FLEXURAL BEHAVIOUR OF UNCURED REINFORCED CONCRETE BEAMS WITH STEEL FIBERS

BY

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LIST OF SYMBOLS

- a = depth of rectangular stress block.
- A_s = area of tension bar reinforcement.
- A'_s = Area of compressive reinforcement steel.
- a/d = the shear span to depth ratio.
- b = width of beam.
- c = distance from extreme compression fiber to neutral axis.
- C_c = compression force in concrete.
- D = total depth of beam.
- d = distance from extreme compression fiber to centroid of tensile bar reinforcement.
- d_e = the equivalent diameter of the fiber bundle.
- d_f = the diameter of fiber.
- d' = distance from extreme compression fiber to centroid of top bar reinforcement.
- e = distance from extreme compression fiber to top of tensile stress block of fibrous concrete.

- E_c = modulus of elasticity of concrete.
- E_f = modulus of elasticity of fiber steel.
- E_s = modulus of elasticity of reinforcement steel bars.
- F_{be} = Bond efficiency factor.
- f_c' = compressive strength of concrete.
- f_r = the flexural tensile strength, also known as modulus of rupture.
- f_s' = the stresses in compressive steel.
- f_y = yield strength of reinforcing bar steel.
- I_g = the moment of inertia of the gross concrete section.
- l_c = critical length of fiber.
- l_e = the equivalent length of bundle.
- l_f = fiber length.
- l_f/d_f = fiber aspect ratio.
- $(l_f/d_f)_{cr}$ = critical fiber aspect ratio.
- M_{cr} = moment of cracking.
- M_t = theoretical moment strength.
- n = number of glued fiber.
- p = percentage by volume of fiber.
- Pe = effective volume percentage of fibers.

- $P_s\sigma_f$ = value represents the force per unit area developed by the bond strength of the fibers that are effective in a unit cross section.

- S_p = section of glued bundles.

- $T_{fc} = \sigma_t b(D - e)$ = tensile force of fibrous concrete.

- $T_{rb} = A_s f_y$ =tensile force of bar reinforcement.

- V_f = volume fraction of fiber.

- V_{fe} = effective volume of fiber in the tensile direction.

- y_t = distance from the centroidal axis of the gross section to the extreme fiber in tension.

Greek Symbols

- ϵ_c = compressive strain in concrete.

- ϵ_s = tensile strain in steel.

- ϵ_{sf} = strain in steel fiber

- ϵ'_s = strain in compressive steel.

- ρ= density of steel in the cross section, it is equal to A_s/bd.

- σ_c = uniaxial tensile strength carried by composite.

- σ_f = stress in the fiber at the assumed bond stress, also known as σ_a

- σ_m = uniaxial tensile strength carried by matrix.

- σ_t = effective tensile stress in fibrous concrete.

- τ = average value of adherence tension.

- τ_d = dynamic bond stress between the fiber and the matrix.

ABSTRACT

This is an experiment to study the behaviour in bending, cracking, deflection, deformation and crack propagation, of uncured reinforced concrete beams with fiber. Tests were carried out on 12 reinforced concrete beams. The beams are divided into two groups according to the percentage of longitudinal steel and shear span ; the first group is designed as under-reinforced concrete beams with $\rho = 1.23\%$, shear span/depth ratio of 2.83 and of the same size of $200 \times 300 \times 2000$ mm., the 2nd group is designed as over-reinforced concrete beams with $\rho = 4.3\%$, shear span/depth ratio of 4.1 and were of the same size of $200 \times 200 \times 2000$ mm., all the beams were tested under two concentrated loads positioned symmetrically .All the beams have the same span of 2000 mm with clear span between supports of1800 mm.

Tests were carried out on reinforced uncured concrete beams, with steel fibers exposed to local environment in Amman, Jordan. The test results on all beams are presented and discussed. Two beams from each group were selected to have no steel fibers ; one was cured in normal curing conditions and the other was uncured and exposed to local environment during the period of investigation outside the laboratory and used as control specimens. The remaining four beams of each group were uncured reinforced concrete with steel fibers and exposed to the local environment. These four uncured reinforced concrete contained four different percentages of steel fibers of 0.5%, 0.75%, 1.0% , and 1.50% of total weight.

During the tests, strains, deflections, first crack load and ultimate load in flexural, cracking and its propagation were recorded for the cured and uncured beams without fibers and the uncured beams with fibers.

A comparison between the ordinary cured conventional concrete beams and the uncured reinforced concrete beams with steel fibers was made with respect to their behaviour in flexural, stress distribution and crack propagation.

OBJECTIVE OF INVESTIGATION

To explain the effect of presence of steel fibers on curing and to study the behaviour, in general, of uncured reinforced concrete beams with steel fibers. And to observe the crack pattern of under-reinforced and over-reinforced cured beams, uncured beams without fibers and those of uncured beams with steel fibers.

To measure the deflections, stress distributions at mid section of the beams, and comparison will be made between the uncured beams with fibers and the cured beams without fibers.

Tests were carried out to show the ability of fibers reinforcement to control cracking and deflection more important consideration apractic than improvements in strength characteristics. The presence of fibre reinforcement is also shown to increase the stiffness and reduce the deflection of conventionally reinforced concrete beams. It is also enables higher grade steel and higher steel stresses to be used without the danger of excessive cracking and deflection.

To know what is the minimum quantity of fibers required to adequately increase the ductility of the concrete to avoid sudden and catastrophic failure.

Chapter 1

INTRODUCTION

1.1 GENERAL

Concrete offers a lot of advantages, proved by its large extended areas of all kind of applications. However concrete is a material with considerable dead weight, and substantial percentage of its weight is inactive when subjected to tension.
To avoid these problems ;a lot of research as example Ramouldi and batson in ref.[15] is done to improve the concrete characteristics. One of the techniques used is to add steel fibers.

Steel fibers concrete is now being considered as a new and totally different concrete construction material with improved strength, ductility and fracture toughness. However the basic materials are the same as for conventional concrete. Specification for high quality ingredients will also apply to steel fibre concrete and the properties of the hardened concrete will be governed by the methods of placing and consolidating of the wet mix.

1.2 HISTORICAL BACKGROUND

Historically fibers have been used to reinforce brittle materials since ancient times; straws were used to reinforce sunbaked bricks, horse hair was used to reinforce plaster and more recently, asbestos fibers are being used to reinforce portland cement. Patents have been granted since the turn of the century for various methods of incorporating wire segments or metal chips into concrete. The low tensile strength and brittle character of concrete have been bypassed by the use of reinforcing rods in the tensile zone of the concrete since the middle of the nineteenth century.

The research by Romualdi and Batson [1], and Romualdi and Mandel on closely spaced wires and random fibers in the late 1950, and early 1960, was the basis for a patent based on fiber spacing. The PCA investigated fiber reinforcement in the 1950,. Another patent based on bond and aspect ratio of the fiber was granted in 1972. In the early 1960, experiments using plastic fibers in concrete with and without steel reinforcing rods or wire meshes were conducted. Experiments using glass fibers have been conducted in the United State since the early 1950, as well as in the United Kingdom and Russia[1]. Applications of fiber reinforced concrete have been made since the mid 1960, for road and floor slabs, refractory materials, and concrete products.

The majority of experience in the United States has been in the use of steel fibers with normal weight aggregate and portland cement as the binder. The state-of-the-art is at an early state wherein the method of proportioning ingredients to achieve a matrix of high bond characteristics is still in development. The methods of mixing, placing, consolidation and finishing have been developed to reasonable degree, particularly for pavements[2]. The greater difficulty in handling steel fiber reinforced concrete requires more deliberate planning and workmanship than established concrete construction producers. Present mechanical methods of producing and handling regular concrete may or may not be appropriate for fiber reinforced concrete depending on the many mix parameters involved.

1.3 DEFINITION OF FIBRE REINFORCEMENT CONCRETE

Steel fibre concrete is the concrete made of hydraulic cement with aggregates of various sizes and amounts, and reinforced with discontinuous and discrete steel fibers[1].

1.4 CLASSIFICATION OF STEEL FIBERS

A number of fibre types are available for reinforcing concrete. Round steel fibers, the earliest examples, are produced by cutting round wire into short lengths. Typical diameters lie in the range 0.25- 0.75 mm. Steel fibers having a rectangular cross section are produced from slitting sheets about 0.25 mm thick, and may be produced cheaply if suitable scrap steel readily available . The melt extract process[3], is capable of producing low cost of stainless and carbon steel fibers from scrap steel in many forms.

A tempts by manufacturers to improve the mechanical bond between fiber and matrix have resulted in the production of indented, crimped, machined and hook-ended fibers. Length/diameter or aspect ratios of fibers which have been employed vary about 30 to 250[3].

1.5 STRUCTURAL BEHAVIOUR

The use of fiber reinforced concrete is gaining importance in the building materials area. Hydraulic cements used in building materials are weak in tension, but the inclusion of suitable fibers, is found to alleviate this weakness. The benefits of using this materials in flexure, shear, dynamic strength, creep and in compression are discussed.

1.5.1 Flexure

The use of fiber reinforced concrete in buildings to achieve an adequate section to resist a flexural failure, has been under investigation by engineers in the past decade. Design and analysis methodologies are formed so that this type of construction can be put into use. From research, it has been shown that the use of steel fibers in reinforced concrete has the following benefits in flexural behaviour:

1. Increase the moment capacity.

   Swamy and Saad [4], have shown that the maximum increase in the ultimate flexural strength due to 1% volume of fibre was only 10.5%, very much less than the effect of the fibers on the modulus of rupture or the effect of fibers on the deformation of beams.

2. Increase the ductility. Shah and Rangan [10], have shown that steel fibers increase, although to a lesser extent than stirrups, the ductility of over-reinforced beams. However, not to the same efficiency of the stirrups . Also, Mansur [11], has shown this to be true in his research. The ultimate concrete strain at failure is also increased.

3. Increase the tensile strength of the material.

4. Improves crack control. when fibrous concrete may crack, its fibrous bridge the crack, limiting its size and restraining concrete separation.[5], [6].
   Hughes and Fattuhi[7], have shown that the maximum increase in the first

crack flexural strength, of nearly 15%, was obtained when the concrete mixes were reinforced with .64 × 59 mm ordinary Duoformor hooked steel fibers.

5. Increases the stiffness.

More recently swamy et. al. [8], have shown by tests on reinforced concrete beams and slabs that steel fibers can be used to increase the stiffness of the beams, control cracking and reduce deflection in concrete structural member.

But walkus, et. all. [9], have showed that the modulus of elasticity of fibre reinforced concrete decreases by 20% when the volume fraction of fibre increases. Also they showed that the addition of cut steel fibers to the concrete increases the tensile strength of fibre reinforced concrete up to 50% greater than that of plain concrete, but only when reinforced below some critical microreinforcement content.

The tests reported by Swamy and AL-Noori [12], showed that the presence of steel fibers enables high strength steel with yield strength of 700 N/mm^2 to be used with both crack width and deflection being controlled to within acceptable limits. They also showed that over-reinforced beams could be made to behave in ductile manner by also use of steel fibers [3]. The basic assumption that the tensile strength of the concrete may be neglected across a cracked section is not valid with respect to fiber reinforced concrete.
The presence of fiber in reinforced concrete member will produce tensile stresses across a cracked section in tensile zone. Equations for the behaviour of ultimate flexural strength has been developed by Swamy and Al- Ta' an [4, 16, 19].

1.5.2 Shear

The shear capacity of a conventional reinforced concrete beam can be increased from 30 to more than 100% of the capacity by the addition of fibers [13]. Steel fibers have some advantages over vertical stirrups or bent up bars and these are:

1. The fibers are randomly distributed through the volume of concrete at much closer spacing than can be obtained by the smallest reinforcing rods.

2. The first crack tensile strength and ultimate tensile strength are increased by the presence of fibers.

3. The shear friction strength is increased. It is evident from test [14], that stirrups and fiber reinforcement can be used more effectively when both types of reinforcement are present.

A comparison of the tests results from AMAL AL-FAR[11], has shown what is lost in strength due to improper or no curing , can be gained by inclusion of fibers in uncured beams. For example, when $a/d^1 = 1$, the uncured beams without fibers, lost 13% in shear strength when compared to normally cured beam.

Whereas, a similar uncured beam containing 1% of steel fibers, gained in shear strength equal to 14%, indicating that what is lost in shear strength due to no curing could be compensated by inclusion of suitable amount of steel fibers.

1.5.3 Dynamic strength

The dynamic strength of concrete reinforced with various types of fibers and subjected to explosive charges, and dynamic tensile and compressive loads have been measured. The dynamic strength for various types of loadings were five to ten times

[1] where a is the shear span, and d over all depth of the beam

greater for fiber reinforced than for plain concrete [1]. The greater energy requirements to strip or pull out the fibers provides the impact strength and resistance to spalling and fragmentation [12].

1.5.4 Creep

The limited test data indicated that wire fiber reinforcement had no significant effect on the creep behaviour of portland cement mortar. For expansive mortars reinforced with steel fibers, the fibers contribute to early no-load creep, termed precreep, [1].

1.5.5 Compression

The use of fibers in reinforced concrete columns has been studied by many investigators [1]. From the tests on axially loaded specimens, it was observed that the ductility increased slightly with the increase in the amount of fibers. The fibers need to be placed in the outer cover of the column in order to help in confining the concrete core.

High shearing forces and axial loads, when applied simultaneously to restrained short column under double curvature bending, cause an unfavourable explosive type of failure. This explosive failure can be controlled by proper proportioning of the column dimensions. The use of fibers helps in reducing such violent failure [11]. It has been seen that, with the use of fibers in the columns, the failure behaviour changed from an explosive type of failure to a bending type of failure [12]. The shear capacity was increased with a less explosive type failure occurring.

1.6 APPLICATION OF STEEL FIBRE CONCRETE

Application of steel-fibre concrete may be classified into six categories, [3].

- Highway and airfield pavements.
- Structural application.
- Fibrous shotcreate.
- Pipes.
- Refractory concrete.
- Miscellaneous precast applications.

1.6.1 Highway and airfield pavements

Three principal benefits have led to steel-fibre concrete pavements being selected as the best engineering solution on a number of major projects.

- A high flexural strength (compared to plain concrete)results in a reduction in the required pavement thickness for given design life, or an extended design life for the same thickness. By a rule of thumb approach, if the flexural strength of fibers concrete is twice that of plain concrete, then the required thickness of the former is about 70% of the latter.
- The transverse and longitudinal joint spacing may be increased. Under conditions of restrained shrinkage, the greater tensile strain capacity of steel fibre concrete results in lower maximum crack widths than in plain concrete.
- The resistance to impact and repeated loading is increased. Mixing , placing and finishing of fibre concrete is very similar to plain concrete, and slab have

been laid using semi-manual methods, concreting train and slip-form paver. Projects have involved new pavement construction or repair to existing pavements by the use of overlays which may be bonded or unbonded to the slab beneath. At McCarran International Airport, Nevada, USA, both types of construction have been employed.

1.6.2 Structural application of steel-fibre reinforcement.

Structural applications where fibers alone provide the reinforcement are rare, although one notable example consisted of precast slabs about 1.1 m square and 65 mm thick, supported by tabular steel space frame for a demountable car park at london airport. Three percent by weight of 0.25 mm diameter by 25 mm long fibre provided the reinforcement for the slab.

The prospects for the uses of fibre reinforcement in structural applications have recently been reviewed by Swamy [15], who outlines the following possibilities.

- fibre reinforcement can inhibit crack growth widening, which may permit the use of high-strength steel without excessive crack widths or deformation at service loads.

- The use of fibre reinforced concrete has been applied to hemispherical domes, using an inflated membrane process. A layer of fibre shotcrete is applied to the inside of a layer of foam first applied to the interior of a membrane. Batson, Naus, and williamson developed a construction technique for inflation forming steel fibre concrete shell structures suitable for shelters and fortifications. Some companies which have built domes using this type of construction include

Monolithic Construction, Inc., EnvironmentalShells, Inc, and Tecton, Inc. in the U.S.A.

- Fibre reinforcement can act effectively as shear reinforcement. Punching shear strength of slabs may be increased and sudden punching shear failure transformed into a slow ductile one.

- In prestressed concrete members, the addition of fibre reinforcement has been found to reduce transmission lengths and prestress losses due to elastic shortening, shrinkage and creep.

- Fiber reinforcement can provide increased impact strength for conventionally reinforced beams. Resistance to local damage and spalling is also increased.

- Fibre reinforcement can provide enhancement and integrity to preserve conventionally-reinforced concrete structures subject to earthquake and explosive loading.

1.6.3 Fibrous Shotcreate

Steel fibrous shotcrete is a mortar or concrete containing steel fibers pneumatically projected at high velocity onto a surface. The addition of steel fibers to shotcrete has considerably improved the flexural strength, direct tensile strength, shock resistance, ductility and failure toughness.

The use of a reinforcing wire mesh can be avoided: placement of a wire mesh is a dangerous and expensive job.

Dramix steel fibrous concrete can be applied using existing equipment with little or no modifications to stabilize rock slopes, line mines, tunnels, irrigation channels, repair existing structure (Dams, Bridges,.....), construction in earthquake areas, steel pipe coatings, refractory coating, shell roofs and other application.

1.6.4 Pipes

Adding steel fibers gives to the pipes a higher first crack resistance. A concrete pipe with steel fibers has better quality; the increase in toughness of steel fibre concrete causes a better distribution of the bending moments and so a higher ultimate resistance.

1.6.5 Refractory Concrete.

Refractory lining of furnaces (petrochemical industries, steel works, cement plants). Lining of steel profiles against fire attack [16].

1.6.6 Miscellaneous Precast Applications

A variety of applications have made use of the improved flexural and impact strengths and resistance to handling misuse of steel fibre concrete. Examples include manhole covers, concrete pipe, burial vaults and machine bases and frames [3].

Chapter 2

EXPIREMENTAL WORK

2.1 GENERAL

Cement, water, reinforces steel, admixtures, and other conventional material to be used for steel fiber reinforced concrete should conform to the same local recognized specification used for conventional concrete[2].

2.2 MIX DESIGN

The mix design adopted here was based on trial error, simply because the workability of the mix does respond to the norms encountered in ordinary mixes. The presence of the fibers interferes significantly with the level of workability needed. The concrete mixes were designed to have a 28 day compressive strength of 20 MPa and to enable the same mix proportions to be used for both conventional and fiber reinforced concrete. The concrete was proportioned to have required strength and adequate workability to enable easy placing, and this was achieved by using 20 mm maximum aggregate size. After doing three trial tests the mix proportions[1] chosen by weight/m^3 is

- cement= 350 kg
- free water = 308 kg

[1]These trials were made to calculate the adequate workability and required strength.

- coarse aggregate= 1225 kg
- fine aggregate = 525 kg
- admixture= 3 ml/ kg cement from superplasticizer.

the description for the materials used in the mix are presented in the following section.

2.2.1 Cement:

Pozzolanic portland cement (Jordanian cement equivalent to type 1) was used in all mixes, and it's chemical composition compiled with relevant J.S 219 . Typically the cement contained about[11]:

- SiO_2=23.38%
- CaO=53.97%
- AL_2O_3=6.31%
- Fe_2O_3=4.91%
- SO_3=2.9%
- MgO=4.94%

The cement was tested in the R.S.S[2] and found to have fineness of 411×10^3 mm^2/g and an initial setting time of 200 minutes and final setting time of 280 minutes. Also the cement has a compressive strength of:

- 3 day compressive strength = 27.5 N/mm^2
- 7 day compressive strength = 39.8 N/mm^2
- 28 day compressive strength = 49.38 N/mm^2

[2]ROYAL SCIENTIFIC SOCIETY

2.2.2 Aggregates

The local aggregate from Amman area was used and contains, fine aggregate and coarse aggregate the description for aggregate are shown below

1. Fine aggregate consists of sweileh sand and Al-Adasia sand.

2. Coarse aggregate used were crushed lime stone and consist of the Al-Folleh and Al-Humsia aggregate.

The properties for all types of aggregate are shown in the following table:

type of aggregate	dry density	water absorption	ruw	moist content
sweileh	2.6	2.53%	1.65	.38%
adassia	2.57	7.32%	1.80	.31%
al humsia	2.52	7.27%	1.49	.14%
al follieh	2.64	5.06%	1.44	.04%

Table 2.1 Description for aggregate.

The mix contains 15% from Sweileh 15% from the Al - Adassia, 45% from Al - Humssia and 25% from Al-folliah. And for the mixed aggregate:

(a) Abrasion resistance using Los angles machine , according to(ASTM -C - 131)= 36%

(b) Specific gravity according to (BS:812)

Dry=2.54 gram/c.c

S.S.D=2.56 gram/c.c

(c) Water absorption % according to (B.S: 812)=6.01%

2.2.3 Water

The tap water used in the mix was calculated to overcome the absorption problem and to complete reactions and to give adequate workability for the mix, and was equal to 88% of the cement weight.

2.2.4 Steel Fibers

One type of steel fibers was used in the eight beams. The fibers obtained was manufactures to have deformed hooks of both end, and are glued together in bundles with a water - soluble adhesive. These fibers are made from low carbon steel wires, corrosion protected. One standard size of fibers were used and had, a nominal length of (60 mm) and a diameter of (.8 mm) with aspect ratio l/d = 75 .

Gluing the loss fibers together, offers them in a compact bundle from which makes it possible to mix longer steel fibers having a higher reinforcing capacity, [16].

The gluing of the fibers creates an artificial aspect ratio of approximately 25 when introduced to the mix. When the glue is dissolved by the water in the mix, the fiber will be separated as individual fibers with an aspect ratio of 75.

To determine the aspect ratio of glued fibers, we calculate the equivalent diameter as if the bundle has a circular section[16, 23]:

$$s_p = n\frac{\pi d_f^2}{4} = \frac{\pi d_e^2}{4} \qquad (2.1)$$

$$n d_f^2 = d_e^2 \qquad (2.2)$$

$$de = \sqrt{nd_f^2} \qquad (2.3)$$

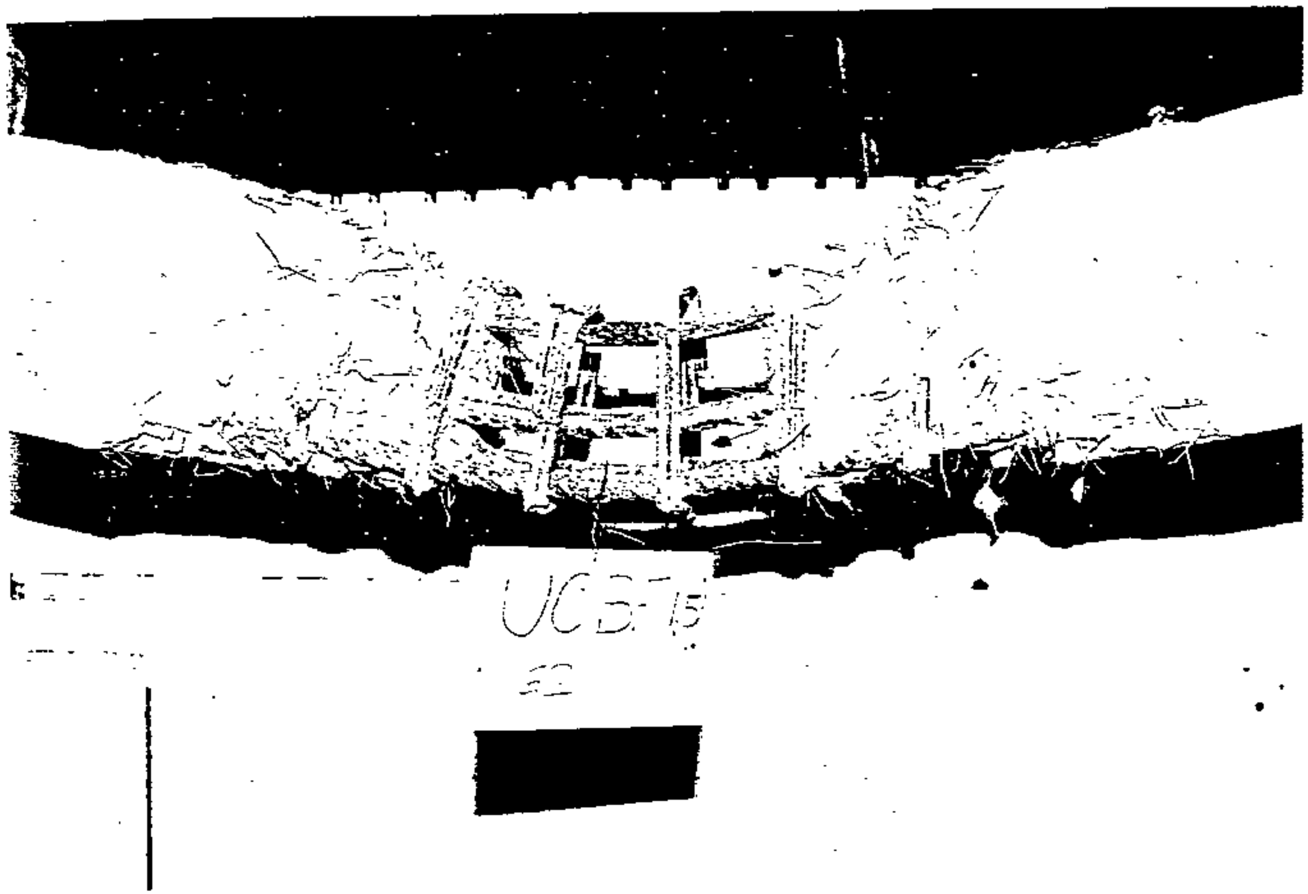
WHERE:

- S_F = section of glued bundles.
- n = number of glued fibers.
- d_f = diameter of each fiber.
- de = equivalent diameter of the bundle.
- l = le = length of fiber / bundle.

Substitute le instead l_f and de from equation (2.3) to find the new aspect ratio in equation (2.4)

$$\frac{le}{de} = \frac{l_f}{d_f} \times \frac{1}{\sqrt{n}} \qquad (2.4)$$

These bundles can be added to the dry aggregates as well as to the already mixed concrete. As soon as mixing process starts the bundles spread immediately throughout the entire mass. They separate once more into separate steel fibers and distributed throughout the concrete to give a homogenous mix. The distribution of the fiber in throughout the beam is illustrated in plate (2.1). From this plate it is clear that the fibers distributed uniformly and finally oriented in three dimensions.



PLATE(2.1) – FIBER DISTRIBUTION THROUGHOUT THE BEAM

2.2.5 Reinforcing Bars

In the first group Grade,[3] Fe 380 mild deformed bars were used as main flexural reinforcement, also Fe 340 mild deformed bars were used as main flexural reinforcement in the second group. Two-ϕ12 mm diameter of Fe 330 used to hang the stirups in all beams and ϕ8 mm mild steel used for stirups in two sets of beams.

The first set was reinforced by 3- ϕ16 mm bars used as flexural reinforcement and 2-ϕ12 mm diameter reinforcing bars used as hanger bars. The flexural reinforcement bars were placed at 247 mm from the extreme compressive fiber. The bars were held in place during casting by using the stirups of ϕ8 mm distributed uniformly throughout the span of the beam; one stirrup each 80 mm., as shown in figure 2.1.

In the second set all beams were reinforcing by 5 ϕ18 mm deformed bar as a flexural reinforcement and 2-ϕ12 mm bars as hanger bars. The flexural bars were placed in two rows with a center located at 147 mm from the extreme compressive fiber. The beams are provided by 2 ϕ8 mm as stirups distributed throughout the length of beam uniformly each 5 cm., as shown in figure 2.2.

[3] Tested In Royal scientific society

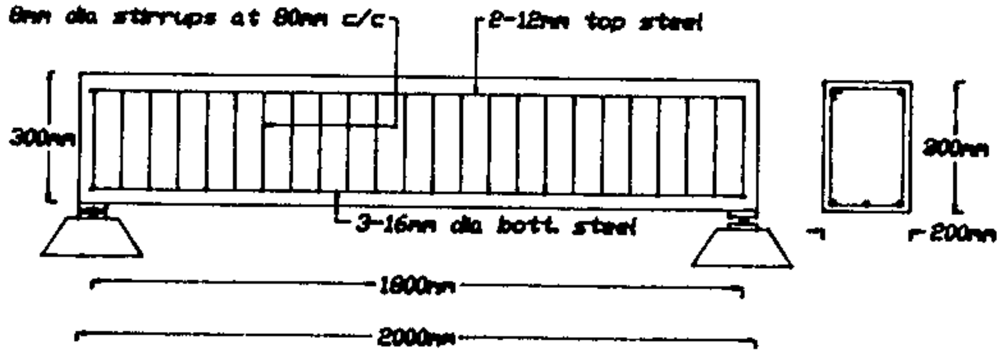


Fig 2-1 Details of the Reinforcement and cross section of beams of first set

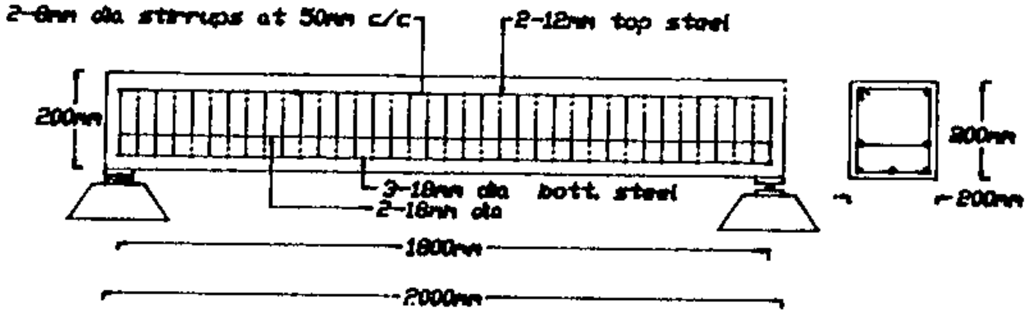


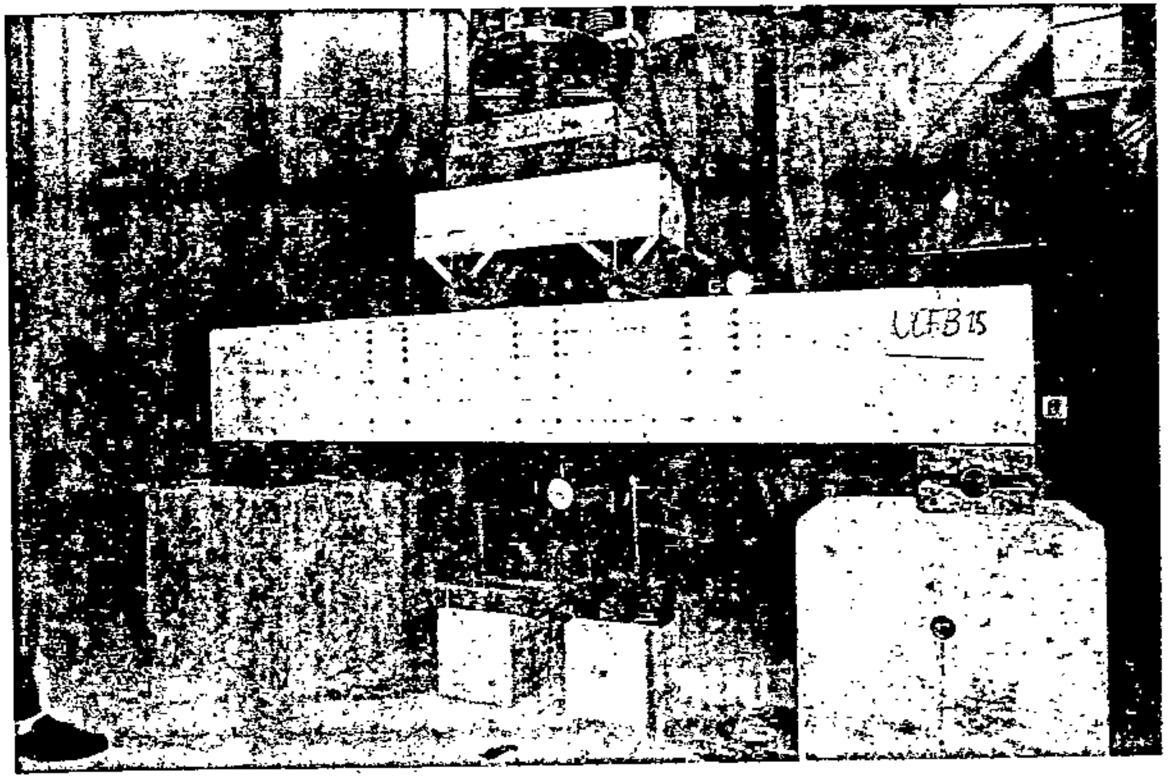
Fig 2-2 Details of the Reinforcement and cross section of beams of second set

2.3 DETAILS OF TEST BEAMS

To study the behaviour in bending, cracking, and deflection of uncured reinforced concrete beams with fiber, the experiment work consisted of tests on 12 reinforced concrete beams. The beams are divided in two sets according to the percentage of longitudinal steel ; the first group designed as under reinforced concrete beams $\rho = 0.0123$ and were of the same size of $200 \times 300 \times 2000$ mm., the 2nd group designed as over-reinforced concrete beams $\rho = .043$ were of the same size of $200 \times 200 \times 2000$ mm.. All the beams are tested under two symmetrically placed concentrated loads as illustrated in plate(2.2). All the beams have the same span = 2000 mm. with clear span of 1800. mm.

The two group, each of six beams will consisted of one cured and one uncured beam without fibers, the remaining four uncured beams were exposed to local environmental directly after opening the molds by keeping them outside the laboratory until tested after 28 days. These beams contained .5%, .75%, 1%, and 1.5% of fiber by total weight of concrete as shown in table (2.2) and table (2.3).

First set of six beams is denoted by CB_1, UCB_1, $UCB_1F.5$, $UCB_1F.75$, UCB_1F1, $UCB_1F1.5$. Second set of six beams is denoted by CB_2, UCB_2, $UCB_2F.5$, $UCB_2F.75$, UCB_2F1, $UCB_2F1.5$.



PLATE(2.2) - TEST SETUP OR LOAD ARRANGEMENT

Beam Name	Beam Size	ρ	Fiber percentage	a/d	type of curing
CB_1	$200 \times 300 \times 2000$	1.22%	0.0%	2.83	moist cured
UCB_1	$200 \times 300 \times 2000$	1.22%	0.0%	2.83	air cured
$UCB_1F.5\%$	$200 \times 300 \times 2000$	1.22%	0.5%	2.83	air cured
$UCB_1F.75\%$	$200 \times 300 \times 2000$	1.22%	.75%	2.83	air cured
$UCB_1F1\%$	$200 \times 300 \times 2000$	1.22%	1.0%	2.83	air cured
$UCB_1F1.5\%$	$200 \times 300 \times 2000$	1.22%	1.5%	2.83	air cured

Table 2.2 properties of different beams of first set

WHERE

- CB cured beam.
- UCB uncured beam.
- UCBF.50% uncured beam with 0.5% fibers.
- UCBF.75% uncured beam with .75% fibers.
- UCBF1.0% uncured beam with 1.0% fibers.
- UCBF1.5% uncured beam with 1.5% fibers.

Beam Name	Beam Size	ρ	Fiber percentage	a/d	type of curing
CB_2	$200 \times 200 \times 2000$	4.3%	0.0%	4.1	moist cured
UCB_2	$200 \times 200 \times 2000$	4.3%	0.0%	4.1	air cured
$UCB_2F.5\%$	$200 \times 200 \times 2000$	4.3%	0.5%	4.1	air cured
$UCB_2F.75\%$	$200 \times 200 \times 2000$	4.3%	.75%	4.1	air cured
$UCB_2F1\%$	$200 \times 200 \times 2000$	4.3%	1.0%	4.1	air cured
$UCB_2F1.5\%$	$200 \times 200 \times 2000$	4.3%	1.5%	4.1	air cured

Table 2.3 Properties of different beams of second set

To check the strength of the mix for each beam is adequate so 44 standard 150×300 mm test cylinders and 44 $150 \times 150 \times 150$ mm cubes and 44 standard prism of $100 \times 100 \times 530$ mm were casted with each batch of beams. They were tested in compression and indirect tension as a check for mix design strength. Also because of the unique properties of steel fiber reinforced concrete, air entertainment and workability measurements or slump requirements somewhat different from those of conventional concrete. Workability of steel fiber reinforced concrete was determined by the following two methods

- Slump Test.
- Vebe test (BS:1881)[17].

Controlled tests such as slump V_b and air entertainment had been done for each cast to check the workability and the percent of air entrained in the mixes.

2.4 MIXING AND CASTING

The quantities of cement, sand and water were weighted on platform scales and the steel fibers were weighted using a triple-beam balance. The total aggregate cement ratio of the mix was 5. and the water cement ratio was .88 . The details of the mixture proportions for one cubic meter of concrete were presented in section 2.1.

One third of the sand, coarse aggregate, cement and water were combined and thoroughly mixed. Two beams, two cylinders and two cubes were casted from each mix, then steel fibers were added as mixer continued running.

Finally the remainder of the cement and aggregate were added. The concrete was mixed in a rotating drum electric mixer with a total capacity of 0.1 of cubic meter. After mixing, the concrete was placed in the forms in approximately three layers. After each layer then the concrete in the forms was compacted by 1" an electric vibrator to achieve air voids as permitted by ASTM C31 and C192 and not by rodding [17].

2.5 TESTING EQUIPMENT

A 2000 KN capacity machine was used for compression tests , a bi-range -load capacity (100 AND 3000 KN) universal testing machine was used for the rupture tests, and compression bending machine test (400 kN capacity) with control capent. Plates(2.3, 2.4 & 2.5) show the testing machine.

The test beams were supported on knife edge supports resting on a large needle beam. The load was applied through a loading beam resting on 25 mm diameter steel rollers.

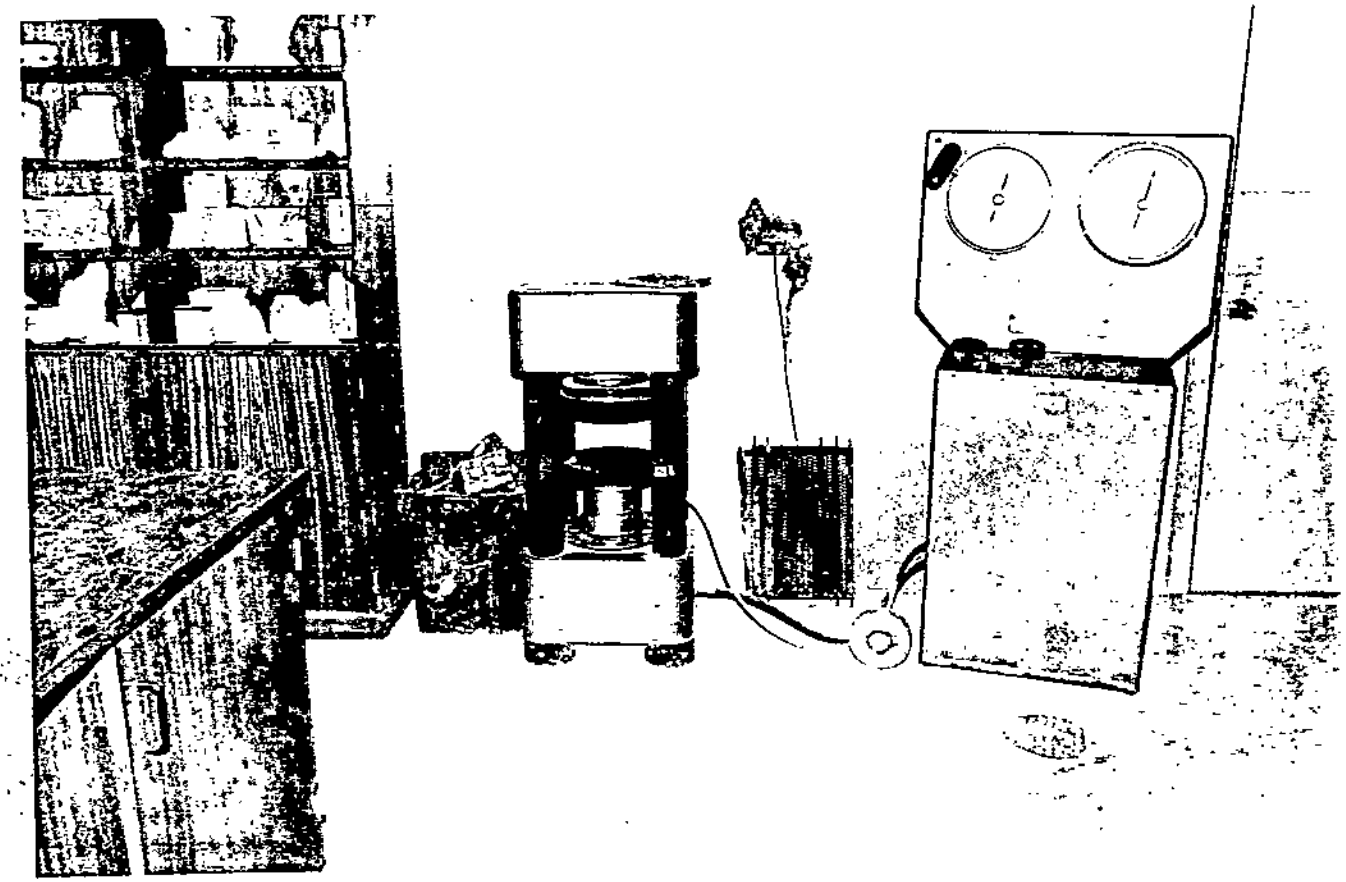
Deflections were measured by dial gauges fixed to the needle beam with clamps. These gauges had .01 mm graduation and were used to read deflection at the mid span of the beam. The beam test set-up is shown in plate (2.2).

2.5.1 Demic Demountable Strain Gauges

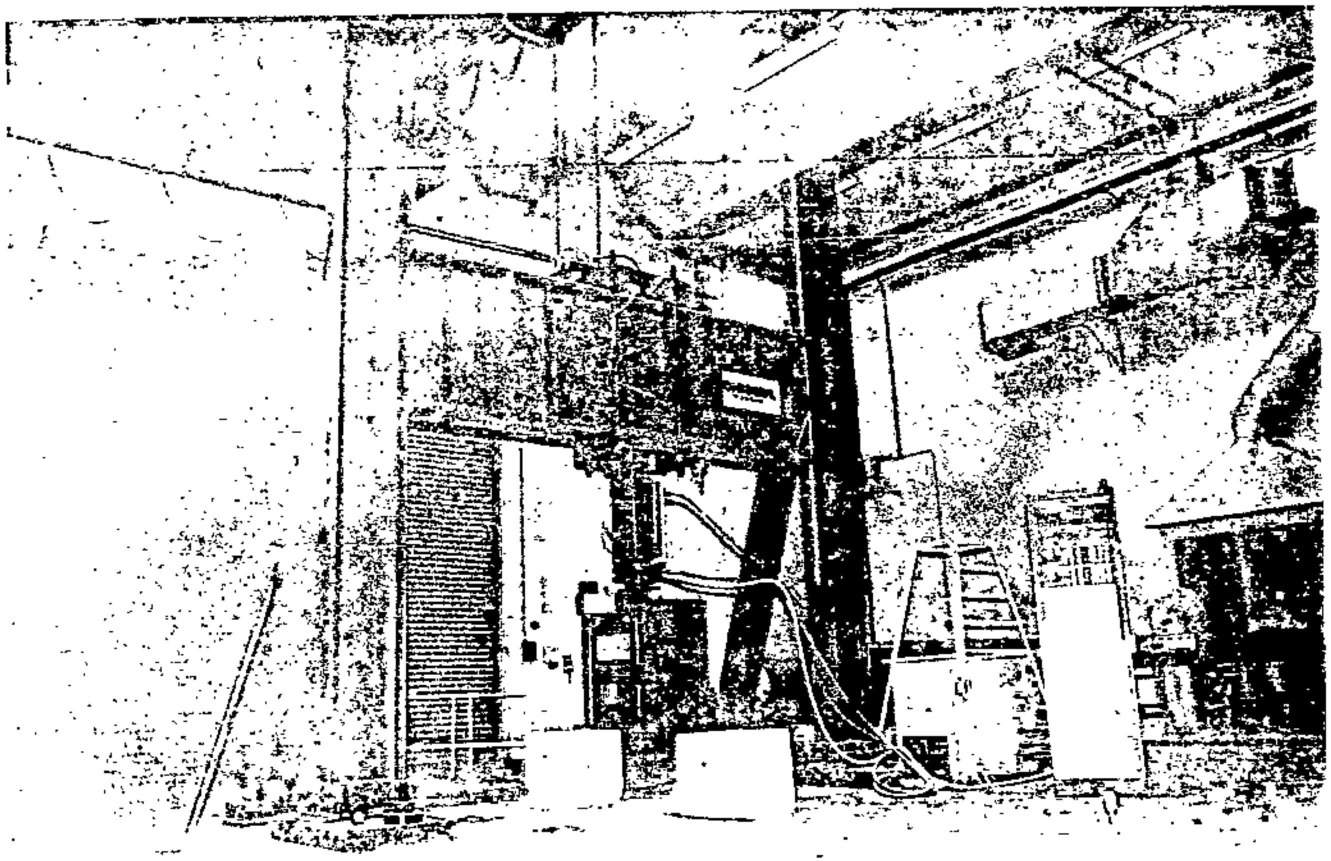
Demic demountable strain gauges of 100 mm gauge length and .001 mm sensitivity was used through the tests for the measurements of strains

2.5.2 Dial Gauges

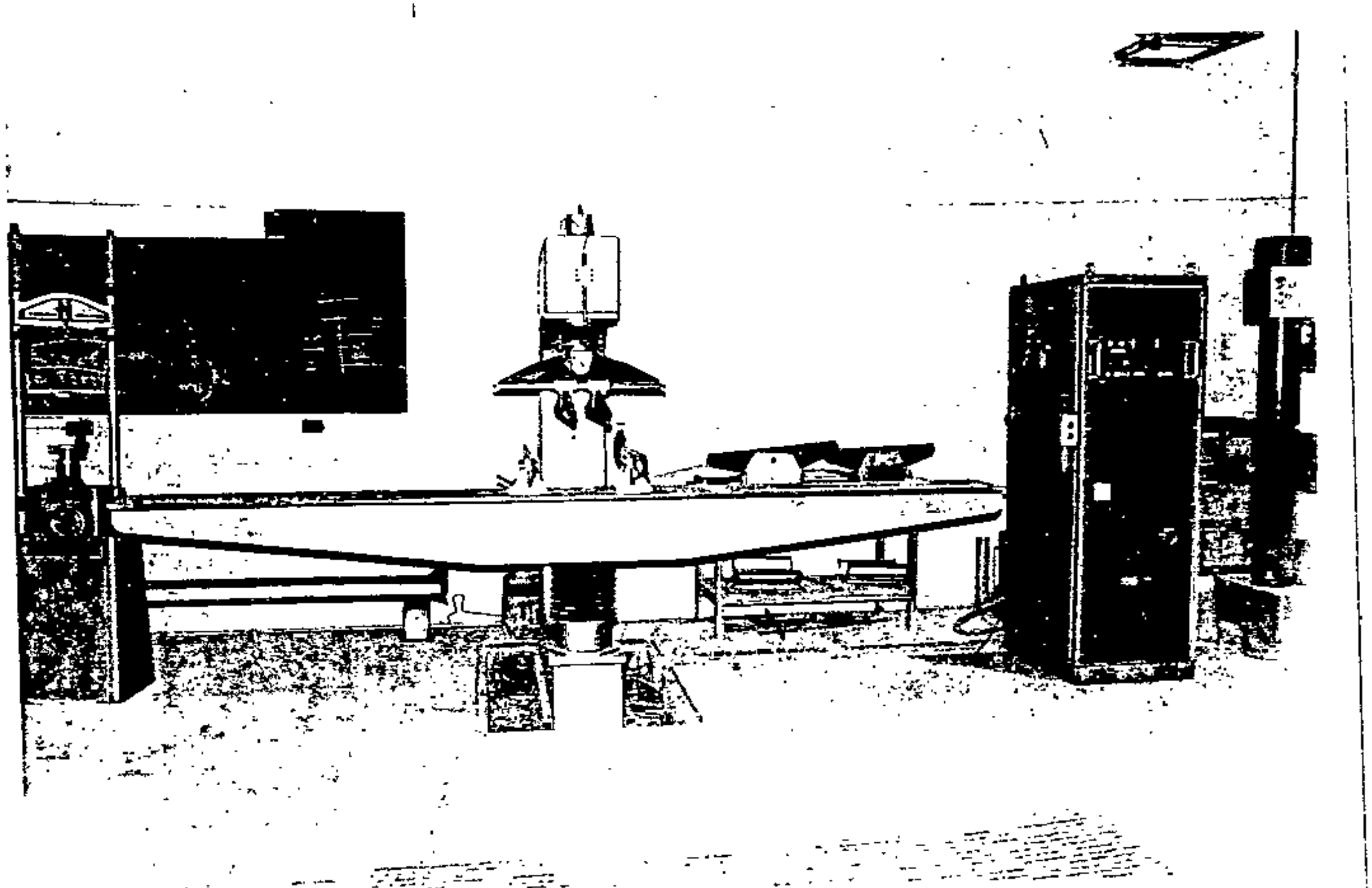
Dial gauges having .01 mm sensitivity were used thought the tests for the measurements of deflections.



PLATE(2.3) - COMPRESSION TESTING MACHINE



PLATE(4.3) - FLEXURAL TESTING MACHINE



PLATE(2.5) - UNIVERSAL TESTING MACHINE

2.6 TESTING PROCEDURE

The test beams were cast in groups of four beams, four cubes, and four cylinders. As previously stated one type of steel fibers was used. Two sets of beams were prepared [4]:

All the fibers were used directly as obtained from the manufacture. The total aggregate -cement ratio was kept constant at 5 and the water cement ratio was kept constant at 0.88. The total number of the beams specimens cast was 12 . After being transported to the laboratory, the beam specimens were measured for mounting on the testing machine. The 2000 mm long beam specimens were tested using two point symmetrical loading on a 1800 mm span.

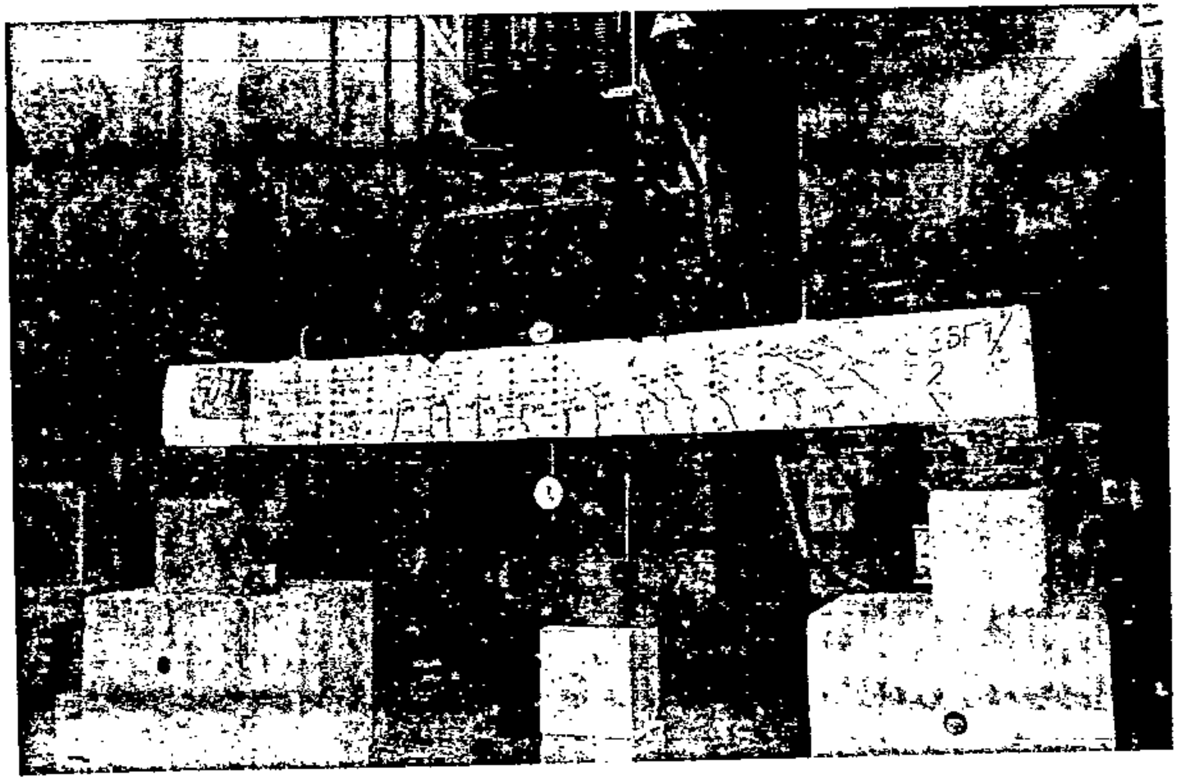
A pencil and flomaster pen were used to mark the points of contact of the knife edge supports, the mid point, to make the grid at about $\frac{2}{3}$ of the span to illustrate the crack location and how it propagates, also located and marked to aid in the placement of the stainless steel locating discs used for strain measurements and dial gages used for deflection measurement.

After sticking the steel discs by super glue,[5] the beam is left for 24 hours to get good bond between the discs and the concrete surface. The beam was then lifted on to the steel needle beam and the knife edge supports were aligned in their proper position. Twenty five mm diameter rollers, which transmitted the external loads to the beams, were positioned on their marked location. Steel bearing plates were used on all points of contact between the beam and specimens, the knife edge supports and the roller supports. The bearing plates

[4]see section 2.5
[5]Super Glue is Made In West Germany

ensured against local crushing of the concrete on knife edge support and roller support were free to move, acting as rollers. The loading beam was then lowered in place. Two plates were placed between the loading beam and the loading head of the testing machine to ensure proper load distribution, see plates (2.2 &2.6). After the dial gages were positioned at mid span The loading cycle began. The load was applied in 2.5 kn increments until ultimate failure was reached in each case. Deflection data and strains were taken at each load increment. After failure, the beam cross section was visually examined to note the steel fiber distribution. For each increment of load, after the first crack, crack pattern and propagation were marked on the beam for all the 12 beams. Load at first crack, and the ultimate load were noted for all beams.

The cylinders were capped, and cylinders, cubes, and beams were tested after 28 days according to ASTM specification C31-64 T to obtain compressive and tensile strengths of concrete used in beam specimens along with the beam tests as seen in plate (2.7).



PLATE(2.6) - LOADING SETUP AND ARRANGEMENT
FOR BEAM FROM SECOND SET

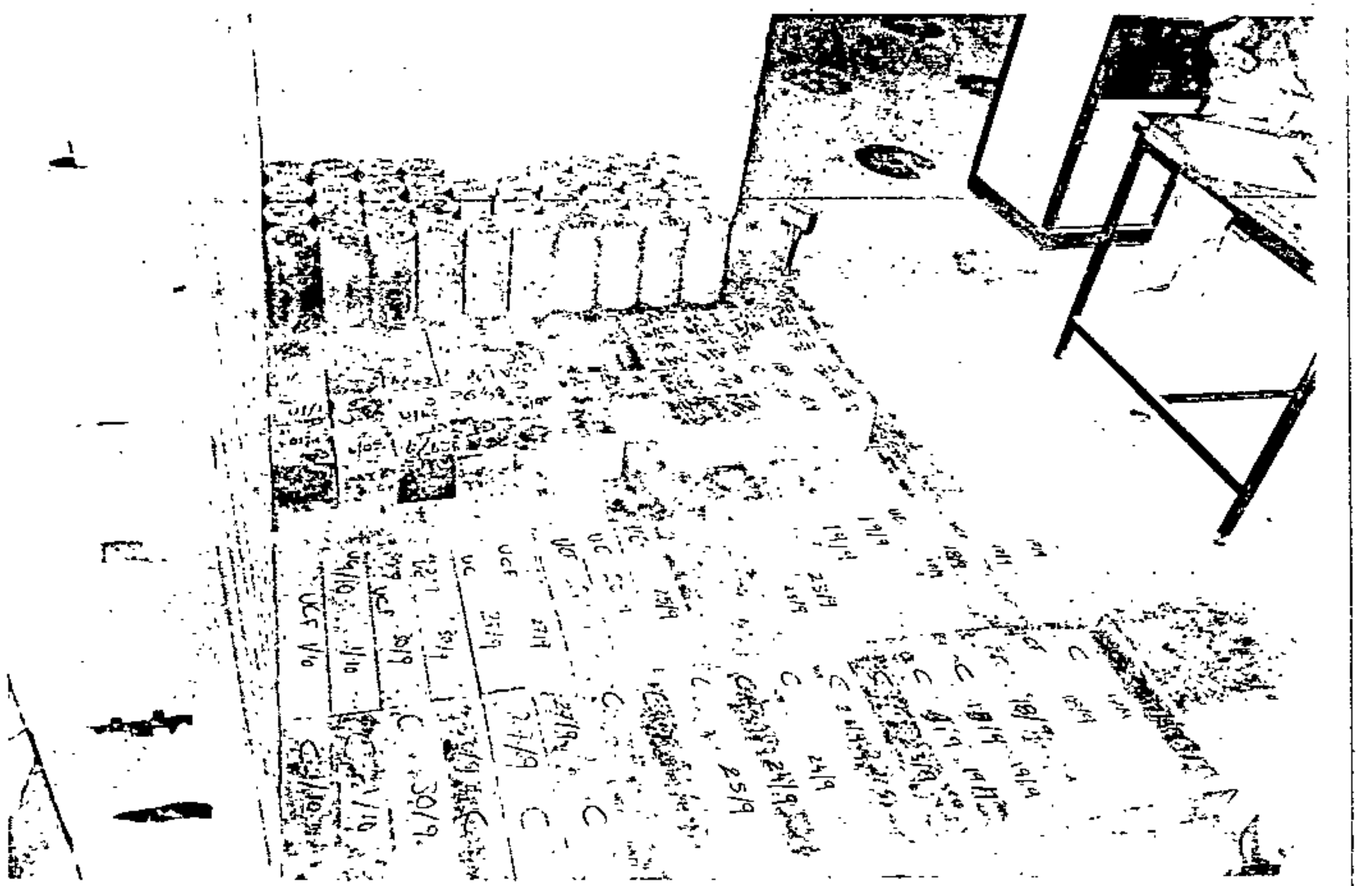


PLATE (2.7) - CURED AND UNCURED SPECIMENS FOR CONTROL TEST

Chapter 3

FLEXURAL THEORY

Information on methods for calculating the flexural strength of reinforced concrete beams and steel fibrous concrete beams is readily available, but there is currently little information available on methods for analysis of beams reinforced with a combination of steel bars and steel fibers.

A method is described for calculating the ultimate moment strength of concrete beams reinforced with a combination of steel bars and randomly distributed steel fibers. The method was developed in a study of structural uses of WIRND concrete for the Battelle Development Corporation, which holds United states and United Kingdom patents on metallic-fiber reinforced concrete.

The method described in this chapter is based on the ultimate strength method and takes into account the bond stress, fiber stress, fiber length diameter ratio, and amount of fiber. The strength computed for the fibrous concrete is added to strength contributed by the reinforcing bars to obtain the theoretical ultimate moment. Henger, and Doherty show good correlation between predicted strength and experimental value.[18]

3.1 ASSUMPTIONS

The following assumptions are made for the analysis method:

(a) The maximum usable strain at the extreme concrete compression fiber is .003 mm/mm.

(b) The strain in the reinforcing steel and concrete are directly proportional to the distance from the neutral axis.

(c) The compressive stress is represented by a rectangular stress block as in the ultimate strength design method.

(d) The tensile contribution of the fibers is represented by a stress block having a force equal to that required to develop the dynamic bond stress that is structurally effective in the tensile portion of the beam. The term " Dynamic bond stress " is used to describe the bond stress developed during fiber pullout, this term is equal to the bond from adhesive, friction and mechanical, see Figure 3.1 .

(e) Dynamic bond stress is taken as 2.297 MP[1]a which gives fiber stresses in the range OF 331 MPa to 586 MPa, depending upon fiber length and diameter. The ultimate strength of the beam occurs along with considerable cracking, indicating that fiber pull out occurring. The dynamic pullout load is somewhat less than the initial static or "breakout" load. From reference [24], the results show that the average bond stress at first crack is 3.57 N/ mm^2 compared to the ultimate bond stress of 4.15 N/ mm^2.

(f) Tensile stress in the fibrous concrete acts on the area having a minimum tensile strain in the fibers of σ_f/E_s, in which σ_f = stress in the fiber at

[1]Note: 1MPa = 1N/mm^2

the assumed bond stress; and E_s = modulus of elasticity of steel.

(g) A bond efficiency factor was illustrated in table 3.1 for different types of fibers, and equal to 1.1 for bekaret fibers.[19]

(h) Figure 3.1 illustrates the design assumptions in which the notation meanings are :

- a = depth of rectangular stress block.
- b = width of beam.
- c = distance from extreme compression fiber to neutral axis.
- d = distance from extreme compression fiber to centroid of tensile bar reinforcement.
- d' = distance from extreme compression fiber to centroid of top bar reinforcement.
- e=distance from extreme compression fiber to top of tensile stress block of fibrous concrete.
- ϵ_s = tensile strain in steel.
- ϵ_c = compressive strain in concrete.
- f_c = compressive strength of concrete.
- f_y = yield strength of reinforcing steel bar.
- A_s = area of tension bar reinforcement.
- C_c = compression force in concrete.
- D = total depth of beam.
- σ_t = tensile stress in fibrous concrete.
- E_s = modulus of elasticity of steel.
- $T_{fc} = \sigma_t b(D - e)$ = tensile force of fibrous concrete.
- $T_{rb} = A_s f_y$ =tensile force of bar reinforcement.

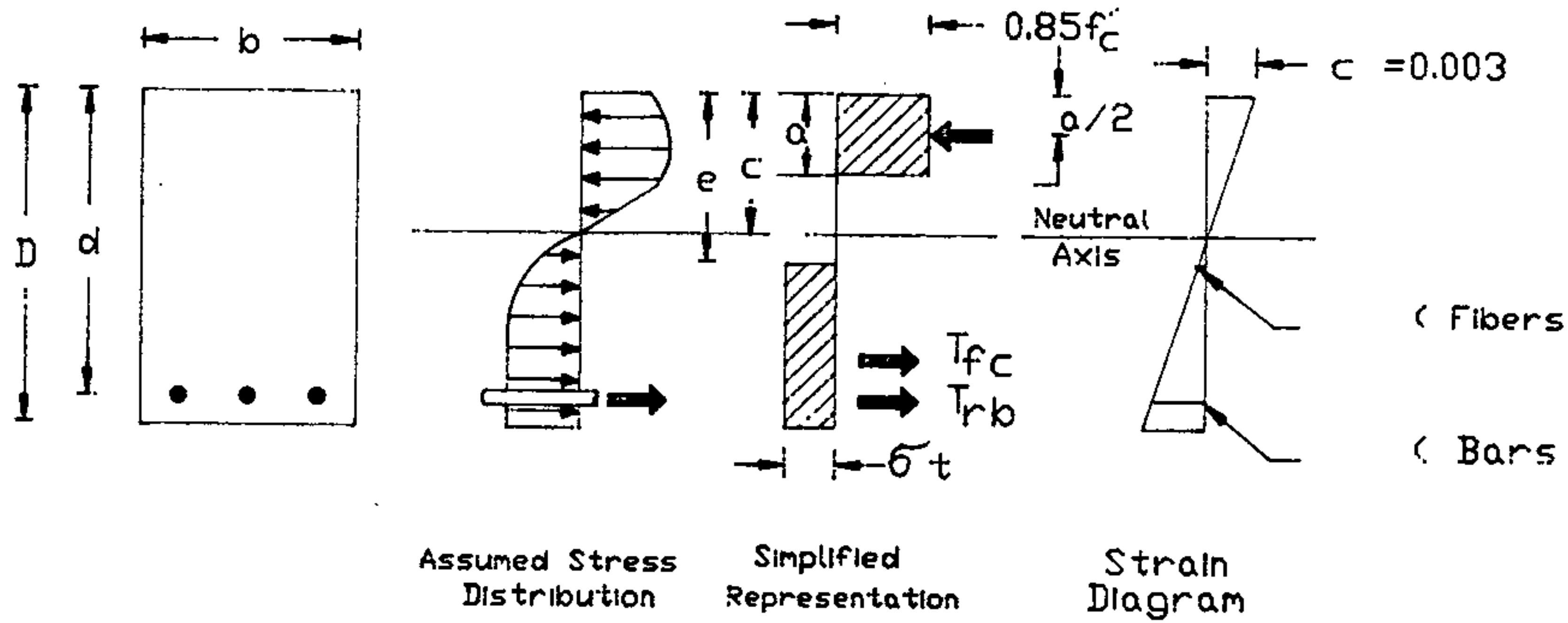


Fig 3.1 Stress - Strain Relationships

3.2 FORMULATION OF THE METHOD

The following equation represents the effective volume percentage of fiber in a matrix. This equation is slightly modified for use in this method

$$\frac{p_e}{p} = \frac{0.41 \times .82 l_f \tau_d d_f}{\sigma_f d_f^2} \tag{3.1}$$

Where

- Pe= effective volume percentage of fibers.
- p = percentage by volume of fiber.
- .41= fraction of fibers effective in random distribution.

  or orientation factor.
- .82= length correlation factor.
- l = fiber length.
- τ_d = dynamic bond stress between the fiber and the matrix.
- d_f = fiber diameter.
- σ_f = tensile stress developed in fiber during pullout.

Equation 3.1 may be reduced to :

$$p_e \sigma_f = .41 \times .82 \times \tau_d \frac{l}{d_f} p \tag{3.2}$$

The term $P_e \sigma_f$ represents the force per unit area developed by the bond strength of the fibers that are effective in a unit cross section. The right hand side of the equation is the net dynamic bond stress times the fiber aspect ratio, l/d_f, times the volume percentage of fibers.

Assuming τ_d to be 2.297 MPa (333 psi) incorporating a bond efficiency factor, F_{be}, and dividing by 100 to correct the units of P as a percentage, Equation 3.2 reduces to[16] :

$$\sigma_t = 0.00772\frac{l}{d_f}pF_{be} \tag{3.3}$$

In which σ_t represents the effective tensile stress in the fibrous concrete or

$$P_e\sigma_f = \sigma_t \tag{3.4}$$

And F_{be} : Bond efficiency factor.

Equating the forces shown in figure 3.1 :

$$C_c = T_{rb} + T_{fc} \tag{3.5}$$

Where[2]

$$C_c = .85\beta_1 f_c' bc \tag{3.6}$$

$$T_{rb} = A_s f_y \tag{3.7}$$

$$T_{fc} = \sigma_t b(D - e) \tag{3.8}$$

And by similar triangles, see strain diagram in figure 3.1

$$e = \frac{.003 + \epsilon_{sf}}{.003}c \tag{3.9}$$

Where

ϵ_{sf} =strain in steel fiber and

$$\epsilon_{sf} = \frac{\sigma_f}{E_s} \tag{3.10}$$

substitute equations 3.6, 3.7 & 3.8 in equation 3.5 so

$$.85\beta_1 f_c' bc = A_s f_y + \sigma_t b(D - e) \tag{3.11}$$

[2]Per ACI 318-71 (para 10.2.7): The fraction β_1 shall be taken as .85 for strengths, f_c', up to 4000 psi (27.6MPa) and shall be reduced continuously at a rate of 0.05 for each 1000. psi (6.9 MPa) of strength in excess of 4000 psi (27.6).

Substitute e from equation 3.9 in equation 3.11 to solve for c

$$c = \frac{A_s f_y + \sigma_t b D}{.85\beta_1 f_c' b + \sigma_t b k_s} \quad (3.12)$$

Where

$$k_s = \frac{.003 + \epsilon_{st}}{.003} \quad (3.13)$$

and

$$a = \beta_1 c \quad (3.14)$$

Equating moments about C_c, using $\left(d - \frac{a}{2}\right)$ as the moment arm for T_{cb} and $\left(\frac{D+e-a}{2}\right)$ as the moment arm for T_{fc}, we obtain the theoretical moment strength M_t:

$$M_t = A_s f_Y \left(d - \frac{a}{2}\right) + \sigma_t b (D - e) \left(\frac{D + e - a}{2}\right) \quad (3.15)$$

If the compression steel is not neglected, as shown in figure 3.2 so equation 3.11 will be modified to the following equation

$$.85\beta_1 f_c' bc + A_s' f_s' = A_s f_y + \sigma_t b(D - e) \quad (3.16)$$

Where

$A_{s'}$ = Area of compressive reinforcement steel.

$f_{s'}$ = the stresses in compressive steel, It is equal to

$$f_s' = \epsilon_s' E_s \quad (3.17)$$

Where ϵ_s' : Strain in compressive steel. From figure 3.2 b and by similar triangles:

$$\epsilon_s' = \frac{c - d'}{c} .003 \quad (3.18)$$

Substitute equation 3.18 in equation 3.17 to get equation 3.19 so :

$$f_s' = E_s \times \frac{c - d'}{c} .003 \quad (3.19)$$

Substitute equation 3.19 & 3.9 into equation 3.16 , from this equation find the neutral axis position by solution of quadratic equation as

$$c = \frac{-\beta + \sqrt{\beta^2 - 4\alpha\gamma}}{2\alpha} \tag{3.20}$$

Where

$$\alpha = .85\beta_1 F_c' b + \sigma_t b\left(\frac{.003 + \epsilon_{sf}}{.003}\right) \tag{3.21}$$

$$\beta = E_s A_s' - A_s f_y - \sigma_t b D \tag{3.22}$$

$$\gamma = -E_s A_s' d' \tag{3.23}$$

Add the effect of the compression steel to equation 3.15 to get

$$M_t = A_s f_Y\left(d - \frac{a}{2}\right) + \sigma_t b(D - e)\left(\frac{D + e - a}{2}\right) + A_s' f_s'\left(\frac{a}{2} - d'\right) \tag{3.24}$$

3.3 EXAMPLE

The following example analysis illustrate the use of Equations 3.3 and 3.15 or 3.24 to determine the strength of the beam, a cross section is shown in figure 3.2, in which :

b=200. mm

D=302. mm

d= 247. mm

p=.462%

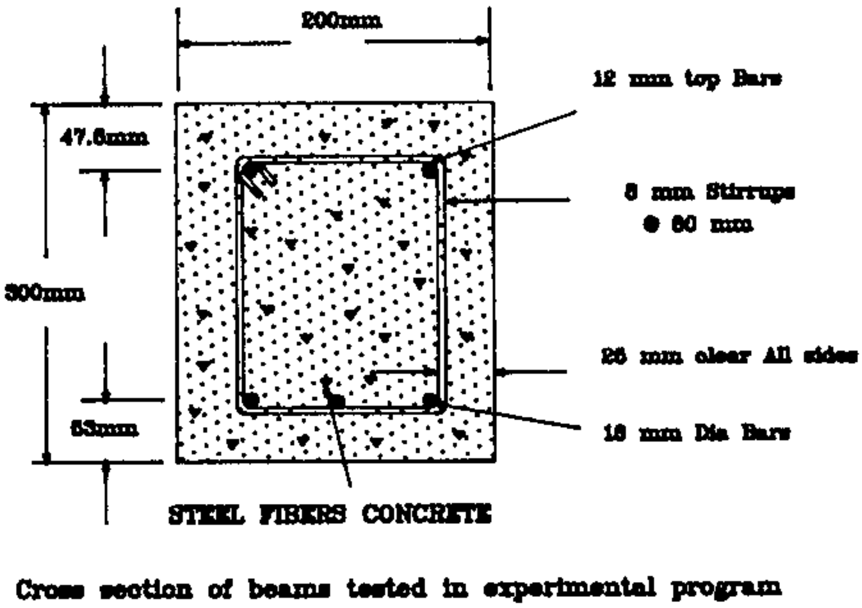


FIG 3.2 Cross section of beams tested in experimental program

As=584 mm^2

A_s'=272.2 mm^2

fy=380 N/mm^2

fibers=DRAMIX .8X60 mm

l/d_f =75

f_c'=21 N/mm^2

τ_d=2.297 N/mm^2

$E_s = E_{sf}$=2×10^5 N/mm^2

And F_{BE} =1.1 (bond efficiency factor for BEKAERT DRAMIX-"ZC" pattern fiber) table(3.1)

(a) Solve for σ_f:

$$\sigma_f = 2\tau_d F_{be}\frac{l}{d_f}$$

$$\sigma_f = 2 \times 2.297 \times 1.1\frac{60}{.8} = 379.0\text{N/mm}^2$$

$$\epsilon_{sf} = \frac{\sigma_f}{E_s}$$

(b) Solve for ϵ_{sf}:

$$\epsilon_{sf} = \frac{379.0}{200 \times 10^3} = 0.001895$$

(c) Solve for σ_t :

$$\sigma_t = 0.00772 \times F_{be} \times p \times \frac{l_f}{d_f}$$

$$\sigma_t = 0.00772 \times 1.1 \times .462 \times \frac{60.}{.8} = 0.2942\text{N/mm}^2$$

(d) Solve for e (by similar triangles, see strain diagram fig 3.1):

$$e = \frac{\epsilon sf + 0.003}{0.003}c$$

$$e = \frac{0.00195 + 0.003}{.003}c = 1.632 \times c$$

(e) solve for c, if we neglect the top bars use equation 3.12 to find c

$$c = \frac{A_s f_y + \sigma_t b D}{.85\beta_1 f_c' b + \sigma_t b k_s}$$

c=65.4 mm

But if we consider the compressive steel use equations 3.20 to 3.23 to find c :

$$k_s = \frac{\epsilon_{sf} + .003}{.003} = 1.662$$

$$c = \frac{-\beta + \sqrt{\beta^2 - 4\alpha\gamma}}{2\alpha}$$

where

$$\alpha = .85\beta_1 F_c' b + \sigma_t b k_s$$

$$\alpha = .85 \times 21 \times 200 + .2942 \times 200 \times 1.632 = 3666.00$$

$$\beta = E_s A_s' - A_s f_y - \sigma_t b D$$

$$\beta = 2 \times 10^3 \times 227.2 - 584.5 \times 380 - .2942 \times 200 \times 302 = 45200238$$

$$\gamma = -E_s A_s' d'$$

$$\gamma = -2 \times 10^3 \times 272.4 \times 50 = 2272 \times 10^6$$

Substitute $\alpha, \beta \& \gamma$ in the above equation to get c so

c= 50.06 mm

Note that c locates the neutral axis. Since the location of the $2 - \phi 12$ mm top bars is coincides to the neutral axis. Their contribution is negligible and may be ignored.

(f) solve for a:

$$a = \beta_1 c = 0.85 \times 65.4 = 55.59\text{mm}$$

(g) solve for e :

$$e = 1.632 \times 65.4 = 106.73\text{mm}$$

(h) find Mt using equation (3.15) :

$$M_t = A_s f_y (d - \frac{a}{2}) + \sigma_t b (D - e)(\frac{D + e - a}{2})$$

$$M_t = 50.684 \text{kN.m}$$

but the value of M_t from equation 3.24

$$M_t = 50.094 + .217 + .0047 = 52.275 \text{ kN.m}$$

Note that method will calculate the theoretical moment strength of conventional (singly - reinforced) beams by formula $M_t = A_s f_y (d - \frac{a}{2})$ if the fiber percentage is set equal to zero. But the value of a changed to

$$a = \frac{A_s f_y}{.85 f_c' b}$$

And in previous example it is equal to 73.19 mm so M_t= 46.732 kN.m

From this example it is clear that the strength of fibrous reinforced concrete from the equation 3.15 is greater than strength of conventional concrete by about 8.25%, and if we add the effect of compression steel to the effect of addition .462% of fiber was increased to 11.86%. So the effect of compression steel in fibrous reinforced concrete is about 3.1%.

Table 3.1 bond efficiency factor for various fiber shapes[19]

Fiber Description	Bond Efficiency Factor, F_{be}
i. Smooth, straight, round steel fibers unplated, brass plated or liquor finsh, e.g., chooped wire	1.0
ii. smooth, or slightly irregular, straight steel fibers, square or rectangular, (e.g. slit- sheet fibers)	1.0
iii. smooth, round, crimped-end steel fibers, BEKAAERT DRAMIX - "ZC" pattern fibers.	1.1
iv. deformed, straight, round steel fibers , unplated or brass plated, national standard duoform fibers.	1.2
v. melt- extraction, irregular shape, carbon steel fibers, light melt oxide finish, FIBTIC "CC" fibers.	1.2

Chapter 4

TEST RESULTS

4.1 Introduction

Two sets of beams were designed to study the flexural behaviour of cured, and uncured conventionally reinforced beams and their results will be compared with other four beams containing fibers of .5%, .75% ,1% and 1.5% of total weight.

All beams in first set were reinforced with 1.22% of ordinary steel with minimum yield strength of 380 N/mm^2 and haven one cross section of $200 \times 300 \times 2000$ mm, but all beams of second set are reinforced with 4.2% of ordinary steel with minimum yield strength of 340.N/mm^2 and have a cross section of $200 \times 200 \times 2000$ mm.

All the beams in both set were instrumented to study the cracking and ultimate load, deflection, strain and cracking characteristic over the entire range of loading up to failure. From the large amount of data obtained, only typical results are presented here but the data presented are representative of the general behaviour of all the beams.

4.2 Cracking load

The load at first flexural crack for each beam was found by visual inspection and recorded, (First crack is defined as the first visible crack at a 30.cm distance from the bare eye). After we finish the experiment work the load associated with the first crack will be checked with the first kink in the load deflection curve of the beam. The observed results were tabulated in table 4.1 and were shown also in the figures (4.1 through 4.6) for the first set, and in table 4.2 and figures 4.8 throut 4.13 for the second set. Crack load divided by ultimate load for each beam is listed in column 4 in table 4.1 &, table 4.2 , also the increment due to inclusion of steel fiber were listed in column 5.

4.3 Load at failure

Ultimate loads detected visually and were associated with concrete crushing, the machine test stopped automatically when the flexural rigidity of the beam became zero or negative. The testing machine recorded the maximum ultimate loads. For the first set the results were tabulated in table 4.1 and were also shown in figures (4.1 through 4.6). But results for the second set were tabulated in table 4.2 and shown in figures 4.8 through 4.13. The increase in the failure load due to inclusion of fiber in different beams is calculated and listed in column 6 in tables 4.1 and 4.2

Beam name	Crack load (KN) column 2	Ultimate load (KN) column 2	Crack divided by ultimate column 4	change in crack load column 5	change in ultimate load column 6
CB_1	23.	140.1	16.4%		
UCB_1	22.9	129.9	17.6%	-.43%	-7.3%
$UCB_1F.5\%$	23.5	160.	14.7%	2.17%	14.2%
$UCB_1F.75\%$	24.8	170.	14.6%	7.82%	21.3%
$UCB_1F1\%$	25.9	172.	15.1%	12.6%	22.8%
$UCB_1F1.5\%$	28.	185.9	15.1%	21.7%	32.7%

Table 4.1 Crack and Ultimate load for different beams in first set.

Beam name	Crack load (KN) column 2	Ultimate load (KN) column 2	Crack divided by ultimate column 4	change in crack load column 5	change in ultimate load column 6
CB_2	23.5	135.0	17.4%		
UCB_2	20.0	120.0	16.7%	-14.89%	-11.11%
$UCB_2F.5\%$	21.5	138.7	15.5%	-8.51 %	2.74%
$UCB_2F.75\%$	23.5	140.1	16.8%	00.00%	3.8%
$UCB_2F1\%$	26.5	120.0	22.1%	12.77%	-11.11%
$UCB_2F1.5\%$	28.	134.2	20.9%	19.15%	-.60%

Table 4.2 Crack and Ultimate load for different beams in second set.

4.4 Deflection characteristics

All beams were gradually loaded at increments of 2.5 kN and deflection were measured by dial gages up to 50mm. Beyond this maximum deflection the results of deflection were taken from the machine itself. The correspondence between the recorded deflections by the dial gages and the deflections given by the machine was satisfactory in the range of 2-50 mm. The typical results of deflection at cracking, at 80 kN and at experimental ultimate load were tabulated for the first set in table (4.3), also for the second set were tabulated in table 4.4.

deflection(mm) at:			
Beam Name	Crack load column 2	80 kN column 3	Ultimate Load Column 4
CB_1	.69	3.5	63
UCB_1	.83	4.15	53
$UCB_1F.5\%$	.76	3.65	58
$UCB_1F.75\%$	.65	3.25	74
$UCB_1F1\%$	.72	3.53	79
$UCB_1F1.5\%$	.71	3.11	98.

Table 4.3 Deflection at different stages of loading in the first set.

The deflection curves for all tested beams were drawn, and in addition to that, the first part up to about 100 kN for each beam were drawn separately using a larger scale, to clarify the characters of this curve, these curves can be seen in figures 4.1 through 4.14, and for the comparison purpose all results for all beams up to about 100 kN were drawn in figures 4.7 and 4.14

deflection(mm) at:			
Beam Name	Crack load column 2	80 kN column 3	Ultimate Load Column 4
CB_2	.69	3.5	72.3
UCB_2	.83	4.15	48.8
$UCB_2F.5\%$	.76	3.65	99.1
$UCB_2F.75\%$	1.5	5.94	107.5
$UC_2BF1\%$	1.62	5.66	118.4
$UCB_2F1.5\%$	1.4	5.43	139.8

Table 4.4 Deflection at different stages of loading in the second set.

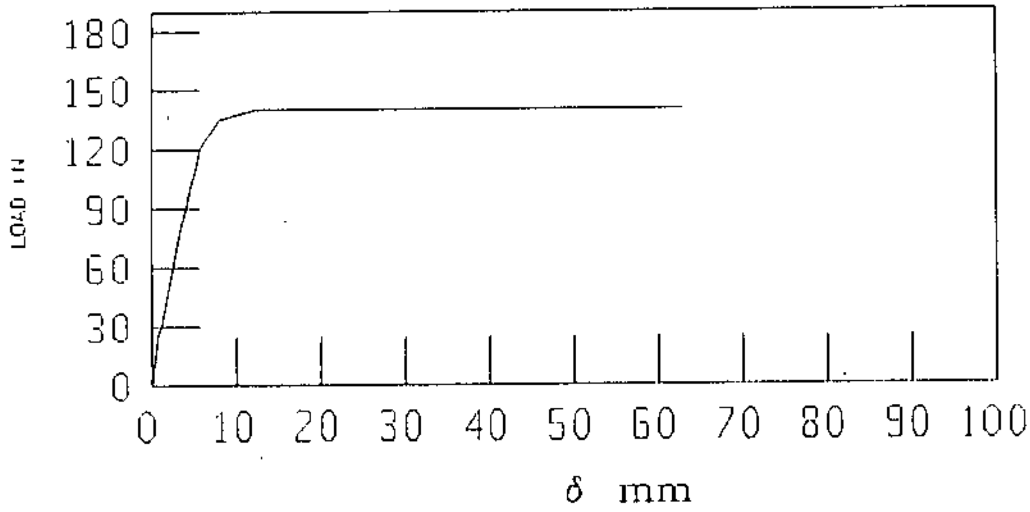


FIG.(4.1a): LOAD DEFLECTION CURVE FOR BEAM # 1

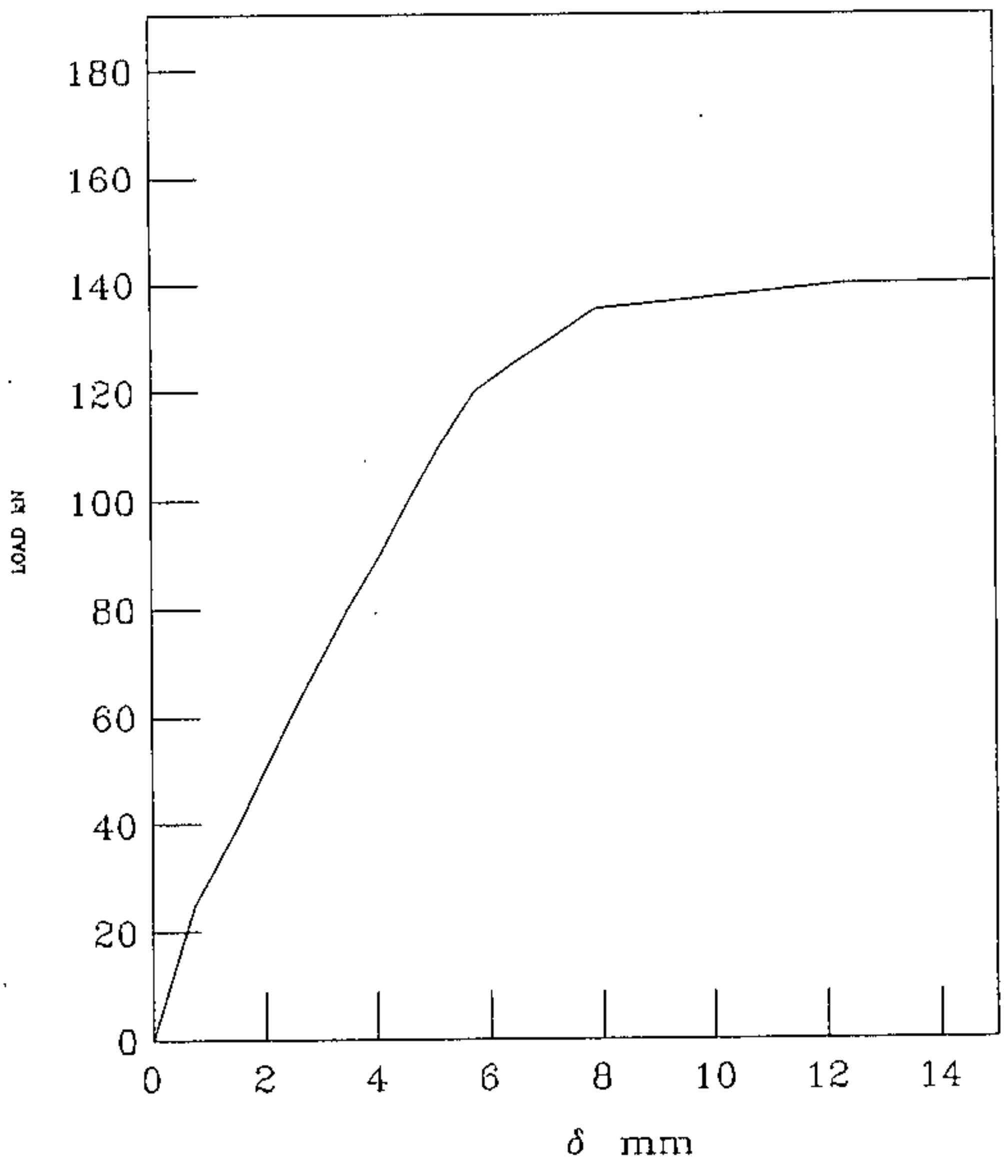


FIG.(4.1b) :LOAD DEFLECTION CURVE FOR BEAM # 1

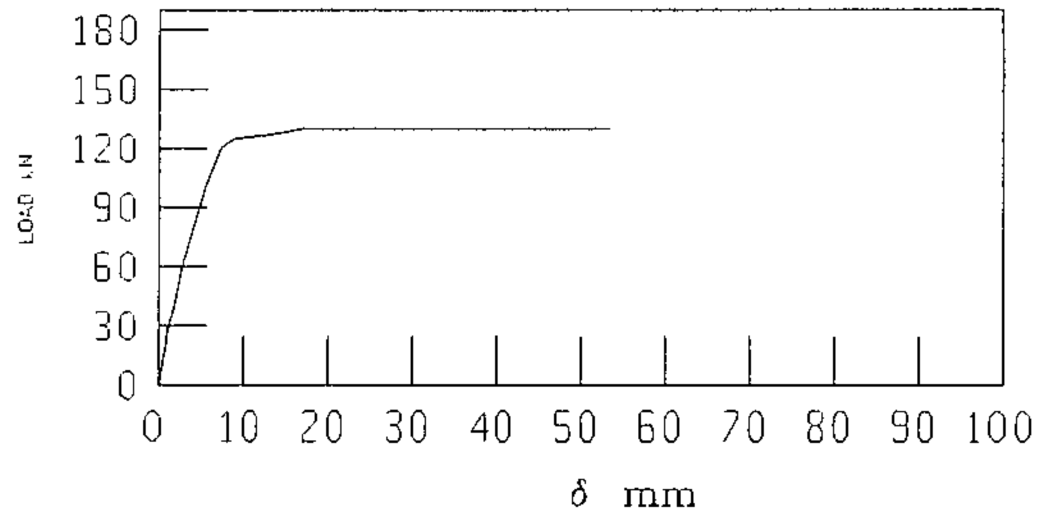


FIG.(4.2a): LOAD DEFLECTION CURVE FOR BEAM # 2

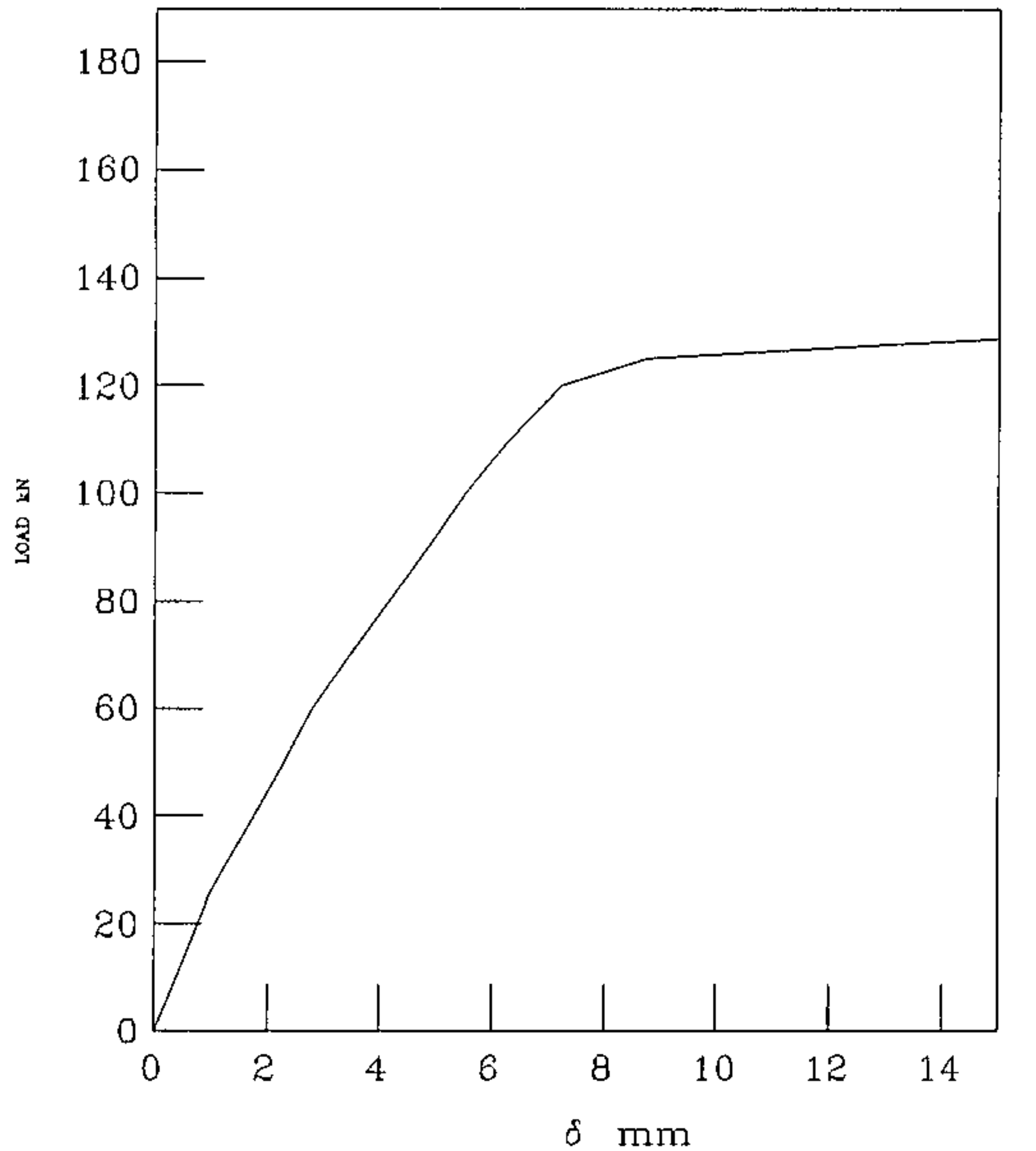


FIG.(4.2b): LOAD DEFLECTION CURVE FOR BEAM # 2

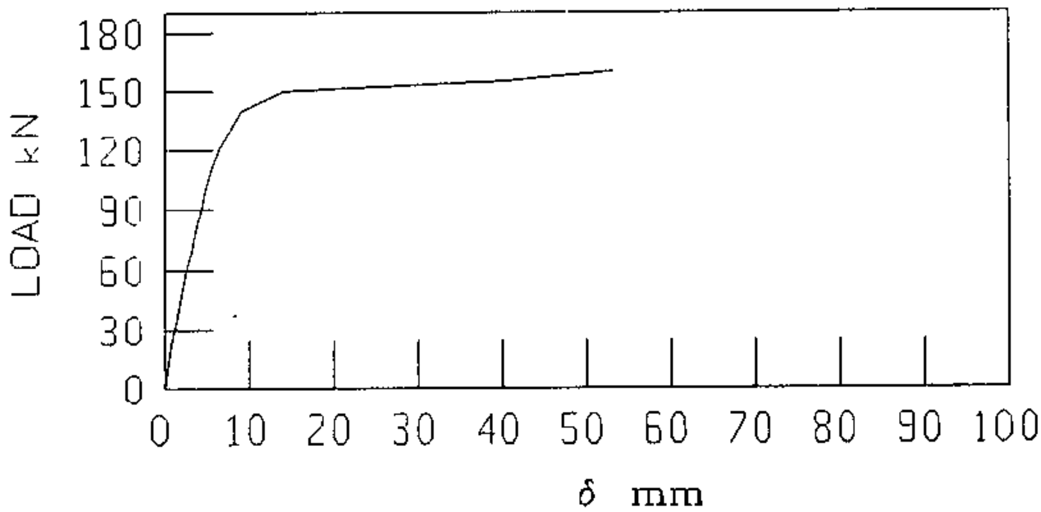


FIG.(4.3a): LOAD DEFLECTION CURVE FOR BEAM # 3

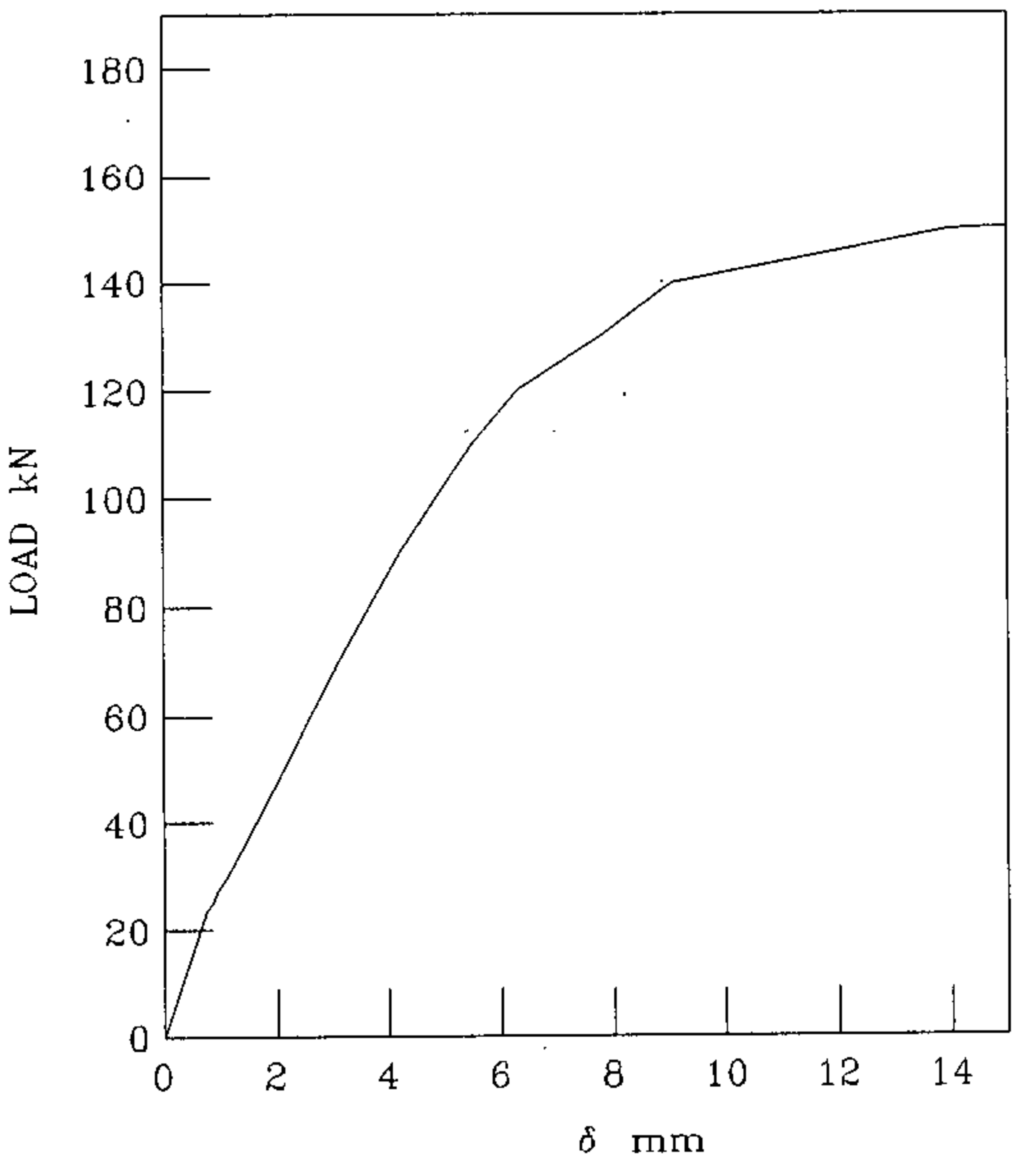


FIG.(4.3b) :LOAD DEFLECTION CURVE FOR BEAM # 3

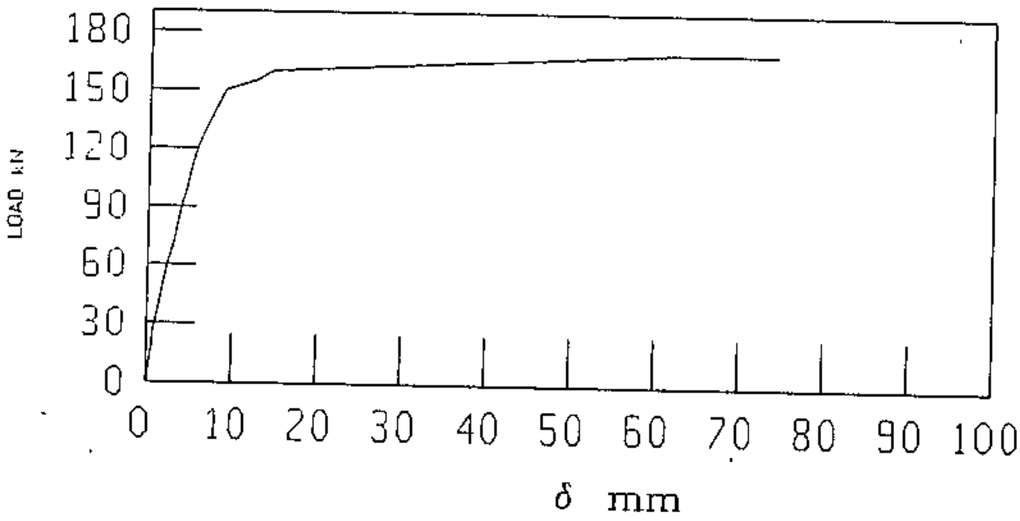


FIG.(4.4a): LOAD DEFLECTION CURVE FOR BEAM # 4

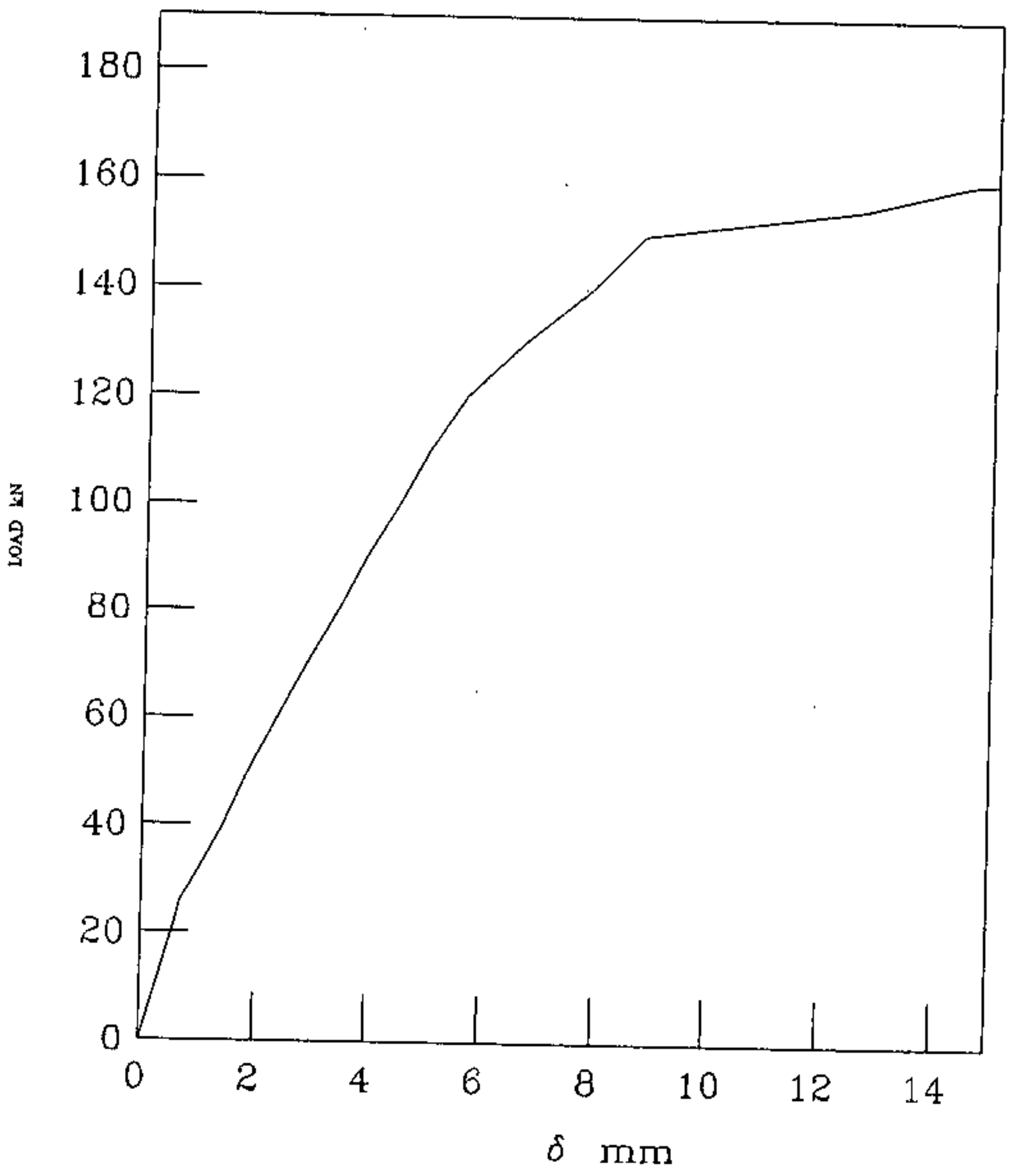


FIG.(4.4b): LOAD DEFLECTION CURVE FOR BEAM # 4

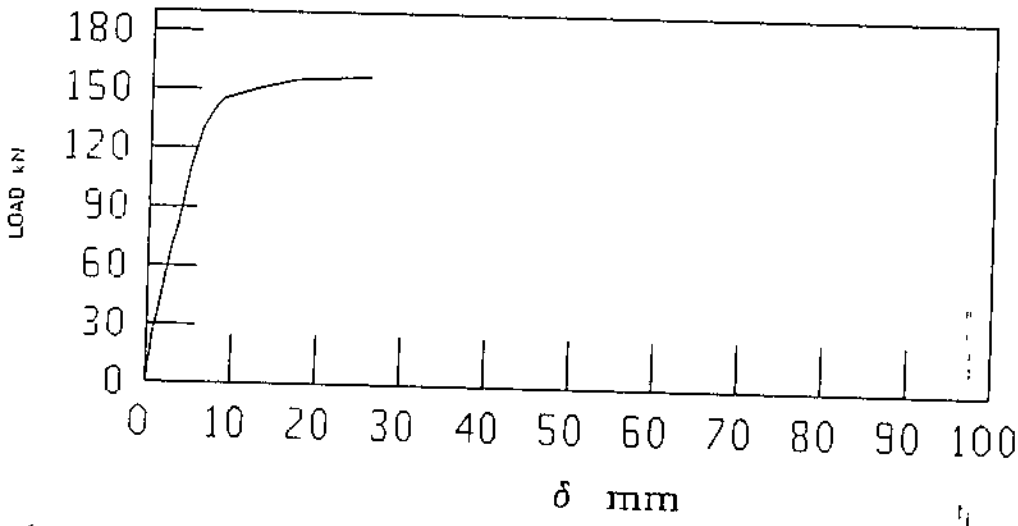


FIG.(4.5a): LOAD DEFLECTION CURVE FOR BEAM # 5

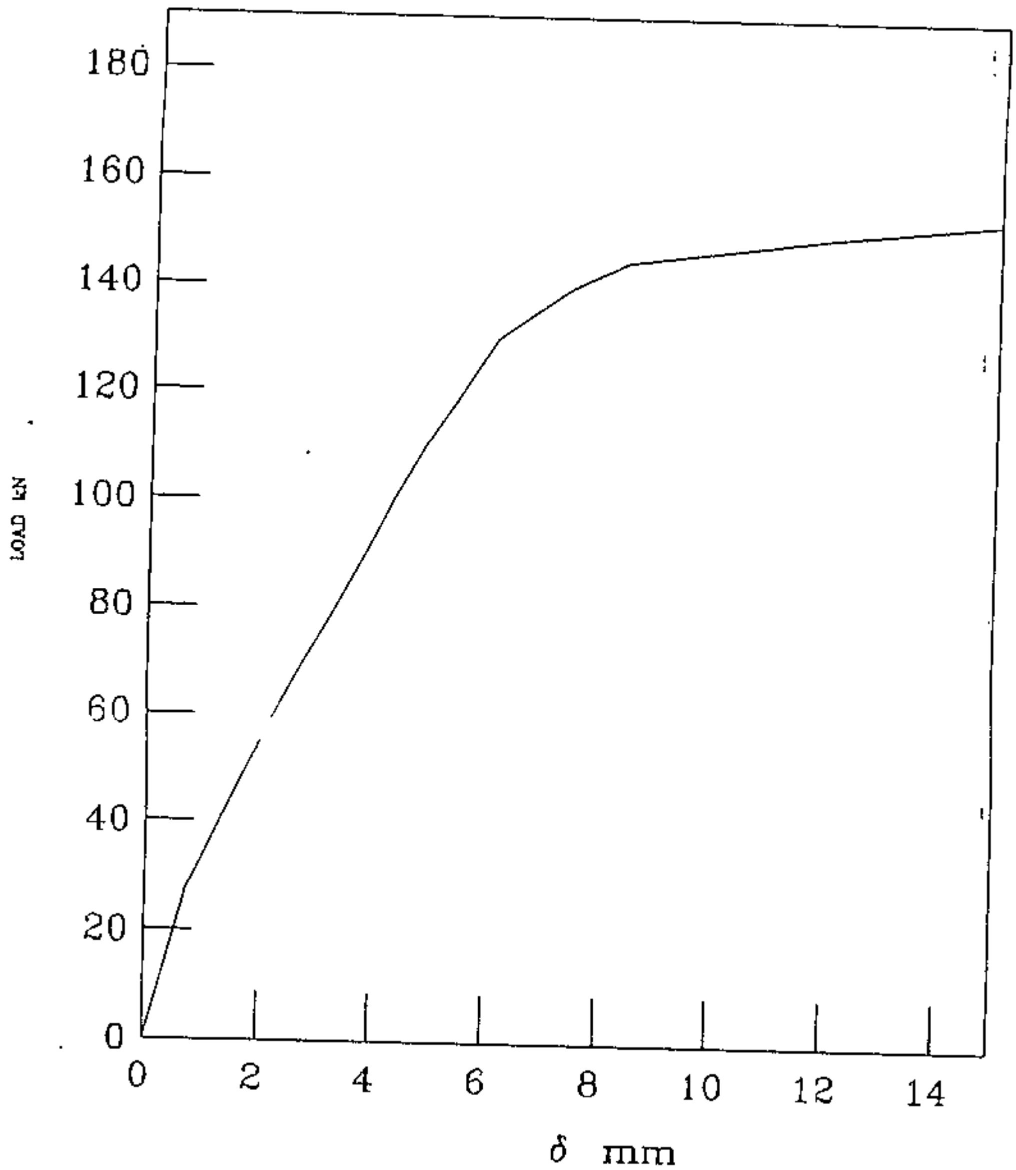


FIG.(4.5b): LOAD DEFLECTION CURVE FOR BEAM # 5

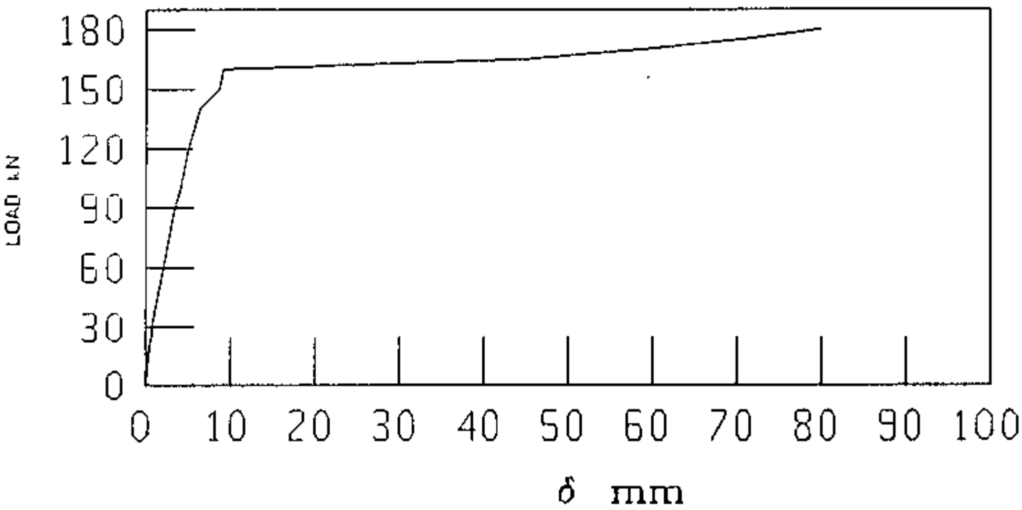


FIG.(4.6a):LOAD DEFLECTION CURVE FOR BEAM # 6

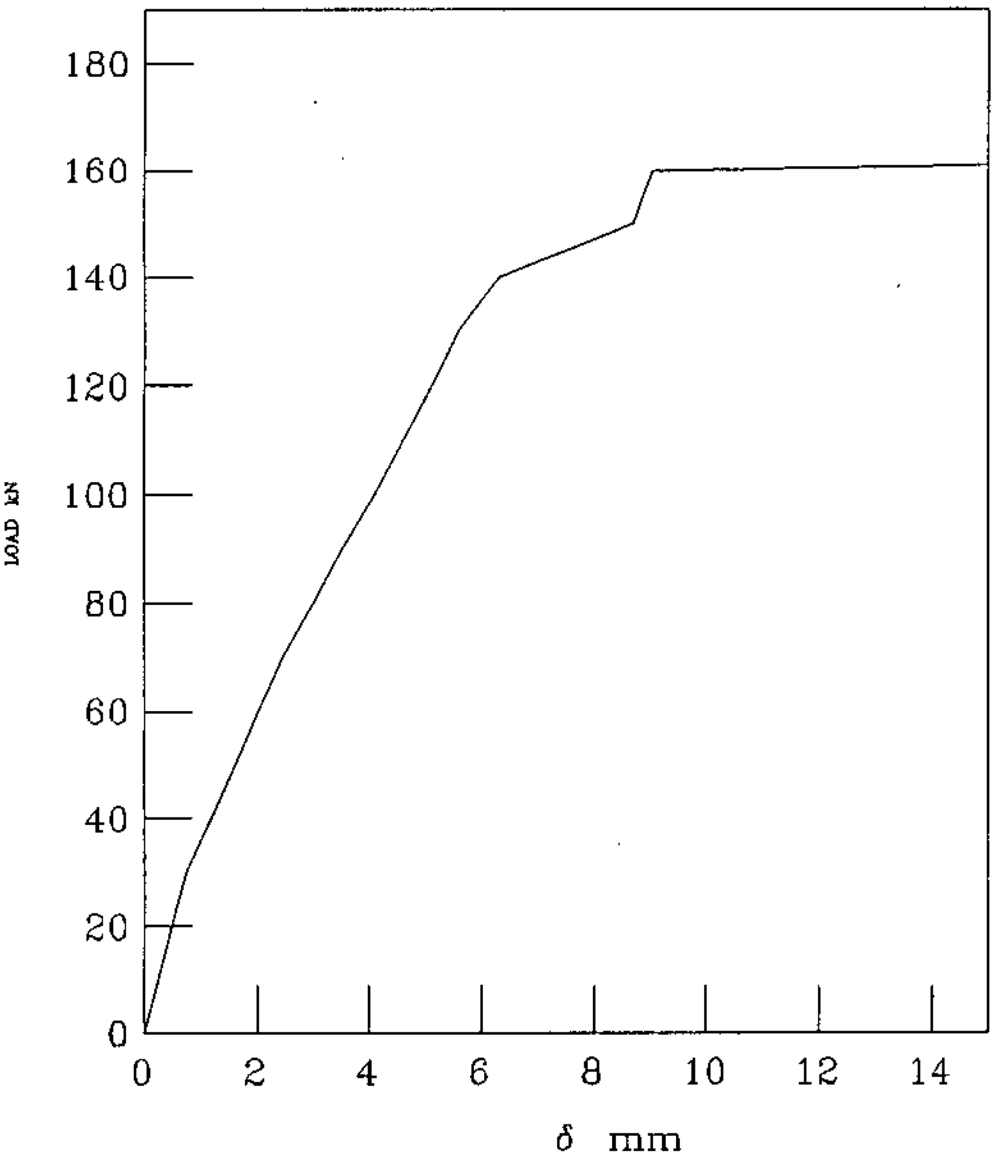


FIG.(4.6b): LOAD DEFLECTION CURVE FOR BEAM # 6

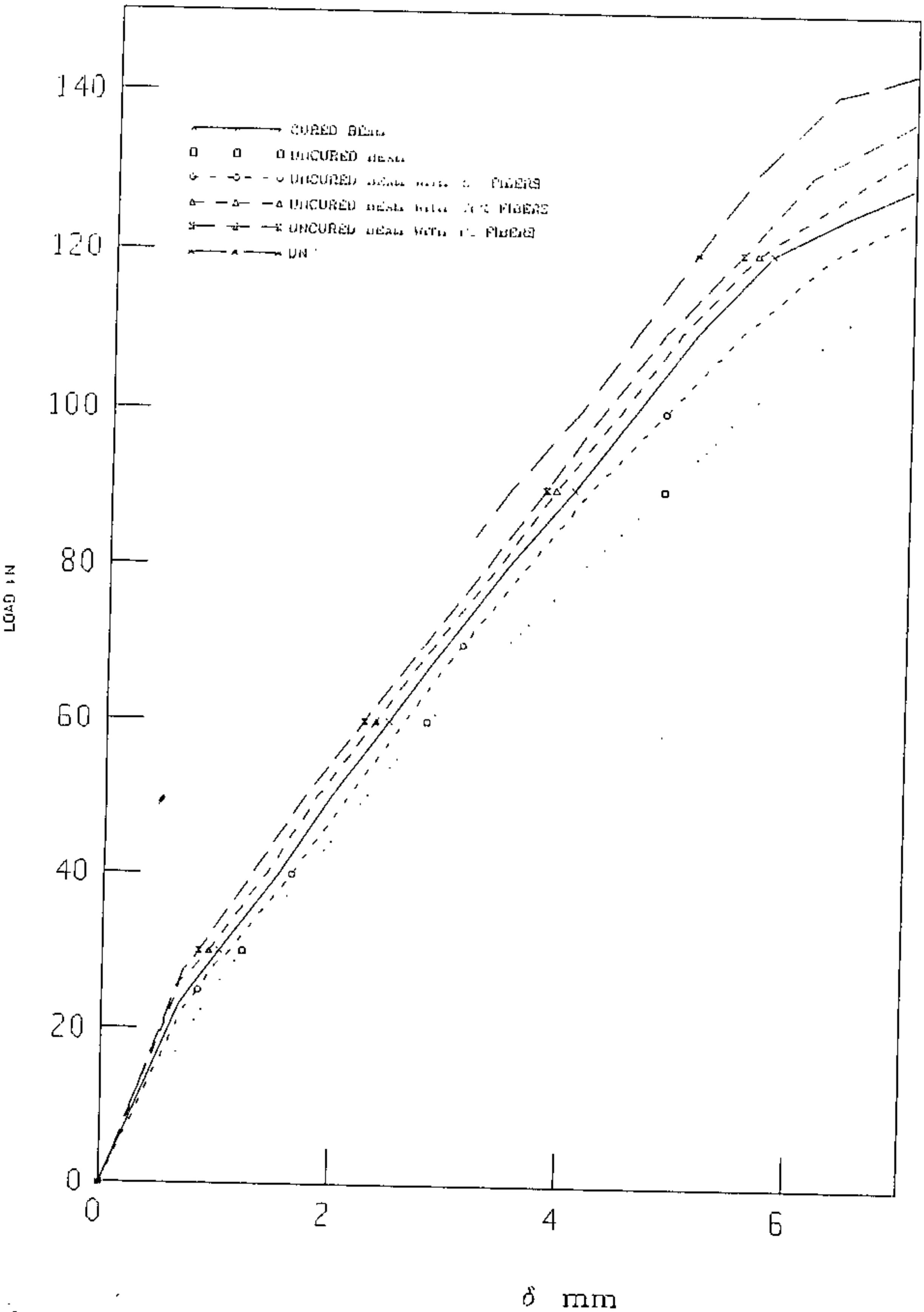


FIG.(4.7): COMPARSION LOAD DEFLECTION CURVES FOR ALL BEAMS IN FIRST SET

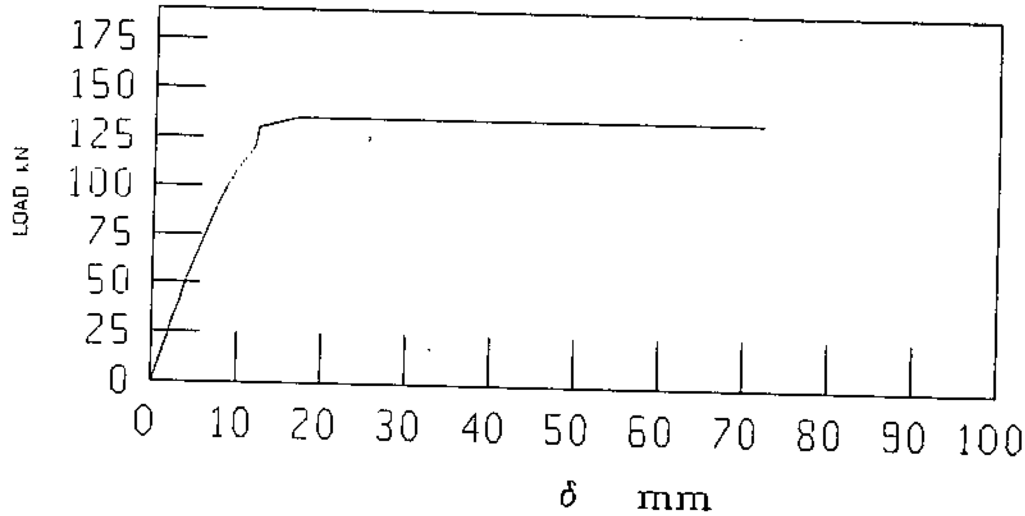


FIG.(4.8a): LOAD DEFLECTION CURVE FOR BEAM # 7

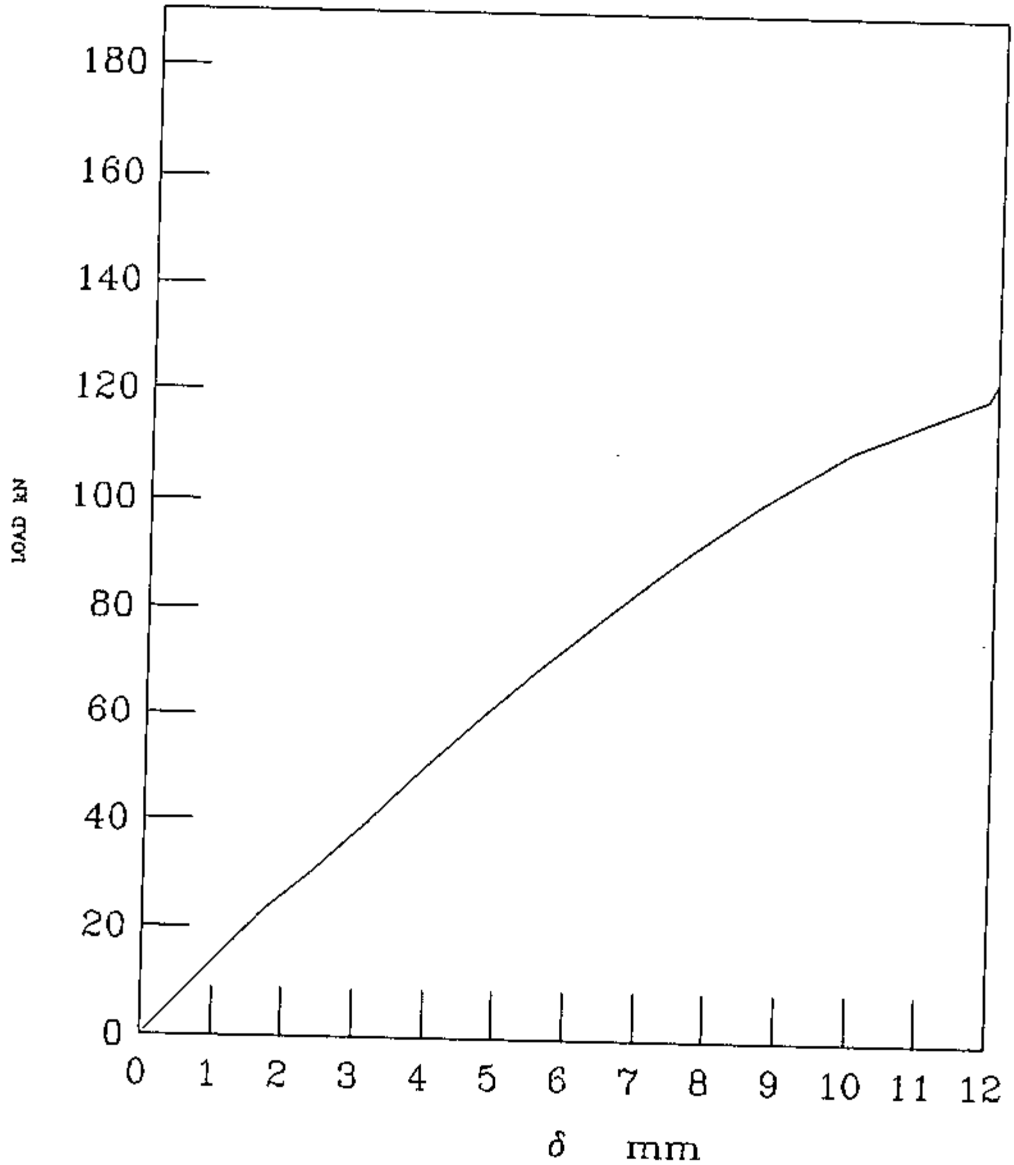


FIG.(4.8b): LOAD DEFLECTION CURVE FOR BEAM # 7

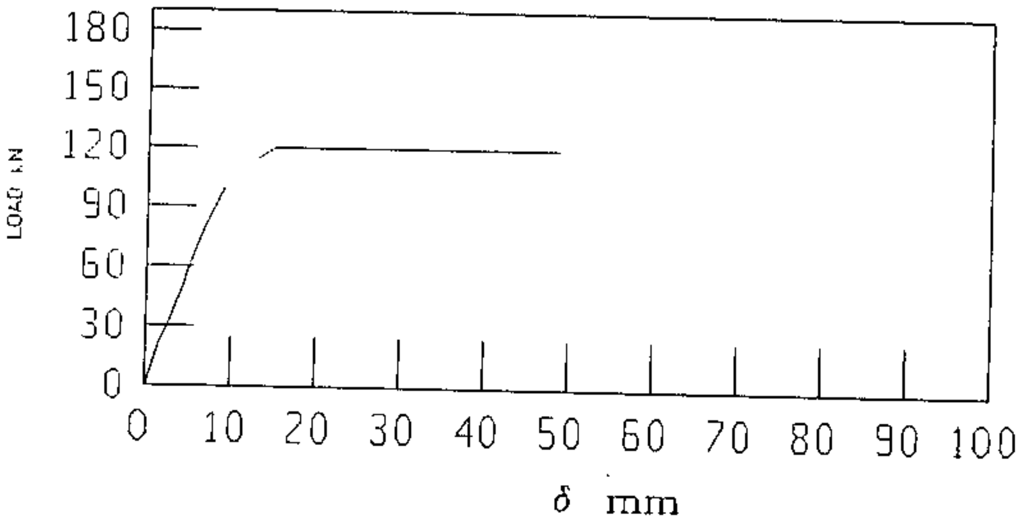


FIG.(4.9a): LOAD DEFLECTION CURVE FOR BEAM # 8

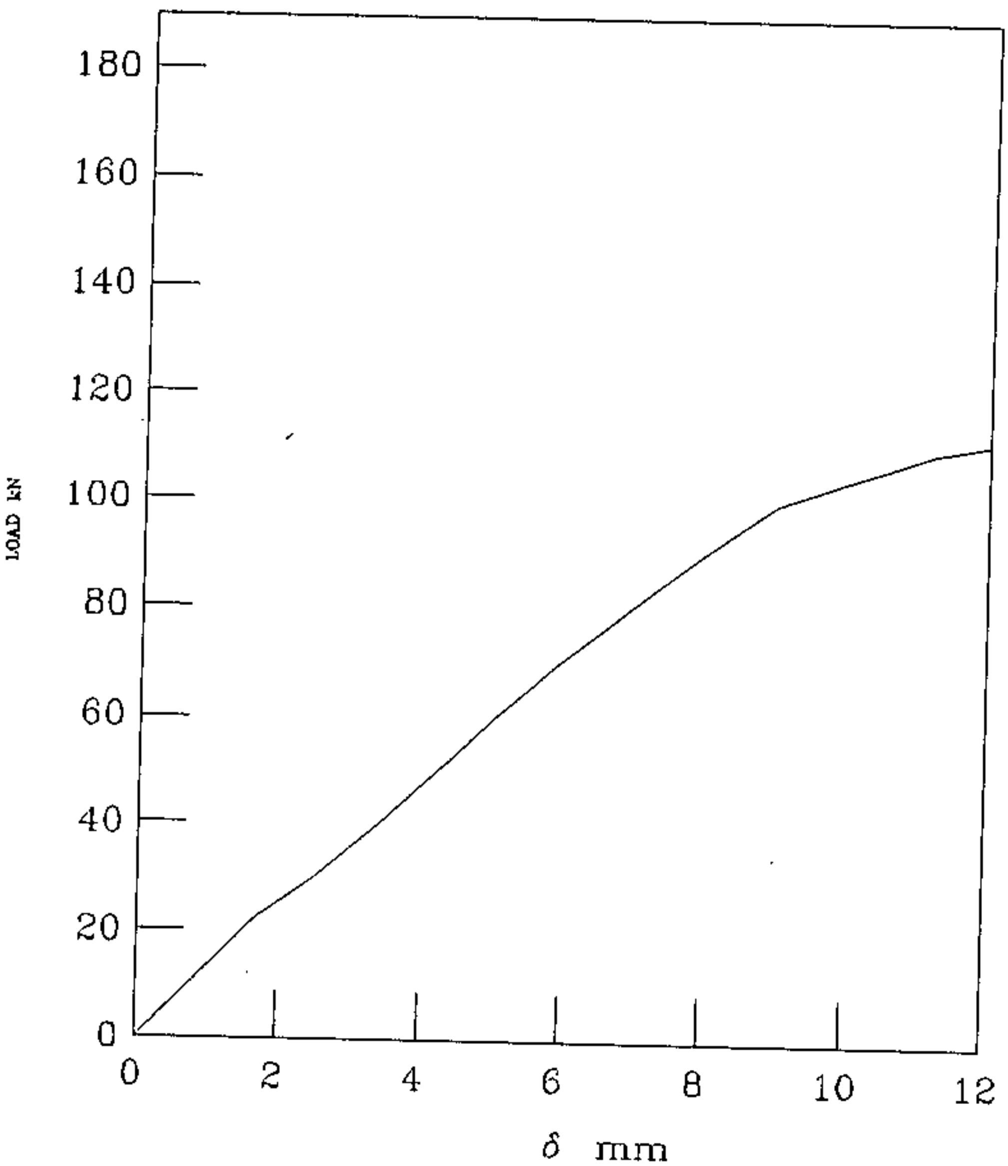


FIG.(4.9b): LOAD DEFLECTION CURVE FOR BEAM # 8

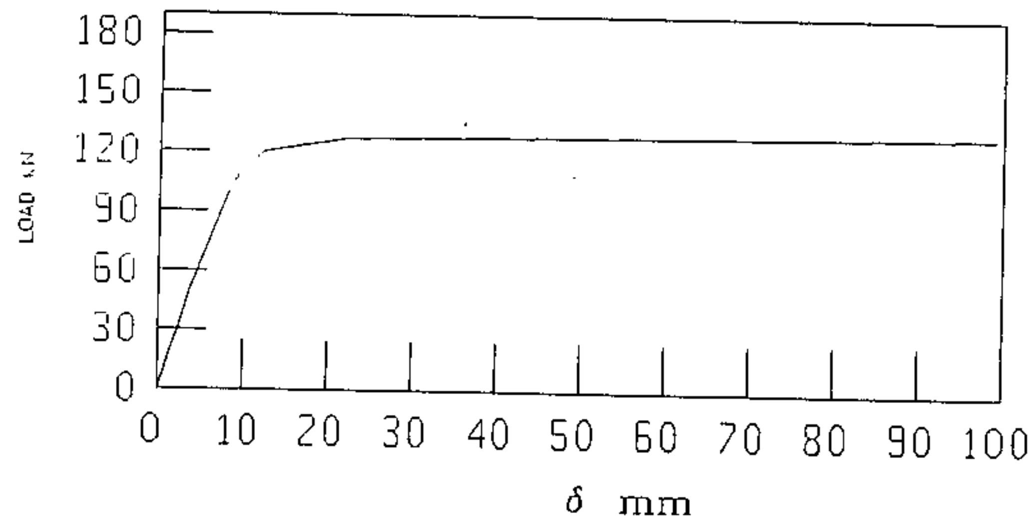


FIG.(4.10a): LOAD DEFLECTION CURVE FOR BEAM # 9

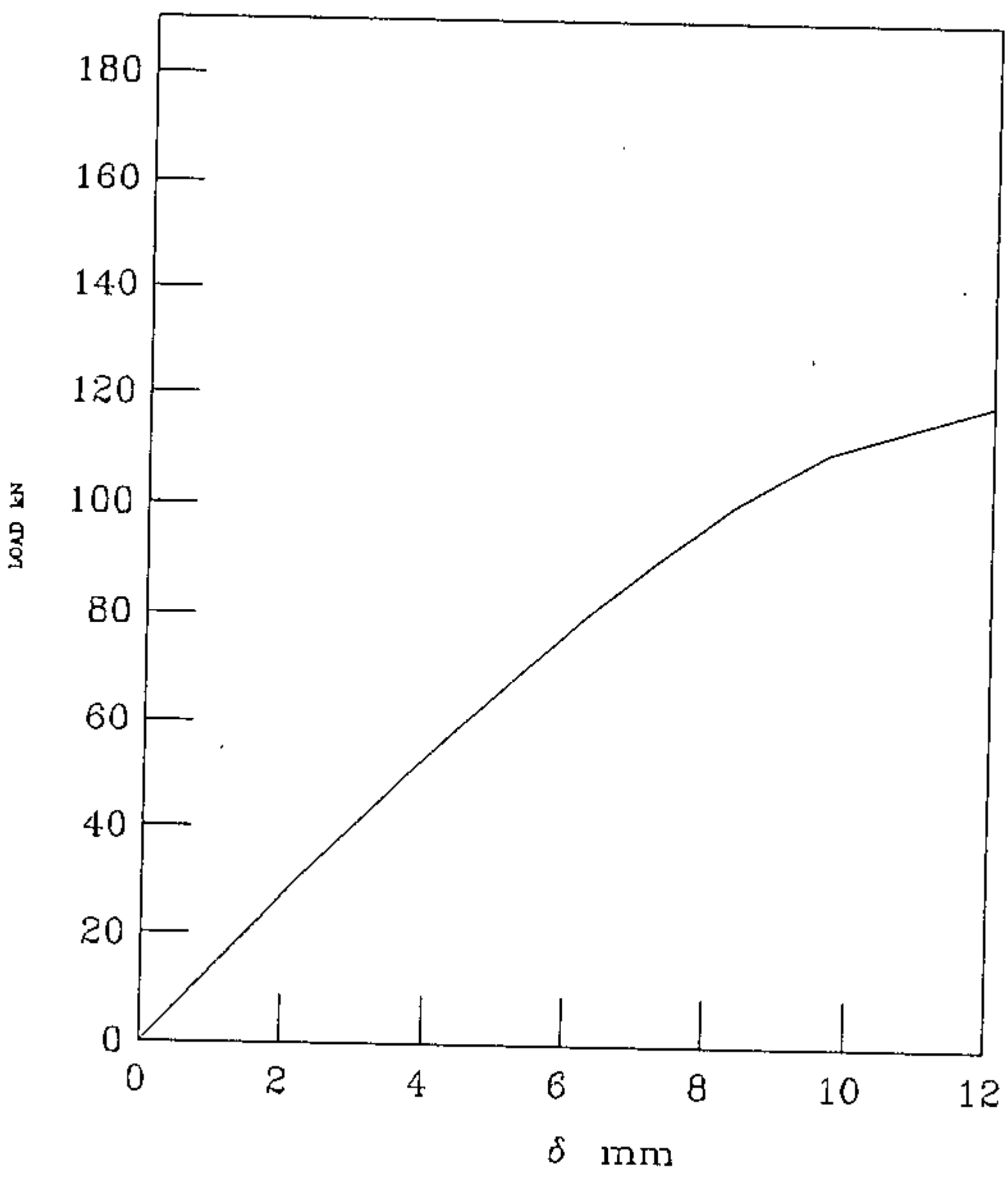


FIG.(4.10b): LOAD DEFLECTION CURVE FOR BEAM # 9

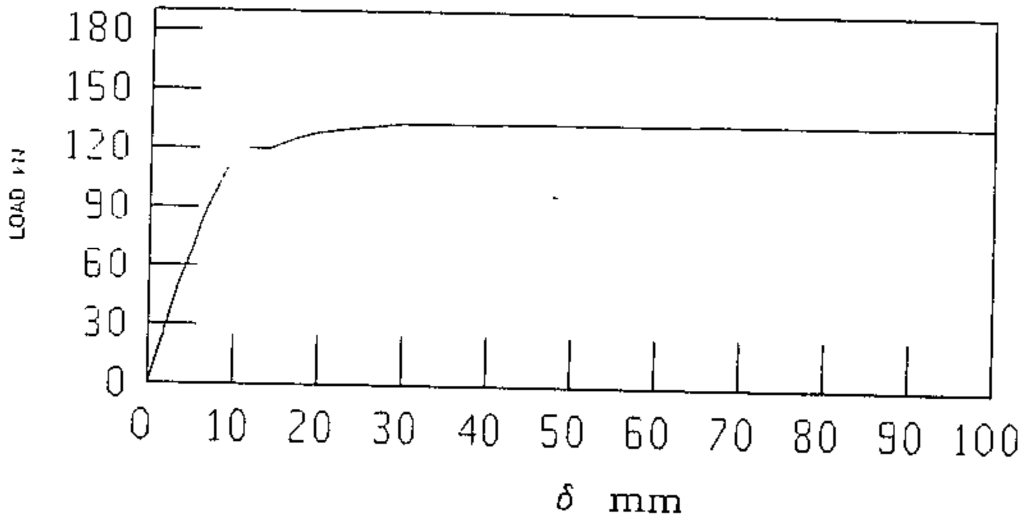


FIG.(4.11a): LOAD DEFLECTION CURVE FOR BEAM # 10

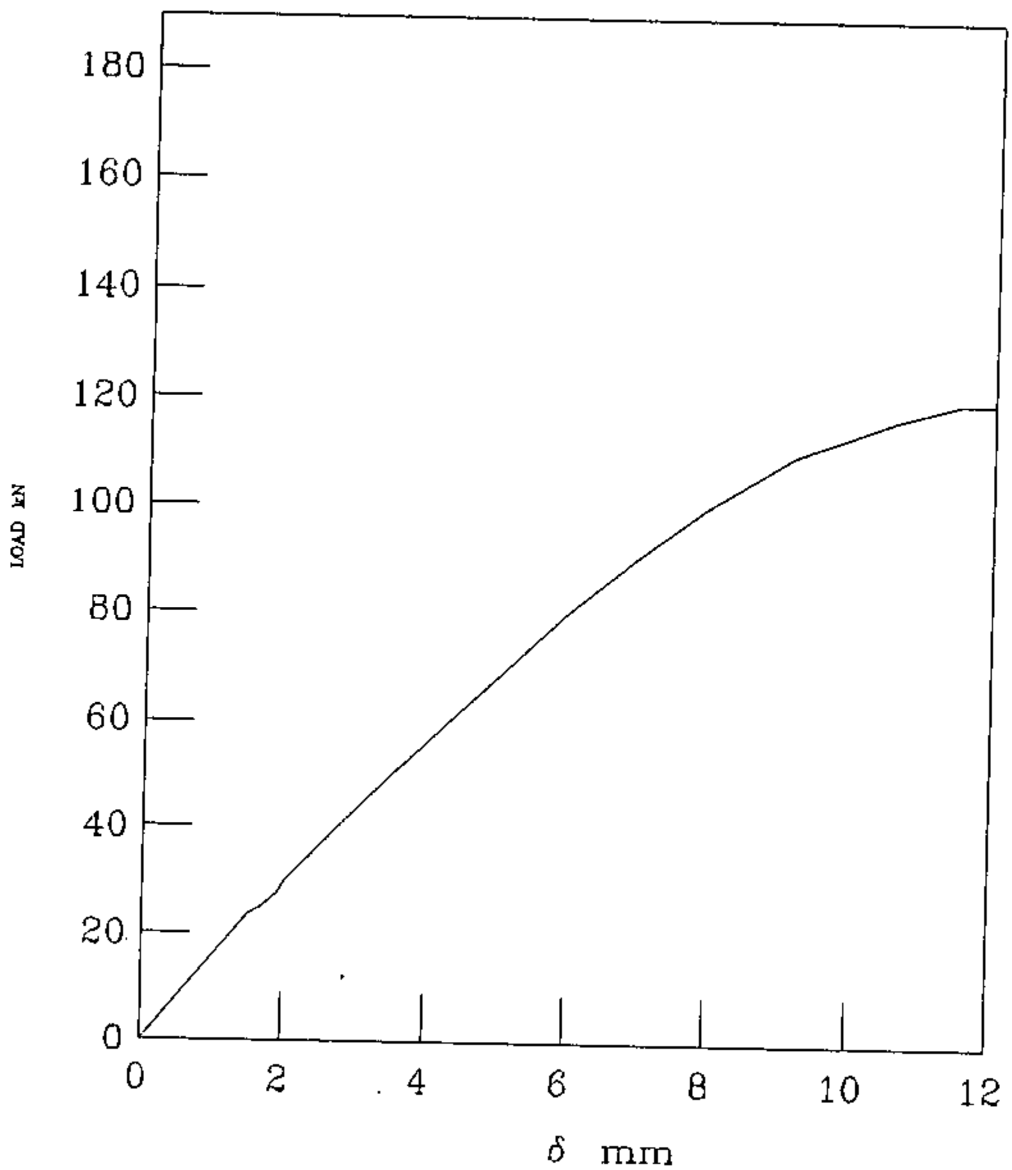


FIG.(4.11b): LOAD DEFLECTION CURVE FOR BEAM # 10

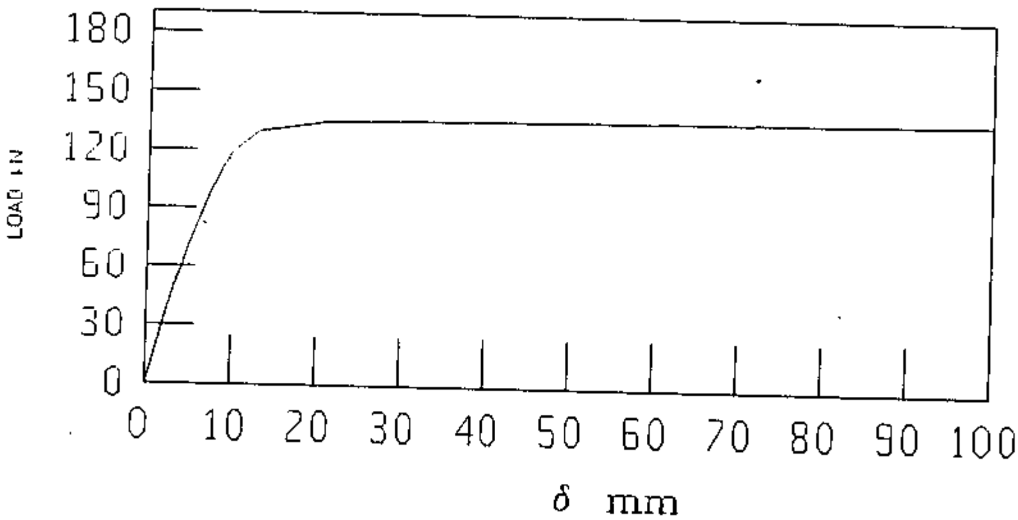


FIG.(4.12a): LOAD DEFLECTION CURVE FOR BEAM # 11

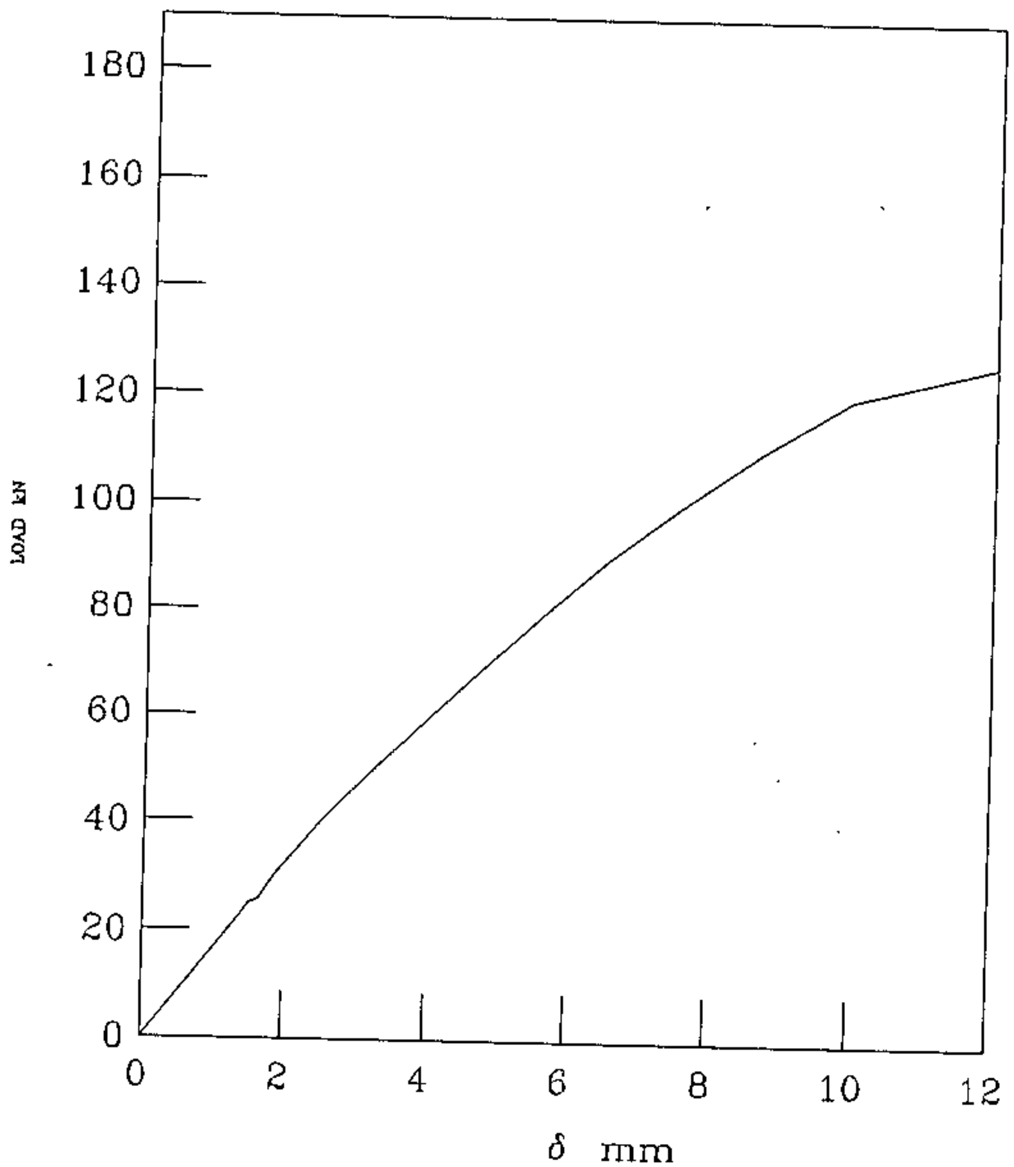


FIG.(4.12b): LOAD DEFLECTION CURVE FOR BEAM # 11

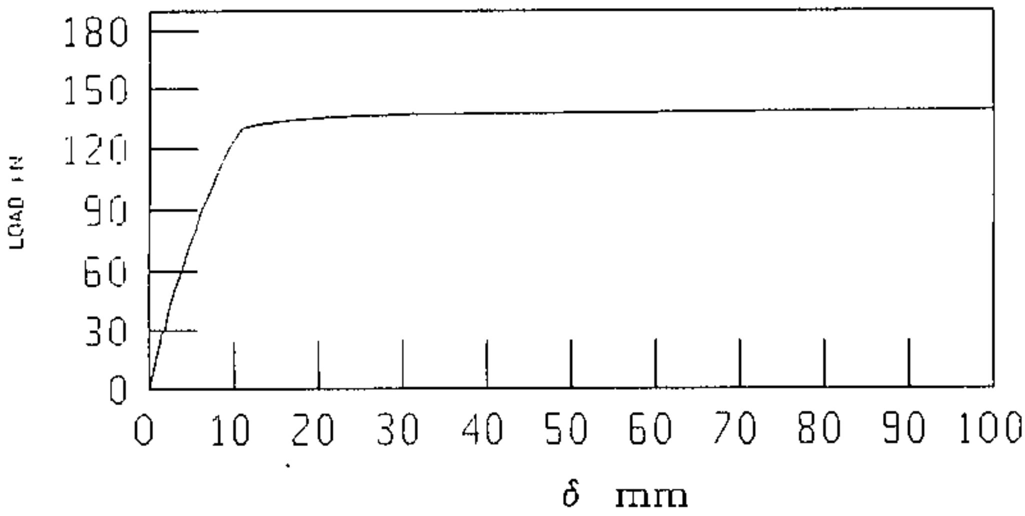


FIG.(4.13a): LOAD DEFLECTION CURVE FOR BEAM # 12

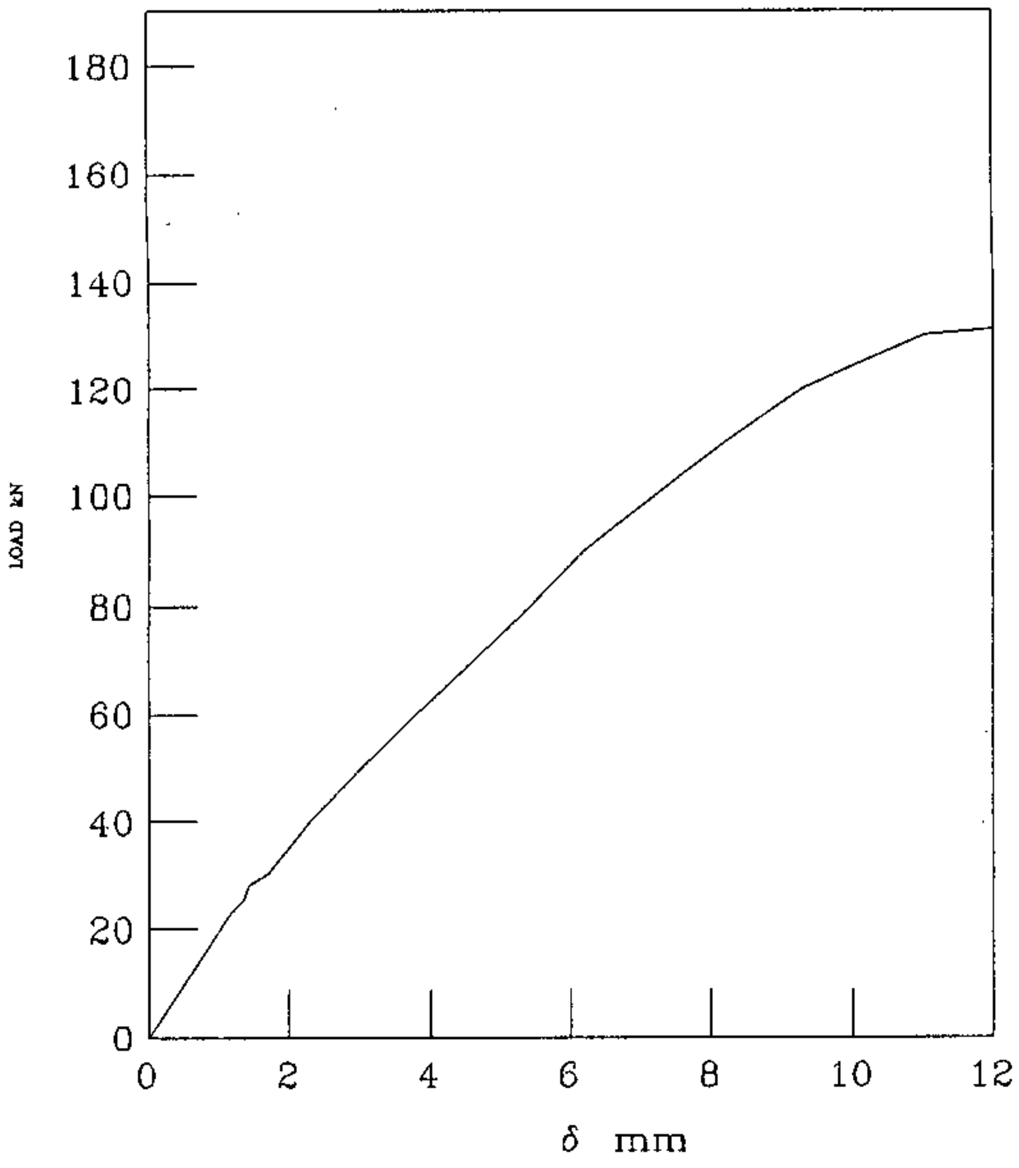


FIG.(4.13b): LOAD DEFLECTION CURVE FOR BEAM # 12

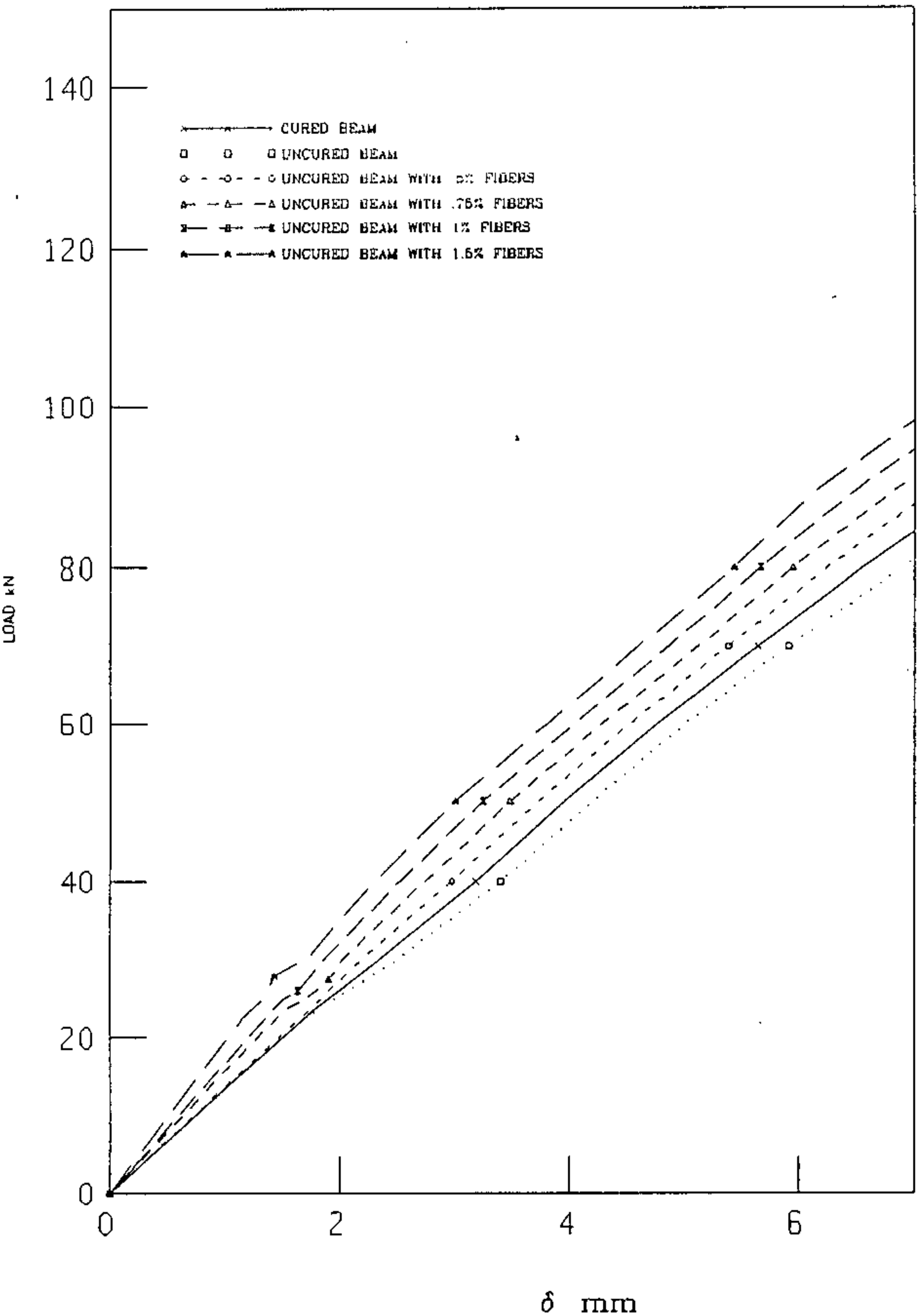


FIG.(4.14): COMPARSION LOAD DEFLECTION CURVES FOR ALL BEAMS IN SECOND SET

4.5 Deformation Characteristics

All beams were instrumented by three sets of demec gages, each set consisted of 12 points in six rows as shown in plate (4.6 & 4.12), strains were measured at increments of 15 kN on compression face by using mechanical strain gauges with a gauge length of 10. cm. For all beams, strains were measured at three sections at six different depths, then the results were recorded. For all the beams, strain distributions across the depth for different loads were plotted as shown in figures 4.15 through 4.26. Curvatures were calculated from the strain diagram by dividing the compression strain at top fiber on the depth of neutral axis for a specified load, then the M-ϕ curves were plotted and can be seen in figures 4.27 through 4.38, for comparison purpose the M-ϕ curves for all beams in first set and second set were shown in figures (4.39 & 4.40).

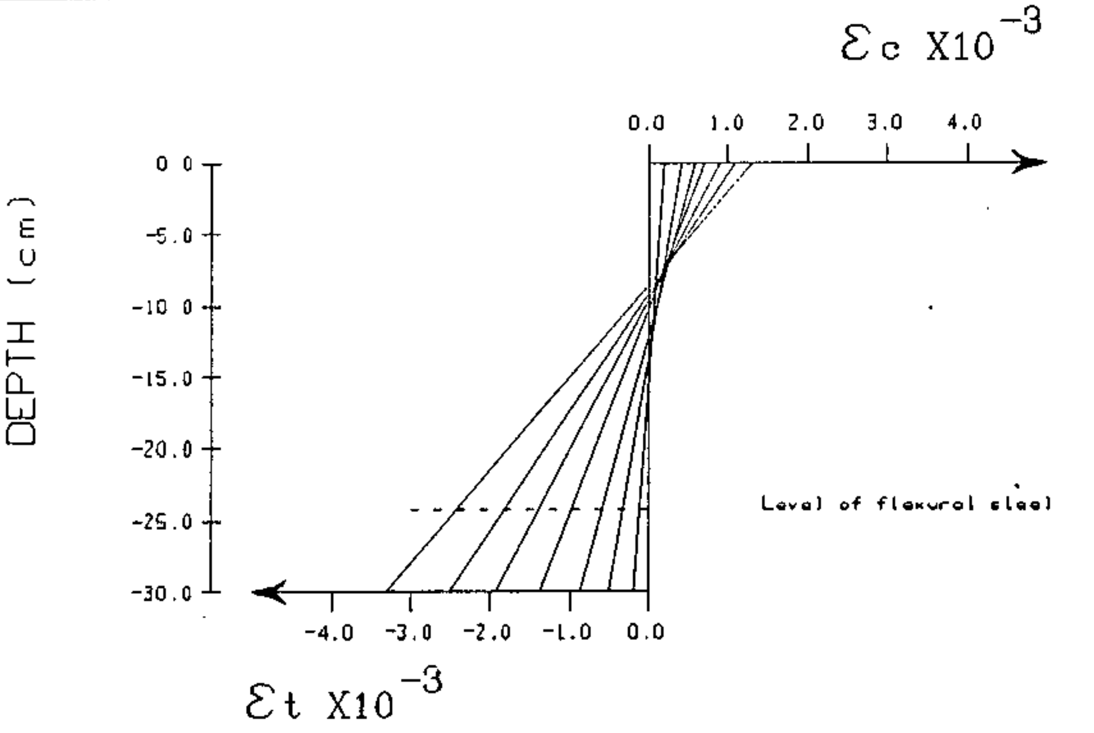


FIG.(4.15): STRAIN DISTRIBUTION FOR BEAM # 1

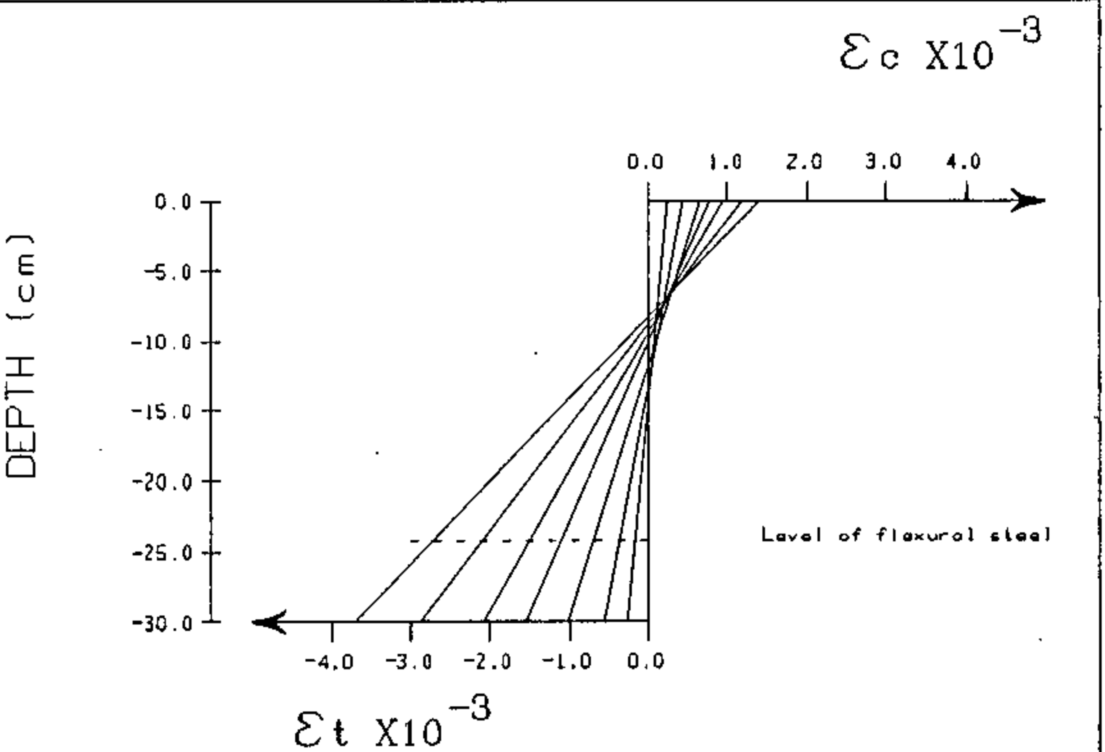


FIG.(4.16): STRAIN DISTRIBUTION FOR BEAM # 2

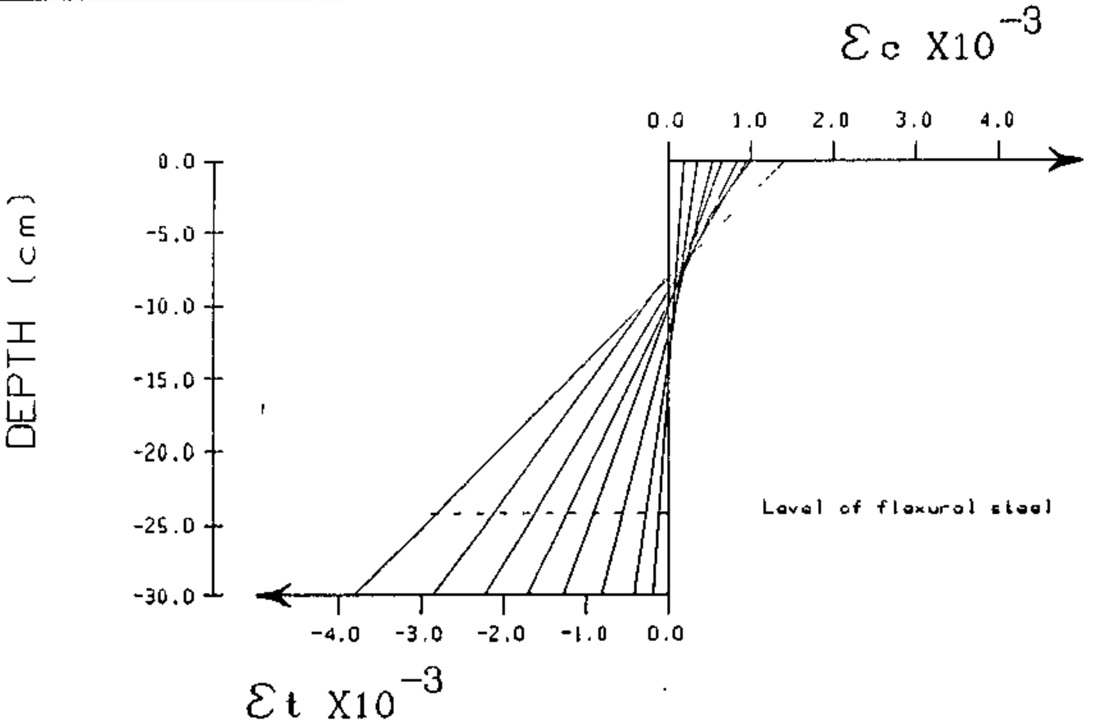


FIG.(4.17): STRAIN DISTRIBUTION FOR BEAM # 3

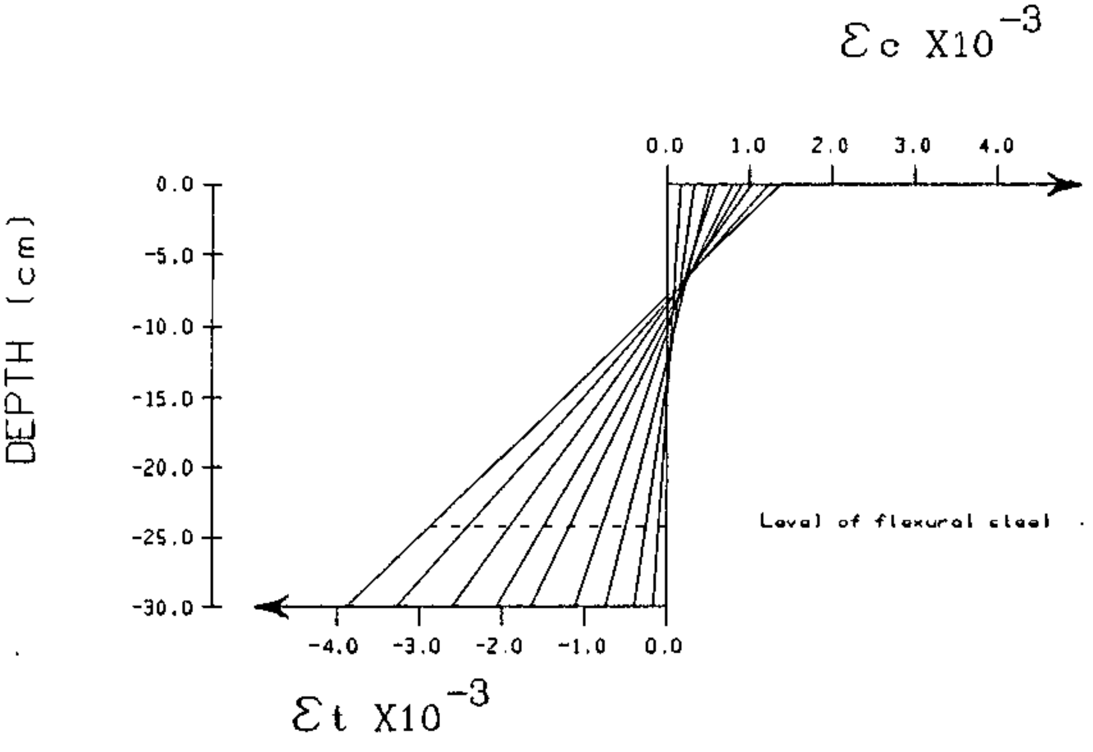


FIG.(4.18): STRAIN DISTRIBUTION FOR BEAM # 4

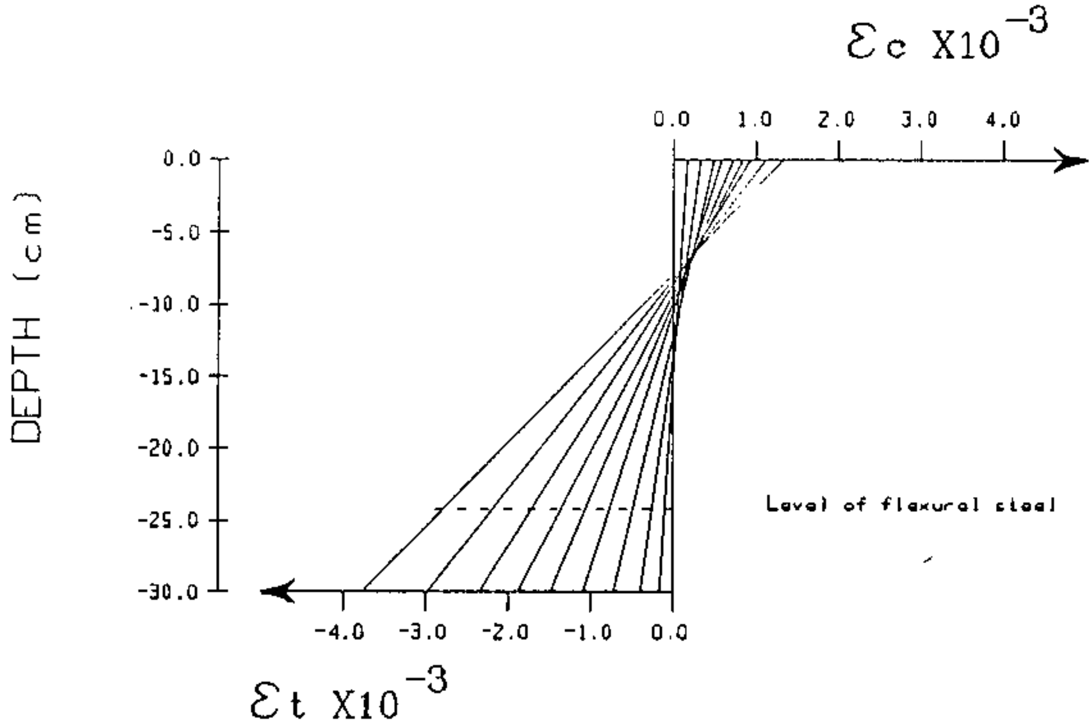


FIG.(4.19): STRAIN DISTRIBUTION FOR BEAM # 5

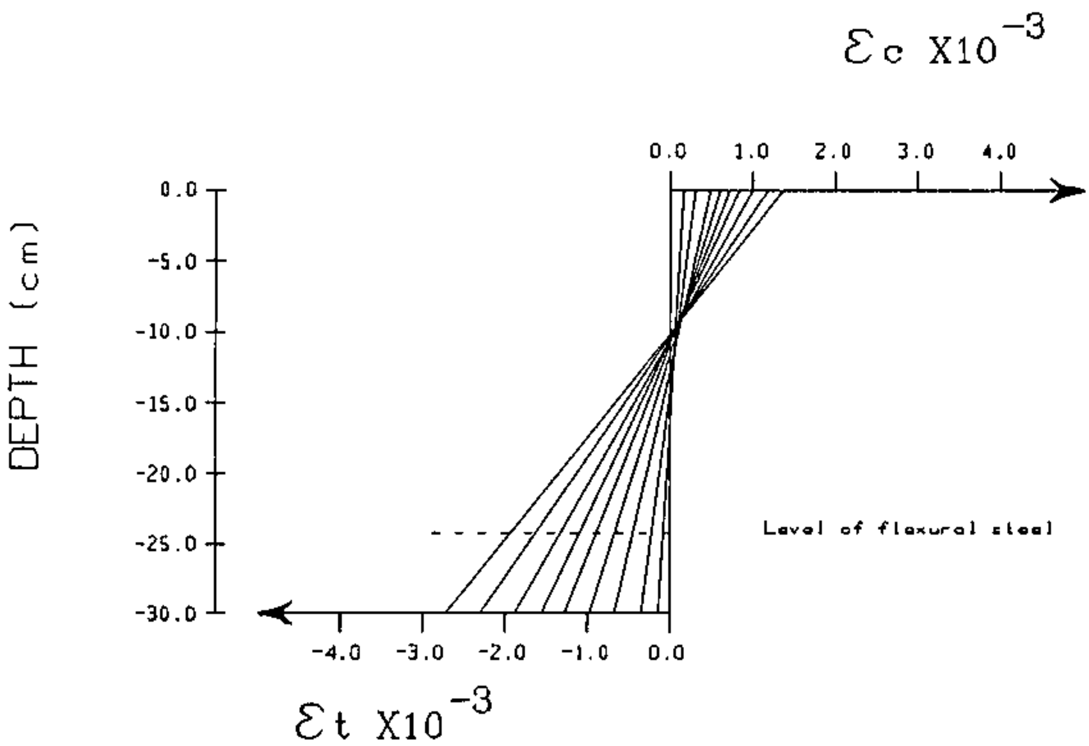


FIG.(4.20): STRAIN DISTRIBUTION FOR BEAM # 6

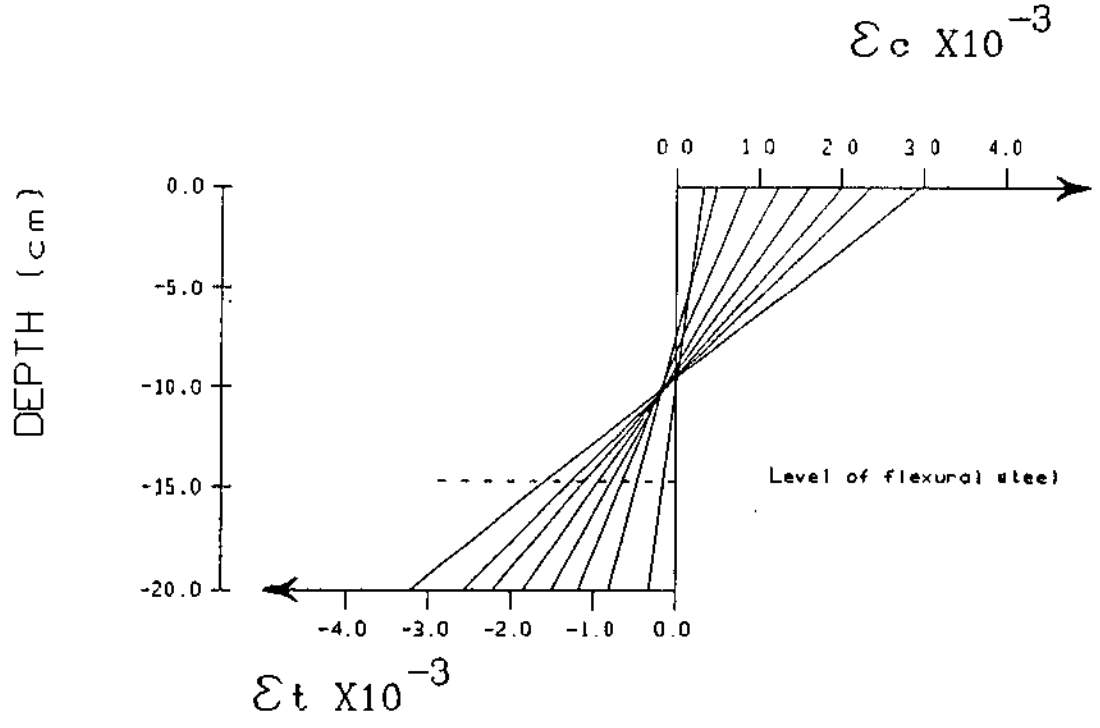


FIG.(4.21): STRAIN DISTRIBUTION FOR BEAM # 7

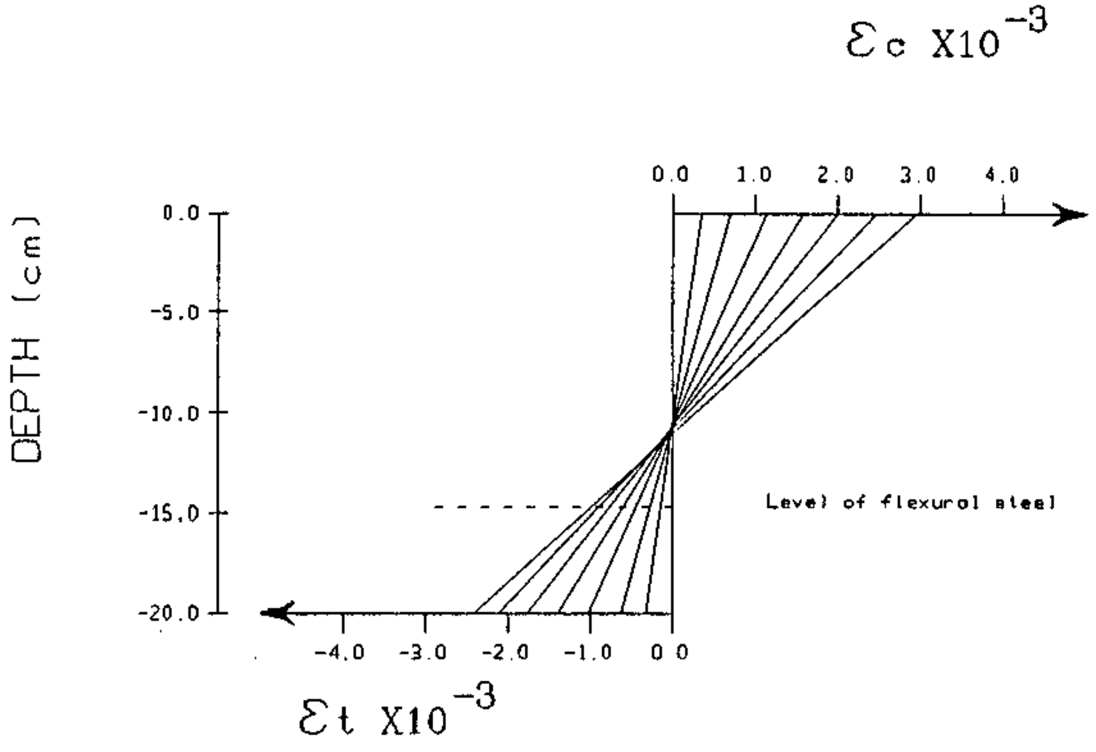


FIG.(4.22): STRAIN DISTRIBUTION FOR BEAM # 8

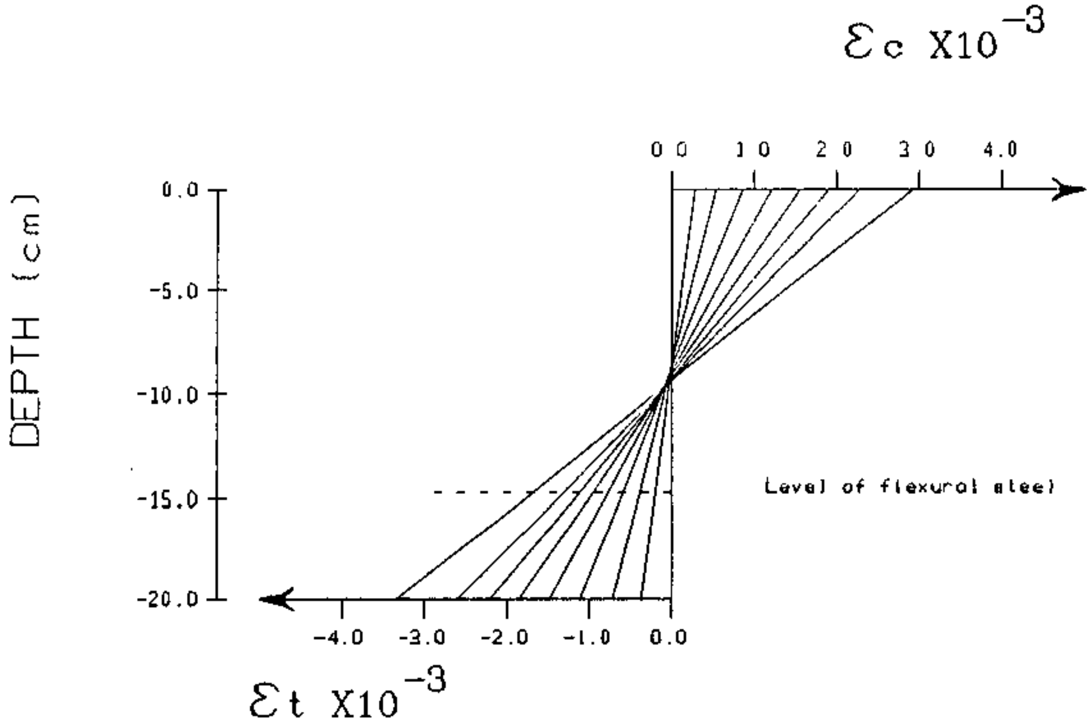


FIG.(4.23):STRAIN DISTRIBUTION FOR BEAM # 9

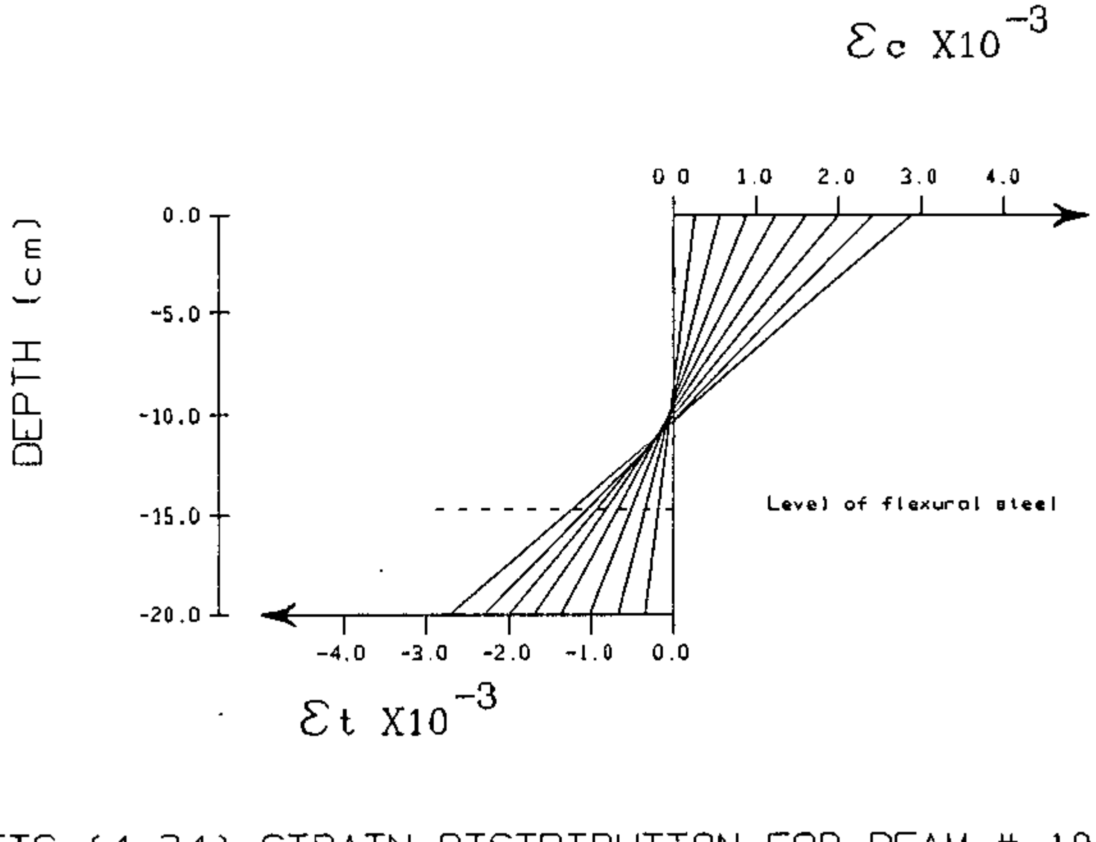


FIG.(4.24):STRAIN DISTRIBUTION FOR BEAM # 10

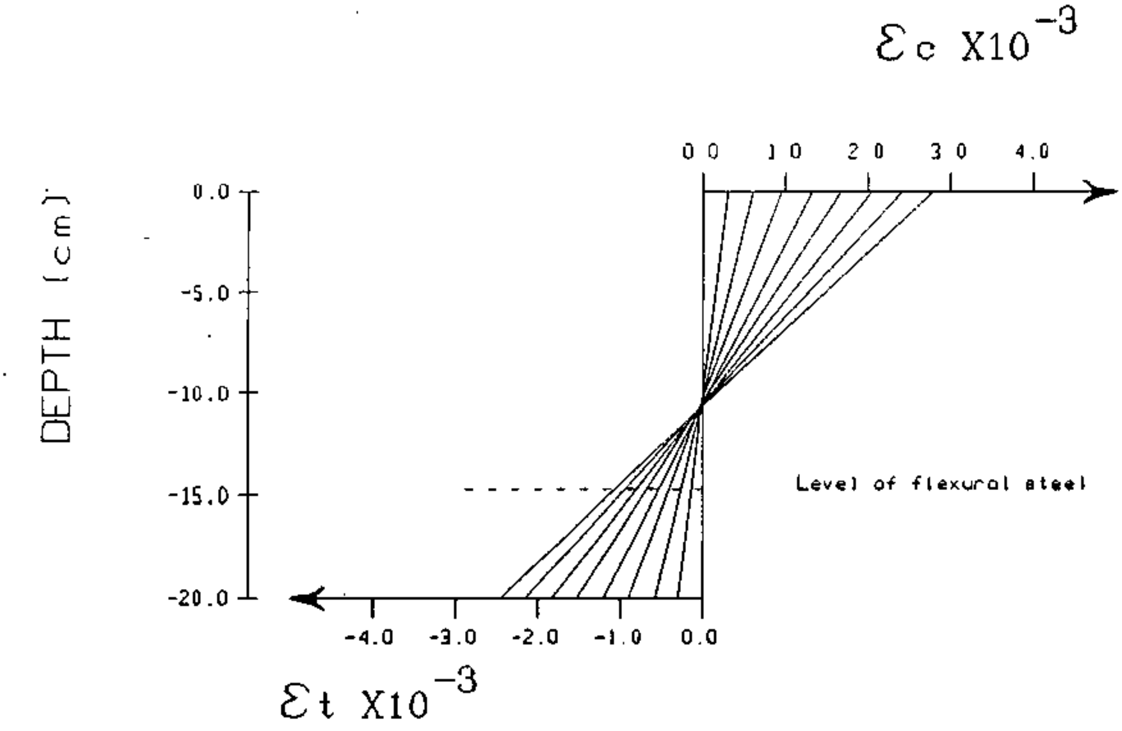


FIG. (4.25): STRAIN DISTRIBUTION FOR BEAM # 11

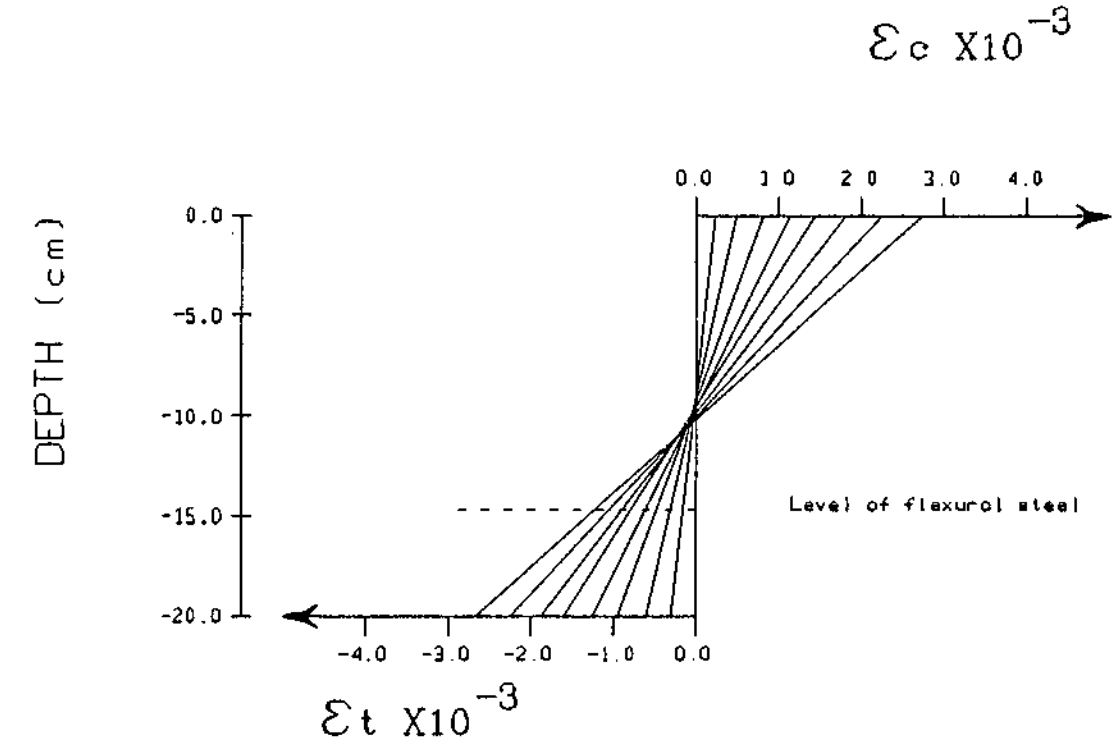


FIG. (4.26): STRAIN DISTRIBUTION FOR BEAM # 12

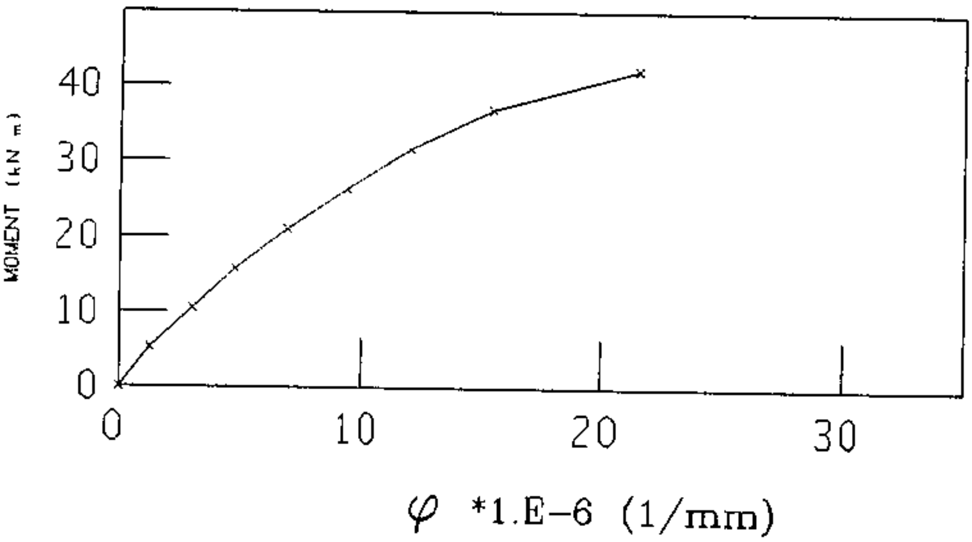


FIG.(4.27) : MOMENT-CURVATURE CURVE FOR BEAM # 1

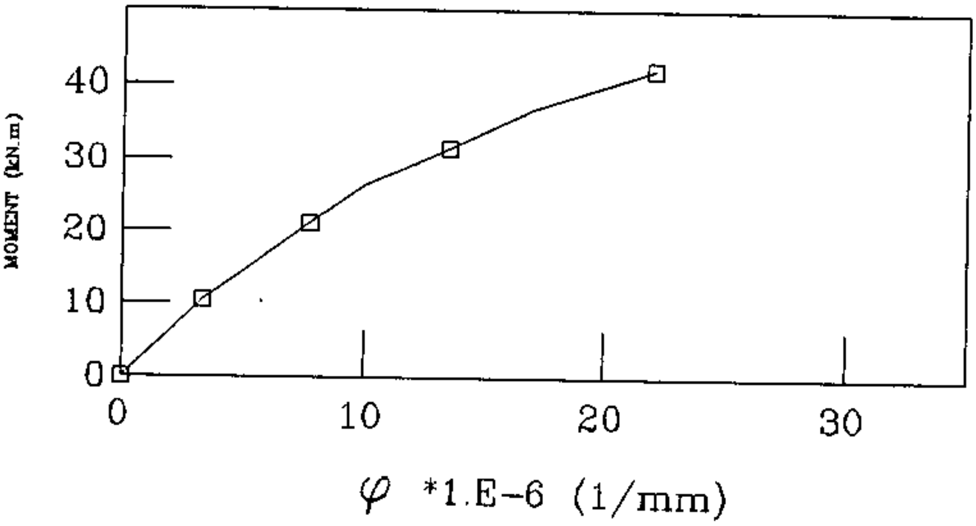


FIG.(4.28): MOMENT-CURVATURE CURVE FOR BEAM # 2

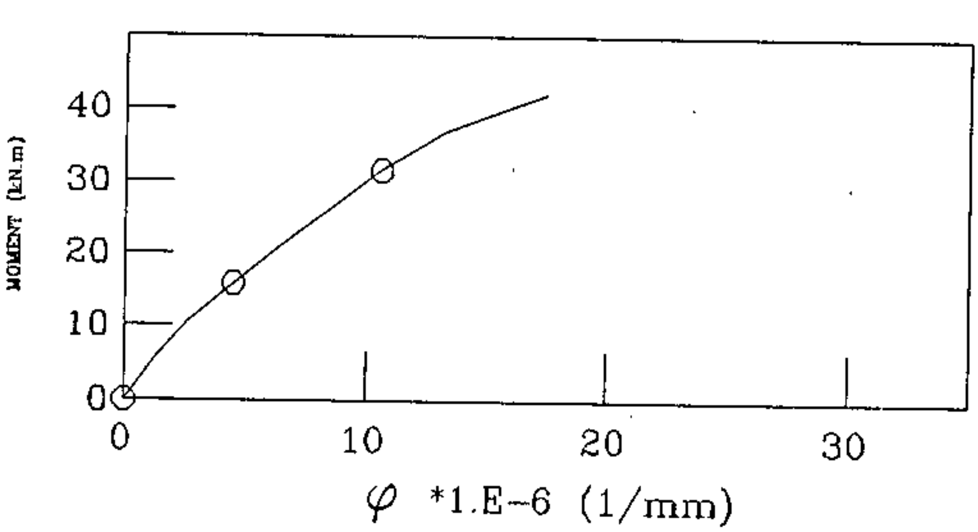


FIG.(4.29): MOMENT-CURVATURE CURVE FOR BEAM # 3

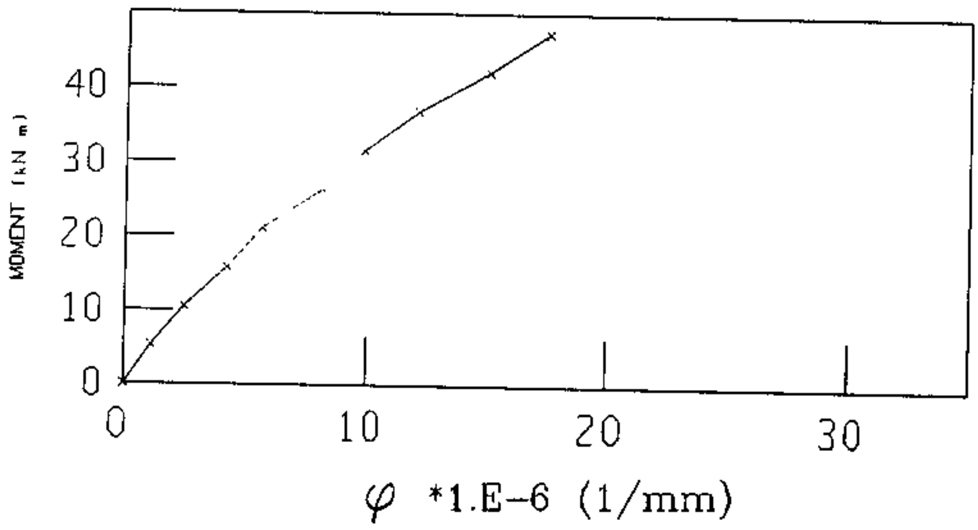


FIG.(4.30): MOMENT-CURVATURE CURVE FOR BEAM # 4

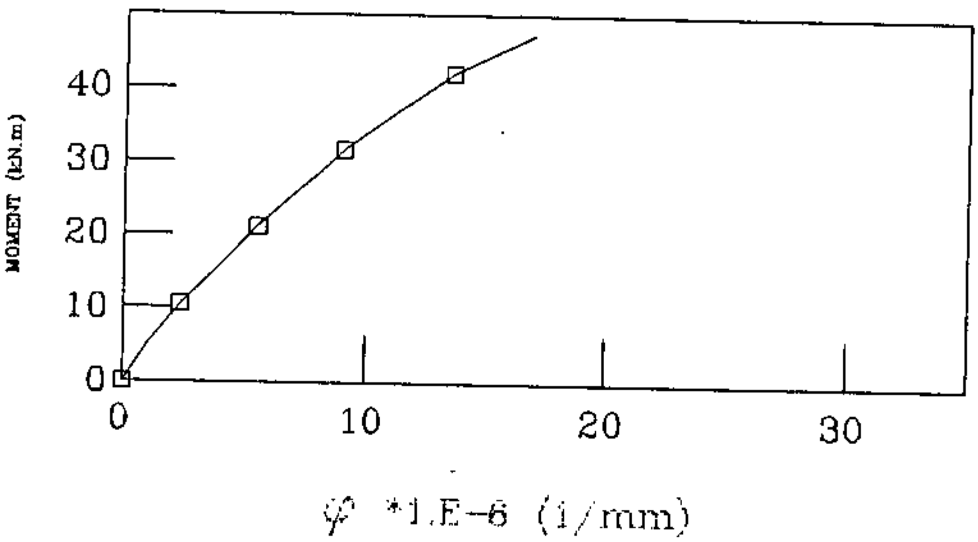


FIG.(4.31): MOMENT-CURVATURE CURVE FOR BEAM # 5

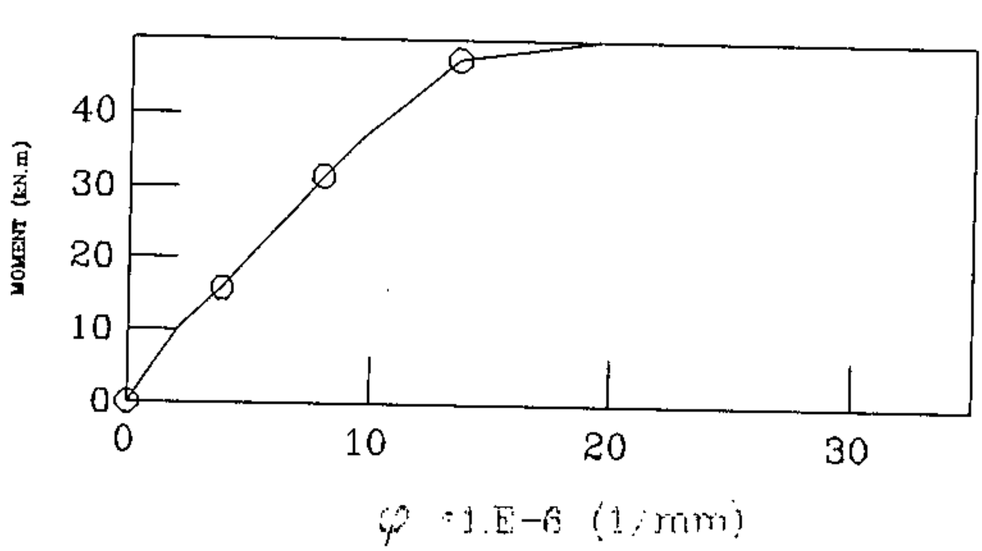


FIG.(4.32): MOMENT-CURVATURE CURVE FOR BEAM # 6

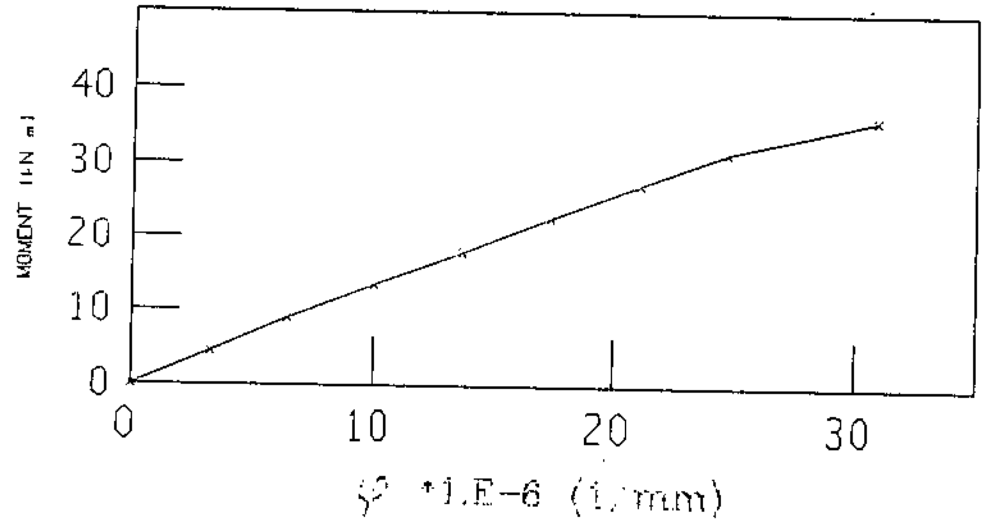


FIG.(4.33): MOMENT-CURVATURE CURVE FOR BEAM # 7

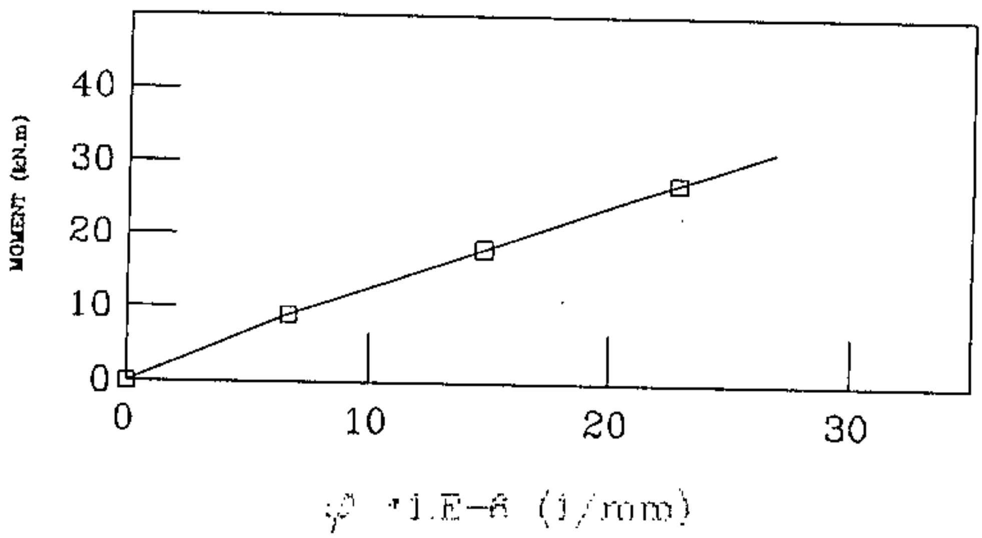


FIG.(4.34): MOMENT-CURVATURE CURVE FOR BEAM # 8

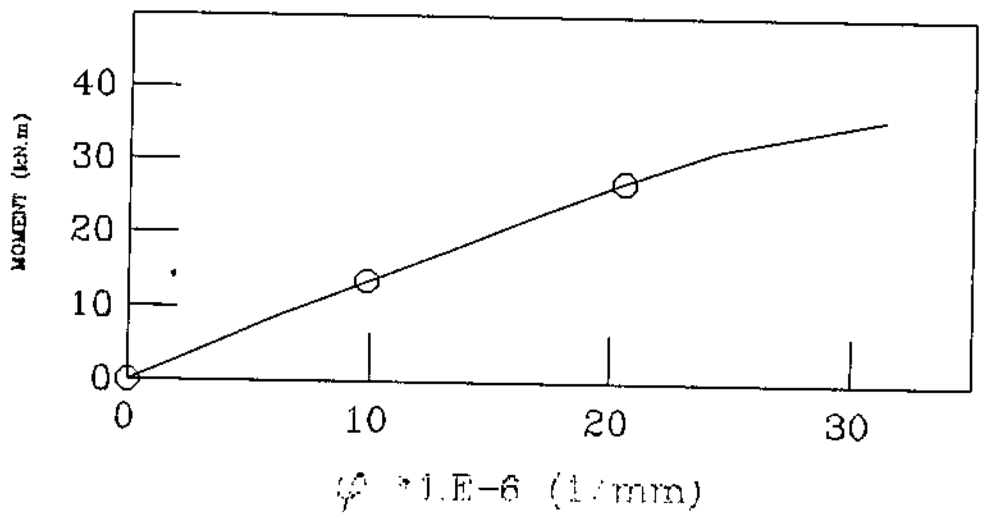


FIG.(4.35): MOMENT-CURVATURE CURVE FOR BEAM # 9

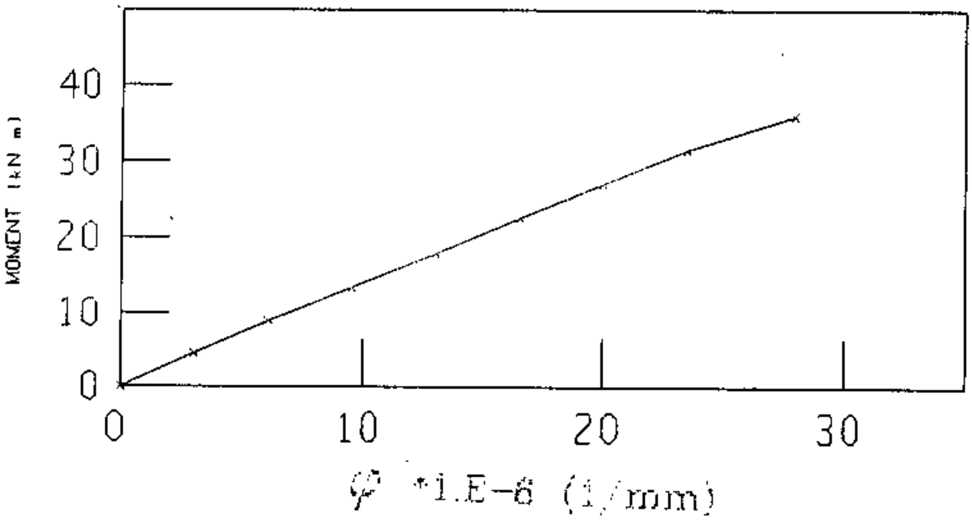


FIG.(4.36): MOMENT-CURVATURE CURVE FOR BEAM # 10

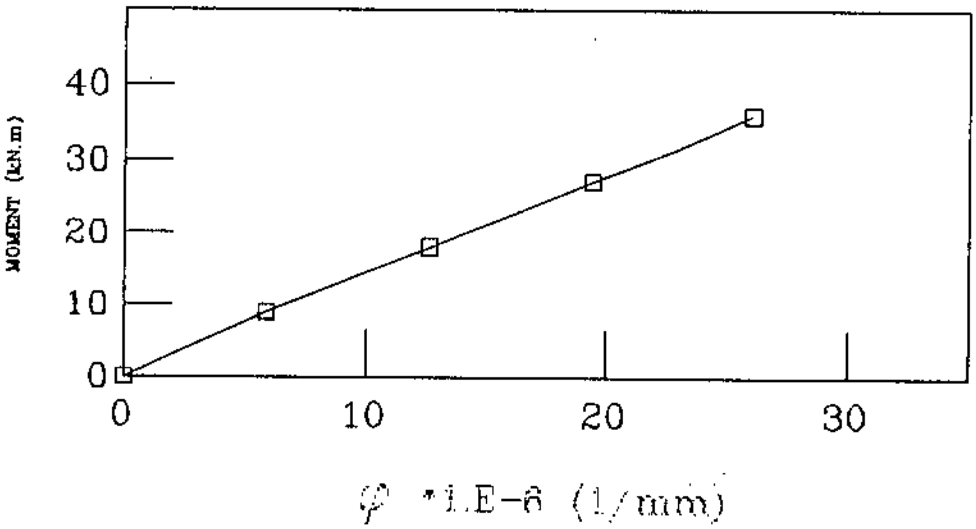


FIG.(4.37). MOMENT-CURVATURE CURVE FOR BEAM # 11

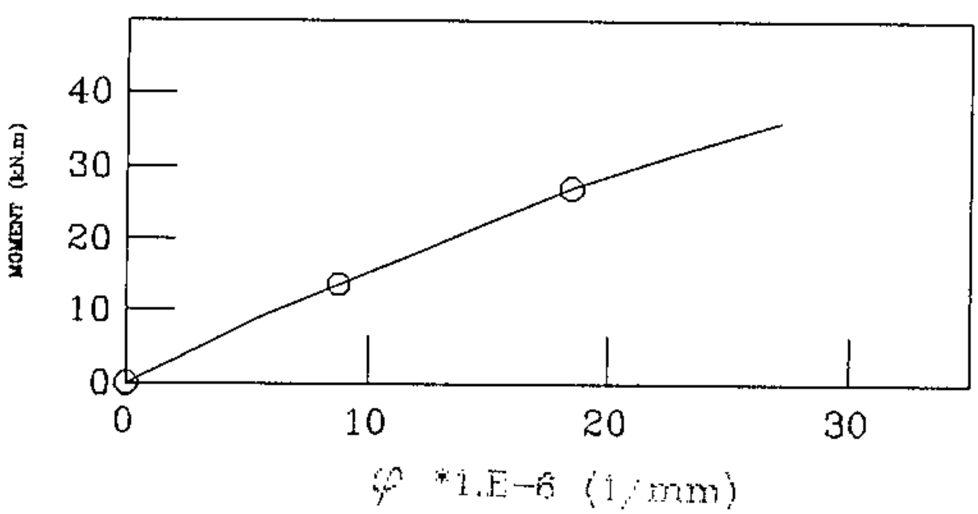


FIG.(4.38): MOMENT-CURVATURE CURVE FOR BEAM # 12

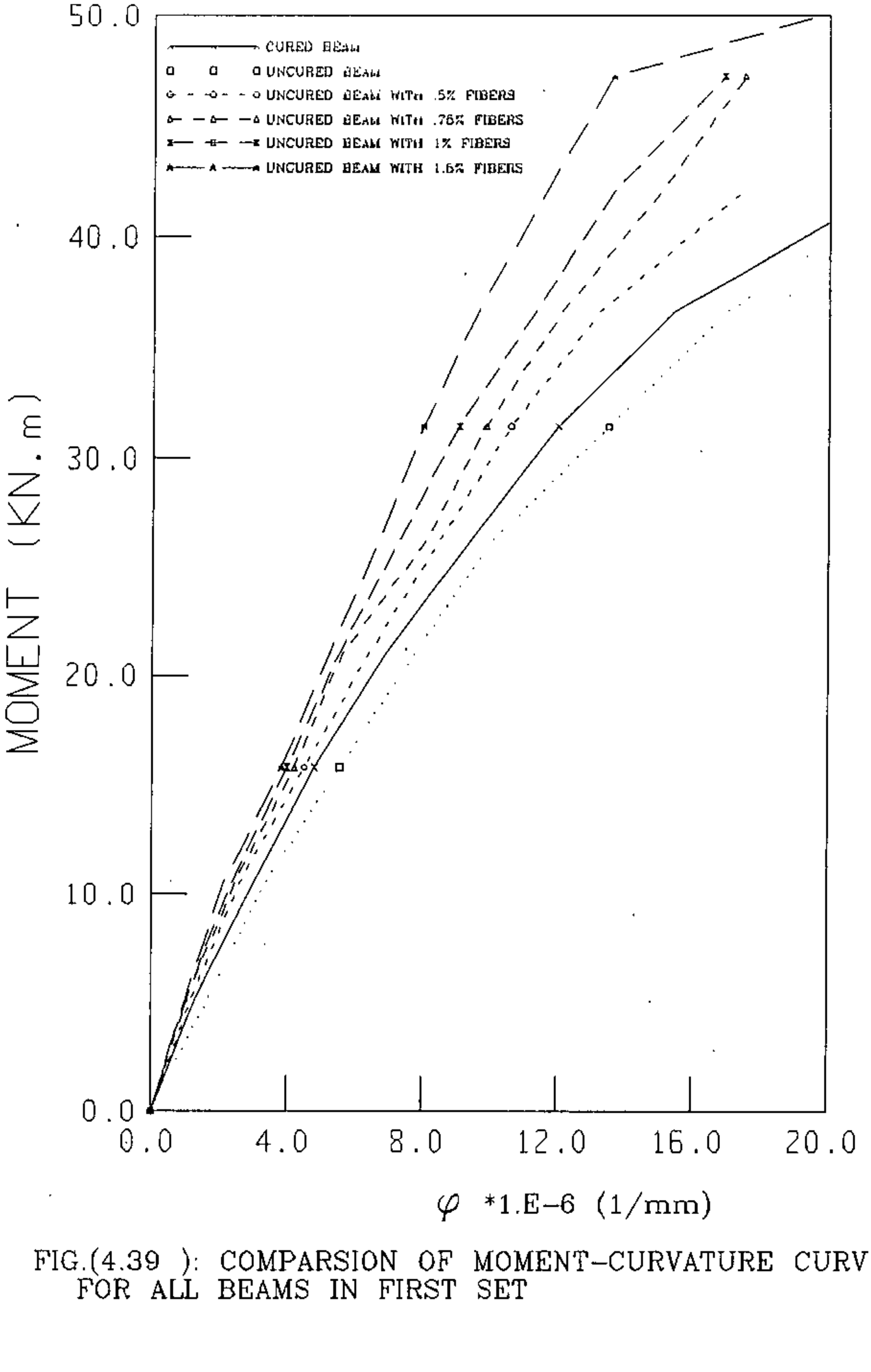


FIG.(4.39): COMPARSION OF MOMENT-CURVATURE CURV FOR ALL BEAMS IN FIRST SET

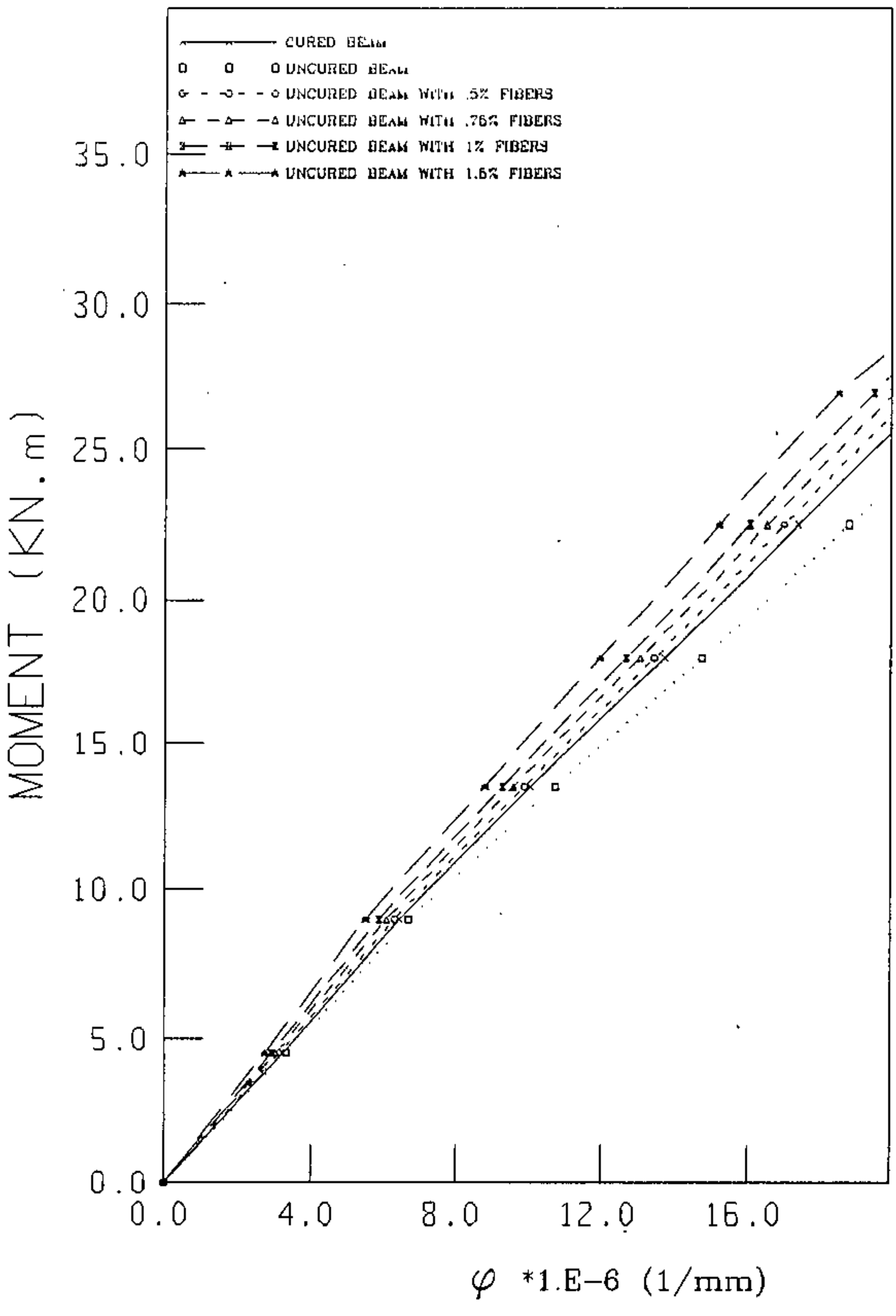


FIG.(4.40): COMPARISON OF MOMENT-CURVATURE CURVE FOR ALL BEAMS IN SECOND SET

4.6 Cracking characteristics

All the crack characteristics such as crack location, crack width, crack shape s, crack spacing and crack hight not measured but the photographs were taken for all stages of loading for all beams. Typical photographs were taken at approximately 120.kN and at ultimate stage shown in plates(4.1 through 4.12), these plates illustrate the crack characteristics, the behaviour of concrete in compression zone and the mode of failure.

In the second group for example the number of crack at each load were counted, the length of crack, and spacing also recorded, for example see table (4.5) & table (4.6).

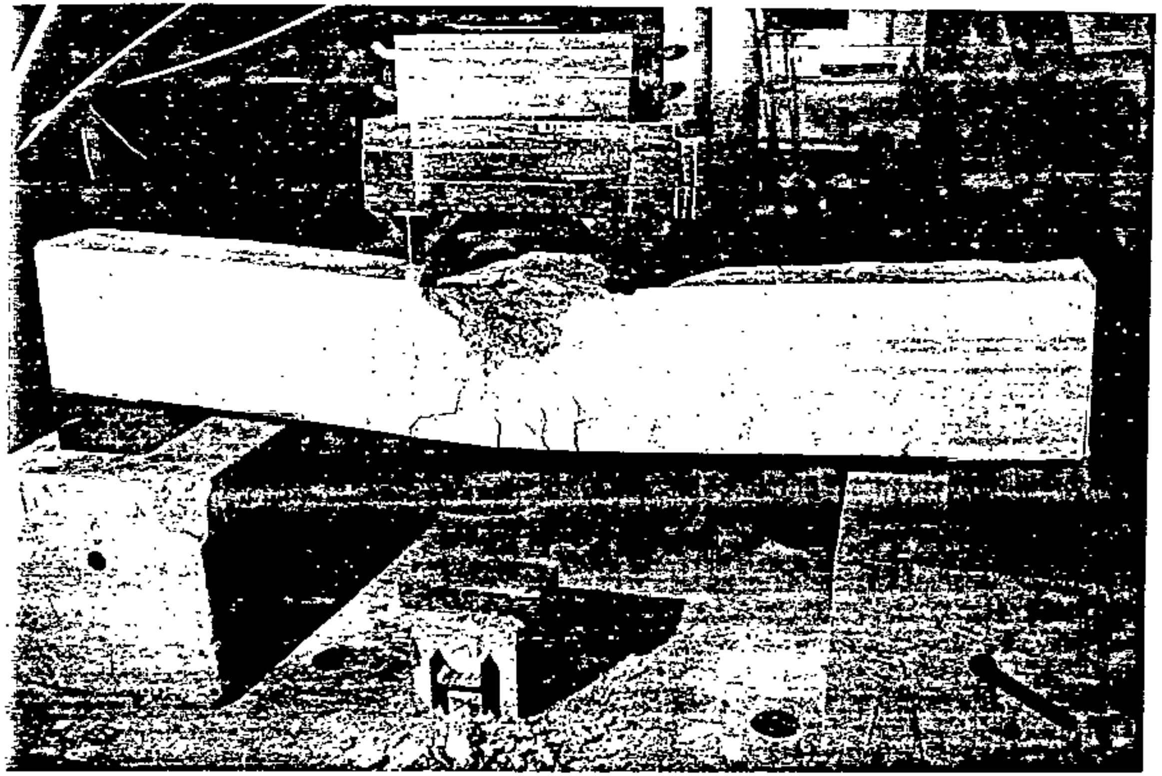
load	number of cracks	averge spacing	averge length	max. length
25.	11			
45.	14			
60.	15	8.77	6.37	11.5
75.	17	8.70	8.22	12.0
90.	19	8.14	8.75	12.1
105.	20	8.40	9.06	12.5
120.	22	8.00	9.49	12.5
135.	24	7.17	9.50	15.0

Table 4.5 Cracking characteristics for $UCB_2F.750\%$.

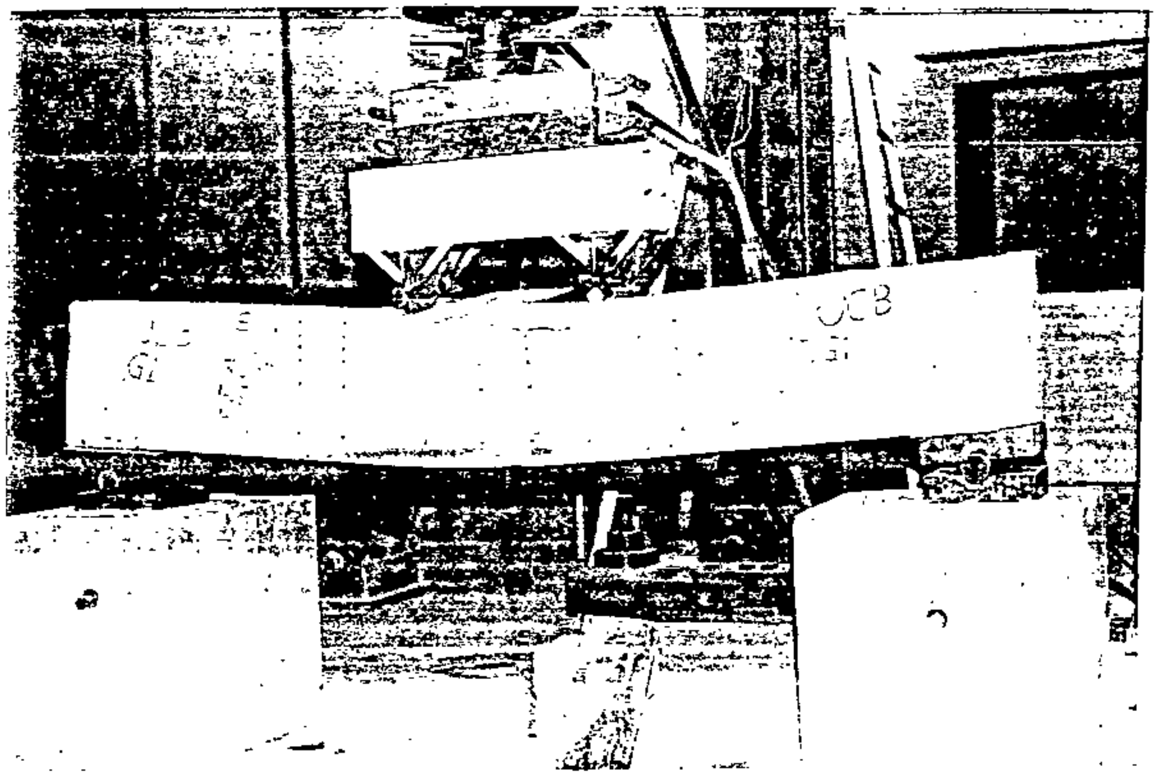
load	number of cracks	average length	max. length
45.	12	6.5	8.
60.	17	9.2	13.
75.	22	9.75	15.
90.	24	9.87	17.
105.	27	9.9	18.5
120.	27	10.8	19.

Table 4.6 Cracking characteristics for $UCB_2F1.\%$.

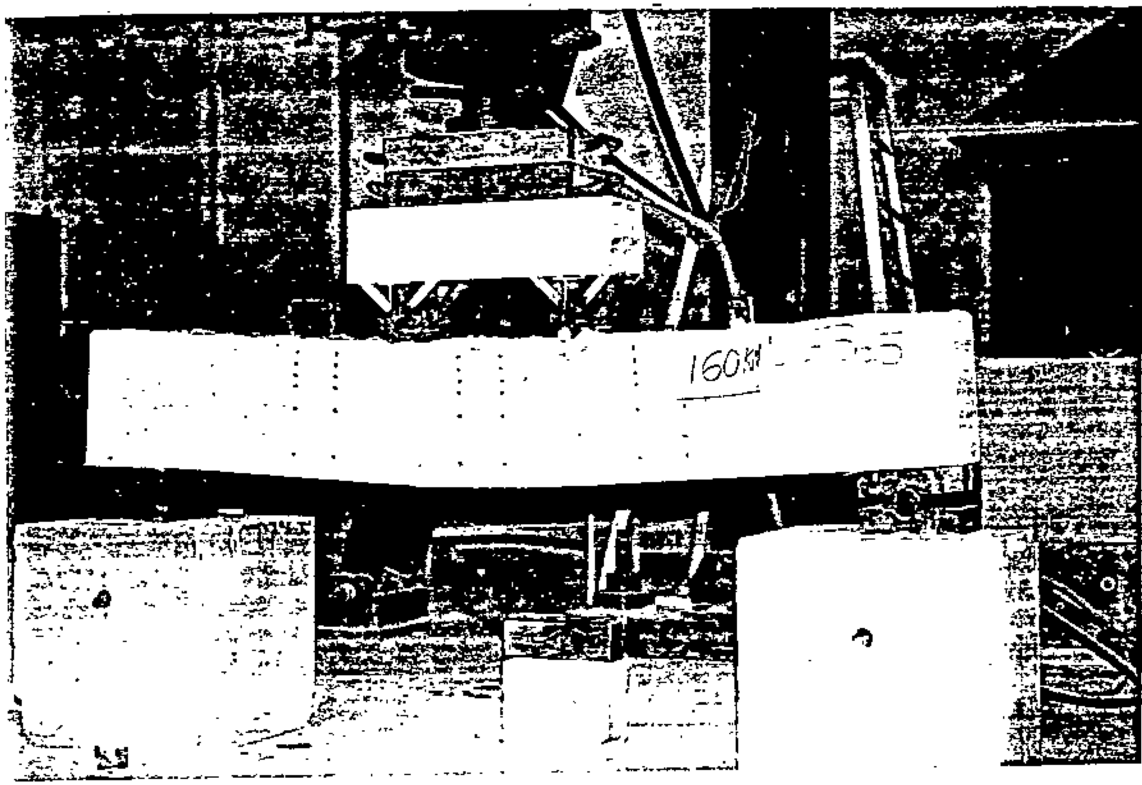
To illustrate how the crack grouth with the load hapen, see plate (4.10) and table (4.11).



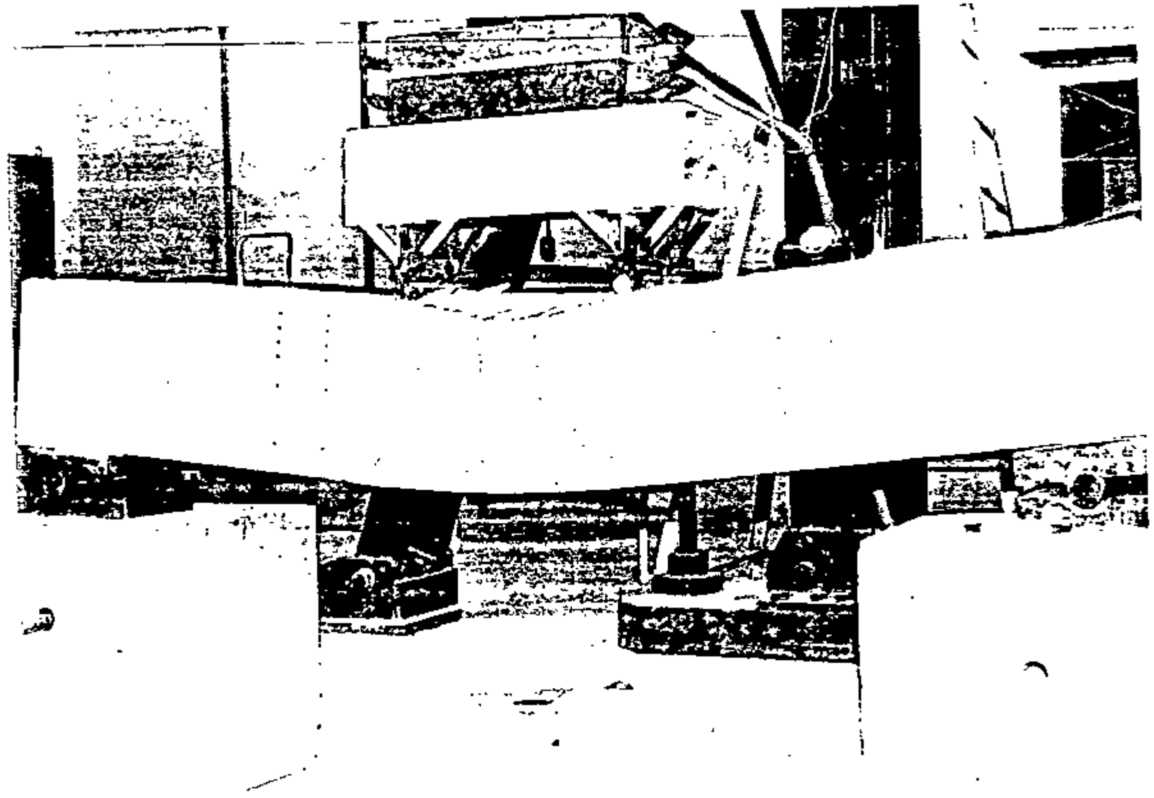
PLATE(4.1a) - TYPICAL CRACK PATTERNS FOR BEAM # 1
AT FAILURE LOAD



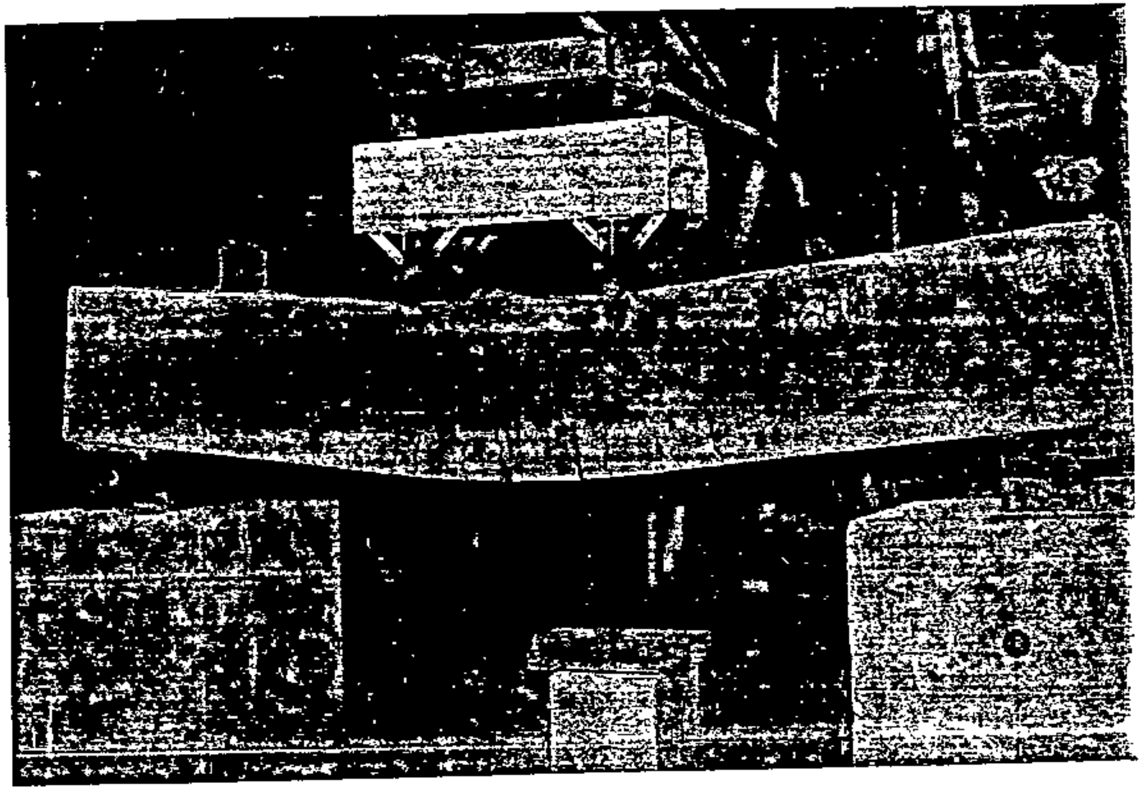
PLATE(4.2b) - TYPICAL CRACK PATTERNS FOR BEAM # 2 AT FAILURE LOAD



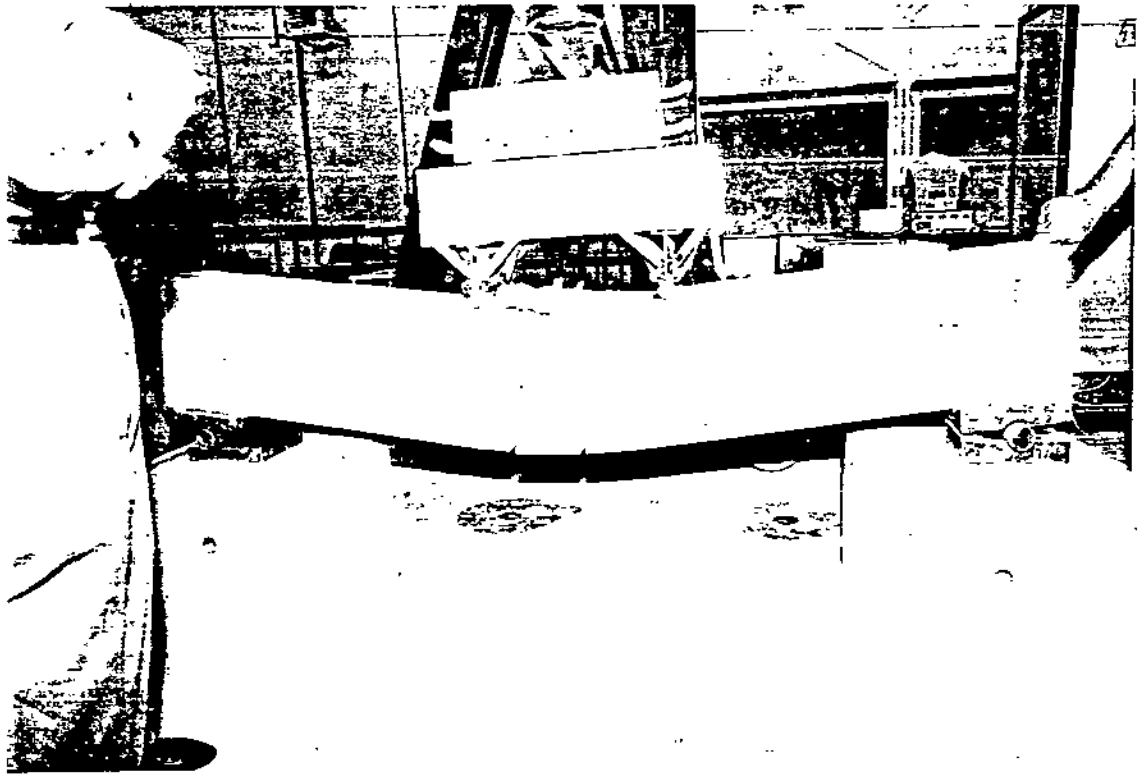
PLATE(4.3b) - TYPICAL CRACK PATTERNS FOR BEAM # 3
AT FAILURE LOAD



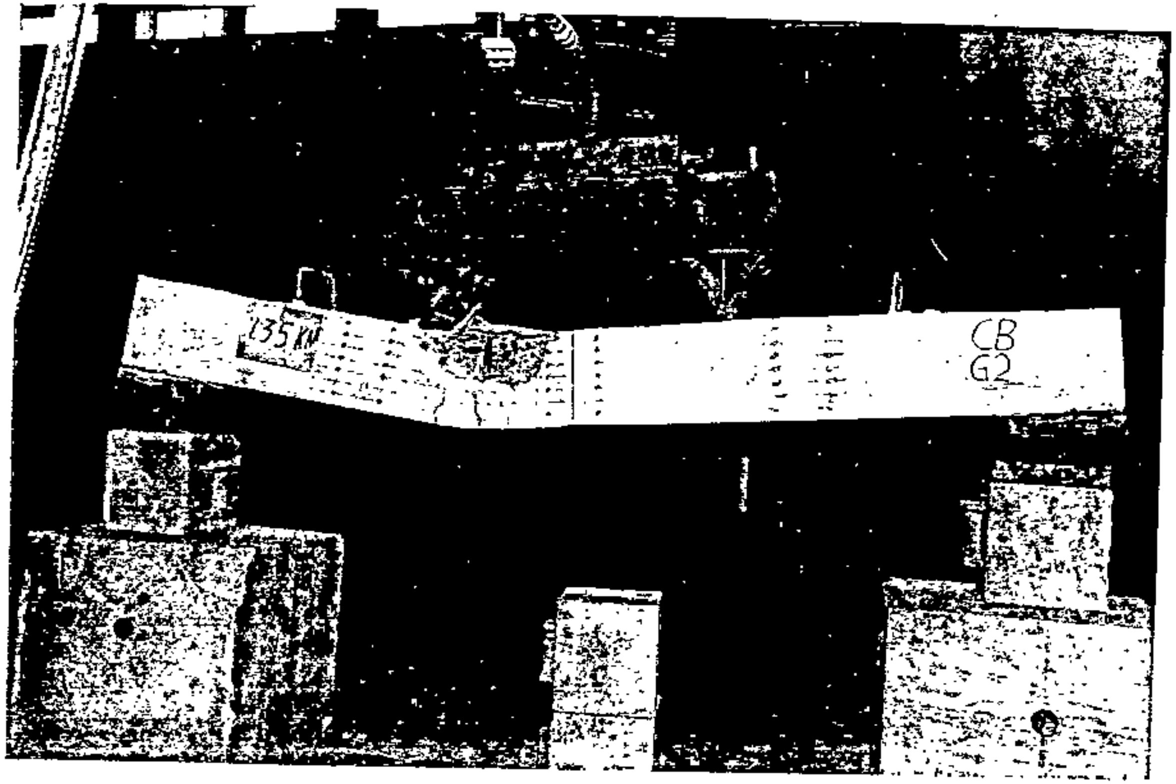
PLATE(4.4b) - TYPICAL CRACK PATTERNS FOR BEAM # 4 AT FAILURE LOAD



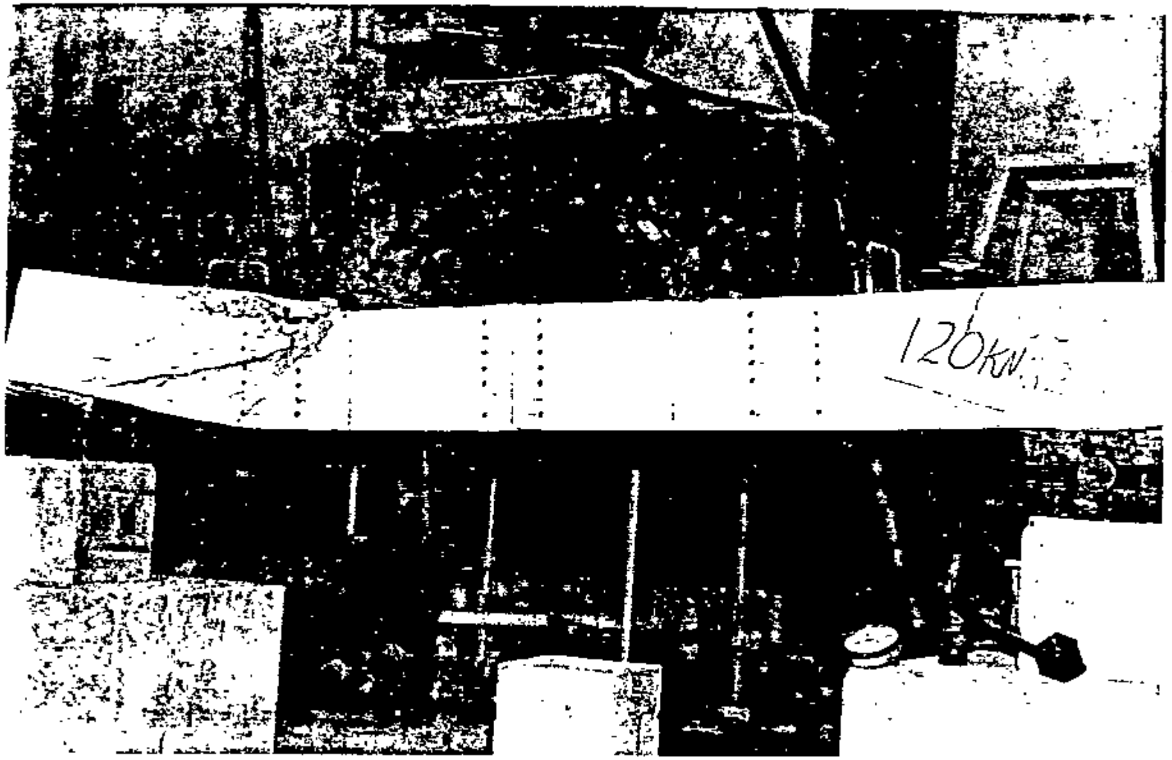
PLATE(4.5b) - TYPICAL CRACK PATTERNS FOR BEAM # 5
AT FAILURE LOAD



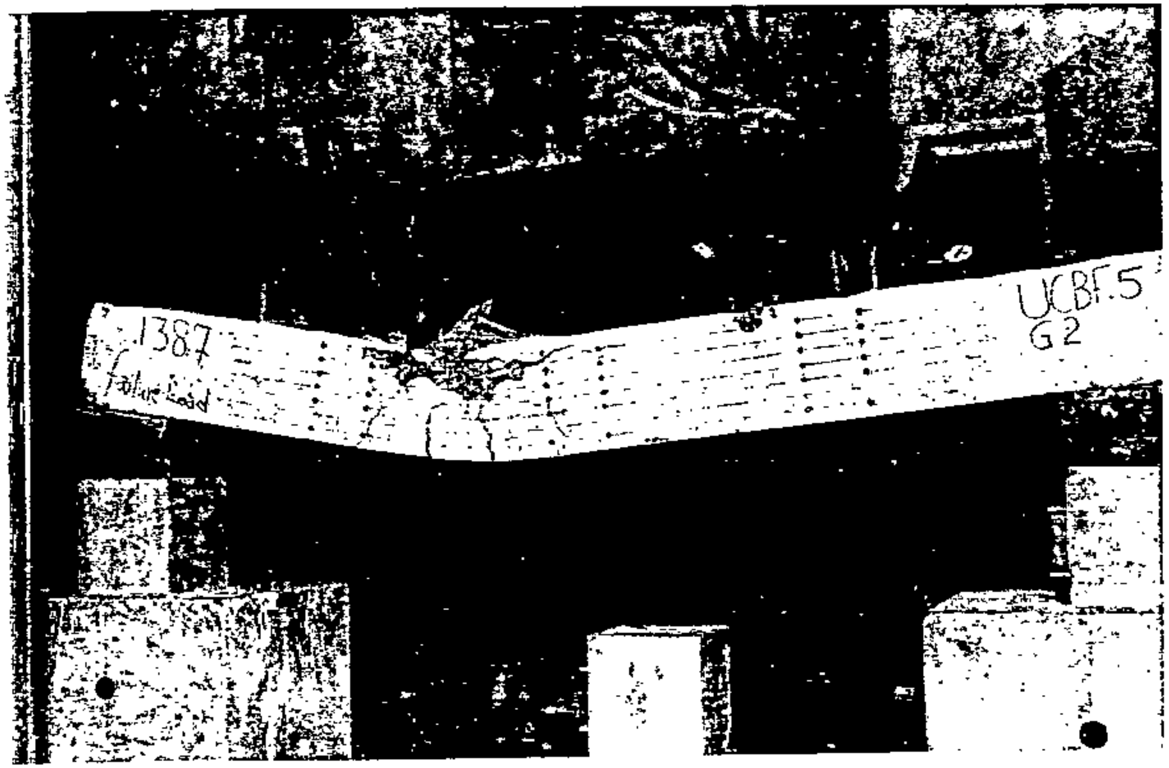
PLATE(4.6b) - TYPICAL CRACK PATTERNS FOR BEAM # 6
AT FAILURE LOAD



PLATE(4.7b) - TYPICAL CRACK PATTERNS FOR BEAM # 7
AT FAILURE LOAD

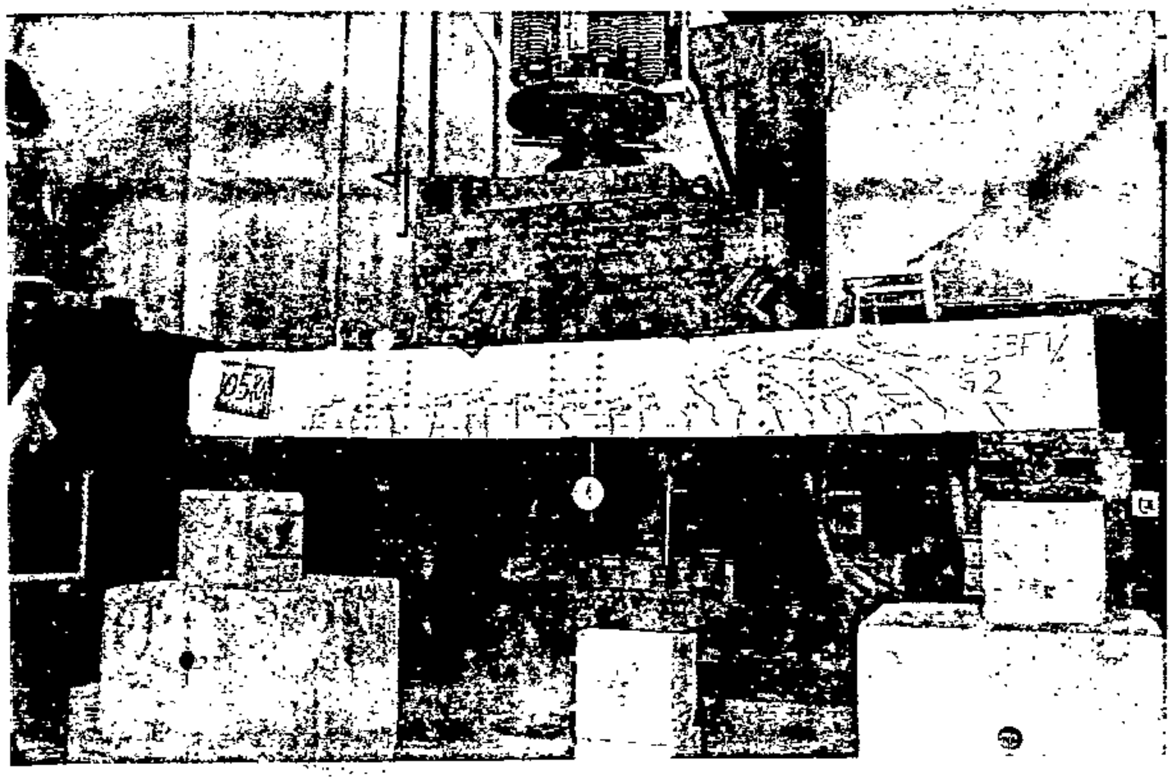


PLATE(4.8b) - TYPICAL CRACK PATTERNS FOR BEAM # 8
AT FAILURE LOAD

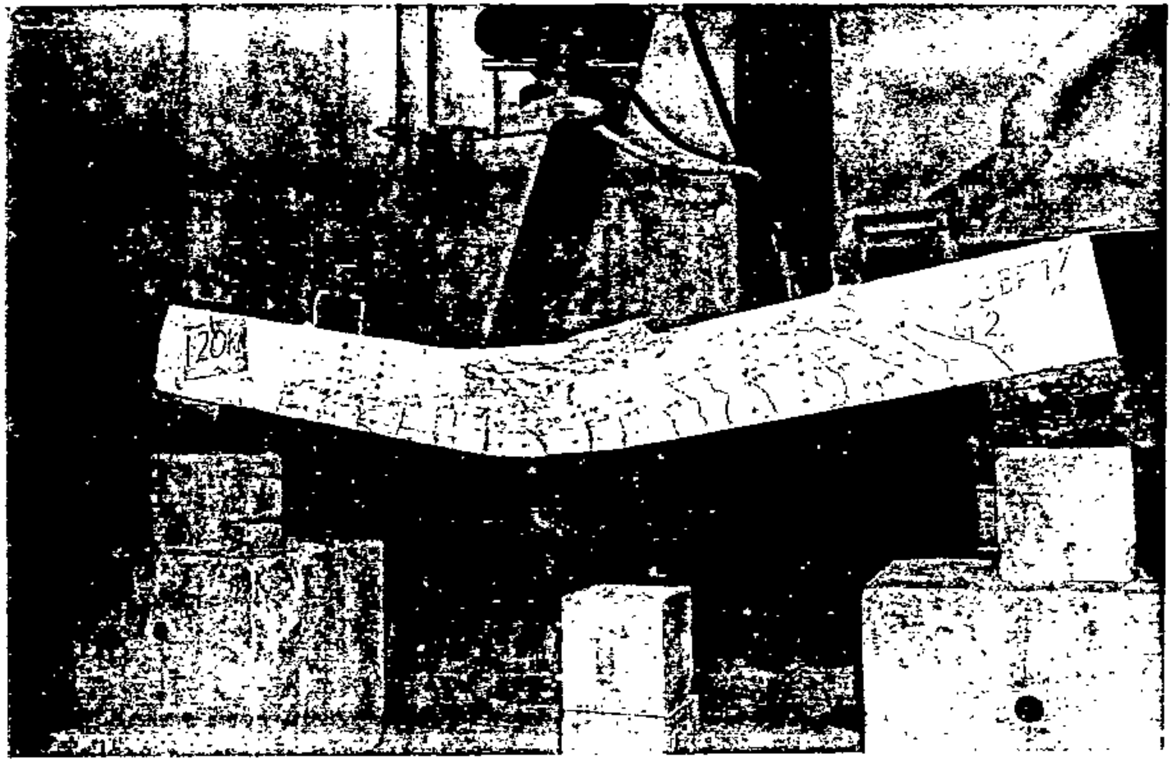


PLATE(4.9b) - TYPICAL CRACK PATTERNS FOR BEAM # 9
AT FAILURE LOAD

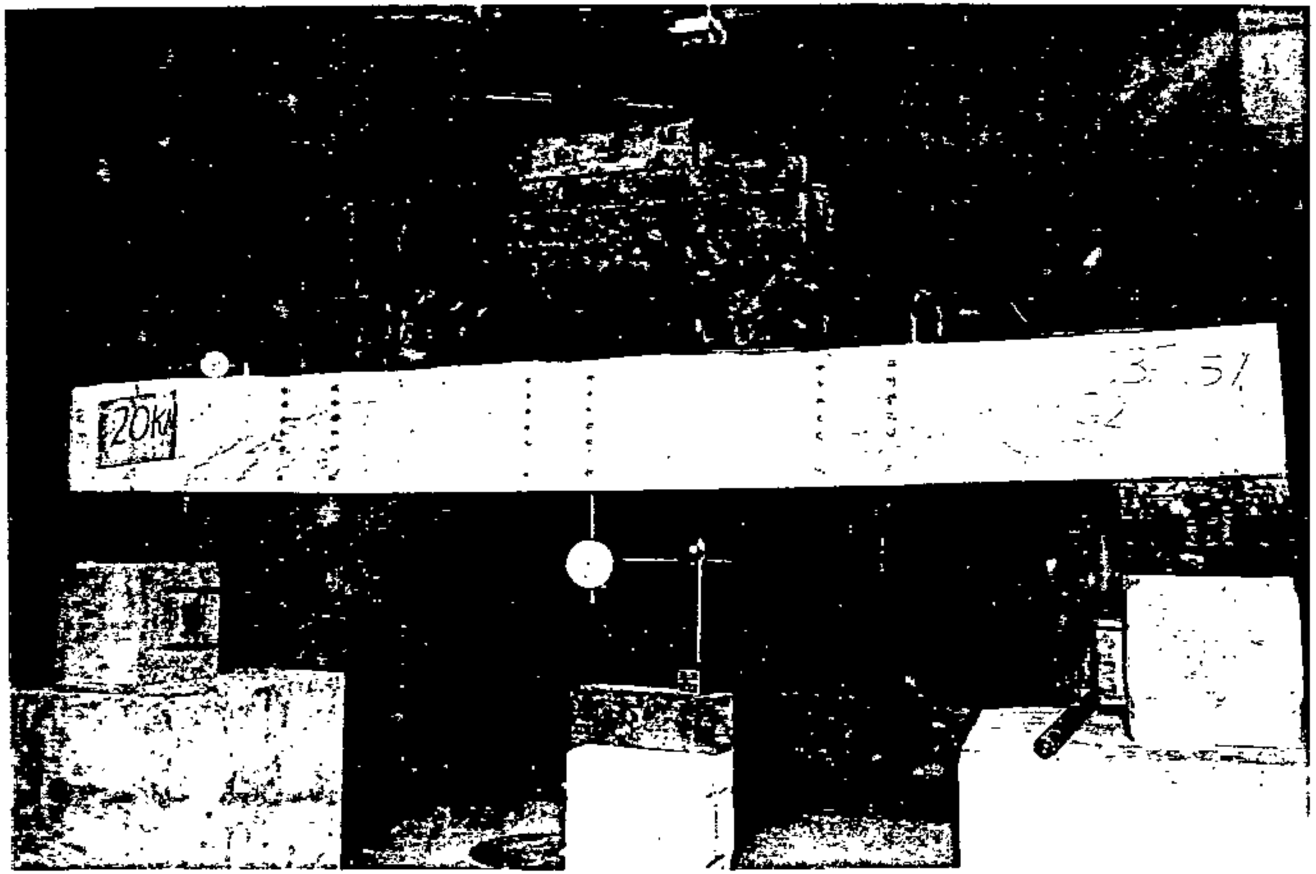
PLATE(4.10) - TYPICAL CRACK PATTERNS FOR BEAM # 10
AT FAILURE LOAD



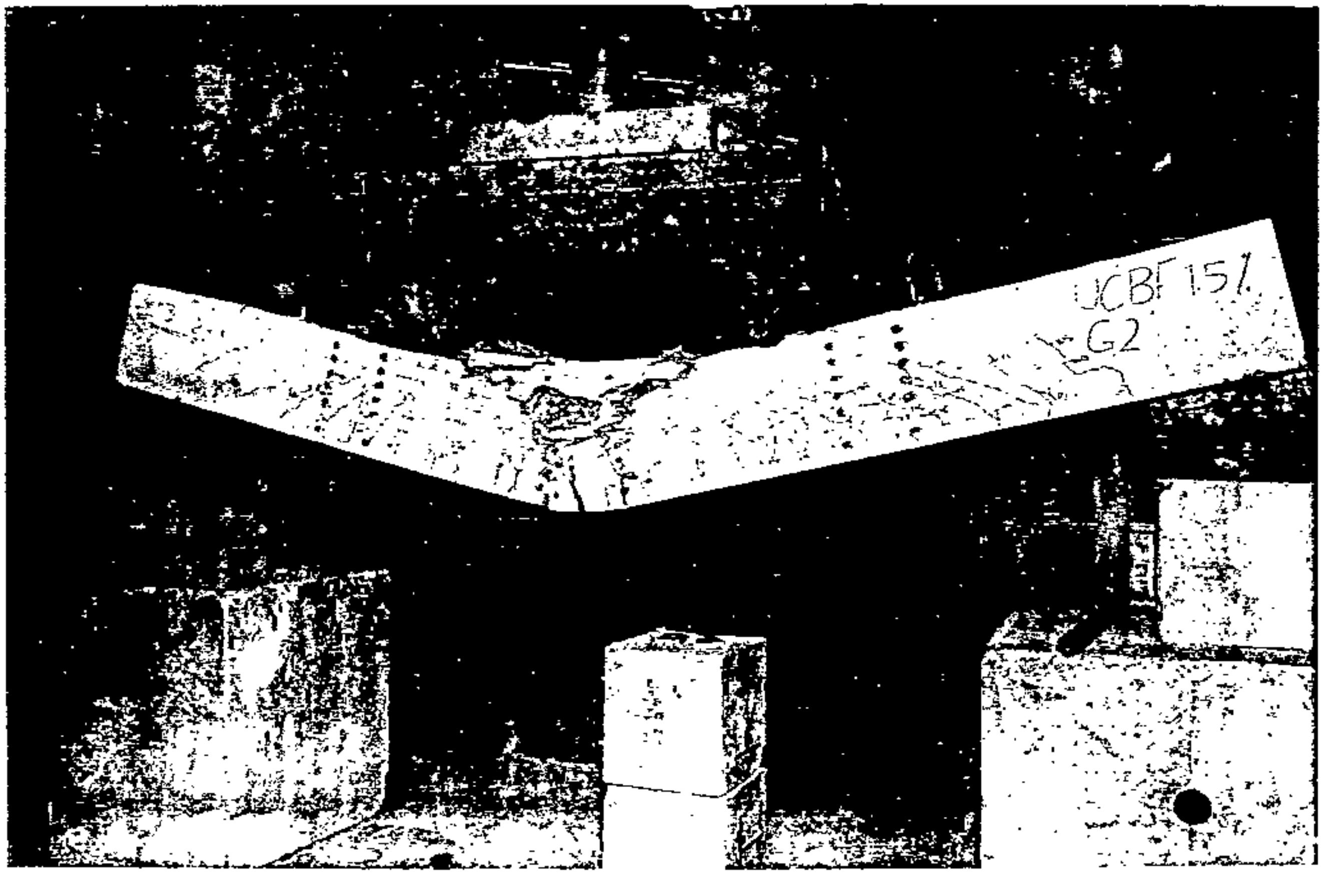
PLATE(4.11a) - TYPICAL CRACK PATTERNSFOR BEAM # 11
AT 105 KN



PLATE(4.11b) - TYPICAL CRACK PATTERNSFOR BEAM # 11
AT FAILURE LOAD



PLATE(4.12a) - TYPICAL CRACK PATTERNSFOR BEAM # 12
AT 120 KN



PLATE(4.12b) - TYPICAL CRACK PATTERNSFOR BEAM # 12
AT FAILURE LOAD

4.7 RESULTS OF CONTROL TESTS

4.7.1 fresh concrete

The freshly mixed concrete was tested for slump (ASTM C 143-78) and for air content to measure the value of workability, according to (ASTM C231-71), but because the slump test wasn't suitable for concrete with fiber so the vebe time was measured and all result for tests carried on fresh concrete are shown in table 4.7 &4.8

BEAM NAME	mixing date	slump mm	air content	V_t second
CB_1	11/9	160	3.1%	3
UCB_1	12/9	170	2.9%	2.5
$UCB_1F.5\%$	17/9	155	2.8%	4
$UCB_1F.75\%$	18/9	120	2.6%	7
$UCB_1F1\%$	19/9	115	2.5%	7.5
$UCB_1F1.5\%$	23/9	70	2.4%	10

Table 4.7 properties of fresh concrete for first series .

The workability of fibered concrete is significantly effected by the fiber, hence the method of mix design should grantee that the workability of fresh fibered concrete is accepted accordingly the water cement ratio adjusted in our mixes based on the vebe test.

BEAM Name	mixing date	slump mm	air content	V_b second
CB_2	24/9	165	3.2%	4
UCB_2	25/9	180	2.9%	3
$UCB_2F.5\%$	26/9	150	2.9%	4
$UCB_2F.75\%$	27/9	125	2.7%	8
$UCB_2F1\%$	30/9	117	2.4%	9
$UCB_2F1.5\%$	01/10	65	2.2%	12

Table 4.8 properties of fresh concrete for second series.

4.7.2 hardened concrete

The hardened concrete was tested for compressive strength and static flexural strength at 28 days. The test for compressive strength was done according to (ASTM C 39-72). In the test for flexural strength, third point loading was applied to the 14in (356 mm) long beam. The tests were made according to (ASTM C 496 -79), the results for each mix were shown in table 4.9 & 4.10.

The results obtained for fibers concrete indicated enhancement in the strength dependent on the percentage of fiber inclusion in concrete.

BEAM Name	symbol	cube strength column 2	cylinder strength column 3	rupture modulus column 4	column 5
CB_1	UCUF	20.1	18.43	2.8	.92
	CUF	24.3	20.5	2.81	.84
UCB_1	UCUF	21.0	22.5	2.8	1.07
	CUF	20.4	18.7	2.95	.91
$UCB_1F.5\%$	UCUF	19.8	15.7	3.1	.79
	CUF	22.0	16.8	3.3	.76
	UCF	23.3	20.6	3.2	.88
	CF	25.3	21.1	3.4	.83
$UCB_1F.75\%$	UCUF	22.7	20.3	2.7	.89
	CUF	21.2	21.8	3.0	1.03
	UCF	26.4	20.7	3.1	.78
	CF	27.4	23.5	3.2	.86
$UCB_1F1\%$	UCUF	21.8	18.6	3.2	.85
	CUF	22.8	20.4	3.4	.89
	UCF	23.1	21.4	3.7	.93
	CF	25.5	22.9	3.9	.9
$UCB_1F1.5\%$	UCUF	21.3	18.1	2.6	.85
	CUF	21.4	17.6	3.3	.82
	UCF	21.8	20.3	3.2	.93
	CF	28.7	23.3	4.5	.81

Table 4.9 properties of hardened concrete for first series.

BEAM Name	SYMBOL column 2	cube strength column 3	cylinder strength column 4	rupture modulus column 5	column 6
CB_2	UCUF	26.6	20.2	2.8	.76
	CUF	27.6	19.0	2.81	.69
UCB_2	UCUF	25.5	22.0	2.8	.86
	CUF	27.9	21.5	2.95	.77
$UCB_2F.5\%$	UCUF	24.7	20.4	3.1	.83
	CUF	23.9	21.6	3.3	.9
	UCF	25.9	21.9	3.2	.85
	UC	28.3	23.9	3.4	.84
$UCB_2F.75\%$	UCUF	24.2	22.5	2.7	.92
	CUF	27.6	22.8	3.0	.83
	UCF	26.4	23.4	3.1	.89
	UC	30.7	24.5	3.2	.85
$UCB_2F1\%$	UCUF	23.5	19.9	3.2	.85
	CUF	25.6	22.2	3.4	.87
	UCF	28.8	20.7	3.7	.72
	UC	23.9	24.9	3.9	1.04
$UCBF_21.5\%$	UCUF	26.3	21.0	2.6	.80
	CUF	25.4	23.7	3.3	.93
	UCF	26.4	22.2	3.2	.84
	UC	24.2	21.0	4.5	.88

Table 4.10 properties of hardened concrete for second series.

Chapter 5

DISCUSSION OF TEST RESULTS

5.1 Cracking Load

From the results shown in tables 4.1 & 4.2, it was observed that for the same longitudinal percent of reinforcement, different first crack moment were obtained.

For all beams with fibers the corresponding cracking moments were larger than the crack moments for cured and uncured conventionally reinforced concrete beam. Also from these tables it is clear that the cracking resistance of concrete members in bending is a function of the percentage of fibers. This occurs because the moment of first crack is a function of tensile strength[20]:

$$M_{cr} = \frac{f_r I_g}{y_t} \tag{5.1}$$

WHERE

- M_{cr}: Cracking Moment.
- f_r : Flexural tensile strength, also known as modulus of rupture.
- Ig : Moment of inertia of the gross concrete section.
- y_t : The distance from the centroidal axis of the gross section to the

extreme fiber in tension.

For conditions of equal elastic strain in fiber and matrix, the tensile strength of a composite σ_c, containing uniaxial continues can be expressed by the law of mixtures[1, 24] as:

$$\sigma_c = \sigma_m(1 - V_f) + \sigma_f V_f \tag{5.2}$$

Where

σ_c :uniaxial tensile strength carried by composite.

σ_m :uniaxial tensile strength carried by matrix.

σ_f :uniaxial tensile strength carried by fibers.

$1 - V_f$:volume fraction of matrix.

V_f :volume fraction of fibers.

In the research only fiber reinforced composites in which randomly oriented short length of fiber are incorporated are used. At the failure of such composite, fiber pull-out invariably occurs since the effective lengths of each fiber in the direction of stress is less than the critical length, l_c. Equation (5.2), can therefore modified to apply to this case, if σ_f is considered to be the stress in fiber caused by the fiber - matrix interfacial bond stress, and V_f is replaced by an equivalent volume of fiber which are effective in the direction of stress. The average tensile stress in the fibers related to the average bond stresses given by [24], is:

$$\sigma_f = 2\tau \frac{l_f}{d_f} \tag{5.3}$$

Where

τ : the averge interfacial bond stress

d_f : is the diameter of fiber.

l_f : the length of fiber.

And the effective fiber length from references [1, 24, 25], is equal to .82, and the effective fiber volume or orientation factor for the fibers distributed in 3 - dimensions is given by

$$V_{fe} = .41V_f \tag{5.4}$$

Substitute Equation(5.3) for σ_f, and Equation(5.4) for V_{fe} instead V_f in Equation (5.2) yields a modified law of mixtures equation (5.5) for composite containing randomly oriented fibers which fail by pull-out :

$$\sigma_c = \sigma_m(1 - V_f) + 2\tau\frac{l_f}{d_f} \times .41V_f \tag{5.5}$$

From the results of rupture test tables 4.7 & 4.8 it was observed that the fibrous beam had higher tensile strength than the plain concrete.

The rupture test for plain concrete beams occurs suddenly , however, for fibrous concrete the rupture test behaves differently, due to the hooking effect of the fiber and their capacity to carry tensile forces. And consequently the failure characteristics for test, show gradual failure associated with pulling out of the failure embedded in the concrete at the failure section.

Tables 4.1 & 4.2 show the crack point for all beams, for cured, uncured conventionally concrete and fiber concrete. It had showed consistently higher cracking loads, for the first set the ratio of moment at first crack to the ultimate moment varied from 14.7% to 15.1% for the uncured beams with fiber concrete compared to 17.6% for uncured conventionally concrete and 16.4% for cured conventionally concrete, also for second set from column 4 in table 4.2 the ratio of moment at first crack to the ultimate moment varied from 15.5% to 20.0% for the uncured beams with fiber concrete compared to 16.7% for uncured conventionally concrete and 17.4% for cured conventionally concrete.

From tables 4.1 & 4.2, it is shown that for cured and uncured conventionally reinforced concrete beam no significant difference in cracking point for beams with low percent of steel and there are significant increases in the first crack load for a fibrous concrete beams over ordinary reinforced beam, the ratio were 2.2%, 7.8%, 12.6%and 21.7% for .5%, .75%, 1.0% and 1.50% weight percent of fiber respectively. So the first crack point increase as the amount of fibers increase, but from table 4.2 it is shown that the uncured beam loss 21.7% from it is crack strength, also the uncured beams with .5% & .75% loss 8.5% & 0.0% from it is strength, but the others uncured beams with 1% &1.5% gain increase in the first crack strength by 12.8% and 19.1%, from these comparison it has shown what is lost in crack strength due to improper or no curing, can be gained by inclusion .5% of total weight if the beams designed as under reinforced and inclusion 1% of total weight if the beams designed as over reinforced.

5.2 ULTIMATE STRENGTH

For the first set it were shown in table 4.1 the increase in the experimental ultimate flexural strength of the uncured reinforced concrete beams due to the presence of steel fibers was significant, the maximum increase was being 32.7% due to inclusion 1.5% of fibers from the total weight of mix. The reduction in the flexural strength due to improper or no curing was 7.3% and can be gained by inclusion of .5% fibers in uncured beams, also from table 4.2 for the second set ' over- reinforced beams ' the increase in the experimental ultimate flexural strength of the uncured reinforced concrete beams due to the presence of steel fibers was only marginal, the maximum increase was being 3.8% due to inclusion .75% of fibers from the total weight of mix. The reduction in the flexural strength due to improper or no curing was 11.11% and can be gained by inclusion of .50% fibers in uncured beams.

From tables 4.1 & 4.2 it is shown that the ultimate flexural strength increase as percentage of fibers increase, this due to the improvement of tensile strength. This increment in the ultimate flexural strength due to the improvement of tensile strength of concrete and increase in the stiffness due to the ability of fiber concrete to arrest cracks and of fiber bridging the cracks to carry loads. From this study and others it is clear the use of fibers may not be the most economical means of achieving hight ultimate strength in conventional reinforced concrete beams especially in over-reinforced concrete beams but the use of steel fibers in beams improve the ductility of the beam and change the brittle failure or sudden failure in beam to ductile manner and give the required time to warning.

To understand how the reinforcement by steel fibers work, a brief explanation is appropriate concerning the differences in the flexural strength of fibrous concrete compared to that of conventional concrete.

Conventional concrete is relatively brittle with load versus deflection properties (which are a measure of flexural strength) that are linear up to an crack load at which a crack is initiated in the concrete and approximately linear up to yield load which is followed by failure load directly.

After first crack happen the tensile strength of concrete becomes zero in tension zone, but In fibered concrete load versus deflection properties are linear up to the first crack, at this load the tensile strength of concrete transmitted to the adjacent fibers in the cross section of concrete. Past this first crack point the load - deflection of fibrous concrete become non-linear, it was seen figure 4.3 through 4.6.

While the load is increased the first crack is propagated towards the failure, the stresses in concrete transmitted to the adjacent reinforcement steel and fibers steel. After reaching an ultimate load at which failure begins to occur, fibrous concrete withstands additional load energy which is necessary to debond the fibers [1].
Since the fibers do not fracture under a continuously heavy load, fibrous concrete does not fail until the fibers bridging the crack, have been pulled out,the hooked ends of the fibers cause a higher efficiency of the reinforcement.

And the balance of forces in the center point of fiber gives by :

$$\frac{l_f}{2}\tau_m \pi d_f = \frac{\pi}{4} d_f^2 \sigma_a \qquad (5.6)$$

Where

τ: average value of adherence tension.

σ_a =tension in the steel fibers.

l_f = fiber length.

d_f = fiber diameter.

The critical aspect ratio of steel fibers is controlled by the fiber failure and equal to

$$\frac{l_f}{d_f} = \frac{\sigma_a}{2\tau} \qquad (5.7)$$

As example if

σ_a=12000 kg/cm^2 Ultimate tensile strength

$$\tau = 24 \text{kg/cm}^2$$

$$\left(\frac{l}{d}\right)_{cr} = \frac{12000}{2 \times 24} = 250$$

The available steel fibers never reach such a high aspect ratio due to economical ' high cost ' and practical mixing reasons.

As a result of the fiber properties fibrous concrete does not fail by fibers fracture but by pulling the fibers out of the concrete. See plate (4.1), & plate (5.1). The stresses in a steel fiber $\frac{6}{.8}$ (aspect ratio 75) being pulled out of the concrete is

$$\sigma_a = 2\tau\frac{l_f}{d_f} = 2 \times 24 \times 75 = 3600 \text{kg/cm}^2$$

$$< 12000 \text{ kg/cm}^2$$

A τ_m (better mechanical adherence) increase causes an increase of σ_a or a better use of the steel quality. The hooked ends aid a lot to increase the value of τ_m.

5.3 DEFLECTION

Figures 4.1 though 4.7 & figures 4.8 though 4.14 showed the load deflection characteristics for the beams of first set & second set respectively. It is shown that the presence of fiber in concrete increase the slope of the load deflection (measure for the value of the effective EI), from these figures it is clear that the deflection for the uncured beam with fiber is less than the deflection of the conventionally beam at the same load, also from figures as the amount of included fiber increases, the smaller the deflection at the same load for all stages. It is clear that the area under load-deflection for the fibered beam is more than ordinary beam (which is measure for the energy required to fracture beam).

To show how fiber work and reduce deflection, it is known that the concrete after crack can't carry any tension strength so the neutral axis go up and the rotation of the beam become more produce large deflection due to increase load. In fibrous concrete the tensile strength of concrete after fracture transmitted to the adjacent fibers, the fiber bridges the crack and redistribution the stresses in the uncracked section of concrete, when the stress in the uncracked section reach the fracture strength also transmitted the stress to the fiber produces the new crack but small so less rotation lead to less deflection.

The flexural rigidity of the beam (EI value - elastic modulus times second moment of area) from load deflection curve for the beam with fibers which is shown to be increased by some 6.5% to 15.9% compared to the cured beam without fibers reinforcement. This increase in EI was due to two reasons, first the lowered of the neutral axis which lead to increase the value of I value , second the increase in the elastic modulus value of composite Ec due to the mixture law [1, 26]

$$E_c = V_f E_f + E_m(1 - V_f) \tag{5.8}$$

Where E_c, E_f, and E_m are moduli of elasticity for the composite, fibers and mixture. V_f is the volume fraction of fiber adjusted for the effect of randomness. So equation 4.8 will be modified to be used for randomly orientation of fiber in 3 - dimension and became [26],

$$E_c = .43 \times V_f E_f + E_m(1 - V_f) \tag{5.9}$$

The difference in deflection in the first stage, uncracked section was moderate, but the the deference in the post cracking is more pronounced. Typical results were tabulated in table 4.3 &4.4 at cracking and ultimate loading for all beams.

5.4 Cracking Characteristic

From plates 4.1 -4.12 it is clear that fibrous reinforced concrete beams showed consistently smaller crack length and width at the same load than the ordinary reinforced concrete beam. This superior resistance to crack propagation was observed for all stages of loading, although near the failure loads difference in crack width tended to decrease. Nevertheless, the crack arrest properties of fibrous concrete observed in conventional modulus of rupture tests were fully reproduced in large structural member.

There is, however, a fundamental difference in the stress redistribution over the cross section due to flexural cracking between plain concrete and fiber concrete. The fibers bridge across the crack and sustain small tensile force, thus it is possible to see that the bond stress at crack does not diminish completely as in the case of ordinary concrete.

The presence of fibers in the tension zone also controls the propagation of the cracks over the depth of the beam and all the results consistently showed that the flexural crack penetration towards the compression zone in fibrous concrete was less than the usual penetration observed in ordinary reinforced concrete beams.

5.5 Behaviour Of Fibrous Concrete In Compression Zone

Generally, the presence of fibers in concrete beams showed higher degree of compressibility and concrete strain prior to failure was higher than in ordinary concrete. They outcome of this will be the ability of attaining larger rotations, curvature and plastic deformations than ordinary concrete. This was illustrated in plates of the tested beams and from the load deflections curves, see plates(4.1 through 4.12), it is clear that all beams will fail flexurally except the 2nd beam (UCB_2) from the second set which fail due to shear failure.

5.6 Deformation Characteristics

From figures (4.15 thought 4.38) which is a plot of strains versus depth of cross section for different load, it was observed that the strains and curvatures are smaller in the beams with fiber than strains and curvatures in the ordinary beams at the same load. But the fibrous beams have larger strains and curvature at failure load.

The effect of fiber become more effective when the section is cracked and this is due to the ability of fibers to mobilize the tension zone section, to resist deformation by the lowered neutral axis and increases the elastic modulus than in ordinary concrete. For the under - reinforced concrete beams the increase in neutral axis depth at failure load varies from 1% to 21.2% in the fibrous concrete beams compared to the neutral axis in ordinary concrete beam. But in the second set in which designed as over - reinforced concrete beams the differences in the neutral axis between uncured fibrous beams and conventionally beams are negligible.

From figures (4.15 through 4.20) it is noticed that the last readings for strain at compression face in the first set varies from .141% to .118% for the uncured fibrous reinforced concrete beams compared to .172% for the cured reinforced concrete beam and .170% uncured reinforced concrete beam. Also from figures (4.21 through 4.26) it is noticed that the last readings for strain at compression face for second set varies from .293% to .273% for the uncured fibrous reinforced concrete beams compared to .294% & .295% for the cured and uncured reinforced concrete beam respectively.

To understand how the fibers reduce the deformation of the fibrous reinforced concrete, see the figures (4.27 through 4.40) for M - ϕ the slope of these curves equal the magnitude of the flexural rigidity EI for different beams, the increase in value of EI for the fibrous beams in the second set varies from 3.8% to 18.9% compared to the value of EI for ordinary concrete. Also after crack the increase in value of EI for the fibrous beams varies from -.7% to 9.6% compared to the value of EI for ordinary concrete. The reduction due to no curing after crack was 12.2% and can gained by inclusion .75% fibers in concrete.

The increase in value of EI for the fibrous beams in the first set was marginal and varies from 8% to 30% compared to the value of EI for ordinary concrete. Also after crack the increase in value of EI for the fibrous beams was very large and varies from 22% to 84% compared to the value of EI for ordinary concrete. The reduction due to no curing after crack was 7% and can gained by inclusion .5% fibers in concrete.
It is known that the deformation depends on the the value of flexural rigidity, also this value depends on the percentage of fibers inclusion in concrete. It is clear from equation(5.8)[1] this increment in stiffness leads to small deflection and delay the propagation of crack [8].

5.7 Control Tests

From the data obtained for the specimens, the relation ship between cube strength and cylinder strength has not been determined. Also the relation ship between flexural strength and direct tensile strength has not been determined. Slumps and air content were lower for fiber reinforced concrete see table (4.7)& table (4.8). There is a strong correlation between slump and air content. So the rate of loss of slump and the air content are slightly higher for fiber reinforced concrete.

[1]Section 5.3 deflection

Chapter 6

CONCLUSION

i. The addition of fibers reduces the slump and air content. The rate of loss of slump and air content are slightly higher for fiber reinforced concrete.

ii. The presence of fiber substantially increases the density of cracking (thereby reducing the crack width).

iii. Even a small amount of fiber reinforcement reduces the suddenness of failure of the unreinforced matrix, and imparts ductility to the composites.

iv. The role of fibers in inhibiting crack growth and crack widening is further confirmed by the increased depth of the neutral axis and increased flexural rigidity of the fiber concrete beam at all stages of loading.

v. The use of steel fibrous concrete in reinforced concrete beams has been shown to increase their ultimate flexural strength.

vi. Crack width and crack spacing were less in reinforced steel fibrous concrete beams, and first cracks occurred at higher loads.

vii. The post cracking stiffness of the reinforced fibrous beams was greater than a conventional reinforced beam.

viii. The results of experimental work have shown that the mode of failure for the uncured beam without fiber in second series was shear failure with no significant flexural cracking. But for all other beams in two groups were flexural behaviour with no significant shear cracking.

ix. What is lost in flexural strength and stiffness due to no curing or improper condition was gained by inclusion of .5% fiber by weight of the total mix.

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بسم الله الرحمن الرحيم

الجامعـــة الأردنيـــة

قسم الهندسة المدنية

"دراسة انحناء الجسور الخرسانية بدون ايناع وذلك بإضافة الألياف المعدنية"

إعداد

قاسم أبو غزلة

بإشراف

د. عبد القادر النجمي

عمان

آيار ١٩٩٠

الجامعــــة الأردنيــــة

كلية الهندسة والتكنولوجيا

قسم الهندسة المدنية

دراسة انحناء الجسور المسلحة الخرسانية بدون إيناع وذلك بإضافة الأليّاف المعدنية

ملخص

أجريت التجارب والفحوصات على جسور خرسانية غير معالجة ومسلحة بالألياف المعدنية ومعرضة للأحوال الجوية السائدة في عمان الأردن ، من أجل دراسة سلوك هذه الجسور تحت تأثير الإنحناء (الإنثناء) ومن أجل ايجاد تأثير إضافة الألياف المعدنية على التشقق وإمكانية استعمالها كبديل للمعالجة .

تم عرض ومناقشة نتائج فحوصات أجريت على مجموعتين من الجسور ، كل مجموعة مكونة من ستة جسور ، جسر مصبوب بخرسانة عادية ومعالجة ، وجسر أخر مصبوب بخرسانة عادية ومعالجة بدون معالجة ، والأربعة الباقية مصبوبة بخرسانة مصلحة مضاف اليها الألياف ، المعدنية بنسب ٥ر٪ ، ٧٥ر٪ ، ١٪ و ٥ر١٪ وجميعها غير معالج ، المجموعة الأولى مسلحة بكمية حديد نسبتها ٢ر١٪ من مساحة المقطع الفعال ، ومقطعها ٢٠٠x٢٣٠x٢٠٠٠ ملم ، والمجموعة الثانية نسبة التسليح فيها ٣ر٤٪ ومقطعها ٢٠٠x٢٠٠x٢٠٠٠ ملم . وكانت جميع الجسور بسيطة الإرتكاز ومعرضة للأحمال مركزة في نقطتين .

تم فحص جميع الجسور الاثني عشر وقد تم تسجيل قراءات لكل من الانفعال ، الترخيم (الانحراف) ، حمل الشق الأول محل التشققات ، حمل المقاومه الحدية (الفشل) ، كما تم مراقبة التشققات وإنتشارها أثناء إجراء الفحوصات تم إجراء مقارنة بين الجسور المعالجة في ظروف مخبرية قياسية ، وغير المسلحة بالياف معدنية (حديدية) من جهة ، وبين الجسور غير المعالجة ومسلحة بالألياف المعدنية من حيث سلوكها في حالات الانحناء (الإنثناء) والتشقق والترخيم .

<u>النتيجة</u> :

بالنسبة للمجموعة الأولى كان الفقدان في مقاومة الانحناء (الانثناء) للخرسانة الناتج عن عدم المعالجة يمكن تعويضه بإضافة الياف معدنية بنسبة ٥ر٪ أما في المجموعة الثانية فإن نسبة ٧٥ر٪ كافية للتعويض بدل الفاقد مما يعني إمكانية الاستغناء عن المعالجة وإدخال الالياف المعدنية خاصة في المناطق الصحراوية التي لا تتوافر فيها المياه ، دون إحداث ضار أو ضعف في الحرسانة .