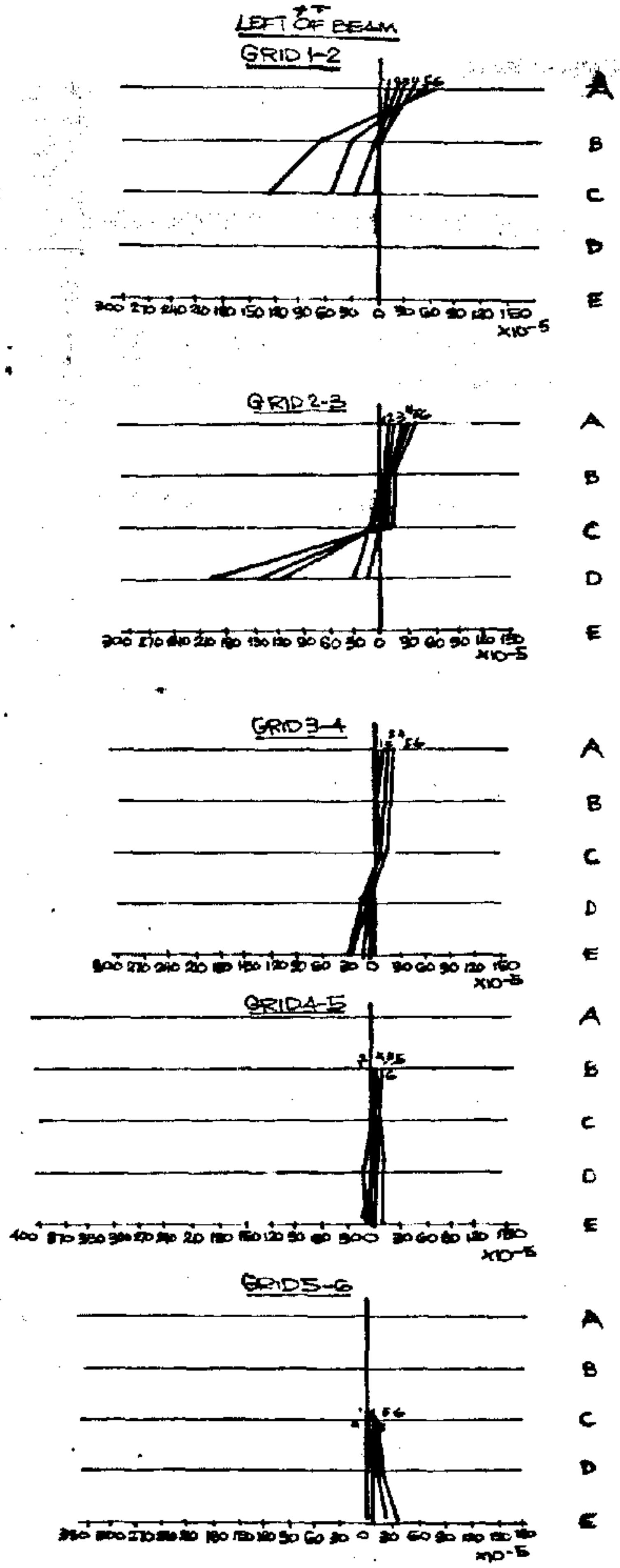


FIG 4.8: Graphs of strain within the various grids at the left hand side of BMG

From the foregoing, it is evident that the strains recorded were generally higher in the beam with ordinary U-stirrups as web reinforcement than those with wire mesh especially the mesh on grid 50mm x 50mm. To make this more elucid, tables showing the strains at maximum loads at different levels of the beams on both the left hand side and right hand side of the beams were drawn. An average of the two was also tabulated. These are as shown in Tables 4.14 to 4.16.

Table 4.14 Strains at Maximum loads for the left hand side of the beams

	Strain Values x 10^{-5} for the beams with;		
Levels	Mesh 50x50	Mesh 100x100 (average BM5 &BM6)	U- Stirrups
A	-40.5	63.18(Compr.)	58.32(compr.)
B	183(Compr.)	-98.82	-59.98
C	-170.10	-222.75	-223.56
D	-225.18	-285.12	-317.52
E	-119.88	-157.98	-315.90

Table 4.15 Strains at maximum loads for the right hand side of the beams

	Strain values x 10^{-5} for the beams with;		
Levels	Mesh 50x50	Mesh 100x100 Average (BM5 &BM6)	U-Stirrups
A	-46.98	75.13(compr.)	105.30(compr.)
B	-71.28	-74.52	-105.30
C	-243.84	-240.57	-262.44
D	-243.00	-296.46	-315.90
E	-270.54	-294.84	-336.96

Table 4.16 Average Strains at maximum loads for both the left and right hand sides of the beams.

	Average strain values x10^{-5} for the beam with;		
Levels	Mesh 50x50	Mesh 100 x 100	U-Stirrups
A	-43.74	69.16(compr.)	81.81(compr.)
B	-71.28; 183 (compr.)	-86.67	-82.64
C	-206.97	-231.66	-243.00
D	-234.09	-290.79	-316.71
E	-195.19	-226.40	-326.43

It is clear from these tables that the strains recorded at the different levels of the beams at maximum load for both the left and right hand side of the beam are generally greater in the beam with ordinary U-stirrups than those with wire mesh. Even an average of the left and right hand sides shown in Table 4.16 gives a greater strain for the beam with U-stirrups than the mesh. This invariables portrays a more shear resistance by the wire mesh than the ordinary U-stirrups.

4.6 GENERAL BEHAVIOUR AND MODE OF RUPTURE OF THE BEAMS

(a) BEAM WITH MESH 50mmx50mm (BM2)

In this beam, cracks started appearing at a load of 40kN at both the left and right hand sides of the beam. They were generally small and uniform at both sides as shown in Plate 1. These cracks started near the support and propagated diagonally towards thepoint of application of load. As the load increased, the cracks widened and failure occured gradually at a load of 78kN. This failure was a diagonal tension failure with the major crack running diagonally from the support to the point of application of load. It must be noted that although some minor cracks appeared outside the shear span; the main concentration of the cracks was within the shear spans and their growth was generally gradual.

PLATE 1: Faces of BM2 at failure showing the crack pattern.

(b) BEAM WITH ORDINARY U-STIRRUPS (BM4)

Here the cracks started appearing at a load of 40kN at both sides of the beam. As the load was increased, the cracks widened and more cracks appeared with the greater number at the right hand side of the beam than the left hand side. This is shown in plate 2. At the right hand side, cracks started near the support and propagated to the point of application of load while on the left hand side of the beam, the cracks started almost at the middle of the shear span and rose to the point of application of load. Thus there was a marked difference in their crack patterns with even major cracks forming outside the shear spans. Unlike other beams, this beam failed almost rapidly at a load of 65.5kN.

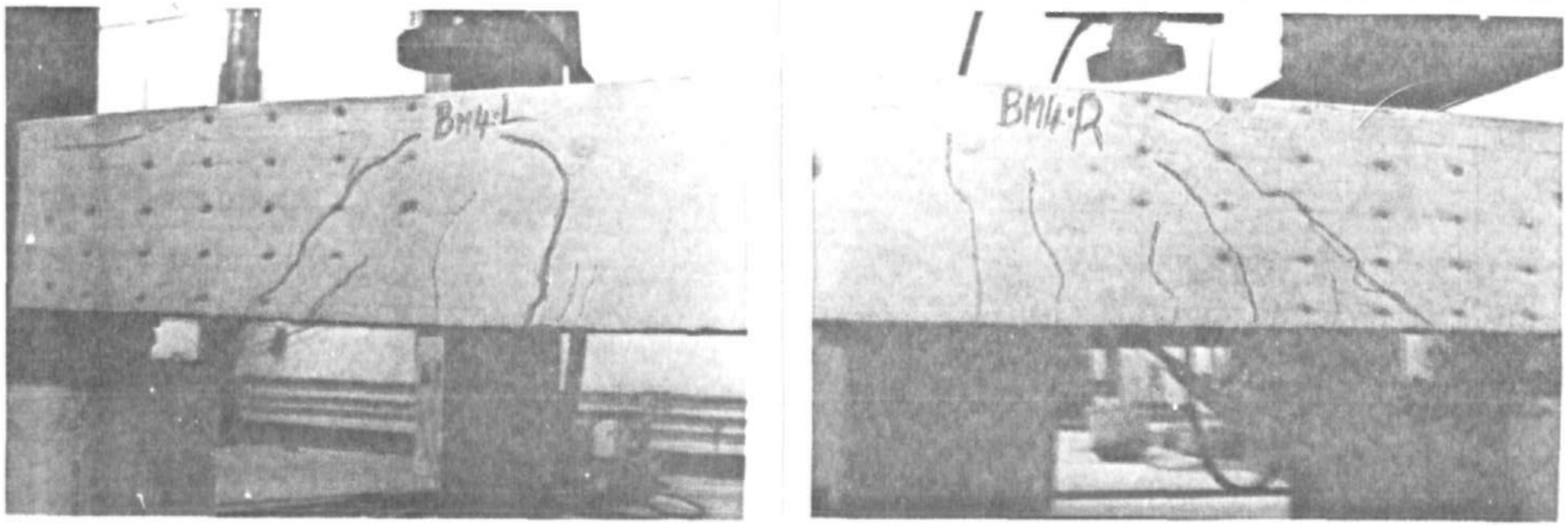
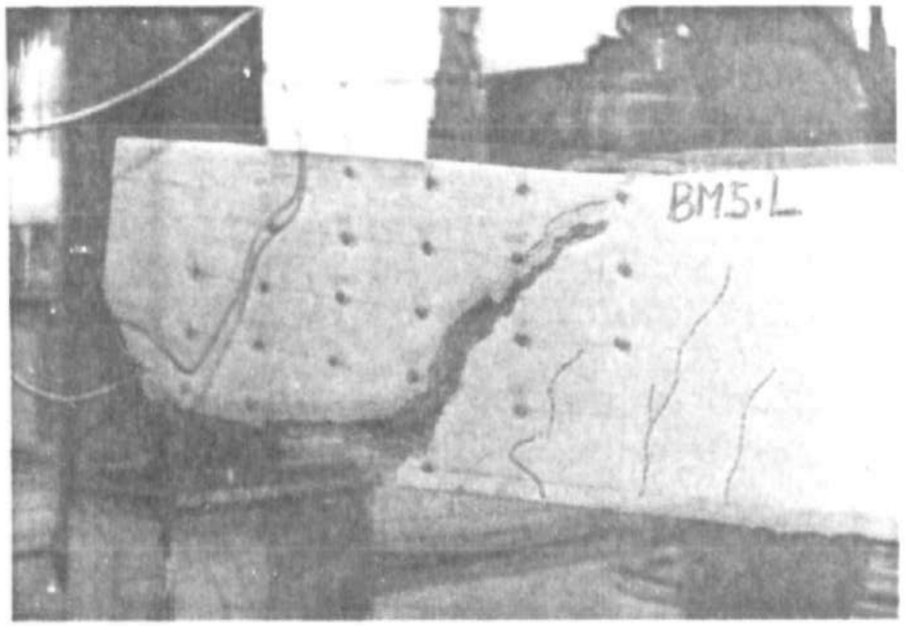


PLATE 2: Faces of BM4 at failure showing the crack Pattern.

(c) BEAM WITH MESH 100mm x100mm (BM5)

In this beam, the cracks started appearing also at a load of 40kN. The cracks here were more pronounced on the right hand side than on the left hand side, as shown in plate 3. The cracks generally started near the support and propagated diagonally to the point of application of load. As the load was increased, the cracks widened with a very remarkable one at the left hand side where failure occurred gradually at a load of 72kN.



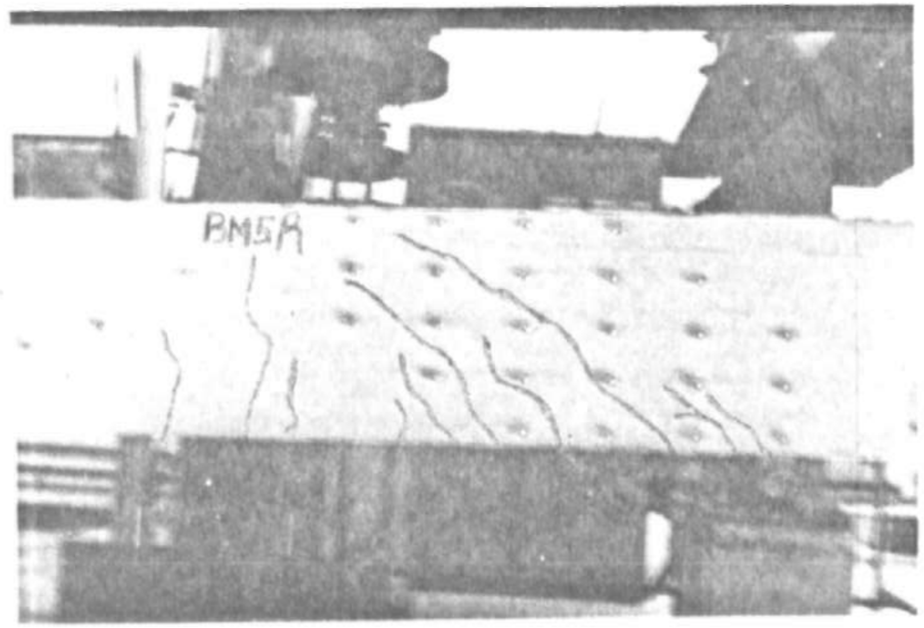


PLATE 3: Faces of BM5 at failure showing the crack pattern

(d) BEAM WITH MESH 100mm x 100mm (BM6)

Like in other beams, cracks started appearing here at a load of 40kN. The cracks were more on the right hand side than on the left hand side where they started propagating from the support. Moving diagonally upwards to the point of applications of load as shown in plate 4. The cracks widened with increase in load and failure occurred at a load of 72.5kN. However, only small minor cracks were recorded outside the shear span.

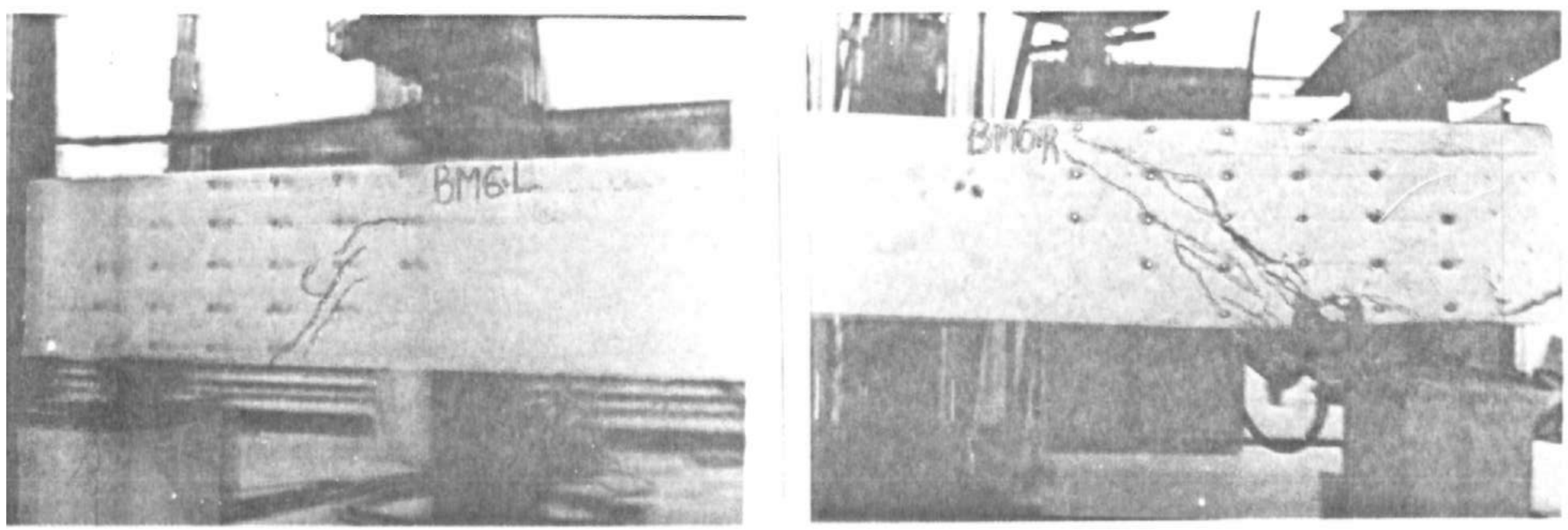


PLATE 4: Faces of BM6 at failure showing the crack pattern

5.0 CONCLUSIONS AND RECOMMENDATION

5.1 CONCLUSIONS

In rounding up this report, the following conclusions based on experimental investigations are hereby presented.

1. The mesh offered better shear resistance than the use of ordinary U-stirrup, though the contributions of the horizontal members of the mesh cannot be quantified. Thus the increase in the failure load due to the beams with mesh can therefore be attributed to the contributions of the horizontal members and other reasons.

2. It appears that mesh reinforcement is not an ideal type of reinforcement to combat shear since the contribution of the horizontal members cannot be actually quantified in the design.

5.2 RECOMMENDATION

Further research should be geared towards finding a way to quantify the contribution of the horizontal members of mesh to shear resistance.

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