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

By

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A thesis submitted to the Post-graduate School, * Ahmadu Bello University, Zaria in partial fulfilment of requirements for the degree of Master of Science in Civil Engineering (Structures).

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September, 1989

THESIS DECLARATION

I hereby declare that this Thesis has been entirely composed by me and that it is a record of my 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 information are specifically acknowledged by means of reference.

UMARU, Bello Sanda
September, 1989

CERTIFICATION

We hereby recommend that this Thesis, prepared under our supervision by UMARU, Bello Sanda entitled "Effectiveness of Overlapping Hoops as Ductility Reinforcement in Short Reinforced Concrete Columns" be 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 its contribution to knowledge and literary presentation.

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ACKNOWLEDGEMENT

All Praise be to Allah, the Lord of all Worlds for his infinite Mercy and Guidance before, during and after this study.

Special thanks and appreciation to my Supervisors Mrs Nytko and Prof. Abatan for their untiring efforts in the effective supervision of this project despite their numerous engagements. Their valuable suggestions and interest in this project is deeply appreciated.

My gratitude also goes to the government and people of Sokoto State for their sponsorship. Special mention must also be made of my Head of Department at the Ministry, Engr. Abdullahi Dalhatu as well as the Director-General and Honourable Commissioner of the Ministry of Works and Transport, Sokoto for their kind understanding and releasing me from all assignments to enable me complete this study.

I also wish to express my profound appreciation to the Staff of Concrete Laboratory of the Department of Civil Engineering, Ahmadu Bello University for agreeing to sweat it out with me in the tedious job of preparing and testing of all specimen. They include Mr. A.R.O. Bello, Alhaji Shonibare, Malam Abdullahi, Malam Samaila and a host of others. To them, I say 'Thank you and God Bless'. Many thanks to Mr. Abiodun for the pains he took in typing out the

manuscripts. Finally, I must not forget the encouragement of my numerous friends, colleagues and well wishers at various stages of this project.

UMARU, Bello Sanda
September, 1989.

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ABSTRACT

The effectiveness of two common arrangements of overlapping hoops, the diamond-shaped and the rectangular overlapping hoops, is studied in relation to the single rectangular overlapping hoops arrangement. Short reinforced concrete columns are used for this purpose. The tests are divided into three parts.

In the first part, the relative effectiveness of the two arrangements of overlapping hoops to the single hoops arranagement is studied. In the second part, the diameter of hoops is changed and its effect on ductility studied. The third part deals with the study of the effect of concrete grade on ductility of confined concrete.

To determine the ductility of each arrangement, stress-strain curves are drawn. From theory, it is known that the ascending part of the curve has no effect on the ductility of confined concrete. But the angle of slope of the falling branch determines its ductility. This angle is therefore calculated and used as a measure of the ductility of each specimen. The smaller the angle, the more ductile is the confined concrete column.

The Universal Testing Machine is used to carry out these tests. Concrete Cube Strength Test and and Tensile Test for Steel are carried out. The strains are measured by Dial Gauge and Electrical Strain

SYMBOLS AND NOTATIONS

A_b - Cross-sectional area of bound or confined concrete

A_c - total cross-sectional area of concrete under compression.

A_s - total cross-sectional area of longitudinal reinforcement in tension.

A_{sn} - cross-sectional area of nth bar of longitudinal reinforcement in tension

A'_s - total cross-sectional area of longitudinal reinforcement in compression.

A''_s - total cross-sectional area of transverse reinforcement.

b - breadth of the specimen

b'' - breadth of the confined concrete cross-section

C_c - Resultant of Compressive force due to concrete

d - the effective depth of the column cross-section

d" - the depth of the confined concrete cross-section

d_n - the distance of the centre of .th bar from the neutral axis

f_c - the compressive stress in concrete

f'_c - the cube strength of concrete

f''_c - the maximum stress achieved in confined concrete $= Kf'_c$

f_s - the tensile stress in steel

f_{si} - the stress in the reinforcement bar

f_y - the yield strength of steel used

k - the ratio of neutral axis depth to the effective depth

K - the ratio by which the strength of concrete increased as a result of confinement

p - the theoritical force of resistance in the column

q" - a measure of the effectiveness of transverse reinforcement (hoops)

s - the spacing of hoops in the column

s_o - maximum spacing of hoops for which confinement is effective.

Z - a parameter used in determining the stress at any given strain in the Kent and Park Stress-Strain Curve.

α - ratio of effective concrete stress to the maximum concrete stress.

γ - ratio of the depth of the resultant compressive concrete force to the effective depth.

ϵ_c - the compressive strain in concrete

ϵ_{ce} - the compressive strain at which the maximum stress is just attrined

ϵ_{cf} - the strain at which the stress has fallen down to 85% of maximum stress in the curve proposed by Soliman and Yu (5).

ϵ_{cm} - the maximum extreme fibre compressive strain attained.

ϵ_{cs} - the strain at maximum stress just before the curve begins to fall in the Soliman and Yu curve

ϵ_{20c} - the confined concrete strain at which the stress has fallen down to 20% of maximum stress in the Kent and Park curve.

ϵ_{50c} - the confined concrete strain at which the stress has fallen down to 50% of maximum stress in the Kent and Park curve

ϵ_{50} - the unconfined concrete strain at 50% of maximum stress in the Kent and Park curve.

ϵ_{50h} - the difference in strain at 50% of maximum stress between the confined and unconfined concrete.

ρ - the ratio of the area of longitudinal reinforcement to the cross-sectional area of column

ρ'' - the volumetric ratio of hoops (transverse) reinforcement to the confined concrete.

CHAPTER ONE

INTRODUCTION

In the design of structures, many factors are taken into consideration. These include safety, economy and aesthetics, among others. To the Engineer, the primary criterion is safety. He is not only to ensure that the structure is safe for everyday use, but also gives adequate warning before collapse in the case of failure.

A structure is usually designed to withstand some perceived characteristic maximum load during its lifespan. To guard against overloading, the load is also multiplied by a factor of safety. Unfortunately, despite these precautions cases of collapse of structures do occur, either as a result of unforseen natural forces or when in some rare cases, the design load is exceeded. There is never a 100% guarantee that the best designed structure will reach its estimated lifespan. Thus, it is essential for the design engineer to see that in the event of failure, the structure does not collapse abruptly. He can do this by recommending the use of construction materials with adequate ductility.

One of the principal materials used for construction of structures is concrete - a composite of gravel or crushed rock, sand, cement and water. It does not possess the required ductility needed to give adequate warning in case of failure. As a remedy, the concrete is confined using transverse reinforcement in the form of spirals or ties. This transverse reinforcement also serves as the shear reinforcement in the structural members.

Previous works have shown that circular spirals are the best in enhancing the ductility of confined concrete. However, rectangular hoops (or stirrups)are more commonly used because they conform with the shape of most structural members. Single rectangular hoops are usually adequate for lightly loaded structures in non-earthquake regions. But when the structure is heavily loaded or is designed to withstand strong earthquakes, overlapping hoops will be required. A lot of research has been done on the single rectangular hoops, whereas little has been done in the case of rectangular overlapping hoops as pointed out by Park and Paulay [1].

It is therefore the object of this study to investigate the effectiveness of overlapping hoops relative to single hoops. To do this, two common arrangements of overlapping hoops are chosen. In addition, the influence of hoop diameter and strength of concrete on the ductility of confined concrete are also studied.

CHAPTER TWO

LITERATURE REVIEW

2.1 DUCTILITY OF CONCRETE

A concrete structure is said to be ductile when it takes considerable time to collapse even after the design load is reached. This is because the structure is able to transfer excess bending moment to other sections where the ultimate moment is not attained through the rotation of plastic hinges formed at this section. The structure collapses only when sufficient hinges are formed as to turn it into a mechanism.

To guard against sudden collapse of structures, most design codes consider the ductility of structures as an important design factor. This is particularly so in areas prone to earthquake where the structure is subjected to repeated seismic loading.

The ductility of a given concrete section is measured by the ratio of ultimate curvature to the curvature at yield of steel (Q_u/Q_y). Also, the angle of slope of the falling branch of the stress-strain curve (after reaching the maximum load) can be used as a measure of ductility. The smaller the angle, the more ductile is the material.

2.2 CONFINED CONCRETE

Concrete is said to be confined when it is restrained in the directions at right angle to the applied stress. Reinforced concrete may be confined by hoops, spirals or lateral reinforcement.

At stresses below the uniaxial compressive strength of concrete, the compressive stress is carried almost entirely by concrete. But as stresses approach the uniaxial strength, progressive internal cracking forces the core concrete to bear on the transverse reinforcement, which then applies a confining reaction to the concrete. As a result, the strength of core concrete increases beyond the uniaxial compressive strength. At higher strains the cover concrete, which cannot carry strains higher than that corresponding to the uniaxial strength, begins to spall and is considered not to contribute to either the strength or ductility of the confined concrete. Spalling is most rapid when the hoops are closely spaced since they provide a plane of weakness between the core and cover concrete.

Confinement is most effective in circular spirals because the axial hoop tension around

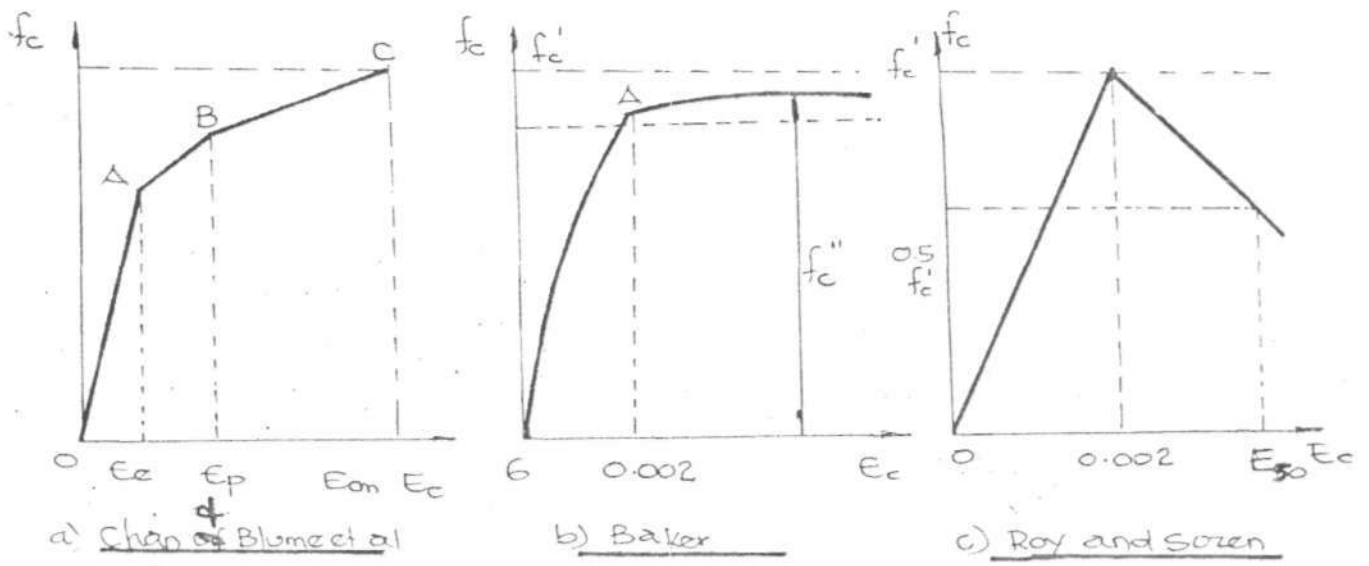
the circumference provides confining pressure. This pressure is, however, only present at the corners in case of rectangular hoops. The unsupported length of the hoops only contributes when the bar dimeter is thick enough to prevent buckling at high stresses.

2.3 Stress-Strain Curves for Concrete Confined by Rectangular Hoops

In order to determine the ductility of a given confined concrete member, it is necessary to know the stress-strain curve of confined concrete. Towards this end, various investigations have proposed stress-strain relationships for concrete confined by rectangular hoops. Some of the important ones are:

(a) Chan (2):- He propcsed a trilinear curve for confined concrete (see Fig. 1a). Region OAB is the same for both confined and unconfined concrete while region BC depends on the content of transverse steel. He considers only the effects of spacing and diameter of transverse steel before arriving at his curve. It therefore does not fit very well with experimental results. He assumes the outer fibre compressive strain at ultimate strain to be 0.0135.

Blume et al (3), also adopted a similar curve in which OA represents the curve for unconfined concrete up to $0.85f_c'$ and ABC depends on the transverse reinforcement.



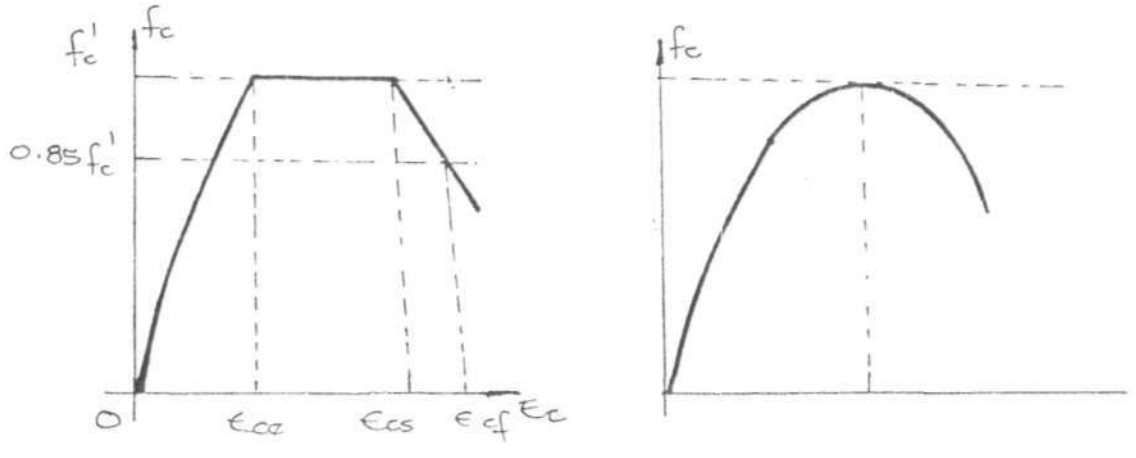


Fig. 1: Some Proposed Stress-Strain Curves for Confined Concrete

b) Baker and Amarakone (4):- The OAB section of Chan's [2] curve now becomes a parabola. This is followed by a plateau at constant stress. This stress has an upper boundary of f_c' and is dependent on the strain gradient and transverse reinforcement. They proposed the maximum compressive strain to be:

$$\epsilon_{cm} = 0.0015 \left[1+150\rho'' + (0.7+10\rho'') \frac{d}{kd} \right] \leqslant 0.01 \ldots \quad (1)$$

in which

ρ'' = volumetic ratio of transverse reinforcement to bound concrete.

d = effective depth of section

kd = neutial axis depth

However, they limited the maximum strain to 0.01; thus making it more conservative than Chan's approximation.

c) Soliman and Yu (5):- Their curve has three regions: a parabola upto a stess of f_c' and a strain of ϵ_{ce}, followed by a flat region at f_c' but with the strain increasing to E_{cs} and a final falling section which includes the point $(0.8f_c', \epsilon_{cf})$, up to the point of failure. ϵ_{ce} is given as:

$$\epsilon_{ce} = 2f_c'/E_c \quad \ldots \quad \ldots \quad (2)$$

and ϵ_{cs}, ϵ_{cf} depend on the spacing of binders.

E_c = concrete modulus of elasticity.

The maximum compressive strain is given as

$$\epsilon_{cu} = 0.003\ (1 + 0.8q'') \quad \ldots \quad \ldots \quad (3)$$

for which the quantity q" is a measure of the effectiveness of transverse reinforcement.

$$q'' = \left(1.4\ \frac{Ab}{A_c} - 0.45\right) \frac{A_s''\ (S_o - S)}{A_s'' + 0.0028BS^2} \quad \ldots \quad (4)$$

where A_b = area of bound concrete under compression

A_c = total area of concrete under compression

A_s'' = cross-sectional area of transverse steel

S_o = maximum longitudinal spacing after which transverse reinforcement is not effective in confining concrete.

S = actual longitudinal spacing of transverse reinforcement.

B = b" - breadth of confined concrete) whichever
or d" - effective depth of confined concrete) is greater

This ultimate strain was found to be closer to experimental results than those of Chan[2] and Baker (4).

d) Roy and Sozen[6]:- They suggested a curve in which the ascending branch is a straight line from the origin to a point with the stress f'_c and a strain of 0.002. The falling branch is also a straight line containing the point $(0.5f'_c, \epsilon_{5c})$. Contrary to previous investigators, they found out that the confinement of concrete does not increase its strength.

e) Sargin, Ghosh and Handa (7):- They propo. a continuous parabolic curve with a maximum stress at a value layer than f'_c.

2.4 The Kent and Park Stress-Strain Curve

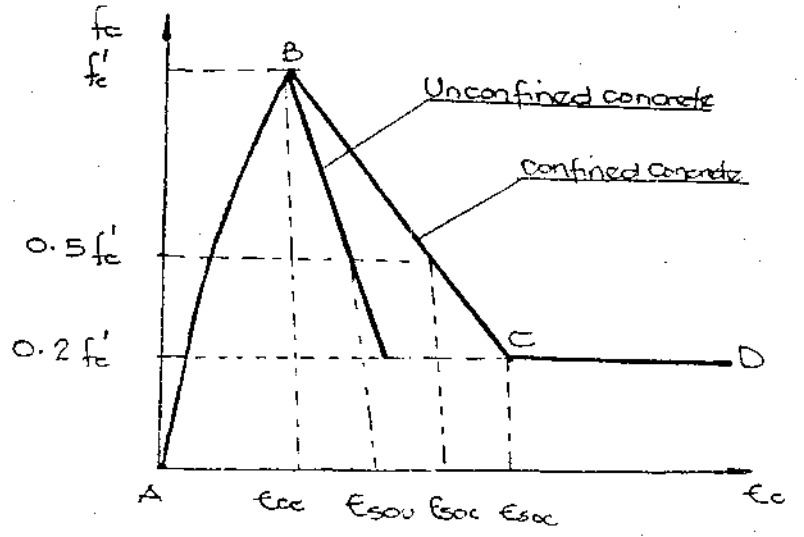


Fig. 2 Kent and Park Stress-Strain Curve

Kent and Park (8) have proposed a stress-strain curve for confined concrete (Fig.2) that contains most of the features of previously proposed curves. It has the following features:

Region AB: - This is an ascending second degree parabola, in common with curves proposed by Baker (4), Soliman and Yu (5) and Sargin et al (7). This curve is common to both the confined and the unconfined concrete. At the peak B, the stress is assumed to have reached f'_c and the strain to be = 0.002. Increase in strength due to confinement is ignored. The stress at any point is given by:

$$f_c = f'_c \left[\frac{2\epsilon_c}{\epsilon_{ce}} - \left(\frac{\epsilon_c}{\epsilon_{ce}} \right)^2 \right] \ldots \quad (5)$$

Region BC: - This is a falling straight line similar to that suggested by Roy and Sozen (6). Its slope is specified by the strain at which the maximum stress is reduced to half. For the unconfined concrete, this strain is

$$\epsilon_{50u} = \frac{3 + 0.002 f'_c}{f'_c - 1000} \quad \ldots \quad (6)$$

in which f_c' is measured in pounds per square inch (PSI). For the confined concrete, the strain is

$$\epsilon_{50c} = \epsilon_{50u} + \epsilon_{50h} \quad \ldots\ldots \quad (7)$$

where $\epsilon_{50h} = \frac{3}{4}\rho'' \sqrt{\frac{b''}{s}}$ ... ... (8)

Thus, equations 7 and 8 can be used to evaluate the confined strain, ϵ_{50c}. The equation for the confined concrete stress is

$$f_c = f_c' \left(1 - Z(\epsilon_c - \epsilon_{ce})\right) \quad \ldots (9)$$

where

$$Z = \frac{0.5}{\epsilon_{50u} + \epsilon_{50h} - \epsilon_{ce}} \quad \ldots (10)$$

The effect of strain gradient is ignored here.

Region CD:- In this region, the confined concrete is assumed to sustain a stress of $0.2f_c'$ from the strain ϵ_{20c} to the point of rapture. No such region exists in unconfined concrete.

2.5 The Modified Kent and Park Stress-Strain Curve

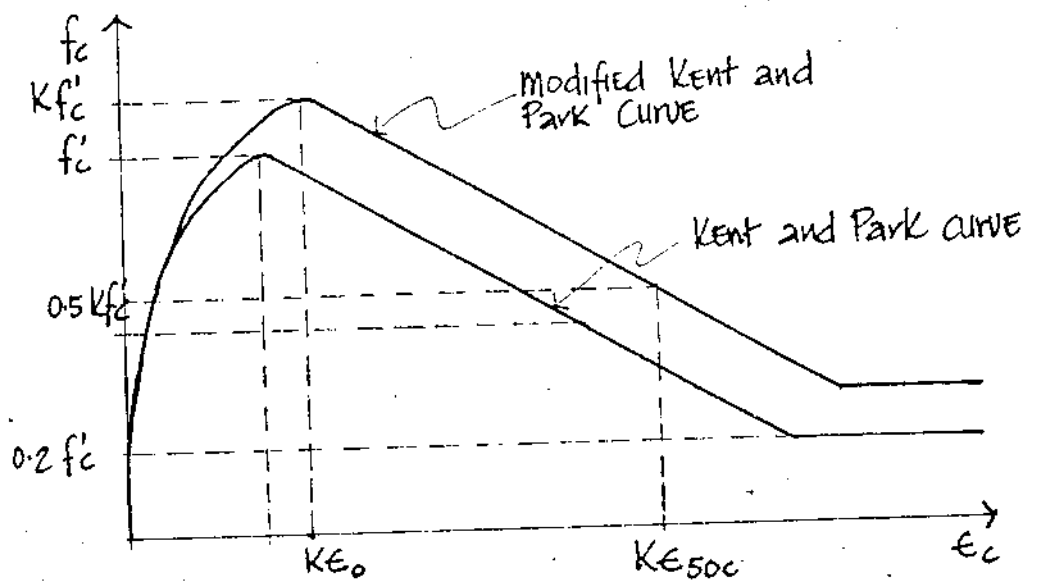


Fig. 3: The Modified Kent and Park Curve

The Kent and Park stress-strain curve did not take into consideration the increase in concrete strength due to confinement. To redress this shortcoming, Scott et al (9) proposed that the cylinder stress be multiplied by a factor K.

$$\text{where } K = 1 + \rho'' \frac{f_y}{f'_c} \quad \ldots\ldots \quad (11)$$

Thus for $\epsilon_c \leq 0.002\ K$,

$$f_c = Kf'_c \left[\frac{2\epsilon_c}{0.002} - \left(\frac{\epsilon_c}{0.002}\right)^2\right] \quad \ldots \quad (12)$$

with f'_c and fy in MP_a.

For $\epsilon_c \geq 0.002\,K$

$$f_c = K f'_c \left[1 - Z\,(\epsilon_c - 0.002k)\right] \quad \ldots \quad (13)$$

where

$$Z = \frac{0.5}{\frac{3+0.29f'_c}{145f'_c - 1000} + \frac{3}{4}\rho''\sqrt{\frac{b''}{s}} - 0.002K} \quad \ldots \quad (14)$$

Equations 11 to 14 are applicable only for low rates of strain application. The authors also suggested that the maximum concrete compressive strain at ultimate be given by

$$\epsilon_{cu} = 0.004 + 0.9\,\rho''\,\frac{fy}{300} \quad \ldots \ldots \qquad (15)$$

with fy in MP_a.

This curve was found to be more in agreement with experimental results than the original Kent and Park curve.

2.6 Factors affecting The Strain-Strain Curve of Confined Concrete

Based on the result of previous investigators, Park and Paulay (1) have identified the following factors as affecting the behaviour of c stress-strain curve of confined concrete.

The ratio of transverse reinforcement to the volume of concrete core (ρ''). A high ratio of transverse steel gives high confining pressure. This is the most important factor identified by most investigators.

2. Size of transverse reinforcement - For high cross-sectional dimensions, small diameter bars merely act as ties between corners. When the strain is high, they buckle out rather than confine the concrete. However, according to Soliman and Yu (5) this factor is of less importance than the spacing of transverse reinforcement.

3. Spacing of transverse reinforcement to the dimension of core concrete (S/b'' or S/d'') - As the spacing is reduced, concrete is confined more effectively. For widely spaced binders, the effect is minimal.

4. Yield strength of transverse steel - this gives an upper limit to the confining pressure. However, due to strain-hardening, the steel does not fail immediately after yielding. Mild steel provides a larger elongation for strain hardening than high yield steel, which is more brittle.

5. Content of longitudinal steel - confinement by longitudinal steel is enhanced by smaller spacing and larger diameter of the bars. However, for effective confinement, the longitudinal bars must be placed tightly against the transverse hoops that prevent them from buckling.

6. Strength of concrete - Ductility reduces with increase in strength of concrete.

7. Rate of loading - Since both stress and strain vary with time, the rate at which load is applied affects the shape of the stress-strain curve.

2.7 THE STRESS BLOCK PARAMETERS

Fig. 4 shows the possible concrete compressive stress blocks for the stress-strain curve as given by Park and Kent (8). The concrete compressive force is

$$C_c = \alpha f''_c \, bkd \quad \ldots \quad \ldots \quad (16)$$

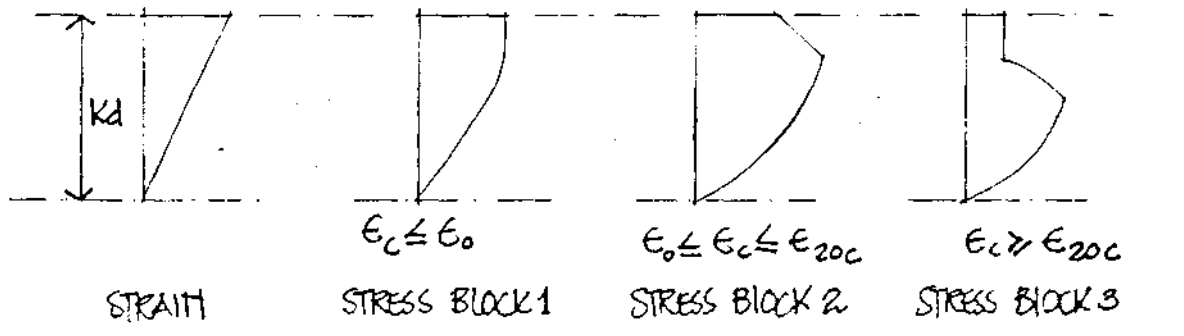
and is acting at a distance γkd from the extreme compression fibre.

f''_c = maximum stress in the stress-strain curve

$= \alpha f'_c$

kd = neutral axis depth

b = breadth of the effective cross section



α and γ are known as the stress block parameters. They determine the magnitude and position of the compressive concrete force.

$$\alpha = \frac{\int_0^{\epsilon_{cm}} f_c \, d\epsilon_c}{f_c' \epsilon_{cm}} \qquad \ldots \quad \ldots \quad (17)$$

$$= 1 - \frac{\int_0^{\epsilon_{cm}} \epsilon_c \, f_c \, d\epsilon_c}{\epsilon_{cm} \int_0^{\epsilon_{cm}} f_c \, d\epsilon_c} \qquad \ldots \ldots \quad (18)$$

Values of α and γ have been tabulated by Kent and Park (8) for different values of Z and ϵ_{cu} using the original Kent and Park Stress-Strain Curve. For the Modified Kent and Park Curve, the values should be evaluated using equations 11 to 15 as well as 17 and 18.

2.4 THE THEORITICAL LOAD AND MOMENT

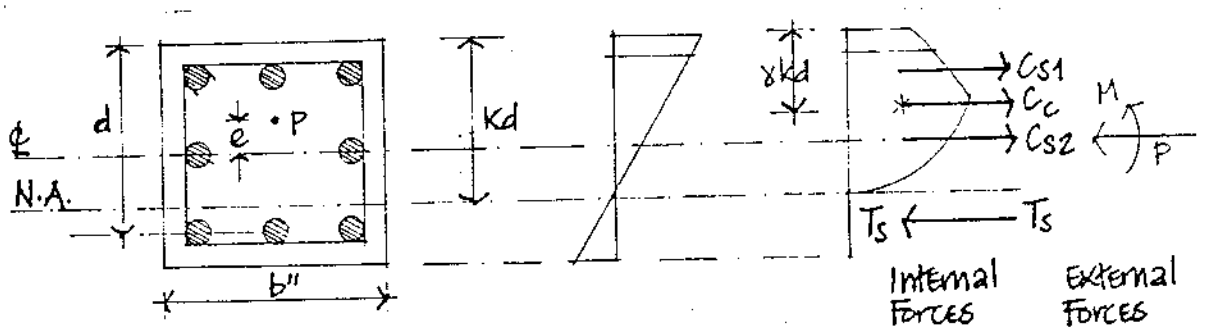


FIG. 5
CROSS SECTIONAL PROPERTIES OF THE COLUMN

The force equilibrium equations for an eccentrically loaded column as shown in Fig. 5 are:

$$P = \alpha f_c'' \, bkd + \sum_{i=1}^{n} f_{si} \, A_{si} \quad \ldots\ldots \quad (19)$$

$$\text{and } M = \alpha f_c'' \, bkd \, (h/2 - \gamma kd) + \sum_{i=1}^{n} f_{si} \, A_{si} \, d_i \ldots \quad (20)$$

where

A_{si} denotes the cross-sectional area of each of the n number of bars in compression and d_i is the corresponding distance from the centre of the bar to the centreline of the column cross-section.

CHAPTER 3

EXPERIMENTAL PROGRAMME

3.1 General

Twenty-one (21) short columns of the size 150 x 150 x 750mm were tested. The tests were divided into three parts.

In the first part, the aim was to find out the relative ductility of different arrangements of overlapping hoops in comparison to single rectangular hoops. For this, three basic arrangements were chosen and numbered A1 to A3. They are the diamond-shaped overlapping hoops, the single rectangular hoops and the rectangular overlapping hoops. For each arrangement, three columns with identical properties were prepared and tested to ensure consistency. Details are provided in Table 1.

In the second part of the test, the effect of hoop diameter on ductility was studied. To do this, the diamond-shaped overlapping hoops and single rectangular hoops were chosen. The hoop diameter was changed from 6mm to 4mm. All other properties were kept constant. As in the first part, three identical columns were produced for each hoop

TABLE 1: DETAILS OF TEST SPECIMEN

CONSTANTS:

Volumetric ratio of hoop reinforcement, $\rho'' = 0.0161$

Ratio of main longitudinal reinforcement, $\rho = 0.0179$

Average Yield strength of reinforcement, steel, $f_y = 301.73 MN/m^2$

Dimensions of specimens = 150x150x750mm

Cover to e reinforcement = 10mm

Area of core concrete = 14,884mm^2

UNIT NUMBER	ARRANGEMENT	LONGITUDINAL STEEL			TRANSVERSE STEEL		DESIGN CONCRETE CUBE STRENGTH (MN/m^2)
		No. of Bars	Diameter mm	Area mm^2	Diameter mm	Spacing mm	
A1		8	8	402.124	6	100	25
A2		8	8	402.124	6	90	25
A3		8	8	402.124	6	58	25
B1		8	8	402.124	4	44	25
B2		8	8	402.124	4	26	25
C1		8	8	402.124	6	100	20
C2		8	8	402.124	6	58	20

arrangement. The arrangements were numbered B1 and B2.

The third part of the experiment entails the study of the effect of concrete grade on the ductility of overlapping hoops. This involved changing the concrete strength from 25 to 20 MN/m^2 for both the diamond-shaped and single rectangular arrangements. All other factors remained constant as in the first part. Here also, three columns were produced for each arrangement and the arrangements were marked C1 and C2 respectively.

To be able to identify the columns after casting, steel-plate tags were tied to the main reinforcement before casting. The three identical columns for each specimen were numbered 1 to 3. Thus, for the diamond-shaped overlapping hoops arrangement in the first test, the columns were tagged A11, A12 and A13. Similarly for all other specimens.

It may be noted that the selection of hoop arrangement, bar diameter and concrete grade is in no way exhaustive. The choice was limited by considerations of finance and time. Similarly, the limitation of the study to only three factors was informed by the same factors.

3.2 MATERIALS

In the first and second part of the experiment, the concrete was designed to provide a nominal strength of 25 MN/m^2 after 28 days. In the third part, the design concrete strength was 20 MN/m^2

Crushed stones aggregates with a maximum size of 10mm, together with ordinary Portland cement and river-bed sand were used in mixing the concrete. 8mm mild steel bars were used as main reinforcement while the hoop reinforcement consisted of 6mm and 4mm mild steel rods.

3.3 INSTRUMENTATION

Electrical resistance gauges were tied to the main reinforcement for the measurement of strain in the reinforcement. Due to the shortage of these gauges, only two were used per column; one at the top, the other at the bottom see Fig. 6 for detail. Dial gauges were also assembled to record the concrete strains at the extreme fibres. This was in addition to the automatic plotting of the Load-Strain diagram by the compression machine used.

3.4 PREPARATION OF SPECIMENS

The columns were cast in horizontal position using partitioned timber formwork prepared for eight (8) columns. Before this, the reinforcement

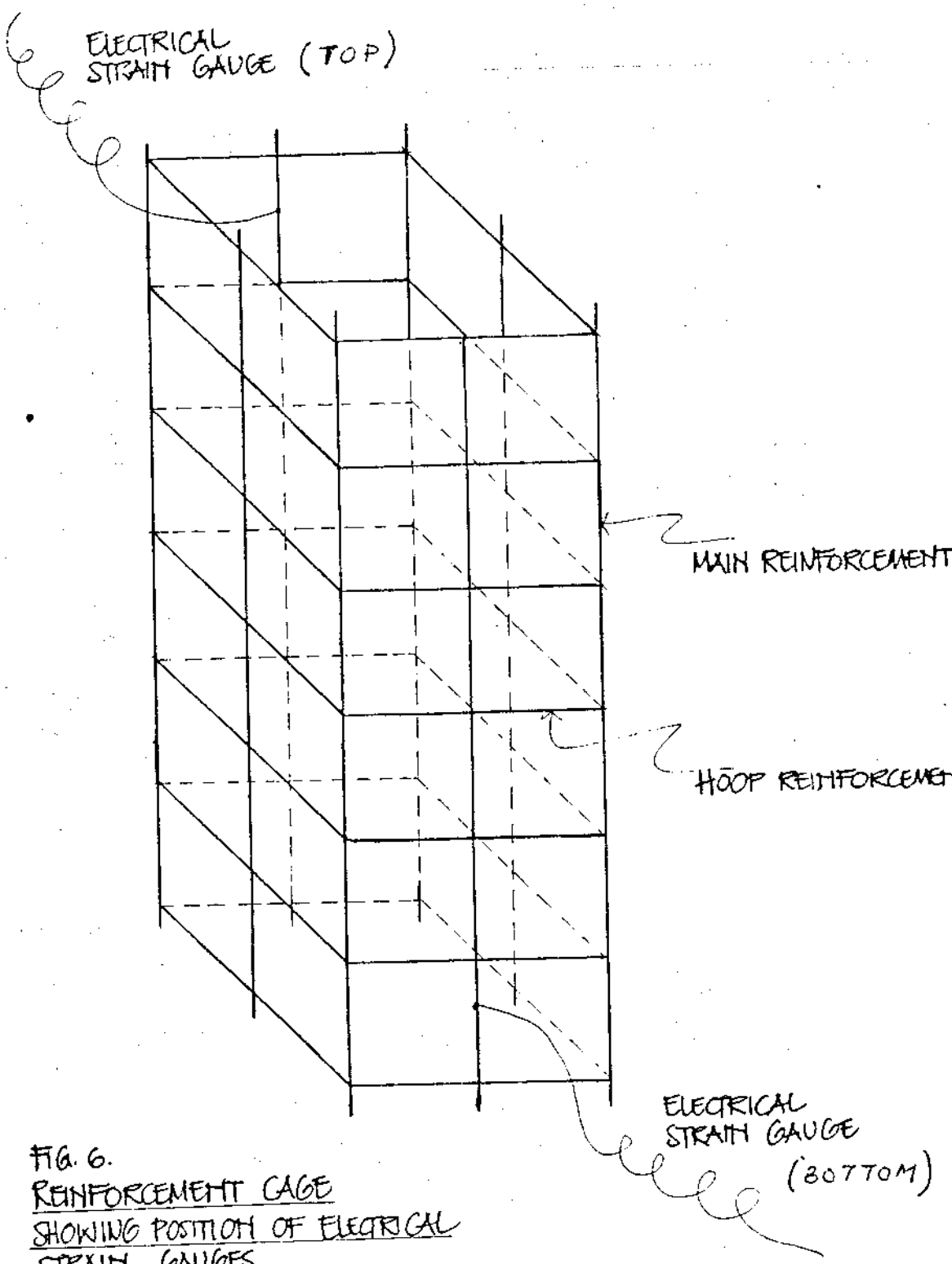


FIG. 6.
REINFORCEMENT CAGE
SHOWING POSITION OF ELECTRICAL
STRAIN GAUGES

cages were prepared separately and labelled. The electrical resistance guages were also attached to the reinforcement. Flat crushed stone aggregates about 10mm thick were then used as biscuits at the bottom and sides so as to provide the required concrete cover of 10mm. The concrete mix was then designed and the designed proportions mixed in a rotary concrete mixer. The mixing was done in three parts of two batches each. In the first part, the design strength was $25MN/m^2$ and enough mix for 8 columns and 4 cubes were prepared and cast. All the columns were among those in the A-series. The last column in this series was cast the following day, together with the 6 columns in the B-series. Finally, the 6 columns in the C-series were cast on the third day with a concrete of design strength of $20\ MN/m^2$. A slump test was carried out to ensure that the water content was within the specified limits. The concrete was then compacted using a vibrating table. The casted columns, together with the concrete cubes prepared were left untouched for 24 hours on the vibrating table. Thereafter, the formwork was removed and the columns taken into the curing room. After 28 days, the column were brought out of the curing room, painted and prepared for testing.

3.5 TEST PROCEDURE

The columns were placed vertically in the Universal Testing Machine. Two dial guages were placed 168cm from the centreline of the column, each on either side of the column as shown in Fig.7. The expected failure load was calculated from the Modified Kent and Park stress-strain equations. Compression load was then applied instalmentally at 50%, 70%, 80%, 90% of this load and the loading was continued until failure was achieved. At each interval, the dial gaugesreadings as well as the strain gaugesreadings were noted.

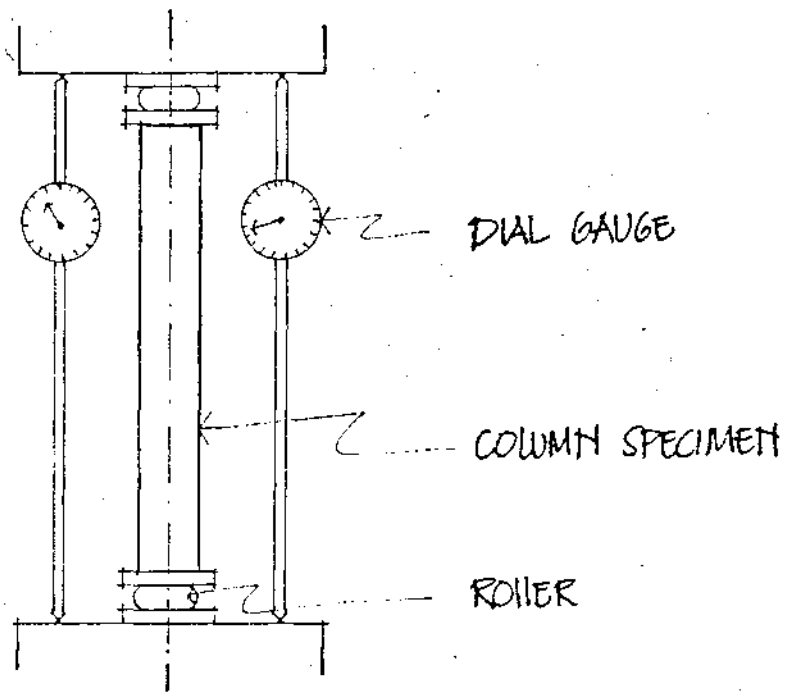


Fig. 7: The Set-up Before Testing

Also, the gauges and load readings at the appearance of the first crack and at failure were recorded. In addition, the automatic graph plotter of the Testing Machine was used to plot the Load-strain curves for each column.

CHAPTER FOUR

OBSERVATIONS AND RESULTS

4.1 OBSERVATIONS

Originally, the tests were to be conducted with the applied load at a uniaxial eccentricity of 10mm. But when one of the columns, A12 was tested using this eccentric loading, the mode of failure was not only one-sided but the column only failed at the top-most 10cm. At this time, only about 50% of the design load was reached. In addition, the column crushed down immediately after failure, almost causing an accident. Since it was observed that very little meaningful data would be obtained by this mode of loading, it was decided to load all other columns axially.

The point of intersection of the lines drawn from one corner to the fartherest corner was taken as the central point of loading. This approximate method was used since it was almost impossible to locate the exact centre of the core concrete. During the test, it was observed in virtually all the specimens that there was a difference in dial gauges readings between the right-hand side and the left-hand side dial gauges. See results in Table 2. The average strain was taken as the strain in the column.

TABLE 2: DIAL GAUGE READINGS

*Multiplying factor = 0.01mm

COLUMN IDENTITY	POSITION	APPLIED LOAD						FAILURE LOAD IN kN
		100KN	200kN	280kN	320kN	360kN	400kN	
A11	LEFT	145	230	305	355	• 420	-	352
	RIGHT	145	220	280	321	• 410		
A13	LEFT	127	196	282	328	382		360
	RIGHT	141	155	251	311	378	-	
A21	LEFT	75	118	147	167	187	220	400
	RIGHT	85	141	175	218	242	280	
A22	LEFT	133	163	196	216	241	266	423
	RIGHT	109	171	211	234	271	293	
A23	LEFT	171	222	261	281	305	331	400
	RIGHT	176	232	278	305	335	376	
A31	LEFT	73	119	167	200	-	-	320
	RIGHT	80	146	202	248			
A32	LEFT	121	198	233	-	-	-	314
	RIGHT	108	185	244				
A33	LEFT	128	183	248	-	-	-	354
	RIGHT	136	198	274	-	-	-	

*reading at failure load

Table 2: DIAL GAUGES READINGS (contd.

COLUMN IDENTITY	POSITION	APPLIED LOAD 100 kN	200kN	280 kN	320kN	360kN	400kN	440kN	FAILURE LOAD
B11	LEFT	203	262	311	339	373	-	-	382
	RIGHT	174	245	302	330	370			
B12	LEFT	163	221	266	300	348			
	RIGHT	183	257	310	347	397	-	-	396
B13	LEFT	117	237	346	-	-	-	-	313
	RIGHT	134	268	380					
B21	LEFT	115	199	250	303	-	-	-	336
	RIGHT	113	198	254	308				
B22	LEFT	106	180	228	252	297	-	-	386
	RIGHT	90	168	216	242	294			
B23	LEFT	158	267	333	363	404			
	RIGHT	165	275	347	382	427	-	-	390
C11	LEFT	93	287	389	454				
	RIGHT	93	286	403	495	-	-	-	320

Table 2: DIAL GAUGESREADINGS (contd.)

COLUMN IDENTITY	POSITION	APPLIED LOAD 100kN	200kN	280kN	320kN	360kN	400kN	440kN	FAILURE LOAD
C12	LEFT	149	217	269	293	318	350	370	
	RIGHT	157	221	269	291	317	348	391	460
C13	LEFT	65	209	271	293	330	–	–	
	RIGHT	88	242	307	341	381	432	–	406
C21	LEFT	122	189	229	251	277	313	356	
	RIGHT	123	209	261	287	315	353	395	476
C22	LEFT	91	157	200	225	257			
	RIGHT	150	219	271	310	350	–	–	360
C23	LEFT	105	120	190	240	280			
	RIGHT	130	175	250	306	355	–	–	360

TABLE 3: STRAIN GAUGE READINGS

*Straing guage factor = 2×10^{-6}

+where no result was shown, the gauge did not function

COLUMN IDENTITY	POSITION	APPLIED LOAD 100kN	200kN	280kN	320kN	360kN	400kN	FAILURE LOAD in kN
A11	TOP	920	995	1058	1034	-	-	352 C
	BOTTOM	1044	1020	992	985	-	-	T
A13	TOP	889	878	891	900	945	-	C
	BOTTOM	1108	1148	1105	1081	-	-	360
A21	TOP	843	869	881	768	765	1123	C
	BOTTOM	1103	1100	1093	1086	1081	-	400 T
A22	TOP	985	1042	1120	1171	1185	1143	C
	BOTTOM	1212	1200	1189	1186	1185	1188	423 T
A23	TOP	-	-	-	-	-	-	-
	BOTTOM	1268	1262	1251	1241	1242	-	400 T
A31	TOP	-	-	-	-	-	-	-
	BOTTOM	935	915	897	-	-	-	320 T

Note: C - Compression
T - Tension

TABLE 3: STRAIN GAUGE READINGS (Contd.

COLUMN IDENTITY	POSITION	APPLIED LOAD 100kN	200kN	280kN	320kN	360kN	400kN	Failure load in kN
	TOP	-	-	-	-	-	-	-
A32	BOTTOM	1161	1140	1128	-	-	-	314 T
	TOP	962	1051	1170	-	-	-	C
A33	BOTTOM	-	-	-		-	-	354 T
	TOP	-	-	-	-	-	-	-
B11	BOTTOM	802	708	-	-	-	-	382 T
	TOP	1004	1080	1069	-	-	-	C
B12	BOTTOM	998	1009	-	-	-	-	396 T
	TOP	843	842	-	-	-	-	T
B13	BOTTOM	878	860	846	-	-	-	313 T
	TOP	791	771	728	-	-	-	T
B21	BOTTOM	-	-	-	-	-	-	336 -
B22	TOP	1161	1188	1261	1299	1343	-	C
	BOTTOM	1331	1327	1339	1342	-	-	386 C

TABLE 3: STRAIN GAUGE READINGS (contd.)

COLUMN IDENTITY	POSITION	APPLIED LOAD 100 kN	200kN	280kN	320kN	360kN	400kN	Failure load in kN	
B23	TOP	1034	1007	952	962	923	-		T
	BOTTOM	1108	1095	1085	1077	1079	-	390	T
C11	TOP	1148	1141	1149	1220	-	-		T
	BOTTOM	906	881	871	-	-	-	320	T
C12	TOP	-	-	-	-	-	-		-
	BOTTOM	754	720	677	658	660	-	460	T
C13	TOP	993	982	974	964	966	1033		C
	BOTTOM	1259	1255	1248	1254	1266	1268	406	C
C21	TOP	1033	1009	993	1002	1020	1052		C
	BOTTOM	1182	1190	1177	1167	1154	1146	476	T
C22	TOP	1200	1292	1285	1287	1298	-		C
	BOTTOM	-	-	-	-	-	-	360	-
C23	TOP	634	621	605	597	664	-		C
	BOTTOM	540	576	647	656	-	-	360	T

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It was also observed that the dial gauge pointer moved very fast as the failure load was approached to the extent that it became impossible to record the dial gauge reading at failure.

The strain gauge readings were obtained using a Linear strain meter and recorded in Table 3. In almost all the cases, 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.

In the columns with rectangular overlapping hoops (A2), it took a higher load for the steel to yield than in other columns. Some of the columns started to fail from the top while others began failing from the bottom. No column was able to carry higher load after spalling of concrete began.

4.2 POSSIBLE SOURCES OF ERRORS

1. Size of Columns - The size of columns casted was much smaller than normal. There was an apparent difficulty in bending the reinforcement to exact size. This increased the margin of error.

2. Casting of columns - Although care was taken to ensure quality control, some lapses did occur during casting. Some of these include the shifting of the reinforcement cage while vibrating the concrete and the tendency for the formwork to bulge even after bracing. It was in an effort to reduce these lapsed that the columns were casted horizontally.

3. Application of Load - The cross-sectional area of the column on which the compressive load was applied was uneven. Thus the initial measurements of strain were much higher than the actual.

4. Instrumental Errors - The gauges used in measuring strain as well as the load meter attached to the Universal Testing Machine were all prone to malfunction. In fact some of the strain gauges malfunctioned while some other did not function at all.

5. Human Errors - These include human errors in measurement and the inability to take down the exact readings at the right time.

TABLE 4: *RESULTS OF CONCRETE CUBE STRENGTH TESTS*

FIRST BATCH - Design Strength = 25MN/m^2

Cube No.	Weight, kg	Crushing Load kN	Strength MN/m^2
A1	7.85	530	23.6
A2	7.40	500	22.2
A3	7.55	520	23.1

Average = 23.0 MN/m^2

SECOND BATCH - Design Strength = 25 MN/m^2

Cube No.	Weight kg.	Crushing Load kN	Strength MN/m^2
B3	7.58	440	19.56
B2	7.58	480	21.33
B1	7.60	510	22.67

Average = 21.197 MN/m^2

THIRD BATCH - Design Strength = 20 MN/m^2

Cube No	Weight,kg.	Crushing Load kN	Strength MN/m^2
C1	7.70	450	20.00
C2	7.80	440	19.56
C3	7.70	440	19.56

Average = 19.71 MN/m^2

TABLE 5: RESULTS OF THE TENSILE TEST FOR STEEL

Bar Identify	Diameter, mm	Yield load kN	Failure load kN	A_s mm^2	f_y MN/m^2
A	8mm	14.7	25.2	50.265	292.45
B	"	14.7	23.8	50.265	292.45
C	"	16.1	24.4	50.265	320.30
				Average =	301.73 MN/m^2
D	6mm	7.6	8.9	28.274	268.80
E	"	7.0	9.0	28.274	247.58
F	"	7.0	9.1	28.274	247.58
				Average=	254.65 MN/m^2
G	4mm	10.0	10.00	12.266	815.26
H	"	10.4	11.0	12.266	847.89
I	"	10.5	10.5	12.666	815.26
				Average =	826.14 MN/m^2

4.3 DISCUSSION OF RESULTS

4.3.1 General

Tables 6 and 7 are the calculated average strains from the dial guage and strains gauge readings respectively. It is decided to take the average strains since it is assumed that the load was acting axially.

In Table 7, there is a marked difference between the strain at the top and that at the bottom of the column. As the loading increases, either the top strain increases while the bottom diminishes or vice-versa. By the time the steel starts yielding, the gap between the top and bottom strains has been considerably reduced. The initial difference may be due to the uneveness of the column surfaces being compressed; thus leading to uneven pressure. The strain values from electrical strain gauge readings terminate at the yield of steel. They seldom exceed 0.00250. They are therefore not reliable for use in evaluating the performance of the columns.

TABLE 6 - AVERAGE STRAIN VALUES FROM DIAL GAUGE READINGS

COLUMN	100 kN	200 kN	280 kN	320kN	360 kN	400 kN	440 kN	failure Load (expt.)	Failure Load (Calc.
A11	0.00191	0.00300	0.00390	0.00450	0.00486	-	-	352	468
A13	0.00179	0.00300	0.00422	0.00479	0.00560	-	-	360	497
A21	0.00106	0.00173	0.00215	0.00257	0.00286	0.00333	-	400	515
A22	0.00148	0.00223	0.00271	0.00300	0.00339	0.00373	-	423	546
A23	0.00232	0.00303	0.00360	0.00391	0.00435	0.00471	-	400	513
A31	0.00102	0.00177	0.00246	0.00299	-	-	-	320	435
A32	0.00106	0.00209	0.00271	-	-	-	-	314	429
A33	0.00176	0.00254	0.00348	0.00381	0.00447	-	-	354	466
B11	0.00218	0.00305	0.00376	0.00413	0.00462	-	-	382	497
B12	0.00231	0.00319	0.00384	0.00446	0.00495	-	-	396	510
B13	0.00168	0.00337	0.00484	-	-	-	-	313	451

TABLE 6: AVERAGE STRAIN VALUES FROM DIAL GAUGE READINGS (CONTD.)

COLUMN	100 kN	200 kN	280 kN	320 kN	360 kN	400 kN	440 kN	FAILURE LOAD (EXPT.)	FAILURE LOAD (CALC.)
B21	0.00152	0.00290	0.00400	0.00408	-	-	-	336	429
B22	0.00124	0.00232	0.00296	0.00330	0.00394	-	-	386	499
B23	0.00216	0.00362	0.00454	0.00497	0.00554	-	-	390	506
C11	0.00124	0.00382	0.00528	0.00633	-	-	-	320	431
C12	0.00204	0.00292	0.00359	0.00390	0.00424	0.00466	0.00507	460	572
C13	0.00102	0.00298	0.00385	0.00423	0.00474	-	-	406	519
C21	0.00164	0.00266	0.00327	0.00359	0.00395	0.00444	0.00501	476	585
C22	0.00161	0.00251	0.00314	0.00357	0.00405	-	-	360	497
C23	0.00157	0.00364	0.00360	0.00437	0.00497	-	-	360	497

TABLE 7: STRAIN VALUES FROM ELECTRICAL STRAIN GAUGE READINGS

COLUMN IDENTITY	100 Kn	200kN	280 kN	320 kN	360 kN	400 kN	440 kN	Failure Load, kN	
	0.00184	0.00199	0.00212	0.00207					C
A11	0.00208	0.00204	0.00198	0.00197				352	T
	0.00178	0.00176	0.00178	0.00180	0.00189			360	C
A13	0.00222	0.00230	0.00221	0.00216	-				T
	0.00169	0.00194	0.00176	0.00154	0.00153	0.00225			C
A21	0.00221	0.00220	0.00219	0.00217	0.00216	-		400	T
	0.00197	0.00208	0.00224	0.00234	0.00237	0.00229			C
A22	0.00242	0.00240	0.00238	0.00237	0.00237	0.00238		423	T
	-	-	-	-	-	-			-
A23	0.00254	0.00252	0.00250	0.00248	0.00248			400	T
	-	-	-						-
A31	0.00187	0.00183	0.00179					320	T
	-	-	-						-
A32	0.00232	0.00228	0.00226					314	T
	0.00192	0.00210	0.00234					354	C
A33	-	-	-						-

TABLE 7: STRAIN VALUES FROM ELECTRICAL STRAIN GAUGE READINGS (CONTD.)

COLUMN IDENTITY	100 kN	200 kN	280 kN	320 kN	360 kN	400 kN	440 kN	FAILURE LOAD, kn	
B11	– 0.00160	– 0.00142	–					– 382	 T
B12	0.00201 0.00200	0.00216 0.00202	0.00214 –					396	C T
B13	0.00169 0.00176	0.00168 0.00172	– 0.00169					313	C T
B21	0.00158 –	0.00254 –	0.00146 –					336	T –
B22	0.00232 0.00266	0.00238 0.00265	0.00252 0.00268	0.00260 0.00268	0.00269 –			386	C T
B23	0.00207 0.00222	0.00201 0.00219	0.00190 0.00217	0.00192 0.00215	0.00216			390	C T
C11	0.00230 0.00181	0.00288 0.00176	0.00230 0.00174	0.00244 –				320	C T
C12	– 0.00151	– 0.00144	– 0.00135	– 0.00132	– 0.00132			460	– T
C13	0.00199 0.00252	0.00198 0.00251	0.00195 0.00250	0.00193 0.00251	0.00193 0.00253	0.00207 0.00254		406	C T
C21	0.00207 0.00236	0.00202 0.00238	0.00199 0.00235	0.00200 0.00233	0.00204 0.00231	0.00210 0.00229		476	C T
C22	0.00240 –	0.00258 –	0.00257 –	0.00257 –	0.00260 –	–		360	C –
C23	0.00127 0.00108	0.00124 0.00115	0.00121 0.00129	0.00119 0.00131	0.00133 –			360	C T

4.3.2 Load-Carrying Capacity - It can be deduced from Table 6 that the overlapping hoops arrangement has higher load-carrying capacity than others. This is followed by the diamond-shaped overlapping hoops arrangement and lastly, the rectangular single hoops arrangement. This can be explained by the fact that the area of the largest loop formed by the hoops is smallest in the rectangular overlapping hoops and highest in the rectangular single hoops. This is further explained in Fig. 8

Also, the results have shown that the columns with 4mm hoops diameter (B series) carry more load than those of similar arrangement but with 6mm hoops diameter. This is because the hoops are more closed spaced in the B-series.



(i) DIAMOND-SHAPED OVERLAPPING (ii) RECTANGULAR OVERLAPPING (iii) RECTANGULAR SINGLE

* SHADED AREAS SHOW THE LARGEST LOOP IN EACH ARRANGEMENT

FIG. 8
COMPARISON OF LARGEST LOOPS IN THE HOOP ARRANGEMENTS

4.3.3 Strain Values - Also, looking at Table 6, one realises that of the three arrangements, the A1-series (diamond-shaped) provides the highest values of strain at failure. This is followed by the rectangular overlapping hoops (A2) and finally, the rectangular single hoops (A3). This suggest that the concrete is able to bear more strain when closely-spaced hoops are used despite the larger diameter of the widely spaced hoops. Similar conclusion can be drawn by comparing columns B1 and B2 as well as C1 and C2.

Comparing the strain values in columns with similar arrangements but different hoops diameter or concrete grade (e.g A1, B1 and C1), it is observed that no clear trend is manifested. This may be because the advantage offered by closer spacing is more or less neutralised by the smaller diameter of bars or weakness of the concrete used.

4.3.4 Ductility - To be able to study the relative ductility of each column, the stress-strain curve is drawn using strain values from dial gauge readings for the ascending portion. These are found in Annex B. For the descending portion, the only alternative was to fall back on the automatically plotted load-strain curves of the Universal Testing Machine. Since the test was terminated at different

stages for each column, it was decided to terminate the descending curve when it comes down to half the maximum stress recorded. As has been explained earlier, the test was not continued until the core concrete disintegrated in order to avoid nasty accidents. It is also realised from the first column tested (A12), that the horizontal portion of the Modified Kent and Park Stress-Strain Curve, may not be very long or may even be nonexistent.

In the absence of the horizontal branch, we may concentrate our studies of ductility on the falling branch. Table 8 shows the range of strains for the stress in the column to fall down to half from the maximum value. The angle of slope of the falling branch is taken as a measure of ductility; the smaller the value, the higher is the ductility of the column. It is pertinent to note that the following columns had no falling branches: A22, A23 and A31. A possible explanation for this is that the operator of the testing machine may have taken off the load from the column immediately after failure.

A close look at Table 8 shows that of the three arrangements, the diamond-shaped overlapping hoops offers the most ductility. The least is the rectangular overlapping hoops arrangement. However, this may be because the falling branches of two out of the three columns in this series are not recorded.

TABLE 8: RELATIVE DUCTILITY OF THE COLUMNS

Column	Maximum stress, MN/m^2	Strain at max. stress	Strain at 50% max. stress	Angle of slope of falling branch	Average value of angle $x10^3 MN/m^2$
A11	15.70	0.00510	0.00660	79.18°	69.38°
A13	17.00	0.00670	0.01210	59.57°	
A21	17.80	0.00400	0.00570	79.19°	79.19°
A22	19.20	0.00430	0.00430		
A23	17.70	0.00430	0.00430		
A31	14.20	0.00280	0.00280		
A32	13.90	0.00380	0.00530	77.82°	73.39°
A33	15.60	0.00300	0.00600	68.96°	
B11	17.00	0.00530	0.00700	78.69	
B12	17.60	0.00530	0.00870	68.89	69.98°
B13	14.90	0.00570	0.00960	62.37	
B21	13.90	0.00260	0.00550	67.38	
B22	17.10	0.00500	0.00970	61.21	68.65°
B23	17.40	0.00540	0.00930	77.36	
C11	14.00	0.00530	0.01000	56.13	
C12	20.40	0.00450	0.01030	60.40	65.48°
C13	18.00	0.00760	0.00920	79.91	
C21	21.00	0.00600	0.00850	76.61	
C22	17.00	0.00410	0.00700	71.16	77.24°
C23	17.00	0.00780	0.00870	83.95	

It may be noted that the average angle of slope in the A-series is higher than the corresponding value in the B-series (compare A1 a with B1 and A3 with B2). It can therefore be concluded that reducing the hoops diameter while maintaining the volumetric hoop reinforcement ratio increases the ductivlity of the column. In other words, the spacing of hoops has to be reduced to accommodate the same volumetric ratio.

To determine how the grade of concrete affects ductility comparison is made between A1 and C2 and A3 and C2. In the first case, it is clear that the average angle of slope is lower in the C1 series than in the A1. However, this is not the case between C2 and A3 due largely to the unusually high value of the C3 column. In fact, minus the 3 columns which had no falling branches, this column has the highest angle. If this column is therefore discarded as faulty, the average value for C2 comes down to 73.88°, which is almost the same ast the 73.39° recorded for A3. We can therefore conclude from the comparison of A1 and C1 that reducing the grade of concrete enhances ductility.

CHAPTER FIVE

CONCLUSION

This project seeks to find out the effectiveness of two types of overlapping hoops arrangement relative to the single hoops arrangment. In addition, effort is made to investigate two other factors that are likely to affect the ductility of confined concrete, namely: the hoops diameter and concrete grade. To achieve this, small-size columns (of the size 150 x 150 x 750mm) are used as specimens. This size is chosen both for economy and ease of handling. In the same vein, the main reinforcement is limited to 8mm diameter bars only while the hoop reinforcement consists of 6mm and 4mm diameter bars. The design concrete grades are 25 and 20 MN/m^2.

Unfortunately, the columns so produced prove too weak to support any load after spalling of cover concrete. The main reason for this is that the spacing of hoops is too far apart, relative to the size of the column. In the absence of any data on the final horizontal portion of the stress-strain curve, the falling branch of the curve is used as a criterion for comparing the relative ductility of the columns. This is so because the effect of confining concrete begins to manifest itself only after the maximum load has been attained.

Based on these curves, the following conclusions can be drawn:

1. The diamond-shaped overlapping hoops arrangement is more effective in confining concrete than both the rectangular overlapping and the rectangular single arrangements. The results also show that the rectangular single, is in turn more effective than the rectangular overlapping. However, this needs further investigation since the conclusion is based on the results of one column only. The other two columns of the rectangular overlapping arrangement have no falling branch recorded.

2. Reducing the hoop diameter while maintaining the volumetric hoop ratio enhances the ductility of confined concrete. This is so because the concrete will now be confined at more places than when the larger diameter hoop is used.

3. A weak concrete grade offers more ductility to confined concrete than a strong one. However, because of its weakness, it requires more longitudinal reinforcement to be able to sustain the same maximum load.

5.2 Suggestions for Further Research

To be able to appreciate the interplay of various factors on the ductility of core concrete, it is suggested that future researches should be carried out using full-size columns. This will also minimise such errors as the one due to inaccurate location of the centre of loading.

In addition, the effectiveness of other arrangements of overlapping hoops should also be studied. Such arrangements may include the star-shaped, the overlapping spirals among others.

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APPENDIX A

STRESS - STRAIN TABLES BASED ON DIAL GUAGE READINGS

A11	f_c	
	4.44	0.00133
	8.89	0.00263
	12.44	0.00360
	14.22	0.00413
	15.64	0.00507
	7.82	0.00680

A13	4.44	0.00933
	8.89	0.00280
	12.44	0.00400
	14.22	0.00467
	16.00	0.00560
	17.42	0.00667
	8.71	0.01210

A21	f_c	
	4.44	0.00093
	8.89	0.00167
	12.44	0.00253
	14.22	0.00313
	16.00	0.00373
	17.78	0.00400
	8.89	0.00573

A22	4.44	0.00107
	8.89	0.00187
	12.44	0.00247
	14.22	0.00300
	16.00	0.00327
	17.78	0.00367
	19.20	0.00430

A23	f_c	
	4.44	0.00133
	8.89	0.00200
	12.44	0.00287
	14.22	0.00333
	16.00	0.00387
	17.78	0.00433

A31	4.44	0.00027
	8.89	0.00107
	12.44	0.00200
	14.22	0.00280

A32	4.44	0.00100
	8.89	0.00193
	12.44	0.00300
	13.96	0.00360
	6.98	0.00533

A33	f_c	
	4.44	0.00033
	8.89	0.00100
	12.44	0.00207
	14.22	0.00233
	15.73	0.00300
	7.86	0.00600

B13	f_c	
	4.44	0.00133
	8.89	0.00267
	12.44	0.00440
	13.91	0.00507
	6.95	0.00867

B23	f_c	
	4.44	0.00100
	8.89	0.00240
	12.44	0.00320
	14.22	0.00387
	16.00	0.00460
	17.33	0.00480
	8.66	0.00933

B11	4.44	0.00120
	8.89	0.00200
	12.44	0.00293
	14.22	0.00340
	16.00	0.00400
	16.98	0.00533
	8.49	0.00707

B21	4.44	0.00020
	8.89	0.00040
	12.44	0.00080
	14.22	0.00140
	14.93	0.00173
	7.46	0.00547

C11 *	4.44	0.00060
	8.89	0.00173
	12.44	0.00330
	14.22	0.00433
	7.11	0.01000

B12	4.44	0.00133
	8.89	0.00253
	12.44	0.00347
	14.22	0.00380
	16.00	0.00460
	17.60	0.00533
	8.80	0.00867

B22	4.44	0.00116
	8.89	0.00227
	12.44	0.00300
	14.22	0.00353
	16.00	0.00427
	17.16	0.00493
	8.58	0.00973

C12 **	4.44	0.00120
	8.89	0.00160
	12.44	0.00227
	14.22	0.00267
	16.00	0.00307
	17.78	0.00333
	19.56	0.00387
	20.44	0.00453
	10.22	0.01030

C13	f_c	
***	4.44	0.00160
	8.89	0.00267
	12.44	0.00480
	14.22	0.00547
	16.00	0.00600
	18.04	0.00760
	9.02	0.00867

C23	f_c	
*	4.44	0.00147
	8.89	0.00180
	12.44	0.00440
	14.22	0.00507
	16.00	0.00573
	17.07	0.00670
	8.53	0.00867

C21		
**	4.44	0.00133
	8.89	0.00273
	12.44	0.00360
	14.22	0.00400
	16.00	0.00453
	17.78	0.00493
	19.56	0.00553
	21.16	0.00600
	10.58	0.00840

C22		
	4.44	0.00080
**	8.89	0.00153
	12.44	0.00240
	14.22	0.00293
	16.00	0.00373
	16.89	0.00507
	8.44	0.00707

APPENDIX A

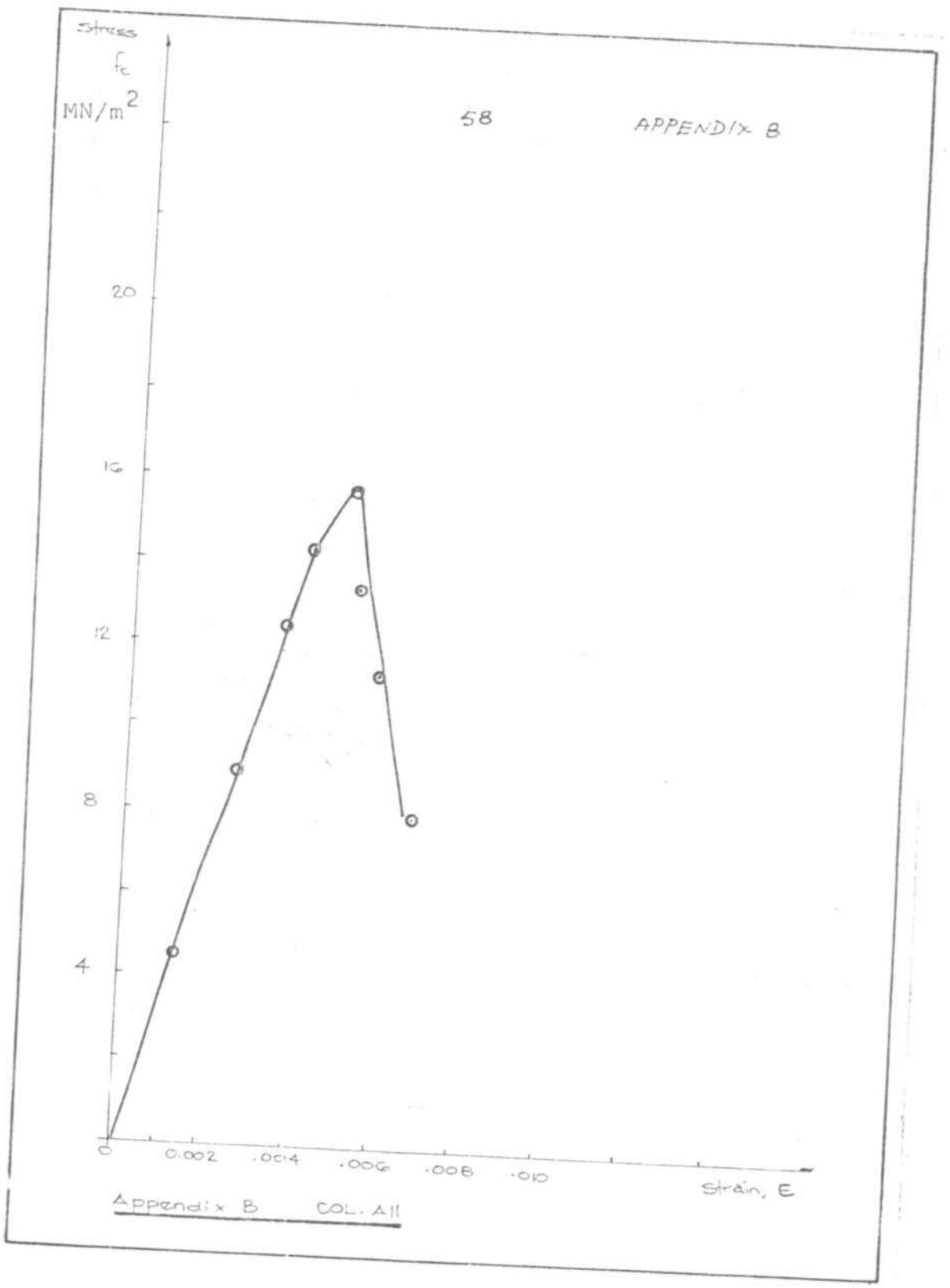
DESCENDING CURVE

	Load	Stress	Strain
A11	300	13.33	0.00533
	250	11.11	0.00587
	200	8.89	0.00640
		7.82	0.0068
A13	350	15.56	0.00797
	300	13.33	0.00903
	250	11.11	0.01023
	200	8.89	0.01183
		8.71	0.01210
A21	350	15.56	0.00447
	300	13.33	0.00483
	250	11.11	0.00513
		8.89	0.00573
A22	350	No falling branch	
A23			
A31			
A32	300	13.33	0.00400
	250	11.11	0.00413
	200	8.89	0.00507
A_{33}	300	13.33	0.00367
	250	11.11	0.00407
	200	8.89	0.00580
		7.86	0.00600

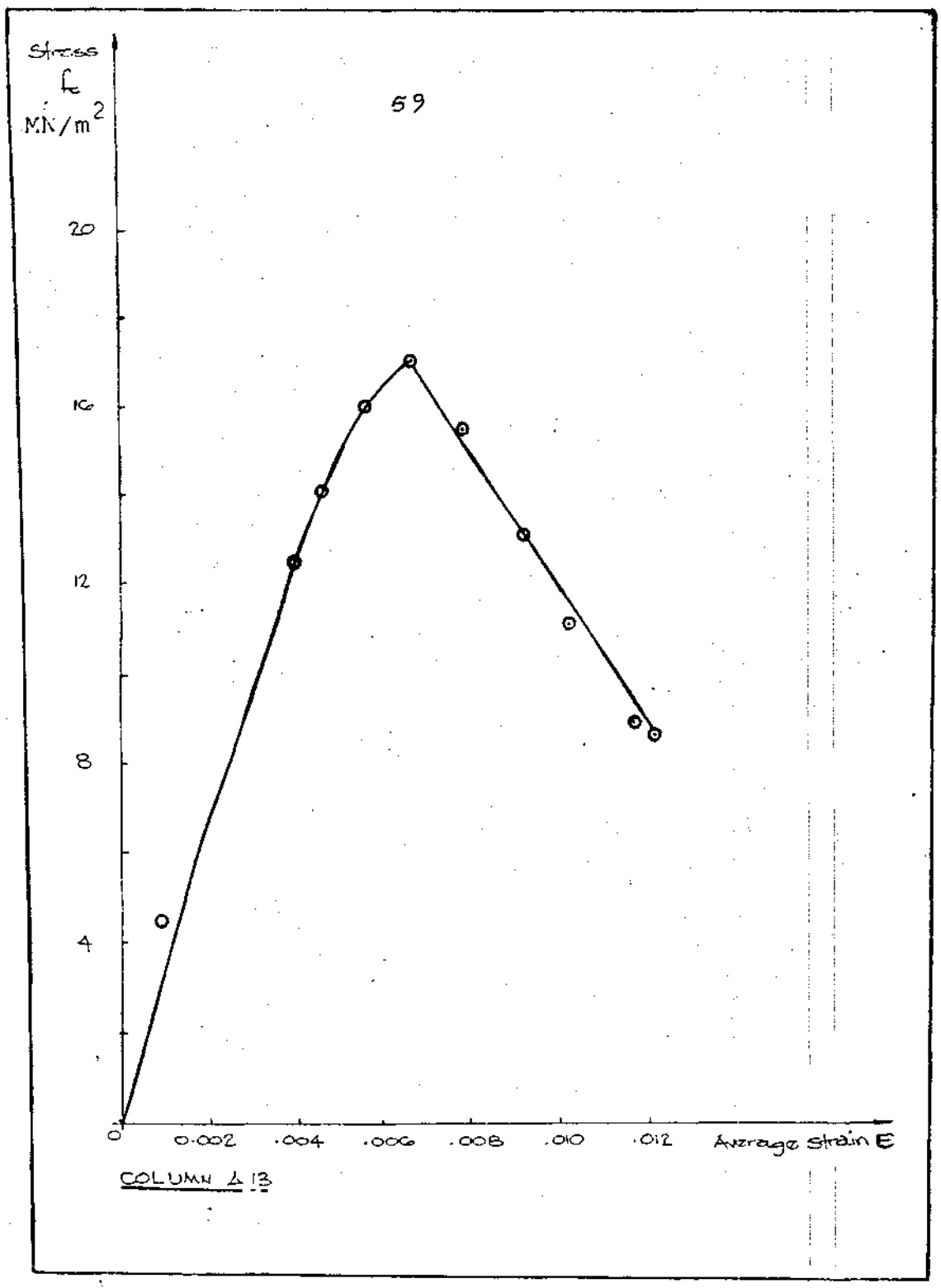
	Load	Stress	Strain
B11	350	15.56	0.00567
	300	13.33	0.00607
	250	11.11	0.00687
		8.49	0.00707
B12	350	15.56	0.00680
	300	13.33	0.00720
	250	11.11	0.00773
	200	8.89	0.00867
		8.80	0.00867
B13	300	13.33	0.00583
	250	11.11	0.00770
	200	8.89	0.00860
		6.95	0.00867
B_{21}	300	13.33	0.00333
	250	11.11	0.00427
	200	8.89	0.00493
		7.46	0.00547
B22	350	15.56	0.00613
	300	13.33	0.00713
	250	11.11	0.00820
	200	8.89	0.00933
		8.58	0.00973
B23	350	15.56	0.00640
	300	13.33	0.00747
	250	11.11	0.00820
	200	8.89	0.00913
		8.66	0.00933

APPENDIX A

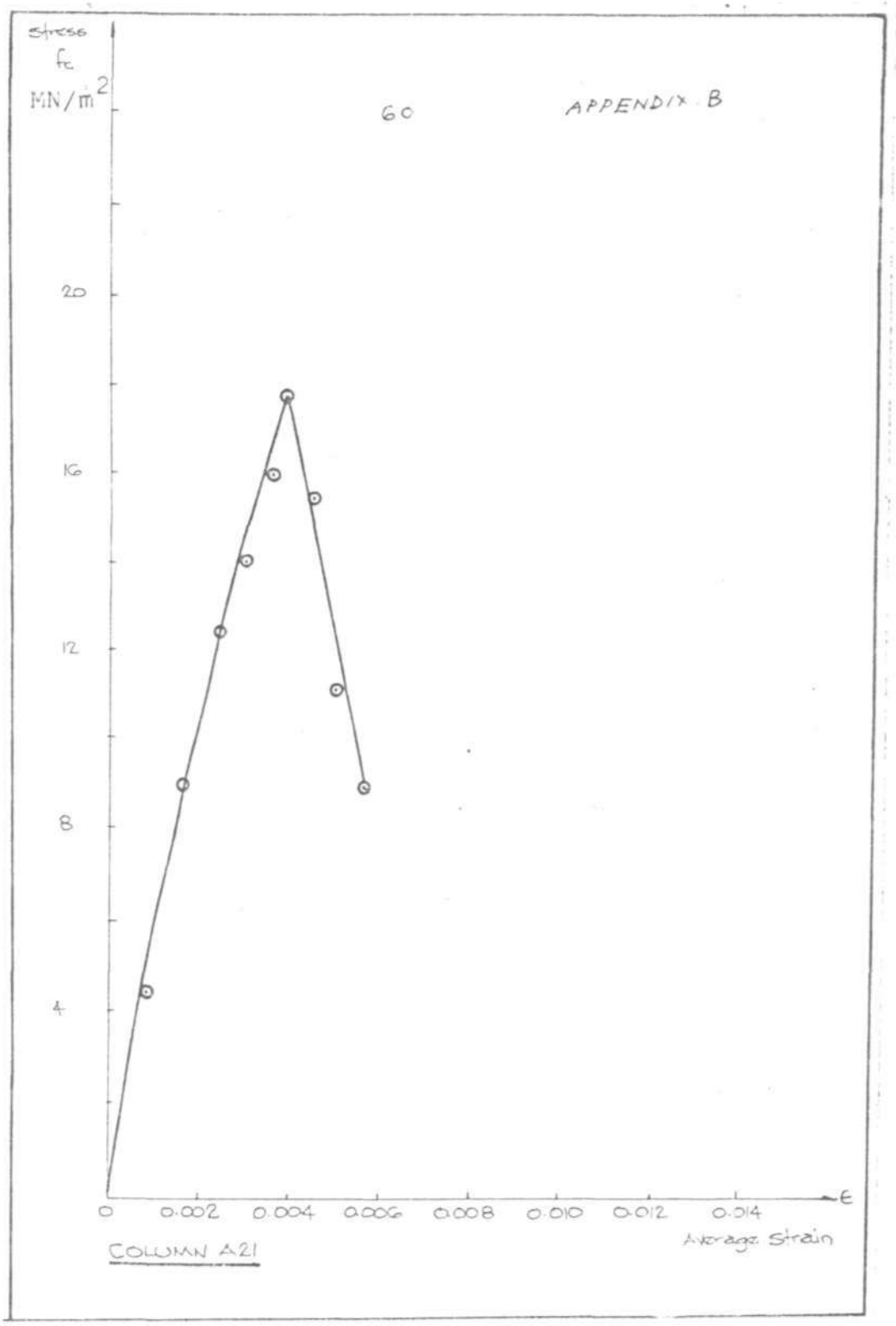
C_{11}	300	13.33	0.00583
	250	11.11	0.00723
	200	8.89	0.00990
		7.11	0.01000
C12	350	15.56	0.00780
	300	13.33	0.00853
	250	11.11	0.00967
		10.22	0.01030
C13	350	15.56	0.00806
	300	13.33	0.00853
	250	11.11	0.00880
		9.02	0.00886
C21	350	15.56	0.00767
	300	13.33	0.00780
	250	11.11	0.00840
	200	10.58	0.00840
C22	350	15.56	0.00453
	300	13.33	0.00533
	250	11.11	0.00600
	200	8.89	0.0068
		8.44	0.00707
C_{23}	350	15.56	0.00810
	300	13.33	0.00830
	250	11.11	0.00840
	200	8.89	0.00867
		8.53	0.00880

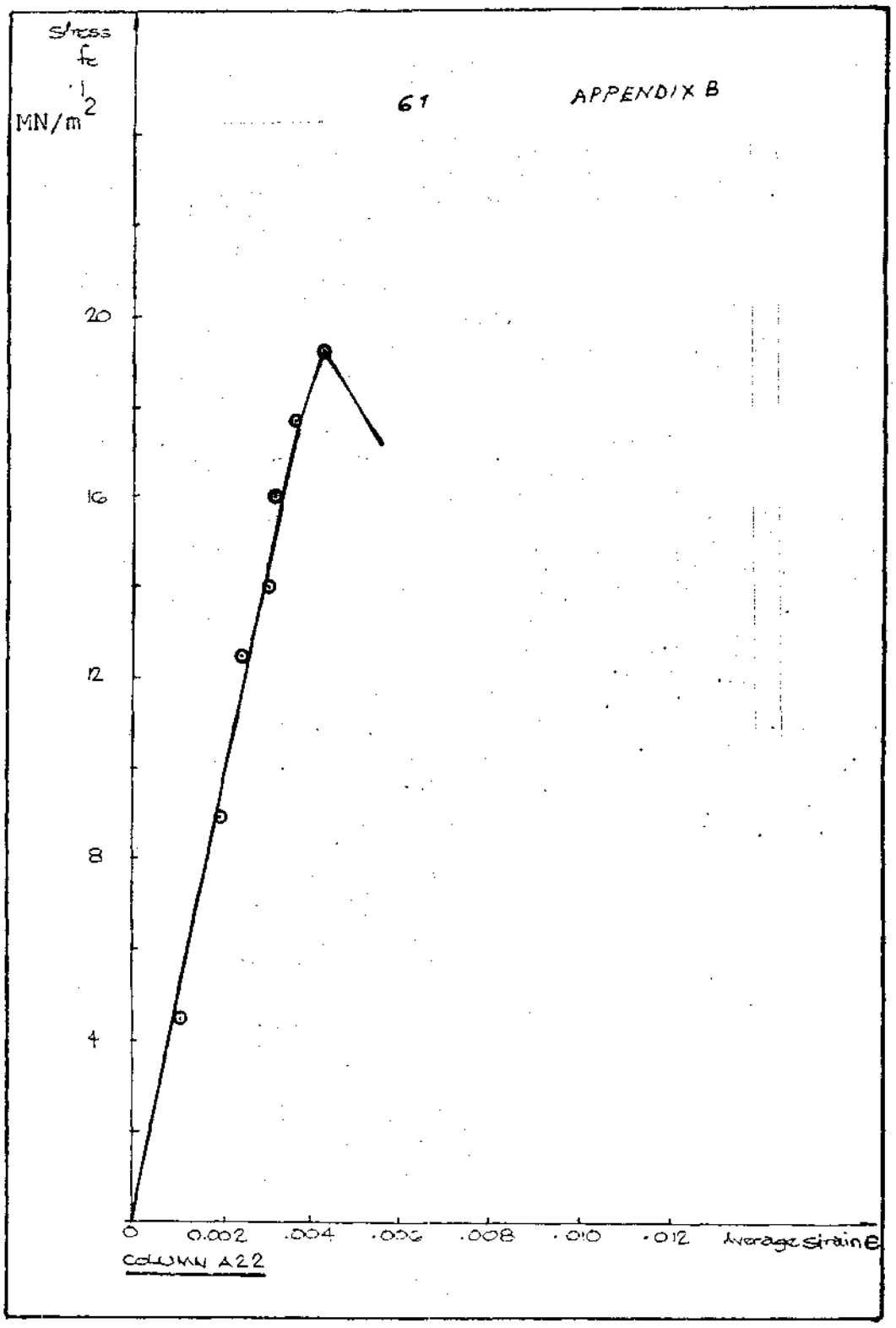


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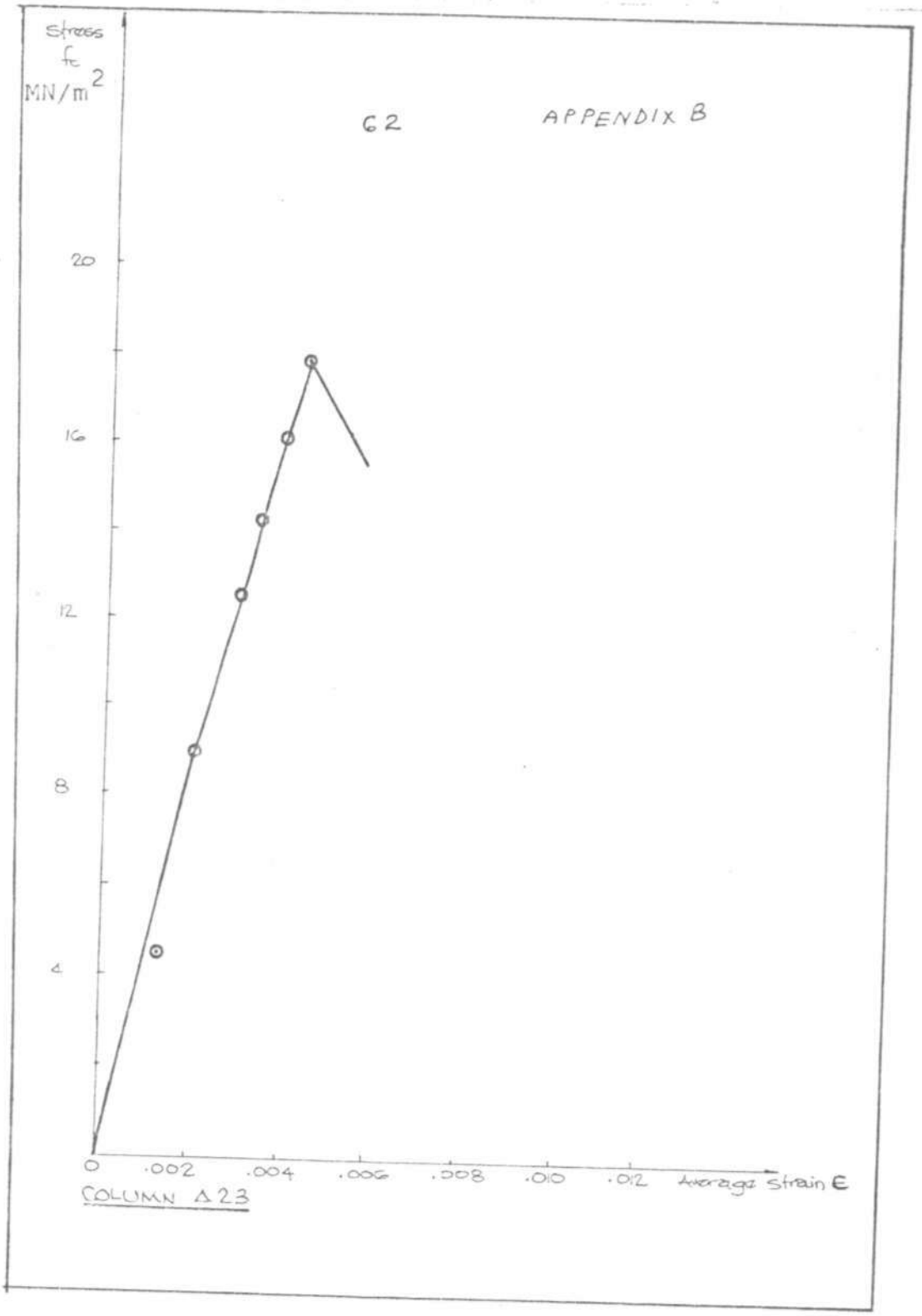

Stress
f
MN/m²
20
16
12
8
4
0
0·002
·004
·006
·008
·010
·012
Average strain E

COLUMN A 13

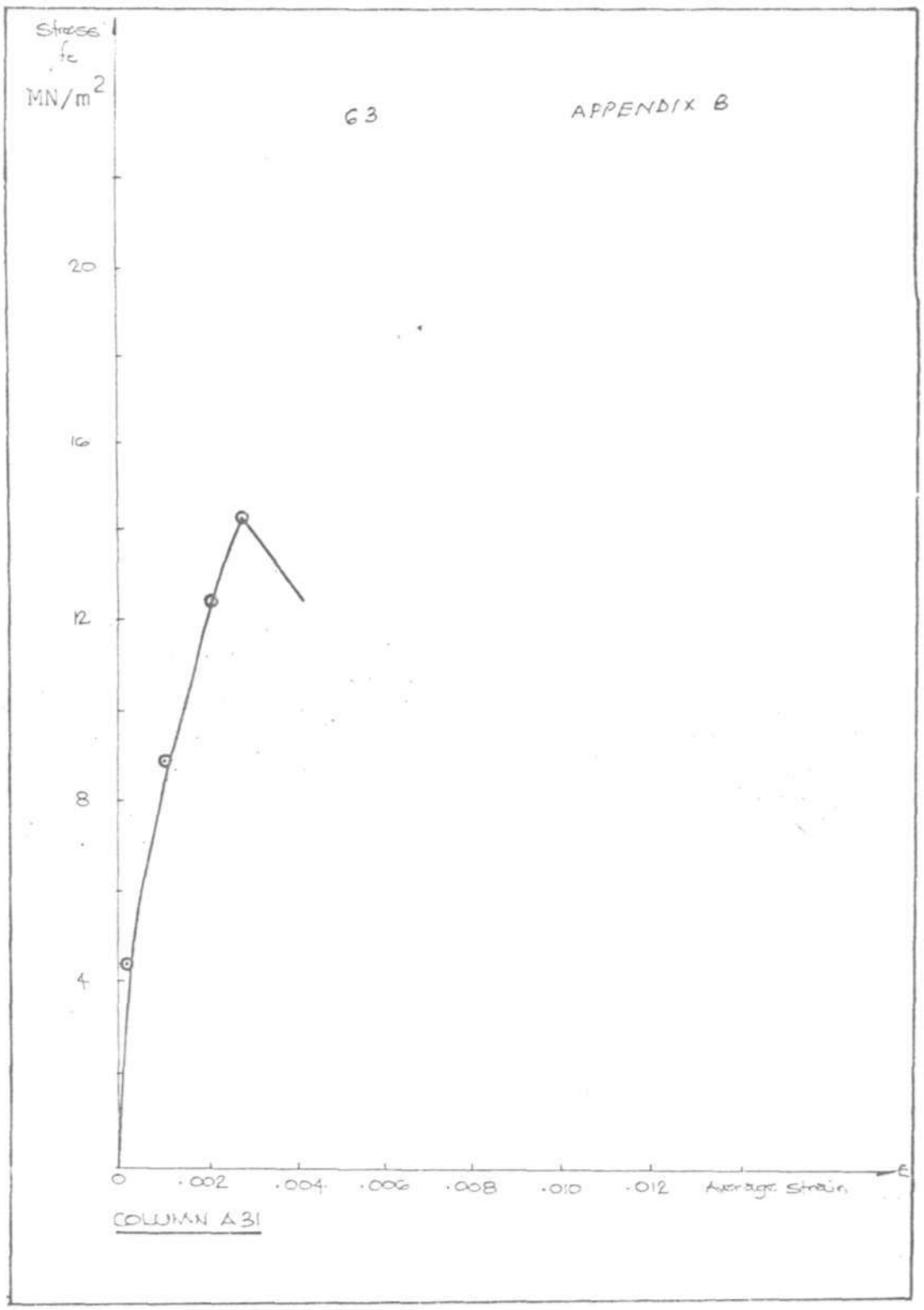

stress
f_c
MN/m^2
20
16
12
8
4
0
0.002
0.004
0.006
0.008
0.010
0.012
0.014
ε
Average Strain
COLUMN A21


Stress
f_c
MN/m^2
20
16
12
8
4
0
0.002
.004
.006
.008
.010
.012
Average Strain ε

COLUMN A22

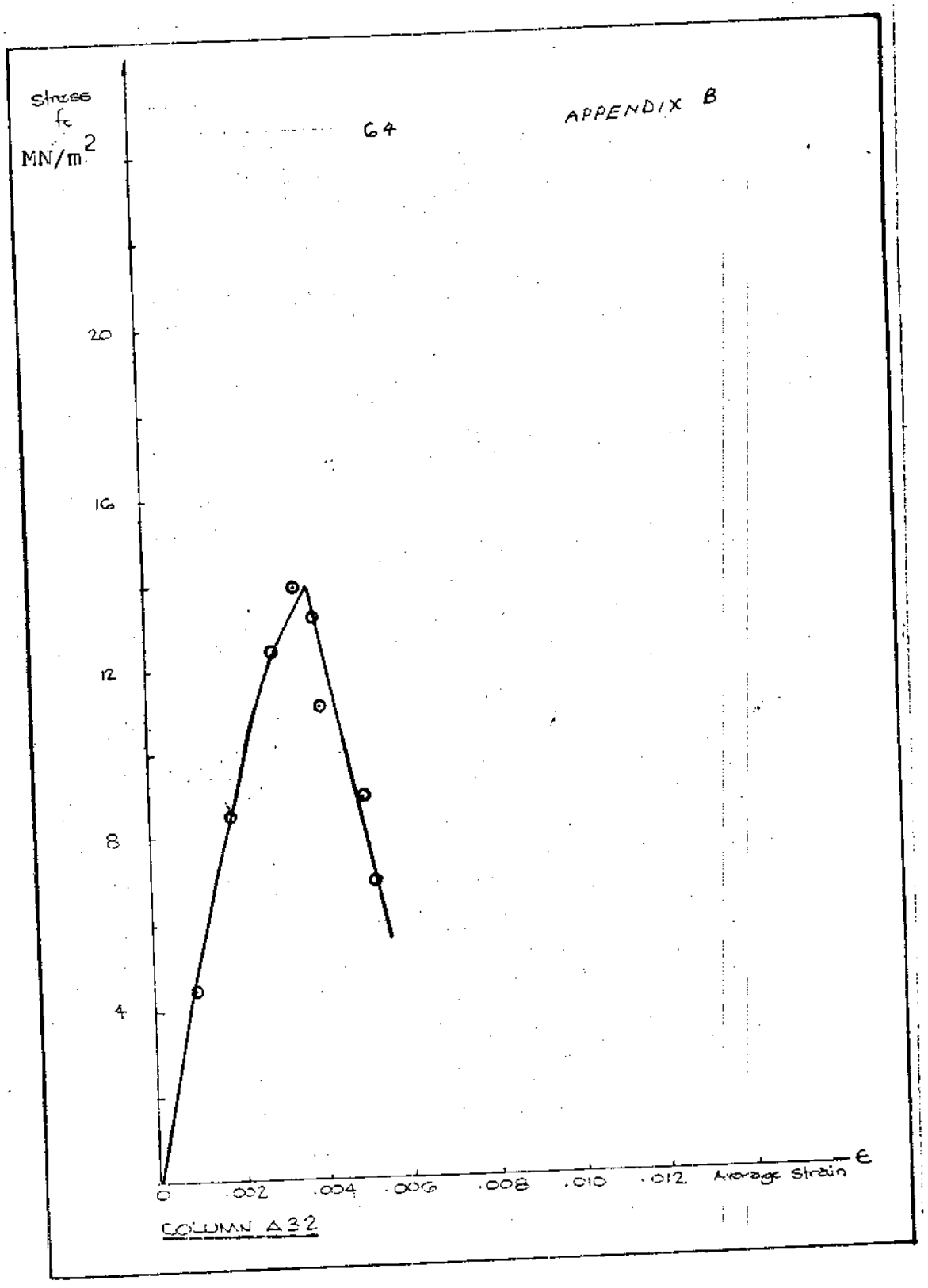

Stress
f_c
MN/m^2
20
16
12
8
4
0
.002
.004
.006
.008
.010
.012
Average Strain ε

COLUMN A23

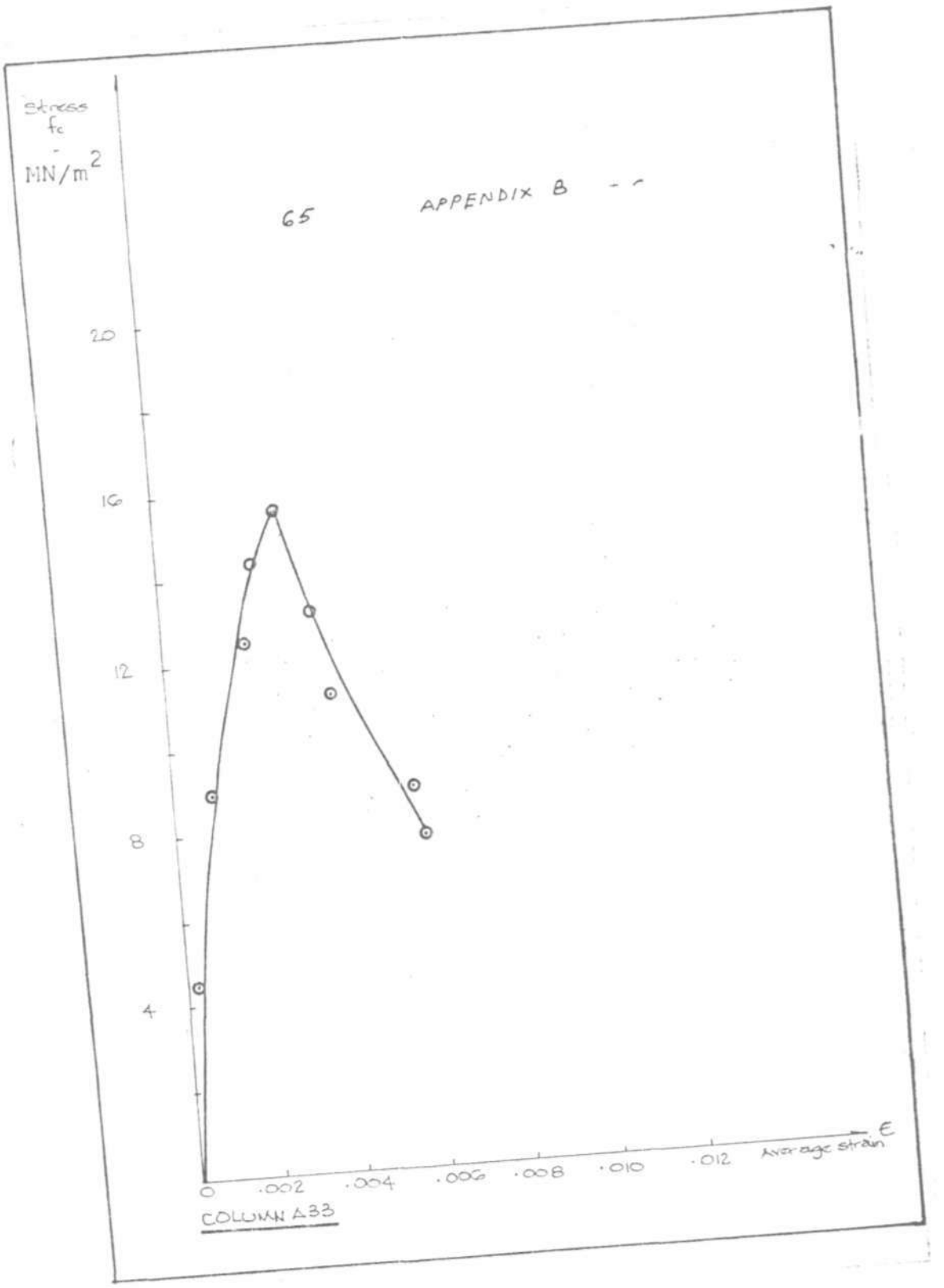

Stress
f_c
MN/m^2
20
16
12
8
4
0
.002
.004
.006
.008
.010
.012
Average strain
E

COLUMN A31

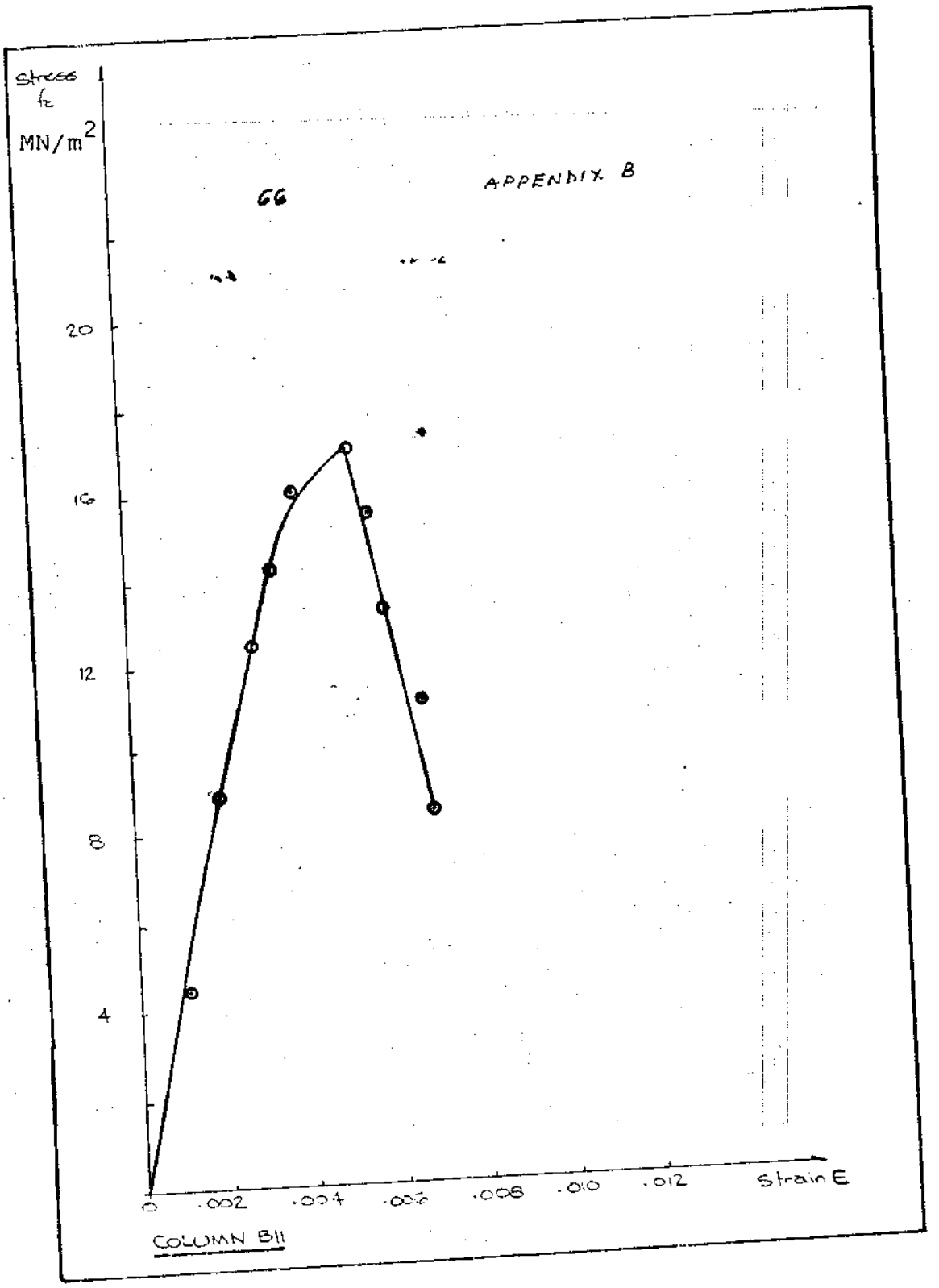
APPENDIX B



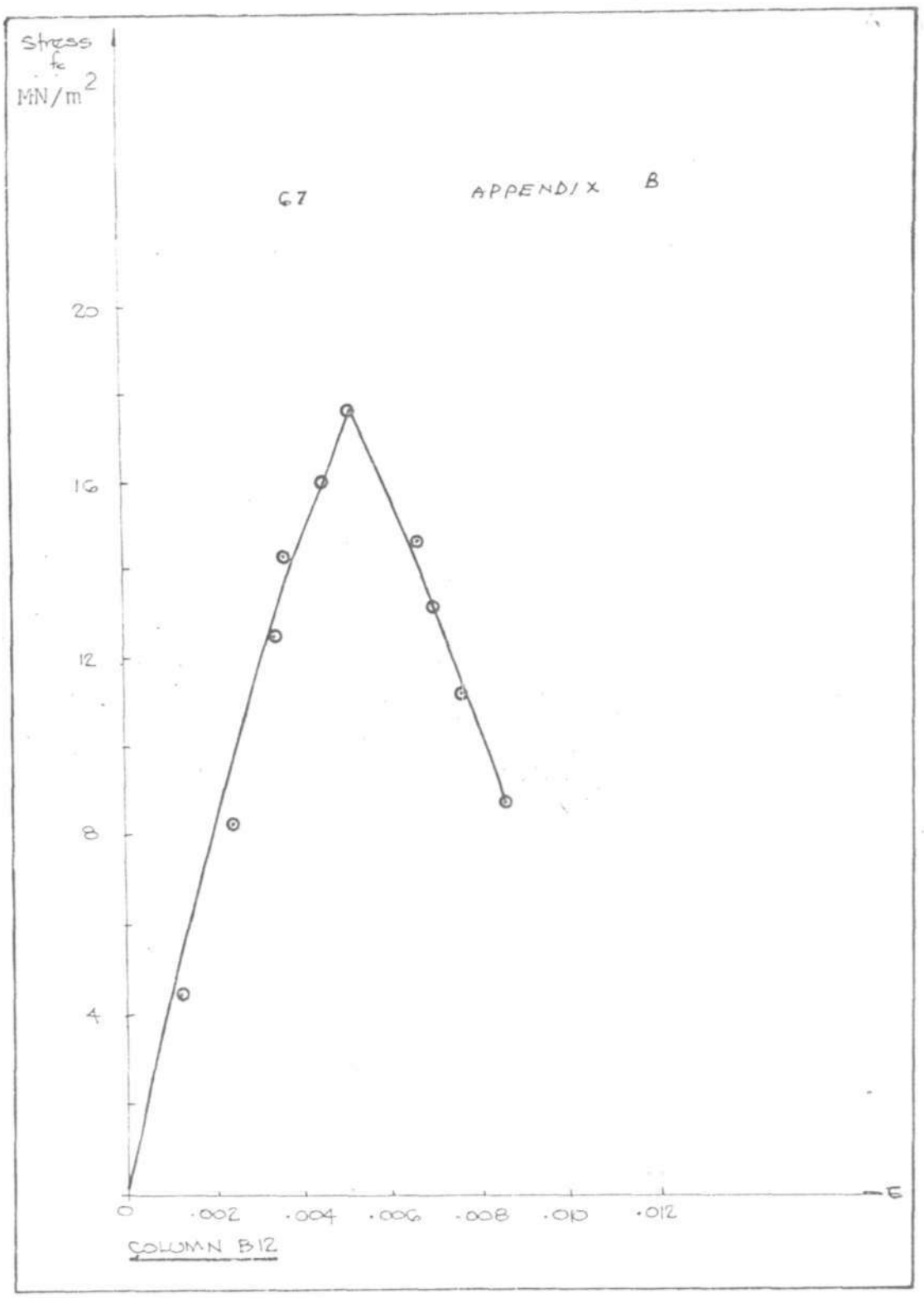
COLUMN A32


stress
f_c
MN/m^2
20
16
12
8
4
0
.002
.004
.006
.008
.010
.012
Average strain
ε

COLUMN A33

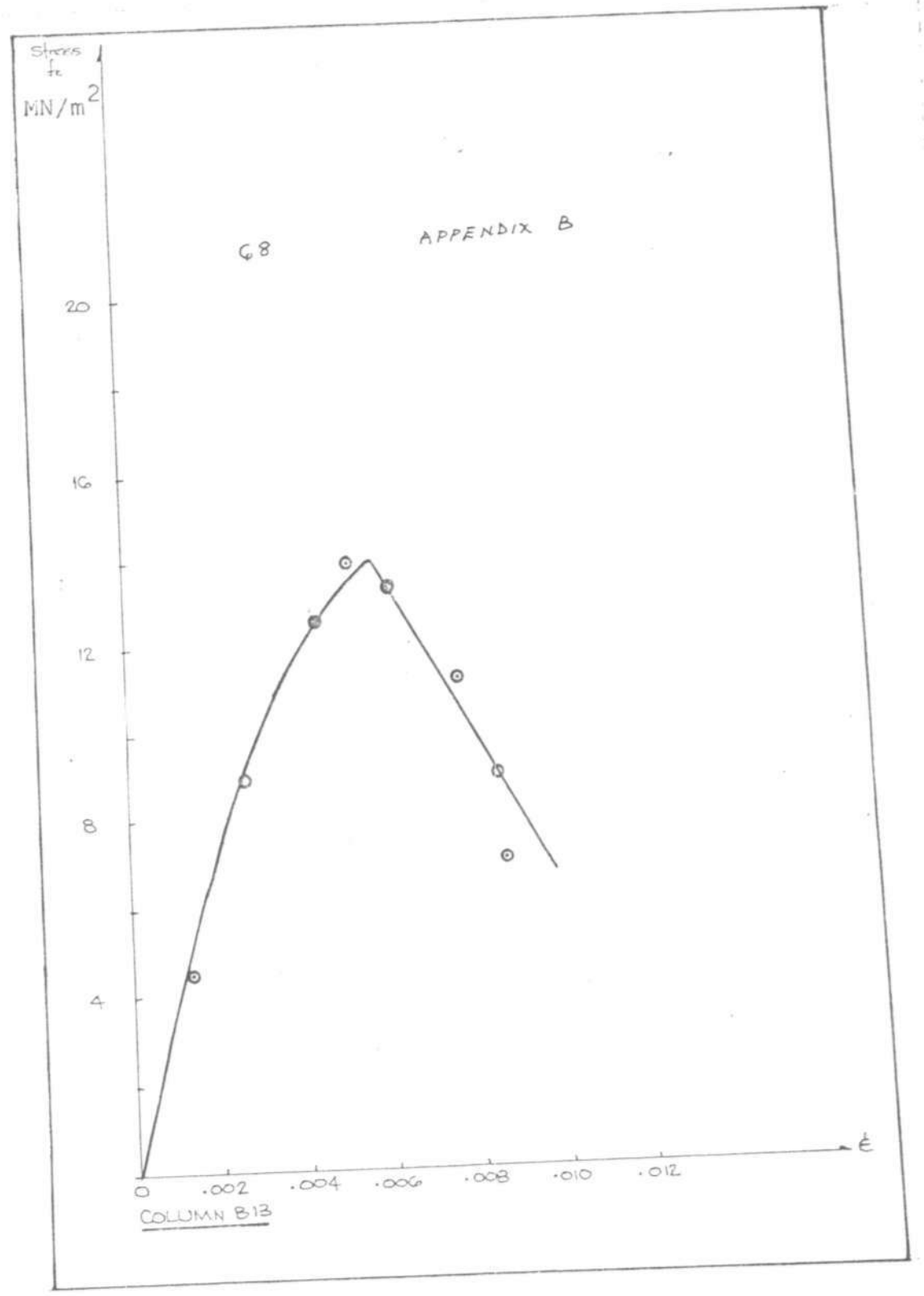


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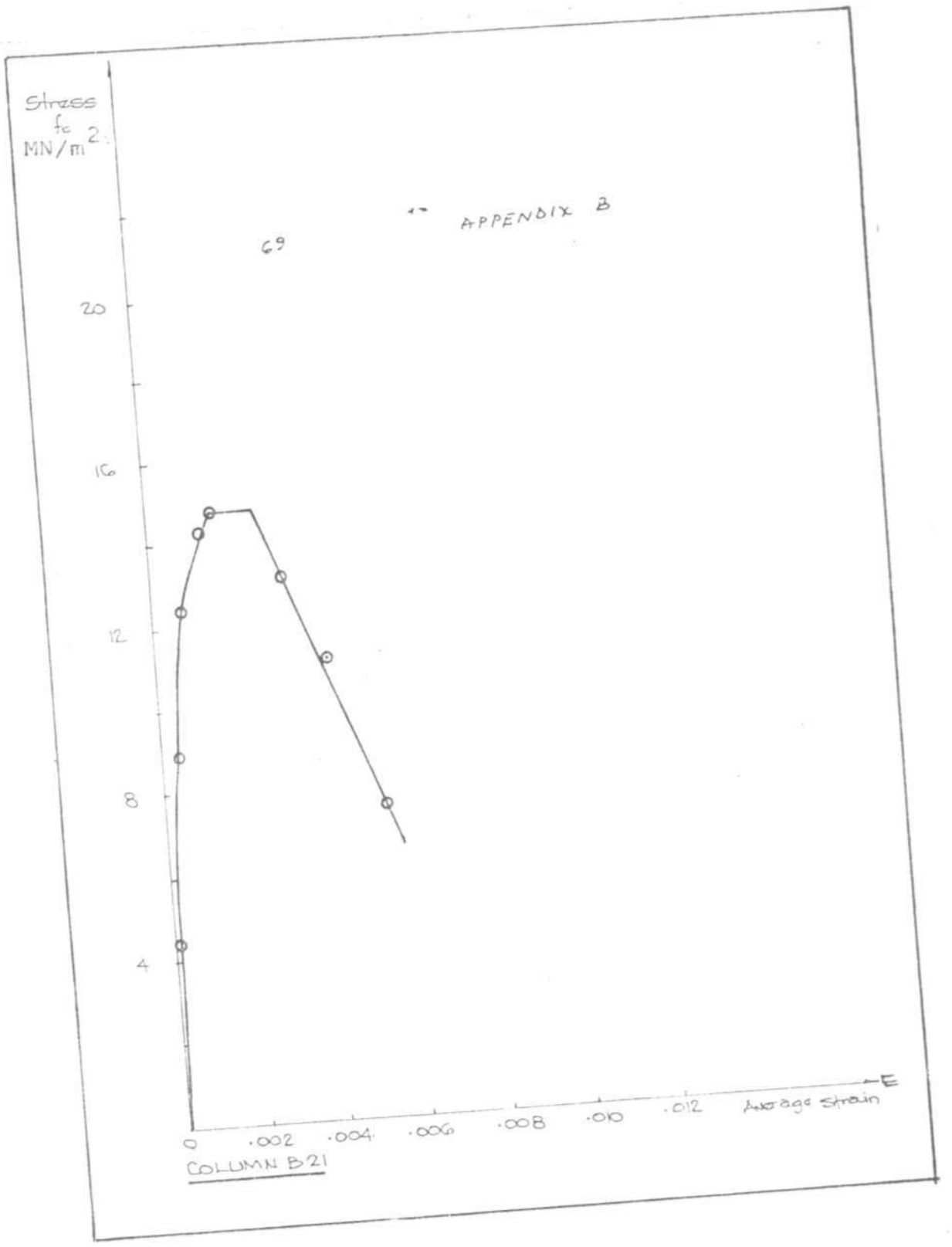


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APPENDIX B

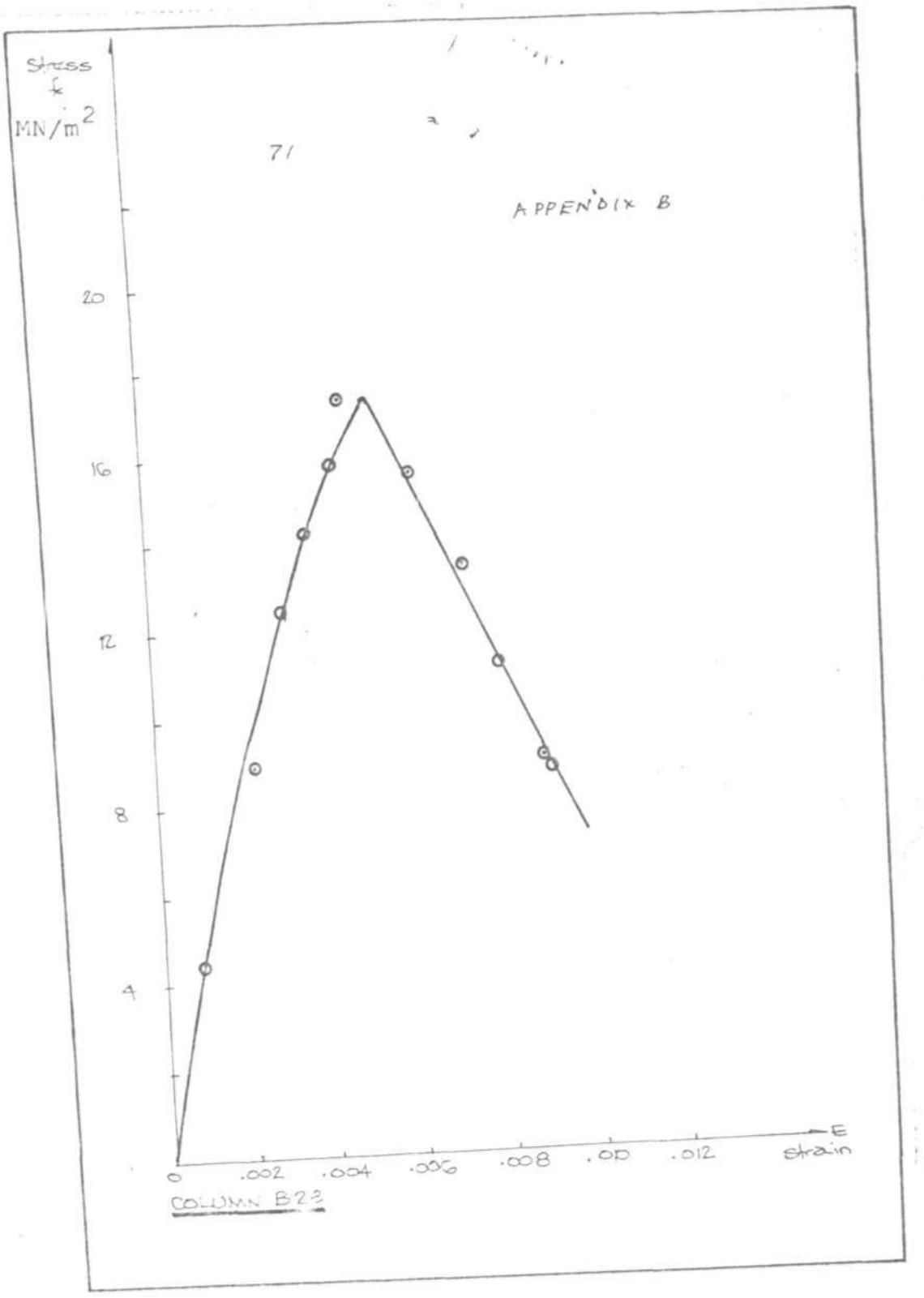


COLUMN B13


Stress
f_c
MN/m^2
20
16
12
8
4
0
.002
.004
.006
.008
.010
.012
Average strain
E

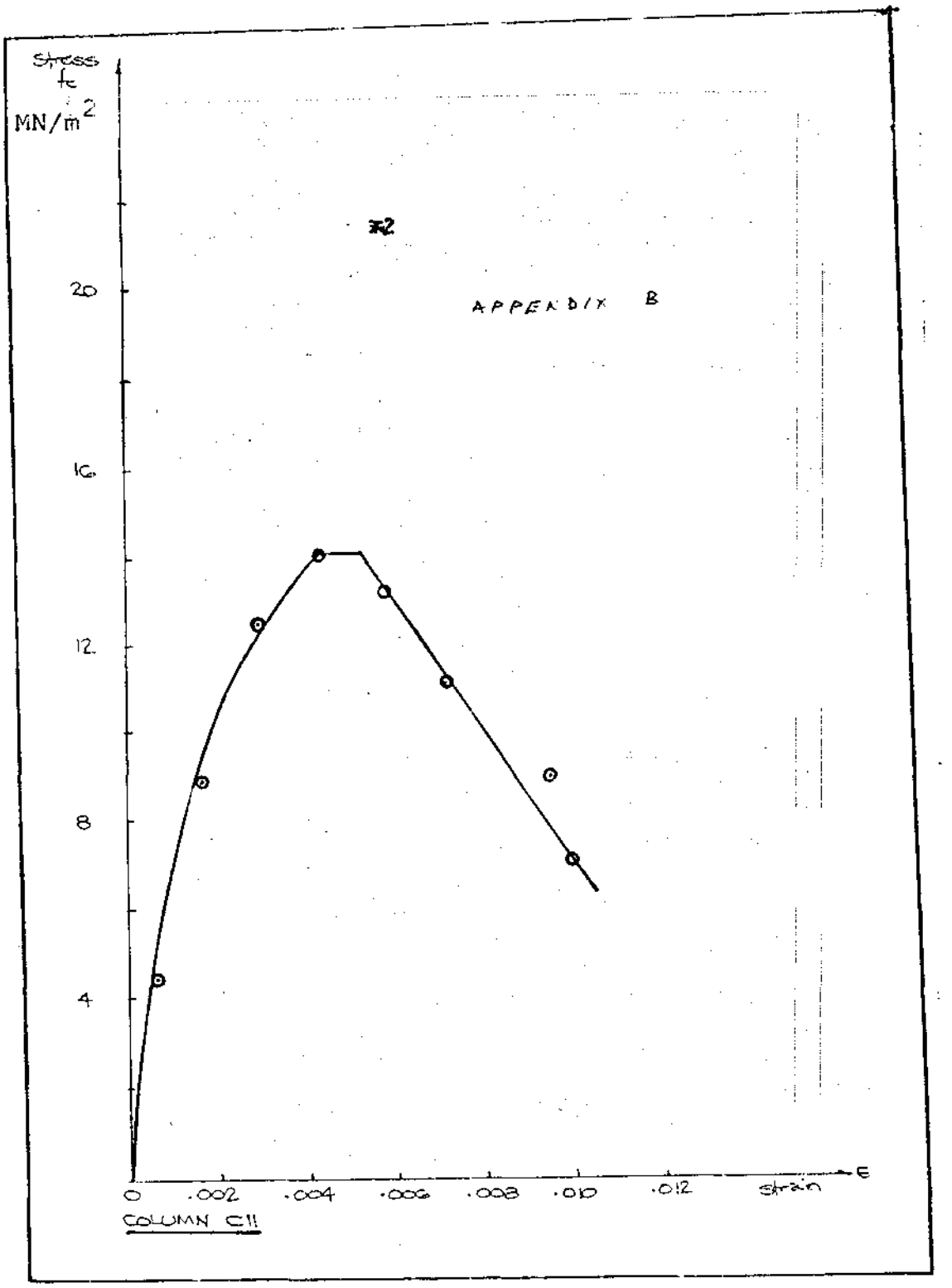
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APPENDIX B



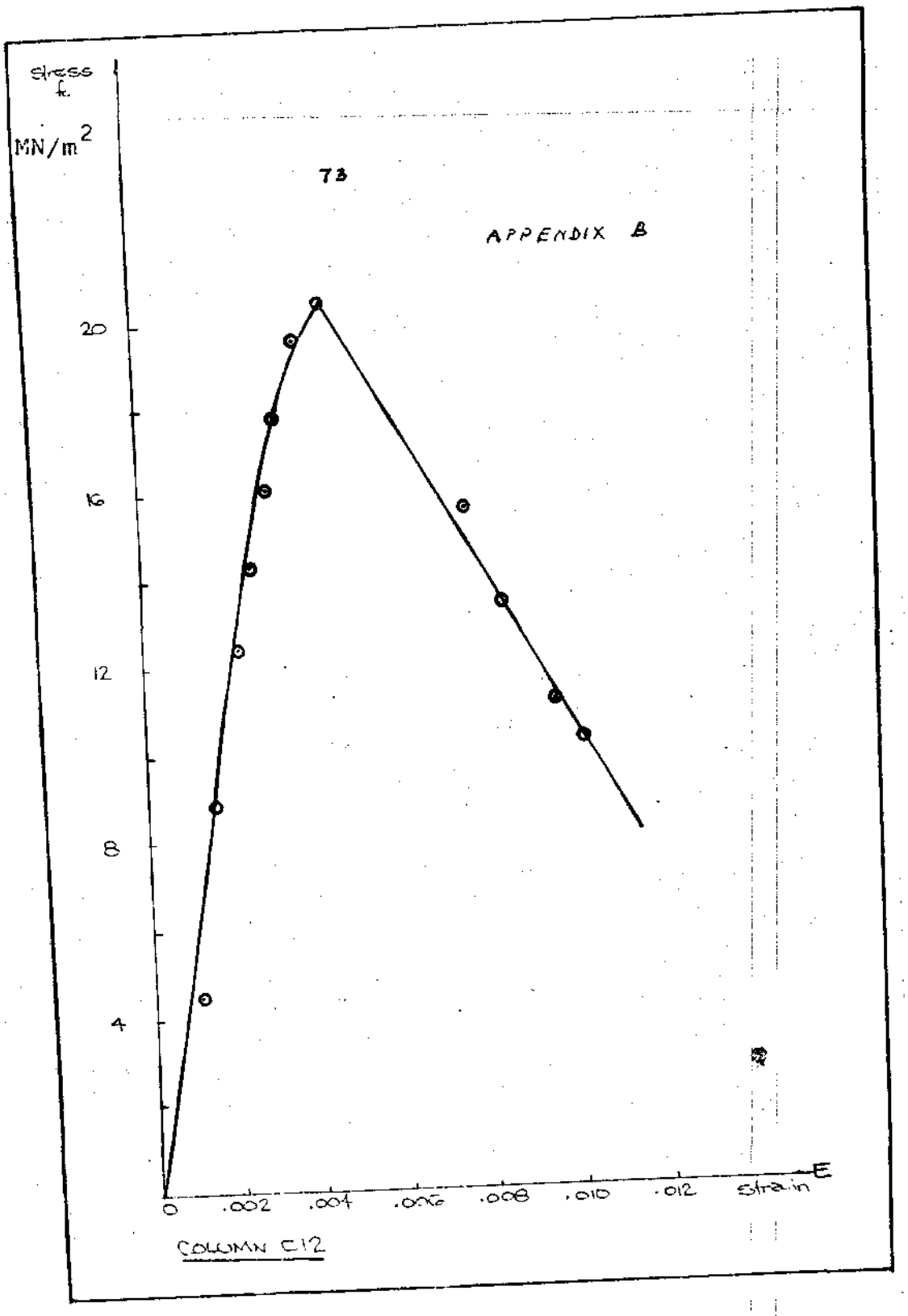
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APPENDIX B



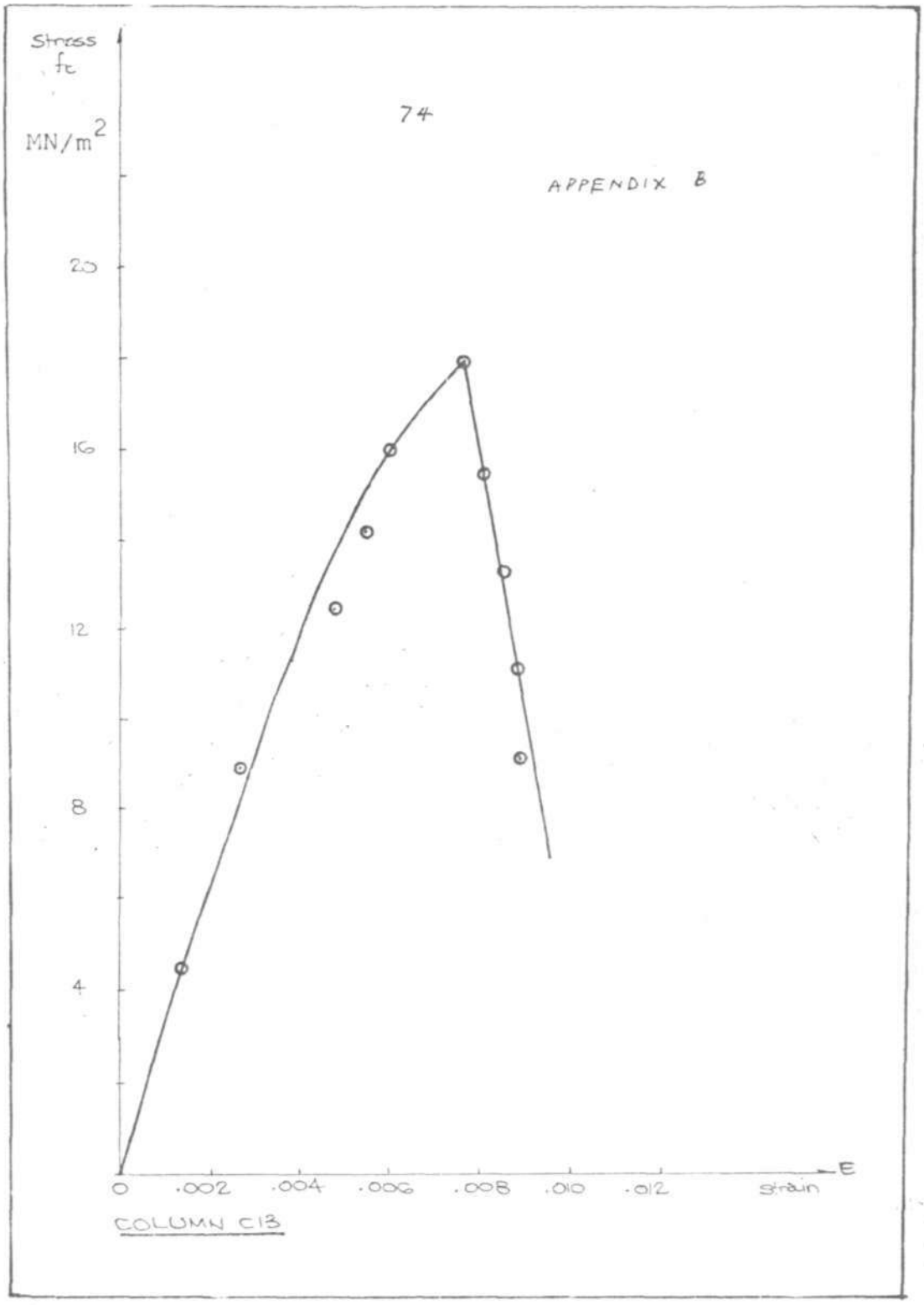
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APPENDIX B



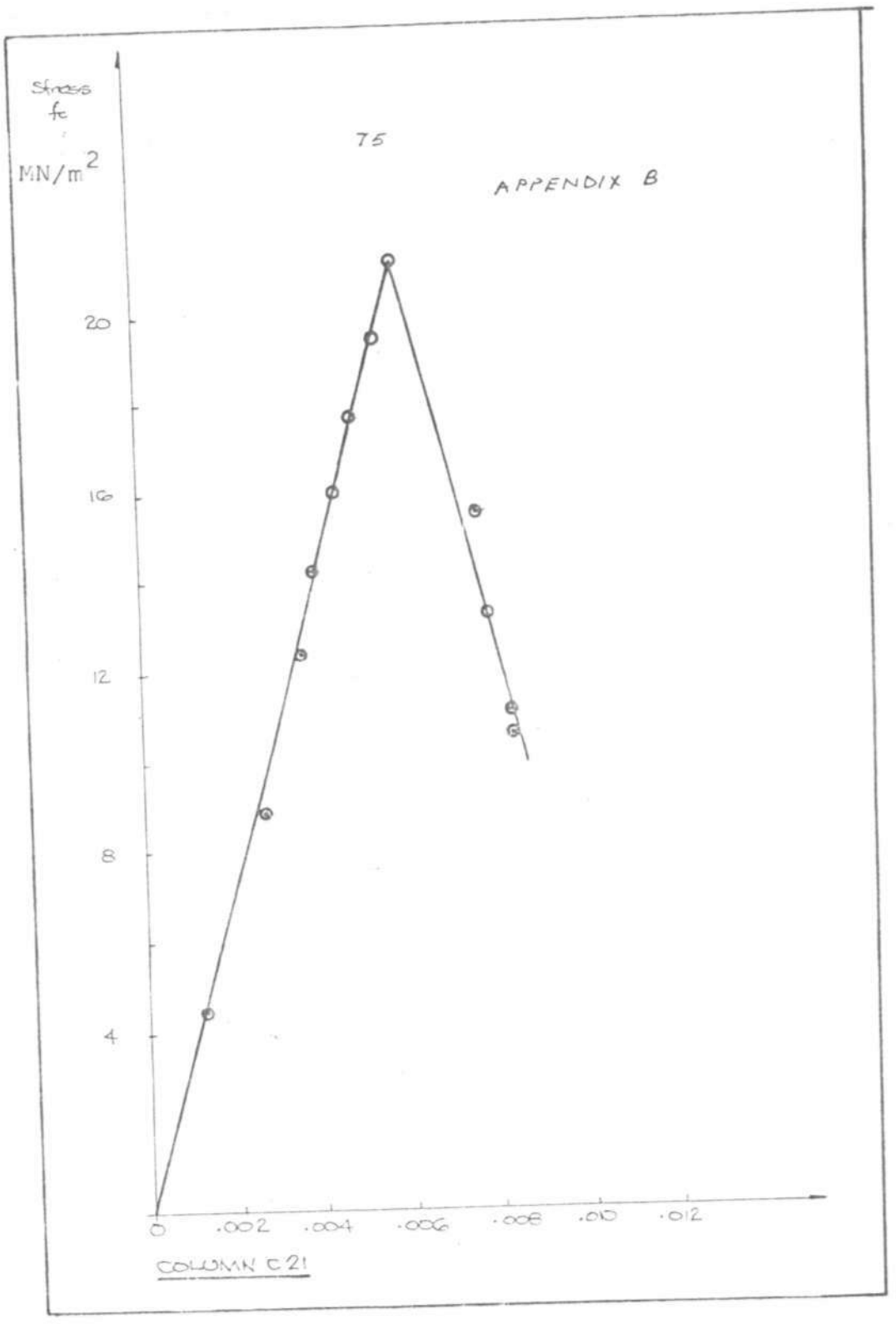
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APPENDIX B



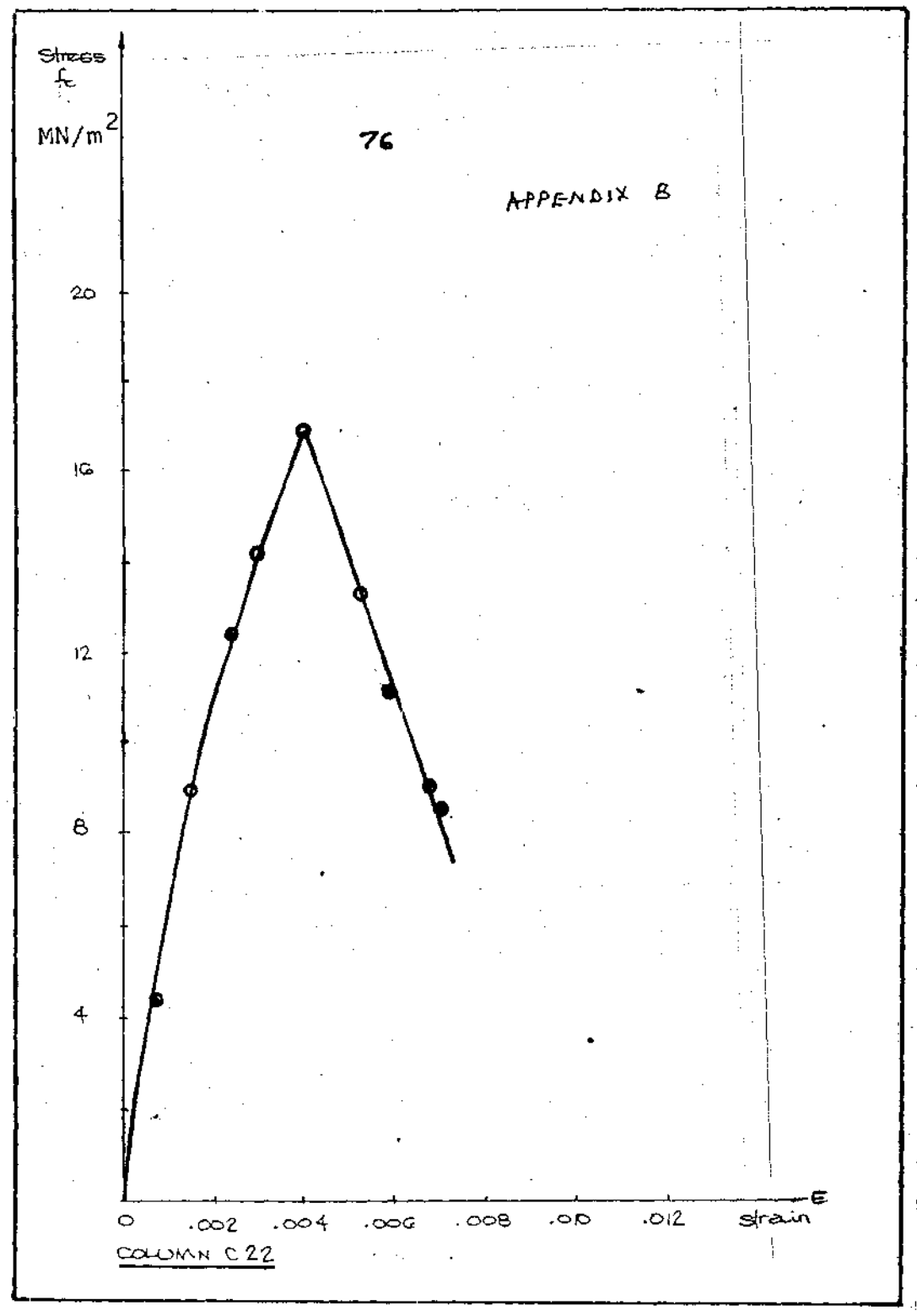
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APPENDIX B



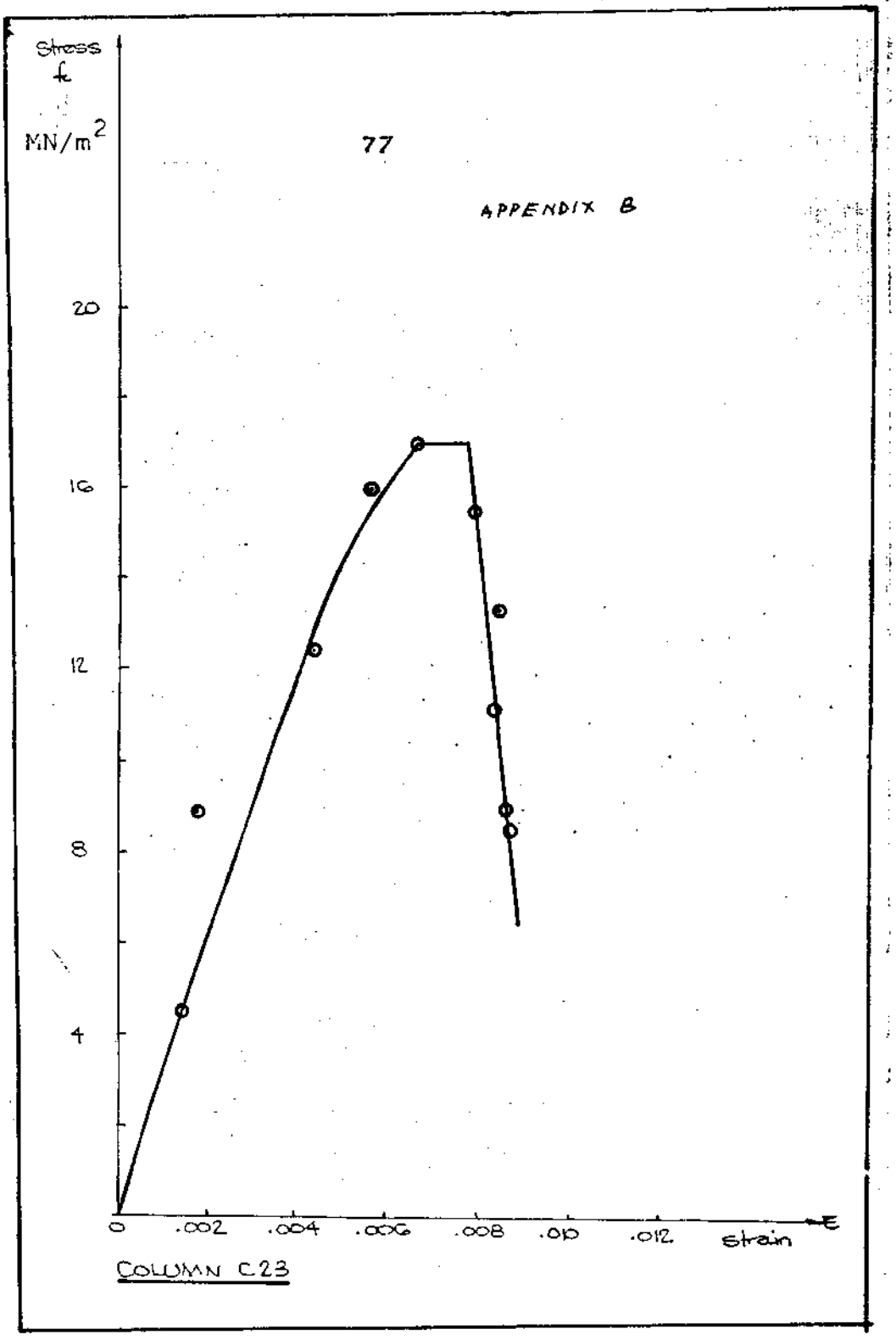
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APPENDIX B



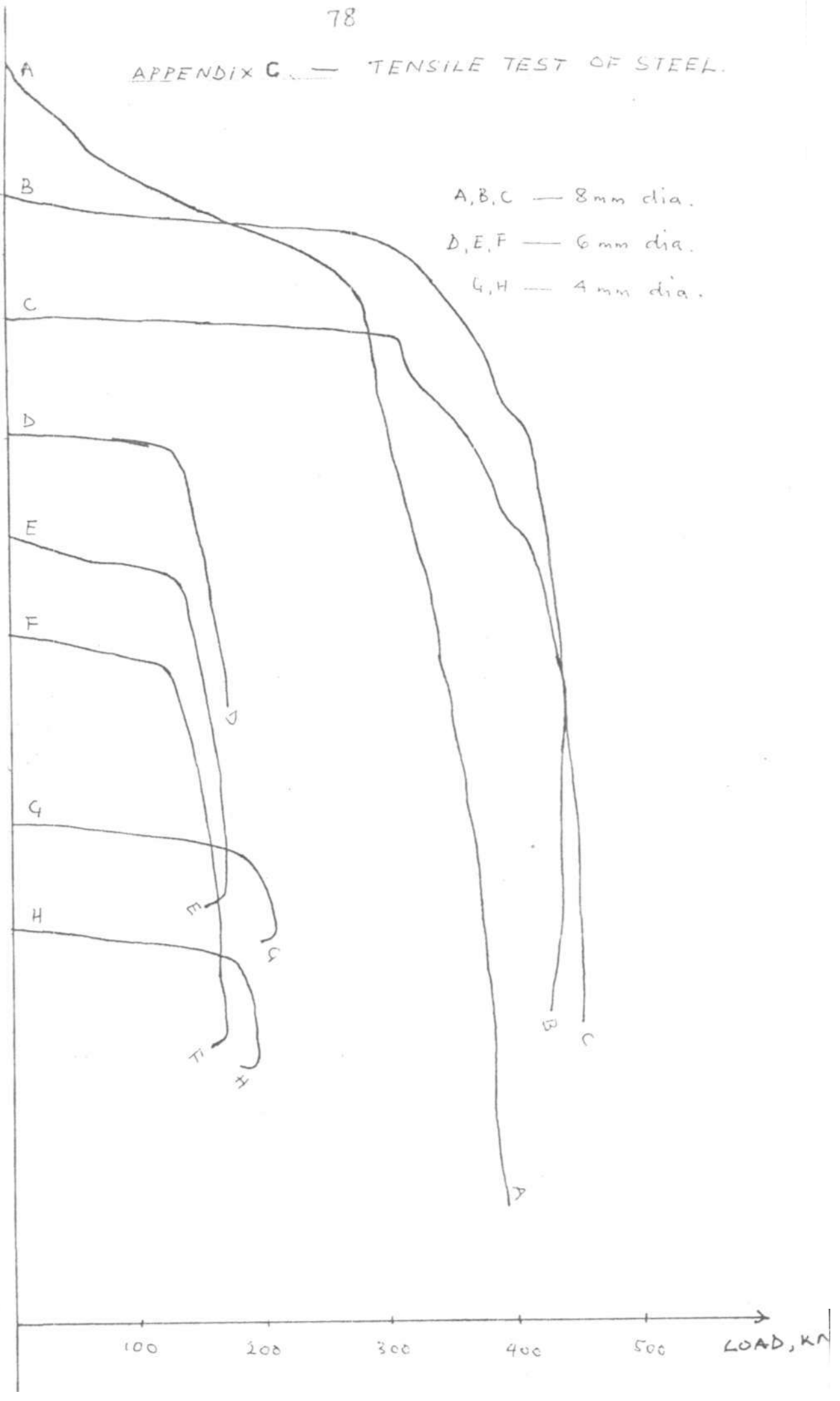
COLUMN C 22

APPENDIX B

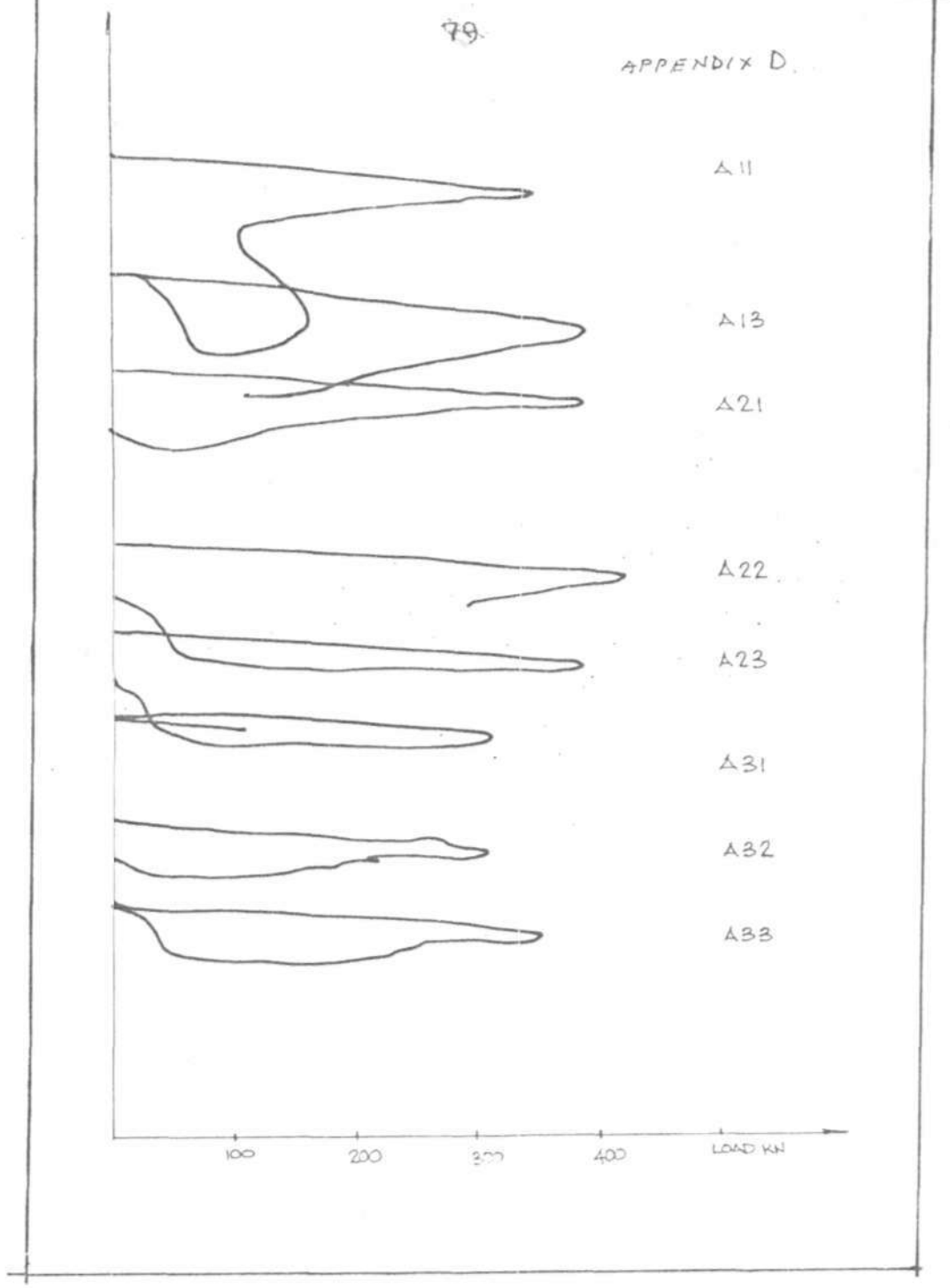


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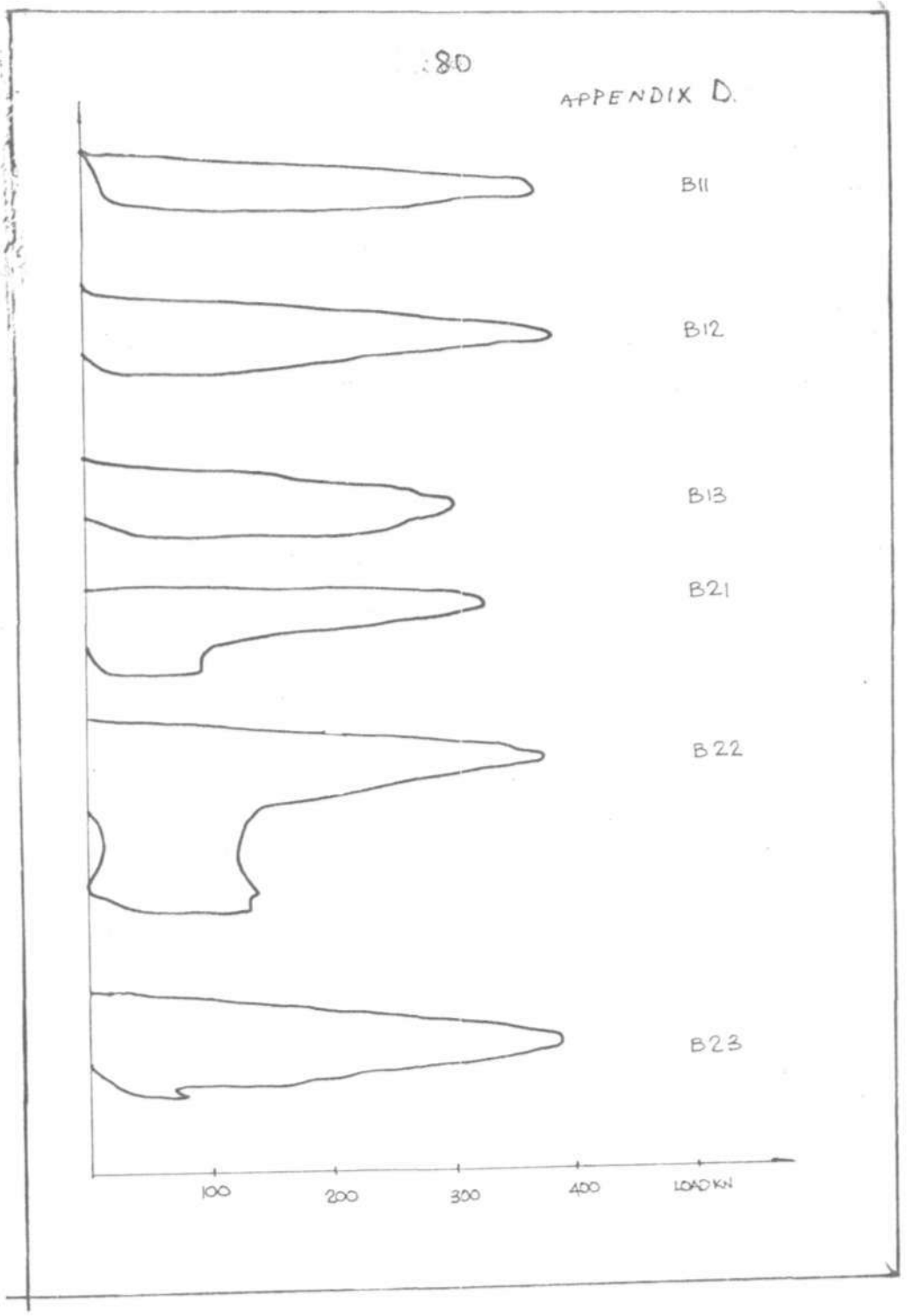
APPENDIX C — TENSILE TEST OF STEEL.



APPENDIX D.



APPENDIX D.



APPENDIX D

