

"EFFECTIVENESS OF OVERLAPPING HOOPS AS DUCTILITY REINFORCEMENT IN
SHORT REINFORCED CONCRETE COLUMNS"

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THESIS DECLARATION

I hereby declare that this Thesis has been entirely composed by me and that it is a record of my own research work. It has not been accepted in any previous publication for the award of higher degree.

Where quotations are made from previous publications, they are distinguished by quotation marks and the sources of informations are specifically acknowledged by means of reference.

YA'U, Alhaji Sanusi

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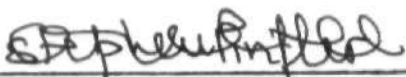
C E R T I F I C A T I O N

We hereby recommend that this Thesis, prepared under our supervision by YA'U, Alhaji Sanusi entitled "Effectiveness of Overlapping Hoops as Ductility Reinforcement in Short-Reinforced Concrete Columns" by accepted as partial fulfilment of the requirements for the award of Master of Science degree in Civil Engineering (Structures) of Ahmadu Bello University, Zaria and is approved for it's contribution to knowledge and literary presentation.



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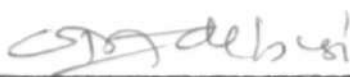
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D E D I C A T I O N

This Thesis and all it's entirety is thereby dedicated to
Galadima Iyama. The source of Alhaji ya'u family and also the first
Galadima of Kaura-Namoda. May His Soul Rest in perfect peace, AMEN.

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Thanks be to Allah, the beneficent and the merciful, for all his Guidance throughout - the period of this research.

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A B S T R A C T

Comparative study of the effectiveness of some typical arrangements of overlapping hoops to ordinary rectangular hoop was carried out. The overlapping arrangements **were:-**

Diamond Shape; Single Crossing; and Double Crossing (See appendix).

During the study, a fixed stirrup spacing of 300mm (Model Size 120mm) and concrete - grade of 25 was used in all the specimen. Twenty four (24) specimens were prepared for this research with each arrangement (Ordinary rectangular hoop, Diamond shape, single crossing and Double crossing) having six (6) specimen.

The concrete mix used in this experiment was designed to have a ratio of 1:1%:3 and a grade of 25 with water-cement ratio of 0.7 ($w/c = 0.7$). Mixing of this concrete was done manually using a dry batching process. The compaction of the cast specimens was by stamping internally, while poker-vibrator was used on the surface, for proper dense settlement of the mixed concrete.

Measurements of stresses and - strains of the specimens under loading was carried out with a strain meter. The use of strain-meter was possible through the help of STRAIN-GAUGES which were soldered on the reinforcement - bars at four (4) different positions (top **front** top-back, middle front and middle-back). These strain-gauges were also connected to a very thin long and flexible electronic wires by use of Araldite glue.

The testing of the prepared specimens was carried out using UNIVERSAL TESTING MACHINE. Care was taken in ensuring proper placements of the specimens on the machine before loading, in order to avoid excessive eccentric loading on the specimens. The loading of all the tested specimens was a gradual process, while the readings of internal stresses and strains of the specimens were

recorded at some interval different loadings, with the aid of a STRAIN METER.

Using the recorded results of the Specimens, "STRESS-STRAIN CURVES" were drawn in order to determine the ductility of each - of the arrangements under study. In theory, the ductility of a confined concrete is determined by measuring and computing the angle of slope of the descending part of it's 'STRESS-STRAIN CURVE.' The more acute the angle of the descending part of the curve, the more ductile is the confined concrete.

It was observed that in all the cases of the Specimen loadings, the steel started yielding before the ultimate load was reached. Thus no readings were obtained for higher loads. Also some of the Strain-Gauges were found to be faulty.

It was also observed that the failure was either at the top or the bottom of the tested Specimen. No column was able to carry higher load after spalling of concrete began.

As a check to the results of the tested specimens, both tensile and - compressive strengths were carried out on prepared samples of the materials of the specimens used. Tensile test was carried out on steel - samples, while compressive-test was done on the concrete cubes.

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CHAPTER 1

INTRODUCTION

1.1 GENERAL

No doubt, the planning, design, and construction of structures is essentially as old as man himself since nature provides on this planet an acceptable environment for man, but seldom are the detailed natural forms of this environment exactly suitable to his needs, convenience, and desires. Early man could probably find a natural cave or a hollow tree which would give him partial shelter from the elements, but it most likely would not be situated properly with respect to his source of food and water. So man started to supplement nature to meet his needs and thereby built his first STRUCTURE.

1.2 STRUCTURAL DESIGN

Structural design involves the most suitable proportions of a structure and dimensioning the structural elements and details of which it is composed.

Specifically, the structural design of any structure first involves the establishment of the loading and other design conditions which must be resisted by the structure and therefore must be considered in its design; then comes the analyses (or computation) of the internal gross forces (thrust, shears, bending-moments, and twisting moments), stress-intensities, strains, deflections, and reactions produced by the loads, temperature, shrinkage, creep, or

other design conditions. The criteria used to judge whether particular proportions will result in the desired behaviour reflect - accumulated knowledge (theory, field and model-tests, and practical experience), intuition, and judgement. For most common Civil - Engineering Structures such as Buildings, Bridges, etc, the usual practice in the past has been to design on the basis of a comparison of allowable stress intensities with those produced by the service loading and other design conditions. This traditional basis for design is called ELASTIC DESIGN, since the allowable stress intensities are chosen in accordance with the concept that the stress or strain - corresponding to the yield point of the material should not be exceeded at the most highly - stressed points of the structure. Ofcourse, the selection of the allowable stresses may also be modified by a consideration of the - possibility of failure due to fatigue, buckling, or brittle fracture, or by consideration of the permissible deflections of the structure.

Depending on the type of structure and the conditions involved, the stress intensities computed in the analytical model of the actual structure for the assumed design conditions may or may not be in close agreement with the stress intensities produced in the actual structure by the actual conditions to which it is exposed. The degree of correspondence is not important, provided that the computed stress intensities can be interpreted in terms of previous experience. By the selection of the service conditions and the allowable stress intensities, a margin of safety against - failure is provided. The selection of the magnitude of this margin depends on the degree of uncertainty regarding loading, analysis,

design, materials, and construction, and on the consequences of failure.

1.3 STRUCTURAL FAILURE

Collapse of a structure may be triggered by any of the following modes of failure as one of its elements: plastic - deformation or rupture of the material in a ductile manner, rupture of the material in a brittle manner, buckling, or fatigue of the material. Seldom is the collapse of a structure triggered by the rupture of material in a ductile manner, since such rupture is preceded by such large strains that excessive deflections would become apparent. Under normal types of loading, there would therefore be opportunities to relieve the structure either by unloading it partially, or by shoring it, or strengthening it temporarily. Ofcourse, in catastrophic - situations, such as Bomb-blasts, Earth-quake, etc. there is no opportunity to relieve the structure, and collapse could be triggered by ductile rupture or excessive plastic deflections. The design Engineer must therefore design his structures so as to obtain a suitable improbability that they will collapse or become unserviceable. He must be prepared to analyse the behaviour of the various - structural forms which he uses so as to estimate the possibilities of collapse or poor performance under the loading and environment to which they are subjected. If the theory is inadequate for such - predictions, the design engineer must make his evaluations and design decisions on the basis of his experience and model, and/or field tests.

Reinforced (and prestressed) concrete share with structural steel the position of the most important Civil-Engineering Structural materials. Although several Builders and Engineers experimented with the use of steel-reinforced concrete in the last half of the 19th century, it's dominant use as a building material dates from the early decades of this present century.

In reinforced-concrete construction, ductile behaviour can be obtained by - proper selection and arrangement of the reinforcement, because plain (Unreinforced) concrete not only is a heterogeneous material but has one very serious defect as a structural material, namely, it's very limited tensile-strength, which is only of the order of one-tenth it's compressive strength. not only is tensile - failure in concrete of a brittle type, but like wise compression failure occurs in a relatively brittle fashion without being preceded by the fore warning of large deformations. Concrete creeps under long-term loading to a degree which must be considered carefully in selecting the design stress conditions. During the curing process and it's early life, concrete shrinks a significant amount, which to a degree can be controlled by properly-proportioning the mix and utilizing suitable construction techniques.

With all these potentially serious disadvantages in concrete, Engineers have learnt to design and build beautiful, durable and economical reinforced concrete structures for practically all kinds of structural requirements. This has been accomplished by careful selection of the design dimensions, and the arrangement of the steel reinforcement, development of proper cements, selection of proper aggregates and mix proportions, careful control of mixing,

placing, and curing techniques, and imaginative - development of construction methods, equipments and procedures.

1.5

COLUMNS

American concrete Institute Code (1983 ACI), Section 2.1, defined column as a member used primarily to support axial compressive loads having a ratio of height to the least lateral dimensions of 3 or greater. A column is first subjected to pure axial load and then may also be subjected to eccentric loading. Reinforced concrete beams, floors, and columns are cast monolithically and this situation - produces some moment on the columns due to end restraint. Perfect alignment of columns is not possible; thus there will be eccentric loading on columns. Therefore it can be said that - axially loaded columns are those with very small eccentricities.

Prior to 1951 reinforced concrete columns were designed exclusively by the elastic theory. Laboratory tests have proven that this theory provide non-satisfactory results and that the ultimate strength theory must be used in order to obtain reasonable - correlation between theory and reality. Because the ultimate strength theory is based primarily on test data, it's usefulness is limited to the data from which it was devised. It is important to note then, that the equations are truly valid for steel ratios of 0.04 or less, and for steel having yield point stress values of 345 Mpa or less. Use of the equations for larger values involves extra-polation beyond the test data range and can often provide results which are grossly in error. As a consequence, the working stress theory is also limited in usefulness to this range.

The main types of reinforced concrete columns are:

1. Tied columns - have longitudinal bars braced by closed ties at intervals.
2. Spiral columns - have longitudinal bars braced with a spaced helix or spiral.
3. Composite columns - contain structural - steel or cast iron members - encased with concrete and may also contain longitudinal bars and spirals.
4. Combination columns - contain structural steel members wrapped with wires and encased in at least 60mm of concrete.
5. Steel pipe Columns - Hollow steel columns filled with concrete. Column sections may be square, rectangular, circular, L-shaped, octagonal, or any desired shape with an adequate side width.

Ties and spirals are provided in columns to serve two main functions: to hold the longitudinal bars in position in the forms during - concreting, and to prevent the slender, highly stressed longitudinal bars from buckling and bursting the concrete cover.

In columns, two different types of failure occur, depending on whether ties or spirals are used. For a tied column, the concrete fails by crushing and shearing outward, the longitudinal steel bars fail by buckling outward between - ties, and the column failure occurs suddenly, much like the failure of a concrete cylinder. A spiral column undergoes a marked yielding, followed by considerable deformation before complete failure. The concrete in the outer shell fails and spalls off. The concrete inside the spiral column is confined and provides little strength before the initiation

of a column failure. A hoop tension develops in the spiral, and for a closely spaced spiral the steel may yield. However, a sudden failure is not expected.

1.6 S T I R R U P S

When a concrete cross-section has insufficient area to maintain shear stresses below the permissible values, additional resistance to shear may be provided. One form of shear reinforcement consists of hoops or STIRRUPS, which may be placed vertically or at some angle with the horizontal.

The shear strength of a reinforced concrete structure is increased by the use of web reinforcement. Prior to the formation of diagonal tension cracks, web reinforcement contributes very little to the shear resistance. After diagonal cracks have developed, web reinforcement - augments the shear resistance of a beam, and a redistribution of internal forces occurs at the cracked section. When the amount of web reinforcement provided is small, failure due to yielding of web steel may be expected, but if the amount of web reinforcement is too high, a shear-compression failure may be expected, which should be avoided.

Concrete, stirrups, and bent-up bars act together to resist transverse shear. The concrete, by virtue of its high compressive strengths, acts as the diagonal compression member of a lattice girder system where the stirrups act as vertical tension members. The diagonal compression force is such that its vertical component is equal to the tension force in the stirrup.

Shear reinforcement or stirrups are also subjected to tensile stresses, and, therefore must be sufficiently anchored into the concrete to prevent slippage. ACI Building Code requires that stirrups should be anchored by at least one of four different ways:

- (1) A standard hook plus sufficient - embedment, with the effective embedment length being the distance between mid-depth of the member and the center of radius of the hook.
- (2) welding to longitudinal reinforcing steel.
- (3) Bending at least 180° around longitudinal reinforcement.
- (4) Embedment in the compression zone as in (1) above to develop bar stress with a minimum of 24 bar diameter.

Of the above, usually (1) or (4) are the most desirable from a construction stand point. In (2) the fitting and welding is a problem and can best be accomplished outside the form work. In (3) it is difficult, if not sometimes impossible, to bend a stirrup tightly enough around another bar to insure anchorage.

CHAPTER 2

LITERATURE REVIEW

2.1 GENERAL

The performance of a reinforced concrete member under load depends, to a great extent on the stress-strain relationship of concrete and steel, and on the type of stress applied to the member.

Stress-strain curves for concrete are obtained by testing a concrete cube or cylinder to rupture at the age of 28 days, and recording the strains at different load increment. The loads are transformed in to direct stresses (P/A), and the stress-strain diagram is then plotted. A typical stress-strain curve consists of an initial relatively straight elastic portion, reaching maximum stress at a strain of about 0.002, then - rupture occurs at a strain of about 0.003.

Before rupture occurs, reinforced concrete structures under load are recommended to have an ability to give warning. The ability for any reinforced concrete structure to give warning, that is taking more time to rupture even when ultimate load is exceeded, is referred as DUCTILITY. This is achieved, when a structure is able to transfer (for stability) excess bending moment to the sections where the ultimate moment is not attained through a rotation of assumed in built plastic hinges formed at that section.

In normal design practices, considerations are always given to the production of reinforced concrete structures that will be ductile enough to prevent hazards of accidents attributed to sudden

failure of such structures. Measurement of ductility of any reinforced concrete section is usually done by computing the ratio of ultimate curvature " ϕ_u ", to the curvature of steel at yield, " ϕ_y ", (Thus ϕ_u/ϕ_y). Another way of measuring the ductility of a concrete structure is by plotting the STRESS-STRAIN curve of the tested sample of the structure. The descending part of the curve will have an angle at the junction of the ascending and descending lines of the curve. This angle indicates the extent of the ductility of the structure. The smaller the angle, the more ductile is the structure.

2.2 DUCTILITY OF CONCRETE

The Ductility of a concrete structure refers to the ability of the structure to withstand to an extent an exceeded design load before failure occurs. This ability is achieved through proper and careful arrangement of transverse reinforcement of the structure.

Design codes and Engineers consider the Ductility of reinforced concrete structures as one of the important factors (in both design and construction) for safe and habitable structures.

Compressive strength of concrete when compared with it's tensile strength is very high. The tensile strength of concrete (very low) varies from as little as one twelfth ($\frac{1}{12}$) to as high as only one-eighth ($\frac{1}{8}$) of it's compressive strength. Primarily it is this weakness in concrete that makes it necessary to reinforce concrete with steel, thus making the concrete ductile. Concrete is a brittle material, and cannot resist the high tensile stresses that are important when considering cracking, shear and, torsional problems. The low tensile capacity can be attributed to the high stress

concentrations in - concrete under load, so that a very high stress is reached in some portions of the specimen causing microscopic cracks, while the other parts of the specimen are subjected to low stresses.

Direct tension tests are not reliable for predicting the tensile strength of concrete, due to minor mis-alignment and stress concentrations in the gripping devices. An indirect tension test in the form of splitting a 150mm x 300mm cylinder or cube is usually carried out. In this test, the concrete cylinder is placed - with it's axis horizontal in a compression testing machine. The load is applied uniformly along two opposite lines on the surface of the cylinder through two - plywood pads, as shown in the figure below. Considering an element on the vertical diameter and at a distance "y" from the top fibers, the element is subjected to a compressive stress:-

$$f_{e_u} = \frac{2P}{LD} \left(\frac{D^2}{yCD - y} - 1 \right) \quad \dots \quad (1)$$

and a tensile stress:-

$$f_{sp}^1 = \frac{2P}{LD} \quad \dots \quad (2)$$

where P = the compressive load on the cylinder

D = the diameter of the cylinder

L = the length of the cylinder

Y = the distance of element considered (see figure)

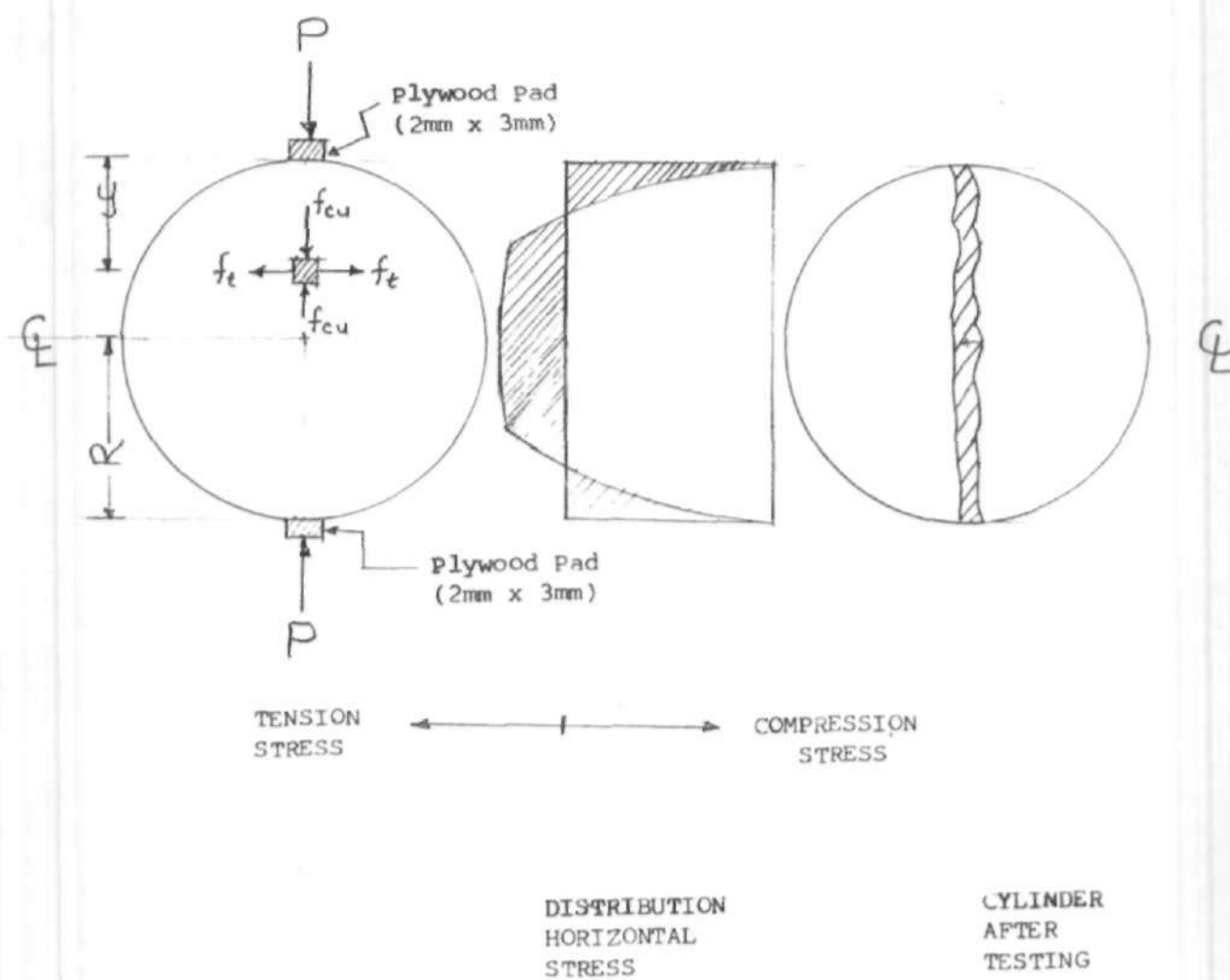


Fig. 0.1: CYLINDER SPLITTING TEST

2.3 CONCRETE CONFINEMENT

Concrete confinement is achieved by the introduction of transverse or lateral reinforcement. The function of these reinforcements is to prevent or reduce to a minimum the movement of concrete in the direction at right angles to the direction of the applied load or stress. This phenomenon when achieved, will help to induce the required reactive strength much needed for the area around the core of the concrete, in order to resist the uniaxial stress along the

structural member. As a result of this induced reactive, strength around the core of the concrete the concrete core now attains a total strength larger than the uniaxial compressive strength.

The concrete found between the outside surface and the surface of the lateral reinforcement is referred as 'CONCRETE COVER'. This concrete cover, under high strains serves no purpose in the ductility or strength of the reinforced concrete structure. The cover functions better in prevention of cracks on the structure due to exposure to atmospheric conditions (internally or external). Rapid spalling of the cover from the main structure can be seen when subjected to high strains. This is so in a much closely spaced transverse reinforcement, since they provide a weak plane between the inside concrete core and the outside surface of the structure. Some studies also found out that effective concrete confinement in reinforced concrete structures can be achieved in - circular spirals, because the axial hoop tension around the circumference provides confining pressure. This pressure is however only present at the corners in the case of rectangular - hoops. The unsupported length of the hoops only contributes when the bar diameter is thick enough to prevent buckling at high stresses.

The ductility of confined concrete can be seen by the relationship of stress and strain of the concrete in question. Such relationship is plotted in the form of graph and referred to as "STRESS-STRAIN CURVE." Curiosity along with the desire to improve the behaviour and ductility of confined concrete having Rectangular transverse reinforcements lead to various investigations by some Engineers. Those who excelled in such investigations

included:- CHAN, W.L.; BLUME, J.A.; PARK, R.; PAULAY, T.; YU, C.W.; and SOLIMAN, M.T.M.; others who also shine in the investigations are:- ROY, H.E.H.; HANDA, V.K.; SOZEN, M.A.; BAKER, A.L.L; SARGIN, M.; GHOSH, S.K.; AMAKAKONE, A.M.N.; KENT, D.C.; SCOTT, B.D.; and PARK, R. to mention a few. Below are some excerpts of their findings, along with their - under standings of the relationships between STRESS and STRAIN of confined concrete having Rectangular hoops as the transverse reinforcement. This is depicted in graphs as follows:-

- (1) ROY and SOZEN - These investigators suggested a curve which has a straight rising line inclined from the original point at an angle and having a unit stress f'_c corresponding to a strain of 0.002. The proposed curve also has a falling line of the curve as straight line. The proposed graph is as shetched (Fig..2.)

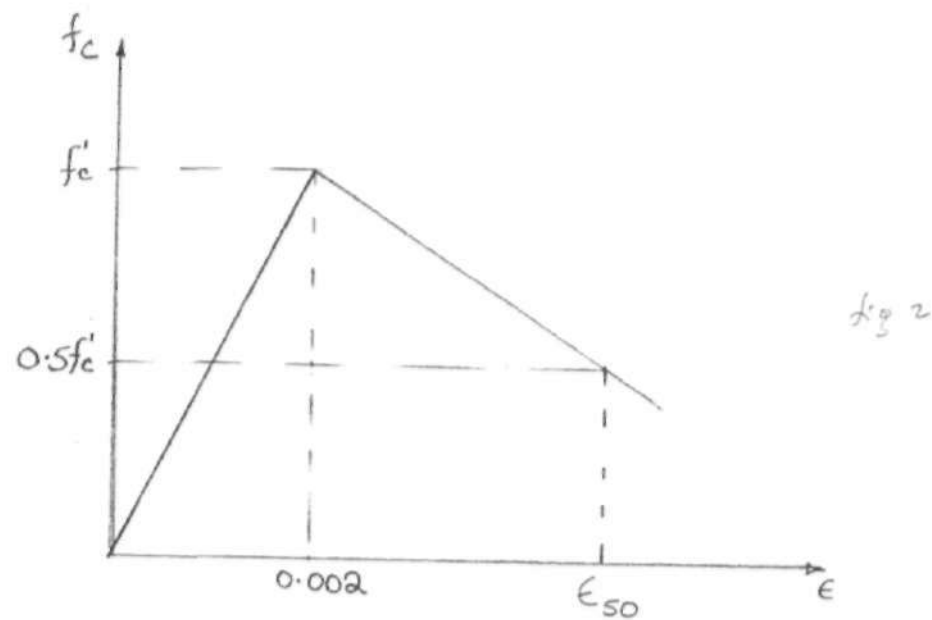


Fig. 0.2: ROY AND SOZEN CURVE

- (2) SARGIN, GHOSH AND HANDA - During the course of their investigations, the three investigators observed and - concluded that the relationship of STRESS-STRAIN should have a graph that is parabolic and continuous in nature. They also proposed that ultimate strength is achieved at the peak of the parabolic curve. Their proposal graph of STRESS-STRAIN relationship is as follows:

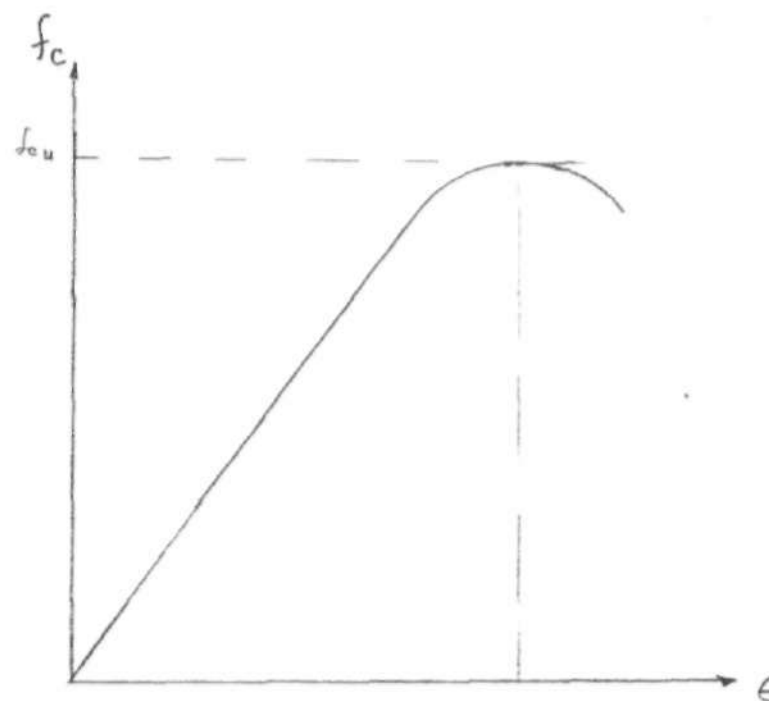


Fig. 0.3: SARGIN, GHOSH AND HANDA

- (3) SOLIMAN AND YU - The two investigators had a different finding with regard to the STRESS-STRAIN relationship. They proposed a curve, which is defined by three (3) distinct regions.

The first region which started from the original point has a parabolic shape up to a unit stress f_c' corresponding to strain ϵ_{ce} , while the second region is horizontal starting at ϵ_{ce} and ending at strain ϵ_{cs} . The final region of this graph has a first degree curve falling down to a stress of $0.85 f_c'$ and corresponding strain ϵ_{cf} at a point of failure. The graph is shown sketched in Fig. 4

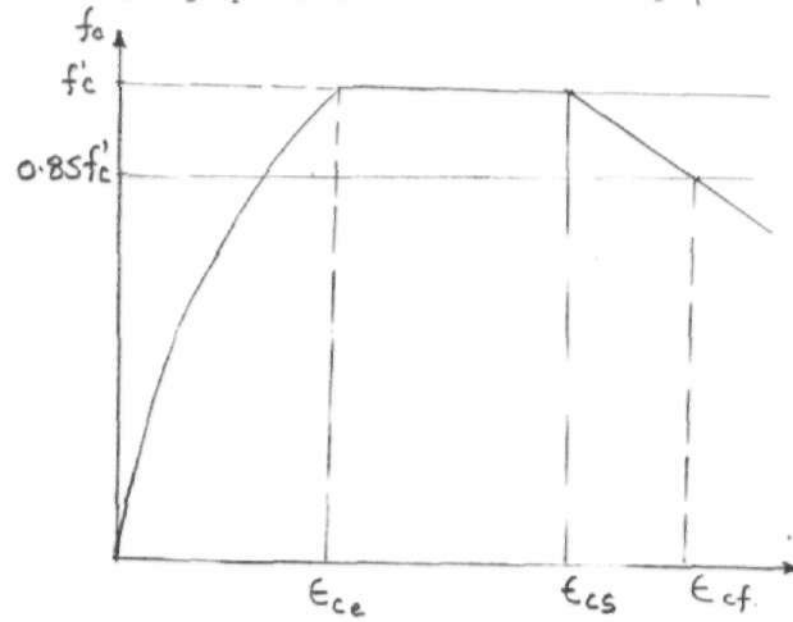


Fig. 0.4: SOLIMAN AND YU CURVE

$$\epsilon_{ce} = 2f_c'/E_c.$$

The investigators concluded that as for ϵ_{cs} and ϵ_{cf} on the graph, their mathematical relationship will depend on the binders spacings. They also gave the mathematical relations of the maximum compressive strain as:-

$$\epsilon_{cu} = 0.003 (1 + 0.8q'')$$

where the notation q'' is representing the effectiveness of the used transverse reinforcement. It's relations to other parameters is as

follows:-

$$q'' = (1.4 \frac{A_b}{A_c} - 0.45) \frac{A_s'' (S_o - S)}{A_s'' + 0.0028BS^2} \dots\dots\dots (4)$$

where: A_b = The area of the confined concrete under compression

A_c = The over-all area of the concrete under compression.

A_s'' = The area of the steel used as transverse reinforcement.

S_o = The ineffective maximum spacing (longitudinally) of transverse reinforcement.

S = The Actual maximum spacing (Longitudinally) of transverse reinforcement.

B = The breadth of the confined - Concrete (=b)

Note If the depth (d) of the confined concrete is larger from use it rather than the breadth for B.

(4) CHAN - This distinguished researcher had a different curve. Although his findings gave him a three region curve like that of Soliman and Yu, the orientation of the curve is unsimilar. Mr. Chan investigated only the variation of the Stirrup diameters and spacings in his Experiments. He also assumed an ultimate strain value of 0.0135 as the compressive strain of the most outer fibre. His idea of STRESS-STRAIN curve (confined or unconfined) of reinforced concrete structure is as follows.

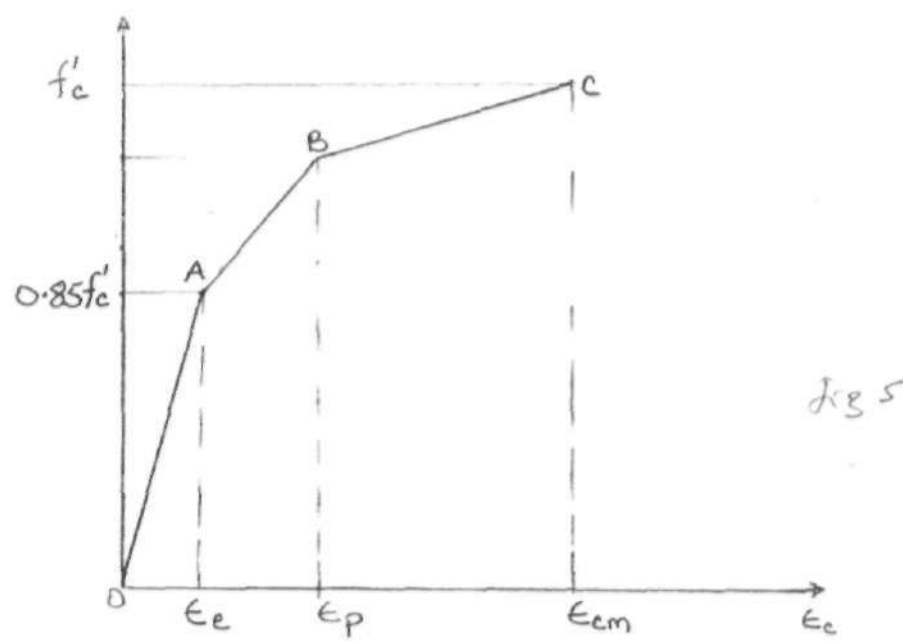


Fig. 0.5: CHAN STRESS - STRAIN CURVE

- (5) AMARAKONE AND BAKER - The combine effort of Baker and Amarakone came out with a different graph. They differ in their assumption with that of Chan when they assumed a maximum compression strain to have a value of 0.01. They backed up this assumption with a numerical equation that reads:

$$E_{cm} = 0.0015 \left[1 + 150P'' + (0.7 + 10P'') \frac{d}{Kd} \right] \leq 0.01 \quad \dots (5)$$

where P'' - Volumetric ratio of transverse reinforcement to that of bounded concrete.

d - Effective depth of the section.

Kd - Neutral axis depth.

The resultant graph came out to have a parabolic shape with a linear rise from the initial point up to the $0.85 f'_c$ point, where from there the graph continue to ascend with another - orientation, which is maintained within the region of $0.85 f'_c$ and f'_c . The graph is depicted as follows.

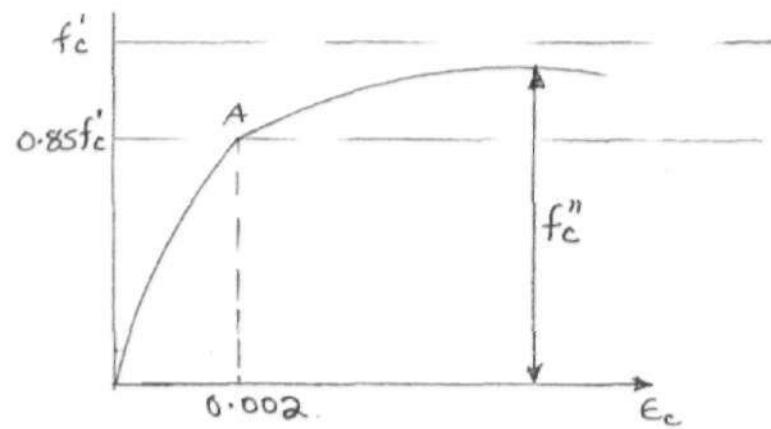


Fig 6.

Fig. 0.6: AMRAKONE AND BAKER

Other investigators who contributed into the research of STRESS-STRAIN curve for reinforced concrete (confined and unconfined) are Kent, Park and Scott to mention a few. Their contributions have been documented and used as a basis for further research in the topic.

In the course of investigations by these individuals, they all agreed that transverse reinforcements are the deriving force behind ductility of reinforced structural members. The degree of ductility differs on factors like the size of the stirrup (diameter), spacing of the stirrup, and the arrangement of the stirrup itself. Other factors includes the characteristic strength of the stirrup, the placement of the main bars, rate of loading on the - structural member, and finally the characteristics strength of the concrete itself.

The performance of a reinforced - concrete member under load depends, to a great extent, on the STRESS - STRAIN relationship of concrete and steel, and also on the type of stress applied to the member. The STRESS-STRAIN diagrams for concrete structures indicate three distinct ranges. These ranges are:-

- (a) The initial range, which is very nearly linear.
- (b) The intermediate range, in which there is increasing curvature, ultimately reaching a point of maximum stress, f_{cu} .

- (c) The final range, in which strain continues to increase while the load carrying capacity decreases.

2.4 LOAD AND MOMENT THEORY OF A COLUMN

Investigators for the effect of load on single rectangular hoop column which is eccentrically loaded theorised that, the force equilibrium equations is as follows.

Earlier investigators theorised that, the following equation represents the force moment Equilibrium equations of an eccentrically loaded column.

$$P = f_c b k d + \sum_{i=1}^n f_{si} A_{si}$$

$$\text{and } M = f_c b k d \left(\frac{h}{2} - k d \right) + \sum_{i=1}^n f_{si} d_i$$

where:- A_{si} - Represents the cross-sectional areas of each of the "n" number of bars in compression.

d_i - Represents the corresponding - distance from the centre of the bar to the centerline of the column cross-section.

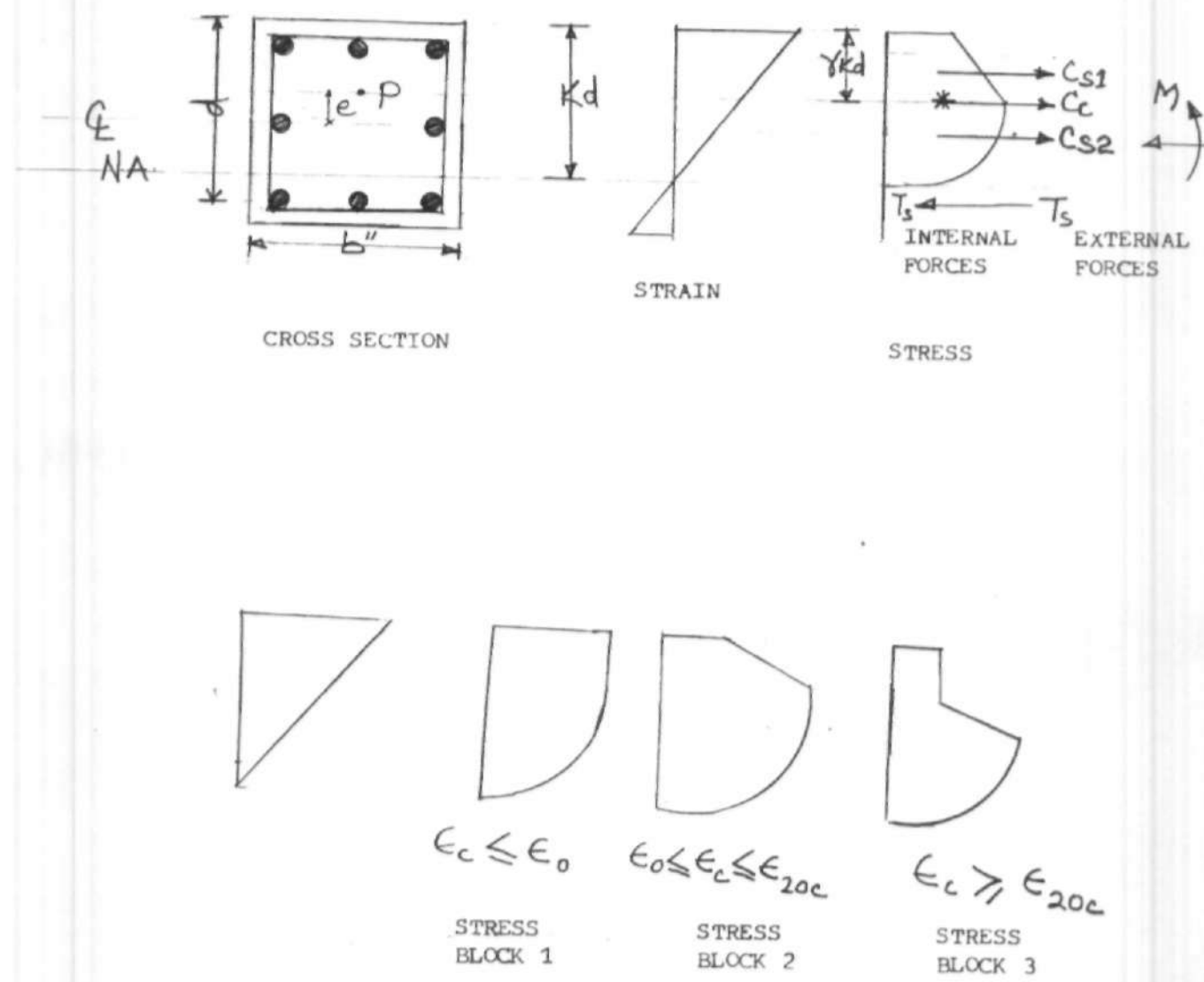


Fig. 0.7:

CHAPTER 3

EXPERIMENTATION

3.1 GENERAL

For the purpose of this study, twenty-four (24) Experimental model sizes (factor 2.5) of short column specimens (120x120x1200mm) were used. The specimens were prepared from three (3) different sets of overlapping hoops and One (1) set of single rectangular hoop, each with a total of Six (6) specimens per set.

Throughout this Experimental study, the hoops spacing, as well as the concrete Grade of all the specimens used were fixed. The model size of the spacing of the hoops was kept at 120mm (Real spacing 300mm), while the Grading of the concrete was fixed at Grade 25.

Casting and Testing of the Test-specimens was divided into two batches. Each of the two batches has two of the four (4) SETS of the Specimens studied. The first batch of the specimens grouped together arrangements numbers One (1), and Three (3), with a total of twelve (12) specimens, while the second batch consisted of the second and the fourth arrangements of the specimens with same equal number of twelve (12). Different dates but same procedures were used to cast and Test the two batches of the specimens. Another point of difference/similarities was on the use of STRAIN-GAUGES to the batches. Arrangements number One (1) and number three (3) (Batch One) used Type 5 - 11 with gauge - factor of 2.0×10^{-6} , while Batch two with arrangements number two (2) and number four (4) used Type 10 - 11 with gauge factor of 2.06×10^{-6} .

Identification marks were attached to each of the specimens during their preparations for the test study. These identification marks were attached to the specimens in order to help correctly identify all the specimens during testing for true representation of their individual behaviours recorded under loadings. During identification process, three (3) subscript numbers and letters were used in order to identify arrangements, specimen numbers and strain-gauge positions respectively. Thus when a tag mark reads C35b, it means 'C' - the column initia; '3'- arrangement/set number 3 (Double crossing); '5' - specimen number 5(out of six); and finally 'b' refers to the back of the top end of the specimen (see Appendix for more details).

3.2 MATERIALS

3.2.1 CONCRETE MIX

The concrete mix used for this research work was designed to have a characteristic strength of 25 MN/m^2 after 28 days. The mix ratios - are $1;1\frac{1}{2};3$ with water-cement ratio of 0.7 ($W/C=0.7$). This designed - strength was maintained in all the specimens prepared for this study. The Batching of the concrete materials was by weight, and the mixing of the concrete was manual but thoroughly done. The mixing was done firstly dry and later wet. The compaction of the concrete mix in the form-work was by rodding, due to the size of the ~~size of the~~ specimen (120x120x1200mm) and then poker vibrator was later applied on the surface and at the top of the form work.

PREPARATION OF THE IRON CASE

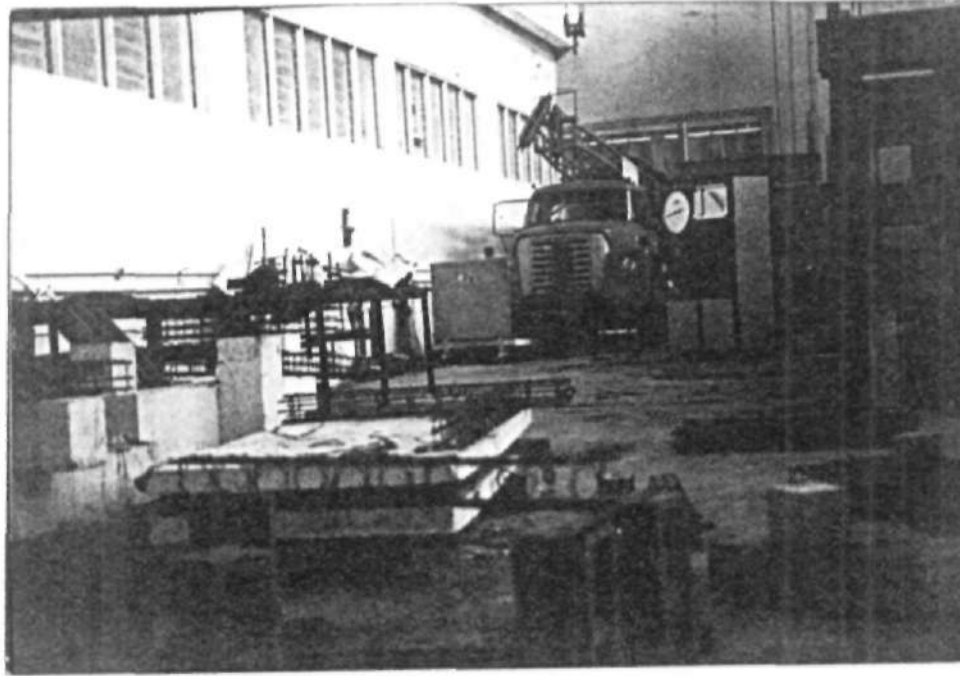


PLATE 01: IRON BENDING

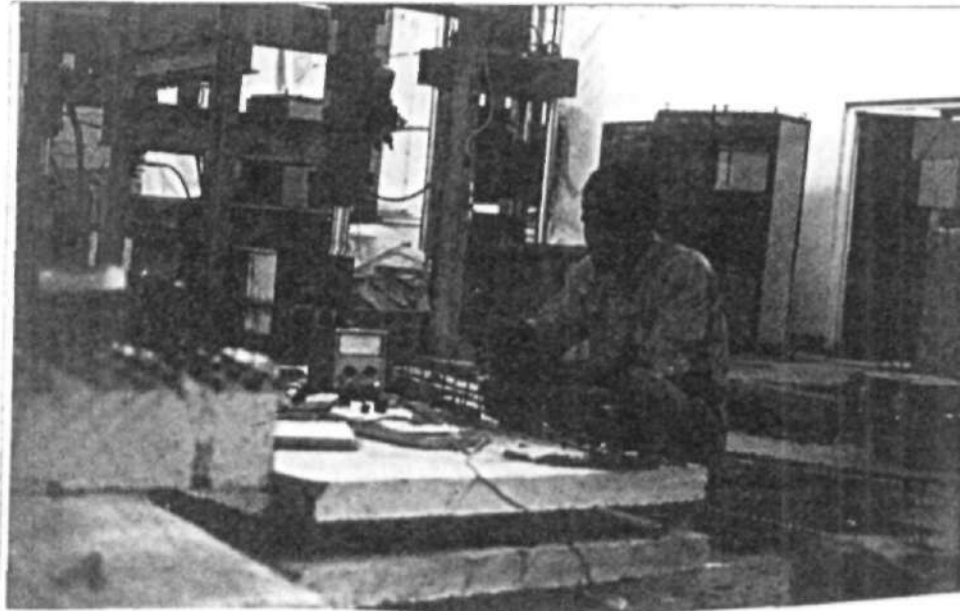


PLATE 02: SOLDERING OF STRAIN GAUGES AT SELECTED POINTS OF THE MAIN BARS.

PREPARATION OF THE FORM WORK BEFORE CASTING OF THE COLUMNS



PLATE 3: ARRANGING OF THE FORM WORK IN READINESS FOR THE CASTING



PLATE 4: PLUMBING OF THE FORM WORK TO MAKE SURE THE COLUMNS ARE CASTED VERTICALLY.

3.2.1.1 C E M E N T

The cement used in preparing the designed concrete mix of this study was ASHAKA Portland Cement.

3.2.1.2 A G G R E G A T E S

Multiple graded crushed stones were used as Aggregates. The maximum size of the used aggregate in this experiment was 10mm, passing through - B.S. Sieve size 12.7 (see table).

3.2.1.3 S A N D

River-bed sand was used in this experiment. The maximum granular size of the sand was passing through B.S. Sieve 3.14 with some percentage retained on B.S. Sieve Size 2.40.

3.2.1.4 W A T E R

Clean water from the tap collected in buckets was used.

3.2.2 S T E E L

Mild steel was used in both the main bars and the stirrups. The main bars used were of 6mm diameters, while the stirrups were of 4mm diameter. The average strength is about 250 MN/m^2 .

3.2.3 F O R M W O R K

The material used for form-works of the specimens was ply-wood. Unused and clean ply-wood was used in order to achieve a very smooth surface.



PLATE 5: MANUAL (HAND) MIX OF THE CONCRETE USED FOR THE CASTING



PLATE 6: THE RODDING METHOD OF - COMPACTION USED DURING THE CASTING

3.2.4 WHITE-WASH/LIME

After 28 days of curing the specimens, white-wash and lime mix was used to laminate the specimens in preparation for tests.

3.3 INSTRUMENTATIONS

The instruments used in this investigations were the STRAIN-GAUGES and STRAIN-METERS. Four Strain-gauges were used on each of the twenty-four (24) specimens prepared for this study. The STRAIN-GAUGES were attached to the main bars of the prepared reinforcement cages at four different positions. The positions are Top back, Top front, middle front, and middle back. They were attached by soldering them at these mentioned positions. Flexible electronic gauge cable was used in stress-strain readings of the positions or points of the specimens - when loaded. The reading was carried out with the help of a strain-meter through these cable connections.

3.4 SPECIMENS

An orchestrated specimens preparation was carried out in order to get a reliable results representations of the study.

A total of twenty-four (24) specimens were prepared and tested during the course of investigations. Reinforcement cages of 8-6mm diameter main bars and 4mm diameter singular rectangular and overlapping hoops arrangements were prepared.

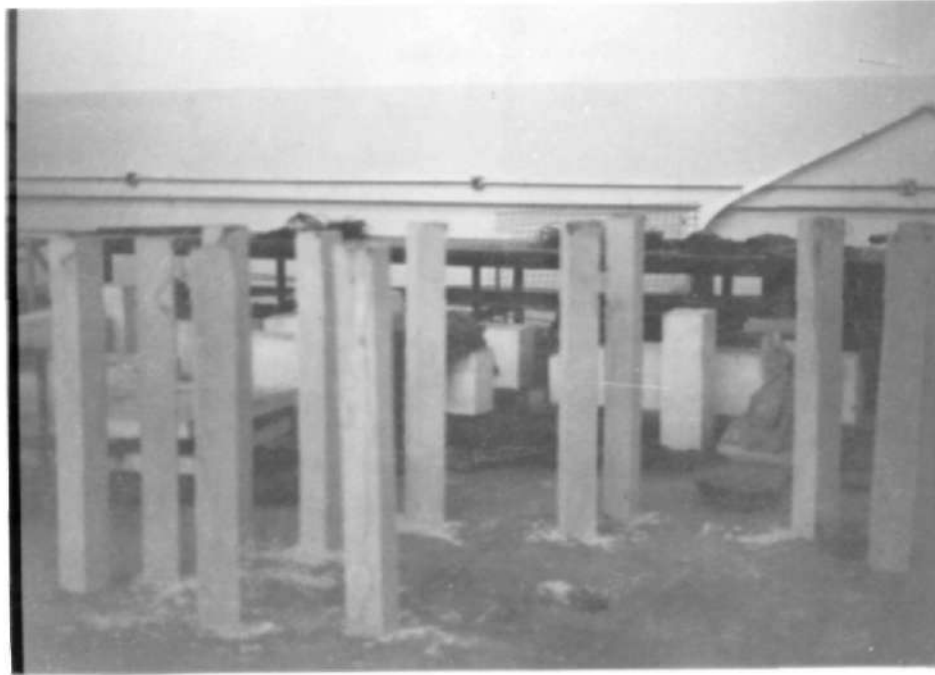


PLATE 7: CURED AND LAMINATED SPECIMENS FOR TESTING (SEE TRAIN-GAUGE CABLE ATTACHED ON THE SPECIMEN).



PLATE 8: SPECIMENS ARRANGED IN ORDER (NEAR U.T.M.) READY FOR TESTING

The sizes of each finished specimen was 120x120x1200mm. It was designed to have a cover of 15mm each on sides, while top and bottom ends had 30mm each as a cover. Strain gauges were soldered to the main bars at four different positions. The positions were near top at both the front of the specimens and the back of the specimens, while the remaining positions were at both mid points of back and front positions of the specimens. A form - work box of plywood of each of the twelve (12) specimens of the two batches were also prepared. A designed concrete mix of Grade 25 with crushed stone aggregates of maximum size of 10mm was prepared and used in the cast. The form-works prepared with cages inside were lined against an upright structure and plumbed vertically. The designed mix was (vertically) casted in to the readied reinforced form-work and then compacted by rodding and poker vibrator. In order to be sure of having proper W/C ratio in the concrete mix, slump test of the concrete was carried out during casting. Concrete cubes specimens were also prepared for testing. The casted specimens were then allowed in the form-work for twenty four (24) hours, and then later removed for curing. The specimen columns were cured through watering in the mornings and evenings and covered with both sack and polythene for 28 days, while the specimen cubes were cured by immersion in a trough for same period (28 days). At the expiration of twenty-eight (28) days, the columns were removed from their stack curing positions and painted with a mix of white-wash and lime. After the painting, the specimens were allowed to dry for a couple of hours.

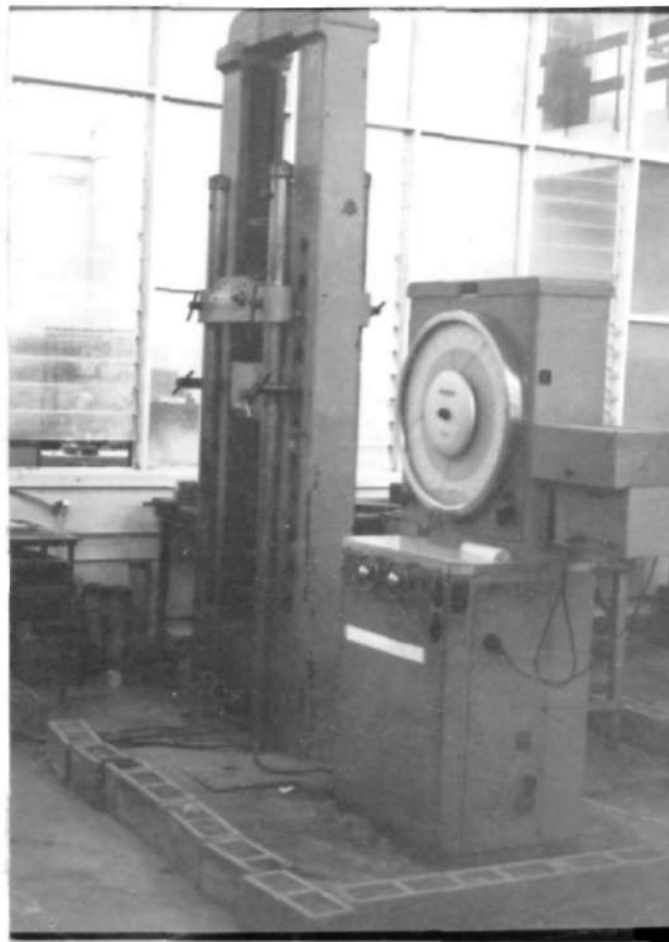


PLATE 9: THE UNIVERSAL TESTING - MACHINE USED FOR MEASURING THE APPLIED LOAD ON THE SPECIMENS.

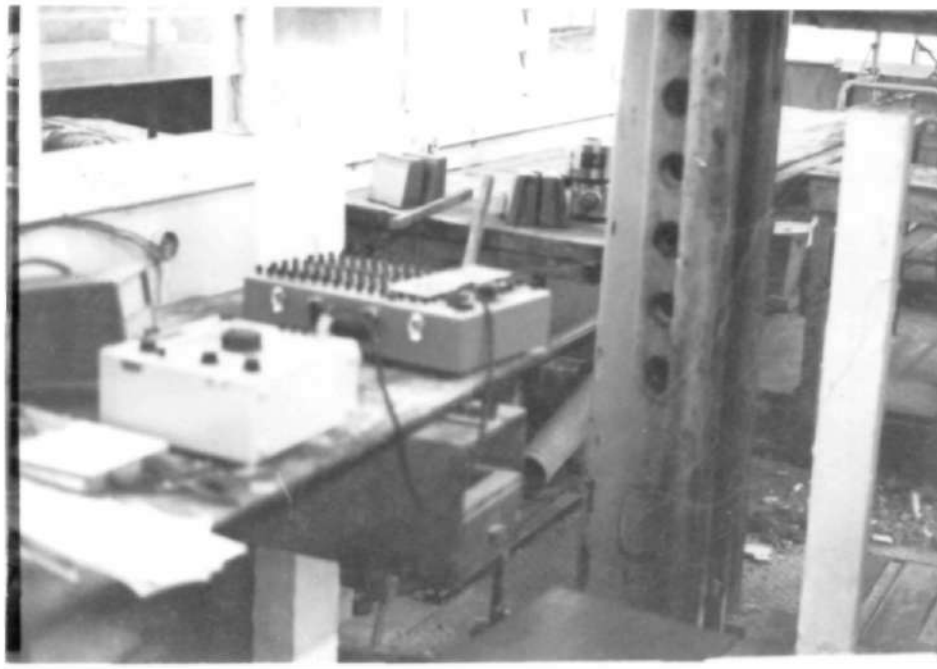


PLATE 10: THE STRAIN-METER DEVICE USED FOR MEASURING THE INTERNAL STRAINS OF THE SPECIMENS.

Testing of prepared columns was carried out with the help of UNIVERSAL-TESTING MACHINE? The specimens were placed centrally on the machine before loading in order to avoid eccentricity on the specimens. Cables of Strain-gauges were then identified through their markings and connected to the Strain meter accordingly. Channel I connection of the Strain meter was connected to the Top-front, while middle-front, Top-Back and Middle Back were connected to channels two (2), three (3) and four (4) of the strain meter respectively. A dummy specimen reading was recorded along with the initial unloaded readings of the four channels of the Test-specimens. When the initial reading were recorded, the specimen were then loaded with the first set of loadings. Readings of the behaviour of the specimen at these four identified positions were noted and recorded. The process continued with Gradual increase of loads at intervals and readings taken untill failure/collapse is achieved. The readings of the internal stress-strains of the specimens were tabulated as shown in the Appendix.



PLATE 11: PLACEMENT OF THE SPECIMEN ON MARKED CENTRE OF THE MACHINE TO AVOID ECCENTRICITY



PLATE 12: HOOKING OF THE CABLES TO THE STRAIN-METER BEFORE APPLYING LOAD TO THE SPECIMEN



PLATE 13: FIRST STAGE OF LOADING THE SPECIMEN DURING TEST



PLATE 14: TYPICAL FAILURE OF THE SPECIMEN ON TEST DURING LOADING

SPECIMENS ON AND AFTER TESTING

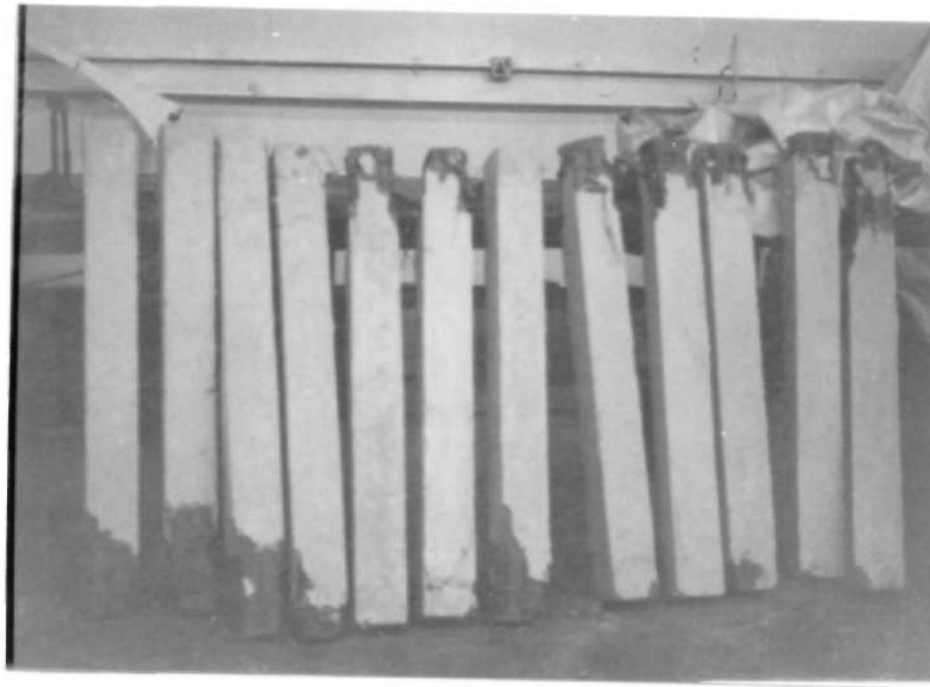


PLATE 15: FIRST BATCH OF THE TESTED SPECIMENS SHOWING FAILURE POINTS OF TOP OR BOTTOM OF THE SPECIMEN



PLATE 16: SECOND BATCH OF THE TESTED SPECIMENS

CHAPTER 4

TEST-RESULTS

4.1 GENERAL

The Experimental procedures applied on this investigative research were highly scientific, care was taken at all the stages of the experiment in order to come out with results and findings that can truly represent the actual conditions of the investigative area of concern.

Recording of the results were both by mechanical and Electrical Instruments. Some of the Instruments used are weight-scales, steel rule and plumb-bob. Others are Universal testing machine for crushing, strain-Gauges for sensitive measurements and strain-Gauge meter for reading recording of strains.

The use of strain gauges came about when it was evident that internal stresses/strains can best and accurately be measured only with a sensitive instrument such as strain-Gauge, it and when placed at the point of action. This idea was - proved to be the right one since most of the readings came out as expected, except only where the strain-Gauge itself did not function due probably to some damages during casting.

Before crushing the test columns, the center of the concrete-core was determined by drawing two lines (at the top and bottom faces) each joining opposing corners. Where the lines intersect is the center of concrete core. This center is also used as a guide for proper positioning on the testing machine.

4.2 OBSERVATION/RECORDING OF RESULTS

4.2.1 MIX - DESIGN

Volume of Concrete required:-

$$\text{One column } 0.12 \times 0.12 \times 1.2 \text{ m} = 0.01728 \text{ m}^3$$

$$\text{For 24 columns} = 0.01728 \times 12 = 0.41472 \text{ m}^3$$

$$25\% \text{ waste and shrinkage} = \frac{0.10368 \text{ m}^3}{}$$

$$\text{Total Volume} = \frac{0.5184 \text{ m}^3}{\text{=====}}$$

$$\text{i) Gross volume of concrete} = 0.5184 \text{ m}^3$$

Gross weight of concrete

$$= 0.5184 \times 2400$$

$$= \frac{1244.16 \text{ kGs.}}{\text{=====}}$$

GRADE 25 - RATIO 1:1½:3

$$\frac{w}{c} = 0.7$$

$$\text{Total ratios} = 6.2$$

$$\text{One part} = \frac{1244.16 \text{ KGs}}{6.2} = 200.671 \text{ KGs}$$

MATERIALS BY PARTS

$$\therefore \text{One(1) part of cement} = 200.671 \text{ KGs.}$$

$$\text{One and a half (1½) parts of sand} = 301.006 \text{ KGs.}$$

$$\text{Three (3) parts of Aggregate} = 602.013 \text{ KGs}$$

$$0.7 \text{ parts of water} = \frac{140.470 \text{ KGs}}{\text{=====}}$$

$$\text{GRAND TOTAL} = \frac{1244.160 \text{ kgs}}{\text{=====}}$$

4.2.2 SIEVE ANALYSIS OF MATERIALS4.2.2.1 A G G R E G A T E

TABLE 4.1

SIEVE ANALYSIS OF MATERIAL

Job GRADING 1 (Aggregate) Operator ALH, SANUSI YAU
 site COMMON Date 23RD JAN, 1991
 Sample No ONE (1) Description RESEARCH WORK
 Total weight of dry sample 5000 grams

B.S. Sieve size	Weight Retained	Weight Passing	Per cent retained	Total passing	Remarks	Max Sieve load
	g	g		%		g
76.3						
63.5						
50.8						4500
38.1						3500
25.4						2500
19.05						2000
12.7	0	5000	0%	100%		1500
9.51	610	4390	12.2%	87.8%		1000
6.35	2235	2155	44.7%	43.1%		750
4.76	788	1367	15.76%	27.34%		500
2.4.	1112	264	22.24%	5.10%		
Riffled sample passing 4.76						
3.14						300
2.40						200
1.20						100
600						75
434						75
300						50
208						50

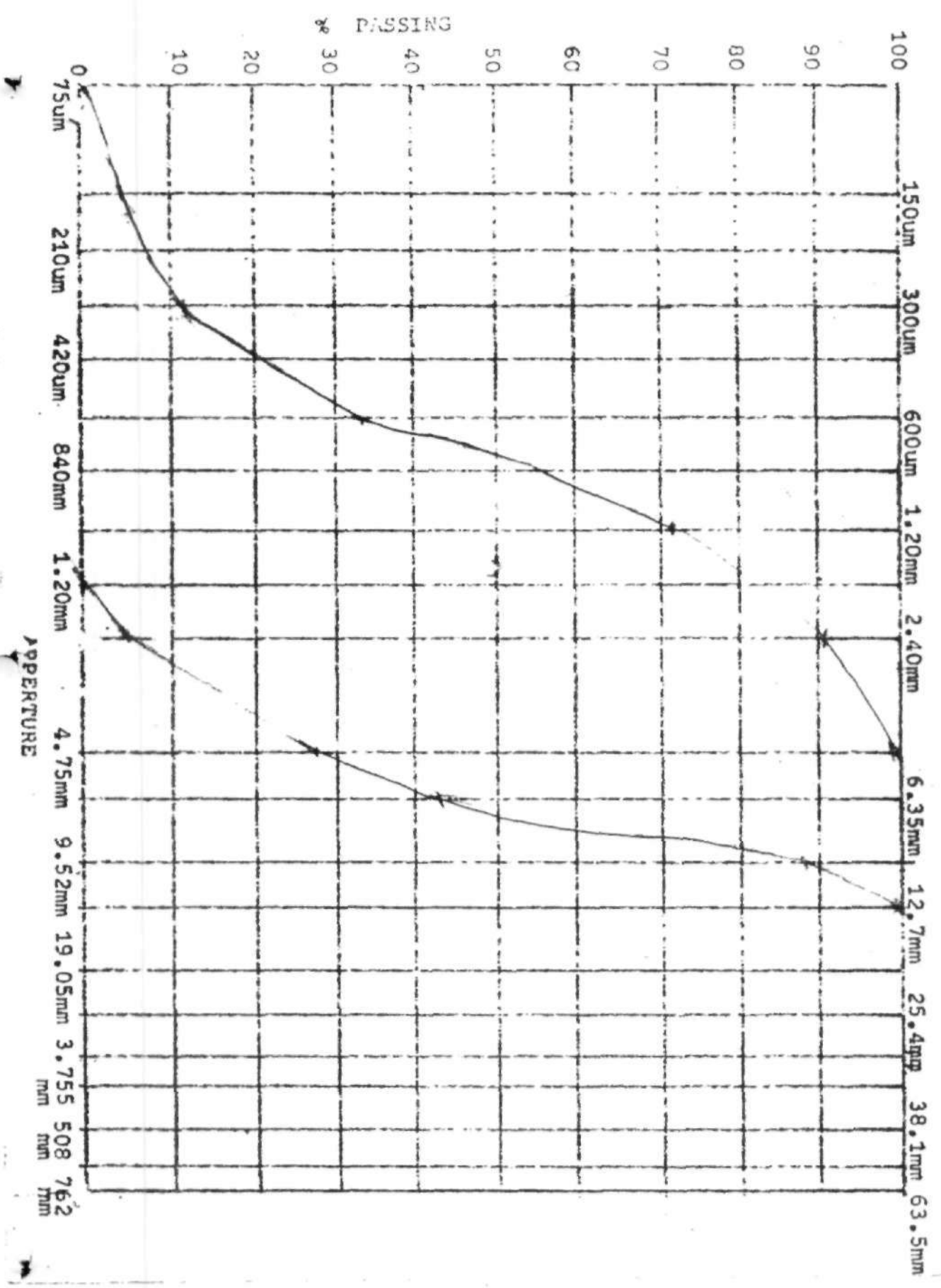
SIEVE ANALYSIS OF MATERIALS

Job GRADING 2 (SAND) Operator ALH. SANUSI YAU
 site COMMON Date 24TH JAN. 1991
 Sample No TWO (2) Description RESEARCH WORK
 Total weight of dry sample 5000 grams

B.S. Sieve size	Weight Retained	Weight Passing	Per cent retained	Total passing	Remarks	Max Sieve load
	g	g	%			g
76.3						
63.5						
50.8						4500
38.1						3500
25.4						2500
19.05						2000
12.7						1500
9.51						1000
6.35						750
4.76						500
Passing 4.76						
Riffled sample passing 5.0kg 4.76						
3.14	0	5000				300
2.40	690	4310		90.83		200
1.20	937	3373		71.09		100
600	1772	1610		33.74		75
434						75
300	1066	535		11.27		50
208						50
150	344	191		4.03		40
75						25
Passing 75	186					

4.2.2.3. Aggregate / Sand Grading Graphs.

GRADING OF AGGREGATE



4.2.3 COMPACTION TEST

Weight of container = 7.7kg.

Partially compact weight of container and concrete = 20.0kgs.

Fully compacted weight of container and concrete = 20.45kgs.

i) Net weight of concrete = (20.0 - 7.7)kg.

$$= \underline{12.30kgs}$$

Length of container = 300mm

Diameter of container = 150mm

$$\text{Area of container} = \frac{(150)^2}{4} = \underline{17671.459 \text{ mm}^2}$$

$$\text{Volume of container} = 17671.459 \text{ mm}^2 \times 300 \text{ mm}.$$

$$= 5301437.6 \text{ mm}^3$$

$$= \underline{5.3014 \times 10^{-3} \text{ m}^3}$$

Density of the concrete: $\frac{\text{Mass}}{\text{Volume}}$

$$= \frac{12.3000 \text{ kgs}}{5.3014 \times 10^{-3} \text{ m}^3}$$

$$= \underline{2320.1254 \text{ kgs/m}^3}$$

4.2.4 SLUMP TEST

$$\text{Slump} = 12.5 \text{ cm} = \underline{0.125 \text{ m}}$$

4.2.5 CHARACTERISTIC STRENGTH OF CONCRETE CUBES

BATCH ONE - DESIGN STRENGTH = 25 MN/M²

TABLE 4.3

CUBE	WEIGHT, kg	CRUSHING LOAD KN	STRENGTH MN/M ²
B 11	7.90	480	21.33
B 12	8.05	500	22.22
BB13	7.75	450	20.00
B 21	7.80	520	23.11
B 22	7.85	440	19.56
B 23	7.60	460	20.44
AVERAGES	7.825	475	21.11

BATCH TWO - DESIGN STRENGTH = 25 MN/M²

CUBE	WEIGHT, kg	CRUSHING LOAD KN	STRENGTH MN/M ²
B 31	7.70	450	20.00
B 32	7.58	510	22.67
B 33	7.55	480	21.33
B 34	7.60	500	22.22
B 35	7.85	460	20.44
B 36	7.80	530	23.56
AVERAGE	7.68	488	21.70

TABLE 4.4

SEMPLE NO	DIAMETER MM	AREA OF STEEL MM ²	YIELD LOAD KN	FAILURE LOAD KN	YIELD STRENGTH (f _y) MN/M ²
61	6 mm	28.2743	7.2000	9.1000	254.6482
62	6 mm	28.2743	7.2000	9.2000	254.6482
63	6 mm	28.2743	7.4000	9.000	261.7217
AVERAGE	6 mm	28.2743	7.2667	9.1000	257.0060
=====					

SAMPLE NO	DIAMETER MM	AREA OF STEEL MM ²	YIELD LOAD KN	FAILURE LOAD KN	YIELD STRENGTH (f _y) MN/M ²
41	4 mm	12.5664	10.3000	11.4000	819.6460
42	4 mm	12.5664	10.2000	10.8000	811.6883
43	4 mm	12.5664	10.3000	10.5000	819.6460
AVERAGE	4 mm	12.5664	10.2667	10.9000	816.9934
=====					

COMPUTATION ANALYSIS

$$\frac{f_{ci}}{0.8f_{cu}} = 1 - \frac{ZE_c}{0.8f_{cu}} + 0.002Z$$

$$ZE_c = 1 - \frac{f_{ci}}{0.8f_{cu}} + 0.002Z$$

$$E_c = \frac{1 - \frac{f_{ci}}{0.8f_{cu}} + 0.002}{Z}$$

$$\text{STRENGTH} = 21.7 \text{ N/mm}^2$$

$$f_{cu} = \underline{3149.5 \text{ psi}}$$

EQUIVALENT CYL STRENGTH

$$f_c^1 = 0.8 f_{cu}$$

$$f_c^1 = 0.8(3149.5) = 2520 \text{ psi } (17.36 \text{ N/mm}^2)$$

$$C_{11} \quad z = 16.3$$

FOR 20%

$$0.2 f_c^1 = 0.2(2520) \text{ psi} = 504 \text{ psi}$$

$$E_{c20} = \frac{1 - \frac{504}{2520} + 0.002}{16.3}$$

$$= \frac{0.0510793}{16.3}$$

$$= \underline{0.05108}$$

FOR 40%

$$0.4 f_c^1 = 0.4(2520) \text{ psi} = \underline{1008 \text{ psi}}$$

$$E_{c40} = \frac{1 - \frac{1008}{2520} + 0.002}{16.3}$$

$$= \underline{0.03881}$$

$$\begin{aligned}
 &\text{FOR 50\%} \\
 &0.5f_c^1 = 0.5 (2520) \text{ psi} \\
 &= \underline{1260 \text{ psi}} \\
 E_{c50} &= 1 - \frac{1260}{2520} + 0.002 \\
 &\quad \underline{16.3} \quad \underline{0.032675}
 \end{aligned}$$

$$\begin{aligned}
 &\text{FOR 80\%} \\
 &0.8f_c^1 = 0.8(2520) \\
 &= \underline{2016} \\
 E_{c80} &= 1 - \frac{2016}{2520} + 0.002 \\
 &\quad \underline{16.3} \\
 &= \underline{0.01427}
 \end{aligned}$$

$$\boxed{C_{12} \quad Z = 51.6}$$

$$\begin{aligned}
 E_{c20} &= \frac{1-8.2}{51.6} + 0.002 \\
 &= 0.0175038 \\
 E_{c20} &\quad \underline{0.01750}
 \end{aligned}$$

$$\begin{aligned}
 E_{c40} &= \frac{1-0.4}{51.6} + 0.002 \\
 E_{c40} &= \underline{0.01363}
 \end{aligned}$$

$$E_{c50} = \frac{1-0.5}{51.6} + 0.002 = \underline{0.01169}$$

$$E_{c80} = \frac{1-0.8}{51.6} + 0.002 = \underline{0.00588}$$

$$\boxed{C_{13} \quad Z = 29.3}$$

$$E_{c20} = \frac{1-0.2}{29.3} + 0.002 = \underline{0.02930}$$

$$E_{c40} = \frac{1-0.4}{29.3} + 0.002 = \underline{0.02248}$$

$$E_{c50} = \frac{1-0.5}{29.3} + 0.002 = \underline{0.01906}$$

$$E_{c80} = \frac{1-0.8}{29.3} + 0.002 = \underline{0.00883}$$

$$\boxed{C_{14} \quad Z = 29.3} \text{ (As } C_{13} \text{ above)}$$

$$E_{c20} = \underline{0.02930}$$

$$E_{c40} = \underline{0.02248}$$

$$E_{c50} = \underline{0.01906}$$

$$E_{c80} = \underline{0.00883}$$

$$\boxed{C_{15} \quad Z = 16.3} \text{ (As } C_{11})$$

$$E_{c20} = 0.05108$$

$$E_{c40} = 0.03881$$

$$E_{c50} = 0.032675$$

$$E_{c80} = 0.01427.$$

$$\boxed{C_{16} \quad Z = 45.00}$$

$$E_{c20} = \frac{1-0.2}{45} + 0.002 = \underline{0.01978}$$

$$E_{c40} = \frac{1-0.4}{45} + 0.002 = \underline{0.01533}$$

$$E_{c50} = \frac{1-0.5}{45} + 0.002 = \underline{0.01311}$$

$$E_{c80} = \frac{1-0.8}{45} + 0.002 = \underline{0.00644}$$

$$C_{21} \quad Z = 14.7$$

$$E_{c20} = \frac{1-0.2}{14.7} + 0.002 = \underline{0.05642}$$

$$E_{c40} = \frac{1-0.4}{14.7} + 0.002 = \underline{0.04282}$$

$$E_{c50} = \frac{1-0.5}{14.7} + 0.002 = \underline{0.03601}$$

$$E_{c80} = \frac{1-0.8}{14.7} + 0.002 = \underline{0.01560}$$

$$C_{22} \quad Z = 24.5$$

$$E_{c20} = \frac{1-0.2}{24.5} + 0.002 = \underline{0.03465}$$

$$E_{c40} = \frac{1-0.4}{24.5} + 0.002 = \underline{0.02649}$$

$$E_{c50} = \frac{1-0.5}{24.5} + 0.002 = \underline{0.02241}$$

$$E_{c80} = \frac{1-0.8}{24.5} + 0.002 = \underline{0.01016}$$

$$C_{23} \quad Z = 32.6$$

$$E_{c20} = \frac{1-0.2}{32.6} + 0.002 = \underline{0.02654}$$

$$E_{c40} = \frac{1-0.4}{32.6} + 0.002 = \underline{0.02041}$$

$$E_{c50} = \frac{1-0.5}{32.6} + 0.002 = \underline{0.01734}$$

$$E_{c80} = \frac{1-0.8}{32.6} + 0.002 = \underline{0.008135}$$

$$C_{24} \quad Z = 24.5$$

$$E_{c20} = \frac{1-0.2}{24.5} + 0.002 = \underline{0.03465}$$

$$E_{c40} = \frac{1-0.4}{24.5} + 0.002 = \underline{0.02649}$$

$$E_{c50} = \frac{1-0.5}{24.5} + 0.002 = \underline{0.02241}$$

$$E_{c80} = \frac{1-0.8}{24.5} + 0.002 = \underline{0.01016}$$

$$C_{25} \quad (\text{NO RESULT})$$

$$C_{26} \quad Z = 37.4$$

$$E_{c20} = \frac{1-0.2}{37.4} + 0.002 = \underline{0.02339}$$

$$E_{c40} = \frac{1-0.4}{37.4} + 0.002 = \underline{0.01804}$$

$$E_{c50} = \frac{1-0.5}{37.4} + 0.002 = \underline{0.01537}$$

$$E_{c80} = \frac{1-0.8}{37.4} + 0.002 = \underline{0.00735}$$

$$C_{31} \quad Z = 17.0$$

$$E_{c20} = \frac{1-0.2}{17} + 0.002 = \underline{0.04906}$$

$$E_{c40} = \frac{1-0.4}{17} + 0.002 = \underline{0.0329}$$

$$E_{c50} = \frac{1-0.5}{17} + 0.002 = \underline{0.03141}$$

$$E_{c80} = \frac{1-0.8}{17} + 0.002 = \underline{0.01376}$$

$$C_{32} \quad Z = 22.2$$

$$E_{c20} = \frac{1-0.2}{22.2} + 0.002 = \underline{0.03804}$$

$$E_{c40} = \frac{1-0.4}{22.2} + 0.002 = \underline{0.022903}$$

$$E_{c50} = \frac{1-0.5}{22.2} + 0.002 = \underline{0.02452}$$

$$E_{c80} = \frac{1-0.8}{22.2} + 0.002 = \underline{0.01101}$$

$$C_{33} \quad Z = 26.2$$

$$E_{c20} = \frac{1-0.2}{26.2} + 0.002 = \underline{0.03253}$$

$$E_{c40} = \frac{1-0.4}{26.2} + 0.002 = \underline{0.02490}$$

$$E_{c50} = \frac{1-0.5}{26.2} + 0.002 = \underline{0.02108}$$

$$E_{c80} = \frac{1-0.8}{26.2} + 0.002 = \underline{0.00963}$$

$$C_{34} \quad Z = 28.9$$

$$E_{c20} = \frac{1-0.2}{28.9} + 0.002 = \underline{0.02968}$$

$$E_{c40} = \frac{1-0.4}{28.9} + 0.002 = \underline{0.02276}$$

$$E_{c50} = \frac{1-0.5}{28.9} + 0.002 = \underline{0.01930}$$

$$E_{c80} = \frac{1-0.8}{28.9} + 0.002 = \underline{0.00892}$$

$$C_{35} \quad Z = 13.2$$

$$E_{c20} = \frac{1-0.2}{13.2} + 0.002 = \underline{0.06261}$$

$$E_{c40} = \frac{1-0.4}{13.2} + 0.002 = \underline{0.04745}$$

$$E_{c50} = \frac{1-0.5}{13.2} + 0.002 = \underline{0.03988}$$

$$E_{c80} = \frac{1-0.8}{13.2} + 0.002 = \underline{0.01715}$$

$$C_{36} \quad Z = 25.7$$

$$E_{c20} = \frac{1-0.2}{25.7} + 0.002 = \underline{0.03313}$$

$$E_{c40} = \frac{1-0.4}{25.7} + 0.002 = \underline{0.02535}$$

$$E_{c50} = \frac{1-0.5}{25.7} + 0.002 = \underline{0.02145}$$

$$E_{c80} = \frac{1-0.8}{25.7} + 0.002 = \underline{0.00978}$$

$$C_{41} \quad Z = 34.4$$

$$E_{c20} = \frac{1-0.2}{34.4} + 0.002 = \underline{0.02526}$$

$$E_{c40} = \frac{1-0.4}{34.4} + 0.002 = \underline{0.01944}$$

$$E_{c50} = \frac{1-0.5}{34.4} + 0.002 = \underline{0.01653}$$

$$E_{c80} = \frac{1-0.8}{34.4} + 0.002 = \underline{0.00781}$$

$$C_{42} \quad Z = 22.5$$

$$E_{c20} = \frac{1-0.2}{22.5} + 0.002 = \underline{0.03756}$$

$$E_{c40} = \frac{1-0.4}{22.5} + 0.002 = \underline{0.02867}$$

$$E_{c50} = \frac{1-0.5}{22.5} + 0.002 = \underline{0.02422}$$

$$E_{c80} = \frac{1-0.8}{22.5} + 0.002 = \underline{0.01089}$$

$$C_{43} \quad Z = 35.4$$

$$E_{c20} = \frac{1-0.2}{35.4} + 0.002 = \underline{0.02460}$$

$$E_{c40} = \frac{1-0.4}{35.4} + 0.002 = \underline{0.01895}$$

$$E_{c50} = \frac{1-0.5}{35.4} + 0.002 = \underline{0.01612}$$

$$E_{c80} = \frac{1-0.8}{35.4} + 0.002 = \underline{0.00765}$$

$$C_{44} \quad Z = 22.4$$

$$E_{c20} = \frac{1-0.2}{22.4} + 0.002 = \underline{0.03771}$$

$$E_{c40} = \frac{1-0.4}{22.4} + 0.002 = \underline{0.02879}$$

$$E_{c50} = \frac{1-0.5}{22.4} + 0.002 = \underline{0.02432}$$

$$E_{c80} = \frac{1-0.8}{22.4} + 0.002 = \underline{0.01093}$$

C₄₅ NO RESULT

C₄₆ Z = 31.6

$$E_{c20} = \frac{1-0.2}{31.6} + 0.002 = \underline{0.02732}$$

$$E_{c40} = \frac{1-0.4}{31.6} + 0.002 = \underline{0.02099}$$

$$E_{c50} = \frac{1-0.5}{31.6} + 0.002 = \underline{0.01782}$$

$$E_{c80} = \frac{1-0.8}{31.6} + 0.002 = \underline{0.00833}$$

TABLE 4.5

COLUMN	% LEVEL STRESS	CORRESPONDING STRESSES ($\times 10^6$)	CORRESPONDING STRAIN VALUE	ANGLE OF SLOPE OF THE FAILING BRANCH	AVERAGE VALUE OF ANGLE
C ₁₁	20	1.63982	0.0510983	0.05108	
	40	3.27964	0.039332	0.03881	
	50	4.09955	0.0326864	0.03268	
	80	6.55928	0.0142745	0.01427	
	100	8.20000	0.0136500	0.00200	
C ₁₂	20	2.72000	0.0175182	0.01750	
	40	5.44000	0.0136385	0.01363	
	50	6.80000	0.0116987	0.01169	
	80	10.88000	0.0058795	0.00588	
	100	13.60000	0.0046000	0.00200	37°
C ₁₃ & C ₁₄	20	1.92000	0.0293325	0.02930	
	40	3.84000	0.0224994	0.02248	
	50	4.80000	0.0190828	0.01906	
	80	7.68000	0.0088331	0.00883	
	100	9.60000	0.0078000	0.00200	
C ₁₅	20	1.66000	0.0510983	0.05108	
	40	3.37000	0.0391332	0.03881	
	50	4.15000	0.0326864	0.03268	
	80	6.64000	0.0142745	0.01427	
	100	8.30000	0.0136500	0.00200	
C ₁₆	20	2.40000	0.0197796	0.01978	
	40	4.80000	0.0153347	0.01533	
	50	6.00000	0.0131127	0.01311	
	80	9.60000	0.0064449	0.00644	
	100	12.00000	0.0048000	0.00200	

TABLE 4.6

COLUMN	% LEVEL STRESS	CORRESPONDING STRESS READING	CORRESPONDING STRAIN VALUE	ANGLE OF SLOPE	AVERAGE VALUE OF ANGLE
C ₂₁	20	1.63982	0.0564542	10°	
	40	3.27964	0.0428407		
	50	4.09955	0.0363390		
	80	6.55928	0.0156135		
	100	8.20000	0.0021000		
C ₂₂	20	1.92000	0.0346885	14°	24.4°
	40	3.84000	0.0265163		
	50	4.80000	0.0224303		
	80	7.68000	0.0101721		
	100	9.60000	0.0020000		
C ₂₃	20	2.27999	0.0265709	39°	
	40	4.55999	0.0204281		
	50	5.69999	0.0173568		
	80	9.11999	0.0081427		
	100	11.40000	0.0100000		
C ₂₄	20	1.92000	0.0346885	15°	
	40	3.84000	0.0265163		
	50	4.80000	0.0224303		
	80	7.68000	0.0101721		
	100	9.60000	0.0020000		
C ₂₆	20	2.60000	0.0233704	44°	
	40	5.20000	0.0180278		
	50	6.50000	0.0073426		
	80	10.40000	0.0073426		
	100	13.00000	0.0092000		

TABLE 4.7

COLUMN	% LEVEL	STRESSES $\times 10^6$	STRAINS $\times 10^{-3}$	ANGLE	AVG. ANGLE
C ₃₁	20	1.760000	0.0491756	10°	10°
	40	3.520000	0.0373817		
	50	4.400000	0.0314848		
	80	7.040000	0.0137939		
	100	8.800000	0.0018000		
C ₃₂	20	2.000000	0.0380440	15°	15°
	40	4.000000	0.0290303		
	50	5.000000	0.0245253		
	80	8.000000	0.0110101		
	100	10.000000	0.0036000		
C ₃₃	20	2.279999	0.0325388	21°	15.17°
	40	4.559999	0.0249041		
	50	5.699999	0.0210867		
	80	9.119999	0.0096347		
	100	11.400000	0.0074000		
C ₃₄	20	2.560000	0.0296436	18°	18°
	40	5.120000	0.0227527		
	50	6.400000	0.0192773		
	80	10.240000	0.0089109		
	100	12.800000	0.0042000		
C ₃₅	20	1.640000	0.0624222	10°	10°
	40	3.280000	0.0473166		
	50	4.100000	0.0397638		
	80	6.560000	0.0171055		
	100	8.200000	0.0072000		
C ₃₆	20	2.240000	0.0331059	17°	17°
	40	4.480000	0.0253294		
	50	5.600000	0.0214412		
	80	8.960000	0.0097765		
	100	11.200000	0.0020000		

4168.

TABLE 4.8

COLUMN	% LEVEL	STRESSES $\times 10^6$	STRAINS $\times 10^{-3}$	ANGLE	AVG. ANGLE
C ₄₁	20	2.520000	0.0252422		
	40	5.040000	0.0194316		
	50	6.300000	0.0165263		
	80	10.080000	0.00781054		
	100	12.600000	0.01060000		
C ₄₂	20	1.880000	0.0375489		
	40	3.760000	0.0286617		
	50	4.700000	0.0242181		
	80	7.520000	0.0108872		
	100	9.400000	0.0028000	12.50°	
C ₄₃	20	2.600000	0.0245102		
	40	5.200000	0.0189576		
	50	6.500000	0.0161314		
	80	10.400000	0.0076525		
	100	13.000000	0.0030000	26°	
C ₄₄	20	2.080000	0.0376926		
	40	4.160000	0.0287694		
	50	5.200000	0.0243079		
	80	8.320000	0.0109231		
	100	10.400000	0.0046000	14°	19.5°
C ₄₅	20				
	40				
	50				
	80				
	100	NO RESULT			
C ₄₆	20	2.320000	0.0272917		
	40	4.640000	0.0209687		
	50	5.800000	0.0178073		
	80	9.280000	0.0083229		
	100	11.600000	0.0050000	25°	

CONCLUSIONS

5.1 GENERAL

This Research carried out an experiment investigations on modelled (short) Reinforced Concrete columns in order to collect informations good enough to be used for predictions of the MOST-DUCTILE hoop arrangement commonly used in construction industry.

During the experiment, some factors were fixed (constant) in order to study satisfactory the effectiveness of the arrangements only. Those factors fixed in this experiment were:-

- (i) Mono Grade of Concrete.
- (ii) Single Size Reinforcement bars.
- (iii) Uniform Specimen Sizes.
- (iv) Single brand of portland Cement (ASHAKA).

After the experiment, it was realised that the results of the descending portion of the STRESS-STRAIN Curve was inevitably absent. This was due to erratic behaviour of the Strain-meter during the experiment. As a result of this development, a method used by an independent research (16) was adopted. This method was the use of descending (falling) branch of the STRESS-STRAIN Curve. The research indicated that the effect of the confining concrete begins to show only after the attainment of the ultimate load (maximum).

5.2 P E R F O R M A N C E

The performance of all the different arrangements studied in this research was carefully noted both manually and electronically. During the observations, it was found out that (see results tables) the Double crossing over lapping hoop arrangement carried the maximum load. While the Double Crossing over lapping arrangement was averaging 200 KN, others were averaging about 175 KN. The closest arrangement that carries higher load was the diamond shape, which averaged about 185 KN.

During crushing of the specimens, it was also observed that the Double crossing over lapping arrangement, resists more load before showing signs of cracking than the other types of arrangement.

Although Aggregate Grading and Amount of Water used in the concrete mix of this Experiment which certainly affected the concrete strength (see cube strength table), it has by no means affected the findings. From the Graphs and computations, you can now make categorical judgement that the DOUBLE CROSSING (TIES) overlapping arrangement offers more ductivity than the remaining arrangements studied.

5.3 S U G G E S T I O N S

Since this is a very wide experimental research with more variables to be investigated, it is now evident that more experimental work is required on confined concrete specimens to provide more data for statistical analysis and to enable the inclusion of the variables. In particular, investigations of a wider arrangement of transverse hoops of different overlapping shapes and sizes are badly needed. The shape of the concrete column (Rectangular, square, round, etc.) is another area of the experiment that can be investigated. Finally the concrete cover of the specimens can also be looked into.

REFERENCES

1. Wynn, A.E. (1951): "Design and Construction of FORMWORK FOR CONCRETE STRUCTURES". Concrete Publications, London.
2. Charles, G.S. (1985): "Reinforced Concrete Design". Harper and Row Publishers, NY., pp. 400 - 486 and 537 - 604.
3. Boguslavsky, B.W. (1956): "Design of Reinforced Concrete". Macmillan, New York.
4. M. Nadim Hassoun (1985): "Design of Reinforced Concrete Structures". P.W.S. Engineering, Boston, pp. 1 - 40 and 280 - 355.
5. Charles Head Morris, Sc.D., John Benson Wilbur, Sc.D., Senol Utku Sc.D., "Elementary Structural Analysis". Third Edition, McGraw-Hill, pp. 621 - 623.
6. Wang, Chu-Kia (1953): "Statistically Indeterminate Structures". McGraw-Hill, New York.
7. Everard, N.J. and Tanner, J. (1966): "Schaum's Outline of Theory and problems of Reinforced Concrete Design". Schaum, New York.
8. Portland Cement Association (1979): "Design and Control of Concrete Mixtures". Twelfth Edition, Skokie, pp. 7 - 15.
9. Case, J. and Chilver, A.H. (1971): "Strength of Materials and Structures". English Language Book Society and Edward Arnold (Publishers), London, pp. 69 - 98 and 346 - 372.
10. Kirby, P.A. and Hethercot, P.A. (1979): "Design for Structural Stability". Constrado Monographs, Collins, London, pp. 16 - 62.

11. Park, R. and Paulay, T. (1975): "Reinforced Concrete Structures". John Wiley and Sons, Inc., New York, pp. 769.
12. Chan, W.L. (Nov. 1955): "The Ultimate Strength and deformation of Plastic Hinges in Reinforced Concrete Framework". Concrete Research Magazine Vol. 7, No. 21, pp. 121 - 132.
13. Blume, J.A.; Corning, L.H.; Newmark, N.M. (1961): "Design of Multi Storey Reinforced Concrete Buildings for Earthquake Motions". Portland Cement Associations, Chicago, pp. 318.
14. / s oi PF elastic Hyper-
nal Symposium
II, Miami,
15. So. str Rel pp. 1 Stress-
Transverse
. 61,
16. Roy, Proce Reinf oncrete".
chanics of
17. Sargir Latera of Con: pp. 99 Effects of
perties
- 76,
18. Scott, B.D.; Park, R. and Priestley, M.J.N. (Jan. - Feb. 1982):
"Stress-strain Behaviour of Concrete Confined by Overlapping Hoops
at Low and High Strains Rates". A.C.I. Journal, No. 79 - 2,
pp. 13 - 27.

Handwritten notes:
 Effectiveness of overlapping
 hoops as ductility reinforcement
 in shear reinforced concrete
 columns by J. A. Alkhatib
 Zaidi, Dept of Civil Engineering
 1992
 X11, 87 Huanes; ill. plates, tables
 Thesis (M.Sc.) Ahmadu Bello Uni
 versity.
 448872

LOAD-STRESS CONVERSION

AREA OF THE COLUMN

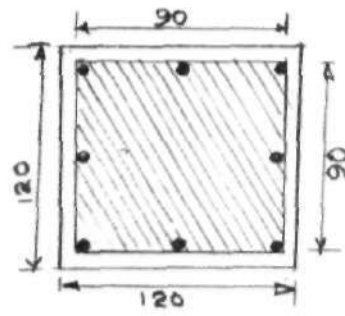
$$= 0.12 \times 0.12 = 0.0144 \text{ m}^2$$

LOAD (KN)	STRESS
50	3.17 N/mm ²
60	3.81 " "
80	5.08 " "
100	6.35 " "
130	8.25 " "
150	9.52 " "
175	11.11 " "
200	12.69 " "
210	13.30 " "
225	14.28 " "
250	15.87 " "

APPENDIX I

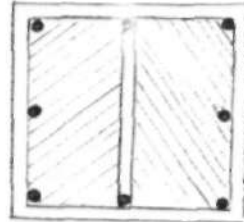
ARRANGEMENT TYPES AND AFFECTED AREAS OF LOOPS

ARRANGEMENT ONE(1) - RECTANGULAR HOOP



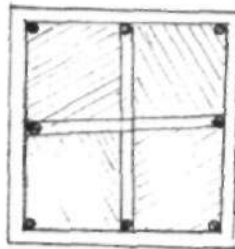
AREA OF CONFINEMENT
 7396 mm^2
 =====

ARRANGEMENT TWO (2) - CROSSING : OVER LAPPING



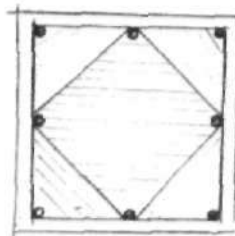
AREA OF CONFINEMENT
 $= 6492 \text{ mm}^2$
 =====

ARRANGEMENT THREE(3) - DOUBLE CROSS OVLAP LAPPING



AREA OF CONFINEMENT
 6432 mm^2
 =====

ARRANGEMENT FOUR(4) DIAMOND OVERLAPPING



AREA OF CONFINEMENT
 6468 mm^2
 =====

APPENDIX II

STRAIN VALUES (GAUGE FACTOR IS 2.0×10^{-6})

APPENDIX TTI

COLUMN TYPE	50 KN	100 KN	130 KN	150 KN	175 KN	200 KN	225 KN	REMARKS
2.44×10^{-4}								
C11 (a) TF	-0.000020	0.003140	*0.006940					FAILED @
(b) TB	0.001000	0.005280	*0.002400					130 KN
* (c) MF	-0.000100	0.000600	*0.013700					
(d) MB	0.000520	0.001720	*0.000820					TOP
C12 (a) TF	0.001280	0.002120	0.002720	0.003100	0.003560	*0.004120		FAILED @
(b) TB	0.001100	0.003120	0.005360	0.006600	0.010400	*		210 KN
(c) MF	0.000700	0.001180	0.001570	-	0.002160	*0.005900		
* (d) MB	0.000740	0.001460	0.001980	0.002200	0.002880	*0.004380		TOP
C13 (a) TF	-0.003660	0.000500	0.005240	0.000940				CRACK @ 100 KN
* (b) TB	0.000280	0.000700	0.001280	0.006140	*			FAILED @ 141 KN
(c) MF	0.000340	0.000780	0.000980	0.000420				TOP AND BOTTOM
(d) MB	0.003160	-0.002760	-0.004360					
C14 (a) TF	-0.005640	-0.000820	-0.005940	-				CRACK @ 130
(b) TB	0.000140	0.000140	-0.003360	0.002960				
(c) MF	0.001380	0.003760	+0.003200	-0.00480				FAILED @ 142
* (d) MB	-0.000040	0.000400	-0.001400	-0.007760				TOP
C15 (a) TF	0.000720	0.001760	0.004160	-				FAILED @
* (b) TB	0.000860	0.002440	0.003800	-*				150 KN
(c) MF	-	-	-	-				
(d) MB	0.000700	0.001980	0.003020	0.001960				TOP
C16 (a) TF	0.007440	-	-	-	-	188 KN		FAILED @
* (b) TB	0.000180	0.000300	0.000600	0.000700	0.001300	-0.000460		188 KN
* (c) MF	0.002260	0.002400	0.002900	0.003300	0.004000	0.002000		
(d) MB	0.000100	0.000860	0.001260	0.003500	0.002180	-0.000080		TOP

STRAIN-VALUES (GAUGE FACTOR 2.06x10 ⁻⁶)								APPENDIX IV
COLUMN TYPE	50 KN	100 KN	130 KN	150 KN	175 KN	200 KN	225 KN	REMARKS
C21 (a) TF (b) TB (c) MF (d) MB	0.00494	0.001030	0.002039	-	*0.001627			CRACK @ 150 KN FAILED @ 162 KN
	-	-	-	-	-			
	0.002204	-0.001174	-0.000412	-	*-0.001854			
	0.001215	0.000391	0.001009	0.001421				
C22 (a) TF (b) TB (c) MF (d) MB	0.000288	0.002781	0.001545	0.003461	*0.005500			CRACK @ 100 FAILED @ 168 KN
	-	-	-	-	-			
	-0.000165	0.000371	0.001318	0.001999	*0.000082			
	0.007766	0.012916	0.015656	0.016686	*-0.001854			
C23 (a) TF (b) TB (c) MF (d) MB	0.001916	0.002740	0.006510	0.008055	0.009558	*0.004058		CRACK @ 130 KN FAILED @ 204 KN
	0.000536	0.001257	0.000235	0.002966	0.003790	-		
	0.005150	0.005665	0.006139	0.004388	0.006592	*0.002081		
	-0.000309	0.000268	-0.000742	-0.000433	0.000412	*-0.010815		
C24 (a) TF (b) TB (c) MF (d) MB	0.000103	0.000412	0.000597	0.000742	0.000391			CRACK @ 150 KN FAILED @ 175
	-0.000185	0.000185	0.000742	0.001792	-0.013040			
	0.000968	0.004079	0.004985	0.003069	-			
	0.000391	0.000927	0.001545	0.002307	-0.001442			
C25 (a) TF (b) TB (c) MF (d) MB	-0.001133	-0.000659	-0.000680	-	-			CRACK @ 150 FAILED @ 165 KN
	0.000391	0.002987	0.001833	0.000268	-0.006345			
	-	-	-	-	-			
	0.000165	0.001092	0.001504	0.000639	-0.000453			
C26 (a) TF (b) TB (c) MF (d) MB	-0.000742	-0.000309	0.0001236	0.000227	-0.000618	*-0.000206		CRACK @ 150 FAILED @ 190
	-0.004614	-0.004182	-	-	-	-		
	0.005624	0.000618	0.001174	0.001545	0.001957	*0.000556		
	0.001339	0.001957	0.002410	0.002946	0.008096	*0.002122		

COLUMN TYPE		50 KN	100 KN	130 KN	150 KN	175 KN	200 KN	225 KN	APPENDIX V
STRAIN-VALUES (GAUGE FACTOR IS 2.0×10^{-6})									
C31 (a) TF									
(b) TB	0.000440	0.000980	0.000740						REMARKS
(c) MF	0.000260	0.000500							
(d) MB	-0.000240	0.000920	0.001500						FAILED
C32 (a) TF									
(b) TB	0.001440	0.004740	0.004840	0.004340	0.005040	0.003640			@ 107 KN
(c) MF									
(d) MB	0.003680	0.002340	0.003360	0.004640	0.006880	0.005680			CRACKS @ 130 KN
C33 (a) TF									
(b) TB	0.000640	0.001880	0.003000	0.006700	0.007100				FAILED @ 185 KN
(c) MF	0.001320	0.001560	0.001780	0.002080	0.002220				
(d) MB	0.002020	0.002200	0.002500	0.003400	0.002360				CRACK @ 120 KN
C34 (a) TF									
(b) TB	0.000580	0.001140	0.001480	0.001680	0.002300	0.004180			FAILED @ 171 KN
(c) MF	0.000240	0.000540	0.000840	0.000840	0.001080				BOTTOM
(d) MB	0.000380	0.001040	0.001520	0.001880	0.002440	0.003140			CRACK @ 150
C35 (a) TF									
(b) TB	0.004800	-0.003000			0.002700	-0.001460			FAILED @ 200 KN
(c) MF	-0.002900	-0.002900	-0.002500	-0.004580					TOP AND BOTTOM
(d) MB	0.000420	0.000760	0.000720	-0.000340					CRACK @ 50 KN
C36 (a) TF									
(b) TB	0.007320	0.008400	0.009400	0.008600					FAILED @ 135 KN
(c) MF	0.002580	0.004460	0.005520	0.004120					CRACK @ 150
(d) MB	0.003720	0.004540	0.004560	0.002060					FAILED @ 150
TOP									
BOTTOM									

COLUMN TYPE		50 KN	100 KN	130 KN	150 KN	175 KN	200 KN	225 KN	REMARKS
STRAIN VALUES (GAUGE-FACTOR IS 2.06×10^{-6})									
C41	(a) TP	0.000886	0.001957	0.002678	0.003090	0.003872	0.003605	* 0.003358	CRACK @ 150 KN
	(b) TB	0.000165	0.000103	0.000453	0.000412	0.000515	0.000783	* -0.002781	
	(c) MP	0.001092	0.003502	0.005686	0.007231	0.008302	0.001051	* -0.001607	
	(d) MB	-0.000103	-0.000165	0.000124	0.0004326	0.000865	0.001298	* -0.001298	
C42	(a) TP	0.000247	-	0.002307	* -0.002122	-	-	-	CRACK @ 130 KN FAILED @ 230 KN
	(b) TB	0.000433	-0.000185	-0.001998	* -0.016995	-	-	-	
	(c) MP	0.000330	0.001627	0.001978	* 0.002328	-	-	-	
	(d) MB	0.000412	0.001298	0.001360	* -0.004429	-	-	-	
C43	(a) TP	0.000680	0.001504	0.002760	0.005232	-	-	-	CRACK @ 155 KN FAILED @ 155 KN
	(b) TB	0.000268	0.000618	0.000968	0.001257	-	-	-	
	(c) MP	0.000597	0.001318	0.002039	0.002699	0.003317	0.003523	* -0.019611	
	(d) MB	0.001751	0.001998	0.002554	0.002997	0.003440	0.004120	* -0.000556	
C44	(a) TP	-0.000330	-0.000041	0.001339	0.000577	-	-	-	CRACK @ 225 FAILED @ 225
	(b) TB	-0.001040	-	-	-	-	-	-	
	(c) MP	-0.001401	0.000659	0.001442	0.001963	* 0.002494	-	-	
	(d) MB	0.000000	0.000391	-0.002266	-0.002369	* 0.009064	-	-	
C45	(a) TP	-	-	-	-	-	-	-	CRACK @ 130 FAILED @ 160 KN
	(b) TB	-0.000783	-0.000103	0.000330	0.000721	0.000062	* -0.003131	-	
	(c) MP	0.000062	0.001627	-	-	-	* -0.000350	-	
	(d) MB	-0.000474	0.000103	0.000453	0.001421	0.005088	* 0.000144	-	
C46	(a) TP	0.000433	0.001215	0.001854	0.002245	-0.002410	-	-	CRACK @ 160 KN FAILED @ 200 KN
	(b) TB	0.000103	0.001236	0.002060	0.002616	-0.004408	-	-	
	(c) MP	0.000515	0.001442	-	-	-	-	-	
	(d) MB	-	-	-	-	-	-	-	

CRACK @ 130
FAILED @ 168 KN

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FAILED @ 168 KN

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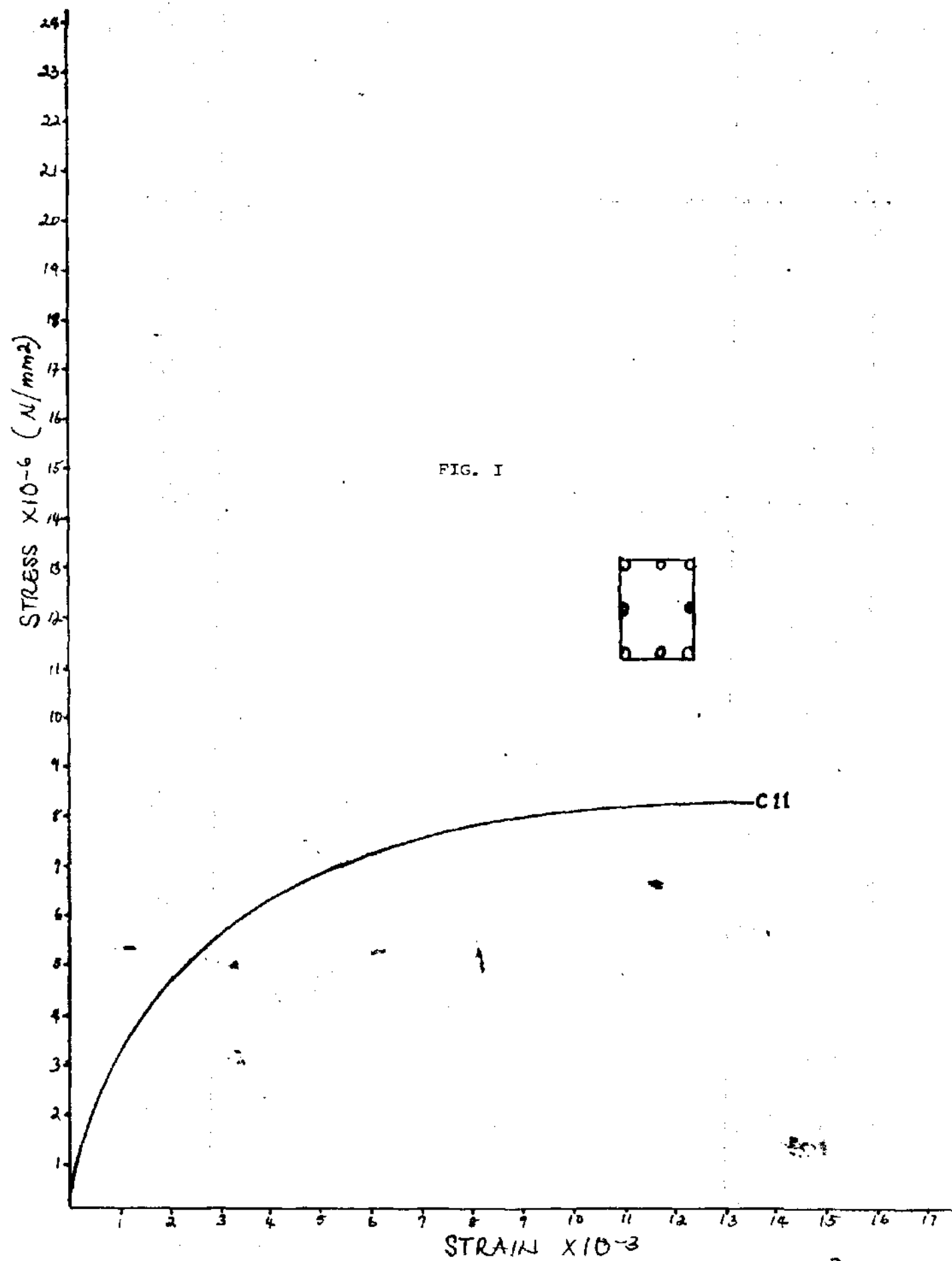
CRACK @ 130
FAILED @ 168 KN

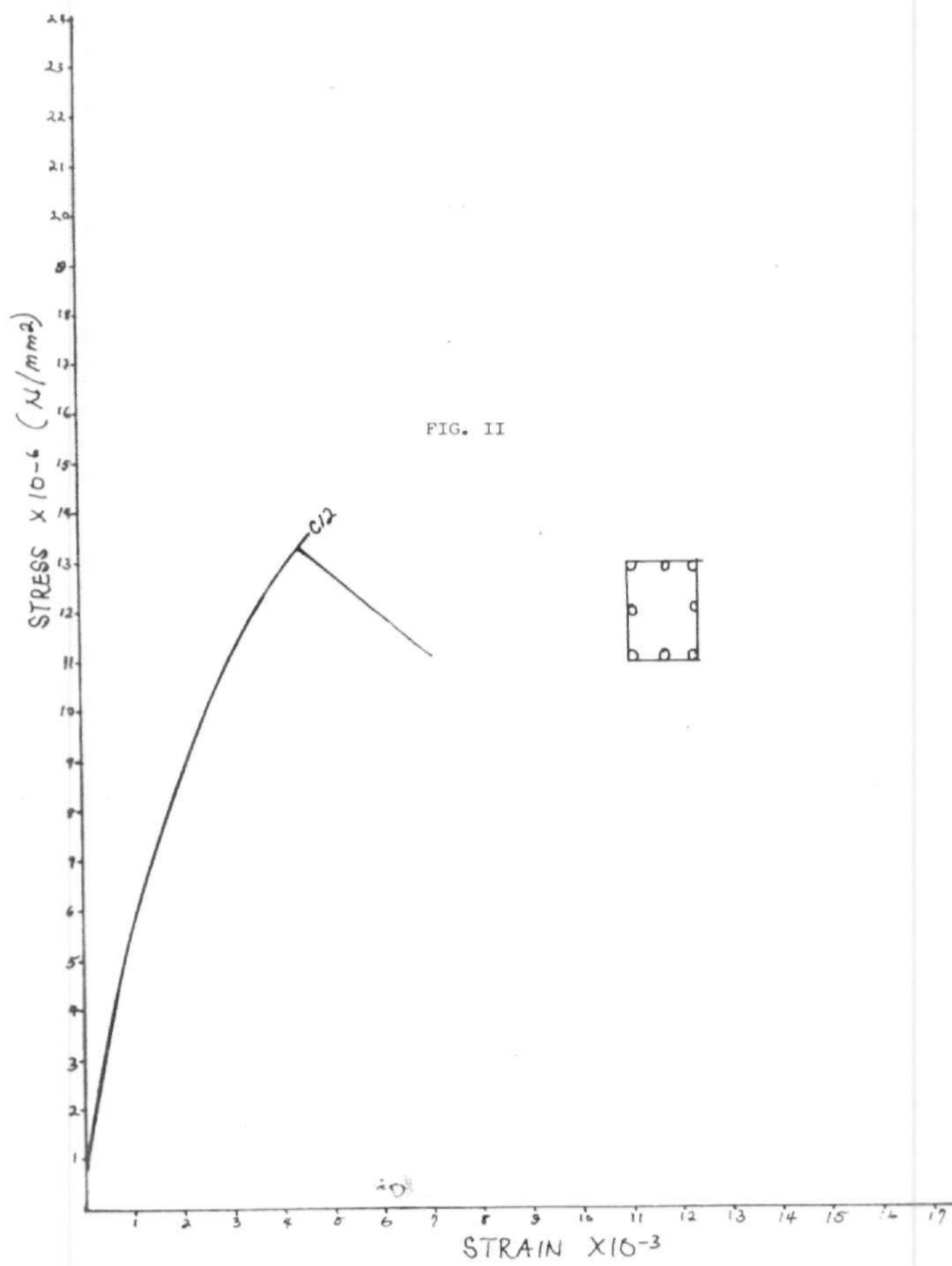
CRACK @ 130
FAILED @ 168 KN

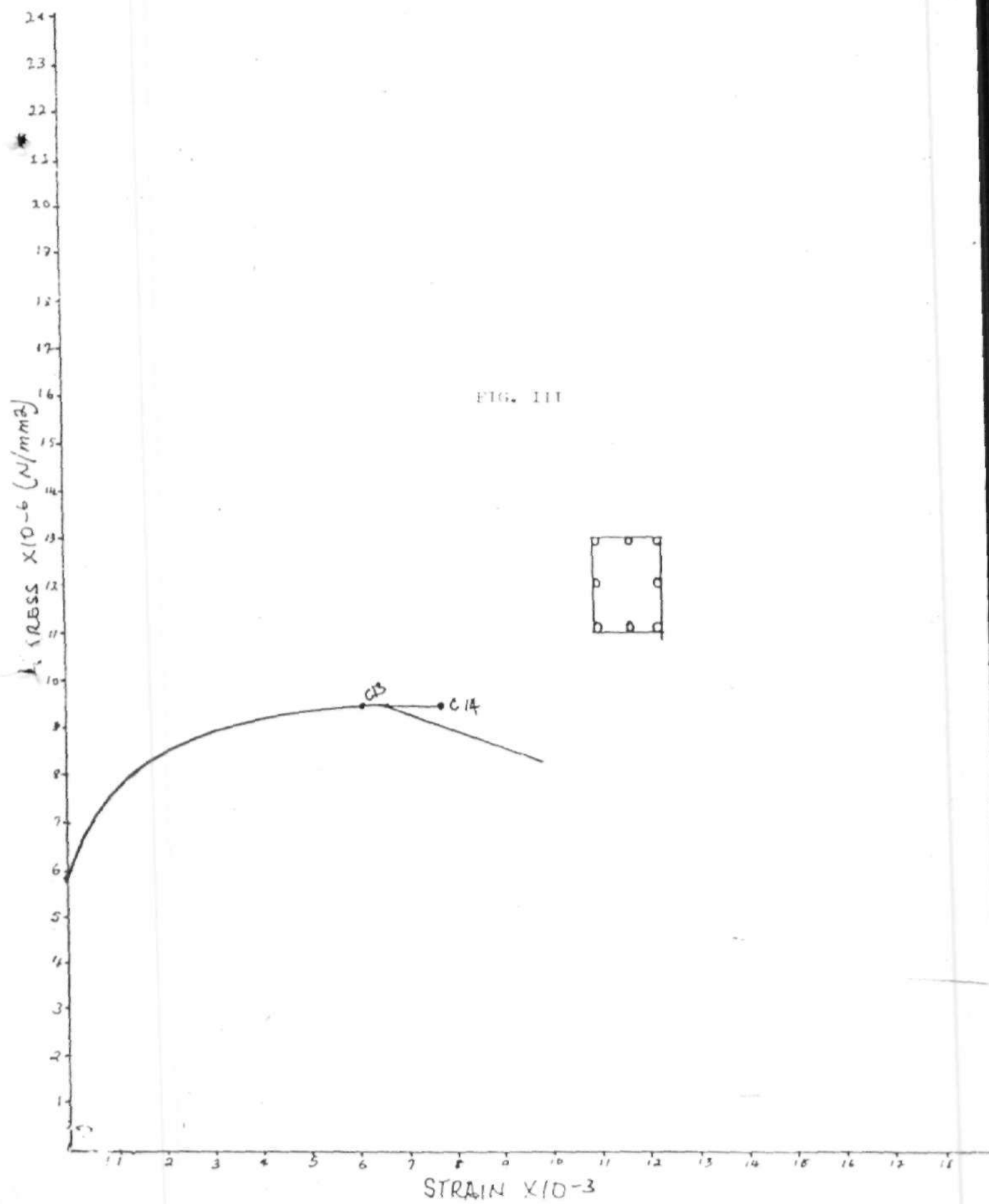
CRACK @ 130
FAILED @ 168 KN

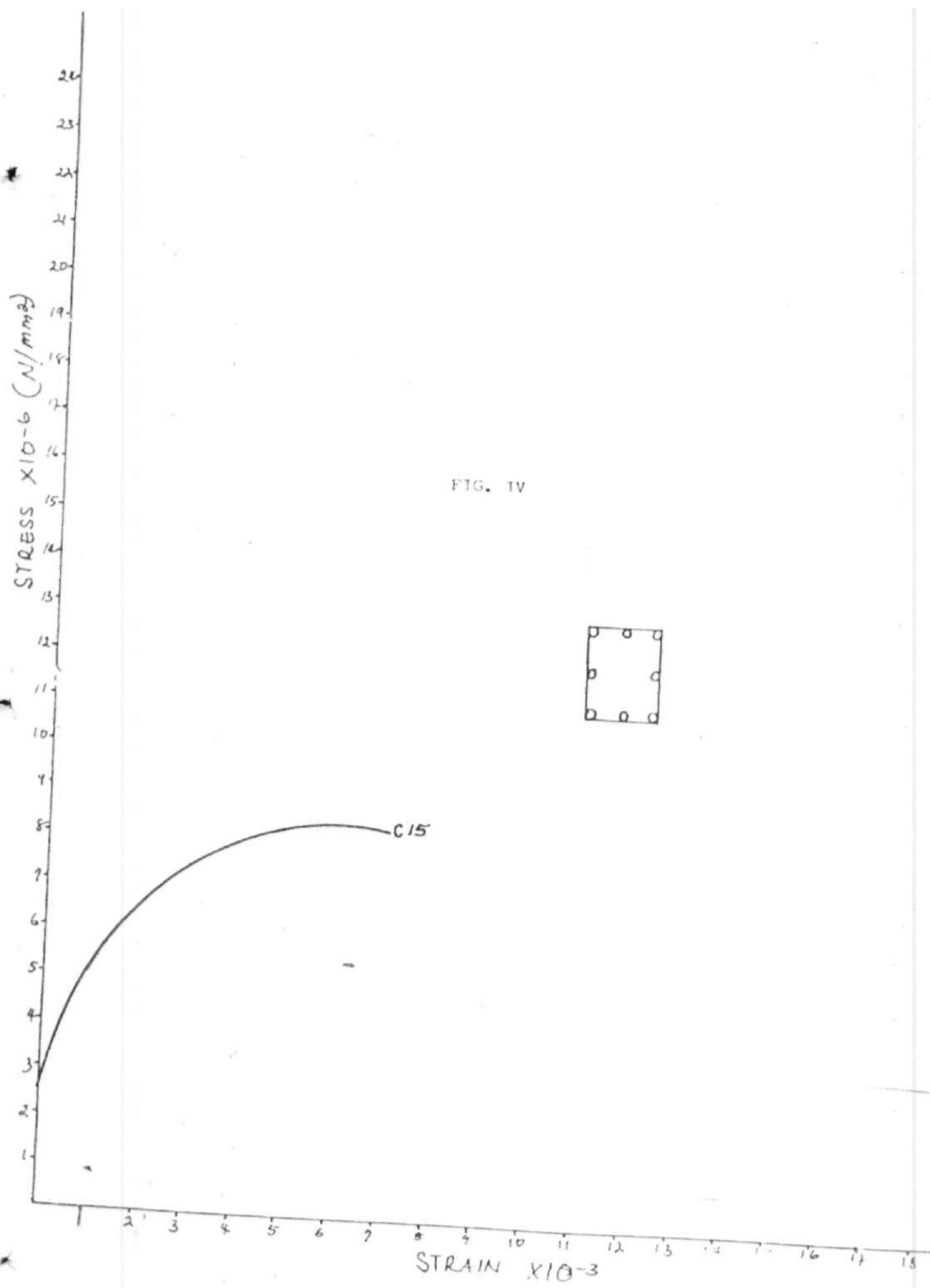
CRACK @ 130
FAILED @ 168 KN

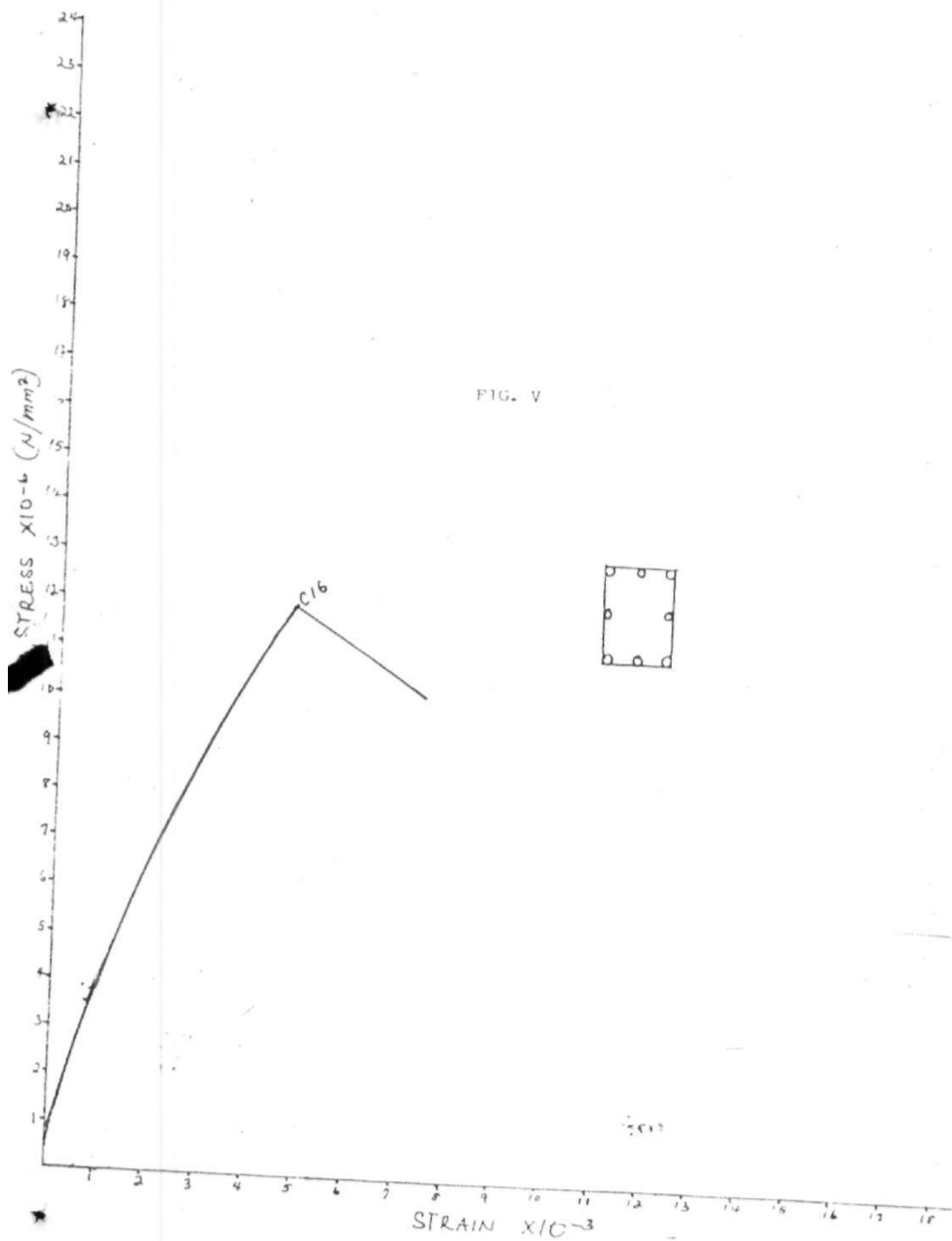
CRACK @ 130
FAILED @ 1

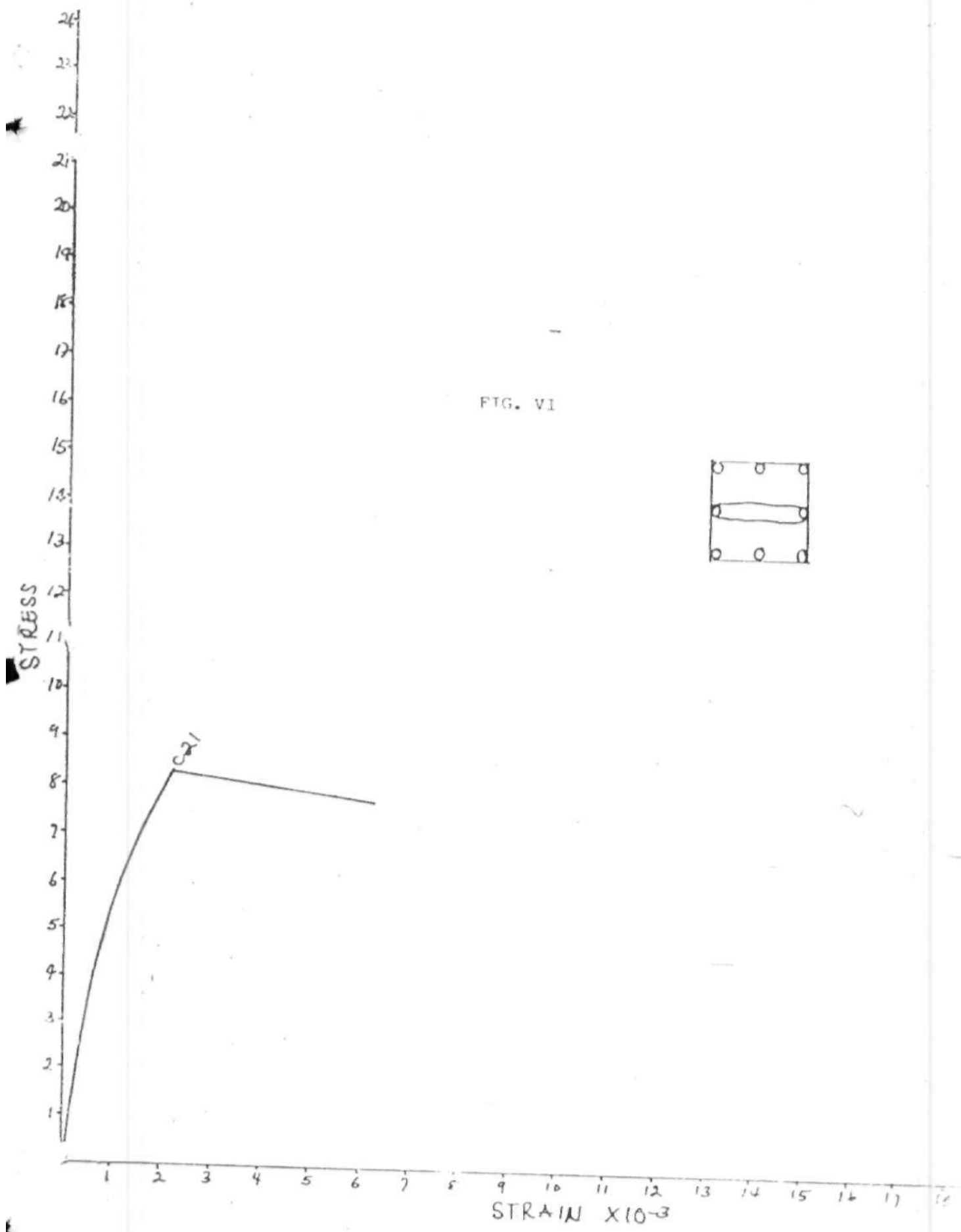


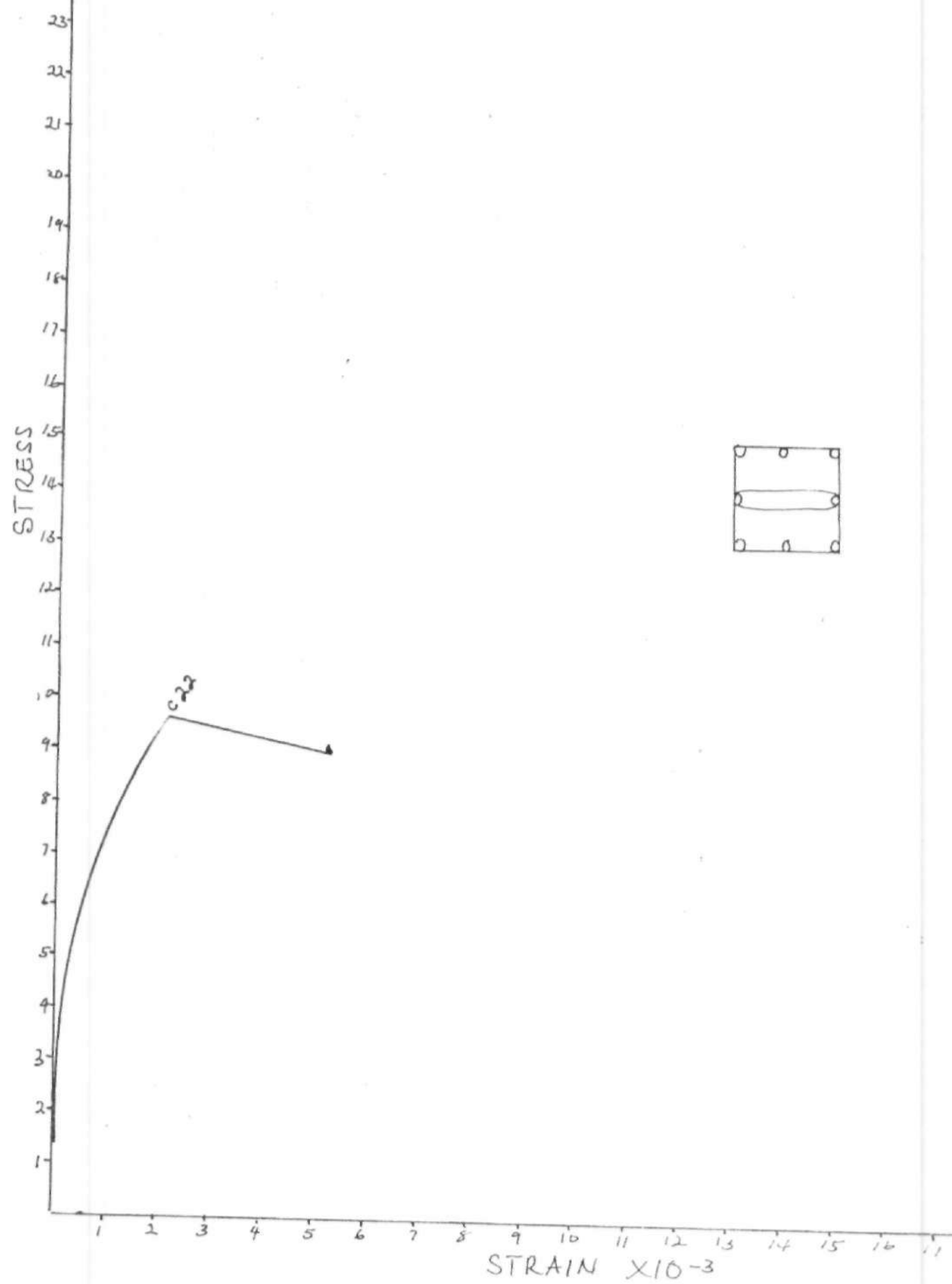


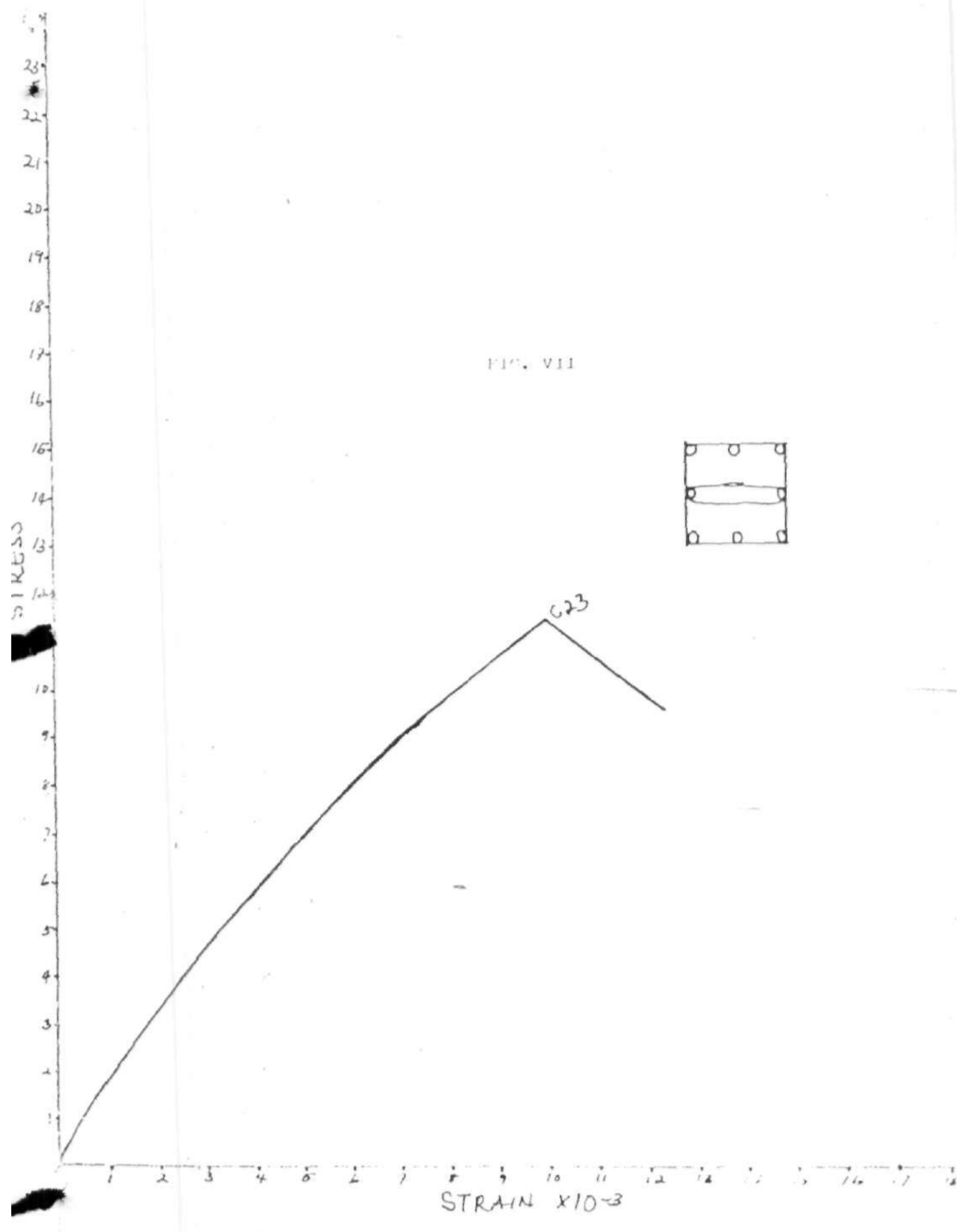


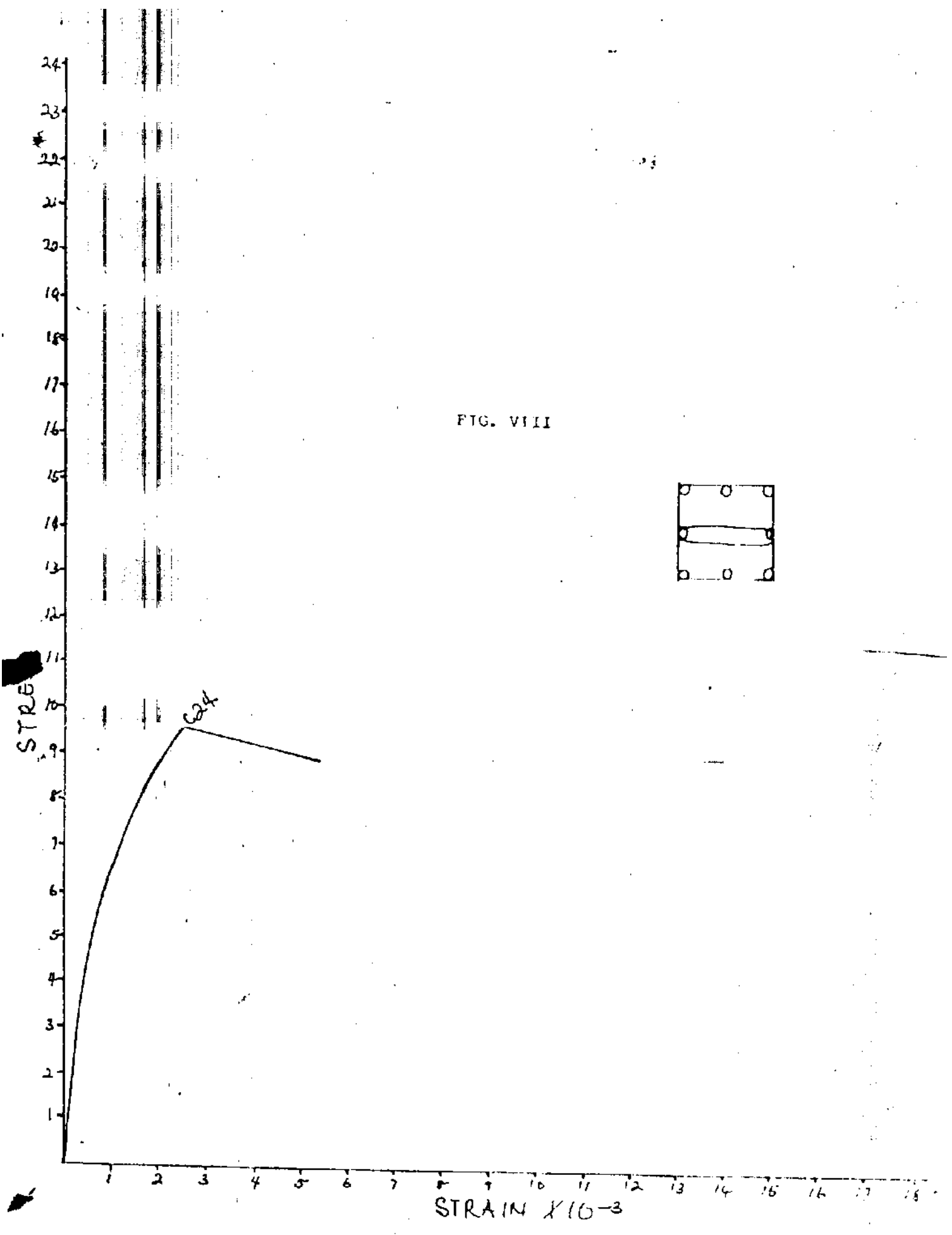


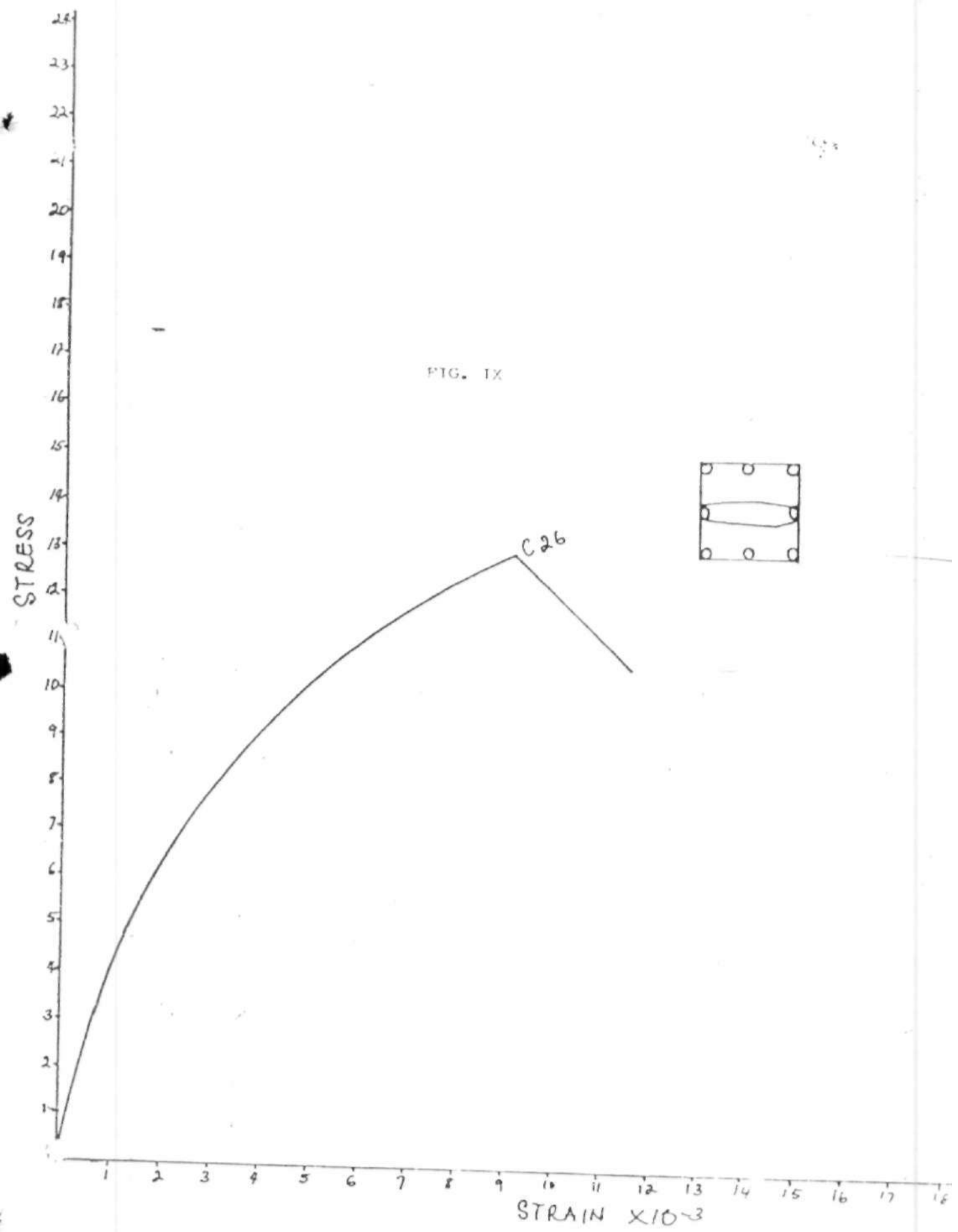


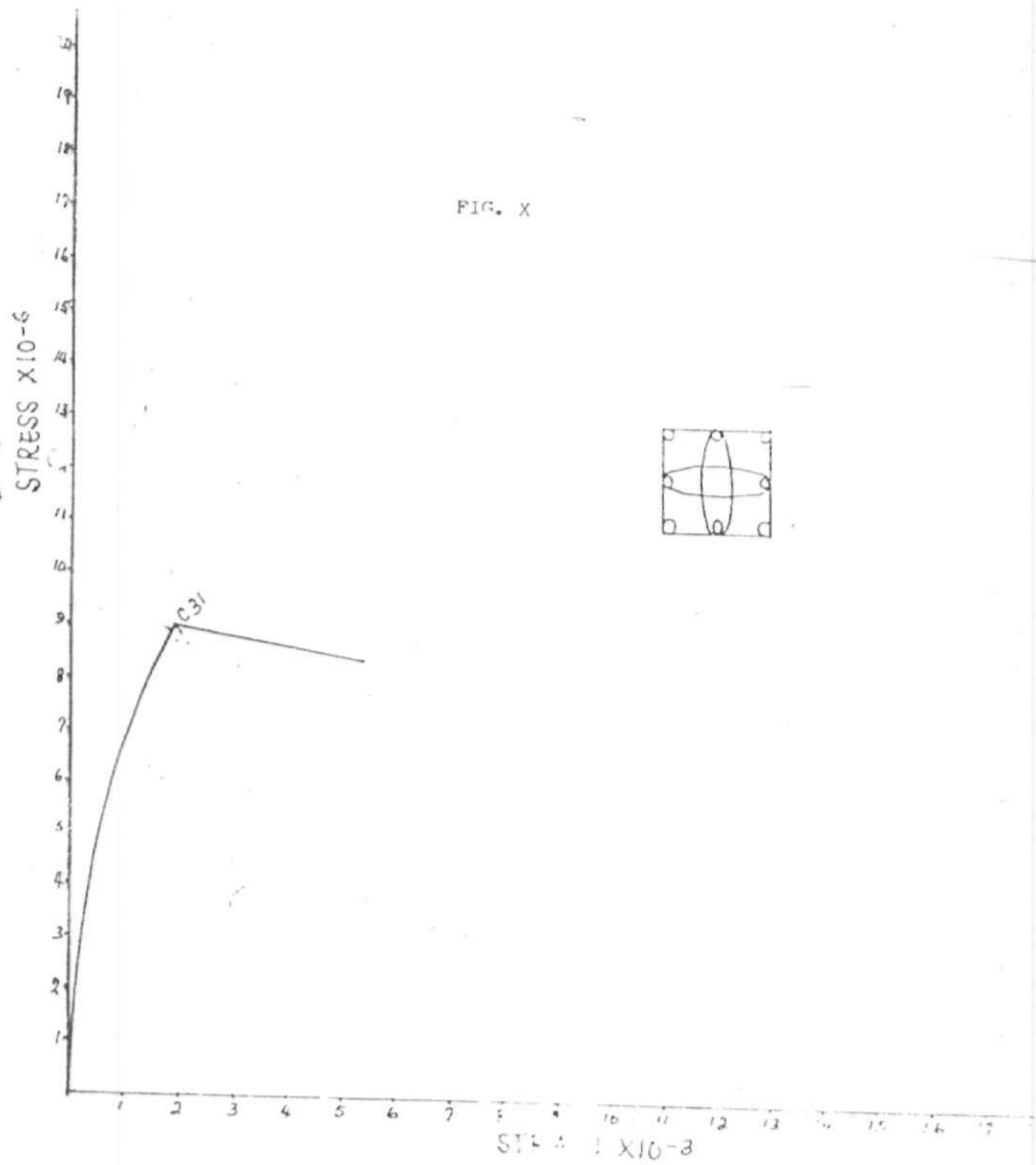












STRESS $\times 10^{-6}$

FIG. XI

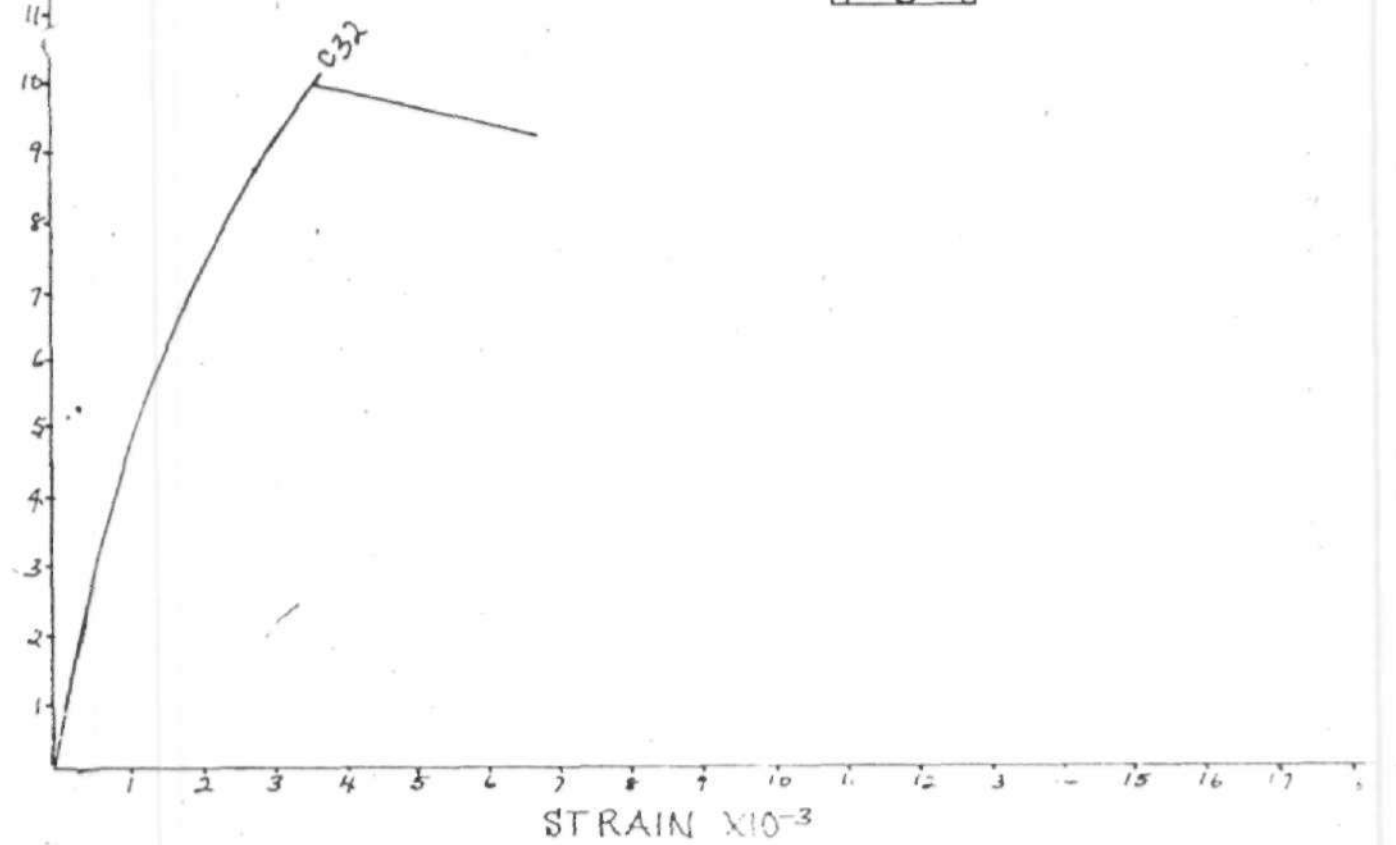
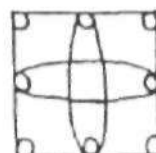
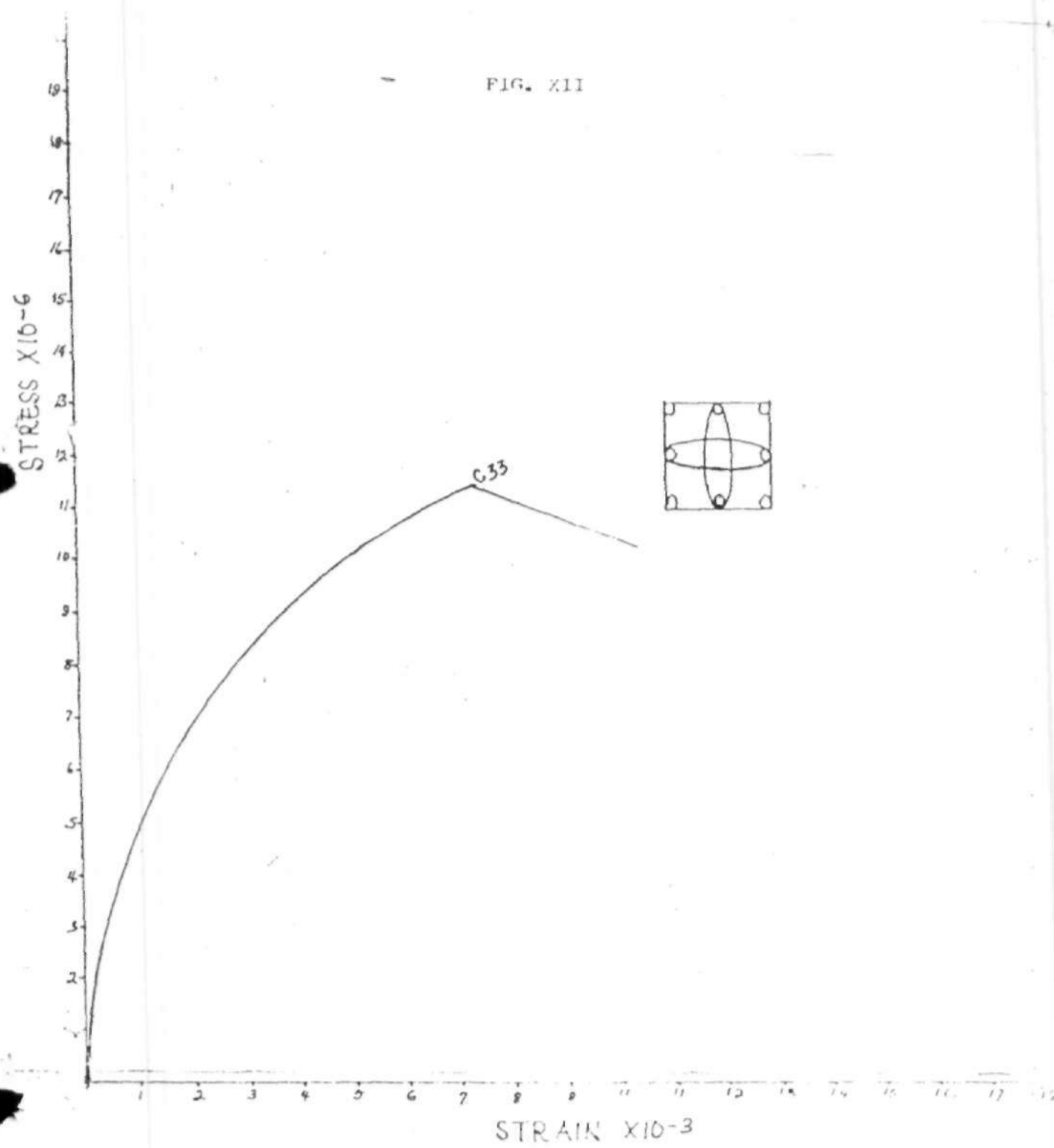
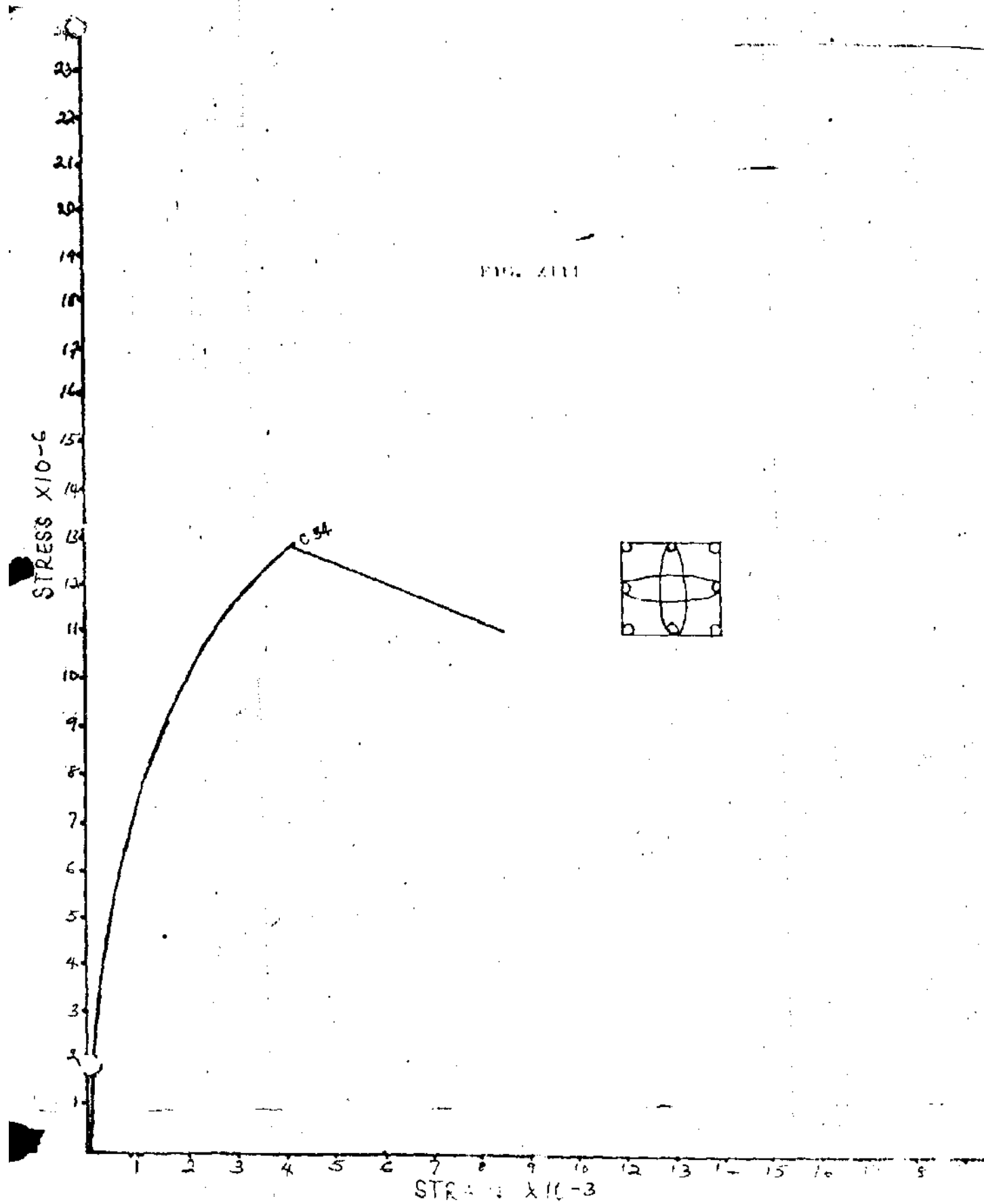
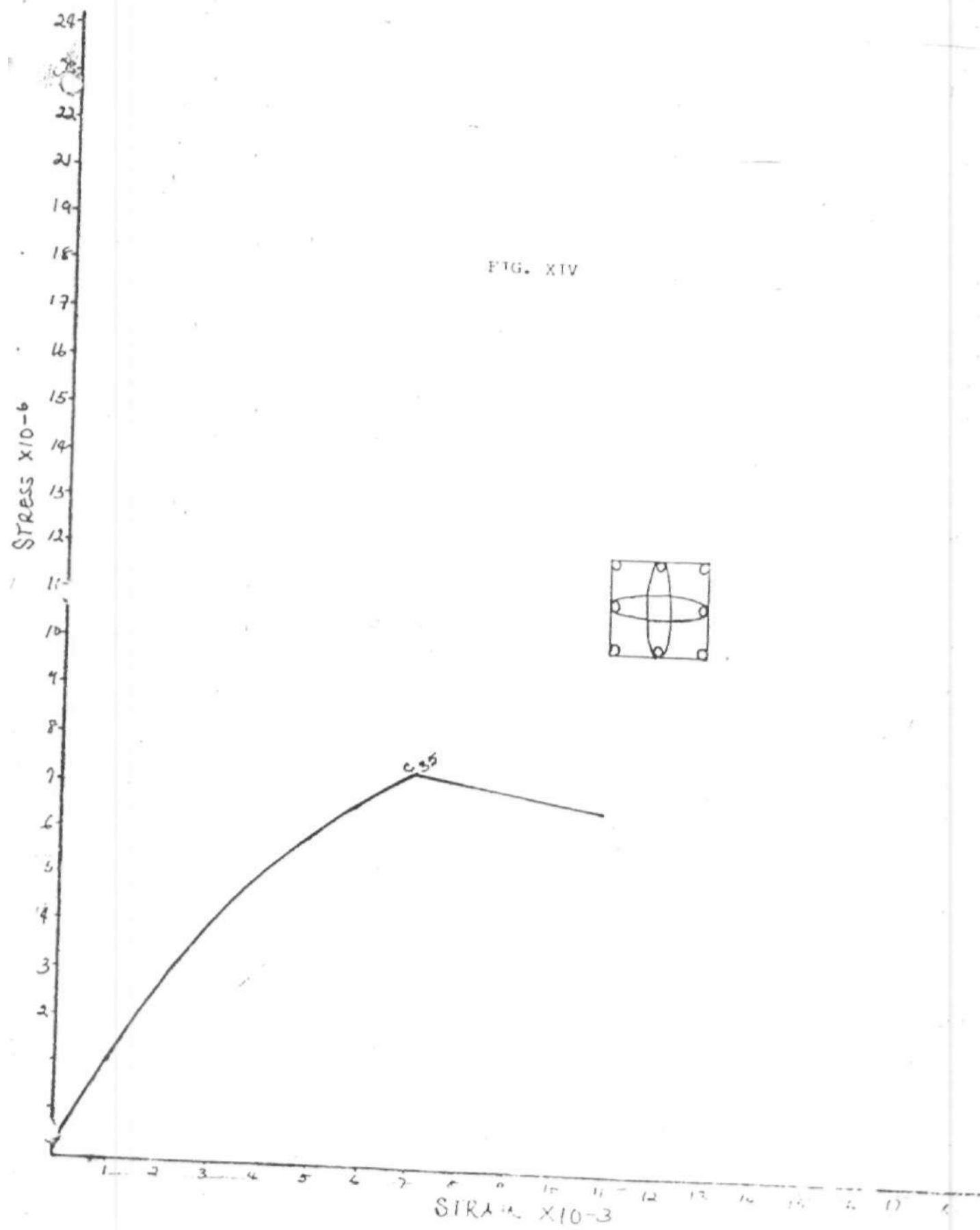
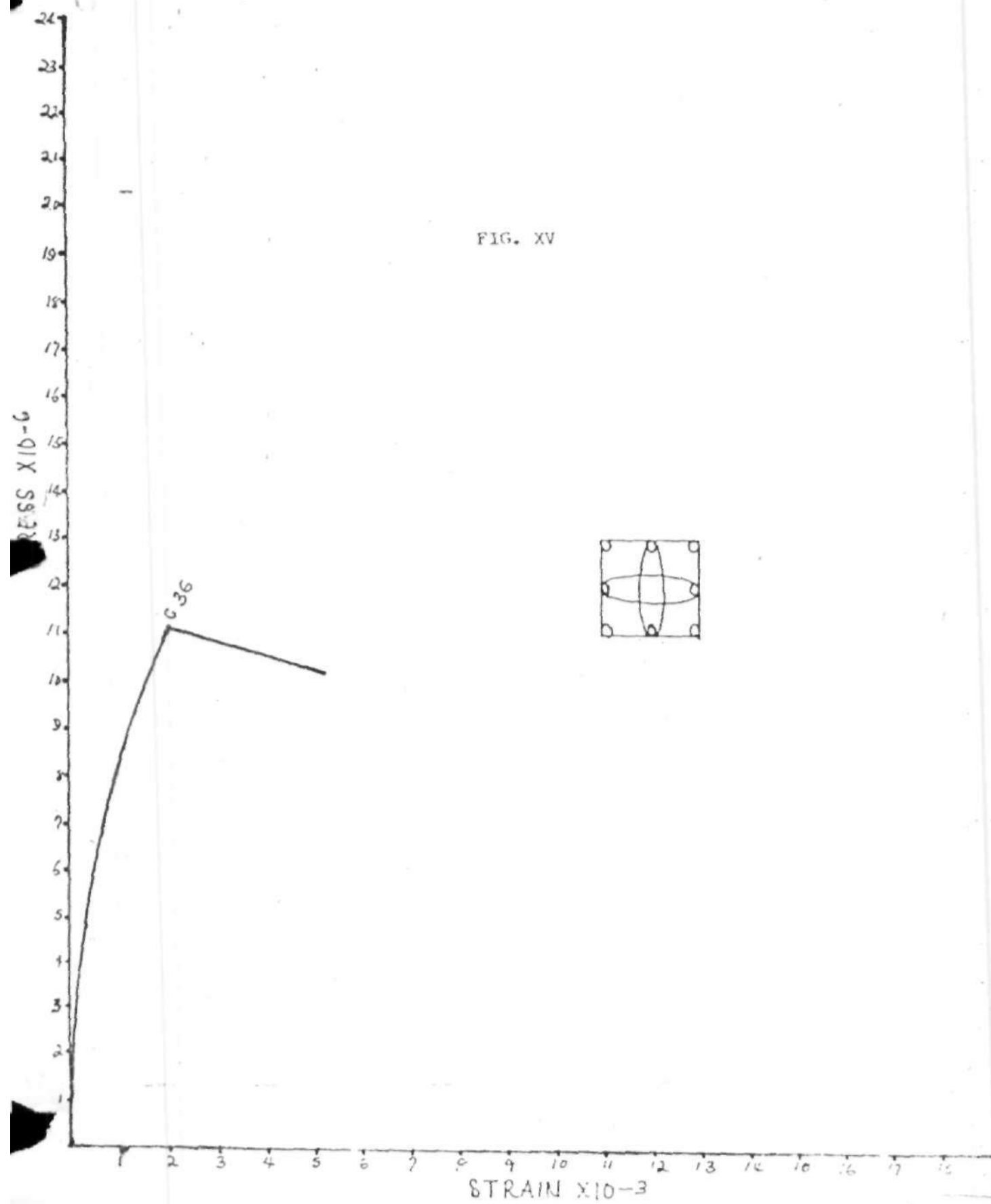


FIG. XII









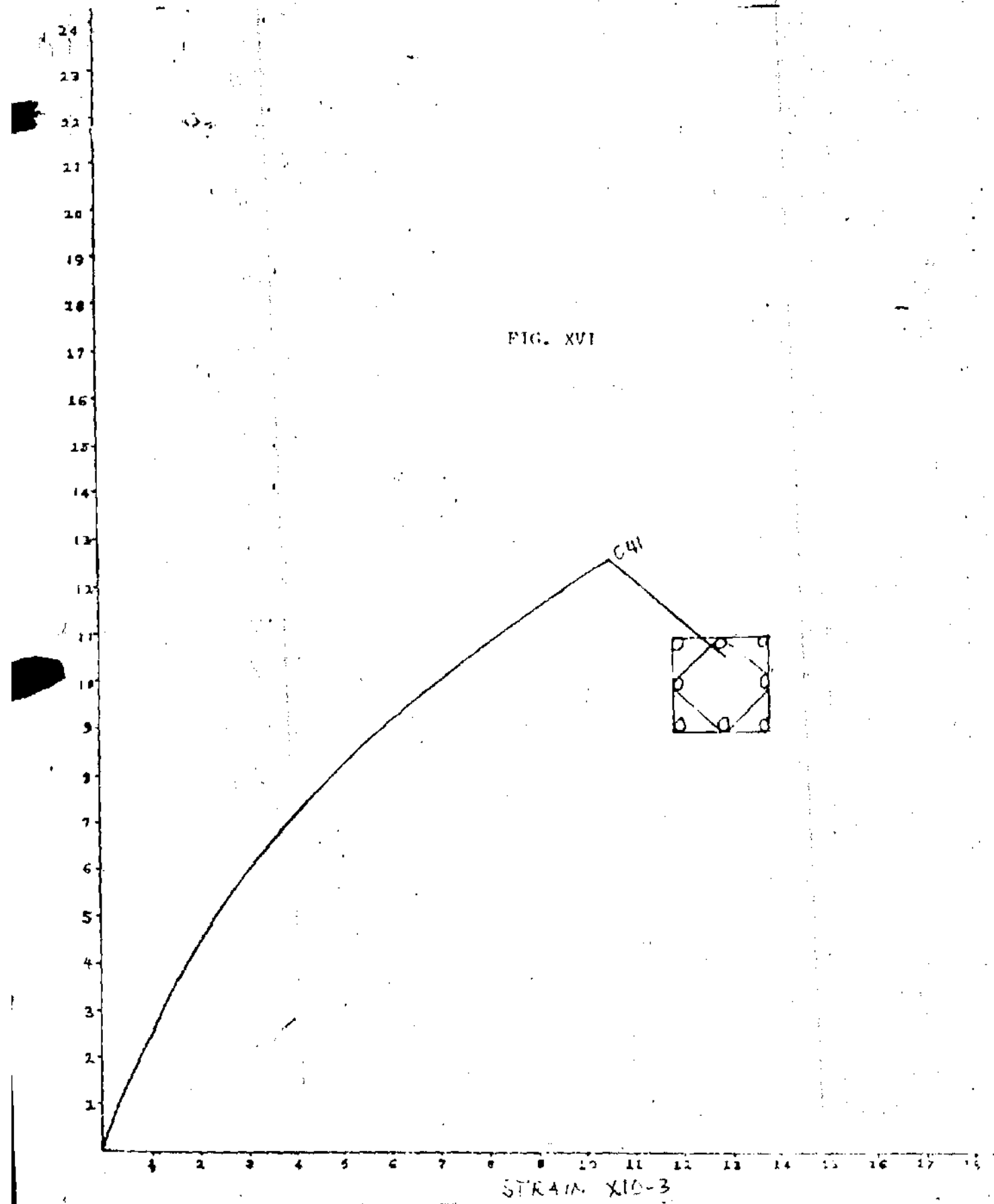


FIG. XVII

